San Juan Water District

Wholesale Master Plan
Water Supply and Treatment

Kennedy/Jenks Consultants

Engineers & Scientists

Final Report
September 2001

In association with

BLACK & VEATCH

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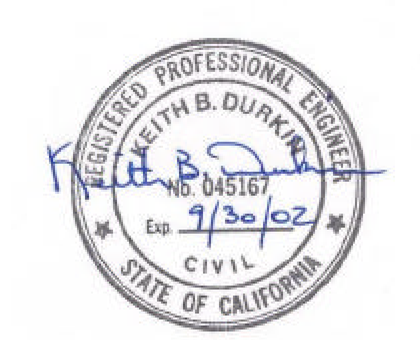

Prepared for

San Juan Water District
9935 Auburn Folsom Road
Granite Bay, CA 95746

K/J Project No. 992509.00

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1.1 Introduction

1.1.1 Background

San Juan Water District (District) is one of the American River Basin Cooperating Agencies developing a Regional Water Master Plan to ensure a reliable, high-quality water supply for the next 30 years and beyond. Partnering with 13 other water providers, the District is working to encourage resource conservation, provide regional planning, and find ways to boost efficiency and productivity. The broad goals of this process are to provide an economic, high quality, reliable water supply while protecting aesthetic and environmental resources. As part of this plan, the District has agreed to a regional conjunctive use program that will optimize the use of surface water during wet years and save groundwater for drier years.

As local agencies continue to explore cooperative regional programs, the District is a logical agency to play a major role. Because of its existing infrastructure (large surface water treatment plant and extensive water transmission and distribution systems), the District is well positioned to participate in the treatment and transfer of water to an expanded customer base.

1.1.2 Master Plan Objectives

The objective of this Master Plan is to assess the District's current water supply and treatment facilities and develop alternative actions where appropriate to accommodate the treatment and transmission of a reliable, high quality water supply for peak supply capacities ranging between 120 million gallons per day (mgd) and 240 mgd. Initially, the District is interested in identifying ways to immediately increase the reliable capacity of the existing Sidney N. Peterson Water Treatment Plant (WTP) to 120 mgd to meet short-term water demands. For the year 2030 planning horizon of this report, the capacity requirement for the WTP to meet the wholesale and retail area water demand is 150 mgd. However, the District estimates that a WTP capacity of as much as 240 mgd might be required to assist in meeting regional demands.

In addition to evaluating short-term and long-term WTP requirements, this Master Plan evaluates the facilities utilized in diverting source water from Folsom Reservoir, transmitting the source water to the WTP, and storing the water for distribution from the WTP. Transmission delivery systems as well as other regulatory, fiscal, administrative, and operational considerations are addressed in other District programs.

Specific goals of this Master Plan are to:

- Assess the current raw water transmission, treatment, and storage facilities for meeting changing capacity and/or treatment requirements.
- Develop alternative actions to ensure the treatment and transmission of an adequate, reliable, high quality water supply.
- Identify a practical approach to project sequencing and an incremental implementation plan that is economical while providing a high degree of reliability and ease of operation.

Ideas and recommendations for this Master Plan reflect consensus from District engineering and operations staff as well as other stakeholders.

1.1.3 Acknowledgements

This Master Plan was prepared by Kennedy/Jenks Consultants, in association with Black & Veatch Corporation. Specialized geosynthetics consulting services were performed by R.K. Frobel & Associates of Evergreen, Colorado. Melissa Blanton provided technical editing and writing guidance.

Although we would like to acknowledge and thank all of the San Juan Water District staff who provided input and assistance in this effort, special mention goes to Ms. Shauna Lorance, Mr. Michael O'Bleness, and Mr. Bill Sadler, who contributed many hours to provide information, guidance, and technical review. Mr. Lyle Hoag, San Juan Water District Board of Directors, deserves recognition for his thoughtful review and "what if" questions that helped to ensure sound recommendations in the report.

1.2 Raw Water Pump and Pipeline Facilities

Surface water from Folsom Lake is currently the only source of raw (untreated) water supply for the District. Water is moved either by gravity or by pumping from the United States Bureau of Reclamation's (Bureau's) Folsom Pumping Plant located at the base of Folsom Dam. An 84-inch pipe (owned by the Bureau) from the Bureau's pumping plant transmits water into District piping consisting of a 72-inch and then a 54-inch diameter pipe. These pipes convey water to the District's 100 mgd WTP.

Folsom Dam. Bureau's Pumping Plant and 84-inch Transmission Pipeline are shown near the base and to the left of the dam.

The Folsom Pumping Plant and raw water pipelines are capable of meeting the District's short-term demands up to approximately 120 mgd without further improvements. The installed pump capacity, however, leaves the District exposed to future water supply shortages, when the level of Folsom Lake is below elevation 392. The risk occurs as the overall demand on the Bureau's facilities approaches 400 cubic feet per second (cfs) (258.5 mgd).

A WTP expansion to 150 mgd will require improvements to the District's leg of the raw water transmission piping system to reduce headloss and velocities exceeding 20 feet per second (fps). In addition, a parallel pipeline to the Bureau's 84-inch line is recommended to further reduce the hydraulic impacts to the pumping plant, provide transmission redundancy, and improve access for maintenance to the 84-inch pipeline. The Folsom Pumping Plant will require the replacement of one of the existing pumps to meet the 150-mgd demand level.

A WTP expansion to 240 mgd requires raw water pipeline improvements similar to, but larger than, the 150-mgd option. Furthermore, significant improvements will be required at the Folsom Pumping Plant and dam intake to accommodate the increased flows. The dam intake improvements include consideration of an anti-vortex feature and possibly additional pipeline capacity through the dam to reduce peak velocities. Detailed analysis of these potential Bureau improvements are not within the scope of this Master Plan.

The condition of the existing District pipelines and the Bureau's 84-inch pipeline lining indicate the linings should be repaired and cathodic protection should be considered. Concerns about the reliability of the single 84-inch pipeline from the Folsom Pumping Plant support the recommendation to parallel the pipeline. Further recommendations for reliability include addition of line valves for isolation and access manways for inspection/repair of the District pipelines.

A review of the Bureau's pumping plant peak power demand indicates that the existing installed horsepower (hp) potentially exceeds an incremental 1000 kilowatts (kW) attributable to pumping the District's share of the total water pumped. Discussions with the District's power consultant indicate that electrical power capacity is available to service whatever installed pump demands are required. This issue is under review outside the scope of this Master Plan.

Costs for recommended improvements to the District's Raw Water Transmission Facilities are summarized in Table 1-1. These costs are shown for a WTP expansion to 150 mgd and for a WTP expansion to 240 mgd in 30-mgd increments.

Table 1-1
Conceptual Level Estimate of Capital Costs
Raw Water Pump Station and Pipeline Improvements

WTP Capacity	150 mgd	240 mgd				240 mgd
Capital Improvement Item	**120-150 mgd**	**120-150 mgd**	**150-180 mgd**	**180-210 mgd**	**210-240 mgd**	**Total**
Folsom Dam Outlet Improvements	0	0	0	(a)	(a)	0
Bureau Folsom Pumping Plant						0
Larger Pump Retrofit	(b)	n/a	n/a	n/a	n/a	0
Plant Reconfiguration	n/a	(b)	(b)	(c)	(c)	0
Bureau Transmission Pipeline						0
Parallel 84	(d)	0	4,845,000	0	0	4,845,000
Lining Repairs	(e)	(e)	(f)	(f)	(f)	0
District Raw Water Piping						0
Rehabilitate Joints	76,000	76,000	0	0	0	76,000
Rehabilitate Linings	110,000	110,000	0	0	0	110,000
Cathodic Protection	54,000	54,000	0	0	0	54,000
54-Inch Gate Valve Replacement	134,000	134,000	0	0	0	134,000
New Manways and Valves	297,000	297,000	0	0	0	297,000
Parallel 48-inch Pipeline	623,000	n/a	n/a	n/a	n/a	0
Parallel 66-inch Pipeline	n/a	805,000	0	0	0	805,000
Subtotal	$1,294,000	$1,476,000	$4,845,000	$0	$0	$6,321,000
Contingency @ 25%	323,500	369,000	1,211,250	0	0	1,580,250
Engineering, Legal, and Administrative @ 25%	323,500	369,000	1,211,250	0	0	1,580,250
Total ($)	**$1,941,000**	**$2,214,000**	**$7,267,500**	**$0**	**$0**	**$9,481,500**

(a) Isolation valve velocities exceed Bureau maximum at Folsom Dam penetration; cost not estimated as part of this work.
(b) Expansion possible with larger pumps retrofit into existing pump bays; cost not estimated as part of this work.
(c) Expansion will require pumping plant reconfiguration; cost not estimated as part of this work.
(d) Parallel pipeline not required for hydraulic capacity, recommended for redundancy and reliability.
(e) Lining repairs not feasible without parallel pipeline.
(f) Lining repairs not estimated as part of this work.

1.3 Water Treatment Plant

The District's WTP was originally completed in 1983. The WTP is characterized as a "conventional filtration treatment process" that includes chemical oxidation and initial disinfection by chlorination, followed by coagulation, flocculation, sedimentation, filtration and final disinfection prior to delivering the treated water to the distribution system. The sedimentation basins and filters remove particles, including microbial contaminants that may be present in the source water. Disinfection provides an additional barrier against microorganisms that pass through the physical removal processes. In addition, lime is added to the treated water to increase the pH as a corrosion inhibition (water stabilization) measure.

San Juan Water District Water Treatment Plant. Pretreatment basins are in foreground, filter basins are in background, with a portion of Hinkle Reservoir on the right of the photograph.

The plant was constructed in three phases. The flocculation-sedimentation (pretreatment) basins were completed in 1975, Hinkle Reservoir was completed in 1980, and the filters were completed in 1983. The WTP has a design capacity of 100 mgd.

1.3.1 Regulatory Requirements

Drinking water regulations in the United States are undergoing significant revisions. The regulatory revisions are due to increasing contamination of water sources, coupled with more definitive knowledge of health risks associated with waterborne contaminants

The District's WTP was designed prior to many of the current state and federal water quality regulations and guidelines. Drinking water regulations that currently, or in the future, will impact the existing and expanded WTP are summarized in Table 1-2.

Table 1-2
Summary of Current, New and Anticipated Drinking Water Regulations and Potential Impact on District

Regulation	Description	Potential Impacts
Current		
Surface Water Treatment Rule (SWTR)	Targets turbidity and microbial contaminants	• Currently in compliance with turbidity requirements. • Disinfection practice must correspond to direct or conventional treatment approach.
Total Coliform Rule (TCR)	Targets microbial contaminants	• Currently in compliance.
Lead and Copper Rule (LCR)	Regulates excessive leaching of lead and copper	• Currently in compliance.
Information Collection Rule (ICR)	Required collection of microbial and DBP information	• No direct impact. • WTP may use data to understand DBP generation at plant.
Partnership for Safe Water Guidelines (PSW)	Recommends average filtered water turbidity =0.1 NTU	• Currently in compliance. WTP has complied with guideline last 5 years.
California *Cryptosporidium* Action Plan (CAP)	Established new turbidity goals for settled, filtered, and return water	• Insufficient monitoring data from WTP to verify impacts. • Return water turbidity likely not in compliance. Will require upgrade to District's filter backwash return treatment system.
Fluoridation (State Assembly Bill 733)	Mandates fluoridation of public water systems under certain circumstances	• Requires fluoridation if funds available from non-ratepayer or taxpayer sources. • Potential impact to site space layout with potential additional cost.
New		
Stage 1 Disinfectants/Disinfection By-Products Rule (D/DBPR)	Targets DBPs, sets limits for disinfection residuals	• Currently in compliance.
Interim Enhanced Surface Water Treatment Rule (IESWTR)	Sets new *Cryptosporidium* removal requirement and turbidity-based removal credit	• Increases monitoring and reporting requirements. • May require filter profile report. • May require disinfection profile. • Return water flow and turbidity must be measured and comply with CAP.
Anticipated		
Filter Backwash Recovery Rule (FBRR)	Sets turbidity standards for returning spent filter backwash to the treatment process	• Will require upgrade to District's return water treatment system. • Final rule requirements unknown. There may be additional impacts.
Arsenic Rule	Will lower arsenic MCL	• No impact to District expected.
Long-Term 2 Enhanced Surface Water Treatment Rule	May include additional turbidity or *Cryptosporidium* disinfection requirements	• Potential impact to District unknown since rule is draft only. • May indicate change in disinfection process.
Stage 2 Disinfectants/Disinfection By-Products Rule	Will focus on contaminant speciation and may reduce DBP MCLs or set individual MCLs for DBPs	• Current draft has compliance with Stage 2 D/DBPR based on local running annual averages. • May increase monitoring requirements. • Potential impact to District unknown since rule is draft only.
Radon and Radionuclides	Targets radon and other radionuclides	• No impact to District's surface water source and WTP. • Potential severe impact to supplemental groundwater supply.

1.3.2 Water Quality Issues

Water quality information provided by the District on source water and treated water indicates that the existing water treatment facilities, with the exception of the filter backwash water treatment system, meet existing, new, and anticipated drinking water regulations.

The District does not monitor treated, return backwash water turbidity. However, discussions with plant staff indicate that the return water turbidity is generally higher than the California *Cryptosporidium* Action Plan (CAP) 2 nephalometric turbidity unit (NTU) goal most of the time. The existing return water pretreatment process should be replaced with a more efficient pretreatment process to reduce return water turbidity to below the recommended 2 NTU goal and to reduce the risk that contaminants, including *Cryptosporidium*, will be returned in a concentrated level to the treatment process.

Replacing the existing return water pretreatment process with a more efficient pretreatment process may also reduce the amount of total organic carbon (TOC) returned to the plant via the filter backwash water recovery system. This may reduce disinfection by-products (DBPs) and should have a beneficial impact on the amount of chlorine required to provide the residual disinfectant levels and DBPs.

The Bureau has proposed installing a Temperature Control Device (TCD) on the outlet structure at Folsom Reservoir. The proposed TCD would permit withdrawing water from the upper zone in Folsom Reservoir for delivery to the District in order to reserve colder water for improving downstream fisheries. Prior experience treating raw water from Folsom Reservoir indicates that warm source water supplies are more vulnerable than cold water supplies to taste and odor causing compounds. The upper zone, warmer source water may contain high levels of DBP precursors. In addition, this water is more vulnerable to both microbial and synthetic organic carbon compound contamination due to recreational uses.

The recommended approach to address water quality issues is as follows:

Filter Backwash Water Treatment System

- Replace existing system with a new treatment system, including flow control, to comply with California CAP goals.

Temperature Control Device

- Notify the Bureau that the proposed TCD operating strategy could adversely impact WTP operations.

- Request/obtain source water quality data with respect to reservoir depth and seasonal variation to assess or predict potential impacts of the TCD.

1.3.3 Existing Water Treatment Plant Capacity

The reliable process and hydraulic capacity of the existing WTP was evaluated, and recommendations were developed to meet a short-term WTP capacity objective of 120 mgd.

1.3.3.1 WTP Process Capacity

The WTP was designed as a “conventional filtration treatment process” incorporating chemical oxidation and initial disinfection by chlorination, followed by coagulation in a three-stage rapid mix system, flocculation, sedimentation, filtration, and final disinfection. Although the original WTP design criteria state the capacity of the WTP is 100 mgd, current United States Environmental Protection Agency (USEPA) and California Department of Health Services (DHS) guidelines indicate the WTP capacity as a conventional filtration process is more on the order of 60 mgd due to limitations of the sedimentation basins.

Based on this observation and WTP operational practices at flows above 60 mgd, the existing plant capacity was also evaluated with the WTP operating as a “direct filtration treatment process.” This process incorporates oxidation and initial disinfection, followed by coagulation in a rapid mix system, flocculation, filtration, and final disinfection. Since the sedimentation step (part of the physical removal process) is eliminated from the conventional treatment process in this approach, the pathogen removal credits are lower (2.0-log *Giardia* removal versus 2.5-log *Giardia* removal and 1.0-log virus removal versus 2.0-log virus removal). Hence additional disinfection credit is required. The process capacity of the WTP in a direct filtration mode is 120 mgd.

1.3.3.2 WTP Hydraulic Capacity

Although from a direct filtration treatment capacity standpoint the WTP is considered rated to 120 mgd, the WTP cannot be operated for sustained periods above about 110 mgd due to hydraulic limitations through the plant. The existing WTP was evaluated to determine what hydraulic bottlenecks might exist and identify improvements that would increase hydraulic capacity. Short-term improvements identified to improve WTP hydraulic capacity include:

- Raising the emergency overflow weir elevation from 420.20 feet to 421.20 to allow for an additional 1.0 foot of filter head without overflow.
- Removing the “blanked” off sections of the sedimentation basin launders to expose additional v-notch weirs. This will double the number of v-notches, slightly reduce the water surface elevation in the sedimentation basin, and help better distribute the flow into the launder.
- Stiffening the sedimentation basin launders against oscillation with horizontal bracing or additional supports.
- Adding additional holes in the sedimentation launders to prevent flow over the weirs.
- Reducing the sloshing and overflow that occurs at the Rapid Mix Boxes at flows of 120 mgd or less by:
  - Increasing the size of the rectangular openings between Rapid Mix Zone 1 and Zone 2 (two openings, one per treatment train).
  - Increasing the size of the 32 inlet holes in the Flocculation Basin Distribution Troughs (or add additional holes).

1.3.4 Water Treatment Plant Capacity Expansion

For the year 2030 planning horizon, without consideration of conjunctive use, a maximum WTP capacity of 150 mgd is required to meet spring-summer-fall water demands of the existing District wholesale and retail service area, and 75 mgd is required for winter-time demands. This assumes full use of existing water rights and contracts. This Master Plan also develops strategies for maximizing the capacity of the WTP at the existing site to an upper limit of 240 mgd for spring-summer-fall demands and 120 mgd for winter-time demands to help meet other potential regional water demands. This Master Plan does not evaluate potential reductions in WTP capacity due to conjunctive use programs.

Alternatives were developed to accommodate the treatment and transmission of high quality potable water for a peak day treatment capacity of a minimum 150 mgd to a maximum 240 mgd by the year 2030. The two expansion scenarios are referred to as Long-Term 75/150 mgd and Long-Term 120/240 mgd.

1.3.4.1 Long-Term 75/150 mgd

The long-term 75/150 mgd (LT 75/150) maximum WTP capacity alternative assumes that the District would limit expansion to full use of existing water rights and contracts and that the future demand pattern will be similar to the existing one. This demand pattern would consist of a winter-time demand of 75 mgd that could be treated with a conventional filtration treatment process and a spring-summer-fall demand of 150 mgd that could be treated with a direct filtration treatment process.

The LT 75/150 expansion implementation could proceed within the District's available property at the existing plant. Hydraulic improvements (including new pipelines and channels) would be necessary within and between the various process units. The expansion would require modifications to the existing flocculation-sedimentation basins, a new filter basin, and new backwash and solids handling facilities along with other identified improvements.

1.3.4.2 Long-Term 120/240 mgd

The long-term 120/240 mgd (LT 120/240) maximum WTP capacity alternative would involve the District changing its existing role to that of a regional agency. Under LT 120/240, the District would continue to deliver treated water to its existing wholesale and retail customers and would also supply treated water to customers within an expanded service area. The evaluations in this Master Plan assume a similar demand pattern to the existing demand pattern, with a much lower demand in winter than in summer.

For this scenario, existing pipelines and channels within the WTP will not be adequate for the hydraulic requirements of LT 120/240. Plant modifications to provide additional hydraulic capacity would be significant, including new plant influent piping, larger channels and piping between the pretreatment basins and filters than required for LT 75/150, and additional piping between the filters and Hinkle Reservoir. Land also would need to be acquired for expanded pretreatment facilities and for filtration facilities for WTP capacities exceeding 180 mgd.

The existing WTP configuration can accommodate modular expansion. Based on our review of the WTP and process requirements, a phased expansion approach of 30 mgd increments is recommended for LT 120/240. The first phase of expansion would be significant. A new flocculation-sedimentation basin, a new filter basin, and the construction of large "backbone" improvements such as piping and channels that would eventually accommodate the ultimate 240 mgd WTP capacity are required. Chemical storage tanks, pumps, and other mechanical equipment could be phased in to the WTP process in a logical, economical fashion.

A comparison of the costs for the LT 75/150 and LT 120/240 WTP expansions is presented in Table 1-3.

Table 1-3
Conceptual Level Estimate of Capital Costs
LT 75/150 and LT 120/240 Water Treatment Plant Expansions

Direct Filtration Treatment Capacity Expansion, MGD	LT 75/150	LT 120/240				LT 120/240
	120-150	120-150	150-180	180-210	210-240	TOTAL ($)
Pretreatment	5,502,000	10,111,000		138,000		10,249,000
Filter Improvements	9,360,000	9,360,000		3,853,000	3,641,000	16,854,000
Backwash Recovery System	2,537,000	2,537,000	1,045,000		1,045,000	4,627,000
Solids Handling	3,792,000	4,340,000		1,296,000		4,486,000
Chemical Feed Systems	1,135,000	1,135,000	2,486,000	615,000	615,000	4,851,000
Sitework	1,800,000	2,100,000	600,000	350,000	350,000	3,400,000
Electrical & Instrumentation	5,430,000	6,657,000	930,000	1,407,000	1,272,000	10,266,000
Subtotal	$29,556,000	$36,240,000	$5,061,000	$7,659,000	$6,923,000	$54,733,000
Contingency @ 25%	7,389,000	9,060,000	1,265,250	1,914,750	1,730,750	13,683,250
Engineering, Legal, and Administrative @ 25%	7,389,000	9,060,000	1,265,250	1,914,750	1,730,750	13,970,750
Totals	**$44,334,000**	**$54,360,000**	**$7,591,500**	**$11,488,500**	**$10,384,500**	**$ 83,824,500**

1.4 Hinkle Reservoir

Hinkle Reservoir is the final component of the District's water supply and treatment system. It is a 62-million gallon (MG) lined and covered earthen reservoir that acts as the clearwell for treated water from the WTP as well as a facility for system storage. Water stored in Hinkle Reservoir flows by gravity to the District's wholesale customers and a portion of its retail service area. Additional water is pumped to the remainder of the retail service area and part of the City of Folsom.

Hinkle Reservoir. The District's WTP can be seen to the left. A portion of the Bureau's 84-inch pipeline and one standpipe can be seen at the top of the photograph.

1.4.1 Hinkle Reservoir Cover

Construction of the floating membrane cover system on the Hinkle Reservoir was completed in 1980. The cover is guaranteed for a period of 25-years. Since it is now over 20 years old, an evaluation of the cover was performed in order to recommend measures to extend the life of the cover, or recommend options to replace the cover if it is nearing the end of its service life.

The Hypalon floating cover system is in very good condition. Laboratory testing of extracted samples indicate that the cover material, seams, and associated attachments appear to have a minimum remaining service life of 20 years with proper maintenance. A comprehensive 20-year maintenance cleaning should be completed with subsequent periodic cleaning approximately once every two years. More frequent cleaning is not recommended due to the increased potential for mechanical damage to the cover. During the 20-year maintenance cleaning, several minor holes and tears should be repaired, calking should be replaced, sumps should be cleaned of debris, and areas where the Hypalon attaches to the inlet and outlet structures should be repaired or replaced.

On the basis of the proven performance of the existing cover and a comparison of alternative costs, a Hypalon floating cover system is recommended for the Hinkle Reservoir when cover replacement is required.

The existing reservoir is configured as a single 62-MG storage reservoir. This does not permit continued delivery of treated water from the reservoir during periods when maintenance and cleaning activities must be conducted. The DHS recommends that Hinkle Reservoir be divided to permit taking one side off-line for cleaning and other maintenance activities while maintaining the other half in service. Dividing the existing reservoir into two sections will result in redundancy and add reliability features to the treated water supply. The reservoir should be divided either before or during cover replacement.

The estimated cost for replacing the Hinkle Reservoir cover and dividing the reservoir into two sections is estimated at $4,755,000.

1.4.2 Cooperative Pipeline Connection

A portion of the treated water currently bypasses Hinkle Reservoir through the Cooperative Transmission Pipeline. Disinfection credit for the WTP must be achieved ahead of Hinkle Reservoir as the water flows through other treatment units. As WTP capacity increases, the available disinfection contact time (CT) will not be sufficient to meet that required. At WTP flow rates above approximately 180 mgd with a third pretreatment basin in operation, additional disinfection CT is required in a direct filtration treatment mode of operation. Without a third basin, there is insufficient disinfection CT in a direct filtration treatment mode of operation at a WTP capacity above approximately 130 mgd. To meet future disinfection credit requirements, the Cooperative Transmission Pipeline connection will need to be moved to the reservoir outlet pipe or to a new outlet structure located to ensure CT credit through the reservoir.

A direct pipeline connection between the existing 78-inch Cooperative Transmission Pipeline and the existing 84-inch reservoir outlet pipeline is the recommended alternative for relocating the cooperative pipeline treated water connection. The estimated cost for relocating the Cooperative Transmission Pipeline Connection is $1,177,000.

1.5 Recommended Improvements Plan

An implementation plan was prepared for the improvements recommended for the raw water transmission facilities, an expanded water treatment plant, and Hinkle Reservoir. Improvements that may be necessary for the Bureau's Folsom Pumping Plant, repairs or rehabilitation of the Bureau's 84-inch transmission pipeline, or a parallel 84-inch transmission line to the Bureau's 84-inch transmission line to provide redundancy under a 150 mgd maximum WTP capacity scenario are outside the scope of this Master Plan and were not included. The implementation schedule also does not account for changes in water use patterns or demands under a conjunctive use water supply approach.

The recommended improvements plan matches the recommendations provided for an ultimate WTP capacity expansion to 240 mgd. The initial capital improvements scheduled through 2002 (backwash and solids handling facilities) are recommended to optimize the existing WTP capacity and address the biggest operational and maintenance issues associated with the District's facilities. The exact timing of capital improvements scheduled for the period of 2002 to 2030 will be driven by actual growth and demand factors.

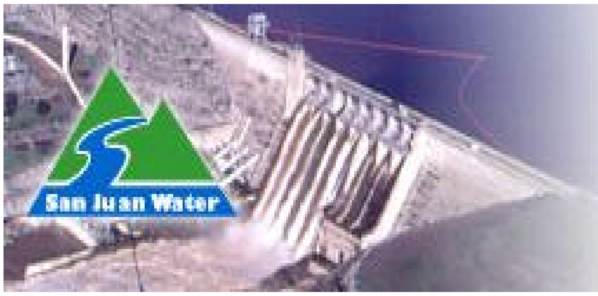
The Recommended Improvements Plan is shown in Table 1-4.

Table 1-4
Project Implementation Schedule
Year 2002 - 2030

Year - WTP Capacity (mgd)	Project Description	Cost
2002 - 60/120	Filter Backwash Hoods	$3,300,000
	New Backwash Treatment and Recovery System	3,805,500
	New Solids Handling System	$6,510,000
	Estimated Capital Improvements Cost Schedule - 2001	$13,615,500
2002 - 60/120	Chlorine System (Structure and Scrubber)	$750,000
	Estimated Capital Improvements Cost Schedule - 2002	$750,000
2002 - 2009 75/150	30 mgd WTP Expansion	$39,994,500
	District Raw Water Pipeline Rehabilitation	1,006,500
	66-inch Raw Water Pipeline within District Property	1,207,500
	Cooperative Pipeline Connection Relocation (Assumes In-line Filtration Desired)	1,177,000
	Estimated Capital Improvements Cost Schedule - 2002 through 2009	$43,385,500
2010 - 2016 90/180	30 mgd WTP Expansion	$7,591,500
	66-inch Raw Water Pipeline (Parallel Bureau 84-inch Pipeline)	7,267,500
	Estimated Capital Improvements Cost Schedule - 2010 through 2016	$14,859,000
2017 - 2023 105/210	30 mgd WTP Expansion	$11,488,500
	Estimated Capital Improvements Cost Schedule - 2017 through 2023	$11,488,500
2023 - 2030 120/240	30 mgd WTP Expansion	$10,384,500
	Hinkle Cover Replacement, Divide Reservoir[3]	4,755,000
	Estimated Capital Improvements Cost Schedule - 2010 through 2016	$15,139,500
	Total Capital Improvement Costs - 2001 through 2030	$99,238,000

1. Costs based on January 2001 Engineering News Record (ENR) Construction Cost Index of 6,281
2. Cost estimates include a 25 percent estimating contingency and a 25 percent allowance for planning, engineering, administrative and legal expenses, and construction management associated with project implementation.
3. The District should consider the benefits of dividing Hinkle Reservoir prior to 2023 as discussed in Section 8.2.
4. Schedule represents the year improvements should be completed.

Chapter 2: Introduction



2.1 Background

San Juan Water District (District) is a community services district created by voters in 1954. Located in Granite Bay, California, the District currently serves more than 180,000 people in eastern Sacramento County and south Placer County. The District wholesales water to Citrus Heights Water District, Fair Oaks Water District, Orange Vale Water Company, and the City of Folsom (north of the American River) and periodically to Northridge Water District. The District also wholesales water to approximately 9,000 customers in Granite Bay and the northeast portion of Sacramento County, which is the District retail area.

The District's sole source of water supply is Folsom Reservoir, which is fed from the North and South Forks of the American River. Water is moved either by gravity or by pumping from the United States Bureau of Reclamation's (Bureau's) pumping plant located at the base of Folsom Dam. An 84-inch pipe from the Bureau's facilities splits into a 72-inch and then into a 54-inch diameter pipe that conveys water to the District's 100-million-gallon-per-day (mgd) Sidney N. Peterson Water Treatment Plant (WTP). At the WTP, it undergoes extensive treatment to ensure the highest quality of water for District customers. From the WTP, the water flows to the 62-million-gallon (mg) Hinkle Reservoir. The District also maintains approximately 163 miles of pipeline, which transports the high-quality, treated water to wholesale and retail customers.

In addition, the District is one of the American River Basin Cooperating Agencies who are developing a Regional Water Master Plan to ensure a reliable, high-quality water supply for the next 30 years and beyond. Partnering with 13 other water providers, the District is working to encourage resource conservation, regional planning, and finding ways to boost efficiency and productivity. The broad goals of this process are to provide an economic, high quality, reliable water supply while protecting aesthetic and environmental resources. As part of this plan, the District has agreed to a regional conjunctive use program that will optimize the use of surface water during wet years and save groundwater for drier years.

As local agencies continue to explore cooperative regional programs, the District is a logical agency to play a major role. Because of its existing infrastructure (large surface water treatment plant and extensive water transmission and distribution systems), the District is well positioned to participate in the treatment and transfer of water to an expanded customer base.

2.2 Objectives and Scope

The objective of this Master Plan is to assess the District's current water supply and treatment facilities and develop alternative actions where appropriate to accommodate the treatment and transmission of a reliable, high quality water supply for peak supply capacities ranging between 120 mgd and 240 mgd. Initially, the District is interested in identifying ways to immediately increase the reliable capacity of the existing WTP to 120 mgd to meet short-term water demands. For the year 2030 planning horizon of this report, the capacity requirement for the WTP to meet the wholesale and retail area water demand is 150 mgd. However, the District estimates that a WTP capacity of as much as 240 mgd might be required to assist in meeting regional demands.

In addition to evaluating short-term and long-term WTP requirements, this Master Plan evaluates the facilities utilized in diverting source water from Folsom Reservoir, transmitting the source water to the WTP, and storing the water for distribution from the WTP. Transmission delivery systems as well as other regulatory, fiscal, administrative, and operational considerations are addressed in other District programs.

Specific goals of this Master Plan are to:

- Assess the current raw water transmission, treatment, and storage facilities for meeting changing capacity and/or treatment requirements.
- Develop alternative actions to ensure the treatment and transmission of an adequate, reliable, high quality water supply.
- Identify a practical approach to project sequencing and an incremental implementation plan that is economical while providing a high degree of reliability and ease of operation.

Ideas and recommendations for this Master Plan reflect consensus from District engineering and operations staff as well as other stakeholders.

2.3 Planning Assumptions

2.3.1 Planning Period

This Master Plan is based on a planning period through the year 2030.

2.3.2 Water Demands

Water demands used in this Master Plan were obtained from other recently prepared environmental documents and reports. In particular, the following references were relied on as definitive sources of water demand data:

- "Increasing Water Supply Pumping Capacity at Folsom Dam," ESA Consultants, Inc., January 1996. 1995, 2020 Annual Supply requirements taken from Report Table 5-2A.
- "American River Basin Cooperating Agencies Regional Water Master Plan Phase I Final Report," Montgomery Watson, et al, 1999. 2030 Annual Supply requirements taken from Table 28, except as noted otherwise.
- "San Juan Water District Schedule of Water Deliveries to Wholesale Agencies for the Period 1985 to 2030," provided by Shauna Lorance, Assistant General Manager, San Juan Water District, email January 5, 2000.

This Master Plan does not evaluate potential reductions or increases in water demands due to conjunctive use programs. The Master Plan also assumes the District can make full use of existing water rights and contracts. The District should review and analyze the impacts of regional planning and conjunctive use on the recommendations contained in this Master Plan.

This Master Plan assumes that demands will grow in a straight-line projection during the study period with the future annual demand profile similar to the existing seasonal demand pattern.

2.3.2.1 Annual Demands

The Bureau facilities provide water to the District, City of Roseville, City of Folsom, and Folsom Prison. Therefore, the total annual supply for each agency was compiled and is listed in Table 2-1 for the years 1995, 2020, and 2030. The District's 2030 demand is listed for both a 150-mgd and 240-mgd plant capacity condition. The 240-mgd demand was assumed to be a maximum day demand. This was projected to an annual required supply using the existing District demand profile.

Table 2-1
Annual Demands

	1995[a] acre feet	2020[a] acre feet	2030[b] acre feet	2030[c] acre feet
San Juan Water District	53,100	82,200	82,200[d]	131,520
City of Roseville	17,855	46,950	54,900	54,900
City of Folsom	15,500	34,400	34,000	34,000
Folsom Prison	2,172	2,900	2,900[a]	2,900[a]
Total	88,627	166,450	174,000	223,320

(a) 1995, 2020 Annual Supply requirements taken from ESA January 1996 Report Table 5-2A.
(b) 2030 Annual Supply requirements taken from Regional Water Master Plan Table 28, except as noted otherwise.
(c) 2030 Annual Supply requirements (240-mgd option) assumed 240-mgd maximum day demand with annual demand profile similar to existing.
(d) 2030 San Juan Supply requirements (150-mgd option) provided by San Juan Water District.

2.3.2.2 Maximum Day Demands

Maximum day demand was used to determine required pumping, pipeline, and WTP capacity. It was assumed that peak hour demands would continue to be provided through treated water storage. The City of Roseville is served by the Bureau through the Folsom Pumping Plant and 84-inch transmission pipeline that supplies the District. Their maximum day demand was considered in the evaluations in this Master Plan. Table 2-2 presents the maximum day demand for the District and the City of Roseville.

Table 2-2
Maximum Day Water Demand

Agency Name	150-mgd District WTP	240-mgd District WTP
San Juan Water District	150 mgd	240 mgd
City of Roseville[a]	100 mgd	100 mgd
Folsom, Others	NA	NA
Total	250 mgd	340 mgd

(a) Nominal 100 mgd used for City of Roseville, 150 cfs = 96.94 mgd actual.

2.3.3 Cost Estimates

Cost estimates are based on an Engineering News Record (ENR) Construction Cost Index of 6281 (in effect January 2001). Estimates include a 25 percent contingency for estimating and construction uncertainties and a 25 percent allowance for planning, engineering, construction management, administrative, and legal expenses associated with project implementation.

2.4 Report Organization

Following this introductory chapter, the Master Plan is divided into the following sections:

Chapter 3: Presents the evaluation for the raw water conveyance system and describes improvements for short term (120 mgd), long-term 150 mgd, and long-term 240 mgd conditions. The raw water demand is documented, and projections are made regarding the existing and future ability of the Bureau's Folsom Pumping Plant to meet these demands. Chapter 3 also evaluates the capacity of the District's raw water transmission pipelines and assesses the condition of these pipelines.

Chapter 4: Presents current and future water treatment regulatory requirements and identifies compliance issues of most relevance to the District.

Chapter 5: Provides recommendations regarding treated water quality objectives, the impact of source water quality on the WTP, the potential impacts of planned changes in lake management practices for Folsom Reservoir, additional water quality monitoring for the expanded WTP, and an approach to address water quality issues.

Chapters 6 and 7: Describe, respectively, current treatment plant capacity/short-term improvements and future treatment plant capacity/long-term improvements. Chapter 6 presents the process capacity evaluation of the existing WTP, a hydraulic capacity evaluation, and short-term process modification alternatives to increase WTP capacity to 120 mgd. Chapter 7 addresses strategies to meet the District's long-term objective of incrementally expanding the capacity of the WTP to meet increasing water demands through build-out of its service area. Two long term (LT) scenarios are developed: Long Term 75/150, through which the WTP would be expanded to 150 mgd to meet the needs of the District's current customers, and LT 120/240, through which the District would develop capacity of 240 mgd in order to provide for regional water needs.

Chapter 8: Presents the results of an evaluation of the condition of the Hinkle Reservoir cover system, including options for extending the life of the cover or replacing it if necessary, and an evaluation of the potential of the reservoir to improve the WTP's ability to comply with disinfection CT requirements and treated water storage goals.

Chapters 9: Provides a summary of the recommended improvements plan, estimate of project costs, and implementation schedule.

Chapter 10: Lists the various references used to complete this report.

2.5 Acknowledgements

This Master Plan was prepared by Kennedy/Jenks Consultants, in association with Black & Veatch Corporation. Specialized geosynthetics consulting services were performed by R.K. Frobel & Associates of Evergreen, Colorado. Melissa Blanton provided technical editing and writing guidance.

Although we would like to acknowledge and thank all of the San Juan Water District staff who provided input and assistance in this effort, special mention goes to Ms. Shauna Lorance, Mr. Michael O'Bleness, and Mr. Bill Sadler, who contributed many hours to provide information, guidance, and technical review. Mr. Lyle Hoag, San Juan Water District Board of Directors, deserves recognition for his thoughtful review and "what if" questions that helped to ensure sound recommendations in the report.

2.6 Key Terminology

Acronyms and abbreviations used in this report are as follows:

AI	Aggressive Index
AL	action limit(s)
Avg.	average
AWWA	American Water Works Association
BAT	Best Available Technology
Bureau	United States Bureau of Reclamation
°C	degree Celsius
Cal/ARP	California Accidental Release Program
CaO	Lime
CAP	*Cryptosporidium* Action Plan
CFE	combined filter effluent
cfs	cubic feet per second
CFU	colony forming unit
CL_2	chlorine
CLI	Colorado Linings International
CSO	combined sewer overflow
CSPE	chlorosulfonated polyethylene (Hypalon)
CT	contact or value time (disinfection concentration times contact time)
DBPPs	DBP precursors
DBPs	disinfection by-products
D/DBPR	Disinfectants/Disinfection By-Products Rule
DHS	California Department of Health Services
District	San Juan Water District
DSE	distribution system evaluation
EBMUD	East Bay Municipal Utility District
EOW	emergency overflow weir
ESWTR	Enhanced Surface Water Treatment Rule
°F	degree Fahrenheit
FBR	Filter Backwash Rule
FBRR	Filter Backwash Recovery Rule
fps	feet per second
ft^3, cf	cubic feet
GAC	granular activated carbon
gpd/ft	gallons per day per feet
gph	gallons per hour
gpm	gallons per minute
gpm/sf, gmp/ft^2	gallons per minute per square foot
Gt	Camp Number (dimensionless value)
HAA5	haloacetic acids, sum of the concentrations of mono-, di-, and trichloroacetic acids and mon- and dibromoacetic acids that are drinking water chlorination by-products
HDT	hydraulic detention time
HGL	hydraulic grade line
HOCL	chlorine (solution)
hp	horsepower
ICR	Information Collection Rule
IESWTR	Interim Enhanced Surface Water Treatment Rule

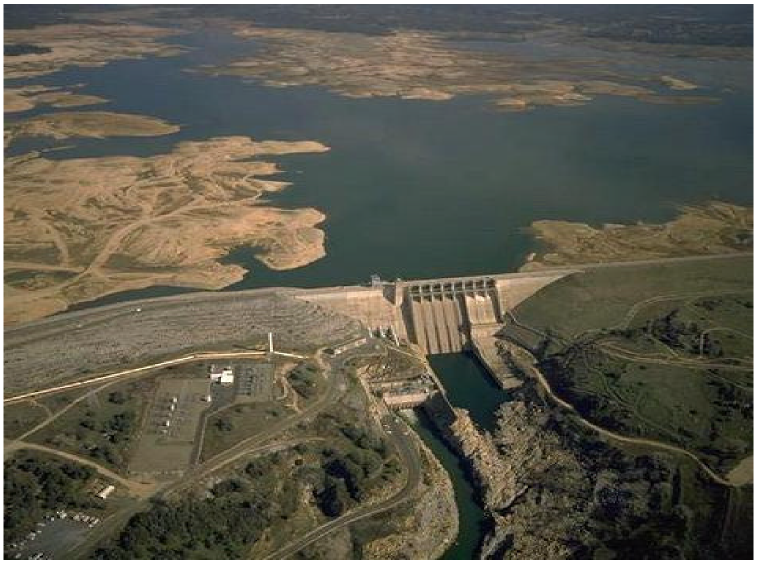
IOCs	inorganic compounds
kW	kilowatt
lb.	pound
LCR	Lead and Copper Rule
LI	Langelier Index
log	Logarithm. In this report (and water treatment) log is typically used to represent a percent removal or inactivation of *Giardia* cysts or enteric viruses. One-half log is equivalent to 68 percent, 1.0-log equals 90 percent, 2.0-log equals 99 percent, 3.0 log equals 99.9 percent, etc.
LRAA	long running annual average
LT	long term
LT 75/150	long-term 75/150 mgd
LT 120/240	long-term 120/240 mgd
LT 2 ESWTR	Long-Term 2 ESWTR
LT 3 ESWTR	Long-Term 3 ESWTR
MCL	maximum contaminant level
MCLGs	maximum contaminant level goals
M/DBP	Microbial/Disinfection By-Product
MG, mg	million gallons
mgd	million gallons per day
mg/L	milligrams per liter
mil	milliliter
mm	millimeter
MRDLGs	MRDL Goals
MRDLs	maximum residual disinfectant levels
MTBE	methyl tertiary butyl ether
NOM	natural organic matter
NSF	National Sanitation Foundation
NTU	nephalometric turbidity unit
O&M	operations and maintenance
ORP	oxidation reduction potential
POE	point-of-entry
PSW	Partnership for Safe Water
Reg-Neg	regulatory-negotiation (process)
RMP	Risk Management Plan
rpm	revolutions per minute
SCVWD	Santa Clara Valley Water District
SDWA	Safe Drinking Water Act
sec.	second
sf	square feet
SFWD	San Francisco Water Department
SMCL	secondary maximum contaminant limits
SNAGMA	Sacramento North Area Groundwater Management Authority
SOC	synthetic organic chemical
SWTR	Surface Water Treatment Rule
T&O	taste and odor
TCD	temperature control device
TCR	Total Coliform Rule
TDS	total dissolved solids
THMs	trihalomethanes
TOC	total organic carbon
TT	treatment technique

TTHMs	total trihalomethanes
μg/L	micrograms per liter
UL	Underwriters Laboratory
USEPA	United States Environmental Protection Agency
UV	ultraviolet light
VFD	variable frequency drive
VOCs	volatile organic compounds
WTP	water treatment plant

Chapter 3: Raw Water Pump Station and Pipeline

3.1 Introduction

This chapter of the Master Plan documents the condition and capacity of the existing raw water pump station and pipelines providing water from Folsom Dam to the District's WTP. The initial task was to confirm that adequate capacity exists in the Folsom Pumping Plant and address required improvements to the District's transmission pipeline to meet 150-mgd and 240-mgd flow requirements. It became apparent during the evaluation that there are limitations to the pumping plant, Bureau pipeline, and District pipelines that impact the ability to provide 150 mgd and, ultimately, 240 mgd. This chapter discusses the evaluation, presents the findings, and provides recommendations for improvements to convey the District's raw water supply up to 240 mgd. Table 3-1 presents a summary of the evaluations and recommendations discussed in this chapter.



Folsom Dam

The facilities evaluated include the Bureau's Folsom Pumping Plant and 84-inch raw water transmission pipeline, and the District's raw water transmission pipelines from the Hinkle Wye to the existing WTP. Figure 3-1 shows the location and description of the facilities evaluated.

In general, the evaluation found that the Folsom Pumping Plant and raw water pipelines are capable of meeting the District's short-term demands up to approximately 120 mgd without further improvements. The installed pump capacity, however, leaves the District exposed to future water supply shortages, when the level of Folsom Lake is below elevation 392. The risk occurs as the overall demand on the Bureau's facilities approaches 400 cfs (258.5 mgd).

A WTP expansion to 150 mgd will require improvements to the District's leg of the raw water transmission piping system to reduce headloss and velocities exceeding 20 fps. In addition, a parallel pipeline to the Bureau's 84-inch line is recommended to further reduce the hydraulic impacts to the pumping plant, provide transmission redundancy, and improve access for maintenance to the 84-inch pipeline. The Folsom Pumping Plant will require the replacement of one of the existing pumps to meet the 150-mgd demand level.

A WTP expansion to 240 mgd requires raw water pipeline improvements similar to, but larger than, the 150-mgd option. However, significant improvements will be required at the Folsom Pumping Plant and dam intake to accommodate the increased flows. The dam intake improvements include consideration of an anti-vortex feature and possibly additional pipeline capacity through the dam to reduce peak velocities.

Table 3-1
Raw Water Transmission System Evaluation and Recommendation Summary

Component Description	Short Term (120 mgd)	Long Term (150 mgd)	Long Term (240 mgd)
Folsom Pumping Plant			
Capacity to meet peak demands	No improvements required.	Hydraulic Limitations - Provide additional pump(s).	Hydraulic Limitations – Expand pumping plant or construct new pumping plant.
Expansion capacity	No improvements required.	Expansion possible with larger pumps retrofit into existing pump bays.	Expansion will require pumping plant reconfiguration.
Hydraulic limitation of intake pipeline and vortex formation at peak flows with low lake level	No improvements required.	No improvements required.	Hydraulic Limitations through dam penetration isolation valve.
Proposed multi-level temperature control intake device (TCD)	Project designed; Bureau reports no hydraulic impact.	Project designed; Bureau reports no hydraulic impact; provide anti-vortex features with TCD.	Project designed; Bureau reports no hydraulic impact; provide anti-vortex features with TCD.
Isolation and emergency shutoff capacity	No improvements required.	No improvements required.	Pumping plant modifications should include isolation and emergency shutoff provisions.
Bureau 84-Inch Raw Water Pipeline			
Velocity limitations at peak flows	No improvements required.	Hydraulic Limitations - 48” parallel pipeline required at District (see below).	Hydraulic Limitations - 84” parallel pipeline required.
Hydraulic grade limitation at twin standpipe surge structures	Approaching maximum water surface elevation.	Hydraulic Limitations - 48” parallel pipeline required at District (see below).	Hydraulic Limitations - 84” parallel pipeline required.
Coal tar lining age and condition	Lining 48-years old and failing. Inspect to confirm condition.	Lining 48-years old and failing. Inspect to confirm condition.	Lining 48-years old and failing. Inspect to confirm condition.
Redundancy and reliability	No redundancy.	Parallel pipeline provides redundancy.	Parallel pipeline provides redundancy.
District Raw Water Pipelines			
Velocity limitations at peak flows	No improvements required.	Hydraulic Limitations - 48” parallel pipeline to 54” plant influent required.	Hydraulic Limitations - 66” parallel pipeline to 54” plant influent required.
Maintenance access and lining repairs	Interior pipeline joint repair required.	Additional manways required; interior relining required.	Additional manways required; interior relining required.
Corrosion Control	Install cathodic protection.	Install cathodic protection.	Install cathodic protection.
Operational flexibility and valve location	Replace existing 54” gate valve. Provide new 54” valves for isolation.	Install valves to isolate new 48” parallel pipeline.	Install valves to isolate new 66” parallel pipeline.

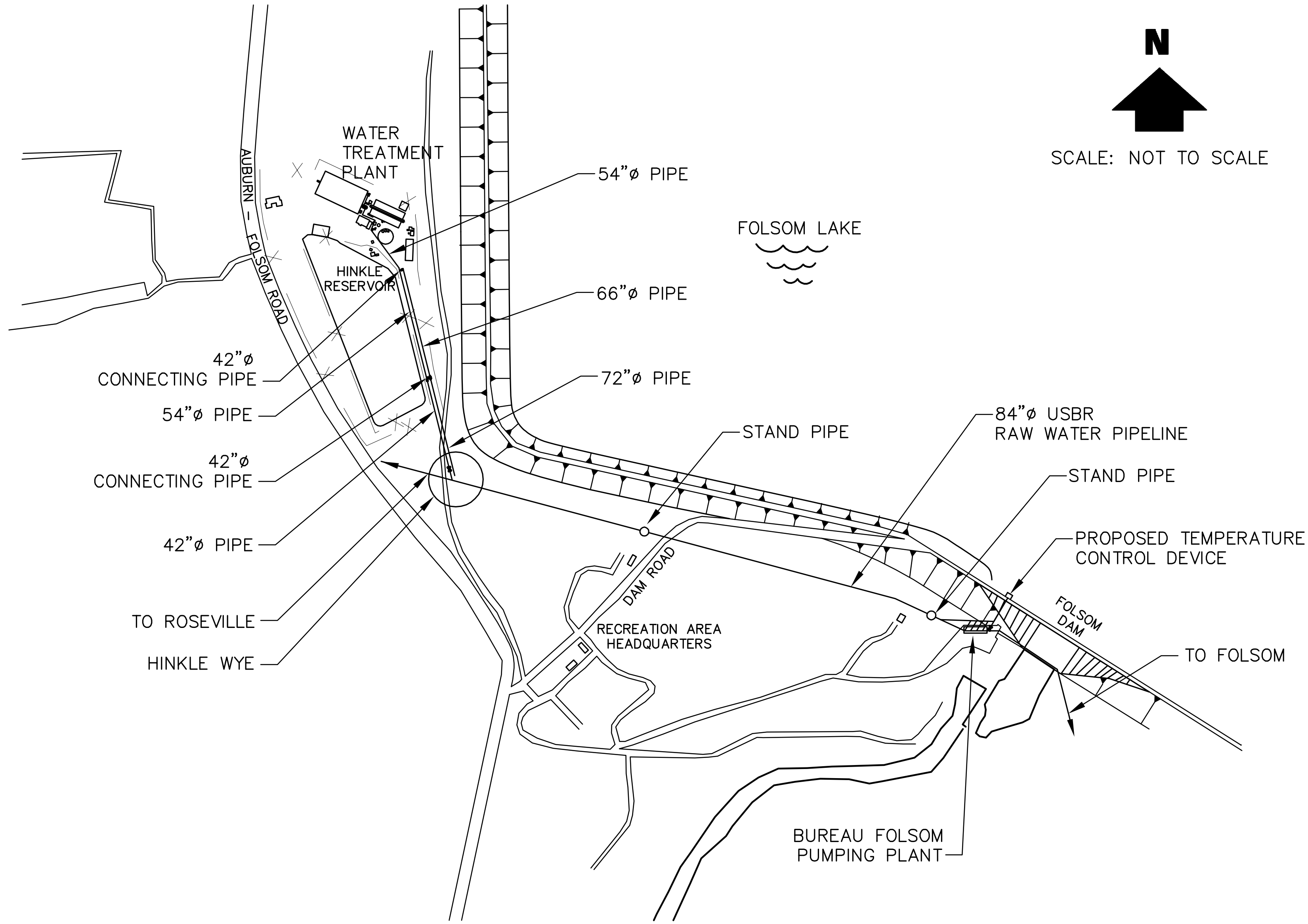


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FIGURE 3-1
EXISTING RAW WATER TRANSMISSION FACILITIES

The condition of the existing District pipelines and the condition of the Bureau's 84-inch pipeline lining have resulted in recommendations to repair the linings and consider cathodic protection. Concerns about the reliability of the single 84-inch pipeline from the Folsom Pumping Plant support the recommendation to parallel the pipeline. Further recommendations for reliability include addition of line valves for isolation and access manways for inspection/repair of the District pipelines.

Peak power demand findings show that the existing installed hp exceeds an incremental 1000 kW attributable to pumping the District's share of the total water pumped. Discussions with the District's power consultant indicate that electrical power capacity is available to service whatever installed pump demands are required. The ultimate electricity provider and power cost are under review outside the scope of this Master Plan.

Additional recommendations contained in this chapter, but not covered in Table 3-1, include:

- Discuss the requirement for an anti-vortex device on the TCD with the Bureau.
- Discuss pump redundancy requirements at the Folsom Pumping Plant with the Bureau. An appropriate criteria is a standby pump equal in size to the largest pump.
- Test VFD flow control operation at the Folsom Pumping Plant as soon as possible to access any impacts from implementing this flow throttling approach.
- Request the Bureau conduct an expansion feasibility study for the Folsom Pumping Plant. The impacts of a major renovation and expansion should consider all users.
- Request the Bureau fully inspect its 84-inch pipeline and develop and implement a rehabilitation program within the next 10 years.

Detailed discussion of the evaluation and recommendations development is presented in the following sections.

3.2 Raw Water Demand

The evaluation of pumping capacity and pipeline adequacy requires the use of instantaneous flow data. Annual water demand projections were converted into gallons per minute (gpm) flow rates for each month as well as a maximum day demand. Two key demand thresholds were evaluated: 150 mgd based on the historic District Year 2030 planning value and the possible increased future demand of 240 mgd. The short-term objective of 120 mgd was also reviewed to determine existing system adequacy.

3.2.1 Annual Demands

In order to evaluate the raw water transmission facilities, it is important to recognize that, in addition to providing water to the District, the Bureau facilities provide water to the City of Roseville, City of Folsom, and Folsom Prison. The total annual supply is listed in Table 3-2 for the years 1995, 2020, and 2030. The District's 2030 demand is listed for both a 150-mgd and 240-mgd plant capacity condition. The 240-mgd demand was assumed to be a maximum day demand. This was projected to an annual required supply using the existing District demand profile.

Table 3-2
Folsom Pumping Plant
Annual Demands

	1995[(a)] acre feet	2020[(a)] acre feet	2030[(b)] acre feet	2030[(c)] acre feet
San Juan Water District	53,100	82,200	82,200[(d)]	131,520
City of Roseville	17,855	46,950	54,900	54,900
City of Folsom	15,500	34,400	34,000	34,000
Folsom Prison	2,172	2,900	2,900[(a)]	2,900[(a)]
Total	88,627	166,450	174,000	223,320

(a) 1995, 2020 Annual Supply requirements taken from ESA January 1996 Report Table 5-2A.
(b) 2030 Annual Supply requirements taken from Regional Water Master Plan Table 28, except as noted otherwis e.
(c) 2030 Annual Supply requirements (240-mgd option) assumed 240-mgd maximum day demand with annual demand profile similar to existing.
(d) 2030 San Juan Supply requirements (150-mgd option) provided by San Juan Water District.

3.2.2 Monthly Average Demands

Monthly demand detail was developed to estimate pumping plant horsepower requirements and ultimately the peak power requirement. Monthly aggregate demand is presented in Table 3-3 for the 150-mgd and 240-mgd levels. The projections include the District's demand (either 150 mgd or 240 mgd) plus the demands of the cities of Folsom and Roseville and Folsom Prison. The maximum demand shown in the table, 535 cfs for July, is equivalent to 345 mgd.

Table 3-3
Folsom Pumping Plant
Monthly Average Total
System Demands[(a)]

Month	2030 (150 mgd) (cfs)	2030 (240 mgd) (cfs)
January	123	160
February	111	142
March	154	203
April	204	273
May	267	350
June	350	471
July	398	535
August	386	514
September	315	417
October	231	299
November	138	170
December	122	154

(a) Monthly percentages of annual demand are based on the report "Increasing Water Supply Pumping Capacity at Folsom Dam" ESA Consultants Inc, January 1996, Table 5-2b.

3.2.3 Maximum Day Demands

Maximum day demand used to determine required pumping capacity and pipeline capacity corresponded to the District's maximum day demand plus 150 cfs for the City of Roseville. The Folsom component of the water demand does not pass through the 84-inch pipeline and was not included in the evaluation. Table 3-4 presents the maximum day flows from the Folsom Pumping Plant to the District and the City of Roseville.

Table 3-4
Maximum Day Water Demand

Agency Name	150-mgd Ultimate[a]	240-mgd Ultimate[a]
San Juan Water District	150 mgd	240 mgd
City of Roseville	100 mgd	100 mgd
Folsom, Others	NA	NA
Total	250 mgd	340 mgd

(a) Nominal 100 mgd used for City of Roseville, 150 cfs = 96.94 mgd actual.

3.3 Folsom Pumping Plant

The Folsom Pumping Plant was constructed in the early 1950s to offset the impact of the dam on existing canal delivery systems. The pumping plant was not required to operate until the 1970s when the District raised the hydraulic grade requirements with the construction of the current WTP. Subsequent growth throughout the region has resulted in increased reliance on the pumping plant to provide water when Folsom Lake's level drops to a point where gravity flow is no longer feasible.

Folsom Pumping Plant

The existing pumping plant remained in its original configuration until the late 1990s when the City of Roseville, in conjunction with the District, City of Folsom, Sacramento Area Flood Control Agency, and the Bureau completed an expansion of the plant to accommodate two variable speed 1,500 hp pumps and to relocate an existing pump (pump 7) to serve as an emergency supply pump.

The evaluation of the Folsom Pumping Plant initially consisted of confirming that adequate capacity existed to meet future District demands. Existing plant capacity limitations identified during the initial evaluation required a more detailed assessment of the future 150-mgd and 240-mgd District demand levels. This section discusses the evaluation approach and findings and provides recommendations for meeting the District's existing and future water demand requirements.

3.3.1 Existing Capacity

The current Folsom Pumping Plant equipment described below is the basis for determining the existing plant capacity. Table 3-5 shows the pump size and performance data reflecting the manufacturer's original pump curve, adjusted for in-plant headloss.

Table 3-5
Installed Pump Equipment – Folsom Pumping Plant

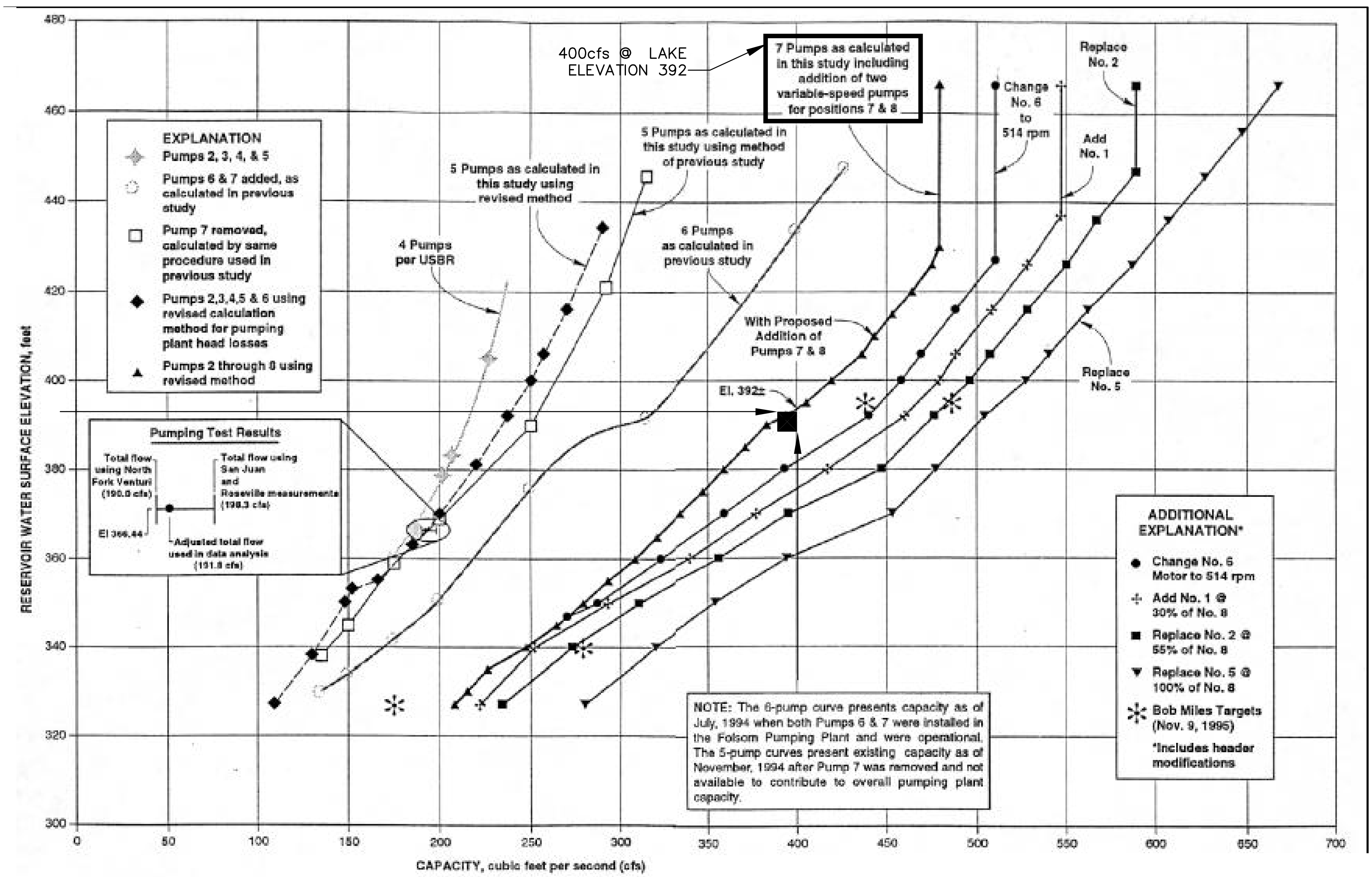
Pump Description	Pump Operating Conditions
Pump 1: No Pump Installed	Best Efficiency Point: Shut off Head
Pump 2: 25 cfs, 250 hp, 1200 RPM	Best Efficiency Point: 21 cfs @ 94 feet Shut off Head: 134 feet
Pump 3: 75 cfs, 600 hp, 720 RPM	Best Efficiency Point: 50 cfs @ 86 Feet Shut off Head: 119 Feet
Pump 4: 50 cfs, 400 hp, 900 RPM	Best Efficiency Point: 36 cfs @ 85 Feet Shut off Head: 111 Feet
Pump 5: 50 cfs, 400 hp, 900 RPM	Best Efficiency Point: 36 cfs @ 85 feet Shut off Head: 111 feet
Pump 6: 100 cfs, 600 hp, 1200 RPM	Best Efficiency Point: 84 cfs @ 21 feet Shut off Head: 80 feet
Pump 7: 110 cfs, 1,500 hp, Variable Speed	Best Efficiency Point: 95 cfs @102 feet Shut off Head: 152 feet
Pump 8: 110 cfs, 1,500 hp, Variable Speed	Best Efficiency Point: 95 cfs @102 feet Shut off Head: 152 feet

In addition to the pumps listed above, there is an existing emergency pump installed with a capacity from 30 to 80 cfs depending on reservoir elevation.

The pumping plant capacity shown in Table 3-5 is 400 cfs at a lake elevation of 392 feet. The District's share of the 400 cfs is 185 cfs (120 mgd) (ESA Consultants 1996). Figure 3-2 is a graphical diagram of alternative pumping plant modifications considered in the ESA January 1996 evaluation. The 7 pump with two variable-speed pumps was the alternative implemented and reflected in Table 3-5. Subsequent improvements have increased pumping capacity to approximately 434 cfs at a lake level of 392. However, the 400-cfs capacity has been used for the basis of this evaluation due to uncertainty regarding availability of the additional 34 cfs to the District.

Figure 3-3 is a plot of the maximum day water demand provided in Section 3.2 assuming straight-line growth through year 2030. The 400-cfs pumping plant capacity is shown as a horizontal line. The point where the demand crosses the 400-cfs capacity line is the approximate year when capacity will be exceeded. As can be seen on the figure, the capacity will be exceeded between 2010 and 2022.

The pumping plant capacity is reported at a specific lake elevation of 392 feet. Folsom Lake level data was reviewed for the period 1922 to 1999 to determine the District's susceptibility to reduced pumping capacity. The data shows that the lake has been below elevation 392 three times during the summer months in the last 78 years. The worst year reviewed was 1977 when lake levels fell to elevation 336 in September. The minimum pool, for reference, is 327 feet.



FOLSOM DAM – WATER SUPPLY SYSTEM
PUMPING CAPACITY WITH VARIOUS PUMP
MODIFICATIONS (MODIFIES FIGURE 1–1 OF
ESA JANUARY 1996)(BETCHART 9126196)

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FIGURE 3-2

FOLSOM PUMPING PLANT - PUMP CAPACITY CURVES

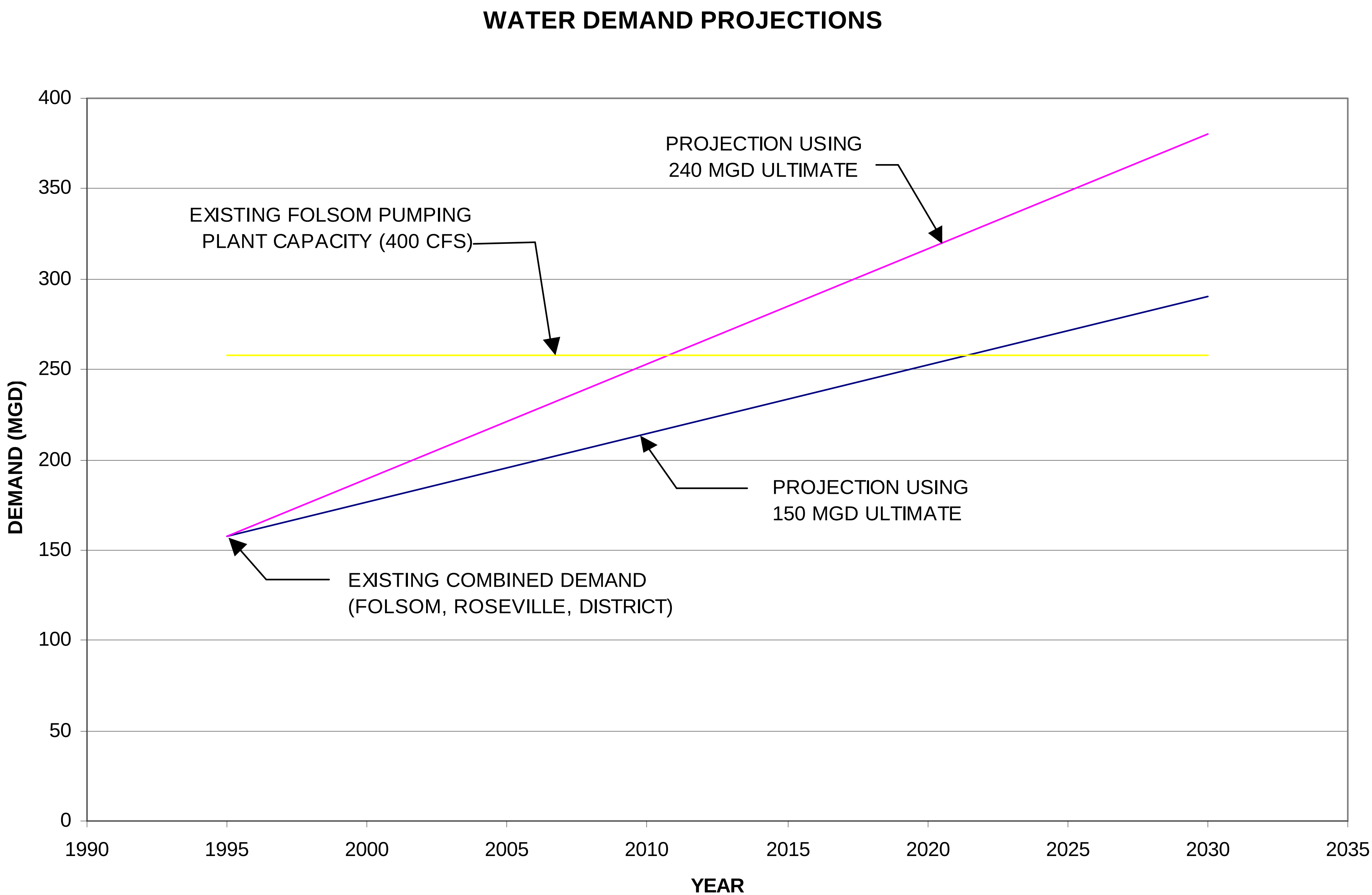

FIGURE 3-3
WATER DEMAND PROJECTIONS

Monthly demand was used to project the minimum hydraulic grade line (HGL) needed to deliver water to the District compared to the 1977 mid-month lake levels. This minimum HGL was used to determine the required pumping head. The analysis confirmed that there is sufficient existing capacity to provide 120 mgd with lake levels nearing the minimum pool elevation of 327 under existing demand conditions for the City of Roseville and Folsom. As these other demands increase, so will the minimum hydraulic grade requirements. Ultimately, a recurrence of lake levels similar to 1977 could result in pumping plant production of less than 70-percent of the normal capacity due to the increased lift required.

Another limitation to pumping at the minimum pool (elevation 327) is the occurrence of air entrapment from breaking suction and vortexing due to insufficient submergence over the dam intake. Prior studies indicated that vortexing may become an issue at lake elevations below 339 feet with pumping rates exceeding 209 cfs. Anti-vortex formation features may be required in the future. However, determining a safe pumping rate at drastically reduced lake levels was beyond the scope of this master plan.

The Bureau is undertaking the design and installation of a TCD on the lake side of the dam. The Bureau has indicated that the hydraulic impact (headloss) through the intake structure is negligible and the water surface elevation will be the same on both sides of the structure. However, the TCD device will be a steel structure which could include anti-vortex features for the dam intake. It is recommended that anti-vortex features be reviewed as part of the TCD project to reduce impacts of low lake levels on the District water supply.

3.3.2 Future Capacity Evaluation Methodology

The future pumping requirements were evaluated for the 150- and 240-mgd future District WTP capacity alternatives. Pipeline improvements alone were reviewed initially to develop pump head criteria, and it was determined that some level of pump improvements will be required for both future capacity scenarios. Similarly, a pumping plant improvements only solution was found to not be feasible due to the cumulative pipeline headlosses at the higher capacity. The results of the evaluation are that pipeline and pump improvements are needed for both future rates. A summary of the pumping plant evaluation, findings, and recommendations is provided below. The raw water transmission pipeline evaluation is discussed in Section 3.4.

3.3.3 150-mgd Future WTP Capacity

Pumping head requirements were identified by determining the downstream pipeline friction losses and minimum water surface elevation required at the WTP. As stated earlier, the Folsom Pumping Plant provides water to multiple users. However, the District has the highest hydraulic grade requirement and therefore sets the controlling pump head criteria. A minimum water surface elevation of 424 feet at the WTP instantaneous mixing chamber was used as the delivery elevation benchmark.

Pressure losses for flow control and friction in the existing transmission pipelines and appurtenances resulted in the HGL plotted on Figure 3-4. As can be seen on the figure, the HGL elevation of 474.6 exceeds the existing surge tower standpipe pump shutoff elevation of 473. In addition, the HGL exceeds the shutoff head of existing pump number 6. Therefore, no flow will be produced by this 600 hp pump. (Pipeline improvements are discussed in Section 3.4.) A new 48-inch diameter pipeline within the District segment will reduce the pumping requirements to within operational limits. The HGL with the District 48-inch pipeline improvement is also shown on Figure 3-4.

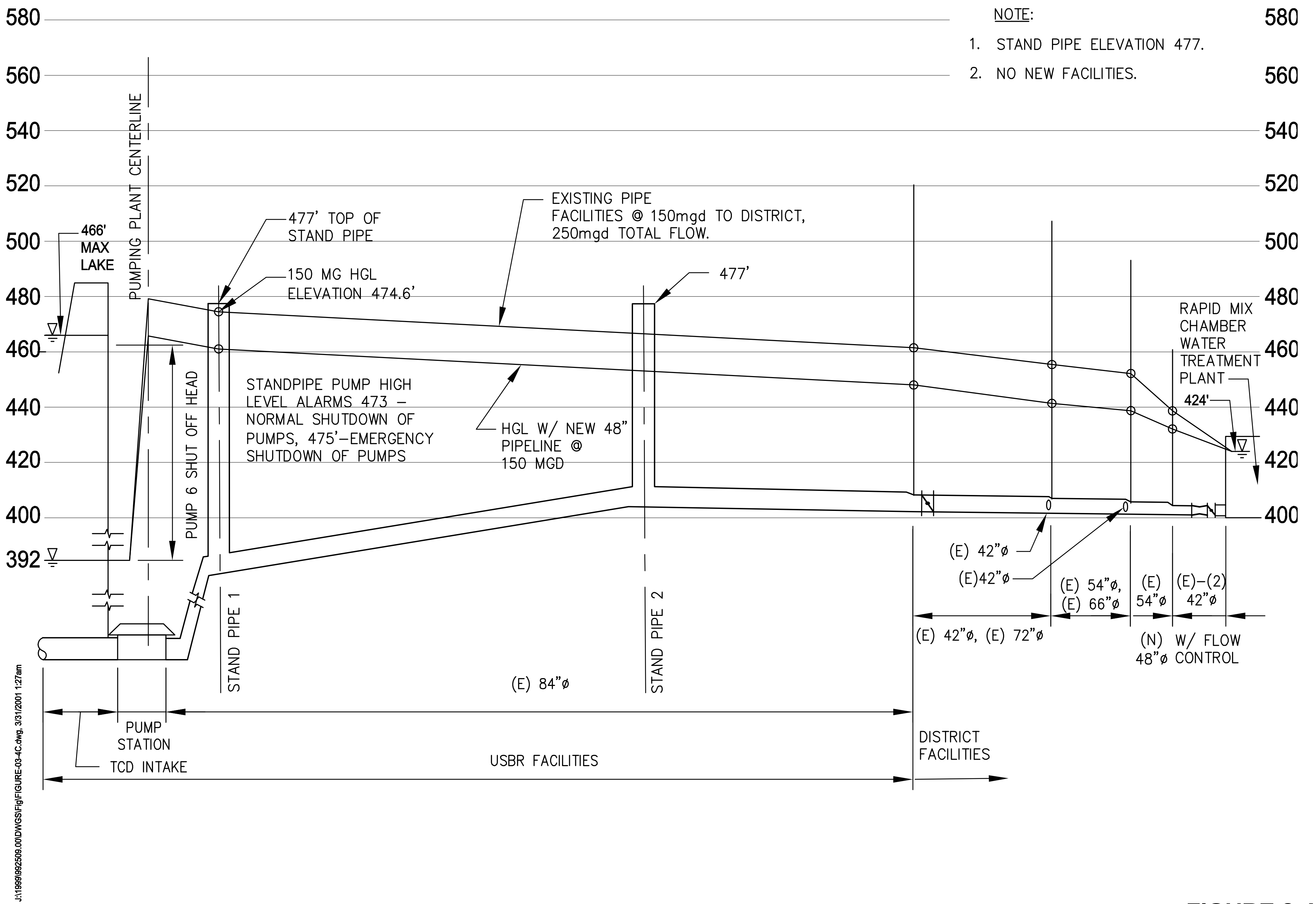

FIGURE 3-4
150-MGD HYDRAULIC PROFILE

Using this similar HGL regime, it is possible to increase the raw water pumping capacity by replacing pump 6 with a 1,500 hp pump similar to the recently installed pumps 7 and 8. This one additional pump brings the total installed pumping capacity to the minimum required to meet the 150-mgd demand level. The pumping capacity with replacement pump 6 is shown in Table 3-6.

Table 3-6
Folsom Pumping Plant
Future Pumping Capacity
150 mgd with One New Pump

Water User Name	Pumping Capacity[a]	Pumping Requirement
San Juan Water District	150 mgd	150 mgd
City Of Roseville	100 mgd	100 mgd
Folsom Users[b]	38.77 mgd	38.77 mgd
Total	288.77 mgd (446.82 cfs)	288.77 mgd (446.82 cfs)

(a) Pumping capacity based on a lake elevation of 392.
(b) City of Folsom, Folsom Prison.

A review of the reliability of the system with one additional pump raised three concerns: lack of pump and piping redundancy, potential for excessive suction side headlosses, and potential difficulties with WTP flow control at the maximum flow rates. These three issues were reviewed further as follows.

3.3.3.1 Redundancy

Additional system capacity is often provided for critical facilities to reduce potential impacts of scheduled outages and emergency conditions such as a pump failure. For example, failure of either pump 7 or 8 would result in a 28 percent reduction in capacity available to the District. In the case of the Folsom Pumping Plant, maintenance of the pumps can be scheduled when demands are below the peak flows. However, emergency failures such as a motor/pump coupling failure, motor failure, or variable frequency drive (VFD) failure may occur during peak demands. In discussions with the Bureau, they have stated they believe that much of the risk of this type of failure is mitigated by preventive maintenance and the availability of spare parts. They indicated that they do follow a preventive maintenance program and that additional capacity for redundancy is not considered necessary by the Bureau. However, a more appropriate criteria for this critical facility would be to have a standby pump equivalent in capacity to the largest single pump. We recommend the District discuss this redundancy requirement with the Bureau.

3.3.3.2 Suction Side Headlosses

Existing suction header (and discharge) improvements will be required with the installation of the new pump. However, modifications to the 84-inch diameter dam penetration and piping are not required. Recent improvements to the 84-inch suction supply pipeline replaced a 60-inch gate valve with a 72-inch diameter butterfly valve. This improvement allows an increase in the flow rate required to develop a similar net suction side headloss to that used in the design of pumps 7 and 8. This supports the assumption that the approximately 10-percent increase in flow through the suction side is allowable without significant impact to the plant performance. The resulting opinion is that the suction side hydraulics should not impact an increase in pumping plant capacity to provide the District with 150 mgd.

3.3.3.3 WTP Flow Control

Pumps 7 and 8 are equipped with VFDs with a future operating goal of providing flow throttling at the Folsom Pumping Plant to minimize pumping energy costs. This will impact the District's current operating practice of controlling the WTP plant influent with the twin 42-inch rate of flow controllers located immediately upstream of the instantaneous mix chamber. This operational change could impact the District's ability to control the WTP hydraulics at peak rates of production.

The composite plant pump performance curve is flattest at the high flows. This results in a higher change in flow for relatively small changes in head. Instability of the VFDs (hunting) while responding to changing flow to Roseville, Folsom, and the District's WTP could result.

It is recommended that the VFD flow control operation be implemented in the near term to both test its impact on the District and develop operating experience prior to implementing pumping plant improvements. If the impacts or experience prove this method of WTP influent flow control to be unacceptable, future pipeline construction schedules could be accelerated to reduce headloss and to allow flow throttling to continue at the District's headworks. Table 3-7 illustrates pumping plant capacity if a second 1,500 hp pump was installed to replace the existing pump 5 were this required to provide for flow control.

Table 3-7
Future Folsom Pumping Plant Pumping Capacity
150 mgd with Two New Pumps

Water User Name	Pumping Capacity[(a)]	Pumping Requirement
San Juan Water District	150 mgd	150 mgd
Replacement Pump 5	45 mgd	N/A
City of Roseville	100 mgd	100 mgd
Folsom Users[(b)]	38.77 mgd	38.77 mgd
Total	333.77 mgd (516.82 cfs)	288.77 mgd (446.82)

(a) Pumping capacity based on a lake elevation of 392.
(b) City of Folsom, Folsom Prison.

