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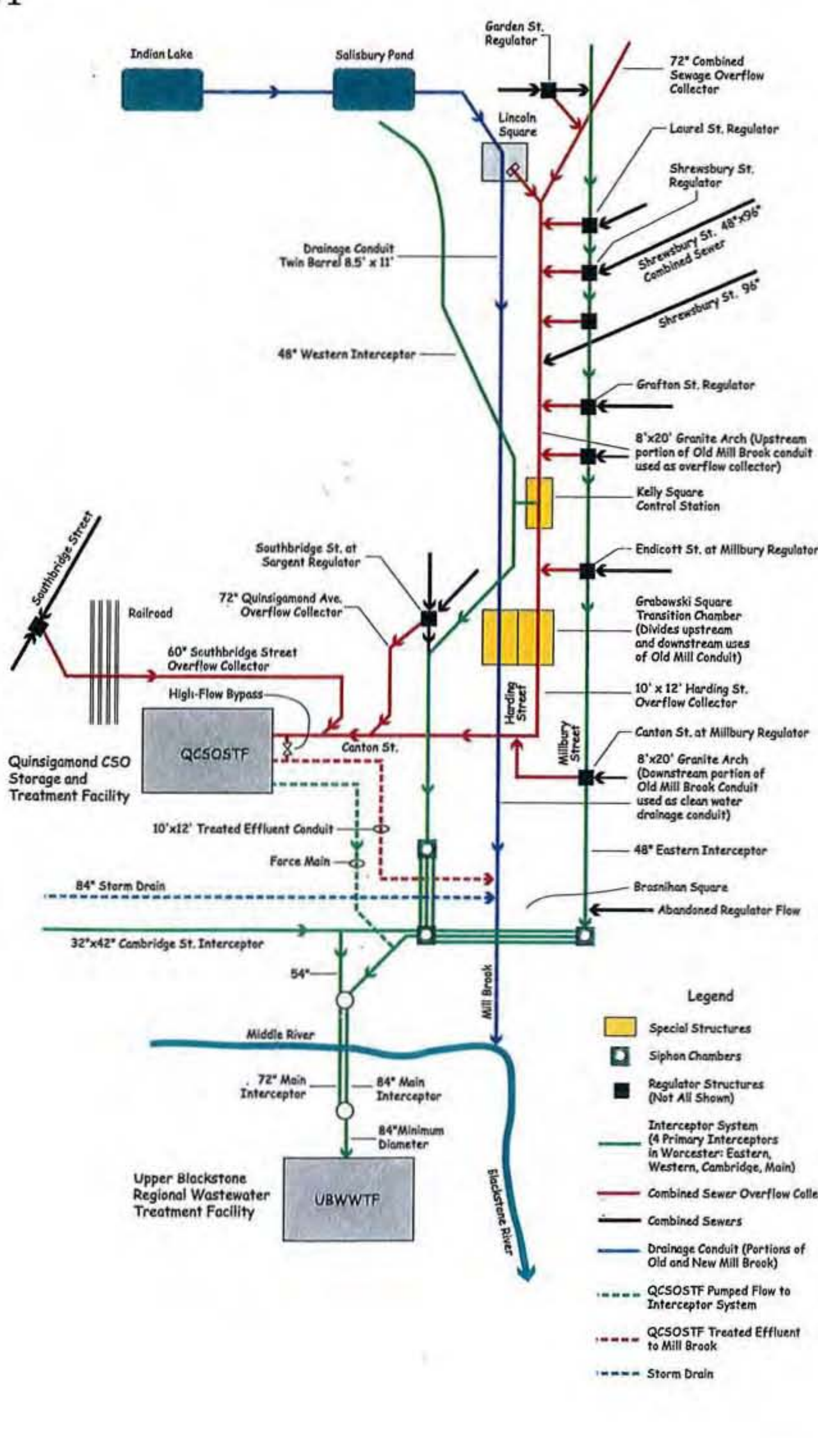
Worcester, Massachusetts
Department of Public Works

Phase II CSO Long-term Control Plan Report

February 2004



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Section 1
Project Background

1.1 Introduction

This report presents Phase II findings and recommendations for the City of Worcester's Long-term Control Plan (LTCP) for further mitigation of the water-quality impacts of its combined sewer system's combined sewer overflows (CSOs). It was prepared to comply with an administrative consent order signed by the EPA and the City on September 18, 2000. The consent order required the City to prepare a two-phased LTCP.

Phase I of the LTCP identified feasible CSO control alternatives. Phase II selects a control plan based on these alternatives.

This section of the report, Project Background, summarizes the results from Phase I to provide the framework for Phase II. This section presents information about:

- Worcester's combined sewer system (CSS), effectiveness of existing facilities, and the CSS's relative impact on the Blackstone River;
- Future planned improvements at the Upper Blackstone Wastewater Treatment Facility (UBWWTF), and how they will further mitigate CSS impacts on the Blackstone River;
- Evaluations of additional alternatives, beyond UBWWTF improvements, to further minimize CSS impacts;
- Financial impacts of potential CSS improvements; and
- An introduction into Phase II.

1.2 Worcester's Combined Sewer System

This is the second facilities plan prepared for Worcester's CSS. The first, in 1975, was fully implemented by 1989, at a cost in 2004 dollars of over $84 million. In addition to reducing the CSS area by 0.5 square mile, CSS facilities built as a result of that plan include four large overflow collectors, a dedicated conduit to carry upstream stormwater through the CSS, and the Quinsigamond CSO Storage and Treatment Facility (QCSOSTF). Figures 1-1 and 1-2 show, respectively, Worcester's four-square-mile CSS and a schematic of the CSS facilities.

These facilities have very effectively mitigated the impact of CSOs. In a typical five-year period, there are no dry weather overflows or untreated bypasses; also, 100 percent of the flow from the CSS is treated. Ninety-four percent of the CSS flow receives secondary treatment or better at the UBWWTF. The remaining flow is treated at the QCSOSTF, where it is screened, stored, disinfected, and dechlorinated. Few communities in Massachusetts or the nation have achieved or will ever achieve this

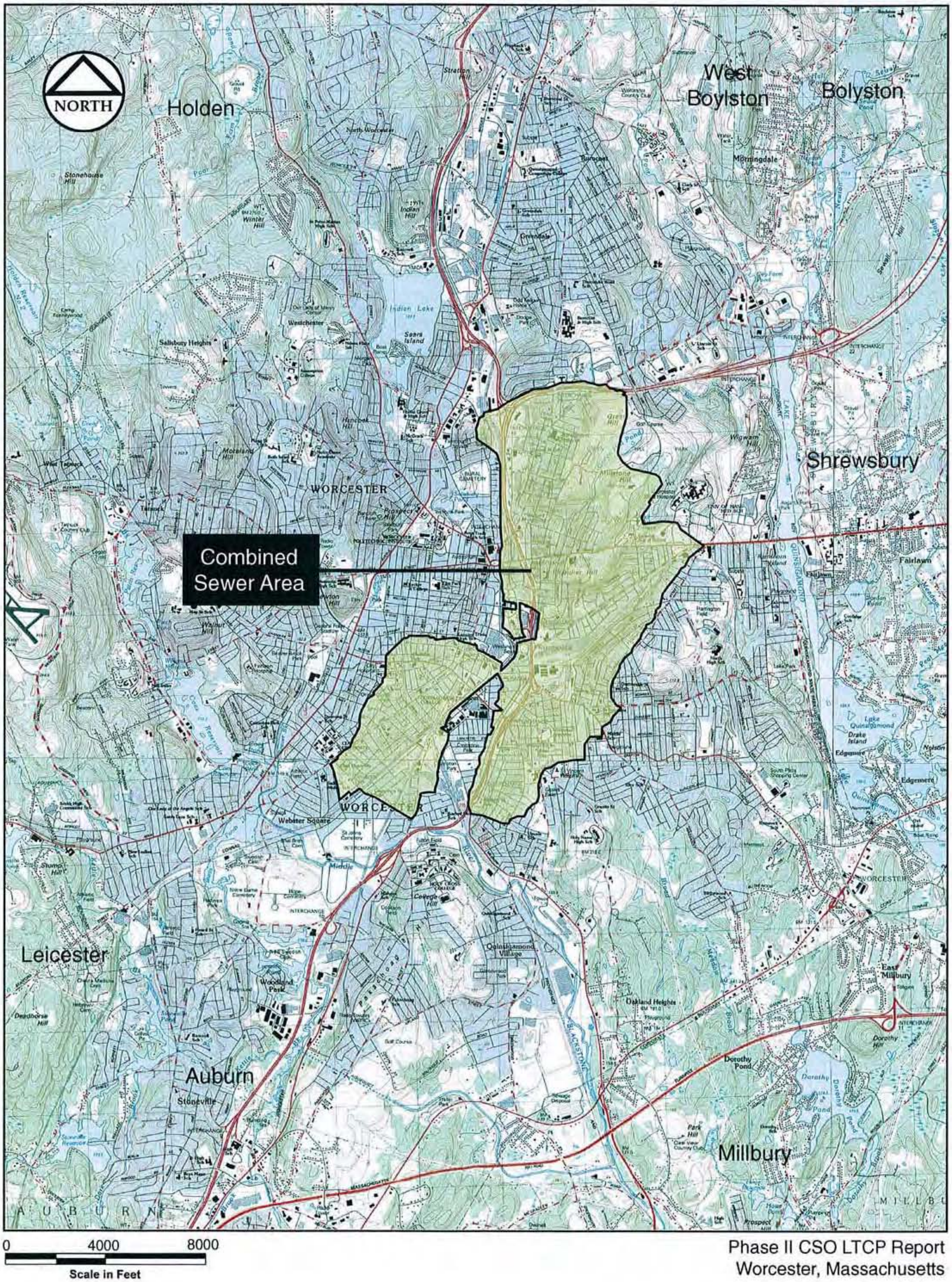

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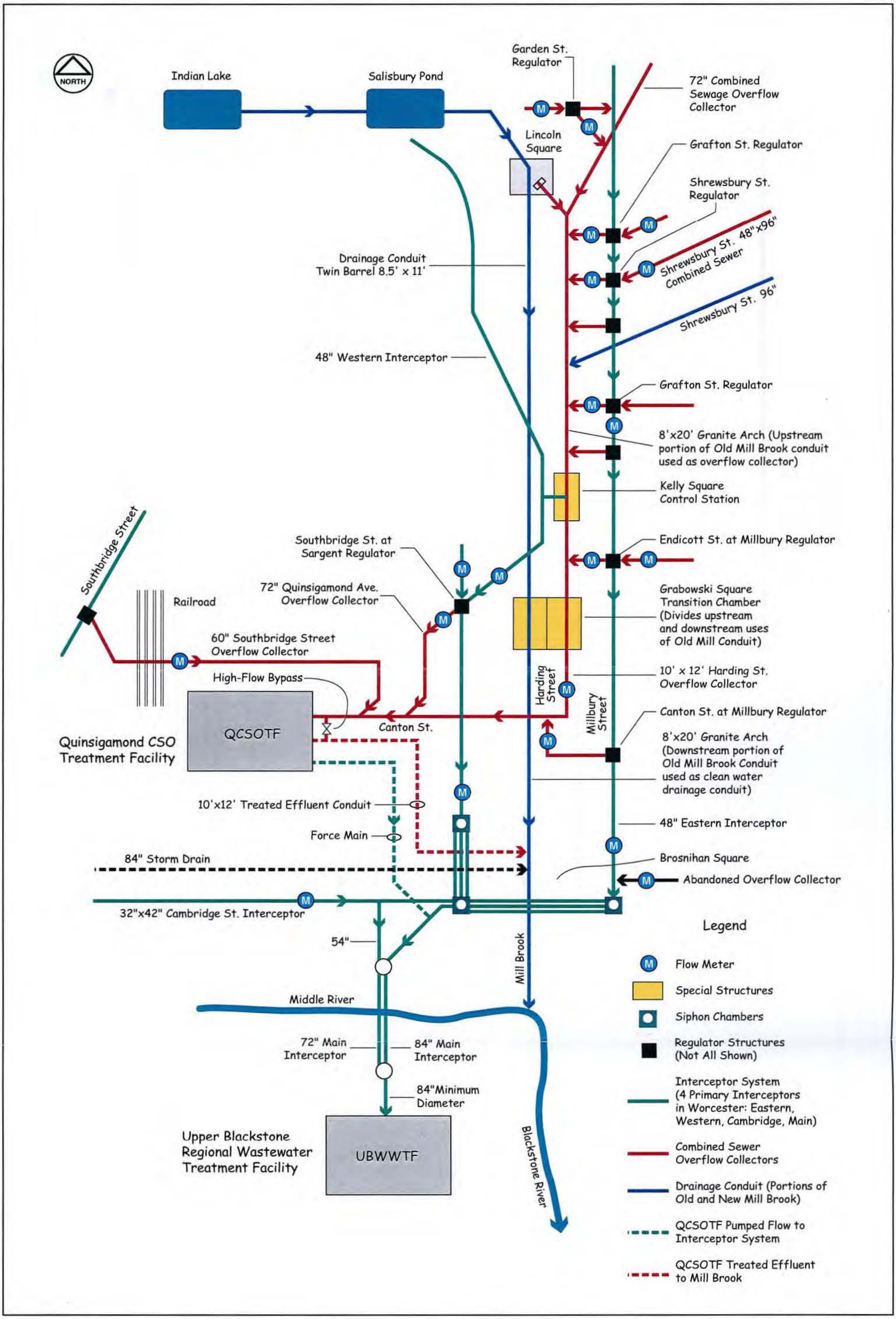
Figure 1-1
Worcester's Combined Sewer Area



Not to Scale

Figure 1-2
Schematic of the Combined Sewer System

level of performance from their CSS control facilities. Table 1-1 compares CSS performance before and after existing CSS facilities were built.

Table 1-1

Comparison of System Performance before and after Construction of Existing CSO Control Facilities

Parameter	Pre-1980	Post-1990
Number of *Untreated* CS Outfalls	>17	0
Number of *Treated* CS Outfalls	0	1
Dry Weather Overflows	Yes	No
Number of *Untreated* Overflow Events Annually	100 (every rainfall)	0
Number of *Treated* Overflow Events Annually	0	12 to 24
Estimated Annual *Untreated* Overflow Volume (Million Gallons)	1,308	0
Estimated Annual *Treated* Volume (Million Gallons)		
Secondary Treatment at UBWWTF (Million Gallons)	0	1,226
Treatment at QCSOSTF (Million Gallons)	0	82

The QCSOSTF acts first as a dry weather sewer pumping station, then as a wet weather combined sewer-pumping station, and then as a 2.5 million gallon wet weather combined sewer storage facility. The QCSOSTF acts as a dry weather pumping station to send any flow collected in the overflow collectors or captured in the storage tanks following a wet weather event to the UBWWTF for treatment. The QCSOSTF acts as a failsafe for preventing dry weather discharges and facilitates system troubleshooting. If higher flows than expected are observed at the QCSOSTF during dry weather, then crews are dispatched to check system regulators for possible malfunctions. Under the conditions described above, all flows reaching the QCSOSTF are pumped for treatment at the UBWWTF to the extent that the UBWWTF has available capacity to treat the flow. All dry weather flows are treated at the UBWWTF and over 75 percent of rainfall events are treated entirely at the UBWWTF. Treated discharges from the QCSOSTF occur only 12 to 24 times each year on average.

When the QCSOSTF does discharge, BOD and TSS are typically reduced by 28 percent and 34 percent, respectively. During high flow periods, UBWWTF primary facilities have BOD and TSS removal efficiencies of 24 percent and 32 percent, respectively. Thus, the QCSOSTF performance is comparable to primary treatment during high flow events. The QCSOSTF bypasses only in extreme conditions. Since going on-line in 1989, only two events have caused bypasses at the facility: Hurricane Bob in August 1991 and a 3.2-inch rainstorm the next month that caused widespread flooding. None of the large storms in October 1996, June 1998, or September 2001 caused a bypass.

Worcester's CSS area covers four square-miles. Effluent from the system is conveyed to the QCSOSTF for pumping to the UBWWTF and/or treatment prior to discharge to the Blackstone River. Treated discharges from the QCSOSTF to the river occur relatively infrequently. Pollutant loads from Worcester's combined sewer system to the Blackstone River therefore are relatively minor, given that the total drainage area to the river at the discharge point is 61.5 square miles.

1.3 Future Planned Improvements at the UBWWTF

The Upper Blackstone Water Pollution Abatement District (UBWPAD) recently completed facilities planning for improvements to the UBWWTF. The recommended improvements consist of a four-phased approach with a total program cost of $130,000,000. Phase I will increase the preliminary and primary treatment capacity of the existing WWTF from a peak of 119 mgd to a peak of 160 mgd, which will result in a significant improvement in CSO control for the City of Worcester. Phase I will also include disinfection and dechlorination facility improvements. Phase II will include upgrades to the existing secondary treatment processes, including addition of biological nutrient removal (BNR). Phase III will include sludge management system improvements. Phase IV will include additional secondary treatment improvements as appropriate following evaluation of facilities implemented in the prior three phases.

Increased WWTP capacity is the foundation of many communities' CSO Long-term Control Plans, and the cost of the increased capacity is typically included in CSO Long-term Control Plan implementation. Phase I improvements at the UBWWTF carry a cost of $44.3M and will be constructed between 2004 and 2006. Worcester's share of this cost as the largest contributor of flows in the UBWPAD service area is approximately $40M. The cost of high flow management facilities to be implemented under Phase I to handle higher peak wet weather flows is approximately $13M. Worcester's share of this cost is $11.7M. Adding the high flow management costs to the $84M already spent on combined sewer system facilities raises the total investment in CSO controls to date to approximately $96M, *prior* to implementation of Long-term Control Plan recommendations.

The planned upgrades will enable the UBWWTF to handle larger future flows at higher treatment levels, and will also accommodate high flows during storm events, thus improving CSS performance. The most significant improvement will be the operational protocol for pumping from the QCSOSTF to the UBWWTF. Currently, the QCSOSTF can pump to the UBWWTF only when there is excess capacity at the UBWWTF. When flow exceeds 54 to 70 mgd into the UBWWTF, pumping from QCSOSTF typically ceases to protect the secondary treatment processes, which are sensitive to wide variations in peak flow. With UBWWTF expansion, pumping will continue until flow at UBWWTF reaches 140 mgd. This is 20 mgd less than the planned new peak capacity of 160 mgd. The 140 mgd cutoff is designed as a safety measure to prevent influent flows from exceeding plant capacity. Flows from Worcester's combined sewer system represent only a small portion of the total influent flow at the UBWWTF. The majority of flow entering the UBWWTF comes from member communities' sanitary sewer systems. During wet weather, the sanitary systems contribute large quantities of infiltration and inflow (I/I). Therefore, pumping from the QCSOSTF needs to be regulated accordingly to reserve capacity for these flows.

Compared to existing conditions, these improvements will further mitigate impacts from the CSS. Instead of discharging treated effluent on the average between once and twice a month, the QCSOSTF will discharge only about once every two months. One-month storms, which currently cause discharges from the QCSOSTF to the Blackstone River, will no longer discharge from the QCSOSTF and will be treated at UBWWTF. Discharge volumes from the QCSOSTF during three-month storms will be halved. Total annual volume of treated discharge from the QCSOSTF will be reduced from 83 million gallons to 34 million gallons, more than a two-fold reduction. The combined sewage portion from the CSS treated at the UBWWTF in a typical year will increase from 94 percent to 97 percent.

UBWWTF improvements will further reduce already infrequent QCSOSTF bypasses. Because of the improvements, it will take approximately a 15-year event assuming all flows are captured in the combined sewer system to cause a bypass at the QCSOSTF. This is a very high level of performance, exceeding the performance of many separated sewer systems.

The UBWWTF improvements will dramatically reduce impacts from the CSS, which is already functioning at a very high level.

1.4 Additional Alternatives to Mitigate CSS Impacts

As required for this LTCP, a full range of CSO control alternatives were evaluated in Phase I. With UBWWTF improvements in place and 97 percent of the combined sewage already treated at the UBWWTF, only three percent of the flow can be mitigated further.

During Phase I, a screening analysis was conducted to screen the full range of improvements. Because CSO control facilities already exist, many potential improvements were already in place, including Nine Minimum Control (NMC) improvements. The NMCs are the minimum technology-based controls required by the Clean Water Act. Other potential improvements were categorized as Hydrologic Response Improvements, Storage Improvements, System Conveyance Improvements, and Treatment Improvements. Table 1-2 presents the screening analysis results from Phase I. This table shows technologies not feasible or appropriate, already in place, and those that were considered further in Phase I evaluations as potential LTCP technologies.

The remaining Phase I alternatives were then evaluated considering cost and pollution removal effectiveness of reduced discharge from the QCSOSTF, and reduced BOD and TSS loadings. These screened technologies are presented in Table 1-2. Table 1-3 shows each alternative evaluated and its Phase I estimated cost (some costs have been revised in Phase II), water-quality benefit from reduced overflow from the QCSOSTF, and if it was carried forward to Phase II as a short-listed alternative. These alternatives all are built on and include the UBWWTF improvements.

Table 1-2
Phase I Screening of CSO Abatement Technologies

CSO Control Technology	Technology Not Feasible or Appropriate	NMC/BMP Technology Continue Current Practice	Potential LTCP Technology
Nine Minimum Control/BMP Measures			
Solid Waste Management		X	
Street Sweeping		X	
Fertilizer/Pesticide Control		X	
Snow Removal and Deicing Practices		X	
Soil Erosion Control		X	
Commercial/Industrial Runoff Control		X	
Animal Waste Removal		X	
Catch Basin Cleaning		X	
Existing System Management		X	
Sewer Cleaning/Flushing		X	
Infiltration/Inflow Control		X	
Hydrologic Response Improvements			
Sewer Separation (full)			X
Sewer Separation (partial)/Flow Diversion			X
Green Hill Pond/Bell Pond Diversion			X
Disconnect 96-inch Shrewsbury St. drain from CSS			X
Downspout Disconnection	X		
Catch Basin Modifications	X		
Urban Parks and Green Spaces	X		
Infiltration Sumps	X		
Storage Improvements			
In-Line Storage			X
Kelly Square Control Station			X
Harding Street Overflow Collector Control Station			X
Real Time Controls			X
Regulator Modification			X
Off-Line Storage			X
Expanded storage at QCSOSTF			X
System Conveyance Improvements			
Increase pumping from QCSOSTF to UBWWTF			X
Flow diversion to interceptors with available capacity			X
Treatment Improvements			
Wastewater Treatment Plant Expansion			X
Screening		X	X
Sedimentation		X	X
Enhanced High-Rate Clarification			X
Swirl and Helix Concentrators	X		
Biological Treatment	X		
Filtration	X		
Disinfection		X	

Table 1-3
Summary of Potential CSO Control Improvements from Phase I

CSO Control Alternative	Estimated Cost	Reduction of Treated Flow at the QCSOSTF- 3 Month Storm, MG	$ per Gallon of Reduced Discharge - 3 Month Storm	Will be Considered in Phase II?	Why Alternative Will or Will Not be Considered in Phase II
Sewer Separation	$ 180,000,000	3.8	$ 47.37	No	Too costly, potentially degrades water quality
Shrewsbury Street - Partial Separation	$ 19,000,000	1.3	$ 14.62	No	Too costly, potentially degrades water quality
Green Hill Pond Diversion	$ 200,000	0.5	$ 0.40	Yes	Low cost, effective at reducing QCSOSTF discharges
Additional 1.25 MG storage tank at QCSOSTF	$ 5,000,000	0.8	$ 6.25	No	Other alternatives just as beneficial at lower cost
Two additional 1.25 MG storage tanks at QCSOSTF	$ 10,000,000	3.0	$ 5.00	No	Other alternatives just as beneficial at lower cost
Modify regulators by raising weirs 0.5 feet	$ 340,000	0.3	$ 1.13	Yes	Low cost, effective at reducing QCSOSTF discharges
Modify regulators by raising weirs 1.0 foot	$ 340,000	0.7	$ 0.49	No	High flood risk
Rehabilitate Kelly Square Control Structure	$ 200,000	0.5	$ 0.40	Yes	Low cost, effective at reducing QCSOSTF discharges
Divert flow to Western Interceptor at Kelly Square	$ 125,000	0.4	$ 0.31	Yes	Low cost, effective at reducing QCSOSTF discharges
Rehabilitate existing pumps at QCSOSTF	$ 500,000	0.6	$ 0.83	Yes	Comparatively low cost, effective at reducing QCSOSTF discharges
Replace existing pumps at QCSOSTF	$ 1,700,000	3.0	$ 0.57	Yes	Low cost, effective at reducing QCSOSTF discharges
Add new pumps at QCSOSTF	$ 1,900,000	3.4	$ 0.56	Yes	Relatively low cost, effective at reducing QCSOSTF discharges
High Rate Clarification at the QCSOSTF	$ 22,000,000	3.8	$ 5.79	No	Other alternatives just as beneficial at lower cost
Alternative 1	$ 865,000	1.4	$ 0.62	Yes	Relatively low cost, effective at reducing QCSOSTF discharges
Alternative 2	$ 2,800,000	3.8	$ 0.31	Yes	Moderate cost, effective at reducing QCSOSTF discharges

Notes:
Alternative 1 = UBWWTF Improvements + Green Hill Pond Diversion + Raise Weirs 0.5' + Activate Kelly Square Gate + Divert Flow to Western Interceptor
Alternative 2 = Alternative 1 + Add New Pumps at QCSOSTF

Findings from the Phase I evaluations include:

- Potential sewer separation improvements (complete sewer separation and separation of the Shrewsbury Street area) are not considered in Phase II because of their cost and their potential for degrading water quality; and

- High Rate Clarification (HRC) is not considered in Phase II because of its cost and because other potential improvements can as effectively or more effectively reduce CSS impacts.

Phase II considers all remaining potential improvements listed in Table 1-3. It is noted that additional storage tanks at the QCSOSTF were not recommended for consideration in Phase II because of their high cost and because other potential improvements can as effectively or more effectively reduce CSS impacts. Nonetheless, storage options are considered further in Phase II to satisfy the request of DEP and EPA that these options be evaluated further.

1.5 Phase II of the LTCP

Phase II investigates the promising alternatives from Phase I to ensure they function satisfactorily and without unintended consequences. The following alternatives, short listed based on the Phase I analysis, were evaluated in Phase II individually and together:

- The Green Hill Pond diversion;

- System regulator modifications;

- Rehabilitation of the Kelly Square control structure;

- Diversion of flows to the Western Interceptor;

- Additional pumping capacity at the QCSOSTF; and

- Additional storage at and near the QCSOSTF.

Section 2
Phase II Alternatives Analysis

2.1 Introduction

This section presents further evaluation of alternatives that were identified in the Phase I Report as feasible, cost effective, and beneficial. These alternatives include Green Hill Pond Diversion, Regulator Modifications, Kelly Square Structure Rehabilitation, Diversion to Western Interceptor at Kelly Square, and Pumping Alternatives at the QCSOSTF. Additionally, expanded storage alternatives were also evaluated further based on comment letters from DEP and EPA. These alternatives were further developed and evaluated individually, as well as in combination with one another, to minimize annual treated discharge events and treated discharge volumes from the QCSOSTF in a cost-effective manner. For each alternative, a description of the additional analysis and refined preliminary planning level cost estimate is provided followed by an evaluation of its CSO abatement and water quality benefits. The goal is to arrive at a recommended plan that includes the most cost-effective combination of alternatives.

