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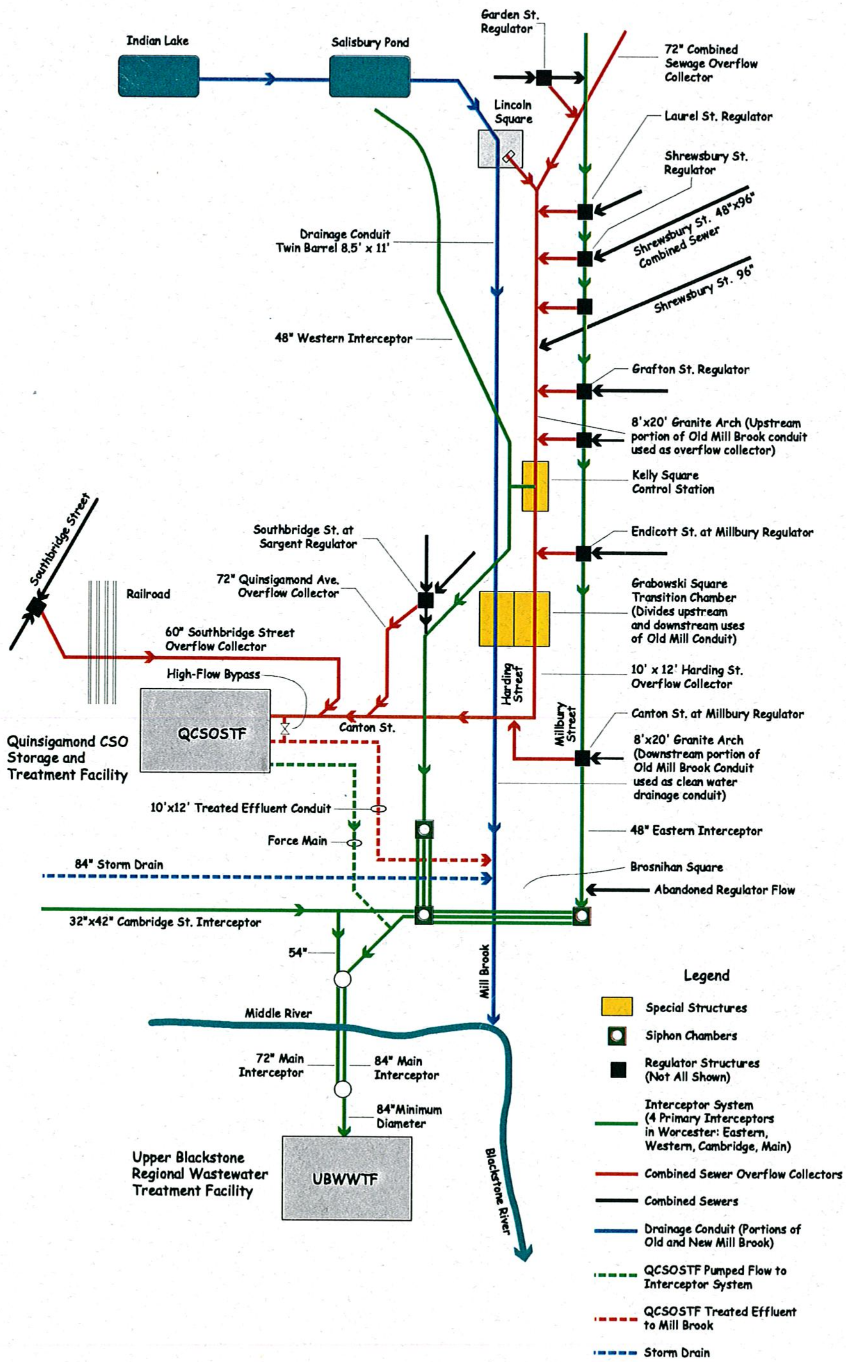
Department of Public Works

Phase I CSO Long-term Control Plan Report

March 2002



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Executive Summary

Introduction

This report presents Phase I findings of the City of Worcester's Long-term Control Plan (LTCP) for mitigating 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 18th, 2000. The consent order required the city to prepare a two-phased LTCP.

Phase I of the LTCP identifies feasible CSO control alternatives. Phase II will select a control plan based on these alternatives.

This executive summary summarizes this report's Phase I CSO Long-term Control Plan (LTCP), presenting 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;
- Regulations affecting the CSS; and
- Phase II of the LTCP.

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 2002 dollars of over $81 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 ES-1 and ES-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 level of performance from their CSS control facilities. Table ES-1 compares CSS performance before and after existing CSS facilities were built.

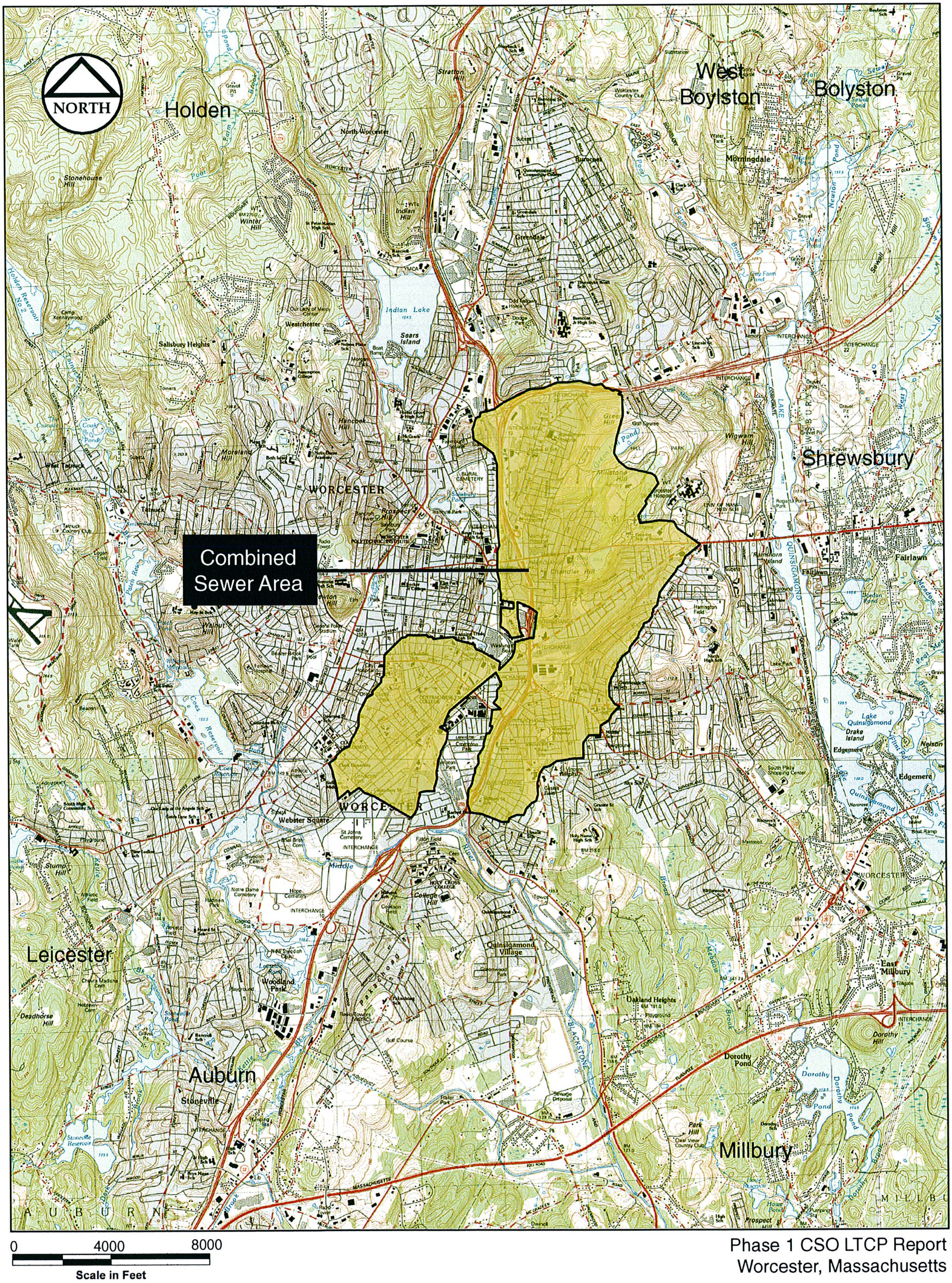

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Figure ES-1
Worcester's Combined Sewer Area

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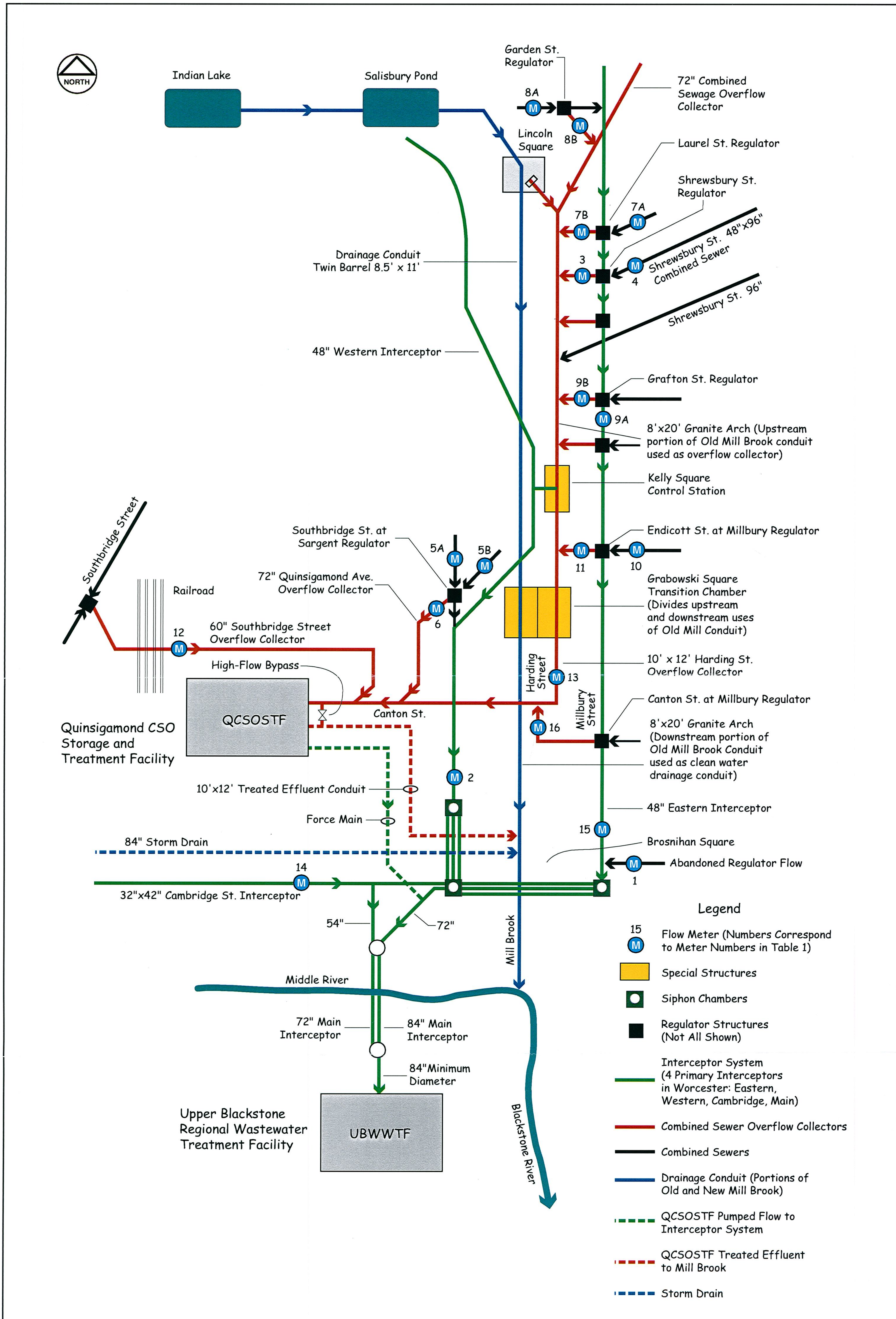

Not to Scale

Figure ES-2
Schematic of the Combined Sewer System

Table ES-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
Estimated Annual *Untreated* Overflow Volume (Million Gallons)	1,300	0
Estimated Annual *Treated* Volume (Million Gallons)		
Secondary Treatment at UBWWTF (Million Gallons)	0	1,218
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. Under these conditions, all flows reaching the QCSOSTF are pumped for treatment at the UBWWTF. Over 75 percent of rainfall events are 100-percent treated at the UBWWTF. Rarely does the QCSOSTF discharge treated effluent directly to the Blackstone River.

When the QCSOSTF discharges, fecal coliform are typically reduced to below the detection limit, and BOD and TSS are typically reduced by 34 percent and 30 percent, respectively. During high flow periods, UBWWTF primary facilities also typically reduce fecal coliform to below the detection limit, and 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.

The QCSOSTF had no dechlorination facilities. Without them it violated its total residual chlorine NPDES permit limit, so the city constructed dechlorination facilities, which are now in start-up phase. Once the start-up phase is complete, the city expects to fully meet all limits in its permit.

Worcester's CSS area covers four square-miles. Effluent from the system rarely discharges directly to the Blackstone River. When it does, it is treated at the QCSOSTF. Pollutant loads 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.

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 are being designed, and will be constructed over the next 13 years. The UBWWTF will be upgraded to handle larger future flows at higher treatment levels. This upgrade also will accommodate high flows during storm events.

The planned upgrades will improve CSS performance. The most significant improvement will be the operational protocol for pumping from the QCSOSTF to the UBWWTF. Currently, the QCSOSTF can only pump to the UBWWTF when there is excess capacity at the UBWWTF. When flow exceeds 54 to 70 mgd into the UBWWTF, pumping from QCSOSTF typically ceases. With UBWWTF expansion, pumping will continue until flow at UBWWTF reaches 140 mgd.

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 receive full treatment 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 rare QCSOSTF bypasses. Because of the improvements, it will take approximately a 15-year event 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.

Additional Alternatives to Mitigate CSS Impacts

As required for this LTCP, a full range of CSO control alternatives were evaluated. 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. This can be done by: treating even more flow at the UBWWTF; increasing the treatment level of QCSOSTF discharges; storing flow; or by separating sewers.

A screening analysis (Section 8) was conducted to screen the full range of improvements for further evaluation during Phase I. Because CSO control facilities already exist, many potential improvements are 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 ES-2 presents the screening analysis results, showing technologies not feasible or appropriate, already in place, and for further consideration in Phase I.

Technologies for further consideration were evaluated in detail (Section 9). The improvements that would result are:

A. To Optimize Hydrologic Response of the CSS

- Sewer separation (complete separation of the entire CSS and partial separation of the Shrewsbury Street area); and
- Diversion of Green Hill Pond and Bell Pond out of the CSS.

B. Designed Primarily to Store Flows

- Rehabilitation of the Kelly Square gates to maximize storage in the overflow collector upstream of Kelly Square;
- Installation of a gate structure in the Harding Street Overflow Collector to maximize storage upstream of the slope break between the QCSOSTF and Grabowski Square;
- Raising of weirs to increase inline storage;
- Real Time Control; and
- Increase in storage capacity at the QCSOSTF.

C. To Enhance System Conveyance

- Increase pumping capacity at the QCSOSTF; and
- Divert more flow into the Western Interceptor.

D. For Better Treatment

- High rate clarification (HRC) at the QCSOSTF.

The evaluations considered cost and pollution removal effectiveness of reduced discharge from the QCSOSTF, and reduced BOD and TSS loadings. Based on their effectiveness, many of the lower cost improvements were combined. Alternative 1 includes Green Hill Pond diversion, regulator modifications, rehabilitation of the Kelly Square control structure, and diversion of flows to the Western Interceptor. Alternative 2 includes all Alternative 1 improvements as well as more pumps at the QCSOSTF. Figure ES-3 presents estimated costs of potential improvements. Table ES-3 shows each alternative evaluated and its cost, water-quality benefit from reduced

Table ES-2
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	

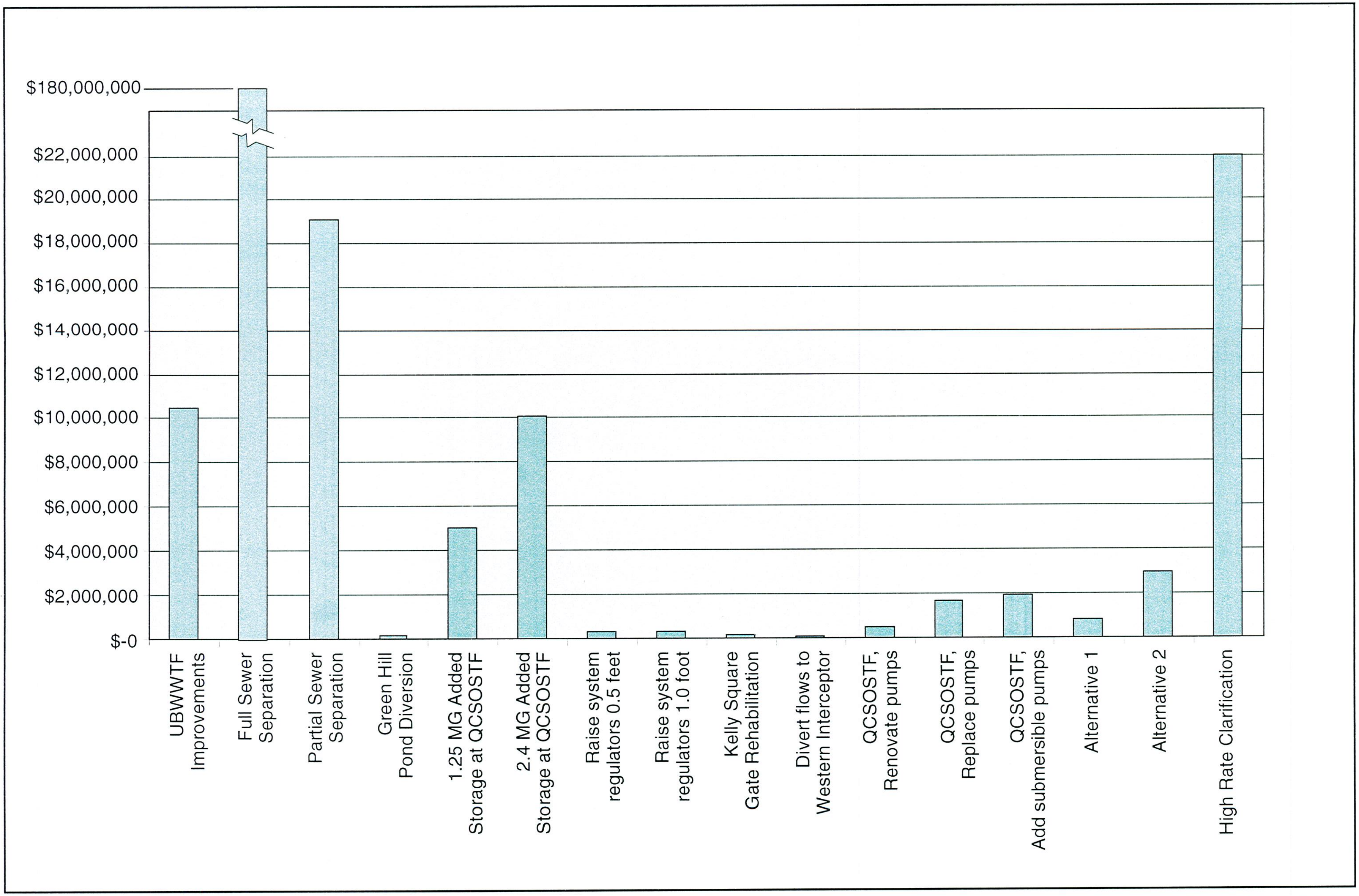

Figure ES-3
Estimated Costs of CSO Control Alternatives

Table ES-3
Summary of Potential CSO Control Improvements

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
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 overflow from the QCSOSTF, and whether it will be evaluated further in Phase II. These alternatives all are built on and include the UBWWTF improvements.

Findings from the evaluation include:

- Potential sewer separation improvements (complete sewer separation and separation of the Shrewsbury Street area) will not be considered further because of their cost and their potential for degrading water quality; and

- Additional storage at the QCSOSTF and high rate clarification (HRC) will not be considered further because of their cost and because other potential improvements can as effectively or more effectively reduce CSS impacts.

Phase II will consider all remaining potential improvements. Alternatives 1 and 2 are particularly effective at reducing CSS impacts and show the most promise as the LTCP cornerstone. With Alternative 1, compared with baseline conditions treated discharges from the QCSOSTF will be reduced from 14 to 5 and annual volume from the QCSOSTF from 83 MG to 27 MG; 99 percent of combined sewage will be treated at the UBWWTF. With Alternative 2, there would be only two treated discharges and 17 MG of discharge annually from the QCSOSTF.

The Financial Impacts of Potential CSS Improvements

Paying for UBWWTF improvements and other planned capital improvement programs will cause a large increase in typical household sewer bills. The average annual household bill is projected to increase from $171 in 2002 to $409 in 2012, a 139 percent increase. These projected rate hikes do not include the cost of new facilities for Worcester's combined sewer system.

Any improvement therefore to the CSS will be a major financial and political challenge. To tolerate any increase, ratepayers will at least need to be convinced their money is well spent and results in significant environmental improvements.

The most costly potential improvement evaluated is complete sewer separation, at an estimated cost of $180M. This alternative, which is likely to have negative water-quality impacts, will raise the average annual 2012 household bill to $671, an almost four-fold increase above current rates. Figure ES-4 shows expected annual household sewer bills with and without sewer separation.

Other potential improvements have quantifiable environmental benefits at more modest cost. As shown in Table ES-3, Alternatives 1 and 2 are estimated to cost $0.9M and $2.8M, respectively. These alternatives will result in much lower rate increases to Worcester ratepayers. Alternative 2 will add about $3 annually to average household sewer bills, with an average in 2012 of $412. The average bill with other planned improvements, including UBWWTF improvements, will be $409.

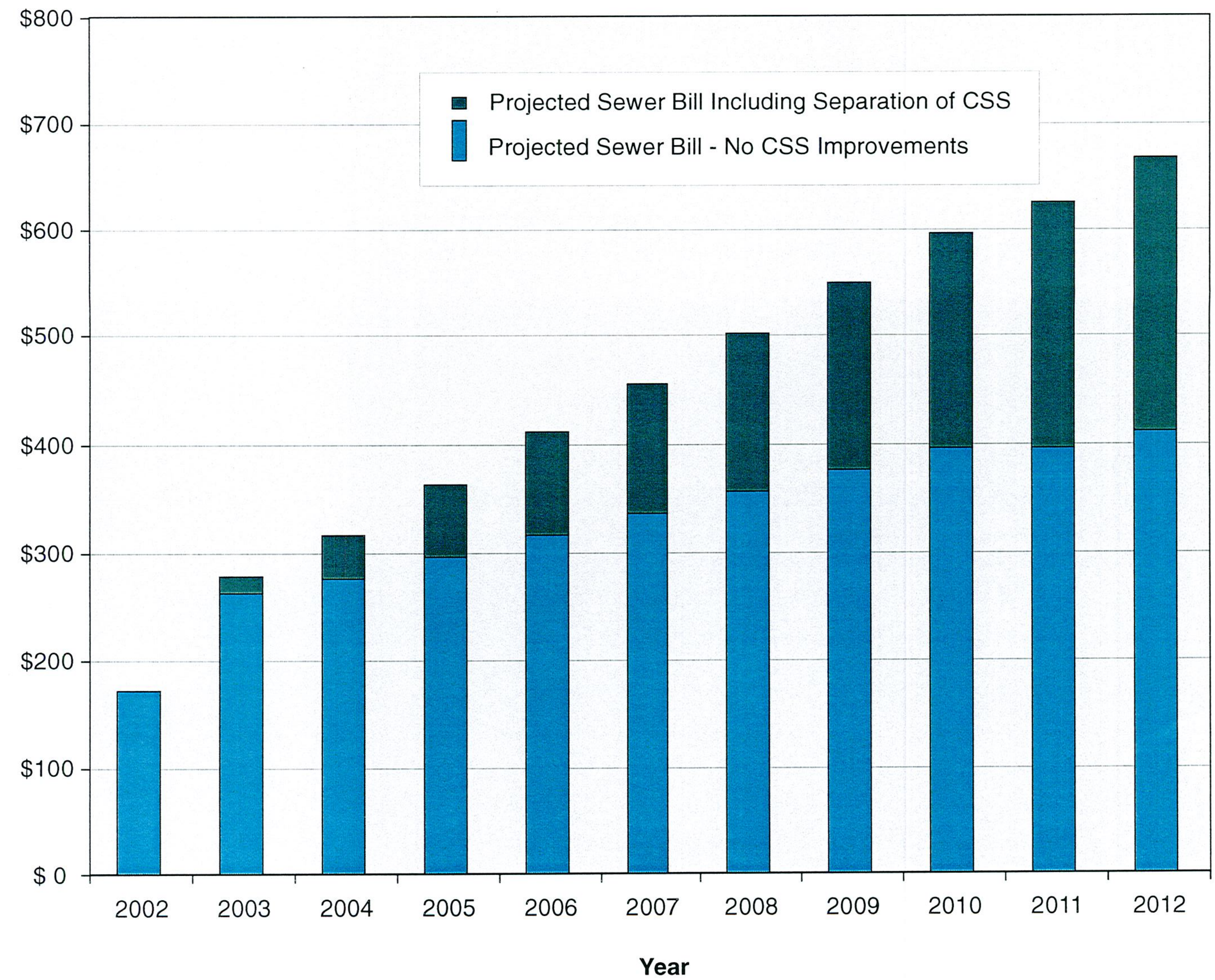

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Figure ES-4
Impact of Sewer Separation on Annual Sewer Bills

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Regulations Affecting the CSS

Massachusetts's regulatory options for CSO control are implemented through different water body classifications, including Class B, Class B (CSO), a variance, a partial use designation, and Class C. Of special interest are the Class B and Class B (CSO) designations:

- Class B - CSOs must be eliminated either by relocation of the outfall or by separation. It is not feasible to relocate the QCSOSTF discharge, since no alternative water bodies are better for the discharge. A water body with CSO discharges cannot have a Class B designation.

- Class B (CSO) - CSOs may remain but must be compatible with goals of the receiving water. The water body must meet uses more than 95 percent of the time. The DEP considers four overflow events per year as satisfying the 95 percent time period. The intent of this designation is to allow up to four untreated discharges per year. Because of past efforts to control CSOs, there are now no untreated discharges from the combined sewer system in a typical five-year period. With UBWWTF improvements, seven treated discharges per year are expected. Potential CSS improvements that will be further investigated in Phase II will reduce this number even more.

When the CSO Policy was drafted, it focused on typical CSSs with untreated overflows. Worcester's CSS is not typical. Its facilities, which pre-date the policy, have only treated overflows. Whether the CSO policy precludes treated CSO discharges, like from the QCSOSTF, is subject to interpretation. This Phase I LTCP demonstrates that:

- Discharges from Worcester's combined sewer system receive very high levels of treatment, either at the UBWWTF or at the QCSOSTF;

- Planned improvements at the UBWWTF and that will be implemented for this LTCP will further reduce CSS impact on the Blackstone River; and

- The city's CSS facilities will be fully functional and operational even during extreme conditions (15-year flood events).

The city contends that given the very high level of treatment of discharges from its CSS and the CSS's ability to function even during extreme conditions, the discharge from the QCSOSTF should not be considered an (untreated) CSO discharge. If untreated CSO discharges and treated CSO discharges are distinguished, the Blackstone River can remain a Class B river.

The alternative is to decommission all CSS facilities and completely separate the entire CSS. At an estimated cost of $180M, this would be a huge financial burden on ratepayers with little or no water-quality benefit, and may even increase pollutant loadings to the Blackstone River.

Phase II of the LTCP

Phase II will investigate the promising alternatives from Phase I to ensure they function satisfactorily and without unintended consequences. The following alternatives will be evaluated 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; and
- Additional pumping capacity at the QCSOSTF.

The LTCP will be explained and subject to modification through a public participation process. After Phase II, the plan will be implemented and the impact of Worcester's CSS on the Blackstone River further mitigated.

Section 1
Introduction

1.1 Background

This report presents the findings of Phase I of the City of Worcester's Long-term Control Plan (LTCP) for mitigating the water quality impacts of combined sewer overflows (CSOs) from its combined sewer system (CSS).

The report was prepared to comply with an administrative consent order signed by EPA and the City of Worcester on September 18, 2000. The consent order required that the City prepare a two-phased LTCP.

This is Phase I of the LTCP. Phase I identifies a range of feasible CSO control alternatives. Phase II will select a control plan based on consideration of the feasible CSO control alternatives.

1.2 The Current State of Worcester's CSS

Worcester is in a unique position because it has already undertaken and completed a facilities planning process for the control of CSOs. More importantly, the City has completed all construction recommended in its facilities plan. The facilities plan was completed in 1975. Construction was completed, and all facilities have been on-line since 1989. This LTCP seeks to improve an already well-designed and functional CSO control system.

The Worcester sewer and drainage system is predominantly a "separated" system. There are typically two pipes in each street, storm drains and sanitary sewers. The catch basins are connected to storm drains and carry stormwater runoff to receiving streams and lakes. The sanitary sewers carry sewage from households, businesses, and institutional uses throughout the City to the Upper Blackstone Water Pollution Abatement District's Wastewater Treatment Facility (UBWWTF).

Fifteen percent of the City (about 4 square miles) is served by a CSS. Figure 1-1 shows the extent of the CSS in Worcester. Combined sewers carry sanitary flows during dry weather and a combination of sanitary flows and stormwater during wet weather. Before improvements to the CSS were constructed, untreated CSO discharges, even during dry weather, discharged to the Blackstone River via the Old Mill Brook.

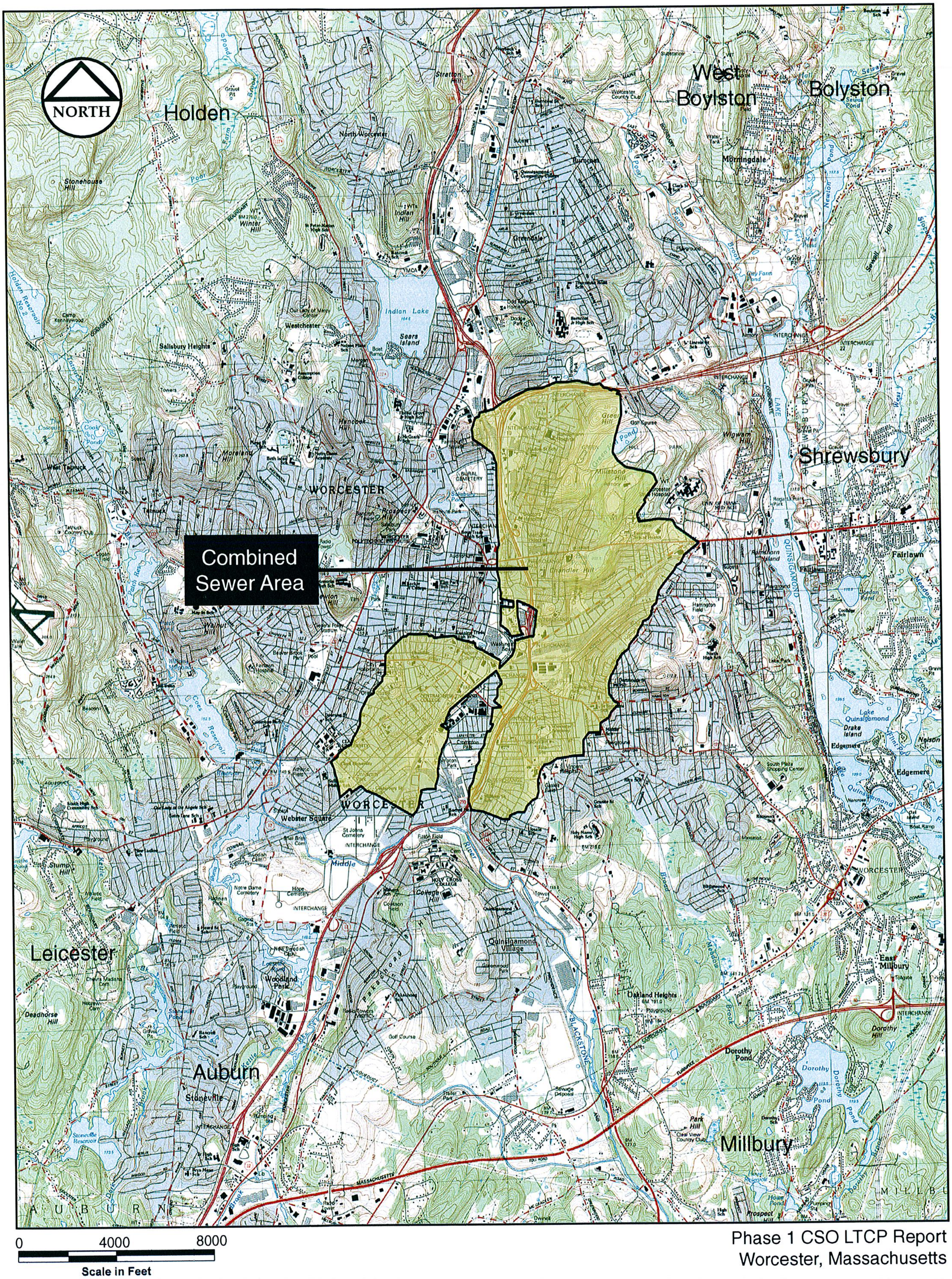

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Figure 1-1
Worcester's Combined Sewer Area

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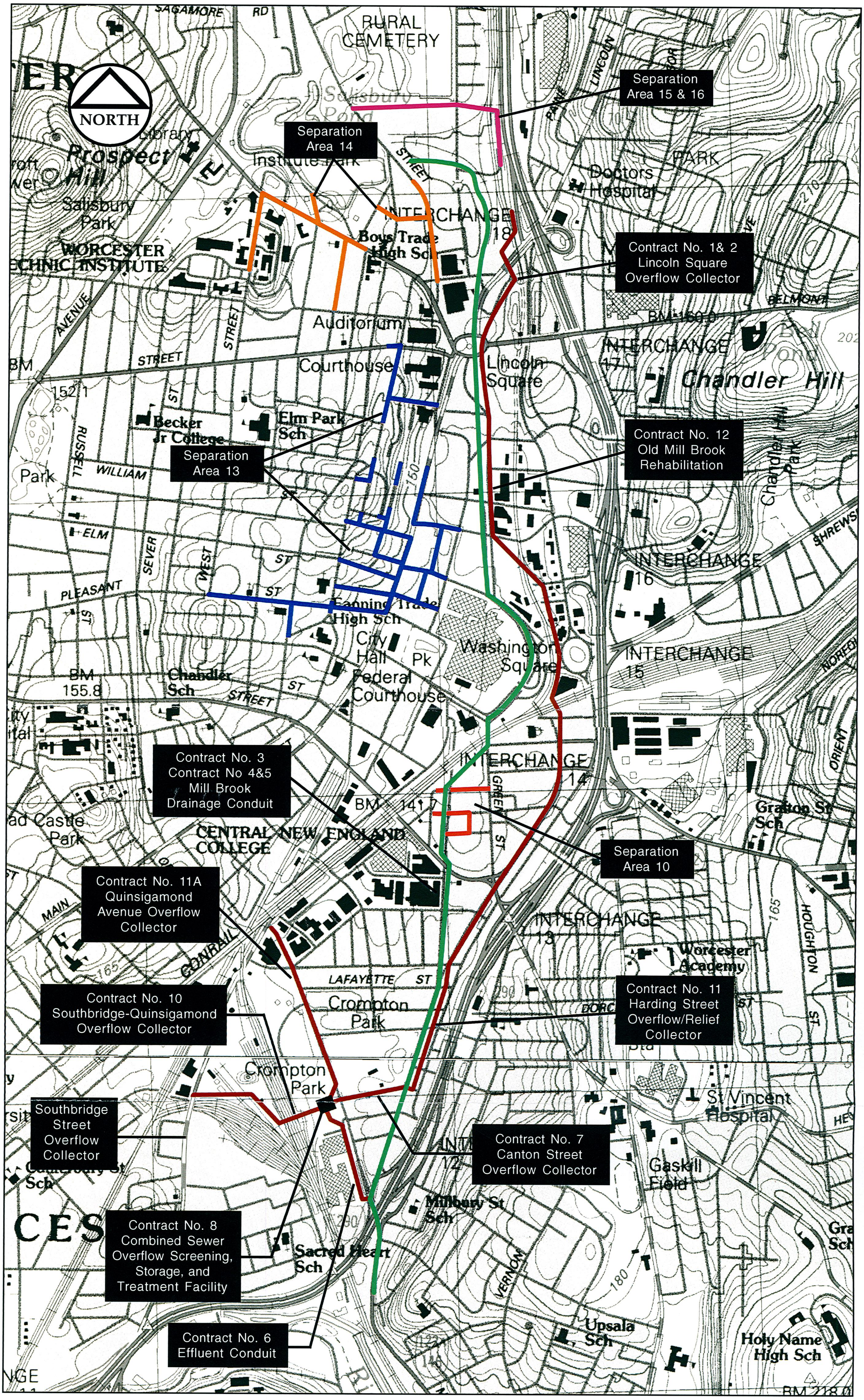
As a result of the facilities planning process, the following facilities, as shown in Figure 1-2, were constructed. The total cost associated with these facilities is estimated to exceed $81M (2002 dollars).

- Sewer separation of 0.5 square miles of formerly combined sewers, reducing the size of the combined sewer system by 11 percent from 4.5 square miles to 4 square miles;
- Harding Street overflow collector: 10-foot by 12-foot box culvert, 2,000 feet long;
- Canton Street overflow collector: 36-inch diameter pipe, 1,100 feet long;
- Quinsigamond Avenue overflow collector: 48-inch pipe, 1,500 feet long;
- Lincoln Square overflow collector: 72-inch pipe, 2,000 feet long;
- Rehabilitation of the old 8-foot by 20-foot granite arch Mill Brook conduit as an overflow collector upstream of Grabowski Square;
- New Mill Brook clean water conduit: 8,000-feet of twin barrel 8.5-feet by 11-feet, to carry stormwater from upstream separated areas through the combined sewer system to the Blackstone River; and
- Quinsigamond Combined Sewer Overflow Storage and Treatment Facility (QCSOSTF). The only remaining CSO discharge point in the CSS is the treated discharge from the QCSOSTF.

The previous facilities planning effort accomplished the following goals, which are recognized as important control goals by the permitting authorities:

- There are no dry weather flows from the Worcester CSS; and
- In a typical 5-year period:
  - 100% of the flow from the CSS is treated. The majority (94%) of this flow is treated at the UBWWTF, the remaining flow is treated at the QCSOSTF; and
  - Over 75% of rainfall events are fully treated at the UBWWTF.

Few communities in Massachusetts or the nation have achieved this level of performance from their CSO control facilities. In fact, most communities are only now (25 years later than Worcester) conducting facilities plans and contemplating construction of CSO control facilities. Indeed, in most communities, the level of CSO control anticipated after full implementation of their LTCPs is less than what Worcester already has in place.



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Scale in Feet

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Figure 1-2
Worcester's CSO Control Facilities

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1.3 Purpose and Approach

The purpose of Phase I is to identify a range of feasible control alternatives that provide an even greater level of protection to Worcester's receiving waters from the impacts of the Worcester CSS. The approach is similar to the recommended approach as identified in EPA's Long-term Control Plan Guidance Document, 1995, and includes the following elements:

- Documentation of implementation of "Nine Minimum Controls";
- Characterization of the combined sewer system;
- Establishment of baseline conditions for CSO and non-CSO loads;
- Development of costs for a range of CSO alternatives and development of a preliminary affordability analysis;
- Assessment of the pollution removal effectiveness from the QCSOSTF;
- Development of a prioritized list of water resources within the City;
- Development of a public participation plan; and
- Development of a Phase II scope and schedule.

Three reports have already been produced to address particular project elements. The first report was in response to EPA's CSO Control Policy. EPA issued the policy in April 1994. The policy was incorporated into the Clean Water Act in 2001. The two key objectives of the policy are implementation of Nine Minimum Control (NMC) measures, and development and implementation of a LTCP. NMC measures represent low cost actions or measures that can help to reduce CSO pollutant discharges and their effect on receiving water quality. Worcester's "Nine Minimum Control Measures " report was published in May 2001. The report documents the City's compliance with NMC requirements.

The second and third reports were produced as part of the characterization of the combined sewer system. The "Draft Metering Summary" report, September 2001 documents the results from the metering program conducted in April, May, and June 2001 to document how the combined sewer system responds during dry weather and during rainfall events. The "SWMM Model Calibration" report, December 2001, presents the computer model used to simulate the performance of the combined sewer system, and its calibration to dry and wet weather events.

This Phase I report focuses on evaluation of alternatives. The remaining sections of this report cover the following topics:

- Section 2 - USEPA and Massachusetts CSO Control Policy;

- Section 3 - River Uses and Water Quality;
- Section 4 - Sewer System Characterization;
- Section 5 - Sewer System Model;
- Section 6 - Precipitation Record and Design Storms;
- Section 7 - Baseline Conditions;
- Section 8 - Assessment of CSO Abatement Technologies and Initial Screening of Alternatives;
- Section 9 - Development and Evaluation of Alternatives; and
- Section 10 – Financial Impacts of Proposed System Improvements.

Section 2
CSO Control Policy

2.1 Background

For the first two decades of implementation of the federal Clean Water Act (CWA), the focus of regulatory agencies was generally the cleanup of continuous waste discharges. Publicly owned treatment works (POTWs) were generally required. Industrial dischargers were required to install best available treatment for their discharges. Although CSOs were known to exist, and were known in some instances to compromise local uses of receiving waters, control was deferred while the larger problems were addressed.

Part of the reason that CSO control was not a high priority was that in many instances their intermittent effects were overshadowed by the continuous discharge of inadequately treated wastewater from point sources. With elimination of inadequately treated continuous discharges, intermittent wet weather discharges - such as CSO's and stormwater - have been brought into the limelight.

Working through CSO solutions has pointed out several key issues:

- CSOs are expensive to eliminate or abate;

- Depending on site-specific circumstances, improvements in water quality from CSO mitigation can be significant but are often marginal. In Worcester, CSS improvements, especially elimination of continuous discharges during dry weather, undoubtedly improved water quality in the Blackstone River. Given the high level of control from existing CSS facilities, the need for even higher levels of control needs to be carefully documented to ensure they improve water quality in a cost effective manner; and

- Given the small size of the CSS compared with the total drainage area (4 square miles vs. 61.5 square miles at the confluence of the Blackstone and Middle Rivers) further improvements to water quality from the CSS may represent a fairly insignificant reduction in loadings and improvement to Blackstone River water quality.