3.3.4 240-mgd Future WTP Capacity

The improvements required to provide 240 mgd to an expanded District WTP are significant. Similar to the 150-mgd option, pipeline improvements will be required to maintain the existing HGL regime. Increasing the Folsom Pumping Plant capacity to provide 240 mgd for the District required additional review of the pumping plant suction side facilities. An increase to 240 mgd for the District requires a 140-cfs (90-mgd) increase, from 400 cfs to 540 cfs, and represents a 35-percent plant capacity increase. This increase is sufficient to raise the peak intake pipeline velocity to approximately 16 fps in the 84-inch pipe and 19 fps through the 72-inch butterfly valve. A 16 fps velocity corresponds to approximately 455 cfs through the 72-inch valve. These velocities are greater than desired and can create excessive headloss and potentially erode the interior of the pipe and valve due to cavitation. However, the affected pipe reach is short, and the duration is limited to approximately three months out of the year. The effects of these high intake velocities were reviewed with the Bureau. They expressed concern about lining damage and cavitation at flows exceeding 16 fps and considered 20 fps a maximum velocity.

The expansion of the Folsom Pumping Plant to accommodate a 240-mgd District demand will require significant improvements. A reconnaissance level review was performed to determine the feasibility of implementing pumping plant improvements. Two general approaches were considered as follows:

1. Installing larger pumps in the existing pumping plant.
2. Constructing a parallel pumping plant adjacent to the existing plant.

The pumping plant demand requirements used in this evaluation are presented in Table 3-4.

3.3.4.1 Install Larger Pumps in the Existing Pumping Plant

To meet a 240-mgd District demand, pumps 1 through 6 must be replaced with larger pumps to meet peak demands at the 392 lake level. The conceptual configuration to obtain a 240-mgd capacity with District and Bureau improvements is as follows:

- Two pumps at 1,500 hp (existing)
- Three pumps at 1,250 hp
- One pump at 600 hp and one pump at 250 hp
- Total installed hp of 7,600
- Peak power demand of 5,670 kilowatt

Friction headlosses through both the suction and discharge pipelines are significantly increased as velocities exceed 10 fps. High velocities result in increased energy costs to overcome friction and result in reduced life of the pipeline linings.

Reconstruction of the pump suction and discharge piping is recommended to reduce the peak velocities. Reconstruction will significantly impact the ability to maintain service during construction. For this reason, a second approach, constructing a parallel pumping plant, was reviewed.

3.3.4.2 Construct a Parallel Pumping Plant Adjacent to the Existing Plant

There is sufficient space for a parallel pumping plant adjacent to the existing plant to provide additional pumping capacity. A second alternative might be to construct a new facility on the opposite (east) side of the stilling basin for the Folsom users. This second alternative could use a new dam intake as a supply.

A new pumping plant adjacent to the existing plant would include a new turnout off the existing 84-inch suction pipeline, parallel pump installations, and a wye configuration tying the recommended parallel transmission pipeline to the existing 84-inch transmission pipeline. The existing pumping plant would remain as is while the new facility provides the additional increment required to reach 240 mgd.

3.3.4.3 Bureau Facilities Plan

The impact of a major renovation and expansion to the Bureau facility should consider all users. It is recommended that the District and the other users of the Bureau's supply facilities at Folsom address the need for a Bureau Master Plan effort. The planning effort should address cavitation and high velocities at the existing 72-inch valve discussed in Section 3.3.4.

3.3.5 Peak Power Demand

The District makes use of federal power at the Folsom Pumping Plant. However, there is a peak demand load limit of 1,000 kW that if exceeded could result in the loss of the District's power contract. To determine the impact of increasing pumping capacity, the peak electrical demand was estimated for both 150 and 240 mgd. The water demand and estimated lift establish the needed installed pump horsepower at approximately 6,750 hp and 7,600 hp, respectively. Estimated power demand is based on the total installed hp.

This broad-brush approach was used to evaluate the peak power demand and is not intended to be sufficient to make actual power supply improvement recommendations. Table 3-8 presents the estimated total pumping plant horsepower and electrical power demand. A straight prorata approach was assumed to estimate the District's share of the power demand.

Table 3-8
Folsom Pumping Plant Power Requirements

Pumping Plant Capacity[(a)]	District Demand	Total Plant Horsepower	Total Plant Power Demand	District Power Demand
258.5 mgd	120 mgd	5,250	3,917 kW	2,428 kW
288.5 mgd	150 mgd	6,750	5,036 kW	3,105 kW
378.5 mgd	240 mgd	7,600	5,670 kW	3,496 kW

(a) City of Roseville demand = 100 mgd; Folsom users demand = 38.5 mgd.

Allocation of power requirements to the volume of water pumped has historically been based on water rights, contract delivery conditions, and project (Folsom Dam) versus non-project deliveries. This allocation is used by the Bureau in its billing cycle to the District for power. Obtaining and evaluating the allocation is beyond the scope of this Master Plan.

3.3.6 Summary of Folsom Pumping Plant Findings and Recommendations

The Folsom Pumping Plant evaluation determined that the plant has sufficient capacity to meet the near term 120-mgd demand requirement without improvements. Increases to 150 mgd or 240 mgd will require new pumps and possible modifications to the dam intake.

Findings and recommendations regarding the Folsom Pumping Plant are summarized as follows.

3.3.6.1 Findings

1. Short-term improvements are not required to meet the 120-mgd demand.

2. The 150-mgd demand can be met with the retrofit of one or possibly two pumps within the existing pumping plant. The replacement pump(s) is estimated to be 1,500 hp and would replace pump 6. If a second 1,500 hp pump replacement is deemed warranted for redundancy, it could replace pump 5.

3. The 240-mgd demand will require a significant pumping plant project. An expanded pumping plant or parallel facility is required with consideration for an expanded dam intake.

4. Total peak power requirements exceed 1,000 kW for all cases. Availability of power and the allocation of power charges to project (Folsom Dam) and non-project waters were not considered in this Master Plan.

3.3.6.2 Recommendations

Conduct Folsom Pumping Plant expansion feasibility study for increasing capacity to 150 and 240 mgd.

3.4 Raw Water Transmission Pipelines

The scope of the raw water system evaluation was to identify the existing capacity and reliability of the pipeline system to support water treatment plant expansion. This included a hydraulic analysis of the raw water transmission capacity and physical inspection of the interior and exterior of the pipelines. The inspections were conducted to establish the current condition and estimate the remaining useful life.

This section presents the hydraulic analysis of the raw water pipelines. Section 3.5 presents the condition assessment.

Bureau's Raw Water pipeline. The Bureau's Folsom Pumping Plant is shown near the center of the photograph. The first surge tower and a portion of the 84-inch pipeline are on the right. The raw water pipeline serving the City of Folsom can be seen in the background.

3.4.1 Transmission Pipeline Hydraulic Capacity Evaluation Methodology

The raw water hydraulics evaluation was coordinated with the review of the pumping plant capacity evaluation discussed in Section 3.3. Recommendations for improvements and repairs were developed and integrated into the development of the pump improvement strategies as necessary.

The transmission pipeline hydraulic performance was analyzed using a U.S. EPANet model prepared to depict the pipelines downstream of the pumping plant. The model was calibrated to the flow test data published in "Increasing Water Supply Pumping Capacity at Folsom Dam," ESA January 1996. The model runs were based on a minimum WTP influent water surface elevation of 424 feet at the instantaneous mix chamber.

The District-owned raw water pipelines were reviewed first to determine if pipeline improvements could be limited to District facilities. The conclusion was that the 120-mgd demand condition can be serviced using the existing raw water transmission facilities. However, improvements to the District's raw water pipelines are required to provide 150 mgd, and improvements to both the District's and the Bureau's transmission pipeline system will be needed to provide 240 mgd.

The following discussion of existing facilities are presented in the direction the water flows; that is, the Bureau's raw water pipeline evaluation is presented first, followed by the District pipelines. The findings and recommendations are presented last. The raw water transmission pipeline is shown on Figure 3-1.

3.4.2 Bureau 84-Inch Raw Water Transmission Pipeline

The existing 84-inch pipeline is an approximately 3,300-foot-long above ground steel pipeline with a coal tar enamel lining. The pipe is equipped with two open topped surge towers that provide pressure relief by allowing overtopping under a surge or water hammer condition. Water can be fed to the line through the Folsom Pumping Plant or by gravity through a bypass. Downstream of the pump station is a venturi meter for flow monitoring.

Bureau's 84-inch pipeline. Location is near Folsom dam, at base of dam road. Second surge tower can be seen in background.

3.4.2.1 Existing Capacity

The feature controlling the maximum flow in the 84-inch pipeline is the existing surge tower elevation. The top rim elevation of the two surge towers is at elevation 477. The maximum lake elevation is 466 feet. The historical pump high-level shutoff has been set at elevation 465 with an all pump shut down at elevation 470. The 1999 maximum surge tower operating condition was 458 or approximately 7 feet below the first pump alarm and high level off switch. The Bureau revised the pump shutoff elevation in 2000 from 465 to 473 with a 475 emergency shutoff level. It is the Bureau's opinion that, if elevation 474 is exceeded, the tower will overtop before the pumps complete a shutdown. Previous overtopping has resulted in erosion damage at the base, which has been repaired.

The existing capacity of the pipeline, considering the new surge tower pump control elevation, is sufficient to provide the District's component of 120 mgd.

The Bureau may provide future flow control using the VFD pumps at the Folsom Pumping Plant limiting the head available for throttling at the District WTP flash mix chamber. No pipeline improvements are required to operate at a District WTP capacity of 120-mgd. However, it is recommended that the modified approach to influent rate of flow control be tested and incorporated into the normal operating procedure. This will allow the District to gain experience with the process before the system reaches capacity.

3.4.2.2 150-mgd Future Plant Capacity

The cumulative headloss through the existing raw water pipelines under the 150-mgd flow rate exceeds the existing capacity of the pipeline surge tower. In addition, the required head to pump 150 mgd exceeds the shutoff head of pump 6 at the Folsom Pumping Plant. Modification of the surge towers to increase available head was not considered a viable alternative without a detailed structural analysis. Such a modification would require a careful surge analysis of the Roseville, Folsom, and District piping. Raising the surge towers will also impact the performance of all the installed pumps and is not recommended.

A review of the District pipelines showed that the pipe reach consisting of a single 54-inch diameter pipe contributed high head losses to the system. A parallel 48-inch pipeline, as recommended in Section 3.4.3, reduces the overall system losses to within operational limits. Improvements to the 84-inch pipeline are not hydraulically required if downstream District pipeline improvements are completed.

A second consideration in looking at improvements to the existing 84-inch pipeline is the fact that it is the only pipeline feeding the City of Roseville and the District. A parallel pipe to the Bureau's 84-inch pipe is not required to hydraulically convey 150 mgd to the District's WTP. However, a single transmission pipeline leaves the District vulnerable to outages due to maintenance or emergencies on the 84-inch pipeline. Given no backup supply, there is a 100 percent loss in water supply if the 84-inch pipe is out of service.

This Master Plan recommends that, for a 150-mgd WTP capacity, the District discuss the feasibility of a parallel 84-inch pipeline with the Bureau to provide redundancy and improve reliability.

3.4.2.3 240-mgd Future Plant Capacity

Limited access, high headloss, and susceptibility to unscheduled outage considerations for the 240-mgd capacity alternative are similar to the 150-mgd condition. Figure 3-5 shows the HGL for the 240-mgd case both with and without improvements. It can clearly be seen that the HGL without improvements exceeds the acceptable operating range. An 84-inch diameter pipeline parallel to the existing 84-inch pipeline is recommended to provide for the 240-mgd alternative. Concurrent pipeline improvements are required in the District segment and are discussed in Section 3.4.3.

3.4.3 District Raw Water Transmission Pipeline

The District's raw water transmission pipeline system consists of five separate segments. The raw water pipelines are shown on Figure 3-6. The first pipe constructed was a 42-inch diameter pipe from the Hinkle Wye to Hinkle Reservoir in 1952. The 42-inch pipe was later extended with a 54-inch pipeline up to the current WTP in 1972. At this time, the Folsom Pumping Plant was first used to deliver raw water to the District. All flows were gravity prior to 1972. Later plant expansion included the installation of a second pipeline paralleling the existing line consisting of a 72-inch pipe and a 66-inch pipe. The 72/66-inch leg is connected at two locations to the original 42/54-inch pipeline with 42-inch piping. The fifth segment is the reach of 54-inch pipe up to the plant headworks and includes a bifurcation to two 42-inch orifice plate rate of flow control throttling valves upstream of the rapid mix chamber.

3.4.3.1 Existing Capacity

The hydraulic evaluation of the composite raw water pipelines indicates there is sufficient capacity to meet the 120-mgd plant capacity without pipeline improvements. However, there is an existing bottleneck in the reach of 54-inch pipe up to the twin 42-inch rate of flow control pipelines once 120 mgd is exceeded. Throttling at this location under the 400-cfs (120 mgd to the District, 100 mgd to Roseville) condition will exceed the hydraulic limitations of the 84-inch pipeline. A shift in flow control to the Folsom Pumping Plant will be required as previously discussed.

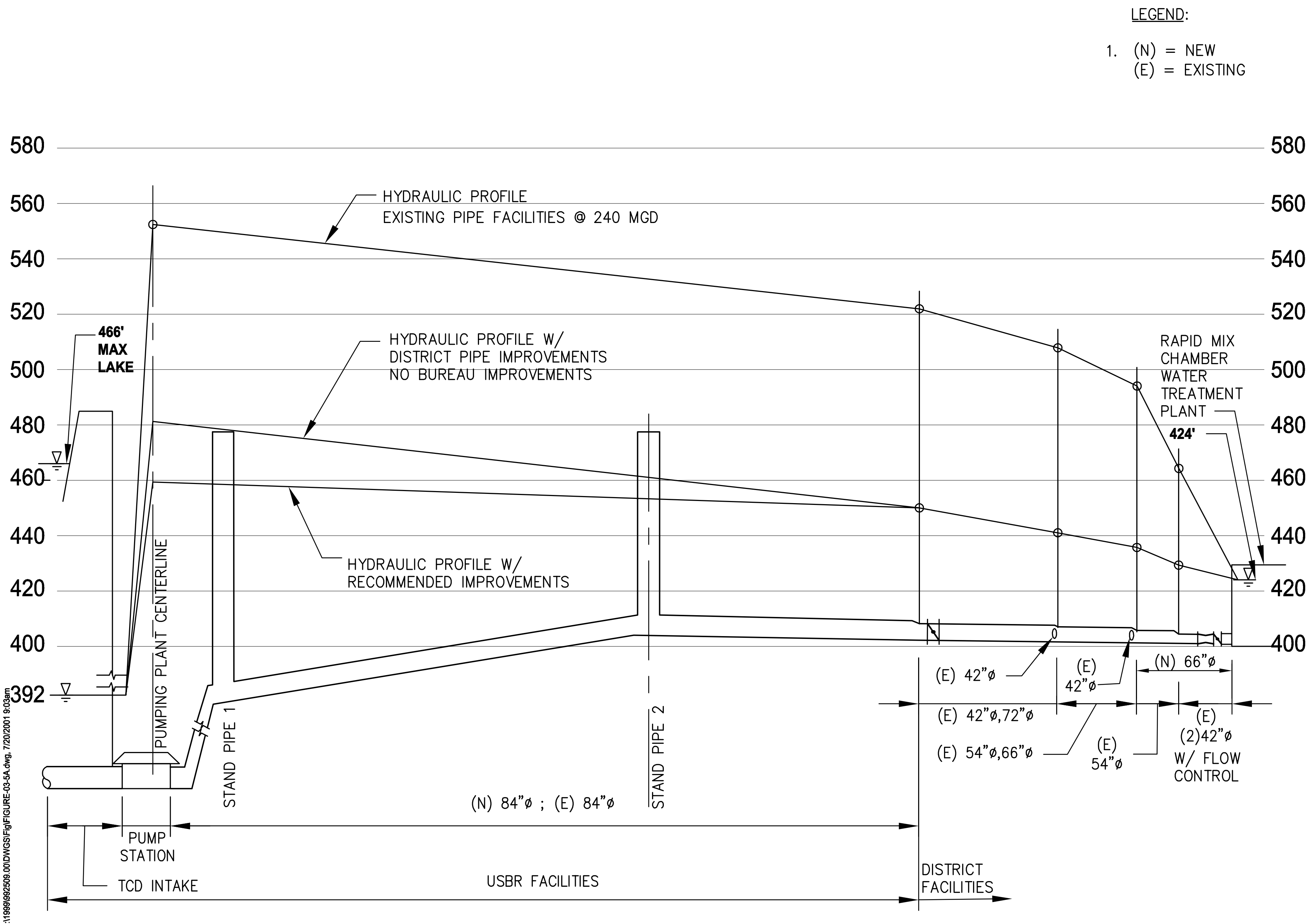

FIGURE 3-5
240-MGD HYDRAULIC PROFILE

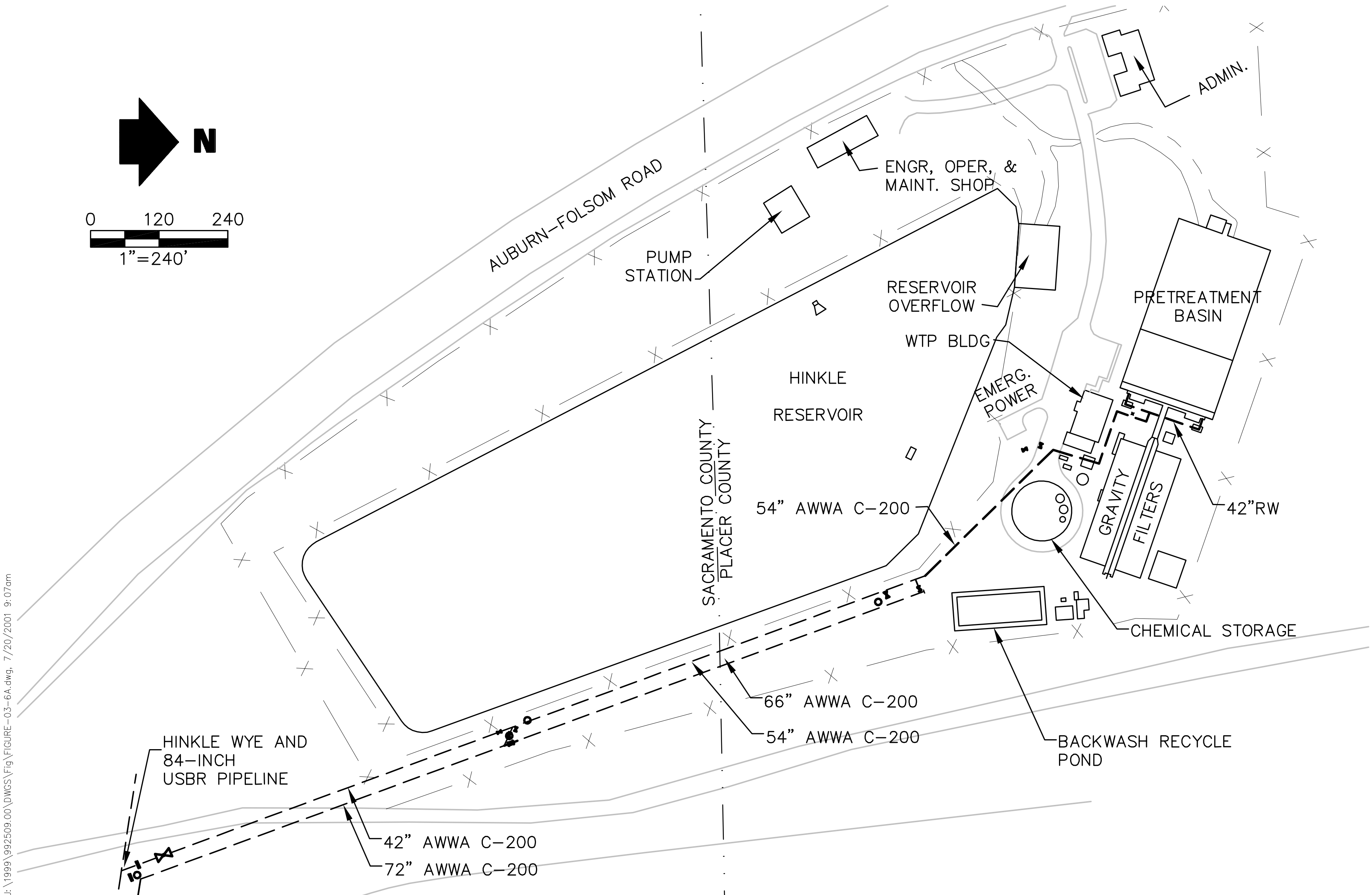

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FIGURE 3-6
EXISTING DISTRICT RAW WATER TRANSMISSION FACILITIES

3.4.3.2 150-mgd Future Plant Capacity

The existing pipelines upstream of the single 54-inch pipeline to the headworks are sufficient to provide 150 mgd. Headloss in the single 54-inch pipe is excessive and will require a second pipeline feeding the plant. The existing 66-inch pipeline has a bumped head outlet, which will accommodate extension to the plant expansion required to increase the capacity to 150 mgd. A 48-inch pipeline is sufficient to provide this additional capacity. Figure 3-7 shows the recommended pipeline improvements.

3.4.3.3 240-mgd Future Plant Capacity

Similar to the 150-mgd option, the existing pipelines are adequate up to the single 54-inch pipeline. The installation of a 66-inch diameter extension from the existing 66-inch pipeline will provide the required additional capacity. Figure 3-8 shows the recommended pipeline improvements.

3.4.3.4 Existing Pipeline Reliability Review

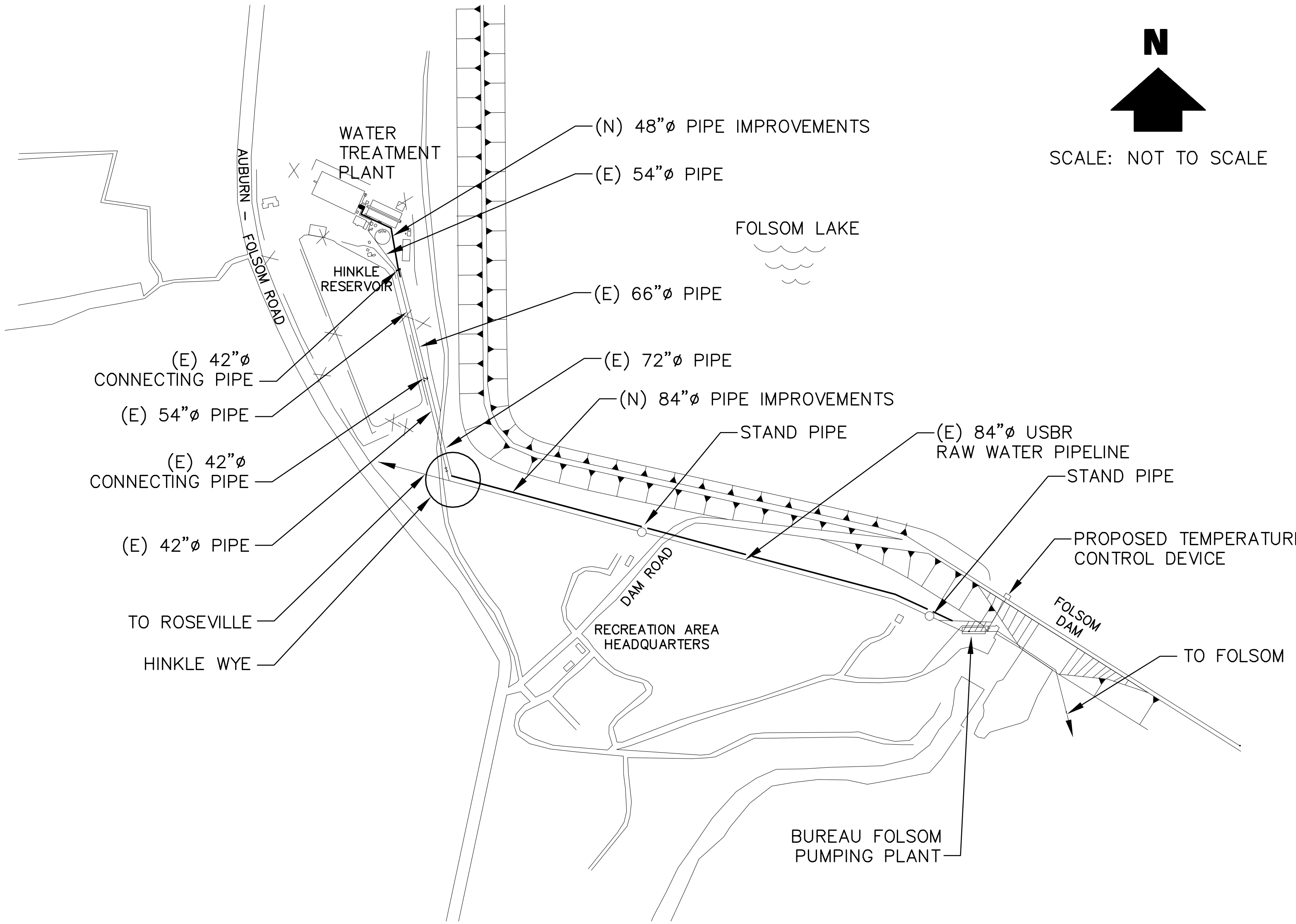
The existing pipelines, although adequate hydraulically, are not equipped with sufficient valving to isolate the 54-inch pipeline for access and maintenance without a complete plant shutdown. Additional valves, and repair or replacement of existing valves, is recommended to provide this capability. The proposed valves include two new valves on the 54-inch pipeline and one replacement valve upstream of the 42-inch wye connecting the 76-inch pipe with the 54-inch pipe. The new valve would replace the existing 54-inch gate valve. The gate valve replacement is based on the condition assessment discussed in Section 3.5.

3.4.4 Summary of Raw Water Transmission Pipeline Findings and Recommendations

The findings and recommendations of the hydraulic evaluation are as follows:

3.4.4.1 Findings

1. The existing pipelines have sufficient capacity to provide the 120-mgd demand; however, this may require a shift of the WTP influent rate of flow control from the WTP to the Folsom Pumping Plant.
2. Meeting a future demand of 150 mgd requires the addition of a 48-inch pipeline paralleling the single 54-inch raw water piping within the existing District property.
3. Meeting a future demand of 240 mgd requires the addition of a 66-inch pipeline paralleling the single 54-inch raw water piping within the existing District property and an 84-inch pipeline paralleling the Bureau's 84-inch pipeline.
4. The existing District pipelines do not have sufficient valving to allow for isolation of the 54-inch pipeline.



J:\1999\992509.00\DWGS\Fig\FIGURE-03-7A.DWG, 7/20/2001 9:09am

FIGURE 3-7

150-MGD RAW WATER TRANSMISSION PIPELINE IMPROVEMENTS

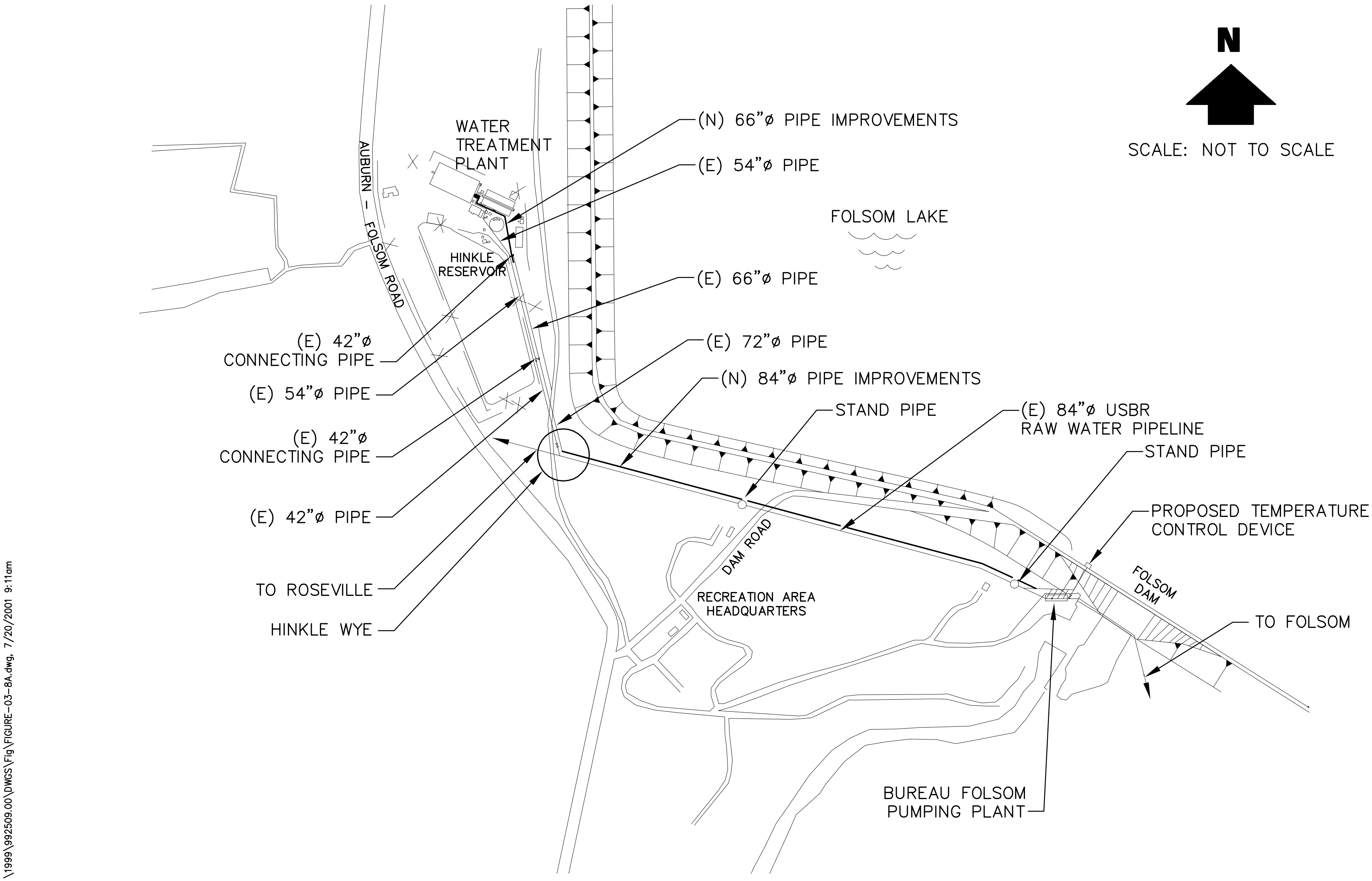


FIGURE 3-8
240-MGD RAW WATER TRANSMISSION PIPELINE IMPROVEMENTS

3.4.4.2 Recommendations

1. Test and operate the WTP using the influent flow control at the Folsom Pumping Plant to provide operational experience prior to reaching full capacity of the Bureau's 84-inch pipeline.

2. Construct a 48-inch diameter pipeline from the existing 66-inch to the expanded WTP headworks if the buildout capacity of the WTP will be 150 mgd or less.

3. Construct an 84-inch pipeline parallel to the Bureau's 84-inch pipeline for redundancy and to increase reliability for a WTP capacity of 150 mgd or less.

4. Construct an 84-inch diameter pipeline parallel to the Bureau 84-inch pipe and a 66-inch diameter pipeline from the existing 66-inch to the expanded WTP headworks if the buildout capacity of the WTP will be between 150 mgd and 240 mgd.

5. Install three new 54-inch diameter butterfly valves in the existing 54-inch line for isolation.

3.5 Raw Water Transmission Pipeline Condition Assessment

As a part of the raw water transmission pipeline evaluation, an inspection was conducted to evaluate the present condition, deterioration, and remaining life of the pipelines to the water treatment plant. The pipelines inspected are shown on Figure 3-9. Summary findings on soil properties and corrosivity, water corrosivity, external and internal corrosion, and a detailed discussion of the inspection and findings are presented in Appendix 3-1.

As-built engineering plans for the pipelines installed in 1976 and 1986 were used to establish the original design criteria. The 42-inch pipeline installed around 1952 is equipped with a coal tar lining and unknown coating below grade. Field inspection determined that a tape wrap coating system had been substituted for the cement mortar in the 1986 installations. The characteristics of the existing pipelines are summarized in Table 3-9.

Table 3-9
Existing Pipeline Construction Material

Pipe Size	Installation Date	Thickness – Inches			Above Grade Exterior Coating
		Steel Pipe Wall[(a)]	Cement Lining[(b)]	Cement Coating[(b)]	
42	1952	Unknown	N/A[(c)]	Unknown	Paint
42	1976	3/16	½[(d)]	¾	-
54	1976	1/4	½	¾	-
72	1986	1/4	½	¾	Paint & Tape Wrap
66	1986	1/4	½	¾	-

(a) AWWA Standard C200
(b) AWWA Standard C205
(c) Coal Tar Lining
(d) Actual measurement was 1 to ¼-inch when inspected 2/10/00

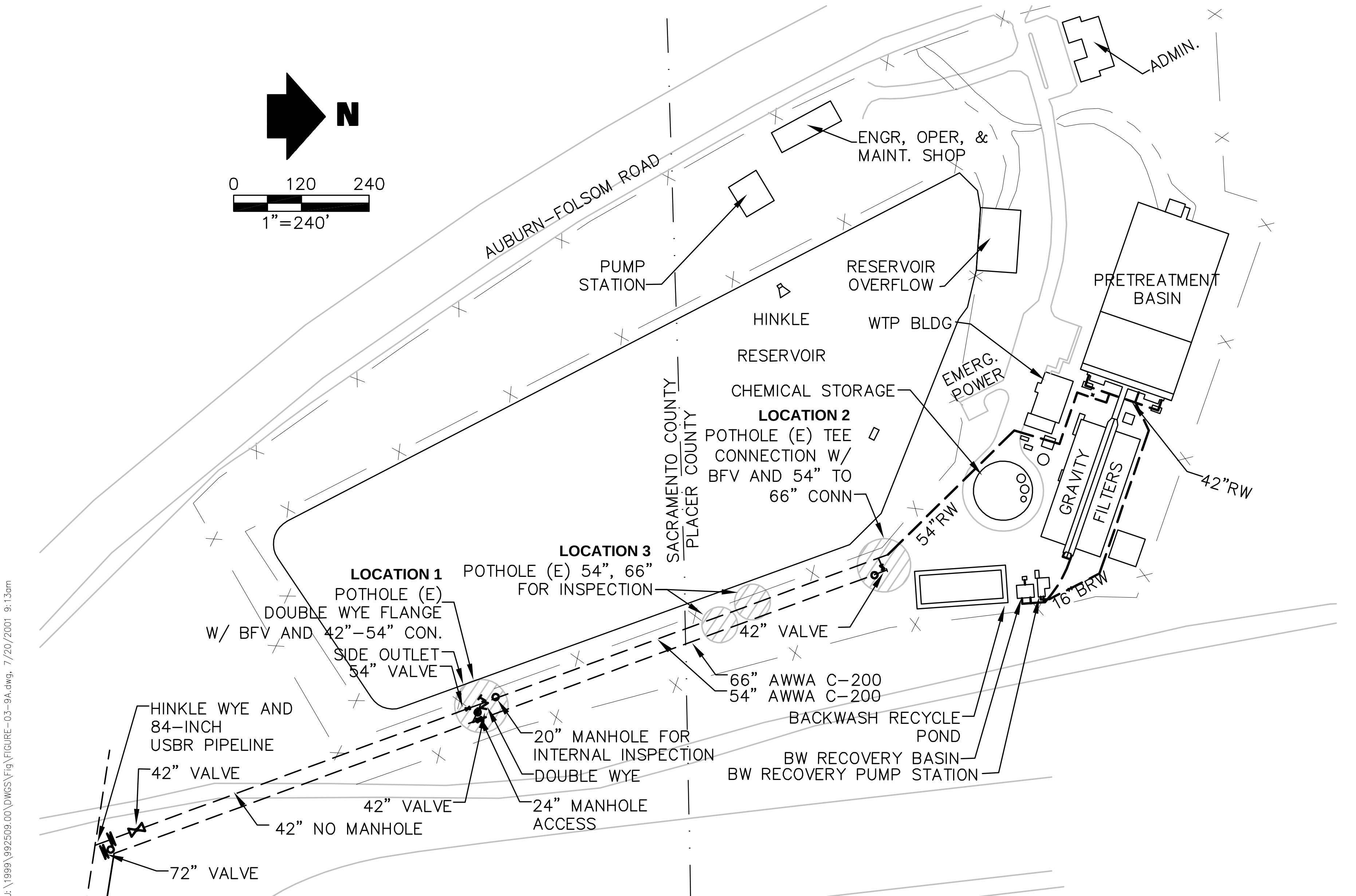


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FIGURE 3-9
RAW WATER TRANSMISSION PIPELINE INSPECTION LOCATION

3.5.1 Soil Properties and Corrosivity

The soils along the raw water transmission pipeline are uniform with depth, consisting of gravelly silty sand with some fine decomposed granite. To determine external corrosion potential to the pipelines, soil resistivities were taken south of excavation location 1 by the 4-pin Wenner method. Results are shown in Table 3-10.

Table 3-10
Soil Resistivity of Intake Pipeline Area at Various Soil Depths

Depth (feet)	Soil Resistivity ohm-centimeters	Probable Corrosivity to Steel
5	12,000	Very Low
10	6,800	Low
15	5,400	Moderate

Soil resistivity is a measure of the conductive salts in soils. Generally, soils are increasingly corrosive with decreasing resistivity with high to severe corrosion occurring where soil resistivities are less than 2,000 ohm-centimeters and increasingly lower corrosivity above 10,000 ohm-centimeters (AWWA 1987). These field tests show a decline in resistivity with depth. The resistivity variance with depth indicates the formation of galvanic potential differences along the pipeline, which can accelerate localized corrosion.

A field measurement of the pipe to soil potential was taken on the 42-inch pipeline. The potential reading was 470 millivolts, which is indicative of active corrosion of the steel and iron portions of the pipeline in that vicinity (Parker & Peattie, 1984). It would be necessary to install cathodic protection to negate the corrosion and corrosion potential that was observed and measured.

3.5.2 External Corrosion and Protective Measures


Failing Exterior Tape Wrap

The external portion of the pipelines is in fair condition. There is moderate corrosion occurring where there are coating defects, and some rust and shallow pitting is apparent.

The condition of the 1-1/4-inch steel bolts removed after 38 years in the ground was inspected. Pit depths to 1/8-inch were observed on the bolts. This would correlate to a pitting penetration rate of 3.3 mils per year (0.0033 inches/year) for exposed metal in the ground, which is a low rate for steel.

The steel thickness of the 42-inch pipeline is 3/16-inch (187.5 mils) and 1/4-inch (250 mils) for the other pipe. This sustained corrosion rate would induce pipe leaks within 60 years, or about 20 years from the present for the 42-inch pipeline. The remaining life of the larger pipelines considering only external corrosion is at least 50 years.

Considering these observations, as well as the desire to preserve the pipelines for more than 100 years, it is recommended that a cathodic protection system be placed to provide protection to all of the buried intake pipelines within the next 5 years.

3.5.3 Water Corrosivity

Historic water quality data from EPA Storet sources was obtained for the American River in the vicinity of Folsom Dam extending back into the 1980s. The primary water characteristics that relate to corrosivity or scaling include pH, temperature, total dissolved solids, calcium, alkalinity, chloride, sulfate, and dissolved oxygen. There is considerable variation in physical properties of Folsom Lake water, such as pH that can range from 6.8 to 8.8, temperature from 5°C to 20°C, and dissolved oxygen from 1 to 12 milligrams per liter (mg/L), and total dissolved solids (TDS) from 10 to 80 mg/L. Chemical characteristics also typically show a 4:1 variation. However, overall, the water is characterized as being cool, slightly alkaline, and low in mineral solids, TDS hardness, and alkalinity.

The average water quality characteristics for the water are listed on Table 3-11, together with calculated corrosion-scaling indices and assessment as to probable corrosivity to piping and valve materials (Ryder and Wagner, 1985). Overall, this data shows a potential for moderate-uniform corrosion to iron and steel; a moderate to high aggressiveness by carbonation to portland cement and concrete; and low corrosivity to copper, copper alloys, stainless steel, and nickel alloys.

Table 3-11
Water Quality Characteristics and Corrosion Potential

Characteristic	Units	American River at Folsom Dam[a]	Desired Range
pH	-	7.3	6.5-8.5
Temperature	°C	14.3	5-20
Total Dissolved Solids	mg/L	43	<500
Calcium	mg/L	7.0	<50
Alkalinity	mg/L $CaCO_3$	23.7	<250
Chloride	mg/L	3.1	<250
Sulfate	mg/L	4.8	<250
Carbon Dioxide	mg/L	3.0 [b]	<5
Corrosivity and Scaling Indices			
pH_S $CaCO_3$ Saturation		9.36	-
Langelier Index		-2.06	-0.5 to +0.5
Ryznar Index		11.42	6-8
Aggressive Index		9.9	>12
Larson Index ($Cl+SO_4/HC0_3$)		0.40	<0.4
SO_4: Cl Ratio		1.54	<3

Table 3-11 (cont.)
Water Quality Characteristics and Corrosion Potential

Characteristic	Units	American River at Folsom Dam[a]	Desired Range
Probable Corrosivity or Scaling to Materials [c]			
Iron and Steel	Moderate Uniform Low Pitting Corrosion		Range 5-10 MPY
Copper	Low Uniform Corrosion		Range 0.5 – 1 MPY
Stainless Steel	Very Low Crevice Corrosion		Range <0.1 MPY
Cement & Concrete	Moderate to High Uniform Corrosion		Range 3-6 MPY

(a) Average of EPA Storet Water Quality
(b) Calculated
(c) Ryder, R.A., "Corrosivity Characteristic Rating for Various Materials, Kennedy/Jenks, 1992.

3.5.4 Internal Corrosion and Protection Measures

The internal condition of the pipelines appears fair, but deteriorating. There is a very smooth gelatinous dark brown film over the concrete lining that is 20 to 30 mils thick. Beneath that, the concrete lining is soft to a depth of 1/16-inch for the newer pipe and 1/8-inch in the 54-inch pipe. This softened cement condition is due to carbonation and loss of calcium and alkalinity. The brown gelatinous film is probably a combination of iron and manganese oxide from that portion of iron in the cement of the pipeline and what may be oxidized on the surface from manganese released from anoxic zones of lower reservoir depths. The brown surface film had no odor, so extensive microbial slime growth is not likely.

Overall, the gelatinous film is beneficial as it maintains a very smooth surface and high Hazen-Williams "C" values to sustain flow capacity, and it also suppresses diffusion of calcium and hydroxide of the cement, the abrasion and loss of sand, and the rate of cement loss with time.

54" "Captains Wheel" Gate

3.5.4.1 54-Inch and 42-Inch Pipelines

Deposits on inside of Gate Valve

There were numerous internal circumferential cracks in the 54-inch pipeline up to 1/16-inch wide at the surface. Some showed steel corroding beneath the surface. No longitudinal cracks were observed. The presence of so many circumferential cracks could be due to displacement or settling of portions of the trench over time, or if soil was disturbed when constructing the parallel nearby 72-inch pipeline.

AWWA C205 does not limit circumferential hairline cracks of cement linings, stating they will autogenously heal and protect the steel wall of the pipe. This is doubtful in this case because these are more than hairline cracks, and rust is observed. The presence of small localized bare steel anodic areas will accelerate corrosion in those locations, and the expanding rust will then spall the adjacent cement lining aggravating corrosion.

The deterioration of the cement caulked joints of the pipeline was the most apparent, and by far the biggest and most immediate corrosion problem. The state of deterioration of all of the joints is severe, with very soft cement lifting away from corroded steel beneath the caulking of each joint.

Cracked and Failed Coal Tar Lining

Corrosion Present under Failed Coal Tar Lining

The interior of the gate valve in the 42-inch pipeline showed extensive tuberculation of about a half-dozen nodules per square foot of surface area. Beneath each tubercle was a pit to 1/8-inch depth, indicating about the same rate of corrosion and condition as for external exposed steel and iron. It is quite likely that the exposed portions of the 3/16-inch steel cylinder of this pipe is corroding at the same rate of 3 mils per year, and serious leakage will occur within the next 20 years.

The original 42-inch-diameter pipeline was beyond the reach of the lifeline cable extraction winch (our limit of inspection). This pipe is reported to be coal tar enamel lined. Based on the condition of the coal tar lining of the gate valve, the original lining is in fair to poor condition.

3.5.4.2 42-Inch Double Wye Pipeline

The interior of the 42-inch double wye pipeline was in considerably better condition. The cement lining was smooth and showed no cracks. The lining had a brown gelatinous film and softened cement to a depth of 1/16-inch. The bright stainless steel valve edge and relatively non-corroded nickel cast iron valve disc (NiResist) appear in excellent condition.

3.5.4.3 72-Inch Pipeline

One circumferential crack was observed about 50 feet south of the entry, with rust showing through a 20 to 40-mil section. Another portion of this pipe had a section of drummy lining and extensive spider cracking extending over a 4-foot-square area of the lower quadrant. A brown gelatinous coating and 1/16-inch soft cement lining was typical.

The joints had an epoxy type of grout that was 1/4- to 3/4-inch thick. The grout was delaminating and breaking into pieces. There are non-welded bell and spigot or Carnegie joints according to Clendennon Engineers' drawings, and there was an apparent substitution of epoxy grout for portland cement grout.

3.5.4.4 66-Inch Pipeline

The condition of the 66-inch pipe north of the wye was similar to the 72-inch pipe. A large circumferential crack with rust staining through a portion was observed, and epoxy grout was loosening from the joints. A large chunk of cement lining was lying on the bottom of the pipe.

There is some concern regarding the state of deterioration of the interior of the 72- and 66-inch pipelines, although they are less than 15 years old. The rate of cement loss is about 6 mils per year, double that for the 42-inch pipeline. This rate of loss may decrease with time. Still, the probable life of the 1/2-inch-thick cement lining is less than 50 years, and relining within 20 years is advisable.

Of more immediate concern is the need to recaulk failed epoxy grout joints and spot repair (regrout) large cracks and areas where lining is spalled. This is fairly urgent work to prevent leaks. These exposed areas become small anodic areas that experience accelerated localized corrosion because they become sacrificial to all other portions of the interior of the pipeline steel. Repairs should be scheduled within 5 years.

3.5.4.5 84-Inch Bureau Pipeline

The existing Bureau 84-inch pipeline was not initially part of the inspection plan and was not evaluated during the inspection of the District pipelines. However, work undertaken by the Bureau in February 2000 resulted in removal of a short segment of pipe. This provided an opportunity to inspect the pipe and make a preliminary determination of condition.

The findings were that the lining is coal tar enamel 1/8- to 3/16-inch in thickness and very brittle. This thickness is consistent with the application standard of the time. The pipe segment inspected was approximately 5 feet long, and cracked lining was seen in three separate areas of the pipe. The lining was removed, and rust is occurring under the lining.

It is quite likely that the coal tar lining failures will accelerate as time proceeds and that the corrosion pitting rate will be about the same as measured from the District pipelines. The observed corrosion rate in the District pipelines is estimated at 3.3 mils per year. Thus a metal loss and pits to 1/4-inch deep will occur in 75 years. Considering that there may already be a 45-year start to the corrosion process, it may be expected that leaks and serious corrosion damage to this 84-inch pipeline will occur in the next 25 years.

Mitigation measures within the next 10 years to protect the existing pipeline's long-term integrity are recommended. The District should enter into discussions with the other users of this intake pipeline and request that the Bureau address rehabilitation of this pipeline.

3.5.5 Summary of Pipeline Inspection Findings and Recommendations

The findings and recommendations of the pipeline inspection effort are summarized below. A more detailed discussion of the inspection and results is presented in Appendix 3-1.

3.5.5.1 Findings

1. The aboveground exterior surfaces of the pipe are now showing indications of coating failure and rust.

2. The soils are moderately corrosive to steel and concrete due to a combination of low pH and resistivity. Deterioration and pitting is occurring on both materials at the rate of about 3 mils per year.

3. The interior cement caulked joints of the 42- and 54-inch pipes have completely softened and failed. Extensive rusting of the steel beneath the joints and disbonding of this softened grout have occurred. There are many circumferential cracks of the 42- and 54-inch pipes' cement lining, which are now showing penetration of rust and probable accelerated corrosion and spalling of the cement lining.

4. The most serious condition in the 66- and 72-inch pipelines is at the interior epoxy grouted joints. Here the epoxy grout has loosened, and steel surfaces are beginning to rust accelerating the spalling of the epoxy grout. There are circumferential cracks in the 66- and 72-inch pipelines at about every 20 feet distance apart as compared to 5 feet for the 42-inch pipeline. Rust is showing through some portions of the cracks. The thickness of the cement lining of the 66- and 72-inch pipelines is typically 1/2-inch, as contrasted to 1- to 1-1/4-inch in the 42- and 54-inch pipelines.

5. Buried access manhole and valve bolts are not stainless steel and are corroding.

6. The Bureau's 84-inch pipeline lining is brittle and cracking with perhaps 25 years of life left before serious leaks occur.

3.5.5.2 Recommendations

1. Install a deep well anode impressed current cathodic protection system within the next 5 years to provide for continuing corrosion protection of all of the buried intake pipelines.

2. Regrout the 42-inch pipeline joints and place a new high calcium cement (1:1 cement-sand ratio) relining over the existing lining within the next 5 years. The same should be done for the existing 54-inch pipeline within the next 10 years. The original coal tar lined 42-inch pipe should be included in the repair and re-lining project.

3. Regrout the joints of the 66- and 72-inch pipelines and patch at spalled and cracked lining locations within 5 years; clean the entire pipeline of softened cement and reline with high calcium cement within 20 years.

4. Replace buried access manhole and valve bolts whenever they are exposed with Type 304 stainless steel with plastic washes and bolt stems to suppress galvanic action with carbon steel flanges.

5. Fully inspect the Bureau's 84-inch pipeline and develop and implement a rehabilitation approach by 2010.

3.5.6 Raw Water Transmission Pipeline Remaining Service Life

The predicted life of the cement linings without rehabilitation and relining is 60 years from the date of installation. The remaining useful life without rehabilitation is estimated as 20 years for the 42-inch, 35 years for the 54-inch, and 45 years for the 66- and 72-inch pipelines. Rehabilitation and pipe relining will extend their service lives for an additional 40 years. The life expectancy of the pipelines with and without corrective action is shown in Table 3-12.

Table 3-12
Raw Water Transmission Pipeline Estimated Remaining Service Life

Pipe Segment	Remaining Life (Years)	
	No Action	Action[a]
42-inch steel installed 1952	20	60
42-inch steel installed 1976	35	75
54-inch steel installed 1975	35	75
72-inch steel installed 1986	45	85
66-inch steel installed 1986	45	85

(a) Recommended action includes joint repair, relining and installation of a cathodic protection system.

Based on the values presented in Table 3-12, the pipeline service life with improvements ends approximately in the years 2060 to 2085. Without corrective action, the service life ends approximately in the years 2020 to 2045.

3.6 Recommended Improvements and Costs

Conceptual level cost estimates prepared for the recommendations presented in this chapter are shown in Table 3-13. The estimates include improvements to District facilities and a new parallel 84-inch pipeline from the Bureau's Folsom Pumping Plant to the Hinkle Wye. The estimates include the improvements recommended to support a 150-mgd treatment plant, a 240-mgd treatment plant, and rehabilitation of existing District pipelines.

Expanded facilities at the Folsom Pumping Plant, and repair and rehabilitation of the existing Bureau 84-inch pipeline have not been estimated.

The recommended improvements required to support the 150-mgd treatment plant alternative are shown in the 120-150 mgd column of Table 3-13. The recommended improvements required to support the 240-mgd treatment plant alternative includes four plant capacity ranges to correspond to discussions in Chapter 7 regarding treatment plant improvements. The 240 mgd capacity ranges are 120 to 150 mgd, 150 to 180 mgd, 180 to 210 mgd, and 210 to 240 mgd.

The estimated capital costs are conceptual level estimates prepared without plans and specifications and actual quantity take-off. The estimates were prepared based on prior bid results, standard estimating guide cost curves, equipment quotes from suppliers, and engineering judgment. The estimates are based on an Engineering News Record (ENR) Construction Cost Index of 6281 (in effect January 2001), and include 25 percent contingencies to provide for reasonable estimating and construction uncertainties. A 25 percent allowance is also included for planning, engineering, administrative, legal expenses, and construction management associated with project implementation.

Table 3-13
Conceptual Level Estimate of Capital Costs
Raw Water Pump Station and Pipeline Improvements

WTP Capacity	150 mgd	240 mgd				240 mgd
Capital Improvement Item	**120-150 mgd**	**120-150 mgd**	**150-180 mgd**	**180-210 mgd**	**210-240 mgd**	**Total**
Folsom Dam Outlet Improvements	0	0	0	(a)	(a)	0
Bureau Folsom Pumping Plant						0
Larger Pump Retrofit	(b)	n/a	n/a	n/a	n/a	0
Plant Reconfiguration	n/a	(b)	(b)	(c)	(c)	0
Bureau Transmission Pipeline						0
Parallel 84	(d)	0	4,845,000	0	0	4,845,000
Lining Repairs	(e)	(e)	(f)	(f)	(f)	0
District Raw Water Piping						0
Rehabilitate Joints	76,000	76,000	0	0	0	76,000
Rehabilitate Linings	110,000	110,000	0	0	0	110,000
Cathodic Protection	54,000	54,000	0	0	0	54,000
54-Inch Gate Valve Replacement	134,000	134,000	0	0	0	134,000
New Manways and Valves	297,000	297,000	0	0	0	297,000
Parallel 48-inch Pipeline	623,000	n/a	n/a	n/a	n/a	0
Parallel 66-inch Pipeline	n/a	805,000	0	0	0	805,000
Subtotal	$1,294,000	$1,476,000	$4,845,000	$0	$0	$6,321,000
Contingency @ 25%	323,500	369,000	1,211,250	0	0	1,580,250
Engineering, Legal, and Administrative @ 25%	323,500	369,000	1,211,250	0	0	1,580,250
Total ($)	**$1,941,000**	**$2,214,000**	**$7,267,500**	**$0**	**$0**	**$9,481,500**

(a) Isolation valve velocities exceed Bureau maximum at Folsom Dam penetration; cost not estimated as part of this work.
(b) Expansion possible with larger pumps retrofit into existing pump bays; cost not estimated as part of this work.
(c) Expansion will require pumping plant reconfiguration; cost not estimated as part of this work.
(d) Parallel pipeline not required for hydraulic capacity, recommended for redundancy and reliability.
(e) Lining repairs not feasible without parallel pipeline.
(f) Lining repairs not estimated as part of this work.

Chapter 4: Treatment Plant - Regulatory Requirements

4.1 Introduction

Drinking water regulations in the United States are undergoing significant revisions. The regulatory revisions are due to increasing contamination of water sources, coupled with more definitive knowledge of health risks associated with waterborne contaminants. The revisions are being driven by:

- The federally enacted Safe Drinking Water Act (SDWA) Amendments of 1986 (PL 99-339) and 1996 (PL 104-182).

- The regulatory negotiation (Reg-Neg) process of health, environmental, and economic issues involving the USEPA.

- Local concerns in the State of California, where the DHS has primacy in implementation of the SWTR, Lead and Copper Rule (LCR), Total Coliform Rule (TCR), the new Interim Enhanced Surface Water Treatment Rule (IESWTR), and Stage 1 - Disinfectants/Disinfection Byproducts Rule (D/DBPR).

The District's WTP was designed prior to many of the current state and federal water quality regulations and guidelines. The WTP is characterized as a "conventional filtration treatment process" that includes oxidation and initial disinfection, followed by coagulation, flocculation, sedimentation, filtration and final disinfection prior to delivering the treated water to the distribution system. The sedimentation basins and filters remove particles, including microbial contaminants that may be present in the source water. Disinfection provides an additional barrier against microorganisms that pass through the physical removal processes. In addition, lime is added to the treated water to increase the pH as a corrosion inhibition (water stabilization) measure.

This chapter discusses drinking water regulations that currently, or in the future, will impact the existing and expanded WTP. These are summarized in Table 4-1. Water quality issues, including recommendations regarding treated water quality objectives, the impact of source water quality, planned changes in lake-management practices, additional water quality monitoring for the expanded WTP, and a recommended approach to address water quality issues are addressed in Chapter 5.

The recommended actions to comply with current, new, and anticipated drinking water regulations are summarized as follows:

- Upgrade filter backwash treatment system to comply with the California *Cryptosporidium* Action Plan (CAP).

- Reserve space at WTP for fluoride storage and feed system in the event funding for fluoridation becomes available to comply with State Assembly Bill 733.

- Add capability to measure return treated backwash water flow and turbidity to comply with the new IESWTR.

Table 4-1
Summary of Current, New and Anticipated Drinking Water Regulations and Potential Impact on District

Regulation	Description	Potential Impacts
Current		
Surface Water Treatment Rule (SWTR)	Targets turbidity and microbial contaminants	• Currently in compliance with turbidity requirements. • Disinfection practice must correspond to direct or conventional treatment approach.
Total Coliform Rule (TCR)	Targets microbial contaminants	• Currently in compliance.
Lead and Copper Rule (LCR)	Regulates excessive leaching of lead and copper	• Currently in compliance.
Information Collection Rule (ICR)	Required collection of microbial and DBP information	• No direct impact. • WTP may use data to understand DBP generation at plant.
Partnership for Safe Water Guidelines (PSW)	Recommends average filtered water turbidity =0.1 NTU	• Currently in compliance. WTP has complied with guideline last 5 years.
California *Cryptosporidium* Action Plan (CAP)	Established new turbidity goals for settled, filtered, and return water	• Insufficient monitoring data from WTP to verify impacts. • Return water turbidity likely not in compliance. Will require upgrade to District's filter backwash return treatment system.
Fluoridation (State Assembly Bill 733)	Mandates fluoridation of public water systems under certain circumstances	• Requires fluoridation if funds available from non-ratepayer or taxpayer sources. • Potential impact to site space layout with potential additional cost.
New		
Stage 1 Disinfectants/Disinfection By-Products Rule (D/DBPR)	Targets DBPs, sets limits for disinfection residuals	• Currently in compliance.
Interim Enhanced Surface Water Treatment Rule (IESWTR)	Sets new *Cryptosporidium* removal requirement and turbidity-based removal credit	• Increases monitoring and reporting requirements. • May require filter profile report. • May require disinfection profile. • Return water flow and turbidity must be measured and comply with CAP.
Anticipated		
Filter Backwash Recovery Rule (FBRR)	Sets turbidity standards for returning spent filter backwash to the treatment process	• Will require upgrade to District's return water treatment system. • Final rule requirements unknown. There may be additional impacts.
Arsenic Rule	Will lower arsenic MCL	• No impact to District expected.
Long-Term 2 Enhanced Surface Water Treatment Rule	May include additional turbidity or *Cryptosporidium* disinfection requirements	• Potential impact to District unknown since rule is draft only. • May indicate change in disinfection process.
Stage 2 Disinfectants/Disinfection By-Products Rule	Will focus on contaminant speciation and may reduce DBP MCLs or set individual MCLs for DBPs	• Current draft has compliance with Stage 2 D/DBPR based on local running annual averages. • May increase monitoring requirements. • Potential impact to District unknown since rule is draft only.
Radon and Radionuclides	Targets radon and other radionuclides	• No impact to District's surface water source and WTP. • Potential severe impact to supplemental groundwater supply.

- Reserve space at WTP for alternative disinfection to chlorine to comply with potential *Cryptosporidium* disinfection requirements of the anticipated Long-Term 2 Enhanced Surface Water Treatment Rule.

4.2 Regulatory Requirements Background

The SDWA was enacted in 1974. Through this legislation, the federal government gave the USEPA authority to set standards for contaminants in drinking water supplies throughout the country.

The 1986 Amendments to the SDWA identified 83 contaminants to be regulated by the USEPA. For each contaminant, the USEPA was required to establish a maximum contaminant level (MCL) or a treatment technique (TT) to limit the level of these compounds in drinking waters. The USEPA was also required to recommend a Best Available Technology (BAT) for removal of each contaminant during treatment. The 1986 Amendments required USEPA to regulate the 83 contaminants within three years of promulgation and to identify 25 additional contaminants for regulation every three years thereafter.

The DHS is responsible for the implementation of federal USEPA drinking water regulations in the State of California. DHS must enforce regulations that are at least as strict as those promulgated by USEPA. Additional requirements and guidelines of the California DHS as mandated by Title 22 of the California Code of Regulations include:

- An average filtered water turbidity goal of 0.2 NTU at new and modified water treatment plants where the design was completed after May 15, 1991.
- A *Cryptosporidium* Action Plan, to protect against *Cryptosporidium* and other pathogens, which includes:
    - A settled water turbidity goal of less than 2 NTU.
    - A filtered water turbidity goal less than 0.3 NTU within the first 4 hours following a backwash.
    - A filtered/treated water turbidity goal of 0.1 NTU beginning 4 hours after a filter backwash.
    - A reclaimed filter backwash water goal of less than 2 NTU.
    - A disinfection system for the reclaimed backwash water system.
- Chemicals used for potable water treatment must have National Sanitation Foundation (NSF) Standard 60 approval (or similar approval from Underwriters Laboratory [UL]) for a purity that is no risk to health from introduced chemicals. The District must ensure that the chemicals added to the water have the required NSF or UL approval and that these chemicals are used in concentrations below the NSF designated maximum concentration limits.
- Recommended Public Health Goals set at or below the MCL for specific contaminants.

4.3 Existing Regulations and Guidelines

Existing drinking water regulations and guidelines include federal and state regulations and guidelines that were in effect on July 31, 2000. A summary of the regulations is provided in Table 4-2.

Table 4-2
Summary of Existing Drinking Water Regulations

Regulation	Year of Promulgation of Final Rule	Number of Contaminants	Targeted Contaminants
National Interim Primary Drinking Water Regulations (NIPDWR)	1975-1981	7	Total Trihalomethanes, Arsenic, Radionuclides
Fluoride	1986	1	Fluoride
Phase I Standards	1987	8	VOCs
Phase II Standards	1991	36	IOCs, SOCs, VOCs
Phase V Standards	1992	23	IOCs, SOCs, VOCs
Surface Water Treatment Rule (SWTR)	1989	5	Turbidity, Microbial Contaminants
Total Coliform Rule (TCR)	1989	1	Microbial Contaminants
Lead and Copper Rule (LCR)	1991	2	Lead and Copper
Information Collection Rule (ICR)	1996	NA	Microbial and DBP Contaminants

The following paragraphs present the relevant features of the existing drinking water regulations and guidelines that impact the District's WTP. These regulations include the SWTR, TCR, LCR and the Information Collection Rule (ICR). The existing drinking water guidelines include the Partnership for Safe Water and the California CAP.

4.3.1 Surface Water Treatment Rule

The SWTR was implemented to provide protection against *Giardia* cysts and pathogenic enteric viruses. For a high quality water source such as Folsom Reservoir, the SWTR requires that the overall treatment process achieve a minimum of 99.9 percent (3-log) removal and/or inactivation of *Giardia* cysts and 99.99 percent (4-log) removal and/or inactivation of enteric viruses. This is to be accomplished through a combination of physical removal and disinfection processes.

Because frequent measurement of *Giardia* cysts and enteric viruses is difficult and costly, the USEPA and DHS have developed functional criteria for determining the effectiveness of surface water treatment processes. These functional criteria are to be used unless more definitive data is presented by operational or pilot plant test results. A well-designed and operated "conventional filtration treatment plant," such as the District's WTP, can receive credit for at least 99.7 percent (2.5-log) and 99 percent (2-log) removal of *Giardia* cysts and enteric viruses, respectively. When the plant operates as a "direct filtration treatment plant," it can receive credit for at least 99 percent (2.0-log) and 90 percent (1-log) removal of *Giardia* cysts and enteric viruses, respectively. These credits apply if the filtered water turbidity is less than or equal to 0.5 NTU for 95 percent of the measurements taken each month.

Disinfection must be used to achieve the rest of the combined removal-inactivation requirement. This requires providing 68 percent (0.5-log) inactivation of *Giardia* cysts and 99 percent (2-log) inactivation of enteric viruses through disinfection when the plant operates as a "conventional filtration" treatment process. Ninety percent (1.0-log) inactivation of *Giardia* cysts and 99.9 percent (3-log) inactivation of enteric viruses are required through disinfection when the plant operates as a "direct filtration" treatment process.

The DHS, with regulatory primacy in California, includes a daily average treated water turbidity requirement of 0.5 NTU for water treatment plants, such as the District's WTP, that were new or upgraded prior to May 15, 1991. (The DHS criteria include a daily average treated water turbidity requirement of 0.2 NTU for water treatment plants that are new or upgraded after May 15, 1991.) Since the filtered water turbidity at the District's WTP is lower than both the USEPA and DHS filtered water turbidity standards throughout the entire year, the plant receives the maximum *Giardia* and virus removal credit associated with the two operating conditions ("conventional filtration" and "direct filtration"). This does require that the disinfection CT credits comply with different pathogen inactivation goals for the two operating conditions.

The SWTR also requires that systems demonstrate, by monitoring and recording, that they continuously maintain a disinfectant residual of at least 0.2 mg/L in water delivered to the public via the distribution system. Chlorine is currently used by the District to satisfy this requirement.

4.3.2 Total Coliform Rule

The TCR provides more stringent control and reduction of all pathogenic bacteria in distributed water. The District is currently in compliance with the TCR. Any improvements to the WTP should enhance the ability to remain in compliance with the TCR, including sample stations to comply with TCR monitoring requirements.

4.3.3 Lead and Copper Rule

The LCR regulates excessive corrosion leaching of these toxic metals from pipe materials, including service piping and customers' on-site piping. Data on lead and copper levels in the District's treated water supply delivered to the public indicates compliance with the requirements of the LCR.

4.3.4 Information Collection Rule

The ICR was a key element in the USEPA's Microbial/Disinfection By-Products (M/DBP) Reg-Neg process and was intended to provide more definitive information on specific source water quality, microorganism contaminants, and treatment plant performance including DBP generation. This regulation required most public water systems serving more than 100,000 people to collect data on their source and treated water and to provide this data to the USEPA for evaluation.

As part of the ICR effort, the District collected data on disinfection byproducts including trihalomethanes (THMs), haloacetic acids (HAA5), TOC, and bromide. The District can use the DBP data to develop an understanding of DBP generation at the WTP.

4.3.5 Partnership for Safe Water Guidelines

The "Partnership for Safe Water," prepared jointly by USEPA, the American Water Works Association (AWWA), and other water industry stakeholders, recommends an average filtered water turbidity of 0.1 NTU or less to ensure protection of the public. This filtered water turbidity goal is also recommended to maximize *Cryptosporidium* oocyst and other pathogenic organism removal. The average combined filtered water turbidity at the District's WTP has been less than 0.05 NTU during each of the most recent 66 months. This indicates the existing WTP is capable of complying with the Partnership for Safe Water guidelines.

4.3.6 California Cryptosporidium Action Plan

The DHS developed the California CAP in response to increased public health concern regarding *Cryptosporidium*. The return of spent filter backwash water and sedimentation basin waste solids have been shown in several studies to contain significantly higher particle concentrations than many source water supplies. Blending these high-risk recycle streams with the source water stream is a particular concern. The CAP established new turbidity goals for settled water, filtered water and return water. The settled (clarified) water turbidity goal includes settled water turbidity between 1 and 2 NTU at all times. The filtered water turbidity goals include both a 0.1 NTU goal for individual filters beginning 4 hours after a filter backwash and for the combined filtered water from all the filters at all times. The filtered water turbidity can be above 0.1 NTU, but should be below 0.3 NTU goal for individual filters during the first 4 hours following a filter backwash. A return (recycle) water turbidity goal was set at 2.0 NTU.

The District does not have sufficient monitoring data to determine the full impact of this guideline on the WTP. However, discussions with plant staff indicate that the return water turbidity is generally higher than the 2.0 NTU CAP guideline.

4.3.7 Fluoridation

The District is mandated by state law (Assembly Bill 733) to install a system to fluoridate its treated water for the protection and maintenance of public dental health when it receives sufficient capital and operational funds from any source (e.g. state, federal or private foundation grants) other than ratepayers or taxpayers. The District should reserve space at the WTP site for a fluoride storage and feed facility in the event funding becomes available.

4.4 New Drinking Water Regulations

New drinking water regulations and guidelines include regulations published in the Federal Register by the USEPA with implementation dates after July 31, 2000. These are summarized in Table 4-3 and discussed in more detail below.

Table 4-3
Summary of New Drinking Water Regulations

Regulation	Compliance Date	Targeted Contaminants	Comments
Interim Enhanced Surface Water Treatment Rule (IESWTR)	1 January 2002	Microbial	Includes a new *Cryptosporidium* removal requirement and a turbidity-based removal credit.
Stage 1 Disinfectants/Disinfection By-Products Rule (D/DBPR)	1 January 2002	Disinfectants, DBPs, and DBP Precursors	Includes new disinfection byproduct MCLs for THMs, HAA5, and bromate as well as new limits for disinfectant residuals.
Filter Backwash Rule (FBR)	---	Microbial	Targets *Cryptosporidium* and other contaminants in treatment process waste streams. Includes turbidity limits. Proposed rule published in Federal Register in April 2000.
Arsenic Rule	---	Arsenic	Proposed rule published in federal register in June 2000. Lowers MCL tenfold.

The new regulations include the IESWTR and Stage 1 D/DBPR, which were published in December 1998. The state primacy agencies have up to three years to adopt the IESWTR and Stage 1 D/DBPR. Public water supply agencies will have an additional two years to comply with these new regulations after they are adopted by the primacy agency. The DHS indicates that the Stage 1 D/DBPR and IESWTR are currently scheduled to be implemented in California on January 1, 2002.

4.4.1 Interim Enhanced Surface Water Treatment Rule

The IESWTR includes protection against *Cryptosporidium* oocysts, and benchmarking of existing disinfection practices at some water treatment plants.

The IESWTR was published in the Federal Register on December 16, 1998. The IESWTR includes a stringent new 2-log *Cryptosporidium* removal requirement and sets a Maximum Contaminant Level Goal (MCLG) at zero for the protozoan genus *Cryptosporidium*. Water treatment plants with a conventional or direct filtration treatment process meet this requirement if they comply with the new filtered water turbidity standards included in the IESWTR.

The IESWTR turbidity standard includes: 1) a combined filter effluent (CFE) turbidity of less than or equal to 0.3 NTU in at least 95 percent of the samples collected each month and 2) a CFE turbidity of less than 1 NTU in all samples collected at 4-hour intervals during each month. Water treatment plants in California modified after May 15, 1991 must produce filtered water with an average turbidity less than or at 0.2 NTU and should also comply with the CAP filtered water turbidity goals. Therefore, modifications to the District's WTP must permit compliance with the new CFE turbidity standards as well as the DHS average filtered water turbidity requirements and CAP turbidity guidelines.

The IESWTR also includes individual filter monitoring and reporting requirements. If the filtered water turbidity from a filter 1) exceeds 1.0 NTU in two consecutive 15 minute intervals or 2) exceeds 0.5 NTU in two consecutive 15 minute intervals after the initial 4 hours of operation following a filter backwash, then a filter profile report must be submitted to the DHS. Also, 1) if the filtered water turbidity from a filter exceeds 1.0 NTU in two consecutive 15-minute intervals during three consecutive months or 2) if the filtered water turbidity from a filter exceeds 2.0 NTU in two consecutive 15-minute intervals during two consecutive months, then a filter exceptions report must be submitted to the DHS, and the District must conduct a filter self-assessment.

In addition, the IESWTR includes new microbial disinfection profiling/benchmarking requirements for surface water treatment systems serving 10,000 or more people to ensure that compliance with the new Stage 1 D/DBPR MCLs will not reduce microbial protection as a result of efforts to reduce DBPs. The District ICR data indicates that both the THM and HAA5 levels are below the thresholds in the Federal IESWTR that would trigger a benchmark study.

The treated water from the District's WTP presently meets the IESWTR requirements.

4.4.2 Stage 1 Disinfectants and Disinfection By-products Rule

The Stage 1 D/DBPR regulates chemical compounds formed when disinfectants used for microbial control in drinking water react with organic and inorganic compounds in the source water. Disinfectants include chlorine, chlorine dioxide, chloramines, ozone and ultraviolet (UV) radiation. The Stage 1 D/DBPR sets new MCLs and MCLGs for selected DBPs, establishes maximum residual disinfectant levels (MRDLs) and MRDL Goals (MRDLGs), and establishes treatment techniques for control of DBP precursors (DBPPs). Surface water systems supplying more than 10,000 people such as the District's must comply with the Stage 1 D/DBPR by January 1, 2002.

The treated water from the District's WTP presently meets the Stage 1 - D/DBPR requirements.

4.4.3 Filter Backwash Rule

The USEPA published the proposed FBR as part of a combined Long-Term 1 ESWTR and FBR in April 2000. The new FBR establishes turbidity standards/criteria that must be met prior to returning spent filter backwash water to the treatment process.

The FBR is expected to include a return water turbidity goal similar to the DHS CAP return water turbidity goal; that is $\leq$ 2 NTU. Discussions with plant staff indicate that the maximum return water turbidity frequently exceeds 2 NTU. This suggests that the existing return water pretreatment process should be replaced with a more efficient pretreatment process in order to reduce the return water turbidity (solids).

4.4.4 Arsenic Rule

An Arsenic Rule was proposed in June 2000 and was scheduled to be promulgated in January 2001. Promulgation has been delayed, however, so the proposed rule can be further reviewed. The proposed rule includes an arsenic MCL of 5 micrograms per liter (µg/L) with a request for comments on MCLs of 3 and 10 µg/L. The surface water supply in Folsom Reservoir is originally from the western slope of the Sierra Nevada Mountains, a source typically free of arsenic. Historical water quality information indicates the water flowing into Folsom Reservoir complies with the new arsenic standard. However, if the District's WTP treatment process has to change in the future in order to comply with a new arsenic MCL, operation of the existing facilities could be modified to achieve a lower arsenic concentration in the treated water.

The most likely plant modification to comply with a low arsenic MCL would be to shift from using alum as the primary coagulant to using ferric chloride. The coagulant storage and metering system could be modified to permit using ferric chloride. It would also be necessary to increase the coagulant dose from the current low level, which is sufficient to meet filtered water turbidity goals, to a higher dose to enhance arsenic removal if the source water arsenic concentration exceeds the new arsenic MCL.

4.5 Anticipated Regulations and Guidelines

For this Master Plan, anticipated regulations and guidelines are those that the USEPA has indicated will be developed and published after July 31, 2000. These are summarized in Table 4-4. The anticipated regulations include a Filter Backwash Recovery Rule (FBRR), a Final (Long-Term 2) ESWTR, a Stage 2 – D/DBPR, and a Radionuclide(s) Rule.

Table 4-4
Summary of Anticipated Drinking Water Regulations

Regulation	Expected Date	Targeted Contaminants	Comments
Phase VIb	Unknown	IOCs, SOCs, VOCs	--
Long-Term 2 Enhanced Surface Water Treatment Rule	May 2002	Pathogens	May include additional turbidity or *Cryptosporidium* disinfection requirements
Stage 2 D/DBP Rule	May 2002	Disinfectants DBPs	--
Radon	August 2000	Radon	--
Phase III	November 2000	Radionuclides	--

4.5.1 Final (Long-Term 2) Enhanced Surface Water Treatment Rule and Stage 2 - Disinfectants and Disinfection By-Products Rule

The IESWTR and Stage 1 D/DBPR include new regulatory requirements that were not covered in the SWTR and other drinking water regulations. The USEPA indicates that additional drinking water regulations, applicable to large public agencies such as the District, will be included in the Long-Term 2 ESWTR and a likely Long-Term 3 ESWTR. The new IESWTR and subsequent Long-Term ESWTRs and their application to water supply agencies are summarized below:

- Interim ESWTR (applies to systems serving >10,000 people such as the District's)
- Long-Term 1 ESWTR (applies to systems serving <10,000 people)
- Long-Term 2 ESWTR (applies to systems serving >10,000 people)
- Long-Term 3 ESWTR (possible, but not definite)

The final (Long-Term 2) ESWTR and Stage 2 - D/DBPR are scheduled for promulgation in May 2002. These two regulations will be based on data collected as part of the ICR and on experience with the IESWTR and Stage 1 - D/DBPR. The potential impact of these rules on the District is not known since the rules are not defined.

The USEPA has indicated that there is a high probability that the Long-Term 2 ESWTR (LT 2 ESWTR), or possibly a later Long-Term 3 ESWTR (LT 3 ESWTR), will include an additional 0.5 to 1.0-log *Cryptosporidium* inactivation requirement. The USEPA also indicates that site-specific *Cryptosporidium* inactivation criteria will be established for each surface water treatment facility based on the presence of *Cryptosporidium* oocysts in the source water supply and the physical removal treatment processes at the plant. Source water quality and treatment plant performance will be used to assess the perceived risk that *Cryptosporidium* oocysts could be present in the treated water delivered to the distribution system. The USEPA indicates that at least two alternative technologies suitable for inactivating *Cryptosporidium* oocysts will be identified as part of this new water treatment requirement. Ozone, chlorine dioxide, and UV light are presently the most likely candidate technologies for *Cryptosporidium* disinfection.

4.5.2 Radon and Radionuclides

The USEPA published the proposed Radon Rule in November 1999 and was expected to promulgate a new rule for radon by August 2000 and a revised rule for other radionuclides by November 2000. Any impacts of the radionuclides rule (e.g., changing the gross alpha screening methods to account for radium-224 and polonium-210) would likely not require action by the District for the Folsom Reservoir source. The radon rule would only impact groundwater sources and not surface water sources such as Folsom Reservoir. Control of radionuclides, therefore, is not anticipated to be necessary for the District's WTP.

Monitoring data from the District's Annual Water Quality Reports for 1989 through 1998 shows that radioactivity levels for gross alpha, gross beta, radium-226, radium-228, radon-222, strontium-90, tritium, and uranium are well within existing federal and state levels.

Chapter 5: Treatment Plant - Water Quality Issues

5.1 Introduction

The previous chapter of this Master Plan summarized existing and anticipated drinking water regulations and their potential impact on the District's existing and expanded WTP. This chapter discusses water quality and the potential impacts on the WTP from the source water from Folsom Reservoir and the returned filter backwash water.

The discussion presented in this chapter includes recommendations regarding treated water quality objectives, the impact of source water quality on the WTP, the potential impacts of planned changes in lake-management practices for Folsom Reservoir, recommended additional water quality monitoring for the expanded WTP, and a recommended approach to address water quality issues.

Water quality information provided by the District on source water and treated water indicates that the existing water treatment facilities, with the exception of the filter backwash water treatment system, meet existing, new, and anticipated drinking water regulations. The backwash water treatment system and potential impacts from the Bureau's proposed Folsom Reservoir Temperature Control Device (TCD) should be addressed as part of the District's water quality management strategies. The recommended approach to address this issue is as follows:

Filter Backwash Water Treatment System

- Replace existing system with a new treatment system, including flow control, to comply with California CAP goals.

Temperature Control Device

- Notify the Bureau that the proposed TCD operating strategy could adversely impact WTP operations.

- Request/obtain source water quality data with respect to reservoir depth and seasonal variation to assess or predict potential impacts of the TCD.

Water quality issues and their potential impacts are summarized in Table 5-1.