Planned improvements at the UBWWTF are included in the evaluation of CSO control alternatives. These improvements are an outgrowth of a separate facilities planning process, have been approved and designed, and will be built. Therefore, all improvements are evaluated assuming the UBWWTF improvements are on-line.

This section is not intended to duplicate information presented in the Phase I report, but to supplement this information. For more background on each alternative, the reader is referred to the Phase I Report.

2.2 Baseline Conditions with UBWPAD Improvements

The UBWPAD facilities plan was completed in October 2001 to identify planned improvements at the UBWWTF in Millbury, Massachusetts. The facility accepts, or soon will accept, wastewater from Worcester, Auburn, Rutland, Holden, Millbury, West Boylston, and the Cherry Valley Sewer District in Leicester. Worcester presently accounts for 90 percent of the total average daily flow to the treatment facility. The UBWWTF is currently designed to handle an average daily flow of 56 mgd, maximum day flow of 83 mgd, and peak-hour flow of 119 mgd. Current average daily flow is about 37 mgd. Wastewater treatment facilities include preliminary treatment, primary treatment, and advanced treatment with seasonal nitrification and disinfection. The plant discharges treated effluent to the Blackstone River downstream of the Worcester QCSOSTF discharge.

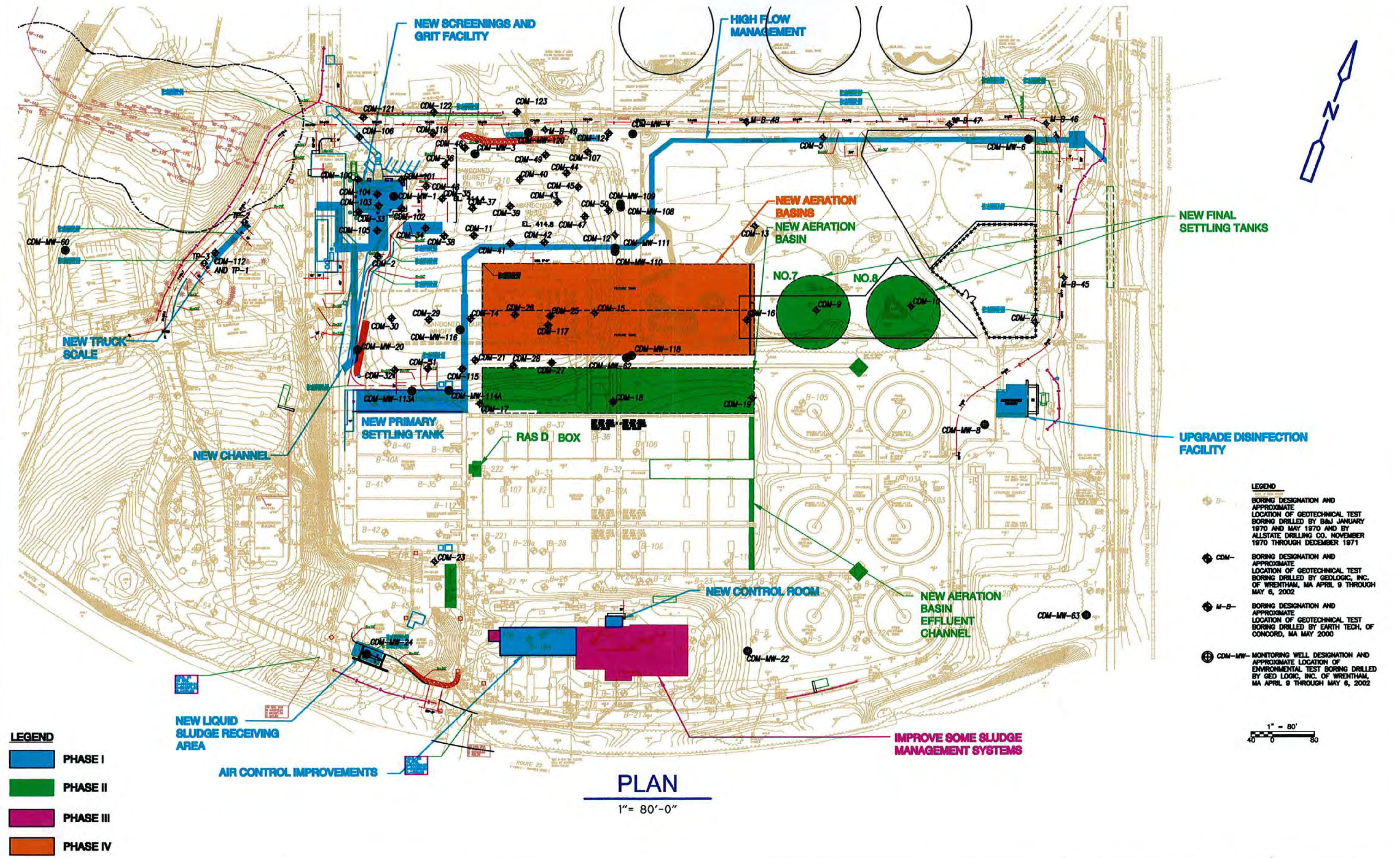
The UBWWTF facilities plan evaluated alternatives for increasing the peak preliminary and primary treatment capacity of the facility to 160 mgd, which matches the peak unsurcharged capacity of the influent interceptor (145 mgd) to the plant plus a peak from the contributing force mains (15 mgd) that discharge to the plant's influent box. The recommended high flow management plan included expanding the preliminary and primary treatment capacity from 119 mgd to 160 mgd, increasing advanced treatment capacity to between 80 and 120 mgd, and constructing a wet weather flow split intended to minimize upsets to the advanced treatment biological processes. The plan would maximize flow receiving advanced treatment and provide flows in excess of advanced treatment capacity with preliminary and primary treatment, disinfection and dechlorination prior to blending with the advanced treatment effluent. Therefore, larger volumes of wastewater will receive higher levels of treatment. Design of the Phase I Wastewater Treatment Facility Improvements including high flow management facilities was completed in July 2003. Construction of these facilities, which will increase the peak preliminary and primary treatment capacity of the UBWWTF to 160 mgd, is scheduled for completion by August 2006.

These improvements, combined with potential lower NPDES permit discharge limits, will significantly improve the water quality of the Blackstone River and will increase the UBWWTF's ability to accept flow from the QCSOSTF. Currently, when influent flows at the UBWWTF reach 54 to 70 mgd, the QCSOSTF stops pumping flow to the UBWWTF to avoid a hydraulic overload of the advanced treatment processes, which are sensitive to wide fluctuations in peak flow. The exact flow at which pumping is stopped varies depending on conditions in the secondary treatment train. By increasing the capacity to accept a peak hour flow of 160 mgd, the QCSOSTF will pump to a much higher cut off point.

For the purposes of the CSO facilities plan, it was assumed that the QCSOSTF will stop pumping when influent flows at the UBWWTF reach 140 mgd. This is based on a peak capacity at the UBWWTF of 160 mgd and a peak pumping capacity at the QCSOSTF of 19.9 mgd. If the influent flow at the UBWWTF reaches 140 mgd, it is considered good practice to discontinue pumping from the QCSOSTF to avoid exceeding the capacity of the UBWWTF. The cutoff point may vary depending on storm-specific conditions and UBWWTF performance.

Flows from Worcester's combined sewer system represent only a small portion of the total influent flow at the UBWWTF. The majority of flow entering the UBWWTF comes from member communities' sanitary sewer systems. During wet weather the sanitary systems contribute large quantities of infiltration and inflow (I/I). Therefore, pumping from the QCSOSTF needs to be regulated accordingly to reserve capacity for these flows.

The total program for the UBWWTF improvements, as shown in Figure 2-1, is estimated to cost $130 million through four phases of implementation as escalated through the mid-point of construction for each of the four phases. The mid-point of construction ranged from 2005 for Phase I to 2012 for Phase IV. Phase I of this program includes the high flow management improvements described above and is estimated to cost $44.3M. Of this Phase I total, approximately $13M is allocated specifically to the high flow management improvements. This includes the cost for a new east headworks, upgrades to the existing west headworks and new junction box,



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UBWWTF IMPROVEMENTS
RECOMMENDED PLAN

FIGURE 2-1

the cost of one new primary settling tank, and the cost of the advanced treatment bypass and associated sampling structure. The high flow management portion does not include upgrades to the existing primary settling tanks and disinfection facilities since these improvements would be required whether or not the peak flow capacity at the plant was increased. The UBWPAD improvements cost is allocated among the UBWPAD member communities. Worcester accounts for approximately 90 percent of the flow to the UBWWTF and so Worcester's share of the $13M high flow management facilities is approximately $11.7M.

Water Quality Benefits

Table 2-1 presents the benefits associated with increasing the UBWWTF capacity to 160 mgd with a high flow management flow plan, as compared to existing and future baseline conditions. The future baseline condition adds forecasted development and population growth to the existing service area, which increases flows and loads in the combined sewer system and at the UBWWTF. The table shows large improvements on both an event basis and an annual basis resulting from the UBWWTF wet weather capacity increase. The improvements entirely prevent any discharges at the QCSOSTF during a one-month storm, and significantly reduce discharges during larger design events. On an annual basis, discharges from the QCSOSTF decrease to about one treated discharge every two months. Treated annual volume discharged from the facility decreases from 83 million gallons to just 34 million gallons. Figure 2-2 shows the volume reduction at the QCSOSTF for the 1-, 3-, and 6-month design storms for this and other alternatives that are examined further later in this Section.

These translate into real water quality benefits. Under Baseline conditions, on average, about 6,700 pounds of BOD and 11,900 pounds of TSS per day would discharge to the Blackstone River from the UBWWTF and the QCSOSTF combined. With these improvements in place, this will be reduced to 6,300 pounds of BOD (4 percent reduction despite the large increase in flows) and 5,500 pounds of TSS (a 53 percent reduction).

These reductions are attributable to better removal efficiencies with the UBWWTF upgrades, and to the fact that a larger portion of flow is treated at UBWWTF instead of QCSOSTF because of the expanded preliminary and primary treatment facilities.

If Worcester's share of the Phase I UBWWTF high flow management improvements is considered ($11.7M), it will cost about $5 for every pound of TSS removed per year, and about $80 for every pound of BOD removed per year. It also represents a cost of about $3 per gallon of discharge reduced from the QCSOSTF during the three-month design storm.

Table 2-1

Comparing Existing, Baseline (2020) and UBWWTF Improvements

Parameter	*1-Month*	*3-Month*	*6-Month*
Treated Discharge at QCSOSTF, Million Gallons			
Existing Conditions	0.4	6.4	10.5
Baseline Conditions	1.2	7.6	12
With UBWWTF Improvements	0	3.8	7.2
Treated Discharge at UBWWTF, Million Gallons (2 Day Simulation)			
Existing Conditions	88.7	90.2	93.4
Baseline Conditions	119.0	120.2	119.7
With UBWWTF Improvements	123.3	124.4	126.1

Comparing Annual Performance: Existing, Baseline, and UBWWTF Improvements

Parameter	*Annual*
Number of Treated Discharges from QCSOSTF	
Existing Conditions	12-24
Baseline Conditions	14
With UBWWTF Improvements	7
Million Gallons/Year Treated Discharge from	
QCSOSTF	82
Existing Conditions	83
Baseline Conditions	34
With UBWWTF Improvements	

2.3 Phase II Alternatives

As discussed in Section 1, the following alternatives were short-listed in Phase I for further evaluation in Phase II.

- Divert flows from Green Hill Pond out of the CSS
- Optimize the CSS through regulator modifications (raise weirs to increase inline storage)
- Use existing Kelly Square gates to maximize storage in the overflow collector upstream of Kelly Square

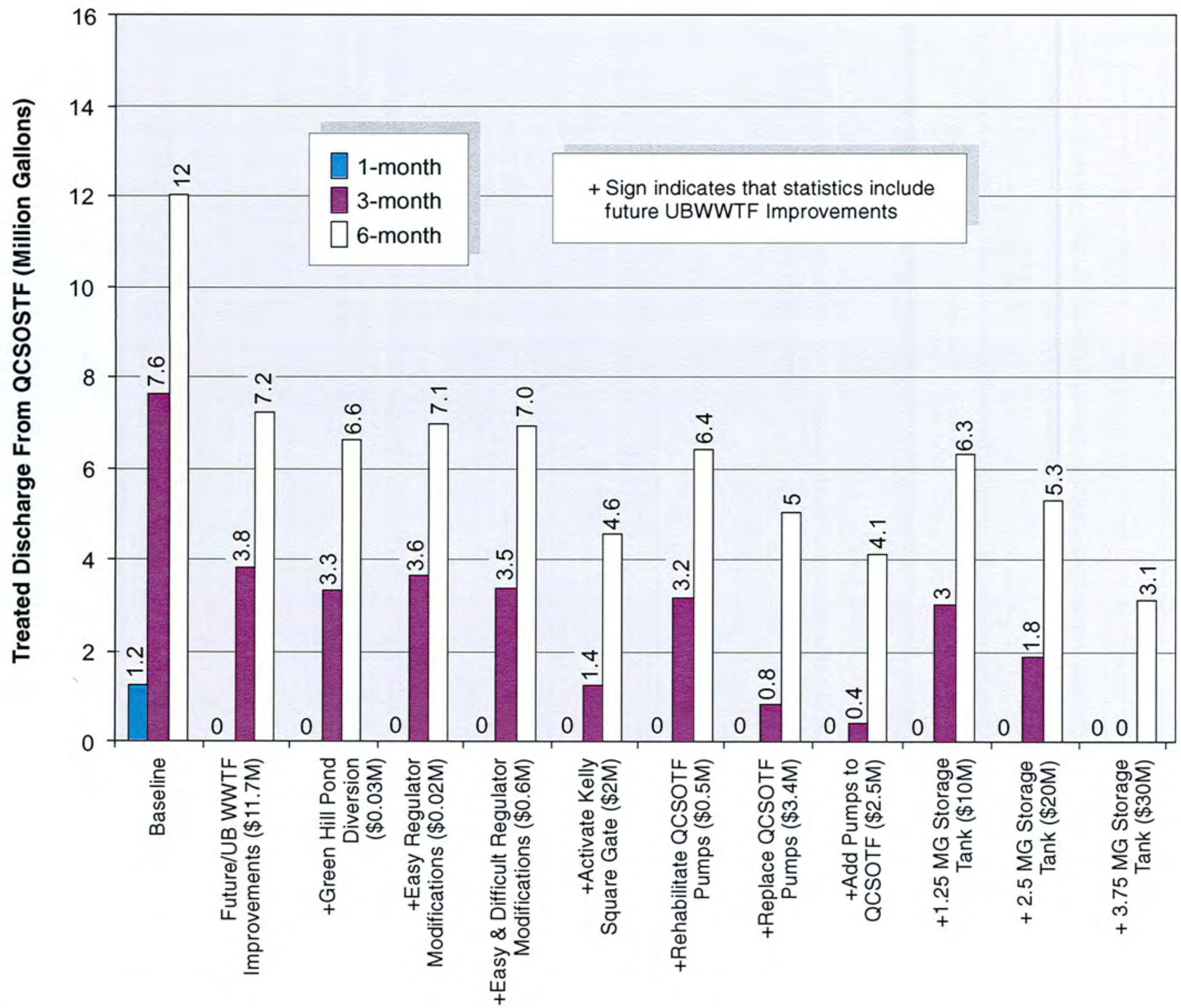

Figure 2-2
Treated Discharge Volume Reduction and Cost Impact of Alternatives

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- Divert additional flow from the Old Mill Brook overflow collector to the Western Interceptor at Kelly Square
- Increase pumping capacity at the QCSOSTF
- Increase the storage capacity at the QCSOSTF

These alternatives are evaluated further in the following sections in terms of feasibility and cost-benefit as stand-alone options and in combination with one another. They are also evaluated assuming the UBWWTF improvements described in Section 2.2 are in place.

2.3.1 Green Hill Pond Disconnection

This alternative involves diverting Green Hill Pond flows from the combined sewer system. As shown in Figure 2-3, Green Hill Pond is located in the northeastern portion of the combined sewer area. Surface water and stormwater runoff (but no combined sewage) from the pond and the area immediately surrounding it flows into the combined sewer system. Disconnecting the pond drainage system from the combined sewer system would remove approximately 266 acres from the CSS.

Green Hill Pond has two outlet structures. The southwestern outlet discharges into the combined sewer drainage area and the northeastern outlet discharges into the separated area and to Coal Mine Brook which drains to Lake Quinsigamond. This alternative would entail modifying the outlet works to both the separated and combined areas. The weir elevation at the outlet to the combined sewer system would need to be raised to prevent flow from discharging to the combined sewer system. Based on topography obtained for the pond and surrounding areas, the weir would need to be raised by approximately 0.2 feet to prevent flow from discharging to the combined sewer area in a 10-year storm. It is recommended that the weir be raised by approximately 0.5 to 1 feet to ensure that pond flows no longer enter the combined sewer system for a variety of storms. Modifications to the southwestern outlet structure would consist of building up the weir with additional mortared blocks.

The outlet works to Coal Mine Brook would also be modified so that the flow discharging to Coal Mine Brook would not exceed current levels. The hydrology of Coal Mine Brook is dominated by flows from sources other than Green Hill Pond. However, to avoid increasing the peak flow in Coal Mine Brook and, it was decided to store flows in excess of current peaks delivered to Coal Mine Brook in Green Hill Pond.

According to the hydraulic model of the system, shortening the outlet works weir from 12 feet (current length) to 6 feet would limit the flow accordingly. The weir may be shortened by closing off a portion of the opening above the weir, so that flow may only pass over 6 feet of weir length. The difference between the flow allowed to

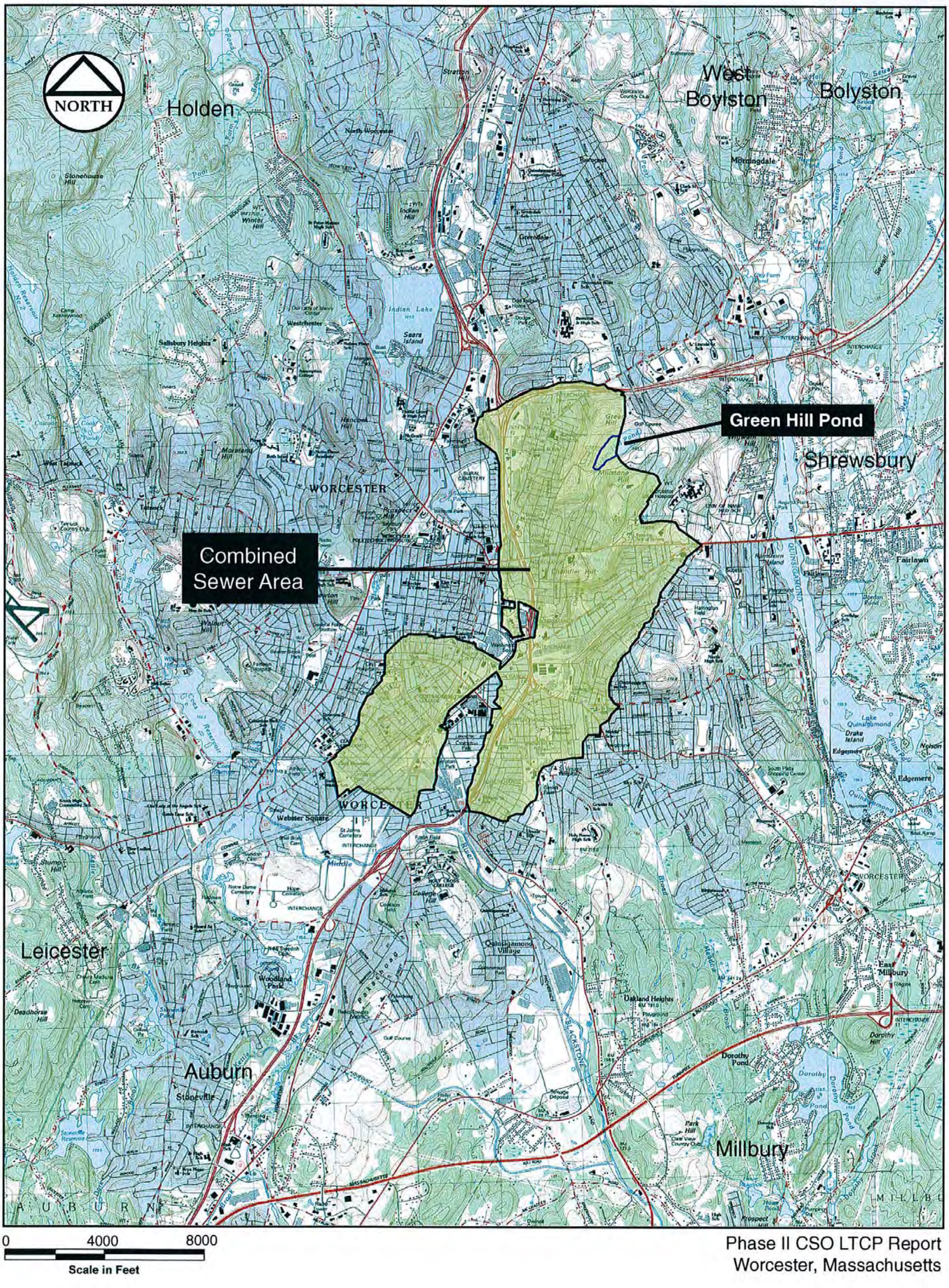

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Figure 2-3
Green Hill Pond Location

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discharge to Coal Mine Brook and the total inflow to the pond would be stored in the pond. Hydrologic and hydraulic modeling of the pond indicates that the water surface elevation would only rise approximately 0.2 feet in a 10-year storm, as controlled by the weir elevation settings.

Water Quality Benefits

Table 2-2 presents the benefits associated with separating Green Hill Pond from the CSS. The table shows that significant reductions in QCSOSTF treated discharge (0.5 MG for the 3-month storm) can be accomplished by diverting Green Hill Pond and its surrounding area from the CSS. The table also shows that this alternative reduces the amount of flow to the UBWWTF. Figure 2-2 presents the benefits associated with separating Green Hill Pond from the CSS.

During a 3-month storm, diverting 0.5 MG of flow from the QCSOSTF and UBWWTF will reduce BOD loadings to the Blackstone River by about 60 pounds and TSS loadings by about 360 pounds.

The cost associated with raising the southwestern outlet weir approximately 0.5 feet and shortening the northeastern outlet weir by 6 feet is minimal. At an estimated cost of $25,000 for the masonry work required, the Green Hill Pond diversion costs about $0.05/gallon of discharge reduced during 3-month storm conditions.

Table 2-2

Comparing Green Hill Pond Diversion to UBWWTF Improvements

Parameter	*1-Month*	*3-Month*	*6-Month*
Treated Discharge at QCSOSTF, Million Gallons			
With UBWWTF Improvements	0	3.8	7.2
With UBWWTF Improvements and Diverting Green Hill Pond	0	3.3	6.6
Treated Discharge at UBWWTF, Million Gallons (2-Day Simulation)			
With UBWWTF Improvements	123.3	124.4	126.1
With UBWWTF Improvements and Diverting Green Hill Pond	122.8	124.2	125.8

2.3.2 Regulator Modifications

Phase I evaluations concluded that raising weirs to force more flow through the interceptor system without spilling into the overflow collectors would be a cost-effective CSO control measure. In the Phase I evaluations, weir elevations were raised first ½ foot, then one foot at all regulators to determine their impact. The Phase II evaluations refined this analysis to consider more site-specific weir settings and conditions, as opposed to generalized raising and lowering of weirs. Regulator modifications were modeled throughout the system to determine these optimal weir elevations and configurations to maximize in-system storage, minimize treated discharges at the QCSOSTF, and manage upstream flooding risk in areas tributary to the regulators.

All 17 regulators in the combined sewer system were evaluated individually. Weirs were raised and lengthened as appropriate to maximize in-system storage and minimize flooding risk at each location. When possible, the weirs were raised until overflows were eliminated in the 3-month storm. The ten-year storm was then used to size the length of the weir. The weir lengths were modified to produce no change in the maximum hydraulic grade line (HGL) during the ten-year storm due to modifications at the regulator. The modeling results indicate that 8 out of 17 regulators may be modified to improve system performance without creating flooding problems. Table 2-3 presents modifications at these 8 regulators to provide a measurable benefit during the 3-month storm, without increasing flooding risk during a ten-year storm. These modifications would reduce overflows to the Harding Street Overflow Collector.

Table 2-3

Potential Regulator Modifications

Location	Weir Elevation	Weir Length	
		Existing	Proposed
Easy Modifications			
Grafton @ Franklin St. Regulator	Raised 1.1 ft	-	-
Endicott @ Millbury St. Regulator	Raised 2 ft	-	-
Pond @ Water St. Regulator	Raised 1 ft	-	-
Vernon @ Millbury St. Regulator	Raised 1 ft	-	-
Difficult Modifications			
Posner Sq. Regulator	Raised 1.6 ft	4 ft	10 ft
Richland @ Millbury St. Regulator	Raised 0.8 ft	2 ft	4 ft
Laurel St. Regulator	Raised 0.5 ft	3 ft	8 ft
Canton @ Millbury St. Regulator	Raise 1 ft	4 ft	14 ft

As shown in Table 2-3, 4 of the 8 regulators listed could be modified relatively easily by raising of weirs. Weirs would need to be lengthened as well as raised at 4 of the 8 regulators proposed for modification. The work required to lengthen the weirs would be much more extensive and disruptive, and would require new regulator structures to accommodate the longer weirs.

The easily modified regulators, requiring only increases in weir elevation, are expected to cost less than $5,000 each for the brick masonry required to raise the weir elevation. The more difficult regulators would require the construction of a special structure in congested, heavily traveled urban areas to both raise and lengthen the weir as needed. Each special structure is expected to cost approximately $150,000.

Water Quality Benefits

The benefits associated with modifying the regulators are presented in Table 2-4 and shown in Figure 2-2 for both easy only, and all regulators. The table shows decreases in treated discharge from the QCSOSTF. During the 3-month storm, easily implemented regulator modifications decrease the discharge from the QCSOSTF by 0.2 MG, while implementing both easy and more difficult regulator modifications decreases the discharge by 0.3 MG.

During 3-month storm conditions, the BOD reduction for implementing the easy regulator modifications is about 30 pounds and the TSS reduction is about 150 pounds. The reductions for implementing both easy and difficult regulator modifications are about 40 pounds BOD and 220 pounds TSS during the 3-month storm.

The cost of completing easy and all regulator modifications is approximately $20,000 and $600,000, respectively. This translates into approximately $0.10 per gallon of discharge reduced for easy only regulator modifications and $2 per gallon for all regulator modifications during 3-month storm conditions. Based on this analysis, the easy modifications provide significant benefit at low cost, and are recommended. However, the difficult modifications, where entire regulator structures would need to be replaced, provide little additional benefit at high cost, and are therefore not recommended.