Over the last several years, there has been a search for a cohesive policy that tries to tie the environmental benefits associated with CSO control and the costs to achieve these benefits. This section discusses important points of current EPA and Massachusetts' policy, and highlights applicability to Worcester.

2.2 USEPA CSO Policy

CSOs are point source discharges to the waters of the United States and subject to the CWA and the National Pollutant Discharge Elimination System (NPDES). Like all other discharges, CSOs are subject to both the technology-based and water quality based requirements in the CWA. The CSO Control Policy, issued in April 1994, provides the EPA guidance for controlling CSOs. The CSO Control Policy was incorporated into the CWA in 2001, and therefore the policy has the force of law. There are two key objectives to the CSO Control Policy: (1) implementation of Nine Minimum Control measures, and (2) development and implementation of a CSO Long-term Control Plan (LTCP).

2.2.1 Nine Minimum Controls

The minimum technology-based controls are the nine minimum controls (NMCs). The CSO Control Policy requires all communities to implement the NMCs. The nine minimum controls are:

1. Monitoring to effectively characterize CSO impacts and the efficacy of CSO controls;
2. Proper operation and regular maintenance programs for the sewer system and the CSOs;
3. Maximum use of the collection system for storage;
4. Review and modification of pretreatment requirements to assure CSO impacts are minimized;
5. Maximization of flow to the POTW for treatment;
6. Prohibition of CSOs during dry weather;
7. Control of solid and floatable materials in CSOs;
8. Pollution prevention programs; and
9. Public notification of CSO occurrences and impacts.

Selection and implementation of actual control measures is based on site-specific considerations including the specific characteristics of the sewer system.

Worcester's compliance with each of the above controls was detailed in a previous report titled "Worcester, Massachusetts Department of Public Works, Nine Minimum Control Measures" prepared by CDM in May 2001.

2.2.2 Long-term Control Plans

The NPDES entity (EPA Region 1 in the case of Worcester) determines whether the NMCs satisfy the technology-based requirements of the CWA. If further controls are necessary to meet water quality standards, then the NPDES authority will require the development of a LTCP. On September 18, 2000, EPA Region I issued a Consent Order to Worcester to prepare a LCTP.

By the requirements in the Clean Water Act, CSO discharges that remain after implementation of the CSO controls must not interfere with the attainment of a State's water quality standards. Under the CSO Control Policy, communities with combined sewer systems are expected to develop LTCPs to provide for attainment of water quality.

The EPA CSO Control Policy presents two alternatives to selecting LTCPs for CSO's: the "presumption approach" and the "demonstration approach".

Presumptive Approach

The "presumptive approach" is based on the presumption that achievement of certain performance criteria will be sufficient to meet currently applicable water quality standards. The presumption approach involves meeting one of the following three criteria:

- No more than an average of 4 untreated overflow events per year. The presumption approach allows untreated overflow events. However, there are no untreated overflow events from Worcester's CSS;

- Elimination or the capture for treatment of no less than 85 percent by volume of the combined sewage collected in the combined sewer system during precipitation events on a system-wide annual average basis. Currently, Worcester captures and treats 100 percent by volume annually. Ninety-four percent receives full secondary treatment, while the remainder receives treatment at the QCSOSTF. This is accomplished through the extensive CSO abatement facilities constructed in the 1980s. The projects included 312 acres of sewer separation, construction of five overflow collectors ranging in size from a 36-inch pipe to a 10-foot by 12-foot box culvert, and construction of the Quinsigamond CSO Storage and Treatment Facility (QCSOSTF). The investment in these projects has allowed the City to treat the majority of combined sewage at the UBWWTF, with the remainder treated at the QCSOSTF; and

- Elimination or removal of no less than the mass of pollutants identified as causing water quality standards impairment. The permit for the current QCSOSTF has limits for fecal coliform bacteria (seasonal) and total residual chlorine (TRC). Fecal coliform has generally been eliminated from the discharge. There were numerous violations of TRC limits. Therefore, the City installed and is testing new dechlorination facilities. The testing is designed to establish proper dosing of the effluent for effective disinfection and dechlorination. During this period, which is on-going, permit limits for fecal coliform bacteria and TRC may be exceeded periodically.

Once the fine-tuning of the dechlorination facilities is completed, Worcester will meet all Clean Water Act presumption approach criteria, without additional CSO controls.

Demonstration Approach

The demonstration approach is intended through a technical and financial analysis to identify the highest level of CSO control that is feasible, and to ultimately demonstrate (by supporting a Use Attainability Analysis if necessary) that the level of control proposed complies with the state water quality standards and the Clean Water Act. Under the demonstration approach, communities collect and present data in the LTCP that is sufficient to show that the proposed control alternative is adequate to meet appropriate water quality standards. The level of control may be more or less than that required under the presumptive approach. The CSO Control Policy lays out four criteria for successful use of the "demonstration approach." A LTCP should show that:

- A CSO control program will protect water quality standards unless the standard can not be met as a result of natural conditions or other pollution sources;
- Overflows remaining after implementation of the control program will not prevent the attainment of water quality standards;
- The planned control program will achieve the maximum pollution reduction benefits reasonably attainable; and
- The planned control program is designed to allow cost effective expansion or cost effective retrofitting if additional controls are subsequently determined to be necessary to meet water quality standards.

Where water quality standards cannot be met because of natural conditions or other pollution sources, a total maximum daily load (TMDL) or other means should be used to apportion pollutant loads within the watershed.

Regardless of whether the presumption or demonstration approach is used, the CSO control program ultimately selected must be sufficient to meet water quality standards and other CWA requirements.

2.3 Massachusetts Policy for Abatement of CSOs

In August of 1997 the Commonwealth issued its own CSO Control policy. This policy is similar to the EPA policy in many ways, but also has several significant differences.

It is important to understand that the states are required to develop water quality standards applicable to their water bodies. While EPA reviews and approves - or disapproves - these standards, the establishment of the standards is the responsibility of the state. In Massachusetts, any NPDES permit for a CSO discharge must comply with Massachusetts Surface Water Quality Standards (314 CMR 4.00). Massachusetts has chosen to designate all waters in the state as fishable and swimmable. For freshwater, all water bodies are either Class A (drinking water source) or Class B (swimmable). The Blackstone River and its tributaries are all Class B waterbodies.

Massachusetts' regulatory options for CSO control are implemented through different water body classifications, as follows:

- <u>Class B</u> - CSOs must be eliminated either by relocation of the outfall or by separation. It is not feasible to relocate the QCSOSTF discharge, since there are no alternative water bodies more desirable for the discharge. A water body with CSO discharges cannot have a Class B designation.

When the CSO policy was drafted, it focused on typical CSS with untreated overflows. Worcester's CSS is not typical. Its facilities, which pre-date the policy, have only treated overflows. It is unclear if the policy is intended to prohibit treated discharges from a Class B waterway. This Phase I LTCP demonstrates that:

- Discharges from Worcester's combined sewer system receive very high levels of treatment, either at the UBWWTF or at the QCSOSTF; and
- The City's CSS facilities will be fully functional and operational even during extreme conditions (15-year flood events).

The City contends that given this very high level of treatment of discharges from its combined sewer system, even during extreme conditions, the discharge from the QCSOSTF should not be considered an (untreated) CSO discharge.

The alternative is to decommission all CSS facilities and completely separate the entire CSS. At an estimated cost of $180M, this would be a huge financial burden with little or no water quality benefits, and may even increase pollutant loadings to the Blackstone River.

- <u>Class B (CSO)</u> - CSOs may remain but must be compatible with water quality goals of the receiving water. The water body must meet uses more than 95 percent of the time. DEP considers 4 overflow events per year as satisfying the 95 percent time period. The intent of this designation is to allow up to 4 untreated discharges per year. Because of past efforts to control CSOs, there are currently no untreated discharges from the combined sewer system.

- <u>Variance</u> - CSOs may remain under a short-term modification of water quality standards. Variances are typically used to gather more data to determine if CSOs are the cause for water quality violations. For example, there is currently a variance on the Charles River. For the term of the variance, MWRA is conducting more studies to identify the causes of Charles River impairment, with the ultimate goal of determining the appropriate classification for the Charles River.

- <u>Partial Use Designation</u> - CSOs may remain with moderate impacts resulting in impairment of water quality goals. Moderate impacts are defined as short-term impairments and water quality standards would be met 75 percent of the time.

- Class C - Where the State is certain that the CSOs will prevent the attainment of national use goals more than 75 percent of the time, the water body is classified as Class C.

If Class B is not attainable, permittees must go through a number of technical and procedural steps to permanently reclassify the receiving water, or to provide temporary modifications to the classification. The steps associated with this process are included in Figure 2-1. The procedural steps involve the notice of proposed changes in the Environmental Monitor, and the conduct of various public meetings and hearings and the official publication of reclassification on the State's Water Quality Standards Regulations.

Underlying these procedural steps are supporting technical analyses that show that fully achieving the designated Class B uses everywhere, all the time is not attainable. These studies are generally called Use Attainability Analyses. In order to permanently reclassify the receiving waters, the Use Attainability Analyses must show that one of the following conditions exist:

1. Human caused conditions or sources of pollution prevent the attainment of the use and cannot be remedied or would cause more environmental damage to correct than to leave in place; or

2. Dams, diversions or other types of hydrologic modifications preclude the attainment of the use, and it is not feasible to restore the water body to its original condition or to operate such modification in a way that would result in the attainment of the use; or

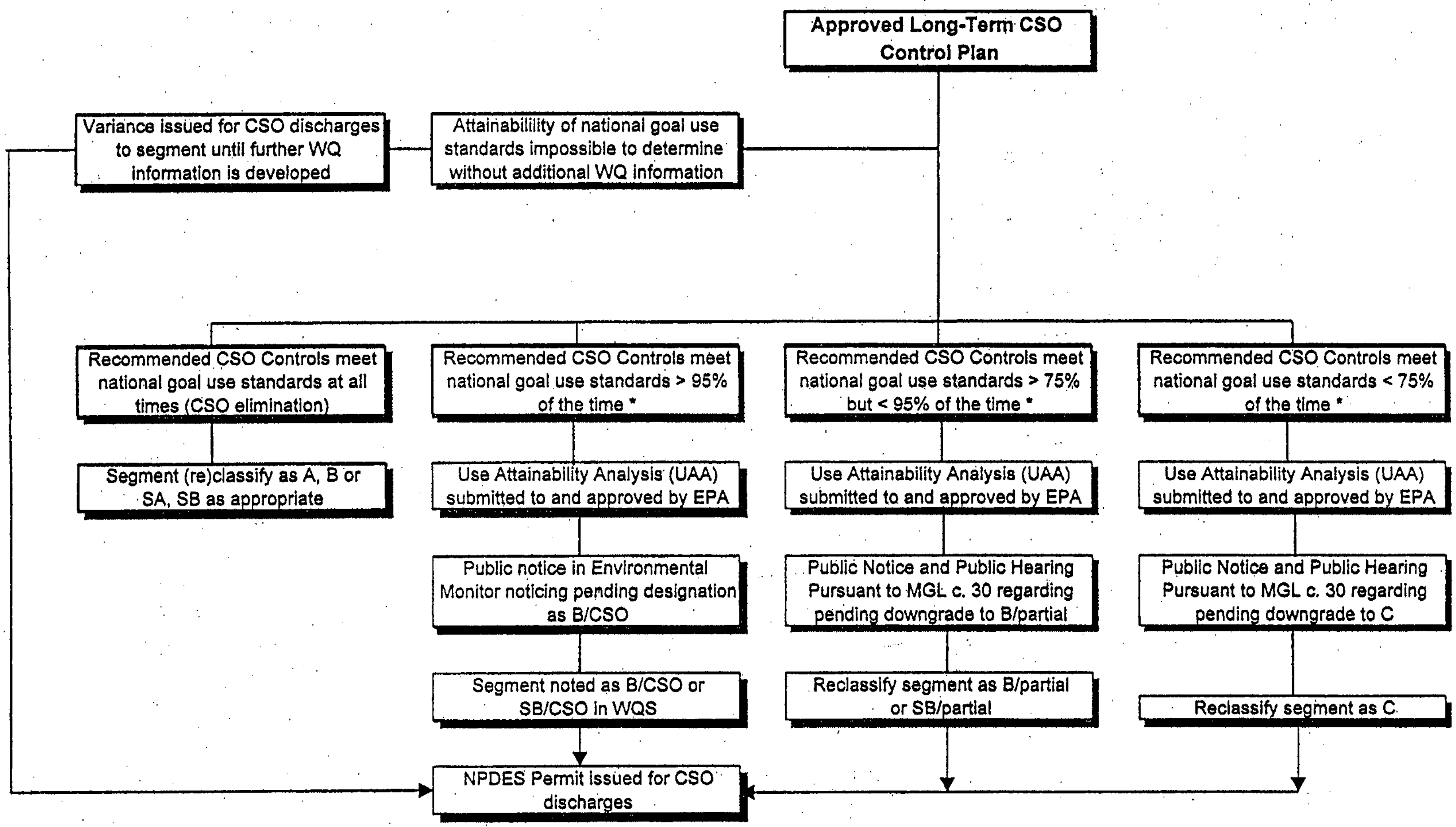
3. Naturally occurring pollutant concentrations prevent the attainment of the use; or

4. Natural, ephemeral, intermittent or low flow conditions or water levels prevent the attainment of the use, unless these conditions may be compensated for by the discharge of sufficient volume of effluent discharges without violating state water conservation requirements to enable uses to be met; or

5. Physical conditions related to the natural features of the water body, such as the lack of a proper substrate, cover, flow, depth, pools, riffles, and the like, unrelated to water quality, preclude attainment of aquatic life protection uses; or

6. Controls more stringent than those required by sections 310(b) and 306 of the Act would result in substantial and widespread economic and social impact.

According to DEP policies, the justification for variances, which are temporary rather than permanent suspensions of the designated uses, involve the same substantive requirements although the showings needed are less rigorous.



* One of the criteria of CMR 4.03 (4) must be met

1. Adapted from the Massachusetts Department of Environmental Protection *Policy for Abatement of Pollution From Combined Sewer Overflow*, dated August 11, 1997

Figure 2-1
CSO Controls - WQS Coordination[1]

2.4 Applicability to Worcester

Worcester's CSO planning is complicated by several factors, as discussed below. Because of these complicating factors, the specific applicability of these CSO policies to the City is unclear. What is clear is that the City must comply with EPA's Nine Minimum Controls, as these are the technology-based control requirements applicable to all CSO communities.

Beyond the nine minimum controls, the application of the CSO policies is complicated. An interpretation that the QCSOSTF represent untreated discharges could mean full sewer separation would be required to meet Class B requirements on the Blackstone River, a National Heritage River. Full separation, aside from being costly in terms of both financial investment and impacts to the community, is not likely to improve water quality in the Blackstone River. However, the sensitivity associated with reclassifying a national heritage river, although in the best interest of the river quality and protection of uses may result in substantial debate.

Section 3
River Uses and Water Quality

3.1 Introduction

This section provides a brief overview of the water bodies in the City, and discusses their general condition. Given that Worcester's Combined Sewer System (CSS) covers only 15% of the city's area, most of the water bodies are not affected by the CSS. These water bodies are discussed briefly, and the Blackstone River, the only water body that receives treated discharges from the CSS, is discussed is greater detail.

3.2 River Classification and Uses

As discussed previously, Massachusetts has designated all waters in the state as fishable and swimmable. For freshwater, all water bodies are either Class A (drinking water source) or Class B (swimmable). All water bodies within Worcester city limits are Class B.

Like almost all urban water bodies and many non-urban water bodies in Massachusetts, the water bodies in Worcester are impaired, and do not fully support their designated uses.

Table 3-1 summarizes river segments, lakes and ponds in Worcester, and their current status, as provided in the Blackstone River Basin 1998 Water Quality Assessment Report, DEP, May 2001 (WQA).

Draft Total Maximum Daily Loads (TMDLs) have been established for a number of lakes and ponds in Worcester, including Indian Lake, Salisbury Pond, Curtis Pond, North, Curtis Pond, South, Green Hill Pond, Lake Quinsigamond, and Flint Pond.

The Blackstone River is formed by the confluence of the Middle River and the culverted Mill Brook Conduit. The drainage area at the confluence is 61.5 square miles. The discharge point of the four-square mile Worcester CSS varies depending on flow conditions. During small to moderate rainfall events, the entire CSS is tributary via an interceptor system to the Upper Blackstone wastewater treatment facility (UBWWTF), which discharges to the Blackstone River in Millbury. During larger rainfall events, the CSS remains tributary to the UBWWTF, but part of the runoff also is diverted to a system of overflow collectors thence to the Quinsigamond CSO Storage and Treatment Facility (QCSOSTF). Flow reaching the facility is either stored and pumped to the UBWWTF, or is screened, settled, disinfected, and dechlorinated, then discharged to the Mill Brook Conduit, then the Blackstone River. Most of the time, the CSS drainage area has no direct runoff to the Blackstone River. At a maximum, the CSS represents less than six percent of the drainage area at the point of discharge to the Blackstone River. Both the UBWWTF and the QCSOSTF discharge to Blackstone River Segment MA51-03.

Table 3-1
Status of Water Bodies in Worcester

Water Body	*Cause of Impairment*
Rivers and Brooks	
Blackstone River	Unknown toxicity, priority organics (PCB), metals, unionized ammonia, chlorine, nutrients, organic enrichment/low DO, flow alteration, pathogens (fecal coliform bacteria), suspended solids, turbidity
Mill Brook	PCB, metals, unionized ammonia, nutrients, organic enrichment/low DO, fecal coliform bacteria, suspended solids, turbidity
Middle River	Unknown toxicity, metals, nutrients, pH, fecal coliform bacteria, turbidity
Kettle Brook	Nutrients, fecal coliform bacteria
Tatnuck Brook	Turbidity
Lakes and Ponds	
Burncoat Park Pond	Noxious aquatic plants, turbidity
Curtis Pond (N. Basin)	Noxious aquatic plants
Curtis Pond (S. Basin)	Siltation, noxious aquatic plants
Flint Pond	Turbidity
Green Hill Pond	Turbidity
Indian Lake	Organic enrichment/low DO, noxious aquatic plants
Leesville Pond	Nutrients, organic enrichment/low DO
Lake Quinsigamond	Noxious aquatic plants
Salisbury Pond	Noxious aquatic plants, turbidity

Source: Blackstone River Basin 1998 Water Quality Assessment Report, DEP, 2001.

The following paragraph is extracted from the WQA report:

The Blackstone River is characterized by numerous impoundments formed by the remains of old mill dams used historically for water power, and still used to generate power. Pollution problems on the Blackstone River date to the industrial era and before. The Blackstone Valley was the birthplace of the American textile industry. Gross sediment contamination also resulted from the discharges of heavy metals from plating operations, oil and grease from machine shops, dyes and prints from textile plants, organics and metals from tanneries. Re-suspension of contaminated sediments located behind many of the dams remains a major concern. The river provides limited dilution for municipal and industrial wastewater discharges. Non-point source pollution associated with urban and agricultural runoff, contaminated sediments, runoff and/or leachate from dumps, junkyards, gravel pits, and automobile graveyards also contribute to the basin's water quality problems.

The Blackstone River (MA51-03) does not support the designated uses for a Class B waterway. It does not support the aquatic life designated use because of habitat alteration, organic enrichment, nutrients, and toxicity caused by channelization, habitat modification, municipal point source, CSO, and urban runoff. The reason the WQA included CSO is not clear.

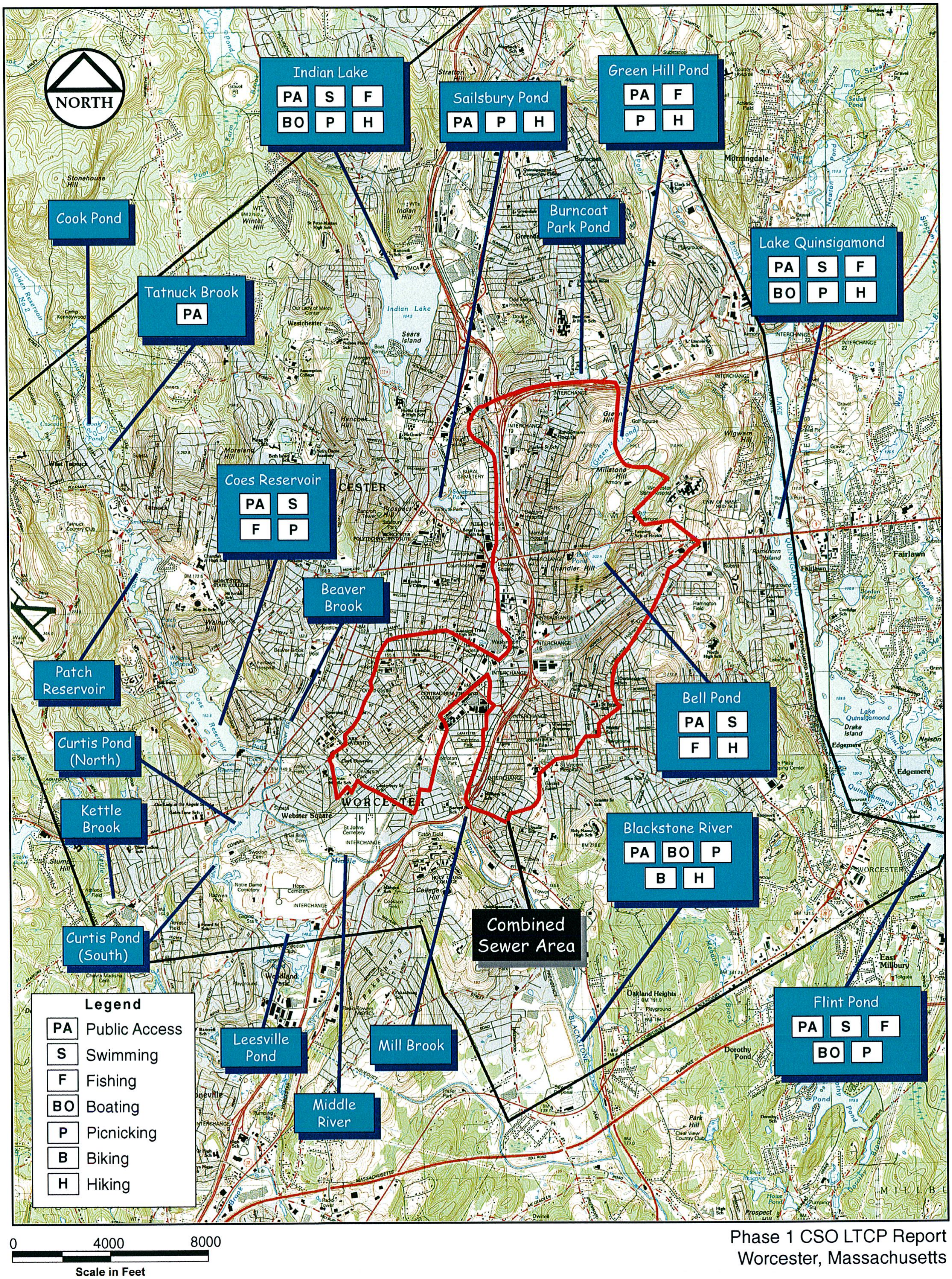
The river segment does not meet its primary contact, secondary contact, or aesthetics designated uses because of pathogens, trash/debris, and turbidity from urban runoff and illicit sewer connections. Given that discharges from the CSS, when they occur, are disinfected, screened, and receive some settling, it does not appear that the CSS contributes to these problems.

3.3 Prioritized List of Water Resources within the City

There are no Outstanding National Resource Waters within the City, nor are there waters known to be habitat for threatened or endangered species. All public drinking water intakes are outside of the city boundaries. No downstream communities obtain their water from surface water intakes, but rather depend on groundwater for water supply. There are a number of recreational uses of water bodies within Worcester, as shown in Table 3-2 and Figure 3-1. The City desires to maintain and improve all of these water bodies, in order to provide continued recreational opportunities, contribute to an improved quality of life for its citizens, and to enhance the environment. The water bodies presented in Table 3-2 are in two tiers. Tier 1 lists highest priority water bodies. They all have public access and many of them have other uses including fishing and boating and have public swimming beaches. Tier 2 lists all other water bodies. They are all Class B waterways, and it is the City's long-term goal to help achieve their designated uses. The only water body in the table impacted by the CSS is the Blackstone River. Despite the fact that it has few of the

Table 3-2
Prioritized List of Water Bodies within the City of Worcester

Water Body	*Public Access*	*Swimming*	*Fishing*	*Boating*	*Picnicking*	*Biking*	*Hiking*
Tier 1							
Bell Pond	X	X	X				X
Blackstone River	X			X	X	X	X
Coes Reservoir	X	X	X		X		
Flint Pond	X	X	X	X	X		
Indian Lake	X	X	X	X	X		
Lake Quinsigamond	X	X	X	X	X		X
Tier 2							
Beaver Brook							
Burncoat Park Pond							
Cook Pond							
Curtis Pond, North							
Curtis Pond, South							
Green Hill Pond	X		X		X		X
Kettle Brook							
Leesville Pond							
Middle River							
Mill Brook							
Patch Reservoir							
Salisbury Pond	X				X		X
Tatnuck Brook	X						



Phase 1 CSO LTCP Report
Worcester, Massachusetts

CDM

Figure 3-1
Water Bodies Within the City of Worcester

uses listed in Table 3-2, the Blackstone River is one of the City's highest priorities, in keeping with the Blackstone Valley's designation as a National Heritage Corridor.

The City has expended considerable resources on the Blackstone River, and will continue to do so in the future. Given that all water bodies listed in the table are tributary to the Blackstone River, all efforts to maintain and improve Worcester waterways will improve the Blackstone River.

Section 4
Sewer System Characterization

4.1 Introduction

Characterization of CSO impacts from a combined wastewater collection system is necessary to document existing conditions and to identify the water quality benefits achieved by CSO mitigation measures. A significant portion of this Phase I CSO Long-term Control Plan consisted of field investigations of the existing wastewater collection system. The focus of these investigations was flow metering. The purpose of the investigations was to collect information regarding the operation and response of the combined sewer system during dry weather and during rainfall events. The data were used to develop and calibrate system models in order to determine impacts to receiving waters under various design storm conditions.

The flow metering was conducted from April to June 2001. The results of the flow metering program were summarized in the September 2001 Draft Flow Metering Summary Report. Other sources of information used for characterization of Worcester's combined sewer system include the City's Nine Minimum Control Measures Report dated May 2001, the July 1975 Report Upon Improvements to the Combined Sewerage System, monthly Discharge Monitoring Reports (DMRs) from the City to USEPA and MADEP, and monthly reports on pumping from the City to the UBWWTF. Of special note is the fact that as the result of the 1975 report, the City has invested heavily in CSO controls, and has substantial in-place infrastructure with a very high level of control that has successfully reduced the impact of CSOs on receiving waters.

This section includes a description of the combined sewer system and briefly discusses the Spring 2001 metering program. Additional detail on the metering program is provided in the metering report. Documentation of the number and characteristics of overflow events, and a summary of CSO impacts are provided in subsequent sections.

4.2 Description of Combined Sewer System

A description of the existing combined sewer system is provided in this section, including the extensive CSO control improvements implemented in the 1980s and 1990s as a result of prior planning efforts.

4.2.1 Overview of the CSS

As indicated in Section 1, the Worcester sewer and drainage system is predominantly a "separated" system. Only 15 percent of the City (about 4 square miles) is served by a CSS. Combined sewers carry sanitary flows during dry weather to the treatment facility. During wet weather, they carry a combination of sanitary flows and stormwater runoff to the treatment facility, but once the conveyance capacity is exceeded, the remaining flow is diverted via a series of combined sewer overflow collectors to the Quinsigamond CSO Storage and Treatment Facility (QCSOSTF). Much of the flow is stored in the collectors and the facility, and pumped to the UBWWTF. During periods of heavy, sustained precipitation (about twice a month), flow exceeding the capacity for storage and pumping to the UBWWTF is discharged to the Mill Brook at the headwaters of the Blackstone River after screening, chlorination, and dechlorination at the QCSOSTF.

Worcester's CSS facilities are shown schematically on Figure 4-1 and consist of the following:

- Approximately 60 miles of combined sewers.

- Three major interceptors:

  - 48-inch Eastern Interceptor, serving the eastern portion of the combined sewer area. This interceptor also carries sanitary flow from points north and east of the CSS;

  - 48-inch Western Interceptor, serving the western portion of the combined sewer area; and

  - 32-inch x 42-inch Cambridge Street Interceptor. This interceptor is located downstream of the combined sewer area, and conveys flows from large separated areas west of the combined area.

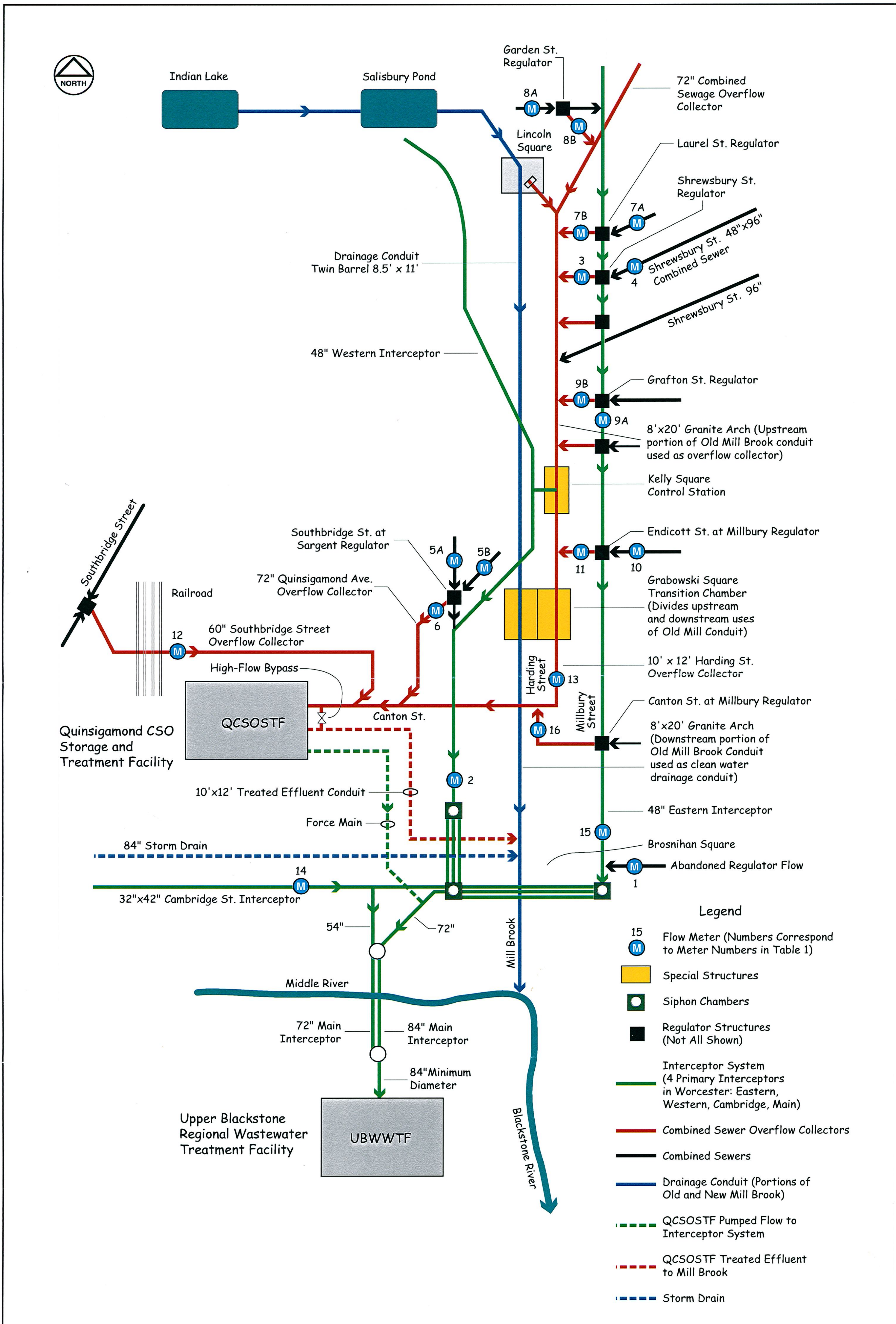
- Six combined sewer overflow collectors designed to store combined sewerage for later conveyance to the UBWWTF, and convey excess combined sewerage to the QCSOSTF. These include:

  - **Lincoln Square Overflow Collector**: 72-inch pipe, 2,000 feet long extending from Garden Street to Lincoln Square, where it connects to the converted Old Mill Brook overflow collector;

  - **Converted Old Mill Brook Overflow Collector**: 8'x20' granite arch (Old Mill Brook), 7,000 feet long extending from Lincoln Square to Grabowski Square, where it connects to the Harding Street overflow collector;

  - **Harding Street Overflow Collector**: 10'x12' box culvert, 2,000 feet long extending from Grabowski Square to the QCSOSTF;

  - **Canton Street Overflow Collector**: 36-inch pipe, 1,100 feet long that connects to the Harding Street overflow collector at the intersection of Harding Street and Canton Street;



Not to Scale

CDM

Figure 4-1
Schematic of the Combined Sewer System

- **Quinsigamond Avenue Overflow Collector:** 48-inch pipe, 1,500 feet long extending from Southbridge Street at Sargent Street to the QCSOSTF; and

- **Southbridge Street Overflow Collector:** 60-inch pipe, 1,400 feet long extending from Southbridge Street under the rail yard to the QCSOSTF.

The Lincoln Square overflow collector, the converted Old Mill Brook overflow collector, and the Harding Street overflow collector act as one overflow collector relieving the Eastern Interceptor. For the purposes of this report, this entire length of overflow collector will be referred to as the Harding Street overflow collector. The other overflow collectors, other than the Canton Street overflow collector, serve the western portion of the combined sewer area.

- Seventeen CSS regulators. At the regulators, low flow is conveyed to the interceptor system to the UBWWTF. Flow that exceeds the capacity of the low flow connections is conveyed to the overflow collectors.

- The QCSOSTF. This facility:

  - Pumps up to 19.9 mgd to the UBWWTF;

  - Stores up to 2.5 million gallons of combined sewage; and

  - Provides treatment consisting of screening, chlorination, and dechlorination of all flow up to 350 mgd before discharging to the Mill Brook at the headwaters of the Blackstone River. Settling in the storage tanks also occurs before discharge.

  As a result of the QCSOSTF, there are no dry weather overflows from the Worcester CSS, and all flows during a typical year receive either advanced treatment at the UBWWTF or are treated at the QCSOSTF.

- One (treated) combined sewer overflow, the discharge from the QCSOSTF, NPDES Permit No. MA0102997

In addition to the CSS facilities, a large drainage conduit (twin 8.5′ x 11′ boxes transitioning into an 8′ x 20′ granite arch downstream of Grabowski Square) carries stormwater through the combined sewer area from the upstream separate storm sewer system to the Blackstone River. All stormwater in the conduit is isolated from the CSS.

4.2.2 Dry Weather and Wet Weather Operations

During dry weather, combined sewers convey flow via one of three interceptors to the UBWWTF. The majority of flow, 76 percent of the combined system (1,924 acres), is routed to the 48-inch Eastern Interceptor. Approximately 18 percent of the combined system (446 acres) is routed to the 48-inch Western Interceptor. The remaining six percent (158 acres) drains to the 32-inch x 42-inch Cambridge Street Interceptor. One

55-acre area in the Southgate Street vicinity discharges directly to the QCSOSTF. The flow from this area is pumped from the QCSOSTF to the UBWWTF.

The three interceptors described above, as well as pumped flows from the QCSOSTF, discharge into the 72-inch Main Interceptor downstream of Brosnihan Square. The Main Interceptor conveys flows for approximately 12,900 feet to the UBWWTF located in Millbury near the Worcester line. The Main Interceptor varies in size from one 72-inch conduit to parallel 72-inch and 84-inch conduits on route to the UBWWTF, and can handle flows up to 145 mgd.

During wet weather, as flow rates increase, low flows continue as described above, and higher flows are diverted at regulators into the CSO collectors, and thence to the QCSOSTF, where the flow is screened and chlorinated and stored until influent levels get too high. Additional higher capacity pumps are turned on that increase the flow from the QCSOSTF to the UBWWTF. When all pumps are on, about 19.9 mgd is conveyed from QCSOSTF to the UBWWTF. This pumping rate is maintained as long as possible. During very heavy rainfalls pumping is reduced or stopped so that the full capacity of the UBWWTF can be used to treat flows from the separated portions of its service area. The QCSOSTF also stores flow in excess of the pumping capacity, filling the 2.5 million gallons of available storage. If the storage available is insufficient to store the runoff, then it passes through a dechlorination facility and discharges through outfall NPDES Permit No. MA0102997.

4.3 Relative Effectiveness of CSO Control Measures Already Implemented

CSS performance contrasts markedly with conditions before the CSS improvements constructed in the 1980s, when the Quinsigamond CSO Treatment Facility and the overflow collectors were built and put on line, and when 312 acres (11% of the original CS area) were separated. Before this work, the system experienced dry weather overflows and virtually all wet weather runoff from the CSS was discharged to the Blackstone River untreated. There are now no dry weather overflows, and all flows are treated. In a typical 5-year period, 94 percent of the flow receives treatment at the Upper Blackstone WWTF, while the remainder receives treatment at the QCSOSTF.

4.4 Flow Metering Program

An eight-week metering program was conducted in Spring 2001 to develop a thorough understanding of how the Worcester combined sewer system functions under dry and wet weather. Results were used to calibrate the SWMM hydraulic computer model of the Worcester sewer system which was used to evaluate system response to storm events and to develop cost-effective CSO control alternatives as discussed later in this report. The basics of the program are described below. Further details and data analysis of this metering program are provided in the September 2001 Draft Metering Summary Report.

4.4.1 Duration

The flow metering program was conducted by Severn Trent Pipeline Services, formerly Utility Pipeline Services, under the direction of CDM. The program extended from April 9, 2001 to June 14, 2001, and included installation and maintenance of 16 continuously recording flow meters, covering 20 key sites in the Worcester sewer system as shown schematically on Figure 4-1. Depth and velocity were recorded at each meter location, and correlated to flow in 5-minute time intervals. Tipping bucket type rain gages used to record rainfall during the monitoring period were also installed at the Worcester Department of Public Works (DPW) and the Fire Station Headquarters on Grove Street to correlate system response to specific rainfall events and to identify spatial variations in rainfall in the combined area. All meters and rain gages were inspected weekly to ensure proper operation.

Of the 20 meter installation sites, three were located in interceptors (Eastern, Western, and Cambridge Street Interceptor) and three in overflow collectors (Harding Street Overflow Collector, Southbridge Street Overflow Collector, and Quinsigamond Avenue Overflow Collector). The rest were installed in combined sewer regulators, either the influent, effluent, or overflow lines. Regulator sites were selected based on tributary area, uniform land use type, and geographical location. Data were collected from 6 of the 17 regulators in the Worcester system. The meter sites were selected to provide a representative look at the performance of the Worcester combined sewer system in dry and wet weather to use in calibrating the SWMM system model.

Other goals of the metering program, aside from providing system performance data for calibration purposes, were geared towards evaluating potential CSO control alternatives. For example, goals included understanding how much flow is generated during wet weather in the interceptor system for conveyance to the UBWWTF, how much flow from the combined area is delivered to the interceptor system and to the QCSOSTF through overflow collectors, how much flow from the QCSOSTF is delivered to the UBWWTF or discharged to the Blackstone River, and performance of regulators in wet weather events.

Hourly influent flow data at the UBWWTP and QCSOSTF performance data, including pumping and effluent data, were also obtained to evaluate wet weather response at these key facilities, and to determine if additional flow could be delivered to the UBWWTF.

4.4.2 Precipitation Data

Table 4-1 presents descriptive statistics and rankings of the rainfall events recorded during the metering period. There were eight significant storm events during this period. Rainfall depths at the DPW rain gage ranged from a minimum of 0.01 inches to a maximum of 2.9 inches on June 2, 2001. The June 2nd storm is estimated to be between a 1-year and a 2-year event, based on peak 15-minute and peak hourly

Table 4-1
Rainfall Statistics for April-June 2001 Metering Program
City of Worcester

Date	Rain Gage[(1)]	Total Precipitation (inches)	Storm Rank[(2)]	Starting Hour	Duration (hours)	Peak 15-Minute Depth (inches/15 minutes)	Peak 1-Hour Depth (inches/hour)
4/10/01	1	0.05	9	9:30	0.5	0.04	0.05
	2	0.00	14	NA	NA	NA	NA
4/12/01	1	0.47	4	5:15	17	0.11	0.23
	2	0.26	6	5:00	13.25	0.05	0.09
4/24/01	1	0.02	13	18:45	0.5	0.01	0.02
	2	0.03	11	18:45	0.5	0.02	0.03
5/12/01	1	0.39	6	17:30	1.5	0.25	0.38
	2	0.22	7	17:30	1	0.13	0.22
5/15/01	1	0.03	12	22:15	1.25	0.01	0.02
	2	0.05	9	22:15	1.5	0.01	0.04
5/16/01	1	0.02	14	9:00	4.75	0.01	0.01
	2	0.03	12	9:15	1.5	0.01	0.02
5/22/01 - 5/24/01	1	1.44	2	4:00	49.5	0.12	0.26
	2	0.93	2	3:45	50.0	0.06	0.14
5/24/01	1	0.28	8	16:45	6.5	0.13	0.13
	2	0.18	8	16:45	5.25	0.07	0.08
5/26/01 - 5/27/01	1	0.42	5	19:45	17.5	0.15	0.26
	2	0.48	4	19:15	15.5	0.09	0.21
5/29/01	1	0.03	11	22:30	0.5	0.02	0.03
	2	0.02	13	22:00	0.5	0.01	0.02
5/30/01	1	0.03	10	10:00	0.5	0.02	0.03
	2	0.03	10	17:00	0.25	0.03	0.03
6/2/01	1	2.90	1	5:30	8.5	0.29	0.89
	2	1.84	1	5:15	9	0.17	0.39
6/3/01	1	0.28	7	3:15	2.5	0.08	0.18
	2	0.31	5	2:45	3	0.14	0.26
6/11/01 - 12/01	1	0.82	3	14:30	10	0.17	0.32
	2	0.85	3	14:00	10.5	0.15	0.42

Notes:
1. Rain Gage No. 1 was installed at the Worcester DPW building on East Worcester Street.
   Rain Gage No. 2 was installed at the Worcester Fire Station Headquarters on Grove Street.
2. Storm rank according to depth recorded at each rain gage.
3. 6-hr minimum separation between storm events.
4. Storms <0.02 inches in total depth not included.

intensity statistics. In general, rainfall recorded at the DPW gage during the metering period was more intense than that recorded at the Fire Station Headquarters rain gage, suggesting that storm events were more intense near the DPW gage than in the northern portion of the study area.