Table 5-1
Summary of Water Quality Issues

Issue	Considerations	Potential Impacts
Source Water Quality	• Turbidity and microbial contaminants	• Existing plant operation meets treatment requirements. No change in operations.
	• Recreational use of Folsom Reservoir increases levels of microbial and SOC contaminants	• Existing plant operation meets treatment requirements. No change in operations.

Table 5-1 (cont.)
Summary of Water Quality Issues

Issue	Considerations	Potential Impacts
Lake Management Practices – Temperature Control Device (TCD)	• TCD operation could increase TOC and trigger enhanced coagulation	• May require enhanced coagulation to reduce TOC.
	• Enhanced coagulation could depress pH and require additional operational changes for LCR compliance	• May require adding lime or caustic soda to raise treated water pH or adding corrosion inhibitor to treated water.
	• Warmer summer-time source water could increase risk for taste and odor (T&O) problems as well as other water quality complaints	• May require adding powdered activated carbon, installing granular activated carbon (GAC) filter media, or using ozone to oxidize T&O compounds.
	• TCD may increase DBP Precursors	• May require enhanced coagulation to reduce DBP Precursors or switch to chloramines to control DBP formation in distribution system.
	• Increased risk of near-surface contaminants from recreation activities	• May require year-round "conventional filtration" treatment to comply with treated water requirements.
	• Winter-time high turbidity events could be reduced	• Could reduce duration that conventional filtration is required during winter.
	• TCD would permit withdrawing warmer water in winter to improve flocculation	• Could permit higher plant flow rates (shorter flocculation time) at times during winter.
Additional Recommended Source Water Quality Monitoring	• Profile turbidity, temperature, pH, dissolved oxygen (DO), particles, *Cryptosporidium,* oxidation-reduction potential (ORP), and TOC with respect to depth in Folsom Reservoir throughout year	• May require additional treatment processes to comply with treated water quality criteria.
	• Methyltertiary-butylethene (MTBE) and perchlorate in Folsom Reservoir	• Could require additional treatment systems.
Additional Recommended Water Quality Monitoring	• Return treated filter backwash water turbidity and particle counts	• If return water turbidity exceeds 2 NTU, would require adding return water pre-treatment process.

5.2 Treated Water Quality Goals

Table 5-2, presented at the end of this chapter, summarizes current water quality standards, historical water quality for the District's WTP, and recommended treated water objectives for the WTP. The recommended treated water quality goals are based on compliance with the current, new, and anticipated federal and state drinking water regulations and guidelines described in Chapter 4. Recommended treated water quality goals also include:

1. Individual filtered water turbidity less than 0.3 NTU for individual filter operation within one hour after a filter backwash;

2. Individual filtered water turbidity that is less than 0.1 NTU for filter operation between 1 hour after a filter backwash until the end of the filter run;

3. A combined filtered water turbidity less than 0.1 NTU at all times;

4. A disinfection CT credit that, in conjunction with the physical removal treatment credits (2.5-log *Giardia* removal and 2-log virus removal for "Conventional Filtration" and 2.0-log *Giardia* removal and 1-log virus removal for "Direct Filtration" treatment), provides at least 3-log *Giardia* removal-inactivation and 4-log virus removal-inactivation at all times;

5. Local running annual average (LRAA) THM and HAA5 concentrations at each of the Initial Distribution System Evaluation sites in the distribution system that are less than 80 μg/L and 60 μg/L, respectively;

6. A non-corrosive treated water supply that has a 90th percentile lead concentration below 1.3 mg/L and a 90th percentile copper concentration below 0.015 mg/L in first-draw water samples collected from vulnerable household faucets every 3 years;

7. A chlorine residual concentration above 0.2 mg/L or a heterotrophic bacteria density that is less than 400 colony forming units (CFU) per 1 milliliter in at least 95 percent of water samples collected each month throughout the distribution systems that receive treated water from the District's WTP;

8. A disinfection system that provides a combination of disinfection CT credit and/or irradiance and exposure time credit complying with the USEPA *Cryptosporidium* inactivation goals for source water from Folsom Reservoir based on the USEPA's proposed *Cryptosporidium* sampling program and the resultant *Cryptosporidium* disinfection goal described by the USEPA in the Stage 2 M/DBP Agreement in Principle;

If the source water *Cryptosporidium* concentration requires using either ozone or chlorine dioxide as a primary disinfectant, the treated water quality goals should also include a bromate concentration in the treated water that is less than 5 μg/L and a chlorite concentration in the treated water that is less than 0.8 mg/L;

If the District replaces the existing chlorine gas system with either on-site hypochlorite generation units or bulk deliveries of hypochlorite solution, then the treated water quality goals should also include a bromate concentration in the treated water below 5 μg/L and a chlorite concentration in the treated water below 0.8 mg/L.

5.3 Source Water Quality

The surface water supply treated at the District's WTP is diverted from Folsom Reservoir. This surface water supply can be generally characterized as a high-quality source water that is low in alkalinity, DBP precursor materials, mineral content, and organic contamination. However, Folsom Reservoir is used for public recreation, and source water stored in the reservoir is vulnerable to contamination. This surface water supply must be treated to reduce turbidity and microbial contaminants to meet current state and federal drinking water regulations and state guidelines. In addition, high raw water turbidity can occur seasonally in Folsom Reservoir due to winter-time storms and spring-time high flows in the American River Watershed.

The existing plant processes, including rapid mix (coagulation) and pre-treatment (flocculation-sedimentation) systems followed by filtration, produce filtered water meeting existing, new, and anticipated regulations and guidelines. The maximum TOC concentration in the Folsom Reservoir source water is currently below the USEPA's 2.0-mg/L limit. Therefore, enhanced coagulation, which requires applying a high coagulant dose to reduce TOC, is not required for the current source water supply.

5.4 Lake Management Impacts

The Bureau has proposed installing a TCD on the outlet structure at Folsom Reservoir. The proposed TCD would permit withdrawing water from the upper, epilimnetic zone in Folsom Reservoir for delivery to the District in order to reserve colder water for improving downstream fisheries. The warmer epilimnetic zone water may increase the average source water temperature by between 5 and 13 degrees Celsius (°C) during the period between April and October each year. Prior experience treating raw water from Folsom Reservoir indicates that warm source water supplies are more vulnerable than cold water supplies to taste and odor causing compounds. The Draft TCD Report also indicates that the warmer epilimnetic source water may contain high levels of DBP precursors. In addition, the epilimnetic water is more vulnerable to both microbial and synthetic organic chemical (SOC) compound contamination due to recreational uses. If TCD operations result in withdrawal of water from the thermocline elevation in the reservoir, this could exacerbate taste and odor (T&O) problems since dead organisms tend to accumulate at this level and release organic compounds as they decay.

The TCD may permit reducing the source water turbidity by positioning the TCD gate(s) to withdraw raw water from Folsom Reservoir at levels with lower turbidity water during and following periods when winter-time and spring-time high turbidity run-off flows into the reservoir. This would reduce the solids load on the treatment processes.

5.5 Recommended Water Quality Monitoring

Based on requirements in the existing, new, and anticipated regulations; on operating experience at the WTP; and on planned changes in lake management practices, it would be prudent to gather additional water quality data. Samples of Folsom Reservoir source water and return water at the WTP should be collected to develop the data.

5.5.1 Folsom Reservoir Source Water Quality

Enhanced coagulation to reduce DBP precursors (DBPPs), measured as TOC, is also a part of the Stage 1 - D/DBPR. The enhanced coagulation requirement applies to water treatment plants with "conventional filtration treatment" and is required if the source water TOC exceeds 2 mg/L. The District's WTP has a "conventional filtration treatment process," but the average source water TOC level is less than 1 mg/L; hence, the District's WTP is not currently required to practice "enhanced coagulation."

On-going water quality monitoring of Folsom Reservoir source water may permit evaluating the impact of planned lake management changes on source water quality. The District should continue to collect water quality data for daily, monthly, and annual water quality reports. In addition, water quality data on temperature, turbidity, particles, pH, TOC, dissolved oxygen, and oxidation-reduction potential with respect to depth in Folsom Reservoir should be collected to develop a basis for evaluating the proposed modifications to current reservoir management strategies on source water quality and plant performance.

The particle density in Folsom Reservoir source water should be evaluated to determine whether there are significant variations in the particle sizes and densities with respect to depth and thermocline depth in Folsom Reservoir. If the new Bureau water withdrawal strategy for Folsom

Reservoir takes water from the reservoir at or just above the thermocline, the source water quality may adversely impact existing WTP performance.

Recent public health awareness and concern about possible source water contamination by the fuel additive MTBE and the solid rocket-fuel component perchlorate warrant adding these two compounds to the list of chemicals monitored as part of the District's regular source water quality monitoring program.

The EPA Stage 2 M/DBP draft Agreement in Principle will require collecting monthly samples of source water in order to develop source-specific *Cryptosporidium* disinfection requirements. The District should coordinate a sampling program with the Bureau and Cities of Roseville and Folsom to gather data on the presence of *Cryptosporidium* at the existing intake elevation as well as at the future TCD inlet elevations.

5.5.2 Settled Water Quality

The settled water turbidity should continue to be monitored to verify compliance with the CAP goal that the settled water turbidity be less than 2 NTU at all times.

5.5.3 Filtered and Treated Water Quality

A review of filtered water data provided by the District indicates that the average combined filtered water turbidity has been at or below 0.05 NTU during the 66-month period between January 1994 and July 1999. The data also indicates that the 95th and 99th percentile and the maximum combined filtered water turbidity have been at or below 0.07, 0.25, and 0.50 NTU, respectively, during this 66-month period. The data suggests that the District's WTP should be capable of complying with the new IESWTR filtered water turbidity goals when operating under current or similar conditions.

The federal IESWTR requires that all public water supply agencies serving more than 10,000 people must collect concurrent sets of data on THMs and HAA5 in order to determine whether the water utility should conduct disinfection profiling data. If the average THM or HAA5 concentrations exceed 80 percent of the Stage 1 D/DBPR MCLs, then the agency is required to conduct a disinfection CT benchmark study. Although the District's ICR data indicates that it would not be required to benchmark disinfection CT performance based on the USEPA criteria, we recommend the District collect disinfection CT credit profile data for twelve months in order to prepare a plant benchmark. The plant benchmark must be completed prior to modifying existing plant processes.

The District should use plant CT data collected during the ICR to determine the current CT disinfection benchmark.

5.5.4 Return Water (Recycled Spent Filter Backwash Water)

Both the USEPA and DHS have indicated concern about elevated risks of recycling microbial contaminants associated with spent filter backwash water. High concentrations of pathogenic microorganisms in spent filter backwash water, as well as other solids which may be present, can challenge treatment process performance and/or capacity if they are returned to the head of the plant and blended with the source water. Both the USEPA and DHS are concerned that elevated levels of pathogenic organisms that may be present in the return water could increase the risk that the treated water would contain unacceptable pathogen concentrations or that elevated levels of solids could adversely affect filter operation/performance.

Particle counters can provide a more sensitive method of monitoring the particle distribution and density in the spent filter backwash water than turbidimeters can. The particle distribution and density in the return water from the two spent filter backwash water clarifiers should be evaluated in order to assess the range of particle densities in the return water in comparison to the particle densities in the source water from Folsom Reservoir.

The District does not monitor return water turbidity. However, discussions with plant staff indicate that the return water turbidity is generally higher than the California CAP 2 NTU goal most of the time. This suggests that the existing return water pretreatment process should be replaced with a more efficient pretreatment process to reduce return water turbidity to below the recommended 2 NTU goal. A more efficient return water pretreatment process should permit delivering return water without causing the severe turbidity and particle loads that the existing return water pretreatment system does.

Replacing the existing return water pretreatment process with a more efficient pretreatment process may also reduce the amount of TOC returned to the plant via the filter backwash water recovery system. This may reduce DBPs and should have a beneficial impact on the amount of chlorine required to provide the residual disinfectant levels and DBPs.

5.6 Recommended Approach To Address Water Quality Issues

Current plant operating data summarized in Table 5-2, located at the end of this chapter, indicates that the treated water complies with existing, new, and anticipated water quality regulations. The high-quality source water permits complying with both microbial removal-inactivation requirements while producing THMs and HAA5 below both the new Stage 1 D/DBPR THM and HAA5 MCLs and the September 2000 draft Stage 2 D/DBPR THM and HAA5 MCLs. However, the aggressive source water requires adding lime to increase the treated water pH to ensure compliance with the LCR. Although the high-quality source water permits compliance with existing, new, and anticipated regulations, the filter backwash and solids processing system(s) should be modified in order to comply with the existing California CAP and the anticipated FBRR.

Planned modifications to the Folsom Reservoir raw water outlet could have an adverse impact on plant operation and performance. The source water quality within the Folsom Reservoir epilimnetic zone should be monitored for turbidity, temperature, *Cryptosporidium*, TOC, alkalinity, pH, dissolved oxygen, and algae as recommended above in Section 5.5.1, in order to evaluate potential impact(s) of changes in these constituents on regulatory compliance and plant operation.

If the raw water TOC concentration increases above 2 mg/L, and local running and annual average THMs exceed 80 μg/L or HAA5 exceed 60 μg/L at any location, the District may want to demonstrate that the TCD impacts compliance with water quality regulations. The DHS has indicated that they would permit the District to operate the plant as a direct filtration treatment process when source water turbidity is below 20 NTU. However, the District could be required to operate the plant as a conventional filtration treatment process if the TOC and THMs or HAA5 exceeds the threshold limits noted above. If the plant operates as a conventional filtration treatment process with enhanced coagulation in order to comply with TOC reduction criteria, it would impact water treatment costs. Pre-treatment facilities would have to be modified to increase conventional pretreatment capacity from about 60 mgd to the plant's filtration capacity (between 150 and 240 mgd depending on future treatment capacity goals). If the raw water TOC concentration exceeds 2 mg/L due to operation of the TCD, the District should discuss the TCD's impact on the cost to treat water with the Bureau.

Although higher average raw water temperatures in May and June due to the TCD could improve flocculation, reduce the required flocculation time, and improve filter performance, higher average raw water temperatures in July through September could increase T&O complaints and adversely impact compliance with new and anticipated THM and HAA5 MCLs.

Water quality information provided by the District on source, settled, filtered, treated, and return water indicates that the existing water treatment facilities, with the exception of the filter backwash water treatment system, meet existing, new and anticipated drinking water regulations for most plant operating conditions. However, there is one existing problem and one potential problem that should be addressed as part of the plant improvements.

1. The filter backwash water treatment system should be replaced with a new treatment system to reduce the risk that contaminants, including *Cryptosporidium*, will be returned in a concentrated level to the treatment process.
2. The Bureau's proposed Folsom Reservoir TCD may adversely impact source water quality with respect to taste and odor compounds, DBPPs, and TOC. The Bureau's proposed TCD operating strategy could force the District to modify current plant operating practices and significantly increase operating costs. The District should notify the Bureau that the proposed TCD operating strategy could adversely impact plant operations. The District should also request that the Bureau provide data on source water quality with respect to both depth and seasonal variation in order to predict the impact of the proposed TCD operation on plant operation and regulatory compliance.

Table 5-2
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[a] Source Water Quality			Recommended Objectives for SPWTP[b]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[j]	Max.	Min.	
GENERAL										
Color	CU	-	15	-	-	15	1	<3	<3	≤3
Corrosivity (Langelier Index)	LI	-	Noncorrosive	-	-	Noncorrosive	-0.87	+0.10	-1.7	Noncorrosive
Corrosivity (Aggressive Index)	AI	-	Noncorrosive	-	-	Noncorrosive	11.17	11.94	10.14	>12
Corrosivity (Larson Index)	LnI	-	Noncorrosive	-	-	Noncorrosive	0.52	0.80	0.19	<0.4
Copper Pitting Propensity	CPP	-	Noncorrosive	-	-	Noncorrosive	-2	-6	+1	<0
Sulfate / Chloride	Ratio	-	Noncorrosive	-	-	Noncorrosive	1.2	4	0.9	<3
Foaming Agents (MBAS)	mg/L	-	0.5	-	-	0.5	0.01	0.015	<0.01	≤0.2
pH	units	-	6.5-8.5	-	-	6.5-8.5	8.3	8.9	7.5	6.5-8.5
Specific Conductance	µmho/cm	-	900	-	-	-	80	96	63	<700
Temperature	°C	-	-	-	-	-	12	22	8	-
Total Alkalinity (as $CaCO_3$)	mg/L	-	-	-	-	-	27	33	21	-
Taste and Odor	TON	-	3	-	-	3	0.6	3	<1	<1
Total Dissolved Solids	mg/L	-	500	-	-	500	52	74	28	<500
Total Hardness (as $CaCO_3$)	mg/L	-	-	-	-	-	34	46	27	<175
Turbidity (Source water-raw)[d]	NTU	-	-	-	-	-	6	508	0.1	N/A
Turbidity (Combined filtered water)[d]	NTU	0.2	5	-	TT	-	0.03	0.49	0.02	≤0.1
Visibility (secchi disk)	feet	-	-	-	-	-	-	-	-	

Table 5-2 (cont.)
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[(a)] Source Water Quality			Recommended Objectives for SPWTP[(b)]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[(j)]	Max.	Min.	
MICROBIOLOGICAL										
Giardia Lamblia	No./100 L	-	-	0	TT	-	-	-	-	0
Cryptosporidium	No./100 L	-	-	0	TT	-	-	-	-	0
Legionella	No./ml	-	-	0	TT	-	-	-	-	0
Heterotrophic Plate Count	CFU/ml	-	-	-	TT	-	-	-	-	<200
Total Coliform	MPN/100 ml	1	-	0	ABS	-	-	-	-	0
Viruses	No./ml	-	-	0	TT	-	-	-	-	0
INORGANIC CHEMICALS										
Aluminum	µg/L	1000	200	-	-	-	33	94	<50	<1000
Arsenic	µg/L	50	-	-	2-20 (TBP)	-	1.5	<4.0	<1	<10[(e)]
Asbestos (>10 µm)	MF/L	7	-	7	7	-	-	-	-	<7
Barium	µg/L	1000	-	2	2	-	14	14	10	<1000
Beryllium	µg/L	4	-	4	4	-	0.5	<1.0	<1	<4
Bicarbonate	mg/L	-	-	-	-	-	25	30	21	
Boron	µg/L	-	-	-	-	-	-	-	-	<1000
Cadmium	µg/L	5	-	5	5	-	0.1	<0.1	<0.1	<5
Calcium	mg/L	-	-	-	-	-	10.6	14	8.5	
Carbonate	µg/L	-	-	-	-	-	1	4	1	
Chloride	mg/L	-	250	-	-	250	4	5	2	<250
Chromium	µg/L	50	-	100	100	-	0.5	<10	<10	<5

Table 5-2 (cont.)
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[a] Source Water Quality			Recommended Objectives for SPWTP[b]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[j]	Max.	Min.	
CO_2	µg/L	-	-	-	-	-	-	-	-	
Copper[f]	µg/L	-	1000	1.3	TT	1	<2	11	<10	<200
Cyanide	mg/L	0.2	-	0.2	0.2	-	<.004	0.006	<0.003	<0.2
Fluoride	mg/L	2	-	4	4	2	0.05	<0.1	<0.1	0.7-1.2
Iron	µg/L	-	300	-	-	300	20	<30	<30	≤200
Lead[f]	µg/L	-	-	0	TT	-	0.8	<1.0	<1	<10
Manganese	µg/L	-	50	-	-	50	8	37	<5	<10
Mercury	µg/L	2	-	2	2	-	0.2	<0.2	<1	<2
Nickel	µg/L	100	-	100	100	-	3	<5.0	<5	<100
Nitrate (as N)	mg/L	10	-	10	10	10				<10
Nitrite (as N)	mg/L	1	-	1	1	1				<1
Selenium	µg/L	50	100	50	50	-	0.8	<0.5	<0.5	<50
Silver	µg/L	-	-	-	-	-	7	30	<10	
Sodium	mg/L	-	-	-	-	-	2.3	3	2	
Sulfate	mg/L	-	250	500	500	250	6	9	4	<250
Thallium	µg/L	2	-	0. 5	2	-	0.5	<1.0	<1.0	<2
Zinc	µg/L	-	5	-	-	5	*8*	<5.0	<5	<1

Table 5-2 (cont.)
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[a] Source Water Quality			Recommended Objectives for SPWTP[b]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[j]	Max.	Min.	
RADIONUCLIDES										
Gross Alpha	pCi/L	15	-	0(TBP)	15	-	0.8	<1.0	<1.8	<15
Gross Beta	pCi/L	50	-	0(TBP)[g]	4(TBP)[g]	-	4	17.5	<0.5	<4
Radon	pCi/L	-	-	0(TBP)	(TBP)	-	-	-	-	<200
Strontium 90	pCi/L	8	-	-	-	-	0.05	0.05	0.05	
Tritium	pCi/L	20,000	-	-	-	-	-5.5	-5.5	-5.5	<10,000
Uranium[h]	pCi/L	20	-	0(TBP)	30(TBP)		-	-	-	<20
ORGANIC CHEMICALS										
Alachlor	µg/L	2	-	0	2	-	<1.0	<1.0	-	<2
Aldicarb	µg/L	-	-	1	3	-	-	-	-	<1
Aldicarb sulfone	µg/L	-	-	1	2	-	-	-	-	<1
Aldicarb sulfoxide	µg/L	-	-	1	4	-	-	-	-	<1
Aldrin	µg/L	-	-	-	-	-	-	-	-	<0.05
Atrazine	µg/L	3	-	3	3	-	<1.0	<1.0	-	<3
Baygon (Dropoxur)	µg/L	-	-	-	-	-	-	-	-	<90
Bentazon (Basagran)	µg/L	18	-	-	-	-	-	-	-	<18
Benzene	µg/L	1	-	0	5	-	<0.5	<0.5	-	<1
a-Benzene hexachloride	µg/L	-	-	-	-	-	-	-	-	<0.7
b-Benzene hexachloride	µg/L	-	-	-	-	-	-	-	-	<0.3
Benzopyrene	µg/L	0.2	-	0	0.2	-	-	-	-	<0.2

Table 5-2 (cont.)
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[(a)] Source Water Quality			Recommended Objectives for SPWTP[(b)]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[(j)]	Max.	Min.	
Captan	μg/L	-	-	-	-	-	-	-	-	<350
Carbaryl	μg/L	-	-	-	-	-	-	-	-	<60
Carbofuran	μg/L	18	-	40	40	-	-	ND	-	<40
Carbon tetrachloride	μg/L	0.5	-	0	5	-	<0.5	<0.5	-	<0.5
Chlordane	μg/L	0.1	-	0	2	-	-	-	-	<0.1
Chlorobenzene	mg/L	-	-	-	-	-	-	-	-	<30
Choramben	μg/L	-	-	-	-	-	-	-	-	
Chloropicrin	mg/L	0.05(AL)	-	-	-	-	-	-	-	<0.05
CIPC (isopropyl N-(3-chlorophenyl carbamate)	μg/L	350	-	-	-	-	-	-	-	<350
Dalapon	μg/L	200	-	200	200	-	-	ND	-	<200
Diazinon	μg/L	-	-	-	-	-	<0.25	<0.25	-	<14
Dibromchloropropane (DBCP)	μg/L	0. 2	-	0	0.2	-	-	-	-	<0.2
1,2-Dibromoethane	μg/L	-	-	-	-	-	-	-	-	<20
1,2-Dichlorobenzene	mg/L	0.6	-	0.6	0.6	-	<0.5	<0.5	-	<0.13
1,3-Dichlorobenzene	mg/L	-	-	-	-	-	-	-	-	<0.13
1,4-Dichlorobenzene	μg/L	5	-	-	-	-	<0.5	<0.5	-	<5
1,1-Dichloroethane	μg/L	5	-	-	-	-	<0.5	<0.5	-	<5
1,2-Dichloroethane	μg/L	0.5	-	0	5	-	<0.5	<0.5	-	<0.5
Di(2-ethylhexyl) adipate	μg/L	400	-	400	400	-	-	-	-	<400
Di(2-ethylhexyl) phthalate	μg/L	4	-	0	6	-	-	-	-	<4

Table 5-2 (cont.)
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[a] Source Water Quality			Recommended Objectives for SPWTP[b]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[j]	Max.	Min.	
1,1-Dichloroethylene	µg/L	6	-	7	7	-	<0.5	<0.5	-	<6
cis-1,2-Dichloroethylene	µg/L	6	-	70	70	-	<0.5	<0.5	-	<6
trans-1,2-Dichloroethylene	µg/L	10	-	100	100	-	<0.5	<0.5	-	<10
1,2-Dichloropropane	µg/L	5	-	0	5	-	<0.5	<0.5	-	<5
1,3-Dichloropropene	µg/L	0. 5	-	-	-	-	<0.5	<0.5	-	<0.5
Dieldrin	µg/L	-	-	-	-	-	-	-	-	<0.05
Dimethoate	µg/L	-	-	-	-	-	<10	<10	-	<140
2,4-Dimethylphenol	µg/L	-	-	-	-	-	-	-	-	<400
Dinoseb	µg/L	7	-	7	7	-	-	-	-	<7
Diphenamide	µg/L	-	-	-	-	-	-	-	-	<40
Diquat	µg/L	20	-	20	20	-	-	-	-	<20
2,4-D	µg/L	70	-	70	70	-	0.02	<0.1	ND	<70
Endothall	µg/L	100	-	100	100	-	-	-	-	<100
Endrin	µg/L	2	-	2	2	-	0.02	<0.1	ND	<2
Epichlorohydrin	µg/L	-	-	0	TT	-	-	-	-	
Ethion	µg/L	-	-	-	-	-	-	-	-	<35
Ethylbenzene	mg/L	0.7	-	0.7	0.7	-	0.25	<0.5	-	<0.3
Ethylene dibromide (EDB)	µg/L	0. 05	-	0	0.05	-	ND	ND	-	<0.02
Ethylparathion	mg/L	-	-	-	-	-	-	-	-	<30
Formaldehyde	mg/L	0.03(AL)	-	-	-	-	-	-	-	

Table 5-2 (cont.)
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[a] Source Water Quality			Recommended Objectives for SPWTP[b]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[j]	Max.	Min.	
Glyphosate	µg/L	700	-	700	700	-	-	-	-	<700
Heptachlor	µg/L	0. 01	-	0	0.4	-	-	-	-	<0.01
Hepachlor epoxide	µg/L	0. 01	-	0	0.2	-	-	-	-	<0.01
Hexachlorobenzene	µg/L	1	-	0	1	-	-	-	-	<1
Lindane	µg/L	0.2	-	0. 2	0.2	-	0.02	<0.1	ND	<0.2
Malathion	µg/L	-	-	-	-	-	-	-	-	<160
Methoxychlor	µg/L	40	-	40	40	-	0.02	<0.1	ND	<40
Methyl parathion	µg/L	-	-	-	-	-	-	-	-	<30
Methyllene chloride	µg/L	-	-	-	-	-	0.35	0.55	ND	<40
Molinate	µg/L	20	-	-	-	-	<2.0	<2.0	-	<20
Monochlorobenzene	µg/L	70	-	100	100		<0.5	<0.5	-	<30
Napthalene	µg/L	-	-	-	-	-	-	-	-	
Oxamyl (vydate)	µg/L	200	-	200	200	-	-	-	-	<200
Penthachlorophenol	µg/L	1	-	0	1	-	-	-	-	<1
Polyaromatic hydrocarbons (PAHs)	µg/L	-	-	-	-	-	-	-	-	
Polychlorinated biphenyls (PCBs)	µg/L	0.5	-	0	0.5		-	-	-	<0.5
Simazine	µg/L	4	-	4	4	-	<1.0	<1.0	-	<4
Styrene	µg/L	100	-	100	100	-	<0.5	<0.5	-	<50
1,1,2,2-Tetrachloroethane	µg/L	1	-	-	-	-	ND	ND	-	<1

Table 5-2 (cont.)
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[a] Source Water Quality			Recommended Objectives for SPWTP[b]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[j]	Max.	Min.	
Tetrachloroethylene	µg/L	5	-	0	5	-	<0.5	<0.5	-	<5
Tetrachlor (Pentachloronitrobenzene)	µg/L	-	-	-	-	-	-	-	-	<0.9
Thiobencarb	µg/L	70	1	-	-	-	<1.0	<1.0	-	<70
Toluene	µg/L	150	-	1000	1000	-	<0.5	<0.5	-	<40
Toxaphene	µg/L	3	-	0	5	-	ND	ND	-	<3
1,1,1-Trichloroethane	µg/L	200	-	200	200	-	<0.5	<0.5	-	<200
1,1,2-Trichloroethane	µg/L	5	-	3	5	-	<0.5	<0.5	-	<5
Trichloroethylene	µg/L	5	-	0	5	-	<0.5	<0.5	-	<5
Trichlorofluoromethane (Freon 11)	µg/L	150	-	0.7	-	-	<5	<5	-	<150
1,1,2-Trichloro-1,2,2-Trifluoroethane (Freon 113)	µg/L	1200	-	4000	-	-	-	-	-	<1200
Trithion	µg/L	-	-	-	-	-	-	-	-	<7
(Dioxin)	µg/L	3.00E-5	-	0	5.00E-5	-	-	-	-	<3.00E-5
(Silvex)	µg/L	50	-	50	50	-	0.02	0.1	ND	<10
Vinyl chloride	µg/L	0. 5	-	0	2	-	<0.5	<0.5	-	<0.5
Xylene (total)	µg/L	1750	-	10000	10000	-	<0.5	<0.5	-	<20

Table 5-2 (cont.)
San Juan Water District – Sidney N. Peterson Water Treatment Plant
Water Quality Standards, Historical Water Quality and Treated Water Objectives

Characteristic	Units	California DHS Standards		Federal EPA Standards			Typical Folsom Reservoir[a] Source Water Quality			Recommended Objectives for SPWTP[b]
		Primary	Secondary	Goals	Primary	Secondary	Avg.[j]	Max.	Min.	
DISINFECTANTS, DISINFECTION BYPRODUCTS AND PRECURSORS										
Chlorine (as Cl_2)	mg/L	-	-	4	4.0	-	-	-	-	0.2-1.0
Chlorite	mg/L	-	-	0.8	1.0	-	-	-	-	<0.8
Haloacetic Acids[i]	µg/L	-	-	48	60	-	19	29	12	<30
Trihalomethanes[i]	µg/L	100	-	64	80	-	32	52	14	<40
Bromate[j]	µg/L	-	-	0	10	-	-	-	-	<5
Total Organic Carbon	mg/L	-	-	-	TT	-	0.8	1	0.7	≤2

NOTES:

(a) From SJWD Annual Reports for 1989 through 1998.
(b) Recommended objectives set at new DHS Action Limits, at or below either the MCL or MCLG, or above the average source water concentration where appropriate.
(c) Not used.
(d) Monthly DHS Reports.
(e) Temporary placeholder.
(f) Lead and Copper Rule MCLs at 90th percentile of consumer's taps are 5 µg/L and 1.3 mg/L, respectively. The EPA requires large systems to optimize the treated water to meet a lead concentration <10 µg/l at the customer's tap.
(g) Federal MCL set at 4 mrem/year.
(h) The EPA Uranium MCL is 20 µg/l which equivalent to 30 pCi/L.
(i) For samples from __ locations in portions of SJWD wholesale distribution system that receive mostly treated water from SPWTP in 199_.
(j) For the calculation of averages containing ND results, a value of ½ the detection limit is used. When no detection limit is available the results are not used. When ND is the only results reported a value of ND is used for average value.

ABBREVIATIONS:

ABS - No more than 5% of samples collected during a month may be coliform positive
AL - Action Level
MCL - Maximum Contaminant Level
ND - Not Detected
TBP - To Be Proposed
TT - Treatment Technique Required in Lieu of Monitoring
MF - Million Fibers
\- - No Available Data

Chapter 6: Current Treatment Plant Capacity and Short-Term Improvements

6.1 Introduction

The District's long-term objective for the WTP is to incrementally expand its capacity to economically meet increasing water demands through build-out of the service area. The District is also participating in regional planning to determine how to use surface water and ground water to meet community water requirements and environmental needs. The District's WTP could play a key role in treating surface water for a regional conjunctive use plan.

For the year 2030 planning horizon of this report, the capacity requirement for the WTP to meet the existing wholesale and retail area water demand is 150 mgd. The District estimates that a WTP capacity of as much as 240 mgd will be required to assist in meeting regional demands. However, the District is also interested in identifying ways to immediately increase the reliable capacity of the existing WTP to 120 mgd to meet short-term water demands. This chapter presents results of process and hydraulic evaluations to determine the existing reliable WTP capacity and recommendations to meet the short-term capacity objective of 120 mgd. Table 6-1 presents a summary of the evaluation results.

Table 6-1
Water Treatment Plant Evaluation Summary and Recommendations for 120 mgd Capacity

Process	Process Capacity mgd	Hydraulic Capacity mgd	Recommendations to Increase Capacity to 120 mgd
Rapid Mix	120	<110	Increase opening size between chambers to improve hydraulics and increase flow through structure. No process modification required.
Pretreatment Flocculation Basins	 130	 <110	 Enlarge openings or add additional openings to the flocculation basin distribution trough to improve hydraulics and increase flow.

Table 6-1 (cont.)
Water Treatment Plant Evaluation Summary and Recommendations for 120 mgd Capacity

Process	Process Capacity mgd	Hydraulic Capacity mgd	Recommendations to Increase Capacity to 120 mgd
Sedimentation	60 for Conventional Filtration Treatment N/A Direct Filtration Treatment	100	Improve hydraulic capacity of the sedimentation basin launders by removing “blanked” off sections of weirs. Improve supports, stiffen launders, or add additional orifices to mitigate oscillation problems as required. Operate WTP as direct filtration rather than conventional WTP when treating more than 60 mgd to eliminate sedimentation basins from process.
Settled Water Channel	NA	<110	Raise emergency overflow weir to increase hydraulic capacity.
Filtration	120	<110	Orient conduits and wiring to backwash hood position indicators to keep conduits above “high water line.” Raise emergency overflow weir to increase available head on filters (hydraulic improvement).
Treated Water Piping	NA	<120	Confirm headloss through treated water piping and discharge structures. Modify piping and structures, or construct parallel piping.
Disinfection System	120 +	120+	Disinfection system capacity sufficient for 120 mgd conventional filtration and direct filtration treatment. Modify Hinkle Reservoir connections to achieve additional chlorine disinfection contact time if in-line filtration capability desired.
Backwash Water Recovery System	1.6 mgd (4.8 mgd required for 120 mgd WTP)	NA	Major improvements required, but not possible short-term. System deficiency will need to be addressed through operator skill and intensive labor efforts for the short-term. New washwater recovery system should be designed and constructed prior to peak demand period in 2002.
Solids Handling	<100	NA	Major improvements required, but not possible short-term. System deficiency will need to be addressed through operator skill and ingenuity for the short-term. New solids handling system should be designed and constructed prior to peak demand period in 2003.
Chemical Feed Systems	120+	NA	No modifications required.

Following the preparation of a draft version of this report chapter in the spring of 2000, the District completed the recommended improvements to the rapid mix basins, removed the blanked off sections of the sedimentation basin launder weirs, and raised the emergency overflow weir. These improvements helped to increase WTP capacity during the summer of 2000 from approximately 108 mgd to approximately 115 mgd.

6.2 Process Capacity Evaluation

The existing plant capacity was evaluated using design standards developed by the USEPA, DHS, AWWA, and other water industry stakeholders to determine the current process capacity of each major treatment process system. Plant staff also provided valuable operational insight used to evaluate the performance of each plant process.

The WTP was designed as a "conventional filtration treatment process" incorporating chemical oxidation and initial disinfection, followed by coagulation in a three-stage rapid mix system, flocculation, sedimentation, filtration, and final disinfection. Although the original WTP design criteria state the capacity of the WTP is 100 mgd, current EPA and DHS guidelines indicate the WTP capacity as a conventional filtration process is more on the order of 60 mgd due to limitations of the sedimentation basins. This is substantiated by operator experience that the sedimentation basin performance deteriorates dramatically when the flow through the basins exceeds about 60 mgd. However, it should be noted that the DHS currently classifies the WTP as a "conventional filtration plant" for flow rates below 100 mgd and as a "direct filtration" plant for flow rates above 100 mgd.

Based on this observation and WTP operational practices at flows above 60 mgd, the existing plant capacity was also evaluated with the WTP operating as a "direct filtration treatment process." This process incorporates oxidation and initial disinfection, followed by coagulation in a rapid mix system, flocculation, filtration, and final disinfection. Since the sedimentation step (part of the physical removal process) is eliminated from the conventional treatment process in this approach, the pathogen removal credits are lower (2.0-log *Giardia* removal versus 2.5-log *Giardia* removal and 1.0-log virus removal versus 2.0-log virus removal) and hence, additional disinfection credit is required. The process capacity of the WTP in a direct filtration mode is 120 mgd.

Figure 6-1 presents a process schematic of the existing system, and Table 6-2 presents the results of the capacity evaluation.

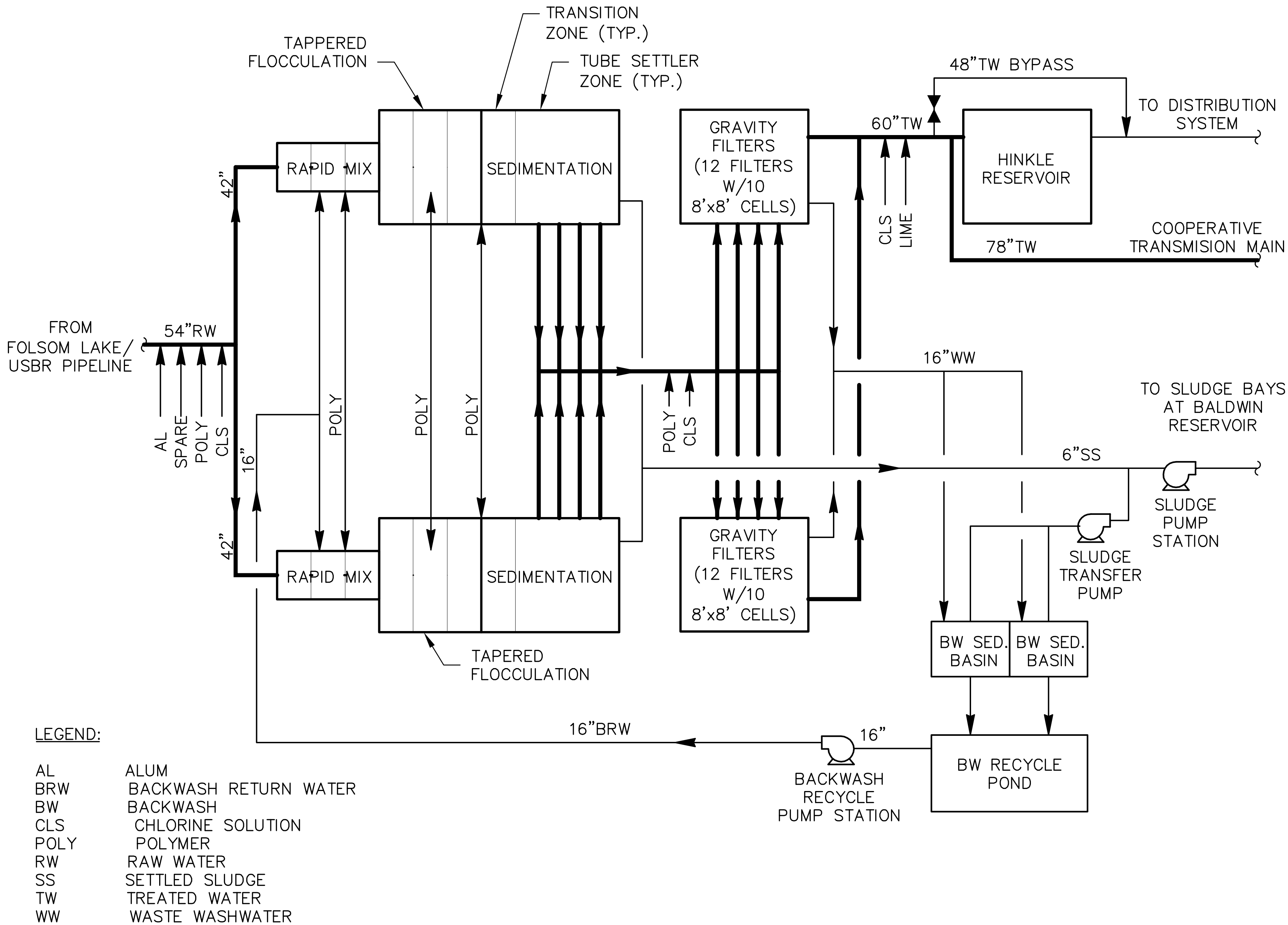

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FIGURE 6-1
EXISTING WTP PROCESS SCHEMATIC

Table 6-2
Water Treatment Plant
Original Design Criteria and Estimated Current Capacity

Description	Original Design Criteria	Capacity[a] as Conventional WTP	Capacity[a] as Direct Filtration WTP
PLANT CAPACITY			
Average Flow Rate, mgd	60		
Maximum Flow Rate, mgd	100	60	120
RAPID MIX SYSTEM			
Combined Rapid Mix System Capacity, mgd	100	120	120
PRETREATMENT SYSTEM			
Flocculation System Capacity, mgd	100	130	130
Total Sedimentation Basin Capacity, mgd	100	60	NA
FILTERS			
Combined Filter Capacity at loading rate			
All Filters in Service, mgd	110	130	130
1 Filter Off-Line each Filter Basin, mgd	100	120	120
Filter Loading Rate, gpm/ft^2	5.0	6.0	6.0
BACKWASH WATER RECOVERY SYSTEM			
Filter Backwash Recovery System Capacity, mgd	4	1.6	1.6
Filter Backwash Recovery System Capacity, % of 100 mgd Flow Rate[b]	4	1.6	1.6

(a) Current Capacity based on criteria developed by AWWA, USEPA and DHS.
(b) Typical design capacity should be approximately 4 percent

6.2.1 Rapid Mix System

The existing rapid mix system includes a high-energy mixing chamber ("Instantaneous Blending Chamber"), which provides a 1,000 sec^{-1} mixing intensity, followed by two subsequent two-stage lower energy "rapid mix chambers," which provide 300 sec^{-1} mixing intensity. Propeller type flash mixers are used in each chamber. The primary metal-salt coagulant, alum, is dispersed into the raw water in the high-energy mixing chamber. A non-ionic polymer is usually added to the chemically destabilized water (as a coagulant aid) with about two-thirds of the polymer dose added at the mid-point between the two rapid mix chambers. Under varying source water quality conditions, operations staff will shift the polymer dose point to the coagulated water distribution trough at a point located between the second rapid mix chamber and the first flocculation basin, or to the second flocculation basin between the two sets of flocculation paddles.

The existing three-stage rapid mix coagulation system should provide satisfactory coagulant mixing at flow rates at least as high as 120 mgd. Current water industry design criteria recommend that rapid mix systems' residence time be less than 30 seconds. The District's combined coagulation system residence time would be approximately 21 seconds with a Camp Number (Gt, dimensionless value) of 15,400 at a 120 mgd plant flow rate, which should be sufficient to ensure proper dispersion of the primary coagulant and coagulant aid polymer.

6.2.2 Flocculation-Sedimentation System

The flocculation-sedimentation system includes three-stage tapered flocculation followed by a transition zone and high-rate sedimentation-clarification with tube settlers. Flocculation takes place with a three-stage paddle system with decreasing energy that utilizes a VFD unit to control revolutions per minute (rpm). The decreased energy effect is achieved by reducing the number of blades per paddle assembly in each successive zone since the rpm in each zone is identical. Each zone in each basin is equipped with eight flocculator paddle assemblies mounted on four steel, chain-driven shafts that are mounted horizontally and run parallel with the direction of flow. Because each shaft runs through the three zones, the rotational speed of the flocculators is constant from zone to zone. The dimensions of the flocculation portion of each basin are 86 feet by 80 feet with an average depth of approximately 13 feet.

The three-stage tapered flocculation basins provide approximately 20 minutes of flocculation time and a Gt of 63,000 at a 50-mgd flow rate in each train. The recommended hydraulic detention time in tapered flocculation basins at water temperatures above 0.5°C and below 10°C is 15 minutes. Therefore, the two existing three-stage tapered flocculation trains would permit operating the plant at flow rates as high as 130 mgd, since raw water temperatures are normally above 10° C.

Sedimentation takes place in two phases. The first is a pure setting zone or the “transition” zone before tube settling. The dimension of the transition zone of each basin is 40.9 feet by 80 feet with an average depth of approximately 16.2 feet. The theoretical detention time in the transition zone is approximately 11 minutes at 100 mgd.

Settling continues in sedimentation basins that include tube settlers. Each sedimentation basin is 176.3 feet by 80 feet with an average depth of 12.9 feet. The sedimentation basins are sloped and begin at a depth of 15.4 fee and end at a depth of 8.3 feet. The theoretical detention time in the sedimentation basins is approximately 36 minutes at 100 mgd.

The original design criteria indicate that the sedimentation basins' settling zone surface loading rate is about 2.5 gallons per minute per square foot (gpm/ft^2) when the combined flow rate through both flocculation-sedimentation trains is 100 mgd. The surface loading rate that is currently recommended by EPA for sedimentation basins that are between 12 and 14 feet deep with tube settlers, without a softening process, is at or below 1.5 gpm/ft^2. Based on a 1.5 gpm/ft^2 surface loading rate for a conventional sedimentation-clarification pretreatment facility, the combined capacity of the two existing sedimentation basin is approximately 60 mgd. Discussions with plant staff indicate that sedimentation basin performance tends to deteriorate when the flow rate in either flocculation-sedimentation train exceeds 30 mgd. Therefore, based on current water industry design standards and operator experience, the combined capacity of the two existing flocculation-sedimentation trains is about 60 mgd.

The launders in the sedimentation basin were modified by adding one-inch diameter (approximately) holes near the bottom of the launder and, in essence, not using the v-notch design. Total weir length is 5,120 feet, which corresponds to an overflow rate of 19,531 gpd/ft. at 100 mgd. The Ten State Standards (standards adopted by the Mississippi Valley States and used as a common reference for WTP design throughout the United States) recommend a maximum overflow rate of 20,000 gpd/ft. After overflowing into launders, the water is sent down a central channel between each of the basins and to the filters.

6.2.3 Filtration

Filtration is accomplished through two modular filter basins. Each filter basin consists of 12 filters with ten 8-foot by 8-foot filter cells. Media consists of anthracite coal, fine sand, and course sand. Ten filter cells constitute one filter. The cells within a filter share the same under drain effluent pipe. Therefore, one basin has 12 filters and 12 corresponding effluent pipes. Each effluent pipe is equipped with a rate-of-flow control valve. These enable each filter to act independently of each other and allow the flow through the filter basins to be equally distributed. The control valves for the filters which experience relatively high head losses remain wide open, whereas valves for the filters with low head losses (i.e., recently backwashed filters) automatically throttle to restrict flow, resulting in a constant filter rate throughout the entire basin.

The original design criteria for the filters indicate that the automatic backwash filters were designed to operate at surface loading rates as high as 5 gpm/ft^2. When the WTP was designed, filter capacity was based on the maximum design filter loading rate with all filters in service. A 5 gpm/ft^2 surface loading rate would permit producing as much as a 110 mgd when all 24 filters are in service. The DHS currently permits dual media filters to operate at surface loading rates as high as 6 gpm/ft^2. However, DHS also requires that combined filter capacity be based on plant operations with one filter off-line for backwashing or maintenance. In addition, DHS recommends that combined filter capacity for plants with more than 12 filters be based on operations with two filters off-line for backwashing or maintenance. Based on current DHS filter operation criteria, the 24 existing dual-media filters should permit producing as much as 120 mgd of filtered water with one filter in each basin off-line.

Backwashing a modular filter basin is performed by a small structure that is mounted on a moving bridge. The structure is equipped with a 15 hp turbine pump, stainless steel surface wash injectors, and a suction hood. The backwash sequence begins with the backwash structure positioning over and lowering the suction hood onto the first filter cell of the first filter. At the same time, the rate controller valve for the filter closes. Once a mechanical seal between the cell and the hood is established, the suction pump is activated, causing water to be drawn through the remaining nine cells of the filter, into the common underdrain, and up through the cell being backwashed. Surface wash injectors are lowered into the expanded media during the backwash in order to reduce the formation of mud balls. The waste backwash water is channeled and then piped to a settling basin.

When the backwash of the first cell is completed, the suction pump is turned off, and the media is allowed to settle back into place. The hood assembly is then moved to the next cell in the same zone. As subsequent cells are backwashed, previously washed cells are filtered-to-waste by providing water for the current backwash. The tenth cell is the only cell that is returned to service prior to being reconditioned.

After an entire filter has been backwashed, the rate controller valve for that filter opens, and the valve for the next filter to be washed closes to prepare for the washing of its cells. This process continues until all cells in all filters of each basin have been washed. The backwash process takes approximately 12 hours per basin.

6.2.4 Disinfection

With the WTP classified as a conventional filtration plant, the plant is entitled to receive 2.5-log *Giardia* removal credit and 2-log virus removal credit (reference Section 4.3.1 Surface Water Treatment Rule). Therefore, the minimum disinfection requirement is 0.5-log of *Giardia* and 2-log of viruses.

The DHS considers the existing WTP to be capable of operating as a full conventional plant at plant flow rates below 100 mgd. The DHS considers the existing WTP to operate as a direct filtration process plant at flow rates above 100 mgd. When the plant is classified as having a direct filtration process, the WTP would receive 2.0-log *Giardia* removal credit and 1.0-log of virus removal credit. In this case, the disinfection process would be required to provide 1.0-log *Giardia* and 3.0-log virus inactivation, respectively.

According to the USEPA SWTR Guidance Manual, the effective contact time T_{10} is the time for 10 percent of the tracer chemical added at the influent end to appear at the effluent end. Tracer studies performed by District staff and reviewed by DHS indicate that the T_{10} to hydraulic detention time (HDT) ratio is nearly 0.5 to 1 for flow rates below 50 mgd through each flocculation-sedimentation train and nearly 0.6 to 1 for flow rates above 50 mgd in a flocculation-sedimentation train. Therefore, the effective T_{10} used for CT calculation is about 50 percent of the hydraulic detention time for flow rates up to 50 mgd per flocculation-sedimentation train and about 60 percent of the HDT for flow rates above 50 mgd.

Based on the dosage rates necessary to meet the CT requirements for both conventional and direct filtration operations, the existing chlorine feed system has adequate capacity to serve the WTP to capacities beyond 120 mgd.

Prior to the implementation of the California SWTR, Title 22, the District often operated the WTP in an in-line filtration mode with low chemical usage, long filter runs, and very low finished water turbidity. During high-demand summer months, in-line filtration (coagulation followed by filtration) may provide/permit the highest turbidity removal. The DHS currently requires using the flocculation and sedimentation basins to provide disinfection contact time since some treated water by-passes Hinkle Reservoir. The pretreatment bypass connection that permits in-line filtration operation is currently only operated under emergencies when basin maintenance must be performed.

In-line filtration is not an acceptable filtration technology in California and would require a petition with supporting filter performance data to the DHS Surface Water Treatment Rule Committee (discussed in Chapter 7). Based on discussions with DHS, in-line filtration could be considered after the Hinkle Reservoir bypass is eliminated (discussed in Chapter 8). As it exists today, the WTP cannot meet the disinfection requirements for in-line filtration.

6.2.5 Backwash Water Recovery System

Backwash water flows down two troughs located on the side of the filters to a settling tank, or spent filter backwash water recovery (return water) system. The return water system consists of two relatively shallow waste filter backwash water recovery treatment basins with tube settlers. Settled water flows though launders to a recovery pond where water is further settled prior to being pumped back to the flocculation basins distribution channel. The return water pump station consists of a 1,400-gpm pump and two 800-gpm pumps.

The two spent filter backwash water recovery basins were designed to operate at surface loading rates as high as 2.5 gpm/ft^2. However, the maximum recommended surface loading rate for shallow basins such as the existing backwash water recovery basins, according to USEPA standards, is 1.0 gpm/ft^2. Therefore, the capacity of the existing spent backwash water reclamation system is considered to be no more than 1.6 mgd. Operating at rates higher than this results in return water with excess turbidity.

The return water system capacity should be capable of handling the combined waste backwash water flow rate produced during two simultaneous filter backwashes. However, based on plant staff experience, the capacity of the return water pretreatment system is not adequate to remove much of the solids present in the spent washwater and would not be adequate to handle waste filter backwash water from two concurrent filter backwashes.

Although the return water turbidity is not monitored, operating experience indicates that the return water turbidity is well above desired levels. In addition, the return water treatment system is, and has been, the most problem-prone and maintenance-intensive plant system. To date, the generally low source water turbidity coupled with extensive plant operator experience and skill have permitted the WTP to comply with existing filtered water turbidity requirements.

6.2.6 Solids Handling

Solids removed during backwash water treatment and recovery, along with sludge withdrawn from the sedimentation basins, are currently pumped offsite across Auburn-Folsom Road to the District's sludge drying facilities at Baldwin Reservoir. Once dried, the sludge is removed from this location and utilized as a soil amendment for agricultural uses. Space on the site is limited, and it has been a labor intensive operation to constantly spread and move around sludge to handle production requirements. The District already considers the solids handling facilities to be operating "beyond capacity." The existing solids handling facilities cannot be reasonably expanded because of the limited site space to adequately process the combined sludge flow.

6.2.7 Chemical Systems

Chemical storage and feed systems at the WTP include chlorine (gas), alum, lime, bulk polymer (cationic), and batch polymer (non-ionic or anionic) systems. Based on a review of chemical demands, chemical feed capacity, and storage facilities, the existing chemical feed systems appear adequate for the WTP for a plant capacity of 120 mgd. A brief description of each system is provided below.

Chlorine System. Chlorine is added to the raw water to oxidize and begin disinfecting the raw water prior to adding the primary coagulant. The existing chlorine system includes a total of 28 trunnions; 20 trunnions for storing one-ton chlorine containers, four trunnions on scales, and four adjacent trunnions. The chlorine gas feed facilities include four chlorinators. Three chlorinators are normally set to feed up to 2,000 pounds of chlorine per chlorinator in the summer and are modified to feed up to 1,000 pounds of chlorine per chlorinator in the winter. The fourth chlorinator is set up to feed up to 500 pounds of chlorine in the summer and up to 250 pounds of chlorine in the winter. The chlorine solution from the three large chlorinators can be added to the source (raw) water ahead of the two rapid mix units at a maximum chlorine dose as high as 6 mg/L at a plant flow rate of 120 mgd. The chlorine solution from the fourth chlorinator is added to the filtered water ahead of Hinkle Reservoir to increase the chlorine residual by up to 0.5 mg/L at a plant flow rate of 120 mgd.

There are usually up to four active chlorine one-ton containers manifolded together to provide the chlorine gas supply for the chlorinators. Two of the one-ton containers are installed on scales to monitor weight, and two are positioned on adjacent trunnions. There are two sets of the four groups of one-ton containers: one set of four one-ton containers is in use ("active" status), and the second set of one-ton containers is in "stand-by" status. The remaining 20 trunnions are used to safely store full and empty one-ton chlorine containers. For design purposes, it is assumed that normally up to 400 pounds of chlorine can be withdrawn from each one-ton container per day. However, discussions with District staff and information provided by Pioneer Chemical Company indicate that up to about 500 and 600 pounds can be withdrawn from each one-ton container when the ambient air temperature is at least 80 and 100° F, respectively.

Aluminum Sulfate (Alum) System. Aluminum sulfate (alum) is added to the oxidized raw water as the primary coagulant. The existing alum system includes two 20,000-gallon capacity alum storage tanks and three chemical (alum) metering pumps. Each of the three alum metering pumps has capacity to feed up to 128 gallons per hour (gph) to the oxidized source water. Each alum metering pump's capacity is equivalent to feeding about 16,500 pounds of alum per day. One of the three alum metering pumps is used to add alum to the raw water at the rapid mix basin in the south coagulation-flocculation-sedimentation train, and a second alum pump is used to add alum to the oxidized raw water at the rapid mix basin in the north coagulation-flocculation-sedimentation train. One of the three alum metering pumps is normally placed in stand-by status. Each of the two operational alum metering pumps is capable of adding a 26.5 mg/L alum dose to the oxidized source water in the rapid mix basin at a maximum flow rate 0f 60 mgd through each coagulation-flocculation-sedimentation train.

Lime System. Slaked lime is added to the filtered water to increase the pH in order to stabilize the water and reduce its corrosivity. The existing lime storage and feed system includes one 99 ton capacity lime storage silo and one 750 pound per hour lime slaker manufactured by Chemco of Monongahela, Pennsylvania. The lime feeder is currently set up to slake up to 500 pounds of lime per hour. The existing lime slaker capacity permits adding up to 12 mg/L of lime to the filtered water at a maximum 120 mgd plant flow rate. The slaker capacity can be increased to 750 pounds per hour by replacing the current lime feeder gears with the original gears to permit adding up to 12 mg/L of lime at plant flow rates as high as 180 mgd.

Cationic Polymer System. The cationic polymer system was designed to improve the settling characteristics of the solids contained in the spent filter backwash water handling system. The cationic polymer system has not been used for about ten years. There are two cationic polymer metering pumps. One metering pump has capacity to feed up to 4 gph, and one metering pump has capacity to feed up to 11 gph.

Batch (Non-Ionic) Polymer System. Non-ionic polymer can be added to the coagulated water as a filter aid and to the settled water as a filter aid. Stock solutions of non-ionic polymer solution are prepared daily in 400-gallon batches. Between 2 and 8 gallons of neat polymer are blended with approximately 400 gallons of water in the batch tank every 24 hours. The amount of polymer blended with the 400 gallons of solution dilution water in the batch tank is adjusted to provide the required non-ionic polymer dose. Typical dosages range from 0.1 mg/L to 0.2 mg/L. The 400-gallon batches of polymer solution are prepared in the mixing tank and then transferred to the polymer solution feed tank. One of two 415 gph capacity polymer metering pumps is used to feed non-ionic polymer as a coagulant aid to the oxidized and coagulated water at one of two locations: 1) between the flash mixer and the first rapid mixer zone, or 2) alternatively to the middle of the second flocculation basin, and/or as a filter aid to one of two locations in the settled water channel.

6.3 Hydraulic Capacity Evaluation

As previously stated, the initial phase of the WTP was designed in 1977 for a capacity of 100 mgd. That phase anticipated an addition of filters that were not a part of the original plant design. The subsequent filter addition project had a design capacity of 100 mgd as well, although by current DHS design standards the filters are considered rated to 120 mgd. However, based on our discussions with the WTP staff, the WTP cannot be operated for sustained periods above about 110 mgd, due to hydraulic limitations through the plant.

The existing WTP was evaluated to determine what hydraulic bottlenecks might exist and identify improvements that would increase hydraulic capacity. The evaluation included reviewing previous analyses of the WTP hydraulics and conducting additional hydraulic analyses to develop a hydraulic profile. The hydraulic profile of the WTP was developed utilizing an in-house computer model tool called Hypro. The underlying calculations are performed as an Excel spreadsheet.

6.3.1 WTP Hydraulic Profile

The results of our analysis of the existing WTP at a flow of 120 mgd are shown on the hydraulic profile on Figure 6-2. A printout of the model can be found in Appendix 6-1.

The WTP hydraulic profile depends, first of all, on the water surface elevation over the filters. According to WTP staff, the water level over the filters is automatically controlled by the filter controls at the lowest practical elevation. Starting with that assumption, the one factor (besides the flow) that will affect the level over the filters (as currently constructed) is the relative condition of the filters (i.e. how clean they are). The filters are divided into cells, which are being continuously backwashed. The longer the period of time that the filters are operated at a high sustained rate, the higher the headloss through the filters and the higher the water surface over the filters. Therefore, the hydraulic model always starts with an assumed elevation for the water level at the filters. (Note: This assumption needs to be verified based on new data obtained in August 2001. Refer to the last paragraph in this section.)

Water passes to the filters from the settled water channel. The WTP has an emergency overflow weir (EOW) that is hydraulically connected to the settled water channel (which receives the flow from the sedimentation basin effluent troughs). The EOW has an elevation reported to be at 420.20 (based on the WTP datum). When the WTP flow is “too high,” flow automatically discharges over the EOW. Discharge over the EOW is non-catastrophic, but this is not a desired condition. The WTP staff has improved erosion protection for the area where the EOW spills to a natural drainage channel.

In order to pass the desired flow of 120 mgd through the filters with no overflow at the EOW, the calculated maximum water surface level over the filters is 419.10 (agreeing with previous WTP analyses).

When the water surface is at elevation 420.20 at the settled water channel, the sedimentation basin effluent troughs are essentially free-flowing (i.e. there is no significant back-up of the flow into the troughs), and there is a considerable drop over the v-notch weirs. This means that any problems at the head end of the plant (from the sedimentation basins back to the rapid mix basins) are not the result of too much depth over the filters.

The only non-standard hydraulic elements in the WTP are the sedimentation basin effluent troughs. These were originally designed as internally hung launders with v-notches, a relatively standard hydraulic element. The effluent troughs had an occasional bottom hole, presumably to allow drainage when a basin was dewatered. However, due to oscillation problems with the troughs as water discharged over the v-notches, numerous holes were added to the bottom sides of the troughs. These holes act as orifices, with practically all flow entering the troughs through the holes until the plant flow rate exceeds about 100 mgd. According to information obtained from WTP staff, there are a total of 4,688 holes (each 1-inch diameter), or 2,344 for each of the two main process trains.

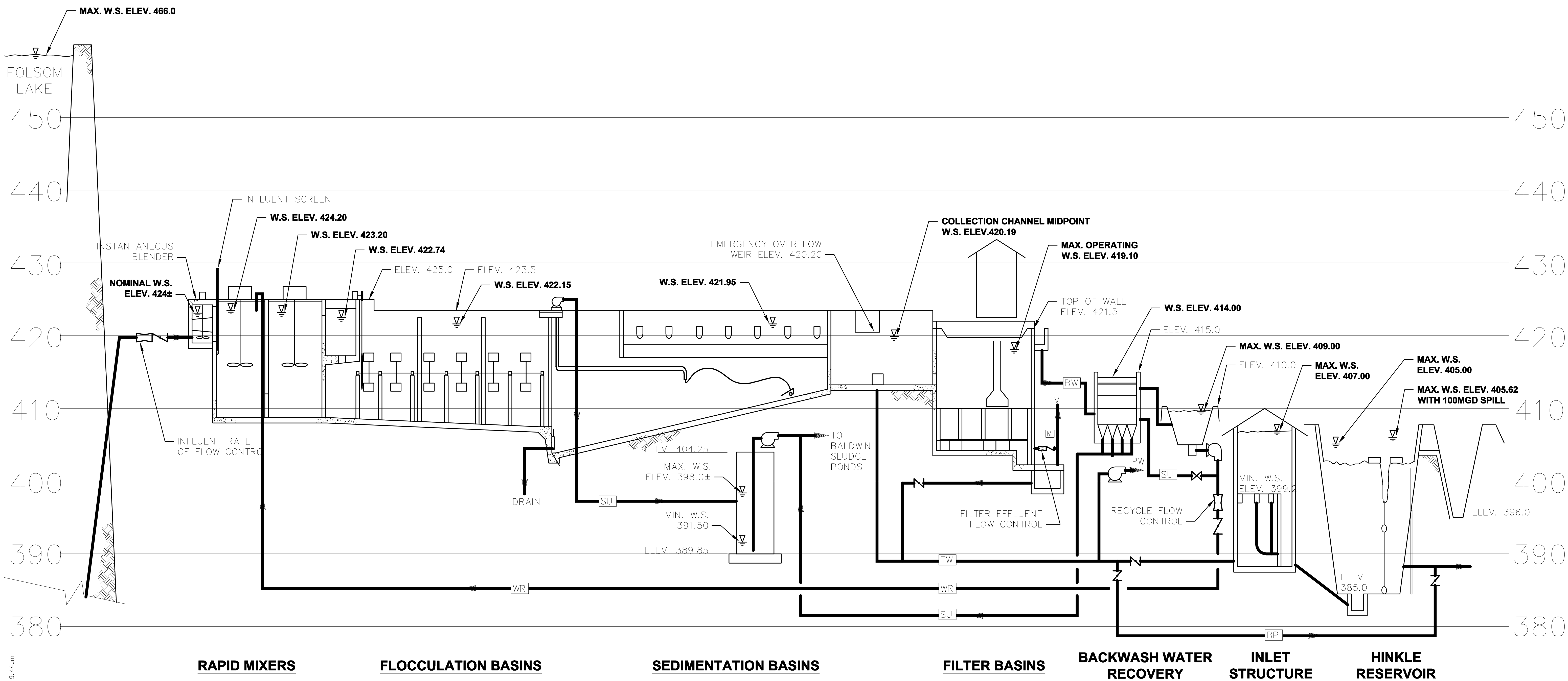

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FIGURE 6-2
EXISTING WTP HYDRAULIC PROFILE AT 120 MGD

The significance of the holes at higher plant flow rates is that they create a variable flow split, with some flow entering the effluent troughs through the v-notches (as originally designed) and with the rest of the flow entering through the holes. Our hydraulic model permits an accurate calculation of the flow split.

The flow split between the v-notch weirs and the holes in the sedimentation basin effluent troughs with a WTP flow rate of 120 mgd calculates to be 17 percent versus 83 percent. That means that most of the flow is leaving the sedimentation basins through the holes, but some flow is still going over the v-notch weirs. This is a desirable condition, because the v-notch weirs are intended to maintain even distribution of flows across the sedimentation basins.

There are a series of headlosses from the WTP influent to the sedimentation basins. There are thirty-two 12-inch by 16-inch rectangular openings in the flocculation basin distribution troughs. These are responsible for approximately 0.59 feet of headloss. Other significant headlosses include 0.47 foot for the sluice gates leading to the flocculation basin distribution troughs and 1.0 foot for the rectangular opening between rapid mix zone 2 and rapid mix zone 1 (one opening for each process train). The rectangular openings were originally 48 inches by 49 inches. The WTP correctly identified the openings as a major bottleneck and expanded the openings to approximately 65 inches by 49 inches.

While none of these headlosses is especially great, the cumulative effect raises the water level at the rapid mix zone 1 (a mixing box) to where it sloshes out onto the deck. The turbulence in a mixing box with a mechanical mixer of this size is such that at least a 1.5-foot freeboard is required to prevent sloshing from reaching the deck. A 2.0-foot freeboard would be desirable. We calculate a freeboard of 0.8 feet, which is not adequate. We observed that, at high flows, sloshing does occur, and some water ends up on the deck and overflows the structure.

New information on the WTP hydraulics became available subsequent to completion of the Final Draft of this report. During late August 2001, the WTP was able to flow more than 120 mgd through the filters when they were relatively clean and the Hinkle Reservoir was below approximately elevation 394 (approximately half full). Above this reservoir elevation capacity was reduced. This indicates excessive head loss between the filters and Hinkle Reservoir may be hydraulically limiting WTP capacity.

The elements between the filters and Hinkle Reservoir consist of the filter media and underdrains, filter control valves and manifold piping, a 60-inch diameter treated water pipeline, a reservoir inlet structure with a control weir, two 48-inch diameter reservoir inlet pipes, and an inlet box with bar screen. At a 120-mgd flow rate, velocity through the 60-inch pipe is approximately 9.46 fps and it is 7.36 fps through the 48-inch pipes. These are reasonable velocities at peak flow and should not create more than a few feet of headloss in these short pipe sections.

This Master Plan recommends additional field observations and testing be conducted to determine what treated water elements may be restricting flow so that recommendations for modifications or additional improvements can be developed.

6.3.2 Short-Term Hydraulic Capacity Improvements

To identify short-term improvements to the existing WTP to increase plant hydraulic capacity to 120 mgd, the hydraulic model was tested with several alternative hydraulic modifications. The recommended improvements are discussed below.

6.3.2.1 Emergency Overflow Weir

It appears that the single best way of preventing overflow at the EOW under high flow conditions is to raise the weir elevation, currently at 420.20. Based on our testing of the hydraulic profile model, raising the EOW to 421.00 would allow for an additional 1.0 foot of filter head without overflow. Under that scenario, the sedimentation basin water level would not be significantly affected. The water level would be higher in the sedimentation basin effluent troughs, but the v-notch weirs would not be submerged to any degree.

6.3.2.2 Sedimentation Basin Effluent Troughs

A majority of the flow into the sedimentation basin effluent troughs currently passes through the 1-inch holes instead of passing over the v-notches as originally designed. These holes mitigated some of the trough oscillation problem that occurred when all flow passed over the v-notch weirs. They have also played an important role in increasing the hydraulic efficiency of the WTP. As peak flow through the plant increases, more flow will pass over the weirs (up to 17 percent at 120 mgd). This may cause the oscillation problem to return. Recommended improvements to address this issue are:

1. Remove the “blanked” off sections of the launders to expose additional v-notch weirs. This will double the number of v-notches, slightly reduce the water surface elevation in the sedimentation basin and help better distribute the flow into the launder.

2. Stiffen the launders against oscillation with horizontal bracing or additional supports.

3. Add additional holes in the launders to prevent flow over the weirs. However, the number of holes should not be increased more than 25 percent because of the possible adverse impact on sedimentation basin performance. The headloss that is incurred through the holes helps to insure flow distribution across the sedimentation basins and into the effluent troughs.

6.3.2.3 Rapid Mix Boxes

The sloshing and overflow that occurs at the rapid mix boxes at flows of 120 mgd or less can be reduced with two improvements. First, by increasing the size of the rectangular openings between rapid mix zone 1 and zone 2 (two openings, one per treatment train) and secondly, by increasing the size of the 32 inlet holes in the flocculation basin distribution troughs (or adding additional holes).

It does not appear practical to consider increasing the size of the sluice gates between the rapid mix boxes and the flocculation basin distribution troughs. These sluice gates are 72-inches wide by 48-inches high, and the cost of replacing them with larger gates would be very high. If the headloss through the rapid mix boxes cannot be reduced sufficiently with the improvements recommended above, it may be necessary to replace these gates.

The 32 existing holes in the flocculation basin distribution troughs are 12-inch by 16-inch rectangular openings. If these were enlarged to 16-inch by 16-inch and rounded on the inlet side (or additional holes were added of equivalent open area), they would still provide effective inlet flow distribution to the flocculation basins. This modification would reduce the headloss at peak flow (120 mgd) from 0.59 feet to 0.33 feet.

The openings between rapid mix zone 1 and zone 2 have already been expanded once. The feasibility of expanding the openings again will require a structural evaluation. Also, the wall between the zones serves a purpose: having two distinct mixing zones, each with its own mixer. This reduces short-circuiting of flow through the mixing zones. The wall insulates each mechanical mixer from the turbulence created by the adjacent mixer. Enlarging the opening further should be reviewed with the mixer manufacturers to determine if the mixers would be adversely affected.

However, assuming that the openings could be widened from 65-inch to 77-inch and rounded on the inlet side, then the headloss could be reduced from an existing 1.0 foot down to 0.71 feet.

6.4 Short-Term Process Modification Alternatives and Capacity

Based on the original plant design criteria and on a need to increase plant capacity to at least 120 mgd as soon as possible, the major plant processes were evaluated in order to determine what could be done to modify existing facilities within the next six months to provide the desired plant capacity. Alternative process modifications were identified and evaluated, where necessary, to permit improving plant performance and/or to increase plant capacity to 120 mgd.

6.4.1 Rapid Mix

The existing three-stage rapid mix system should not require modifications in order to provide satisfactory service at plant flow rates as high as 120 mgd. However, hydraulic improvements are necessary to permit the higher flow rate as discussed in Section 6.3.

6.4.2 Flocculation-Sedimentation

Based on current water industry design criteria for flocculation and sedimentation systems, the combined capacity of the two existing three-stage flocculation basins is approximately 130 mgd when the source water temperature is above 10° C. The two existing sedimentation basins' performance would not be (and has not been) satisfactory at plant flow rates above 60 mgd. Although methods of increasing the sedimentation basins' capacity should be evaluated for the long-term, the only practical approach to increasing plant capacity to 120 mgd in the short-term is to operate the plant in a direct filtration mode. This eliminates the need for a sedimentation step, but increases disinfection requirements.

6.4.3 Filtration

Although the original filter design capacity (100 mgd) was based on all 24 filters operating with a 5 gpm/ft^2 surface loading rate, the DHS permits operating dual media filters with surface loading rates as high as 6 gpm/ft^2. The DHS has developed additional filter design criteria that impact filter capacity. The DHS filter reliability/redundancy design criteria include determining plant capacity based on at least one filter being off-line for backwashes or maintenance. Based on the DHS current filter design and plant capacity criteria, the WTP capacity would be about 120 mgd with one filter in each filter basin off-line and the remaining 22 filters operating at a 6 gpm/ft^2 surface loading rate. Therefore, the two existing filter basins and 24 filter units should not require modification in order to operate the plant at 120 mgd.

6.4.4 Disinfection

The existing chlorine feed system has adequate capacity for disinfection purposes with the WTP operating in either a conventional or direct filtration mode. When operating in a direct filtration mode, the chlorine feed rate at the head of the plant will need to be increased based on calculated CT requirements.

6.4.5 Solids Handling and Backwash Water Recovery System

The existing spent filter backwash system is inadequate and is a major operational problem for plant staff. At times, the spent filter backwash water returned to the front of the plant is the major source of particles. However, there is no recommended short-term “fix” for the existing system. The existing spent filter backwash water recovery system should be replaced with a more reliable and robust treatment process at the earliest opportunity in order to permit the WTP to remain in compliance with existing, new, and anticipated regulations and guidelines. In the short-term, continued utilization of plant operator experience, skill, and ingenuity will need to be relied on to manage the backwash water recovery system.

6.4.6 Chemical Systems

The existing chemical feed systems have been maintained in good operating condition since the plant was originally constructed.

The chlorine system was converted from a liquid/evaporation system to the current gas system. The original powdered activated carbon system was never utilized, so it was removed by plant staff, and the carbon feed room was converted to parts storage. A new lime slaker/feeder replaced the original split system. The remaining chemical storage and feed systems have largely remained in place. A third alum metering pump was recently added by plant staff.

Based on plant operating data for 1993 through July 1999 on average and maximum chemical doses, the existing chlorine, alum, non-ionic polymer, and lime storage and feed systems provide adequate capacity for plant flow rates as high as 150 mgd. For plant flow rates above 150 mgd, additional storage and feed capacity will be required. In addition, a thorough code and safety review should be performed to identify modifications necessary to bring the existing chemical storage and feed systems up to current code requirements. The Risk Management Plan (USEPA RMP) and the California Accidental Release Program (Cal/ARP) recently completed by the District documented the need for enclosing the chlorine storage area to contain an uncontrolled release of chlorine gas. A chemical scrubber system was also recommended to neutralize a release of gas.

Chapter 7: Future Plant Capacity and Long-Term Improvements

7.1 Introduction

For the year 2030 planning horizon, without consideration of conjunctive use, a maximum WTP capacity of 150 mgd is required to meet spring-summer-fall water demands of the existing District wholesale and retail service area, and 75 mgd is required for winter-time demands. This assumes full use of existing water rights and contracts. This Master Plan also develops strategies for maximizing the capacity of the WTP at the existing site, to an upper limit of 240 mgd for spring-summer-fall demands and 120 mgd for winter-time demands, to help meet other potential regional water demands. This Master Plan does not evaluate potential reductions in WTP capacity due to conjunctive use programs.

This chapter describes the WTP expansion scenarios for the two capacity requirements and presents results of a screening of long-term treatment process alternatives. The screening was conducted in two phases: 1) a preliminary non-economic, qualitative evaluation of treatment alternatives to identify feasible alternatives and 2) a quantitative matrix evaluation of the remaining alternatives. Finally, a discussion of the recommended long-term improvements is provided for two expansion scenarios: a long-term maximum WTP capacity of 150 mgd and a long-term maximum WTP capacity of 240 mgd. Table 7-1 presents a summarized narrative of the recommended plant improvements. Process improvements are summarized later in this chapter in Table 7-3. Site plans for the alternatives are also provided.

7.2 Water Treatment Plant Expansion Scenarios

The objective of this Master Plan is to develop alternatives to accommodate the treatment and transmission of high quality potable water for a peak day treatment capacity of a minimum 150 mgd to a maximum 240 mgd by the year 2030. The two expansion scenarios are referred to herein as Long-Term 75/150 mgd and Long-Term 120/240 mgd.

7.2.1 Long-Term 75/150 mgd

The LT 75/150 maximum WTP capacity alternative assumes that the District would limit expansion to full use of existing water rights and contracts and that the future demand pattern will be similar to the existing one. As discussed in Chapter 6, this demand pattern would consist of a winter-time demand of 75 mgd that could be treated with a conventional filtration treatment process and a spring-summer-fall demand of 150 mgd that could be treated with a direct filtration treatment process.

The LT 75/150 expansion implementation could proceed within the District's available property at the existing plant. Hydraulic improvements (including new pipelines and channels) would be necessary within and between the various process units. The expansion would require modifications to the existing flocculation-sedimentation basins, a new filter basin, and new backwash and solids handling facilities along with other identified improvements.

Table 7-1
Narrative of Recommended Improvements

Process	LT 75/150 Recommended Improvement	LT 120/240 Recommended Improvement
Coagulation	Replace existing mechanical turbine mixers with more efficient pump injection type mixing system for dispersion of primary coagulant. Maintain existing coagulant aid (polymer) feed points within the rapid mix flocculation zone and settled water channel to aid in optimizing floc formation.	Construct a third rapid mix, flocculation and sedimentation train on the north side of the two existing rapid mix, flocculation, and sedimentation basins.
Flocculation	Replace the existing flocculation basin horizontal turbines with new horizontal paddle flocculators. The horizontal paddle flocculators should be designed to provide higher mixing energies to form small filterable pin floc during the summer when source water turbidity is low and conventional filtration is not required. Install redwood walls between each of the five parallel flocculation trains to improve flocculation performance. Install a perforated flow distribution wall between each flocculation basin and the adjacent sedimentation basin similar to the existing perforated walls between existing flocculation zones 1 and 2 and zones 2 and 3.	Construct a third rapid mix, flocculation, and sedimentation train on the north side of the two existing rapid mix, flocculation, and sedimentation basins.
Sedimentation Basins	Replace existing shallow 2-foot deep tube settler/launder system with new tube settlers and launders equipment. New 4-feet deep tube settler modules could be installed in the deepest (up to 126 feet) portion of each of the two existing sedimentation basins and new 2-feet deep tube settler modules in the shallowest (minimum 50 foot) section in each existing sedimentation basin. This would increase the conventional pretreatment capacity of the existing sedimentation basins to a nominal 75 mgd. Increase launder size from 18-inch X 21-inch to 24-inch X 24-inch. Launder bracing and supports should be improved. Construct a new settled water conveyance channel on the north side of the two existing rapid mix, flocculation, and sedimentation basins. The channel would convey settled water from the north sedimentation basin. The existing basin would convey water from the south sedimentation basin. Total settled water channel capacity should be sized to provide for a	Construct a third rapid mix, flocculation, and sedimentation train on the north side of the two existing rapid mix, flocculation and sedimentation basins. The third pretreatment train sedimentation basin would have deeper (4-foot) tube settler modules to provide capacity of 60 mgd for the third basin. This would provide a total conventional filtration treatment pretreatment capacity of at least 120 mgd with all three flocculation-sedimentation basins in service. Construct a new settled water conveyance channel on the north side of the two existing rapid mix, flocculation, and sedimentation basins between the existing and new pretreatment basins. Total settled water channel capacity should be sized to provide for a hydraulic capacity of 240 mgd to accommodate initial and future conventional and direct filtration treatment capacity requirements.