Table 2-4

Comparing Regulator Modifications to UBWWTF Improvements

Parameter	*1-Month*	*3-Month*	*6-Month*
Treated Discharge at QCSOSTF, Million Gallons			
With UBWWTF Improvements	0	3.8	7.2
Plus Easy Regulator Modifications	0	3.6	7.1
Plus Easy and Difficult Regulator Modifications	0	3.5	7.0
Treated Discharge at UBWWTF, Million Gallons (2-Day Simulation)			
With UBWWTF Improvements	123.3	124.4	126.1
Plus Easy Regulator Modifications	123.3	124.6	126.3
Plus Easy and Difficult Regulator Modifications	123.3	124.8	126.8

2.3.3 Kelly Square Control Station Modifications

As described in the Phase I report, the Kelly Square Control Station was designed and constructed in the 1980s as a means to store flows in the Harding Street Overflow Collector/Old Mill Brook upstream of Kelly Square and to divert lower flows from the overflow collector to the Western Interceptor to take advantage of available capacity. Figure 2-4 illustrates how the station was intended to operate. The control station is equipped with an 18-foot wide by 5-foot high hinged leaf gate, three 12-inch

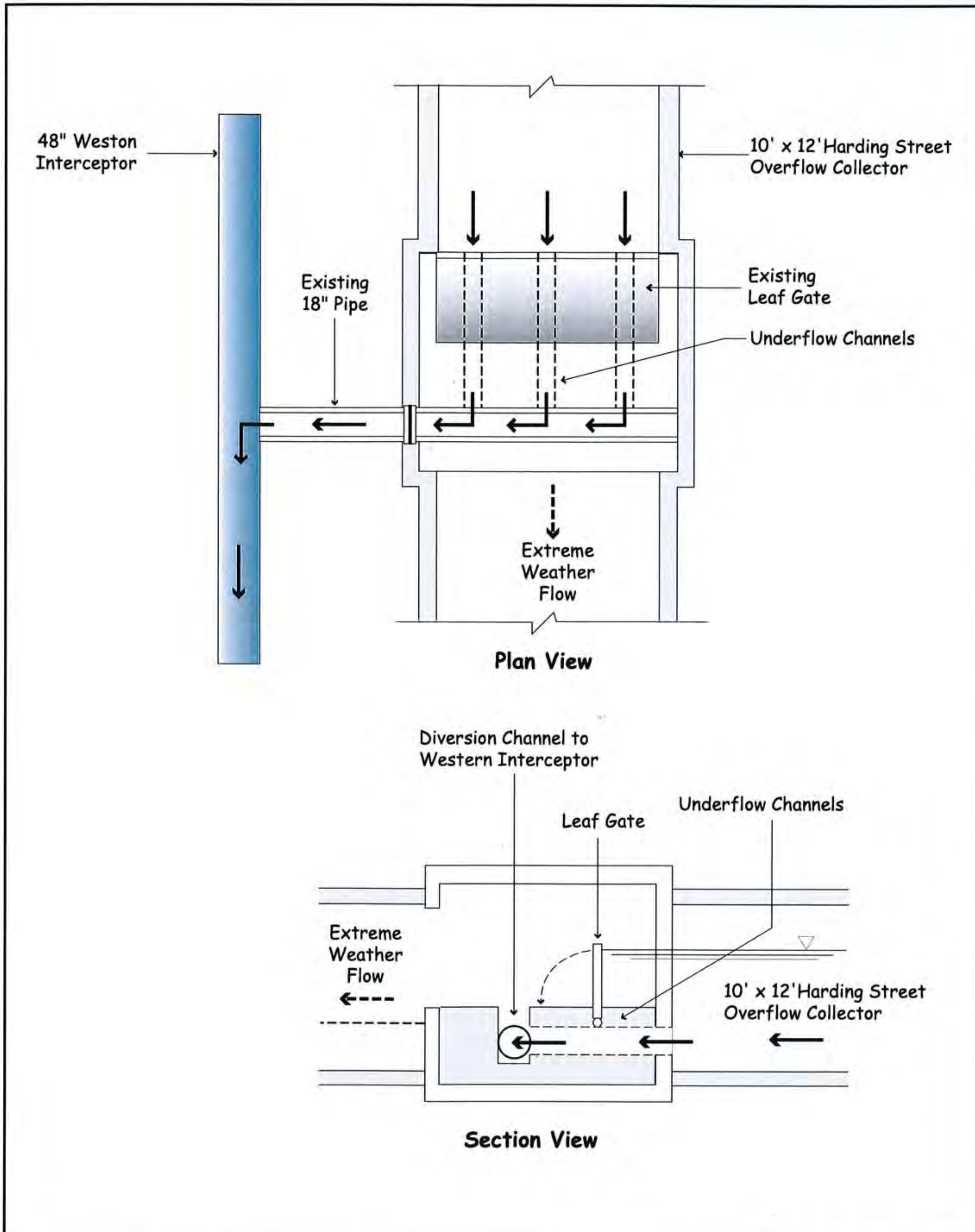

Figure 2-4
Kelly Square Control Structure

to 16-inch square underflow channels, and an 18-inch flow diversion connection between the overflow collector and the 48-inch Western Interceptor. As shown in Figure 2-4, flow passes through the underflow channels and drops into the diversion connection. Flows in excess of the 18-inch diversion or available capacity in the 48-inch Western Interceptor continue downstream through the overflow collector to the QCSOSTF. The Kelly Square Control Station effectively diverts flows from the surcharged 48-inch Eastern Interceptor to the 48-inch Western Interceptor. The Western Interceptor has capacity available in many storm events due to the separation projects completed in areas tributary to this interceptor.

The hinged leaf gate currently is not raised to hold flows back. Instead it lies flat over the top of the underflow channels. This is in large part due to the uncertainty associated with the hydraulic response of the system when the station was designed. The risk of flooding was a significant concern that outweighed the value of activating the gate. The Phase II evaluations further evaluated the impacts, benefits, and costs of activating the gate. Using the hydraulic model of the system, an operational protocol was developed to maximize storage effectiveness while making sure gate operations do not cause peak hydraulic grade lines to exceed current levels experienced during a 10-year storm without the gate activated. Therefore, the gate operation will not lead to elevated water surface elevations beyond those currently experienced for more extreme storms. In addition, field evaluations of the control station and Old Mill Brook conduit were conducted to assess feasibility, impacts, and upgrade requirements associated with the Kelly Square Control Station gate activation. The following describes the operating policy and summarizes the field investigations.

GATE OPERATING POLICY

Model simulations were performed for a variety of design storms to evaluate the feasibility and hydraulic impacts associated with activating the Kelly Square structure to provide storage during storm events. An appropriate operating policy was developed from this analysis based on remote control from the QCSOSTF. The gate would be operated automatically through controls at the QCSOSTF, but manual operation at Kelly Square would also be possible. The policy is presented in Table 2-5 and is based on wetwell levels at the QCSOSTF. A water level sensor would also be used to check depth of flow upstream of the gate. The operating rules presented reduce overflows by storing storm flows behind the Kelly Square gate without increasing the hydraulic grade line beyond what is currently experienced in a 10-year storm. A 10-year design storm will not surcharge the overflow conduit, and so the protocol maintains the HGL below the crown of the Old Mill Brook for all smaller design storms. A peak of approximately 3.9 MG of storage is available when the gate is fully raised. It is noted that flow may overtop the gate even when it is fully raised due to the opening between the top of the gate and the crown of the pipe.

Table 2-5

Kelly Square Operating Policy

Condition	QCSOSTF Wetwell Depth	Kelly Square Operation
Prior to storm		Gate Open
Wetwell depth increases to 6 ft	6.0	Gradually raise gate to 50 percent closed (2.25 ft)
Wetwell depth increases to 12 ft OR signal to stop pumping is received	12.0	Gradually raise gate to 100 percent closed (5.5 ft)
Wetwell depth recedes to 6 ft	6.0	Gradually lower gate to 50 percent closed (2.25 ft)
Wetwell depth recedes below 2 ft	2.0	Gradually lower gate to fully open
Severe Storm		
Flow above Kelly Square gate reaches top of pipe		Lower gate in 1-foot increments until surcharging condition subsides

This operating policy is designed to accommodate different storm sizes. For very small storms that do not increase the QCSOSTF influent wetwell depth above 6 ft, the gate will not be activated. In larger storms, the gate is activated partially (50 percent) and if necessary fully (100 percent) to minimize the volume of pumping necessary at the QCSOSTF. This prevents overflows during many medium-sized storms. The gate is raised fully when the signal is received from the UBWWTF to stop pumping because the facility's pumping capacity is significantly reduced due to this signal. In most cases, the gate reduces but does not eliminate overflows.

The operating policy developed also provides an optimized means of utilizing the storage available when it is needed most. The gate remains in the full open position (lying flat on the conduit) until flow levels rise to a certain point in the QCSOSTF influent wetwell. If the gate were activated prematurely, the available storage would become fully utilized before the QCSOSTF needs assistance with keeping up with the storm. In general, discharges may be minimized at the QCSOSTF by pumping as much flow to the UBWWTF as possible before resorting to storage of flows. This means of operation effectively shaves off the peak of the storm and is consistent with the current operating policy at the QCSOSTF, which does not allow flow to enter the storage/contact tanks until the sewage pumps cannot keep up with influent flows.

During severe storms, when the depth of flow above the gate at Kelly Square reaches the crown of the structure, the gate should be lowered by 1-foot to prevent surcharging. Lowering the gate by 1-foot should continue as necessary to prevent surcharging and flooding due to the raised gate. Lowering the gate to the 50 percent position creates a 3.25 foot opening above the gate, which can pass over 500 cfs (323 MGD) of flow. It is unlikely that many storms will produce such inflows to the overflow conduit upstream of the Kelly Square structure. Model simulations using a

25-year storm indicate that the gate opened to 50 percent was adequate to convey the flow without surcharging.

FIELD INVESTIGATIONS

Once a suitable operating policy was developed for the Kelly Square Control Structure, the next step was to investigate the impacts of consistently raising the HGL in the conduit and the upgrades that would be required in order to place the gate in operation. Field evaluations were conducted to address these questions. The investigations focused on the following three areas: 1) evaluating the structural condition of the Old Mill Brook Conduit; 2) evaluating potential impacts to existing service connections along the Old Mill Brook Conduit; and, 3) evaluating the condition of existing controls and other equipment to identify rehabilitation or replacement needs required to place the gate in service.

Old Mill Brook Structural Inspection

The structural inspection was conducted in November 2003 to determine whether the Old Mill Brook Conduit could handle the consistently higher hydraulic grade line and increases in internal hydraulic pressures resulting from raising the gate. The Old Mill Brook Conduit is generally an 8 foot high by 20 foot wide granite archway constructed over 100 years ago. Raising the five-foot high gate would increase the HGL in the conduit by five feet or more at the downstream end and would have an impact on the HGL extending approximately 5,000 feet upstream.

The structural inspection consisted of an above ground walk-over and interior inspections of specific sections of the conduit. The above ground walk-over involved visual observation of the surface roadway and sidewalks on Harding Street from the Kelly Square Control Station to Franklin Street for voids, sinkholes or depressions that could be indicative of settlement near the conduit. The sections of the conduit selected for interior inspection included the reach immediately upstream of the gate, which would see the greatest increase in hydraulic grade line, and sections of conduit rehabilitated in 1986 to assess the performance of the repairs. The internal inspection consisted of visual observation of the conduit and photography of selected areas. In total, approximately 900 feet of conduit was inspected in sections extending from the Kelly Square Control Station to the corner of Harding and Harrison Streets, and from the corner of Harding and Winter Streets to the corner of Harding and Franklin Streets.

The observations collected during the inspection indicate that the arch has maintained a consistent shape throughout with no visible deflection or major fractures in the stone. Substantial voids were identified in 4 locations, with smaller voids present throughout the inspected portions of the conduit. The larger voids were approximately 2 ft x 2ft x 1ft and should be repaired with stone and mortar or other appropriate means. The smaller voids do not require repair. Prior repairs of mortared joints observed at several locations appear to be performing adequately.

The bottom of the conduit could not be inspected due to the presence of sediment and water.

The structural inspection of the selected sections of conduit did not identify any significant structural deficiencies in the pipe that would preclude activation of the gate. It is noted that the total length of conduit impacted by the gate activation is approximately 5,000 feet. Conclusions are based on approximately 900 feet of this reach near the downstream end closest to the gate where the increase in hydraulic grade line would be greatest. It is possible that voids or other conditions could be found in sections of pipe not inspected that warrant repairs. Further inspections of other pipe sections impacted by the gate activation may be required during preliminary design to confirm that there are no significant structural deficiencies in these other reaches. It is expected that spot repairs required throughout the conduit may be completed for less than $50,000.

Service Connections

An evaluation of service connections to the Old Mill Brook was conducted concurrently with the internal structural inspection. Contract 12 drawings of the Old Mill Brook rehabilitation completed in 1986 show many 8-inch to 12-inch VCP service connections to the Old Mill Brook in the area that will be impacted by the increased HGL. There are no drawings available for these service connections to help determine whether they are still active or whether they are inactive lines that were installed at the time of the Old Mill Brook conduit construction. Therefore, inspectors conducted visual observations of service connections encountered in the reach of conduit inspected for structural condition. Based on these observations, it was determined that approximately 12 out of 40 service connection lines encountered in the 900 feet of pipe inspected were active, and showed signs of recent flow activity. The majority of the inactive service connections were silted up. These results suggest that one active service connection can be expected every 75 feet or so. Extrapolating these results to the 5,000 feet of conduit that would be affected by the raised HGL suggests that a total of about 65 active service connections might be expected, assuming the other reaches of pipe would have similar characteristics.

The majority of the service connections were found to be below what the new water surface elevation would be with the gate fully raised. Therefore, there may be some impact to these lines. However, since the HGL would not be increased above what is currently experienced in a 10-year storm, the City should identify whether sewer backup complaints in the vicinity of the Old Mill Brook have been documented in the past. If none have been reported, the potential impacts may not be as significant as inspection results suggest. Nonetheless, a more detailed study of individual service connections and flooding risk is needed to determine site-specific conditions for each service connection and the appropriate course of action. Connections may need to be re-routed to tie in to the Eastern or Western Interceptors, to avoid potential backups. Re-routing these service connections may be accomplished through direct connection to the interceptors where possible, through connecting to existing 8-inch sewers running in parallel to the Old Mill Brook in localized areas of the conduit, or through constructing new larger intercepting lines that would connect to the Eastern and Western Interceptor. Conservatively assuming new 24-inch service connection interceptors would be needed along both east and west sides of the Old Mill Brook conduit for approximately 5,000 feet, yields an estimated cost of $1.7M. This includes allowances for completion of additional service connection studies, reconnection costs, and appropriate contractor overhead and profit, engineering, and construction contingencies. The cost to re-route the service connections could be lower, but additional study is required to refine the requirements and associated costs.

There are also active regulator overflow connections to the Old Mill Brook in the reach that will be impacted by the higher HGL. These regulators are the Harrison Street, Pond Street, Grafton Street, Franklin Street, Shrewsbury Street, Thomas Street, Laurel Street, and Goldsberry Street regulators. The weir elevation at all of these regulators with the exception of those at Harrison Street and Pond Street is above the expected HGL in the Old Mill Brook for the 10-year storm. Therefore, there would be no increase in flooding risk at these regulators due to the activation of the Kelly Square gate. The Harrison Street and Pond Street regulators were looked at more closely using the hydraulic system model to determine the impacts of the raised gate here. Although the weir elevations are below the expected HGL in the conduit, the increase in HGL required upstream of the weir to drive the flow through the overflow connection does not pose a measurable increase in flooding risk during the 10-year storm. In both cases, over 10 feet of freeboard is available between the maximum HGL experienced in the upstream pipe connections to the regulator and the ground surface.

Lastly, there appeared to be a few 12-inch to 24-inch connections (likely stormwater) to the Old Mill Brook that may or may not be active. It was difficult to determine the status of these lines during the conduit inspection due to standing water in the Old Mill Brook. Separate storm drains were apparently disconnected from the Old Mill Brook and connected to the drainage conduit that runs through the combined sewer area. However, there may be a few connections remaining that should be investigated further to determine if these drains would be adversely impacted by the activation of the Kelly Square gate, and if so, how they would be disconnected.

Kelly Square Controls

Field inspections of the Kelly Square Control Station equipment and controls and the QCSOSTF controls related to the Kelly Square operation were conducted in May and November 2003. The purpose of these inspections was to determine if the gate could be activated and controlled remotely from the QCSOSTF using existing equipment or if equipment rehabilitation and/or replacement are required.

Mechanical devices, controls, electrical installation and remote SCADA control condition were evaluated. In general, the Kelly Square controls appeared to be in poor condition, but with appropriate rehabilitation and upgrades, they could be restored to operable condition. Further findings of the inspections are summarized below.

Description of Controls

There are three devices that have been installed at Kelly Square for combined sewer flow control: an 18-foot by 5-foot leaf gate, a tipping plate regulator, and a manually operated sluice gate. As described above, the leaf gate is designed to control flow during a storm event by manually raising and lowering the gate to increase and decrease storage. The gate is located in the Harding Street Overflow Collector, which is also the Old Mill Brook in this reach of pipe. A hydraulic cylinder and power pack moves the gate. The power pack is located in a control room below the street. The power pack includes an electrical enclosure, pressure switches and other electrical components. An instrumentation cabinet on the sidewalk above the structure houses the local control and telemetry equipment. The telemetry equipment was installed to allow remote monitoring and control from the QCSOSTF. Other than some debris on and around the gate, the gate appears to be in good condition.

The tipping plate regulator is located in a manhole between the Harding Street Overflow Collector and the 18-inch connection to the Western Interceptor. Its purpose is to regulate normal wastewater flow diversion from the overflow collector to the Western Interceptor, and to prevent excess flow during a storm event from surcharging the Western Interceptor. The regulator appears to be stuck in the open position. Raising the gate will not increase the amount of flow diverted to the Western Interceptor, since the gate is located upstream of the flow diversion channel and will not increase the head on the channel flows.

The manually operated sluice gate is located between the Harding Street Overflow Collector and the Western Interceptor flow connection. The gate may be closed manually from the street to prevent flow diversions from the overflow collector to the Western Interceptor. The gate box in the road is broken and the gate does not appear to have been operated recently. It is in the full open position. It should be rehabilitated as needed to allow closing off of diversion flows in the event the Western Interceptor cannot take any more flows, especially since the tipping plate regulator appears to be stuck in the open position.

The leaf gate hydraulic system uses a hydraulic operating system to raise (close) and lower (open) the gate. The system is located in an underground control room isolated from the Harding Street Overflow Collector. The shaft of the leaf gate penetrates the wall between the overflow collector and the control room and is connected to a lever operated by a hydraulic cylinder. The cylinder uses pressure greater than 1700 psi to raise the gate in one direction of travel in 2-1/2 minutes. The hydraulic power pack provides the oil and pressure required for operation. The power pack includes an electrical panel, electric motor, hand pump and other components required for control of the oil.

Access to the control room is provided through a standard manhole with rungs set in the concrete walls. The control room contains the hydraulic system along with monitoring equipment, electric power for the power pack, lighting, electrical receptacles, and a sump pump to remove water from the space. The sump pump is no longer functional and a few inches of water were present on the floor at the time of inspection. There is no heating system to keep the space dry.

Exposure over time to the damp and corrosive environment in the control room has rendered the hydraulic system inoperative. All of the electrical devices on the power pack are corroded and the electrical panel has water inside. Water also appears to be in the oil reservoir. The cylinder and power pack are rusted, and the power pack disconnect switch appears to be corroded although it may work if put in service. The junction boxes, conduit and receptacles are badly corroded from prolonged exposure to the damp environment.

The control room also contains monitoring equipment providing an operator with information required for operation of the gate. A pressure (level) transmitter provides remote indication of the water level behind the gate. A combination position transmitter and switch unit provides remote gate position indication and local full open and/or closed position.

The level transmitter is mounted on a pipe through the wall into the main gate channel. A butterfly valve is provided to close off the pipe if the transmitter starts to leak or is out for repair. The pipe and valve are heavily rusted and the transmitter housing is badly corroded. The position transmitter and switch unit is mounted at the end of the main gate shaft. The unit appears to be in good condition; however it was not tested to prove its functionality.

The transmitter and switch units are wired to the instrumentation cabinet on the sidewalk above and the power pack electrical enclosure. Existing tone telemetry equipment in the cabinet was designed to send a level and gate position signal to the QCSOSTF for remote monitoring and control.

Controls Recommendations

The hydraulic system at Kelly Square is in poor condition and needs considerable work to put the station back in service. The lack of operation and electric power combined with the damp conditions has adversely affected the electrical components and produced rusting and corrosion on the equipment.

The age of the instrumentation equipment, the availability of improved equipment and changes at the QCSOSTF leads to the recommendation that the instrumentation system be upgraded through a combination of rehabilitation of some existing equipment and complete replacement of other equipment. The newer instrumentation would allow for better and more reliable remote control of the Kelly Square facility according to the operating policy developed above.

Listed below are specific recommendations with some options to consider in placing the facility into service:

Crest / Leaf Gate - The crest /leaf gate needs to be lubricated and operated. Operation may require repair of the power pack first. (Alternatively, an initial operational check could be done by moving the gate with jacks.) Provided the gate is not found to be stuck in the open position and lubrication is provided, the gate should be suitable for service.

Tipping Plate Regulator - The tipping plate regulator also needs to be cleaned, lubricated and operated to be sure it is suitable for service. It is expected that the plate can be freed up to work as designed.

Manually Operated Sluice Gate - The curb box in the street used for manual operation of the sluice gate will need to be repaired and the gate stem checked for damage. The gate itself should then be inspected, cleaned, lubricated and adjusted to ensure it is suitable for service.

Note: The initial work on the gates, regulator and power pack (discussed below) should be done by a manufacturer's service person. Complete operation and maintenance manuals should be provided and City staff trained on the required operation and maintenance.

Leaf Gate Hydraulic System - The leaf gate hydraulic system will require significant cleaning and rebuilding in order to be put back in service. The hydraulic cylinder needs to be cleaned, primed and painted. This could be done in place. To ensure continued reliable service all hydraulic hoses should be replaced even though they do not appear to have deteriorated.

The entire power pack unit should be removed from the control room and sent back to the manufacturer for evaluation, rebuilding or replacement.

Control Room - The existing control room is a confined space that requires special measures for entry which complicates operations and maintenance at the station. Also, the damp environment in the existing control room is corrosive for the equipment. In order to simplify operations and maintenance at the station and increase the life of the equipment, it is recommended that a new building be constructed above ground to house new and/or rehabilitated station power pack, electrical, and instrumentation equipment. This building will also contain the equipment currently provided in the sidewalk enclosures, which have been subject to vandalism. The only equipment that will remain in the below ground control room is the hydraulic cylinder (new and/or rehabilitated) with position transmitter to track gate position and a new level transmitter to track water surface elevation upstream of the gate. The hydraulic lines and electrical conduit will be run between the building and the control room. The sump pump in the control room will also need to be replaced and a ball float added for a flood alarm to protect the equipment from water damage.

The new above-ground building would be approximately 10 feet by 20 feet. Finding an appropriate site for this building is a major consideration. One option is a private parking lot adjacent to the control structure. The building would provide an ideal space for personnel and most of the equipment, and would also provide room for a larger accumulator to allow for gradual manual lowering of the gate in the event of a power outage.

If an above-ground control building cannot be sited, a vault could be installed below the sidewalk next to the control room to house the power pack. The vault should be as small as possible to minimize the size of ventilation equipment required. The electrical and instrumentation enclosure would remain on the sidewalk. Minimizing the depth of the vault and providing a large hatch would further reduce or eliminate the confined space condition. This option would require further evaluation to see if there are any obstructions in the sidewalk area.

If neither of the two options discussed above can be sited and equipment will need to remain in the existing control room, then all wire, conduit, lights, switches and receptacles in the control room should be replaced in addition to replacing and/or rehabilitating the existing power pack, hydraulic cylinder and instrumentation for detecting level and gate position. The addition of heat, ventilation and dehumidification is also recommended to reduce the potential for equipment damage, although the volume of the control room and its location under the road would make it difficult to heat and ventilate properly.

Instrumentation and Control

The existing tone telemetry equipment, transmitters and associated instruments should be replaced due to the age of the equipment, the exposure to the corrosive environment, and the fact that the equipment was manufactured by a company, Bristol Babcock, that is no longer in business. Therefore, the existing equipment would no longer be supported and spare parts and service would not be available. It is recommended that a new instrument panel with programmable logic controller (PLC) and equipment needed to communicate with the QCSOSTF be installed. The PLC would replace the function of the existing tone telemetry and provide local intelligence for local automatic control.

The local automatic control would provide protection for upstream locations by monitoring the level and override the leaf gate position control if for any reason the position requested by the QCSOSTF is causing a backup of water above a set maximum limit. The protection will be provided even if the communication with the QCSOSTF is lost. The communication link between the control station and the QCSOSTF could be telephone or radio. A radio survey is recommended to determine the available frequencies and the functionality of a radio link.

The normal leaf gate control from the QCSOSTF would be a gate position setpoint telling the Kelly Square programmable controller where to move the gate. The controller would then move the gate to the requested position as determined by the position feedback, and in accordance with the operating policy described previously.

The local programmable controller would monitor upstream water level and gate position for local automatic control of the gate and communicate the level and position data to the QCSOSTF. Full open and full closed switches should also be provided to replace the existing switches. The full travel switches, low oil level (from hydraulic system) would be connected to the programmable controller for transmission to the QCSOSTF. A flood alarm float and unauthorized entry alarm switches should also be added to the facility for transmission to the QCSOSTF.

The QCSOSTF will also need new equipment to communicate with the equipment at Kelly Square. The control logic will need to be added in the facility control system as required to provide the needed control and monitoring functions. Display screens would also need to be developed to provide operator interface to the Kelly Square functions.

Control redundancies and manual operation options will need to be provided so that the gate may be lowered gradually in the event of a power outage. A larger accumulator should be included so that the gate may be lowered gradually in manual mode using the hydraulic pressure from the stored flows balanced by the hydraulic cylinder pressure. The gate and hydraulic equipment will need to be maintained on a regular basis to prevent mechanical interferences with gate operation.

Lastly, both sidewalk enclosures have been damaged by vandalism. The enclosures will need to be repaired or most likely replaced if they are needed in the final station upgrade.

Cost

An opinion of probable cost for the Kelly Square controls improvements was developed based on the above recommendations. The cost for these improvements is estimated to be approximately $250,000, which includes the cost of replacement and/or rehabilitation of required equipment, construction of a new above-ground building adjacent to the Kelly Square Control Station to house most equipment, and controls upgrades described above. This cost also includes appropriate allowances for contractor overhead and profit, engineering, and construction contingencies.

Water Quality Benefits

The benefits associated with activating the Kelly Square Gate are presented in Table 2-6 and shown in Figure 2-2. The table shows significant decreases in treated discharge from the QCSOSTF. During the 3-month storm, discharge from the QCSOSTF is reduced by 2.4 MG. This represents about a 300 pound reduction in BOD and a 1,800 pound reduction in TSS to the Blackstone River.

The total cost of the Kelly Square Control Station activation including estimates for structural spot repairs to the Old Mill Brook Conduit, disconnection of active service connections, and controls and equipment upgrades is approximately $2M. This equates to a cost-benefit of about $0.85 per gallon of discharge reduced during 3-month storm conditions. It is noted that the majority of this cost ($1.7M) is attributed to a conservative estimate for disconnecting active service connections to the Old Mill Brook.

Table 2-6

Comparing Kelly Square Gate Alternatives to UBWWTF Improvements

Parameter	*1-Month*	*3-Month*	*6-Month*
Treated Discharge at QCSOSTF, Million Gallons	0	3.8	7.2
With UBWWTF Improvements	0	1.4	4.6
UBWWTF Improvements Plus Activating the Kelly Square Gate			
Treated Discharge at UBWWTF, Million Gallons (2-Day Simulation)	123.3	124.4	126.1
With UBWWTF Improvements	123.2	128.1	128.7
UBWWTF Improvements Plus Activating the Kelly Square Gate			

2.3.3.1 Diversion of Flow at Kelly Square to the Western Interceptor

Another option that was considered at the Kelly Square Control Station was increasing the size of the diversion channel and pipe diverting flow to the Western Interceptor from the Harding Street Overflow Collector. After further evaluation in Phase II, it was found that this option would not yield significant improvements. Rather than replacing approximately 500 feet of 18-inch pipe with larger pipe, it was determined that additional flow may be diverted to the Western Interceptor merely by increasing the head on the diversion channel flows. This would likely be less expensive than pipe replacement, and would also prevent excessive flow from entering the Western Interceptor during large storm events should the tipping plate regulator stick in the open position again.