CSS performance in response to these storms and the SWMM model calibration is described further in Section 5.

Section 5
Sewer System Model

5.1 Introduction

CDM developed a hydrologic and hydraulic model to simulate flows in the City's combined sewer area. The model was developed using the US EPA Storm Water Management Model (SWMM). SWMM is a dynamic hydrologic and hydraulic simulation model designed for urban areas. The Runoff, Transport and Extran modules of SWMM were used to simulate flows in Worcester's combined sewer area. Detailed discussion of the model development and calibration are presented in the calibration memorandum (See December 2001 SWMM Model Calibration memorandum). This section presents an overview of model development and calibration. This section also presents wastewater flow projections at the UBWWTF.

5.2 SWMM Model Development

The City's combined sewer area model represents all of the key components of the City's combined sewer collection system. These components comprise the interceptors, overflow collectors and other flow control, conveyance and storage structures in the system. These structures are:

- Interceptors:
  - Eastern;
  - Western;
  - Cambridge Street; and
  - Main.
- Overflow Collectors:
  - Lincoln Square;
  - Southbridge Street;
  - Harding Street;
  - Quinsigamond Avenue; and
  - Canton Street.
- Flow Control, Conveyance and Storage Structures:
  - Quinsigamond Avenue Combined Sewer Overflow Storage and Treatment Facility (QCSOSTF);
  - Kelly Square Control Station;
  - Brosnihan Square Siphon;
  - Grabowski Square Weir; and
  - Combined Sewer System Regulators.

In total, the City's model represents 287 manholes and 15.7 miles of pipe. The model includes a detailed representation of the QCSOSTF. The pumps, wetwell, storage and treatment tanks, inlet and outlet gates and operating procedures at the facility are explicitly represented in the model. As-built plans for the interceptors and overflow collectors and construction plans for the QCSOSTF obtained from the City were used to develop the model. In addition to the City's combined sewer area that was represented in detail, less detailed representations of other areas tributary to City's collection system and the UBWWTF were represented in the model. They include the separated portion of Worcester's sewer system, as well as the sewered areas in the neighboring communities of Auburn, Holden, Leicester, Millbury, Rutland, and West Boylston. Tributary flows for the towns of Shrewsbury, Oxford and Sutton are also included in the model.

5.3 Model Calibration

The model was calibrated using data from the metering program conducted during April, May and June of 2001 at 20 locations within the collection system, as described briefly in Section 4 and in more detail in the September 2001 Draft Metering Summary report. Long-term data (1992–1999) from the UBWWTF was also used in the model calibration. This data was used to develop flow patterns for seasonal flow variation in the City's collection system. These flow patterns were incorporated into the dry weather calibration. The dry weather calibration was performed using periods with at least two previous days of no precipitation. This ensured that the dry weather calibration was not affected by storms.

Storm event calibration was performed after the dry weather calibration was completed. The storm event calibration utilized data collected during the entire monitoring period. Both dry weather and storm event calibration were performed comparing storm volumes, discharge rates, and depth at each meter.

The future flows were incorporated into the City's model by increasing the dry weather flows that represent the sanitary and infiltration components of flow in the collection system.

Figure 5-1 presents typical wet weather calibration results, comparing metered and simulated results at a location on the Eastern Interceptor. The figure shows the results from the June 2, 2001 storm, the largest storm during the metering period, including the rainfall pattern, flow hydrograph and depths in the interceptor for the duration of the storm. Flow at this location represents flow from 73 percent of the combined sewer area. The calibration memorandum presents comprehensive results during wet and dry weather at all meter locations for the entire metering period.

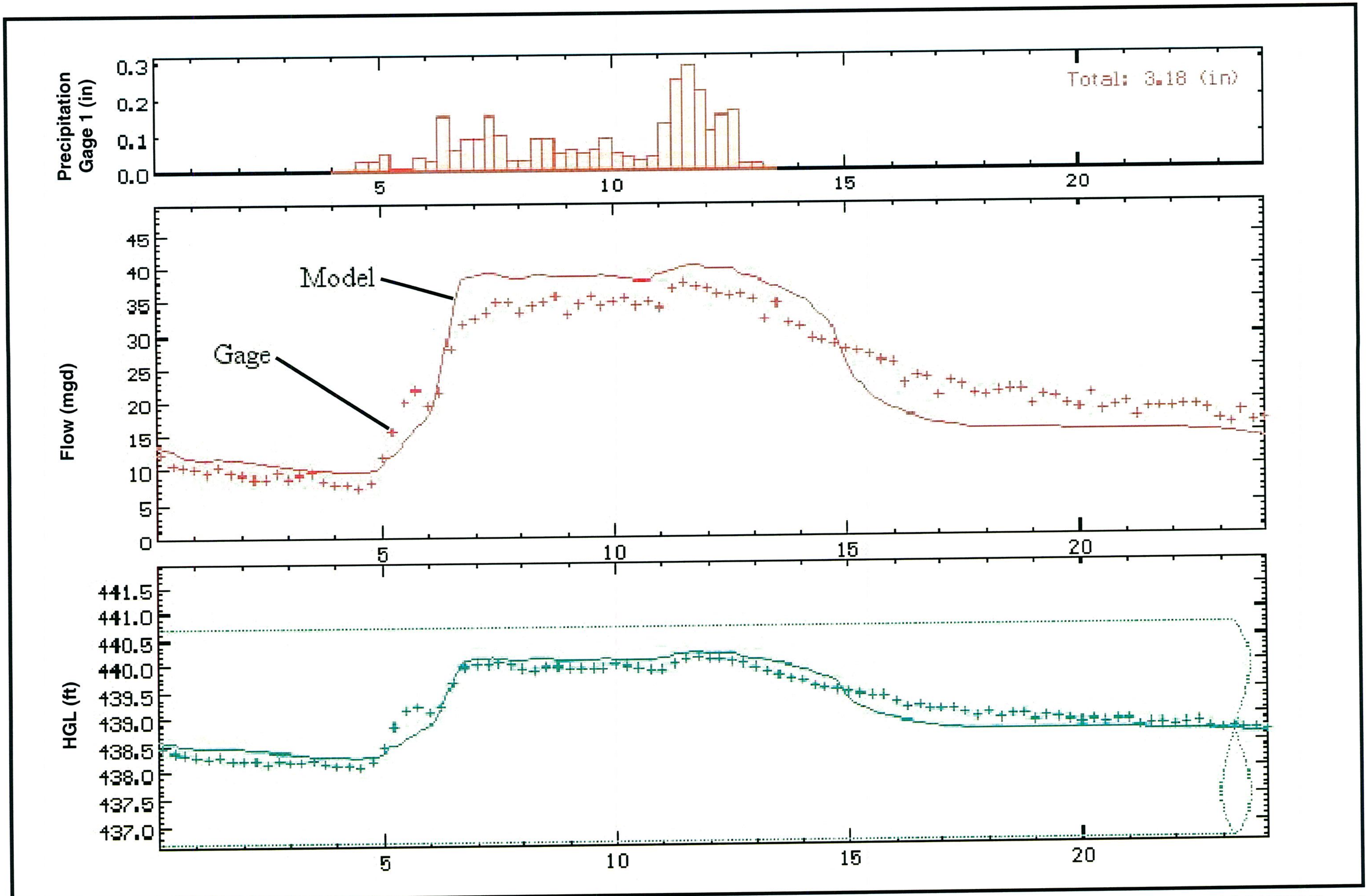


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Figure 5-1
June 2, 2001 Measured and Modeled Flow in Eastern Interceptor

CDM

5.4 Future Flow Projections

Future flow projections were developed for use in the model to simulate 2020 conditions. These flows were adapted from projections developed for the UBWWTF as part of the October 2001 Regional Wastewater Treatment Facilities Plan. Table 5-1 below presents year 2000 and projected 2020 average day flows. These flow projections are based on an analysis of MISER and CMRPC population projections.

Table 5-1
Existing and Future Average Day Flows by Community

Community	*2000 Flow (gpd)*	*2020 Flow (gpd)*
Worcester	34,112,000	37,610,000
Auburn	1,792,000	1,932,999
Millbury[1]	------------	2,258,000
Rutland	459,000	569,000
Holden	900,000	1,600,000
West Boylston	------------	776,200
Oxford	26,000	126,000
Shrewsbury	12,000	12,000
Leicester	------------	360,000
Total	**37,301,000**	**45,245,000**

[1]Includes Sutton flows.

Source: Upper Blackstone Water Pollution Abatement District Regional Wastewater Treatment Facilities Plan, October 2001, Volume 1 of 2.

Section 6
Precipitation Record and Design Storms

Historical Worcester precipitation data were used to establish a five-year representative precipitation record and to identify design storms to be used in facilities planning screening simulations.

6.1 Historic Precipitation Data

Hourly data for Worcester airport (NCDC station 9923) are available for the period from 1957 to the present, with the exception of the period from October 1989 to June 1991, for which only daily totals are available.

While the airport gage is located at an elevation of 986 feet NGVD, most of the city is situated several hundred feet lower. However, daily precipitation data were collected at a gage in the city (NCDC station 9928, elevation 620) until 1962. Figure 6-1 compares annual totals for the period when both gages were in operation. The average annual precipitation for the concurrent 13-year period was 47.3 inches at the city gage, and 47.1 inches at the airport gage. There was no significant difference in precipitation totals between the two locations.

6.1.1 Selection of Representative Five-Year Record

The methodology for selecting the five-year period follows the method used to select a representative period that has been used by CDM for CSO studies in Haverhill, Lawrence, and Lowell. The ideal representative period has the following criteria:

- Excludes storms with return periods exceeding 10 years for durations of 1, 2, 3, 6, 12, and 24 hours;
- Has average annual precipitation close to the long-term mean;
- Is a contiguous five-year period with complete data;
- Includes one wet year and one dry year, where a wet or dry year has precipitation differing from the long-term mean by one standard deviation;
- Includes five 1-year storms at durations of 1, 2, 3, 6, 12, and 24 hours;
- Includes one 5-year storm and two or three storms with a 2-year return period at each duration listed above; and
- Has wintertime temperatures representative of the long-term record.

6.1.2 Screening for Large Storms

Table 6-1 lists storms with return periods of 10 years and longer. Data from Worcester Regional Airport for the period 1957-2000 were compiled for this analysis.

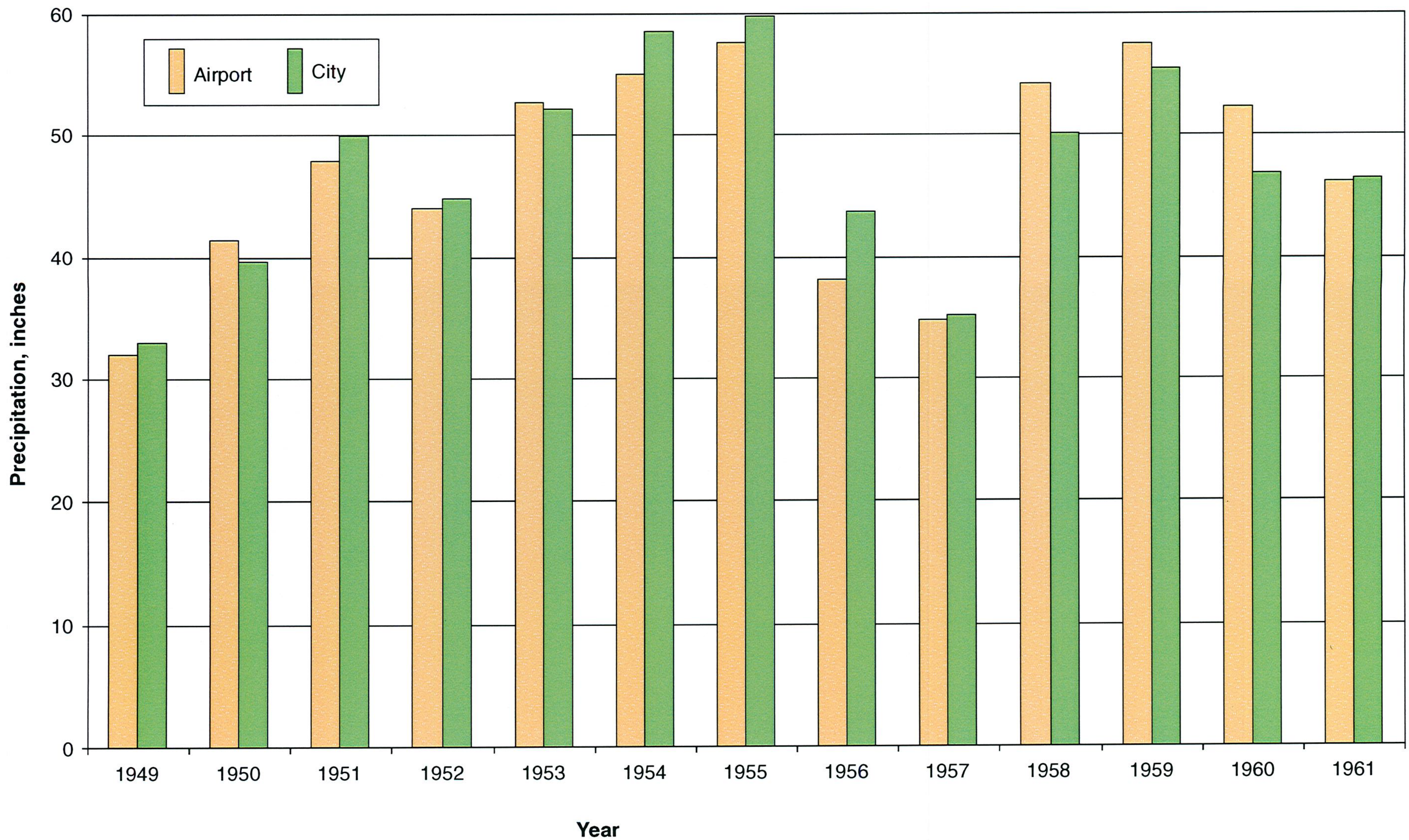

Figure 6-1
Worcester Airport and City Gage Annual Precipitation 1949-1961

CDM

Return periods were computed based on CDM's analysis of the long-term record. CDM's Rainmaster program was used to identify storm events, and storm data were fitted to statistical distributions to obtain return periods. The process used was similar to that described by Wilks and Cember (1993) for the "Atlas of Precipitation Extremes for the Northeastern United States and Southeastern Canada." Previous investigations have shown that CDM's analysis yields results consistent with the data reported in the Atlas.

Table 6-1
Worcester Storms with 10-year and Longer Return Periods 1957-2000

Date	Return Period (y) at Interval (h)							Precipitation (inches)	
	1	2	3	6	12	24	48	Peak Hour	Storm
July 19, 1959	30							1.38	2.64
September 11, 1960		20	30	40	30	30	20	1.13	5.31
October 5, 1962							40	0.44	6.21
March 17, 1968							10	0.33	4.91
June 4, 1972	10	20	20					1.42	2.63
July 3, 1972	20							1.51	1.70
July 19, 1972	60							1.79	1.85
September 13, 1974		20	20					1.18	2.57
July 16, 1979	20	60	30					1.48	2.80
June 2, 1982				20				0.68	3.28
July 31, 1985			20		10			0.99	3.90
October 5, 1985	10							1.38	1.82
June 5, 1986							10	0.29	4.77
March 30, 1987						20		0.45	4.56
August 19, 1991			20	40	60	40	30	0.92	5.00
September 27, 1993		20	20	20				1.14	3.16
September 15, 1999					20	10		0.81	4.50

Years with two or more exceedances at any duration were dropped from consideration (whether both exceedances occurred in one storm or in two separate storms). Based on the storms in Table 6-1, the years 1960, 1972, 1974, 1979, 1985, 1991, 1993, and 1999 were excluded from the candidate list of years for inclusion in the typical five-year period. This leaves the five-year periods ending from 1965-1971, 1984, 1990, and 1998 as candidate periods. Since there are no hourly data for the period from October 1989 to June 1991, the five-year period ending in 1990 was also excluded.

6.1.3 Annual Precipitation Statistics

The second principal criterion is that the representative period should have typical precipitation. Figure 6-2 shows annual precipitation recorded in Worcester from 1930-2000, the running 5-year mean precipitation, long-term average precipitation, and +1 and −1 standard deviations from the mean. The graph shows that the periods

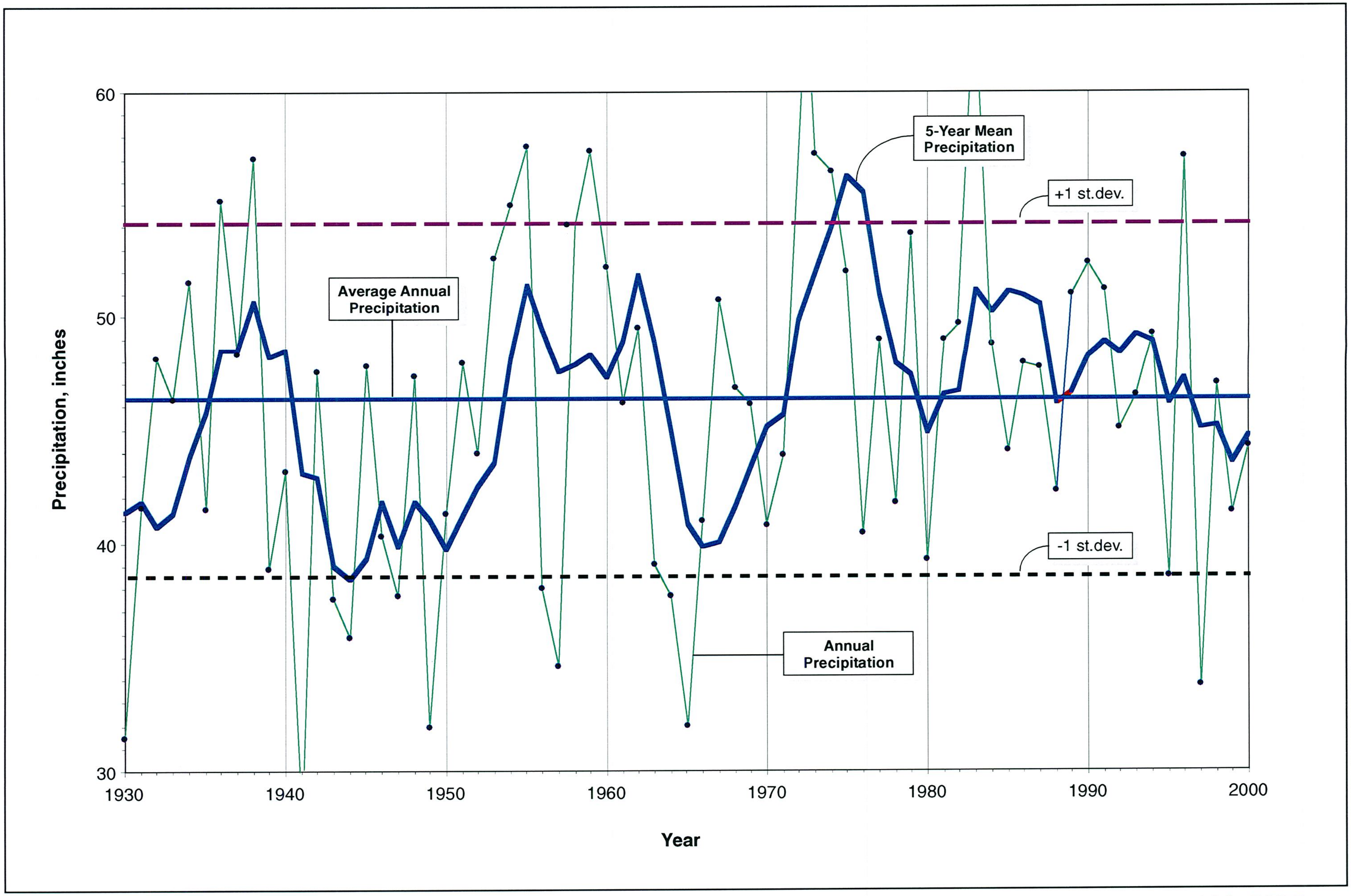

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Figure 6-2
Worcester Annual Precipitation 1930-2000

CDM

ending between 1965 and 1969 are not representative of the long-term record as they include the severe drought years of 1964 and 1965, resulting in well-below average precipitation. The period ending in 1984 is also not typical, as it includes 1983, which was the second wettest year on record. Based on excluding years with large storms and five-year periods with non-representative total precipitation, the remaining candidate typical periods are 1966-1970, 1967-1971, and 1994-1998. The 1990s period meets an additional criterion, as it includes a wet year (1996) and a dry year (1997). The three candidate 5-y periods were next evaluated against the criteria for number of large events.

6.1.4 Storms with 1- to 5- Year Return Periods

The 1966-1971 period has close to the ideal number of storms with return periods of at least one year and of at least two years. For intervals of 1, 2, 3, 6, 12, and 24 hours, the 5-year period from 1967-1971 had 4, 5, 5, 5, 3, and one-year events respectively, of which 3, 3, 3, 2, 1, and 2 events had return periods of two years or more. This closely parallels the ideal condition where there would be five 1-year events at each duration and 2.5 2-year events at each duration.

The 1994-1998 period presents slightly skewed conditions, with an abundance of long-duration large storms, but a scarcity of short-duration intense storms. The numbers of 1-year or larger events in this period were 3, 3, 2, 1, 5, and 8 for durations of 1, 2, 3, 6, 12, and 24 hours respectively. The number of events exceeding a 2-year threshold was 2, 2, 0, 0, 2, and 4. The 1994-1998 period was thus most notably deficient at durations of 3 and 6 hours. There were only two events exceeding a 1-year 3-hour storm, of which none was larger than a 2-year event.

6.1.5 Winter Temperatures

Winter temperatures were considered as a minor factor in selecting the representative period. Average temperatures in Worcester are typically below freezing from early December until mid-March, with daily low temperatures below the freezing mark from mid-November until early April.

The winters of 1970 and 1971 (from November to April) were the two coldest since 1904 with mean seasonal temperatures of 30°F. The winters from 1994 to 1998 were unremarkable with the exception of very warm months in December 1994 (34°F, 5° above normal) and February 1998 (33°F, 7° above normal), and cold months in January 1994 (16°F, 8° below normal) and November 1995 (35°F, 5° below normal). Simulations of the 1970 and 1971 winters would thus be expected to result in lower than normal CSO volumes during periods with accumulating snow, followed by elevated flows during snowmelt events.

6.1.6 Representative Five-Year Record

The period from 1967-1971 was selected as the representative record for use in model simulations of typical system performance. While it does not include ideal wet and dry years as the period from 1994-1998 does, it has a better population of large storms than the alternate period in the 1990s.

6.2 Design Storm Selection

Design storms for Worcester were selected from the historic record. Storms were sought that could be classified as typical 1, 3, and 6-month, and 1, 2, and 5-year storms. The use of historic events instead of synthetic hyetographs produces more realistic facilities planning scenarios. For instance, peak 1-hour precipitation in New England usually occurs during brief thunderstorms (convective storms), while peak 24-hour precipitation occurs during rainfall associated with northeasters and hurricanes (cyclonic storms). The use of a synthetic design storm that combines the characteristics of both of these storm types yields unrealistic conditions that can lead to overly conservative facilities planning.

Design storms were selected from the historic record based upon the criteria that the selected storms should conform as closely as possible with the statistical rainfall depth for the selected return period at 6 to 24 hour durations, and that the selected storms should include monotonically increasing rainfall totals at all time intervals. Thus while the selected 1-year storm need not have a 1-hour rainfall total that matches the 1-year 1-hour statistical rainfall depth, it must have more rainfall in its peak hour than the selected 6-month storm and less rainfall than the selected 2-year storm.

Table 6-2 lists rainfall depths at return periods from 1 month to 5 years for durations of 1 to 24 hours as computed using CDM's Rainmaster program from the 44-year Worcester hourly record.

Table 6-2
Rainfall Duration-Return Period Depths for Worcester Computed by CDM

Return Period	*Rainfall Depth (in.) at Durations (h)*					
	1	*2*	*3*	*6*	*12*	*24*
1-month	0.3	0.4	0.5	0.8	1.0	1.1
3-month	0.5	0.7	0.9	1.2	1.5	1.8
6-month	0.7	0.9	1.1	1.5	2.0	2.3
1-year	0.9	1.1	1.3	1.8	2.3	2.6
2-year	1.0	1.4	1.6	2.0	2.6	3.2
5-year	1.2	1.6	1.9	2.4	3.1	3.7

The computed values generally match those presented in the "Atlas of Short-Duration Precipitation Extremes for the Northeastern United States and Southeastern Canada" (McKay and Wilks, 1995) and "Atlas of Precipitation Extremes for the Northeastern United States and Southeastern Canada" (Wilks and Cember, 1993) to within 0.1 inches. Reported values in the atlases are presented in Table 6-3. The Northeast Precipitation atlases update the older values reported in TP-40 (NWS, 1961).

Table 6-3
Rainfall Duration-Return Period Depths for Worcester from Northeast Precipitation Atlases

Return Period	Rainfall Depth (in.) at Durations (h)					
	1	2	3	6	12	24
2-year	1.1	1.3	1.6	2.1	2.6	3.1
5-year	1.3	1.6	2.0	2.6	3.2	3.7

Six storms were selected from the historic record to use as design storms, ranging from a 1-month event on April 3, 1982 to a 5- to 10-year storm on July 31, 1985. It was not possible to identify storms that could consistently be characterized as 2- and 5-year events. Instead, a 3-year storm was substituted for the 2-year event, and the 5-year design storm was synthesized from the hyetograph of the larger July 31, 1985 storm.

Depth-duration curves for the six selected storms are presented in Figure 6-3. The figure shows that each storm is larger than its predecessor at each duration, and that the storms generally correspond with the statistical depth-duration curves.

Hyetographs for each storm are presented in Figures 6-4 through 6-9. The hyetographs show that the storms have widely varying temporal distributions. The 1- and 3-month storms (April 1982 and December 1976) approximate typical triangular hyetographs, although the 1-month event peaks in its 5th hour while the 3-month storm peaks at its 11th hour. The September 1958 (6-month) storm is similar in shape to the 1-month event, although it has a distinctive second small rainfall peak period occurring 10 hours after the first peak. The June 1974 (1-year) storm has two hours with nearly identical peak intensity at the beginning of the event, followed by 10 hours of moderate rainfall. The October 1973 (3-year) storm presents a very different hyetograph, with 16 hours of moderate rainfall followed by 3 hours of intense rain.

The modified July 1985 (5-year) storm is similar in shape to the October 1973 event, with 9 hours of moderate rainfall followed by 3 hours of very intense rain.

The hourly rainfall data for both the design storms and the long-term record were synthetically disaggregated into 15-minute data using a method described by Ormsbee (1989) and used by CDM in several previous studies. This procedure results in higher peak storm intensities, resulting in more realistic peak flows in parts of the sewer system that have short times of concentration. Since the hydraulic model is calibrated to 15-minute rainfall data, it is important to use data with the same temporal resolution for planning purposes to avoid underestimating peak runoff rates.

Table 6-4 summarizes the rainfall totals for each design storm, including the 15-minute and 30-minute peaks obtained via synthetic disaggregation.

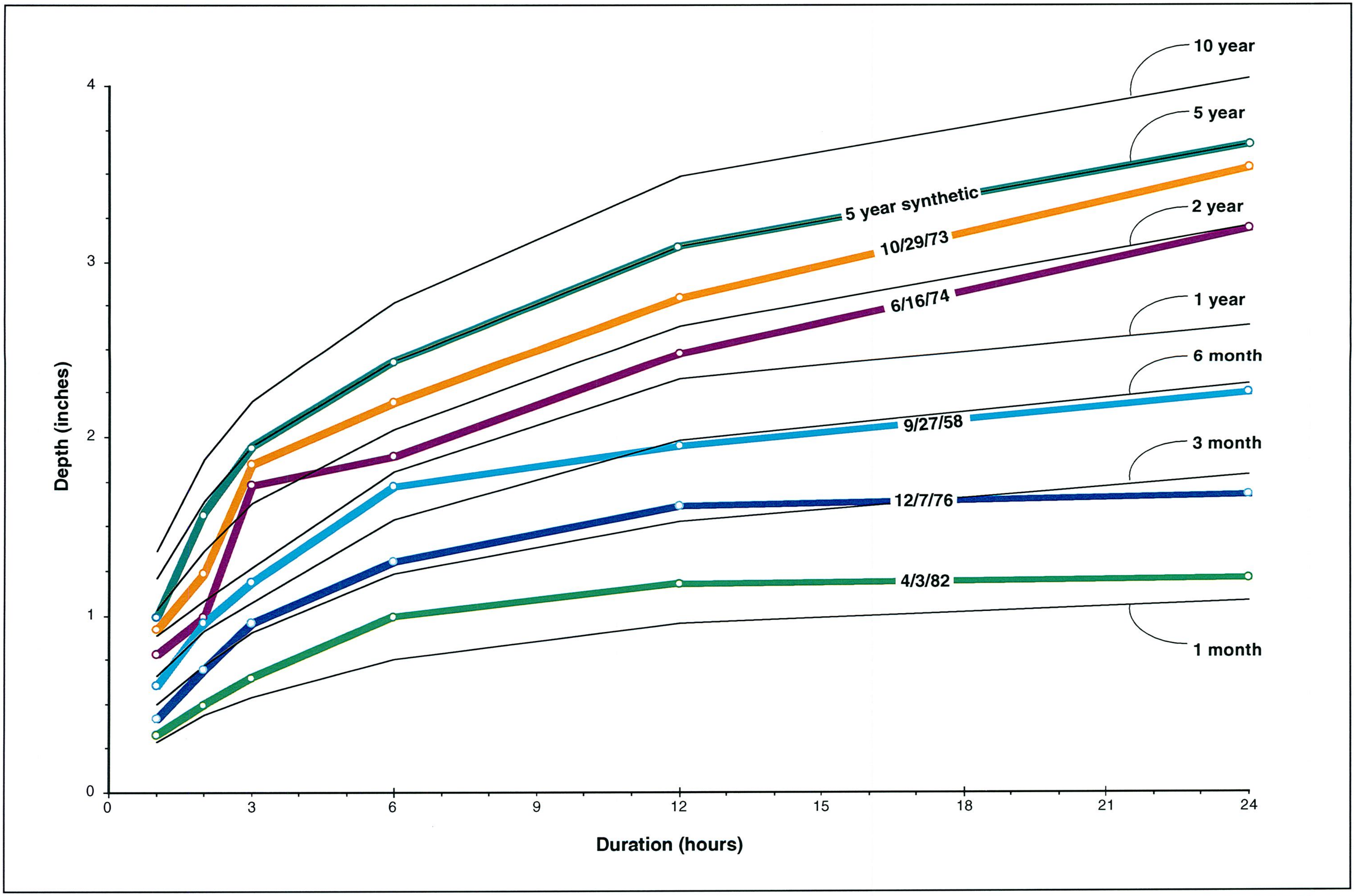

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Figure 6-3
Worcester Depth-duration Curves and Selected Design Storms

CDM

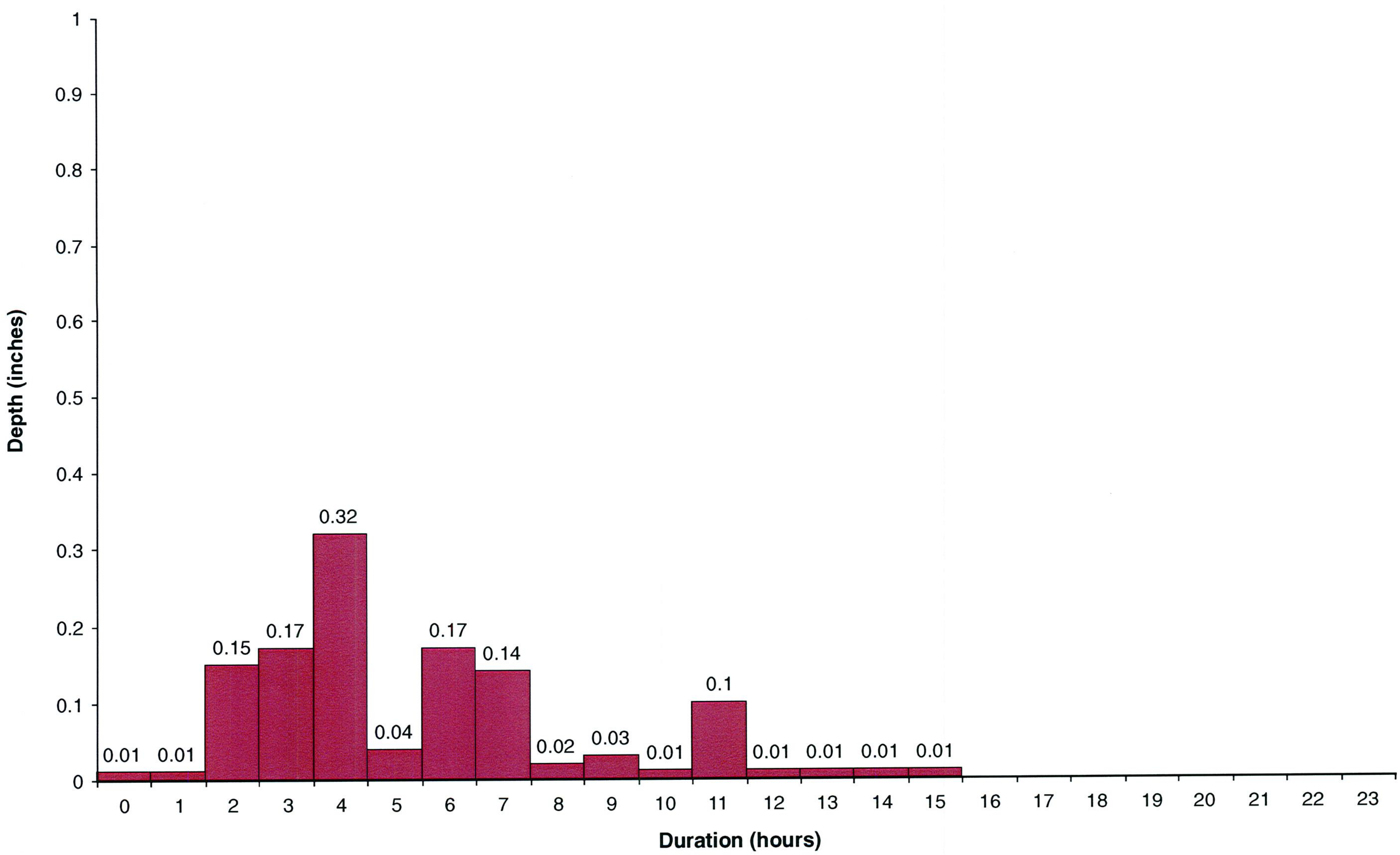

Figure 6-4
1-Month Storm: 1.21 inches April 3, 1982

CDM

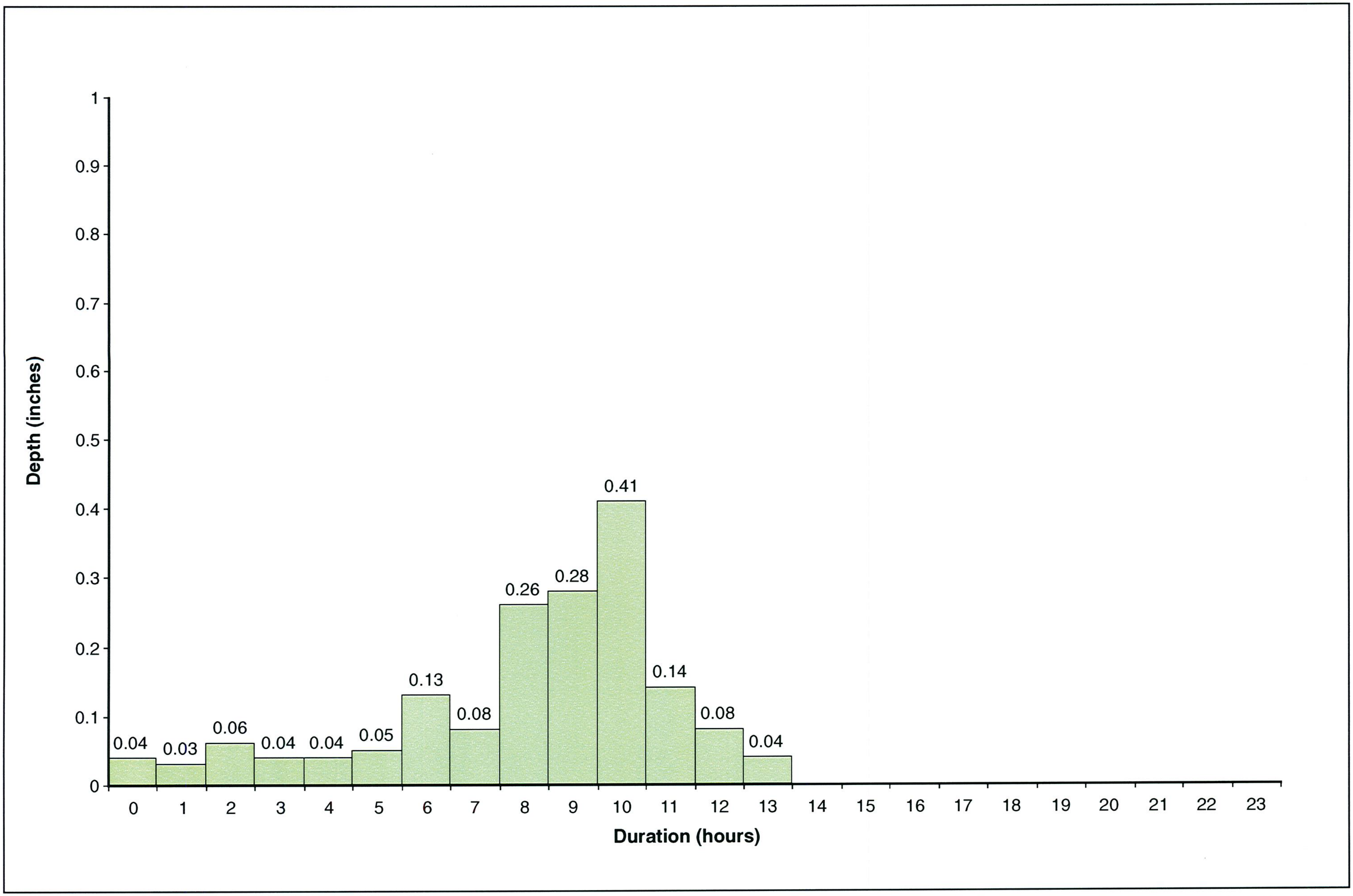

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Figure 6-5
3-Month storm: 1.68 inches December 7, 1976

CDM

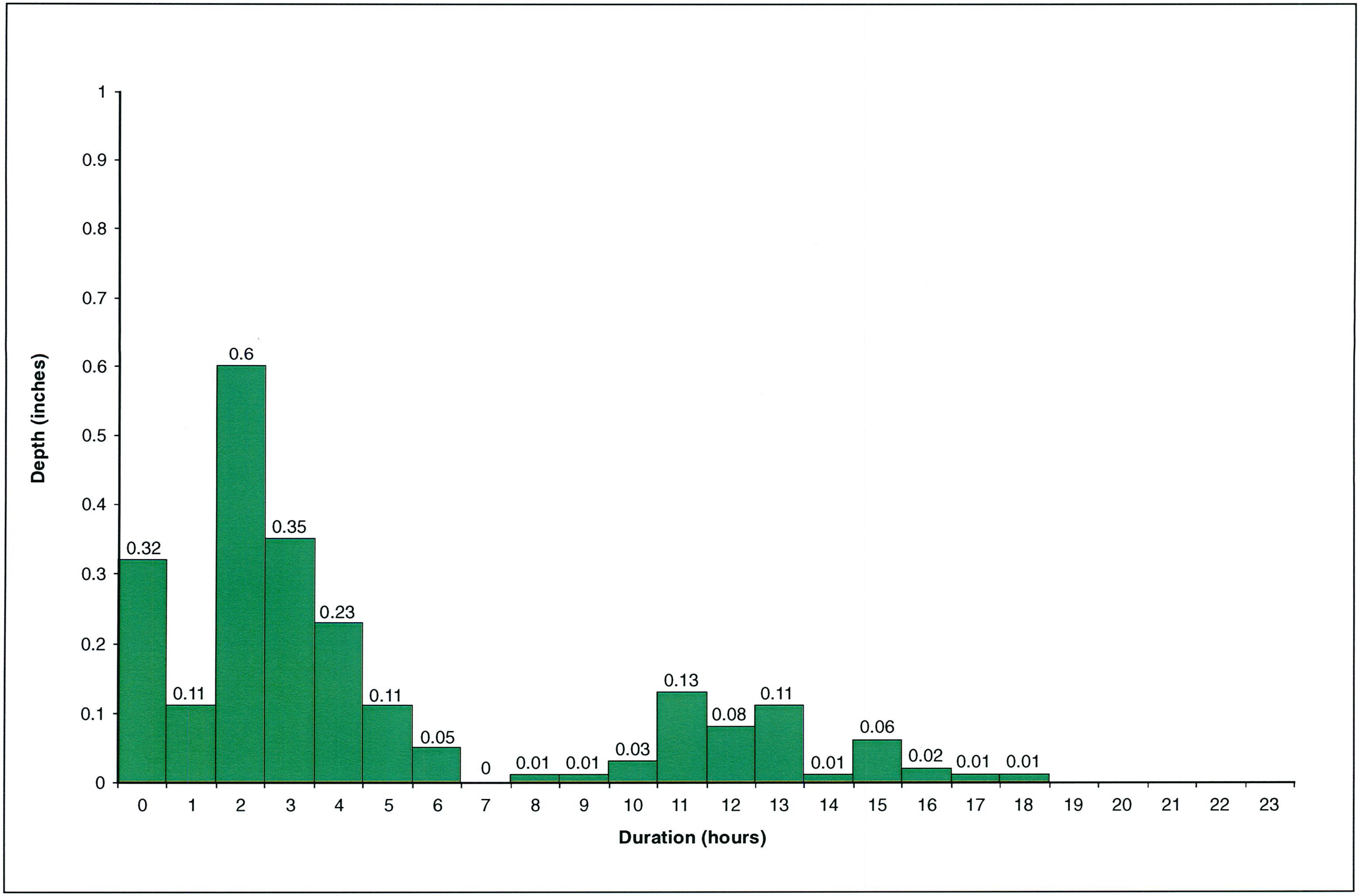

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Figure 6-6
6-Month Storm: 2.25 inches September 27, 1958