Table 7-1 (cont.)
Narrative of Recommended Improvements

Process	LT 75/150 Recommended Improvement	LT 120/240 Recommended Improvement
Sedimentation Basins (cont.)	hydraulic capacity of 150 mgd to accommodate initial and future conventional and direct filtration treatment capacity requirements.	If additional conventional pretreatment capacity is desired in the future, replace the existing sedimentation basin 2-foot deep tube settler modules with 1) new 4-feet deep tube settler modules in the first (deepest) 126 feet of each of the two existing sedimentation basins and 2) new 2-feet deep tube settler modules in the last (shallowest) 50 foot section in each existing sedimentation basin. This would increase the pretreatment capacity of the existing sedimentation basins to between 40 and 50 mgd each, for a maximum total conventional pretreatment capacity of 160 mgd.
Filtration	Add a new filter basin, divided into two 30-mgd capacity "half" filter basins, with similar design and loading rate to existing design. Filter loading rate is recommended to stay at, or below, 6 gpm/sf with one filter unit in backwash and one filter unit at a reduced flow rate during filter-to-waste and "start-up mode" in each 30-mgd basin. Add an additional filter backwash unit to each existing filter basin to facilitate faster backwash sequencing during high plant flow rates and poor source water/ settled water quality events. Add filter-to-waste capability to the existing filters.	Stage the addition of two more 30-mgd capacity filter basins with similar design and loading rate to existing design. Filter loading rate is recommended to stay at, or below, 6 gpm/sf with one filter unit in backwash and one filter unit at a reduced flow rate during filter-to-waste and "start-up mode" in each 30-mgd basin. An additional "half" filter basin and filter backwash unit is required for each 30-mgd increment of filter expansion. Additional land would need to be acquired to the east of the third filter basin for additional filter basins.
Disinfection	Retain gaseous chlorine. Construct chlorine storage improvements to provide secondary containment of one-ton chlorine containers and a scrubber system.	Expand or replace disinfection system as plant flow rate increases. For expansions beyond 150 mgd, consider converting to onsite generation of sodium hypochlorite.
Future Disinfectants	In order to prepare for future regulatory requirements, reserve space for a chlorine dioxide system, ozone, or UV. These three disinfectants are identified by USEPA for inactivation of *Cryptosporidium*.	In order to prepare for future regulatory requirements, reserve space for a chlorine dioxide system, ozone, or UV light. These three disinfectants are identified by USEPA for inactivation of *Cryptosporidium*.

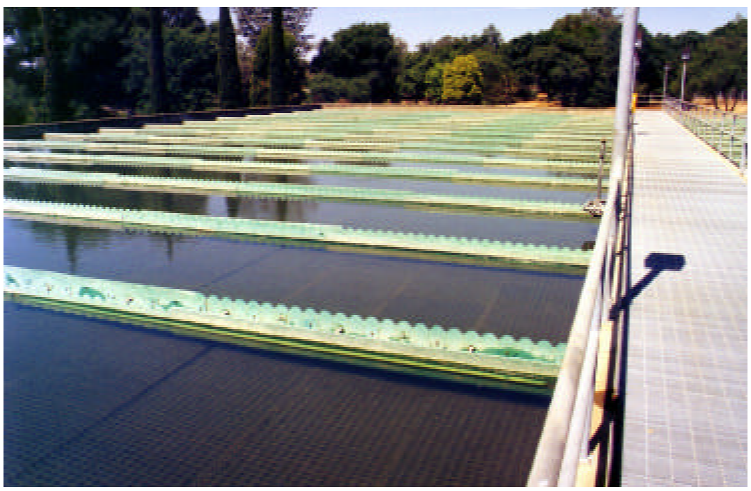
Table 7-1 (cont.)
Narrative of Recommended Improvements

Process	LT 75/150 Recommended Improvement	LT 120/240 Recommended Improvement
Backwash Water Recovery System	Replace the existing undersized system with a new system designed to handle the increased demand from the expanded WTP. A recovery/equalization basin will provide equalization and continuous (versus batch) treatment operation. Parallel modules (sedimentation with plate settlers) for solids separation will provide solids capture so that the decant stream meets the requirements of the CAP and FBR.	Construct improvements similar to those described for LT 75/150. Add an additional equalization basin and treatment module for plant flows greater than 180 mgd to provide filter backwash water treatment capacity of a minimum of 4 percent of plant flow.
Residuals Handling	Implement mechanical dewatering to address limited site space and hydraulic limitations at the existing residuals handling facility west of Auburn-Folsom Road. An approximate 4,000 square foot building located west of the sedimentation basins will be required to house the mechanical dewatering equipment. Two sludge thickeners will be required.	Construct improvements similar to those described for LT 75/150 except building size will be approximately 5,000 square feet. Add additional mechanical dewatering equipment and sludge thickener for a plant capacity expansion above 180 mgd.
Chemical Feed Systems	Retain chlorine as primary and residual disinfectant. Modify existing alum, lime, and polymer chemical feed systems to provide additional storage and/or feed points as required. Add polymer system for sludge conditioning and processing.	Implement required disinfection systems. Chlorine will be retained as residual disinfectant. Existing alum, lime, and polymer chemical feed systems will be expanded to provide additional storage, injection capacity, and/or feed points as required by the phased expansion. Add polymer system for sludge conditioning and processing.
Additional Site Improvements	Implement improvements to increase the hydraulic capacity of the existing facilities: • A parallel plant influent line to increase capacity to 150 mgd while meeting existing head conditions. • New pipeline from new filters to Hinkle Reservoir. • Expanded overflow channel capacity of 150 mgd. • Enlarged inlets/outlets to rapid mix units and pretreatment basins. Replace the existing in-plant pump station for process water. Replace the existing orifice plates on the 42-inch inlet water pipelines used for rate of flow control with magnetic flow meters	Implement improvements similar to those described for LT 75/150, including: • A parallel plant influent line to increase capacity to 240 mgd while meeting existing head conditions. • New pipeline from new filters to Hinkle Reservoir. • Expanded overflow channel capacity of 240 mgd. • Enlarged inlets/outlets to rapid mix units and pretreatment basins. Replace the existing in-plant pump station for process water. Replace the existing orifice plates on the 42-inch inlet water pipelines used for rate of flow control with magnetic flow meters

7.2.2 Long-Term 120/240 mgd

The LT 120/240 maximum WTP capacity alternative would involve the District changing its existing role to that of a regional agency. Under LT 120/240, the District would continue to deliver treated water to its existing wholesale and retail customers and would also supply treated water to customers within an expanded service area. Future water demands within this expanded service area are currently not as well defined as those for the District's existing family of users. However, the evaluations in this Master Plan assume a similar demand pattern to the existing demand pattern, with a much lower winter demand than that in the summer.

For this scenario, existing pipelines and channels within the WTP will not be adequate for the hydraulic requirements of LT 120/240. Plant modifications to provide additional hydraulic capacity would be significant, including new plant influent piping, larger channels, and piping between the pretreatment basins and filters than required for LT 75/150, and additional piping between the filters and Hinkle Reservoir. Land also would need to be acquired for expanded pretreatment facilities, and for filtration facilities for WTP capacities exceeding 180 mgd.



The sedimentation basin launders and the settled water channel are examples of existing WTP hydraulic elements that will require significant improvements.

The existing WTP configuration can accommodate modular expansion. Based on our review of the WTP and process requirements, a phased expansion approach of 30 mgd increments is recommended for LT 120/240. The first phase of expansion would be significant. A new flocculation-sedimentation basin, a new filter basin, and the construction of large "backbone" improvements such as piping and channels that would eventually accommodate the ultimate 240 mgd WTP capacity are required. Chemical storage tanks, pumps, and other mechanical equipment could be phased in to the WTP process in a logical, economical fashion.

Filters units could be constructed in smaller capacity increments than 30 mgd, but a backwash hood, piping, and electrical and instrumentation would need to be constructed during any initial phase to accommodate the ultimate sized basin. Additional common walls between filter units in each basin would be required, and redundancy and reliability would suffer during initial, smaller phases.

Expanding the WTP in a minimum of 30 mgd increments is also sensible for the 30-year planning period of this Master Plan. An expansion would be required approximately every 10 years, which would be about the minimum desired time for proper planning, design, and construction. This approach would give the District the ability to make incremental increases in capacity as future demands become more clearly defined. The proposed expansion increments are as follows:

120 mgd expanded to 150 mgd
150 mgd expanded to 180 mgd

180 mgd expanded to 210 mgd
210 mgd expanded to 240 mgd

7.3 Evaluation Approach

Kennedy/Jenks Consultants and Black & Veatch treatment specialists evaluated several alternative treatment technologies, oxidation/disinfection processes, backwash water recovery systems, and residuals handling methods to determine the recommended treatment processes for the WTP. The treatment specialists held a workshop to develop preliminary and detailed screening criteria and screen the alternatives. A subsequent workshop was held with District staff to incorporate their feedback and insight. A detailed discussion of the alternatives evaluation is presented in Appendix 7-1. The two-step evaluation results are summarized in Table 7-2.

7.3.1 Preliminary Screening of Treatment Plant Expansion Alternatives

As shown in Table 7-2, the preliminary screening evaluated eight treatment technologies, six oxidation/disinfection processes, four backwash recovery systems, and five residuals handling methods. The preliminary screening of these alternatives was based on the following criteria as further described in Appendix 7-1:

- USEPA/DHS Listed Technologies.
- Site Adaptability.
- Present and Future Regulations.
- Water Quality.
- Operations and Maintenance (O&M) Requirements.
- Reliability/Proven Technology.
- Compatibility with Existing Plant Facilities.

Cost issues were deferred to the detailed screening summarized in Section 7.3.2.

The preliminary screening of alternatives included a rating system using the following scoring classifications: excellent (satisfies all screening criteria), good (satisfies most criteria), fair (satisfies some criteria), and poor (does not satisfy criteria).

As shown in Table 7-2, four of the eight treatment technologies were determined to be suitable for additional evaluation in the detailed screening. Five of the six disinfection/oxidation processes were carried forward, as were all four backwash water recovery systems and four of the five residuals handling methods.

Table 7-2
Summary of Preliminary and Detailed Screening

	Carried Forward from Preliminary Screening	Carried Forward from Detailed Screening
Treatment Technology		
Conventional Filtration	Yes	Yes
Conventional Filtration with DAF	No	--
Direct Filtration	Yes	Yes
DE Filtration	No	--
Slow Sand Filtration	No	--
Serial Filtration	No	--
Ballasted Floc Sedimentation	Yes	Yes
Membranes (No Pretreatment)	Yes	No
Oxidation/Disinfection		
Free Chlorine	Yes	Yes
Chloramines	Yes	No
Chlorine Dioxide	Yes	Yes
Ozone	Yes	Yes
Ultraviolet Light (UV) Radiation (Disinfection Only)	Yes	Yes
Potassium Permanganate (Oxidation Only)	No	--
Backwash Water Recovery Systems		
Ballasted Floc Sedimentation	Yes	Yes
Plate Settler Sedimentation	Yes	Yes
Membrane Filtration	Yes	Yes
Roughing Filters	Yes	Yes
Residuals Handling		
Sand Beds	No	--
Belt Press	Yes	Yes
Centrifuge	Yes	Yes
Wedge Wire	Yes	No
Wedge Wire with Vacuum	Yes	No

7.3.2 Detailed Screening of Treatment Plant Expansion Alternatives

The detailed screening included a weighted evaluation of the alternatives selected for further evaluation in the preliminary screening. The evaluation criteria and weighting factors are listed below. A detailed description of the criteria is provided in Appendix 7-1.

Regulatory Impact	25%
Source Water Quality	25%
Operations	20%
Adaptability/Compatibility	20%
Costs	10%

The rating system used a scale from 1 to 10 with 10 being "excellent" and 1 being "poor." As shown in Table 7-2, the following alternatives were carried forward from the detailed screening:

Treatment Technology
Conventional Filtration Treatment Process
Direct Filtration Treatment Process
Ballasted Floc Sedimentation Pretreatment

Oxidation/Disinfection
Free Chlorine
Chlorine Dioxide
Ozone
UV

Backwash Recovery System
Ballasted Floc Sedimentation
Plate Settler Sedimentation
Membrane Filtration
Roughing Filters

Residuals Handling
Belt Press
Centrifuge

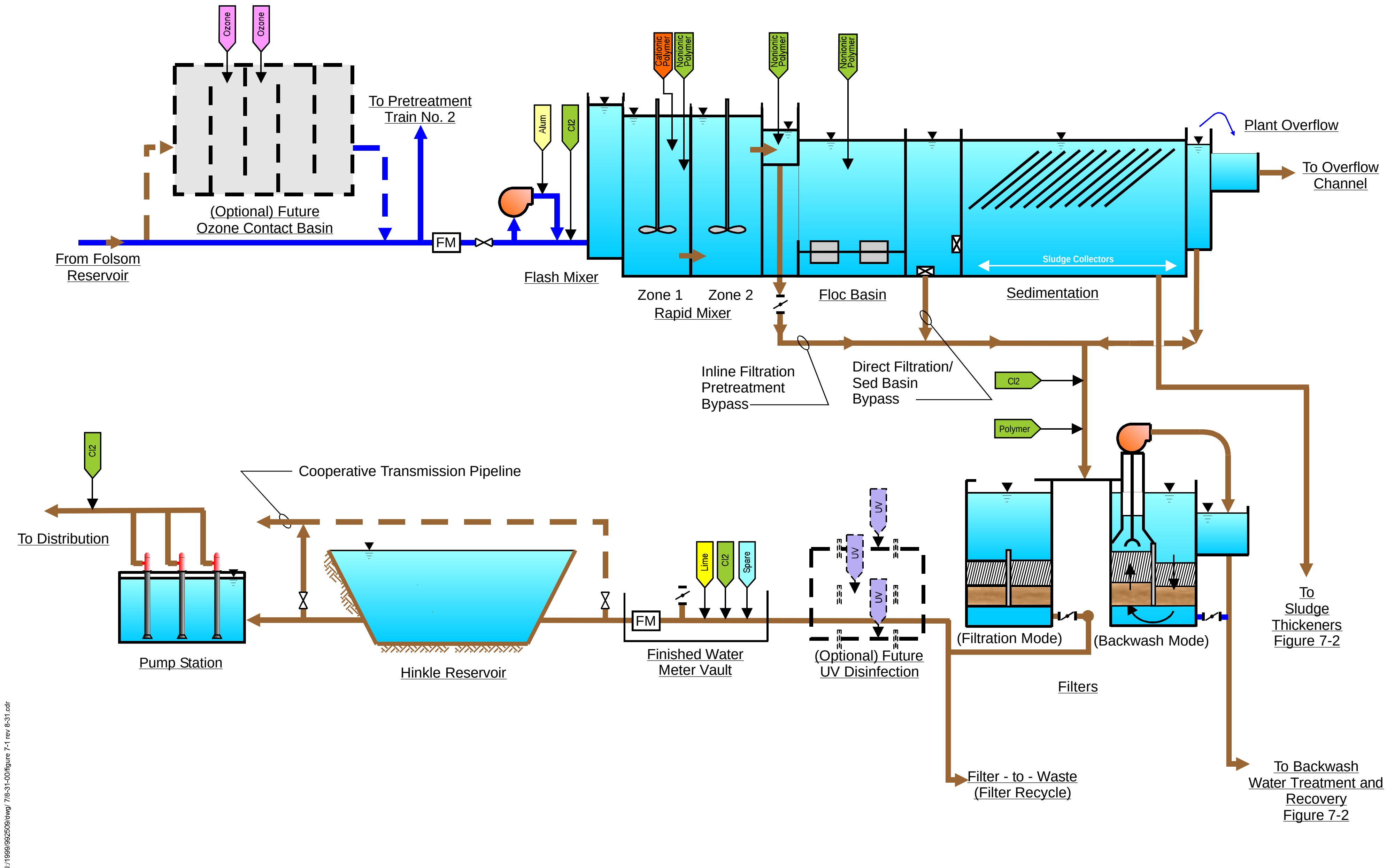
7.3.3 Recommended Treatment Process

The treatment technologies carried forward from the detailed screening were evaluated to determine the recommended treatment process and approach to expanding the existing WTP. The detailed analysis is provided in Appendix 7-1. The recommendations are summarized in the following paragraphs and depicted on the process schematics shown on Figures 7-1 and 7-2.

The treatment process recommended for future expansion(s) is similar to that of the existing facilities: conventional treatment using coagulation, flocculation, sedimentation and filtration during periods of high source water turbidity and direct filtration treatment during warmer months when higher source water quality permits. The WTP would be rated at its maximum capacity while operating in a direct (or in-line) filtration mode during spring-summer-fall, e.g., 150 mgd to up to 240 mgd. However, in a conventional treatment mode during winter time, the plant would be rated at 50 percent of its direct (or in-line) filtration capacity, e.g., 75 mgd to up to 120 mgd.

If the District's water demand pattern changes in the future, additional conventional filtration treatment capacity could be achieved by adding a third coagulation-flocculation-sedimentation train for the LT 75/150 alternative, or by making additional modifications to the existing coagulation-flocculation-sedimentation basins for the LT 120/240 alternative. These alternatives were not fully evaluated as part of this Master Plan. Meeting a different demand pattern than that assumed by this Master Plan will require re-evaluating conventional treatment requirements and expansion needs.

Recommended major process modifications to the existing WTP facilities include: modifying the two flash mix units in the coagulation system; modifying the two pretreatment (flocculation-sedimentation) basins; adding a new pretreatment basin rated to a higher capacity than the existing basins for the LT 120/240 mgd alternative, adding new filter basin(s); adding additional filter



San Juan Water District Wholesale Master Plan -
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Figure 7-1

DETAILED PROCESS SCHEMATIC
PRETREATMENT AND FILTRATION

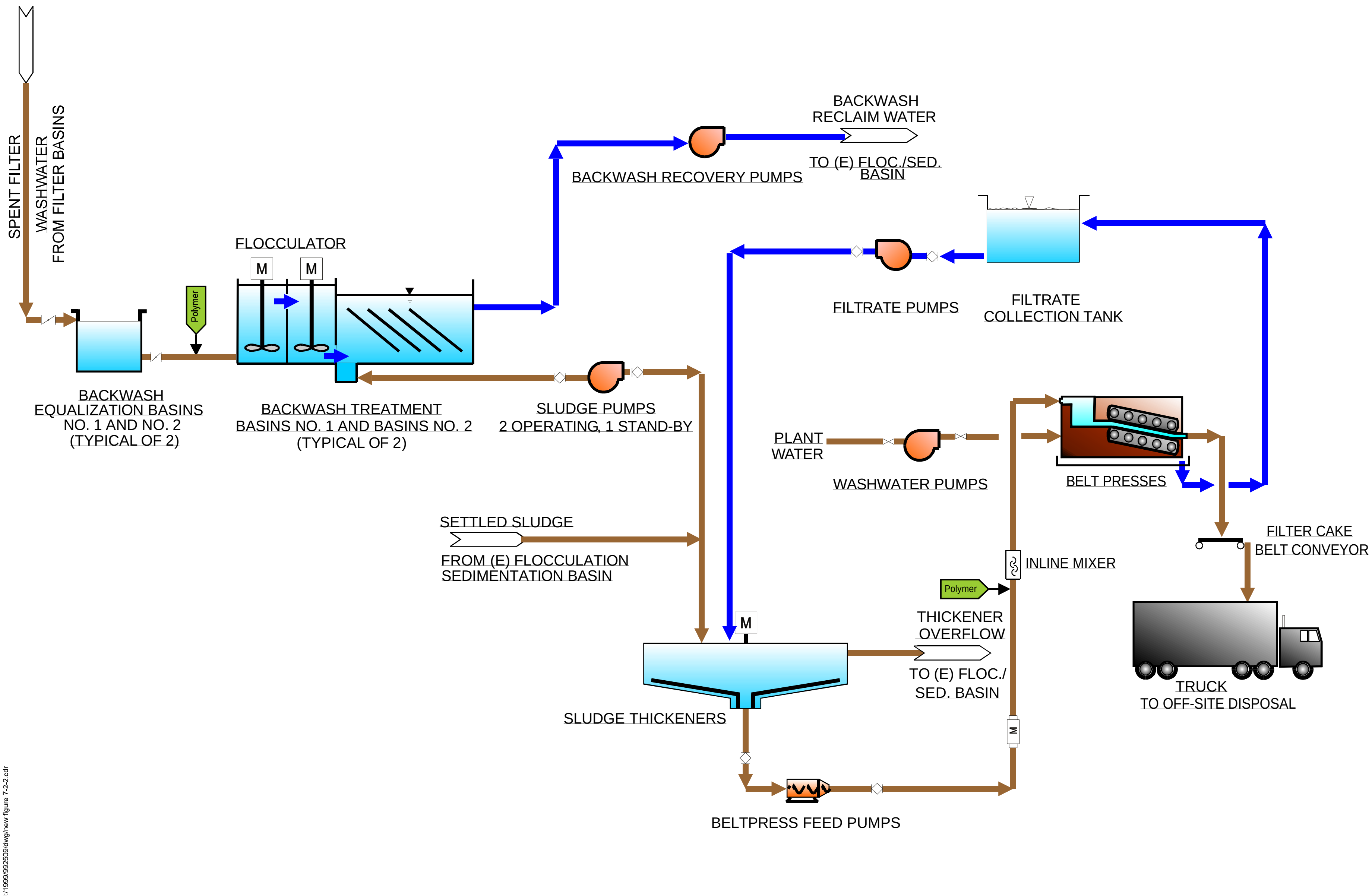


Figure 7-2
DETAILED PROCESS SCHEMATIC
BACKWASH AND SOLIDS HANDLING

backwashing capability in the existing filter basins; and replacing the existing filter backwash recovery and residuals handling systems.

7.4 Plant Capacity Expansions

This section discusses the plant capacity expansion for both LT 75/150 and LT 120/240. Table 7-3 presents information on process requirements for LT 75/150 and the phased components of LT 120/240.

The WTP capacity expansion recommendations include consideration of recommended design criteria for reliable operation. These design criteria include:

Redundant process units. To the extent feasible, new processes should be designed with a minimum of two units to allow taking one unit out of service for maintenance while maintaining a minimum level of capacity. Individual process units should be designed conservatively to permit "overloading" when other units are off-line while still meeting minimum treatment requirements.

Redundant process equipment. Process equipment such as sludge dewatering units, chemical feeders, and pumps should be designed with a backup unit. Standby pumps should be sized equivalent to the largest duty pump. Where it is not feasible to incorporate a backup unit, such as with sludge collectors in basins, adequate spare parts should be kept on hand to permit quick response to emergency maintenance.

Chemical storage. Storage facilities should be designed with a minimum of two tanks or containers.

Controls and instrumentation. All processes should be designed with a backup manual operation made to address SCADA or control system outages.

7.4.1 LT 75/150

Process requirements to expand the WTP capacity to 150 mgd direct filtration treatment, 75 mgd conventional filtration treatment include rapid mix (coagulation), pretreatment (flocculation-sedimentation), filtration, disinfection, backwash recovery system, residuals handling, chemical feed systems, and additional site improvements. The recommended improvements for the LT 75/150 Expansion are summarized in Table 7-3 and shown on Figure 7-3. Although all the recommended LT 75/150 improvements are listed as occurring at the same time, some of the more critical improvements, such as the backwash recovery and solids handling systems, should be constructed independently and earlier than other improvements.

The existing instantaneous blending pump (background) and rapid mix pumps (foreground) should be replaced with a pump jet-injection mixing system for better performance.

Rapid Mix (Coagulation). The existing WTP facilities include three stages of mixing for chemical coagulation. Three

Table 7-3
Summary of Recommended Process Improvements
For LT 75/150 and LT 120/240 Water Treatment Plant Expansions

Process	LT 75/150	LT 120/240			
Capacity, mgd	**120-150**	**120-150**	**150-180**	**180-210**	**210-240**
Conventional Capacity, MGD	75	75	90	105	120
Direct Filtration Capacity, MGD	150	150	180	210	240
Flash Mix					
Number of Units	2	2	2	2	2
Type	Pumped Jet	Pumped Jet	Pumped Jet	Pumped Jet	Pumped Jet
Number of Pumps/Horsepower	2-10 (1 duty)	2-10 (1 duty)	2-10 (1 duty)	3-10 (2 duty)	3-10 (2 duty)
Velocity Gradient, sec^{-1}	1,000	1,000	1,000	1,000	1,000
Flocculation Basins					
Number of Basins	2	3	3	3	3
Flocculation Trains per Basin	5-Parallel baffled	5-Parallel baffled	5-Parallel baffled	5-Parallel baffled	5-Parallel baffled
Avg. Side Water Depth, Feet	13.5	13.5	13.5	13.5	13.5
Cell Width, Feet	16	16	16	16	16
Flocculation Cell Length, Feet	86	86	86	86	86
Flocculation Time, All Trains in Service (Minutes at maximum flow)	13.3	20	16.7	14.3	12.5
Avg. Detention Time, Minutes, Conventional Operation	26	40	33.4	28.6	25
Transition Zone					
Number of Basins	2	3	3	3	3
Avg. Detention Time, Minutes, Conventional Operation, Basin 1 & 2 / Basin 3	15	30.4/15.2	25.3/12.7	21.7/10.9	19/9.5
Sedimentation Basins					
Number of Basins	2	3	3	3	3
Type	Tube Settler	Tube Settler	Tube Settler	Tube Settler	Tube Settler
Tube Settler Loading Rate, gpm/sf/day, Conventional Operation, Basin 1 & 2 / Basin 3	1.85	0.93/1.85	1.1/2.2	1.3/2.6	1.5/3
Basin Length, Settling Zone, Feet	176	176	176	176	176
Detention Time, min., Conventional Operation, Basin 1 & 2 / Basin 3	52	104/51	87/43	74/37	65/32

Table 7-3 (cont.)
Summary of Recommended Process Improvements
For LT 150 and LT 240 Water Treatment Plant Expansions

Process	LT 75/150	LT 120/240			
Capacity, mgd	**120-150**	**120-150**	**150-180**	**180-210**	**210-240**
Filters					
Number of Filter Basins	3	3	3	3.5	4
Number of Filters per Filter Basin	12	12	12	12	12
Number of Cells per Filter	10	10	10	10	10
Area per Cell, sf	64	64	64	64	64
Total Filter Media Area, sf	23,040	23,040	23,040	26,880	30,720
Design Loading Rate, gpm/sf,	5.4	5.4	5.4	5.4	5.4
One Filter Off-line Per Basin	5.92	5.92	5.92	5.92	5.92
Backwash Recovery and Treatment					
Flow Equalization Basin					
Number	2	2	2	3	3
Capacity (Each), gal	60,000	60,000	60,000	60,000	60,000
Treatment Modules, Number	2	2	3	3	4
Capacity, mgd	3	3	3	3	3
Total Capacity, mgd	6	6	9	9	12
Percent of Total Plant Production	4.0%	4.0%	5.0%	4.2%	5.0%
Residuals Treatment					
Number of Sludge Thickeners (50' diameter)	2	2	2	3	3
Capacity per thickener, gpm	200	200	200	200	200
Number of 2 Meter Belt Presses, duty-standby	2+1	2+1	2+1	3+1	3+1
Hours of Operation per Day[(a)]	9	9	10.5	8	9
Sludge Production (lbs/day)	28,000	28,000	33,000	39,000	45,000
Equipment Building	80' x 50'	100' x 50'	100' x 50'	100' x 50'	100' x 50'

Table 7-3 (cont.)
Summary of Recommended Process Improvements
For LT 150 and LT 240 Water Treatment Plant Expansions

Process	LT 75/150	LT 120/240			
Capacity, mgd	**120-150**	**120-150**	**150-180**	**180-210**	**210-240**
Chemical Feed Systems **Chlorine Gas**					
Total Capacity of One-Ton Containers, pounds	40,000	40,000	40,000	40,000	40,000
No. of One-Ton Containers Required On-line (based on a 550[b] lb/day withdrawal rate per 1-ton container)	7	7	8	10	11
One-Ton Containers per Day, Max Feed Rate of 3.0 mg/L	1.8	1.8	2.2	2.6	3.0
Chlorine: Onsite Hypochlorite Generation System					
lbs/day Cl_2 @ 2 mg/L lbs/day Cl_2 @ 3.3 mg/L	2,500 4,130	2,500 4,130	3,000 4,500	3,500 5,780	4,000 6,600
1,500 lb/day Generator Units (Duty & Standby)	2+1	2+1	3+1	3+1	4+1
Brine Tanks HOCl Tanks	3 3	3 3	4 4	4 5	5 6
Equipment Building	60’ x 30’	60’ x 30’	60’ x 30’	60’ x 30’	60’ x 30’
Brine Solution Metering Pumps @ 1,000 gph (sets)	4	4	4	5	5
Hypochlorite Metering Pumps (Duty & Standby)	3+1	3+1	3+1	4+1	4+1
Dosage average, mg/L maximum, mg/L	 2.0 3.3	 2.0 3.3	 2.0 3.3	 2.0 3.3	 2.0 3.3
Alum					
Metering Pump Capacity Required at Max Dose and Max Flow (gph)	96	96	116	135[c]	154[c]
Dosage average, mg/L maximum, mg/L	 7 20	 7 20	 7 20	 7 20	 7 20
Storage at average (Days)	30	30	30	30	30
Number of 13,500 Gallon Bulk Tanks	4	4	6	6	6

Table 7-3 (cont.)
Summary of Recommended Process Improvements
For LT 150 and LT 240 Water Treatment Plant Expansions

Process	LT 75/150	LT 120/240			
Capacity, mgd	**120-150**	**120-150**	**150-180**	**180-210**	**210-240**
Nonionic Polymers					
Dosage average, mg/L maximum , mg/L	0.1 0.5	0.1 0.5	0.1 0.5	0.1 0.5	0.1 0.5
Storage (Days)	30	30	30	30	30
Number of 275 gal. tote bins	2	2	2	3	3
Lime					
Dosage average, mg/L maximum, mg/L	6 12	6 12	6 12	6 12	6 12
Number of Slakers	1	1	2	2	2
Slaker Capacity, lb. of 90% CaO/hr.	695	695	834	975	1,112
Storage (Days)	30	30	30	30	30
Volume Required, ft^3 60 lb/ft^3 of 90% CaO	4,170	4,170	5,000	5,840	6,675
Number of 3,600 cf silos	2	2	2	3	3
Major Site Piping Parallel Plant Influent, Inches	72	84	84	84	84
North Basin to Filters, Inches	84	108	108	108	108
South Basin to Filters, Inches	48	72	72	72	72
Filters to Hinkle Reservoir, Inches	72	84	84	84	84

(a) Based on BFP capacity at 800 lb. solids per meter width per hour.
(b) Maximum chlorine gas withdrawal rate for ambient temperature at 80° F and gas withdrawn at ∅ psig.
(c) Existing alum metering pumps' capacity is 128 gph. Replace metering pumps for this plant capacity increase.

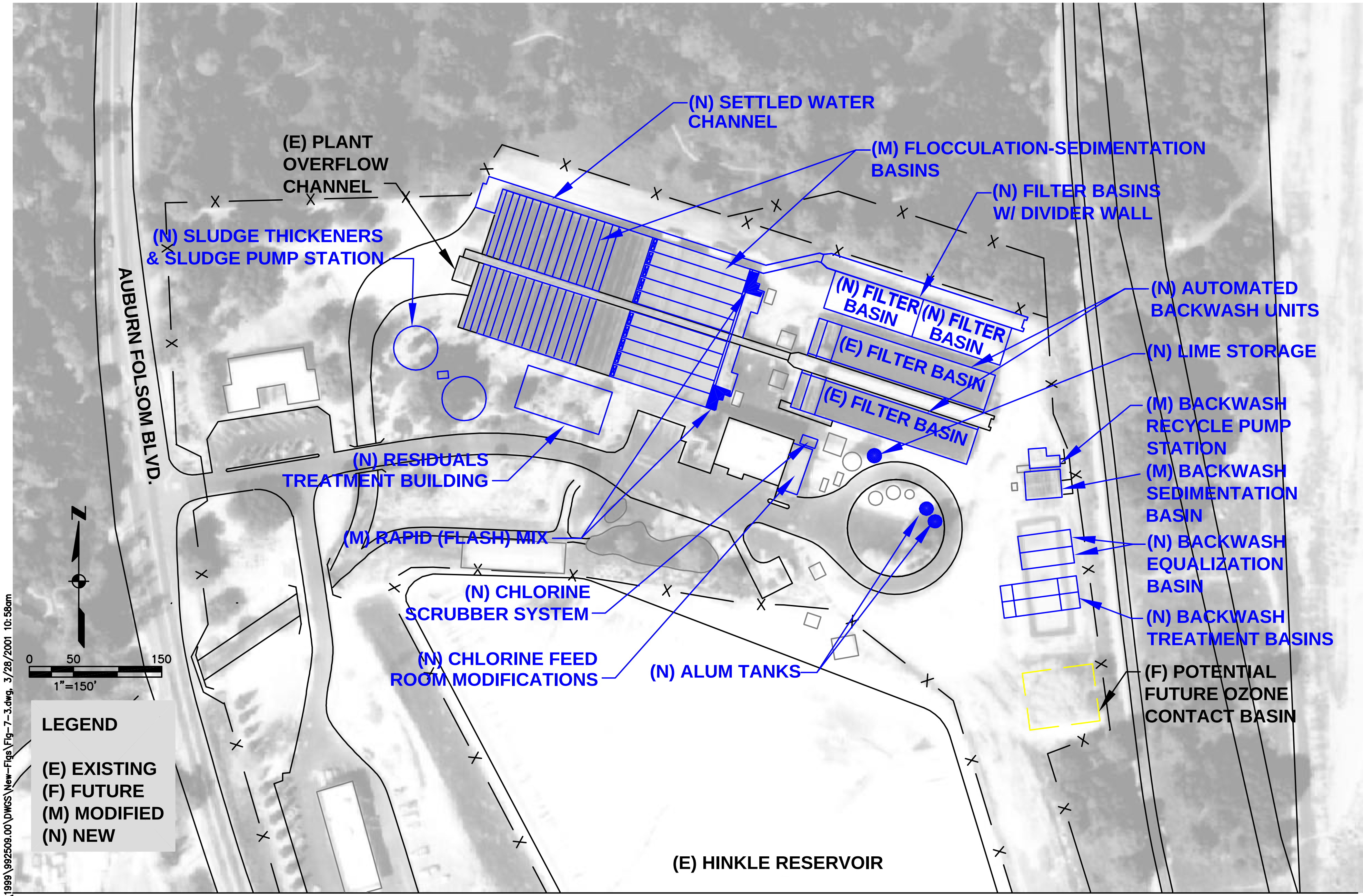


FIGURE 7-3
RECOMMENDED IMPROVEMENTS FOR LT 75/150 EXPANSION

vertical turbine mixers were included in the original design: one in the instantaneous (flash) blending chamber, and one in each of the two rapid mix chambers in each process train. However, the mixers in the rapid mix chambers are not used at plant flows greater than about 60 mgd because the excess turbulence they induce at high flows causes overtopping of the basin walls. The present operation, without additional mechanical mixing for coagulation, will continue to provide satisfactory coagulant mixing through hydraulic turbulence in the existing basins at plant flow rates up to 120 mgd. For plant capacities above 120 mgd, flash mixing is recommended for coagulation.

Flash mixing options include in-line mixers such as water-champs, static mixers, pump injection mixers, and back-mix reactors with vertical mixers. Static mixers require available energy from the incoming water line and therefore are not favored for a pumped supply. The District has experience with vertical mixers which, if properly designed, would be preferred; however, they would require adjustable frequency drives to accommodate the wide range of flows. Water champs are in-line mixers that are effective in retrofit applications; however, they usually result in higher O&M costs.

To minimize inlet pipeline headloss and provide adequate mixing over a wide range of flows, a pump jet-injection mixing system is recommended. For LT 75/150-mgd, one pump jet unit would be required for each coagulation train. This approach has been successfully used at a number of facilities treating similar source water. The primary coagulant is injected through a diffuser that is inserted in the middle of a fan spray created by the jet-injection nozzle. This type of mixer provides rapid dispersion of the primary coagulant over a wide range of plant flow rates. Supplemental coagulant chemicals and oxidants/disinfectants that can be blended without high energy will be introduced into the first two stages of the flocculation basins.

Pretreatment (Flocculation-Sedimentation). Optimization of both existing flocculation and sedimentation basins will be required for LT 75/150.

<u>*Flocculation Basins with In-line Bypass*</u>. The best optimization of the treatment process will be achieved at the flocculation basins. During the summer, the low turbidity Folsom Reservoir source water can be filtered following limited pretreatment with low coagulant doses. During high-demand summer months, in-line filtration (coagulation followed by filtration) may provide/permit the highest turbidity removal. Prior to the implementation of the California SWTR, Title 22, the District often operated the WTP in an in-line filtration mode with low chemical usage, long filter runs, and very low finished water turbidity. The DHS currently requires using the flocculation and sedimentation basins to provide disinfection contact time since some treated water by-passes Hinkle Reservoir. The pretreatment bypass connection that permits in-line filtration operation is currently only operated under emergencies when basin maintenance must be performed.

In-line filtration is not an acceptable filtration technology in California and would require a petition with supporting filter performance data to the DHS Surface Water Treatment Rule Committee. Since in-line filtration previously served the District well, it is recommended that the existing in-line filtration bypass be maintained and that the District develop protocol and assistance from DHS for implementation at a future date. In-line filtration could be considered after the Hinkle Reservoir bypass is eliminated (discussed in Chapter 8) and the existing backwash water pre-treatment facility is replaced.

During the winter, the raw water temperature and alkalinity decrease, and turbidity increases, primarily due to runoff from winter storms into Folsom Reservoir. The WTP cannot be operated in a direct or in-line filtration mode during this period and meet production requirements and water quality objectives due to the higher turbidities. When water is colder than 10^{o} C, it is more difficult to form a large, heavy, settleable floc without providing more flocculation time. Therefore, it may be necessary to reduce the plant flow rate to increase flocculation time to as much as 30 minutes during the winter when source water turbidity is high and water temperature is below 10^o C in order to form a larger heavier floc that can be removed by sedimentation.

The longer detention times required to form large settleable floc in conventional treatment can be detrimental to preparing the durable pinpoint floc required for optimal in-line or direct filtration operation. Flocculation for direct filtration is best accomplished in well-baffled basins with mechanical agitation. Basin designs that rely on hydraulic flocculation without mechanical mixers are suitable for plants with small variations in source water quality and plant flow rate. However, the District's summer to winter variations of flow, water quality, and treatment requirements (conventional versus direct filtration) require installation of mechanical flocculation equipment. Current state-of-the-practice is to provide large diameter vertical hydrofoil flocculators in baffled basins. Large vertical flocculators with variable speed drives permit flocculation tapering and, with seasonal variations, can turn at higher revolutions to create pinpoint floc. A larger settable floc can be formed by slowing down the flocculators and/or increasing coagulant dosages. However, vertical flocculators would be expensive to retrofit into the existing basins due to overhead support requirements and associated major structural modifications. Alternatively, the five existing flocculators could be replaced with reel (horizontal paddle wheel) type flocculators that are supported from the floor. This type of flocculator has provided good process results and would be less expensive than vertical flocculators, but it has extensive submerged moving parts that require regularly scheduled maintenance. Considering the advantages and disadvantages of each type of flocculator, we recommend horizontal flocculators for the expanded facilities.

Multiple parallel baffles within each existing flocculation basin would optimize flocculation basin performance. Redwood walls would be installed between each of the five parallel flocculation trains. A perforated flow distribution wall would be installed between each flocculation basin and the adjacent sedimentation basin similar to the existing perforated walls between existing flocculation zones 1 and 2 and zones 2 and 3 to improve flocculation performance.

While 25 to 30 minutes of flocculation is recommended ahead of sedimentation treatment, 10 to 15 minutes should prove more appropriate for direct filtration. Less than 10 minutes has been used successfully at larger facilities using ozone as a pre-oxidant. For LT 75/150, a minimum detention time of 13 minutes is provided at the maximum plant flow rate of 150 mgd (direct filtration treatment) and 26 minutes is provided at the conventional treatment capacity of 75 mgd.

Sedimentation Basins. Historically, the District has been able to treat high turbidity events by reducing the flowrate through the pretreatment and filtration processes. This reduced loading rate has allowed the WTP to consistently produce high-quality treated water. As mentioned in Chapter 6, the sedimentation basin design is not conservative and results in very poor turbidity removal during high turbidity source water winter conditions. Floc carryover occurs at flow rates well below the sedimentation basins' original design capacity of 100 mgd. Part of the problem is due to the shallow basins. The basins reduce in depth as the flow travels through them and water is "skimmed" by the effluent launders. This shallow basin depth could contribute to the solids carryover reported by plant staff at flowrates above about 30 mgd (per basin).

Three alternative approaches to increasing the treatment capacity of the flocculation-sedimentation pre-treatment processes were evaluated as part of the Master Plan. The alternatives included combinations of constructing a third rapid mix (coagulation)-flocculation-sedimentation treatment train parallel to the existing two pretreatment trains, and/or constructing modifications to the existing two pretreatment trains to gain capacity. The evaluation of alternatives is presented in Appendix 7-2. Based on a review of the alternatives with the District at a workshop on January 30, 2001, the selected approach for increasing sedimentation capacity for LT 75/150 is to modify the existing pretreatment basins to correct deficiencies and increase total conventional treatment capacity to a nominal 75 mgd.

Pre-treatment capacity of the existing basins would be increased by replacing the existing 2-foot deep tube settler modules with 1) new 4-feet deep tube or plate settler modules in the first (deepest) 126 feet of each of the two sedimentation basins and 2) new 2-feet deep tube settler modules in the last (shallowest) 50 foot section in each sedimentation basin. The existing 18-inch by 21-inch launders would be replaced with 24-inch by 24-inch launders to increase hydraulic capacity. The launder supports and bracing would also be improved.

A new settled water conveyance channel on the north side of the two existing pretreatment basins is required to provide additional hydraulic capacity to at least 150 mgd to accommodate initial and future direct filtration treatment capacity requirements. The launders from the north pretreatment basin would be re-directed towards the new settled water channel.

An alternative technology that receives the same pre-treatment credit from DHS as conventional sedimentation pre-treatment is ballasted floc sedimentation. This technology is not new, but installation and operational experience in the United States is limited. Ballasted floc sedimentation utilizes silica sand added as a "weighting" agent in the coagulant feed, causing floc to settle much more rapidly than traditional chemical floc. This permits sedimentation basins 20 to 25 percent the size of conventional sedimentation basins. However, the high loading rate and relatively short hydraulic resident time in these basins provide very little time for the plant operator to respond to a unit failure.

Ballasted floc sedimentation provides a high level of pre-treatment and requires a reduced footprint compared to conventional sedimentation pre-treatment. It should be considered as an option for the expansion to 150 mgd (or beyond) if additional U.S. experience is available to better evaluate the process by the time the decision needs to be made.

Filtration. Three alternative methods of increasing filtration capacity from 120 mgd to 150 mgd could be implemented. The first approach is to construct an additional filter basin similar to the two existing automatic backwashing filter basins. Each of the three filter basins would normally be used to treat 50 mgd. This approach would have a high degree of redundancy and reliability. As described in Chapter 6, the original plant design criteria and current DHS design standards permit filtration capacity as high as 60 mgd from each filter basin. If a basin needs to be removed from service, the remaining two basins could be operated to achieve 120 mgd, or 80 percent of total plant capacity.

As a critical piece of process equipment, two filter backwash units should be provided in each existing and new filter basin.

A second filter backwash unit should be added to each of the two existing filter basins, and two filter backwash units should be provided with the new basin, in order to further improve redundancy and reliability. This would also reduce backwash time for each basin from more than 12 hours to as low as 6 hours.

As a variation to the first approach discussed above, constructing a half basin (with six rather than 12 filters) would also permit increasing plant capacity to

150 mgd. This approach to increasing filtration capacity would be more economical, but would not offer the same level of reliability and redundancy as constructing a full basin. With one of the larger basins out of service, plant capacity would be reduced to 90 mgd, or 60 percent of total plant capacity. WTP operators would also not have the same flexibility to deal with certain poor raw water quality events with the smaller filter area in this approach. If this alternative were considered, we still recommend adding a second filter backwash unit to each of the two existing filter basins and including one filter backwash unit with the new "half-basin."

The second approach to increasing filtration capacity is to increase the filter surface loading rate in the two existing filter basins from the DHS-permitted 6 gpm/ft^2 to as high as 7.5 gpm/ft^2. Pilot testing would be required by DHS to demonstrate performance at the higher loading rate. However, based on the hydraulic capacity of existing facilities and filter operating goals, increasing the maximum filter surface loading rate to 7.5 gpm/ft^2 is not considered an appropriate strategy to increase plant capacity to 150 mgd *at this time*.

The third approach is to construct four individual high-rate deep-bed filter basins with capacity to filter at least 30 mgd with one filter off-line, in accordance with current DHS guidelines. Based on the need to provide more valves and controls for each filter plus air-wash blowers and backwash supply pumps, as well as having to increase the washwater recovery system capacity to accommodate a greater instantaneous waste filter backwash washwater volume, this approach is considered to be unsatisfactory.

Based on District goals of capacity and reliability and DHS redundancy requirements for critical processes, the recommended approach to increasing filtration capacity from 120 to 150 mgd is to construct a filter basin similar to the two existing filters basins on the north side of the two existing basins. The new filter basin should be constructed with a divider wall between each group of six filters to permit removing as few as six filters (30 mgd of capacity) from service for maintenance at any time. Two filter backwash units should be provided with the new basin.

Filter-To-Waste. The California Code of Regulations Title 22, Chapter 17, Surface Water Filtration and Disinfection Treatment, Article 4., Design Standards includes Section 64658, New Treatment Plants. This section includes requirements that are applicable to both new filtration and disinfection facilities and to existing facilities that will be modified. Section 64658 includes paragraph (b), (8) which states: "Provide for filter-to-waste for each filter unit or addition of coagulant chemicals to the water used for backwashing."

The District's filter design uses filtered water from nine of the ten 8-foot by 8-foot filter cells in each filter unit as the filter backwash water supply for the one 8-foot by 8-foot filter cell being washed. In essence, this filter backwash method provides equivalent filter-to-waste operation for nine of the ten filter cells in each filter unit. However, the last cell to be backwashed in a filter unit does not filter-to-waste before being placed back in service. Therefore, the filter-to-waste system required by Section 64658 (b) (8) should be included in the filter improvements implemented as part of LT 75/150. The filter-to-waste water should have a relatively low turbidity and should not be commingled with the spent filter backwash water prior to blending with the raw water.

Disinfection. Free chlorine should continue to be the primary oxidant and disinfectant at the WTP. However, although chlorine gas is a proven technology with a good track record, safety concerns have resulted in more stringent ordinances for toxic gases and secondary containment requirements. For these reasons, modifications to the existing chlorine system will be required to bring it up to code. These modifications include construction of improvements to the chlorine storage and feed facility to ensure that it is gas-tight. In addition, a scrubber system capable of neutralizing the accidental release of chlorine from a full one-ton container will be required. Alternatively, the District could change its chlorine gas system to a bulk hypochlorite or onsite sodium hypochlorite generation facility. This Master Plan recommends chlorine gas for LT 75/150.

The chlorine area will need to be enclosed and provided with a scrubber to comply with current safety requirements.

Disinfection Contact Time. The District's WTP currently operates in compliance with the DHS 3-log *Giardia* and 4-log virus removal-inactivation requirement for surface water supplies using a multi-barrier combination of physical removal and disinfection. The plant currently receives 2.5-log *Giardia* and 2-log virus removal credit when it operates in a conventional filtration mode, and receives a 2.0-log *Giardia* and 1-log virus removal credit when it operates in a direct filtration mode. It would also receive a 2.0-log *Giardia* and 1-log virus removal credit operating in an in-line filtration mode, if approved. Disinfection is used to meet the remaining inactivation requirement.

Since a portion of the treated water currently bypasses Hinkle Reservoir through the Cooperative Transmission Pipeline, most of the disinfection credit must be achieved ahead of Hinkle Reservoir as the water flows through other treatment units. (A small amount of disinfection credit is received in the 78-inch Cooperative Transmission Pipeline before the first service connection.) Therefore, the disinfection credit needed to comply with the required combination of 3-log *Giardia* and 4-log virus removal-inactivation is achieved by maintaining an adequate chlorine residual in the water as it flows through the two existing flocculation and sedimentation trains and two filter basins.

Table 7-4 shows the disinfection CT required and the existing disinfection CT available when the WTP operates in various treatment modes. As WTP capacity increases, the available disinfection CT is reduced. With two pretreatment basins in service and the plant operating in a direct filtration mode, there is sufficient disinfection CT up to a WTP capacity of approximately 130 mgd. Above this capacity, chlorine residual through the WTP may need to be increased or the Cooperative Transmission Pipeline connection should be relocated to allow additional disinfection CT credit through Hinkle Reservoir. In an in-line filtration treatment operating mode, there is only sufficient disinfection CT credit available up to a capacity of about 36 mgd with two pretreatment basins in service. This indicates the WTP cannot operate in an in-line mode without disinfection CT credit through Hinkle Reservoir. This is discussed in more detail in Chapter 8.

Table 7-4
WTP Disinfection Requirements and Available Disinfection CT

Plant Flow Rate (mgd)	WTP Operating Classification	Disinfection Required[(d)] (mg/L-min.)	Disinfection Contact Time Required (Min.)	Available Contact Time[(e) (f) (g)] (Min.)
60	Conventional[(a)]	18	22.5	116
60	Direct Filtration[(b)]	37	46	116
60	In-line Filtration[(c)]	37	46	45
120	Conventional[(a)]	18	22.5	53
120	Direct Filtration[(b)]	37	46	53
120	In-line Filtration[(c)]	37	46	23
150	Conventional[(a)]	18	22.5	55
150	Direct Filtration[(b)]	37	46	55
150	In-line Filtration[(c)]	37	46	23
180	Conventional[(a)]	18	22.5	47
180	Direct Filtration[(b)]	37	46	47
180	In-line Filtration[(c)]	37	46	17
210	Conventional[(a)]	18	22.5	41
210	Direct Filtration[(b)]	37	46	41
210	In-line Filtration[(c)]	37	46	16
240	Conventional[(a)]	18	22.5	36
240	Direct Filtration[(b)]	37	46	36
240	In-line Filtration[(c)]	37	46	14

(a) Conventional Treatment receives 2.5-log *Giardia* removal credit and 2.0-log enteric virus removal credit. Disinfection CT for the remaining 0.5-log *Giardia* inactivation is the controlling condition for treated water pH between 6 and 9 and a water temperature as low as 10°C.

(b) Direct Filtration Treatment receives 2.0-log *Giardia* removal credit and 1.0-log enteric virus removal credit. Disinfection for the remaining 1.0-log *Giardia* inactivation is the controlling condition for treated water pH between 6 and 9 and a water temperature as low as 10°C.

(c) In-line Filtration Treatment receives 2.0-log *Giardia* removal credit and 1.0-log enteric virus removal credit. Disinfection for the remaining 1.0-log *Giardia* inactivation is the controlling condition for treated water pH between 6 and 9 and a water temperature as low as 10°C.

(d) Disinfection requirement based on a chlorine residual concentration of 0.8 mg/l and a pH of 7.0.

(e) Available CT based on 2 flocculation-sedimentation trains and 2 filter basins for plant flows to 120 mgd and 3 trains and basins for flows above 120 mgd.

(f) Disinfection contact time based on a T_{10} to HDT ratio through the pretreatment basins of 0.49 to 1 for plant flow rates less than 50 mgd and 0.59 to 1 for plant flow rates greater than 50 mgd, and 0.3 through the settled water channel and filters at all flow rates based on District tracer studies.

(g) Disinfection contact time in the 78-inch cooperative transmission pipeline based on a T_{10} to HDT ratio of 1, a pipeline volume of approximately 116,140 cubic feet before the first connection, and up to 50-percent of the WTP flow through the pipeline. (Source: DHS Annual Inspection Report, August 1999.)

Future Cryptosporidium Inactivation. The USEPA indicates that *Cryptosporidium* inactivation will be required in future regulations for some water supplies. Because research indicates that free chlorine or chloramine are unsuitable for *Cryptosporidium* inactivation, the District should plan for the addition of ozone, chlorine dioxide, or UV light as a possible future disinfectant. For direct filtration plants, ozone would be fed prior to coagulation to reduce construction costs. In conventional treatment plants, the preferred location for ozone would be between sedimentation and filtration. The space allocated for an ozone disinfection facility should also be adequate for a chlorine dioxide and/or UV light disinfection facility.

Backwash Water Recovery System. The capacity of the existing backwash water recovery system is not adequate to remove much of the solids present in the spent backwash water and is not adequate to handle waste filter backwash water from two simultaneous filter backwashes (one from each filter basin). In addition, the return water treatment system has been the most problem-prone and maintenance-intensive system at the WTP. The existing spent filter backwash water recovery system should be replaced with a more reliable treatment process at the earliest opportunity in order to permit the WTP to remain in compliance with existing regulations and meet anticipated regulations and guidelines.

The 150-mgd expansion should include a new 6-mgd backwash water recovery system that would be capable of treating approximately 4 percent of total plant production. As indicated in Section 7.3.2, Detailed Screening and Appendix 7-1, several alternative filter backwash pre-treatment options are suitable for this treatment process. Therefore, the District could consider pilot-testing. However, a sedimentation process with plate settlers can be designed and constructed without pilot testing and can meet the performance, operations, and maintenance requirements of the District. It is therefore recommended and used as the basis for planning in this Master Plan.

The existing backwash water recovery system should be demolished and replaced with a new, properly sized, reliable system.

Two equalization basins should be provided to allow handling the normal starts and stops of the backwashing process throughout the day. The system should also include two treatment modules, each with a 3-mgd capacity capable of meeting the normal requirements for backwash treatment. The new facilities would be constructed in the same location as the existing backwash recovery pond. Both the existing backwash recycle pump station and sludge pump station will need to be modified or replaced. The existing plant air system for the air diaphragm sludge pumps, and the polymer feed system (both in the bottom floor of the Control Building) will need to be upgraded. Alternatively, new, dedicated air and polymer feed systems could be located in a new building near the backwash equalization basins.

Residuals Handling. Solids that are removed during backwash water treatment and recovery are currently pumped offsite across Auburn-Folsom Road to the District's sludge drying facilities at Baldwin Reservoir. Once dried, the sludge is removed from this location and utilized as a soil amendment for agricultural uses. Space on the site is limited, and it has been a labor intensive operation to constantly spread and move around sludge to handle production requirements. The existing solids handling facilities cannot be reasonably expanded because of the limited site space and lack of capacity at Baldwin Reservoir. No other large land areas are available near the existing WTP for construction of similar low-tech approaches to solids handling. Consequently, mechanical dewatering facilities are recommended at the existing WTP site.

Based on the evaluation of alternatives in Appendix 7-1, belt filter presses are recommended. Two sludge thickeners would be used to further thicken sludge from the backwash treatment system and the sludge withdrawn from the sedimentation basins. Thickened sludge would be processed by three 2-meter belt filter presses, two duty presses and one standby. A new residuals handling building, approximately 4,000 square feet in size, would be required to house the belt presses, a control room, and a polymer feed system. Dewatered cake from the belt presses would be trucked to a landfill for disposal. The sludge thickeners and building can be located between the District Administration Building and the existing WTP facilities as shown on Figure 7-3.

Chemical Feed Systems. All existing chemical feed systems will be retained and modified for the WTP expansion. No new chemicals, except solids conditioning polymer and sodium hydroxide (for chlorine gas neutralization in the new scrubber), will be required in the initial expansion. The following chemical systems will be retained:

- Alum - primarily coagulant.
- Non-ionic polymer – coagulant aid.
- Cationic polymer - system needs to be replumbed and returned to service.
- Batch polymers - non-ionic and anionic, as coagulant and filter aids.
- Lime - pH adjustment, stabilization.
- Chlorine - oxidation and primary and residual disinfection.

As discussed under Regulatory Requirements in Chapter 4, the District is required by California State law to provide fluoridation if funding becomes available from sources other than ratepayers or taxpayers. Space should be reserved in the chemical storage area and control building basement for fluoride storage and feed. There is sufficient space for these facilities if required.

Operations Buildings. The existing plant control building modifications for a 30 mgd capacity increase would be minor. Discussions with plant staff indicate that the existing operations/plant control area, offices, crew quarters, laboratory, and file storage area are adequate and should continue to be adequate for plant operations after plant capacity increases to at least 150 mgd. An additional bathroom facility is required on the bottom floor of the building. Although the existing kitchen and staff break area is inadequate, improvements to these facilities are already scheduled and do not need to be included in an LT 75/150 expansion project.

The existing laboratory is adequate for existing plant operation’s in-house water quality testing. Plant staff presently perform regular water quality analysis for alkalinity, pH, hardness, turbidity, color, chlorine residual(s), particle counts, color, conductivity, and temperature. Since the existing lab area is adequate and an increase in plant capacity to 150 mgd will not have much impact on the quantity of these tests, the existing laboratory space should continue to be suitable for plant operations. The District currently sends water samples to a contract laboratory for analysis of microbial contaminants, including coliforms and heterotrophic bacteria, DBPs, SOCs, VOCs, inorganic compounds as well as radionuclides. The plant staff does not anticipate conducting water quality analysis for these constituents, even after the plant capacity increases.

Besides the lower floor bathroom, some money should be budgeted for control building modifications to accommodate control system upgrades and replacements.

Additional Site Improvements. To accommodate increased flows, it will be necessary to increase onsite hydraulic capacity by providing parallel piping between basins and by modifying process units to increase hydraulic capacity by enlarging openings and increasing capacity of weirs and launders.

On-site pipelines should be modified to permit bypassing process units during plant expansion construction activities, as well as to maximize plant capacities during peak flow events. The addition of isolation gates inside each flocculation basin, and an isolation bulkhead in the settled water channel, will allow for isolation of the existing settled water channel to accommodate construction, as well as provide a direct filtration bypass of the sedimentation basins.

Another recommended site improvement is a new in-plant pump station. The pumps in the existing pump station are installed “in-line,” with the suction bell penetrating into the treated water pipe section. There is insufficient separation between adjacent pumps, and the approach velocity to the pump intakes can exceed 8 feet per second. Each of these design elements can create turbulence and vortexing. The WTP staff report that when multiple pumps operate there are problems with cavitation and air entrapment. This forces operating only one pump, limiting the available plant water.

The in-plant pump station should be replaced with a properly designed side-stream pump station.

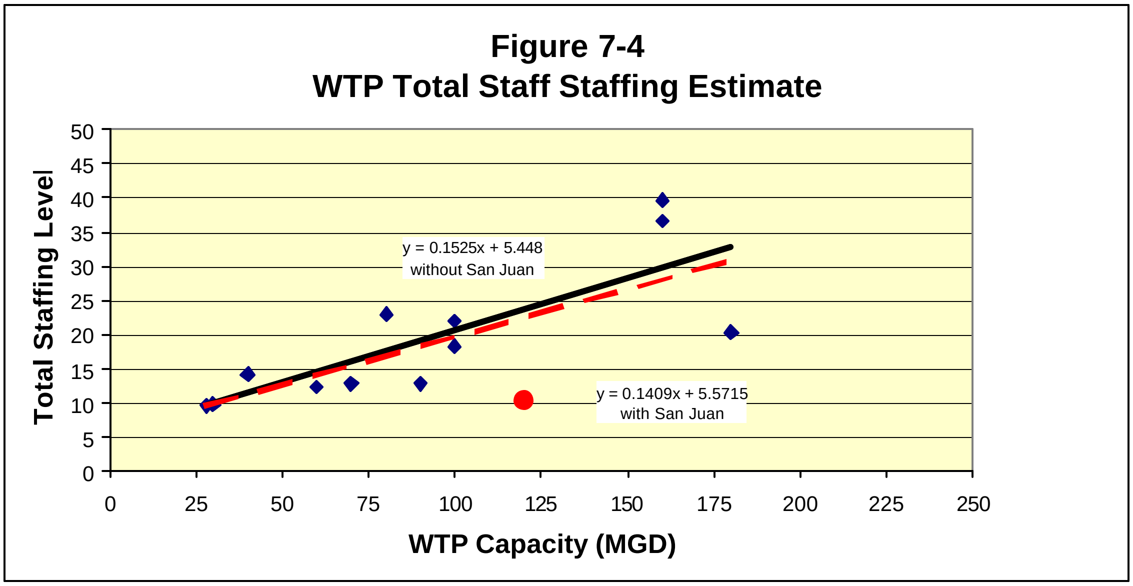
Although there are options for improving the operation of the existing pump station, some of the reported problems would likely still exist to a certain degree when there is a high demand for plant water. It is recommended that a new side-stream clearwell sump with can-type vertical turbine pumps be constructed following Hydraulic Institute Standards.

A final recommended site improvement is replacement of the orifice plate flow meters located on the 42-inch inlet water pipelines. These meters are used for rate of flow control. Orifice plates are a highly functional, inexpensive method of measuring flow. However, they offer a limited range of flows and become a major headloss restriction at higher flow rates. Magnetic flowmeters are recommended for this application because of their proven performance, reliability, lack of exposed metal in the pipeline, and relative ease to maintain.

Recommended Plant Staffing. Several large water supply agencies were contacted during the Master Plan Study to obtain information on plant staffing at their water treatment plants, including supervisory, operational, laboratory, and maintenance personnel. The agencies contacted included East Bay Municipal Utilities District (EBMUD), Santa Clara Valley Water District (SCVWD), City of Sacramento, and City of San Francisco Water Department (SFWD). All of these agencies operate conventional filtration treatment plants similar to the District's WTP. Some of these agencies also operate direct filtration or in-line filtration treatment plants. The capacities of these plants vary from 30 mgd to 180 mgd. The plant operating staff at each of these agencies belong to unions. The data from this survey is shown in Table 7-5 and on Figures 7-4 and 7-5.

Table 7-5
WTP Staffing Survey Results

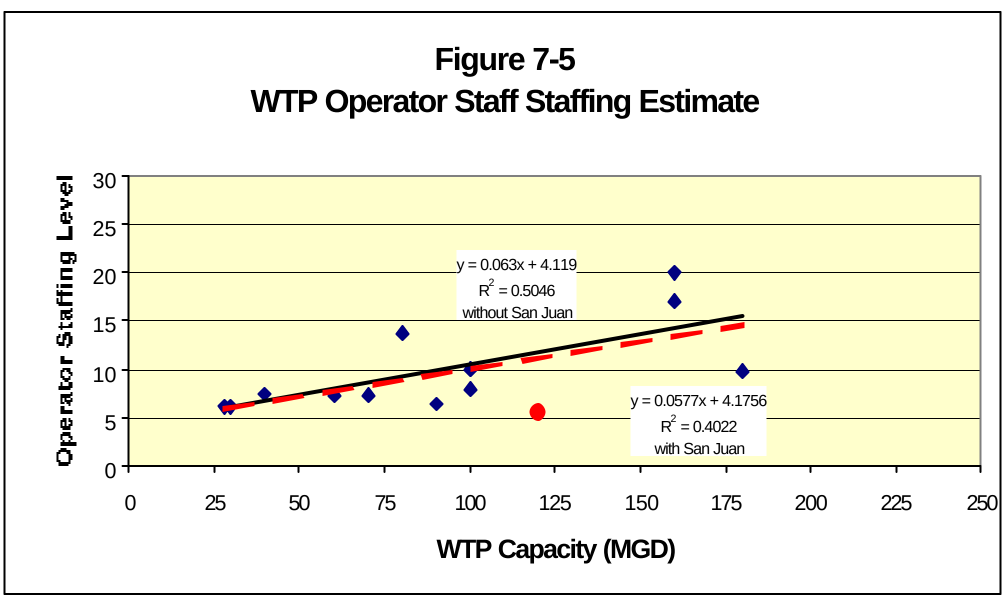
WTP/AGENCY	WTP Capacity	operator	mechanics	instrument technician	electrician	misc.	lab personnel	TOTAL
Lafayette/EBMUD	28	6.1	0.5	0.5	0.25	1	1.3	9.65
San Pablo/EBMUD	30	6.1	0.5	0.5	0.25	1	1.4	9.75
Penitenica/SCVWD	40	7.4	2	1	0.5	1	2.4	14.3
USL/EBMUD	60	7.3	0.5	0.5	0.25	1	2.8	12.35
El Sobrante/EBMUD	70	7.3	0.5	0.5	0.25	1	3.3	12.85
Rinconada/SCVWD	80	13.7	2	1	0.5	1	4.7	22.9
Walnut Crk/EBMUD	90	6.4	0.5	0.5	0.25	1	4.2	12.85
Santa Teresa/SCVWD	100	7.9	2	1	0.5	1	5.9	18.3
Fairbairn WTP	100	10	3	2	1	4	2	22
Sacramento WTP	100	10	3	2	1	4	2	22
Sunol/SFWD	160	17	0.9	2.7	0.5	1	14.5	36.6
Tracy/SFWD	160	20	0.9	2.7	0.5	1	14.5	39.6
Orinda/EBMUD	180	9.8	0.5	0.5	0.25	1	8.4	20.45
San Juan Water District	120	5.5	2.67	0.1	0	1.5	0.5	10.27



Notes:

Electrician time for SFWD & SCVWD is 50% (0.5) for each plant; 25% (0.25) for EBMUD.
Mechanics time for SFWD is 90% (0.9); 50% (0.5) for EBMUD.
Instrumentation technician time for SFWD is 2.7; 0.5 for EBMUD.
Misc. support personnel time for each plant (gardener, painter, carpenter, plumber, groundskeeper, etc.) is 100% (1.0) for all three utilities.

Utility/plant-specific Issue:
SVWD's Santa Teresa WTP is highly automated so that it does not require a high level of staffing (18.3) relative to its capacity (100 mgd).
There is a labor union factor that may significantly influence staffing at EBMUD, besides normally required based on capacity of the plant.



Based on the plant staff survey, the total number of staff at a 120 mgd capacity WTP should be between 15 and 20 staff persons, and the number of operators should be about 12. The plant staffing survey data suggests that the current staffing level for the District is about two-thirds the staffing level at plants with similar treatment processes and similar capacities. Since these plants have a unionized workforce and the District does not, the total number of plant staff at the District WTP should be expected to increase, but not by a 50 percent increase. The survey data also indicates that the total number of plant staff should increase by three or four additional staff persons and the operator staff should increase by at least two additional operators when the plant capacity increases from 120 mgd to 150 mgd.

Although the recommended facility improvements include addition of new treatment units in parallel with some existing units, the plant operator's duties should not change significantly. It should be noted that installing belt filter presses to handle the waste sludge stream will require additional operator time, in the range of two hours per day. The new spent filter backwash water recovery system will also require some operator attention, but should improve the return water quality and may actually reduce overall operator time required for this system. However, the District should plan to hire two additional operators, one new equipment/mechanical maintenance technician, and one electrical/instrumentation and control system technician to improve plant operational adequacy.

7.4.2 LT 120/240

Process modifications required to increase the WTP capacity up to a maximum of 240 mgd direct filtration treatment, 120 mgd conventional filtration treatment include rapid mix, pretreatment, filtration, disinfection, backwash recovery system, residuals handling, chemical feed systems, and additional site improvements. The recommended process improvements, in 30 mgd increments, are summarized in Table 7-3. The LT 120/240 scenario will require additional process "modules" and/or process equipment for each treatment process compared to an LT 75/150 WTP capacity requirement. The most significant differences are WTP hydraulics (sizing of pipes and channels), a third pretreatment train, additional filters, and the disinfection system.

Design of the first increment of expansion will depend in many ways on the ultimate required future capacity of the WTP. For an LT 120/240 expansion, piping, channels, and basin inlets and outlets should be modified, or added, to accommodate an ultimate capacity of 240 mgd during the initial 150-mgd expansion. The residuals treatment building should be constructed large enough to accommodate additional belt filter presses. Space should also be allocated for additional pumps, process equipment, and parallel basins.

For the LT 75/150 scenario, it was recommended that the WTP continue to use chlorine gas for disinfection, with the implementation of safety improvements. The same approach is recommended for the first phase expansion (150 mgd) of the LT 120/240 scenario. However, for larger WTP capacities, additional chlorine storage requirements, more frequent one-ton container changeout, additional safety concerns, and code requirements may dictate changing to an onsite hypochlorite generation system. Changing regulations may also drive the need to add supplemental disinfection facilities such as ozone, chlorine dioxide, or UV.

The following paragraphs address requirements to modify the WTP beyond a 75/150 mgd capacity, up to a capacity of 120/240 mgd. Where only a single capacity is shown below (i.e. "150 mgd"), it refers to the direct filtration treatment capacity of the WTP.

The recommended phased improvements for the LT 120/240 expansion are shown on Figures 7-6 through 7-9.

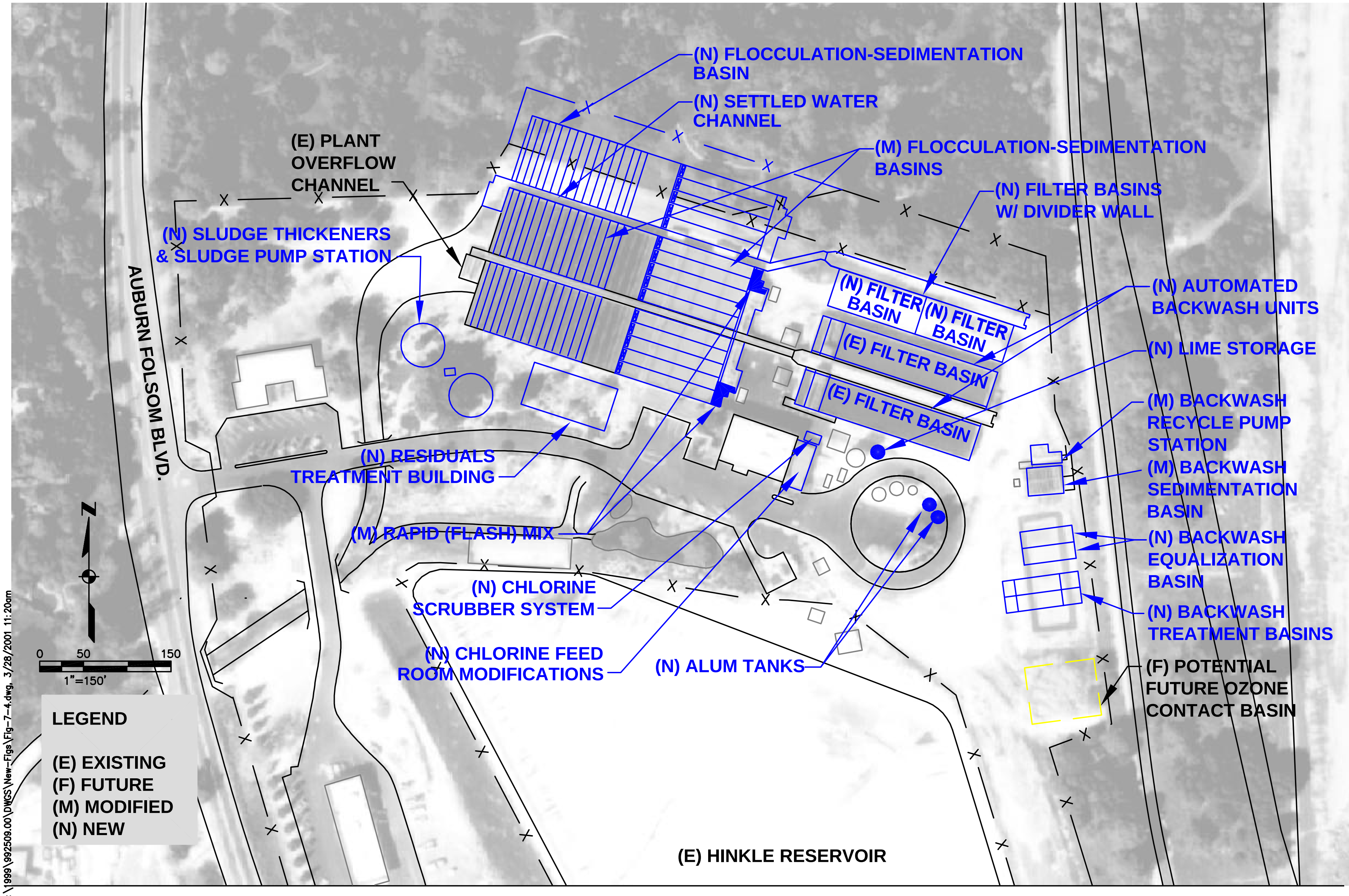


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FIGURE 7-6
RECOMMENDED IMPROVEMENTS FOR LT 120/240 EXPANSION PHASE 1 (75/150 MGD)

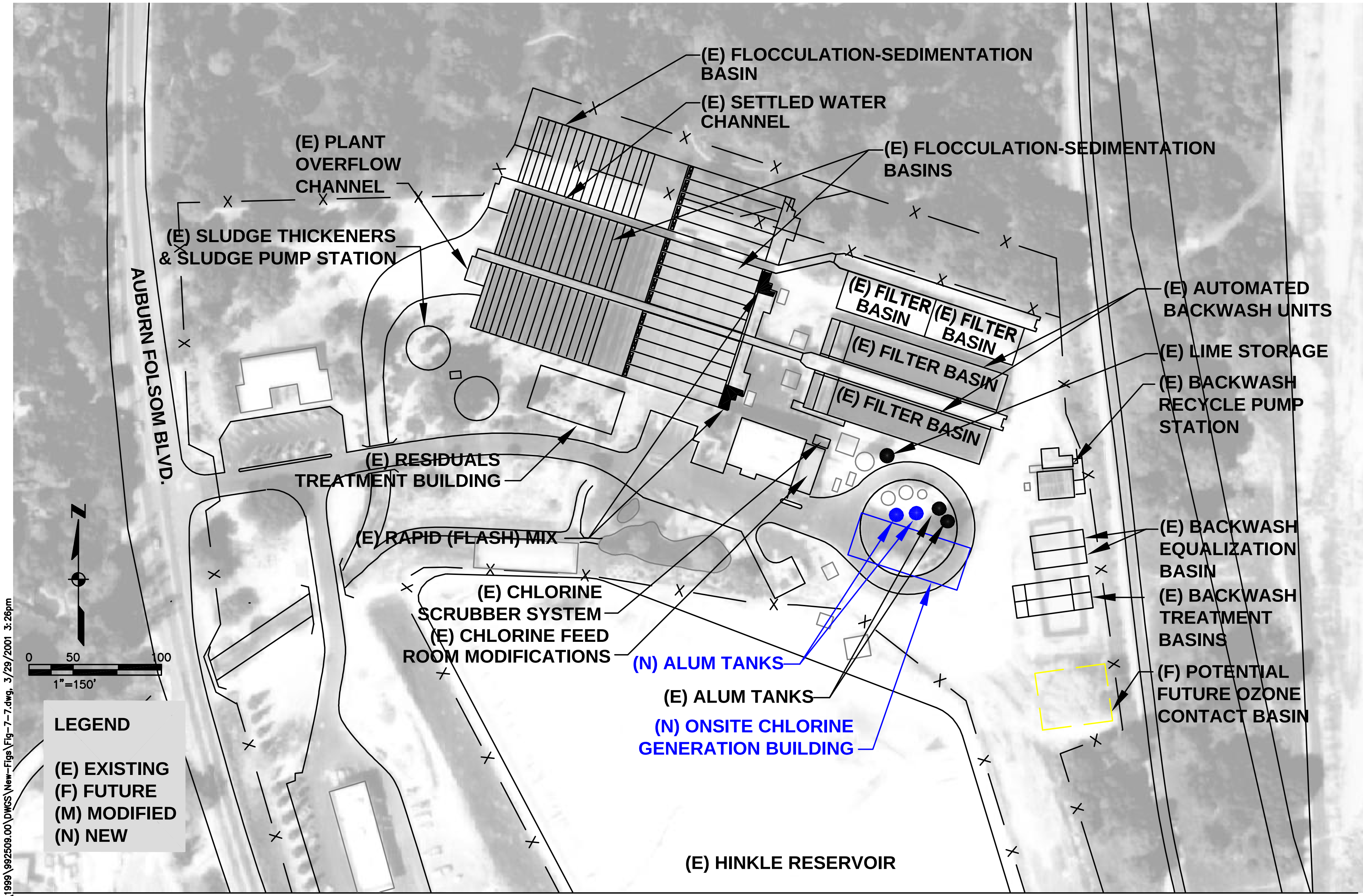

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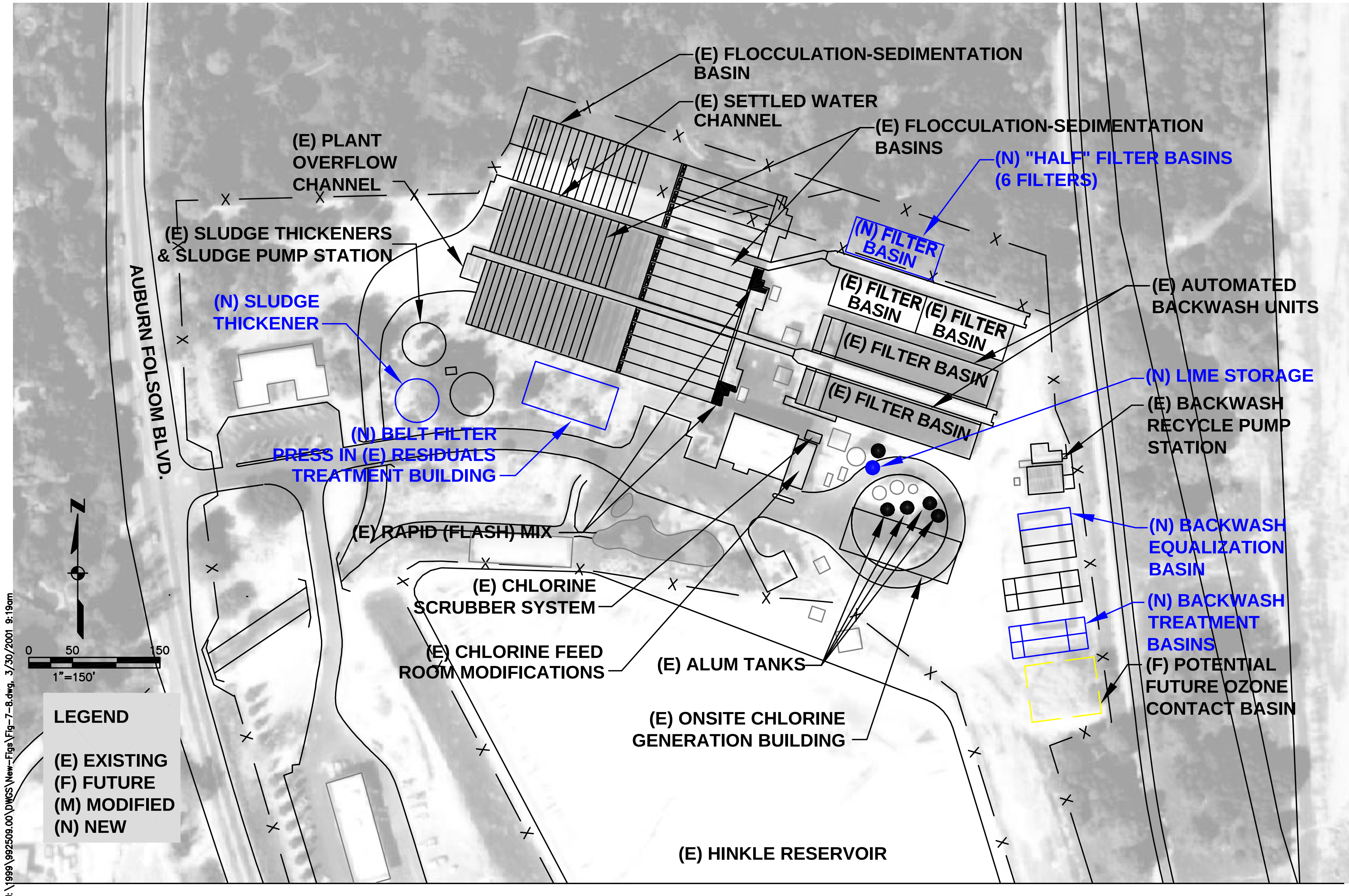
FIGURE 7-7
RECOMMENDED IMPROVEMENTS FOR LT 120/240 EXPANSION PHASE 2 (90/180 MGD)



San Juan Water District Wholesale Master Plan - Water Supply and Treatment

FIGURE 7-8
RECOMMENDED IMPROVEMENTS FOR LT 120/240 EXPANSION PHASE 3 (105/210 MGD)

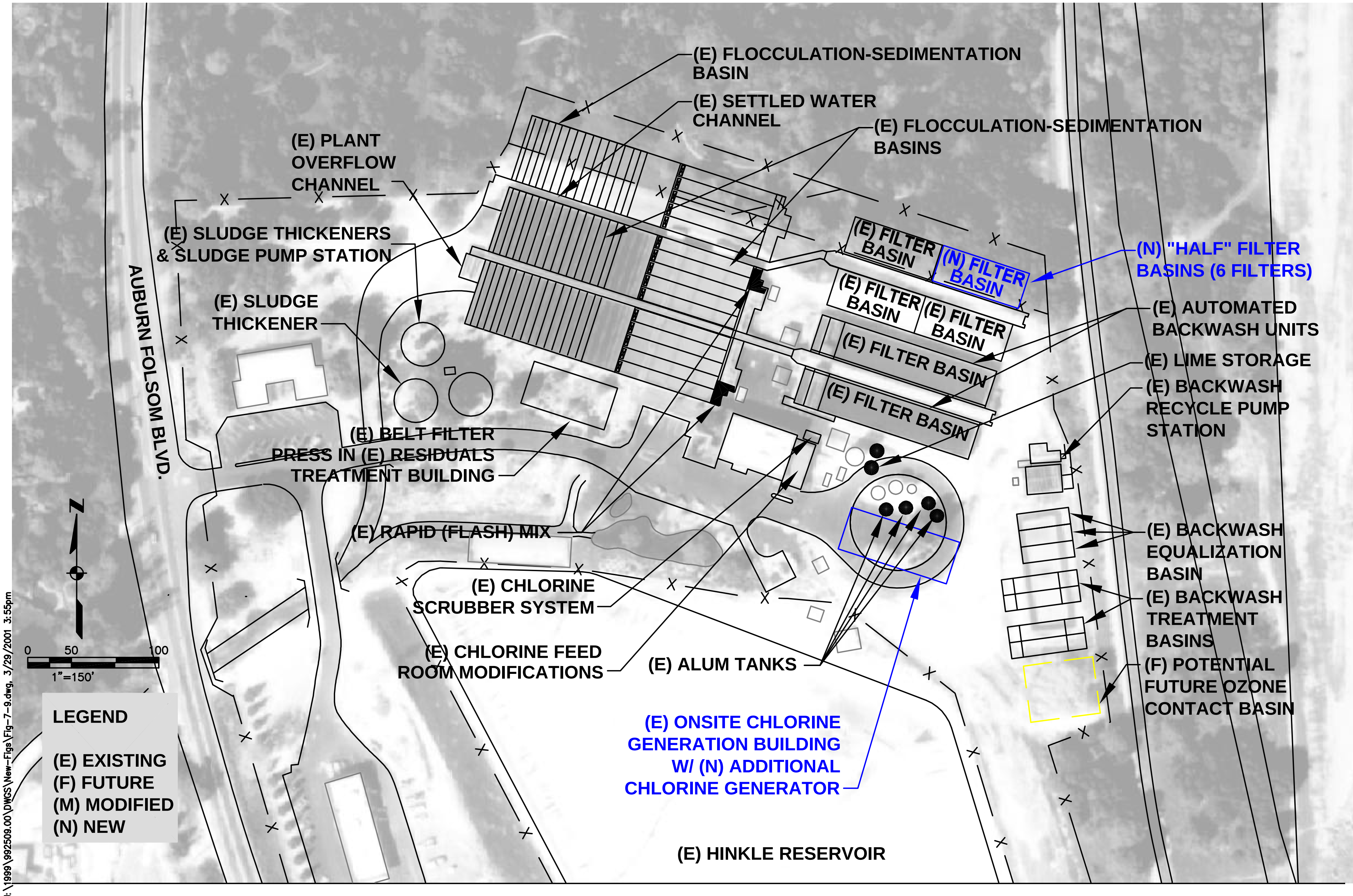


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FIGURE 7-9
RECOMMENDED IMPROVEMENTS FOR LT 120/240 EXPANSION PHASE 4 (120/240 MGD)

Rapid Mix (Coagulation). Similar to LT 75/150, flash mixing using a pump jet-injection mixing system is recommended for coagulation for plant capacities above 120 mgd.