The Kelly Square gate is upstream of the diversion channel and so this does not increase the head on the diverted flows. An inflatable dam or secondary gate would need to be installed downstream of the diversion channel. This would complicate the operation of the existing Kelly Square Control Station and also lead to increased operation and maintenance demands for relatively small return given the amount of available capacity in the interceptor for increased diversion flows. In light of these considerations, it was determined that the Kelly Square Control Station and flow diversion should be activated as originally designed before implementing additional enhancements that may complicate O&M to the point that the gate is not used regularly. Increasing the flow diversion to the Western Interceptor will not be considered further as part of this Phase II evaluation. However, once the station is activated and operated for some time, the City may choose to enhance the performance of the station with the addition of an inflatable dam or secondary gate to maximize the amount of flow diverted to the Western Interceptor at Kelly Square in the future.

2.3.4 QCSOSTF Pump Capacity Increase

The QCSOSTF currently pumps flows collected in the overflow collectors back into the interceptor system for gravity flow to the UBWWTF. If sufficient capacity is not available at the UBWWTF, the QCSOSTF large sewage pumps are deactivated and the water surface elevations at the QCSOSTF begin to rise. The pumps are deactivated to protect the advanced treatment processes and to reserve capacity for flows conveyed from other areas of the District. The water surface elevations also begin to rise if the pumps cannot keep up with flow entering the QCSOSTF via the overflow collectors. Once the wetwell elevations rise above a certain level, the QCSOSTF effluent gates open and treated flows are allowed to discharge to the Mill Brook. Therefore, if the capacity at the UBWWTF is increased, as planned, allowing more wet weather flows to be pumped to the UBWWTF and the pump capacity is increased to keep up with incoming flows, the frequency and volume of treated overflows at the QCSOSTF will decrease. The QCSOSTF pump modification options, costs and benefits are discussed further in this section.

Existing Conditions

As shown in Figure 2-5, the QCSOSTF is equipped with 2-6,000 gpm sewage pumps, 2-600 gpm drain pumps, and 1-600 gpm scum pump, for a total pumping capacity of 19.9 mgd. The drain pumps run as needed to pump flow collected in the overflow collectors during dry weather to the 48-inch Western Interceptor in Quinsigamond Avenue via an 8-inch diameter, 240-foot long force main. The scum pump and sewage pumps activate only during wet weather events. The scum pump sends flow to the Western Interceptor via a 6-inch diameter, 200-foot long force main. The sewage pumps send flow to the 72-inch Main Interceptor downstream of the siphons in Brosnihan Square via a 24-inch diameter, 1,200-foot long force main. The interceptors convey flow by gravity to the UBWWTF. As influent flows at the UBWWTF approach treatment capacity, the 6,000 gpm pumps at the QCSOSTF are deactivated

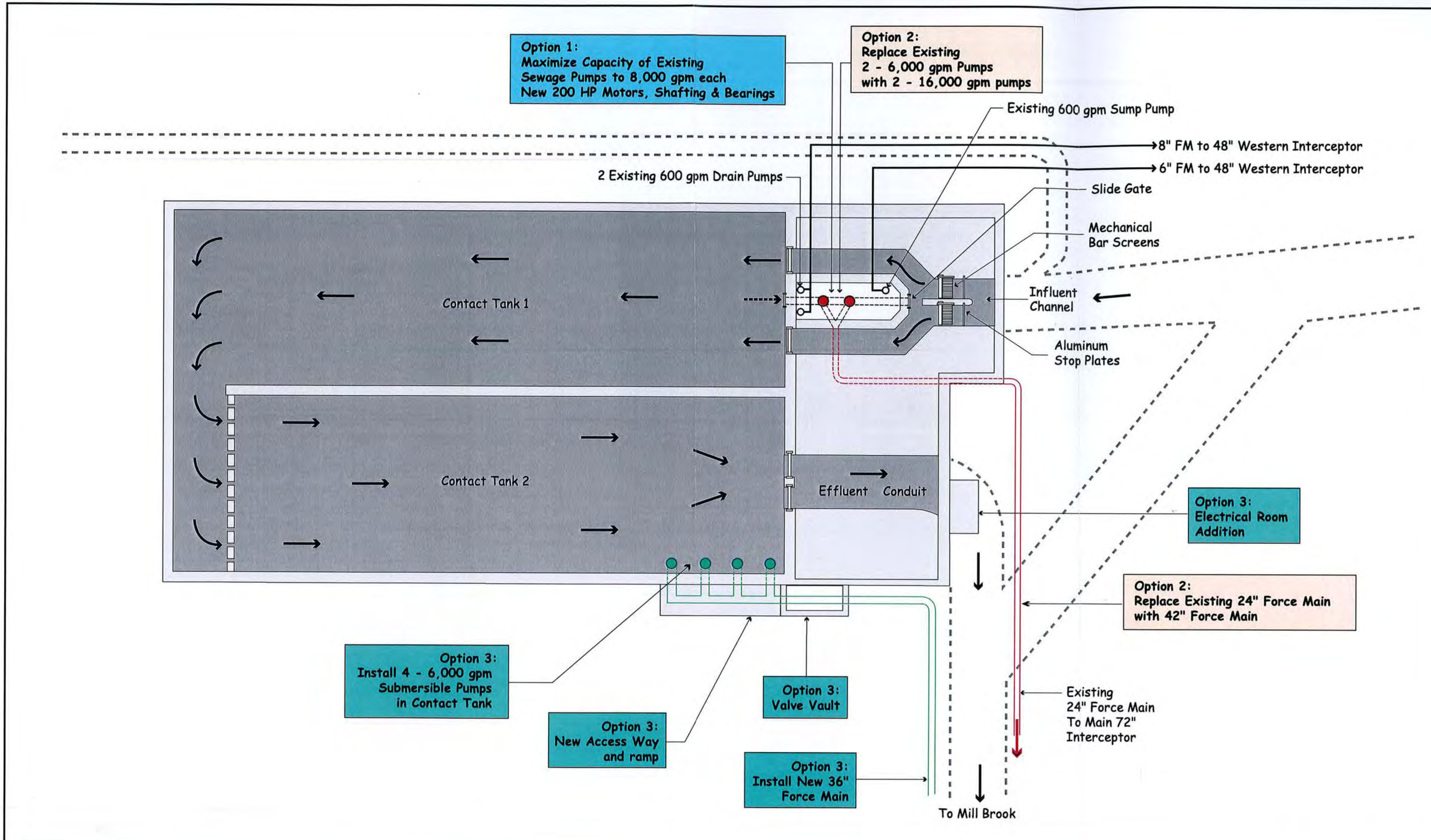

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Figure 2-5
Quinsigamond CSO Storage and Treatment Facility Pumping Alternatives

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to prevent washing out the treatment process. The deactivation currently happens when UBWWTF influent flows reach 54 mgd to 70 mgd. This shutdown point is expected to increase to 140 mgd with the planned expansion at the UBWWTF.

All five pumps draw flow from suction headers connected to a 4-foot wide by 8-foot long by 6.5-foot deep wetwell and the 2.5 million gallon chlorine contact tanks. The wetwell is located downstream of 1-inch bar screens. Given the relatively small size of the wetwell or sump, the influent channel to the QCSOSTF and overflow collectors generally act as the wetwell in storm events. There are currently two 6-inch diameter suction headers, from which the two drain pumps and one scum pump draw their flow, and one 30-inch suction header, from which the two sewage pumps draw their flow. During dry weather, the suction header valves to the contact tanks typically remain closed and the valves to the wetwell are open. The suction header valves to the contact tanks are opened if the contact tanks require dewatering.

Pump Specifications
All five pumps are Wemco pumps. The 600 gpm drain and scum pumps are equipped with 10 HP, fixed speed motors, operating at 1150 rpm. The total dynamic head (TDH) for the 600 gpm scum and drain pumps is 42 feet. The majority of the TDH appears to be associated with friction losses in the 8-inch and 6-inch force mains, since the static head is relatively low (<14 feet).

The lead 6,000 gpm sewage pump is equipped with a 2-speed, 100 HP motor. The lead pump low speed (585 rpm) comes on at a wetwell water surface elevation of 426.75. The higher speed (885 rpm) is activated once the wetwell reaches 427.0. The lag pump is equipped with a fixed speed (885 rpm) 100 HP motor, which is activated when the wetwell reaches an elevation of 427.5. The total dynamic head for the 6,000 gpm sewage pumps is 44 feet. The majority of the TDH appears to be associated with friction losses in the 1,200 foot long, 24-inch force main, since the static head is relatively small (<13 feet).

Pump Capacity Improvements

Essentially the goal is to increase the wet weather pumping capacity of the QCSOSTF to deliver more flow to the UBWWTF and reduce treated discharges at the QCSOSTF. There are three options that were considered for increasing the pumping capacity in the Phase I report. These options are shown in Figure 2-5. The first is to maximize the pumping capacity of the existing 2-6,000 gpm pumps through modifications to the motor and/or impeller. The second is to replace the existing 2-6,000 gpm pumps with higher capacity pumps. The third is to install submersible pumps in the existing chlorine contact tanks to supplement the existing pumps. These three options were evaluated further during Phase II to determine the most viable approach in terms of feasibility, cost, and effectiveness in reducing treated discharges at the QCSOSTF.

Option 1 – Optimize Existing Sewage Pump Capacity
According to the Phase I analysis, the first option considered increasing the existing sewage pump capacity by approximately 1/3 of their current capacity to 8,000 gpm each. The new capacity of the QCSOSTF would increase to a total of 25.6 MGD, assuming the 3-600 gpm pumps remain unchanged.

The pump capacity increase would be possible by increasing the speed of the pumps from 850 rpm to approximately 1100 rpm through replacing the 2-100 HP motors with 2-250 HP motors. Shaft and bearing frame replacement would also be required to handle the higher loads that come with the higher speeds. The risk of vibration issues increases at higher speeds. These concerns would need to be evaluated with the assistance of the pump manufacturer to ensure the recommended increase would be reliable. The impellers in the existing Wemco Hidrostal pumps are currently at their maximum size and, therefore, additional capacity may not be gained from replacing the impellers.

No new force main is required with this option.

This option is technically feasible with minimal modifications to the facility. The main issue is whether the CSO benefit gained by optimizing the existing pumps is worth the investment compared to other options. The modifications would be limited to replacement of the existing motors with higher speed motors, replacement of the existing shafts with larger shafts and pillow blocks to handle the higher speed/HP/torque, and potentially adding bracing to the structure to handle the higher weight load and the harmonic load. The weight of the shafting would be supported by the motor floor slab.

Available drawings of the QCSOSTF identify two support columns in the pump room at approximate elevations of 433.50 and 439.50. The need for additional support for the motor room floor or for the bearing supports would need to be determined through review of structural calculations prepared during design. A vibration analysis may also be conducted on the site to determine if the natural frequency of the floor and supports would require additional support to handle the higher rotational speeds and forces. The natural frequency of the structure could be tested through hiring a vibration test specialist. The test would consist of striking the floor with a hammer and recording the vibrations at sensors placed on the floor. The cost of such a resonance test would be approximately $2,000 per day. This testing could be delayed until design.

Concerns associated with this option include the "discharge head limit" identified on the existing pump curves, which could potentially create a problem with increasing the capacity of the existing pumps through increasing the speed of the pumps. Also, the existing Hidrostahl pumps are subject to wear with abrasives. The City may need to evaluate the pump condition to determine if the replacement of the entire pump is warranted with this option as opposed to just installing new motors, shafts and bearings. It is generally more cost-effective to replace an entire pump than individual parts of a pump. The age of the pumps is also of concern. If the pumps were installed in the 1980s, then they are approximately 20 years old. The hours of usage would need to be evaluated further to estimate remaining pump life. Also, the existing electrical components at the facility may need to be upgraded to handle the higher loads associated with the increase from 2-100 HP motors to 2-250 HP motors.

The concern with the unvented high point in the existing force main raised during Phase I evaluations was evaluated and determined not to be a problem. Based on Phase II analysis, there is little chance for entrapped air bubbles to form at the high point under Cambridge Street. The pumped flow velocities would overwhelm any air that gets trapped in the high point for either current flows (peak 12,000 gpm) or proposed flows (peak 16,000 gpm). There may be a slight reduction in capacity early on caused by potential trapped air in the high point, but this will be overcome as the pumps send more flow through the force main. Therefore, no air vent is required for the existing force main. Phase II evaluations also clarified the discharge characteristics in the force main. The pumps first discharge to the high point under Cambridge Street and then flow continues by gravity downstream to the 72-inch Main Interceptor. As the gravity pipe fills, a siphon forms. The siphoning effect will change the operating point of the pumps. This reduces the static head requirement for the pumps, and shifts the operating point.

Costs
According to the local Wemco pump representative, the cost associated with this option is estimated to be $30,000 per pump. This includes the cost to replace the motors, shafts and bearings. It may also be advisable to increase the size of the discharge piping between the pump and the 24-inch force main to maintain recommended velocities. The current configuration shows 16-inch discharge piping necking up to a 20-inch by 16-inch wye. The wye connects to 20 feet of 20-inch pipe before increasing to the 24-inch force main, which runs 1,200 feet. In general, vertical discharge piping is designed for velocities ranging between 6 and 10 ft/s. The velocities in the 16-inch discharge piping are approximately 13 ft/s with the increased flow. The 20-inch section would see a peak velocity of 16 ft/s with both pumps contributing flow.

Velocities in the existing 16-inch diameter suction piping are also high. Typically, suction piping is designed to maintain velocities at 3 to 5 ft/s. At current flow rates of 6,000 gpm, the velocity is already above the upper recommended threshold. A 33 percent increase in flow rate would result in a velocity of 13 ft/s. However, this is not a major concern considering the configuration of the suction piping. The suction piping is a direct uptake from the suction header into the pump, rather than a bell intake from a wetwell. The suction piping also matches the 16-inch pump size, which is a good rule of thumb to follow. Consequently, the high velocities in the suction piping are not considered a problem, especially in light of the fact that the pumping would not be continuous, but only during wet weather events as needed.

Furthermore, it would be more difficult and costly to replace the suction piping since it is built into the pump room floor, whereas the discharge piping and valves would be easier to replace. Velocities in the 30-inch diameter suction header are acceptable.

The velocities in the 24-inch force main would be 11 ft/s with the new flows, which is also on the high side. However, given the length of pipe that would need to be replaced and the fact that the pumping would be intermittent (the sewage pumps are limited to approximately 60 hours of usage annually), it is not recommended that the 24-inch force main be replaced for this option.

The total cost estimate associated with optimizing the existing sewage pump capacity is $500,000, including contractor overhead and profit, construction contingencies, and engineering and implementation allowances as indicated in previous sections.

Structural allowances were included in the cost estimate in case the motor room flooring requires reinforcement due to the increased loads from the heavier motors and higher pump speeds. Also, the larger shafts and bearings may require motor/pump room floor modifications. The cost of these modifications is expected to be relatively low, and would likely consist primarily of addition of bearing supports and steel beams beneath the motor room floor.

The cost also assumes that the power supply to the QCSOSTF is sufficient to handle the increased demands of the higher horsepower motors. It assumes that the existing MCC is adequate. Electrical allowances may vary significantly depending on specific requirements of the facility. These details would need to be evaluated in design if this option is shown to be cost-effective.

Water Quality Benefits
The benefits associated with increasing the sewage pumping capacity by 33 percent to a total facility pumping capacity of 25.6 mgd are presented in Table 2-7 and in Figure 2-2.

During the 3-month storm, discharge from the QCSOSTF is reduced by 0.6 MG. This will reduce BOD loadings to the Blackstone River by about 75 pounds and TSS loadings by about 440 pounds. At a cost of $500,000, this is about $0.83 per gallon of discharge reduced during 3-month storm conditions.

Table 2-7

Comparing QCSOSTF Pumping Alternatives to UBWWTF Improvements

Parameter	1-Month	3-Month	6-Month
Treated Discharge at QCSOSTF, Million Gallons			
With UBWWTF Improvements	0	3.8	7.2
UBWWTF Improvements Plus + Optimize Existing QCSOSTF Pumping Capacity	0	3.2	6.4
UBWWTF Improvements Plus + Replacing Existing QCSOSTF Pumps	0	0.8	5
UBWWTF Improvements Plus + Installing New Submersible Pumps	0	0.4	4.1
Treated Discharge at UBWWTF, Million Gallons (2-Day Simulation)			
With UBWWTF Improvements	123.3	124.4	126.1
UBWWTF Improvements Plus + Optimize Existing QCSOSTF Pumping Capacity	123.3	119.4	127.1
UBWWTF Improvements Plus + Replacing Existing QCSOSTF Pumps	123.6	121.6	130.3
UBWWTF Improvements Plus + Installing New Submersible Pumps	123.8	122.4	132.9

Option 2 – Replace Existing Pumps with Higher Capacity Pumps

This option would involve replacing the existing 6,000 gpm sewage pumps with larger capacity pumps, as shown in Figure 2-5. As described in the Phase I report, the new total capacity of the sewage pumps would be 32,000 gpm, leading to a total pumping capacity of 48.6 mgd if the drain and scum pumps remain at 600 gpm each. A new 1,200 lineal foot force main would need to be constructed to handle the increased pump capacity since the velocities in the existing 24-inch force main would be excessive (23 ft/s), and head losses would be unacceptable. A 42-inch force main ($215/LF) would bring velocities into the acceptable range (7.4 ft/s). The 30-inch suction header and 16-inch suction piping and discharge piping would also need to be upsized to accommodate the higher flows.

Based on further evaluation of this option in Phase II, a number of concerns exist that outweigh the benefit gained from replacing the existing pumps with higher capacity pumps, and reduce the feasibility of this option being successfully implemented.

The Phase I report suggests replacing the existing pumps with 2-16,000 gpm pumps in the 30 foot by 15 foot pump room. After further evaluation and site visits, it was determined that fitting the new pumps in the pump room would be difficult, and would likely require removal of the existing scum pump to create more room. The pump platforms would also need to be upsized.

Also, it would be difficult to increase the size of the suction header and suction piping since this piping is built into the floor of the pump room. Consequently, there would be significant construction and maintenance of plant operations (MOPO) considerations associated with this option. The pump station will need to be taken out of service during construction, which would increase the number of discharges that could occur. Also, the contact tanks would take longer to drain since the 600 gpm pumps would need to handle all of the flow. It appears the pump room could be isolated from the flow; however, this would impact the performance of the facility during storm events that occur during construction.

The resonance issue raised under pumping option 1 is not as significant with this option since the existing pumps operate at a comparable speed to the proposed pumps, but the other issues are more significant. Structural supports would most likely still be required to handle the heavier pumps, motors, and shafts.

The power supply to the QCSOSTF and the existing MCC may also be insufficient to handle the increased demands of the new pumps and motors. Electrical allowances would need to be included to upgrade equipment as needed.

Costs

The total estimated cost of this option developed in Phase I was $1.7 M, including allowances discussed previously. A new cost allowance was not developed during Phase II due to the feasibility issues raised above. If this option were considered further, it would likely cost significantly more than $1.7M. For the purposes of this analysis, it is assumed that the cost would be twice that presented in Phase I, or $3.4M.

Water Quality Benefits

Figure 2-2 and Table 2-7 show a decrease of 3.0 MG during 3-month storm conditions. This would reduce BOD loadings by 375 pounds and TSS loadings by 2,200 pounds. At a cost of $3.4M, this is about $0.88 per gallon of discharge reduced during 3-month storm conditions.

Option 3 – Install New Submersible Pumps

The third option is to install submersible pumps in the existing chlorine contact tanks, as shown in Figure 2-5. This option would involve the installation of separate submersible pumps with control system and electrical upgrades and a separate force main. The force main would discharge into the 72-inch Main Interceptor, near the existing 24-inch force main discharge point. This option would enable a significant increase in pumping capacity without modification to the existing pumps and piping. In light of the significant improvements gained and the fact that existing suction piping would not need to be replaced, this option is the most viable of the three pumping alternatives under consideration.

Phase II evaluations included developing an optimized maximum pumping capacity assuming pumps were shut down when flows reached 140 mgd at the upgraded UBWWTF; further developing the submersible pump layout at the QCSOSTF; further developing a force main route for the new force main; and evaluating electrical and structural considerations with adding new submersible pumps to contact tank No. 2 at the QCSOSTF. These evaluations are discussed further below.

Allowable Submersible Pump Capacity

The Phase I analysis assumed that the submersible pumps would be sized to deliver the same flow and TDH as the existing sewage pumps at the QCSOSTF (6,000 gpm at 44 feet TDH). Assuming four 6,000 gpm pumps could be added to Contact Tank No. 2, the total capacity increase would be 24,000 gpm. The new total pumping capacity of the QCSOSTF would be 54 mgd, if the existing sewage pumps remain at 6,000 gpm and the drain and scum pumps remain at 600 gpm each.

The Phase II analysis utilized the hydraulic model of the system to simulate varying pumping capacities under a variety of design storm flows to determine whether the pumping capacity could be increased further for improved benefit or if the pump shutoff point could be varied. Results indicated that 4-6,000 gpm pumps provide the optimum benefit for the 3-month, 6-month, and 1-year design storms. The capacity varies from storm event to storm event, but for the purposes of setting a design criterion applicable for a variety of storms, 4-6,000 gpm pumps provide the greatest benefit for the widest range of flows. It was also determined that the pumps should be shut down when influent flows at the UBWWTF reach 140 mgd. Both of these results are consistent with the assumptions developed in Phase I.

The addition of 4-6,000 gpm pumps to Contact Tank No. 2 would increase the total pumping capacity at the QCSOSTF from approximately 19.9 mgd to approximately 54.4 mgd, or a 34.6 mgd increase. This is the maximum allowable increase in pumping capacity; so other enhancements, such as optimizing the existing sewage pumps to deliver more flow, would not add any additional value.

Pump Layout Considerations

The addition of the 4-6,000 pumps to Contact Tank No. 2 is considered feasible with modifications to the existing facility. The proposed planning level configuration of pumps is presented in Figure 2-5. The location of the pumps was moved to the south side of the tank at the downstream end, as opposed to on either side of the effluent gates, as proposed in the Phase I report. After further evaluation of the structural and physical space constraints, this was considered a better location for the new submersible pumps. It avoids potential conflicts with effluent gates, minimizes the length of discharge piping needed, and provides the least complicated means for installing and accessing the pumps. Given the volume of the contact tank, maintaining a consistent flow path to each pump, a primary reason for positioning the pumps along the effluent gate wall, is not as significant a concern.

Pumps were selected on a preliminary basis assuming each pump would deliver 6,000 gpm at a total dynamic head (TDH) of 44 feet, which is comparable to the existing pumps. The Flyght 3356-810 pump with 100 horsepower motor is one pump that fits this criterion, and is available with special coatings for exposure to sodium hypochlorite. This pump is approximately 6 foot high by 3 feet wide by 7 feet deep, and weighs approximately 3,400 pounds. Discharge piping would add another 600 pounds to this weight. The total additional weight if four pumps are installed is about 16,000 pounds. Access to the contact tanks for installation and periodic maintenance of these large pumps is limited with the equipment that would likely be required. A new garage door, access ramp, and structural platform are proposed along the south wall of the QCSOSTF, at the downstream end of Contact Tank No. 2. This would provide adequate access to the tanks for installation and periodic maintenance purposes. The structural slab would also provide support to the existing tank cleaning water well in the vicinity of the pumps to support the raising and lowering of the pumps. The pumps would need to be mounted on a platform in the contact tank itself. A disadvantage of this configuration is that the traveling bridge and tank flushing equipment would need to be stopped approximately 25 feet short of the end of the contact tank to avoid interfering with the pumps. It may be necessary to extend or adjust the traveling bridge spray nozzles, or provide some other means of flushing for the downstream end of Contact Tank No. 2.

If the pumps were positioned along the same wall as the effluent gates, space would be a bigger concern, and there may be some structural concerns near the access hatch and grating for the platform drains. Discharge piping would also be more difficult to route in this area. If the pumps were installed along the center wall dividing the two contact tanks, sufficient space and structural support would be available. However, pump installation would likely be more difficult since the installation point is further away from the access point, and maneuvering the large pumps around effluent gate motors may be difficult. Also, discharge piping length would be increased. Another consideration is raising and lowering the pumps. If the pumps are installed near the new proposed access way along the south side of the building, it may be possible to raise and lower the pumps using a mobile truck mounted system, as opposed to constructing a monorail system with support piers in the tank itself.

Installation of pump equipment would need to be coordinated during periods of dry weather as much as possible, or a temporary dam would need to be constructed around the work space to prevent wet weather flows from interfering with construction activities.

Electrical Considerations

Other considerations include electrical and controls modifications required to operate the pumps. According to existing drawings of the facility, the existing electrical service extends from a utility transformer in a separate building through an underground ductbank to the basement and then up into the main switchboard in the electrical room on the first floor. The main switchboard and motor control center (MCC) are close coupled and installed along the north wall of the electrical room. There are no spare conduits between the main switchboard and the transformer. Existing service is 480/277 volt, 3 phase, 4 wire. The existing electrical room is approximately 19 feet by 12 feet, with limited floor space available for new equipment.

Proposed electrical improvements were developed to power 4-100 HP motors. These improvements were developed through review of available facility drawings. Preliminary design would allow for site visits to document existing conditions of electrical equipment and a more detailed load analysis. The electrical demand for four new 100 to 125 HP motors was added to an assumed peak demand for the base load of 950 amperes, which is the existing facility's main disconnect. The result was a new service requirement of 2,000 amperes at 480 volts. The existing switchboard and disconnect are not adequately sized to handle this load. A new service switchboard and a new 480 volt MCC would be required to house the motor starters and controls for the new submersible pumps.

The existing electrical room is not adequately sized to accommodate the new switchboard and MCC. Therefore, an addition to the existing electrical room would also be required. The new electrical room addition would be approximately 16 feet by 14 feet, and would be constructed adjacent to the existing electrical room with an interior connection, as shown in Figure 2-5. The new electrical room would house the 2000-ampere main switchboard and the MCC for the new pumps. A new utility service is required and new ductbank into the switchboard from a new pad mounted transformer. Once the new electrical service, switchboard and MCC are installed and operational, the existing switchboard in the old electrical room would be shut down and back-fed from the new switchboard via a new pullbox and feeder conduits. Staging the electrical installation in this manner would enable the facility to remain in operation with minimal shut down time. This shut down may be coordinated during dry weather. When the existing switchboard is connected to the new service, the existing utility service (transformer, cables, etc.) will be removed. New conduit and wiring will be required between the MCC and the pump motors located in the contact tank area.

A more detailed evaluation of electrical upgrade requirements would need to be performed during design to determine, among other items, whether a 2000 ampere service is necessary. Depending on the actual facility peak demand load (as determined by the utility) and the final size of the pump motors, it may be determined that the existing switchboard of 1600 amperes is adequate. In that case, the new switchboard may not be required; however, maintenance of plant operation (MOPO) issues may be more difficult, requiring longer power shutdowns or temporary power provisions as needed.