CDM

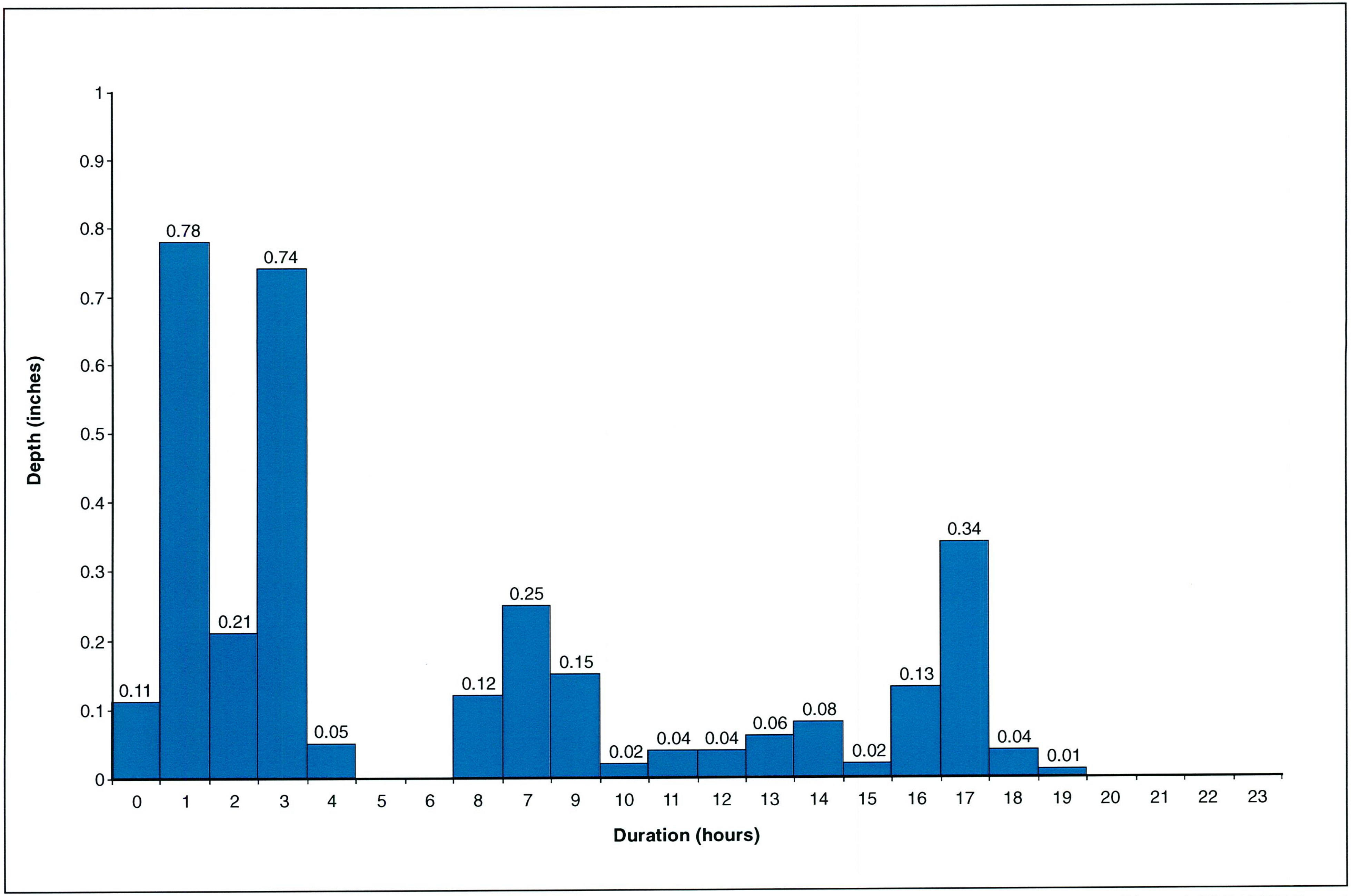

Figure 6-7
1-Year Storm: 3.19 inches June 16-17, 1974

CDM

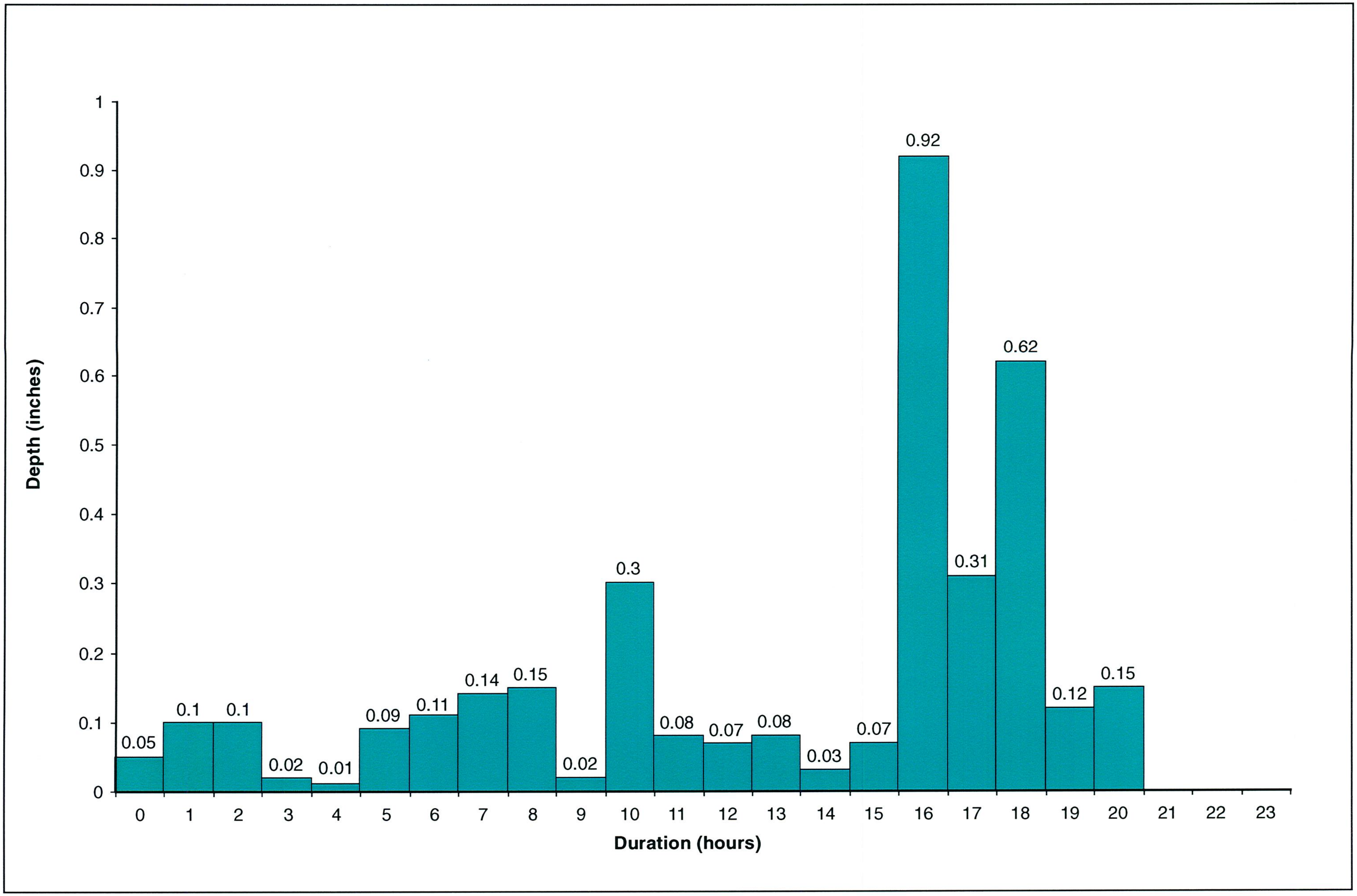

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Figure 6-8
3-Year Storm: 3.54 Inches October 29, 1973

CDM

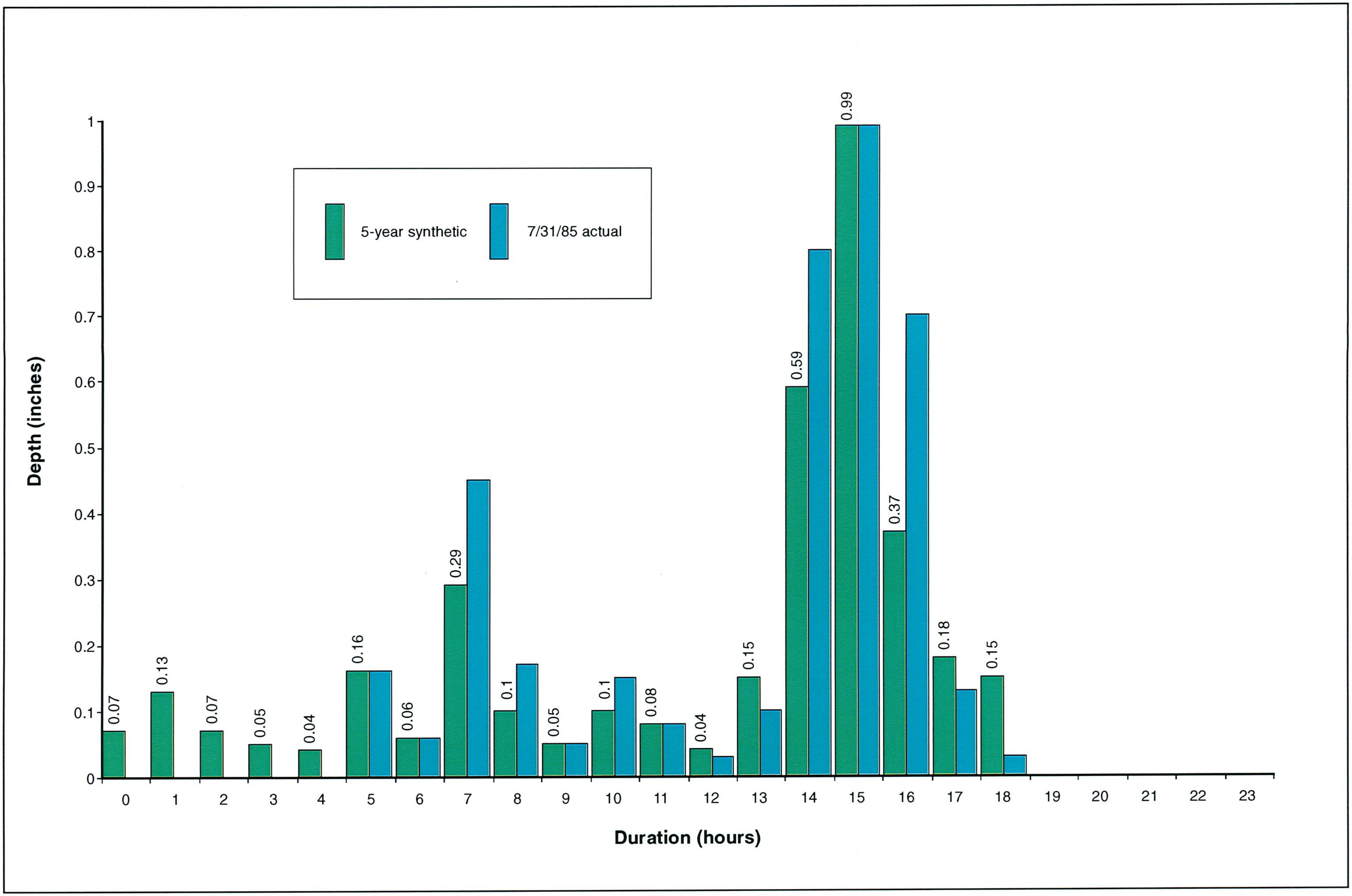

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Figure 6-9
5-Year Storm: 3.90 inch Synthetic Hyetograph Based on July 31,1985 Storm

CDM

Table 6-4
Design Storm Rainfall Totals

Storm	***Date***	***Duration (hours)***	***Maximum Rainfall Depth (inches)***							
			15-minute	***30-minute***	***1-hour***	***2-hour***	***3-hour***	***6-hour***	***12-hour***	***24-hour***
1-month	April 3, 1982	21	0.13	0.23	0.32	0.49	0.64	0.99	1.17	1.21
3-month	December 7, 1976	14	0.15	0.26	0.41	0.69	0.95	1.30	1.61	1.68
6-month	September 27, 1958	19	0.21	0.41	0.60	0.95	1.18	1.72	1.95	2.25
1-year	June 16, 1974	20	0.30	0.53	0.78	0.99	1.73	1.89	2.47	3.19
3-year	October 29, 1973	21	0.33	0.61	0.92	1.23	1.85	2.19	2.79	3.54
5-year	Adjusted July 31, 1985	19	0.36	0.59	0.99	1.58	1.95	2.43	3.09	3.67

Section 7
Existing and Baseline Conditions

7.1 Introduction

The purpose of this section is to report on how the existing system performs, then to project how it will perform in the future (2020) if no improvements are made. This sets the baseline condition, and establishes the framework for comparing the effectiveness of potential CSO controls.

7.2 Basis for Comparisons

In Section 6, Worcester's precipitation record was examined in order to select design storms and a 5-year continuous simulation period (1967-1971). The calibrated SWMM model is used in this section to simulate the response of the CSS to design storms and to the continuous simulation period. The output from the model that is particularly useful includes (for each design event):

- Total volume discharged from the QCSOSTF;
- The volume during the event discharged from the UBWWTF; and
- During a typical year, 100 percent of the combined sewage is treated at either UBWWTF or the QCSOSTF. Results from the model show the portion of combined sewage that receives full secondary treatment at the UBWWTF, and the portion that receives treatment at the QCSOSTF. (Section 7.3.3 demonstrates that QCSOSTF essentially achieves a level of performance matching primary treatment).

The volumes generated by the SWMM model multiplied by typical BOD and TSS effluent concentrations provide BOD and TSS loadings in pounds. The loadings from the QCSOSTF added to the UBWWTF loadings establish total loadings (excluding stormwater) to the Blackstone River, the ultimate beneficiary of any proposed pollution control measure.

For continuous simulations, important statistics developed over the 5-year period include:

- Average number of times the QCSOSTF discharges per year;
- Annual volume from the QCSOSTF; and
- Annual volume from the UBWWTF.

These volumes are also multiplied by typical concentrations to determine average annual BOD and TSS loadings from the two treatment facilities to the Blackstone River.

This methodology for design events and continuous simulations is also used in Section 9 to evaluate the effectiveness of potential alternatives. In addition, the cost of the alternatives is compared to BOD and TSS reductions, in order to form a basis for establishing their cost effectiveness.

The impact of stormwater on total loadings to the Blackstone River is not explicitly considered. Stormwater loadings from Worcester's stormwater drainage system are discussed in detail in Worcester's Phase I Stormwater Permit Application, 1993. As a result of that application, the City now has in place a strong stormwater management plan consisting of best management practices designed to reduce pollution from its stormwater system.

7.3 Performance of Existing CSO Control Facilities

This section discusses the effectiveness of existing CSO control facilities in terms of water quality improvements.

7.3.1 Comparison of System Performance Before and After Construction of Existing CSO Control Facilities

Past efforts to control CSOs in Worcester have been very successful in reducing the impact of CSOs on the Blackstone River. Table 7-1 compares the performance of the system before and after the construction of CSO controls. It clearly demonstrates extraordinary improvements in system performance.

Table 7-1
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
Estimated Annual *Untreated* Overflow Volume (Million Gallons)	1,300	0
Estimated Annual *Treated* Volume (Million Gallons)		
Secondary Treatment at UBWWTF (Million Gallons)	0	1,218
Treatment at QCSOSTF (Million Gallons)	0	82

7.3.2 Design Storm Performance

Table 7-2 demonstrates how the system performs during the design storms. The table compares the volume treated at the QCSOSTF with the volume treated at the UBWWTF during two-day periods. It also provides estimates of BOD and TSS loadings, based on typical effluent concentrations during wet weather at the UBWWTF (see Table 6.3-2 from the October 2001 UBWPAD Regional Wastewater

Treatment Facilities Plan) and typical effluent concentrations from the QCSOSTF as given in monthly Discharge Monitoring Reports from 1995 to 2000. Fecal coliform data are not provided as a measure of performance since the QCSOSTF and UBWWTF effluent is disinfected.

Given that during dry weather the UBWWTF treats about 27 mgd (or 54 million gallons in two days), the table demonstrates that the UBWWTF treats a large portion of the flow even during major storms. The table also demonstrates that even during very large storms, the QCSOSTF contributes comparatively minor flows and loads to the Blackstone River.

The one-month storm barely causes an overflow at the QCSOSTF. Storms smaller than the one-month storm (approximately a two-week storm) cause no discharge at the QCSOSTF, with 100 percent of the runoff receiving advanced treatment at the UBWWTF.

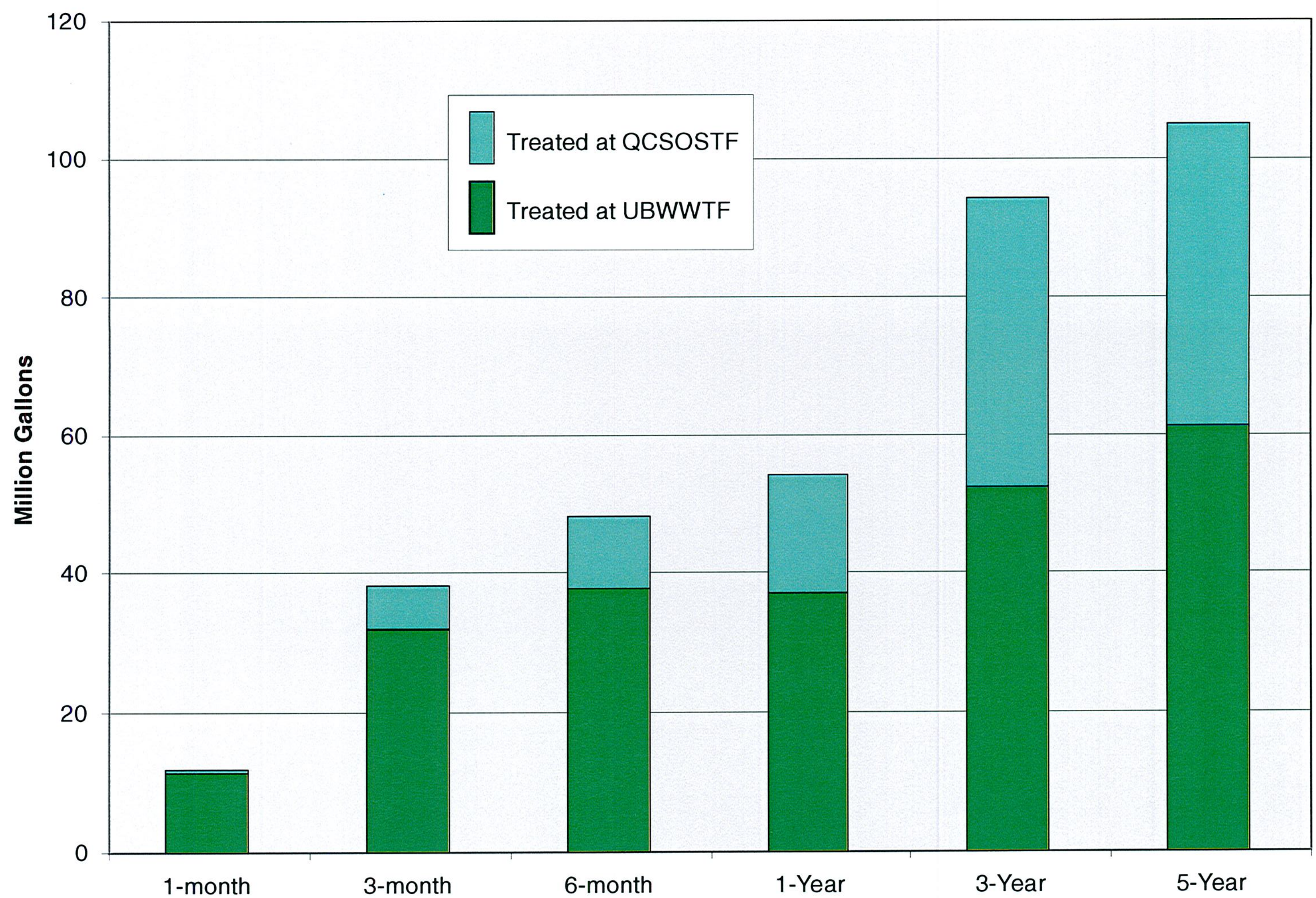
Table 7-2
Existing Conditions – Design Storm Performance Over Two-Day Simulation Period

Parameter	1-Mo.	3-Mo.	6-Mo.	1-Yr.	3-Yr.	5-Yr.
Volume Treated at QCSOSTF, MG	0.4	6.4	10.5	16.9	41.8	44.1
Pounds of BOD to Blackstone River	104	1,657	2,718	4,375	10,820	11,415
Pounds of TSS to Blackstone River	337	5,397	8,855	14,253	35,252	37,192
Volume Treated at UBWWTF, MG	88.7	90.2	93.4	88.1	109.4	108.8
Pounds of BOD to Blackstone River	12,591	12,804	13,258	12,506	15,529	15,444
Pounds of TSS to Blackstone River	22,219	22,595	23,397	22,069	27,405	27,254

Figure 7-1 shows the total volume of combined sewage generated from Worcester's combined sewer area during the design storms and whether it is treated at the UBWWTF or the QCSOSTF. Fully 97 percent of the combined sewage from a one-month storm is treated at the UBWWTF. Even during a 5-year storm, 59 percent of the flow from the combined sewer area is treated at UBWWTF.

As shown in Table 7-1, on an annual basis, 94 percent of combined sewage flow is treated at the UBWWTF. Comparing the QCSOSTF and the UBWWTF, the QCSOSTF contributes only 1 percent of the BOD and 2 percent of the TSS to the Blackstone River annually.

Based on annual and event flow volumes, annual and event loadings, level of treatment, and any other measurement of CSO performance, past improvements to Worcester's combined sewer system have been extremely effective.



Phase 1 CSO LTCP Report
Worcester, Massachusetts

Figure 7-1
Existing Conditions Where Combined Sewage Receives Treatment During Design Storms

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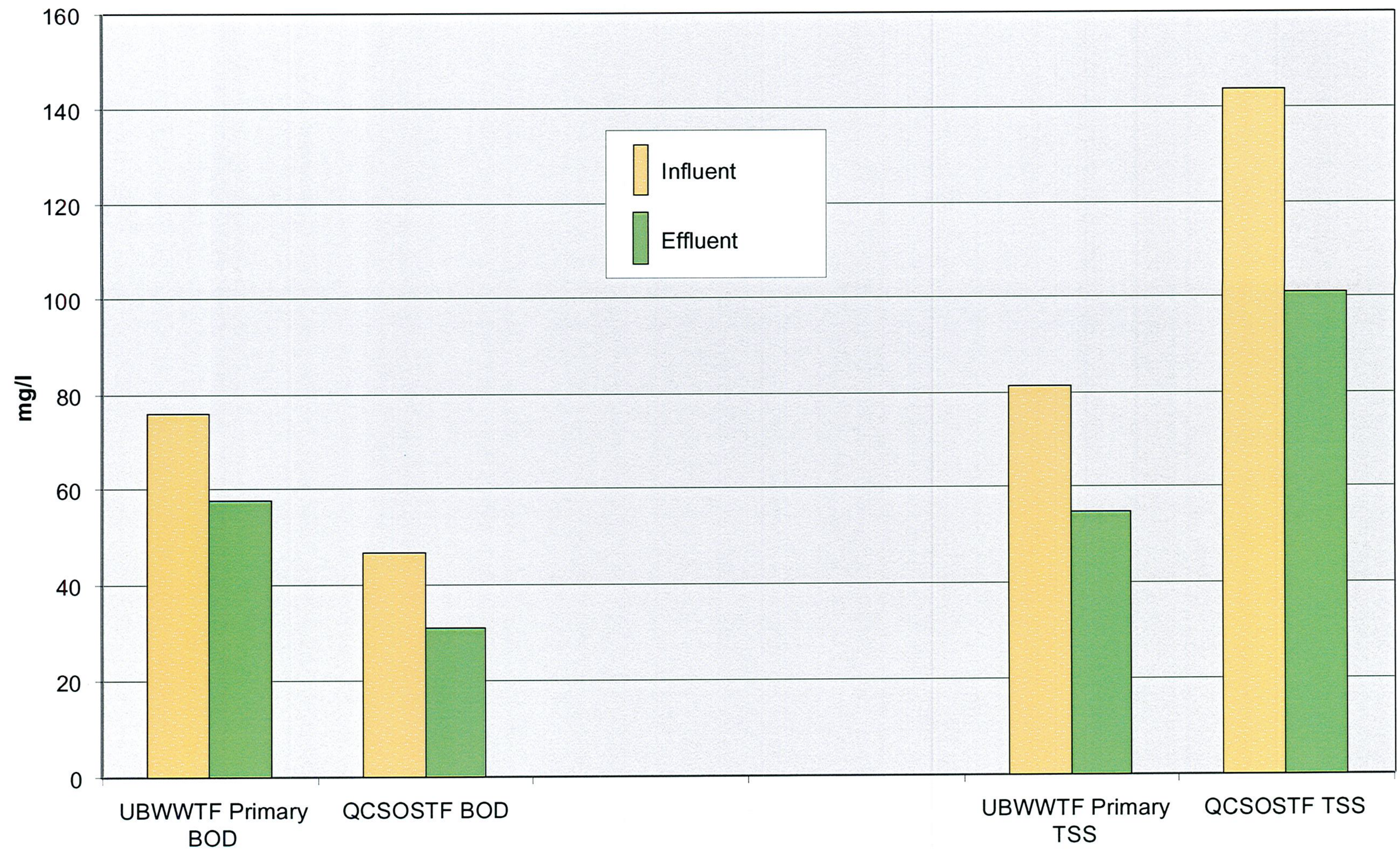

Phase 1 CSO LTCP Report
Worcester, Massachusetts

Figure 7-2
Comparison of UBWWTF Primary Quinsigamond CSOSTF BOD and TSS Influent and Effluent

CDM

7.3.3 Treatment Performance of the QCSOSTF

The Discharge Monitoring Reports (DMRs) for the QCSOSTF from January 1995 to December 2000 were examined to assess treatment performance for BOD and TSS removal. The performance was compared to the performance of primary treatment at the UBWWTF during high flow events to determine QCSOSTF effectiveness compared to a primary treatment facility.

Small and Medium Storms

As discussed above, assuming approximately 100 runoff events per year, only 12 cause discharges from the QCSOSTF, thus 88 events are fully treated at the UBWWTF.

BOD and TSS Effluent Concentrations and Removal Efficiency

For the 1995-2000 period, there were 58 events where concurrent 4-hour composite influent and effluent BOD measurements were made at the QCSOSTF. The data were analyzed by establishing the flow weighted mean of the influent and effluent samples. The mean influent and effluent BOD was 47 mg/l and 31 mg/l, respectively, representing an average removal efficiency of 34 percent. For TSS, there were 73 events in the data set. The mean influent and effluent TSS were 144 mg/l and 101 mg/l, respectively, a 30 percent removal efficiency.

The UBWWTF Facilities Plan 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 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 removal efficiency.

Figure 7-2 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 very 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.

First Flush

The data collected from the DMRs include four-hour composite samples of influent and effluent for BOD and TSS. Samples are analyzed only if and when the QCSOSTF discharges for four or more hours. It is widely accepted that most runoff events are characterized by a "first flush" effect, where the runoff at the beginning of the storm is washing off pollutants that have built up since the previous storm event. In the case of the QCSOSTF, portions of the entire first flush are temporarily stored at the QCSOSTF then pumped to the UBWWTF before influent sampling begins. Consequently, the most contaminated wet weather flows, known as the first flush, are not included in the sample analysis and removal efficiency analysis at the QCSOSTF. Thus, it is likely that the QCSOSTF actually performs better than the data indicate.

7.3.4 Performance of the CSS During Flood Events

The QCSOSTF has a capacity of 350 mgd. Flows exceeding 350 mgd are bypassed untreated to the Blackstone River. However, this rarely occurs. Since the facility went on-line in 1989, there have been only two bypass events during Hurricane Bob in August 1991, then during a 3.2-inch rainstorm on September 25^{th} and 26^{th}, 1991.

To establish the frequency of bypass events, the largest storms from the historical record (from 1958) were evaluated using the SWMM model. Based on the results, had the facility been on-line since 1958, 7 overflow events would have occurred. They include Hurricane Donna in 1960, the July 16, 1979 flood, and Hurricane Bob in 1991, and four other smaller storms that caused very small bypasses. These four smaller events were rerun assuming the planned improvements at the UBWWTF (see Section 9.3) are in place. Under these conditions, the four smaller storms would no longer cause bypasses at the QCSOSTF, and only 3 storms in the 44-year record would cause a bypass.

Based on this analysis, it will take approximately a 15-year event to cause a bypass at the QSCOSTF. This is a very high level of performance, considering untreated overflows are often allowed during 3-month storms in other approved facilities plans.

7.4 Future (Baseline) Conditions

The CSS and the Upper Blackstone Water Pollution Abatement District (UBWPAD) treatment system will undergo significant changes in the future (2020). As explained in the UBWPAD Regional Wastewater Facilities Plan, the population of the existing service area is expected to grow, and the size of the service area is expected to expand. This will result in increases in flows and loads requiring treatment at the UBWWTF. Table 7-3 compares existing and baseline flow volumes in the service area. The table shows what will happen if service area expansion occurs, but no improvements are made to either the UBWWTF or to the combined sewer system. The impacts will be much greater on smaller storms than larger storms. Overflow volumes during design storms will increase by between 0.8 and 1.5 million gallons, a significant increase at the 1-month level (300 percent), but only a marginal increase to the 5-year storm (less than 3 percent).

Planned improvements at the UBWWTF as documented in the October 2001 Facilities Plan are not included in the baseline condition. As will be demonstrated later, these improvements will benefit the entire service area in general and the combined sewer system in particular. Thus planned UBWWTF improvements are included in later sections of this report as alternatives that improve combined sewer system performance. This is consistent with the CSO long-term control planning process, which typically evaluates WWTP expansion as an alternative.

Table 7-3
Comparing Existing and Baseline (2020) Conditions During Design Storms

Parameter	*1-Mo.*	*3-Mo.*	*6-Mo.*	*1-Yr.*	*3-Yr.*	*5-Yr.*
Volume Treated at QCSOSTF, MG						
Existing Conditions	0.4	6.4	10.5	16.9	41.8	44.1
Baseline Conditions	1.2	7.8	12	17.6	43.1	45.3
Volume Treated at UBWWTF, MG						
Existing Conditions	88.7	90.2	93.4	88.1	109.4	108.8
Baseline Conditions	119.0	120.2	119.7	115.0	136.6	135.5

Comparing Annual Performance During Existing and Baseline (2020) Conditions

Parameter	*Annual*
Number of Treated Overflows from QCSOSTF	
Existing Conditions	12
Baseline Conditions	14
Million Gallons/Year Discharged from QCSOSTF	
Existing Conditions	82
Baseline Conditions	83

7.5 Summary

Past efforts to mitigate the impacts of CSOs in Worcester have been highly successful. Combined sewage from small storms (approximately 2-week storms and less) are fully treated at the UBWWTF. All storms during a typical year are treated, either at the UBWWTF or at the QCSOSTF, where treatment efficiencies are on par with primary treatment. Even during very large storms, the majority of flow from the combined sewer service area is treated at the UBWWTF. The CSO facilities function well beyond typical design levels. It will take flood events at the 15-year level to cause a QCSOSTF bypass. This level of control exceeds expectations even for most separated sewer systems, where untreated sanitary sewer overflows (SSOs) occur much more frequently. Even combined sewer systems that are separated will often discharge untreated SSOs during 15-year flood events.

As population increases and the service area expands, the challenge for the future is to ensure the performance of the CSS facilities remains high, or even improves beyond their current performance.

Section 8
Assessment of CSO Abatement Technologies

8.1 Introduction

This section describes available CSO abatement technologies and assesses their applicability to the City of Worcester CSO control objectives. Many alternative strategies are available to control pollutants discharged from CSOs ranging from no action to complete separation of the combined sewer system into separate sanitary and stormwater systems. This assessment considers technologies presented in EPA guidance manuals and other technologies considered appropriate for application in Worcester. The purpose of this assessment is to initially select appropriate alternatives for further evaluation and comparison in Section 9.

This screening of technologies and alternatives is based on current conditions and existing facilities in Worcester. Applicable technologies that will build on the current high level of CSO control will be considered further. As described previously, Worcester already has separated significant portions of the city, and constructed five overflow collector conduits that feed the QSCOSTF. All flows from the combined sewer system are treated either here or at the UBWWTF. Consequently, some technologies that may be applicable for combined sewer communities without this infrastructure in place would not provide the same benefit in Worcester.

8.2 Screening of CSO Abatement Technologies

To organize the screening analysis, potential CSO abatement technologies were grouped according to shared characteristics, as follows:

- Improvements to optimize hydrologic response of the system.
- Improvements designed primarily to store flows.
- Improvements that enhance system conveyance.
- Improvements for better treatment.

This section identifies and screens these technologies. It goes beyond the general source control measures and collection system controls identified in the EPA guidance manuals. Many of these general measures are typical best management practices (BMPs) that are already performed by the City of Worcester as discussed in the Report On Nine Minimum Control (NMC) Measures. Consequently, these NMC measures are not discussed further in this report, unless considered as part of a LTCP alternative.

Each applicable technology is described below and evaluated in general terms of effectiveness and feasibility in Worcester. Technologies that are infeasible for implementation, or that offer no benefit to the CSO mitigation program were eliminated from further consideration. Technologies that should be considered as Long-term CSO mitigation alternatives are evaluated further in Section 9.

Table 8-1 lists the CSO abatement technologies considered for this report and identifies the results of the technology evaluation/screening. This table also includes NMC/BMP type controls for completeness, even though these controls are not discussed as part of this report. For more information on these NMC/BMP controls, please refer to the Nine Minimum Control Measures report (CDM, May 2001). The technologies have been categorized as follows:

- Technology Not Feasible. These technologies will not work effectively in Worcester or will not improve water quality.
- Continue Current Practice. These technologies are typical best management practices and were, for the most part, addressed in the Nine Minimum Controls report submitted in May 2001. These technologies will help to optimize system operations and minimize CSO discharges and impacts to the rivers.
- Potential LTCP Technology. These technologies are feasible structural controls that can reduce and/or eliminate Worcester CSO discharges and impacts.

8.3 Hydrologic Response Improvements

Alternatives in this category generally lead to improved hydrologic response of the combined sewer system through removing inflow sources and thereby reducing the volume and peak flows during storm events. Such quantity control measures would reduce the amount of flow entering the overflow collectors in Worcester for pumping to the Upper Blackstone WWTF or storage and treatment at the QCSOSTF. This in turn would free capacity at both the UBWWTF and the QCSOSTF for more effective CSO control. The alternatives include full sewer separation, partial sewer separation, downspout disconnection, catch basin modifications, urban parks/green spaces, and infiltration sumps.

8.3.1 Sewer Separation

Sewer separation is defined as the reconstruction of an existing combined sewer system into non-interconnected sanitary and storm sewer systems. The sanitary sewer system is tributary to the wastewater treatment facility, and the storm sewer system discharges directly to the receiving waters.

Typically, to separate an existing combined sewer area, either a new drainage system is constructed or new sewer pipelines are installed and the existing combined sewer is used as the sanitary or separate storm drain, respectively. If portions of the Worcester combined sewer system were found to be susceptible to structural failure, they would likely require complete replacement and two new pipes would need to be constructed for the separate sewer and drain systems.

Table 8-1
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	

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Unlike storage and treatment alternatives, which reduce the frequency of CSO discharges, sewer separation eliminates CSOs by diverting all sanitary flow to the wastewater treatment facility. The EPA and DEP CSO abatement policies require that combined sewer system separation be evaluated as a step in CSO facilities planning. Although separation eliminates CSOs, it may not, in all cases, be the most appropriate alternative in terms of addressing site-specific water quality objectives. With separation, pollutant loadings to receiving waters caused by the sanitary flow are eliminated; however, impacts caused by stormwater borne pollutants are not. In the case of Worcester, CSOs currently are treated at the UBWWTF or the QCSOSTF prior to discharge, significantly reducing pollutant loading from this source and weakening the argument for full separation.

System wide sewer separation, although costly and arguably a step backwards in terms of improving receiving water quality, is considered as a potential long-term control plan technology and will be discussed further in Section 9.

Also discussed in Section 9 as a viable alternative is the concept of partial separation. Partial separation targets specific areas where the CSO reduction benefits may outweigh the construction costs and other impacts. Two areas that are considered viable for partial separation projects include separating the Green Hill Pond and Bell Pond flows from the combined sewer system, and separating areas tributary to the 96-inch Shrewsbury Street drain. The 96-inch pipe drains stormwater and pond flows primarily, but also acts as a relief sewer for smaller combined sewers in Shrewsbury Street. It is currently connected to the Harding Street Overflow Collector, but if the combined sewer relief connections could be removed, the 96-inch drain may be connected to the 8.5'x11' twin box stormwater conduit. These partial separation projects are discussed further in Section 9.

8.3.2 Downspout Disconnection

In urban areas, such as Worcester, roof leaders from gutters or roofs are often connected to the combined sewer system. Direct connection to the system avoids excessive surface runoff across properties to the catch basins or street drainage collection system. In winter, this helps prevent ice build-up on driveways, sidewalks, and roadways. However, these direct inflow connections increase the peak flow rates and volumes during storm events by decreasing the time of concentration within the drainage basin and preventing some of the runoff from infiltrating into pervious surfaces.

Roof leader disconnection is most effective when roof leaders can be re-directed to pervious surfaces, where the runoff can infiltrate into the soil, thereby reducing the volume of runoff that enters the combined sewer system. The combined area of Worcester is highly urbanized with little pervious area available for disconnected roof drain flows to percolate into the soil.

Also, there is no direct discharge point, such as a river, into which the disconnected flows may be directed. Instead, disconnected flows would need to be routed to the twin 8.5′ x 11′ drainage conduit. In many cases, this would require constructing siphons beneath the large overflow collector separating the eastern portion of the combined from the drainage conduit.

In light of this discussion and the significant administrative burden and public relations component associated with downspout disconnection, this alternative will no longer be considered.

8.3.3 Catch Basin Modifications

Modifications to existing catch basins can be made to reduce peak stormwater inflows to the combined sewer system. Catch basins within a drainage area can be retrofitted with devices, such as a vortex valve, that will retard the surface water runoff entering the sewer system. The flow may backup in catchbasins lacking capacity and divert overland to a downstream catchbasin with available capacity. This is commonly referred to as flow slipping, and has been successfully implemented in several cities across the country.

Such inlet control devices can also be designed to provide a level of surface detention in flat areas to reduce peak flows entering the combined system. Surface detention has been implemented in streets, parking lots, and parks using berms to trap storm water temporarily until capacity opens up in the combined sewer system. There are risks, however, linked to street and parking lot detention, particularly during the winter months when snow and ice raise further concerns. This risk is of particular concern in highly urbanized areas subject to severe winter weather such as Worcester. In addition, much of the combined sewer area is flood prone. Insufficient catch basin capacity has been identified as one of the reasons the area is subject to flooding during high intensity events. Reducing capacity is counter-productive to flood mitigation efforts.

In light of the above discussion, catch basin modifications restricting inflow are not considered feasible in Worcester.

8.3.4 Urban Parks and Green Spaces

The construction of urban parks and green spaces would help reduce impervious area draining to the combined sewer system, thereby decreasing the volume and peak flows of CSOs. These features would promote infiltration and groundwater recharge, while providing attractive recreational areas for the community. Planting of trees and other vegetation would also add to the effectiveness of these areas in decreasing inflow to the combined sewer system. Conceivably, these areas could also provide a level of surface detention with the proper landscape design and implementation of inlet control devices, as discussed above.

However, there are few realistic opportunities for construction of new parks and green spaces in the highly developed urban combined sewer area of downtown Worcester. The use of existing parks is not a practical solution due to community impacts. Consequently, this alternative will not be considered further as a CSO control technology.

8.3.5 Infiltration Sumps

The construction of infiltration sumps represents a technology highly dependent on subsurface soil conditions. Infiltration sumps, which are essentially large, deep catchbasins with no pipe outlet, collect storm water and allow it to infiltrate into the soil. Consequently, these relatively low-cost structures are not effective in areas with soils exhibiting low permeability. It is not expected that the soils in the urban areas of downtown Worcester will be favorable for implementation of infiltration sumps. Consequently, this technology will not be considered further.

8.4 Storage Alternatives

Storage of CSO flows can be performed either at a local site adjacent to a regulator or other control device or downstream at a central site that consolidates the need for several facilities. Storage facilities are typically used to store CSO discharges for eventual treatment at the WWTP after the storm. However, storage facilities can also be designed to provide some sedimentation treatment capacity for flows greater than the storage volume.

The QCSOSTF is a storage and treatment facility for combined sewer overflows. It provides 2.5 million gallons of storage in two chlorine contact tanks. The effluent gates to the tanks are only opened when capacity is not available at the UBWWTF to receive pumped flow from the QCSOSTF. The gates are closed at the first sign that the storm is receding. This measure minimizes the amount of treated discharge at the QCSOSTF, and maximizes storage at the QCSOSTF until capacity becomes available at the UBWWTF. In-line storage is maximized in the Overflow Collectors as a function of how the QCSOSTF operates. The operating water surface elevations allow flows to back up in the Overflow Collectors, which act as the wet weather wet well for the QCSOSTF.

Storage technologies generally represent larger, more costly structural modifications to a combined sewer system. These technologies are presented below and include inline storage and off-line storage.

8.4.1 In-Line Storage

The use of inline storage is considered a cost-effective method of reducing combined sewer overflows by utilizing available pipeline storage volume. The storage volume helps to both dampen peak flows and detain combined wastewater for later treatment at the WWTP. Inflatable dams, control gates or other devices, such as weirs, can be used to create or optimize inline storage in existing pipes during a rainfall event.

Also, new oversized pipes may be constructed in the interceptor or collection system for improved inline storage capacity.

Utilization of control devices provides a means to optimize inline storage depending on capacity needs. The intent of the control devices, which can be remotely controlled, is to hold back wet weather flows in the combined sewers to delay flow peaks in the downstream flow reaches. The devices may also be used to maximize flow into the interceptor system before overflowing to the receiving water. In both cases, the goal is to maximize inline storage in either upstream or downstream sewer reaches with available capacity. This controlled surcharging of the sewer system can be effective in minimizing CSOs through maximizing inline system storage. When relief is needed to avoid upstream flooding, the devices may be deactivated either manually or through an automated system to release the stored flow.

Inline storage technologies may increase combined sewer system cleaning and maintenance requirements since the storage of flow in pipes reduces flow velocities and increases the possibility of solids deposition. Surcharging pipes may also lead to more frequent rehabilitation requirements, particularly in older reaches of pipe.