One pump jet unit would be required for each coagulation train. Additional jet pumps for the flash mix unit will be required to increase flash mixing energy if flow through any rapid mix basin were to exceed approximately 90 mgd.

Pretreatment (Flocculation-Sedimentation)

As discussed under LT 75/150, three alternative approaches to increasing the treatment capacity of the flocculation-sedimentation pre-treatment processes were evaluated as part of the Master Plan. The alternatives included combinations of constructing a third rapid mix (coagulation)-flocculation-sedimentation treatment train parallel to the existing two pretreatment trains, and/or constructing modifications to the existing two pretreatment trains to gain capacity. The evaluation of alternatives is presented in Appendix 7-2. They are summarized as follows:

- Alternative 1: Modify existing pretreatment basins to correct deficiencies and increase conventional treatment capacity to approximately 50 mgd for each basin. Initial total conventional treatment capacity will be 100 mgd. When capacity needs increase, construct third pretreatment basin similar to the existing two, for a total conventional treatment capacity of 150 mgd.

- Alternative 2: Construct a third rapid mix-flocculation-sedimentation basin with a conventional treatment capacity of 60 mgd. Replace equipment in the existing pretreatment basins to correct deficiencies. Conventional treatment capacity of the existing pretreatment basins will remain 30 mgd each. Total conventional treatment capacity will be 120 mgd.

- Alternative 3: Modify the existing pretreatment basins to increase capacity of each to 60 mgd, for a total conventional treatment capacity of 120 mgd.

The costs for the three alternatives are summarized in Table 7-6.

Table 7-6
Conceptual Level Capital Cost Estimate
Flocculation - Sedimentation Basin Comparisons

Alternative	LT 120/240 WTP Capacity Expansion, mgd				Total Cost
	120-150	150-180	180-210	210-240	
Alternative 1 Replace equipment in existing basin. Add 3rd basin in future					
Coagulation, rapid mix	$240,000	$110,000	$439,000		$789,000
Modify Flocculation Basin	1,088,000				1,088,000
Modify Sedimentation Basins	3,086,000				3,086,000
Additional effluent & equalization channels	1,919,000				1,919,000
Third Basin w/4' tube settlers			4,534,000		4,534,000
Instrumentation & Electrical	760,000	14,000	597,000		1,371,000
Contingency @ 25%	1,773,250	31,000	1,392,500		3,196,750
Total	**$8,866,250**	**$155,000**	**$6,962,500**		**$15,983,750**
Alternative 2 Add Third Basin w/4'tube settlers and min. mods. Exist. Basins now.					0
Coagulation for new basin	$439,000				$439,000
Flocculation & Sedimentation Basin, 4'plates	4,534,000				4,534,000
Modify exist floc basin, walls, floc equip.	932,000				932,000
Modify exist set basin, launders & tubes	2,594,000				2,594,000
Modify Coagulation to exist basins	240,000	110,000			350,000
Additional effluent & equalization channels	1,919,000				1,919,000
Instrumentation & Electrical	1,279,000	14,000			1,293,000
Contingency @ 25%	2,984,250	31,000			3,015,250
Total	**$14,921,250**	**$155,000**			**$15,076,250**
					0
Alternative 3 Demo & Build within Existing Flocculation - Sedimentation Basins, Install 14' plate settlers					0
Coagulation	$240,000	$110,000			$350,000
Modify Flocculation Basin	1,788,000				1,788,000
Modify Sedimentation Basins	11,752,000				11,752,000
Additional effluent & equalization channels	1,919,000				1,919,000
Instrumentation & Electrical	1,884,000	14,000			1,898,000
Contingency @ 25%	4,395,750	31,000			4,426,750
Total	**$21,978,750**	**$155,000**			**$22,133,750**

Based on a review of the alternatives with the District at a workshop on January 30, 2001, the selected approach for increasing sedimentation capacity for LT 120/240 is Alternative 2, constructing a third pretreatment train and correcting deficiencies in the existing pretreatment trains. This alternative has the lowest total capital cost, offers the highest level of redundancy and reliability, and offers the District flexibility to increase conventional treatment capacity in the future if necessary by further modifying the existing pretreatment basins.

Under this approach to increasing pre-treatment capacity, a third rapid mix-flocculation-sedimentation train would be constructed to the north of the existing northern rapid mix-flocculation-sedimentation train as part of the initial 30 mgd LT 120/240 plant improvements. The design of the new rapid mix-flocculation-sedimentation train would be different than the design of the two existing basins to provide at least 60 mgd of additional rapid mix-flocculation-sedimentation capacity. This approach would not require modifying the existing sedimentation basins to increase conventional filtration treatment pretreatment capacity. However, new horizontal paddle flocculators and redwood baffles would be installed in the existing flocculation basins to improve direct filtration treatment performance.

Recommended improvements for Alternative 2 are summarized as follows:

- A new pretreatment train consisting of a third rapid mix, flocculation, and sedimentation basin constructed on the north side of the two existing rapid mix, flocculation, and sedimentation basins. The third pretreatment train would have 4-foot tube or plate settler modules to provide a capacity of 60 mgd for the third basin. This would provide a total conventional filtration treatment pretreatment capacity of at least 120 mgd with all three flocculation-sedimentation basins in service.

- A new settled water conveyance channel on the north side of the two existing rapid mix, flocculation, and sedimentation basins between the existing and new pretreatment basins. The channel should be sized to provide additional hydraulic capacity to at least 240 mgd to accommodate initial and future conventional and direct filtration treatment capacity requirements.

- A new jet mix coagulation system to replace the existing rapid mix coagulation system in the existing pretreatment trains.

- New horizontal paddle flocculators to replace the existing flocculation basin horizontal turbines. The horizontal paddle flocculators would be designed to provide higher mixing energies than the existing flocculators to form small filterable pin floc during the summer when source water turbidity is low and conventional filtration is not required.

- New redwood walls between each of the five parallel flocculation trains in the existing pretreatment basins to improve flocculation performance.

- A new perforated flow distribution wall between each flocculation basin and the adjacent sedimentation basin similar to the existing perforated walls between existing flocculation zones 1 and 2 and zones 2 and 3.

- New 2-feet deep tube settler modules to replace the existing sedimentation basin 2-foot deep tube settler modules. (The existing modules are brittle and near the end of their useful life.)

- New 24-inch by 24-inch launders to replace the existing 18-inch by 21-inch launders to improve the hydraulics in the existing pretreatment basins.

Filtration. The filtration alternatives considered for LT 75/150 apply to LT 120/240. Similar to LT 75/150, the initial expansion would include a filter basin with two 30-mgd halves constructed on the north side of the two existing basins. The new filter basin should be provided with a divider wall between each group of six filters to permit removing as few as six filters (30 mgd of capacity) from service for maintenance at any time. Two filter backwash units should be provided with the new basin.

Beyond a WTP capacity of 150 mgd, 30-mgd filter basins similar to the existing basin would be constructed in phases. Thus, filtration capacity would increase from 120 to 150 mgd, from 150 to 180 mgd, from 180 to 210 mgd, and from 210 to 240 mgd. A filter backwash unit would be required for each 30 mgd of filter capacity in order to maintain reliability and redundancy.

Disinfection. For the initial phase of expansion for LT 120/240, it is recommended that free chlorine, from chlorine gas, continue to be the primary disinfectant at the WTP similar to the discussion for LT 75/150. However, as WTP capacity increases beyond 150 mgd, the need for additional one-ton chlorine container storage and changeout will increase. Safety concerns and code requirements may require the District to change to a bulk hypochlorite storage or onsite sodium hypochlorite generation facility.

Bulk liquid sodium hypochlorite is most commonly used and is generally safer than chlorine gas. However, hypochlorite solution is a severe irritant, corrodes ferrous metals, and disintegrates concrete. The solution decomposes as a function of time, temperature, and concentration and produces off-gases which can affect pump operation. In addition, future regulations for chlorite and chlorate may impact bulk hypochlorite users. Storage and maintenance is more expensive than chlorine gas. There are also public health concerns with bromate present in commercial hypochlorite. For large facilities, the O&M cost for hypochlorite can be two to three times the cost of using gaseous chlorine on an annualized cost basis.

Onsite hypochlorite generation systems are a proven technology in Europe and are becoming more common and cost effective in the United States. The only raw materials used are bromide-free, food grade table salt and hardness-free water. Hypochlorite is only generated as required, which minimizes decomposition losses and allows for closer residual control. The process produces a product solution that is very dilute (less than one percent). The units are relatively simple, compact, and automated. Hydrogen gas is produced as a byproduct, but, in a properly designed system, can be safely discharged to the atmosphere.

For the initial 150-mgd expansion phase, the District should consider reserving space for a future onsite hypochlorite generation facility. In this scenario, the 180-mgd expansion would phase out chlorine gas, and the initial onsite hypochlorite generation facilities would be constructed. These facilities would be expanded in subsequent WTP capacity expansions.

As discussed under LT 75/150, future regulations may require *Cryptosporidium* inactivation. The LT 120/240 scenario must also provide for the possible addition of ozone, chlorine dioxide, or UV facilities for *Cryptosporidium* disinfection.

Disinfection Contact Time. Disinfection CT credit used to meet *Giardia* and virus inactivation requirements were discussed in LT 75/150. Table 7-4 presented the disinfection CT required and the existing disinfection CT available when the WTP operates in various treatment modes. For LT 120/240, with <u>three</u> pretreatment basins in service and the plant operating in a direct filtration mode, there is sufficient disinfection CT up to a WTP capacity of approximately 180 mgd. In an in-line filtration treatment operating mode, there is only sufficient disinfection CT credit available up to a capacity of about 60 mgd. Above these capacities, chlorine residual through the WTP may need to be increased or the Cooperative Transmission Pipeline connection should be relocated to allow additional disinfection CT credit through Hinkle Reservoir. This is discussed in more detail in Chapter 8.

Backwash Water Recovery System. The LT 75/150 discussion for backwash water recovery applies to the first phase, 150 mgd expansion of the WTP. An additional equalization basin, for a total of three, and two additional treatment modules (over the initial two) would be required for a WTP capacity above 180 mgd. Ultimately, four 3-mgd treatment modules with a combined capacity of 12 mgd, or 5 percent of total plant production, would be required for a WTP capacity of 240 mgd.

Residuals Handling. The approach to residuals handling for LT 120/240 would be identical to that for LT 75/150. A third sludge thickener would be required for WTP capacities above 180 mgd. Similar to LT 75/150, a new residuals handling building would be constructed for the 150-mgd phase of LT 120/240 to house belt filter presses for solids dewatering. However, the building would be approximately 5,000 square feet, large enough to accommodate all ultimately required belt filter presses and chemical feed equipment.

Three 2-meter belt filter presses would be provided for the initial expansion, two duty and one standby unit. Up to a maximum of four belt filter presses would be required in the residuals handling building for WTP expansions to 240 mgd.

Chemical Feed Systems. The existing chemical feed systems, as discussed in LT 75/150, will be expanded in phases to accommodate the increased flows. Additional bulk storage tanks will be required, as shown in Table 7-3.

Adding a third flocculation-sedimentation basin would require adding a fourth coagulant (alum) metering pump and additional non-ionic polymer feed pipelines, but it would not require replacing the three existing alum metering pumps when plant capacity exceeds 150 mgd.

Additional Site Improvements. Similar site improvements to those described in LT 75/150 would be required for LT 120/240. However, during the initial 150-mgd expansion phase, piping, channels, and other plant hydraulic elements would need to be sized for an ultimate capacity of 240 mgd. Additional space would have to be reserved for equipment, tanks, and other support facilities.

7.5 Recommended Improvements and Costs

Table 7-7 presents the recommended improvements for LT 75/150 and LT 120/240 and their associated costs. The estimated capital costs are conceptual level estimates prepared without plans and specifications and actual quantity take-off. The estimates were prepared based on prior bid results, standard estimating guide cost curves, equipment quotes from suppliers, and engineering judgment. The estimates are based on an Engineering News Record (ENR) Construction Cost Index of 6281 (in effect January 2001), and include 25 percent contingencies to provide for reasonable estimating and construction uncertainties.

Table 7-7
Conceptual Level Estimate of Capital Costs
LT 75/150 and LT 120/240 Water Treatment Plant Expansions

Direct Filtration Treatment Capacity Expansion, MGD	LT 75/150	LT 120/240				LT 120/240
	120-150	120-150	150-180	180-210	210-240	TOTAL ($)
Coagulant Flash Mix System	240,000	679,000		138,000		817,000
Flocculation Basin Modifications, Option 1	1,088,000					0
Sedimentation Basin Modifications, Option 1	4,174,000					0
Flocculation Basin Modifications, Option 2		932,000				932,000
Sedimentation Basin Modifications, Option 2		2,594,000				2,594,000
New Floc/Sedimentation Basins, Option 2		5,906,000				5,906,000
Filter Improvements						0
Filter Backwash Hoods	2,200,000	2,200,000				2,200,000
Filter To Waste Piping	726,000	726,000				726,000
New Filter Units	6,434,000	6,434,000		3,853,000	3,641,000	13,928,000
Backwash Recovery System						0
Demolish Existing (includes temp. system)	250,000	250,000				250,000
New System Basins and Equipment	2,137,000	2,137,000	995,000		995,000	4,127,000
New Polymer Feed and Control Building	150,000	150,000	50,000		50,000	250,000
Residuals Treatment						0
Sludge Thickeners	700,000	750,000		400,000		1,150,000
Belt Presses and Related Equipment	2,150,000	2,150,000		896,000		3,046,000
Belt Filter Press Building	942,000	1,440,000				1,440,000
Chemical Feed Systems:						0
Chlorine System (Structure and Scrubber)	500,000	500,000	50,000			550,000
On-Site Generation Chlorine System			2,331,000	615,000	615,000	3,561,000
Alum System	175,000	175,000	80,000			255,000
Coagulants (polymers)	160,000	160,000				160,000
Lime System	300,000	300,000	25,000			325,000
Ozone or UV Systems						
Administration Building	100,000	100,000				100,000
Sitework	250,000	250,000	100,000	100,000	100,000	550,000
Yard Piping/Channels	1,450,000	1,750,000	500,000	250,000	250,000	2,750,000
Instrumentation	1,810,000	2,219,000	310,000	469,000	424,000	3,422,000
Electrical	3,620,000	4,438,000	620,000	938,000	848,000	6,844,000
Subtotal	$29,556,000	$36,240,000	$5,061,000	$7,659,000	$6,923,000	$ 55,883,000
Contingency @ 25%	7,389,000	9,060,000	1,265,250	1,914,750	1,730,750	13,970,750
Engineering, Legal, and Administrative @25%	7,389,000	9,060,000	1,265,250	1,914,750	1,730,750	13,970,750
Totals	**$44,334,000**	**$54,360,000**	**$7,591,500**	**$11,488,500**	**$10,384,500**	**$ 83,824,500**

Chapter 8: Hinkle Reservoir

8.1 Introduction

Hinkle Reservoir is the final component of the District's water supply and treatment system. It is a 62-million-gallon (MG) lined and covered earthen reservoir that acts as the clearwell for treated water from the WTP as well as a facility for system storage. Water stored in Hinkle Reservoir flows by gravity to the District's wholesale customers and a portion of its retail service area. Additional water is pumped to the remainder of the retail service area and part of the City of Folsom.

Hinkle Reservoir. The District's WTP can be seen to the left. Folsom Reservoir is at the top left of the photograph.

The scope of this Master Plan included evaluating the condition of the Hinkle Reservoir cover system, evaluating options for extending the life of the cover or replacing it if necessary, and evaluating the potential of the reservoir to improve the WTP's ability to comply with disinfection CT requirements and treated water storage goals.

Table 8-1 presents a summary of the evaluations and findings presented in this chapter. All recommended actions for the District covered in this chapter are included in this table.

8.2 Reservoir Cover Evaluation

8.2.1 Background

Construction of the floating membrane cover system on the Hinkle Reservoir was completed in 1980. The cover is guaranteed for a period of 25 years. Since it is now over 20 years old, the District is concerned with the remaining life of the cover and what alternatives should be considered when the cover needs replacing. An evaluation of the cover was performed in order to provide recommendations for extending the life of the cover, or recommend options to replace the cover if it is nearing the end of its service life.

According to District records, the cover is composed of 45 mil (1.14mm) thick chlorosulfonated polyethylene (CSPE), also known as Hypalon. The Hypalon cover is internally reinforced with two plies of scrim (a durable, woven fabric) sandwiched between three layers of Hypalon resulting in a

Table 8-1
Recommended Hinkle Reservoir Improvements Summary

Component Description	Recommended Short Term Improvements	Recommended Long Term Improvements
Hinkle Reservoir Cover	Conduct a comprehensive 20-year maintenance cleaning. Inspect and replace existing patches as necessary. Patch any un-repaired damage. Inspect and clean every factory and field seam of debris. Inspect and repair all perimeter attachments, structure attachments, and hatch covers as necessary. Clean and flush sump drain pipe headers. Inspect and repair sand ballast tubes. Remove and flush algae growth with a chlorine solution. Redesign/replace the Hypalon cover at the inlet and outlet structure to properly accommodate cover movement and eliminate the un-drained sump. Remove and replace the caulking around the entire perimeter. Add supplemental weights to areas requiring better tensioning and improved drainage to reduce ponding rainfall. Remove trapped air by 'walking' to the hatches.	Conduct a comprehensive 20-year maintenance cleaning no more frequently than once every two years. (More frequent cleaning is not recommended due to the increased potential for mechanical damage to the cover.) Modify the reservoir to provide two separate treated water storage sections to increase redundancy and add reliability features to the treated water supply. When required, replace the existing Hinkle Reservoir cover with a similar Hypalon floating cover system.
Cooperative Transmission Pipeline Connection	***If petitioning to DHS for in-line filtration treatment approval***, relocate the cooperative pipeline treated water connection with a direct pipeline connection between the existing 78-inch Cooperative Transmission Pipeline and the existing 84-inch reservoir outlet pipeline to obtain additional disinfection CT.	Relocate the cooperative pipeline treated water connection with a direct pipeline connection between the existing 78-inch Cooperative Transmission Pipeline and the existing 84-inch reservoir outlet pipeline when either: • WTP capacity exceeds 130 mgd and only two flocculation-sedimentation basins are in service; • WTP capacity exceeds 180 mgd and three flocculation-sedimentation basins are in service.

five-ply construction. The top surface is colored a tan or earth tone, and the underside is black. The original Hypalon roll goods were manufactured by Burke Rubber Company, San Jose, California. The roll stock was fabricated into panels, delivered to the site, and field assembled into the reservoir liner and floating cover system. The Hinkle Reservoir cover was the first to use a self draining design where rainwater is removed through flexible hoses fitted with penetration fittings at the Hypalon cover pipe manifold (bottom of sumps) and at the base of the reservoir.

8.2.2 Initial Field Inspection and Cover Evaluation

A site visit and initial inspection was conducted on September 13, 1999 to inspect the cover and evaluate its general condition. The site visit included meeting with Mr. Michael J. O'Bleness, the Water Quality Manager, and Mr. Joe Batt, Lead Worker. Cover history, maintenance procedures, and known problem areas associated with the Hypalon cover system were discussed. In addition, maintenance inspection procedures and forms, original construction drawings, O & M manual, and underwater photos of the cover system and drain pipe connections were reviewed. A complete report of the initial inspection and cover evaluation is provided in Appendix 8-1.

8.2.2.1 Hypalon Cover General Condition

The initial inspection indicated the condition of the Hypalon cover system is very good considering its 20-year plus life and constant exposure to the elements. The tan surface exhibits surface oxidation, surface crazing (near surface cracking), stiffening (surface hardening), and general aging, characteristics typical of Hypalon. However, Hypalon polymer typically becomes stronger with age due to continued cross-linking of the polymer. Other than discoloration and distortion at factory seam areas (over water surface only), there were no obvious surface areas that exhibited deterioration. The only damage noted was due to mechanical puncture at the upper slope surface and broken or split sand ballast tubes. There were no major distorted or wrinkled areas other than stressed areas at the slope where the sumps and weights rest on the slopes. A dark gray discoloration was observed on the north end of the cover that may be attributed to standing water over time.

The initial inspection report and cover evaluation, included as Appendix 8-1, provides specific findings and recommendations for cover maintenance and repairs. These are summarized below.

8.2.2.2 Initial Inspection Findings and Recommendations

<u>Findings:</u>

The initial inspection determined that there was no apparent reason that the cover material, seams, and associated attachments would not provide an additional five years minimum of service life, the approximate remaining warranty period. However, to determine a more realistic projected life expectancy for the existing cover system, the initial inspection recommended that samples of the cover be extracted to help determine the aged physical/mechanical properties and percent change in properties of the cover after almost 20 years of service. A complete testing program is outlined in the cover evaluation report contained in Appendix 8-1.

In addition to the physical/mechanical properties test program and evaluation, the initial inspection also determined that a thorough 20-year comprehensive inspection and maintenance cleaning/repair of the cover system should be completed. This determination was based on the following observations:

- A number of old patches were observed to be loose, un-bonded, or easily lifted from the cover surface.

- Some minor un-repaired damage was noted on the cover system at the top of slope on the east side of the reservoir.

Sumps within the Hinkle Reservoir cover should be cleaned of all debris and algae.

- Several sections of the rainwater collection sumps were full of water at the time of inspection. The sumps should have been fully drained at the time of the inspection. Water in the sumps may be an indication of debris or biological growth clogging or blocking the header drain pipes.

- Green algae was covering some of the rainwater collection channels. Algae will attack and distort the surface if allowed to remain and dry on the cover surface.

- Sand ballast tubes were damaged at two areas on the upper slope/channel connections at the south end of the reservoir.

- Accumulated dirt, dust, and small debris were collecting in the factory seam channels that have formed on the surface over water areas.

- A significant amount of surface water was present at the outlet structure due to reported leaks in this area.

<u>Recommendations:</u>

Several other recommendations resulted from the initial inspection of the cover system and are summarized as follows:

- All existing patches should be inspected and replaced as necessary, and any un-repaired damage should be patched. The original cover manufacturer, Burke Rubber Company, should be contacted for current repair procedures and materials recommendation. They should also be contracted for on site instruction in repair of old Hypalon.

- Every factory and field seam should be inspected and cleaned of debris.

- All perimeter attachments, structure attachments, and hatch covers should be inspected and repairs made as necessary.

- The sump drain pipe headers must be cleaned and flushed.

- All sand ballast tubes should be inspected and repaired.

- Algae growth should be immediately removed and flushed with a chlorine solution.

The current maintenance inspection program and reporting forms are acceptable. It is important that daily visual perimeter observations be continued and that the weekly cover inspection and recorded observations and repairs be kept current. The weekly inspections should be augmented with a thorough yearly detailed inspection of all cover areas, hatches, connections, and sumps. A yearly underwater inspection program is currently being accomplished for all underwater connections and is recommended to be continued for future inspections. The top cover inspection should be completed in concert with the underwater inspection.

Once the 20-year inspection and cleaning is complete, it is not recommended to clean the surface of the cover more than once every two years. More frequent cleaning is not recommended due to the increased potential for mechanical damage. Because access to the reservoir is controlled by fencing, the site has 24-hour operations personnel present, and air blown debris is limited to fine material, the potential for damaging objects or material accumulation on the cover is small.

8.2.3 Cover Sampling and Testing and 20-Year Inspection

Subsequent to the initial inspection, the District authorized the extraction and testing of samples to help determine the aged physical/mechanical properties and percent change in properties of the cover. The District also authorized a thorough inspection of the entire cover and test cleaning of a limited portion of the cover. This work was completed in October and November of 2000, when lower system demands allowed the reservoir to be drawn down to approximately 8 feet.

The inspection and testing included a thorough physical assessment of the condition of the cover, the collection of material samples from the cover, laboratory testing of the samples, test cleaning of a small portion of the cover, and preparation of a summary report. The physical inspection and sampling was completed by Colorado Linings International (CLI) under contract to the District.

Four coupons were cut from the cover for materials testing. These samples were analyzed by the Burke Rubber Company, supplier of the original Hypalon cover material, and Precision Geosynthetic Laboratories, an independent third party laboratory. The test results and inspection report were evaluated by Mr. Ron Frobel of R.K. Frobel & Associates (RKF), a recognized expert in Hypalon materials and membrane systems. The complete CLI inspection report and RKF summary report is provided in Appendix 8-2. The findings and recommendations are summarized as follows:

- The Hypalon floating cover system is in very good condition. Laboratory testing of the extracted samples indicate that the cover material, seams, and associated attachments appear to have a minimum remaining service life of 20 years with proper maintenance. A comparison of material properties with typical average property values for Hypalon manufactured by Burke Rubber Company when the Hinkle Reservoir liner and cover were installed generally show an increase in tensile, burst, and seam strength, with a subsequent decrease in elongation properties.

- The detailed inspection identified the location of 60 to 70 small holes or failing repairs (patches). All holes and failing repairs should be patched using the recommended procedure described in the CLI report.

All holes and failing patches should be repaired.

- Perimeter edge caulking has cracked and pulled away from the concrete edge beam at the top of the slope. This may allow water to seep under the edge and into the reservoir. Caulking should be removed and replaced around the entire perimeter.

- Several areas would benefit from supplemental weights to provide better tensioning and to improve drainage to reduce ponding rainfall. Thirty additional weights were provided as part of the inspection and sampling contract and could be used for this purpose. Supplemental weights should be placed near the northeast sump and other areas identified following rainfall events.

- Trapped air exists under the cover and can allow the cover to lift and tear during high wind events. Trapped air should be 'walked' to the hatches.

- The inlet and outlet structure geometry creates areas of significant stress in the Hypalon material. The geometry also creates an undrained sump which collects debris and supports biological growth. The Hypalon cover at these structures should be redesigned and replaced to properly accommodate cover movement and eliminate the undrained sump.

The Hypalon cover at the outlet structure should be redesigned to eliminate the undrained sump.

- A comprehensive 20-year maintenance cleaning should be completed with subsequent periodic cleaning no more frequent than once every two years. More frequent cleaning is not recommended due to the increased potential for mechanical damage to the cover.

- Updated AWWA recommendations for inspection and reporting (April 1999) should be reviewed and selectively incorporated into the District's maintenance program as appropriate.

A 50-foot test section of the rainwater drainage sump was cleaned to determine the level of effort required to remove accumulated debris and to estimate the volume of material present in the sump. The total length of sump is 1,950 feet. The reservoir was drawn down to approximately 8 feet to allow access to the northwestern reach of the sump. This was the only reach of the sump exposed at the 8-foot level. The reservoir will need to be drawn down several more feet to provide similar access to the rest of the sump when the 20-year maintenance cleaning is completed.

The test cleaning indicated that the entire sump contains a substantial volume (an estimated 10 plus cubic yards) of debris consisting of dirt, pine needles, and leaves. The contractor was able to clean the Hypalon with a moderate effort using a mild soap and brushes. Given proper access, the sump should clean up with moderate effort during the 20-year maintenance cleaning.

A budget level cost estimate was provided by CLI to complete the repairs and cleaning outlined above. Their estimated cost is $200,000.

8.2.4 Replacement Alternatives

Although the existing Hinkle Reservoir flexible liner and floating membrane cover system appears to have a minimum 20-year life expectancy with proper maintenance, the District should begin planning its replacement now. This includes planning for the capital cost of replacement as well as developing an alternative water supply plan during the period when the reservoir will be out of service for construction. This Master Plan evaluated two options for replacing the existing Hinkle Reservoir flexible liner and floating membrane cover system: a new flexible membrane system and a rigid roof system.

The existing reservoir is configured as a single 62 MG storage reservoir. This does not permit continued delivery of treated water from the reservoir during periods when maintenance and cleaning activities must be conducted. The DHS recommends that Hinkle Reservoir be divided to permit taking one side off-line for cleaning and other maintenance activities while maintaining the other half in service. Dividing the existing reservoir into two sections will result in redundancy and add reliability features to the treated water supply. The cost for dividing the reservoir was included in each alternative evaluated.

8.2.4.1 Flexible Membrane System

This alternative would consist of replacing the Hypalon cover with a similar cover when the need arises. Although DHS has general concerns with the integrity and health protection aspects of floating membrane covers, the Hinkle Reservoir has a flawless track record of reliable service. This is due not only to the performance of the Hypalon material and cover design, but also to the security of the site, regular monitoring, and maintenance of the cover. With the additional improvements in the monitoring and maintenance program recommended above, there is no reason to believe a replacement Hypalon cover would not provide the reliable service required by the District.

Our evaluation of this replacement alternative assumed the existing reservoir liner would remain in place. The existing liner is not under the tension and stress the cover is subjected to as the reservoir water level rises and falls. Also, it is not subjected to the effects of ozone and UV radiation from continuous exposure to sunlight and the environment. Photomicrographic examination of the underside of the extracted cover samples indicated the Hypalon surface and cut sections showed no surface deterioration. This is a good indication of the current material condition and its ability to resist significant degradation. Therefore, it is estimated that the liner should have a remaining life more than two times that of the cover.

The cost of a replacement Hypalon cover system is shown in Table 8-2. The cost includes a lined berm to create two sections of reservoir, a new (second) reservoir inlet and outlet structure for the new reservoir section, and interconnecting piping and valves to isolate one section from the other during maintenance. Each section would be baffled to promote plug flow and improve disinfection CT credit through the reservoir. The north section of the reservoir could be constructed first and brought back on-line to limit the reservoir outage period to approximately 1½ to 2 months.

Table 8-2
Conceptual Level Capital Cost Estimate
Hinkle Reservoir Replacement Hypalon Cover System

Item	Capital Cost
Mobilization, demobilization, and set-up costs	$100,000
Demolish existing cover	195,000
Earthwork (Reservoir divider)	400,000
Liner repair and modifications	150,000
Cover	1,150,000
Interior baffles	60,000
Outlet structure	35,000
Inlet structure	30,000
60-inch inlet piping, 84-inch outlet piping, valves, and appurtenances	980,000
Drain system	50,000
Site restoration	20,000
Subtotal	**$3,170,000**
Contingency @ 25%	792,500
Engineering, legal, administrative @ 25%	792,500
Total	**$4,755,000**

8.2.4.2 Rigid Roof System

Replacing the existing Hypalon cover with a rigid roof system could be accomplished with steel, concrete, or fiberglass. However, previous studies of lifecycle costs for large reservoir systems (those greater than about 10 MG) have demonstrated that reinforced concrete structures are much more economical. The regular painting, corrosion protection, and general maintenance requirements associated with systems other than concrete make them unattractive for a reservoir the size of Hinkle.

A rigid roof system would require columns to support the roof structure. Typical column spacing is estimated to be approximately 24 feet on center. Because of the extensive work within the reservoir and the requirement for column footings, it is assumed the existing lined bottom would be removed and the bottom of the reservoir would be constructed of reinforced concrete. It is also estimated the reservoir would be off-line for a minimum of 10 to 12 months before at least one half of the reservoir could be reconstructed and utilized for storage.

The cost of a concrete reservoir system is shown in Table 8-3. The cost includes a concrete divider wall between two sections of reservoir, a new (second) reservoir inlet and outlet structure for the new reservoir section, and interconnecting piping and valves to isolate one section from the other during maintenance. A Hypalon or membrane fabric curtain wall would be used in each section of the reservoir to provide baffling and improved disinfection CT credit.

Table 8-3
Conceptual Level Capital Cost Estimate
Hinkle Reservoir Replacement Concrete Liner and Cover System

Item	Capital Cost
Mobilization, demobilization, and set-up costs	$500,000
Demolish existing cover and liner	400,000
Earthwork (Foundation preparation)	250,000
Concrete base slab	3,900,000
Suspended Roof Slab	10,225,000
Side walls	3,550,000
Center wall	225,000
24-inch diameter columns @ 24-feet cc	7,400,000
Outlet structure	35,000
Inlet structure	30,000
60-inch inlet piping, 84-inch outlet piping, valves, and appurtenances	980,000
Hatches, vents and accessories	50,000
Site restoration	20,000
Subtotal	**$27,565,000**
Contingency @ 25%	6,890,000
Engineering, Legal, Administrative @ 25%	6,890,000
Total	**$41,345,000**

8.2.4.3 Recommended Reservoir Replacement

On the basis of the proven performance of the existing cover and a comparison of alternative costs, a Hypalon floating cover system is recommended for the Hinkle Reservoir when cover replacement is required. The recommended improvements are shown on Figure 8-1.

Neither cost estimate for the considered alternatives included a cost factor for implementing an alternative water supply plan during reservoir construction activities. A portion of the base water supply could be provided directly through the reservoir bypass pipeline. The District would need to explore the possibility of meeting the remaining water demand through wells and agency water system interties. Careful planning, cooperative agreements, and public notification will be necessary during the reservoir outage.

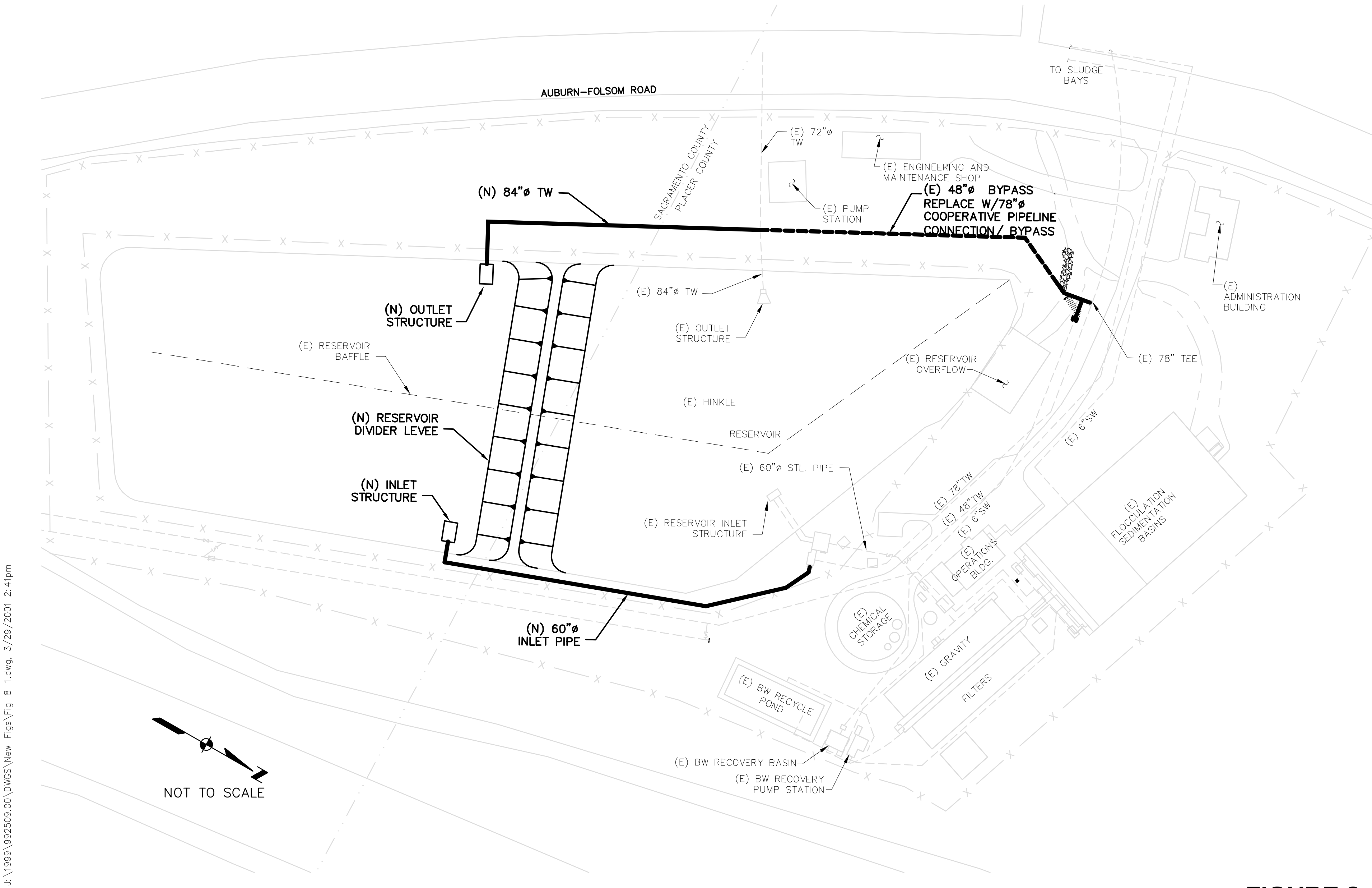

FIGURE 8-1

HINKLE RESERVOIR IMPROVEMENTS

As mentioned previously, the cost for bifurcating the reservoir was included in each cover replacement alternative evaluated. However, the District may wish to consider completing this improvement prior to replacing the reservoir cover system given the remaining life of the cover. As demands within the District service area increase, it may become increasingly difficult to take the entire reservoir out of service. If designed properly, dividing the reservoir into two sections initially should allow only taking one-half of the reservoir out of service at a time when cover replacement becomes necessary.

8.3 Cooperative Transmission Pipeline Connection

The District's WTP currently operates in compliance with the DHS 3-log *Giardia* and 4-log virus removal-inactivation requirement for surface water supplies using a multi-barrier combination of physical removal and disinfection. The plant currently receives 2.5-log *Giardia* and 2-log virus removal credit when it operates in a conventional filtration mode and receives a 2.0-log *Giardia* and 1-log virus removal credit when it operates in a direct filtration mode. Disinfection is used to meet the remaining inactivation requirement.

Since a portion of the treated water currently bypasses Hinkle Reservoir through the Cooperative Transmission Pipeline, most of the disinfection credit must be achieved ahead of Hinkle Reservoir as the water flows through other treatment units. (A small amount of disinfection credit is received in the 78-inch Cooperative Transmission Pipeline before the first service connection.) Therefore, the disinfection credit needed to comply with the required combination of 3-log *Giardia* and 4-log virus removal-inactivation is achieved by maintaining an adequate chlorine residual in the water as it flows through the two existing flocculation and sedimentation trains and two filter basins.

As discussed in Chapter 7, as WTP capacity increases, the available disinfection CT will not be sufficient to meet that required. Table 8-4 shows the disinfection CT required and the existing disinfection CT available when the WTP operates in various treatment modes. To meet future disinfection credit requirements, the Cooperative Transmission Pipeline connection will need to be moved to the reservoir outlet pipe or to a new outlet structure located to ensure CT credit through the reservoir.

Table 8-4
San Juan Water District Water Treatment Plant
Disinfection Requirements and Existing Available Disinfection CT

Plant Flow Rate (mgd)	WTP Operating Classification	Disinfection Required[d] (mg/L-min.)	Disinfection Contact Time Required (Min.)	Available Contact Time[e] [f] [g] (Min.)
60	Conventional[a]	18	22.5	116
60	Direct Filtration[b]	37	46	116
60	In-line Filtration[c]	37	46	45
120	Conventional[a]	18	22.5	53
120	Direct Filtration[b]	37	46	53
120	In-line Filtration[c]	37	46	23

Table 8-4 (cont.)
San Juan Water District Water Treatment Plant
Disinfection Requirements and Existing Available Disinfection CT

Plant Flow Rate (mgd)	WTP Operating Classification	Disinfection Required[d] (mg/L-min.)	Disinfection Contact Time Required (Min.)	Available Contact Time[e] [f] [g] (Min.)
150	Conventional[a]	18	22.5	55
150	Direct Filtration[b]	37	46	55
150	In-line Filtration[c]	37	46	20
180	Conventional[a]	18	22.5	47
180	Direct Filtration[b]	37	46	47
180	In-line Filtration[c]	37	46	17
210	Conventional[a]	18	22.5	41
210	Direct Filtration[b]	37	46	41
210	In-line Filtration[c]	37	46	16
240	Conventional[a]	18	22.5	36
240	Direct Filtration[b]	37	46	36
240	In-line Filtration[c]	37	46	14

(a) Conventional Treatment receives 2.5-log *Giardia* removal credit and 2.0-log enteric virus removal credit. Disinfection CT for the remaining 0.5-log *Giardia* inactivation is the controlling condition for treated water pH between 6 and 9 and a water temperature as low as 10°C.
(b) Direct Filtration Treatment receives 2.0-log *Giardia* removal credit and 1.0-log enteric virus removal credit. Disinfection for the remaining 1.0-log *Giardia* inactivation is the controlling condition for treated water pH between 6 and 9 and a water temperature as low as 10°C.
(c) In-line Filtration Treatment receives 2.0-log *Giardia* removal credit and 1.0-log enteric virus removal credit. Disinfection for the remaining 1.0-log *Giardia* inactivation is the controlling condition for treated water pH between 6 and 9 and a water temperature as low as 10°C.
(d) Disinfection requirement based on a chlorine residual concentration of 0.8 mg/L and a pH of 7.0.
(e) Available CT based on 2 flocculation-sedimentation trains and 2 filter basins for plant flows to 120 mgd and 3 trains and basins for flows above 120 mgd.
(f) Disinfection contact time based on a T_{10} to HDT ratio through the flocculation-sedimentation basins of 0.49 to 1 for plant flow rates less than 50 mgd, and 0.59 to 1 for plant flow rates greater than 50 mgd, and 0.30 through the settle water channel and filters of all flow rates, based on District tracer studies.
(g) Disinfection contact time in the 78-inch Cooperative Transmission Pipeline based on a T_{10} to HDT ratio of 1, a pipeline volume of approximately 116,140 cubic feet before the first connection, and up to 50-percent of the WTP flow through the pipeline. (Source: DHS Annual Inspection Report, August 1999.)

Table 8-4 is also based on a chlorine residual of 0.8 and a pH of 7.0 through the flocculation-sedimentation basins. A higher chlorine residual or lower pH reduces the required disinfection contact time. At WTP flow rates above approximately 180 mgd, additional disinfection CT may be required in a direct filtration treatment mode of operation. Table 8-4 is also based on an additional pretreatment basin on-line by a WTP capacity of 150 mgd. Without a third basin, there is insufficient disinfection CT in a direct filtration treatment mode of operation at a WTP capacity of approximately 130 mgd.

8.3.1 New Cooperative Transmission Pipeline Reservoir Outlet

When the Cooperative Transmission Pipeline was constructed, a 78-inch tee and blind flange were provided on the pipeline near the northwest corner of the reservoir, just west of the existing reservoir overflow. (Refer to Figure 8-1.) As-built drawings also indicate that approximately 30 feet of blasting was performed to provide a future trench for a reservoir outlet pipe. It was anticipated that a new connection to the reservoir would be constructed by installing new pipeline penetrating into the reservoir along with a new outlet structure complete with slide gate. The estimated capital cost of this alternative is provided in Table 8-5.

Table 8-5
Conceptual Level Capital Cost Estimate
Cooperative Transmission Pipeline/Hinkle Reservoir Connection

Item	Capital Cost
Mobilization, demobilization, and set-up costs	$50,000
Earthwork	50,000
78-inch pipe and fittings	200,000
Cover and liner repair and modifications	150,000
Outlet structure	35,000
78-inch outlet slide gate and appurtenances	25,000
Site restoration	20,000
Subtotal	**$530,000**
Contingency @ 25%	132,500
Engineering, legal, administrative @ 25%	132,500
Total	**$795,000**

It should be noted that the new outlet structure would connect the Cooperative Transmission Pipeline to only one section of the reservoir if it was divided into two segments in the future. If the north half of the reservoir was out of service, the pipeline would need to be off-line or operated on reservoir bypass, which would limit the available disinfection CT for the WTP.

This cost estimate does not include a cost factor for implementing an alternative water supply plan during reservoir construction activities. Reservoir outage during construction of the new outlet structure and reservoir tie-in is estimated to take a minimum of 45 days.

8.3.2 New Cooperative Transmission Pipeline- Reservoir Outlet Pipe Connection

View down west side of reservoir. Existing 48-inch bypass is located between toe of reservoir berm and pump station shown on right.

Another alternative for relocating the Cooperative Transmission Pipeline connection is to connect it directly to the existing 84-inch reservoir outlet pipe. A new 78-inch pipe could be constructed parallel to the existing 48-inch bypass pipe along the west side of the reservoir. The new pipe would connect to both the 84-inch pipe and the existing tee on the 78-inch Cooperative Transmission Pipeline.

A variation of the pipeline connection alternative would be to replace the 48-inch bypass pipe with the 78-inch pipeline connection. This would save on blasting and trenching and would minimize conflicts with constructing the two pipes in parallel. The new 78-inch pipeline could act either as bypass piping (flowing south) or cooperative pipeline supply (flowing north). This alternative is depicted on Figure 8-1. The estimated capital cost of this alternative is provided in Table 8-6.

Table 8-6
Conceptual Level Capital Cost Estimate
Cooperative Transmission Pipeline/Hinkle Reservoir Outlet Pipe Connection

Item	Capital Cost
Mobilization, demobilization, and set-up costs	$50,000
Trench excavation and 48-inch pipe demo	250,000
78-inch pipe and fittings	410,000
78-inch gate valve with electric operator	70,000
Site restoration	5,000
Subtotal	**$785,000**
Contingency @ 25%	196,000
Engineering, legal, administrative @ 25%	196,000
Total	**$1,177,000**

This connection alternative offers the greatest advantage in limiting reservoir outage time. The only reservoir outage would occur during the 78-inch to 84-inch pipeline tie-in. This tie-in could be limited to a 24-hour duration.

8.3.3 Recommended Cooperative Transmission Pipeline Connection

A direct pipeline connection between the existing 78-inch Cooperative Transmission Pipeline and the existing 84-inch reservoir outlet pipeline is the recommended alternative for relocating the cooperative pipeline treated water connection. Although this alternative has the higher capital cost, it has the following benefits:

- The pipeline connection can be made with a very short reservoir outage compared to more than a month with a direct reservoir outlet connection.
- It has the greatest operating flexibility. If the reservoir is divided into two sections in the future, the cooperative pipeline connection can remain in service regardless of either section being out for maintenance.
- The pipeline connection will add a 78-inch reservoir bypass, either replacing or supplementing the existing 48-inch bypass. During future reservoir outages, this can provide added capacity to directly feed the transmission pipelines.

Chapter 9: Recommended Improvements Plan

9.1 Introduction

This section provides a scheduled implementation plan for the improvements recommended in Chapters 3, 7, and 8 for the raw water transmission facilities, an expanded water treatment plant, and Hinkle Reservoir. The implementation schedule does not include improvements that may be necessary for the Bureau's Folsom Pumping Plant, repairs or rehabilitation of the Bureau's 84-inch transmission pipeline, or a parallel 84-inch transmission line to the Bureau's 84-inch transmission line to provide redundancy under a 150 mgd maximum WTP capacity scenario. The implementation schedule also does not account for changes in water use patterns or demands under a conjunctive use water supply approach as discussed in Chapter 2.

9.2 Basis of Cost

The estimated capital costs presented in the implementation plan are conceptual level estimates prepared without plans and specifications and actual quantity take-off. The estimates were prepared based on prior bid results, standard estimating guide cost curves, equipment quotes from suppliers, and engineering judgment. The cost estimates include a 25 percent contingency to provide for reasonable estimating and construction uncertainties. The total capital cost estimates also include a 25 percent allowance for planning, engineering, construction management, administrative, and legal expenses associated with project implementation.

The cost estimates are in 2001 dollars corresponding to the January 2001 Engineering News Record (ENR) Construction Cost Index of 6,281.

Environmental documentation and mitigation costs have not been estimated for the various projects due to the uncertainty regarding these potential costs. It is recommended that the District review each project on a case-by-case basis during preliminary design to minimize potential environmental impacts.

9.3 Implementation Schedule

The implementation schedule matches the recommendations provided for an ultimate WTP capacity expansion to 240 mgd, as described in Chapters 3, 7, and 8. The backwash and solids handling facilities capital improvements scheduled for 2002 are key to optimizing the existing WTP capacity and addressing the biggest operational and maintenance issues with the District's facilities. The actual timing of capital improvements scheduled for the period of 2002 to 2030 will be driven by actual growth and demand factors.

The planning, environmental documentation, design, and construction of the first 30 mgd phase expansion of the WTP will likely take a minimum of three years. These improvements are projected to be necessary between the years 2002 and 2009. Although the actual timing of the expansion is dependent on many factors, it appears that the initial steps for planning and financing this first phase expansion should begin soon.

Table 9-1
Project Implementation Schedule
Year 2002 - 2030

Year - WTP Capacity (mgd)	Project Description	Cost
2002 - 60/120	Filter Backwash Hoods	$3,300,000
	New Backwash Treatment and Recovery System	3,805,500
	New Solids Handling System	$6,510,000
	Estimated Capital Improvements Cost Schedule - 2001	$13,615,500
2002 - 60/120	Chlorine System (Structure and Scrubber)	$750,000
	Estimated Capital Improvements Cost Schedule - 2002	$750,000
2002 - 2009 75/150	30 mgd WTP Expansion	$39,994,500
	District Raw Water Pipeline Rehabilitation	1,006,500
	66-inch Raw Water Pipeline within District Property	1,207,500
	Cooperative Pipeline Connection Relocation (Assumes In-line Filtration Desired)	1,177,000
	Estimated Capital Improvements Cost Schedule - 2002 through 2009	$43,385,500
2010 - 2016 90/180	30 mgd WTP Expansion	$7,591,500
	66-inch Raw Water Pipeline (Parallel Bureau 84-inch Pipeline)	7,267,500
	Estimated Capital Improvements Cost Schedule - 2010 through 2016	$14,859,000
2017 - 2023 105/210	30 mgd WTP Expansion	$11,488,500
	Estimated Capital Improvements Cost Schedule - 2017 through 2023	$11,488,500
2023 - 2030 120/240	30 mgd WTP Expansion	$10,384,500
	Hinkle Cover Replacement, Divide Reservoir[3]	4,755,000
	Estimated Capital Improvements Cost Schedule - 2010 through 2016	$15,139,500
	Total Capital Improvement Costs - 2001 through 2030	$99,238,000

1. Costs based on January 2001 Engineering News Record (ENR) Construction Cost Index of 6,281
2. Cost estimates include a 25 percent estimating contingency and a 25 percent allowance for planning, engineering, administrative and legal expenses, and construction management associated with project implementation.
3. The District should consider the benefits of dividing Hinkle Reservoir prior to 2023 as discussed in Section 8.2.
4. Schedule represents the year improvements should be completed.

Chapter 10: References

The following references were used in the preparation of this Master Plan:

American River Watershed Sanitary Survey - 1998 Update.

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Surface Water Resources, Inc., “Draft Report, Assessment of the Beneficial and Adverse Impacts of Operating a Temperature Control Device (TCD) at the Water Supply Intakes of Folsom Dam,” Prepared for the U.S. Bureau of Reclamation, March 1997.

United States Environmental Protection Agency, “Optimizing Water Treatment Plant Performance Using the Composite Correction Program,” 1998 Edition.

Appendix 3-1

Technical Memorandum:
San Juan Water District Wholesale Master Plan
Intake Pipeline Inspection
Robert A. Ryder, P.E.

7 March 2000

TECHNICAL MEMORANDUM

To: Alex Peterson, P.E., Project Engineer
Keith Durkin, P.E. Project Manager

From: Robert A. Ryder, P.E., Corrosion Consultant

Subject: San Juan Water District
Intake Pipeline Condition Inspection for Deterioration
K/J 992509.00-G91

INTRODUCTION

As a part of the engineering study for the Wholesale Master Plan Project for the San Juan Water District (SJWD), an inspection was conducted to evaluate the present condition and remaining life of the two intake pipelines to the water treatment plant (WTP). These pipelines begin at the Hinkle Wye junction of the 84-inch Bureau of Reclamation (USBR) pipeline several hundred feet westerly of Folsom Dam and extend about 1,200 feet to the north to the SJWD's WTP.

The first pipeline is a 42-inch-diameter pipeline constructed in 1962, then extended by a 54-inch pipeline in 1976, and then paralleled by a 72-inch pipeline which transitions to a 66-inch pipeline constructed in 1986. Both the 42-inch and 72-inch pipelines transverse above grade from the USBR junction for several hundred feet, and then are buried beneath the earth for the remaining distance to the WTP. The pipelines are interconnected at several locations with valved crossover pipes. Near the WTP, the pipes converge to a single 54-inch diameter pipe, as shown on Figure 1.

INSPECTION PREPARATION

K/J staff conducted an inspection of the intake pipelines on 10 February 2000. The weather was extremely rainy. Alex Peterson and Robert Ryder made a visual inspection of the exterior parts of the pipeline, and Paul Peterson and Robert Ryder inspected the interior of portions of the pipeline that day.

A letter dated 24 January 2000 from K/J to SJWD described the configuration of the pipelines, access to be provided by excavation of buried parts of the pipelines, and safety precautions. The SJWD staff had excavated and shored the earth to the top of the pipe entrance manholes, closed and chain locked the inlet valves on each pipeline, and had dewatered the pipes prior to entry for interior inspection. The SJWD staff conducted a confined space entry checkout and protocol with K/J personnel and provided a gas measurement field instrument to monitor oxygen concentration in the interior of the pipes continuously during entry. Ladders, a winch cable, and harness with cable attachment were provided for the safety and emergency removal of K/J personnel who entered the pipes for interior inspection.

Inspection was conducted of the exterior of the pipes starting at 9:00 a.m. It was necessary to burn off corroded carbon steel manhole bolts and nuts to enter the pipe, and this delayed the anticipated pipe entrance from 10:30 to 11:00 a.m. The interior inspection of the dewatered pipes was conducted between 11:00 a.m. and 1:30 p.m.

PIPE CONSTRUCTION

Engineering plans prepared by Clendennon Engineers for the SJWD in 1984-85 indicated the following type of pipe:

		Thickness - Inches			
Pipe Size	**Installation Date**	**Steel Pipe Wall (1)**	**Cement Lining (2)**	**Cement Coating (2)**	**Above Grade Exterior Coating**
42	1962	Unknown	N/A (3)	Unknown	Paint
42	1976	3/16	1/2 (4)	¾	-
54	1976	1/4	1/2	¾	-
72	1986	1/4	1/2	¾	Paint & Tape Wrap
66	1986	1/4	1/2	¾	-

(1) AWWA Standard C200
(2) AWWA Standard C205
(3) Coal Tar Epoxy Lining
(4) Actual measurement was 1 to ¼-inch when inspected 2/10/00

The southerly portion of the original 42-inch pipeline was constructed with a coal tar lining, and extends to a distance of 1780 feet from Location 1 as shown on Figure 1. We did not traverse far up the 42-inch pipeline and did not observe its condition. However, based upon observations of the similarly lined 84-inch steel intake pipelines at Folsom Dam, the coal tar lining has reached its useful life.

Upon excavation and observation, it was found that a substitution had been made of the exterior coating for buried portions of the 72-inch and 66-inch pipe and crossover pieces constructed in 1986. An exterior tape wrap had been installed, which probably was like AWWA C214, a three-layer system of a butyl rubber primer with intermediate polyolefin tape and an outer wrap of polyolefin tape to provide an overall thickness of 50 mils.

The AWWA C214 specifications indicate that for this size pipe the tape width is 12 inches and should be wrapped in a spiral manner with 1-inch overlaps, and the seams of the intermediate and finished coats spread apart so that there would be barrier continuity.

A portion of the 72-inch pipeline just north of the flexible coupling is exposed prior to entering an earth berm (Photo #3) and shows the tape. This photograph also shows that the tape width is 6 inches for that portion of the pipe rather than 12 inches, as recommended in the current AWWA C214 specification.

The Clendennon Engineers plans also indicated that the joints of the 72-inch and 66-inch pipe were to be bonded by 12-inch-long #4 AWG copper conductor straps thermite welded around each joint. This type of bonding cable provides pipe electric continuity for installation of cathodic protection and corrosion testing when there are discontinuous non-welded joints as shown for the 72-inch and 66-inch pipes. A bonding cable of this type was observed around the flexible coupling on the exposed section of 72-inch pipeline.

The 54-inch pipeline fabrication and lay diagrams were obtained from American Pipe and show #8 TW copper wire bonding jumpers on the bell and spigot joints.

There was no indication if the original 42-inch inch pipeline had continuously welded joints or if there were similar bonding straps. Design drawings and manufacturer's key diagrams show that the 54-inch-diameter pipe had Carnegie joints and bonding straps. It will be necessary to determine, prior to installation of any cathodic protection in the future, if there is electrical continuity through the joints and, if not, to excavate and install bonding straps.

SOIL PROPERTIES AND CORROSIVITY

Observations of the surface soils and of the excavation pits and piles showed the soils to be uniform with depth and consisting of a gravelly silty sand with some fine decomposed granite. There was no discernable groundwater in the pits that were excavated to a depth of up to 15 feet below the surface. The soils also showed a great deal of permeability despite heavy rainfall, as there were no standing water puddles.

Soil resistivities were taken south of excavation Location 1 (Photo #7) by the 4-pin Wenner method. Results are shown in Table 2.

Table 2

Soil Resistivity of Intake Pipeline Area at Various Soil Depths

Depth	Soil Resistivity ohm-centimeters	Probable Corrosivity to Steel
5	12,000	Very Low
10	6,800	Low
15	5,400	Moderate

The resistivity is a measure of the conductive salts in soils. Generally, soils are increasingly corrosive with decreasing resistivity, with high to severe corrosion occurring where soil resistivities are less than 2,000 ohm-centimeters and increasingly lower corrosivity above 10,000 ohm-centimeters (AWWA 1987). These field tests show a decline in resistivity with depth. These readings taken during the wet season are probably nearly annual minimum values, as when soils become seasonally dry, the resistivity increases. The resistivity variance indicates that formation of galvanic potential differences both horizontally and vertically are probable, which can accelerate localized corrosion.

The Soil Construction Service (SCS 93) has mapped, tested, and described surface soil characteristics to a depth of 5 feet for all of Sacramento County. The soils to the west of Folsom Dam in the vicinity of the intake pipeline are in the Andregg-Urban Complex of 2 to 8% slope at elevations of 300 to 420 feet. These are moderately deep and well drained and formed from weathered granite rock. Surface soils to 21 inches depth are brown, coarse sandy loam or loamy sand, and below that depth, weathered granitonite to bedrock located at 20 to 40 feet below the surface. The SCS describes the soils as having moderately rapid permeability of 2-6 inches per hour; clay in the range of 7 to 11%; a moderate water holding capacity of 0.1 to 0.13 inches per inch; a pH range below 32-inch depth of 5.6 to 6.5; an organic content of 1 to 3%; a low shrink-swell potential; and a water table depth of more than 6 feet. The SCS also rated the probable corrosivity to uncoated steel or concrete, based upon the pH, as moderate.

Our visual assessment of the character of the soils, together with the resistivity measurements, would correlate with that of the SCS, as soils of low to moderate corrosivity.

A field measurement of the pipe to soil potential of the 42-inch pipeline was taken at Alternate Location 1 by attaching one lead to an exposed flange of the gate valve, and the other of a high impedance potentiometer to a copper-copper sulfate electrode placed on the ground within the pit. The potential reading was –470 millivolts, which is indicative of active corrosion of the steel and iron portions of the pipeline in that vicinity (Parker & Peattie, 1984). It would be necessary to elevate the potential to above –850 mv or to achieve a –100 mv instant-off potential shift by installation of cathodic protection to negate the corrosion and corrosion potential that was observed and measured.

WATER CORROSIVITY

Historic water quality data was obtained from EPA Storet sources for the American River in the vicinity of Folsom Dam extending back into the 1980's. The primary water characteristics that relate to corrosivity or scaling include pH, temperature, total dissolved solids, calcium, alkalinity, chloride, sulfate, and dissolved oxygen. There is considerable variation in physical properties of the lake water, such as pH that can range from 6.8 to 8.8, temperature from 5°C to 20°C, dissolved oxygen from 1 to 12 mg/L, and total dissolved solids (TDS) from 10 to 80 mg/L. Chemical characteristics also typically show a 4:1 variation. However, overall, the water is characterized as being cool, having a slightly alkaline pH, and low mineral solids, TDS hardness, and alkalinity.

The average water quality characteristics for the water are listed on Table 3, together with calculated corrosion-scaling indices and assessment as to probable corrosivity to piping and valve materials (Ryder and Wagner, 1985). Overall, these data show a potential for moderate-uniform corrosion to iron and steel; a moderate to high aggressiveness by carbonation to portland cement and concrete; and low corrosivity to copper, copper alloys, stainless steel and nickel alloys.

This assessment is useful to understand the reasons for the extent of internal corrosion that was observed.

Table 3

Water Quality Characteristics and Corrosion Potential

Characteristic	Units	American River at Folsom Dam	Desired Range
pH	-	7.3	6.5-8.5
Temperature	°C	14.3	5-20
Total Dissolved Solids	mg/L	43	<500
Calcium	mg/L	7.0	<50
Alkalinity	mg/L $CaCO_3$	23.7	<250
Chloride	mg/L	3.1	<250
Sulfate	mg/L	4.8	<250
Carbon Dioxide	mg/L	3.0 (2)	<5
Corrosivity and Scaling Indices			
pH_S $CaCO_3$ Saturation		9.36	-
Langelier Index		-2.06	-0.5 to +0.5
Ryznar Index		11.42	6-8
Aggressive Index		9.9	>12
Larson Index ($Cl+SO_4/HC0_3$)		0.40	<0.4
SO_4: Cl Ratio		1.54	<3

Probable Corrosivity or Scaling to Materials (3)		
Iron and Steel	Moderate Uniform Low Pitting Corrosion	Range 5-10 MPY
Copper	Low Uniform Corrosion	Range 0.5 – 1 MPY
Stainless Steel	Very Low Crevice Corrosion	Range <0.1 MPY
Cement & Concrete	Moderate to High Uniform Corrosion	Range 3-6 MPY

Notes:
(1) Average of EPA Storet Water Quality
(2) Calculated
(3) Ryder, R.A., "Corrosivity Characteristic Rating for Various Materials, Kennedy/Jenks, 1992.

EXTERNAL INSPECTION

Overall, the external conditions of the pipelines appear generally good. There is moderate corrosion occurring where there are coating defects, and some rust and shallow pitting.

The condition of the 1-1/4-inch steel bolts removed after 38 years in the ground are shown in Photo #27. Pit depths to 1/8-inch were observed on the bolts. This would correlate to a pitting penetration rate of 3.3 mils per year (0.0033 inches/year) for exposed metal in the ground, which is a low rate for steel.

However, the steel thickness of the 42-inch pipeline is 3/16-inch, or 187.5 mils. The steel thickness of the other pipe is 1/4-inch (250 mils). This sustained corrosion rate would induce pipe leaks within 60 years, or about 20 years from the present for the 42-inch pipeline. Considering these observations, as well as the need to preserve the pipelines for more than 100 years, it is recommended that a cathodic protection system be placed to provide protection to all of the buried intake pipelines within the next 5 years. This is now frequently occurring to preserve and extend the life of many cement and dielectric coated pipelines throughout North America (Gammow, 2000).

An impressed current, deep well anode cathodic protection system utilizing a buried reference cell is recommended considering the type of coatings and relatively high soil resistivity. It will be necessary to provide insulating joints before the USBR pipeline and WTP structures to isolate the cathodic protection and to minimize current requirements.

The installation of a cathodic protection system will require all underground pipe joints to be bonded. The pipeline should be megger tested for electrical continuity, and additional joint bonding added if required. Cathodic protection test stations should be installed near the ends and at the centrally located anode site.

Exposed Pipeline (72-Inch)

Photographs of the external portions of the 72-inch pipeline are shown on Photos #1 through #3. There is a paint coating (probably a two- or three-coat epoxy) from the turnout past the butterfly valve extending a few hundred feet to the flexible coupling. Then the tape wrap begins and extends beneath the earth berm.

The pipe is butt-welded, and the exterior is in generally good condition. A little rust is evident at the butterfly valve flange bolts, and there is some minor rust over 10 to 20% of the surface of the flexible coupling. There were two areas of more extensive rust splotches on either side of the lower quadrant of the pipeline about 50 feet from the USBR turnout. On the west side of the pipeline there was an area of about 1 square foot where there were numerous 1- to 2-inch round rust spots with the paint coating blistered off. Opposite this rust patch was a larger (4 feet square) area of numerous rust splotches and deteriorated paint below the springline.

Pit penetration into the steel pipe wall is not deep (less than 10 mils) at these rust locations. Otherwise, this exposed external portion of the pipeline appeared in good condition.

It is assumed that the original exterior paint coating was placed at the time of pipe construction. It is now 14 years old, and is near its normal life expectancy of 10 to 15 years.

Beyond the flexible coupling is the tape-wrapped portion of the 72-inch pipeline. The deterioration of the exposed tape wrap, particularly on the top of the pipeline where it is exposed to the direct sun, is evident from Photo #3. However, bare pipe or rust was not observed, so the tape is still providing surface protection. Nevertheless, cleaning up the delaminated sections and providing a new tape overwrap within the near future is desirable.

Exposed Pipeline (42 Inch)

The coating system of the 42-inch exposed pipeline appeared to be the same as the 72-inch pipeline, and was probably also done in 1986. Photos #4 through #6 are photographs of this pipeline, which is of spiral welded steel pipe. There are, however, more rust spots and imperfections. There are numerous rust spots on the sides of the pipe, particularly on the west side, which could have been caused by gravel nicks from traffic or thrown from the WTP pond access road. These occur about every 5 feet. Typically, pitting is shallow (<10 mils) in the rusted area. However, like the 72-inch pipeline, recoating should be scheduled in the near future to preserve the integrity of the pipeline.

Buried 42-Inch Pipeline

There were several areas observed from excavation pits along the 42-inch pipeline, as shown on Photos #7 through #12. The external coal-tar coating on the gate valve and flanges was failing, and about 30% was exposed and beginning to rust, with pits as deep as 1/8 inch. About five pits per square foot were observed in the thick cast iron flange and body of the valve.

A typical plastic diaphragm diaper used to place cement mortar at a field joint is shown on Photo #12, and it and the 3/4-inch cement coating appears generally in very good condition, with little surface deterioration.

Buried 54-Inch Pipeline

The surface of the concrete mortar coating of the 54-inch pipeline was observed at Location 3. The cement coating again showed little deterioration, but had a drummy sound near a joint, indicative of partial disbonding to the steel.

Buried 72-Inch Pipeline

The 72-inch pipeline was observed at Location 1. It had a tape-wrapped exterior coating that appeared to be in good condition, as was the 42-inch wye pipe shown in Photo #8.

Buried 66-Inch Pipeline

The 66-inch pipeline was exposed at Location 2. It has an exterior tape wrap, shown in Photo #12 that appears to be in fair condition. The edges of the outer wrap are now delaminating and lifting from the intermediate wrap by as much as 3/4-inch. This is certainly an indication of coating deterioration and loss of adhesive bonding, which will, in time, lead to pipe exposure and aggravated corrosion.

INTERNAL INSPECTION

Overall, the internal condition of the pipelines appears fair, but deteriorating. The specific condition of each section of pipe is discussed below.

42/54-Inch Pipeline Near the Double Wye

The 54-inch pipeline was entered through a 20-inch manhole at Location 1. There was a very smooth gelatinous dark brown film over the concrete that was 20-30 mils thick. Beneath that was the concrete lining, which was soft to a depth of 1/8-inch of the measured 1-inch original thickness. This softened cement condition is no doubt due to carbonation and loss of calcium and alkalinity due to the passage of Folsom Lake water, which as previously described, tends to be undersaturated with calcium carbonate (a negative Langelier Index and at times relatively low <7.5 pH). Holtschulte (1985) and Leroy, et al (1996) describe this condition of carbonation deterioration of cement lining in pipelines conveying aggressive water. The brown gelatinous film is probably a combination of iron and manganese oxide from that portion of iron in the cement of the pipeline, and what may be oxidized on the surface from manganese released from anoxic zones of lower reservoir depths. The brown surface film had no odor, so extensive microbial slime growth is not likely.

Overall, the gelatinous film is beneficial as it maintains a very smooth surface and high Hazen-Williams "C" value to sustain flow capacity. It also suppresses diffusion of calcium and hydroxide of the cement, the abrasion and loss of sand, and suppresses the rate of cement loss with time.

There were numerous circumferential cracks occurring at about 5-feet intervals, as shown on Photo #13. These cracks were up to 1/16-inch wide at the surface and some showed that steel was corroding beneath the surface. No longitudinal cracks were observed. The presence of so many circumferential cracks could be due to displacement or settling of portions of the trench with time, or if soil was disturbed when constructing the parallel nearby 72-inch pipeline.

AWWA C205 does not limit circumferential hairline cracks of cement linings, stating they will autogenously heal and protect the steel wall of the pipe. This is doubtful in this case because these are more than hairline cracks and rust is observed. The presence of small localized bare steel anodic areas will accelerate corrosion in those locations and the expanding rust will then spall the adjacent cement lining, aggravating corrosion.

Deterioration of the cement caulked joints of the pipeline was very apparent, and by far the biggest and most immediate corrosion problem. The state of deterioration of all of the joints is severe, with very soft cement lifting away from corroded steel beneath the caulking of each joint. Photos #14 and #24 depict the severity of this condition, which was typical of all of the joints observed.

The field joint lining accelerated deterioration, as contrasted to the centrifugally spun factory-applied pipe lining, is most likely due to excessive water and lack of bond to the pipe lining. Also, the joint mortar was probably not packed nearly as dense nor had the low permeability of the adjacent pipe lining. AWWA C205 specifies the same cement sand ratio as the pipe lining for joints, but whether that occurred is questionable considering the relative condition of the two types of lining.

42-Inch Pipeline

The interior of the gate valve in the 42-inch pipeline showed extensive tuberculation of about a half-dozen nodules per square foot of surface area. Each tubercle was about 3/4-inch in diameter, rising to 1/2-inch above the surface. Beneath each tubercle was a pit to 1/8-inch depth, and about the same rate of corrosion and condition as for external exposed steel and iron.

The original 42-inch-diameter pipeline was approximately 189 feet from the manway, beyond the reach of the cable connection winch used during inspection. This pipe is reported to be coal tar epoxy lined. Based on the condition of the coal tar lining of the gate valve the original lining is in fair to poor condition. Although the initial portion of the 42-inch pipeline with the original coal tar lining was not inspected, it is likely that this type of lining has a life of less than 50 years, and as solvents volatize will crack, creating water penetration to the steel. The small portions of exposed steel will then act as sacrificial anodes with accelerating corrosion and rust expansion to spall off more and more lining.

The interior joints and lining of this pipe should be rehabilitated within the next 5 years. The grout joints should be replaced and a high calcium (1:1 cement-sand ratio) spun in pipe lining of a 3/8-inch minimum thickness be applied for the full length of the pipeline. It has been found by Leroy and Holtschulte that a 1:1 cement sand ratio, if used in relining pipe, will last three times as long as the original 1:3 pipe lining. This is the most recommended means of repair and rehabilitation.

The interior of the 42-inch double wye pipeline was in considerably better condition. The cement lining was smooth, showing no cracks; the brown gelatinous film and softened cement penetrated to 1/16-inch depth.

A photo of this lining and of the butterfly valve disc is shown on Photo #16. The bright stainless steel valve edge and relatively non-corroded nickel cast iron valve disc (NiResist) appear in excellent condition.

54-Inch Pipe

The 54-inch pipe was less severely deteriorated than the 42-inch pipeline. However, circumferential cracking and joint deterioration approaching that of the 42-inch pipeline was evident. We would advise joint repacking and relining the pipe within 10 years.

72-Inch Pipeline

One circumferential crack was observed about 50 feet south of the Location 1 entry, shown in Photo #20. Another portion of this pipe had a section of drummy lining and extensive spider cracking extending over a 4-foot-square area of the lower quadrant. A brown gelatinous coating and 1/16-inch soft cement lining was typical.

The joints had an epoxy type of grout that was 1/4- to 3/4-inch thick. The grout was delaminating and breaking into pieces as shown on the photo on Photo #19. There are non-welded, bell and spigot or Carnegie joints according to Clendennon Engineers' drawings, and there was an apparent substitution of epoxy grout for portland cement grout.

66-Inch Pipeline

The condition of the 66-inch pipe north of the wye was similar to the 72-inch pipe. A large circumferential crack with rust staining through a portion was observed and epoxy grout was loosening from the joints as shown on Photo #23. A large chunk of cement lining was lying on the bottom of the pipe. Photographs of the epoxy grout are shown on Photos #25 and #26.

Overall, there is some concern regarding the state of deterioration of the interior of the 72- and 66-inch pipelines, although they are less than 15 years old. The rate of cement loss is about 6 mils per year, double that for the 42-inch pipeline, although it may decrease with time as the gelatinous coating builds up. Still, the probable life of the 1/2-inch-thick cement lining is less than 50 years, and relining within 20 years is advisable.

However, a more immediate need is the recaulking of the failed epoxy grout joints, and spot repairs to regrout large cracks and areas where lining is spalled. This is fairly urgent work to prevent leaks, as these become small anodic areas that experience accelerated localized corrosion because they become sacrificial to all other portions of the interior of the pipeline steel. There are, perhaps, between 40 and 45 joints to repair. It appears relatively urgent considering the rusted areas of pipe wall found beneath the epoxy grout. These will expand as rusting continues, further loosening the grout in a continuing accelerating corrosion condition. This work should be scheduled within 5 years.

SUMMARY OF Findings and Recommendations

1. The aboveground exterior surfaces of the pipe are now showing indications of coating failure and rust. It is recommended that both the 42- and 72-inch pipes be recoated within the next few years.

2. The soils are moderately corrosive to steel and concrete due to a combination of low pH and resistivity. Deterioration and pitting at a rate of about 3 mils per year is occurring on both materials.

3. The 72- and 66-inch pipelines were provided with a tape wrap as an alternate to specified cement coating, and the wrapping is beginning to shrink and delaminate at the edges.

4. Installation of a deep well anode impressed current cathodic protection system is recommended to provide for continuing corrosion protection of all of the buried intake pipelines within the next 5 years.

5. The water conveyed in the interior of the pipelines from Folsom Lake is of low TDS, hardness, alkalinity, and periodically pH. It is undersaturated with calcium carbonate and is aggressive to cement and concrete causing leaching of calcium leaving a softened paste. This water is also moderately corrosive to steel and iron and has a tendency toward more uniform corrosion rather than deep pitting.

6. The interior of the pipelines have a dark brown, very slick film coating overlying cement softened from 1/16- to 1/8-inch depth by carbonation from aggressive water conveyed from Folsom Lake. This film tends to suppress the rate of carbonation and deterioration of the cement lining beneath its surface.

7. The interior cement caulked joints of the 42- and 54-inch pipes have completely softened and failed, and extensive rusting of the steel beneath and disbonding of this softened grout have occurred.

8. Unprotected steel and iron in the interior of the pipelines are corroding at a rate of about 3 mils per year, as is the pipe cement lining.

9. There are many circumferential cracks of the 42- and 54-inch pipes' cement lining, which are now showing penetration of rust and probable accelerated corrosion and spalling of the cement lining.