Valve Vault and Force Main Considerations

A valve vault would be constructed just outside of the QCSOSTF in a below ground structure, as shown in Figure 2-5. This valve vault would enable easy access to discharge piping valves to isolate pump flows as needed without entering the contact tank.

The discharge piping would combine into a header in the valve vault and feed a new 36-inch diameter force main. The velocities in the 36-inch diameter force main would remain within the acceptable range of 2 ft/s to 10 ft/s for a full range of flow, with a peak velocity of 7.6 ft/s with all four 6,000 gpm pumps running and 1.9 ft/s with one pump running. As shown in Figure 2-6, the route of the force main would extend south and east from the valve vault to the edge of the QCSOSTF property. Portions of the route on the QCSOSTF property would be through a parking lot currently used by J.J. Nissen to park their trucks. The force main would run in parallel to the facility effluent conduit as it approaches the property boundary. After leaving the QCSOSTF property, the force main would be constructed in an existing City of Worcester effluent conduit and utility easement extending from the QCSOSTF property south to Cambridge Street. After crossing Cambridge Street, the force main route would bend southeast to connect with a new discharge structure. The new discharge structure would be constructed adjacent to the existing discharge structure connecting the existing 24-inch diameter QCSOSTF sewage pump force main with the 72-inch diameter Main Interceptor.

Based on site visits and review of available drawings, there appears to be adequate space within the City of Worcester effluent conduit and utility easement to construct the 36-inch force main for the majority of the proposed pipe route. This route runs primarily through paved parking lots adjacent to the Quinsigamond Avenue on-ramp to Route 290. The most constrained area along the proposed route extends approximately 250 feet from the edge of the QCSOSTF property line to the southeastern edge of the Castle Metals industrial building. At its most constrained point at the southeast corner of the Castle Metals building, there is only 8 feet between the edge of building and the existing 24-inch force main. Portions of an existing 12-inch water line may need to be relocated and/or reinstalled above the new force main in this reach in order to accommodate the new force main. Alternative layouts may need to be considered in this area, including tunneling beneath the QCSOSTF effluent conduit to the east side of the conduit.

The City may want to consider constructing the new force main in combination with the new Cambridge Street on-ramp under consideration to minimize community impacts associated with both of these construction projects.

As discussed further in the next section evaluating storage options, it has been documented that hazardous waste is present on the QCSOSTF site. The source and type waste is described further below. It is not anticipated that hazardous waste will

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Figure 2-6
QCSOSTF Submersible Pump Option Proposed Force Main Route

impact the construction of the force main since the force main would likely be constructed with minimum cover (approximately 5 feet) above the depth of groundwater and potentially away from the contaminated soils. Also, the volume of soils excavated is not as large as with the storage options discussed below. Nonetheless, a hazardous waste allowance has been included in the cost estimate for this option to account for the possibility of encountering some hazardous waste during construction.

Costs
The estimated cost for this alternative is $2.5M, including appropriate allowances. This cost is slightly higher than that presented in the Phase I report, primarily due to the hazardous waste allowances added in for Massachusetts Contingency Plan (MCP) submittals and construction activities that may be required for working with hazardous waste that could be encountered on the QCSOSTF site during valve vault and force main construction. Also, part of the increased cost is attributed to the tight space constraints for force main construction in the vicinity of the Castle Metals building south of the QCSOSTF.

Water Quality Benefits
The water quality benefits associated with increasing the pumping capacity through the installation of submersible pumps are significant. This option provides the greatest increase in pumping capacity at the QCSOSTF with the fewest impacts to the QCSOSTF operation. Figure 2-2 and Table 2-7 show a decrease of 3.4 MG during 3-month storm conditions, almost eliminating the overflow. Expected reductions in BOD and TSS are 430 pounds and 2,500 pounds, respectively. At a cost of $2.5M this is about $0.75 per gallon of discharge reduced during 3-month storm conditions.

2.3.5 QCSOSTF Storage Capacity Expansion

The existing QCSOSTF features two chlorine contact basins that act as storage tanks as well as providing adequate contact time for flow through disinfection. The effluent gates to the contact tanks only open after the water surface elevation in the influent wetwell reaches 441. Once the water surface elevation drops to 439, the effluent gates close. The gates are rarely 100 percent open for extended periods of time, but rather fluctuate with the varying influent water surface elevation to the facility. This maximizes the existing storage capacity of the QCSOSTF facility. It is possible to increase the storage capacity by adding to the tank volume at the facility. Currently, each contact tank can store 1.25 million gallons. This equates to a total of 2.5 million gallons of storage for the entire facility.

The storage capacity at the QCSOSTF may be increased through adding additional tanks adjacent to the existing tanks. The Phase I report evaluated increasing the capacity by 1.25 MG and 2.5 MG. The Phase II analysis re-evaluated these options in light of the hazardous waste present on-site, as well as looking at utilizing additional space available on the QCSOSTF property to increase storage capacity still further. It was determined that the existing capacity may be increased by approximately 3.75

MG if all available and usable space was utilized on-site taking into account interferences from force main, utility and effluent conduit routes. As shown in Figure 2-7, flow would be diverted into the storage tanks through a new gate structure where each tank would fill up before the effluent gates to the contact tanks would open.

Costs

The approximate cost for the storage tank construction, engineering, and implementation was estimated in the Phase I report as $5M per 1.25 MG tank without any allowance for hazardous waste. The hazardous waste allowance developed in Phase II essentially doubles this cost. The hazardous waste costs were developed through review of a Preliminary Site Investigation Report (March 1997) and Activity and Use Limitation (AUL) documentation for the QCSOSTF site that documents the presence of contaminated soils on-site.

According to the available documentation, the QCSOSTF property was previously occupied by a gas plant, tar distilling companies and motor freight terminals and used for storage of gas plant and tar distilling materials and petroleum products. Subsurface investigations conducted during the Preliminary Site Investigation identified the presence of polyaromatic hydrocarbons (PAHs) and phenolic compounds on-site. PAHs and phenolics are indicator compounds of Manufactured Gas Plan (MGP) wastes. The Preliminary Site Investigation concluded that the source of contamination detected in the soil was most likely related to the release of coal tar during previous ownership and use of the property by gas manufacturers and tar distillers. A former above-ground storage tank was located along the west border of the property south of the QCSOSTF building where additional storage tanks would be constructed.

The hazardous waste allowance cost estimates associated with constructing additional storage tanks were developed assuming contaminants in soil and groundwater are consistent with typical MGP sites and consist of polycyclic aromatic hydrocarbons, petroleum hydrocarbons, benzene, cyanide, metals and phenols, and that all excavate is considered to be contaminated. Excavation dimensions for each 1.25 MG tank are approximately 200 feet long by 65 feet wide, by 30 feet deep, generating approximately 400,000 to 500,000 cubic feet of excavate. Most of this excavate was assumed to be suitable for disposal at an out-of state Subtitle D landfill, with some assumed to be suitable for disposal at a Subtitle C landfill. Groundwater removed as part of construction would require on-site treatment consisting of oil/water separation and granular activated carbon adsorption. Approximately 25 feet of excavation will be in groundwater.

Based on these assumptions, the total hazardous waste allowance carried per 1.25MG storage tank was estimated to be $5M. This cost includes Massachusetts Contingency Plan (MCP) submittals, and construction activities associated with excavation, handling and disposal of contaminated soils and dewatering and treatment of contaminated groundwater expected to be on-site. The cost also includes appropriate

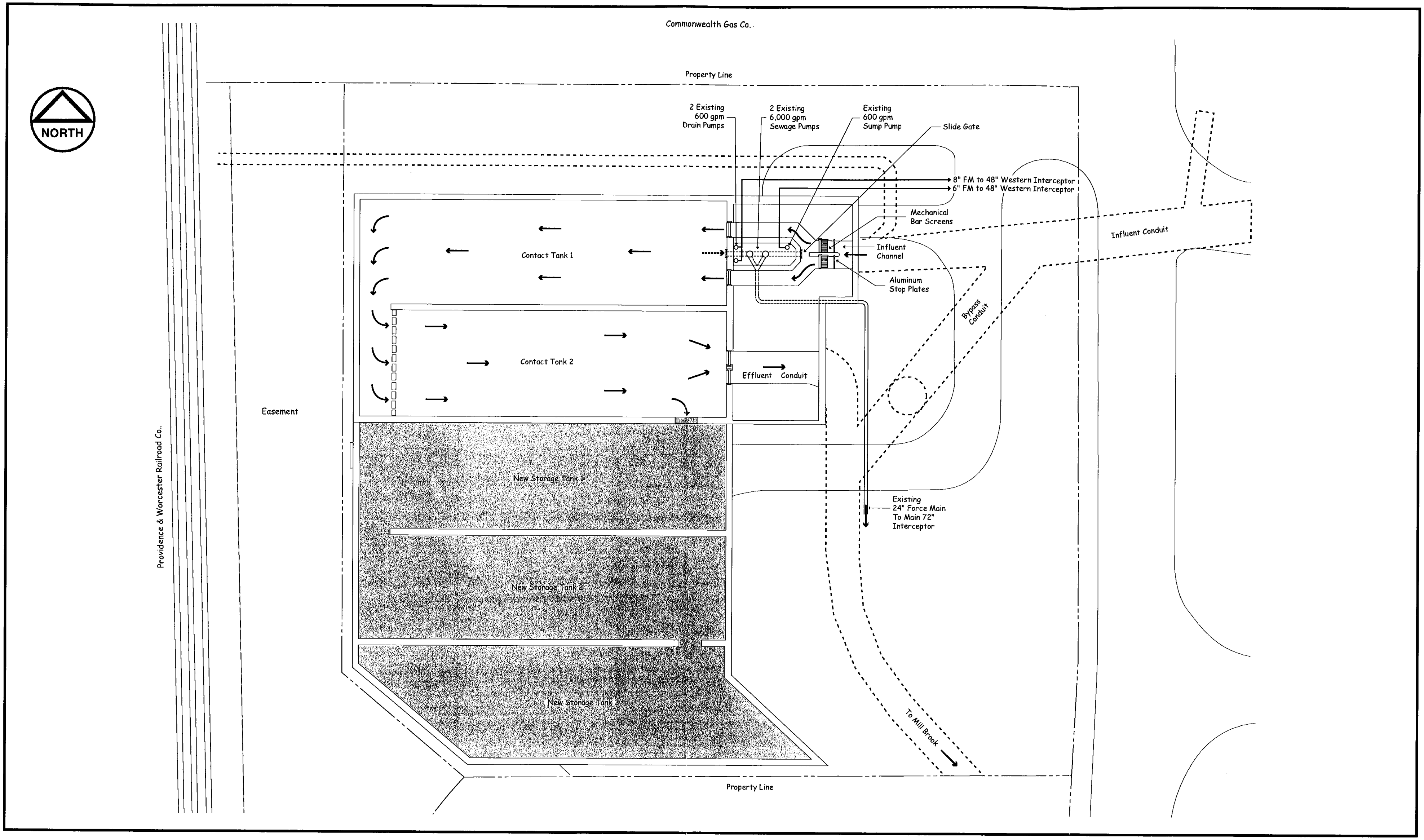

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Figure 2-7
QCSOSTF Storage Capacity Expansion

allowances for contractor overhead and profit, engineering and construction contingencies associated with the hazardous waste component of the storage option. The Phase II Comprehensive Site Assessment would need to be reviewed to refine the assumptions this estimate is based on.

In summary, the total cost for increasing the storage capacity at the QCSOSTF by 1.25MG is estimated at $10M. The total cost associated with a 2.5MG capacity increase is $20M, and the total cost for a 3.75MG tank is $30M. Roughly half of these total costs are associated with hazardous materials mitigation and disposal.

Water Quality Benefits

The benefits associated with increasing the storage capacity at the QCSOSTF are presented in Table 2-8 and shown in Figure 2-2. With one new tank, 3-month storm volumes from the QCSOSTF decrease by 0.8 MG. With two tanks, volumes decrease by 2.0 MG. With three tanks, volumes decrease by 3.8 MG. For one tank under 3-month storm conditions, diverting 0.8 MG from QCSOSTF to UBWWTF will reduce BOD and TSS loadings by 100 and 580 pounds, respectively. For two tanks, diverting 2 MG will reduce BOD and TSS loadings by 250 pounds and 1,450 pounds, respectively. For three tanks, diverting 3.8 MG will reduce BOD and TSS loadings by 480 pounds and 2,760 pounds, respectively. At $10M, one tank costs about $12.50 per gallon of discharge reduced during 3-month storm conditions. At $20M, two tanks cost about $10 per gallon reduced during the 3-month storm. At $30M, three tanks cost about $8 per gallon reduced during the 3-month storm.

Table 2-8

Comparing Increased Storage at QCSOSTF to UBWWTF Improvements

Parameter	*1-Month*	*3-Month*	*6-Month*
Treated Discharge at QCSOSTF, Million Gallons			
With UBWWTF Improvements	0	3.8	7.2
Plus 1.25 MG New Storage (one new tank) at QCSOSTF	0	3.0	6.3
Plus 2.50 MG New Storage (two new tanks) at QCSOSTF	0	1.8	5.3
Plus 3.75 MG New Storage (three new tanks) at QCSOSTF	0	0	3.1
Treated Discharge at UBWWTF, Million Gallons (2-Day Simulation)			
With UBWWTF Improvements	123.3	124.4	126.1
Plus 1.25 MG New Storage (one new tank) at QCSOSTF	123.3	124.4	127.7
Plus 2.50 MG New Storage (two new tanks) at QCSOSTF	123.3	124.5	128.9
Plus 3.75 MG New Storage (three new tanks) at QCSOSTF	123.3	125.1	130.0

2.4 Combining Alternatives

The alternatives discussed above were also evaluated in combination with one another to further enhance the performance of the system with UBWWTF improvements in place. The alternatives that are most cost-effective include Green Hill Pond diversion, regulator modifications at Grafton, Endicott, Pond, and Vernon Streets, Kelly Square Control Station activation, and submersible pumps at the QCSOSTF. Additional simulations were performed for design storms and average annual conditions to evaluate the effectiveness of combining the alternatives listed above. The results from these simulations are presented below as Alternative 1. They are compared to conditions assuming UBWWTF improvements are in place.

Increasing storage capacity is not a cost-effective solution. Nonetheless, the Alternative 1 combination of alternatives plus storage was evaluated to eliminate treated discharges from the QCSOSTF during a 1-year storm at the request of MADEP and EPA. This option is evaluated as Alternative 2 below.

2.4.1 Alternative 1

Alternative 1 combines the following CSO control measures:

- <u>Green Hill Pond Diversion</u>: Diverting the Green Hill Pond drainage out of the CSS at an estimated cost of $25,000;

- <u>Regulator Modifications</u>: Raise system regulator weirs at the Endicott at Millbury Street (raise 2 feet), Pond at Water Street (raise 1 foot), Grafton at Franklin Street (raise 1.1 feet), and Vernon at Millbury Street (raise 1 foot) regulators at an estimated cost of $20,000;

- <u>Kelly Square Control Station Modifications</u>: Rehabilitate and activate the Kelly Square gate, estimated to cost $2M; and

- <u>Add New Pumps at the QCSOSTF</u>: Install new submersible pumps in the chlorine contact tanks at the QCSOSTF, at an estimated cost of $2.5M.

The total estimated cost for Alternative 1 improvements is $4.5M. This cost would be in addition to the $84M invested in 2004 dollars for the improvements implemented in 1989 and Worcester's share of the UBWWTF high flow management improvements ($11.7M).

2.4.2 Alternative 2

Alternative 2 combines the CSO control measures in Alternative 1 plus additional storage to eliminate treated discharges at the QCSOSTF in the 1-year storm. Using the hydraulic model of the system, it was determined that 6.5 MG of additional storage beyond that currently provided at the QCSOSTF would be required in order to eliminate treated discharges at the QCSOSTF in a 1-year storm with the Alternative 1 combination and UBWWTF improvements in place. Assuming 3.75 MG of storage may be constructed on the QCSOSTF site adjacent to the existing tanks, as described in Section 2.8, an additional 2.75 MG would need to be constructed elsewhere. One possible location for the satellite tank would be under Crompton Park, adjacent to the Harding Street Overflow Collector. A gate would be installed between the storage tank and the overflow collector to control flow into the tank. The gate would be operated off of the QCSOSTF influent wetwell elevations to avoid filling up the tank until needed. The tank may need to be dewatered back into the overflow collector using pumps unless the tank elevations can be designed to flow by gravity back into the overflow collector or the QCSOSTF.

During excavations completed in the 1980s during construction of CSO facilities, it was determined that contamination from the former ComGas property had migrated east over the property boundary. Therefore, when determining costs for storage at Crompton Park, it was considered prudent to include the cost of hazardous material mitigation. The total cost of this option is approximately $57M, including $4.5M for the Alternative 1 combination of control measures, $30M for the 3.75 MG tank volume constructed adjacent to the QCSOSTF facility, and $22M for the additional 2.75 MG of storage to be constructed under Crompton Park. The $22M cost for the 2.75MG tank is based on a detailed cost estimate for constructing a 2.5 MG tank scaled up to 2.75 MG, including appropriate allowances for contractor overhead and profit, construction contingencies, and engineering. The estimate also includes an allowance for handling and disposal of hazardous wastes.

The $57M total estimated cost for Alternative 2 is in addition to the $84M invested in CSO control from improvements implemented in 1989 and Worcester's share of the UBWWTF high flow management improvements ($11.7M).

2.4.3 Water Quality Benefits

Figure 2-8 shows the impact of Alternatives 1 and 2 for the 1-, 3-, and 6-month design storms. Table 2-9 presents the benefits from the alternatives for both design event simulations and annual simulations. To determine the true impact of these improvements, baseline conditions are also included. Alternative 1 eliminates discharge from the QCSOSTF in a 3-month storm (a 3.8 MG reduction compared to conditions with UBWWTF improvements in place), and reduces discharge by 5.4 MG in a 6-month storm. At a cost of about $4.5M, this represents a cost-benefit of about $1.2 per gallon of discharge reduced in the 3-month storm and $0.84 per gallon reduced for the 6-month storm. Alternative 1 is expected to reduce BOD loadings by 475 pounds and TSS loadings by about 2,770 pounds during 3-month storm conditions. Reductions of 675 pounds and 3,930 pounds of BOD and TSS, respectively, are expected in 6-month storm conditions.

Alternative 2 eliminates treated discharges in 3-month, 6-month, and 1-year storm conditions; however, the cost is significantly higher. At a cost of $57M, this is about $15 per gallon and $8 per gallon of discharge reduced in the 3-month and 6-month storm conditions. Alternative 2 would result in comparable reduction in BOD

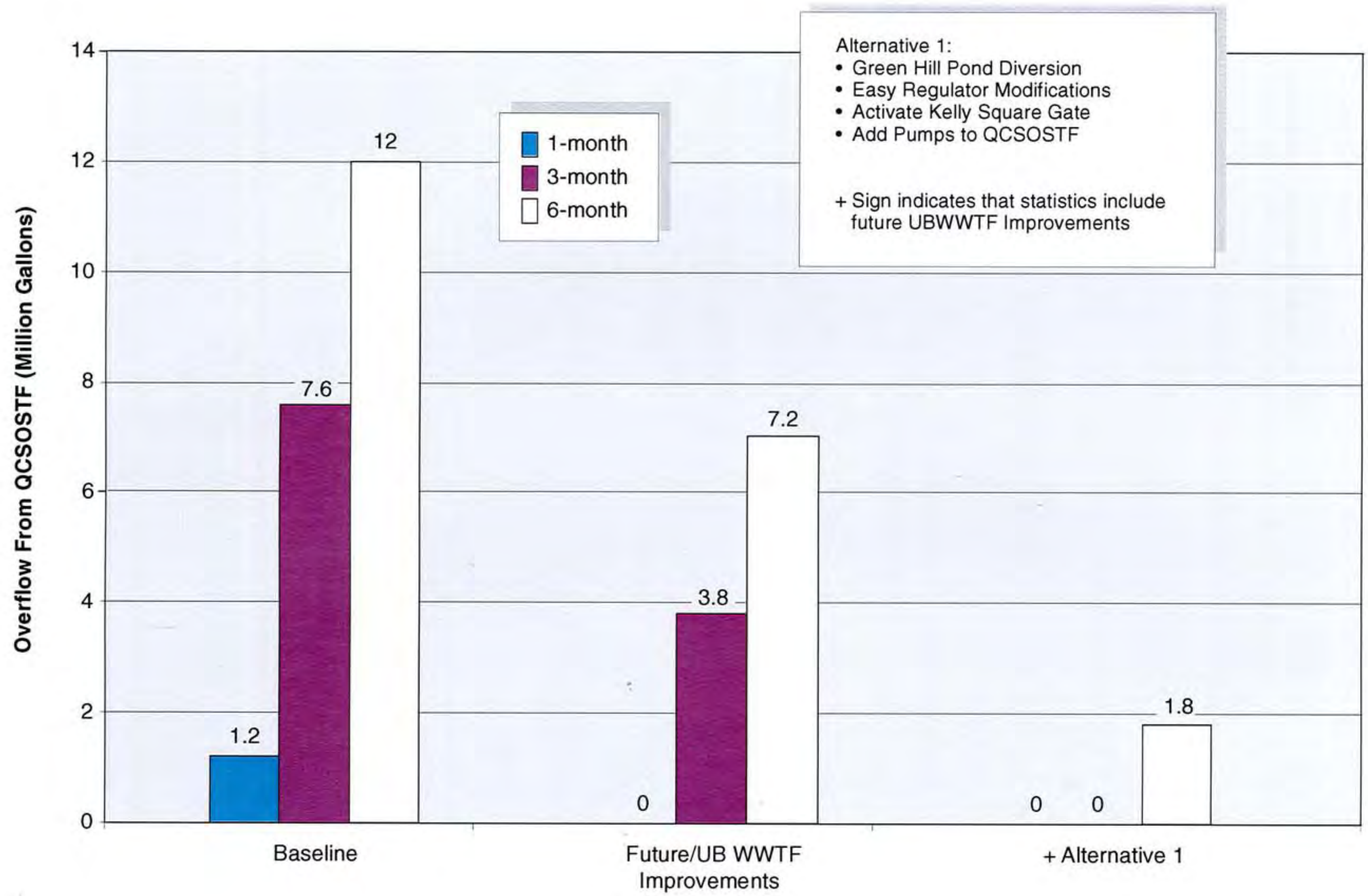

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Figure 2-8
Volume Reduction from Combination of Promising Alternatives Compared with Baseline Conditions

loadings and TSS loadings presented for Alternative 1 since Alternative 1 also eliminates discharges during the 3-month storm and eliminates all but 1.8 MG in a 6-month storm.

As shown in Table 2-9, the average annual reduction in treated discharges at the QCSOSTF with Alternative 1 and the UBWWTF improvements in place is significant. Only two discharges would be expected in a typical year, which is closer to a 6-month level of control than 3-month, discharging a total of 13 MG. This represents an 83 percent reduction in treated discharge volume at the QCSOSTF compared to baseline conditions and a 62 percent reduction compared to baseline conditions with UBWWTF improvements in place. The effluent would receive screening, seasonal disinfection, dechlorination, and partial settling of solids. Given the fact that effluent from the UBWWTF and the QCSOSTF is disinfected seasonally, it may be argued in terms of bacteria impacts that two discharges per year at the QCSOSTF approach a one-year level of control if one discharge occurs in the non-disinfection season and the other occurs in the disinfection season.

While these reductions are significant, they do not translate into significant reductions in BOD and TSS loadings to the Blackstone River on an average-day basis, primarily because the QCSOSTF does not represent a large percentage of the flow discharged to the Blackstone River. With UBWWTF improvements in place, on average, 5,500 pounds per day TSS and 6,300 pounds per day BOD will be discharged from the combined QCSOSTF and UBWWTF facilities. With Alternative 1 in place, loads would be reduced by an average of 6 pounds per day BOD and 34 pounds per day TSS. In terms of annual loadings reduction, Alternative 1 will cost about $2,100 for every pound per year of BOD removed and about $360 for every pound per year of TSS removed.

Table 2-9

Comparing Alternative 1 to UBWWTF Improvements

Parameter	*1-Month*	*3-Month*	*6-Month*
Treated Discharge at QCSOSTF, Million Gallons			
Baseline Conditions	0.0	7.8	12.0
With UBWWTF Improvements	0.0	3.8	7.2
With UBWWTF Improvements and Alternative 1	0.0	0.0	1.8
Treated Discharge at UBWWTF, Million Gallons (2 day simulation)	119.0	120.2	119.7
Baseline Conditions	123.3	124.4	126.1
With UBWWTF	124.7	128.0	131.0
With UBWWTF Improvements and Alternative 1			

**Comparing Annual Performance:
UBWWTF and Alternative 1**

Parameter	Annual
Number of Treated Discharges from QCSOSTF	
Baseline Conditions	14
With UBWWTF Improvements	7
With UBWWTF Improvements and Alternative 1	2
Million Gallons / Year Treated Discharge From QCSOSTF	
Baseline Conditions	83
With UBWWTF Improvements	34
With UBWWTF Improvements and Alternative 1	17

2.5 Comparing Costs and Effectiveness of Alternatives

Figure 2-9 shows the estimated costs of the CSO control alternatives considered under Phase II. Figure 2-10 summarizes the cost effectiveness in terms of dollars spent per gallon of discharge reduced at the QCSOSTF under 3-month storm conditions for the most promising alternatives discussed in each section above. Per dollar spent, the lower cost improvements are much more effective. Alternative 2 is not cost-effective.