Inline storage can be a viable CSO abatement technology if the existing sewer system pipelines are large enough and deep enough to provide significant storage volume. Pipes which are steeply sloped require numerous flow control devices at regular grade changes to maximize the use of available storage. With numerous flow control devices, inline storage is more difficult to control and less cost-effective than downstream controls.

Kelly Square Control Station

Based on evaluation of the Worcester combined sewer system, most pipe reaches are already utilized at maximum capacity. However, some areas, such as the Western Interceptor, and upstream portions of overflow collectors may be more fully used for storage or conveyance. This potential will be evaluated further, especially with respect to the Kelly Square Control Station. This structure was designed to divert flows in the Harding Street Overflow Collector to the Western Interceptor. It also is equipped with a leaf gate for storing flow in the upstream reaches of the overflow collector. The leaf gate has not been used to date due to uncertainty associated with system hydrology and the hydraulic response. Nonetheless, the capability is there and its use will be evaluated further as a potential CSO control technology.

Constructing similar control structures in other reaches of the overflow collectors will also be considered to take advantage of storage capacity in the large conduits. The effectiveness of a remotely controlled gate structure installed in the reach of pipe downstream of the slope break in the Harding Street Overflow Collector is of particular interest.

Real Time Controls

Diverting flow from one drainage basin having limited hydraulic capacity to a drainage basin having excess capacity can reduce the volume and frequency of CSO discharge. Available and existing pipeline capacity may be used to convey flow or used as inline storage. Components include: a data gathering system to monitor rainfall, pumping rates, treatment rates and regulator positions; a central computer processing center to provide real time control; and an instrumentation and control system to remotely regulate pumps, gates, valves and regulators. Real-time control programs are available for providing system-wide control or localized control. System-wide, real-time control is capable of providing integrated control of regulators, gates, and pump station operations based on anticipated flows from rainfall events and actual flow conditions within the system. System response to control commands may be evaluated prior to execution through the use of computer models linked to the real-time control system. Such global real-time control provides a means for optimization of the entire collection system (i.e. maximized flow to the treatment plant for maximized treatment) for a variety of conditions. Local real-time control impacts specific dynamic regulators, based on feedback control from flow monitoring points located upstream or downstream of the point of interest.

The Worcester combined sewer system is equipped with flow sensors. However, the sensors have not been used to their full potential and their functionality is in question. Nonetheless, real-time control, particularly with respect to the Kelly Square Control Station and Harding Street Overflow Collector in-line storage structures, will be evaluated further as a CSO control technology for Worcester. The cost of implementation (if further implementation is required), training, and maintenance will be considered as well.

Regulator Modifications

Modifications to the operation of regulators also represent a form of in-line storage. Regulators can be modified to pass more flow through to the interceptor or to take advantage of upstream pipeline storage. Such measures maximize the flow to the treatment plant through the interceptor system; however, in Worcester they do not prevent untreated CSO discharges to the receiving water due to the CSO control infrastructure in place. This infrastructure, consisting of five overflow collectors and the QCSOSTF, allows overflows occurring at regulators to be captured and treated, either at the UBWWTF if capacity is available, or at the QCSOSTF, prior to discharge. Consequently, regulator modifications will not necessarily lead to improved CSO control. However, they may improve operating efficiencies since fewer flows would need to be pumped from the QCSOSTF to the UBWWTF.

All of the existing regulators in the Worcester system function as static flow control devices, either as weirs or orifice controls. Therefore, overflow weirs could be raised to decrease the potential CSO discharge to overflow collectors. These modifications are relatively low in cost and simple to implement.

In general, downstream conveyance and in-line storage capacity is well utilized in the Worcester interceptor system during most storm events, with few bottlenecks. The Western Interceptor has capacity available resulting from separation projects in areas tributary to this interceptor. Flows are currently transferred from the Harding Street Overflow Collector to the Western Interceptor via the Kelly Square Control Station to maximize use of the Western Interceptor. Other than this, the effect of passing more flow through one regulator to reduce CSO discharges locally may increase CSO discharges at the next regulator downstream. Also, raising weirs could lead to increased risk of flooding in the upstream sewer collection system.

Nonetheless, regulator modifications will be considered further as a potential CSO control technology since the benefits could be significant for a relatively low cost improvement. Weir modifications will need to be evaluated in terms of flooding risk. Furthermore, modifications to the Kelly Square Control Station, which effectively transfers overflows from the capacity-constrained Eastern Interceptor to the Western Interceptor, as described above, will be considered further. Also, utilizing the capabilities of the station to store upstream flows in the overflow conduit will be evaluated.

8.4.2 Off-Line Storage

Off-line storage and pumpback to the interceptor system is an effective method for CSO mitigation. Similar to inline storage, off-line storage facilities temporarily store wet weather overflow volumes until the flow can eventually be conveyed and treated at the WWTP. Types of storage facilities include underground tanks or conduits, abandoned pipelines, or deep tunnels. Off-line storage is usually located at individual overflow points for storage of localized CSOs. The storage facilities may also be located near dry weather or wet weather treatment facilities or pump stations in lower reaches of the system, where the off-line storage would consolidate overflows conveyed in the collection system from upstream locations. These facilities can be relatively simple in design and operation and can effectively reduce the frequency of overflows.

Storage facilities can also be designed to remove settleable solids, with periodic cleaning by dredging, mechanical chain and flight scrapers, or other means. In effect, some primary treatment (sedimentation) takes place due to quiescent conditions. The settled solids can typically be handled by collecting and pumping to the interceptor to be handled at the WWTP after the event.

Excessively long detention times can result in the settled solids causing offensive odors. Accordingly, prompt solids removal is necessary along with proper odor control equipment.

The Worcester combined sewer system is equipped with five overflow collectors and the QCSOSTF, each providing a significant degree of storage. The overflow collectors consolidate overflows from the combined sewer regulators and convey them to the

QCSOSTF. The QCSOSTF provides 2.5 million gallons of storage in its chlorine contact basins. The operation of the QCSOSTF limits treated discharges to the Blackstone River and maximizes storage in the contact tanks and the overflow collectors, which act as a large wet well. Off-line storage facilities, including maximized use and/or expansion of existing facilities and addition of new facilities, will be further evaluated as a potential CSO Long-term control technology, as discussed in Section 9. Specifically, the addition of new storage tanks at the QCSOSTF will be considered as a consolidated storage site.

8.5 System Conveyance Improvements

These alternatives result in improved system conveyance to the UBWWTF, which reduces the volume of treated discharges at the QCSOSTF. These alternatives involve pumping improvements at the QCSOSTF that allow more flow to be pumped from the QCSOSTF to the UBWWTF as well as utilizing available capacity in the interceptor system.

8.5.1 Pump More Flow from the QCSOSTF to the UBWWTF

This alternative involves routing more flow from the QCSOSTF to the UBWWTF through pump operation and/or capacity modifications. As discussed under the treatment improvement alternatives section, facilities planning was completed in October 2001 for the expansion of the UBWWTF. This expansion will allow more flows to be pumped from the QCSOSTF to the interceptor system for treatment at the UBWWTF. Currently, 19.9 mgd is pumped from the QCSOSTF during wet weather events until flows approach the secondary treatment capacity of the UBWWTF (54-70 mgd, depending on plant conditions). With the expansion, it is expected that pumping may continue from the QCSOSTF to the UBWWTF until influent flows at the UBWWTF approach 140 mgd. Increasing the pumping capacity at the QCSOSTF will also be evaluated as a potential LTCP alternative.

8.5.2 Divert More Flow to the Western Interceptor

Separation of areas tributary to the Western Interceptor in the 1980's and 1990's has increased the available conveyance capacity of this interceptor. As discussed in Section 8.4.1, the Kelly Square Control Station was designed to divert flows from the Harding Street Overflow Collector to the Western Interceptor to take advantage of the available capacity. The potential to increase the amount of flow diverted to the Western Interceptor through the Kelly Square Control Station connection will be evaluated as a potential LTCP technology.

8.6 Treatment Improvements

Treatment technologies target reduction of pollutant loads in combined sewer overflows to receiving waters. Specific technologies address specific pollutants, such as suspended solids, floatables, chemicals, or bacteria. Treatment residuals must be accounted for as part of the implementation plan. Technologies used for treating CSOs prior to discharge are presented below.

It is noted that Worcester has already implemented a CSO storage and treatment facility. This facility, which has been in operation since 1989, provides screening, storage, sedimentation, disinfection and dechlorination. The facility is capable of treating a peak flow of 350 mgd and storing 2.5 million gallons in its chlorine contact tanks. Enhancement of the QCSOSTF and its capacity for storage and treatment will be evaluated further as part of the long-term control planning process. In addition, the UBWPAD examined high flow management alternatives as part of the facilities planning for upgrades to the UBWWTF. Alternatives development for CSO control have been coordinated with the UBWPAD Regional Wastewater Facilities Planning.

8.6.1 Wastewater Treatment Plant Expansion

Expanding the capacity of the existing wastewater treatment plant to handle higher peak wet weather flows would reduce the frequency and volume of untreated CSO discharges upstream of the treatment plant. A variety of high flow management alternatives were considered as part of an improved wet weather management plan at the UBWWTF. The recommended alternative expands the preliminary and primary peak treatment capacity from 119 to 160 mgd and upgrades the advanced treatment capacity to between 80 to 120 mgd. The specific advanced treatment capacity will be determined during preliminary design. Wet weather flows exceeding the advanced treatment capacity would receive preliminary and primary treatment and disinfection but would be routed around the advanced treatment train to minimize upsets of the biological system during peak flow events. The wet weather flows not receiving advanced treatment would be blended with the advanced treatment effluent. An analysis performed in the Wastewater Treatment Facilities Plan determined that NPDES permit limits would consistently be achieved even with the blended effluent. In addition to the treatment capacity improvements discussed above, two of the proposed total of eight primary clarifiers could be used as in-line storage during high flow events.

This alternative represents an attractive means for reducing CSOs and maximizing flow through the treatment plant, and will be evaluated further as a CSO control alternative.

8.6.2 Screening

Screens can be installed at either inline or at off-line facilities. Inline facilities must be closely monitored and cleaned to prevent loss of hydraulic capacity, which could cause flooding. Screens for wastewater treatment are available in various types and sizes ranging from bar racks to coarse/fine screens or microstrainers. Screens are effective in removing large solids and floatables from the wastewater flow -- the effectiveness being dependent on the clear opening of the screen. The size of the screen openings determines the level of treatment achieved. Microstrainers can achieve primary treatment levels by removing 60 percent of the suspended solids but lead to high headlosses.

Bar screens are almost always installed at the entrance to storage and treatment facilities for removal of large objects, trash and debris, and to protect treatment and pumping equipment. Often two sets of screens in series are used. The first set usually consists of coarse screens with 1 ½ -inch bar spacing. Finer screens with ¼-inch to 1-inch spacings are located just downstream. A double screen set-up is also less likely to be blocked than one fine screen.

Screening is a viable treatment alternative to meet CSO control strategies. The QCSOSTF currently employs two one-inch opening mechanically cleaned bar screens at the inlet to the facility. Since screening is already implemented at the QCSOSTF, this technology will not be considered further.

8.6.3 Sedimentation

Gravity sedimentation using high surface overflow rates (to conserve space) can achieve 20 to 40 percent removal of BOD and 50 to 70 percent removal of TSS in CSOs. Land requirements and residual solids handling are important considerations in determining the feasibility of sedimentation.

Sedimentation reduces solids loadings from CSOs by gravitational settling and removal of suspended solids. As a result, metals and BOD loadings are also reduced. In addition, the process is used in many wastewater treatment applications providing an extensive base of full scale operating data.

The major disadvantage of sedimentation is that the land requirements are relatively high. Because the availability of land is usually limited in urban areas, siting of CSO abatement facilities that include sedimentation basins can be an important issue.

Because experience has shown sedimentation to be a reliable, cost-effective CSO abatement technology, expansion of existing chlorine contact tank volume by addition of additional tanks for improved settling and storage will be considered as a means of enhancing treatment processes at the QCSOSTF.

8.6.4 Enhanced High-Rate Clarification

A relatively new concept for treating storm flows is enhanced high-rate clarification. This technology, which could be operated intermittently during storm conditions, is essentially a physical-chemical process in which ferric chloride and polymer are added to the wastewater. The resulting floc particles adsorb onto either very fine sand added to the wastewater, or re-circulated solids. If fine sand is used, it is removed in the sludge and reused. The fine sand or recirculated sludge acts as ballast and increases the settling rate of the adsorbed floc. The process is also known as "ballasted flocculation".

A typical ballasted flocculation system consists of addition of ferric chloride, polymer, and "microsand" (sand approximately 100 microns in diameter) to the wastewater. The wastewater and additives are rapidly mixed (flash mixing) and then slowly stirred in a maturation tank before settling in a clarifier. The sludge from settling is passed though a hydrocyclone, where the microsand is removed from the sludge and recycled.

At least three suppliers provide enhanced high-rate clarification systems. Suppliers of the ballasted flocculation process include US Filter Kruger, with their Actiflo process, and U.S. Filter, which supplies the Microsep process. Ondeo Degremont provides a high-rate clarification process using recirculated solids called DensaDeg 4D.

Whichever process is selected, the BOD and TSS removal rates associated with high-rate clarification have been shown to be roughly double those of traditional clarification, while the area requirements are only one-tenth of the traditional area requirements.

In summary, enhanced high-rate clarification is able to provide significantly higher treatment capacities than conventional primary treatment with significantly higher BOD and TSS removals. Therefore, enhanced high-rate clarification is considered a potential alternative to evaluate further for providing expanded wet weather treatment flow capacity at the QCSOSTF. However, potentially higher O&M requirements associated with this technology may limit its applicability.

8.6.5 Swirl and Helix Concentrators

Swirl regulators/concentrators operate as a solids/liquid separator removing both suspended solids and floatables through rotationally induced forces. Swirls have been reported to remove up to 50 percent of the suspended solids from the combined sewer flow. Helical concentrators are similar in design but are more effective as an inline device (rather than an off-line device). The flow is separated into overflow, which is discharged to the receiving water (typically after chlorination) and underflow (a concentrated low volume of wastewater that is intercepted for treatment at a treatment plant).

The swirl/helical bend concept has several advantages over other treatment/storage options including:

1. The unit regulates flow to the interceptor systems and treats CSO discharges;
2. Land area requirements are lower than conventional sedimentation or off-line storage;
3. The unit contains virtually no mechanical equipment and, because solids remain in suspension, no removal facilities are required; and
4. Low operations and maintenance costs.

Swirl and helical bend concentrators represent a potentially low cost and efficient technique to regulate and treat combined wastewater. These units, however, have some limitations and potential drawbacks, including:

1. The rate of underflow diversion is subject to design limitations relative to the incoming combined flow;

2. The relatively short detention time requires high rate disinfection or construction of contact tanks to provide adequate detention time for bacteria kill before discharge to the receiving water;

3. The configuration of the swirl concentrator results in a large hydraulic headloss requirement between the influent combined sewer and the underflow pipe; and

4. There is relatively little long-term data on performance and reliability.

Drawbacks 1 and 3 above can be satisfied by storage and pump back facilities in conjunction with a concentrator. Interceptor and treatment capacity must be available for underflow during a storm event. If underflow rates exceed the available interceptor capacity or sufficient grade is not available, the underflow may need to be stored and pumped back following the storm.

The application of swirl/helical concentrators in Worcester would most likely need to be developed in-line with the QCSOSTF for enhanced treatment. This application poses several challenges and does not appear to be practical at this time considering the lack of data to demonstrate the effectiveness of the swirl/helical concentrators. Therefore, this technology will not be considered further in this study.

8.6.6 Biological Treatment

Biological treatment processes, including contact stabilization, trickling filters, rotating biological contactors, treatment lagoons, and land application, have been most successfully used in the treatment of sanitary sewage and industrial wastewater. Their exclusive use for the treatment of combined sewer overflows has several drawbacks including difficulty in maintaining the life of the biomass used to assimilate the nutrients in the combined sewage during dry weather, erratic loading conditions inherent to combined sewer overflows, costly operation and maintenance, and the need for highly skilled operators. Potentially, discharge of CSOs into wetlands may provide some level of biological treatment; however, this is not considered appropriate for the Worcester area. Consequently, biological treatment will not be considered further in this study.

8.6.7 Filtration

Filtration is a physical treatment process that removes solids by straining wastewater through a filter medium such as sand, charcoal (carbon adsorption), or membranes. Its major disadvantage is the tendency to clog rapidly during use, thus limiting its hydraulic capacity and ability to remove solids. It can be used after sedimentation to reduce clogging, but this level of treatment is typically not required for CSO applications. Consequently, filtration will not be considered.

8.6.8 Disinfection

Disinfection is used to destroy pathogenic microorganisms. Many disinfection technologies are available including chlorination, ozonation, and ultraviolet radiation. The most common method is chlorine addition, although its apparent toxicity to aquatic life is a concern. For this reason, dechlorination is often required.

The QCSOSTF currently employs sodium hypochlorite for disinfection and sodium bisulfite for dechlorination. Current practices are considered adequate and should be continued.

8.6.9 Summary of Treatment Technologies

No treatment technology alone is adequate to meet all water quality objectives. However, various combinations of treatment methods, as implemented at the QCSOSTF, and various levels of treatment plant capacity expansions may be used to meet CSO abatement goals. These concepts are discussed further in Section 9.

8.7 Summary

This assessment has eliminated many technologies from further consideration for the Worcester Long-Term CSO Control Plan. These eliminated technologies will not directly address the CSO impacts identified in the receiving water quality and use evaluation. Other technologies in this list have already been identified as recommended nine minimum control measures. These technologies will incorporate good maintenance practices to ensure that system operation is maximized to the extent possible before more expensive structural controls are implemented. The remaining controls are more significant in implementation and capital requirements and costs. These will discussed further in Section 9.

Section 9
Development and Evaluation of Alternatives

9.1 Introduction

This section presents the development and evaluation of alternatives that have passed the initial screening effort described in Section 8. For each alternative, a brief description and preliminary planning level cost estimate is provided followed by an evaluation of its CSO abatement and water quality benefits. The goal is to present a range of feasible alternatives with the costs and benefits associated with each. The alternatives that are considered feasible, cost effective, and beneficial will be further developed in Phase II of the Long-term Control Plan.

Planned improvements at the UBWWTF are given special consideration in this section. These improvements are an outgrowth of a separate facilities planning process, have been approved, are under design, and will be constructed. This section first describes UBWWTF improvements, and compares their water quality benefits to the Year 2020 Baseline Condition described in Section 7. Next, all other improvements are evaluated, assuming the UBWWTF improvements are on-line.

9.2 Screened Alternatives

The UBWWTF improvements recommended in the October 2001 UBWPAD Regional Wastewater Facilities Plan are evaluated further as a CSO control alternative in Section 9.3. The remaining alternatives that have passed the initial screening in Section 8 were grouped into four CSO abatement categories, are listed below and described further in Sections 9.4 through 9.7.

A. Improvements to Optimize Hydrologic Response of the CSS

- Sewer separation.
- Reduce flow to the overflow collectors through:
  - Diverting the 96-inch Shrewsbury Street connection from the overflow collector to the clean water conduit; and
  - Diverting flows from Green Hill Pond and Bell Pond and other separated areas upstream of Shrewsbury Street out of the CSS.
- Downspout disconnection program.

B. Improvements Designed Primarily to Store Flows

- Use existing Kelly Square gates to maximize storage in the overflow collector upstream of Kelly Square.

- Install a gate structure in the Harding St. Overflow Collector to maximize storage upstream of the "slope break" between the QCSOSTF and Grabowski Square.

- Optimize the CSS through regulator modifications (raise weirs to increase inline storage).

- Real Time Control.

- Increase the storage capacity at the QCSOSTF

C. Improvements that Enhance System Conveyance

- Pump from the QCSOSTF until flow in UBWWTF reaches 140 mgd.

- Increase pumping capacity at the QCSOSTF.

- Divert more flows into the Western Interceptor.

D. Improvements for Better Treatment

- High rate clarification (HRC) at the QCSOSTF.

These alternatives are evaluated further in this section in terms of feasibility and cost-benefit.

9.3 UBWWTF Expansion

A facilities plan was completed in October 2001 to identify planned improvements at the UBWWTF in Millbury, MA. 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, 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 capacity of the facility to 160 mgd, which matches the peak 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. A series of high flow management alternatives were considered, including storage and a wet weather flow split, with maximized flow receiving advanced treatment and flows in excess of advanced treatment capacity receiving preliminary and primary treatment and disinfection prior to blending with the advanced treatment effluent. The recommended alternative shown in Figure 9-1 was a wet weather flow split, intended to minimize upsets to the advanced treatment biological processes, with in-line storage. The current plan calls for injection of sodium hypochlorite at the upstream end of the wet weather treatment train and dechlorination at the downstream end. Chlorination and dechlorination details will be further refined in preliminary design.

The facility currently provides advanced treatment. The facilities plan also provides for a higher level of treatment, and for an increase in the capacity of the preliminary and primary treatment processes, so that larger volumes of water will be treated to a higher level. The recommended alternative expands the preliminary and primary peak treatment capacity from 119 mgd to 160 mgd. The advanced treatment capacity would be increased to between 80 to 120 mgd. The capacity of the advanced treatment system, and thus the capacity of the wet weather flow split, depends on the results of the ongoing biological phosphorous removal pilot evaluation. The ultimate capacity will be determined in preliminary design. For the purposes of this report, two alternatives were evaluated; first assuming advanced treatment capacity of 80 mgd (wet weather treatment train capacity = 80 mgd), and second assuming an advanced treatment capacity of 120 mgd (wet weather treatment train capacity = 40 mgd).

These improvements will significantly benefit the water quality of the Blackstone River since they will increase the UBWWTF's ability to accept flow from the QCSOSTF. Currently, when influent flows at the UBWWTF reach 50 to 70 mgd, the QCSOSTF stops pumping flow to the UBWWTF to avoid a hydraulic overload of the advanced treatment processes. The exact flow at which pumping is stopped varies depending on storm conditions and conditions in the advanced treatment train. By increasing the capacity to accept a peak hour flow of 160 mgd, the QCSOSTF may continue to pump to a higher cut off point. In addition, one or two of the primary settling tanks could initially be operated as peak flow storage tanks. The potential may also be available to use the existing abandoned trickling filters as off-line storage in the future, if necessary. For the purposes of this CSO facilities plan, it is assumed that the QCSOSTF stops 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. In some cases, pumping may be discontinued before the 140 mgd cut off point, especially if the pumping capacity at the QCSOSTF is increased as an alternative, as discussed later in this section.

The total estimated cost of the UBWWTF expansion is $99.1 million, with $10.5 million allocated for the wet weather flow split alone, based on December 2000 dollars. Portions of the headworks, primary settling tanks, and disinfection improvements totaling $34.6M could also be considered as part of the high flow management cost. However, some of these improvements were needed anyway.

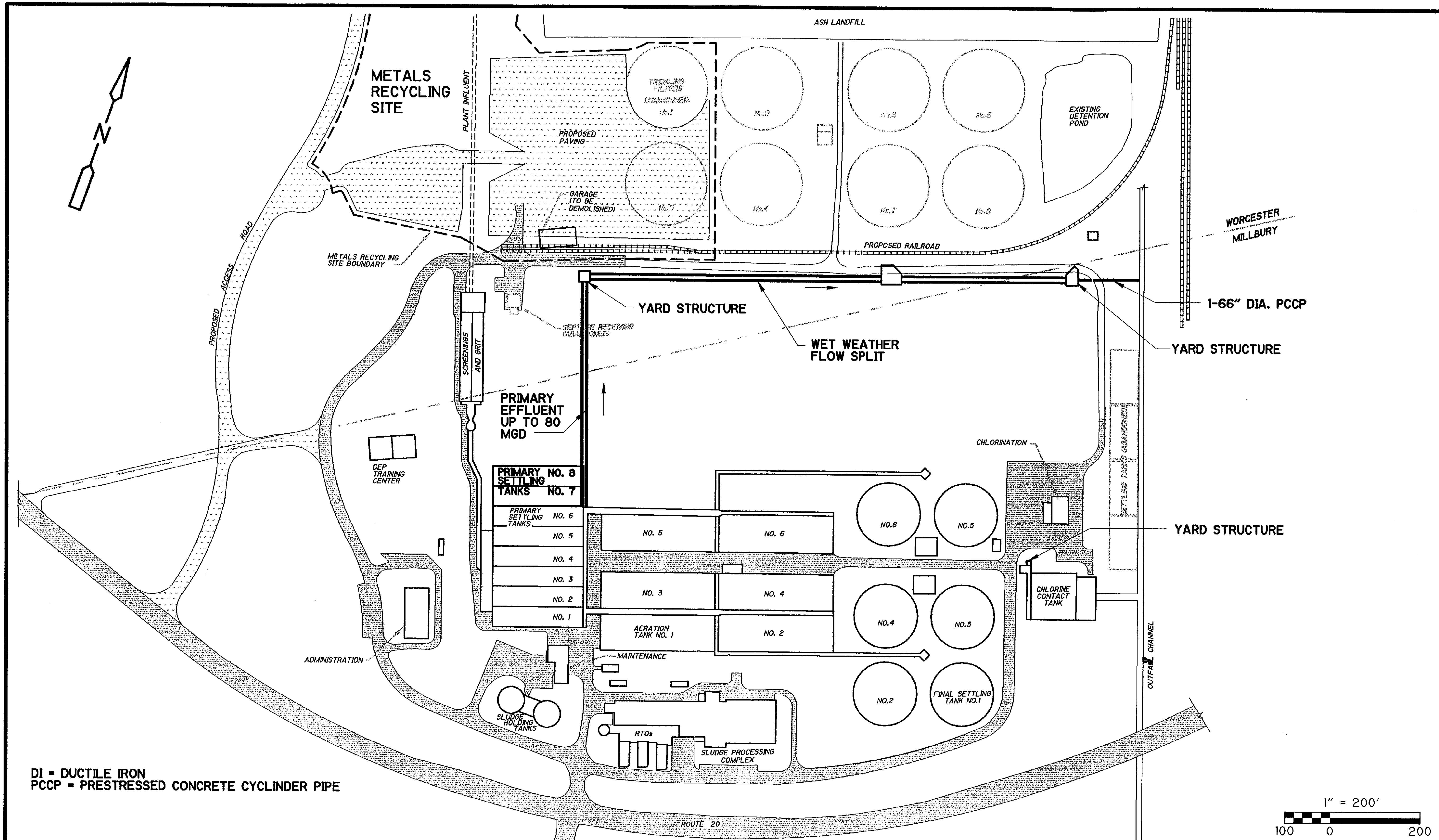

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FIGURE 9-1
UBWWTF HIGH FLOW
MANAGEMENT RECOMMENDED PLAN

Figure No. 10.2-1

Since it is difficult to breakout these costs in terms of high flow management versus non-high flow management improvements, this report only considers the $10.5M flow split cost in its analysis. The UBWWTF cost estimates include a 35 percent allowance for engineering and contingencies. Implementation of the high flow management improvements is planned for completion in 2006. Full compliance with the newly issued permit will be achieved in 2009. Other aspects of the expansion will be completed in subsequent years. If capital costs are escalated to the midpoint of construction, the total project amounts to $120 million with $11.8 million allocated for the wet weather flow split alone. It is noted that the overall expansion and enhanced treatment capability of the UBWWTF, and not just the high-flow treatment train, contributes to the improvement in high flow management and reduction in CSO impacts.

Further details are provided in the October 2001 UBWPAD Regional Wastewater Facilities Plan.

9.3.1 Water Quality Benefits

Table 9-1 presents the benefits associated with increasing the UBWWTF capacity to 160 mgd with a high flow management flow split. The table shows large improvements on both an event basis and an annual basis. The improvements entirely prevent 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 discharge every two months. Annual volume discharged from the facility decreases from 83 million gallons to just 34 million gallons. Figure 9-2 shows the volume reduction benefits at the QCSOSTF for the 1-, 3-, and 6-month design storms and costs for this and other alternatives that are examined in Section 9.9.

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 only the cost of UBWWTF high flow management facilities is considered ($10.5M), it will cost about $4 for every pound of TSS removed per year, and about $54 for every pound of BOD removed per year. It also represents a cost of about $2.76 per gallon of discharge reduced from the QCSOSTF during the three-month design storm.

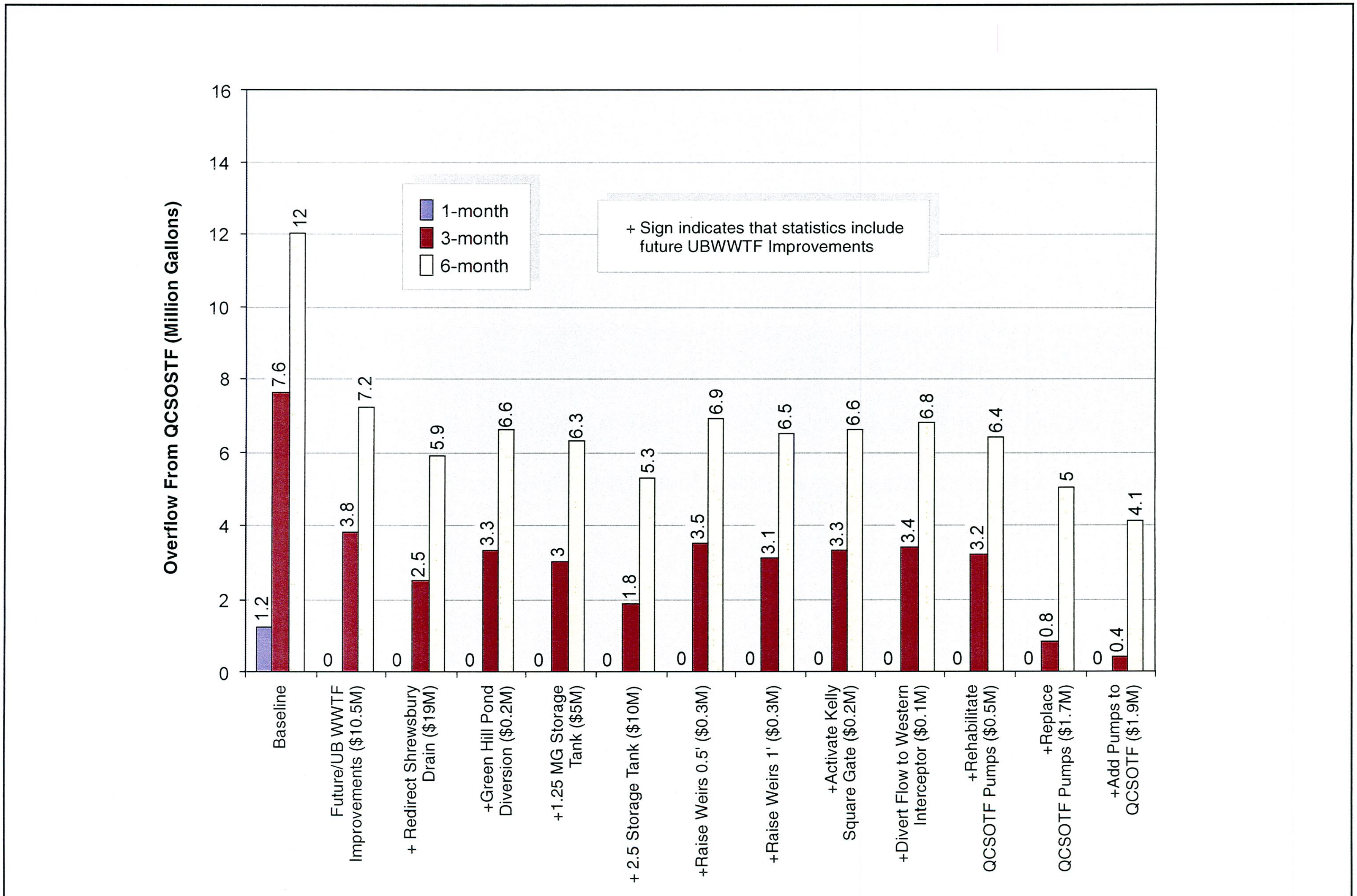

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Figure 9-2
Treated Discharge Volume Reduction and Cost Impact of Alternatives

CDM

Table 9-1
Comparing Existing, Baseline (2020) and UBWWTF Improvements

Parameter	*1-Month*	*3-Month*	*6-Month*
Volume Treated at QCSOSTF, Million Gallons			
Existing Conditions	0.4	6.4	10.5
Baseline Conditions	1.2	7.8	12
With UBWWTF Improvements	0	3.8	7.2
Volume Treated 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 Overflows from QCSOSTF	
Existing Conditions	12
Baseline Conditions	14
With UBWWTF Improvements	7
Million Gallons/Year Discharged from QCSOSTF	
Existing Conditions	82
Baseline Conditions	83
With UBWWTF Improvements	34

9.4 Hydrologic Response Improvements

Alternatives in this category generally lead to improved hydrologic response of the combined sewer system through removing inflow sources and thereby reducing the volume and peak flows during storm events. The alternatives include full sewer separation, partial sewer separation or flow diversions, and downspout disconnection.

9.4.1 Full Sewer Separation

Sewer separation involves constructing new drains or sewers to collect storm and sanitary flows separately, rather than in one combined sewer pipe. The storm drains would discharge directly to the receiving water, while the sanitary sewers would convey flow to the wastewater treatment facility for treatment prior to discharge. This prevents overflows in the combined sewer pipe, which may be undersized to handle both sanitary and storm flows. However, it also results in increased stormwater pollutant loading to the receiving water, and often results in high economic and community impacts from construction in developed areas such as Worcester.

Much of Worcester is already separated. Only four-square miles remain as combined sewers near the highly developed downtown area, where construction impacts would be high. Many opportunities for separation have already been implemented as the result of previous CSO abatement projects and system optimization efforts. This is particularly true for areas tributary to the Western Interceptor, which may be more easily connected to the twin 8.5 foot by 11 foot box stormwater drainage conduit running through the middle of Worcester from Salisbury Pond to the Blackstone River, as shown schematically in Figure 4-1. Combined areas tributary to the Eastern Interceptor, however, are much more difficult to separate because the 8 foot by 20 foot granite arch overflow collector is positioned between these combined areas and the drainage conduit. Storm drains from the east side of the city would likely need to be routed beneath the large overflow collector and connected to the stormwater conduit, complicating construction.

This section discusses full separation of the entire four-square mile combined area remaining in Worcester. Partial separation or flow diversion alternatives for CSO abatement are discussed subsequently. Costs and benefits are assigned to each.

Approach for Full Separation

The following describes the methodology used to develop cost estimates for separating the 2,500-acre combined sewer area in Worcester. Essentially, costs were developed by determining what it would take to separate specific areas with relatively uniform land uses and applying these costs throughout the entire combined area, according to land use. The evaluation consisted of preparing two conceptual drainage layouts and cost estimates for two representative catchment areas within two different land use zones in the City of Worcester. The goal of this work was to develop typical sewer layouts and cost estimates that could be applied to other catchment areas in the city. For the purposes of the Worcester estimate, it was assumed that the pipe sizes would be roughly the same as the existing combined sewer pipe and that the layout would be similar. Delineated catchment areas in Worcester were taken from the catchment and land use information used to develop the hydraulic model of the sewer system.

The developed costs were then compared to costs from other similar separation projects in New England.

Areas Selected for Conceptual Layouts

The two locations selected as representative catchments were the North Southbridge Street area and the Canton Street area, both in the southern portion of the Worcester combined area. These catchment areas were selected due to their nearly uniform land use. The first catchment area, North Southbridge Street (catchment 32 in Figure 9-3), is comprised mostly of industrial land use properties. The second representative area, Canton Street (catchment 17 in Figure 9-3), was selected given its primarily residential land use. The following describes each catchment in greater detail.

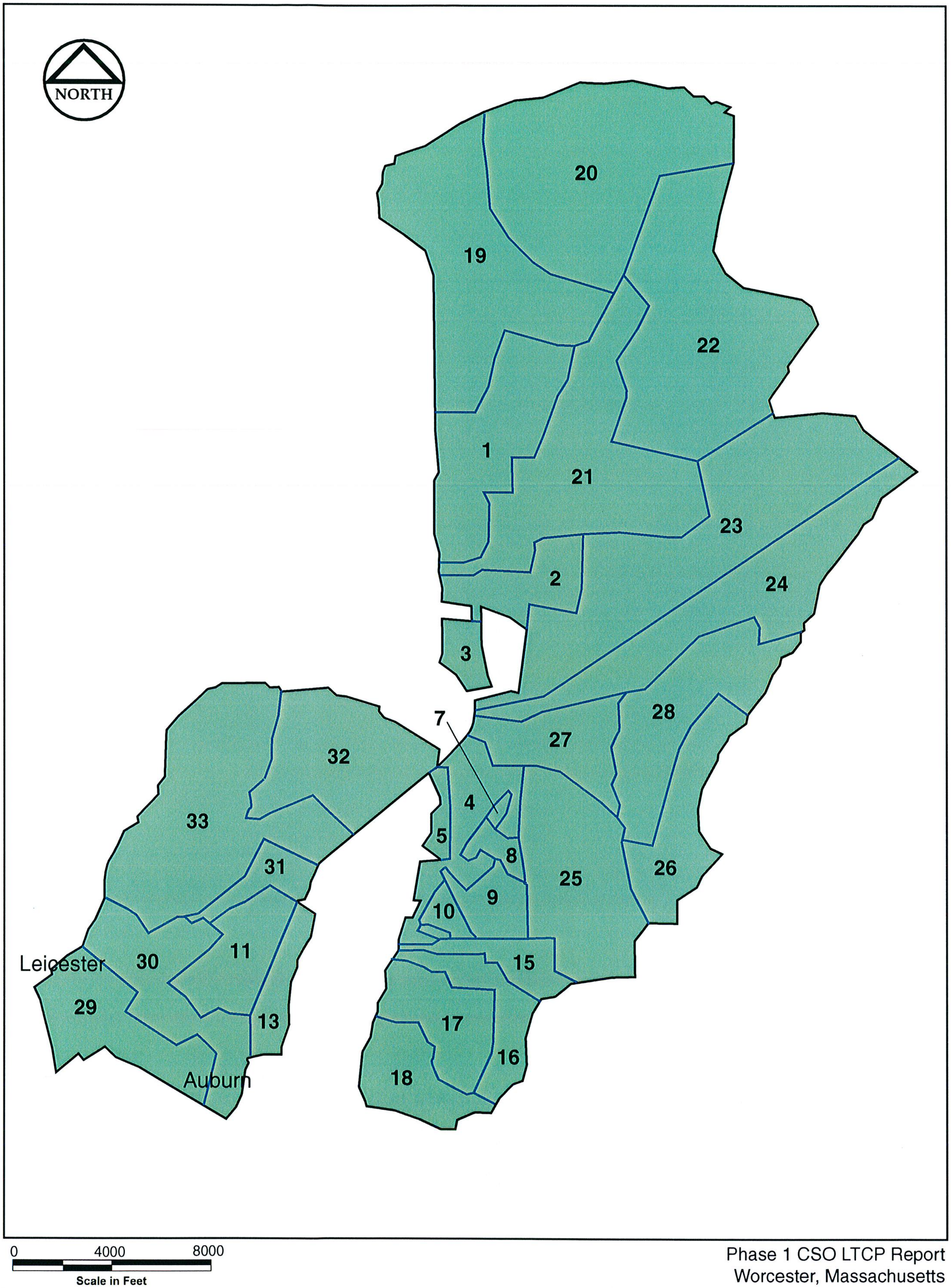

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Figure 9-3
CSS Catchment Areas

The North Southbridge Street area is primarily comprised of industrial and commercial properties. It is located in the central to southwestern portion of the combined system, and has an area of 106 acres. It is bounded by Francis J. McGrath Boulevard to the east, Franklin Street to the north, Crown/Wellington/Dale Streets to the west, and Hermon Street to the south. It is tributary to the Western Interceptor and the Quinsigamond Avenue Overflow Collector. This commercial/industrial area is generally more densely developed with a higher potential for utility conflicts.

The Canton Street area is primarily residential. It is located in the southeastern portion of the combined system, north of Brosnihan Square, and has an area of 57 acres. It is bounded by Vernon Street to the east, Suffield and Endicott Streets to the north, Harding Street to the west, and Ashmont/Stone/and Harlem Streets to the south. It is tributary to the Eastern Interceptor and the Canton Street/Harding Street Overflow Collector. This residential area consists mainly of one, two and three family homes with driveways on less than 0.25-acre lots. Some more densely developed residential areas are interspersed with the less densely developed neighborhoods characteristic of the Canton Street area. Route 290 runs through the western portion of the catchment, where there are some commercial properties.

The separation costs developed for each area provided a range of costs that were applied to all remaining combined areas in the city. The North Southbridge Street cost estimate was applied to areas with similar industrial/commercial land uses. The Canton Street cost estimate was applied to areas with similar residential areas. The cost estimates were adjusted accordingly for areas with a mix of both industrial/commercial and residential properties. This was done by estimating the percentage of each land use in each catchment and adjusting the cost estimates accordingly.

Procedure for Drainage Layouts

Based on previous studies it was assumed that the approximate length of combined sewer would be roughly equal to the approximate length of storm drain required for the areas selected. It was also assumed that the storm drain size required would be approximately the same as the existing combined sewer pipe designed to carry stormwater flows as well as a small sanitary flow component. These assumptions are realistic based on experience in other areas. The lengths of each pipe size were then tabulated and the approximate cost to construct each pipe size was calculated according to unit prices taken from previous separation estimates. The unit prices were adjusted to current day prices by escalating each year by 2.5 percent or if the ENR was known, by converting to the January 2002 ENR index of 6462.

It was assumed that manholes were placed roughly 300-400 feet apart with special consideration for roadway intersections. Manholes were also assumed to measure 10 feet in depth. It was assumed that there were two catch basins for every manhole and that the 12-inch PVC connection between a given manhole and catch basin measured 15 feet.

Cost Estimation of Conceptual Drainage Layouts

Cost estimates were prepared with unit costs based on past CDM construction bids and a preliminary cost estimation. The construction bids were from the 1998 Lynn, MA sewer separation project (contract SS-7) and the preliminary cost estimation was the 1999 Lowell, MA Humphrey's Brook sewer separation preliminary design report. These costs were escalated by 2.5% each year to get a 2002 estimate. This escalation corresponds to a January 2002 ENR of 6462. The cost estimates are included as Tables 9-2 and 9-3.