10. It is recommended that the 42-inch pipeline joints be regrouted and a new high calcium cement (1:1 cement-sand ratio) relining placed over the existing lining within the next 5 years, and the same done for the existing 54-inch pipeline within the next 10 years.

11. The 66- and 72-inch pipelines also show the same type of brown surface film over deteriorating cement that is softened to a depth of 1/16-inch by loss of calcium. This initial rate of deterioration is about 6 mils per year, but will most likely decrease with time to be closer to what is measured for the 42-inch pipeline.

12. The most serious condition in the 66- and 72-inch pipelines is at the interior epoxy grouted joints, where the epoxy grout has loosened, and steel surfaces are beginning to rust accelerating the spalling of the epoxy grout.

13. There are circumferential cracks in the 66- and 72-inch pipelines at about every 20 feet distance apart as compared to 5 feet for the 42-inch pipeline. Rust is showing through some portions of the cracks.

14. The thickness of the cement lining of the 66- and 72-inch pipelines are typically 1/2-inch, as contrasted to 1- to 1-1/4-inch in the 42- and 54-inch pipelines, so lining replacement is more urgent with respect to time of initial installation.

15. There is a 5- by 8-inch portion of the interior cement lining that has spalled off of the 66-inch pipeline, and other areas where there is a drummy sound of disbonded cement.

16. The butterfly discs of stainless steel edged nickel cast iron show no corrosion, which is evidence of the superior resistance of these materials to cast iron or steel.

17. The joints of the 66- and 72-inch pipelines should be regrouted as well as patched at spalled and cracked lining locations within 5 years, and the entire pipeline be cleaned of softened cement and relined with high calcium cement within 20 years.

18. The life expectancy of the original cement linings is 60 years. Remaining service life for the pipelines are 20 years for the 42-inch, 35 years for the 54-inch, and 45 years for the 66- and 72-inch pipelines. Rehabilitation and pipe relining will extend their service lives for an additional 40 years.

19. Buried access manhole and valve bolts should be replaced whenever they are exposed for maintenance with Type 304 stainless steel with plastic washes and bolt stems to suppress galvanic action with carbon steel flanges.

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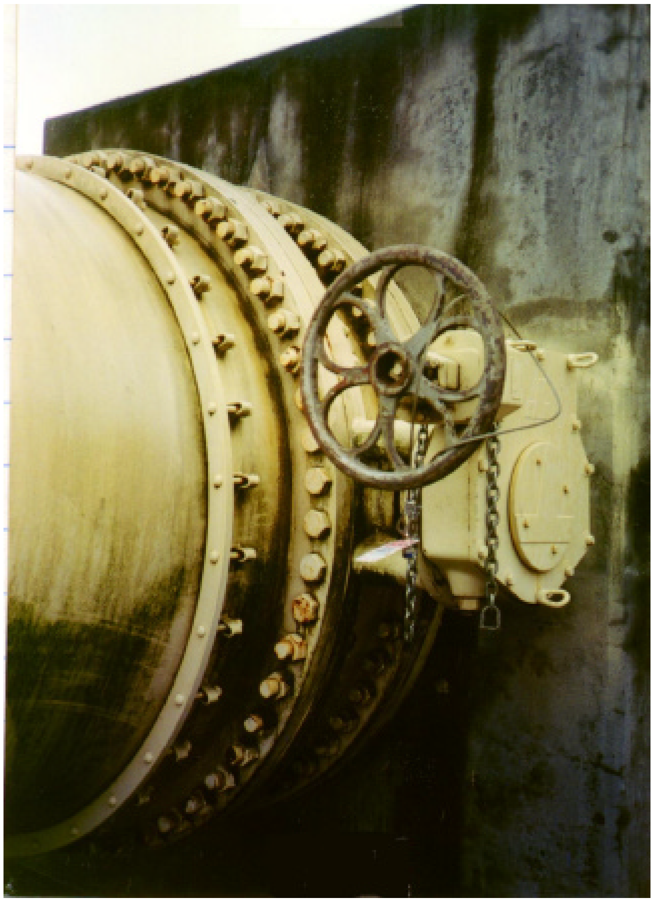


Photo #1: 72" Butterfly valve at turnout from USBR 84" P.L.

Photo #2: Corrosion, deterioration, and pitting on surface of 72" P.L. about 50 feet from USBR turnout.

Photo #3:

Exterior tape wrap protective coating of 72" P.L. beyond flex. coupling and into embankment mound cover.

Photo #4: 42" Gate valve on intake pipeline near 84" USBR P.L.

Photo #5: Exterior coating deterioration and pitting typical of many areas of 42" intake pipeline.

Photo #6:

Corrosion pits near 42" Venturi section of intake pipeline.

Photo #7: Excavation dirt piles and soil resistivity test site south of Location 1.

Photo #8: 42" Wye pipe, butterfly valve and tape wrap coating at Location 1.

Photo #9 Diaper and Coating on 66" intake pipe at Location 3.

Photo #10: 54” gate valve south of Location 1, 54” CMC Steel with epoxy at FCA, gate valve protected with tape wrap and coal tar.

Photo #11: Tape wrap on 42” P.L. at Location 2 with ¾” of tape delaminated at FCA.

Photo #12:

Diaper on joint of 66” CLS pipe at Location 2.

Photo #13: Circumferential crack in 42" P.L. south of Location 1.

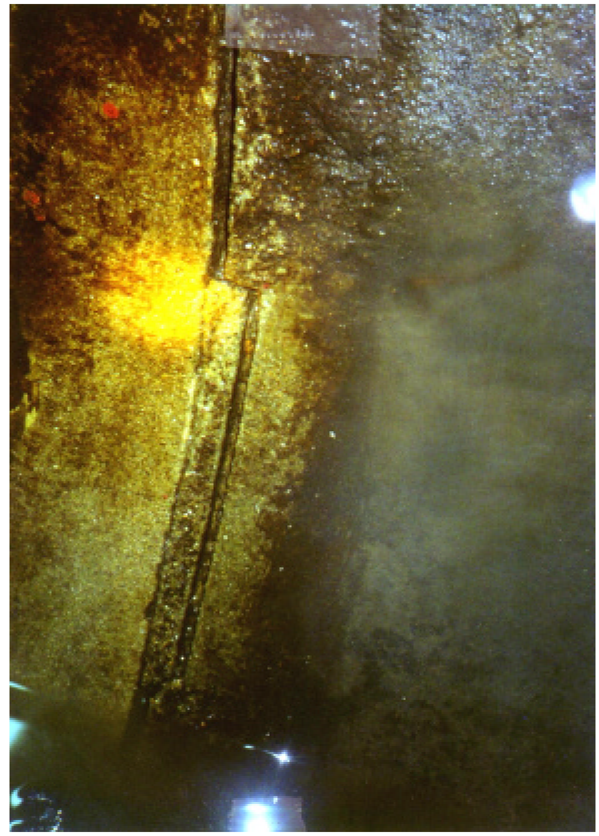


Photo #14: Deteriorated concrete grout in joint of 42" P.L. - typical.

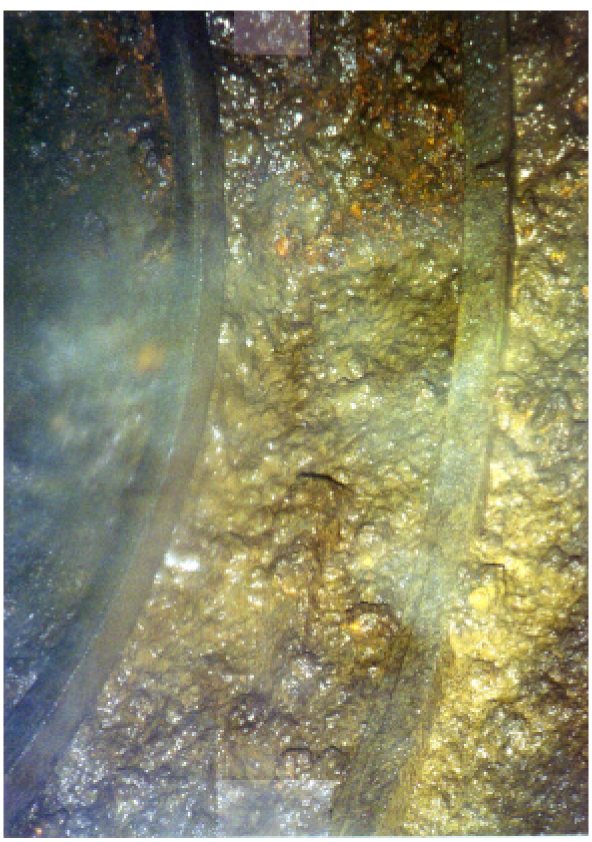


Photo #15: Tubercles on interior of cast iron gate valve body in 54" P.L. south of Location 1.

Photo #16: Stainless steel edge of 42” butterfly Disc at Crossover Y.

Photo #17: Rust on interior of 42” steel pipe cylinder where cement grout loosened. Knife scraping on brown slime coating.

Photo #18: Tubercles and rust on interior of 42-inch steel pipe cylinder north of Location 1.

Photo #19: Deteriorated epoxy grout in joint and rust in interior of 72" P.L.

Photo #20: Circumferential crack in 72" P.L. south of Location 1.

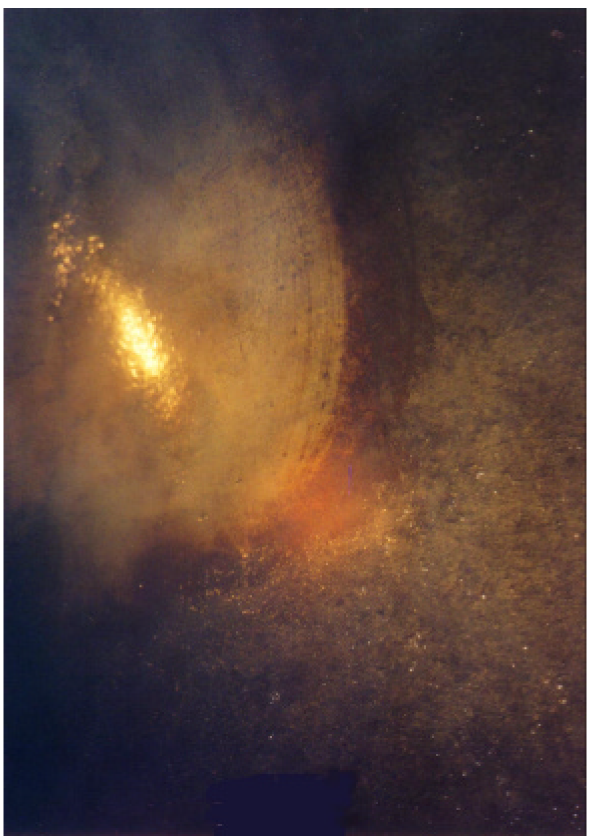


Photo #21: Circumferential crack and rust spotting in interior of 72" P.L. south of Location 1.

Photo #22: A chunk of concrete lining in bottom of 66" P.L. north of Location 1.

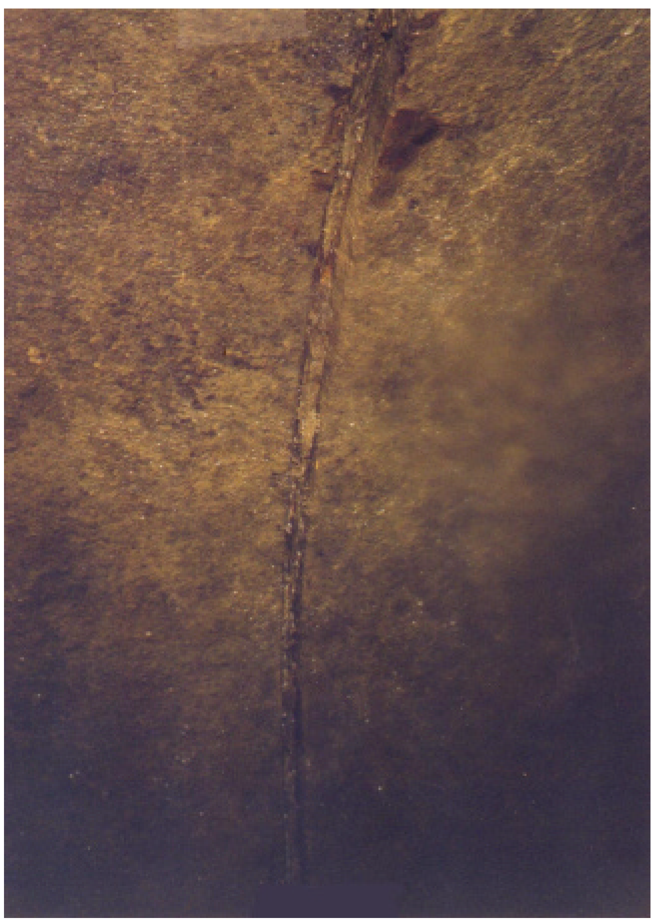


Photo #23*:*

Loose epoxy grout in joint of 66" P.L. north of Location 1 (typical).

***Photo #24**:* Chunk of grout which fell away from interior joint of 42" P.L. (16" x 1").

***Photo #25**:* Loose chunk of ½" cement lining (8" x 5") found lying on bottom of 66" P.L..

***Photo #26**:* Flake of epoxy grout from 66" intake pipe - (5" x 1/16").

***Photo #27**:* Rusted 1 ¼" steel bolts cut from 42" pipeline access manhole.

Appendix 4-1

Drinking Water Regulations and Guidelines

Appendix 4-1 Drinking Water Regulations and Guidelines

Existing Drinking Water Regulations and Guidelines

The existing drinking water regulations and guidelines include federal and state regulations and guidelines that were in effect on July 31, 2000. The existing drinking water regulations include the Surface Water Treatment Rule (SWTR), Total Coliform Rule (TCR), Lead and Copper Rule (LCR) and the Information Collection Rule (ICR). The existing drinking water guidelines include the Partnership for Safe Water and the California Cryptosporidium Action Plan.

Surface Water Treatment Rule (54 FR 27486; June 29, 1989) and Title 22 CCR Sections 64650 through 64700)

For a good quality water source, the SWTR requires that the overall treatment process achieve a minimum of 99.9 percent (3-log) removal and/or inactivation of *Giardia* cysts and 99.99 percent (4-log) removal and/or inactivation of enteric viruses. This is to be accomplished through a combination of physical removal treatment and disinfection processes. Because frequent measurement of *Giardia* cysts and enteric viruses is difficult and costly, the USEPA and DHS have developed functional criteria for determining the effectiveness of surface water treatment processes. These functional criteria are to be used unless more definitive data is presented by operational or pilot plant test results.

The guidance criteria developed by USEPA allow up to 99.7 percent (2.5-log) removal credit for *Giardia* cysts and 99 percent (2-log) removal credit for enteric viruses at water treatment plants with Conventional Filtration treatment if the filtered water turbidity is less than or equal to 0.5 NTU for 95 percent of the measurements taken each month. These guidance criteria also allow 99 percent (2-log) removal credit for *Giardia* cysts and 90 percent (1-log) removal credit for enteric viruses at water treatment plants with Direct Filtration treatment. The DHS, with regulatory primacy in California, includes a daily average treated water turbidity requirement of 0.2 NTU for water treatment plants that are new or upgraded after May 15, 1991.

Disinfection is used to achieve the rest of the combined removal-inactivation requirement. This would require an additional 68 percent (0.5-log) reduction of *Giardia* cysts and 99 percent (2-log) reduction of enteric viruses through disinfection when the plant operates in a Conventional Filtration treatment condition and 90 percent (1-log) reduction of *Giardia* cysts and 99.9 percent (3-log) reduction of viruses when the plant operates in a Direct Filtration condition. The SWTR also requires that systems demonstrate, by monitoring and recording, that they continuously maintain a disinfectant residual of at least 0.2 mg/L in water delivered to the public via the distribution system. It is anticipated that chlorine would be used to satisfy this requirement.

Appropriate disinfection is based upon the product of disinfectant residual concentration (C) and contact time (T) expressed as CT in units of mg/L-minutes. The CT required is a function of the type of disinfectant, residual disinfectant concentration, water temperature, and pH. Tables 4-1 and 4-2 summarize CT requirements for chlorine and ozone disinfection at 10°, 15°, 20° and 25°C, which are the range of temperatures in the source water from Folsom Reservoir.

Table 4-1
CT Requirements for Chlorine Disinfection at 10°C, 15°C, 20°C and 25°C [a]

Parameter	Units	@10°C	@15°C	@20°C	@25°C
Giardia Cysts [a] 90 percent (1-log inactivation)	mg/L-min	50	33	25	17
Enteric Viruses [a] 99.9 percent (3-log inactivation)	mg/L-min	4	3	2	1
Design CT Goal	mg/L-min	50	33	25	17
Design Residual	mg/L	0.4	0.4	0.4	0.4
Required Contact Time (T_{10}) [c]	min	125	83	63	43

Table 4-2
CT Requirements for Ozone Disinfection at 10°C, 15°C, 20°C and 25°C [b]

Parameter	Units	@10°C	@15°C	@20°C	@25°C
Giardia Cysts 70 percent (0.5 log inactivation)	mg/L-min	0.23	0.16	0.12	0.08
Enteric Viruses 99 percent (2.0 log inactivation)	mg/L-min	0.5	0.3	0.25	0.15
Design CT Goal	mg/L-min	0.5	0.3	0.25	0.15
Design Residual (average)	mg/L	0.05	0.05	0.05	0.05
Required Contact Time (T_{10}) [c]	min	10	6	5	3

[a] For pH = 8.0, and chlorine residual ≤ 0.4 mg/L.
[b] Assume dissolved ozone concentration in water leaving first transfer cell ≥ 0.3 mg/L.
[c] T_{10} = Contact Time of the first 10% of water passing through detention facility.

Total Coliform Rule (54 FR 27544; June 29, 1989)

Coliforms are found in human and animal wastes, as well as in soils. The presence of coliforms, which may not necessarily be disease causing, often indicates that gastroenteric infection-causing organisms may be present. Therefore, coliforms are used as a surrogate for all potentially pathogenic bacteria because of prevalence, resistance and relative ease of monitoring. The Total Coliform Rule (TCR) established monitoring and sanitary survey requirements for surface and groundwater systems. Current regulations require that suppliers monitor water quality in the distribution system through a routine sampling program approved by DHS. This sampling schedule changes if the system turbidity exceeds 1 NTU or the system is "out of compliance" with the SWTR. The tests are based strictly on presence or absence of coliform organisms. If a sample is positive, a repeat sample must be analyzed for fecal coliforms or *E. Coli*.

Lead and Copper Rule (56 FR 26460; June 7, 1991)

Lead solder and copper tubing are common materials used in household plumbing and/or customer service connection pipe. Lead and copper are soluble in water and can be leached from pipe, solder and/or fixtures under corrosive water quality conditions. The presence of these metals in drinking water, especially lead, can cause adverse impacts on health, particularly in children. Lead is associated with retarding physical development and interfering with mental development.

The USEPA's Lead and Copper Rule (LCR) is intended to protect the public not just from the water delivered to the consumers' service pipe connection, but also after it has flowed through the consumers' plumbing to the tap. The LCR establishes action levels (AL) to be lower than 0.015 mg/L for lead and 1.3 mg/L for copper in at least 90 percent of the most likely consumer tap samples in first draw samples after overnight stagnation. Sampling must also be conducted at points of entry (POE) to the distribution system to verify that lead and copper in the source of supply do not exceed the USEPA criteria.

In addition, the water supplier may be required to treat the water to reduce corrosivity to minimize leaching lead and/or copper. If the lead and/or copper levels are still above action levels after optimum treatment technology and/or corrosion control techniques have been implemented, the water supplier must:

- provide additional corrosion control strategies
- initiate a lead service line replacement program (LSLRP) if lead services are present
- and/or begin a public education program aimed at minimizing consumer exposure to these metals.

Information Collection Rule (61 FR 24354; May 14, 1996)

The Information Collection Rule (ICR) was published in the Federal Register on May 14, 1996. The ICR is a key element in the USEPA's Microbial/Disinfection Byproducts (M/DBP) Reg-Neg process and was intended to provide more definitive information on specific source water quality, microorganism contaminants and treatment plant performance including disinfection by-product generation. This regulation required most public water systems serving more than 100,000 people to collect data on their source and treated water and that they provide these data to the USEPA for evaluation.

Partnership for Safe Water Guidelines

The "Partnership for Safe Water", prepared jointly by EPA, AWWA and other water industry stakeholders, recommends an average filtered water turbidity of 0.1 NTU or less to ensure protection of the public. This filtered water turbidity goal is also recommended to maximize *Cryptosporidium* oocyst and other pathogenic organism removal.

California *Cryptosporidium* Action Plan

The California CAP established new turbidity goals for settled water, filtered water and return water. The settled (clarified) water turbidity goal includes settled water turbidity between 1 and 2 NTU at all times. The filtered water turbidity goals include a 0.1 NTU goal for both individual filters beginning 4 hours after a filter backwash and for the combined filtered water (from all the filters) at all times, and a 0.3 NTU goal for individual filters within 4 hours following a filter backwash. The CAP also includes a return (recycle) water turbidity goal set at 2.0 NTU.

New Drinking Water Regulations

The new drinking water regulations and guidelines include regulations published in the Federal Register by the USEPA with implementation dates after July 31, 2000. These new regulations include the Interim Enhanced Surface Water Treatment Rule (IESWTR) and Stage 1 Disinfectants/Disinfection Byproducts Rule (D/DBPR), the Long Term 1 Enhanced Surface Water Treatment Rule and Filter Backwash Rule, and the Arsenic and Clarifications to Compliance and New Source Contaminants monitoring; Proposed Rule. The state primacy agencies have up to three years to adopt the IESWTR and Stage 1 D/DBPR. Public water supply agencies will have an additional 2 years to comply with these new regulations after they are adopted by the primacy agency. The DHS indicates that the Stage 1 D/DBPR and IESWTR are currently scheduled to be implemented on January 1, 2002 in California.

Stage 1 Disinfectants and Disinfection By-products Rule (63 FR 69389; December 16, 1998)

The Stage 1 Disinfectants and Disinfection By-Products Rule (Stage 1 D/DBPR) was published in the Federal Register concurrently with the new Interim Enhanced Surface Water Treatment Rule (IESWTR) on December 16, 1998. The Stage 1 D/DBPR set new MCLs for selected disinfection by-products, establishes maximum residual disinfectant levels (MRDLs), and treatment techniques for control of DBP precursors (DBPPs). Surface water systems supplying more than 10,000 people must comply with this new rule by January 1, 2002.

The Stage 1 D/DBPR revised the existing THM MCL, created a new MCL for HAA5, and also included MCLs for bromate and chlorite as part of the new regulations. On the basis of the Reg-Neg rulemaking process, in which the USEPA participated, the Total THM (TTHM) MCL was reduced from 0.1 mg/L (100 µg/L) to 0.080 mg/L (80 µg/L), the new HAA5 MCL was set at 0.060 mg/L (60 µg/L), the new bromate MCL was set at 0.010 mg/L (10 µg/L) and the chlorite MCL was set at 1.0 mg/L in the Stage 1 - D/DBPR. In addition, the Stage 1 D/DBPR includes maximum residual disinfectant levels (MRDLs) for chlorine, chloramines and chlorine dioxide. The chlorine concentration in the treated water delivered to SJWD customers is well below the new 4.0 mg/L chlorine (as Cl_2) MRDL.

Enhanced Coagulation (EnCoag) to reduce DBPPs, measured as TOC, is also a part of the Stage 1 - D/DBPR. The enhanced coagulation requirement applies to water treatment plants with “conventional filtration treatment”, which includes a sedimentation step, and is required if the source water TOC exceeds 2 mg/L.

Interim Enhanced Surface Water Treatment Rule (63 FR 69477; December 16, 1998)

The Interim Enhanced Surface Water Treatment Rule (IESWTR) was published in the Federal Register on December 16, 1998. The IESWTR includes a stringent new 2-log *Cryptosporidium* removal requirement and sets a Maximum Contaminant Level Goal (MCLG) at zero for the protozoan genus *Cryptosporidium*, not the species *Cryptosporidium parvum*. Water treatment plants with a conventional or direct filtration treatment process automatically meet this requirement if they comply with the new filtered water turbidity standards included in the IESWTR.

The primacy agency in each state has three years to adopt the new IESWTR. The DHS staff indicates that the IESWTR will be adopted in California on January 1, 2002. Information presented by DHS staff indicates that the California IESWTR will include some provisions that are more restrictive than the Federal IESWTR. These provisions include an individual filtered water turbidity standard set at “0.3 NTU at all times after 30 minutes of filter run time” and continuous monitoring of CFE turbidity and recording combined filter effluent (CFE) turbidity at 15 minute intervals. The DHS developed these modifications to the IESWTR in conjunction with stakeholders including water utilities.

The new IESWTR turbidity standard include: 1) a CFE turbidity of less than or equal to 0.3 NTU in at least 95 percent of the samples collected each month and 2) a CFE turbidity less than 1.0 NTU in all samples collected at 4 hour intervals during each month.

The IESWTR also includes individual filter monitoring and reporting requirements. If the filtered water turbidity from a filter 1) exceeds 1.0 NTU in two consecutive 15 minute intervals or 2) exceeds 0.5 NTU in two consecutive 15 minute intervals after the initial 30 minutes of operation following a filter backwash, then a filter profile report must be submitted to the primacy agency. Also, 1) if the filtered water turbidity from a filter exceeds 1.0 NTU in three consecutive months or 2) if the filtered water turbidity from a filter exceeds 2.0 NTU in two consecutive months, then a filter profile report must be submitted to the primacy agency.

The Federal IESWTR requires that disinfection CT profile data be collected for at least a 12-month period and permits using up to 36 months of CT data. The lowest monthly CT will be used (or lowest average monthly CT if 36 months of data are used) to establish a plant CT credit benchmark. The plant CT benchmark must be used in consultation with DHS prior to making significant changes to disinfection practices.

The IESWTR includes sanitary survey requirements and reduces the interval between follow-up sanitary surveys for most systems from the 5 years required by the TCR to 3 years. The required interval between sanitary surveys can increase to 5 years if the primacy agency determines that prior sanitary surveys indicate “outstanding performance.”

Long Term 1 Enhanced Surface Water Treatment Rule and Filter Backwash Rule; Proposed Rule (65 FR 19045; April 10, 2000)

The USEPA published the proposed Filter Backwash Rule (FBR) in the Federal Register as part of a combined Long Term 1 ESWTR and FBR in April 2000. The intent of the FBR is to reduce the risk that contaminants removed in the pretreatment and filtration processes are not returned with recycle water flow. The new FBR requires that large in-plant recycle streams be blended with source water “prior to the point of primary coagulant addition.” The USEPA states in the FBR that “Given the above limiting factors, the Agency does not believe it is prudent to establish a national recycle flow treatment requirement until additional data becomes available.” The proposed FBR is less stringent that the California CAP.

Arsenic Rule; Proposed Rule (65 FR 38887; June 22, 2000)

The proposed Arsenic Rule was published in the Federal Register as part of an “Arsenic and Clarifications to Compliance and New Source Contaminants Monitoring; Proposed Rule” on June 22, 2000. The proposed Arsenic Rule includes a proposed arsenic MCL set at 5 µg/L and includes a request for comments on setting the arsenic MCL at 3, 10 and 20 µg/L.

Anticipated Drinking Water Regulations

The USEPA has indicated that additional regulations intended to protect public health will be developed and published after December 31, 2000. The anticipated regulations include a Final (Long Term 2) ESWTR, a Stage 2 – D/DBPR, and a Radionuclide(s) Rule. The USEPA issued the Draft Microbial/Disinfection By Products (M-DBP) Stage 2 M-DBP Agreement in Principle on 12 September 2000.

Long Term 2 ESWTR and Stage 2 - D/DBPR

The Long Term 2 ESWTR LT2ESWTR and Stage 2 - D/DBPR are scheduled for promulgation in May 2002. These two regulations will be based on data collected as part of the ICR, and experience with the IESWTR and Stage 1 - D/DBPR. The 12 September 2000 draft "USEPA Microbial/Disinfection By-Products (M/DBP) Federal Advisory Committee Stage 2 M/DBP Agreement in Principle" indicates that:

1) The THM and HAA5 MCLs will remain at 80 µg/L and 60 µg/L, respectively, but compliance will be based on Local Running Annual Averages (LRAA). In addition, each Community Water System serving more than 10,000 people must conduct an "Initial distribution system evaluation (IDSE)." The IDSE would include sampling for THMs and HAA5 at locations where maximum levels are likely to occur. Systems using free chlorine for oxidation and disinfection should collect samples at eight locations. The eight locations would include: one near the entry (connection) to the distribution system, two with an average residence time and five locations at the maximum residence time. The IDSE results will not be used for compliance purposes unless these sample locations are already used for this purpose.

2) The Long Term 2 ESWTR (LT2ESWTR) will require all systems serving more than 10,000 people to develop source water quality data on *Cryptosporidium*, *E. coli* and turbidity during a 24-month period. The 10-liter samples collected for *Cryptosporidium* must be analyzed using USEPA Method 1622/23.

3) The source water *Cryptosporidium* data will be used to determine which of three alternative *Cryptosporidium* treatment requirements apply to a water treatment facility.

4) A "Microbial Toolbox Table" provides alternative strategies for complying with the applicable *Cryptosporidium* treatment requirements.

5) The proposed LT2ESWTR will include disinfection CT Tables for *Cryptosporidium* using ozone and chlorine dioxide; and the final LT2ESWTR will include disinfection "Intensity-Time" (IT) Tables for 2, 3 and 4-log inactivation of *Giardia lamblia, Cryptosporidium* and virus."

Appendix 6-1

Water Treatment Plant Hydraulic Analysis

DRAFT TECHNICAL MEMORANDUM

Prepared by: Howard Hoffman
Reviewed by: Keith Durkin

Date: October 22, 1999
Project No. 992509.00

Subject: **SAN JUAN WATER DISTRICT**
WATER TREATMENT PLANT HYDRAULICS ANALYSIS

PURPOSE

The purpose of this technical memorandum is to:

1. Summarize our review of previous analyses of the water treatment plant hydraulics
2. Review the hydraulics of the existing water treatment plant
3. Recommend improvements to the water treatment plant that would improve hydraulic capacity

HYDRAULICS ANALYSIS

Kennedy/Jenks Consultants reviewed a Technical Memorandum by Montgomery Watson dated January 21, 1999 prior to our own independent analysis. In general, our analysis agrees with most of the Montgomery Watson analysis, although our recommendations for improving the water treatment plant hydraulics are somewhat different.

According to the construction plans, San Juan Water District's Sidney N. Peterson Water Treatment Plant (WTP) was designed in 1977 to have a capacity of 100 million gallons per day (MGD). That project anticipated an addition of filters that were not a part of the original plant design. The subsequent filter addition project had a design capacity of 100 MGD as well, although by current design standards they are considered rated to 120 MGD. Based on our discussions with the WTP staff, the WTP cannot be operated for sustained periods at 120 MGD, even though some hydraulic bottlenecks have been reduced by modifications to the WTP.

Kennedy/Jenks produced a hydraulic profile computer model utilizing an in-house tool called Hypro, which was developed specifically for hydraulic profiling. The underlying calculations are performed as an Excel spreadsheet. A printout of the model can be found in Appendix A. The results of our analysis at a flow of 120 MGD are shown in Figure 1. The discussion that follows will be easier to understand if the reader is familiar with and has at hand Appendix A and Figure 1.

The only non-standard hydraulic elements in the WTP are the Sedimentation Basin Effluent Troughs. These were originally designed as internally hung launders with v-notches, a relatively standard hydraulic element. The Effluent Troughs had an occasional bottom hole, presumably to allow drainage when a basin is dewatered. However, due to the hydraulic limitations at the plant, numerous bottom holes have been added to the Effluent Troughs. According to information obtained from WTP staff, there are a total of 4,688 holes (each 1" diameter), or 2,344 for each of the two main process trains.

The significance of the holes is that they create a variable flow split, with some flow entering the Effluent Troughs through the v-notches (as originally designed) and with the rest of the flow entering through the holes. Our hydraulic model permits an accurate calculation of the flow split.

EXISTING HYDRAULIC PROFILE

The WTP hydraulic profile depends, first of all, on the water surface elevation over the filters. According to the WTP staff, the level over the filters is automatically controlled by the filter controls at the lowest practical elevation. Starting with that assumption, the one factor (besides the flow) that will affect the level over the filters (as currently constructed) is the relative condition of the filters (i.e. how clean they are). The filters are divided into cells, which are being continuously backwashed. The longer the period of time that the filters are operated at a high sustained rate, the higher the head loss through the filters and the higher the water surface over the filters. Therefore, in our hydraulic model, we always start with an assumed elevation for the water level at the filters (Montgomery Watson followed this same approach).

The WTP has an emergency overflow weir (EOW) that is hydraulically connected to the Sedimentation Basin Collection Channel (which receives the flow from the Sedimentation Basin Effluent Troughs). The EOW has an elevation reported to be at 420.20 (based on the WTP datum). When the WTP flow is "too high", flow automatically discharges over the EOW. Discharge over the EOW is non-catastrophic, but this is not a desired condition. The WTP staff has improved erosion protection for the area where the EOW spills to a natural drainage channel.

In order to pass the design flow of 120 MGD through the filters and to have no overflow at the EOW, we calculate the maximum water surface level over the filters to be 419.10 (agreeing with calculations by Montgomery Watson).

When the water surface is at elevation 420.20 at the Sedimentation Basin Collection Channel, the Sedimentation Basin Effluent Troughs are essentially free-flowing (i.e. there is no significant back-up of the flow into the troughs) and there is a considerable drop over the v-notch weirs. This means that any problems at the head end of the plant (from the Sedimentation Basins back to the Rapid Mix Basins) are not the result of too much depth over the filters.

There are a series of head losses from the WTP influent to the Sedimentation Basins. There are 32 12"x16" rectangular openings in the Flocculation Basin Distribution Troughs, and these are responsible for 0.59 ft of head loss. Other significant head losses include 0.47 ft for the sluice gates leading to the Flocculation Basin Distribution Troughs and the 0.57 ft for the rectangular opening between the Rapid Mix Zone 2 and Rapid Mix Zone 1 (one opening for each process train). The rectangular openings were originally 48” x 49”. The WTP correctly identified that as a major bottleneck and expanded the openings to approximately 65” x 49”. Given the turbulence in the mixing zones, the actual head losses may be greater than is calculated here.

While none of these head losses is especially great, the cumulative effect is to raise the water level at the Rapid Mix Zone 1 (a mixing box) to where it sloshes out onto the deck. The turbulence in a mixing box with a mechanical mixer of this size is such that at least a 1.5 ft. freeboard is required to prevent sloshing from reaching the deck. A 2.0 ft. freeboard would be desirable. We calculate a freeboard of 1.23 ft., which is not really adequate. We observed that at high flows some sloshing does occur and some water ends up on the deck and overflows the structure.

If the water level in the Sedimentation Basin rises for any reason, then the sloshing at the Rapid Mix Zone 1 will get worse. So, the additional holes that were drilled in the Sedimentation Basin Effluent Troughs have reduced the sloshing problem in the mixing boxes. However, under typical conditions, the level at the filters will not rise high enough (due to the Emergency Overflow Weir) to back up the flow into the Sedimentation Basins and worsen the sloshing at the mixing boxes. This means that hydraulic improvements to the WTP will need to be made at more than one location.

The flow split between the v-notch weirs and the holes in the Sedimentation Basin Effluent Troughs calculates to be 29% vs. 71% (29% over the v-notches and 71% through the holes). That means that most of the flow is leaving the Sedimentation Basins through the holes, but some flow is still going over the v-notch weirs. This is a desirable condition, because the v-notch weirs are intended to maintain even distribution of flows across the Sedimentation Basins.

SUGGESTED HYDRAULIC IMPROVEMENTS

Emergency Overflow Weir

It appears that the single best way of preventing overflow at the EOW under high flow conditions is to raise the weir elevation, currently at 420.20. Based on our testing of the hydraulic profile model (Appendix B), raising the EOW to 421.00 would allow for an additional 1.0 ft of filter head without overflow. Under that scenario, the Sedimentation Basin water level and the mixing box water levels would not be significantly affected. The water level would be higher in the Sedimentation Basin Effluent Troughs, but the v-notch weirs would not be submerged to any degree.

Under this scenario, the flow split between the v-notch weirs and the holes in the Sedimentation Basin Effluent Troughs calculates to be 39% vs. 61%. However, there would be less head loss through the holes, and the gain from drilling more holes (as suggested by Montgomery Watson) would be diminished.

Issues that would need to be addressed before raising the Emergency Overflow Weir include:

- Structural evaluation to determine if the higher water level would have any adverse impact on any of the structures, including the EOW itself, that would see a higher water level.
- Impact of a higher water level on filter performance.

Additional Holes for the Sedimentation Basin Effluent Troughs

Montgomery Watson suggested that it would be beneficial to drill additional holes in the Sedimentation Basin Effluent Troughs. Montgomery Watson overestimated the actual head loss through the holes by not taking into account the backwater effect of the flow from the Sedimentation Basin Collection Channel and the flow split between the v-notches and the holes. As it is, a majority of the flow already passes through the existing holes, instead of passing over the v-notches as originally designed. The existing holes have played an important role in increasing the hydraulic efficiency of the WTP. However, we do not recommend additional holes. By raising the Emergency Overflow Weir, there will be even less head loss through the holes. The head loss that is incurred through the holes helps to insure flow distribution across the Sedimentation Basins and into the Effluent Troughs.

If the EOW cannot be raised for any reason, then the additional holes should be considered. Montgomery Watson evaluated increasing the number of holes by 10%, 25% or 50%. Each of these options would reduce the water surface in the Sedimentation Basins (and upstream structures) to an increasing degree. However, the benefits would not be as great as calculated by Montgomery Watson. We would be reluctant to increase the number of holes more than 25% because of the possible adverse impact on Sedimentation Basin performance. Even increasing the number of holes by 50% would not permit a 1-ft increase in head over the filters without overflow at the EOW.

Sloshing at the Rapid Mix Boxes

The sloshing that occurs now at flows of 120 mgd or less can be reduced by making the following improvements:

1. Increase the size of the 32 inlet holes in the Flocculation Basin Distribution Troughs
2. Increase the size of the rectangular openings between Rapid Mix Zone 1 and Zone 2 (two openings, one per treatment train)

It is probably not practical to consider increasing the size of the sluice gates between the Rapid Mix boxes and the Flocculation Basin Distribution Troughs. These sluice gates are 72” wide by 48” high gates and the cost of replacing these with larger gates would be very high.

The 32 existing holes in the Flocculation Basin Distribution Troughs are 12” by 16” rectangular openings. If these were enlarged to 16” by 16” (or equivalent) and if they were rounded on the inlet side, they would still provide effective inlet flow distribution to the Flocculation Basins. This modification would reduce the head loss at peak flows from 0.59 ft (at present) down to 0.33 ft., a savings of ¼ ft. Enlarging the holes further would reduce the head loss even more. However, the effectiveness of the Distribution Troughs would be compromised if the head loss were reduced too much.

It will be desirable to look at the construction shop drawings for the Flocculation Basin Distribution Troughs to determine the best way of expanding the existing openings. However, it appears that widening the opening by 4” should have no significant adverse impact on the troughs.

The openings between the Rapid Mix Zones 1 and Zones 2 have already been expanded once. The feasibility of expanding the openings again will require a structural evaluation. Also, the wall between the zones serves a purpose: having two distinct mixing zones, each with its own mixer. This reduces short-circuiting of flow through the mixing zones. Also, the wall insulates each mechanical mixer from the turbulence created by the adjacent mixer. Enlarging the opening further should be reviewed with the mixer manufacturers to determine if the mixers would be adversely affected.

However, assuming that the openings could be widened from 65” to 70” and rounded on the inlet side, then the head loss could be reduced from an existing 0.57 ft down to 0.37 ft.

Appendix C is a printout of the hydraulic model modified to allow for the two modifications recommended above, in addition to raising the Emergency Overflow Weir as previously recommended.

One other approach that could be considered for the Rapid Mix Zone 1 boxes would be to install a raised splashguard around the openings where the water sloshes out. This would have to be done very carefully to make sure that this did not create a tripping hazard.

SUMMARY

In order to increase the reliable sustained hydraulic capacity of the San Juan Water District’s Sidney N. Peterson Water Treatment Plant to 120 MGD, it will be necessary to make some improvements. The recommended improvements include:

- Raise the existing Emergency Overflow Weir elevation from 420.20 to 421.00
- Enlarge and round the 12” x 16” holes in the Flocculation Basin Distribution Troughs
- Enlarge and round the 65” x 48” opening between the Rapid Mix zones

APPENDIXES

A. Hydraulic Profile model of the existing water treatment plant at 120 MGD without overflow at the Emergency Overflow Weir
B. Hydraulic Profile model of the existing water treatment plant at 120 MGD with the Emergency Overflow Weir raised 0.8 ft.
C. Hydraulic Profile model of the existing water treatment plant at 120 MGD with the Emergency Overflow Weir raised 0.8 ft. and with other improvements.

FIGURES

Client	**San Juan Water District**		
Project Title	**WTP Study**		
K/J/C	Job No.	995037.00	BY: HLH
FILE:	Hypro_r2		

on HYPRO release 2.0 by Howard L. Hoffman

AVE. DAILY FLOW =	120.00	MGD
PEAKING FACTOR =	1.00	
PEAK DAILY FLOW =	**120.00**	**MGD**

HYDRAULIC PROFILE FOR **WTP Existing Condition**

******* *********************** ****************** ************ ************ ************* **************

			FLOW (MGD)	HEAD LOSS (FT)	ELEVATION (FT)
PEAK DAILY FLOW =			**120.00**		
Filter Level (PER MW MEMO)					419.10
Flow per Inlet			2.50		
CIRCULAR INLET LOSS:					
DIA.=	16.00	INCHES			
ADD. LOSS=	0.50	VEL. HEADS			
CIRCULAR INLET LOSS =				0.30	
Filter Distribution Channel					419.40
			60.00		
CHANNEL HEAD LOSS:					
BOTTOM WIDTH=	5.30	FEET			
SIDE SLOPE=	0.00	HORIZ/VERT			
BOTTOM ELEV=	414.00				
SURFACE EL=	419.40				
DEPTH=	5.40	FEET			
CR SEC AREA=	28.61	SQ FT			
HYDR RADIUS=	1.78	FEET			
LENGTH=	214.00	FEET			
N=	0.013				
VELOCITY=	3.25	FT/SEC			
CHANNEL HEAD LOSS =				0.08	
Gate Contraction	1.00				

Split Entrance	1.00			
Expansion	0.50			

FORM LOSS=	2.50	x V^2/2G =	0.41	

Sed Basin Collection Channel Discharge	419.89

FULL FLOW FROM BOTH TRAINS		120.00		
CHANNEL HEAD LOSS:				
BOTTOM WIDTH=	5.00	FEET		
SIDE SLOPE=	0.00	HORIZ/VERT		
BOTTOM ELEV=	414.00			
SURFACE EL=	419.89			
DEPTH=	5.89	FEET		
CR SEC AREA=	29.44	SQ FT		
HYDR RADIUS=	1.755	FEET		
LENGTH=	210.00	FEET		
N=	0.013			
VELOCITY=	6.31	FT/SEC		
CHANNEL HEAD LOSS =			0.30	

Sed Basin Collection Channel Mid Point	420.19

Number of Troughs		32.00		
FLOW PER SED BASIN EFFLUENT TROUGH		3.75		
TROUGH FUNCTIONS LIKE A BROAD-CRESTED WEIR @				419.96
WEIR LENGTH =	1.50 FEET			
WEIR BREADTH=	2.00 FEET			
WEIR LOSS W/O SUBMERGENCE=	1.13			
WEIR SUBMERGENCE =	0.23			
WEIR LOSS =			1.14	

Sed Basin Effluent Trough Mid-point	421.10

Flow Split: Through Trough Holes	71%	85.20	
Number of Holes	4,688		
Flow per Hole		0.02	
Assumed Elevation in Sed Basin			421.95
Differential Head for Holes			0.85

CIRCULAR HOLE LOSS:				
DIA.=	1 INCHES			
ADD. LOSS=	0 VEL. HEADS			
CIRCULAR HOLE LOSS =			0.83	
Flow Split: Over V-Notch Weirs		34.80		
Sed Basin Effluent Trough Mid-point				421.10
V-NOTCH WEIR @				421.87
WEIR LENGTH =	5120.00 FEET			
V-NOTCH SPACING	6.00 INCHES			
NO. OF V-NOTCHES	10240			
FLOW PER V-NOTC	0.003 MGD			
WEIR LOSS W/O SUBMERGENCE=	0.08			
WEIR SUBMERGENCE =	0.00			
WEIR LOSS =			0.08	
Sedimentation Basin				421.95
Baffle Walls, assume			0.20	
Flocculation Basins				422.15
Number of Inlet Square Holes	32			
Flow Per Inlet		3.75		
RECTANGULAR HOLE LOSS:				
WIDTH=	16.00 INCHES			
HEIGHT=	12 INCHES			
ADD. LOSS=	0 VEL. HEADS			
RECTANGULAR HOLE LOSS =			0.59	
Flocculation Basin Distribution Trough				422.74
Flow Per Train		60.00		
RECTANGULAR SLUICE GATE LOSS:				
GATE WIDTH=	72 INCHES			
GATE HEIGHT=	48 INCHES			
ADD. LOSS=	0 VEL. HEADS			
RECTANGULAR SLUICE GATE LOSS =			0.47	
Rapid Mix Zone 2				423.20

RECTANGULAR OPENING:		
WIDTH=	65 INCHES	
HEIGHT=	48 INCHES	
ADD. LOSS=	0 VEL. HEADS	
RECTANGULAR OPENING LOSS =		0.57
Rapid Mix Zone 1		423.77

Client	**San Juan Water District**		
Project Title	**WTP Study**		
K/J/C	Job No.	995037.00	BY: HLH
FILE:	C:///San Juan WD/Hydraulic Profile.xls		
on HYPRO release 2.0 by Howard L. Hoffman			

AVE. DAILY FLOW =	120.00	MGD
PEAKING FACTOR =	1.00	
PEAK DAILY FLOW =	**120.00**	**MGD**

HYDRAULIC PROFILE FOR **WTP with Emergency Overflow @421.00**

******* *********************** ****************** ************ ************ ************* **************

				FLOW (MGD)	HEAD LOS: (FT)	ELEVATION (FT)
PEAK DAILY FLOW =				**120.00**		
Filter Level (PER MW MEMO)						420.10
Flow per Inlet				2.50		
	CIRCULAR INLET LOSS:					
	DIA.=	16.00	INCHES			
	ADD. LOSS=	0.50	VEL. HEADS			
	CIRCULAR INLET LOSS =				0.30	
Filter Distribution Channel						420.40
				60.00		
	CHANNEL HEAD LOSS:					
	BOTTOM WIDTH=	5.30	FEET			
	SIDE SLOPE=	0.00	HORIZ/VERT			
	BOTTOM ELEV=	414.00				
	SURFACE EL=	420.40				
	DEPTH=	6.40	FEET			
	CR SEC AREA=	33.91	SQ FT			
	HYDR RADIUS=	1.87	FEET			
	LENGTH=	214.00	FEET			
	N=	0.013				
	VELOCITY=	2.74	FT/SEC			
	CHANNEL HEAD LOSS =				0.05	
	Gate Contraction	1.00				

Split Entrance	1.00			
Expansion	0.50			

FORM LOSS=	2.50	x V^2/2G =	0.29	

Sed Basin Collection Channel Discharge 420.74

FULL FLOW FROM BOTH TRAINS		120.00		
CHANNEL HEAD LOSS:				
BOTTOM WIDTH=	5.00	FEET		
SIDE SLOPE=	0.00	HORIZ/VERT		
BOTTOM ELEV=	414.00			
SURFACE EL=	420.74			
DEPTH=	6.74	FEET		
CR SEC AREA=	33.71	SQ FT		
HYDR RADIUS=	1.824	FEET		
LENGTH=	210.00	FEET		
N=	0.013			
VELOCITY=	5.51	FT/SEC		
CHANNEL HEAD LOSS =			0.22	

Sed Basin Collection Channel Mid Point 420.96

Number of Troughs		32.00		
FLOW PER SED BASIN EFFLUENT TROUGH		3.75		
TROUGH FUNCTIONS LIKE A BROAD-CRESTED WEIR @				419.96
WEIR LENGTH =	1.50 FEET			
WEIR BREADTH=	2.00 FEET			
WEIR LOSS W/O SUBMERGENCE=	1.13			
WEIR SUBMERGENCE =	1.00			
WEIR LOSS =			1.38	

Sed Basin Effluent Trough Mid-point 421.34

Flow Split: Through Trough Holes	61%	73.20		
Number of Holes	4,688			
Flow per Hole		0.02		
Assumed Elevation in Sed Basin				421.95
Differential Head for Holes				0.61

CIRCULAR HOLE LOSS:				
DIA.=	1 INCHES			
ADD. LOSS=	0 VEL. HEADS			
CIRCULAR HOLE LOSS =			0.61	
Flow Split: Over V-Notch Weirs		46.80		
Sed Basin Effluent Trough Mid-point				421.34
V-NOTCH WEIR @				421.87
WEIR LENGTH =	5120.00 FEET			
V-NOTCH SPACING	6.00 INCHES			
NO. OF V-NOTCHES	10240			
FLOW PER V-NOTC	0.005 MGD			
WEIR LOSS W/O SUBMERGENCE=	0.09			
WEIR SUBMERGENCE =	0.00			
WEIR LOSS =			0.09	
Sedimentation Basin				421.96
Baffle Walls, assume			0.20	
Flocculation Basins				422.16
Number of Inlet Square Holes	32			
Flow Per Inlet		3.75		
RECTANGULAR HOLE LOSS:				
WIDTH=	16.00 INCHES			
HEIGHT=	12 INCHES			
ADD. LOSS=	0 VEL. HEADS			
RECTANGULAR HOLE LOSS =			0.59	
Flocculation Basin Distribution Trough				422.75
Flow Per Train		60.00		
RECTANGULAR SLUICE GATE LOSS:				
GATE WIDTH=	72 INCHES			
GATE HEIGHT=	48 INCHES			
ADD. LOSS=	0 VEL. HEADS			
RECTANGULAR SLUICE GATE LOSS =			0.47	
Rapid Mix Zone 2				423.21

RECTANGULAR OPENING:			
WIDTH=	65	INCHES	
HEIGHT=	48	INCHES	
ADD. LOSS=	0	VEL. HEADS	
RECTANGULAR OPENING LOSS =			0.57

Rapid Mix Zone 1	423.78

Client	**San Juan Water District**			
Project Title	**WTP Study**			
K/J/C	Job No.	992509.00	BY:	HLH
FILE:	C:///San Juan WD/Hydraulic Profile.xls			

on HYPRO release 2.0 by Howard L. Hoffman

AVE. DAILY FLOW =	120.00	MGD
PEAKING FACTOR =	1.00	
PEAK DAILY FLOW =	**120.00**	**MGD**

HYDRAULIC PROFILE FOR **WTP with Recommended Improvements**

******* *********************** ****************** ************ ************ ************* **************

			FLOW (MGD)	HEAD LOSS (FT)	ELEVATION (FT)
PEAK DAILY FLOW =			**120.00**		
Filter Level (PER MW MEMO)					420.10
Flow per Inlet			2.50		
CIRCULAR INLET LOSS:					
DIA.=	16.00	INCHES			
ADD. LOSS=	0.50	VEL. HEADS			
CIRCULAR INLET LOSS =				0.30	
Filter Distribution Channel					420.40
			60.00		
CHANNEL HEAD LOSS:					
BOTTOM WIDTH=	5.30	FEET			
SIDE SLOPE=	0.00	HORIZ/VERT			
BOTTOM ELEV=	414.00				
SURFACE EL=	420.40				
DEPTH=	6.40	FEET			
CR SEC AREA=	33.91	SQ FT			
HYDR RADIUS=	1.87	FEET			
LENGTH=	214.00	FEET			
N=	0.013				
VELOCITY=	2.74	FT/SEC			
CHANNEL HEAD LOSS =				0.05	
Gate Contraction	1.00				
Split Entrance	1.00				

Expansion	0.50			

FORM LOSS=	2.50	x V^2/2G =	0.29	

Sed Basin Collection Channel Discharge 420.74

FULL FLOW FROM BOTH TRAINS		120.00		
CHANNEL HEAD LOSS:				
BOTTOM WIDTH=	5.00	FEET		
SIDE SLOPE=	0.00	HORIZ/VERT		
BOTTOM ELEV=	414.00			
SURFACE EL=	420.74			
DEPTH=	6.74	FEET		
CR SEC AREA=	33.71	SQ FT		
HYDR RADIUS=	1.824	FEET		
LENGTH=	210.00	FEET		
N=	0.013			
VELOCITY=	5.51	FT/SEC		
CHANNEL HEAD LOSS =			0.22	

Sed Basin Collection Channel Mid Point 420.96

Number of Troughs		32.00		
FLOW PER SED BASIN EFFLUENT TROUGH		3.75		
TROUGH FUNCTIONS LIKE A BROAD-CRESTED WEIR @				419.96
WEIR LENGTH =	1.50	FEET		
WEIR BREADTH=	2.00	FEET		
WEIR LOSS W/O SUBMERGENCE=	1.13			
WEIR SUBMERGENCE =	1.00			
WEIR LOSS =			1.38	

Sed Basin Effluent Trough Mid-point 421.34

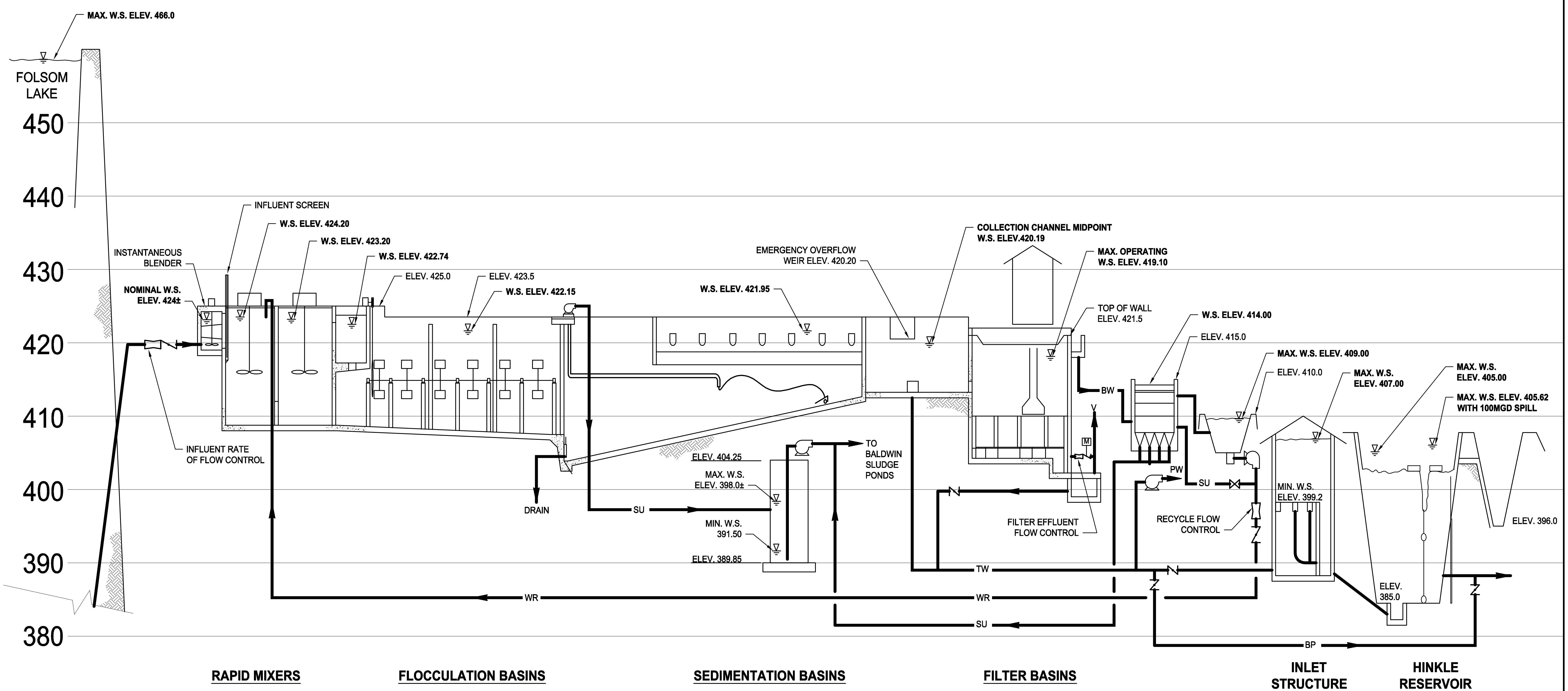
Flow Split: Through Trough Holes	61%	73.20		
Number of Holes	4,688			
Flow per Hole		0.02		
Assumed Elevation in Sed Basin				421.95
Differential Head for Holes				0.61
CIRCULAR HOLE LOSS:				
DIA.=	1	INCHES		

ADD. LOSS=	0 VEL. HEADS			
CIRCULAR HOLE LOSS =			0.61	
Flow Split: Over V-Notch Weirs		46.80		
Sed Basin Effluent Trough Mid-point				421.34
V-NOTCH WEIR @				421.87
WEIR LENGTH =	5120.00 FEET			
V-NOTCH SPACING	6.00 INCHES			
NO. OF V-NOTCHES	10240			
FLOW PER V-NOTC	0.005 MGD			
WEIR LOSS W/O SUBMERGENCE=	0.09			
WEIR SUBMERGENCE =	0.00			
WEIR LOSS =			0.09	
Sedimentation Basin				421.96
Baffle Walls, assume			0.20	
Flocculation Basins				422.16
Number of Inlet Square Holes	32			
Flow Per Inlet		3.75		
RECTANGULAR HOLE LOSS:				
WIDTH=	16.00 INCHES			
HEIGHT=	16 INCHES			
ADD. LOSS=	0 VEL. HEADS			
RECTANGULAR HOLE LOSS =			0.33	
Flocculation Basin Distribution Trough				422.49
Flow Per Train		60.00		
RECTANGULAR SLUICE GATE LOSS:				
GATE WIDTH=	72 INCHES			
GATE HEIGHT=	48 INCHES			
ADD. LOSS=	0 VEL. HEADS			
RECTANGULAR SLUICE GATE LOSS =			0.47	
Rapid Mix Zone 2				422.96
RECTANGULAR OPENING:				
WIDTH=	70 INCHES			

HEIGHT=	48	INCHES	
ADD. LOSS=	-0.5	VEL. HEADS	

RECTANGULAR OPENING LOSS = 0.37

Rapid Mix Zone 1 423.33



LEGEND

BW	BACKWASH
BP	BYPASS
PW	PROCESS WATER
SU	SLUDGE
TR	TREATED WATER
WR	RETURN WATER

DRAFT

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USE OF DOCUMENTS

NO.	REVISION	DATE	BY

SCALES

0 — 1"

0 — 25mm

IF THIS BAR IS NOT DIMENSION SHOWN, ADJUST SCALES ACCORDINGLY.

DESIGNED K. DURKIN

DRAWN D. MATCHELL

CHECKED K. DURKIN

APPROVED XXX

SAN JUAN WATER DISTRICT

WHOLESALE MASTER PLAN

Kennedy/Jenks Consultants

3336 BRADSHAW ROAD, SUITE 140 SACRAMENTO, CALIFORNIA 95827

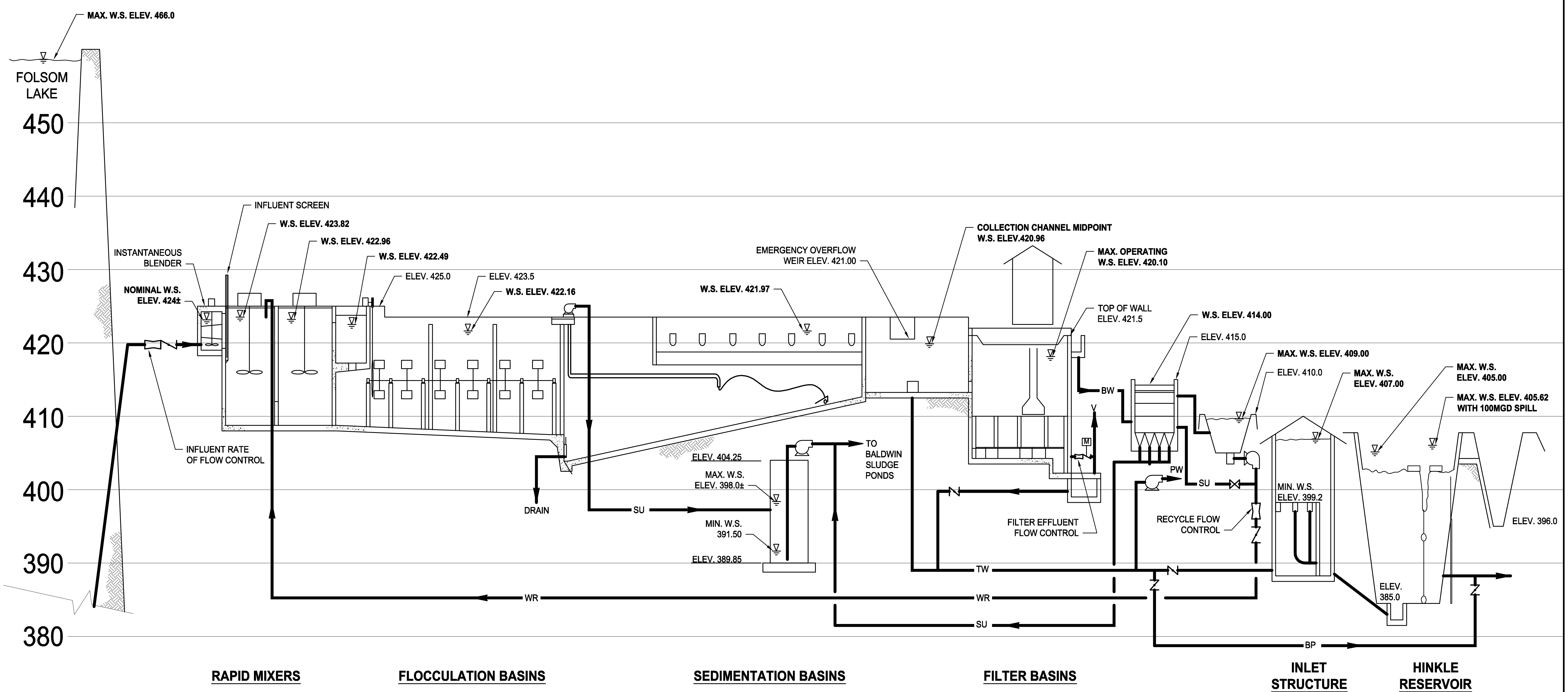
FIGURE 1
EXISTING WATER TREATMENT PLANT
HYDRAULIC PROFILE
AT 120 MGD

FILE NAME HYDRO-PROFLIE

JOB NO. 992509.00

DATE JANUARY 2000

SHEET H1 OF



LEGEND

BW	BACKWASH
BP	BYPASS
PW	PROCESS WATER
SU	SLUDGE
TR	TREATED WATER
WR	RETURN WATER

DRAFT

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DESIGNED K. DURKIN

DRAWN D. MATCHELL

CHECKED K. DURKIN

APPROVED XXX

SAN JUAN WATER DISTRICT

WHOLESALE MASTER PLAN

Kennedy/Jenks Consultants

3336 BRADSHAW ROAD, SUITE 140 SACRAMENTO, CALIFORNIA 95827

FIGURE 2
WATER TREATMENT PLANT
WITH RECOMENDED IMPROVEMENTS
HYDRAULIC PROFILE
AT 120 MGD

FILE NAME HYDRO-PROFLIE-6.2

JOB NO. 992509.00

DATE JANUARY 2000

SHEET H2 OF

Appendix 7-1

Analysis of Long-Term Expansion Alternatives

Appendix 7-1 Analysis of Long-Term Expansion Alternatives

The results of a screening of long-term treatment process alternatives for the expansion of the San Juan Water District's Water Treatment Plant is presented in this Appendix. The screening was conducted in two phases: (1) a preliminary non-economic qualitative evaluation for a wide range of alternatives to identify feasible alternatives, and (2) a quantitative evaluation of the remaining alternatives.

Kennedy/Jenks Consultants and Black & Veatch water treatment experts performed the preliminary evaluation in a workshop to develop the screening criteria and conduct the screening. A subsequent workshop was held with District staff to obtain their feedback and insight. This Appendix incorporates the comments of District staff.

Preliminary Screening

Identification of Preliminary Treatment Technologies

A workshop was conducted by Kennedy/Jenks and Black & Veatch water treatment experts to review treatment technologies and to identify feasible alternatives for further review. The treatment technologies evaluated in the preliminary screening included USEPA-listed treatment technologies and DHS-approved alternative filtration processes. The oxidation-disinfection evaluation included oxidants that may be required in the future.

Evaluation Criteria

The treatment processes were evaluated using the following performance criteria:

- EPA/DHS Approval.
- Site Adaptability.
- Present and Future Regulations.
- Water Quality.
- Operations and Maintenance Requirements.
- Reliability/Proven Technology.
- Compatible with Existing Facility.

Cost issues were deferred to the detailed screening described in the Detailed Screening section of this Appendix. The preliminary screening criteria are discussed below.

EPA/DHS Approval

The USEPA issued the *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems Using Surface Water Sources* in October 1989. This is referred to as the Surface Water Treatment Rule (SWTR). This document defined the multiple barrier concept for treatment of surface water. Four water treatment technologies were listed and given credits for removal of *Giardia* and virus:

- Slow Sand Filtration.
- Diatomaceous Earth Filtration.
- Conventional Filtration.
- Direct Filtration.

DHS has adopted, with some modifications, the SWTR into the California Surface Water Treatment Rule, Title 17.

Any filtration technology, such as membrane filtration, that is not specifically listed in the SWTR must be approved by DHS as an Alternative Filtration Technology (AFT).

The ratings referred to as "EPA listed" are mentioned in the SWTR. A "DHS approved" rating means this technology has been approved by DHS as an AFT for use in treating similar source water.

Site Adaptability

Each alternative treatment technology was reviewed to determine adaptability to the existing site and compatibility with existing processes.

Present and Future Regulations

Each treatment technology was reviewed for the ability to meet existing and future regulations, including *Cryptosporidium* removal/inactivation, reduction of disinfectant by-product precursors, and turbidity standards.

Water Quality

Although the current source water is excellent, consideration was given to future operational changes which could have an impact on treatment processes. For example, if the plant were required to operate as a conventional filtration treatment process with enhanced coagulation, the pretreatment facilities would have to be modified. In addition, higher raw water temperatures in the summer could increase taste and odor complaints and impact compliance with THM and HAA5 MCLs. If the proposed TCD at Folsom Reservoir necessitates operating in an enhanced coagulation mode, then the pretreatment system capacity would have to match the filtration system capacity. If the proposed TCD at Folsom Reservoir necessitates operating in an enhanced coagulation mode, then the pretreatment system capacity would have to match the filtration system capacity.

Operation and Maintenance Requirements

Another important consideration in the selection of a treatment process was whether plant operators could easily integrate the process into existing operations or whether additional personnel would be required to operate and maintain the equipment. The potential impact of power and chemical costs was also considered.

Reliability/Proven Technology

Each alternative treatment technology was reviewed as to whether it has a proven track record for reliable operation and whether there are similarly sized installations in the United States, particularly California and other western states, treating similar source water supplies.

Compatibility With Existing Facility

The existing site offers some limitations for expansion. In fact, expanding with the existing filtration technology, at similar filter loading rates, will require acquisition of additional land. This criterion would apply if the proposed technology could be employed with some land acquisition. Slow sand filtration, for example, could not be constructed due to lack of site space.

Rating System

A rating system was established to provide a logical basis for comparing the above non-economic criteria. The rating system used for the preliminary screening was:

E – Excellent	Satisfies all performance criteria.
G – Good	Satisfies most performance criteria.
F – Fair	Satisfies some performance criteria.
P – Poor	Does not satisfy performance criteria.

Preliminary Screening Results

The results of the preliminary screening are presented in Table 1.

Table 1
Preliminary Screening of WTP Expansion Alternatives

	EPA Listed	DHS-Approved Alternative	Site Adaptability	Present Regs	Future Regs	Water Quality	O&M	Reliability	Proven Technology	Compatible with Existing Facility	Remarks	Carried Forward
Treatment Technology												
Conventional	✓		F	E	E	E	E	E	E	E	Provides most flexibility to meet varying water quality. Generally more expensive to design, build, and operate than other technologies.	Y
Conventional with DAF	✓		G	E	E	F	G	E	E	P	Pilot study required. Incompatible with winter-time turbidity.	N
Direct Filtration	✓		E	F	E	F	E	G	E	E	Seasonal raw water turbidity issue. GAC media may be required for T&O/DBPs. Additional disinfec tion credits may be required.	Y

Table 1 (cont.)
Preliminary Screening of WTP Expansion Alternatives

	EPA Listed	DHS-Approved Alternative	Site Adaptability	Present Regs	Future Regs	Water Quality	O&M	Reliability	Proven Technology	Compatible with Existing Facility	Remarks	Carried Forward
DE Filtration	✓		G	G	G	F	F	G	P	P	Seasonal raw water turbidity issue. No comparable capacity facility.	N
Slow Sand Filtration	✓		P	G	F	P	F	F	P	P	Seasonal raw water turbidity issue. No comparable capacity facility.	N
Serial Filtration		✓	E	E	E	F	F	F	F	P	Peak winter raw water turbidity issue. Additional head required.	N
Ballasted Floc		✓	E	E	E	E	G	F	F	E	Issue with microsand separation system. Pretreatment process only. Limited comparable capacity facilities.	Y
Membrane		✓	E	E	G	E	E	G	F	E	Limited TOC removal. Good candidate for parallel treatment.	Y
Disinfection/Oxidation												
Free Chlorine				E	F/P	E/F	F	E	E	E	D/DBP regulatory compliance depends on TCD impact on TOC. Gas scrubber or liquid solution required.	Y
Chloramine				N/A	E/G	G/F	P	P	G	P	Only necessary if TOC increases.	Y
Chlorine Dioxide				G	G	E/G	F	G	G	G	Chlorite issue. Potential future *Cryptosporidium* disinfectant. DHS-health effects issue.	Y
Ozone				E	E	E	G	E	E	G	Proven *Cryptosporidium* disinfectant.	Y
Ultraviolet Radiation (UV)				N/A	E	E	E/G	G	F	E	Unproven technology in regulatory development. Potential *Cryptosporidium* disinfectant.	Y
Potassium Perman-ganate				N/A	N/A	E	G	G	E	E	Not a disinfectant (oxidant only) so eliminated from further evaluation. Could be used in future if TCD raises TOC and for taste & odor.	N
Backwash Water Recovery Systems												
Ballasted Floc			E	E	E		G	G	G	E	Requires pilot study.	Y
Plate Settlers			E	E	E		E	E	E	E	Recommend pilot study to verify suitable loading rates.	Y

Table 1 (cont.)
Preliminary Screening of WTP Expansion Alternatives

	EPA Listed	DHS-Approved Alternative	Site Adaptability	Present Regs	Future Regs	Water Quality	O&M	Reliability	Proven Technology	Compatible with Existing Facility	Remarks	Carried Forward
Membranes			E	E	E		F	G	G/F	E	Requires pilot study.	Y
Roughing Filters			E	E	E		E/G	E/G	E	E	Requires pilot study.	Y
Residuals Handling												
Sand Beds			P				G	E	E	P	Requires large footprint. Environmental concerns.	N
Belt Press			E/G				G	G	E	E	Proven technology.	Y
Centrifuge			E/G				G	G	E	E	Few applications for WTP solids. Requires pilot study.	Y
Wedgewire			G				F	F	F	F	Requires pilot study.	Y
Wedge Wire with Vacuum			G				F	F	F	G	Requires pilot study.	Y

The retained treatment technologies and disinfectant/oxidation alternatives were further evaluated in a detailed screening process, as described below

Detailed Screening

Identification of Treatment Technologies for Further Evaluation

The treatment technologies carried forward for a detailed screening evaluation are listed below.

Treatment Technology
- Conventional Filtration
- Direction Filtration
- Ballasted Floc
- Membrane Filtration

Oxidation/Disinfection
- Free Chlorine
- Chloramination
- Chlorine Dioxide
- Ozone
- Ultraviolet Radiation (disinfectant only)
- Potassium Permanganate (oxidant only)

Backwash Water Recovery System
- Ballasted Floc
- Plate Settlers
- Submerged Membranes
- Roughing Filter

Residuals Handling
- Belt Presses
- Centrifuges
- Wedgewire (with and without Vacuum)

Evaluation Criteria

A weighted evaluation was performed on the selected treatment technologies based on impact of regulations, source water quality, operations, compatibility with existing plant, and capital and O&M costs. Regulatory impacts and source water quality were given equal and highest weighting. Operations and adaptability/compatibility were given equal but less weight. Economic considerations were given the least weight because the evaluation included only order of magnitude costs and not actual construction costs. The evaluation criteria and associated weighting factors are listed below:

Regulatory Impacts	25%
Source Water Quality	25%
Operations	20%
Adaptability/Compatibility	20%
Costs	10%
	100%

Regulatory impacts, source water quality, and adaptability/compatibility were described in the previous section. The operations category combined reliability of the process with ease of operation and maintenance. Costs included both capital and O&M.

The rating system used included a scale from 1 to 10 as follows:

10	Excellent
9	Very Good
8	Good
6-7	Above Average
5	Average
3-4	Below Average
2	Fair
1	Poor

Table 2 summarizes the detailed screening of these treatment technologies.

Table 2
Summary of Detailed Screening

	Regulatory (25% Weight)					Water Quality (25% Weight)			Operations (20% Weight)				Compatibility with Existing (20% Weight)	Costs (10% Weight)			Weighted Score	Remarks	Carried Forward
	Existing	Future	Health, Safety, ADA	Piloting Required?	Average Score	Turbidity	Organics Removal	Average Score	O&M	Reliability	Flexibility	Average Score	Score	Capital	O&M	Average Score			
Treatment																			
Conventional	10	10	9	10	**9.75**	10	10	**10**	8	8	10	**8.67**	**8**	7	8	**8**	**9.02**	1	Y
Direct Filtration	8	8	8	10	**8.5**	8	9	**9**	9	8	9	**8.67**	**10**	10	10	**10**	**8.98**	2	Y
Ballasted Floc	10	10	8	5	**8.25**	10	10	**10**	7	8	9	**8.00**	**9**	8	7	**8**	**8.51**	3	Y
Membrane (No Pretreatment)	8	7	10	5	**7.5**	10	5	**8**	7	9	7	**7.67**	**7**	6	9	**8**	**7.43**	4	N
Disinfection & Oxidation																			
Chlorine	10	7	5	10	**8**	8	8	**8**	7	10	8	**8.33**	**10**	7	8	**8**	**8.42**		Y
Chloramines	N/A	6	5	5	**5.33**	8	7	**8**	6	8	8	**7.33**	**7**	7	7	**7**	**6.78**		N
Chlorine Dioxide	N/A	8	5	5	**6.00**	8	8	**8**	7	9	9	**8.33**	**7**	7	9	**8**	**7.37**		Y
Ozone	N/A	10	8	5	**7.67**	10	10	**10**	8	8	9	**8.33**	**8**	7	8	**8**	**8.43**		Y
UV (Disinfection Only)	N/A	8	10	5	**7.67**	N/A	N/A	**10**	9	9	9	**9.00**	**9**	8	9	**9**	**8.87**		Y
Backwash Recovery																			
Ballasted Floc	N/A	8	N/A	5	**6.5**	10	10	**10**	8	8	8	**8.00**	**10**	8	8	**8**	**8.53**		Y
Plate Settlers	N/A	8	N/A	10	**9**	10	10	**10**	9	9	9	**9.00**	**10**	10	9	**10**	**9.50**		Y
Membranes	N/A	9	N/A	5	**7**	10	10	**10**	9	9	9	**9.00**	**8**	8	9	**9**	**8.50**		Y
Roughing Filters	N/A	8	N/A	5	**6.5**	8	8	**8**	8	8	8	**8.00**	**10**	10	9	**10**	**8.18**		Y
Residuals Handling																			
Belt Press	N/A	8	N/A	5	**6.5**	8	8	**8**	8	8	8	**8.00**	**10**	10	7	**9**	**8.08**		Y
Centrifuge	N/A	8	N/A	5	**6.5**	8	8	**8**	8	8	8	**8.00**	**10**	10	6	**7**	**8.03**		Y
Wedgewire	N/A	8	N/A	5	**6.5**	8	8	**8**	8	8	8	**8.00**	**6**	6	6	**6**	**7.03**		N

Remarks:

1. Site constraints require that plate settlers be used. Enhanced coagulation could be required if source water TOC exceeds 2 mg/L. Chemical usage and solids production will be higher. Pretreatment facilities ultimate plant capacity will be limited in the conventional filtration mode.
2. For TOC above 2 mg/L, conventional treatment with enhanced coagulation could be required. Seasonal high raw water turbidity would require operation at low rates or pre-treatment ahead of filters.
3. Although promising, this process was eliminated because pilot testing will be required, there is a limited number of suppliers, and experience at this capacity is limited.
4. TOC removal capacity is limited. The process is less sensitive to water quality variations and is preferred as a parallel treatment process, not as an integrated process. There is limited capacity in plants with this process.

Analysis of Treatment Technologies

Conventional Filtration

Conventional Filtration treatment was given the highest rating because it is currently used at the plant and because of its adaptability to future regulations and changing raw water quality. This process could also be run with enhanced coagulation if TOCs increase.

The cost of conventional filtration treatment is higher than for a direct filtration process due to the sedimentation basin and sludge removal equipment. The existing sedimentation basins could be equipped with plate settlers (i.e., stacks of inclined plates), which occupy about 10 percent of the area needed for conventional settling basins. Inclined plate settlers were developed in Europe, and installations in large facilities in Europe are over 30 years old. Inclined plates provide for increased surface area for floc to accumulate, reduce basin short circuiting, and have no moving parts requiring maintenance. Conservative designs for plate sedimentation report loading rates of 4 gpm/sf (gross area below plates). Actual installations show good process performance at nearly double that rate.

Direct Filtration

Direct filtration was given the second highest rating because it is currently being used as a seasonal process and requires less space than conventional filtration treatment. The process would become less attractive if TOC and turbidity increase. As detention times decrease, the process will become more sensitive to changes in water quality. Direct filtration receives less *Giardia* and virus removal credit than conventional filtration. Therefore, additional *Giardia* and virus inactivation is required.

Ballasted Floc

Ballasted-floc sedimentation, also known by its trade name ACTIFLO, is a proprietary process developed by Kruger. Microsand is introduced with the primary coagulant and polymer and, as the flow passes through the flocculation step, the microsand produces a “ballast” for the forming floc that settles out much more readily. The microsand is separated from the floc by pumping through a hydrocyclone. The segregated microsand is returned to the process stream, and the removed floc is sent to the solids handling system. Kruger claims basin loading rates, using plate settlers in the sedimentation tanks, as high as 20 gpm/sf or higher. This results in a footprint for the basin that is four or five times less than a high-rate sedimentation basin. This could be attractive to the District due to limited site space available for basin expansion.

Ballasted-floc sedimentation technology is not new, but it does not have an O&M track record in the United States. We recommend considering ballasted floc for future evaluation during the preliminary design of the plant expansion. By that time, it could be expected that more large facilities may be operating in the United States.