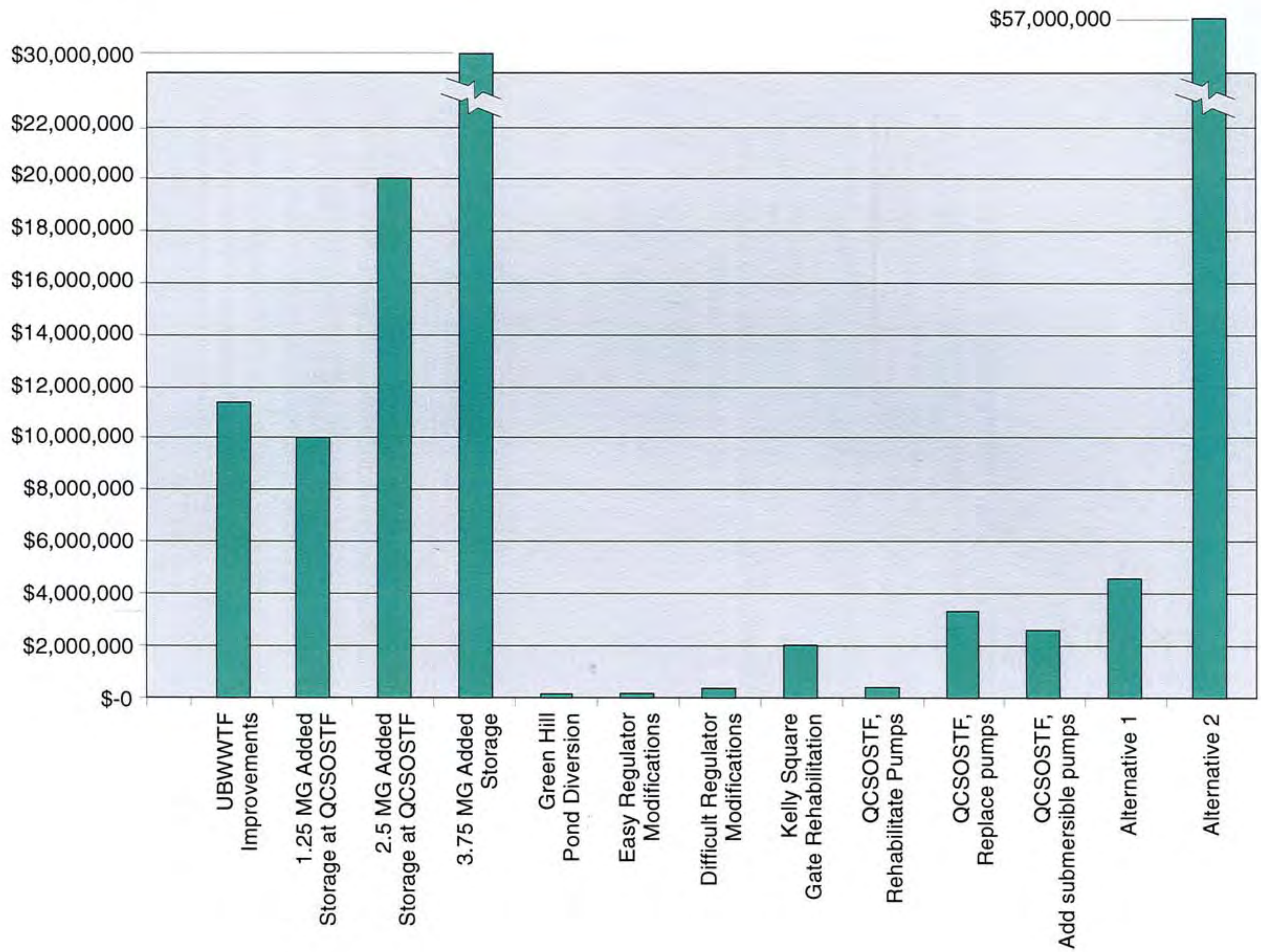

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Figure 2-9
Estimated Costs of CSO Control Alternatives

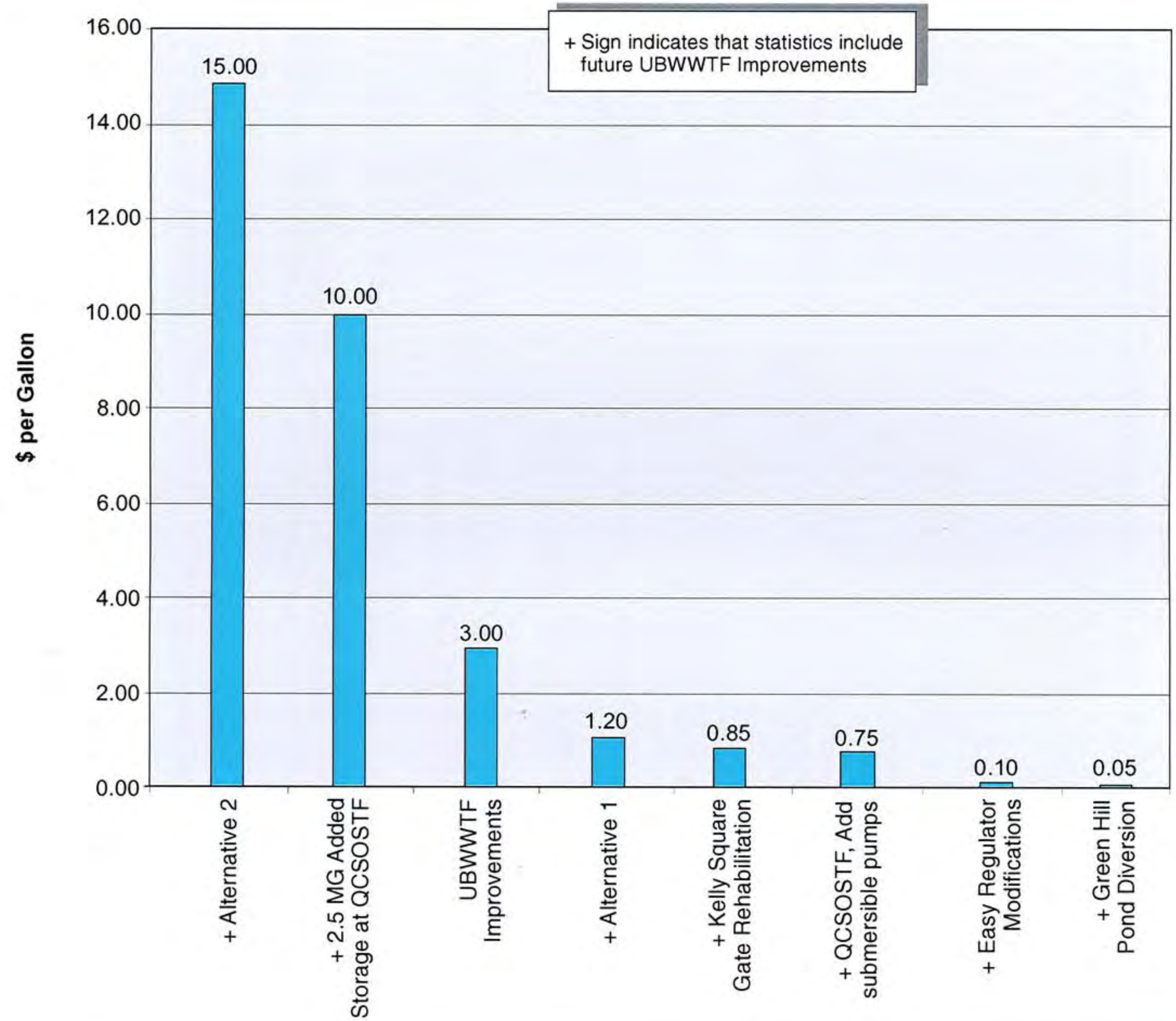

Figure 2-10
3-month Storm: $ per Gallon Reduction in Treated Discharge at QCSOSTF

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Section 3
Water Quality Considerations

3.1 Introduction

This section establishes the relative water quality impact of Worcester's Combined Sewer System (CSS) on the Blackstone River in the past (before CSO facilities were built), under present conditions, and once Long-term Control Plan recommendations are implemented. It also addresses the treatment effectiveness of the facility, now and in the future. Finally, it discusses Blackstone River water quality considerations in context with the City's Long-Term CSO Control Planning, and the recommended plan.

3.2 Water Quality Impact of Worcester's CSS

The relative impact of Worcester's CSS on the Blackstone River was compared to other sources for BOD, TSS and fecal coliform. The total load of these pollutants to the Blackstone River comes from three general sources, the CSS, the Upper Blackstone WWTF, and stormwater runoff from both urbanized areas served by municipal separate storm sewer systems and from more rural, forested areas that have no formal stormwater collection system. The loadings computed herein are meant to quantify stormwater from Worcester's urbanized and rural areas, combined sewage from Worcester's combined sewer system, and effluent from the Upper Blackstone WWTF. This does not represent the total loading to the river, but approximately represents the total loading from the City of Worcester from its CSS and urban and rural stormwater systems. It also includes the total wastewater loading from the City plus other areas served by the UBWPAD. BOD and TSS loadings were computed on an annual basis. Disinfection at the QCSOSTF and the UBWWTF is seasonal, from April 1 to October 15. Therefore, fecal coliform loadings were computed only for the disinfection season. This allowed direct comparisons of loadings from different sources.

3.2.1 Before Construction of CSO Facilities

CSO facilities were constructed in the 1980s. Prior to their construction, the CSS routinely polluted the Blackstone River during both wet and dry weather periods. Therefore, pollutant loads for BOD, TSS, and fecal coliform before CSS improvements were implemented were much larger than today.

CSS loadings were estimated by multiplying the annual discharge estimate from the CSS (1,308 MG) by average *influent* BOD (51 mg/l) and TSS (180 mg/l) concentrations taken from discharge monitoring reports at the QCSOSTF. For fecal coliform, no direct estimates were available for CSS concentrations. However, typical CSO concentrations for three Massachusetts communities were available: 180,500 / 100 ml for Haverhill; 165,000 / 100 ml for Greater Lawrence Sanitary District; and 28,000 / 100 ml for the City of Lowell (CDM, Draft Merrimack River Water Quality Assessment Study, September, 2003). Lowell's concentrations are thought to be relatively dilute; therefore, 173,000/100 ml, the average of the Haverhill and GLSD values, was used.

Stormwater loadings are based on land uses that were assigned event mean concentrations and multiplied by annual runoff volumes. The event mean concentrations for BOD and TSS were based on data collected by the City for its Stormwater Permit for industrial, commercial, and residential land uses and default values from CDM's Water Management Model for agricultural, forested, and open space land uses. Much of the loading within the stormwater category is from forested areas with no formalized storm drainage system.

The stormwater loadings were adjusted to account for the recent reductions from the citywide rehabilitation of twin invert manholes. This adjustment factor (32 percent) was computed based on estimates of the impact of twin invert manholes in the City's Phase I Stormwater Permit Application.

Loadings for the UBWWTF were based on current average day flow estimates (37 mgd) multiplied by the permit limits for BOD and TSS, and the seasonal (April 1 to October 15) average day permit limit for fecal coliform of 200 / 100 ml. Based on these computations, the annual loadings before CSO facilities were built are estimated as shown in Table 3-1:

Table 3-1

Annual Pollutant Loading Estimates before CSO Facilities Were Built

	BOD (pounds per year)	***TSS (pounds per year)***	***Fecal Coliform (counts during disinfection season)***
Combined Sewer System	560,000	1,970,000	4.6 E 15
Stormwater	960,000	2,720,000	1.5 E 15
UBWWTF	2,060,000	2,590,000	5.5 E 13
Total	3,580,000	7,280,000	6.2 E 15

Prior to construction of CSO facilities, the CSS was not the largest source of BOD and TSS loadings to the Blackstone River, but was a significant source, especially considering its relatively small area. It was the most significant source of fecal coliform during the disinfection season.

3.2.2 Present Conditions

The major quantifiable differences between present conditions and before CSO facilities were built are in the city's stormwater system and the construction of the CSO facilities. Since the CSO facilities were built, the city has instituted a stormwater management program under its Phase I Stormwater Permit. The Stormwater

Management Program consists of over 20 best management practices (BMPs) designed to reduce stormwater pollution to the maximum extent practical. These BMPs include regulations, education/outreach programs, source controls, storm drainage system maintenance procedures, and storm drainage system infrastructure improvements. Some of these BMPs lead to quantifiable improvements, while others are harder to quantify. For example, in addition to retrofitting twin invert manholes, the city has removed over 100 illicit connections from its storm drainage system.

CSS loadings for BOD and TSS were estimated by multiplying the average annual discharge volume under present conditions (82 MG) by *effluent* concentrations from the QCSOSTF based on evaluation of discharge monitoring reports (37 mg/l BOD and 119 mg/l TSS). Fecal coliform loadings during the disinfection season were estimated based on the estimated discharge volume during the disinfection season multiplied by the permit limit (400 / 100 ml).

Stormwater loadings were computed based on the product of annual runoff and event mean concentrations for each land use adjusted to apply only to the disinfection season. The stormwater loadings were also adjusted to account for improvements to the City's stormwater system.

The loadings from the UBWWTF were estimated using the same methods as before CSO facilities were built. The estimates for annual loadings under present conditions are presented in Table 3-2:

Table 3-2

Annual Pollutant Loading Estimates – Present Conditions

	BOD (pounds per year)	***TSS (pounds per year)***	***Fecal Coliform (counts during disinfection season)***
Combined Sewer System	25,000	81,000	6.7 E 11
Stormwater	730,000	2,060,000	1.1 E 15
UBWWTF	2,060,000	2,590,000	5.5 E 13
Total	2,815,000	4,731,000	1.2 E 15

The construction of existing CSS facilities has had a dramatic effect on loadings from the CSS. BOD and TSS have been reduced by 96 percent, and fecal coliform during the disinfection season by over 99.9 percent.

3.2.3 With Recommended Plan in Place

In the future, the recommended plan (Alternative 1 described in Section 2) arising from the current Long-term CSO control planning process will result in smaller annual overflow volumes from the combined sewer system. Stormwater loadings will be lower as a result of further improvements to the stormwater system. To estimate CSS loadings, the annual CSO volume (13 MG) with Alternative 1 recommendations in place was multiplied by BOD and TSS effluent concentrations estimated from the discharge monitoring reports (37 mg/l BOD and 119 mg/l TSS). This is a conservative estimate because effluent concentrations in the future may be lower given the significantly higher detention times provided by the QCSOSTF tanks with the recommended plan in place. The detention time under current conditions, only considering storage volume in the two existing 1.25 MG tanks, is at least 15 minutes for peak 1-year storm flows. With the recommended plan in place, this detention time increases to at least 40 minutes. Fecal coliform loadings during the disinfection season were estimated based on the seasonal permit limit of 400/100 ml from April 1 to October 15.

Stormwater loadings will be lower as a result of further improvements to the stormwater system. To be conservative, and since these reductions are hard to quantify, the stormwater loadings with the recommended plan in place were held constant.

Future UBWWTF loadings were assumed to remain the same as current conditions. The additional expected average annual flow (45 mgd compared to 37 mgd present day) will be approximately offset by stricter permit limits expected in the future. The loading estimates once the recommended plan is in place are shown in Table 3-3:

Table 3-3

Annual Pollutant Loading Estimates with Recommended Plan

	BOD (pounds per year)	***TSS (pounds per year)***	***Fecal Coliform (counts during disinfection season)***
Combined Sewer System	4,000	13,000	1.1 E 11
Stormwater	730,000	2,060,000	1.1 E 15
UBWWTP	2,060,000	2,590,000	5.5 E 13
Total	2,794,000	4,663,000	1.16 E 15

With the recommended plan in place, the CSS will account for less than an estimated 0.15 percent of the BOD load, 0.32 percent of the TSS load, and 0.01 percent of the disinfection season fecal coliform load. Figures 3-1 through 3-3 provide bar charts of

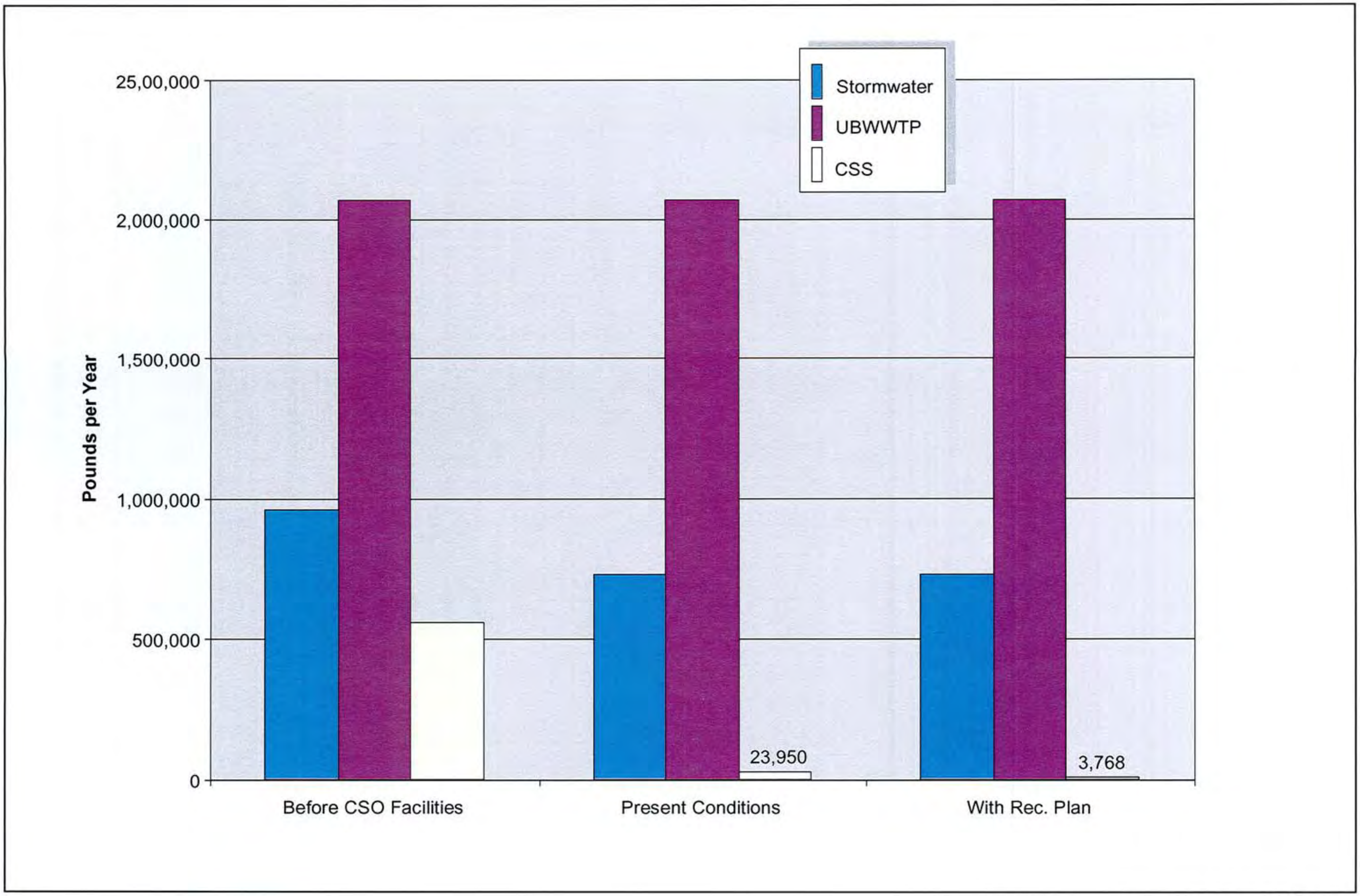

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Figure 3-1
Estimated Annual BOD Loads

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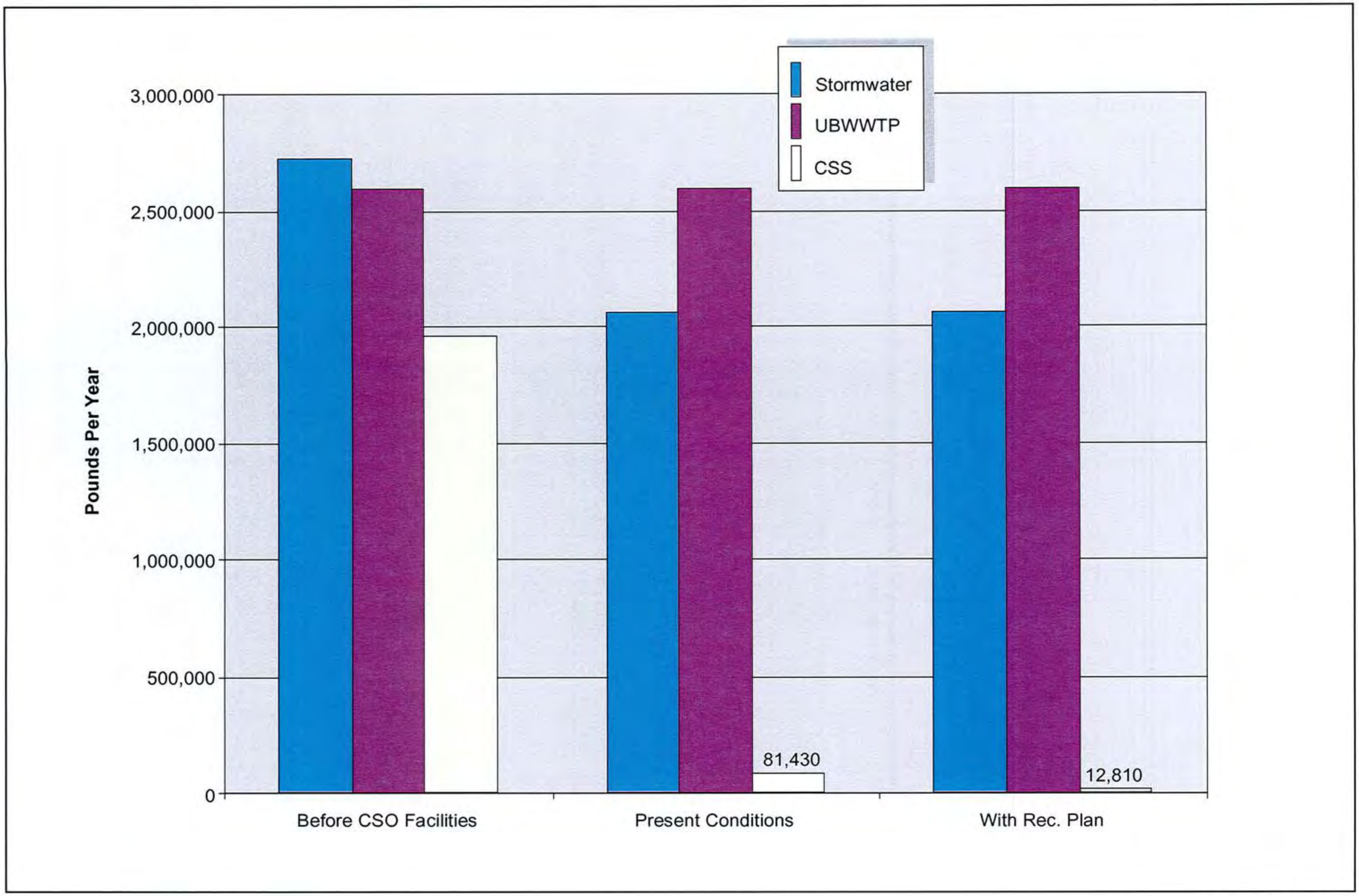

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Figure 3-2
Estimated Annual TSS Loads

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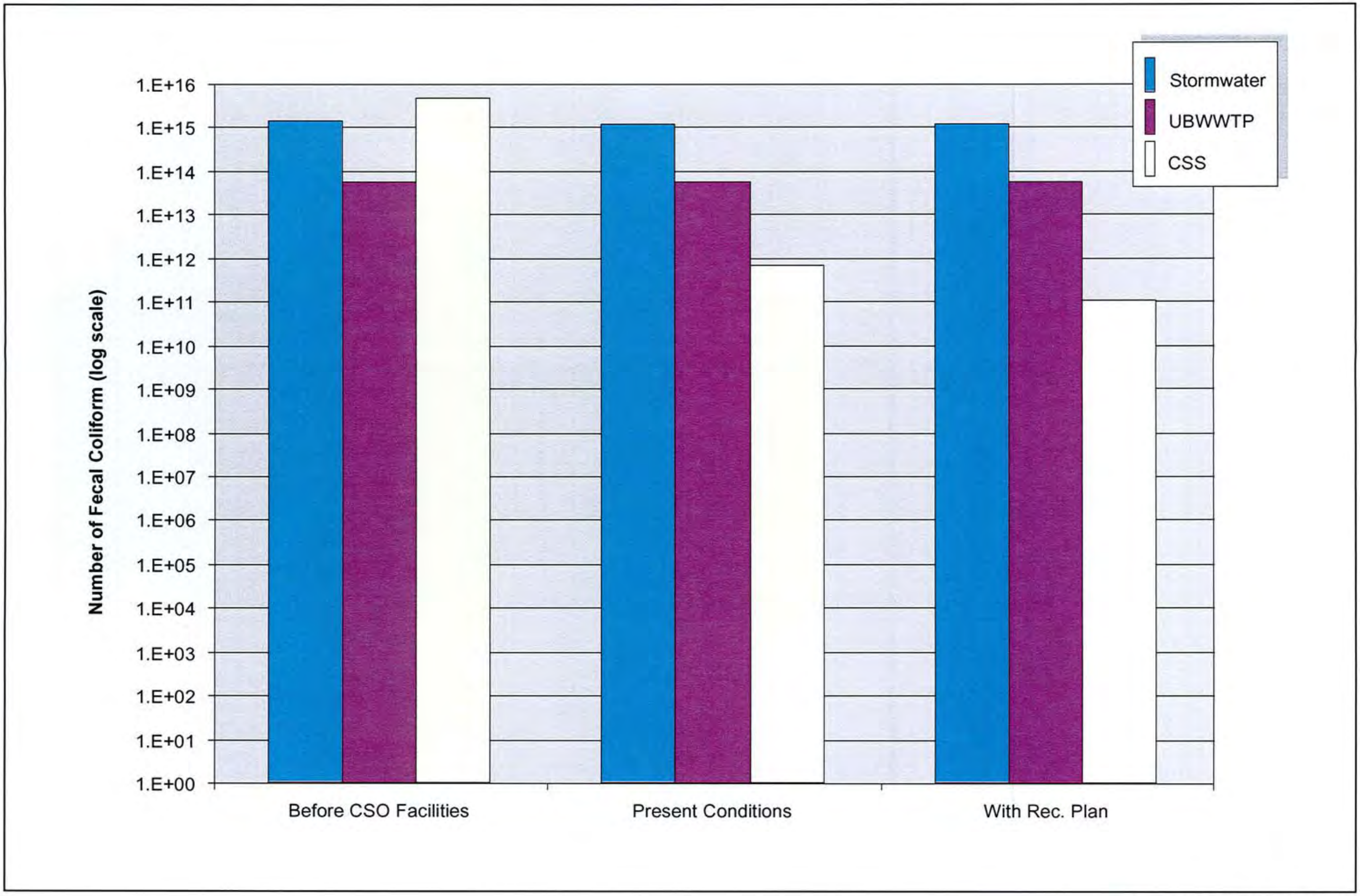

Figure 3-3
Estimated Fecal Coliform Loads During Disinfection Season

these estimates, and clearly demonstrate that additional improvements to reduce CSS loadings will have virtually no impact on Blackstone River water quality.

3.3 Treatment Effectiveness of the QCSOSTF

Information on the treatment effectiveness of the QCSOSTF was presented in Section 7 of the Phase I CSO Long-term Control Plan Report. This section updates the information presented there specifically to account for the longer record available since the Phase I report was published.

3.3.1 Small and Medium Storms

Assuming approximately 100 runoff events per year, most receive full treatment at the UBWWTF. Under current conditions, the effluent gates at the QCSOSTF open only about 12 to 24 times per year. Thus, most of the time, runoff from the CSS receives full secondary treatment at the UBWWTF.

Under the recommended plan, the number of events treated at the UBWWTF will increase, and the QCSOSTF will discharge treated effluent only about twice annually.

3.3.2 First Flush

The pollutant removal efficiency for BOD and TSS calculated below is based on the QCSOSTF Discharge Monitoring Reports (DMRs). The data collected from the DMRs include four-hour composite samples of influent and effluent, analyzed only if and when the QCSOSTF discharges for four or more hours. It is likely that the QCSOSTF actually performs better than the data indicate, because much of the "first flush" from runoff events is stored temporarily and pumped from the QCSOSTF to the UBWWTF before samples are taken.

3.3.3 BOD and TSS Effluent Concentrations and Removal Efficiency at the QCSOSTF

The basic data for removal efficiency was taken from the Discharge Monitoring Reports (DMRs) submitted to EPA monthly from December 1994 to August 2003. For this time period, there were 78 events where both influent and effluent BOD were reported. The data were analyzed by establishing the flow weighted mean of the influent and effluent samples. The mean influent and effluent BOD was 51 mg/l and 37 mg/l, respectively, representing an average removal efficiency of 28 percent. For TSS, there were 104 events in the data set. The mean influent and effluent TSS was 180 mg/l and 119 mg/l, respectively, a 34 percent efficiency.