Generally, the miscellaneous work (i.e. water/sewer improvements and other impacts) associated with the separation projects in the North Southbridge Street area was assumed to be more extensive and costly than in the Canton Street catchment. It was assumed that the larger diameter pipes in the North Southbridge Street area would result in larger amounts of disruption to the existing water mains, sewers, and other utilities. This was translated into assuming larger allowances for the miscellaneous water and sewer improvements in the North Southbridge Street area (15 and 10 percent of the construction cost, respectively, for North Southbridge Street as opposed to 12.5 and 7.5 percent, respectively, for Canton Street). The remaining miscellaneous work, covering all utility improvements and other general improvements to the area, was also assumed to be a higher percentage of the total construction cost for the North Southbridge Street area than the Canton Street area.

Allowances were made for contractor overhead and profit (15 percent) and construction contingencies and engineering and implementation costs. Contingencies were assumed to be 25 percent of the construction cost in both catchment areas. Engineering and implementation costs (i.e. design, survey, permitting, borings, reports, easement concerns, etc.) were assumed to be 20 percent of the total construction cost for both areas. Costs are not escalated to the midpoint of construction.

The total cost, cost per acre, and cost per foot of new pipe for each of the two representative catchment areas are presented in Table 9-4.

Table 9-4
City of Worcester Separation Estimates

Street	Total Cost	Cost Per Acre	Cost Per Foot of Pipe
Canton Street	$3,805,000	$66,750	$354
North Southbridge Street	$9,592,000	$90,490	$403

Separation Cost Estimates Allocated to Other Combined Areas

The cost per acre values for constructing new drainage pipes were applied to all other combined catchments in the City of Worcester. The cost per foot of sewer pipe was

Table 9-2
North Southbridge Area (Catchment 32) Separation Cost Summary

Item	Unit	Quantity	Unit Price	Total
12" PVC	lf	8,375	$65	$541,138
12" PVC (CB connections)	lf	2,850	$65	$184,148
15" PVC	lf	7,064	$81	$570,537
18" PVC	lf	3,852	$97	$373,336
21" RC	lf	2,110	$108	$227,224
24" RC	lf	187	$124	$23,159
27" RC	lf	412	$129	$53,241
30" RC	lf	1,164	$145	$169,223
33" RC	lf	522	$156	$81,510
36" RC	lf	142	$172	$24,467
48" RC	lf		$215	$0
Drain Pipe Subtotal		26,678		$2,247,982
5' Manholes	vf	950	$232	$219,955
Catch Basins	Each	190	$2,154	$409,218
Bank Run Gravel	CY	9,881	$8	$74,483
Temporary Paving	SY	29,642	$8	$223,450
Full-Width Paving	SY	79,427	$10	$769,804
Drain Outlet	Each		$53,845	$0
Special Structures	Each	1	$21,538	$21,538
Seal regulator	Each	1	$5,384	$5,384
48" TEE Manhole	Each		$5,384	$0
MH, CB, Gravel, Paving, Structure Subtotal				$1,723,833
Sum of Pipe and MH/CD/Grading Subtotals				$3,971,816
Miscellaneous Water Improvements	LS	1		$595,772
Miscellaneous Sewer Improvements	LS	1		$397,182
Miscellaneous Work	LS	1		$397,182
Mobilization	L	1		$198,591
Miscellaneous Subtotal				$1,588,726
Subtotal				$5,560,542
Contractor OH&P (15%)				$834,081
Subtotal				$6,394,623
Construction Contingency (25%)				$1,598,656
Total Construction Cost				$7,993,279
Engineering & Implementation (20%)				$1,598,656
Total Project Cost				**$9,591,935**
Total Acreage				106
Total New Pipe Length (ft)				26,678
Cost per Acre				$90,490
Cost per Foot				$403

Notes:

1) 1999/1998 unit prices escalated 2.5% each year for 3 years to reflect inflation.
2) Drain pipe lengths were assumed to be the same length as the existing combined sewer pipe in the subbasin, with the exception of additional CB connections.
3) Drain pipe sizes were assumed to be the same diameter as combined sewer pipe diameters or widths.
4) Pipe lengths and catchment areas taken from Worcester GIS data and hydraulic model catchment information.

Table 9-3
Canton Area (Catchment 17) Separation Cost Summary

Item	Unit	Quantity	Unit Price	Total
12" PVC	lf	8,021	$65	$518,264
12" PVC (CB connections)	lf	1,290	$65	$83,351
15" PVC	lf	1,386	$81	$111,943
18" PVC	lf	868	$97	$84,127
21" RC	lf	15	$108	$1,615
24" RC	lf	36	$124	$4,458
27" RC	lf		$129	$0
30" RC	lf		$145	$0
33" RC	lf	228	$156	$35,602
36" RC	lf	187	$172	$32,221
48" RC	lf		$215	$0
Drain Pipe Subtotal		12,031		$871,581
5' Manholes	vf	430	$232	$99,559
Catch Basins	Each	86	$2,154	$185,225
Bank Run Gravel	CY	4,456	$8	$33,590
Temporary Paving	SY	13,368	$8	$100,769
Full-Width Paving	SY	35,803	$10	$347,006
Drain Outlet	Each		$53,845	$0
Special Structures	Each	1	$21,538	$21,538
Seal Regulator	Each	1	$5,384	$5,384
48" TEE Manhole	Each		$5,384	$0
MH, CB, Gravel, Paving, Structure Subtotal				$793,072
Sum of Pipe and MH/CD/Grading subtotals				$1,664,653
Miscellaneous Water Improvements	LS	1		$208,082
Miscellaneous Sewer Improvements	LS	1		$124,849
Miscellaneous Work	LS	1		$124,849
Mobilization	L	1		$83,233
Miscellaneous Subtotal				$541,012
Subtotal				$2,205,665
Contractor OH&P (15%)				$330,850
Subtotal				$2,536,515
Construction Contingency (25%)				$634,129
Total Construction Cost				$3,170,644
Engineering & Implementation (20%)				$634,129
Total Project Cost				$3,804,773
Total Acreage				57
Total New Pipe Length (ft)				12,031
Cost per Acre				$66,750
Cost per Foot				$354

Notes:

1) 1999/1998 unit prices escalated 2.5% each year for 3 years to reflect inflation.
2) Drain pipe lengths were assumed to be the same length as the existing combined sewer pipe in the subbasin, with the exception of additional CB connections.
3) Drain pipe sizes were assumed to be the same diameter as combined sewer pipe diameters or widths.
4) Pipe lengths and catchment areas taken from Worcester GIS data and hydraulic model catchment information.

not used since the total length of pipe required for the four-square mile area was more difficult to estimate than the area of each catchment.

Each catchment was categorized by percentage of residential and commercial/ industrial land use. Costs for separation were generated using the Canton Street estimate for predominantly residential land use areas and the North Southbridge Street estimate for predominantly industrial/commercial land use areas. If there was a mix of development density or land use within a catchment, then the cost per acre figure was adjusted accordingly to correspond with the approximate percentage mix of land use. For example, if a catchment were 50 percent residential and 50 percent industrial/commercial, then the cost per acre applied to the catchment area would be the average of the Canton Street and North Southbridge Street cost per acre estimate, or $78,600 per acre. It should be noted that separation costs assume that new drains will be designed and constructed and that the existing combined pipes will remain as sanitary pipes. Table 9-5 summarizes the costs of separation for each catchment area in the Worcester combined sewer system. The catchment areas for the entire combined system are identified in Figure 9-3. Based on this analysis, the estimated cost to fully separate the four-square mile combined sewer system in Worcester is $180,000,000.

Comparison with Other New England Communities

The separation cost estimates prepared for Worcester compare well with separation estimates from other communities in New England, as shown in Table 9-6 and are considered reasonable for evaluation of separation throughout Worcester.

Table 9-6
Separation Cost Estimates for Other New England Communities

Municipality/ Sewer District	Combined Area (acres)	Total Separation Cost Estimate ($)	Cost per Acre ($/acre)		Cost Per Foot ($/LF)	
			Res.	Indust./ Comm.	Res.	Indust./ Comm.
Worcester (2002 Estimate)	2,528	180,000,000	66,800	90,500	354	403
GLSD (2001 Estimate)	2,285	157,000,000	67,600	75,400	323	324
Haverhill (2000 Estimate)	2,354	105,000,000	50,000	70,000	310	420
Lowell (2000 Estimate)	4,990	366,000,000	65,000	85,000	NA	NA
Dorchester CSO 090 (2001 Estimate)	361	34,650,000	NA	96,000	NA	NA

Table 9-5
Summary of Separation Costs for the Worcester Combined Sewer System

Catchment	Area (Acres)	Land Use	Cost per Acre	Separation Cost According to Cost/Acre
1	105	50% res, 50% ind	$78,600	$8,253,000
2	45	50% res, 50% ind	$78,600	$3,537,000
3	16	ind	$90,500	$1,448,000
4	55	ind	$90,500	$4,977,500
5	9	ind	$90,500	$814,500
7	3	ind	$90,500	$271,500
8	11	50/50	$78,600	$864,600
9	27	res	$66,800	$1,803,600
10	11	65%ind/35%res	$82,200	$904,200
11	55	ind	$90,500	$4,977,500
13	27	ind	$90,500	$2,443,500
14	1	ind	$90,500	$90,500
15	27	res	$66,800	$1,803,600
16	41	res	$66,800	$2,738,800
17	57	res	$66,800	$3,807,600
18	50	res	$66,800	$3,340,000
19	184	ind	$90,500	$16,652,000
20	216	res	$66,800	$14,428,800
21	174	res	$66,800	$11,623,200
22	220	20% res, 80% park	$13,400	$2,948,000
23	224	res	$66,800	$14,963,200
24	132	ind	$90,500	$11,946,000
25	123	65%res, 35%ind	$75,100	$9,237,300
26	70	res	$66,800	$4,676,000
27	58	ind	$90,500	$5,249,000
28	104	60%res, 40%ind	$76,200	$7,924,800
29	78	res	$66,800	$5,210,400
30	87	50/50	$78,600	$6,838,200
31	25	50/50	$78,600	$1,965,000
32	106	ind	$90,500	$9,593,000
33	187	60%res, 40%ind	$76,200	$14,249,400
Total	**2528**			**$179,579,700**

Water Quality Benefits

However, stormwater runoff has fecal coliform concentrations that exceed water quality standards. Currently, stormwater runoff from the CSS is disinfected either at the QCSOSTF or at the UBWWTF. This level of treatment is unusual for stormwater runoff. The treatment facilities also help reduce BOD and TSS loadings.

This option potentially has negative water quality benefits, at a cost of $180M. For the three-month storm, this represents a cost of $23.68 per gallon of discharge reduced from the QCSOSTF.

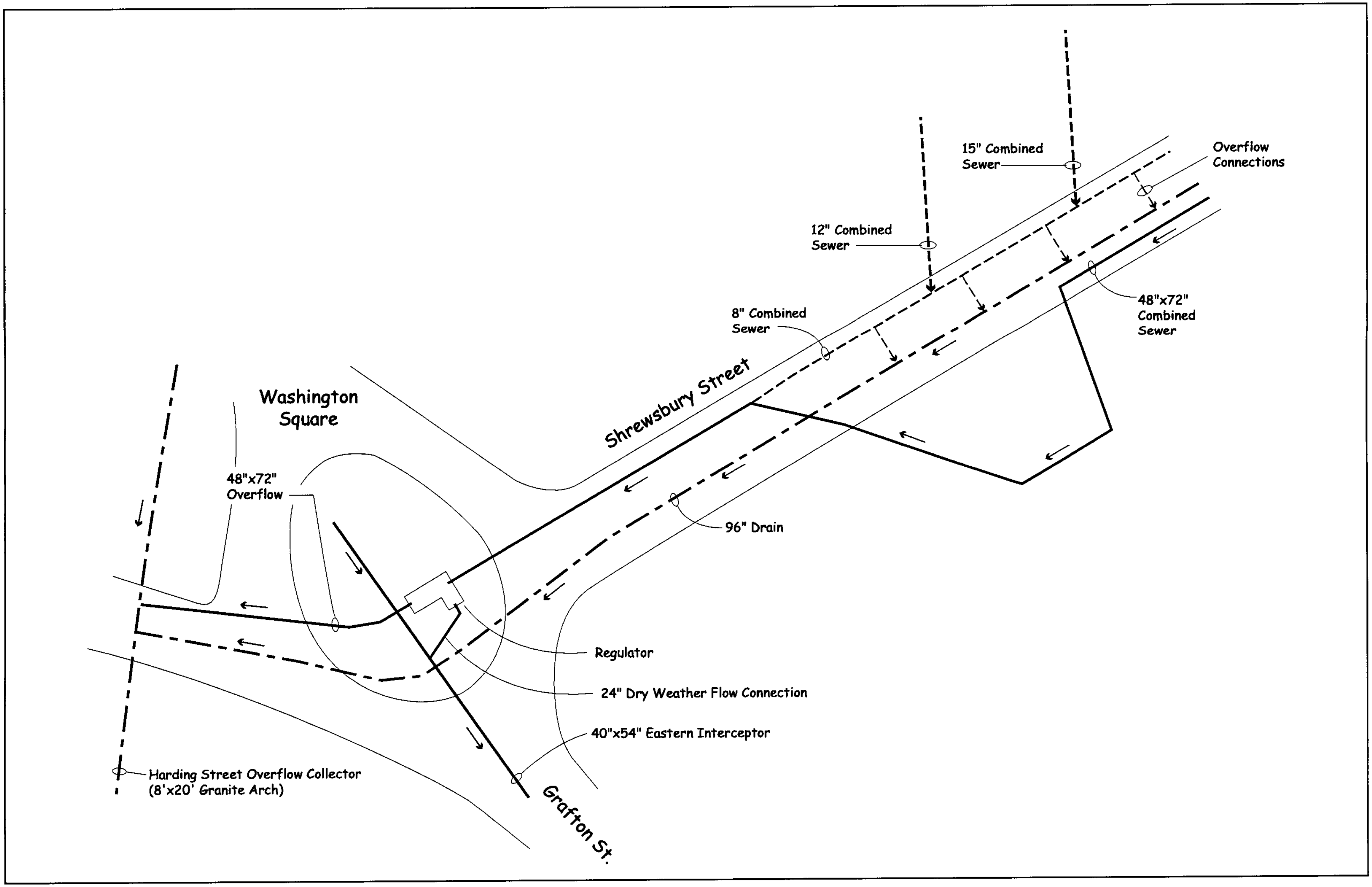
9.4.2 Partial Separation/Flow Diversion

As shown above, full separation is often expensive and difficult to implement in highly developed urban areas. Targeted separation of specific areas prioritized in terms of ease of implementation or system performance benefit represents a more cost-effective means of achieving CSO control goals. Worcester has already implemented many targeted separation projects through its history of CSO control and system improvement projects. There are not many easy-to-separate areas left in the highly developed downtown combined area. However, there are two targeted separation or flow diversion projects that have been evaluated in this study in terms of their potential to provide CSS performance benefits through diverting flows from the overflow collectors. The first project involves the disconnection of the 96-inch drainage/combined sewer conduit in Shrewsbury Street from the combined system. The second involves disconnecting Green Hill Pond and Bell Pond flows from the combined sewer system. Each of these projects is discussed further below.

Shrewsbury Street Drain Disconnection

As shown previously on Figure 4-1, the 96-inch drainage conduit in Shrewsbury Street currently discharges to the Harding Street Overflow Collector. The majority of the flow collected by this large pipe is storm and surface water drainage from Green Hill Pond, Bell Pond, and the north Shrewsbury Street drainage area. However, it also acts as a relief sewer for combined pipes along Shrewsbury Street. The 96-inch pipe was not disconnected previously because of the difficulty and cost associated with connecting to the twin 8.5 foot by 11 foot box culvert to the west of the 8 foot by 20 foot granite arch overflow collector. As described previously, a large siphon structure would be required to route flows beneath the overflow collector and for discharge to the drainage conduit.

Nonetheless, disconnecting this large pipe from the combined sewer system would free up valuable capacity in the overflow collector and at the QCSOSTF for storage and/or pumping to the UBWWTF or treatment at the QCSOSTF. The challenge in disconnecting the 96-inch diameter pipe from the overflow collector is twofold. As shown in Figure 9-4, the 96-inch pipe acts as a relief conduit for the smaller 8-inch diameter combined sewer pipe that runs parallel to the 96-inch pipe in Shrewsbury Street. Flows from larger combined sewers tributary to the 8-inch pipe overflow into



Not to Scale

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Figure 9-4
Shrewsbury Street Sewers

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the 96-inch pipe through various sewer relief connections. The 96-inch pipe also relieves the 48-inch by 72-inch Shrewsbury Street interceptor where the 15-inch Lyon Street connection ties into the interceptor.

In order to disconnect the 96-inch pipe from the combined sewer overflow collector, these combined sewage inputs would need to be removed. This may be accomplished in one of two ways, or through a combination of the two. Areas tributary to Shrewsbury Street may be separated, or a larger combined sewer intercepting pipe may be installed in Shrewsbury Street to prevent overflows into the 96-inch drain. The second, and perhaps more difficult task, would be to route the 96-inch pipe flows under the existing overflow collector to discharge into the twin 8.5 foot by 11 foot box drainage conduit on the west side of the overflow collector. This would be difficult construction in the congested Washington Square area. A special siphon structure would need to be constructed immediately upstream and downstream of the overflow collector. The siphon would most likely be constructed between the two structures via pipe jacking. A new section of large diameter pipe crossing beneath Worcester Center Boulevard would also be needed between the downstream siphon structure and the 8.5 foot by 11 foot twin box drainage conduit. Approximately 300 feet of pipe would be required to connect the existing 96-inch pipe to the drainage conduit. A hydraulic analysis of the twin box drainage conduit and 96-inch Shrewsbury Street drain/siphon structure is recommended should this alternative be pursued further to evaluate potential flow or surcharging concerns. Sediment deposition in the siphon would also be of concern.

The costs of each of these components to the disconnection of the 96-inch drain conduit option, shown schematically in Figure 9-5, are presented below.

- <u>Separation of Areas Tributary to Shrewsbury Street</u>: The combined sewer area tributary to Shrewsbury Street is approximately 234 acres. Applying the cost per acre values developed under the full separation option to this mix of residential (40 percent), commercial/industrial (30 percent), institutional (10 percent), and government land (30 percent), yields a separation cost of $16 million. This cost may be lower if specific street-by-street drainage needs are evaluated for the areas tributary to Shrewsbury Street.

- <u>Siphon Structure and Connection to Drainage Conduit</u>: The cost of siphoning beneath the overflow collector and connecting to the drainage conduit is $3M.

  The total cost of the 96-inch Shrewsbury Street drain disconnection alternative is $19M. This includes allowances for contractor overhead and profit (15 percent), construction contingencies (25 percent), and engineering and implementation (20 percent). Costs were not escalated to the midpoint of construction. Unless otherwise noted, these same allowances were applied to all alternatives evaluated in this study.

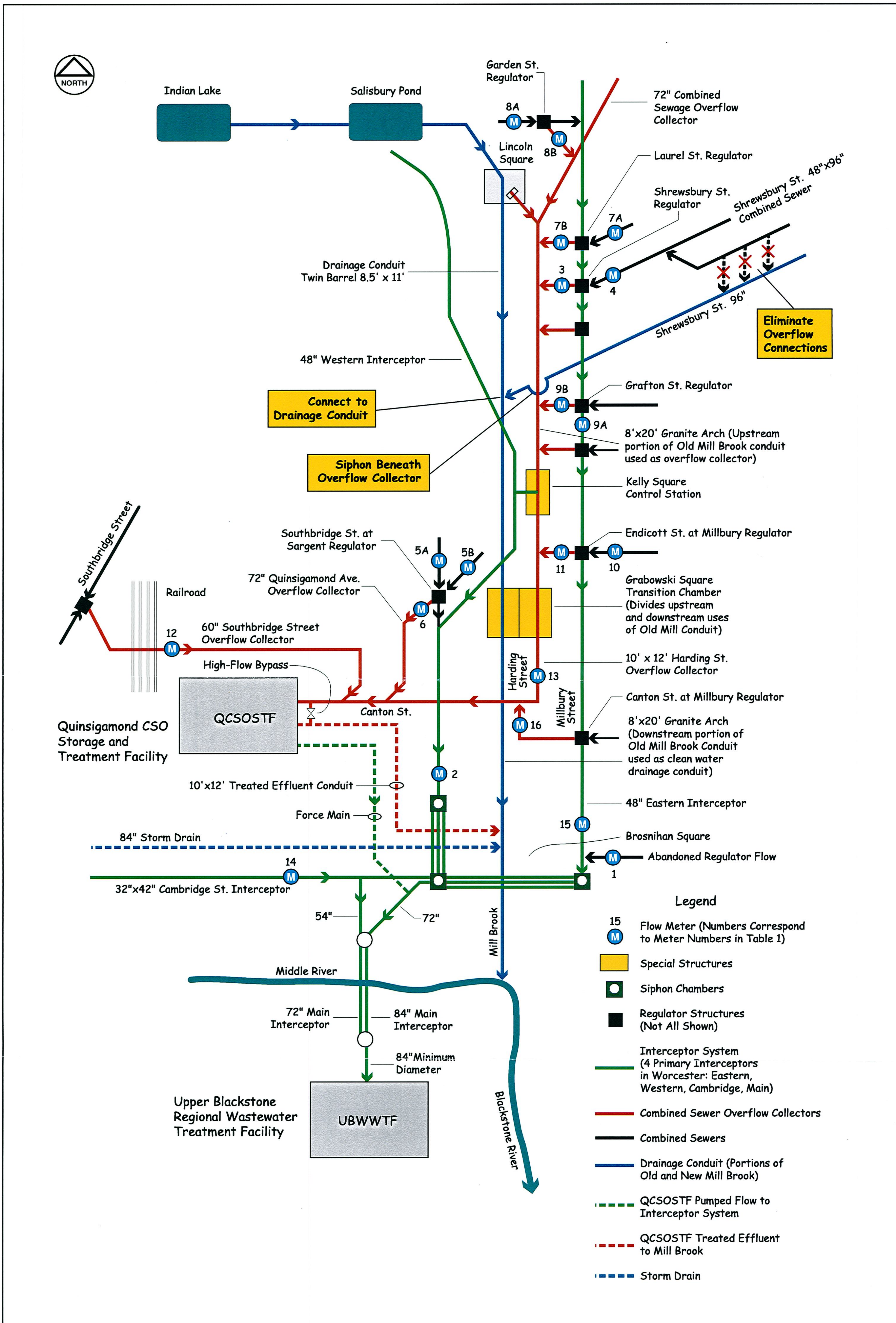

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Figure 9-5
Shrewsbury St. 96" Drain Disconnection

Constructing a new intercepting pipe in Shrewsbury Street to replace the undersized 8-inch diameter pipe would likely offer a lower cost solution to the separation of the Shrewsbury Street tributary area option presented above. This new pipe would be designed to handle the flow from the combined areas north of Shrewsbury Street and eliminate the overflow connections to the 96-inch drain pipe in appropriate design storms. The approximate length of pipe required is 4,500 linear feet. The specific pipe size and length required and the feasibility of constructing it in Shrewsbury Street would need to be evaluated further in the next phase of work. It is noted that improvements along Shrewsbury Street and Washington Square involving road paving will commence in the near future. These planned improvements will result in significant community impacts. Since the design of the Shrewsbury Street 96-inch drain disconnection, if selected as a recommended plan, is not likely to be completed in time to include in the implementation of these planned improvements, it is likely that additional work in Shrewsbury Street and Washington Square shortly after completion of the planned improvements would meet public resistance.

Consequently, this work would most likely need to be delayed until the next round of street improvements are scheduled approximately 10 years from now.

Water Quality Benefits

Table 9-7 and Figure 9-2 present the benefits associated with disconnecting the 96-inch Shrewsbury Street drain from the CSS. According to the table, the QCSOSTF discharge volume decreases from 3.8 million gallons with just planned UBWWTF improvements to 2.5 million gallons in the 3-month storm. Results for the 1-month and 6-month storms are also presented in the table. The results show that there would be a reduction in the total flow treated at the UBWWTF, because this alternative diverts runoff from the sewer system to discharge directly to the Blackstone River.

Table 9-7
Comparing 96-inch Drain Disconnection to UBWWTF Improvements

Parameter	1-Month	3-Month	6-Month
Volume Treated at QCSOSTF, Million Gallons			
With UBWWTF Improvements	0	3.8	7.2
With UBWWTF Improvements Plus Partial Separation	0	2.5	5.9
Volume Treated at UBWWTF, Million Gallons (2-Day Simulation)			
With UBWWTF Improvements	123.3	124.4	126.1
With UBWWTF Improvements Plus Partial Separation	122.0	117.9	125.2

This option, like full separation, potentially has negative water quality benefits. At a cost of $19M, the 96-inch Shrewsbury Street drain disconnection costs about $14.62/gallon of discharge reduced during 3-month storm conditions.

Green Hill Pond and Bell Pond Flow Diversion

This alternative (which would only be considered if the Shrewsbury Street alternative discussed above is not recommended) involves diverting Green Hill Pond and Bell Pond flows from the combined sewer system. As shown in Figure 3-1, Green Hill Pond and Bell Pond are located in the northeastern portion of the combined sewer area. Surface water and stormwater runoff (but no combined sewage) from the ponds and the areas immediately surrounding them flow downstream into the combined sewer system. Flows eventually drain to the 96-inch pipe in Shrewsbury Street discussed above. Disconnecting these pond drainage systems from the combined sewer system was investigated to determine the CSO control benefits associated with this alternative. If this were accomplished, then approximately 266 acres, as shown in Figure 9-6 would be removed from the CSS.

Green Hill Pond has two outlet structures. The western outlet discharges into the combined sewer drainage area and the eastern outlet discharges into the separated area and to Coal Mine Brook which drains to Lake Quinsigamond. This alternative would entail modifying the outlet works in the vicinity of Green Hill Pond Park to the separated area. A new drainage channel may be required to connect flows to Coal Mine Brook. Further field investigations would be required to evaluate the feasibility of this option, but if possible, is considered to be a relatively low cost alternative for improving the hydrologic performance of the CSS.

Diverting Bell Pond out of the combined area is much more difficult to accomplish since Bell Pond is at a lower elevation. Pond flows and runoff from this area would possibly require pumping or significant drainage projects to route flows by gravity out of the combined sewer system, and would be considerably more expensive than diverting Green Hill Pond and its drainage from the CSS.

Water Quality Benefits

Table 9-8 presents the benefits associated with separating Green Hill Pond and Bell Pond from the CSS. The table shows that significant reductions in QCSOSTF discharge (0.5 MG for the 3-month storm) can be accomplished by diverting Green Hill Pond and its surrounding area from the CSS, with little additional benefit (0.1 MG) from also diverting the Bell Pond area. The table also shows that these alternatives reduce the amount of flow to the UBWWTF. In fact, diverting the pond flows approaches the effectiveness of the Shrewsbury Street alternatives, at much lower cost. Figure 9-2 presents the benefits associated with separating Green Hill Pond from the CSS.

Because of the limited benefits of diverting Bell Pond, and the significant engineering challenges and costs, it was not considered further.

During a 3-month storm, diverting 0.5 MG of flow from the QCSOSTF to the UBWWTF will reduce BOD loadings to the Blackstone River by about 60 pounds and TSS loadings by about 360 pounds. At an estimated cost of $200,000, the Green Hill

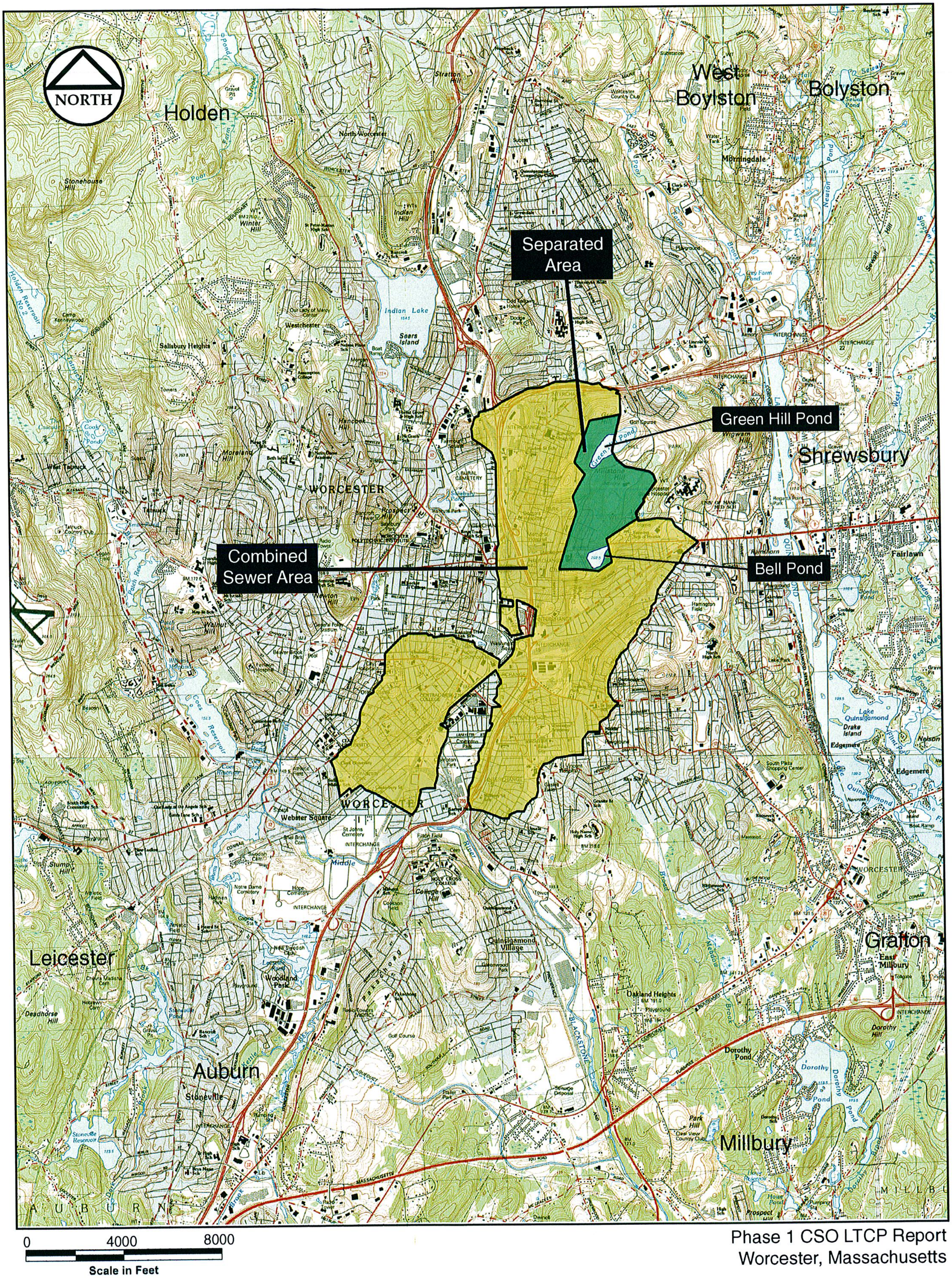

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Figure 9-6
Separated Pond Areas

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Pond diversion costs about $0.40/gallon of discharge reduced during 3-month storm conditions.

Table 9-8
Comparing Green Hill Pond and Bell Pond Alternatives to UBWWTF Improvements

Parameter	1-Month	3-Month	6-Month
Volume Treated 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
With UBWWTF Improvements and Diverting Both Ponds	0	3.2	6.5
Volume Treated 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
With UBWWTF Improvements and Diverting Both Ponds	122.6	124.1	125.8

9.5 Storage Alternatives

Alternatives in this section lead to improved in-line or off-line storage capacity in the Worcester CSS. These alternatives include expanded QCSOSTF storage capacity, regulator modifications, Kelly Square Control Station modifications, and construction of a Harding Street control station.

9.5.1 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 wet well 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. This capacity may be doubled without extending beyond the City property line of the QCSOSTF. As shown in Figure 9-7, flow would be diverted into the storage tanks through a new gate structure and each tank would fill up before the effluent gates to the contact tanks would open.

Costs

The approximate cost of this option assuming that an additional 2.5 million gallons of storage were added to the facility, as shown in Figure 9-7, is $10 million. This cost includes the same allowances discussed in the previous section. However, this does not include hazardous waste allowances, or consideration for other site-specific

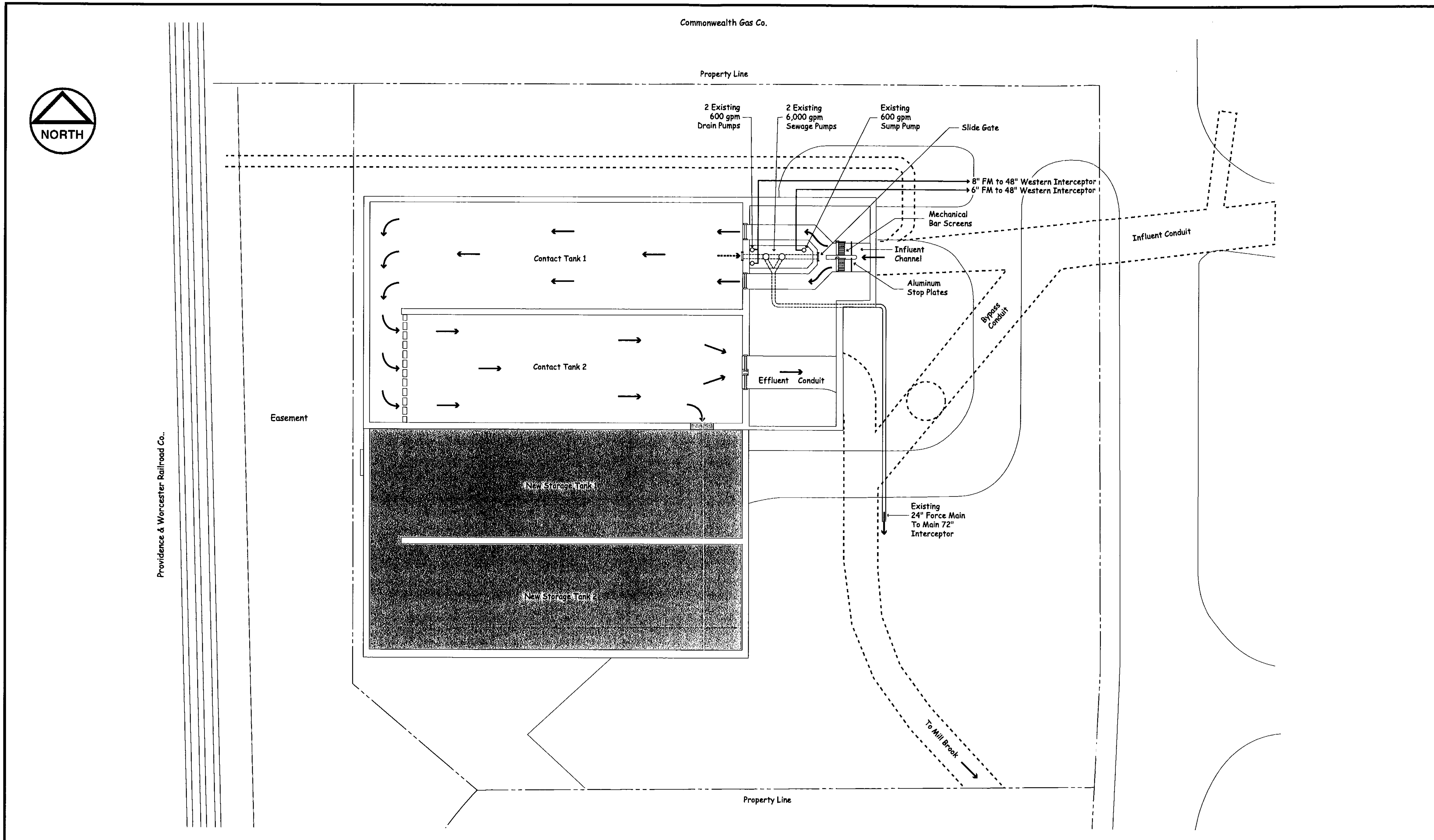

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Figure 9-7
QCSOSTF Storage Capacity Expansion

conditions that may affect the cost estimate. The cost would be approximately half of the $10 million estimate if only one 1.25 million gallon tank were constructed.

Water Quality Benefits

The benefits associated with increasing the storage capacity at the QCSOSTF are presented in Table 9-9 and shown in Figure 9-2. The table shows significant decreases in discharge from the QCSOSTF. 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. 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. At $5M, one tank costs about $6.25 per gallon of discharge reduced during 3-month storm conditions. At $10M, two tanks cost about $5.00 per gallon reduced during the 3-month storm.

Table 9-9
Comparing Increased Storage at QCSOSTF to UBWWTF Improvements

Parameter	1-Month	3-Month	6-Month
Volume Treated 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
Volume Treated 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	119.6	127.7
Plus 2.50 MG New Storage (two new tanks) at QCSOSTF	123.4	121.6	128.2

9.5.2 Regulator Modifications

Regulator modifications were modeled throughout the system to determine their effectiveness in terms of CSO abatement. Raising weirs forces more flow through the interceptor system without spilling into the overflow collectors. Weir elevations were raised first ½ foot, then one foot at all regulators to determine their impact. If this alternative is carried forward, it will be necessary to determine optimum increases in weir height at each regulator to prevent upstream flooding. Nonetheless, raising weirs offers a relatively low cost option for optimizing the performance of the combined sewer system. In order to avoid upstream flooding, it may be necessary to lengthen weirs in addition to raising them. This may require the construction of new manhole structures with the elevated and lengthened weirs.

Water Quality Benefits

The benefits associated with elevating the weirs are presented in Table 9-10 and shown in Figure 9-2. The table shows significant decreases in discharge from the QCSOSTF. During the 3-month storm, raising the weirs ½ foot decreases the discharge from the QCSOSTF by 0.3 MG, while raising them 1 foot decreases the discharge by 0.7 MG. During 3-month storm conditions, the BOD reduction for raising weirs ½ foot is about 40 pounds and the TSS reduction is about 220 pounds. Raising the weirs one foot is expected to reduce BOD by 90 pounds and TSS by 510 pounds. Conservatively assuming a cost of $340,000 for regulator modifications ($20,000 per regulator), raising weirs ½ foot costs about $1.13 per gallon of discharge reduced during 3-month storm conditions, and raising weirs 1 foot costs about $0.49 per gallon of discharge reduced during the 3-month storm.

Table 9-10
Comparing Regulator Modifications to UBWWTF Improvements

Parameter	1-Month	3-Month	6-Month
Volume Treated at QCSOSTF, Million Gallons			
With UBWWTF Improvements	0	3.8	7.2
Plus Raising All Regulator Weirs 0.5 Feet	0	3.5	6.9
Plus Raising All Regulator Weirs 1.0 Foot	0	3.1	6.5
Volume Treated at UBWWTF, Million Gallons (2-Day Simulation)			
With UBWWTF Improvements	123.3	124.4	126.1
Plus Raising All Regulator Weirs 0.5 Feet	123.2	124.7	126.4
Plus Raising All Regulator Weirs 1.0 Foot	123.5	125.0	126.8

9.5.3 Kelly Square Control Station Modifications

The Kelly Square Control Station is highlighted in yellow on Figure 4-1 and shown in more detail in Figure 9-8. This station, constructed in the Harding Street Overflow Collector as part of the CSO control improvements implemented in the 1980's, was designed to store flows in the overflow collector upstream of Kelly Square and to divert lower flows from the overflow collector to the Western Interceptor to take advantage of available capacity. In order to accomplish this, the station is equipped with an 18-foot wide by 5-foot high hinged leaf gate, three 12-inch 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 9-8, 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 is a significant concern that outweighs the value of activating the gate. This alternative evaluates the value of using the gate for storage and assumes an operating protocol can be implemented to ensure gate operations do not cause flooding.

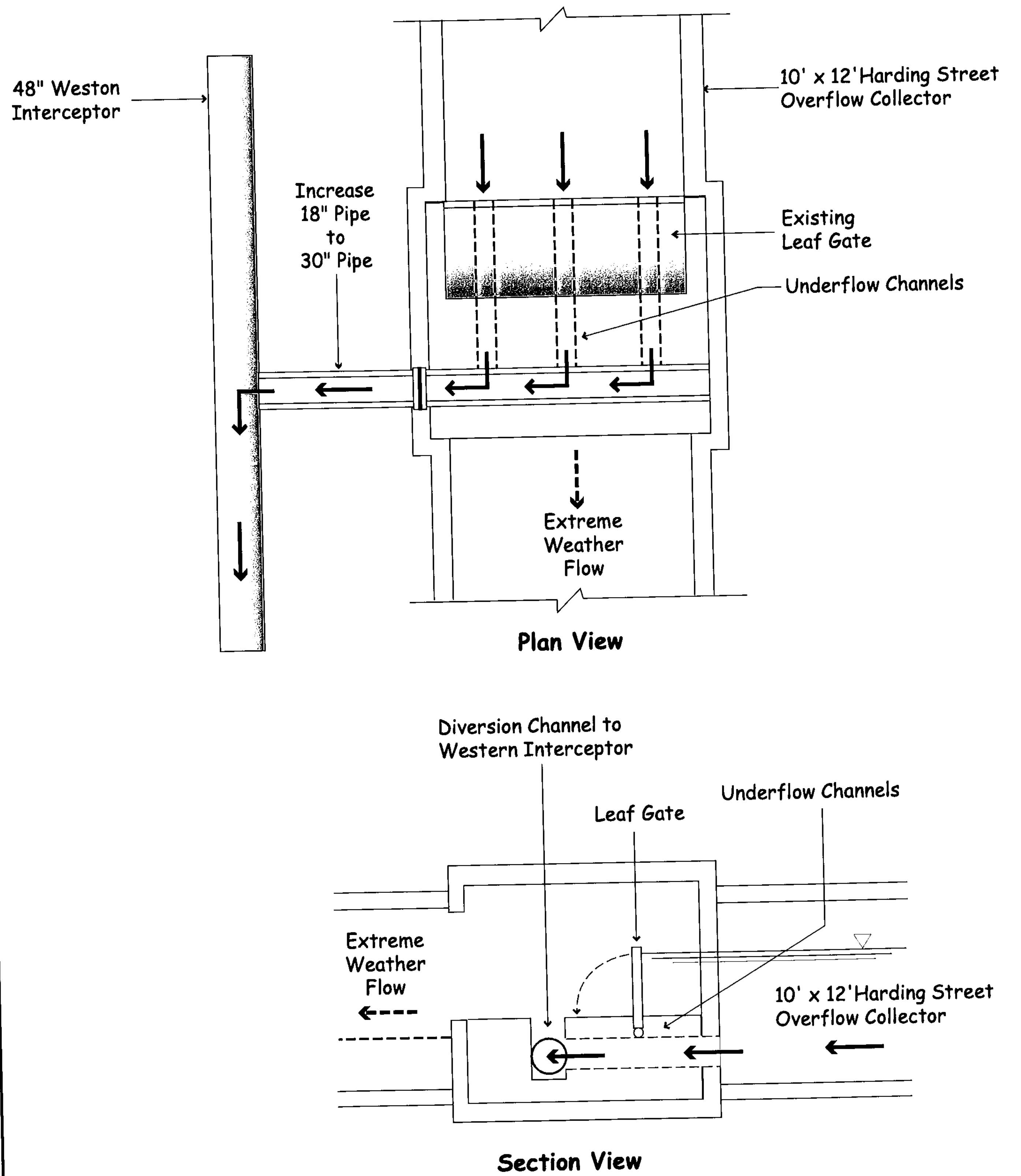

Figure 9-8
Kelly Square Control Structure

Real Time Controls (RTC) and sensors were installed as part of previous CSO control efforts. These controls may be activated or replaced as appropriate to enable the gate to be raised and lowered remotely from the QCSOSTF as conditions dictate. This option should be evaluated further in future phases of CSO control planning. The condition of the 8 foot by 20 foot granite archway should also be evaluated further in future phases to determine if the structure can handle the hydraulic forces associated with the proposed increased depth. Rehabilitation of the overflow collector beyond previous maintenance efforts may be required.