Membrane Filtration

Smaller utilities are increasingly using membrane filtration processes for surface water treatment. The interest in membrane filtration is partially due to the SWTR requirement for conventional filtration that the filtered water turbidity not exceed 0.5 NTU for 95 percent of the samples. In addition, many utilities have an internal guideline to meet a turbidity limit of 0.1 NTU to maximize the removal of microbial contaminants such as *Giardia* cysts and *Cryptosporidium* oocysts. Membrane filtration can produce water with very low turbidity. Other advantages include ease of operation and effective treatment of *Giardia* and *Cryptosporidium* without the addition of coagulant chemicals.

Larger utilities are also converting to membranes, primarily due to ease of operation and good turbidity/cyst removal. Costs for membranes have dropped significantly in the past 5 years.

Membrane filtration processes use a relatively thin media to filter water primarily by sieving action based on size exclusion. Dissolved materials, such as color or DBP-precursors, are not removed. In some cases, chemical pretreatment can increase the removal of dissolved material. Because of the uncertainty related to removal of dissolved materials, pilot testing of membranes is recommended prior to selection of this alternative.

Since the District has an existing plant that can meet most water quality challenges and an existing, well-trained staff, and since we estimate the cost of converting to membranes for the full 150 mgd capacity would be over $45,000,000, membranes are not deemed suitable as an expansion option.

Should the District desire to site a remote WTP or parallel WTP on the same site, then the ease of construction/operation would justify consideration of membranes for this source water.

Conclusions and Recommendations

As a result of the detailed screening, conventional treatment and direct filtration were determined to be the recommended treatment technologies. Ballasted floc should be considered in the future if additional operational data becomes available and if facility costs decrease. Membrane filtration was eliminated from further consideration because of (1) limited experience in plants of this size, (2) higher capital costs, and (3) the need for actual pilot data to allow for proper process sizing.

Analysis of Oxidation/Disinfection Alternatives

Several chemical oxidants are used in the drinking water industry, including free chlorine, chlorine dioxide, ozone, and potassium permanganate. Oxidants are used for control of certain taste and odor problems, for oxidation of iron and manganese, for improving the filterability of water, and for oxidation of organic compounds. The plant does not currently practice oxidation per se, but the free chlorine fed prior to pretreatment satisfies the oxidant demand of the raw water.

Chemical disinfection and irradiation with ultraviolet (UV) light are two techniques used for disinfection. The vast majority of water utilities use chemical disinfection, employing free chlorine, chloramine, ozone, or chlorine dioxide. Of these, free chlorine is used most commonly, with chloramine next in popularity. The District uses free chlorine in the form of chlorine gas.

Free Chlorine

Free chlorine is the most commonly used oxidant and disinfectant. Chlorine reacts with natural organic compounds to form DBPs and is not a proven disinfectant for *Cryptosporidium*. The District benefits by having a pristine watershed with high quality source water and highly effective water filtration as barriers to the passage of *Cryptosporidium* into the finished drinking water. Some research conducted at 22^{o}C indicates that free chlorine is more effective in inactivating *Crypto-sporidium* when it is followed by chloramination. Additional testing needs to be performed at lower water temperatures to confirm these findings.

Chlorine is also a strong oxidizing agent. Its effectiveness can be influenced by pH due to chlorine speciation. Some taste and odor compounds require a stronger oxidant than free chlorine. Pre-chlorination has proven beneficial for filtration of some waters.

The existing gaseous chlorine system, if retained, would need modifications to bring it into compliance with existing codes.

Chloramines

Chloramination is accomplished by combining free chlorine with ammonia or an ammonium salt to form chloramines. Chloramines are not as strong as chlorine when used as primary disinfectants, and are not recommended as primary disinfectants by the USEPA. Chloramines do, however, form a persistent disinfectant residual and are used by numerous water utilities for maintenance of a residual in the distribution system. Because they are slower to react with substances on the walls of water mains, chloramines have a better opportunity to penetrate tubercles and biofilms and then to kill bacteria sheltered or hidden in them.

Chloramines have been tested in research on sequential disinfection for the inactivation of *Cryptosporidium*, in which a strong disinfectant such as free chlorine, chlorine dioxide, or ozone is used first and then followed by an extended period of contact with chloramines. Preliminary research suggests that sequential disinfection with chloramines as the second disinfectant is more effective against *Cryptosporidium* than the use of only a strong disinfectant.

Since the District has not had difficulty maintaining chlorine residuals in its distribution system, has not had troubles with disinfection byproducts, and blends surface water with other supplies, chloramines would not be necessary at this time.

Chlorine Dioxide

Chlorine dioxide is a highly effective disinfectant that equals or exceeds free chlorine in its inactivation capabilities for bacteria, viruses, and *Giardia* cysts. It has been shown to be very effective for *Cryptosporidium*, with limited disinfection byproducts created; however, more studies are needed. The USEPA has not published chlorine dioxide CT tables for *Cryptosporidium*.

A disadvantage of chlorine dioxide is the relatively low MRDL established by the USEPA. This level was set to control the formation of chlorite in water distribution systems, which is a breakdown product of chlorine dioxide. The Stage 1 D/DPR set an MCL of 1.0 mg/L for chlorite. Formation of chlorite is of concern, as evidenced by establishment of an MRDL.

DHS has indicated a willingness to consider chlorine dioxide for water treatment in California; however, current experience only includes an experimental basis at one treatment plant and a limited extended period at another small capacity plant.

Chlorine dioxide is an excellent oxidant and is effective for controlling taste and odor episodes. However, if a chlorine dioxide residual is carried into the distribution system, or if use of chlorine dioxide is followed by free chlorine in the distribution system, odor problems could occur in homes, particularly those with new carpet, because formaldehyde vapors mix with gaseous chlorine dioxide. Use of chloramines as the distribution system disinfectant could prevent this type of problem. If chlorine dioxide is used in the future, consideration of chloramines may be advantageous to the District.

The very low organic content of Folsom Lake water makes chlorine dioxide a suitable alternative disinfectant for the District's WTP.

Ozone

Ozone is the most powerful oxidant and disinfectant available to the water industry. It has been demonstrated to be more effective than any other chemical disinfectant against both *Giardia* and *Cryptosporidium*. When used as a pre-oxidant before coagulation and filtration, ozone improves the effectiveness of filtration in many waters. Preoxidation using ozone has been shown to improve filtered water quality in terms of lower turbidity and lower particle counts.

Other advantages of ozone are its ability to combat tastes and odors and its ability to break down organic matter so it can be removed from water by biological filtration.

Disinfection research suggests that ozone followed by chloramines is more effective for inactivating *Cryptosporidium*; however, this effect has not been evaluated in natural waters. Some more recent tests do not consistently indicate a benefit to chloramination after ozonation. As a result, development of disinfection regulations for *Cryptosporidium* will be difficult to complete until further research is performed to explain the differences or to demonstrate consistency.

An important requirement of the SWTR is that a disinfectant residual must be maintained in the distribution system of water utilities treating surface water. Because ozone dissipates rapidly and a consistent long-term residual cannot be maintained, free chlorine, chloramine, or chlorine dioxide must be added to achieve the desired residual.

Ozone is currently the most expensive disinfectant to install; however, as more ozone systems are installed, costs can be expected to decrease. Application of ozone would require installation of ozone generation facilities and contact basins, which would occupy a significant amount of space near head end of the plant.

Ultraviolet Radiation

This promising disinfection technology is not yet approved by the USEPA. As a disinfectant, UV irradiation would be applied to filtered water. Turbidity in the water could attenuate the UV radiation and thus reduce the effectiveness of the process. This problem could be avoided by applying the UV treatment after water has been filtered and when turbidity is minimal.

UV equipment is currently only cost effective for small-scale plants; however, large-scale UV disinfection equipment is currently in development. Larger scale units would consist of groups of UV lamps placed inside a large pipe so that all of the water to be treated would flow by the UV equipment. This would eliminate the need to have numerous UV disinfection units piped in parallel, with the attendant problems of equalizing and measuring flow.

Since UV has such promise for inactivation of *Cryptosporidium*, we recommend that space be allocated for its possible future installation.

Conclusions and Recommendations

Based on the detailed screening analysis, free chlorine is the recommended oxidant and disinfectant because of its proven track record and current usage. The existing system uses chlorine gas; however, there are newer, safer technologies available that could be considered. In addition, chlorine dioxide (and chloramines), ozone, and UV should be considered if future regulations require additional inactivation of *Cryptosporidium.*

Analysis of Backwash Water Recovery Systems

Plate Settlers

This system is very similar to the main treatment process using coagulation, flocculation, and a sedimentation basin with inclined plate settlers. The treated backwash water would be returned to the head of the plant, as required by DHS. This technology has a proven track record in this application. Recycled water will meet the requirements of the Filter Backwash Rule.

Ballasted Floc Sedimentation (ACTIFLO)

This proprietary system was described in the Analysis of Treatment Technologies section, above. This system has been strongly considered by many utilities, including East Bay Municipal Utilities District (EBMUD), to help gain compliance with the Filter Backwash Rule. This system shows good performance with variable influent water quality. The treated backwash water would be returned to the head of the plant, as required by DHS.

Submerged Membrane

In this system, backwash water would be filtered by membranes, filtered backwash water would be chlorinated, and then the treated backwash water returned to the head of the plant. There have been some scale-up problems from pilot to full-scale membrane installations, so if submerged membranes are selected, design flux rates need to be carefully selected.

Roughing Filter

In this system, a coarse media is used to reduce backwash turbidity to acceptable levels. A coagulant is fed to the backwash water to assist in solids separation. The treated backwash water is returned to the head of the plant, as required by DHS. Although industry experience with this technology for wash water treatment is limited, Kennedy/Jenks has designed these units at several installations.

Conclusions and Recommendations

All four technologies are promising and should be evaluated in more detail, including consideration of pilot-scale evaluation of their performance. While all four technologies are applicable, only the inclined plate settler could be designed without a pilot study. Black & Veatch has designed plate settlers for backwash recovery systems at several large plants, including the 96 mgd Mesa, Arizona, WTP and the 240 mgd Detroit Water Park WTP, Detroit, Michigan. For master planning purposes, inclined plate settlers are the assumed approach.

Analysis of Residual Handling Technologies

The existing solids handling facilities cannot be reasonably expanded because of the large land area that would be required and because of the limited land adjoining the existing site. No other large land areas are available near the existing WTP for construction of similar low-tech approaches to solids handling. Consequently, mechanical dewatering facilities are recommended at the existing site with onsite storage and eventual disposal in a landfill. All dewatering options would require sludge flow equalization and thickening (2 to 3 percent solids) prior to dewatering.

Belt Presses

Under this approach, thickened solids would be applied to belt presses for dewatering. A belt press consists of two belts running over a series of rollers. Sludge is pressed between the two belts as the pressure increases between the sequence of rollers. Dewatered sludge, typically 20 to 24 percent solids by weight, can be discharged into a dump truck for transport or dropped onto a slab for pickup and transported by a loader to a storage pile for further dewatering. The belt presses and auxiliary facilities would be housed in a new structure.

Centrifuges

Centrifuges provide dewatering of thickened sludge by high-speed rotation within a drum. A helical scroll scrapes the dewatered solids from the centrifuge as it rotates. Dewatered sludge, typically 24 to 26 percent solids, can be stockpiled locally for further dewatering. Although the higher solids content cake generated by a centrifuge requires more power than a belt press, centrifuges are a proven, reliable technology. Construction costs for belt presses and centrifuges are similar. The centrifuges and auxiliary systems would be housed in a new structure.

Wedgewire (With and Without Vacuum)

Wedgewire blocks are placed in shallow concrete basins with a lower plenum to collect water draining from the sludge applied to the top of the wedgewire. Sludge cake is typically concentrated to 8 to 12 percent solids within two to three days. Improved dewatering can be obtained by providing a vacuum assist, resulting in a cake with 14 to 16 percent solids within two to three days. This low solids concentration would require further dewatering on the existing drying beds and constant double handling of the material with a front-end loader. This operation would be difficult to implement at the existing site and would require extensive manual operator and maintenance attention.

Conclusions and Recommendations

Wedgewires would not be appropriate for residuals handling because they would be difficult to implement at the existing site and would require extensive manual labor. For these reasons, wedgewires were eliminated from further consideration. Due to limited site space and lack of capacity at the District’s residual processing facility across Auburn-Folsom Road, technologies that provide reliable, thickened sludges are preferred. Belt presses and centrifuges are reliable technologies that should be considered further. For purposes of site planning, belt presses will be used.

Chemical Feed Systems

The existing chemical feed systems (i.e., liquid alum, quicklime, and polymer) will be expanded as required for capacity. The only new chemicals required for initial expansion are polymers for sludge conditioning/processing. We recommend that the existing cationic polymer system be returned to service to serve as a coagulant aid.

Conclusions

As a result, the following options were retained for the plant capacity expansion:

Treatment Technology
- Conventional
- Direction Filtration
- Ballasted Floc Sedimentation

Disinfection/Oxidation
- Free Chlorine
- Chlorine Dioxide
- Ozone
- Ultraviolet Radiation

Backwash Water Recovery System
- Ballasted Floc
- Plate Settlers
- Membranes
- Roughing Filters

Residuals Handling
- Belt Press
- Centrifuge

Appendix 7-2

Kennedy/Jenks Consultants Memorandum
Summarizing Pretreatment Alternatives Evaluation

30 March 2001

MEMORANDUM

To: San Juan Water District

From: Keith Durkin

Subject: San Juan Water District Wholesale Master Plan
K/J 992509.00 file 6.01

This memorandum summarizes pretreatment alternatives for increasing WTP capacity evaluated as part of the Master Plan. These are submitted for your review and input. Each alternative has not only a different capital cost associated with it, but also different levels of reliable conventional treatment capacity and redundancy.

Three alternative approaches to increasing the treatment capacity of the pre-treatment processes (rapid mix-flocculation-sedimentation) were evaluated. The alternatives included combinations of constructing a third rapid mix (coagulation)-flocculation-sedimentation treatment train parallel to the existing two pretreatment trains, and/or constructing modifications to the existing two pretreatment trains to gain capacity. The alternatives are summarized as follows and further described below. Estimates of the cost of construction for each alternative are provided at the end of this memorandum.

- Alternative 1: Modify existing pretreatment basins to correct deficiencies and increase conventional treatment capacity to approximately 50 MGD for each basin. Initial total conventional treatment capacity will be 100 MGD. When capacity needs increase, construct third pretreatment basin similar to the existing two, for a total conventional treatment capacity of 150 MGD.

- Alternative 2: Construct a third rapid mix-flocculation-sedimentation basin with a conventional treatment capacity of 60 MGD. Replace equipment in the existing pretreatment basins to correct deficiencies. Conventional treatment capacity of the existing pretreatment basins will remain 30 MGD each. Total conventional treatment capacity will be 120 MGD.

- Alternative 3: Modify the existing pretreatment basins to increase capacity of each to 60 MGD, for a total conventional treatment capacity of 120 MGD.

The evaluation of the three following pre-treatment facility improvement alternatives include providing at least 15 minutes of flocculation time for Direct Filtration treatment and could require increasing the coagulant dose to at least 20 mg/L of alum to form settleable floc particles when the plant operates in a Conventional Filtration treatment mode and is treating source water with turbidity above 2 NTU. Adding a third flocculation-sedimentation basin would require adding a fourth coagulant (alum) metering pump and additional non-ionic polymer feed pipelines, but would not require replacing the three existing alum metering pumps when plant capacity exceeds 150 MGD.

Alternative 1

Pre-treatment capacity of the existing basins would be increased by replacing the existing 2-feet deep tube settler modules with 1) new 4-feet deep tube settler modules in the first (deepest) 126 feet of each of the two sedimentation basins and 2) new 2-feet deep tube settler modules in the last (shallowest) 50 foot section in each sedimentation basin. The existing flocculation basins provide 15 minutes of hydraulic residence time for flow rates of nearly 70 MGD per flocculation-sedimentation basin train. This approach would require modifications to the existing flocculation basins to increase performance during direct filtration treatment modes. A sketch depicting these modifications is attached.

Recommended improvements are summarized as follows:

1. Replace the existing rapid mix coagulation system with a jet mix coagulation system.

2. Replace the existing flocculation basin horizontal turbines with new horizontal paddle flocculators. The horizontal paddle flocculators would be designed to provide higher mixing energies to form small filterable pin floc during the summer when source water turbidity is low and conventional filtration is not required.

3. Install redwood walls between each of the five parallel flocculation trains to improve flocculation performance.

4. Install a perforated flow distribution wall between each flocculation basin and the adjacent sedimentation basin similar to the existing perforated walls between existing flocculation zones 1 and 2 and zones 2 and 3.

5. Replace the existing sedimentation basin 2-foot deep tube settler modules with 1) new 4-feet deep tube settler modules in the first (deepest) 126 feet of each of the two sedimentation basins and 2) new 2-feet deep tube settler modules in the last (shallowest) 50 foot section in each sedimentation basin.

6. Replace the existing 18-inch by 21-inch launders with 24-inch by 24-inch launders.

7. Construct a new settled water conveyance channel on the north side of the two existing rapid mix, flocculation and sedimentation basins to provide additional hydraulic capacity to at least 240 MGD to accommodate initial and future conventional and direct filtration treatment capacity requirements.

8. Items 1 through 7 would permit increasing the existing conventional filtration treatment pretreatment capacity from 60 MGD to about 100 MGD (50 MGD per flocculation-sedimentation train).

9. To further increase conventional filtration treatment capacity, a third rapid mix, flocculation and sedimentation train should be constructed on the north side of the two existing rapid mix, flocculation and sedimentation basins. The third pretreatment train would be similar to the existing (modified) basins. This would provide a conventional filtration treatment

pretreatment capacity of at least 150 MGD with all three flocculation-sedimentation basins in service.

Alternative 2

A second approach to increasing pre-treatment capacity would be to construct a third rapid mix-flocculation-sedimentation train to the north of the existing northern rapid mix-flocculation-sedimentation train as part of the initial (LT 75/150) plant improvements. The design of the new rapid mix-flocculation-sedimentation train would be different than the design of the two existing basins to provide at least 80 MGD of additional rapid mix-flocculation-sedimentation capacity. This approach would not require modifying the existing sedimentation basins to increase conventional filtration treatment pretreatment capacity, however the new paddle flocculators and redwood baffles in the existing flocculation basins described in Alternate 1 above should be installed to improve direct filtration treatment performance.

Recommended improvements for Alternative 2 are summarized as follows:

1. To further increase conventional filtration treatment pretreatment capacity, a third rapid mix, flocculation and sedimentation train would be constructed on the north side of the two existing rapid mix, flocculation and sedimentation basins. The third pretreatment train would have deeper (4-foot) tube settler modules to provide capacity of 60 MGD for the third basin. This would provide a total conventional filtration treatment pretreatment capacity of at least 120 MGD with all three flocculation-sedimentation basins in service.

2. Construct a new settled water conveyance channel on the north side of the two existing rapid mix, flocculation and sedimentation basins between the existing and new pretreatment basins. The channel should be sized to provide additional hydraulic capacity to at least 240 MGD to accommodate initial and future conventional and direct filtration treatment capacity requirements.

3. Replace the existing rapid mix coagulation system with a jet mix coagulation system.

4. Replace the existing flocculation basin horizontal turbines with new horizontal paddle flocculators. The horizontal paddle flocculators would be designed to provide higher mixing energies to form small filterable pin floc during the summer when source water turbidity is low and conventional filtration is not required.

5. Install redwood walls between each of the five parallel flocculation trains to improve flocculation performance.

6. Install a perforated flow distribution wall between each flocculation basin and the adjacent sedimentation basin similar to the existing perforated walls between existing flocculation zones 1 and 2 and zones 2 and 3.

7. Replace the existing sedimentation basin 2-foot deep tube settler modules with new 2-feet deep tube settler modules. (The existing modules are brittle and near the end of their useful life.)

8. Replace the existing 18-inch by 21-inch launders with 24-inch by 24-inch launders.

9. Items 1 through 8 would permit increasing the existing conventional filtration treatment pretreatment capacity from 60 MGD to about 120 MGD.

Alternative 3

The third approach to increasing pre-treatment capacity would be to modify the existing flocculation basins to provide at least 15 minutes of flocculation time at the maximum direct filtration treatment flow rate and to also modify the sedimentation basins to increase the conventional filtration treatment pre-treatment capacity to at least 120 MGD. A sketch depicting these modifications is attached.

Recommended improvements for Alternative 3 are summarized as follows:

1. Replace the existing rapid mix coagulation system with a jet mix coagulation system.

2. Increase the capacity of the existing flocculation basins by modifying the first 28 feet of the sedimentation basin transition zone to increase the length of each flocculation basin from 87 feet to 115 feet. This will provide 15 minutes of flocculation time at the maximum 240 MGD direct filtration treatment flow rate. This modification would also require relocating the existing sedimentation basin cleanout connection to the new flocculation basin-sedimentation basin interface.

3. Convert each of the two existing flocculation basins with five 3-stage tapered flocculation units to four separate parallel flocculation trains. Replace the existing flocculation basin horizontal turbines with new horizontal paddle flocculators. The horizontal paddle flocculators would be designed to provide higher mixing energies to form small filterable pin floc during the summer when source water turbidity is low and conventional filtration is not required.

4. Install redwood walls between each of the four parallel flocculation trains to improve flocculation performance.

5. Install a perforated flow distribution wall between each flocculation basin and the adjacent sedimentation basin similar to the existing perforated walls between existing flocculation zones 1 and 2 and zones 2 and 3.

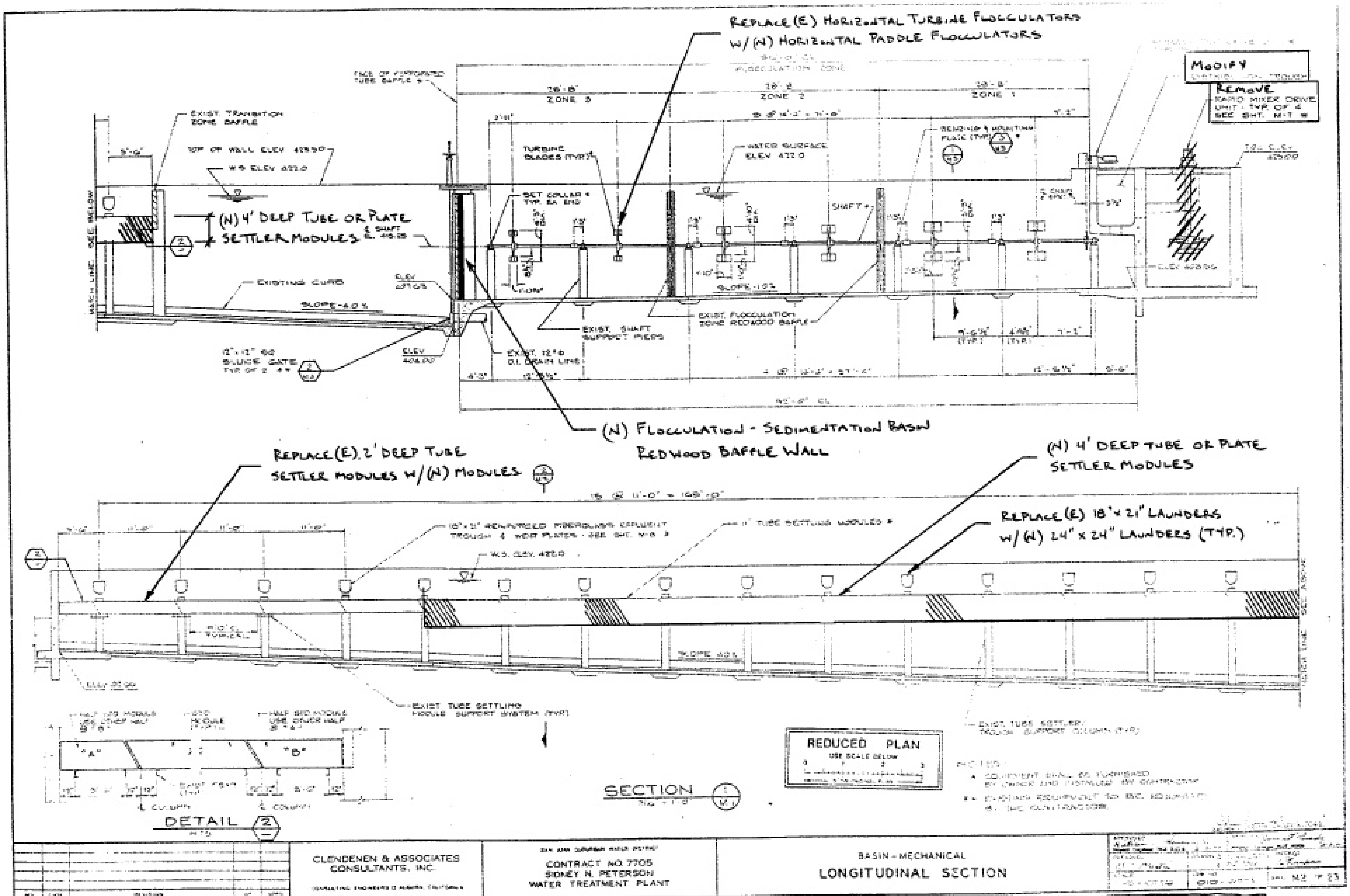
6. Deepen both of the two existing sedimentation basins to provide 18-feet side water depth to accommodate 14-foot long plate settlers. The plate settlers would permit increasing the sedimentation basin surface loading rate and the conventional filtration treatment pretreatment capacity from 60 MGD to at least 120 MGD.

7. Construct a new settled water conveyance channel on the north side of the two existing rapid mix, flocculation and sedimentation basins between the existing and new pretreatment basins. The channel should be sized to provide additional hydraulic capacity to at least 240 MGD to accommodate initial and future conventional and direct filtration treatment capacity requirements.

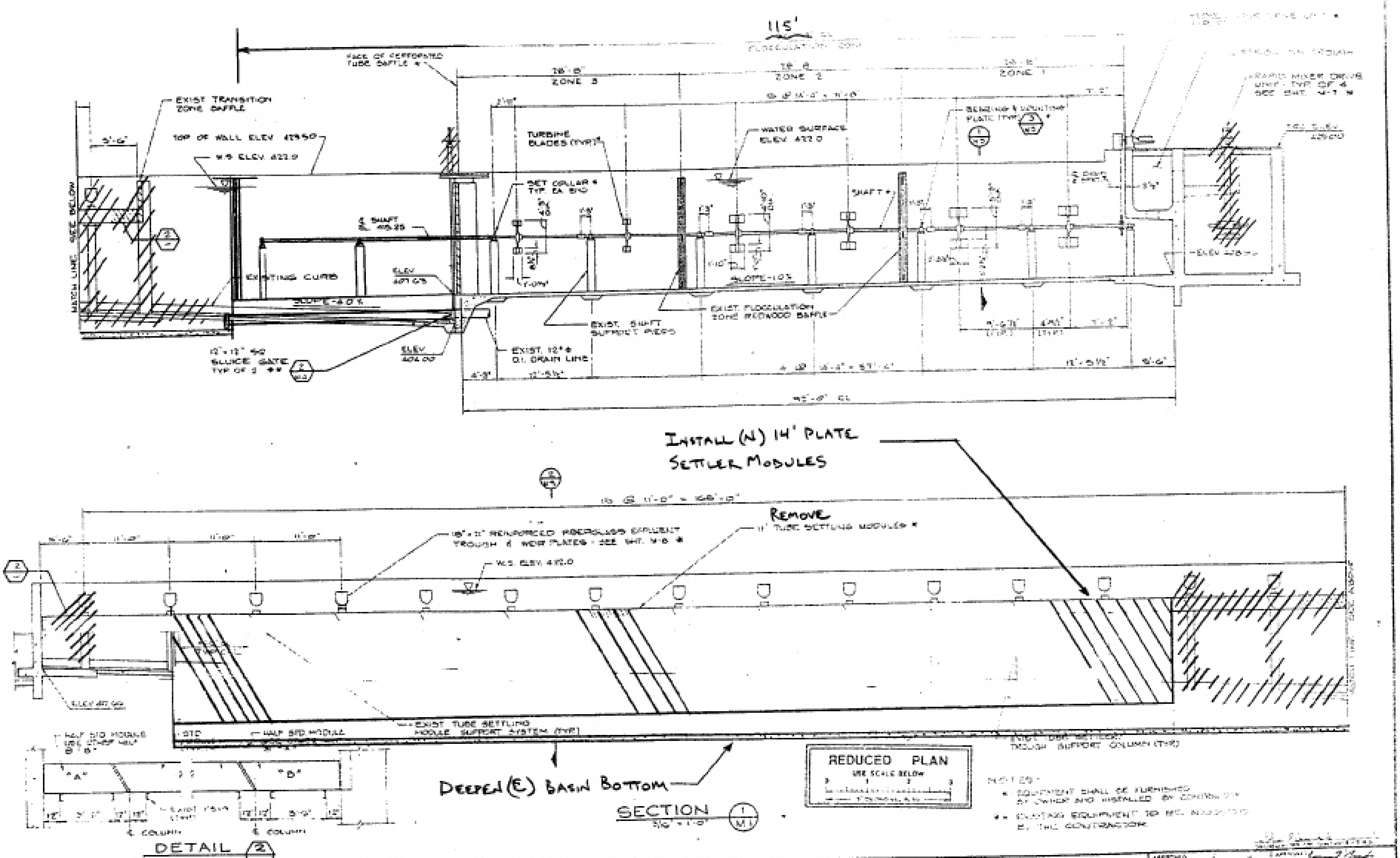
Enclosure(s) (#)

Table 7-5
Conceptual Level Capital Cost Estimate
Flocculation - Sedimentation Basin Comparisons

Alternative	LT 120/240 WTP Capacity Expansion, mgd				Total Cost
	120-150	150-180	180-210	210-240	
Replace equipment in existing basin. Add 3rd basin in future					
Coagulation, rapid mix	$240,000	$110,000	$439,000		$789,000
Modify Flocculation Basin	1,088,000				1,088,000
Modify Sedimentation Basins	3,086,000				3,086,000
Additional effluent & equalization channels	1,919,000				1,919,000
Third Basin w/4' tube settlers			4,534,000		4,534,000
Instrumentation & Electrical	760,000	14,000	597,000		1,371,000
Contingency @ 25%	1,773,250	31,000	1,392,500		3,196,750
Total	**$8,866,250**	**$155,000**	**$6,962,500**		**$15,983,750**
Add Third Basin w/4'tube settlers and min. mods. Exist. Basins now.					0
Coagulation for new basin	$439,000				$439,000
Flocculation & Sedimentation Basin, 4'plates	4,534,000				4,534,000
Modify exist floc basin, walls, floc equip.	932,000				932,000
Modify exist set basin, launders & tubes	2,594,000				2,594,000
Modify Coagulation to exist basins	240,000	110,000			350,000
Additional effluent & equalization channels	1,919,000				1,919,000
Instrumentation & Electrical	1,279,000	14,000			1,293,000
Contingency @ 25%	2,984,250	31,000			3,015,250
Total	**$14,921,250**	**$155,000**			**$15,076,250**
					0
Demo & Build within Existing Flocculation - Sedimentation Basins, Install 14' plate settlers					0
Coagulation	$240,000	$110,000			$350,000
Modify Flocculation Basin	1,788,000				1,788,000
Modify Sedimentation Basins	11,752,000				11,752,000
Additional effluent & equalization channels	1,919,000				1,919,000
Instrumentation & Electrical	1,884,000	14,000			1,898,000
Contingency @ 25%	4,395,750	31,000			4,426,750
Total	**$21,978,750**	**$155,000**			**$22,133,750**



PRETREATMENT ALTERNATIVE 1



CLENDENEN & ASSOCIATES CONSULTANTS, INC. CONSULTING ENGINEERS · AUBURN, CALIFORNIA	SAN JUAN SUBURBAN WATER DISTRICT CONTRACT NO. 7705 SIDNEY N. PETERSON WATER TREATMENT PLANT	BASIN - MECHANICAL LONGITUDINAL SECTION

SHT. M2 OF 23

PRETREATMENT ALTERNATIVE 3

Appendix 8-1

Initial Inspection Report and Cover Evaluation

R.K. Frobel & Associates
Geosynthetics Consulting Engineers

Mr. Keith B. Durkin, P.E.
Vice President
Kennedy/Jenks Consultants
3336 Bradshaw Road, Suite 140
Sacramento, CA 95827

September 21, 1999

RE: San Jaun Water District, Granite Bay, CA
Sidney N. Peterson Water Treatment Plant
Hinkle Reservoir Hypalon Floating Cover Evaluation Report

Dear Mr. Durkin:

At the request of Kennedy/Jenks Consultants, I accompanied Mr. Keith Durkin on a site visitation and floating cover inspection/evaluation at the San Jaun Water District on September 13, 1999. The following is a summary of the site observations and recommendations as related to the Hypalon Floating Cover on the Hinkle Reservoir.

Introduction

The floating cover system on the Hinkle Reservoir is now 20 years old and is composed of 45 mil (1.14mm) thick Hypalon or Chlorosulfonated Polyethylene (CSPE). The Hypalon Geomembrane Cover is internally reinforced with 2 plys of scrim sandwiched between three layers of Hypalon resulting in a 5 ply construction. The two reinforcing scrim layers are each 16 x 8, 2:1 Leno Weave Polyester (8 x 8, 250 denier Apparent). The top surface is colored a tan or earth tone and the underside is black. The original Hypalon roll goods were manufactured by Burke Rubber Company, San Jose, CA as product number M-153 (Potable Water Grade). The roll goods were then fabricated into panels, delivered to the site and field assempled into the floating cover. The floating cover is a defined sump tensioned plate Burke floating cover design (patent no. 3,991,900). The design engineer for the Hinkle reservoir liner system and floating cover was Clendenen & Assocates Consultants, Inc., Auburn CA. It should be noted that the Hinkle Reservoir was the first to use a self draining design where the rainwater is removed through flexible hoses fitted with penetration fittings at the Hypalon cover pipe manifold (bottom of sumps) and the base of the reservoir.

Upon arrival at the San Jaun Water District, we met with the Water Quality Manager, Mr. Michael J. O'Bleness and Mr. Joe Batt, Lead Worker. We briefly discussed history, maintenance proceedures and any problem areas associated with the Hypalon cover system. In addition, we reviewed maintenance inspection proceedures and forms, original drawings, O & M manual as well as underwater photos of the cover system and drain pipe connections. The following evaluation is based on the September 13 site visual inspection. Reference Photos are included as Attachment 1 and are refered to in the following text.

Hypalon Floating Cover Evaluation

The reservoir was in operation during our evaluation and was near capacity. Thus, it was necessary to walk out on the cover system at several locations to observe the conditions. As a general observation, the condition of the Hypalon cover system is very good considering its 20 year life and constant exposure to the elements. The tan surface exhibits surface oxidation, surface crazing (near surface cracking), stiffening (surface hardening) and general aging characteristics typical of Hypalon. The Hypalon polymer becomes stronger with age due to continued cross-linking of the polymer. Other than discoloration and distortion at factory seam areas (over water surface only), there were no obvious surface areas that exhibited deterioration. The only damage noted was due to mechanical puncture at the upper slope surface and broken or split sand ballast tubes (upper slope of rainwater collection channel south end of reservoir). There were no major distorted or wrinkled areas other than stressed areas at the slope where the sumps and weights rest on the slopes. A dark gray discoloration was also noted primarily on the North end of the cover and may be attributed to standing water over time. *(Refer to Figures 1 and 2)*

Observations - Mechanical

The upper connection detail with SS batten bars is in very good condition and may only require cleaning and replacement of the exterior edge sealant and tightening of the SS bolted connection. There was no distress or mechanical damage of the cover material noted at the connection. (*Refer to Figure 3*)

The original factory and field patches that were placed with bodied solvent chemical fusion methods are in excellent condition considering the age of the cover. All observed patches, vents, and butt seams were intimately bonded to the top surface of the Hypalon and all edges that were observed were sealed with edge sealant. (*Refer to Figure 4*)

The top of slope surface air vents appear to be in good condition. However, considering the age of the installation, the function of the one way valve should be examined for proper operation on all vents to be assured of no surface leakage. (*Refer to Figure 4*)

The factory fabricated seams are in excellent condition with the exception of distortion or "channeling" over the water surface which may be associated with aging and temperature at the near water surface. Distortion was not noted on the side slopes. The bond at the edge of the seams could not be mechanically loosened. All field seams appear to be in excess of 2 inches in width (2 inches scrim to scrim bonding) and apparently were fabricated with thermal welds. (*Refer to Figure 5*)

The field seams are also in generally good condition and are noticably wider than the factory seams. These seams apparently were field fabricated using bodied solvent chemical fusion. No blistering was noted in the seam areas.

The Hatch Covers, connections to the cover and associated float system appear to be in good condition and operational. (*Refer to Figure 6*)

There are numerous recent and old repair patches many of which are small circular pieces. Repairs were accomplished on some of these using a contact adhesive which does not adhere well as the material ages and is not recommended for future patching. Many of these patches were observed to be loose, unbonded or easily lifted from the surface. (*Refer to Figures 7,8 and 9*)

There was only minor unrepaired damage noted on the cover system at the top of slope, East Side, apparently caused by a puncturing object or rock. (*Refer to Figure 10*)

Defined sumps and Rainwater Removal System

The defined sumps on the tensioned plate appear to be positioned well and reportedly remove excess rainwater as designed. There was little or no standing water on the cover at the time of the inspection. The following observations were made in examining the cover system:

1. The cover system and defined sumps were positioned as originally designed and the cover is in a tensioned condition. (*Refer to Figures 1 & 2*)

2. Some of the rainwater collection channels were full of water at the time of inspection. The channels should be fully dewatered and may be an indication of blockage in the header drain pipes. At least one of the channels (NE channel) was noted to be covered with green algae which must be cleaned and flushed. Algea will attack and distort the surface if allowed to remain and dry. It is obvious that these areas have had standing water for some time. (*Refer to Figure 12*)

3. It was noted that the sand ballast tubes were damaged at two areas on the upper slope/channel connections reservoir south end and should be repaired. (*Refer to Figure 11*)

4. Distortion of the floats and wrinkling of the cover, although not detrimental, was noted at the upper slope/channel intersection areas. (*Refer to Figure 11*)

5. The defined sump floats appear to be intact and serving their design function

6. The condition of the submerged sump drain pipe headers could not be observed but may be clogged in some areas with debris or biological growth.

7. The condition of the underwater connections of drain pipes to flex drains and connections to the reservoir outlets appear to be in good condition after observing the underwater inspection photographs provided during our site visit. The connections were reportedly tested with dye (white milk) for leaks and none were found. (*Refer to Underwater Inspection Report for San Jaun Water District - 1999*)

Observations - Surface Discoloration

Accumulated dirt, dust and small debris was noted to be collecting in the factory seam channels that have formed on the surface over water areas only. Upon cleaning some of the channel areas and observing the Hypalon seam surface and edges of seam, no deterioration is evident. However, these seam areas should be cleaned by low water pressure/vacuum or dry brush/vacuum when the seam areas are inspected (See Recommendations). (*Refer to Figures 13, 14 and 15*)

There were several areas that exhibited brownish discoloration and were tacky to the touch. It is not known if this is a surface spill of liquid or if it is extruding from the Hypalon surface. However, these areas should be investigated for surface damage or deterioration. Visual observation of these areas does not indicate physical damage, only strong discoloration. (*Refer to Figures 16 & 17*)

Observations - Leakage

A significant amount of surface water was noted at the outlet structure due to reported leaks in this area. It is vitally important that the cover system not be sucked into the outlet area during drawdown as this will stress the cover and especially stress the numerous field seams in this area causing possible leaks. (*Refer to Figure 18*)

CONCLUSIONS AND RECOMMENDATIONS

Based on the September 13, 1999 site visitation, discussions with water district personnel, visual cover system inspection and observations, the following conclusions and associated recommendations are offered:

1. The general condition of the Hypalon Floating Cover System is very good considering the almost 20 years of continuous operation of the Hinkle Reservoir. There is no reason that the cover material, seams and associated attachments will not last an additional 5 years minimum in service life.

Recommendation. In order to determine a more realistic projected life expectancy for the existing cover system, samples of the cover should be extracted as soon as practical to help determine aged physical/mechanical properties and percent change

in properties after almost 20 years of service. Samples with factory seam should be taken from the south facing slope and locations at different quadrants on the cover. A minimum of four samples, each approximately 18 inches in width by 30 inches in length with the seam centered on the 30 inch length should be cut from the cover system. It is further suggested that the manufacturer (Burke) of the cover material be contacted for cooperative testing and evaluation as well as on site sampling/repair of sample cut-out areas. As the original supplier of the material for one of the largest floating covers, they may be interested in a case history evaluation and technical publication.

As a minimum, the following tests should be run on extracted samples:

Thickness/Mass
Water Absorption
Ply Adhesion
Tensile Breaking Strength & Elongation
Tear Resistance
Bonded Seam Strength - Percent of Parent Material
Shore Hardness - Bottom and Top Surface
Surface and Edge Photomicrographs - Bent Strip and Flat
Analytical Component on Extractables

The above testing should be carefully coordinated with comparisons made between original values and existing values as well as comparisons of upper slope vs. over water material properties.

In addition to the Physical/Mechanical properties test program and evaluation, a thorough 20 year comprehensive inspection and maintenance cleaning/repair of the cover system should be completed. This may require complete drawdown of the reservoir. At the very least, every factory and field seam should be inspected (and cleaned of debris). All perimeter attachments, structure attachments and hatch covers should be inspected and repairs made. The sump drain pipe headers must be cleaned and flushed and ballast tubes inspected. (Again this may require drawdown). Drain pipes may be accessed by underwater vacuum cleaning equipment from above the sumps, however this may not thoroughly clean the pipes.

2. The current maintenance inspection program and reporting forms is acceptable. It is imperative that daily visual perimeter observations be continued and that the weekly cover inspection with recorded observations and repairs be kept current.

Recommendation. A comprehensive 20 year maintenance inspection and cleaning should be completed as outlined above in item 1.

The weekly inspections should be augmented with a thorough yearly detailed inspection of all cover areas, hatches, connections and sumps. A yearly underwater

inspection program is currently being accomplished for all underwater connections and is recommended to be continued for future inspections. The top cover inspection should be completed in concert with the underwater inspection.

Once the 20 year inspection and cleaning is complete, it is not recommended to clean the surface of the cover more than once every 2 years. More frequent cleaning is not recommended due to the increased potential for mechanical damage. Because the reservoir is limited to access, fenced, has 24 hour operations personnel present and air blown debris is limited to fine material, the potential for damaging objects or material accumulation is small.

3. The current Hypalon material repair methods using patches and contact adhesive is not acceptable and should be reevaluated.

Recommendation. The original manufacturer, Burke Rubber Company, should be contacted for current repair proceedures and materials recommendation. They should also be contracted for on site instruction in repair of old Hypalon.

4. It was noted that several sections of the rainwater collection sumps were full of water and that algae growth was prevalent.

Recommendation. The rainwater removal system should be inspected. The header pipes in the sump bottom may be clogged with debris or biological growth. The header pipes are reportedly 4 inch schedule 80 PVC with 1/2 inch holes drilled every 6 inches. Some of the header pipes may not be draining and may account for the standing water and discoloration on the cover surface. These pipes can be cleaned and/or replaced during the suggested 20 year inspection. Algae growth should be immediately removed and flushed with chlorox. Temporary dewatering and cleaning of sump areas can be accomplished with small submersible pumps.

5. The upper slope and cover surface air vents appear to be in good condition. However, actual operation of the one way valve is unknown.

Recommendation. Test each valve for one way operation (no drainage into reservoir). Valves that fail can be replaced with similar one way vents or conventional top of slope vent details.

6. Some minor damage was noted on the upper slope of the east side.

Recommendation. Make a thorough inspection of upper slopes and repair damaged areas with new patch material and methods as per Burke Rubber Company Recommendations.

7. Numerous old and more recent maintenance field repair patches were observed to be unbonded or becoming unbonded.

Recommendation. Remove the unacceptable patches and replace with new patch materials and methods as per Burke Rubber Company Recommendations.

8. Significant surface water was noted at the outlet structure and may be associated with cover leakage.

Recommendation. Draw Reservoir down to below the suspected areas around the outlet structure and inspect all field seam areas. It is suggested that an installation crew recommended by Burke be contacted to inspect and test this area for leaks and to make repairs. Again, this area is subject to significant stress during drawdown and the numerous field seams can become unbonded at areas of high stress.

This concludes the report on the San Jaun Water District Hinkle Reservoir Cover Visual Inspection, evaluation and recommendations. If you have any questions, please give me a call at 303-679-0285.

Sincerely Yours,

Ronald K. Frobel, P.E.

Attachment 1 - Site Photographs 1 - 18

ATTACHMENT 1

SITE PHOTOGRAPHS

Figure 1. General View of Hypalon Floating Cover - Note Tensioned Condition

Figure 2. General View of Hypalon Cover Taken from the North. Note the Dark Grey Discoloration of the Surface.

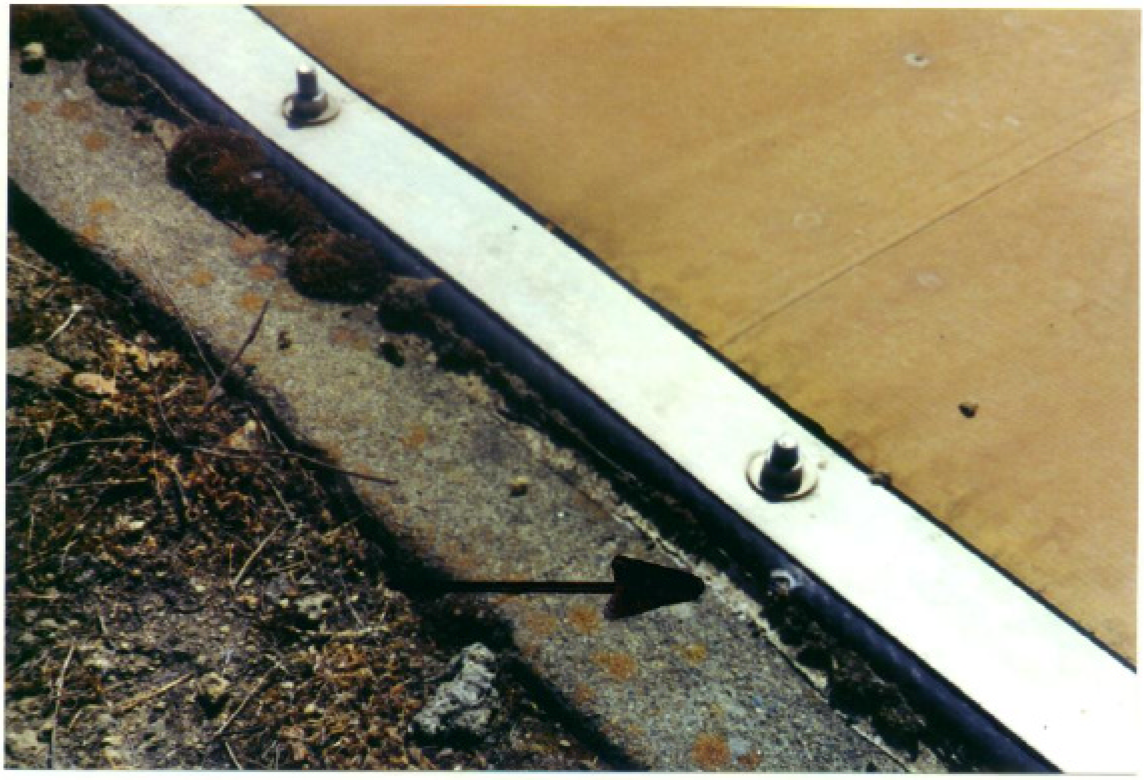


Figure 3. Typical Top of Slope Batten Bar Connection Detail. Note Deteriorating Condition of Edge Seal.

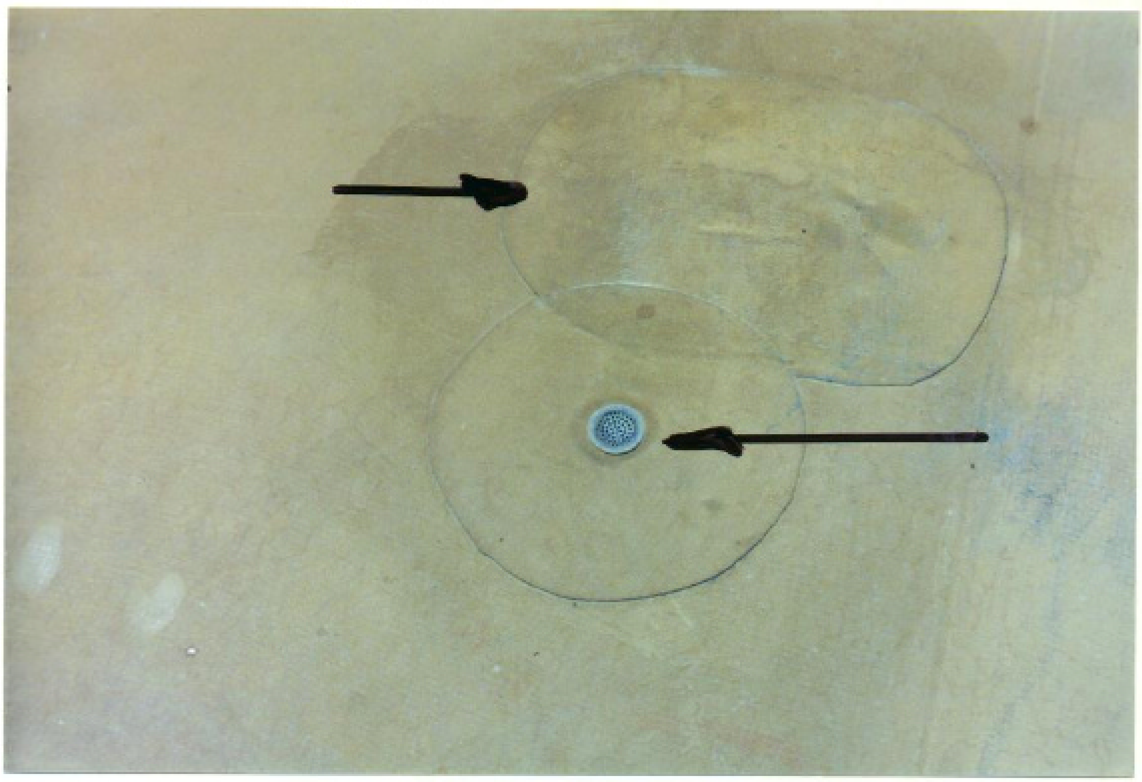


Figure 4. Photo Illustrating Typical Slope Vent Detail as well as Typical Original Field Patch. Note Flat Condition and Tight Bond to Surface.

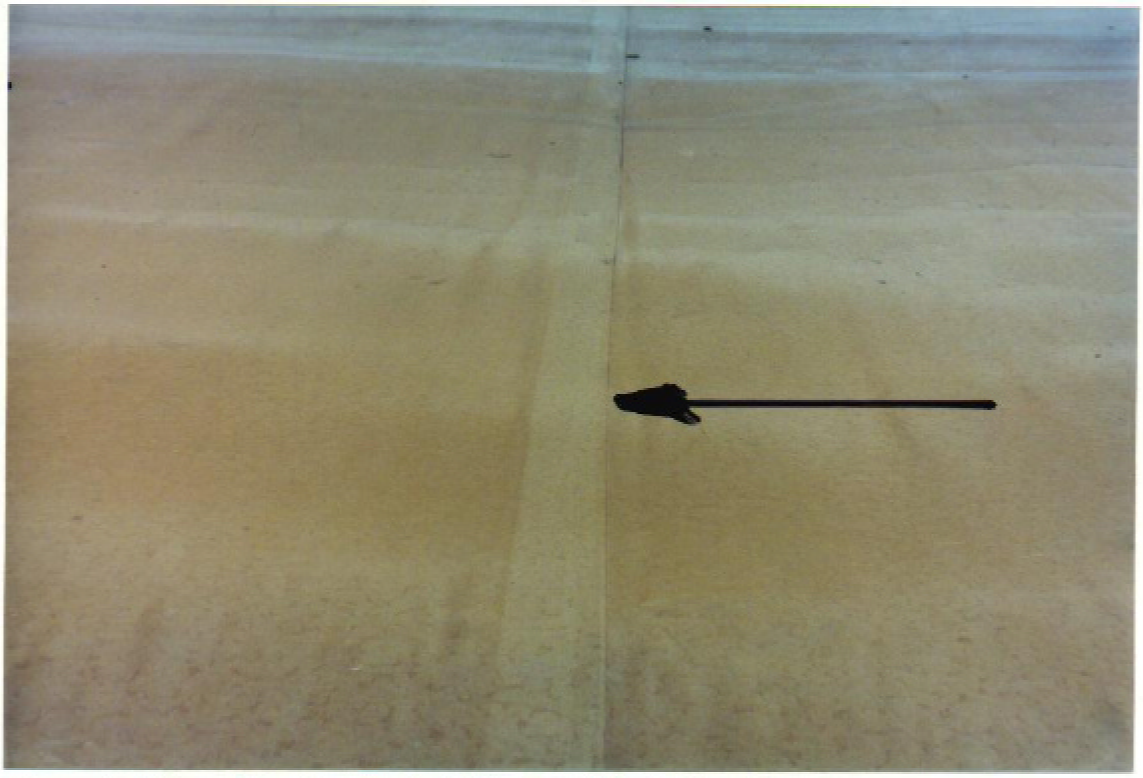


Figure 5. Photo Showing Typical Factory Fabrication Seam.

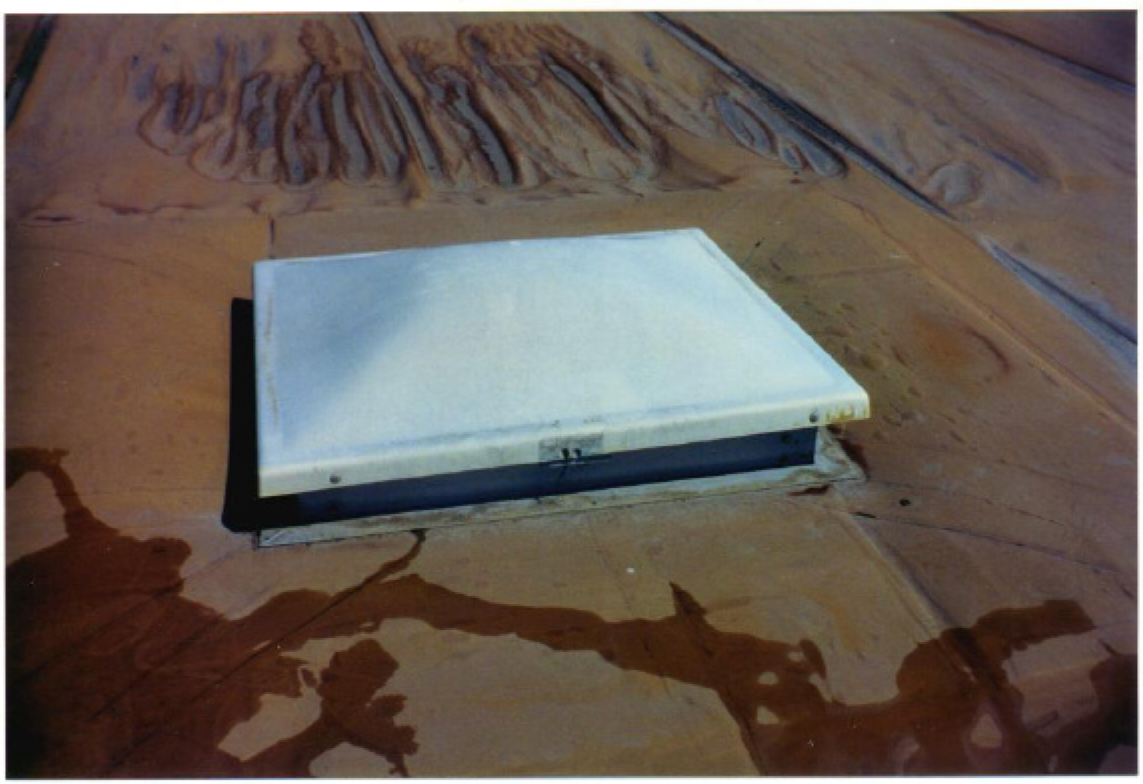


Figure 6. Typical Access Hatch Cover Installation.

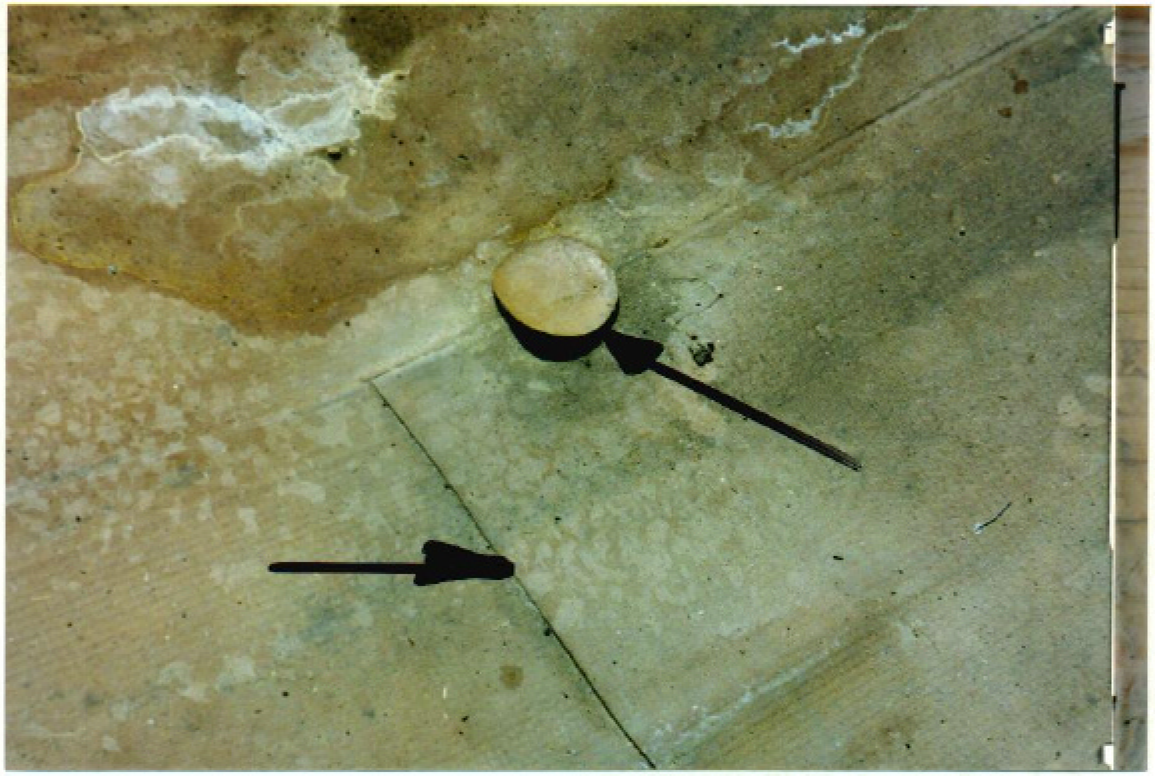


Figure 7. Photo Illustrating Unacceptable Field Repair Patch Using Contact Adhesive. Note Factory Fabrication Butt Seam.

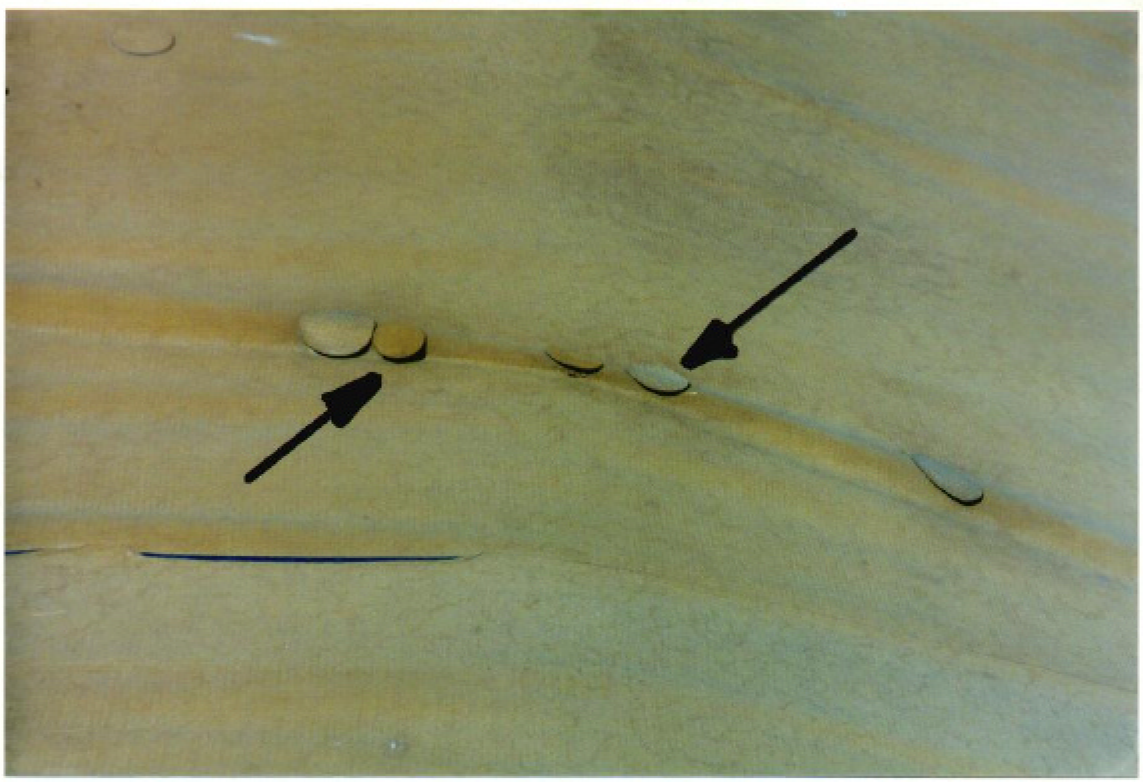


Figure 8. Photo Showing Small Loose Patches for Field Repairs Using Contact Adhesive.

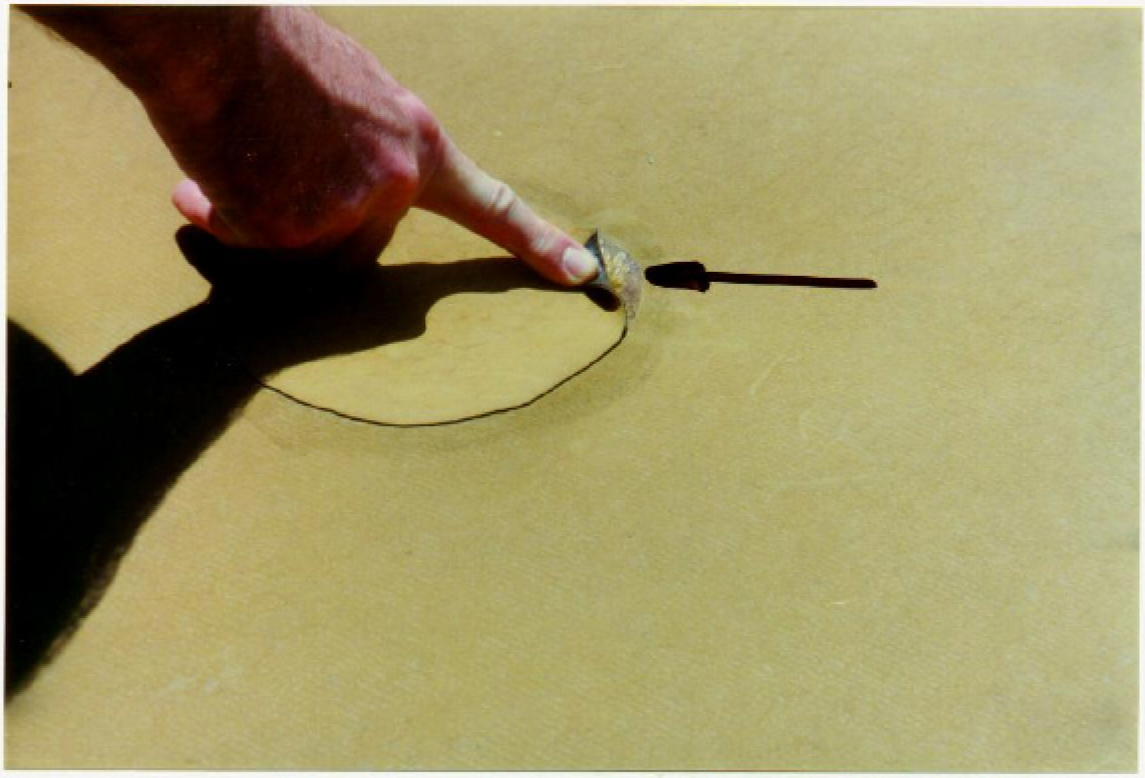


Figure 9. Recent Field Repair Patch Using Contact Adhesive. Note that Edge is Easily lifted and will Become Unbonded over Time.

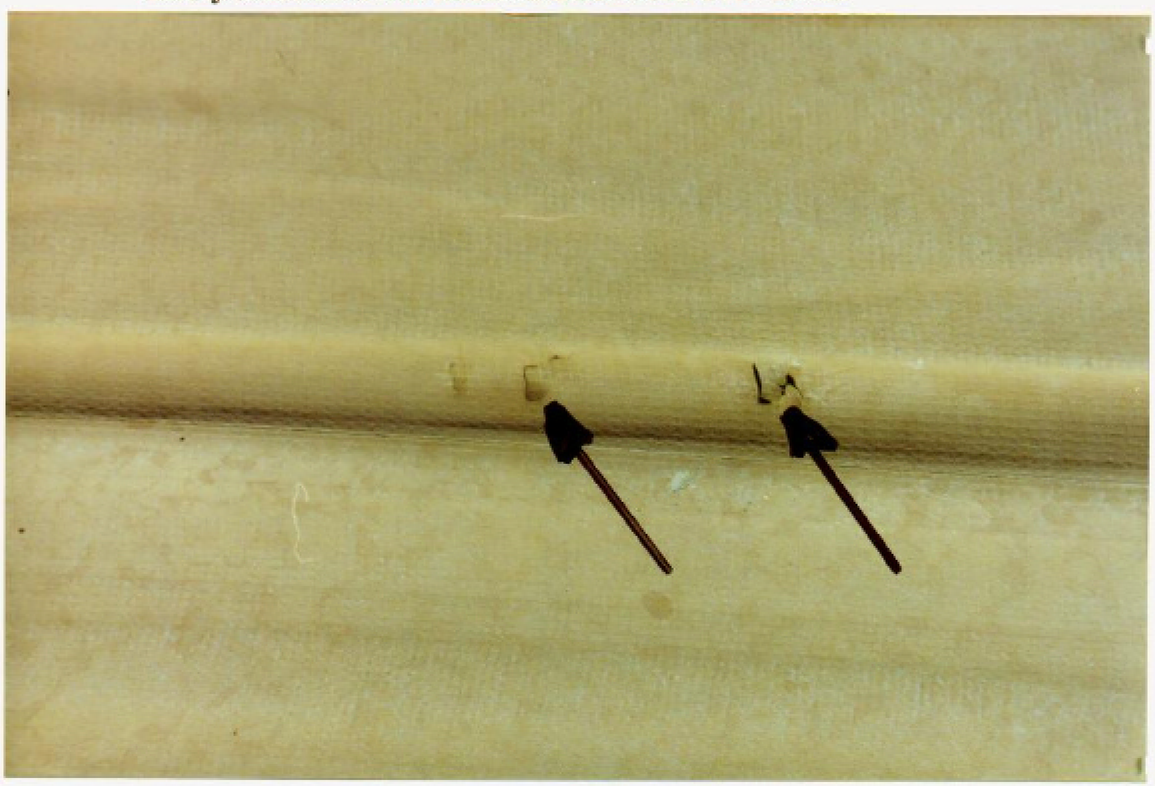


Figure 10. Puncture Damage Noted at Top of Slope East Side.

Figure 11. Photo Illustrating Sand Ballast Tube Damage and Cover Wrinkling - South End of Reservoir at Slope.

Figure 12. Rainwater Collection Sump With Algae Growth - NE Corner.

Figure 13. Factory Seam Area Over Water with Accumulated Debris in Channel. Note that Channel Appears to be a Seam Distortion over Water Only.

Figure 14. Photo Illustrating Debris in Factory Seam Area as well as Dark Brown Discoloration.

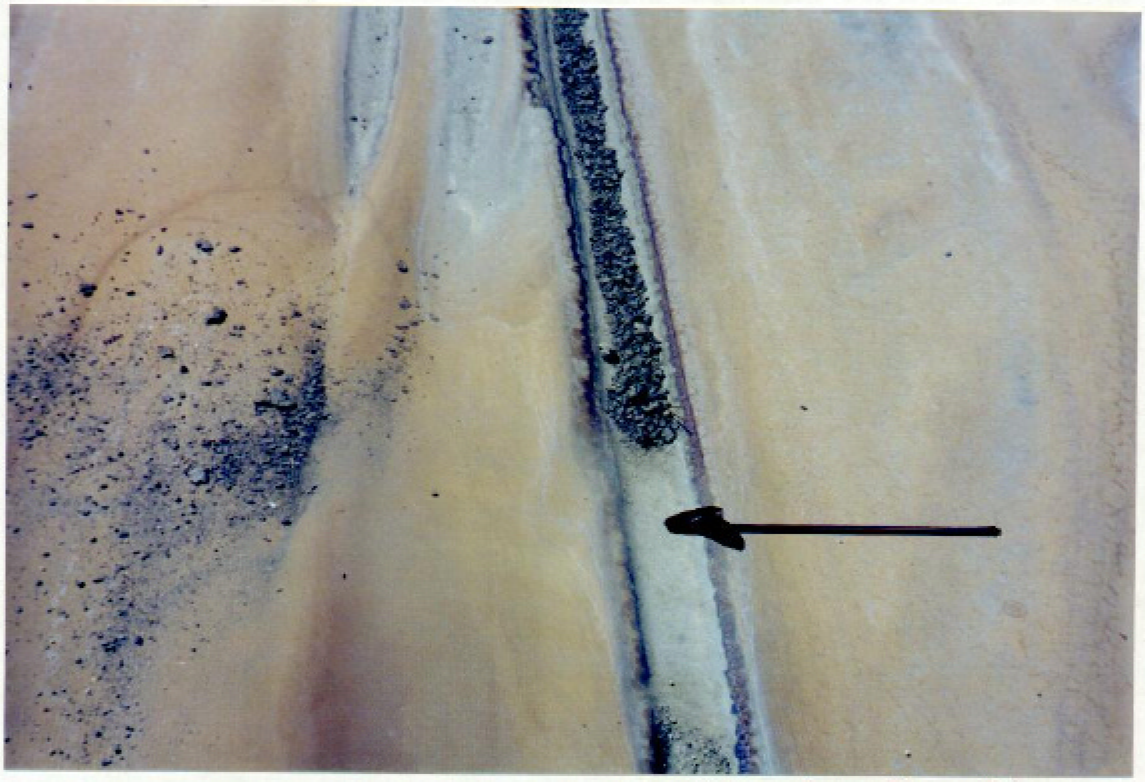


Figure 15. Photo Showing Debris Removed from Factory Seam Channel Area. Note that Hypalon Surface is Lighter in Color but not Damaged.

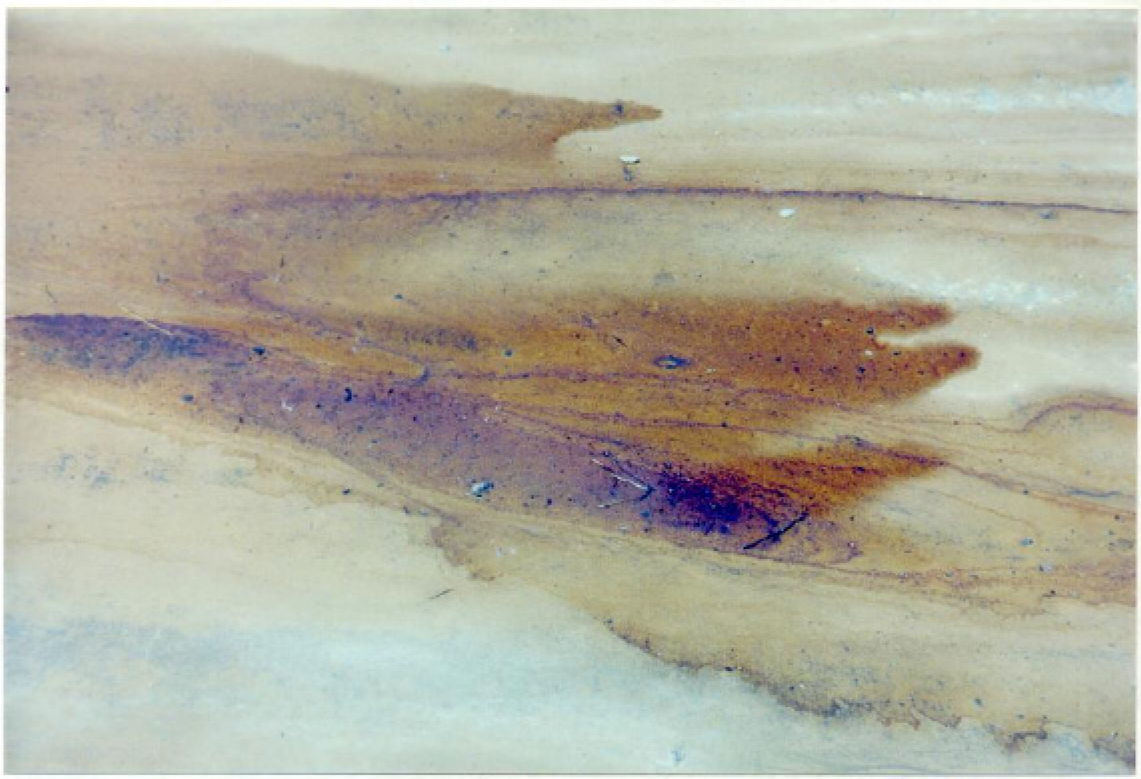


Figure 16. Photo Showing a Dark Brown Surface Accumulation of a Tacky Material.

Figure 17. Photo Illustrating Another Area of Dark Brown Surface Contamination.

Figure 18. Water Accumulation at Outlet Structure. Note that this Area Contains Numerous Field Seams Due to Structure Attachment.

R.K. Frobel & Associates

Geosynthetics Consulting Engineers

Mr. Keith B. Durkin, P.E.
Vice President
Kennedy/Jenks Consultants
3336 Bradshaw Road, Suite 140
Sacramento, CA 95827

November 9, 1999

RECEIVED
BY KENNEDY/JENK
NOV 10 1999
SACRAMENTO

RE: San Jaun Water District, Granite Bay, CA
Sidney N. Peterson Water Treatment Plant
Hinkle Reservoir Hypalon Floating Cover Test Program

Dear Mr. Durkin:

At the request of Kennedy/Jenks Consultants, I have finalized a test program for the sample extraction and testing of the 45 mil (1.14 mm) Hypalon Floating Cover currently installed on the Hinkle Reservoir. The purpose of the test program is to determine the physical/mechanical properties of the cover system after 20 years of service and to use the data to help determine a projected life expectancy of the existing cover system.

Hypalon Cover Sampling

It is recommended that 4 samples be extracted from the cover system from 4 quadrants approximately as shown on the attached drawing. Each sample shall be a minimum of 20 inches in width by 36 inches in length with a factory seam centered on the 36 inch length. All samples are to be taken from floating cover sections that are normally in operation over water and not on the slopes as shown on the attached drawing. All samples must be identified as to location, i.e., quadrant A, B, C or D, distance from top of slope anchor and approximate location along side of reservoir perimeter.

It is further recommended that the samples be extracted by an authorized representative of Burke Industries, the original manufacturer of the cover system and that the repairs for the cut out areas be completed by Burke or Burkes representative immediately after sampling with their approved repair methods for aged Hypalon. Mr. Steven Roades of Burke Environmental, San Jose, CA has agreed to participate in the sampling, testing and repair of the cover system on the Hinkle Reservoir. Mr. Roades can be contacted at 408-297-3500. Also, Mr. Bob Pitman, technical manager in charge of testing at Burke has been notified and has agreed to the test program as outlined below.

At the time that the sampling and repairs are made, San Jaun maintenance personnel should be available for on site training in the repair proceedures for aged Hypalon.

1153 Bergen Parkway, Suite M 240 • Evergreen, CO 80439 • ph 303-679-0285 • fax 303-679-8995

Burke may suggest that the sampling and repair of sampled areas be performed by one of their approved fabricator/installers such as C.W. Neal Corporation, Santee, CA or R.T.D. Enterprises, Hollister, CA

Hypalon Cover Material Testing

Each of the samples extracted from the cover shall be cut into two equal pieces, each approximately 20 inches in width by 18 inches in length. One sample shall be tested by Burke and the other sample shall be sent to an independent laboratory. Precision Laboratories, Orange, CA has experience in testing Hypalon and has agreed to complete testing as required. Mr. Ron Belanager or Ms. Cora Aquino can be contacted at 714-744-0357. All samples must be fully identified as to original location taken from the reservoir surface.

The following tests should be run on each of the 20 in. x 18 in. sample sections:

Thickness	ASTM D 751 (1599)	5 replicates
Water Absorption*	ASTM D 471	3 replicates
Ply Adhesion	ASTM D 413/type A	3 replicates
Tensile Strength/ Elongation	ASTM D 751/Grab	2 replicates MD 2 replicates CMD
Seam Shear Strength	ASTM D 751/NSF 54	2 replicates
Hydrostatic Burst	ASTM D 751/method A	4 replicates
Surface Cracking	Photomicrograph-bent strip	1 @ 30X
Cut Edge Section	Photomicrograph	1 @ 30X

* Use the D 471 proceedure to extract as received moisture from the sample, dry to equilibrium and determine % water in the sample.

The approximate cost for testing each section at Precision Laboratories is $250.00 or a total of $1000.00 for all four test samples. It is not known if there will be costs associated with testing at the Burke laboratories, however Burke is very interested in obtaining aging data and publishing a case history. Burke may also perform analytical testing such as FTIR on the samples to help determine ageing characteristics.

Sample extractions, cutting, identification and testing at both Burke and Precision must be carefully coordinated as the samples are small. Once the samples are taken, they should be properly identified, photographed and packaged flat in heavy plastic bags for shipment to the labs. It is imperative that the samples be packaged immediately upon removal from the cover and protected by plastic until ready for specimen cutting and testing. Attached to this letter is a suggested specimen layout for each of the 20 in. x 18 in. sample sections. Due to specimen sizes for scrim reinforced Hypalon, we are limited to the number of specimens that can be cut from the samples.