The UBWWTF Facilities Plan dated October 2001 analyzed primary treatment efficiency during high flow events. The data at the UBWWTF were grouped so that the 98th percentile highest flow days could be analyzed. As reported in Table 6.3-2 of the UBWWTF Facilities Plan, the mean influent and effluent BOD during high flow conditions were 76 mg/l and 58 mg/l, a 24 percent removal efficiency while the mean influent and effluent TSS were 81 mg/l and 55 mg/l, a 32 percent efficiency.

Figure 3-4 compares the BOD and TSS influent and effluent concentrations from the QCSOSTF and the UBWWTF primary facilities. Influent BOD into the QCSOSTF is relatively dilute and the facility is effective at BOD removal. TSS influent and effluent exceed UBWWTF levels, but the removal efficiencies are nearly identical.

The data demonstrate that the QCSOSTF performance is comparable to primary treatment during high flow events. In addition, the removal efficiencies with the recommended plan in place are expected to increase, given the significantly higher detention times that will be provided by the QCSOSTF tanks. The detention time in the two existing 1.25 MG tanks under current conditions during a 1-year storm is 15 minutes or more even without accounting for timing of effluent gate opening and closing, or considering storage in the overflow collectors. With the recommended plan in place, this detention time for the 1-year storm increases to 40 minutes or more.

3.3.4 Fecal Coliform and Total Residual Chlorine Levels in QCSOSTF Treated Discharge

As with TSS and BOD levels, most combined sewage and stormwater runoff from the Worcester CSS is pumped from the QCSOSTF and treated at the UBWWTF. When the CSO facility discharges, it is subject to a seasonal permit limit from April 1 to October 15 of 400 / 100 ml for fecal coliform and 0.02 mg/l for total residual chlorine (TRC).

When the QCSOSTF was put in service, it had no dechlorination facilities, and consequently frequently exceeded the TRC limit. Therefore, the City added dechlorination facilities. These facilities were put on-line in August 1998. A major start-up and trouble-shooting period to refine processes extended until October 1999.

The facility met fecal coliform permit limits 92 percent of the time before dechlorination facilities went on line. This dropped to 52 percent during the start-up period and has improved to 65 percent since then.

The median TRC level prior to dechlorination was 0.44 mg/l, exceeding the TRC limit 22-fold. During dechlorination start-up, the median TRC level dropped to 0.01 mg/l, and the facility met its 0.02 mg/l limit 74 percent of the time. The City achieved a major reduction in residual chlorine discharged from the facility, but struggled with correct dosing to meet both fecal coliform and TRC limits. The median TRC level increased to 0.03 mg/l following the dechlorination start-up period to enable the facility to better meet fecal coliform limits. The facility has met its TRC limit 48 percent of the time post start-up.

The City continues to monitor performance closely, and meets after every event to discuss system performance and make adjustments for the next event.

The City believes the facility could meet its fecal coliform limit consistently if relief were provided regarding the very low TRC limit. The water quality standards established for chlorine are 0.019 mg/l daily maximum and 0.011 mg/l monthly

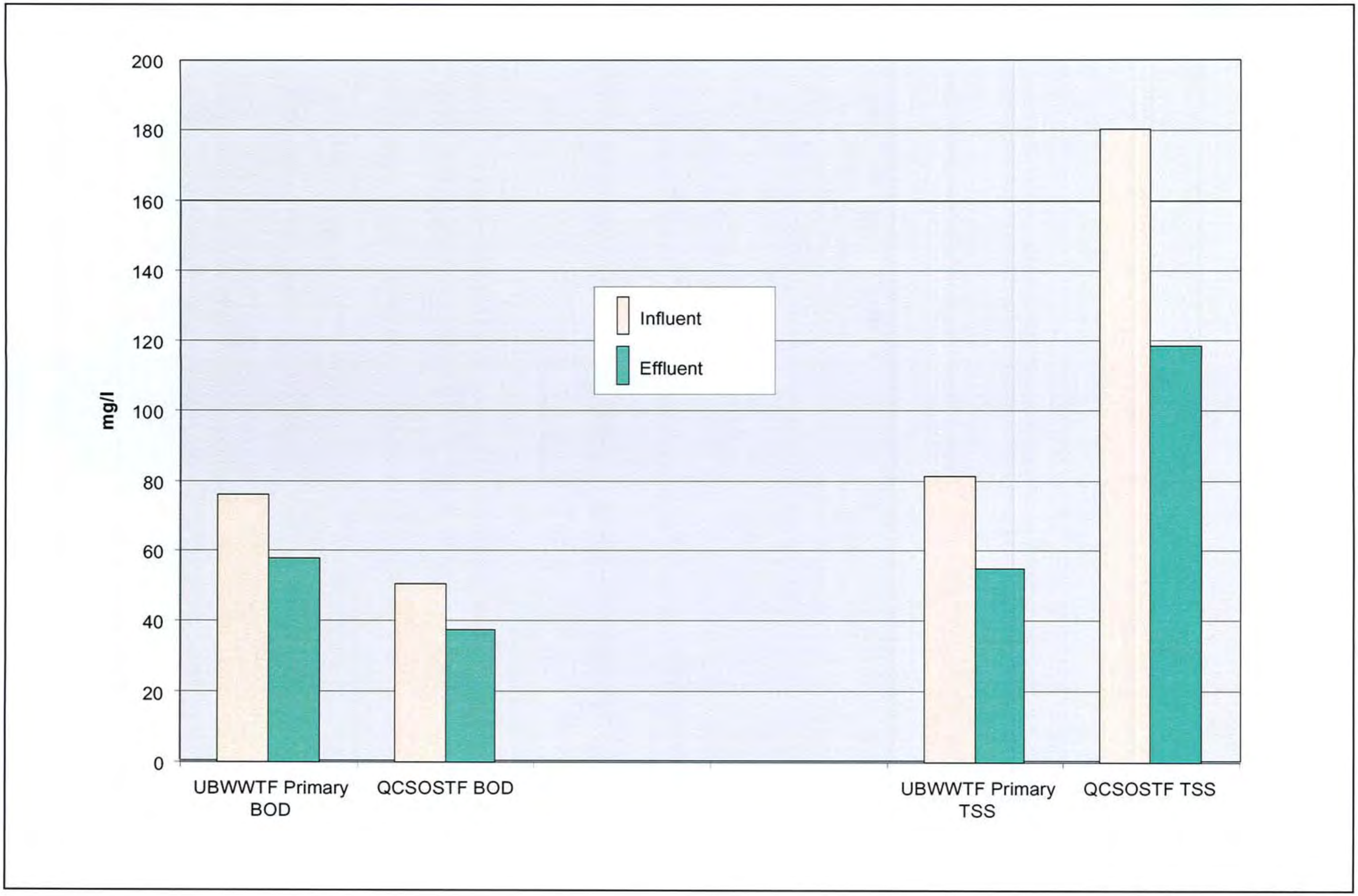

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Figure 3-4
Comparison of UBWWTF Primary and Quinsigamond CSOSTF BOD and TSS Influent and Effluent Concentrations

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average in the receiving waters. Typically when issuing permits, EPA includes a fact sheet where it establishes and sets permit limits based on the available dilution. The fact sheet attached to the QCSOSTF permit contained no such calculations, and since the limit (0.020 mg/l) was set approximately equal to the daily maximum for chlorine (0.019 mg/l), it is assumed that no dilution was allowed.

The City believes that allowances for dilution are justified, given the fact that the QCSOSTF discharges only during significant storm events, when flow rates in the receiving waters are high. A preliminary analysis was conducted to determine dilution available at the headwaters of the Blackstone River (the confluence of Mill Brook and Middle River). The discharge during project design storm events generated by the drainage area (less the 4 square mile combined sewer system) was estimated using flow-duration information at the USGS Kettle Brook gage (adjusted to account for an upstream flood diversion) and prorating the data collected there for the larger drainage area for the Blackstone River. Because of the urbanized nature of the watershed, this method may underestimate peak flows and available dilution. Table 3-4 presents these estimates alongside estimates of peak discharge from the QCSOSTF from the SWMM model of the combined sewer system, assuming Long-term Control Plan recommendations are in place. The table computes dilution assuming concurrent peaks and presents a TRC concentration in the effluent from the QCSOSTF that would still meet water quality standards. A more detailed analysis accounting for both watershed urbanization and timing of peak flows may result in slightly different TRC concentrations.

Table 3-4
Available Dilution in the Blackstone River for Discharges From the QCSOSTF with Recommended Plan

	Peak Discharge in Receiving Water (mgd)	*Peak Discharge from QCSOSTF (mgd)*	*Total Discharge Recommended Plan (mgd)*	*Available Dilution*	*TRC Limit (mg/l)*
1-month storm	175	0	175	N/A	N/A
3-month storm	259	0	259	N/A	N/A
6-month storm	350	86	436	5.1	0.096
1-year storm	452	90	542	6.0	0.114
5-year storm	905	326	1,231	3.8	0.072

The most severe case once Long-term Control Plan recommendations are in place is a 3.8:1 dilution, where a TRC limit of 0.072 mg/l would adequately protect the water quality of the receiving water. A similar analysis was performed for present conditions, where the most severe case was a dilution of 2.9:1 during the 1-year storm, resulting in a TRC limit of 0.055 mg/l. Therefore, the City requests consideration of a higher TRC permit limit at 0.055 mg/l now and 0.072 mg/l once Long-term Control Plan recommendations are in place. This will greatly enhance the ability to meet the fecal coliform permit limit without adversely impacting water quality.

This Phase II Long-term Control Plan summarized the literature review conducted in Phase I to assess the capability of the QCSOSTF to provide adequate disinfection considering potential particle interference. This literature review is contained as Appendix A.

3.4 Water Quality Impacts of the CSS on the Blackstone River

Phase I of the LTCP evaluated the water quality impacts of the CSS on the Blackstone River in depth. Phase II provides additional information further supporting the Phase I conclusions, which are best summarized in Figures 3-1, 3-2, and 3-3. Given its relatively small four-square-mile area, the water quality impact of the CSS on Blackstone River water quality was once disproportionately large. However, the water quality impacts under current conditions are much smaller and once LTCP recommendations are in place will be relatively insignificant.

As shown above, the QCSOSTF provides a level of treatment approximately equal to primary treatment during high flow periods. Treatment level and effectiveness at the QCSOSTF will only improve with the implementation of the recommended plan. Detention time and contact time in the QCSOSTF storage tanks will increase significantly. Instead of 12 to 24 treated discharges, only two treated discharges will occur in a typical year. During one-year storm conditions, the detention/contact time in the QCSOSTF storage tanks will increase to 40 minutes or more, compared to 15 minutes today for peak instantaneous flows. This detention/contact time will increase for lower, more sustained peaks. This enhances both sedimentation and disinfection capabilities of the facility.

With CSO Long-term Control Plan recommendations in place, treated discharges from the QCSOSTF will not cause or contribute to exceedences of WQS on the Blackstone River. Untreated discharges will be rare. All flow from Worcester's CSS will receive treatment at either the UBWWTF or the QCSOSTF up to a 15-year return period, even exceeding the performance of most separated systems. Therefore, the City contends that the current Class B classification of the Blackstone River is appropriate.

In light of this discussion, the affordability analysis conducted for Phase I was not updated for this Phase II Long-term Control Plan. Combined sewer flows from Worcester's combined sewer system will receive treatment at either the UBWWTF or the QCSOSTF for flows up to a 15-year return period and the level of treatment provided will not cause or contribute to exceedences of WQS. Furthermore, the affordability analysis completed for Phase 1 is still applicable for Phase 2, since the cost of the Phase 2 recommended plan does not differ markedly from the costs presented during Phase 1; therefore, the Phase I conclusions remain unchanged. In light of the limited value of an updated affordability analysis and the fact that discharges from Worcester's combined sewer system will not cause or contribute to exceedences of Water Quality Standards, it was considered appropriate to forego updating the affordability analysis.

As demonstrated in this report, the expenditure of additional funds to further reduce pollutant loadings from the CSS beyond LTCP recommendations would provide little benefit, and is not recommended.

Section 4
Recommended Plan

This section presents the Long-term Control Plan recommendations, the benefits attributable to the plan, and a proposed implementation schedule.

4.1 The Recommended Plan

The recommended plan calls for improvements to existing combined sewer system (CSS) facilities. They are in addition to Upper Blackstone WWTF improvements that will help mitigate CSS impacts. They consist of four distinct projects. Locations for these projects are shown in Figure 4-1.

Divert Green Hill Pond flows from the combined sewer system. Green Hill Pond currently has two outlets, one that discharges surface water and stormwater runoff to the combined sewer system, the other that discharges to Coal Mine Brook and then Lake Quinsigamond. This project raises the weir at the outlet to the combined sewer system so that all flow up to a 10-year storm is discharged to Coal Mine Brook. The estimated cost is $25,000. This will reduce flow discharged from the QCSOSTF by about 0.5 MG, at a cost of about $0.05/gallon, during 3-month storm conditions.

Raise weirs at four regulator structures:

- Raise the weir at the Grafton at Franklin Street regulator by 1.1 foot.
- Raise the weir at the Endicott at Millbury Street regulator by 2 feet.
- Raise the weir at the Pond at Water Street regulator by 1 foot.
- Raise the weir at the Vernon at Millbury Street regulator by 1 foot.

The cost of raising the weirs is expected to be less than $5,000 each. This will reduce flow discharged from the QSCSOSTF by about 0.2 MG, at a cost of about $0.10/gallon, during 3-month storm conditions.

Rehabilitate and Activate the Kelly Square Control Structure.

This improvement includes:

- Establishing a Gate Operating Policy to ensure operations maximize storage without causing flooding.
- Spot repairs in the Old Mill Brook conduit upstream of the Kelly Square Control Structure to maintain its structural integrity.
- Redirecting service connections to the Old Mill Brook Conduit, to maintain sewer services without the threat of sewer backups.

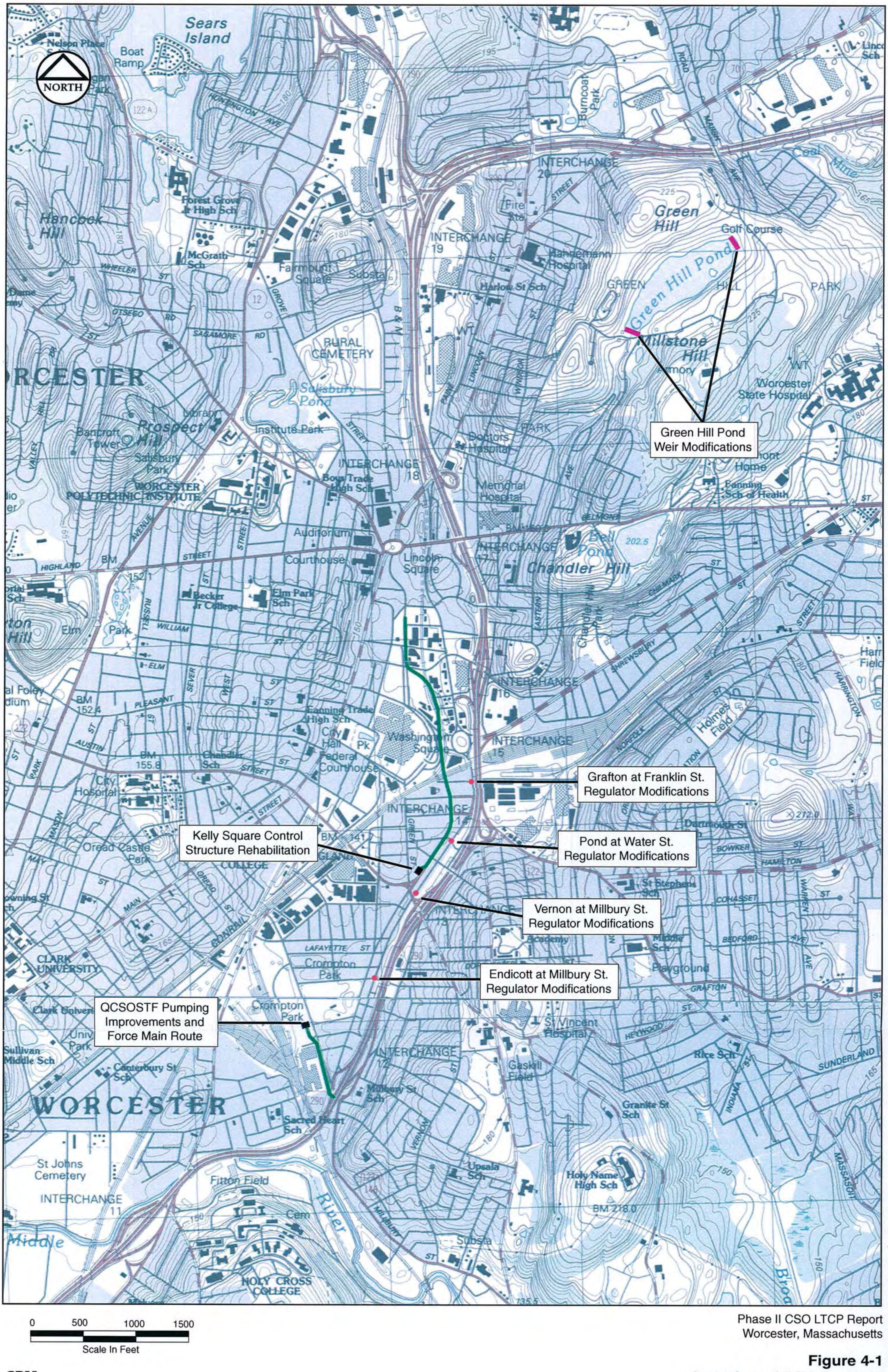

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Figure 4-1
Location of CSO Projects

- Rehabilitation of the Kelly Square facility to restore and update its functionality.
- Replacing instrumentation and control equipment.

The estimated cost to rehabilitate and activate the Kelly Square control structure is approximately $2M. This will reduce flow discharged from the QCSOSTF by about 2.4 MG, at a cost of about $0.85/gallon, during 3-month storm conditions.

Install 4-6,000 GPM Submersible Pumps at the QCSOSTF

This improvement will increase the pumping capacity from the QCSOSTF to the UBWWTP from 19.8 mgd to 54 mgd. This improvement will be most beneficial *after* Phase II improvements at the UBWWTP to increase its capacity are completed. It will allow much greater pumping from the QCSOSTF to the UBWWTF, thereby reducing the frequency and duration of overflows at the QCSOSTF. The estimated cost of this improvement, which includes construction of a 1,200 foot force main from the QCSOSTF to the discharge conduit, is $2.5M. This will reduce flow discharged from the QCSOSTF by about 3.4 MG, almost eliminating the overflow during 3-month storm conditions. The cost of the improvement is about $0.75 / gallon reduced during 3-month storm conditions.

Combined Impact of the Improvements

The recommended plan in total has a greater impact than each improvement individually. The recommend plan eliminates discharge during the 3-month event. At a cost of $4.5M, it costs about $1.20/gallon reduced during 3-month storm conditions. The plan reduces discharge during the 6-month storm by 5.4 MG, from 7.2 MG to 1.8 MG, and costs about $0.85/gallon reduced during 6-month storm conditions.

The average annual reduction in treated discharges at the QCSOSTF with the recommended plan and the UBWWTF improvements in place is significant. Only two discharges would be expected in a typical year, which is closer to a 6-month level of control than 3-month, discharging a total of 17 MG. This represents an 80 percent reduction in treated discharge volume at the QCSOSTF compared to baseline conditions and a 50 percent reduction compared to baseline conditions with UBWWTF improvements in place.

Furthermore, the recommended improvements will enhance treatment capabilities at the QCSOSTF by increasing the detention time provided for peak flows resulting from various storms. For example, with the recommended plan in place, the existing 2.5 million gallon storage tanks at the QCSOSTF will provide over 40 minutes of detention time for peak instantaneous treated discharge flows resulting in a 1-year storm event, compared to 15 minutes today. The recommended improvements effectively reduce the peak treated discharges at the QCSOSTF. The increased detention time provided for these lower peak flows will enhance both sedimentation and disinfection effectiveness at the QCSOSTF.

The recommended plan provides an incremental improvement to the already significant improvements to Worcester's CSS. To show the improvements in total, Table 4-1 compares Worcester's combined sewer system before 1976 facilities plan recommendations were constructed with expected conditions once these Long-term Control Plan recommendations are in place. It demonstrates that the City continues to invest heavily in water quality improvements in its CSS, and the investment is dramatically reducing CSS impacts on the Blackstone River.

Table 4-1

Comparison of System Performance and Cost before Construction of CSO Control Facilities and after Implementation of the Long-term Control Plan

Parameter	Before CSO Facilities	After LTCP
Number of Untreated CS Outfalls	>17	0
Number of Treated CS Outfalls	0	1
Dry Weather Overflows	Yes	No
Number of Untreated Overflow Events Annually	100 (every rainfall)	0
Number of Treated Overflow Events Annually at the QCSOSTF	0	2
Estimated Annual Untreated Overflow Volume (Million Gallons)	1,308	0
Estimated Annual Treated Volume (Million Gallons)		
Treatment at UBWWTF (Million Gallons)	0	1,291
Treatment at QCSOSTF (Million Gallons)	0	17
Annual Loading from the QCSOSTF to the Blackstone River:		
BOD (Pounds)	490,000	3,800
TSS (Pounds)	1,735,000	10,000
Fecal Coliform (Number)	8.5 E 15	3.9 E 13
Cost (including existing facilities @ $84M, Phase I UBWWTP high flow management improvements @ $11.7M, and LTCP recommendations @$4.5M, 2004 dollars)	$0	$100.2M

4.2 Proposed Implementation Schedule

The Green Hill Pond Diversion can be accomplished once this LTCP is approved and any necessary permits are obtained. To allow for permitting, construction of the diversion is scheduled for the summer of 2005.

Raising weirs at four regulator structures can also begin after LTCP approval. No permitting issues are expected; therefore, construction is slated tentatively for summer, 2004.

Rehabilitation of the Kelly Square control structure requires further field studies and design prior to construction. The proposed schedule is:

- Design (including permitting and field studies): July 2004 - March 2005
- Construction: July 2005 - December 2005

Installation of submersible pumps and associated force main construction is dependent on Phase I and Phase II UBWWTF improvements. Phase I improvements are under a Consent Order to be completed by August 2006. Phase II improvements are planned for completion in 2009. It is preferred that the pumping capacity at the QCSOSTF not be increased until after implementation of Phase II so that construction of the secondary treatment system improvements is not impacted by the higher influent flows. It also would provide a period of time for UBWWTF operators and engineers to evaluate secondary bypass performance and permit compliance with real operating data. Thus the proposed schedule for the QCSOSTF pumping capacity improvement is:

- Design (including permitting and field studies): July 2008 - March 2009
- Construction: June 2009 - December 2009 (after Phase II UBWWTF improvements are completed)

This proposed schedule is shown in Figure 4-2.

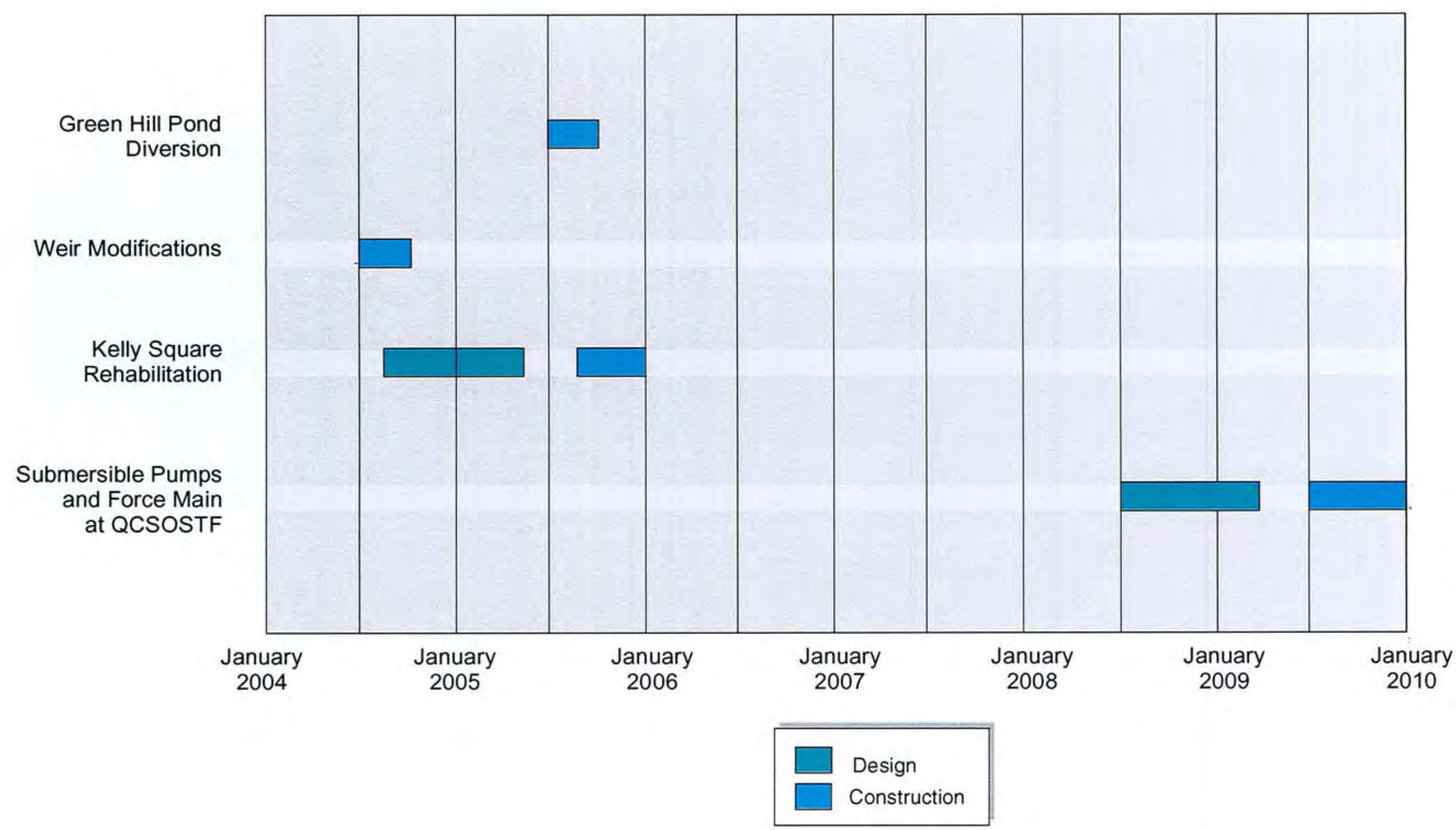

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Figure 4-2
Proposed Schedule
Design and Construction of
Long-term Control Plan Recommendations

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Appendix A
Evaluation of the QCSOSTF Effectiveness

A.1 Literature Review

CDM conducted a literature review to provide insight into the QCSOSTF and its capability to provide adequate disinfection considering potential particle interference. The literature review extended beyond the literature available that was specific to CSO facilities. It included literature on particle shielding and regrowth of coliforms in receiving waters following chlorination.

A.1.1 Summary

The most important conclusions from a review of the literature are:

- Coliform levels, as a group, can increase after chlorination.

- Although coliform levels can increase after chlorination and discharge, chlorination is thought to reduce the number of coliform organisms originally discharged, thereby limiting the peak aftergrowth. Thus, even with regrowth, coliform counts are lower with chlorination than without chlorination.

- Although total coliform levels have been shown to increase after chlorination, many studies report that fecal coliform levels do not increase after chlorination. Unfortunately, there is contradictory evidence regarding fecal coliform. Perhaps regrowth is not as important when considering fecal coliform.

- Some organisms survive chlorination because they are protected from contact by aggregates of suspended solids.