It would also be beneficial to divert more flow to the Western Interceptor. This may be accomplished by increasing the capacity of the diversion connection between the Kelly Square Control Station and the Western Interceptor and would involve constructing approximately 500 feet of 30-inch pipe to replace the existing 18-inch diameter pipe.

Cost

The estimated cost to activate the Kelly Square Control Station gate to increase in-line storage is $200,000. The estimated cost to increase the capacity of the size of the diversion connection from an 18-inch diameter pipe to 30 inches is approximately $125,000.

Water Quality Benefits

The benefits associated with activating the Kelly Square Gate are presented in Table 9-11 and shown in Figure 9-2. The table shows significant decreases in discharge from the QCSOSTF. During the 3-month storm, discharge from the QCSOSTF is reduced by 0.5 MG. This represents about a 60 pound reduction in BOD and a 360 pound reduction in TSS to the Blackstone River. At a cost of $200,000, this is about $0.40 per gallon of discharge reduced during 3-month storm conditions.

The benefits associated with diverting more flow to the Western Interceptor are also shown in the table. The discharge reduction is similar to the reduction provided by activating the Kelly Square gate. During the 3-month storm, discharge from the QCSOSTF is reduced by 0.4 MG representing a 50 pound reduction in BOD and 290 pound reduction in TSS to the Blackstone River. At a cost of $125,000, this is about $0.31 per gallon of discharge reduced.

Table 9-11
Comparing Kelly Square Gate Alternatives to UBWWTF Improvements

Parameter	*1-Month*	*3-Month*	*6-Month*
Volume Treated at QCSOSTF, Million Gallons			
With UBWWTF Improvements	0	3.8	7.2
UBWWTF Improvements Plus Activating the Kelly Square Gate	0	3.3	6.6
UBWWTF Improvements Plus Directing flow to the Western Interceptor	0	3.4	6.8
Volume Treated at UBWWTF, Million Gallons (2-Day Simulation)			
With UBWWTF Improvements	123.3	124.4	126.1
UBWWTF Improvements Plus Activating the Kelly Square Gate	123.2	128.1	133.1
UBWWTF Improvements Plus Directing flow to the Western Interceptor	123.3	128.1	133.2

9.5.4 Harding Street Control Station

This alternative involves the construction of a new control station to take advantage of the slope break in the Harding Street Overflow Collector near the intersection of Harding Street and Canton Street. The structure would be similar to Kelly Square with a hinged leaf gate that could be raised and lowered remotely as conditions dictated to maximize in-line storage in the overflow collector. No diversion connection would be constructed.

Cost

The preliminary cost estimate associated with this alternative is $450,000. This includes the cost to construct a special structure similar to the Kelly Square Control Station, contingencies, and engineering and implementation allowances, as described with previous alternatives.

Water Quality Benefits

The simulation results showed that there is not sufficient storage in the reach between Harding Street and Kelly Square to reduce overflows. Consequently, this alternative will no longer be considered.

9.6 System Conveyance Improvements

These alternatives result in improved system conveyance to the UBWWTF, which reduces the volume of treated discharges at the QCSOSTF. These alternatives involve pumping improvements at the QCSOSTF that allow more flow to be pumped from the QCSOSTF to the UBWWTF as well as utilizing available capacity in the interceptor system. Two of the improvements in this category, utilizing expanded UBWWTF capacity and diverting more flow to the Western Interceptor, were already discussed in Section 9.3 and Section 9.5.3, respectively.

9.6.1 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 water surface elevations also begin to rise if the pumps cannot keep up with flow entering the QCSOSTF via the overflow collectors. Once the wet well 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, to allow 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 should decrease. The QCSOSTF pump modification options, costs and benefits are discussed further in this section.

Existing Conditions

As shown in Figure 9-9, 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 continuously, sending flow 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 only activate 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 to prevent washing out the treatment process. The deactivation currently happens when UBWWTF influent flows reach 50 mgd to 70 mgd. This shut down 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 wet well and the 2.5 million gallon chlorine contact tanks. The wet well is located downstream of 1-inch bar screens. Given the relatively small size of the wet well or sump, the influent channel to the QCSOSTF and overflow collectors generally act as the wet well 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 wet well 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

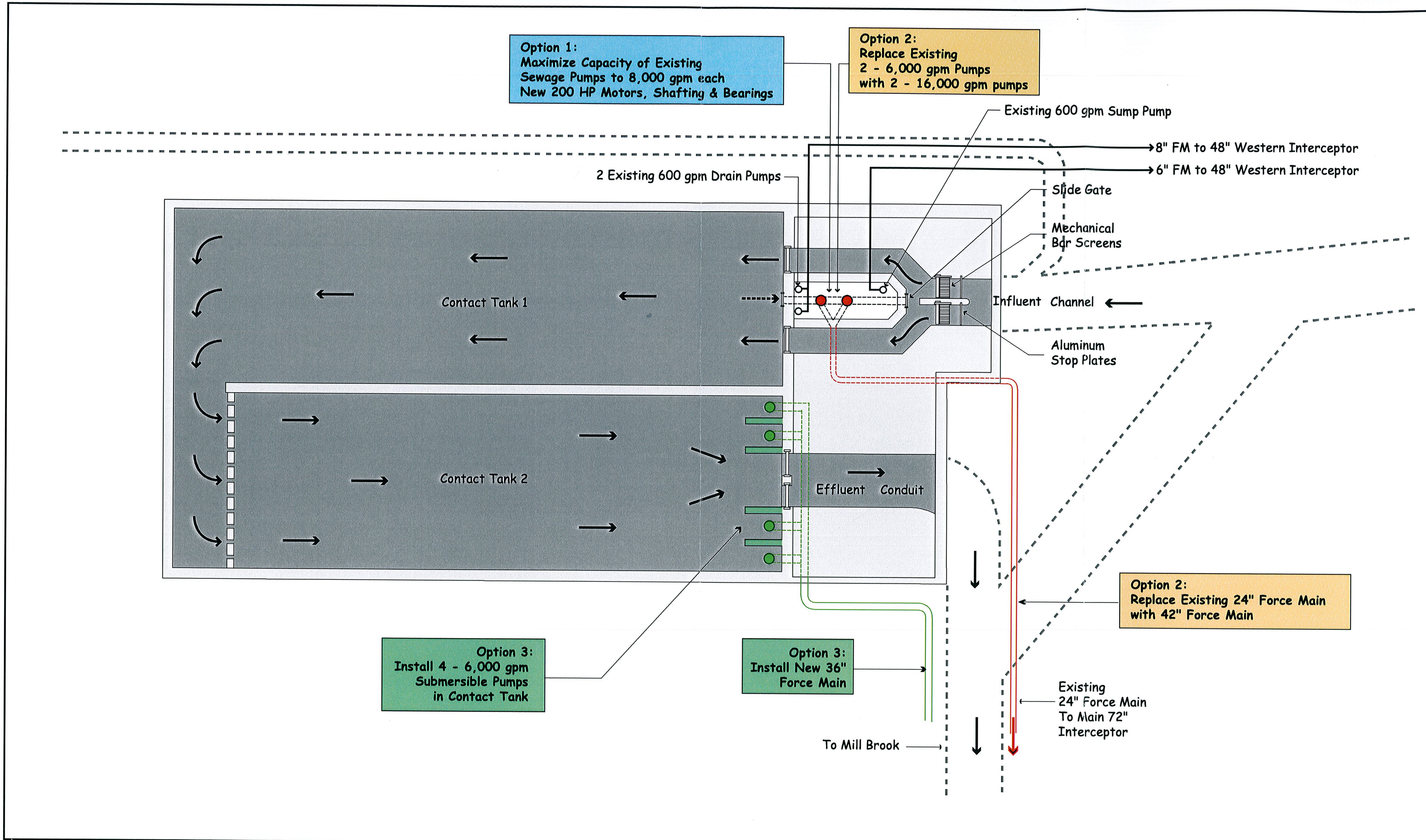

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Figure 9-9
Quinsigamond CSO Storage and Treatment Facility
Pumping Alternatives

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 wet well water surface elevation of 426.75. The higher speed (885 rpm) is activated once the wet well reaches 427.0. The lag pump is equipped with a fixed speed (885 rpm) 100 HP motor, which is activated when the wet well 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 under consideration for increasing the pumping capacity as shown in Figure 9-9. 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. The selected approach will depend on feasibility, cost, and effectiveness in terms of reduction in treated discharge at the QCSOSTF and relative improvements to water quality.

Option 1 - Optimize Existing Sewage Pump Capacity

Based on preliminary analysis and confirmation with the local Wemco representative, it was determined that the capacity of the 2-6,000 gpm pumps may be increased by approximately 1/3 of their current capacity to 8,000 gpm each. This was based on an evaluation of available system and pump curve information. 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 is possible by increasing the speed of the pumps from 850 rpm to approximately 1100 rpm through replacing the 2-100 HP motors with 2-200 HP motors. Shaft and bearing frame replacement would also be required to handle the higher loads that come with the higher speeds. It is noted that the risk of vibration issues increases at higher speeds as well. 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 flow capacity may not be gained from replacing the impellers.

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 wet well. 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 event that the motor room flooring would require 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 assumes that the power supply to the QCSOSTF is sufficient to handle the increased demands of the higher horsepower motors. It also assumes that the existing MCC is adequate. Electrical allowances may vary significantly depending on specific requirements of the facility. These details should be evaluated in future phases of planning and/or design.

A lower cost way of increasing the effective pumping capacity it to raise the pump operating levels in the wet well, since the static head requirement is reduced. This option would involve modifying the operating levels of the QCSOSTF, and increasing the wet well elevation at which the bypass gate is opened. This is not recommended due to the increased potential for flooding upstream.

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 9-12 and in Figure 9-2.

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 9-12
Comparing QCSOSTF Pumping Alternatives to UBWWTF Improvements

Parameter	1-Month	3-Month	6-Month
Volume Treated 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	3.5
UBWWTF Improvements Plus + Installing New Submersible Pumps	0	0.4	4.1
Volume Treated 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 9-9. Based on review of design drawings and photographs of the QCSOSTF, there appears to be sufficient room in the 30 foot by 15 foot pump room for larger capacity pumps to be installed. Based on discussions with the Wemco representative, it is considered feasible to replace the existing pumps with 2-16,000 gpm pumps. The recommended pumps are L-20K-SS Wemco pumps, operating at 800 rpm with 250 HP motors.

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. It is likely that the 1,200 lineal foot 24-inch force main would need to be replaced if the sewage pump capacity were increased to 32,000 gpm, since the velocities would increase to 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). This may also decrease the motor horsepower requirements. The 16-inch suction and discharge piping would also be upsized to match the pump size. It is noted that the suction lines may be difficult to upsize, since the suction piping and header are built into the floor of the pump room. This would require significant structural allowances and difficult installation. The feasibility of this option should be evaluated further in light of this point.

Costs
The total estimated cost of this option is $1.7 M, including allowances discussed previously.

This assumes that the power supply to the QCSOSTF is sufficient to handle the increased demands of the new pumps and motors, and that the existing MCC is adequate. Electrical allowances may vary significantly depending on specific requirements of the facility. These details should be evaluated in future phases of planning and/or design as they could add significant cost to the estimates below.

Water Quality Benefits
Figure 9-2 and Table 9-12 above 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 $1,700,000, this is about $0.57 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 9-9. This option would involve the installation of separate submersible pumps with control system and a separate force main. The force main would likely discharge into the 72-inch Main Interceptor, near the existing 24-inch force main discharge point. There may be siting issues associated with constructing a new 1,200 foot long force main due to the rail yard to the West of the facility and the congested area to the south and east of the facility. However, this option would enable a significant increase in pumping capacity without modification to the existing pumps and piping.

It was assumed that the submersible pumps would be sized to deliver the same flow and TDH as the existing pumps (6,000 gpm at 44 TDH). Assuming four pumps, the total capacity increase would be 24,000 gpm. The new total pumping capacity of the QCSOSTF would be 54 mgd, if the sewage pumps remain at 6,000 gpm and the drain and scum pumps remain at 600 gpm each.

Costs
The estimated cost for this alternative is $1.9M, including appropriate allowances.

Water Quality Benefits
Figure 9-2 and Table 9-12 above 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 $1,900,000, this is about $0.56 per gallon of discharge reduced during 3-month storm conditions.

Other Pumping Related Issues
It may be advisable to install an air vent at the high point of the existing 24-inch force main beneath Cambridge Street, if one is not already in place. Based on available Contract No. 6 records drawings, there does not appear to be an air vent at this high point, which may lead to reduced capacity in the force main.

It is also advisable to confirm the pressure and headloss through the force main to ensure that the pump evaluation is realistic. This may be accomplished through inserting a pressure gage in the line. Given the age of the system and potential wear of the force main, it is good practice to re-evaluate friction and headloss assumptions.

The Wemco-Hidrostal pumps installed at the QCSOSTF are not known to handle grit very well. The performance and wear of the existing pumps should be discussed with DPW staff to determine if grit in the combined sewer flows has created any problems. If so, it may be prudent to replace the pumps for more than capacity reasons.

9.7 Treatment Improvements

These alternatives result in improved treatment for CSO flows and include high-rate clarification (HRC) as enhanced treatment at the QCSOSTF.

9.7.1 High Rate Clarification at QCSOSTF

The following describes the development and evaluation of the high-rate clarification alternative at the QCSOSTF. The concept, costs, and recommended approach are detailed below.

Process Description
High-rate clarification, or ballasted flocculation as it is also commonly called, is a relatively new process for treating wet weather flows. This technology, which could be operated intermittently as needed during storm conditions, is essentially a physical-chemical process in which coagulant and polymer are added to the influent

wastewater. The resulting floc particles adsorb onto a heavier material such as very fine sand added to the wastewater, or recirculated solids. The heavier material acts as ballast and increases the settling rate of the adsorbed floc.

High-rate clarification is able to provide significantly higher treatment capacities than conventional primary treatment due to the chemically enhanced and ballasted settling. This results in significantly higher BOD and TSS removals in a much smaller footprint. The BOD and TSS removal rates associated with high-rate clarification have been shown to be roughly double those of traditional clarification, while the area requirements are only one-tenth of the traditional area requirements. All of this means that treatment would be enhanced over the existing screening, storage/sedimentation, and disinfection provided at the QCSOSTF for wet weather flows.

In light of this discussion, high-rate clarification is considered a viable alternative to evaluate further for providing a higher level of treatment of CSOs delivered to the QCSOSTF. However, the cost of this proprietary technology and siting considerations will have to be weighed against the benefits to determine its true viability in the project area. Furthermore, potentially higher O&M requirements associated with this technology may limit its applicability. These points are discussed further in the following sections.

Suppliers

At least two suppliers, U.S. Filter Kruger and Ondeo Degremont, provide high-rate clarification systems. The U.S. Filter Kruger system is called Actiflo and the Ondeo Degremont system is called DensaDeg. The Acitflo process uses a recirculated microsand as ballast, whereas the DensaDeg process uses recirculated solids that have been settled out of the wastewater.

A typical Actiflo system consists of addition of ferric chloride, polymer, and "microsand" (sand approximately 100 microns in diameter) to the wastewater. The wastewater and additives are rapidly mixed (flash mixing) and then slowly stirred in a maturation tank before settling in a clarifier. The sludge from settling is passed though a hydrocyclone, where the microsand is removed from the sludge and recycled. The dilute sludge, which is roughly 0.6 percent solids, may be pumped to a solids handling facility for thickening, to a holding tank, or directly to the interceptor if capacity is available in the combined sewer system.

The DensaDeg process is very similar to the Actiflo process except that settled solids are recirculated to act as the ballast, rather than adding microsand. This eliminates the need to include a hydrocyclone and other sand handling/replacement needs. However, it requires maintenance of a sludge blanket and dependence on other processes, as well as increasing odor issues. DensaDeg essentially acts as a gravity sludge thickener, which puts out a higher solids concentration that requires no thickening. This may be an advantage if solids would be handled on-site or stored, but it may create problems if solids are to be pumped back into the interceptor.

Both systems are capable of treating intermittent flows and loads associated with CSOs and require a relatively short amount of time (approximately 20 minutes) to reach full treatment capacity. Options for handling the first 20 minutes of flow may need to be considered further, if this is a concern. Although both technologies are relatively new, Actiflo appears to have more installations across the US. Two of these applications, a 15 MGD Bremerton, WA CSO application and a 110 MGD Forth Worth, TX wet weather treatment train application, are discussed briefly below as a comparison to the potential Worcester application under consideration.

Based on the above discussion, Actiflo will be considered as the preferred method of high-rate clarification for the purposes of this CSO facilities plan.

Preliminary Layout

Figure 9-10 shows a preliminary layout of a potential Actiflo facility. The facility shown is for a 200 MGD capacity. This is below the rated capacity of the QCSOSTF, but is high enough to handle a 5-year storm, as modeled. The Actiflo process is capable of handling flows up to 150% of its rated capacity without significant decline in treatment effectiveness. Therefore, a 200 MGD system could handle flows up to 300 MGD. Flows above this amount would be bypassed through the existing effluent conduit.

As shown in Figure 9-10, flow is routed from the existing 10-foot by 12-foot effluent conduit through fine screens and into the Actiflo process. The fine screens (10 mm spacing) are required to prevent the hydrocyclones from clogging. It may be possible to install the fine screens in the existing QCSOSTF to reduce the HRC footprint. It also may be possible to avoid fine screens altogether if it is shown that the settling that occurs in the QCSOSTF contact tanks is sufficient to remove particles larger than 10 mm on a consistent basis for a variety of storm conditions. These options would need to be evaluated further if this option is carried into Phase 2 of this LTCP. Another option is to install fine screens in the influent channel to the existing QCSOSTF, possibly downstream of the existing coarse screens. Headloss and space constraints would need to be evaluated further in the next phase of planning.

As discussed above, coagulant, polymer and microsand are added to the flow stream in the rapid mixing injection tank, before passing under a baffle into the maturation tank. Flow passes over a weir wall into the clarifier equipped with lamellar plates and a scraper. Upwelling through the lamellar plates polishes the effluent before it flows over v-notched weirs and into collection troughs. The collection troughs then discharge to the HRC effluent conduit, which would connect back to the existing 10-foot by 12-foot effluent conduit shortly downstream.

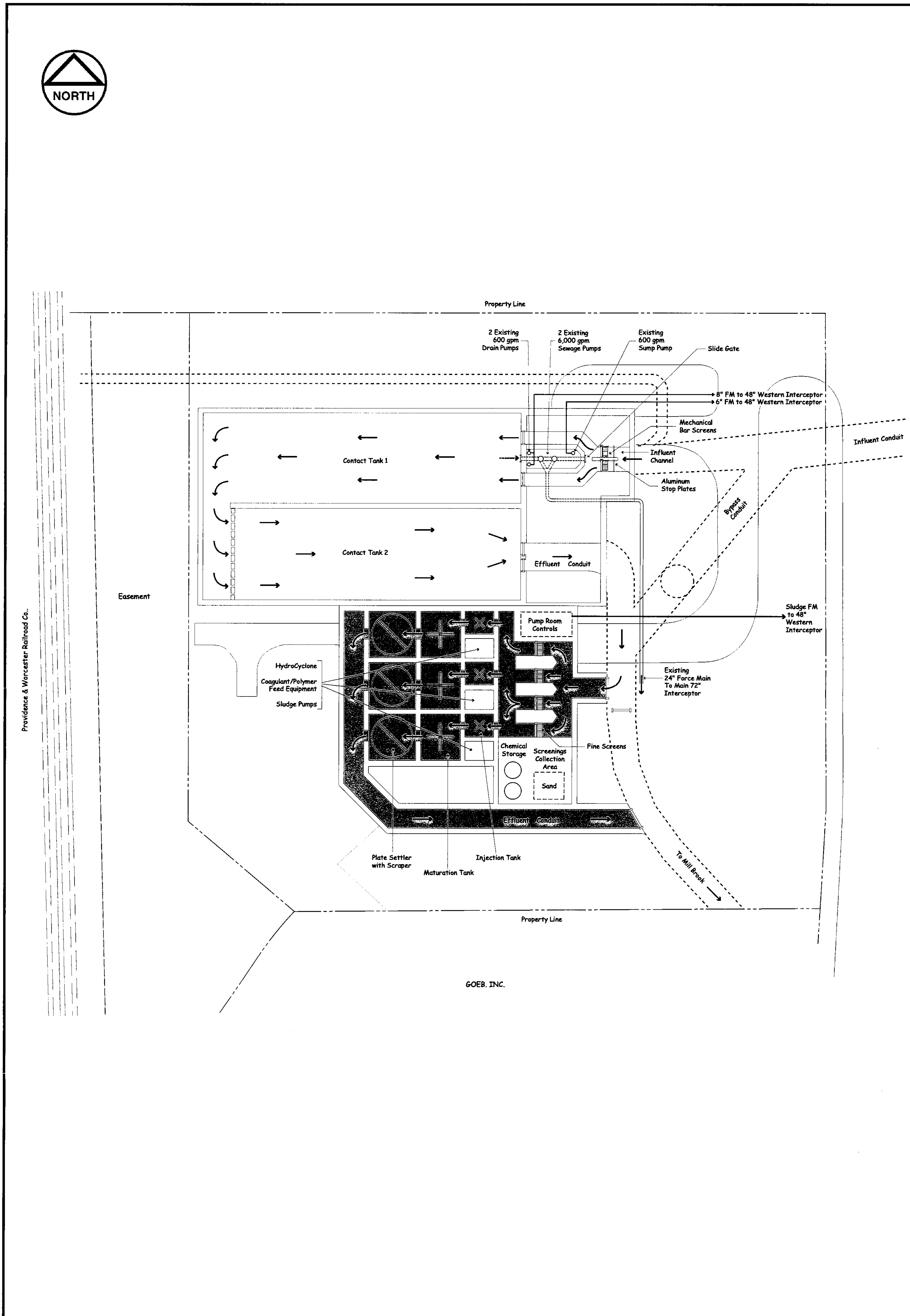

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Figure 9-10
Quinsigamond CSO Storage and Treatment Facility
High Rate Clarification Option

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Siting

The facility would most likely be sited on City land immediately to the south and downstream of the QCSOSTF, as shown in Figure 9-10. There do not appear to be any physical conflicts with the proposed site, since the railroad tracks and Nissen Baking Company flour unloading facility shown on the QCSOSTF drawings are reportedly no longer in service. The facility would only treat flows discharged through the effluent gates at the QCSOSTF. All other flows would be pumped back to the interceptor for treatment at the UBWWTF. Although the sequence of operations (disinfection followed by clarification) is not traditional, it is not considered to be a problem for treating the CSO flows. However, it is recommended that dechlorination operations be moved to the discharge point of the HRC facility to maximize chlorine contact time from the QCSOSTF through the HRC.

Installing an Actiflo system in the existing contact tanks at the QCSOSTF was also considered. However, this would require demolishing portions of the superstructure and tanks to excavate to the depth required for the Actiflo process. An additional 10 feet in depth would be required to meet the 27-foot depth requirement. This would require that the QCSOSTF be taken off-line until construction is completed and it would also complicate construction. Therefore, it is not considered practical and construction of a new high rate clarification system adjacent to the QCSOSTF is the preferred option. The HRC facility would be located immediately south of the QCSOSTF and would draw flows from the disinfected effluent in the existing 10-foot by 12-foot effluent conduit.

Among the challenges posed by siting the facility adjacent the QCSOSTF is the presence of contaminated or hazardous waste reported on-site. Also, new construction would most likely need to be limited to City land immediately south of the QCSOSTF. There is little room available to construct west of the QCSOSTF. Also, the HRC effluent conduit would most likely need to tie into the existing 10-foot by 12-foot effluent conduit upstream of the buildings sited immediately south of the QCSOSTF property line.

Preliminary Hydraulic Assessment

Preliminary hydraulic evaluations indicate that flow-through treatment at the HRC facility and discharge to the Mill Brook via the existing 10-foot by 12-foot effluent conduit may be possible by gravity. However, there are a number of questions and assumptions that require resolution and confirmation before the necessity for a large pumping station is ruled out. Such a large influent or effluent pumping station would add significant cost to the preliminary HRC facility estimate, as discussed below.

A detailed hydraulic profile should be completed to determine the hydraulic grade line and headlosses for a variety of flows. In order to establish this profile, water surface elevations and flows would need to be determined in the Mill Brook to evaluate downstream conditions. The hydraulic analysis would establish upstream water surface elevations through the HRC facility and the QCSOSTF from the downstream set points. The Actiflo representatives estimate that 1 to 2 feet of headloss may be expected through the Actiflo process.

For the purposes of this report, costs of a HRC facility will be presented assuming influent pumping is not required, but this assumption would need to be confirmed. Other alternatives to constructing a 200 MGD pumping station may also be considered. These include modifying the wet well operating levels in the QCSOSTF to increase the hydraulic grade line upstream of the HRC facility. Also, routing the HRC effluent conduit to a lower connection point in the Mill Brook is another option, although this will be difficult to site. The planned extension of the Mill Brook to a lower discharge point on the Blackstone River will also be valuable in terms of alleviating any tailwater effects and improving potential for gravity flow through the HRC.

Costs

The preliminary cost estimate for constructing a HRC facility to the south of the QCSOSTF as shown in Figure 9-10 is $22M, including allowances for hazardous waste as well as contractor overhead and profit, construction contingencies, and engineering and implementation. Again, if influent or effluent pumping is required, this cost will increase significantly. The $22M cost assumes that the electrical power supply to the existing QCSOSTF is sufficient to power the HRC facility. New power supply issues may increase the cost estimate significantly. Such design level electrical evaluations will need to be resolved further in the next phase of planning. Although not considered necessary by the process representative, grit removal options may also be worth considering in future phases of work. O&M costs and pilot study allowances should also be included in the overall cost estimate.

Actiflo Application Examples

Some other applications in the US are described below as a comparison for the potential Worcester application.

Bremerton, WA

The Bremerton facility was installed to treat CSO discharges to Puget Sound. The Actiflo process was sized for a peak flow of 15 MGD, but could handle 20 MGD hydraulically. The total cost of the facility was $2 million. It includes a fine screening facility upstream of the Actiflo process and UV disinfection downstream. The UV disinfection is made possible for treating CSO discharges by the high TSS removal effectiveness of the HRC process. The total size of the facility is roughly 70-foot by 50-foot and includes the Actiflo process and aboveground electrical and chemical buildings. The solids are pumped to a holding tank until the storm event subsides and the sludge may be pumped to the interceptor system for treatment at the WWTP. The facility has been online since December 2001.

Forth Worth, TX

The Fort Worth Actiflo facility was designed to handle a peak flow of 110 mgd but has not yet been constructed. The preliminary design of the Fort Worth application was completed in October 2000 to handle wet weather flows in excess of the secondary treatment capacity at the existing WWTP. The approximate footprint for the facility is 260-foot by 60-foot and includes the HRC equipment and tankage, an influent lift station, a chemical/sludge handling building, as well as space allowance for future grit removal equipment. Flow is diverted to the HRC facility after passing through a new coarse and fine screens facility at the head of the plant once influent flows approach the peak capacity of the traditional WWTP. This approach was viewed as a much lower cost alternative to upgrading the existing traditional treatment train to handle the excess wet weather flows. Solids are sent to on-site sludge thickening facilities. Also, effluent is routed to the aeration basins for the first 20 minutes of start-up to allow for ramping up to full treatment efficiency.

According to the opinion of probable cost in the October 2000 preliminary design manual, the Fort Worth HRC facility construction cost estimate is approximately $18 million. This cost includes a coarse and fine screenings facility with odor control, a pump station, and all required Actiflo process equipment and chemical handling facilities. It does not include the disinfection and sludge thickening aspects of the plant improvements project also included in the preliminary cost estimate.

Water Quality Benefits

The measurement used to quantify water quality benefits for the hydrologic response, storage, and conveyance alternatives do not apply to this HRC treatment alternative. The effectiveness of those alternatives was judged by how much of the combined sewage that discharges from the QCSOSTF could be transported and treated at the UBWWTF instead, where it would receive a higher level of treatment. However, the goal of HRC is to increase the treatment effectiveness at the QCSOSTF, and reducing discharge at the facility is not necessary.

For HRC, the measure of effectiveness is the reduction of BOD and TSS from the QCSOSTF, based on HRC treatment efficiencies. Section 7 presented the current effectiveness of the QCSOSTF, where BOD is typically reduced by 34 percent and TSS by 30 percent. HRC will significantly increase treatment effectiveness, and is reported to be capable of producing the same effluent quality as secondary treatment (30 mg/l BOD and 30 mg/l TSS). For this analysis, BOD from the QCSOSTF is already low (31 mg/l). Therefore, effectiveness was assumed to double, to 64 percent. With typical influent of 47 mg/l, typical effluent would then be 15 mg/l. For TSS, effluent was assumed to meet secondary levels of 30 mg/l, a 78 percent reduction.

With UBWWTF improvements in place, 3.8 MG will discharge from the QCSOSTF during 3-month storm conditions. Without new HRC facilities, BOD and TSS loadings from the QCSOSTF will be 980 and 3,200 pounds, respectively. With HRC in place, BOD will be reduced to 480 pounds, a 500 pound reduction, and TSS to 950 pounds, a 2,250 pound reduction.

Though these are significant reductions from the QCSOSTF, they are not significant when considering the combined impact of the QCSOSTF and the UBWWTF on the Blackstone River. On an annual basis, with UBWWTF improvements in place, about 6,300 pounds per day of BOD and 5,500 pounds per day of TSS are expected to discharge to the Blackstone River from the combined UBWWTF and QCSOSTF facilities. With HRC in place, this will be reduced by about 12 pounds per day BOD and 55 pounds per day TSS, 0.2 percent and 1.0 percent reductions, respectively.

In Section 9.3, it was established that UBWWTF high flow management facilities would cost about $54 for every pound of BOD removed per year, and about $4 for every pound of TSS removed. The cost of HRC facilities is estimated at $22M. This is about $2,100 per pound of BOD removed annually, and $450 per pound of TSS.

9.8 Combining Alternatives

Two additional simulations were performed, to evaluate the effectiveness of combining some of the more promising, relatively low cost CSO control measures. The results from these simulations are presented below. They are compared to conditions assuming UBWWTF improvements are in place.

9.8.1 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 $200,000;
- <u>Regulator Modifications</u>: Raise all system regulator weirs 0.5 feet at an estimated cost of $340,000; and
- <u>Kelly Square Control Station Modifications</u>: Rehabilitate and activate the Kelly Square gate, estimated to cost $200,000, and divert flow to the Western Interceptor, estimated to cost $125,000.

The total estimated cost for Alternative 1 improvements is $865,000.

9.8.2 Alternative 2

Alternative 2 combines the following CSO control measures:

- <u>Green Hill Pond Diversion</u>: Divert the Green Hill Pond drainage out of the CSS at an estimated cost of $200,000;
- <u>Regulator Modifications</u>: Raise all system regulator weirs 0.5 feet at an estimated cost of $340,000;

- **Kelly Square Control Station Modifications**: Rehabilitate and activate the Kelly Square gate, estimated to cost $200,000, and divert flow to the Western Interceptor, estimated to cost $125,000; and
- **Add New Pumps at the QCSOSTF**: Install new submersible pumps in the chlorine contact tanks at the QCSOSTF, at an estimated cost of $1,900,000.

Alternative 2 is identical to Alternative 1 except submersible pumps are added to the QCSOSTF. The total estimated cost for Alternative 2 improvements is $2,765,000.

9.8.3 Water Quality Benefits

Figure 9-11 shows the impact of Alternatives 1 and 2 for the 1-, 3-, and 6-month design storms. Table 9-13 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. During a 3-month storm, Alternative 1 reduces discharge from the QCSOSTF by 1.4 MG (compared to conditions with UBWWTF improvements in place). At a cost of about $865,000, this is about $0.62 per gallon of discharge reduced. Alternative 1 is expected to reduce BOD loadings by 175 pounds and TSS loadings by about 1,020 pounds during 3-month storm conditions

Alternative 2 completely eliminates the 3-month discharge. At a cost of $2.8M, this is about $0.74 per gallon of discharge reduced. Alternative 2 would reduce BOD loadings by about 475 pounds and TSS loadings by about 2,700 pounds.

On an average-day basis, these improvements do not translate into significant reductions in BOD and TSS loadings 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 2 pounds per day BOD and 14 pounds per day TSS. Even with Alternative 2 in place, the reduction in loads is expected to be only 6 pounds per day BOD and 34 pounds per day TSS. In terms of annual loadings reduction, Alternative 1 will cost about $990 for every pound per year of BOD removed and about $170 for every pound per year of TSS removed. Alternative 2 will cost about $1,315 for every pound per year BOD removed and $225 for every pound per year of TSS removed.

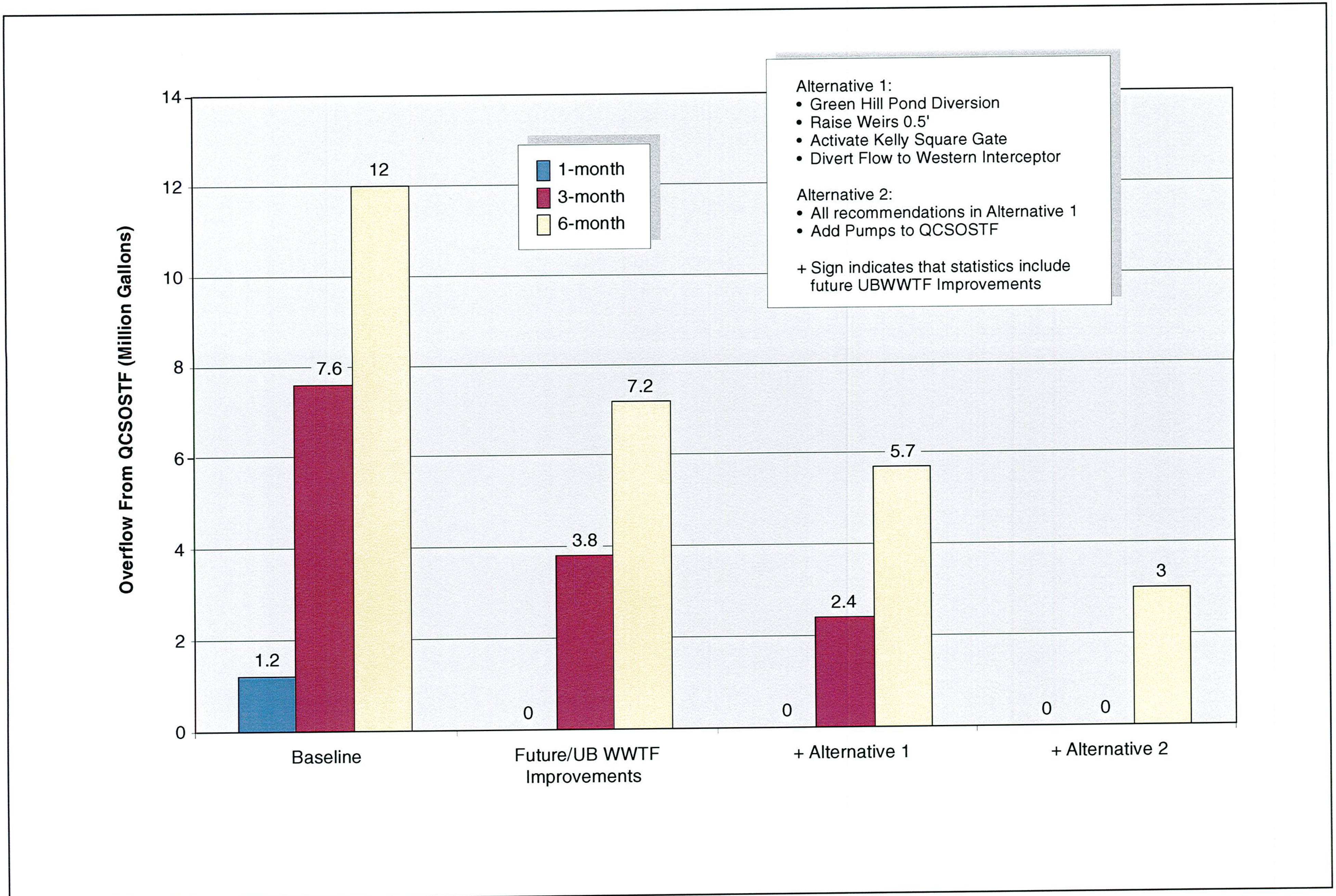

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Figure 9-11

Volume Reduction from Promising Alternatives Compared with Baseline Conditions

CDM

Table 9-13
Comparing Alternatives 1 and 2 to UBWWTF Improvements

Parameter	1-Month	3-Month	6-Month
Volume Treated at QCSOSTF, Million Gallons			
Baseline Conditions	1.2	7.8	12.0
With UBWWTF Improvements	0.0	3.8	7.2
With UBWWTF Improvements and Alternative 1	0.0	2.4	5.7
With UBWWTF Improvements and Alternative 2	0.0	0.0	3.0
Volume Treated at UBWWTF, Million Gallons (2 day simulation)			
Baseline Conditions	119.0	120.2	119.7
With UBWWTF	123.3	124.4	126.1
With UBWWTF Improvements and Alternative 1	123.3	125.1	126.3
With UBWWTF Improvements and Alternative 2	124.7	127.7	129.6

Comparing Annual Performance:
UBWWTF, Alternative 1 and Alternative 2

Parameter	Annual
Number of Treated Overflows from QCSOSTF	
Baseline Conditions	14
With UBWWTF Improvements	7
With UBWWTF Improvements and Alternative 1	5
With UBWWTF Improvements and Alternative 2	2
Million Gallons / Year Discharged From QCSOSTF	
Baseline Conditions	83
With UBWWTF Improvements	34
With UBWWTF Improvements and Alternative 1	27
With UBWWTF Improvements and Alternative 2	17

9.9 Comparing Costs and Effectiveness of Alternatives

Figure 9-12 shows the estimated costs of the CSO control alternatives. Note that the figure is truncated at $20,000,000, and the cost of full separation is $180,000,000. It shows the cost of separation, storage, and treatment improvements is large compared to other alternatives. Figure 9-13 shows cost effectiveness in terms the number of gallons of discharge reduced at the QCSOSTF per dollar spent for each alternative under 3-month storm conditions. Per dollar spent, the lower cost improvements are much more effective. Since high rate clarification does not reduce flow at QCSOSTF, it is not shown. Figure 9-14 shows reduced TSS loadings to the Blackstone River during the 3-month storm. It does not show alternatives that are expected to add loadings to the Blackstone River. It shows that pumping alternatives can be more effective than more expensive storage alternatives and as or more effective than high rate clarification. It also shows that Alternative 2, which combines many of the lower cost alternatives is the most effective TSS removal alternative considered.