20 Year Inspection and Cleaning

As recommended in my inspection report dated September 21, 1999, the cover system should be thoroughly inspected and cleaned in so far as practical without damaging the cover material. A 20 year maintenance inspection, cleaning and repair should be performed by an experienced subcontractor familiar with Hypalon and floating covers. Two experienced companies would be C.W. Neal and R.T.D Enterprises as follows:

C.W. Neal Corp., Santee, CA - 619-562-6438 - Mr. John Glitch

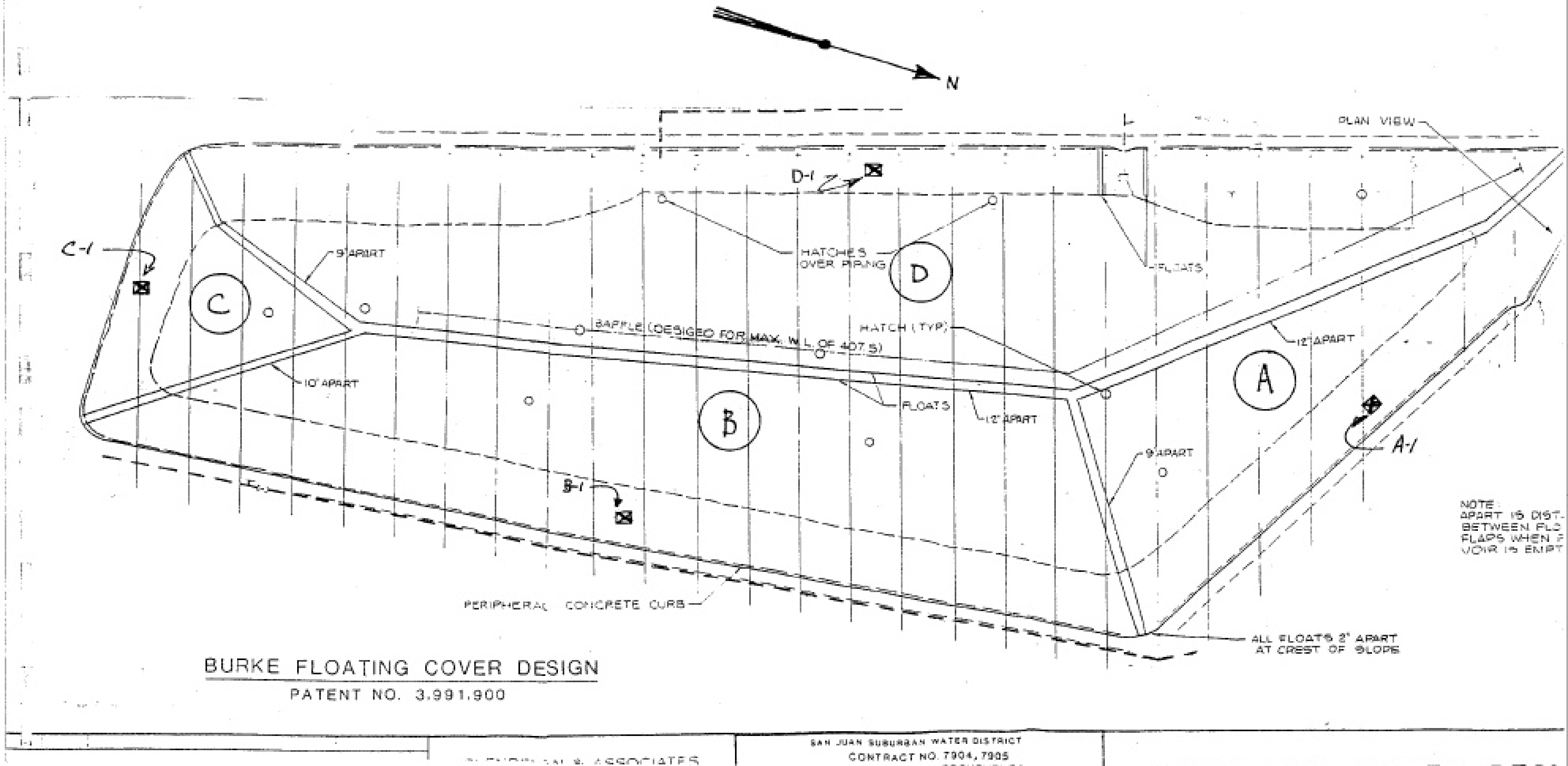
R.T.D. Enterprises, Hollister, CA - 831-636-0861 - Mr. Ed Parker

This completes my recommendations for sampling, testing and maintenance cleaning as well as specific contacts for Kennedy/Jenks Consultants. If you have any questions, please give me a call at 303-679-0285.

Sincerely Yours,

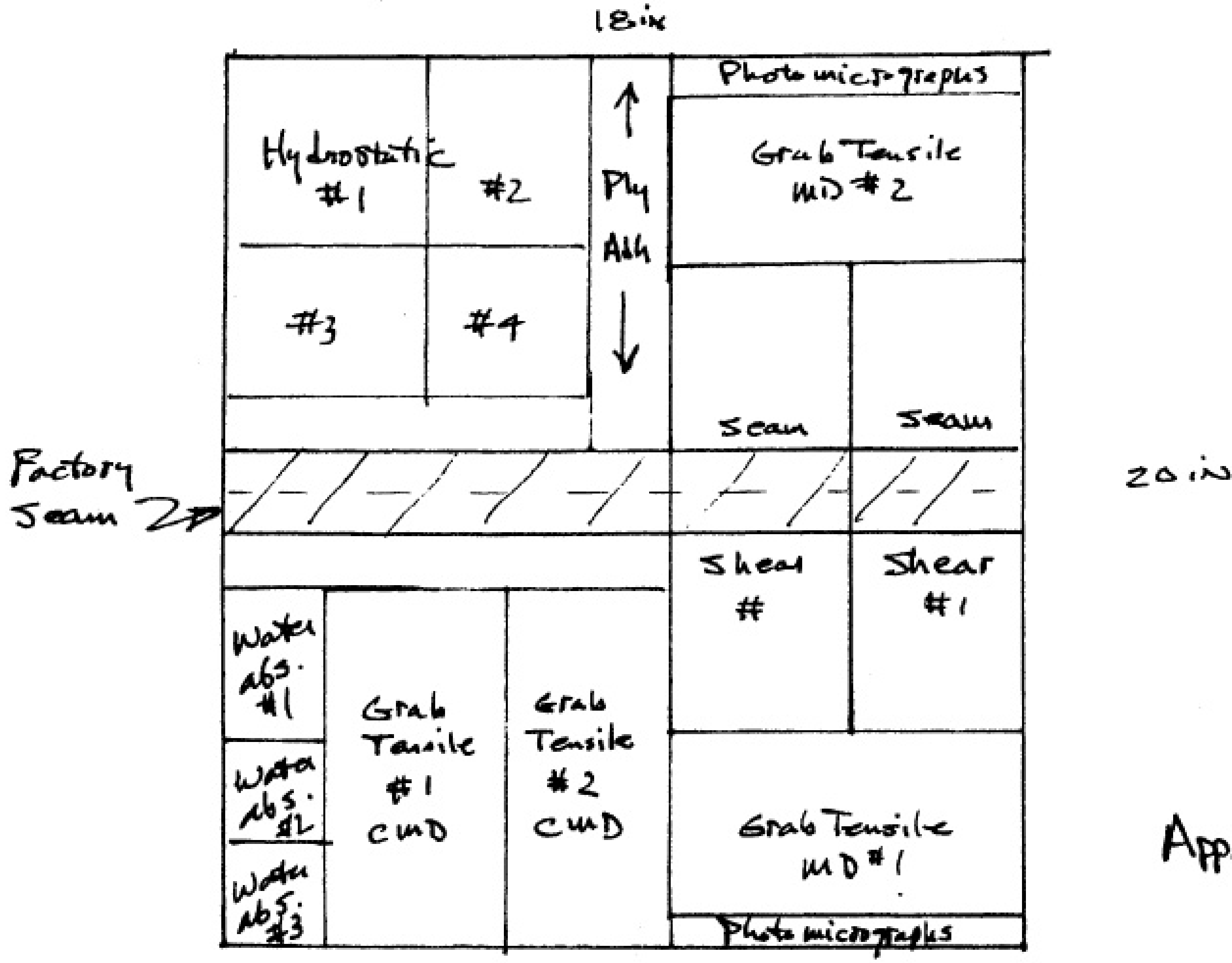
Ronald K. Frobel, P.E.

Enclosures - 2



Approximate Sampling Locations
Hinkle Reservoir Cover

NTS



Approximate Specimen Layout
18" x 20" Samples

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Appendix 8-2

CLI Inspection Report and RKF Summary Report

2 January 2001

Ms. Shauna Lorance
Assistant General Manager
San Juan Water District
9935 Auburn Folsom Road
Granite Bay, CA 95746

Subject: Wholesale Master Plan Project
Hinkle Reservoir Hypalon Cover – 20 Year Inspection
K/J 992509.01 file 6.01

Dear Shauna:

Kennedy/Jenks Consultants has completed the 20-year inspection and sampling program of the Hinkle Reservoir Hypalon Cover. The inspection included a thorough physical assessment of the condition of the cover, collection of material samples from the cover, laboratory testing of the samples, test cleaning a small portion of the cover, and preparation of a summary report. The physical inspection and sampling was completed by Colorado Linings International (CLI) under contract to the District.

Four coupons were cut from the cover for materials testing. These samples were analyzed by the Burke Rubber Company, supplier of the original hypalon cover material, and Precision Geosynthetic Laboratories, an independent third party laboratory. The test results and inspection report were evaluated by Mr. Ron Frobel of R.K. Frobel & Associates (RKF), a recognized expert in hypalon materials and membrane systems. RKF provided a summary report which is enclosed for your review along with CLI's report.

This letter summarizes the findings and recommendations of the 20-year inspection as follows:

- The Hypalon Floating Cover System is in very good condition. The cover material, seams, and associated attachments have a minimum remaining service life of 15 to 20 years with proper maintenance.

- The detailed inspection identified the location of 60 to 70 small holes or failing repairs (patches). All holes and failing repairs should be patched using the recommended procedure described in the CLI report.

- The outlet structure geometry creates areas of significant stress in the hypalon material. The geometry also creates an undrained sump which collects debris and supports biological growth. The hypalon cover at the outlet structure should be redesigned to properly accommodate cover movement and eliminate the undrained sump.

- A comprehensive 20-year maintenance cleaning should be completed with subsequent periodic cleaning no more frequent than once every two years. More frequent cleaning is not recommended due to the increased potential for mechanical damage to the cover.

- Perimeter edge caulking has cracked and pulled away from the concrete edge beam at the top of the slope. This may allow water to seep under the edge and into the reservoir. Caulking should be removed and replaced around the entire perimeter.

- Several areas would benefit from supplemental weights for better tensioning and to improve drainage to reduce ponding rainfall. Thirty additional weights were provided as part of the inspection and sampling contract and could be used for this purpose. Supplement weights should be placed near the northeast sump and other areas identified following rainfall events.

- Trapped air exists under the cover and can allow the cover to lift and tear during high wind events. Trapped air should be 'walked' to the hatches.

- Updated AWWA recommendations for inspection and reporting (April 1999) should be reviewed and selectively incorporated into the District's maintenance program as appropriate.

A fifty-foot test section of the rainwater drainage sump was cleaned to determine the level of effort required to remove accumulated debris and to estimate the volume of material present in the sump. The total length of sump is 1,950 feet. The reservoir was drawn down to approximately 8 feet to allow access to the northwestern reach of the sump. This was the only reach of the sump exposed at the 8-foot level. The reservoir will need to be drawn down several more feet to provide similar access to the rest of the sump when the 20-year maintenance cleaning proceeds.

The test cleaning indicated that the sump contains a substantial volume (10+ cubic yards) of debris consisting of dirt, pine needles and leaves. The contractor was able to clean the hypalon with a moderate effort using a mild soap and brushes. Given proper access, the sump should clean up nicely.

The challenge for the District will be to provide proper access to the sump to complete the maintenance cleaning while maintaining service to its customers. Several alternatives were discussed with CLI, generally falling into two categories as follows:

1. Completely drain the reservoir causing the cover to layout flat and allow a thorough cleaning and inspection of the sump.

2. Partially drain the reservoir reducing the sump depth to less than 3 feet and clean with a fire hose and grinder pump. The use of a temporary rigid sump insert would create access for the grinder pump to safely draw water and debris from the cover sump. This technique could successfully remove the bulk of the debris but does not allow for brushing, cleaning or a thorough inspection.

The recommended approach is to take the reservoir off line if possible and completely expose the hypalon material for cleaning.

Please review the enclosed reports and call me to discuss the findings and next steps to complete work on the Hinkle Reservoir cover. We are prepared to proceed with preparation of contract documents to obtain specialized contracting services to complete the 20-year maintenance cleaning.

Very truly yours,

KENNEDY/JENKS CONSULTANTS

Keith B. Durkin, P.E.
Project Manager

enclosures

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Colorado Lining
INTERNATIONAL

Your Inside Line On Containment

San Juan Water District
9935 Auburn-Folsom Road
Granite Bay, CA 95746

December 7, 2000

Subject: Hinkle Reservoir Floating Cover Inspection Summary

Observations

<u>Floating Cover Surface Condition </u>(Tensioned Plate)

The overall condition of the cover surface is in very good condition considering how long it has been in service. There are approximately 60-70 areas that need small patches. Some of these areas are very small holes, some are field repairs that could be peeled off, and a few are leaking tee joints. All of these areas are documented and will be shown on a scaled plan view of the reservoir.

There are several areas that would benefit from supplemental weights for better tensioning and draining. The primary area is at the NE sump near the access hatch over the inlet area (between section 1 and 2). Another less problematic area would be between section 4 and 5. 30 – 6" diameter weights were delivered to the district as part of the inspection contract and might be used as the supplemental weights for these areas.

Dirt and debris has collected in some seam areas and around the inlet near the NE sump and at the outlet structure.

The hatches all appear to be in good order.

Air vents appear to be working. Although there seams to have been no apparent detrimental effects, the cover vents do not appear to be venting all of the air collecting under the cover.

The cover at the outlet structure needs some attention. Due to it's design it collects water in the folds around the structure.

Recommendations:

7051 Mesa Drive Aptos, CA 95003 831-684-1411 877-578-5000 Fax 831-684-1441 www.coloradolining.com

Hinkle Reservoir Inspection

1.) Clean the dirty cover areas with a solution of Simple Green and water.
2.) Patch all suspect areas marked during the inspection using the attached patching method.
3.) It may be useful to install floating air vents under the cover to help vent trapped air.
4.) Clean the cover at the outlet structure area and devise a method of dewatering the folds there. An alternative might be to re-design and replace the cover over the outlet structure to obtain a more desirable cover to outlet structure interface.
5.) Install supplemental floating air vents in areas where air is collecting under the cover.
6.) Install supplemental weights in slack areas to help drain surface water to sumps more efficiently.

Defined Sump

The sump appears to be working since it rained several times during this inspection and each time the collected water "drained" away successfully over time.

Due to the amount of dirt and debris evident in the sumps it is quite probable that the drains are not working efficiently. Part of the debris cross section is a layer of pine needles which could add to the clogging of the drain system.

Approximately 20 buckets (5 cubic feet) of debris was taken from the first 50' LF of the north west sump as part of this contract. The debris was left on the ground adjacent to the sump and photos were taken for future reference. The Hypalon was easily cleaned using "Simple Green" and nylon pot scrubbers and brushes. The dirt and algae came off easily. Most off the mineral deposits remained.

Recommendations:

1.) Remove all dirt and debris from the sumps.
2.) Clean the surface areas of the sumps with Simple Green and water.
3.) Clean and inspect the perforated drain pipe systems in the sumps.

Inlet Hatch and Air Capture Float Area

This area captures water and debris between the floats.

Recommendations:

1.) Clean the cover area between and around the floats and hatch.
2.) Devise a method to stop water from pooling between the floats.

Hinkle Reservoir Inspection

Perimeter Attachment

The perimeter attachment is in good condition. No cover pull out is evident and there are no areas of obvious strain. At the "point" of the reservoir between sections 1 and 7 it appears as though some non-stainless steel nuts and washers were used. The caulking between the rope hem and the concrete curb appears intact but isn't adhered very well to the concrete.

Recommendations:

1.) Clean and re-caulk perimeter between the cover material and the concrete curb.
2.) Clean road dirt back away from the concrete curb and re-grade the surface to drain away from the reservoir.
3.) Compact or resurface the roadway to eliminate small angular rocks from migrating to the cover surface.

Notes:

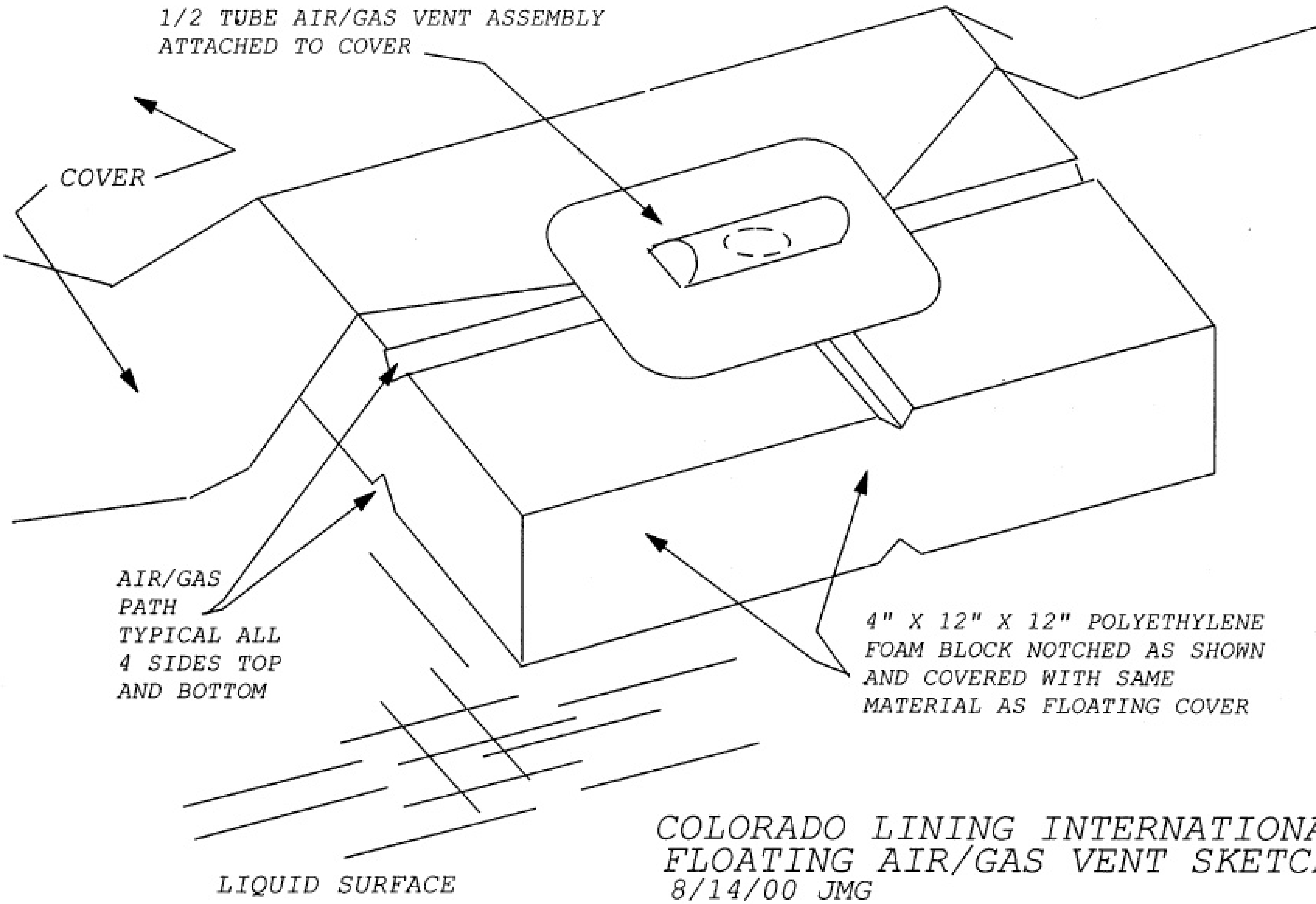
Over the course of this inspection several ideas on how to clean the sumps have been discussed. The following is a collection of those ideas in no order of significance. I should add that during the inspection we attempted to open the sump by using approx. 1000 pounds of weight of either side of about 30 lf of sump in an attempt to "offset" the sump weights thereby raising or stretching out the sump. We were unable to open the sump using this method.

1.) Use a type of dredging equipment to suck the dirt out from the sumps.
2.) Use a fire hose to liquefy the dirt and pump it out.
3.) Lower the reservoir all the way to expose the sumps, then clean them by hand.
4.) Use a hose to liquefy the dirt over the drain pipes and let it drain out the drains.
5.) Use a coffer dam to isolate ½ of the reservoir at a time, then lower that ½ and clean by hand.

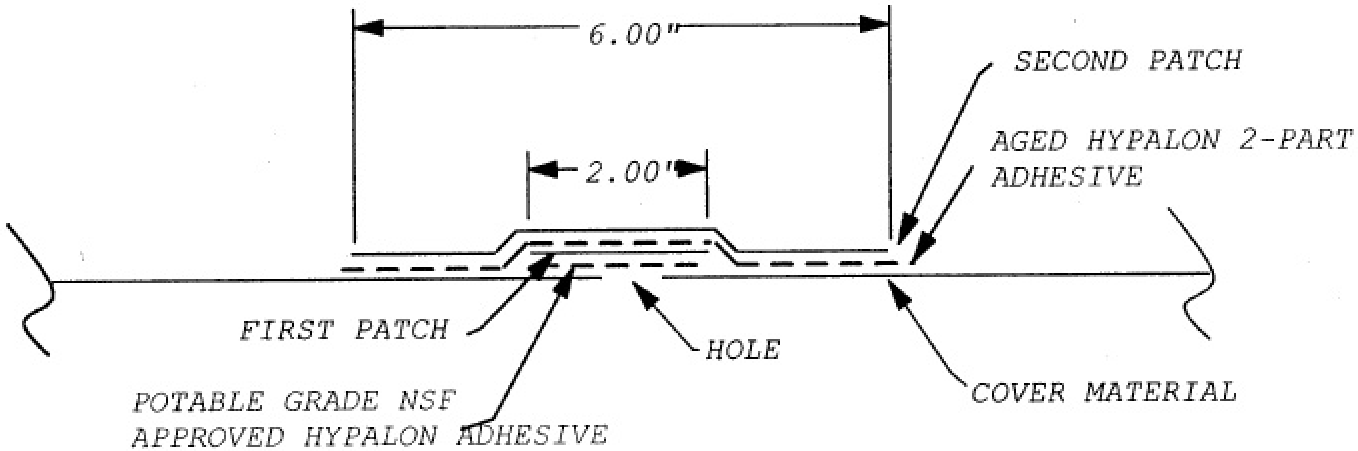
Conclusion:

In addition to the recommendations outlined above, we would suggest the implementation of a regular maintenance program which would include inspection, repair and cleaning on a yearly or as needed basis.

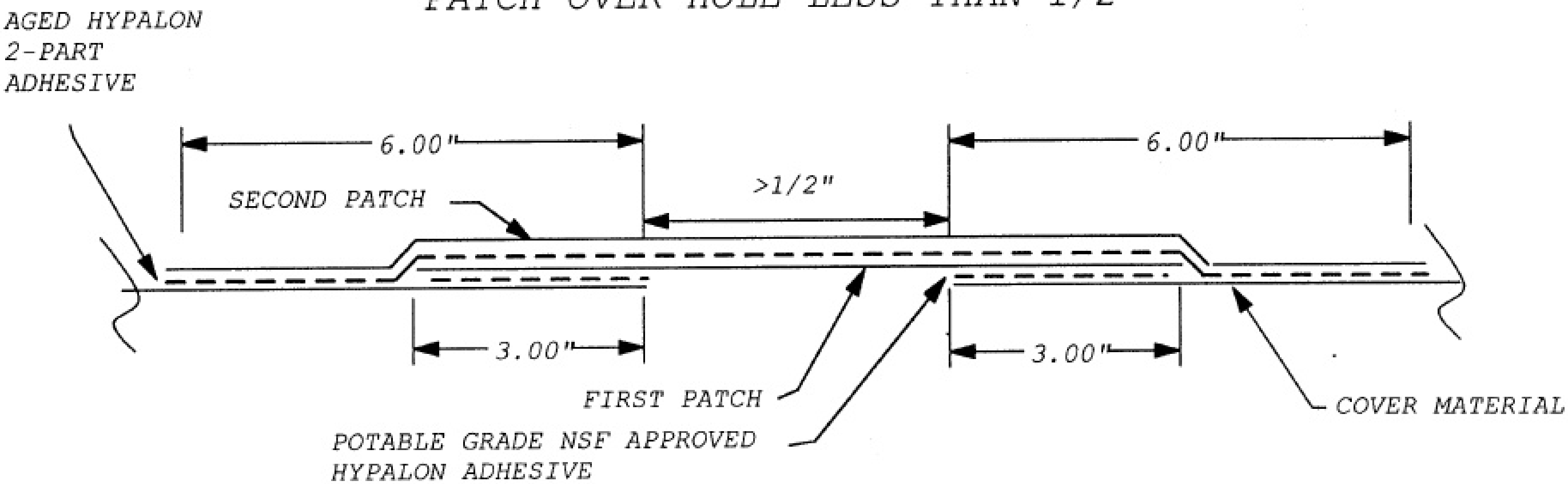
John Glitsch
Colorado Lining International
7051 Mesa Drive
Aptos, CA 95003



COLORADO LINING INTERNATIONAL
FLOATING AIR/GAS VENT SKETCH
8/14/00 JMG



PATCH OVER HOLE LESS THAN 1/2"



PATCH OVER HOLE GREATER THAN 1/2" (4" SHOWN)

Procedure for Patching an Aged Hypalon Floating Cover

This procedure uses 2 patches. The first patch is applied over the hole using NSF approved Hypalon adhesive and functions as a non structural barrier between the water and the latter applied "structural" patch which is applied with non-NSF approved 2 part adhesive. We recommend using a heat gun during this entire procedure if the ambient temperature is below 75 degrees F (the adhesive cures in 24 hrs. at 70 degrees F). The hot air is also good for drying excess xylene solvent from the patch areas.

1.) Place patching floats under the area to be patched if the repair is to be made with water in the reservoir.

2.) Clean area to be patched as thoroughly as possible without damaging the Hypalon membrane. Nylon pot scrubbers with Simple Green can be used. See below for further information regarding the size of the area to be cleaned.

3.) For holes up to ½" diameter: Cut a patch 3" in diameter. Cut another patch 9" in diameter. Clean both patch surfaces to be welded with xylene solvent and a clean rag. (For holes over ½" diameter, the
4.) Mix the appropriate amount of aged Hypalon repair adhesive adhesive.

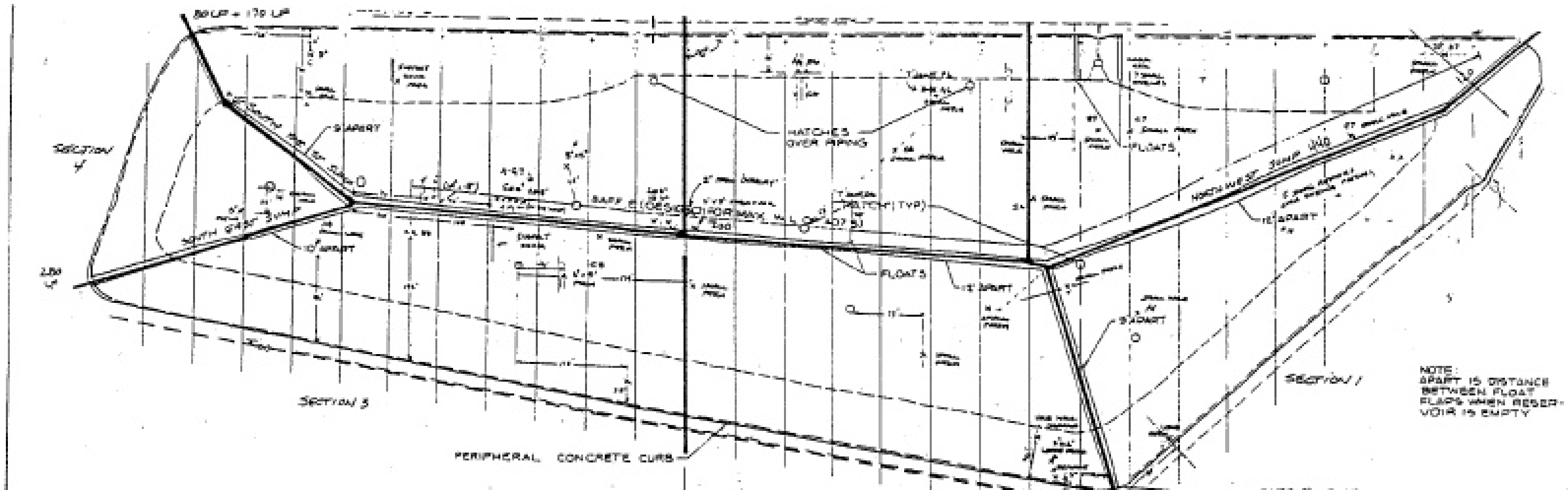
5.) Clean the cover area to be patched again, this time using xylene solvent being extremely careful not to get any solvent into the hole, then apply the 3" diameter patch to the cover using NSF approved Hypalon adhesive using the standard patching procedure (heat gun and 2" nylon or rubber seaming roller).

6.) Clean the entire 9" diameter area over the patch, including the top of the first patch.

7.) Paint the entire area to be patched as well as the bottom of the patch with the aged Hypalon adhesive allowing it to dry to the point where you can touch it without it coming off on your finger.

8.) Paint both surfaces again with the adhesive and roll with a 2" flat nylon or rubber roller until all air and excess adhesive has been squeezed out.

9.) Check for loose edges with fingernails or a probe. Do not be too vigorous with this check as the adhesive has not cured yet. You may check with a probe more vigorously 24 hrs. after the patch was installed.



BURKE FLOATING COVER DESIGN

PATENT NO. 3.991.900

	08/80	AS BUILT IN THE FIELD		
NO.	DATE	REVISION	BY	APPR.

CLENDENEN & ASSOCIATES CONSULTANTS, INC
CONSULTING ENGINEERS · AUBURN, CALIFORNIA

SAN JUAN SUBURBAN WATER DISTRICT
CONTRACT NO. 7904, 7905
HINKLE RESERVOIR IMPROVEMENTS
FOR
SIDNEY N. PETERSON
WATER TREATMENT PLANT

RESERVOIR COVER DESIGNS

R. K. FROBEL & ASSOCIATES
GEOSYNTHETICS CONSULTING ENGINEERS

Mr. Alex Peterson
Kennedy/Jenks Consultants
3336 Bradshaw Road, Suite 140
Sacramento, CA 95827

December 8, 2000

RE: San Jaun Water District, Granite Bay, CA
Sidney N. Peterson Water Treatment Plant
Hinkle Reservoir Hypalon Floating Cover Test Evaluation Report

Dear Mr. Peterson:

At the request of Kennedy/Jenks Consultants, I accompanied Mr. Alex Peterson on a site visitation and floating cover inspection at the San Jaun Water District on October 25, 2000 for the purpose of directing sample extraction, meeting with the lining subcontractor to discuss inspection methods and to coordinate test sample testing at both the manufacturer (Burke Rubber Company) and an independent third party laboratory (See the site visit report dated October 26, 2000). The following is a summary of the test program results, estimated life expectancy and recommendations as related to the Hypalon Floating Cover on the Hinkle Reservoir.

Introduction

The floating cover system on the Hinkle Reservoir is now 20 years old and is composed of 45 mil (1.14mm) thick Hypalon or Chlorosulfonated Polyethylene (CSPE). The Hypalon Geomembrane Cover is internally reinforced with 2 plys of scrim sandwiched between three layers of Hypalon resulting in a 5 ply construction. The two reinforcing scrim layers are each 16 x 8, 2:1 Leno Weave Polyester (8 x 8, 250 denier Apparent). The top surface is colored a tan or earth tone and the underside is black. The original Hypalon roll goods were manufactured by Burke Rubber Company, San Jose, CA as product number M-153 (Potable Water Grade). The roll goods were then fabricated into panels, delivered to the site and field assempled into the floating cover. The floating cover is a defined sump tensioned plate Burke floating cover design (patent no. 3,991,900). The design engineer for the Hinkle reservoir liner system and floating cover was Clendenen & Assocates Consultants, Inc., Auburn CA. It should be noted that the Hinkle Reservoir is reportedly the first to use a self draining (gravity feed) design where the rainwater is removed through flexible hoses fitted with penetration fittings at the Hypalon cover pipe manifold (bottom of sumps) and the base of the reservoir.

Upon arrival at the San Jaun Water District, we met with San Jaun District personnel and the Inspection/repair subcontractor, Colorado Lining International (CLI). Due to the inclement weather and water collecting on the cover, only one sample (sample B-1) was extracted and on site training of maintenance personnel was initiated. All remaining samples were identified as to location, extracted at a later date and forwarded to the manufacturer and independent laboratory for testing by CLI.

Hypalon Test Program and Results

Colorado Lining International (CLI) extracted four samples of the Hypalon cover material from preselected quadrants as shown on the CLI inspection drawing. Each sample was approximately 20 inches in width by 40 inches in length with either a factory or field seam centered on the length. The samples were each cut in two equal parts with one sent to Burke Environmental (manufacturer) and one sent to Precision Geosynthetic Laboratories (third party independent laboratory). The four samples extracted were as follows:

A-1 North Quadrant - discolored surface area taken over water (factory seam)
B-1 East Quadrant - on slope just above surface water (field seam)
C-1 South Quadrant - on slope just above surface water (factory seam)
D-1 West Quadrant - on slope just above surface water (factory seam)

The reader is referred to the CLI Inspection report summary and as built drawing for exact locations of sample extraction and subsequent repair and repair proceedures.

Each of the samples were extracted, photographed, cut in half, immediately packaged in plastic and submitted to both the manufacturer and independent laboratory for testing. The following tests were requested to be run on the samples:

Thickness	ASTM D 751	5 replicates
Water Absorption	ASTM D 471	3 replicates
Ply Adhesion	ASTM D 413/A	3 replicates
Tensile Strength/ Elongation	ASTM D 751/Grab	2 MD replicates 2 TD replicates
Seam Shear Strength	ASTM D 751	2 replicates
Hydrostatic Burst	ASTM D 751/A	4 replicates
Surface Cracking	Photomicrograph	1 @ 30X
Cut Edge Section	Photomicrograph	1 @ 30X

Test Results

Tables 1 and 2 sumarize the test results from both the manufacturer and the independent laboratory for samples A-1, B-1, C-1 and D-1. Table 3 sumarizes the average values for both laboratories as well as typical average material properties taken from the original manufacturer data sheets for 5 ply 45 mil Hypalon as a comparison. Percent changes in properties are also shown in Table 3. It should be noted that differences in values between the manufacturer and independent lab may be due to technique as well as statistical variance.

Table 1
Summary of Test Results - Manufacturer Laboratory
Hypalon - 5 ply 45 mil

Property	Test Method	Units	Sample Number A-1	B-1	C-1	D-1
Thickness	ASTM D 751	mil	48.6	45.6	47.0	47.6
Tensile Strength	ASTM D 751	lbs				
Ultimate MD			254.5	229	243	269.5
Ultimate TD			200	167	220	251
Tensile Strain	ASTM D 751	%				
Ultimate MD			105	71	78.5	70
Ultimate TD			87	72	90	80
Seam Strength	ASTM D 751	lbs	183.5	142.5	189	195
Seam Efficiency		%	91.7	85.3	85.9	77.6
Burst Strength	ASTM D 751/A	psi	166.2	176.2	186.2	191.2
Ply Adhesion	ASTM D413	lb/in	10.0	10.3	10.6	10.3
Water Absorption	ASTM D 471	%	6.2	4.5	2.6	8.5

Notes: 1. Sample B-1 includes a field fabricated chemical fusion seam. All other samples include factory thermal seams. Sample A-1 is discolored taken over water.
2. Seam Efficiency is measured as a percentage of material strength in the TD.
3. MD = machine or long direction; TD = cross machine or transverse direction.
4. Water Absorption test was used to measure as received moisture content.

Table 2
Summary of Test Results - Independent Laboratory
Hypalon - 5 ply 45 mil

Property	Test Method	Units	Sample Number A-1	B-1	C-1	D-1
Thickness	ASTM D 751	mil	40.9	40.7	44.5	45.0
Tensile Strength	ASTM D 751	lbs				
Ultimate MD			234	245	302	292
Ultimate TD			210	210	244	233
Tensile Strain	ASTM D 751	%				
Ultimate MD			113	100	86	103
Ultimate TD			143	101	137	97
Seam Strength	ASTM D 751	lbs	190	174	219	219
Seam Efficiency		%	90.5	82.3	89.7	94
Burst Strength	ASTM D 751/A	psi	207	195	208	228
Ply Adhesion	ASTM D413	lb/in	NA	NA	NA	NA
Water Absorption	ASTM D 471	%	3.3	3.8	3.85	3.4
Surface Cracks	30X Photomicrographs		NC	NC	NC	NC
Section/Plys	30X Photomicrographs		5ply	5ply	5ply	5ply

Notes: 1. Sample B-1 includes a field fabricated chemical fusion seam. All other samples include factory thermal seams. Sample A-1 was discolored taken over water.
2. Seam Efficiency is measured as a percentage of material strength in the TD.
3. MD = machine or long direction; TD = cross machine or transverse direction.
4. Water Absorption test was used to measure as received moisture content.
5. NC = no cracking at the Hypalon surface

Table 3
Summary of Typical 1980 and Average of 2000 Measured Values
Hypalon - 5 ply 45 mil

Property	Test Method	Units	1980 T.V.	Man A.V.	%	Ind A.V.	%
Thickness	ASTM D 751	mil	45	48.6	+8	42.8	-4.8
Tensile Strength	ASTM D 751	lbs					
Ultimate MD			200	249	+25	268	+34
Ultimate TD			200	210	+5	224	+12
Tensile Strain	ASTM D 751	%					
Ultimate MD			245	81	-67	100	-59
Ultimate TD			245	82	-66	120	-51
Seam Strength	ASTM D 751	lbs	175	177	+1	200	+14
Seam Efficiency		%	87.5	85	-3	89	+2
Burst Strength	ASTM D 751/A	psi	175	180	+3	209	+19
Ply Adhesion	ASTM D413	lb/in	12	10.3	-14	NA	NA
Water Absorption	ASTM D 471	%	5	5.45	+9	3.6	-28

Notes: 1. Typical Average values (T.V.)are taken from Burkes original published data sheet. Manufacturers average values (A.V.) and Independent Lab average values (A.V.) are averages taken from tables 1 and 2.
2. The % column is % change from original published values.
2. Seam Efficiency is measured as a percentage of material strength in the TD.
3. MD = machine or long direction; TD = cross machine or transverse direction.
4. Water Absorption test was used to measure as received moisture content.

Of particular note, sample A-1 which was taken over water and was discolored due to standing water, showed no major loss in properties and in fact was higher in seam efficiency than the other samples. Sample B-1 which contained the only field fabricated chemical fusion seam showed lower overall seam strength and seam efficiency than the factory seams in the other samples. However, at an average seam efficiency of 84% the efficiency still exceeds todays typical specifcation requirements of 80%.

When examining the comparisons of typical 1980 values and the average year 2000 test values, there is obviously an overall increase in mechanical strength due to the age and cross-linking of the CSPE polymer. There is little trend in decreased values other than elongation which was to be expected with increased tensile strength. The ply adhesion values are reported as less than expected based on the 1980 reported value of 12 ppi. However, even today, manufacturers report less than 10 ppi for new Hypalon. The original actual value for the Hinkle Reservoir material may have been in the range of 8 ppi based on the authors experience with similar material.

In general, the testing shows that the material is in excellent physical/mechanical condition considering the 20 years of continuously exposed service life. Although original material property values were not available, the typical average property values published by Burke Rubber Company were available for comparison and generally show an increase in mechanical strength due to the CSPE or Hypalon curing over time. As the CSPE cures and cross-links, chemical bonds between polymeric chains are formed to yield an insoluble, three dimensional structure. The cross-linked CSPE has higher tensile, burst and seam strength because the breaking strength of the CSPE has increased over time with a subsequent decease in elongation properties.

Photomicrographic examination of the Hypalon surface and cut sections showed no significant surface cracking which is also an indication of excellent resistance to the effects of ozone and ultraviolet radiation due to the continuous exposure to sunlight and the environment. The photomicrographs indicate little or no deterioration of the surface which is a good indication of the current material condition and resistance to significant degredation for continued service life.

Hypalon Cover Inspection Report by CLI

The summary of the report by CLI, "Hinkle Reservoir Floating Cover Inspection Summary", was reviewed and discussed with Mr. John Glitsch. The report generally characterized the 20 year old cover as in very good condition considering the lack of maintenance cleaning and proper repair proceedures. The only obvious area that needs repair and correction is the outlet structure. Other than that, approximately 60-70 small areas need repair patches and the entire cover and sumps need to be cleaned. In addition, some areas of the cover require entrapped air removal and float correction for proper surface drainage into the sumps. There was no indication during the inspection of cover material deterioration or weak seams even at the discolored areas noted on the cover.

CONCLUSIONS AND RECOMMENDATIONS

1. Based on the above test program results and summary of the 20 year surface inspection, the general condition of the Hypalon Floating Cover System is very good considering the almost 20 years of continuous operation of the Hinkle Reservoir. There is no reason that the cover material, seams and associated attachments will not last an additional 15 to 20 years minimum in service life with proper continued maintenance.

In addition to the 20 year third party inspection completed by CLI, a thorough 20 year comprehensive maintenance cleaning/repair of the cover system should be completed as soon as practical. This may require complete drawdown of the reservoir. At the very least, the cover should be cleaned of debris, all repairs made and all perimeter attachments, structure attachments and hatch covers repaired as required. The sump drain pipe headers must be cleaned and flushed and ballast tubes inspected. (Again this may require drawdown). Drain pipes may be accessed by underwater vacuum cleaning equipment (similar to portable gold dredges) from above the sumps to remove most of the accumulated debris.

2. The current maintenance inspection program and reporting is acceptable but needs to be improved and updated to current AWWA requirements. It is imperative that daily visual perimeter observations be continued and that the weekly cover inspection with recorded observations and repairs be kept current.

Recommendation. A comprehensive 20 year maintenance cleaning should be completed as outlined above in item 1.

The weekly inspections should be augmented with a thorough yearly detailed inspection of all cover areas, hatches, connections and sumps. A yearly underwater inspection program is currently being accomplished for all underwater connections and is recommended to be continued for future inspections. The top cover inspection should be completed in concert with the underwater inspection. The California - Nevada Section of the AWWA has recommendations for maintenance and inspection in their recently published manual on reservoir covers (April, 1999). At the very least, routine maintenance should be documented with inspection forms similar to those included in the appendix of this report.

Once the 20 year cleaning is complete, it is not recommended to clean the surface of the cover more than once every 2 years. More frequent cleaning is not recommended due to the increased potential for mechanical damage. Because the reservoir is limited to access, fenced, has 24 hour operations personnel present and air blown debris is limited to fine material, the potential for damaging objects or material accumulation is small.

3. It was noted again on the October 25, 2000 site visit that several sections of the rainwater collection sumps were full of water and that algae growth was prevalent. In

addition, it was obvious that surface water was not draining properly into the sump at the northeast end of the reservoir (area where the cover is discolored due to standing water).

Recommendation. The rainwater removal system should be inspected. The header pipes in the sump bottom may be clogged with debris or biological growth. The header pipes are reportedly 8 ft. long 4 inch schedule 80 PVC with 1/2 inch holes drilled every 6 inches. Some of the header pipes may not be draining and may account for the standing water and discoloration on the cover surface. These pipes can be cleaned and/or replaced during the suggested 20 year maintenance cleaning. Again, algae growth should be immediately removed and cleaned with methods as recommended by CLI. It was also noted that water could not flow into the sump at the northeast end due to restriction at the float area. This problem should be corrected as soon as possible to eliminate standing surface water.

4. Significant surface water was again noted at the outlet structure and may be associated with cover leakage due to damage or open seams.

Recommendation. Draw Reservoir down to below the suspected areas around the outlet structure and inspect cover and all seam areas - clean and repair with CLI recommended methods. Again, this area is subject to significant stress during drawdown and the numerous field seams can become unbonded at areas of high stress. This entire outlet area should be redesigned to properly accomodate the cover movement.

5. Numerous areas of entrapped air were noted, especially during the rain event on October 25.

Recommendation. Move air to hatch areas or install supplemental floating vents as recommended by CLI. In any event, trapped air must be removed to prevent wind uplift.

This concludes the report on the San Jaun Water District Hinkle Reservoir Cover testing and evaluation as well as recommendations. If you have any questions, please give me a call at 303-679-0285.

Sincerely Yours,

Ronald K. Frobel, P.E.

attachments 1 - 5

ATTACHMENT 1

SITE PHOTOGRAPHS

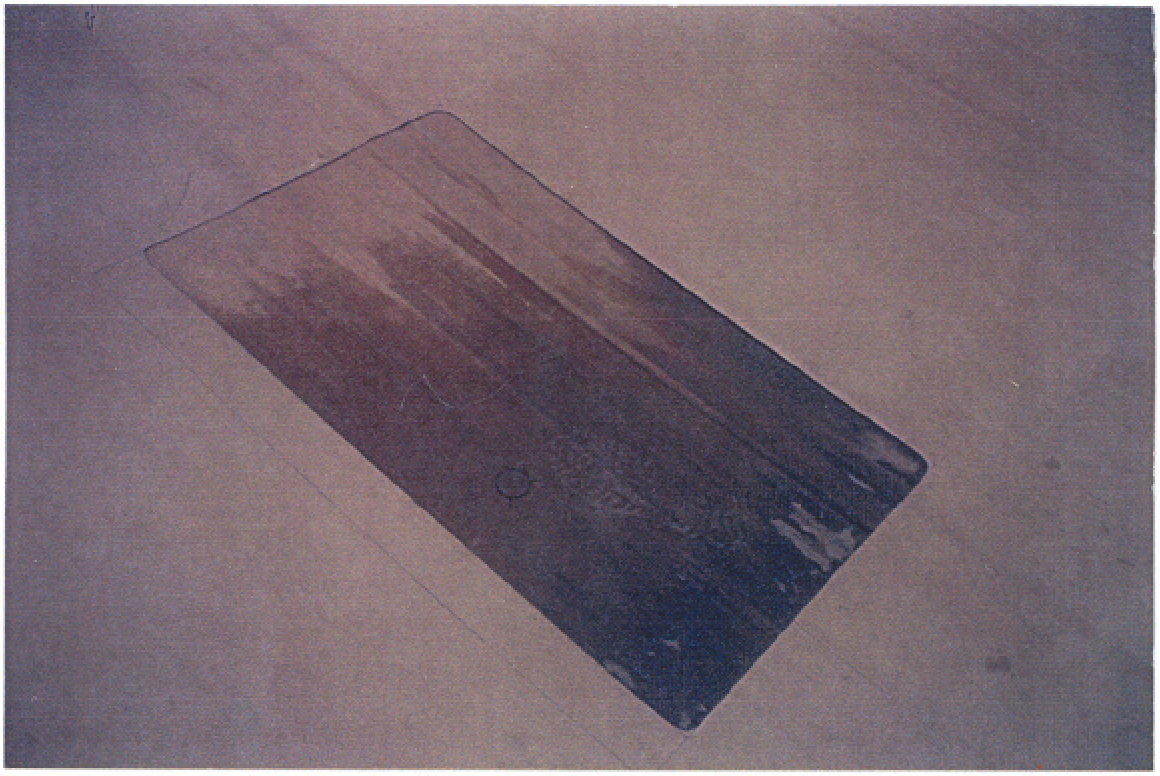


Figure 1. Photo showing typical test sample cutout area on a slope (note original bottom lining material which appears to be in excellent condition)

Figure 2. Photo showing numerous entrapped air bubbles under the cover

Figure 3. Photo showing the discolored cover area due to standing water - NE quadrant near inlet

Figure 4. Photo showing poor surface drainage into the sump - NE sump

ATTACHMENT 2

ORIGINAL BURKE LITERATURE

Burke Rubber Company
a division of Burke Industries

2250 South Tenth Street, San Jose, California 95112
(408) 297-3500

M-153 Black, Potable Grade
M-303 Black, Industrial Grade
45 Mil, 8x8-250d Scrim
Hypalon® Flexible Membrane

45 MIL SUPPORTED HYPALON (CSPER) MEMBRANE SPECIFICATION GUIDE

The following is a specification for Burke's flexible membrane pond lining material, and is recommended for use in specifying lining materials.

PHYSICAL PROPERTIES: (These are interim values and subject to change).

Property	Test Method	Minimum Specification*	Typical Avg. Values
Thickness			
1. Total, overall (mils)	ASTM D751	41	45 nominal
2. Min. over scrim (mils)	Optical Method	11	pass
Tensile Properties (each direction)	ASTM D751 Grab Method		
1. Breaking Strength (pounds)			
Fabric		90	145
Membrane Rupture		120	200
2. Elongation at Break			
Fabric		15%	22%
Membrane Rupture		125%	245%
Tear Propagation (pounds)	ASTM D751, Tongue Tear (8"x8" sample)	12	23
Hydrostatic Resistance (psi)	ASTM D751, Method A Procedure 1	140	175
Puncture Resistance (pounds)	FTMS 101B Method 2031	–	195
Bonded Seam Strength (pounds)	ASTM D751, Modified (12 in./min.)	96	175
Ply Adhesion (lbs./in. width)	ASTM D413 Machine Method, Type A (12 in./min.)	12 (or film tearing bond)	pass
Ozone Resistance	ASTM D1149, 1/8" bent loop, 100 pphm, 104°F, 7 days	No cracks at 7x magnification	pass
Low Temperature (refer to para. following)	ASTM D2136, 1/8" mandrel, 4 hrs. @ −40°F	pass	pass at −45°F

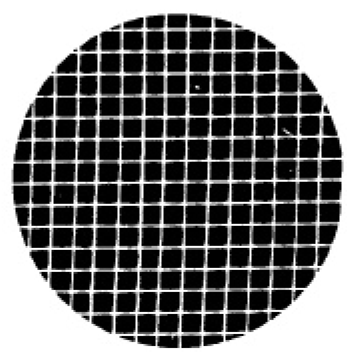
These specification tables represent current opinion of the data points to characterize the membrane product as produced and are not necessarily appropriate for product performance or installation or engineer-ing design criteria 'per se'. (For example, the low temperature resistance numbers represent qualities for a few minutes at a given temperature and must not be interpreted or extrapolated into installation temperature qualities or comparisons").

**Minimum specification limits are currently proposed industry standards for this type of flexible membrane product. Burke Quality*

OTHER MATERIAL SPECIFICATIONS AND CHARACTERISTICS:

A. The thermoplastic elastomer lining material shall be manufactured from a synthetic rubber compound designed to contain Hypalon Type 45 synthetic rubber as the principal elastomer. The compound used in manufacture of the laminate shall conform to the specification of Burke 30 Mil Unsupported Hypalon® M-148 Potable Grade or M-321 Industrial Grade material. (Specification Sheet BR00208).

B. The thermoplastic elastomer lining shall be manufactured totally by calendering, with each ply of rubber laminated to the next ply through the openings in the scrim weave to produce a pinhole-free construction. The open-weave supporting fabric (scrim) shall have a 16 x 8, 2:1 leno weave 140 warp/250 fill denier (8 x 8-250d apparent) construction. The two plies of supporting fabric shall be totally encapsulated within three plies of rubber, giving a 5 ply construction of nominal 45 mil thickness. Exposed fabric or indication of delamination will not be permitted.



16 x 8, 2:1 leno weave
140 warp/250 fill denier
(8 x 8-250d apparent)

C. "Potable Grade" Hypalon is suitable for the storage of potable water. A colored top ply in white, blue, tan or green is available at additional cost. Operational service temperature should not exceed 120°F maximum. "Industrial Grade" Hypalon for non-potable use is available only in black. Operational service temperature should not exceed 160°F maximum. Brief or intermittent exposure to higher temperatures may occasionally be tolerated, but may reduce the effective service life of the liner. The required grade must be specified, and the liner manufacturer shall certify compliance.

D. All membrane lining materials transmit water vapor at a very low rate, which is related to the temperature and relative humidity. The permeability of Burke Hypalon, as expressed in Metric Perm-Centimeters is less than 3×10^{-3} for a 30 mil thickness. (This translates to approximately one gallon per acre per year @ 72°F and 50% R.H.). Permeability can also be expressed as a "K" factor in centimeters/second, a test generally used for more porous materials. Burke Hypalon has a "K" factor in the range of 10^{-12}.

A STRONG CASE FOR BURKE

Defined Sump Tensioned Plate Floating Covers

- Durable
- Weather, Ozone and Chemical Resistant
- Easy to Maintain

A STRONG CASE FOR BURKE

HINKLE: A PIONEER IN FLOATING RESERVOIR COVERS

Since its reconstruction in 1980, the Hinkle Reservoir, Roseville, CA has attracted many onlookers from both the U.S. and abroad. Engineers, administrators and legislators are drawn to the site to learn about one of the country's most successful designs for floating reservoir covers and liners; while residents are impressed with improved water quality and the reservoir's appearance.

"We've had hundreds of people come to the site before planning their own reservoir construction," said Jim English, assistant general manager of the Sidney N. Peterson Water Treatment Plant, San Juan Suburban Water District. "In fact, after touring Hinkle, officials from the government of South Australia decided to use the identical floating cover and liner design we used based on Du Pont HYPALON® synthetic rubber in their reservoir construction," he said.

"We considered other alternatives, such as steel and concrete tanks and rigid covers, but being a municipal facility, the costs were too high. The range was between $6 million and $12 million," English said. "The total cost of reconstructing the 62-million gallon capacity reservoir was only $2 million (or $.04 a gallon) including the liner and cover."

"But the cost alone doesn't matter if the product doesn't work," English said. "At Hinkle, we've been completely satisfied with the liner and the cover's performance in terms of effectiveness, ease of maintenance and repair, improved water quality and appearance."

The proprietary patented design principles of the "Defined Sump Floating Cover" manufactured by Burke Industries of San Jose, CA were adopted by the district's engineers, Clendenen Associates-Consultants, Inc., Auburn, CA. "Essentially, the membrane, when attached to the perimeter of the reservoir, has enough slack to rise and fall with varying water levels," said Dennis Gerber, manager, Research and Development, for Burke. "The slack is taken up by sandbag weights to form a rainwater sump down the center of the cover."

This U.S. patented design enables rainwater to flow toward the center of the cover. Collector headers built into the sump allow the water to drain by gravity through 4-inch flex hoses outside the reservoir so that contact is never made with the water supply. This feature differs from most floating covers which use pumps to remove rainwater.

Another unique feature of the cover is the use of a baffle based on HYPALON. The baffle extends from the bottom of the cover to the floor of the reservoir at the central sump, spanning the length of the reservoir. It is held in place by sand-filled tubes of HYPALON. The baffle forces the water flow from the inlet to the far end of the reservoir which greatly improves water circulation.

Proper water circulation within the reservoir and an efficient drainage system for the cover are both crucial in preventing contamination. According to English, "Since there is a 12.5-acre surface area, every one inch of rainfall leaves 385,000 gallons of water on the cover. In 1984, we had 15.6 million gallons of water on the cover. This cover design allowed for easy water drainage. After a storm, water has never remained on the cover for more than three days."

In addition to easy water removal, the district has experienced ease of maintenance and repair. "We had an accident after a storm, where a piece of metal danced across the cover, creating 30 holes. Because the cover design provides stability for safe walking, we were able to walk on top and easily isolate and repair the holes in less than three days," English said. "That was in our first year. Since then, we have not needed to budget for maintenance on this reservoir. And we've only had to clean the cover once."

The floating cover based on Du Pont HYPALON synthetic rubber provides ease of maintenance and improved water quality for the 62-million gallon Hinkle Reservoir.

The cover has also prevented the degradation of the potable water from the elements. "Before we had the cover, we needed to use 650 tons of chlorine per year to purify the water. That's because there was a significant loss of chlorine from exposure to sunlight," said English. "Now, we only need to use 250 tons per year."

He continued, "We're particularly impressed that the quality of the water that goes into the reservoir is identical to its quality going out. The level has remained constant at .03 Nephlometric Turbidity Units (NTU). Taste and odor problems are gone. And HYPALON is one of the few materials certified by the FDA for potable water contact."

A final consideration for the district was to maintain the beauty of the natural environment. "Since the reservoir spans such a broad area, we needed a cover that could be designed to blend in with the surroundings," said English. "The cover based on HYPALON was able to be manufactured in an 'earth' color. It literally looks like a part of the environment.

"But most importantly, we've exceeded the EPA and FDA standards of improving water quality for our community at a reasonable cost," English said.

Burke Rubber Company
a division of Burke Industries
2250 South Tenth Street, San Jose, California 95112
(408) 297-3500

ATTACHMENT 3

RECOMMENDED INSPECTION FORMS - AWWA 1999

Reservoir Floating Cover
Sample Perimeter Inspection Report

Date: ______________________

Time: ______________________

Operator: ______________________

Avg WS Elev: ______________________

☑ Check all that apply

Adverse Weather (check/complete only those that apply):

☐ Extreme heat   High: ____________ F

☐ Extreme cold   Low: ____________ F

☐ High Winds   Direction: ________

    Speed: __________ MPH

☐ Rain   Rainfall: __________ inches

☐ Other   ______________

Reservoir Perimeter Check:

No   Yes

☐   ☐ Damage to perimeter fencing or gates?

☐   ☐ Debris on cover?

☐   ☐ Visible damage to cover?

☐   ☐ Rainwater removal system malfunctioning?

☐   ☐ Ponded water?

☐   ☐ Visible damage at structures?

☐   ☐ Other?

Provide details for those items checked "Yes" above:

____

____

____

____

____

Complete the table for each required repair

Required Repair	Date Reported to O&M Supervisor	Repair Assigned To	Date Assigned	Completion Date

Reservoir Floating Cover
Sample Detailed Inspection Report

Date: ______________________________
Time: ______________________________
Operator: ___________________________

☑ Check all that apply

Daily Inspection Report completed? ☐ No ☐ Yes

Detailed Inspection:

No	Yes	
☐	☐	Unsecured hatches?
☐	☐	Debris on cover or in troughs?
☐	☐	Damage to rainwater removal system?
☐	☐	Areas of ponded surface water?
☐	☐	Standing water in trough?
☐	☐	Leakage at previous repairs?
☐	☐	Membrane damage/pinholes/abrasion?
☐	☐	Seam failure?
☐	☐	Excessive air pockets under cover?
☐	☐	Damage or wear at structures?
☐	☐	Damage to vent screens
☐	☐	Other?

Provide details for those items checked "Yes" above:

____
____
____
____
____

Complete the table for each required repair

Required Repair	Date Reported to O&M Supervisor	Repair Assigned To	Date Assigned	Completion Date

Reservoir Floating Cover
Sample Maintenance Report

Date: ______________________________
Time: ______________________________
Operator: ___________________________

☑ Check all that apply

Maintenance Period: ☐ Bi-Annual ☐ Annual ☐ Other ____________

Maintenance Performed:

Bi-Annual
- ☐ Trough flushing
- ☐ Clean rainwater removal pumps
- ☐ Service rainwater removal pumps
- ☐ Clean pump on/off probes

Annual
- ☐ Cover washdown

☐ Other ____
____
____

List items requiring further maintenance or repair:

____
____
____
____
____

Complete the table for each required repair

Required Repair	Date Reported to O&M Supervisor	Repair Assigned To	Date Assigned	Completion Date

ATTACHMENT 4

LABORATORY TEST RESULTS

Precision Geosynthetic Laboratories

CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

VERIFICATION OF MATERIAL PROPERTIES
(PGL Job No. 001429)

MATERIAL DESCRIPTION: Hypalon seam

ORIGIN: COLORADO LINING

DATE RECEIVED: November 2, 2000   **DATE REPORTED:** November 7, 2000

SAMPLE IDENTIFICATIONS:

SAMPLE ID	PRECISION CONTROL NUMBER
A	57819

TESTS REQUIRED:

TEST METHOD	DESCRIPTION
ASTM D751, *NSF Modified*	Thickness
ASTM D570	Water Absorption
ASTM D751	Grab Tensile
ASTM D751	Bonded Seam Strength
ASTM D751, *Method A*	Hydrostatic Resistance
Photomicrograph	Photograph by Microscope
ASTM D413, *Method A*	Ply Bond Adhesion

TEST CONDITIONS: The sample was conditioned for a minimum one hour in the laboratory at 22 $\pm$ 2°C (71.6 $\pm$ 3.6°F) and at 60 $\pm$ 10% relative humidity prior to test.

TEST RESULTS:

The test results are summarized in Tables 1. The units in which the data are reported are included on the tables. A 4 by 5 in. black and white photograph, taken is attached to table 1A.

PRECISION GEOSYNTHETIC LABORATORIES

CAL-LAP
Initials: #: 013
Date: 11/8/00

Edith Pintor
Quality Assurance

Cora B. Queja
Vice President

1160 North Gilbert Street, Anaheim, CA 92801, Tel # 714-520-9631, Fax # 714-520-9637

TABLE 1.

MATERIAL PROPERTIES

CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

Date Received : **11/2/00**
Date Reported: **11/7/00**
Client Sample ID : **A**
Material Description: **Hypalon Seam**

QC'd by:
PGL Job No. : **001429**
PGL Control No. : **57819**

METHOD	DESCRIPTION	SPECIMENS 1	2	3	4	5	6	7	8	9	10	Avg.	Std. Dev.	Proj. Specs.
ASTM D751	Thickness (mils)													
NSF modified		40.4	40.4	41.1	41.5	41.0						40.9	0.5	
ASTM D751	Grab Tensile													
	Tensile Strength (lbs)													
	MD	247	222									234	18	
	TD	209	212									210	2	
	Elongation at Break (percent)													
	MD	123	104									113	13	
	TD	141	146									143	3	
ASTM D751	Hydrostatic Resistance (psi)													
Method A		205	211	205	205							207	3	
ASTM D570	Water Absorption - Weight Gain (percent)													
		3.33	3.34	3.15								3.28	0.11	
ASTM D413	Ply Bond Adhesion (lbs/ in.- width)													
Method A	*MD* (NOTE: CAN'T PEEL)													
	TD													
ASTM D751	Bonded Seam Strength (lbs)													
		188	191									190	2	
	Break Type [Locust of Break Code]													
		FTB[BRK]	FTB[BRK]											

MD - MACHINE DIRECTION
TD - TRANSVERSE DIRECTION
FTB - FILM TEAR BOND
BRK - BREAK IN THE SHEET THROUGH BOTH THE FABRIC AND THE PLIES OF THE POLYMER

Precision Geosynthetic Laboratories

Precision Geosynthetic Laboratories

CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

VERIFICATION OF MATERIAL PROPERTIES
(PGL Job No. 001431)

MATERIAL DESCRIPTION: Hypalon

ORIGIN: COLORADO LINING

DATE RECEIVED: November 2, 2000 **DATE REPORTED:** November 7, 2000

SAMPLE IDENTIFICATIONS:

SAMPLE ID	PRECISION CONTROL NUMBER
B	57822
C	57823
D	57824

TESTS REQUIRED:

TEST METHOD	DESCRIPTION
ASTM D751, *NSF Modified*	Thickness
ASTM D570	Water Absorption
ASTM D751	Grab Tensile
ASTM D751	Bonded Seam Strength
ASTM D751, *Method A*	Hydrostatic Resistance
Photomicrograph	Photograph by Microscope
ASTM D413, *Method A*	Ply Bond Adhesion

TEST CONDITIONS: The samples were conditioned for a minimum one hour in the laboratory at 22 $\pm$ 2°C (71.6 $\pm$ 3.6°F) and at 60 $\pm$ 10% relative humidity prior to test.

TEST RESULTS:

The test results are summarized in Tables 1 through 3. The units in which the data are reported are included on the tables. A 4 by 5 in. black and white photograph, taken is attached to tables 1A through 3A.

PRECISION GEOSYNTHETIC LABORATORIES

GAI-LAP
Initials: [illegible] #: 013
Date: 11/8/00

Edith Pintor
Quality Assurance

Cora B. Queja
Vice President

TABLE 1.

MATERIAL PROPERTIES

CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

Date Received : **11/2/00**
Date Reported: **11/7/00**
Client Sample ID : **B**
Material Description: **Hypalon**

QC'd by:
PGL Job No. : **001431**
PGL Control No. : **57822**

METHOD	DESCRIPTION		SPECIMENS 1	2	3	4	5	6	7	8	9	10	Avg.	Std. Dev.	Proj. Specs.
ASTM D751	Thickness (mils)														
NSF modified			40.4	40.6	41.0	40.7	40.7						40.7	0.2	
ASTM D751	Grab Tensile														
	Tensile Strength (lbs)														
		MD	224	266									245	30	
		TD	202	217									210	11	
	Elongation at Break (percent)														
		MD	107	92									100	10	
		TD	97	104									101	4	
ASTM D751	Hydrostatic Resistance (psi)														
Method A			190	190	205	195							195	7	
ASTM D570	Water Absorption - Weight Gain (percent)														
			3.62	3.80	3.89								3.77	0.14	
ASTM D413	Ply Bond Adhesion (lbs/ in.- width)														
Method A		*MD*	(NOTE: CAN'T PEEL)												
		TD													
ASTM D751	Bonded Seam Strength (lbs)														
			179	170									174	6	
	Break Type [Locust of Break Code]														
			FTB[BRK]	FTB[BRK]											

MD - MACHINE DIRECTION
TD - TRANSVERSE DIRECTION
FTB - FILM TEAR BOND
BRK - BREAK IN THE SHEET THROUGH BOTH THE FABRIC AND THE PLIES OF THE POLYMER

Precision Geosynthetic Laboratories

TABLE 2.

MATERIAL PROPERTIES
CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

Date Received : **11/2/00**
Date Reported: **11/7/00**
Client Sample ID : **C**
Material Description: **Hypalon**

QC'd by:
PGL Job No. : **001431**
PGL Control No. : **57823**

		SPECIMENS												
		1	2	3	4	5	6	7	8	9	10	Avg.	Std. Dev.	Proj. Specs.
METHOD	**DESCRIPTION**													
ASTM D751	Thickness (mils)													
NSF modified		44.0	45.0	44.7	44.0	44.7						44.5	0.5	
ASTM D751	Grab Tensile													
	Tensile Strength (lbs)													
	MD	322	281									302	29	
	TD	243	246									244	2	
	Elongation at Break (percent)													
	MD	70	102									86	23	
	TD	132	143									137	7	
ASTM D751	Hydrostatic Resistance (psi)													
Method A		210	210	205	205							208	3	
ASTM D570	Water Absorption - Weight Gain (percent)													
		3.95	3.82	3.78								3.85	0.09	
ASTM D413	Ply Bond Adhesion (lbs/ in.- width)													
Method A	*MD* **(NOTE: CAN'T PEEL)**													
	TD													
ASTM D751	Bonded Seam Strength (lbs)													
		222	216									219	4	
	Break Type Locust of Break Code													
		FTB BRK	FTB BRK											

MD - MACHINE DIRECTION
TD - TRANSVERSE DIRECTION
FTB - FILM TEAR BOND
BRK - BREAK IN THE SHEET THROUGH BOTH THE FABRIC AND THE PLIES OF THE POLYMER

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TABLE 3.

MATERIAL PROPERTIES

CLIENT: KENNEDY / JENKS CONSULTANTS

PROJECT: Hinkle Reservoir Floating Cover

Date Received : **11/2/00**
Date Reported: **11/7/00**
Client Sample ID : **D**
Material Description: **Hypalon**

QC'd by:
PGL Job No. : **001431**
PGL Control No. : **57824**

METHOD	DESCRIPTION		SPECIMENS 1	2	3	4	5	6	7	8	9	10	Avg.	Std. Dev.	Proj. Specs.
ASTM D751	Thickness (mils)														
NSF modified			45.0	45.1	45.0	45.1	45.0						45.0	0.1	
ASTM D751	Grab Tensile														
	Tensile Strength (lbs)														
		MD	283	301									292	13	
		TD	231	235									233	3	
	Elongation at Break (percent)														
		MD	97	108									103	8	
		TD	93	101									97	5	
ASTM D751	Hydrostatic Resistance (psi)														
Method A			230	225	230	225							228	3	
ASTM D570	Water Absorption - Weight Gain (percent)														
			3.55	3.31	3.23								3.36	0.16	
ASTM D413	Ply Bond Adhesion (lbs/ in.- width)														
Method A		*MD*	(NOTE: CAN'T PEEL)												
		TD													
ASTM D751	Bonded Seam Strength (lbs)														
			215	223									219	6	
	Break Type Locust of Break Code														
			FTB BRK	FTB BRK											

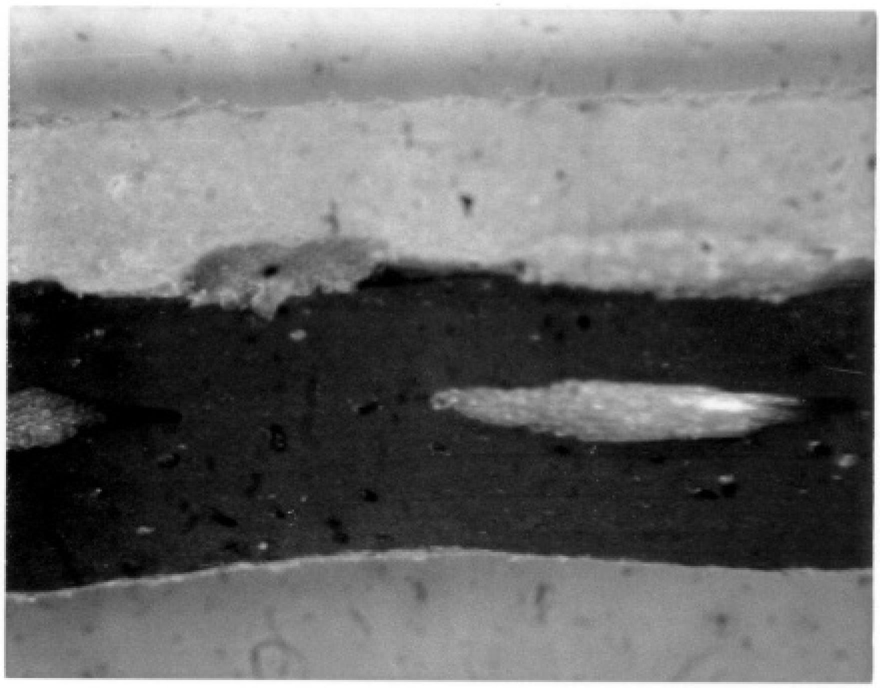
MD - MACHINE DIRECTION
TD - TRANSVERSE DIRECTION
FTB - FILM TEAR BOND
BRK - BREAK IN THE SHEET THROUGH BOTH THE FABRIC AND THE PLIES OF THE POLYMER

Precision Geosynthetic Laboratories

TABLE 1A.
MATERIAL PROPERTIES
CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

VERIFICATION OF MATERIAL PROPERTIES
(PGL Job No. 001429)

QC'd by: [signature]
November 7, 2000



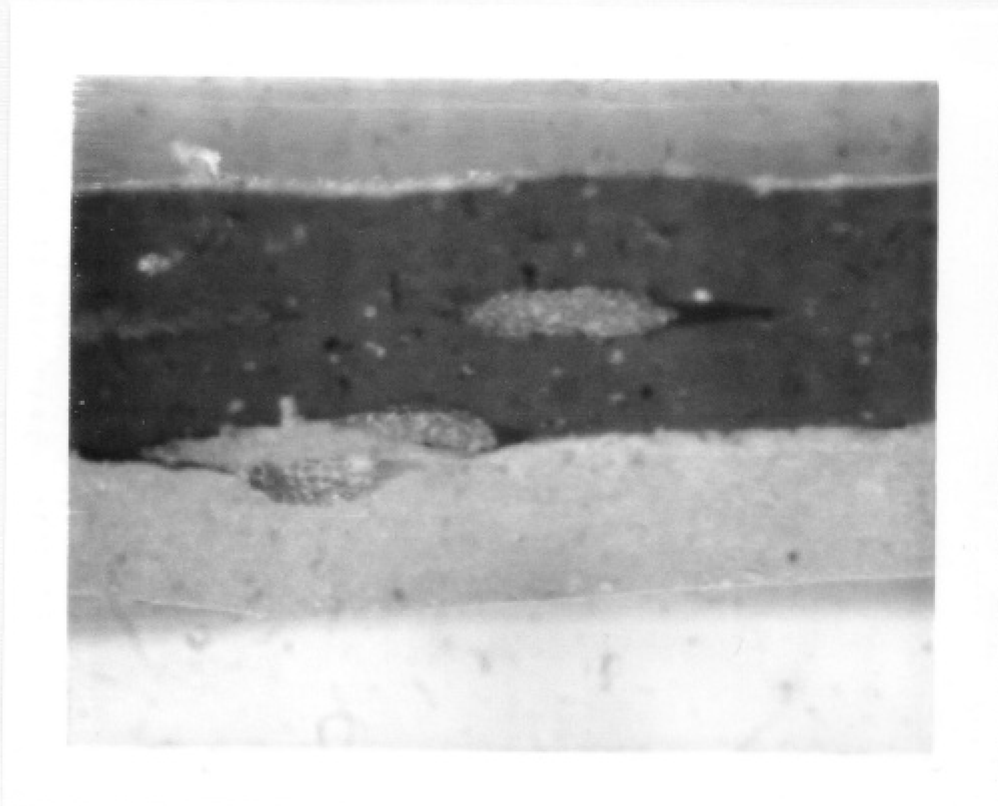
SAMPLE ID	CONTROL NUMBER	OBSERVATIONS
A	C #57819	

Precision Geosynthetic Laboratories

TABLE 1A.
MATERIAL PROPERTIES
CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

VERIFICATION OF MATERIAL PROPERTIES
(PGL Job No. 001431)

QC'd by: [signature]
November 7, 2000



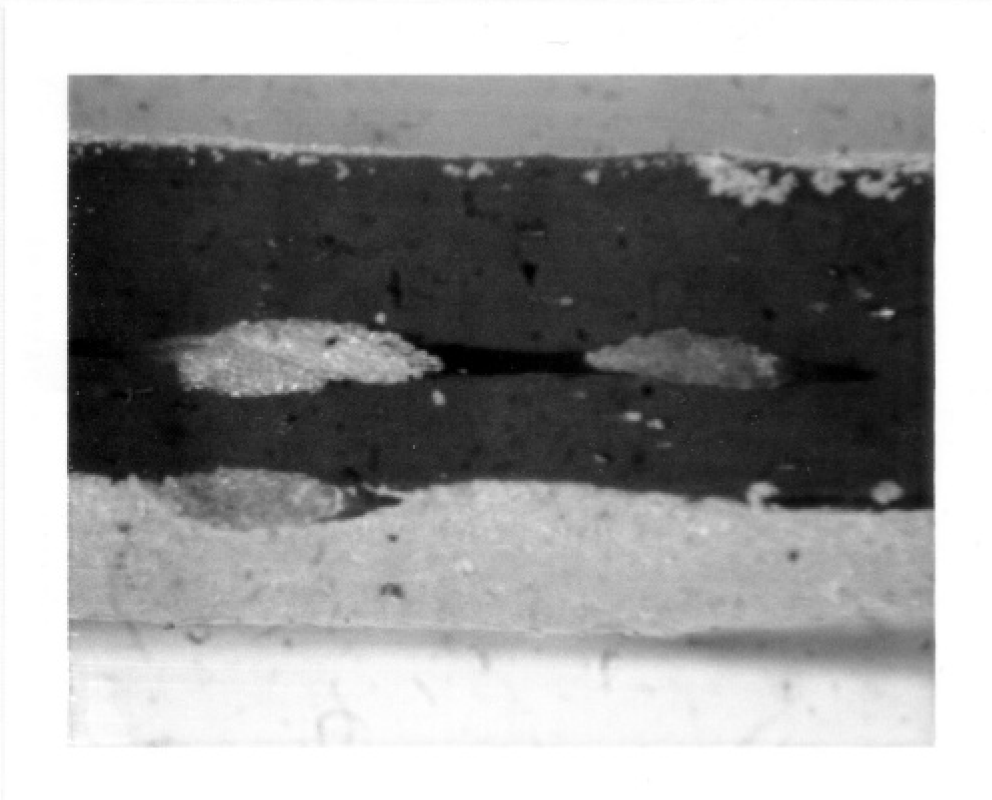
SAMPLE ID	CONTROL NUMBER	OBSERVATIONS
B	C #57822	

Precision Geosynthetic Laboratories

TABLE 2A.
MATERIAL PROPERTIES
CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

VERIFICATION OF MATERIAL PROPERTIES
(PGL Job No. 001431)

QC'd by: ____
November 7, 2000



SAMPLE ID	CONTROL NUMBER	OBSERVATIONS
C	C #57823	

TABLE 3A.
MATERIAL PROPERTIES
CLIENT: KENNEDY / JENKS CONSULTANTS
PROJECT: Hinkle Reservoir Floating Cover

VERIFICATION OF MATERIAL PROPERTIES
(PGL Job No. 001431)

QC'd by:
November 7, 2000

SAMPLE ID	CONTROL NUMBER	OBSERVATIONS
D	C #57824	

FAX TO RON [illegible]
(303) 679-0285
FAX (303) 679-8955

Burke mercer
A DIVISION OF BURKE INDUSTRIES, INC.

November 15, 2000

Kennedy Jenks Consultants
3336 Bradshaw Road, Suite 140
Sacramento, CA 95827

Re: Test results for tests samples pulled from the Hinkle reservoir floating cover

Alex,

Attached to this correspondence, please find test results for the materials submitted to Burke by John Glitsch of Colorado Lining. I have also included an original specification for 45 mil 8 X 8 - 5 ply material for your examination. I think you'll find the tests results confirm just how well Hypalon performs as a long term water containment membrane.

If you have any questions please call 800-669-7010 ext. 447.

Regards,

Bradley Roades
Technical Engineer.

Cc: F. Cote
S. Roades

HINKLE RESERVOIR FLOATING COVER
HYPALON COVER MATERIAL TESTING

	A-1		B-1		C-1		D-1	
THICKNESS ASTM D751	0.047		0.047		0.045		0.045	
	0.049		0.044		0.047		0.049	
	0.048		0.043		0.050		0.050	
	0.047		0.045		0.047		0.048	
	0.052		0.049		0.046		0.046	
PLY ADH. ASTM D413 TY A	10		10		11		10	
	10		11		11		11	
	10		10		10		10	
	LBS-FAB.	%-FAB.	LBS-FAB.	%-FAB.	LBS-FAB.	%-FAB.	LBS-FAB.	%-FAB.
TENS./ELONG. ASTM D751 GRAB MACH.DIR.	136	17	139	23	165	17	176	17
	176	17	141	22	163	17	174	20
	LBS-RUB	%-RUB	LBS-RUB	%-RUB	LBS-RUB	%-RUB	LBS-RUB	%-RUB
	238	110	229	82	268	77	271	73
	271	100	229	63	218	80	268	67
	LBS-FAB.	%-FAB.	LBS-FAB.	%-FAB.	LBS-FAB.	%-FAB.	LBS-FAB.	%-FAB.
CROSS DIR.	169	28	125	23	152	28	183	28
	LBS-RUB.	%-RUB.	LBS-RUB.	%-RUB.	LBS-RUB.	%-RUB.	LBS-RUB.	%-RUB.
	200	87	167	72	220	90	251	80
SHEAR STRENGTH ASTM D751	173		141		190		192	
	194		144		188		198	
HYDROSTATIC BURST	160		175		185		185	
	170		175		195		195	
	165		180		185		190	
	165		175		180		195	
WATER ABSORPTION % ASTM D-471	6.2%		4.5%		2.6%		8.5%	

Anthony Ruiz

BURKE MERCER DATE:11/14/00