- Disinfection effectiveness is improved and regrowth is minimized if chlorine residual is maintained.

- Disinfection effectiveness is related to contact time, dosing, and mixing among other factors.

- There is little relationship between indicator organism concentration (coliform or fecal coliform) and pathogen concentration (viruses, etc.).

- Chlorination reduces pathogens. The bactericidal effect is considered good, the viricidal effect is considered moderate.

- Secondary or advanced treatment is more effective at removing pathogens prior to chlorination than primary treatment.

- Since pathogenic viruses need host cells, they will not multiply outside the host, and regrowth is not an issue for pathogenic viruses. Viruses released to the environment are also susceptible to inactivation by temperature, pH, sunlight and other factors.

- Chlorination of effluent may not improve microbiological water quality during certain times of the year and beyond a certain zone downstream of the discharge.

- Dechlorination is generally successful in reducing chlorine residual, but it is extremely difficult to reduce TRC levels to .02 mg/l.

A.1.2 Application to the QCSOSTF

The facility is designed to store the flow before discharging to the Upper Blackstone River. The facility approximates primary treatment. Thus:

- Pathogens are more likely to be removed at the UBWWTF, which provides primary and secondary treatment, than at the QCSOSTF, which does not provide secondary treatment. The QCSOSTF pumps as much flow as possible to the UBWWTF.

- The QCSOSTF removes pathogens through sedimentation in the storage tanks and through chlorination. It is less effective at pathogen removal than a secondary treatment plant, but more effective than typical CSO treatment facilities with no storage and sedimentation.

- Given that CSO influent is sewage diluted with rainfall-runoff, pathogen concentrations in influent are not nearly as high as at a WWTP, and some of these diluted levels of pathogens are removed during treatment.

- The QCSOSTF pumps, which pump as much flow as possible to the UBWPAD; the chlorine contact tanks, which store as much flow as possible before discharging; and the large overflow collectors, which act as a wet well for the QCSOSTF pumps; all enable the QCSOSTF to either pump or settle out the "first flush" flows, which have the highest concentrations of pathogens, suspended solids and other pollutants. Therefore, only the more dilute flows will be discharged following disinfection and dechlorination.

- The QCSOSTF provides 15-minutes of contact time or more for peak instantaneous flows up to a 1-year frequency. With the recommended plan in place, the QCSOSTF will provide 40 minutes or more of contact time for peak instantaneous flows up to a 1-year frequency.

- The TRC limit at the QCSOSTF (0.02 mg/l) is extremely low and difficult to meet. Higher allowable TRC levels would increase disinfection effectiveness.

- QCSOSTF flow velocities through the influent gates to contact tank 1 and the baffle wall in contact tank 2 provide mixing and enhance disinfection effectiveness.

- Seasonal chlorination is appropriate.

- Pathogens similar to coliform bacteria may increase before decreasing. Viral pathogens are likely to decrease by natural processes since they require a host.

- None of the references established if regrowth is a problem (it would only be a problem if pathogenic bacteria regrow, not indicator bacteria). Many of the pathogens cannot regrow.

- None of the references established at what dose pathogens (bacterial or viral, subject to regrowth or not) are problematic.

- The best way to move forward is to maximize treatment at the UBWPAD WWTF, and treat the remaining flow that exceeds WWTF capacity at the QCSOSTF.

A.1.3 References

The following is the list of references, including some of the major conclusions reached from each reference. The literature covered a period from 1951 to present. The list is in chronological order.

1. Heukelekian, H. "Disinfection of Sewage with Chlorine," *Sewage and Industrial Wastes*. March 1951.

    - Laboratory experiments to show if the residual total coliform organisms in chlorinated effluents multiply when discharged to a receiving water.

    - When chlorinated sewage is diluted with stream and distilled water, an increase in total coliform organisms occurs.

    - The higher the concentration of sewage or effluent, the greater the increase in total coliform. The increase occurs within one day after dilution.

    - When diluted with seawater, no material increase in total coliform numbers occurs.

    - *Relevance to QCSOSTF: Total coliform organisms in chlorinated effluents to fresh water multiply after disinfection. Influent concentrations of bacteria are expected to be lower for dilute CSOs treated at the QCSOSTF than for dry weather sanitary flows. Therefore, the increase in total coliform levels will be lower.*

2. Eliassen, Rolf. "Coliform Aftergrowths in Chlorinated Storm Overflows." *Journal of the Sanitary Engineering Division*. Proceedings of the American Society of Civil Engineers. April 1968.

    - Bacteriological studies of CSO effects on the Charles River basin. Sample from the Cambridge CSO at the Boston University Bridge.

    - Organic matter in wastewater and river basins promotes the metabolism of coliform, resulting in total coliform aftergrowth in receiving water following chlorination.

- Chlorination to the 15-min chlorine demand of the overflow mixture will appreciably reduce the numbers of total coliform organisms discharged to the river and will also limit the peak aftergrowth.

- Chlorination to the 15-minute chlorine demand will result in average MPN aftergrowth values in the basin from 10 percent to 30 percent of those which would develop if unchlorinated overflow were discharged to the river in the normal ranges of summer dilutions.

- The aftergrowth phenomenon must be studied in establishing design criteria for treatment facilities for overflows from combined sewer systems.

- *Relevance to QCSOSTF: Chlorination will reduce the number of coliform organisms discharged, and will limit the peak aftergrowth. The QCSOSTF provides 15 minutes of contact time or more for peak instantaneous flows up to a 1-year frequency, and just under 15 minutes for peak 2-hour average flows in a 5-year storm under current conditions.*

3. Evans, F.L., Geldreich, E.E, Weibel, S.R., Robeck, G.G. "Treatment of Urban Stormwater Runoff." *Water Pollution Control Federation.* Vol. 40, no. 5 part 2. May 1968.

- Study of urban stormwater runoff as pollution source. Bench-scale settling and chlorination experiments for treatment.

- Urban storm runoffs appear to be significant sources of nutrients.

- Fecal streptococci do not demonstrate the aftergrowth shown by the total coliform group, emphasizing the importance of fecal coliforms, rather than total coliforms as a more realistic indicator of the downstream effects of chlorinated discharges.

- Aftergrowth is defined as the apparent increase in bacterial count that may result following a mixture of chlorinated sewage effluent and receiving waters. This condition is a specialized type of bacterial growth response to nutrient discharges into the receiving stream.

- Generally, aftergrowth phenomena are associated with a non-fecal coliform segment of total coliforms of which *Aerobacter aerogenes* is the major strain. *A. aerogenes* is the coliform most responsive to the stimulation of available nutrients because it can grow with a very minimal amount of nutrient and does not require the complex amino acids or other additives that are necessary for *Escherichia coli* or other fecal strains.

- Aftergrowth is a product of many interrelated factors associated with bacteria and their environment. Some organisms survive chlorination to become the inoculum that utilizes the available nutrients. These organisms include strains that are protected from contact with chlorination by aggregates of suspended matter. As the aggregates disintegrate, viable cells are released into the partially treated stormwater.

- *Relevance to QCSOSTF: (1) Fecal coliform do not demonstrate the same regrowth potential as other coliform groups. (2) Among the reasons organisms survive chlorination: disinfection is less than 100% effective; some organisms survive because they are protected from contact by aggregates of suspended matter.*

4. Chambers, Cecil W. "Chlorination for Control of Bacteria and Viruses in Treatment Plant Effluents." *Water Pollution Control Federation*. Vol. 43, no. 2. 1968.

   - Overview study of previous work done.

   - Since effluent stream contributes its bacteria content to the receiving water, it cannot greatly exceed water quality standards if the watercourse is already at the maximum limit, in such a case, the receiving water and effluent quality would be the same.

5. Deaner, David G., Kerri, Kenneth D. "Regrowth of Fecal Coliforms." *Journal of American Water Works Association*. September 1969.

   - Study to see if fecal coliforms experience regrowth after stream discharge, in light of evidence that nonfecal coliforms do.

   - No fecal regrowth occurred below the outfall of a highly treated wastewater during the sampling period.

   - Significant inhibitory factors: short travel time of organisms within study section; lack of bacterial nutrients; physiographic features of the river, including shallowness, swiftness, and low turbidities.

   - *Relevance to QCSOSTF: Can prevent fecal coliform regrowth. Inhibitory factors including lack of nutrients and physiographic features of the receiving water.*

6. Shuval, Hillel I., Cohen, Judith, Kolodney, Robert. "Regrowth of Coliforms and Fecal Coliforms in Chlorinated Wastewater Effluent." *Water Research.* Vol. 7, 1973.

   - Laboratory and field experiments of bacterial regrowth in chlorinated/ dechlorinated wastewater.

   - Field observations indicated the greater the residual chlorine in the storage reservoir, the less likely there would be regrowth.

   - Fecal coliforms generally showed less regrowth than total coliforms.

   - Pathogenic viruses cannot multiply in sewage or water outside the living host cell.

- *Relevance to QCSOSTF: (1) There will be more regrowth in dechlorinated effluents, but this may not be an issue because fecal coliform generally show less regrowth than coliforms in general. (2)Pathogenic viruses will be reduced to some extent by chlorination; they cannot multiply outside the living host cell, therefore regrowth is not an issue for pathogenic viruses.*

7. Silvey, J.K.G., Abshire, R.L., Nunez, W.J. III. "Bacteriology of Chlorinated and Unchlorinated Wastewater Effluents." *Water Pollution Control Federation.* Vol. 46, no. 9. September 1974.

   - Nonfecal coliform strains exhibit significant aftergrowth, whereas fecal strains, as a rule, do not reproduce in wastewater.

8. Kinney, Crispin E., Drummond, David W., Hanes, N. Bruce. "Effects of Chlorination on Differentiated Coliform Groups." *Water Pollution Control Federation.* October 1978.

   - Increase in numbers of coliforms after chlorination is result of recovery of damaged cells rather than bacterial growth.
   - Total coliform group does not necessarily simulate behavior of microbial pathogens.
   - Fecal coliform group does not necessarily simulate behavior of microbial pathogens.
   - All coliform groups (including fecal) increased in number following chlorination (even with advanced wastewater treatment).

9. Berg, Dahling, Brown, and Berman. *Applied and Environmental Mircobiology.* 12/78

   - Disinfected effluents free of non-spore forming bacteria, such as fecal coliform, may still contain viruses.
   - *Relevance to QCSOSTF: Fecal coliform is not a good measure of the presence of viruses in the effluent.*

10. LeChevallier, Evans, and Seidler. *Applied and Environmental Microbiology.* 11/80

   - Turbidity (especially if total organic carbon (TOC) is present) makes disinfection less efficient. TOC creates a chlorine demand.
   - *Relevance to QCSOSTF: Although TOC is not measured at the QCSOSTF, TOC levels are expected to be relatively low in discharge from the QCSOSTF (TSS, if it can be considered a surrogate for TOC, is low). Thus, there is no reason to expect disinfection to be inefficient.*

11. LeChevallier, Mark W., Evans, T.M., Seidler, Ramon J. "Effect of Turbidity on Chlorination Efficiency and Bacterial Persistence in Drinking Water. " *Applied and Environmental Microbiology.* July 1981.

    - Experiments evaluating bacterial survival, chlorine demand, and interference with microbiological determinations in order to assess relationships between high turbidities and drinking water chlorination efficiency.
    - Disinfection efficiency was negatively correlated with turbidity and was influenced by season, chlorine demand of the samples and the initial coliform level.
    - Scanning electron photomicrographs showed some bacteria embedded in turbidity particles or appeared to be coated with amorphous material or both. Mixing chlorinated turbid water increased the number of standard plate count bacteria, indicating the physical separation of cells attached to common particles.
    - Total organic carbon was found to be associated with turbidity and was shown to interfere with maintenance of free chlorine residual by creating a chlorine demand.
    - *Relevance to QCSOSTF: Suggests that particle shielding is possible; however, disinfection of drinking water and disinfection of wastewater may have different sensitivities due to different flow and load characteristics.*

12. Shuval, Cohen, and Kolodney. Hebrew University. 5/82.

    - Viruses cannot multiply outside the living host cell, and so regrowth cannot occur.
    - Coliform and fecal coliform are capable of regrowth in chlorinated sewage.
    - Regrowth is minimized if high chlorine residuals are maintained in the effluent.
    - *Relevance to QCSOSTF: This reference says fecal coliform can experience regrowth in chlorinated sewage. It also reaffirms that viruses cannot regrow. Extremely low chlorine residual permit limits at the QCSOSTF may reduce disinfection effectiveness.*

13. Hass, Charles N., Sheerin, John G., Lue-Hing, Cecil, Rao, K.C., O'Brien, Parnell. "Effects of Discontinuing Disinfection on a Receiving Water." *Water Pollution Control Federation.* Vol. 60, no. 5, 1988.

    - Study of receiving water quality after discontinuation of disinfection.

- Supports the concept that beyond a certain zone, and during certain times of year, chlorination of an effluent may not improve microbiological water quality.

- Fecal coliforms do not necessarily simulate microbial pathogen behavior in aquatic environments.

- *Relevance to QCSOSTF: Supports the concept of seasonal chlorination. Reaffirms that fecal coliform levels have little to do with pathogen levels.*

14. LeChevallier, Mark W., Cawthon, Cheryl D., Lee, Ramon G. "Factors Promoting Survival of Bacteria in Chlorinated Water Supplies." *Applied and Environmental Microbiology*. March 1988.

    - Study of disinfection resistance mechanisms with respect to the survival of indicator bacteria in potable water supplies.

    - The attachment of bacteria to surfaces provided the greatest increase in disinfection resistance.

    - Disinfection by free chlorine is affected by type of surface, age of biofilm, encapsulation, and nutrient effects.

    - *Relevance to QCSOSTF: Bacteria attached to surfaces are more difficult to disinfect.*

15. Hurst, Christon J. "Fate of Viruses during Wastewater Sludge Treatment Processes." *CRC Critical Reviews in Environmental Control*. Vol. 18, issue 4. 1989.

    - Overview of viruses in wastewater.

    - Viruses released into the environment are susceptible to inactivation by temperature, pH, sunlight, inorganic cations and anions, loss of moisture through evaporation, and antagonism by aerobic microorganisms and microbial products.

    - Regrowth cannot occur for human viruses in wastewater sludges.

    - Enteric viruses adsorb well onto particulate organic materials, thus removal of viruses during wastewater treatment is correlated with partitioning onto removed sludge fractions.

    - *Relevance to QCSOSTF: Viruses that adsorb onto particulate organic materials and settle will not be discharged from the facility. Viruses that do survive the treatment process are susceptible to inactivation by a number of natural processes.*

16. Narkis, Nava, Armon, Robert, Offer, Regina, Orshanksky, Frieda, Frieland, Eugenia. "Effect of Suspended Solids on Wastewater Disinfection Efficiency by Chlorine Dioxide." *Water Research*. Vol. 29, No.1, 1994.

- Study of chlorine dioxide disinfection of effluent enriched with suspended solids and survival of microorganisms after crushing of solids.
- After crushing, a fraction of indicator organisms were found intact as a result of chlorine dioxide disinfection.
- Intact fraction was able to regrow as was shown for all bacterial indicators, such as coliforms, fecal coliforms, enterococci and heterotrophic count, despite high disinfectant concentrations.
- Two factors allowed regrowth: resistant indicator organisms entrapped in suspended solids which survived disinfection, and oxidation of complex organics to lower molecular weight organics, which in turn are metabolically more accessible to the surviving bacteria.
- Study indicates that some microorganisms entrapped in suspended flocs can survive disinfection with chlorine dioxide, depending on indicator type; therefore, their prior removal by coagulation, sedimentation and filtration is a prerequisite for successful disinfection.
- *Relevance to QCSOSTF: Regrowth is attributable to organisms within suspended solids that survive disinfection and oxidation of complex organics to lower molecular weight organics, which are accessible to the surviving bacteria. Fecal coliform were able to regrow. Removal prior to disinfection is a prerequisite for successful disinfection. At the QCSOSTF, some removal is expected because of settling in upstream storage.*

17. Francy, Donna S., Hart, Teresa L., Virosteck, Cathy M. "Effects of Receiving-Water Quality and Wastewater Treatment on Injury, Survival, and Regrowth of Fecal-Indicator Bacteria and Implications for Assessment of Recreational Water Quality." *US Geological Survey, Water-Resources Investigations Report* 96-4199. 1996.

    - Field studies on bacterial injury, survival and regrowth from wastewater and CSO effluents in receiving stream or lake.
    - Samples analyzed by standard and enhanced-recovery membrane filtration methods. Standard methods support the growth of healthy organisms; enhanced-recovery supports the growth of both healthy and injured organisms.
    - In wastewater effluent, dechlorination following chlorination enhanced repair of chlorine-injured fecal coliforms and regrowth on culture media and in the lake, but not the river.
    - In CSO effluent, the percent injury indicated that dechlorination after chlorination reduced the ability of organisms to recover and regrow on culture media compared to chlorination alone.

- Dechlorination after chlorination was found to be less effective than chlorination alone in reducing the survival of fecal coliforms in wastewater effluent, but not in CSO effluent.

- Patterns of concentration increases and decreases in CSO effluents were atypical, so the effects of wastewater treatment on injury, survival, and regrowth in CSO effluents could not be determined.

- Characteristics of receiving water and the effluent affect organism response to chlorination/dechlorination.

- *Relevance to QCSOSTF: Results were inconclusive, and in some cases counterintuitive, in terms of chlorination and dechlorination of CSO effluents and the effect on bacteria injury, survival and regrowth.*

18. *Wastewater Disinfection*. WEF. 1996.

- Chlorine is not effective at disinfecting cysts of *Entamoeba histolytica* and *Giardia lamblia* and eggs of parasitic worms.

- Regrowth of bacteria after chlorination are presumed to be a result of the destruction of large numbers of protozoa by chlorination. This permits subsequent multiplication of the surviving bacteria unhampered by predatory protozoa such as the ciliates and flagellates.

- Chlorine has limitations, and so other treatment methods of treatment should be utilized for improved virus removal.

- Assumed virus concentration in raw wastewater is 7,000 / L.

- *Relevance to QCSOSTF: Given that the QCSOSTF does not provide advanced treatment, there is probably little removal of entamoeba histolytica and Giardia lamblia and eggs of parasitic worms from the facility.*

19. Tree, J.A., M.R. Adams and D.N. Lees. "Virus Inactivation during Disinfection of Wastewater by Chlorination and UV Irradiation and the Efficacy of F+ Bacteriophage as a 'Viral Indicator'." 1997.

- Disinfection was rapid for fecal coliform, not as rapid for poliovirus, and still less rapid for F+ bacteriophage.

- *Relevance to QCSOSTF: Supports idea that bacterial indicators may not accurately represent the behavior of viruses.*

20. Emerick, Robert W., Loge, Frank J., Ginn, Tim, Darby, Jeannie L. "Modeling the Inactivation of Particle-Associated Coliform Bacteria." *Water Environment Research*, Vol. 72, no. 4. 1999.

- Modeling equation derived for describing the measured inactivation of particle-associated coliform bacteria in wastewater secondary effluent exposed to UV light disinfection.

- A minimum particle size governs the ability of a particle to shield coliform bacteria from UV light. Particles smaller than that size do not contain regions shielded from UV light.

- At sizes greater than the critical particle size, size is not significant in determining shielding of coliform bacteria.

- *Relevance to QCSOSTF: The performance of UV disinfection is more sensitive to the presence of particles than chlorination processes.*

21. EPA. "Combined Sewer Overflow Fact Sheet." *Chlorine Disinfection.* 9/99.

- When breakpoint chlorination is practiced properly, the bactericidal effect is considered good and the viricidal effect is considered moderate.

- Disinfection capability is dependent on contact time.

- Suspended solids (SS) can inhibit the disinfecting agent, thus disinfection is usually used in conjunction with an additional technology that reduces SS in solution.

- Strong initial chlorine mixing is critical in high rate disinfection processes where contact times are short.

- Higher total residual chlorine (TRC) concentrations may be more effective at inactivation of viruses, spores, and cysts.

- It is recommended that TRC levels not exceed 0.2 mg/l for a period of 2 hours per day where more resistant species of fish are known to persist, or .04 mg/l for trout and salmon.

- In CSOs with low SS concentrations, pathogens are killed with a quick dose of disinfectant.

- When SS concentrations are high, the initial disinfection kills most bacteria; however, residual bacteria entrapped in solids were not found to be affected. The amount of bacteria remaining is a function of SS concentration and particle size.

- *Relevance to QCSOSTF: Supports idea that bacterial indicators may not represent viral behavior accurately. Storage at QCSOSTF enhances contact time. The QCSOSTF provides 15-minutes of contact time or more for peak instantaneous flows up to a 1-year frequency. With the recommended plan in place, the QCSOSTF will provide 40-minutes or more of contact time for peak instantaneous flows up to a 1-year* *frequency. SS in influent is probably low enough (and then it gets some additional removal in storage tanks) that chlorination works well. The QCSOSTF pumps, which pump as much flow as possible to the UBWPAD; the chlorine contact tanks, which store as much flow as possible before discharging; and the large overflow collectors, which act as a wetwell for the QCSOSTF pumps; all enable the QCSOSTF to either pump or settle out the "first flush" flows, which have higher concentrations of suspended solids and other pollutants. Therefore, only the more dilute flows will be discharged following disinfection and dechlorination. Chlorination is expected to work better on these more dilute flows. The QCSOSTF permit limit for TRC (0.02 mg/l) is exceptionally low in light of the limits recommended in this study (0.2 mg/l and 0.04 mg/l). Higher allowable TRC levels increase disinfection effectiveness.*

22. Zeghal, Slim, Bourbigot, Marie-Marguerite, Sibony, Jacques. "New Developments in Water and Wastewater Treatment." *Water Supply*, Vol. 17, nos. 3, 4. 1999.

    - Overview of new methods.
    - Studies have shown that more than a chlorine residual in distributed water is necessary for good water quality; suspended solids carry and protect bacteria, and organic matter provides the substrate for re-growth in the distribution system.
    - *Relevance to QCSOSTF: The QCSOSTF is designed to limit SS levels in the disinfected and discharged flow.*

23. Perdek, Joyce M. and Borst, Michael. "Microbial Particle Association and Combined Sewar Overflow Disinfection." *Water Environment Federation.* 2000.

    - One disinfection research project investigated the effects of particle association on measurements of microbial indicator concentrations in CSOs through mixing of samples to break up particles.
    - Both mixing time and speed affected measured indicator concentrations.
    - Measured fecal coliform (FC) and *E. coli* (EC) concentrations in blended samples were up to 10 times greater than FC and EC concentrations in untreated samples.
    - No correlations between increased indicator microorganism concentration and decreased particle size. Results suggest that indicator microorganisms are associated with the particles and removing the larger-diameter particles will lower the total indicator microorganism concentration in CSO.
    - FC and EC concentrations in all samples mixed between 0.5 and 3 min either increased or remained the same as those in unmixed samples. The EC concentration increased in the sample mixed for 10 minutes while the FC concentration decreased.

- *Relevance to QCSOSTF: Removing larger-diameter particles will lower the total microorganism concentration in CSO. This supports the idea that storage at the QCSOSTF helps reduce microorganism concentrations.*

24. Payment, Plante, and Cejka. *Canadian Journal of Microbiology.* 8/2000.

    - Removal efficiencies from large primary plant (2,000 mgd).
    - The following removal efficiencies were achieved without disinfection
        - Fecal coliform 25%
        - Fecal Strep 29%
        - *E.coli* 12%
        - *Clostridium perfringens* 51%
        - *Giardia* cysts 76%
        - *Cryptosporiduium* oocyst 27%
        - Enteric viruses 0%
    - *Relevance to QCSOSTF: Since the QCSOSTF mimics primary treatment, there is some removal of pathogens, though no enteric virus removal. Disinfection, as is provided at the QCSOSTF, would improve many of the removal efficiencies presented.*

25. Johnson, Igwe, Mitcell, Kaunelis. "Operating Experience with Large CSO Control Facilities." WEFTEC 2000.

    - Nine storage and treatment facilities in Rouge River basin.
    - TRC goal was 1 mg/l, fecal coliform limit was 400/100 ml.
    - Chlorination used sodium hypochlorite.
    - Facilities could generally meet fecal coliform limit, but usually saw 2 mg/l TRC.
    - Water quality studies are being conducted to determine the stream reach where TRC plume exceeds 1 mg/l.
    - *Relevance to QCSOSTF: It is very difficult to achieve the current TRC limit of .02 mg/l at the QCSOSTF. Maintaining appropriate TRC levels improves disinfection effectiveness.*

26. Moffa, Davis, and LaGorga. *Disinfection of CSOs.* WEFTEC 2001.

    - High rate disinfection is defined as the application of high rate mixing in combination with a chemical disinfectant to achieve disinfection within five minutes.

- *Relevance to QCSOSTF: Flow velocities through the influent gates to contact tank 1 and the baffle wall in contact tank 2 provide mixing and enhance disinfection effectiveness. Contact time at the QCSOSTF is 15 minutes or more for peak instantaneous flows up to a 1-year frequency.*

27. CDM Report. "Phase II CSO Disinfection Pilot Study Final Report." April 2001.

- Pilot study on disinfection of CSOs using UV, ozone, chlorination/ dechlorination, and chlorine dioxide.
- All technologies except E-beam, where successful in 3-4 log reductions in bacteria.
- 5 minutes contact time was sufficient for chlorination.
- TRC was generally below 0.1 mg/l after dechlorination (the detection limit) as compared to .0075 mg/l water quality standard.
- Generally suspended solids limit the exposure of embedded bacteria by shielding them from contact with the disinfectant, though chlorine has the ability to penetrate suspended solids.
- Study showed no apparent trend between chlorine disinfection effectiveness and suspended solids, in samples following grinding of particles. This is particularly evident at a dose of 24mg/l for TSS concentration ranging from 200 to 500mg/l.
- *Relevance to QCSOSTF: TSS concentrations in influent to CSOSTF generally much lower than 200 to 500 mg/l. Thus, "shielding" may not be a significant problem.*

28. Ormeci, Banu and Linden, Karl G. "Comparative Effectiveness of UV and Chlorine for Inactivation of Particle Associated Coliform." Water Environment Federation. 2002.

- Study to compare effectiveness of UV and free chlorine on naturally-occurring particle-associated coliform (PAC) and non-particle-associated coliform (NPAC) in wastewater.
- Secondary effluent was used in both NPAC and PAC experiments.
- Microorganisms attached to surfaces or associated with particles are more resistant to chlorine and UV disinfection.
- In water quality characterized by a high concentration of particulate matter, microorganisms may be associated with particles and shielded from the disinfectant.

- Total disinfection of PAC could be achieved only when the contact time was 45 minutes and the initial chlorine concentrations were relatively high (10 and 15 mg/L).

- Contact time plays an important role in effectiveness of chlorine disinfection in wastewater. Since wastewater flocs have a porous structure, chlorine may be able to reach the protected PAC given enough time. Since the inactivation of PAC does not take place immediately, it is important to assure that the initial chlorine concentration in wastewater is high enough to cover the chlorine demand of the wastewater and provide enough residual chlorine for the disinfection of PAC.

- Particle associated coliform can survive UV and chlorine disinfection at doses that are typically encountered in a wastewater treatment plant.

- *Relevance to QCSOSTF: Contact time and appropriate dosing play an important role in disinfection effectiveness. With the recommended plan in place, the QCSOSTF will provide 40 minutes or more of contact time for peak instantaneous flows up to a 1-year frequency. Dosing is fine-tuned by QCSOSTF staff following storm events as needed.*

A.1.4 Other Related Publications:

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