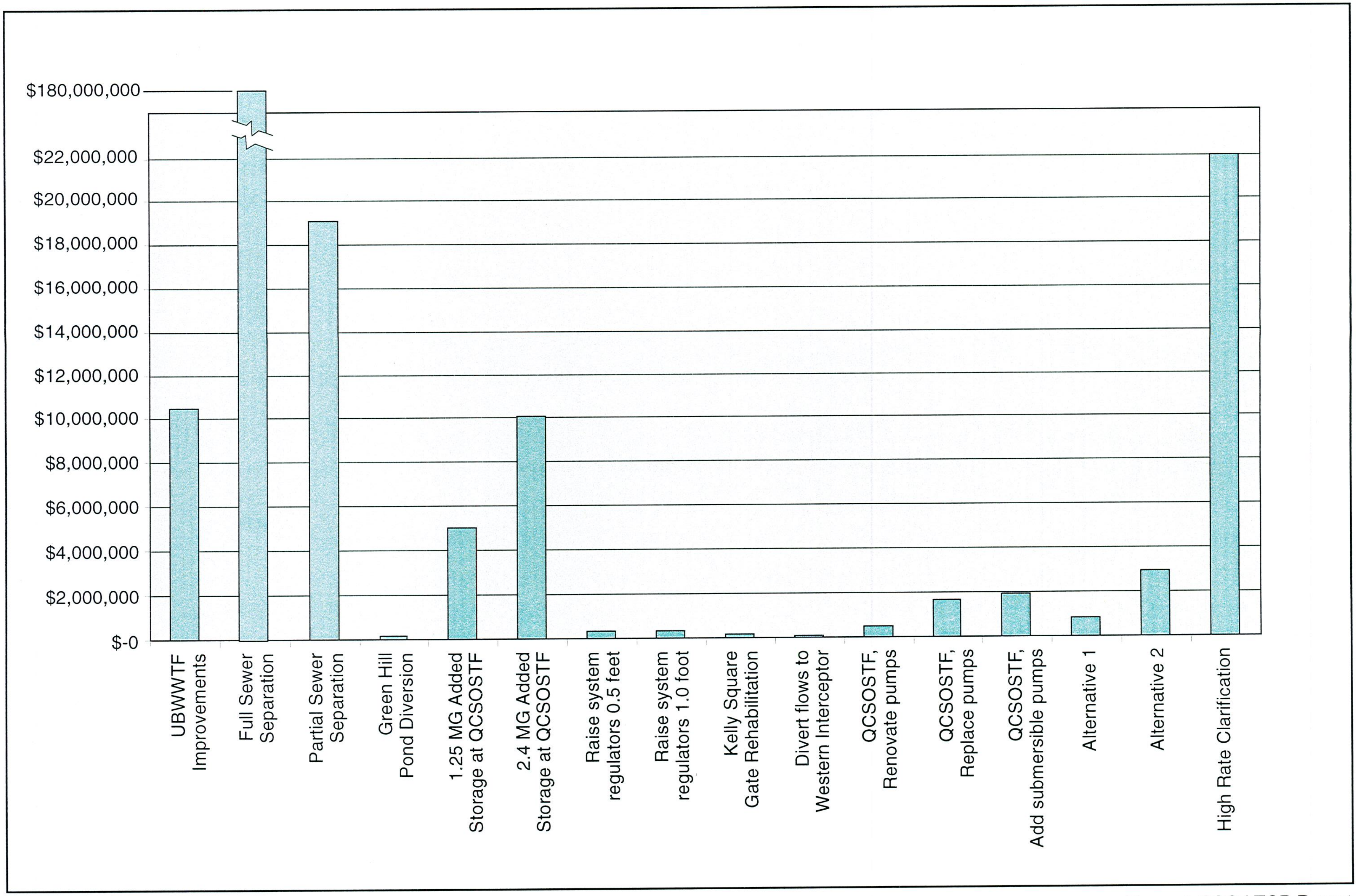

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Figure 9-12
Estimated Costs of CSO Control Alternatives

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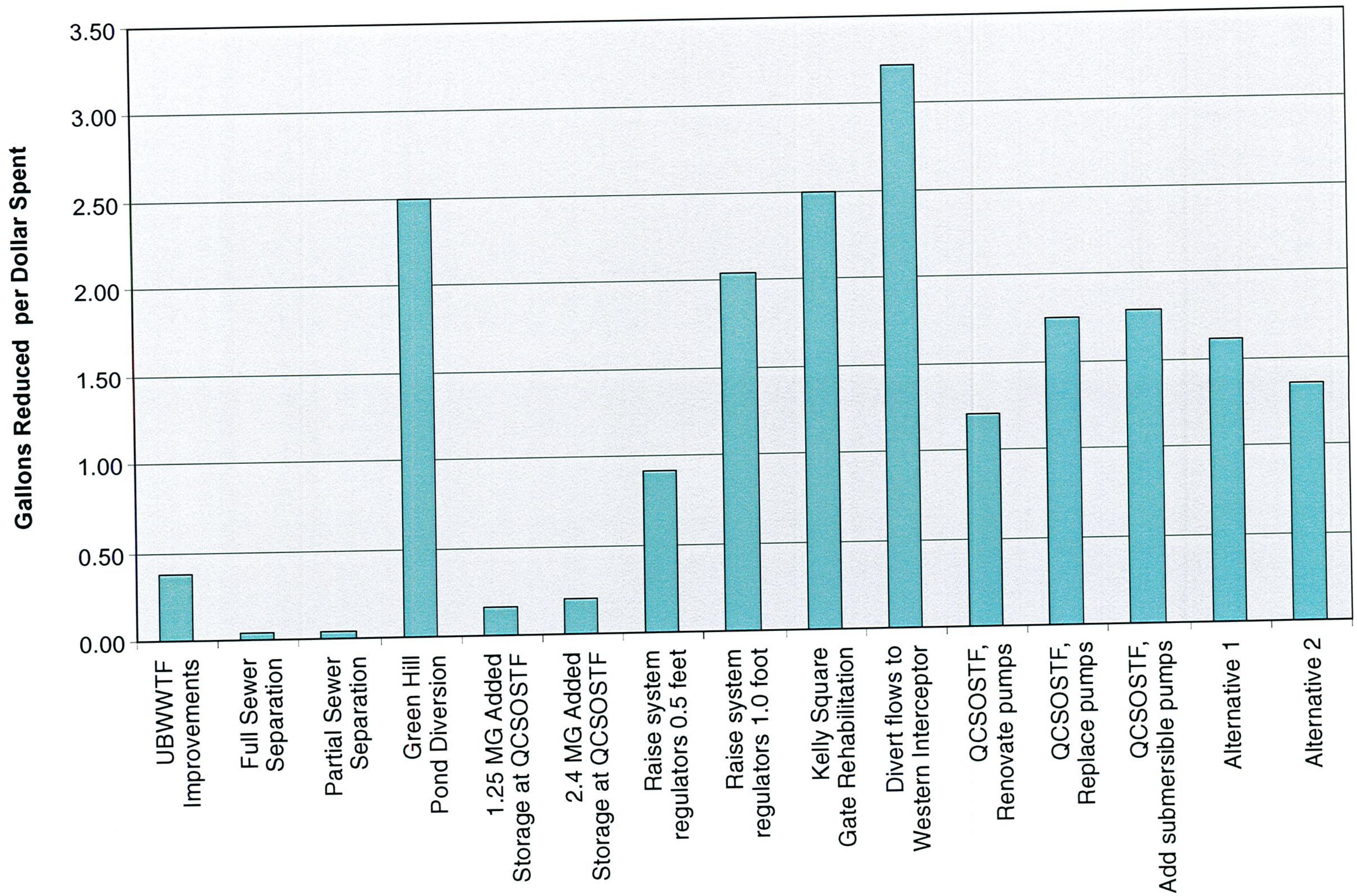

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Figure 9-13
3-month Storm: Gallons Reduced From QCCOSTF per $ Spent

CDM

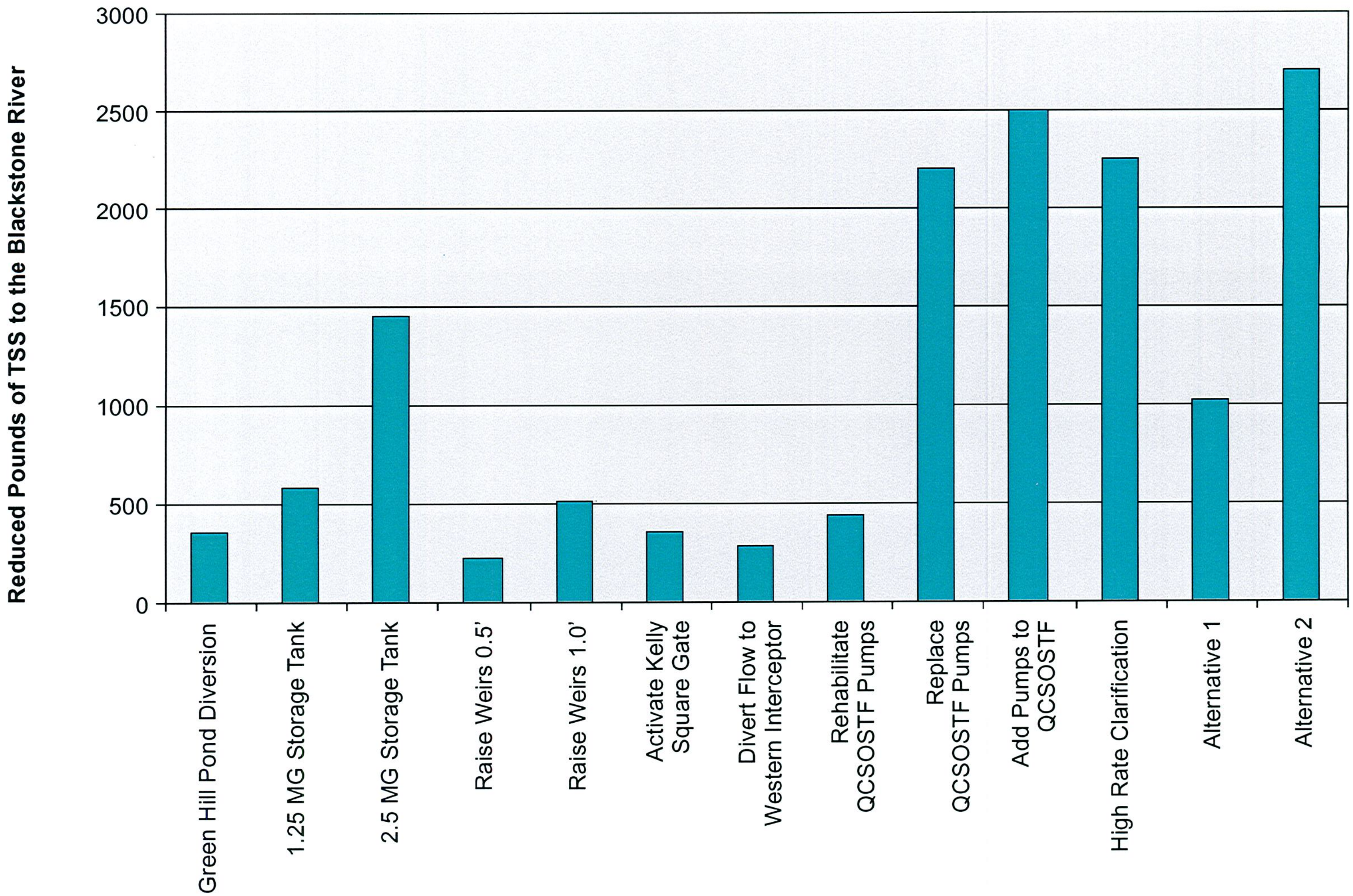

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Figure 9-14
Reduced TSS Loadings to the Blackstone River 3-month Storm

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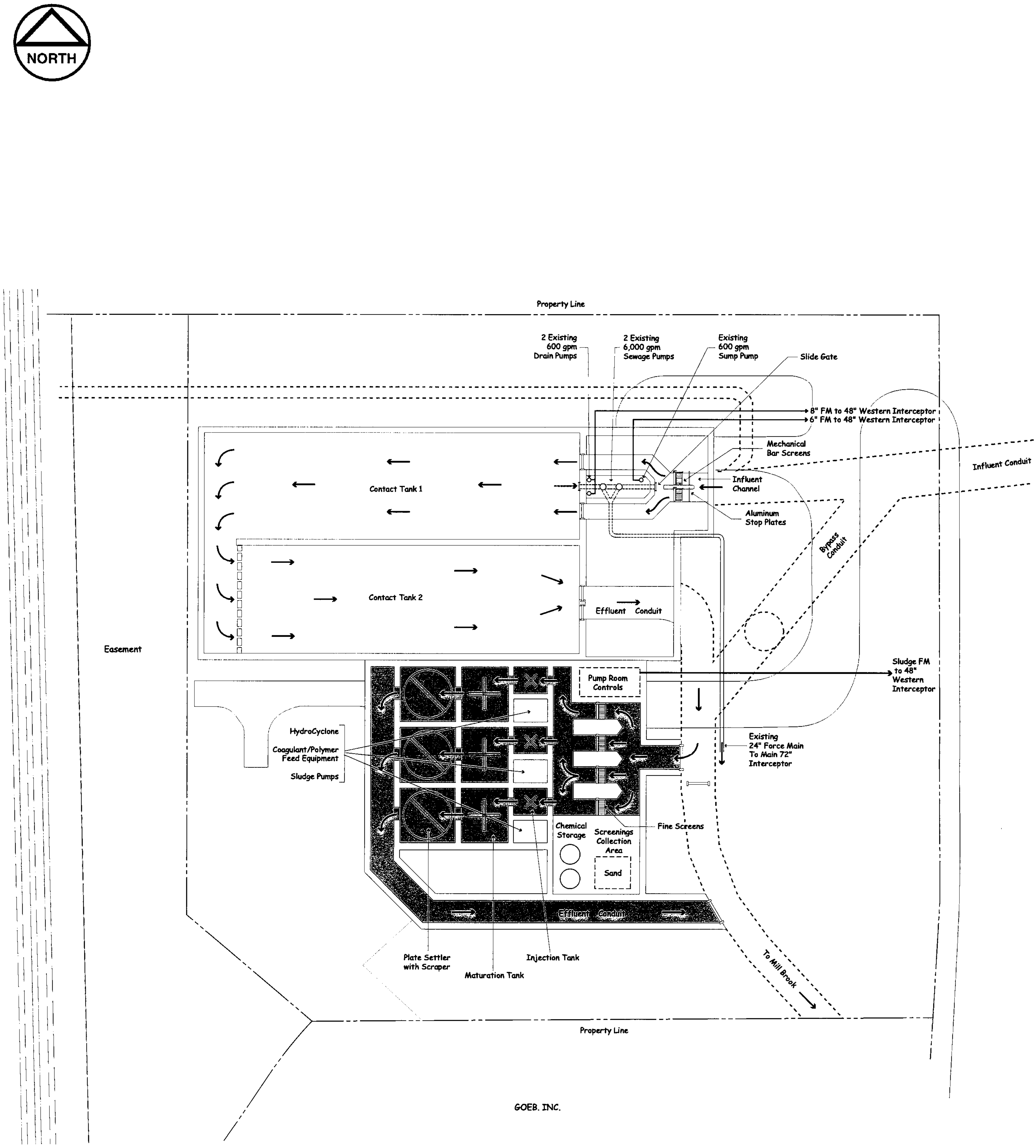

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Figure 9-10
Quinsigamond CSO Storage and Treatment Facility
High Rate Clarification Option

CDM

Section 10
Financial Impacts of Proposed System Improvements

10.1 Introduction

The City of Worcester is conducting Phase I of a two-phase CSO Long-term Control Plan. Unlike most communities, the City already has extensive CSO control facilities. During Phase I, the City is evaluating the need for additional programs to further mitigate the impacts of combined sewer overflows. At the same time, the City is also undertaking capital improvements to upgrade and improve the efficiency of its sewer system. The Upper Blackstone Water Pollution Abatement District (the "UBWPAD"), which provides wastewater treatment services to the City, is also undertaking a $120 million capital improvement program that affects the City's cost of sewer services. The purpose of this section is to develop a baseline projection of sewer expenses and rates for the City in advance of the CSO program being developed, then to evaluate the rate impact of alternative CSO improvement programs.

The accounting information as maintained by the City enables evaluation of the City's revenues and expenses for the purpose of determining appropriate rates and anticipated impacts from the proposed capital improvements.

Based on Worcester's current capital plan, the City anticipates spending approximately $20 million over the next five years for sewer system related improvements. (This is independent of any improvements undertaken by the UBWPAD.) Annual capital improvement projects for the City range from new sewer construction to building rehabilitations to various sewer structure improvements.

10.2 Methodology

CDM projected expenses, revenue requirements, and rates using standard industry methods. The analysis relies heavily on data and information provided by the City. Sewer revenue requirements are projected for a ten-year forecast period (through 2012). These costs are then allocated to retail and wholesale ratepayers in accordance with current City funding principles, and the impact on typical households is illustrated.

CDM assessed the City's required revenue requirement taking into account likely changes in capital and operating costs, outstanding debt service and likely changes in sewer demand. The basis for these projections is the City's fiscal year (FY) 2002 approved budget, as well as projections developed by the UBWPAD. CDM developed a spreadsheet based forecasting model that allowed quick and systematical alternatives evaluation, including the impact of the UBWPAD's improvement program.

CDM determined the annual revenue requirements for sewer revenues. Among the issues that were addressed were the evaluation of capital financing requirements based upon the proposed capital improvement programs versus capital funding sources that were utilized. The amount raised through the rate structure was calculated based on annual cost and projected revenues raised through the continued application of the current rate structure.

10.3 General Assumptions

CDM developed projections of the potential impacts of the planned wastewater improvements for FY 2003 through 2012 using the following key assumptions:

- The cost of operating and maintaining Worcester's sewer system will be recovered through sewer user fees assessed to Worcester's current wholesale and retail customers.
- The City's non-CSO related capital improvements would continue throughout the entire forecast period at the rate planned in the City's current 5-year plan.
- Annually, the City's share of the UBWPAD operating and maintenance costs will be 91.2 percent from 2003 through 2009 declining to 85.0 percent from 2010 through 2019. The City's share of UBWPAD's operating and maintenance costs is based on inflow projections developed by the UBWPAD through FY 2019.
- Beginning in FY 2003, the UPWPAD is assumed to undertake a $120 million improvement program. The costs include $107 million for the treatment plant upgrade and $13 million for the design costs. It is assumed that the District will finance that improvement program using general obligation debt carrying a 5.5 percent interest rate over a 20-year term.
- UBWPAD's supplemental funds will increase by 7.5 percent in 2003, 5.0 percent in 2004, 2.5 percent in 2005, and the growth rate will remain at 0 percent annually thereafter.
- Labor costs, operation and maintenance expenses and other expense data used in the wastewater rate model are based on the City's FY 2002 sewer budget and inflated to future years assuming a 3 percent annual inflation rate.
- Miscellaneous revenues and other revenue data used in the wastewater rate model are based on the FY 2002 sewer budget and held constant at the FY 2002 levels.
- Total billable consumption will remain constant at 7,383,000 hundred cubic feet (HCF) as in FY 2002.
- Existing debt service is based on the current debt schedules provided by the City of Worcester.

- The City will fund its capital improvements with General Obligation (GO) debt assumed to carry a 6 percent interest rate for a 20-year term.
- As of FY 2002, the current sewer user rate is $1.78 per hundred cubic feet (HCF) billed quarterly and the annual sewer cost per 120 HCF of water usage is $171. (Sewer bills are based on 80 percent of metered water usage.)

10.4 Data Sources

CDM used the following data sources for these baseline projections:

- Fiscal Year 2002 Sewer Enterprise Budget
- UBWPAD's Fiscal Year 2002 Budget
- 2000 Annual Water and Sewer Retail Rate Survey

10.5 Financial Analysis

In this section, the impacts existing planned improvements have on the financial requirements of the City are described. These planned improvements include the impacts of improvements at the Upper Blackstone WWTF. They do not include the impacts of further improvements to Worcester's combined sewer system. Combined sewer system improvements are evaluated in Section 10.6.

10.5.1 Revenue Requirements and Projections

This section defines revenue requirements for the sewer system. The three main components of revenue requirements include operations and maintenance expenses, capital costs, and miscellaneous revenues. For purposes of this presentation, the total revenue requirement is projected.

The costs associated with operations and maintenance expenses are departmental salaries, sewer maintenance expenses, UBWPAD's sewer maintenance expenses, costs allocation, fringe costs, and capital equipment costs. The capital costs include existing debt service of principal outside debt limit, long and short-term debt; capital outlay consists of improvement projects and equipment replacement; and new CIP system improvements. The last main component of revenue requirements is miscellaneous revenues that consist of utility interests and liens and other miscellaneous revenues that offset total expenses.

Table 10-1 summarizes the operations and maintenance costs for FY 2002 and FY 2012. Total operating and maintenance expenses are projected to increase from approximately $11.5 million in FY 2002 to nearly $28.4 million in FY 2012, an average annual increase of approximately 10.3 percent. UPWPAD costs include operations and maintenance costs and District debt service.

Table 10-1
Operations and Maintenance Costs

Expenses	FY 2002	FY 2012
Salary	$2,253,764	$3,028,870
Sewer Maintenance	$1,471,935	$1,978,158
Sewer Maintenance- UBWPAD	$2,871,377	$18,646,513
Costs Allocation	$1,911,482	$2,568,872
Fringe Costs (Health Etc.)	$1,562,052	$2,099,267
Capital Equipment Costs	$1,406,304	$1,889,955
Total Expenses	**$11,476,914**	**$30,211,635**

Table 10-2 lists the capital improvement projects assumed for the City excluding any work that may be undertaken as a result of the City's CSO Long-term Control Plan. The projects to be undertaken are new sewer construction, improvements for infiltration control, sewage pumping reconstruction, sewer reconstruction, building rehabilitation, and other sewer improvement projects. Total capital improvement projects will be $3.3 million in FY 2002. The annual capital improvement project costs will average approximately $4.0 million through FY 2012.

Table 10-2
City's Capital Improvement Projects

Expenses	FY 2002	FY 2012
New Sewer Construction	$100,000	$300,000
Infiltration Control	$400,000	$500,000
Sewage Pumping Reconstruction	$200,000	$200,000
Sewer Reconstruction	$1,300,000	$1,500,000
Building Rehabilitation	$500,000	$600,000
Other Sewer Improvements	$800,000	$900,000
Total Costs	**$3,300,000**	**$4,000,000**

Table 10-3 summarizes the existing and new debt service. The total debt service for the City will increase from $3.8 million in FY 2002 to $4.7 million in FY 2012. Existing debt service will decrease from $3.6 million in FY 2002 to $0.6 million in FY 2012. New GO debt service will increase from $0.2 million in FY 2002 to $4.1 million in FY 2012 due to capital improvement projects. (These figures exclude any debt service associated with the City's CSO Long-term Control Plan, beyond debt already issued for planning studies.)

Table 10-3
Debt Service

Expenses	FY 2002	FY 2012
Total Existing Debt Service	$3,594,437	$594,437
New GO Debt	$198,000	$4,090,750
New SRF Debt	$0	$0
Total Debt Service	**$3,792,437**	**$4,685,187**

Miscellaneous revenues are the third element of revenue requirement. In 2002, the City estimates that it will receive approximately $1.6 million from these sources including payments received as part of the MWRA's wastewater protection program. Miscellaneous revenues will remain at $1.6 million through 2012.

Table 10-4 summarizes the total operational and maintenance costs, existing and new debt service, and miscellaneous revenues to calculate rate revenue requirements for FY 2002 and FY 2012. Total expenses are projected to increase from $15.3 million in FY 2002 to $33.0 million in FY 2012, a 9.3 percent average annual increase. Net rate revenue requirements will increase from approximately $13.7 million in FY 2002 to $31.5 million in FY 2012, a 9.3 percent average annual increase.

Table 10-4
Sewer Revenue Requirement

Expenses	FY 2002	FY 2012
O&M Costs	$11,476,914	$28,338,071
Existing Debt Service	$3,594,437	$594,437
New Debt Service	$198,000	$4,090,750
Total Expenses	**$15,269,351**	**$33,023,258**
Miscellaneous Revenues	$1,577,698	$1,577,698
Net Rate Revenue Requirement	**$13,691,653**	**$31,445,560**

10.5.2 Impact on Customers

We evaluate the impact on customers based on total expenses and the net rate revenue requirement.

10.5.3 Sewer Rate Projections

Sewer customers are obligated to pay through user fees the costs of operating and maintenance expenses in addition to total debt service. As seen above, the net rate revenue requirement totals are projected to increase from approximately $13.7 million in 2002 to $31.5 million in 2012. Sewer use fees will need to generate a total of $31.5 million to maintain the solvency of the sewer fund in 2012.

The sewer customer user rate is based on the net rate revenue requirement and the total amount of consumption. The sewer rate will increase from $1.78 per HCF in FY

2002 to $4.24 per HCF in FY 2012. Based on average annual household consumption of 120 HCF, the sewer household bill will more than double from $171 in FY 2002 to approximately $409 in FY 2012. This is an average annual increase of 9.1 percent. The FY 2002 through FY 2012 sewer bill will average approximately $325 per year. Figure 10-1 shows the expected average sewer bill for FY 2002 through FY 2012.

10.6 Rate Impact of Potential CSO Control Alternatives

In this section, the added impact on the financial requirements of the city and its residents attributable to additional CSO control facilities is described. Two scenarios are evaluated. The first scenario describes the impact assuming the Worcester CSS is separated, the most costly CSO alternative evaluated. The second scenario describes the impact assuming a combination of potential CSO control alternatives are constructed, as described below in Section 10.6.2.

10.6.1 Rate Impact of Sewer Separation

As described in Section 9, the cost of total sewer separation in Worcester is estimated to be $180M. To determine the impact of this significant capital expense on sewer bills, the same general assumptions as described in Section 10.3 were used. In addition, sewer separation construction was assumed to take place over a 10-year period starting in FY 2003, and the City will finance the sewer separation program using general obligation debt carrying a 5.5 percent interest rate over a 20-year term.

With sewer separation, the sewer rate will increase from $1.78 per HCF in FY 2002 to $6.98 per HCF in FY 2012. Based on average annual household consumption of 120 HCF, the sewer household bill will increase from $171 in FY 2002 to approximately $671 in FY 2012, an average annual increase of 14.7 percent, and an almost a four-fold increase about current levels. The FY 2002 through FY 2012 sewer bill will average approximately $450 per year. Figure 10-2 shows the expected average household sewer bill for FY 2002 through FY 2012 with and without sewer separation.

10.6.2 Rate Impact of Lower Cost CSO Alternatives

Section 9 showed that significant improvements in the performance of an already-well-performing CSS can be achieved with a combination of improvements at a low cost compared to sewer separation. The cost of the following improvements were evaluated to determine their impact on average household sewer bills:

- Rehabilitate Kelly Square Gate structure;
- Add weirs to system regulators;
- Increase flow to the Western Interceptor; and
- Add pumping capacity to the Quinsigamond CSOSTF.

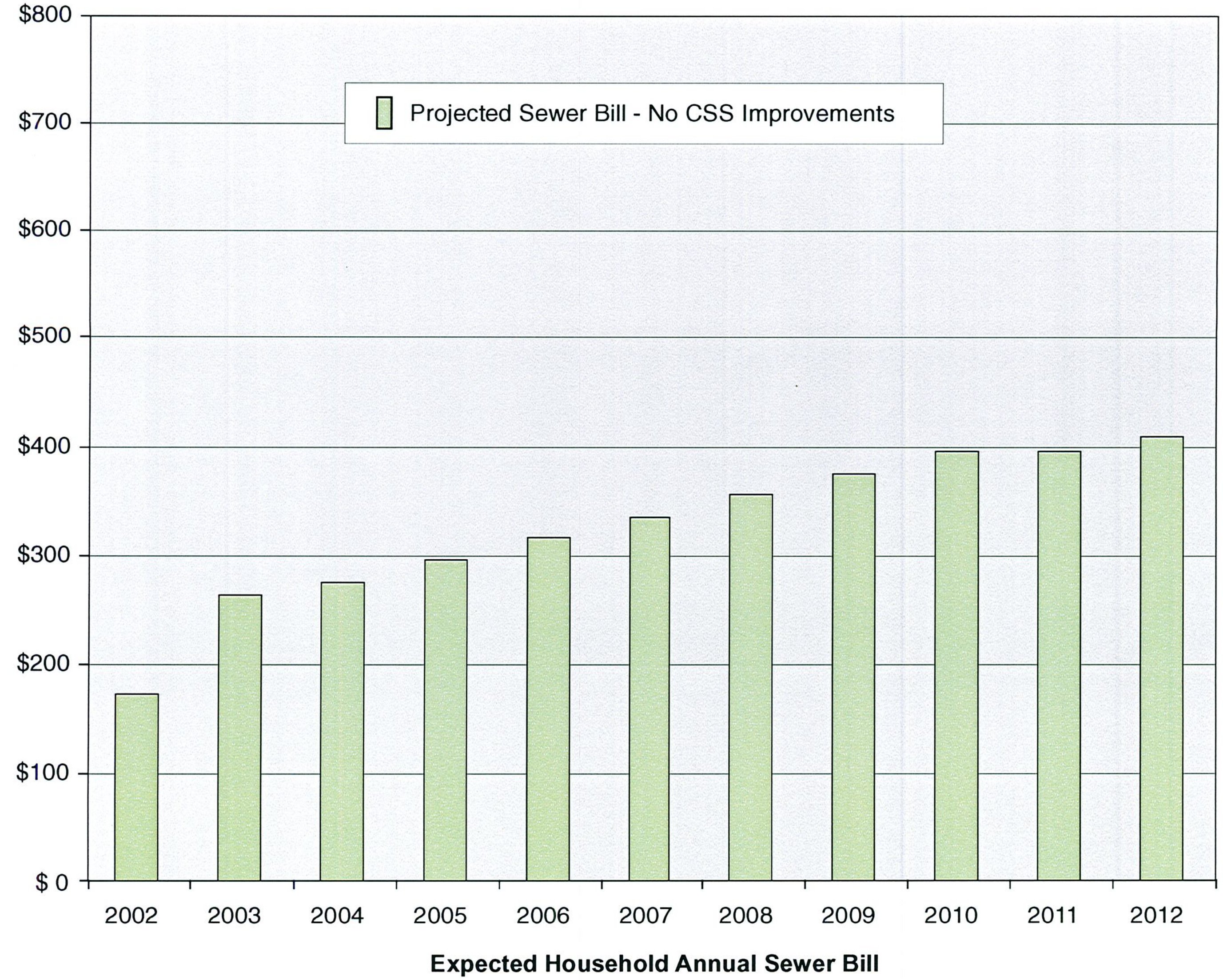

Figure 10-1
Expected Household Annual Sewer Bill With No CSS Improvements

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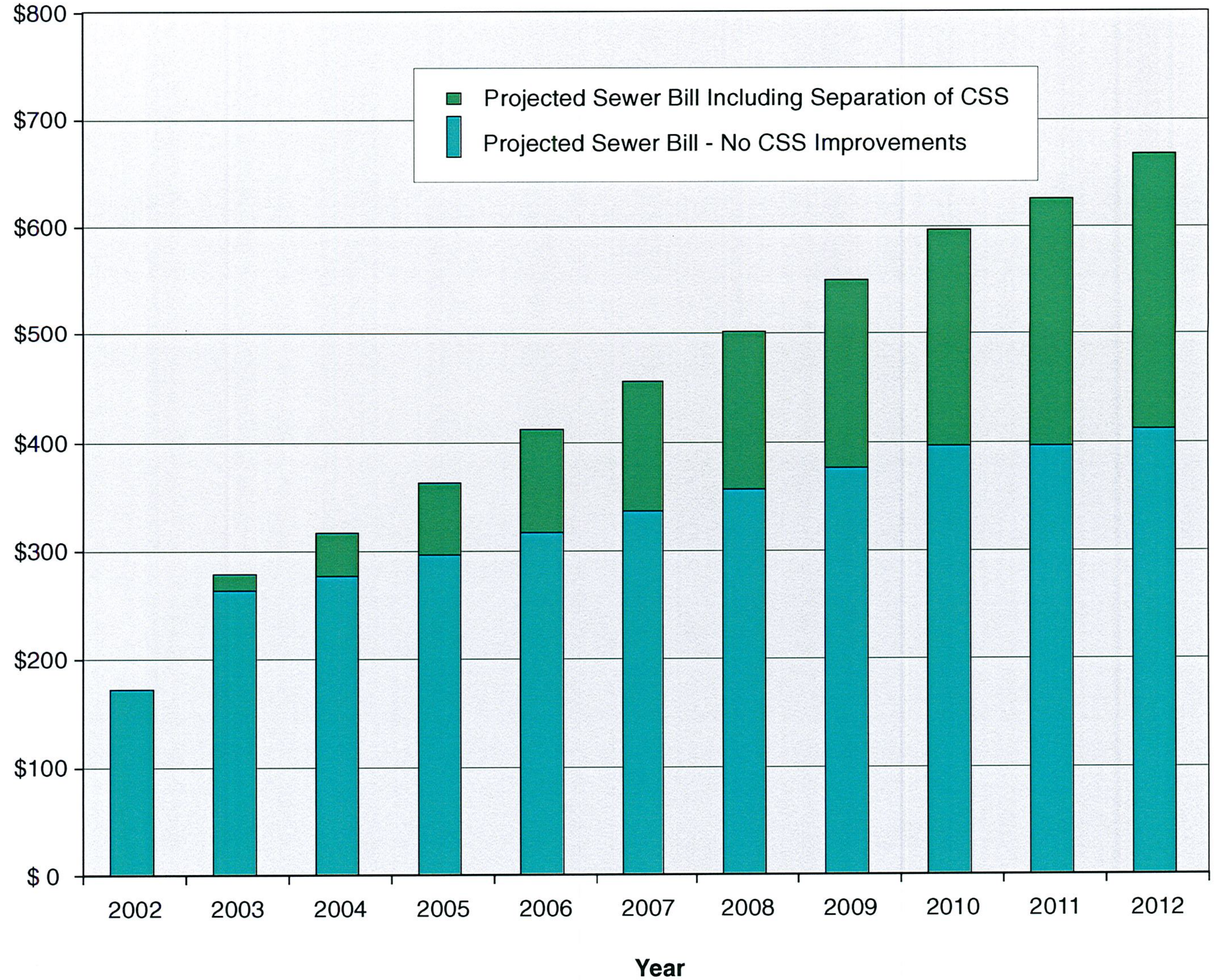

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Figure 10-2
Impact of Sewer Separation on Annual Sewer Bills

CDM

The estimated cost of these improvements is about $2.8M. A similar evaluation was performed as for sewer separation to determine the impact of this program on sewer bills. The capital cost of the program was assumed to take place over a 20-year period starting in FY 2003, and the City will finance the program using general obligation debt carrying a 5.5 percent interest rate.

Based on average annual household consumption of 120 HCF, the sewer household bill will increase from $171 in FY 2002 to approximately $412 in FY 2012, about a $3 increase in annual household bills as compared to the baseline without CSS improvements. Figure 10-3 shows the expected average household sewer bill for FY 2002 through FY 2012 if these alternatives are constructed.

10.7 Summary

The City of Worcester, including the impact of the UBWPAD's program, faces a major capital improvement program to rehabilitate and upgrade its existing sewer system. This program will have a significant impact on the City's ratepayers. The total costs for capital improvement programs will increase at an average of $4 million annually between 2002 and 2012. The annual household bill will more than double from $171 to approximately $409 during this same time frame.

Thus, any improvement to the combined sewer system will be a major financial and political challenge to the City. To tolerate any increase, at a minimum, ratepayers will need to be absolutely convinced the increased revenue is well spent and results in significant environmental improvements. Previous sections of this report demonstrate that modest cost improvements result in quantifiable environmental benefits, but costly improvements such as sewer separation will be difficult to justify.

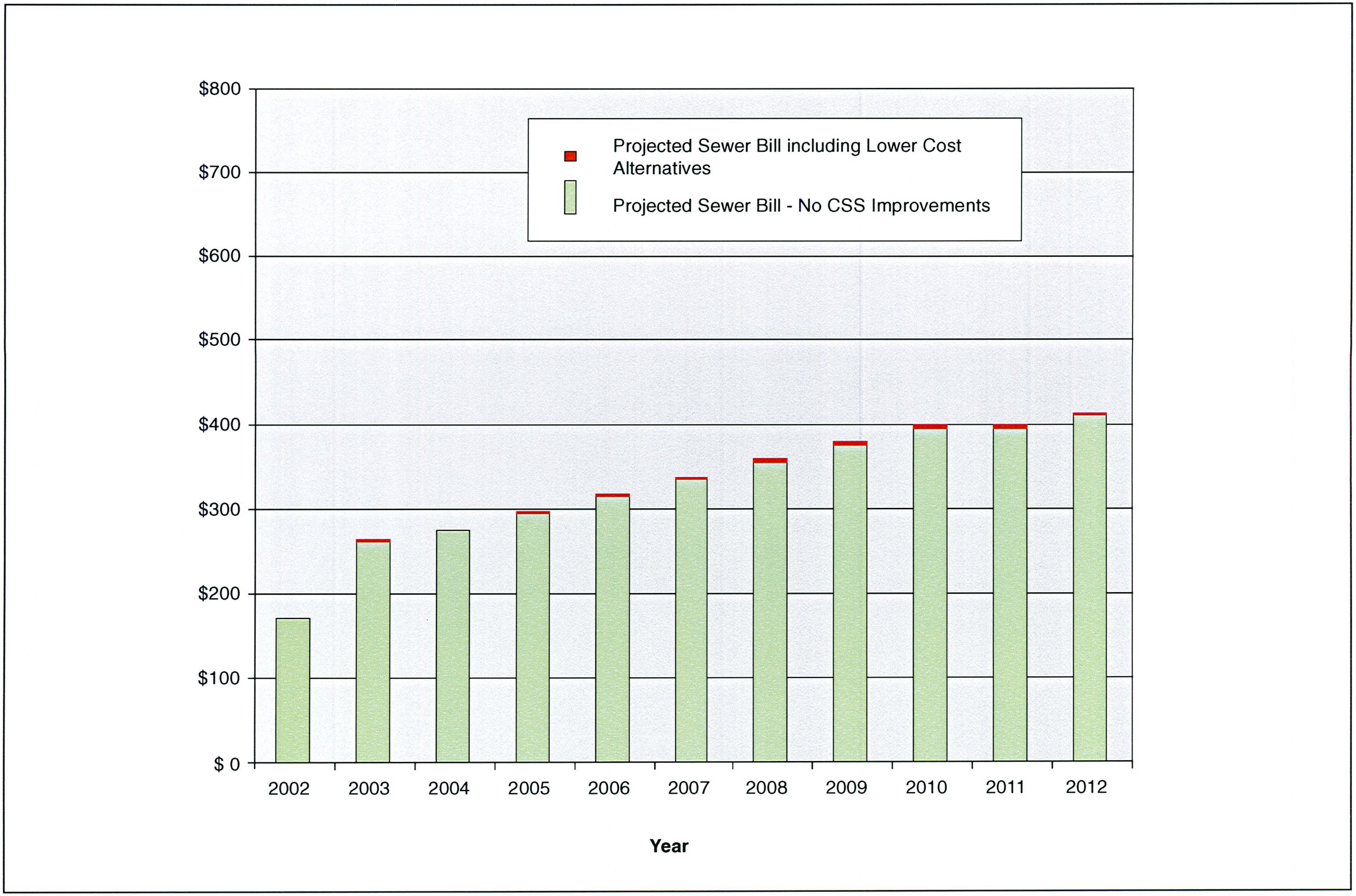

Figure 10-3
Impact of Lower Cost CSO Alternatives on Annual Sewer Bills

Section 11
Summary and Alternatives to Carry Forward to Phase II

11.1 Introduction

The purpose of this section is two-fold. It is intended to summarize important points in the report, and to present the alternatives proposed for evaluation in Phase II of the LTCP.

11.2 Current Status of Worcester's Combined Sewer System (CSS)

This is the second facilities plan prepared for Worcester's four-square mile CSS. The first, in 1975, was fully implemented by 1989, at a cost in 2002 dollars of over $81M. In addition to reducing the CSS area by 0.5 square mile, CSS facilities built as a result of that facilities plan include four large overflow collectors, a dedicated conduit to carry stormwater through the CSS, and the Quinsigamond CSO Storage and Treatment Facility (QCSOSTF). These facilities have been extremely effective at mitigating the impact of CSOs. In a typical 5-year period, there are no dry weather flows, no untreated bypasses, and 100 percent of the flow from the CSS is treated. Ninety-four percent of the CSS flow receives advanced treatment at the Upper Blackstone Wastewater Treatment Facility (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 level of performance from their CSO control facilities.

11.3 The QCSOSTF and Water Quality Standards

The QCSOSTF has permit limits for fecal coliform (400/100 ml maximum daily, 400/100 ml average weekly, and 200/100 ml average monthly) and total residual chlorine (TRC) (.02 mg/l maximum daily) valid during the non-winter period between April 1 and October 15. The facility historically routinely meets its fecal coliform limit, with only five recorded exceedances in the 1995-2000 period. Typically, fecal coliform levels in the discharge are less than 10/100 ml. The facility had difficulty meeting its total residual chlorine limit. Therefore, the City voluntarily constructed dechlorination facilities, which are now in start-up phase. During start-up, violations of fecal coliform and TRC limits are expected. However, once the start-up phase is complete, the City expects to meet both its fecal coliform and TRC limits.

The QCSOSTF discharges to the Blackstone River, a Class B river that does not currently support its designated uses. Compared to other sources such as stormwater and wastewater treatment plant effluent, on an annual and event basis, the QCSOSTF contributes very small flows and pollutant loads to the Blackstone River. Major capital expenditures will only marginally reduce already small contributions, and will do little to help the river meet its designated uses.

The QCSOSTF discharges to the Blackstone River during about one in four rainfall events. During smaller rainfall events, combined sewage reaching the QCSOSTF is pumped to the UBWWTF. The one CSS discharge point, the QCSOSTF, discharges treated effluent to the Blackstone River, at a level of treatment approximating primary treatment during high flow conditions. It also provides treatment under extreme conditions. During a typical 5-year period, no untreated bypasses are expected. The approximate return period of a bypass is once every fifteen years. To keep the Blackstone River's Class B designation, Massachusetts' only regulatory option is to eliminate CSO discharges. This would require separation of the entire CSS, at an estimated cost of $180M. Because the majority of stormwater runoff from Worcester's CSS now typically receives greater than secondary treatment and 100 percent of the runoff is treated, continuing to treat stormwater reduces pollutant loads to the Blackstone River. Therefore, sewer separation would not only have enormous impact on Worcester's ratepayers, it would likely not improve Blackstone River water quality.

Even with the very high level of controls, the City is continuing its efforts to further mitigate the impacts of its CSS. As shown below, UBWWTF improvements, currently under design, will considerably improve CSS performance. Beyond that, the City will move forward with Phase II, where it will evaluate additional alternatives with the goal of virtually eliminating combined sewer system impacts.

The City believes untreated CSO discharges and treated CSO discharges should be distinguished, especially given the existing very high level of control from Worcester's CSS, and the extremely high level of control once UBWWTF and LTCP improvements are in place. The City believes that its CSS's impact on the Blackstone River is marginal, and will be even less so in the future. If untreated CSO discharges are distinguished from treated CSO discharges, the Blackstone River can remain a Class B waterway, as it should.

11.4 UBWWTF Facilities Planning and the Impact on Worcester's CSS

The UBWPAD recently completed facilities planning for improvements to the UBWWTF. The facilities plan has been approved and the improvements are under design, and will be constructed over the next 11 years. The UBWWTF will be upgraded to handle larger future (2020) flows at higher treatment levels. The flow increase is from increased flows from the current service area and geographic expansion into new service areas. The upgrade will include a wet weather flow split, to accommodate higher flows during storm events.

The planned upgrades will improve the performance of the CSS. The most significant improvement will be the operational protocol for pumping from the QCSOSTF to the UBWWTF. Currently, the QCSOSTF can only pump to the UBWWTF when there is excess capacity at the UBWWTF. Typically, when flow exceeds 54 to 70 mgd into the

UBWWTF, pumping from QCSOSTF ceases. With UBWWTF expansion, pumping will continue until flow at UBWWTF reaches 140 mgd.

Compared to existing conditions, these improvements will further mitigate impacts from the CSS. Instead of discharging more than once per month, the QCSOSTF will discharge only about once every two months. One-month storms will receive full treatment at UBWWTF, and discharge volumes from the QCSOSTF during 3-month storms will be cut in half. The total annual volume discharged from the QCSOSTF will be reduced from 83 million gallons to 34 million gallons, more than a two-fold reduction. The portion of combined sewage from the CSS treated at the UBWWTF in a typical year will increase from 94 to 97 percent.

The UBWWTF improvements will have a large rate impact on typical household sewer bills. The average annual household bill is projected to increase from $171 in 2002 to $409 in 2012, a 139 percent increase. These projected rate hikes are for UBWWTF improvements only, and do not include the cost of new facilities for Worcester's combined sewer system.

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

11.5 Additional CSO Control Alternatives and Phase II of the LTCP

As required for this LTCP, a full range of CSO control alternatives were evaluated. With UBWWTF improvements in place, and 97 percent of the combined sewage already treated at the UBWWTF, only 3 percent of the flow can be mitigated further, by treating even more flow at the UBWWTF, or increasing the treatment level of QCSOSTF discharges. Alternatively, sewer separation would entirely eliminate stormwater from the CSS from either the QCSOSTF or the UBWWTF.

Table 11-1 presents each of the alternatives evaluated, its cost, water quality benefit, and whether it will be considered in Phase II for further evaluation. These alternatives all are built on and include the UBWWTF improvements.

At this time, combining the lower cost CSO control improvements appear to have the most promise. For example, Alternative 1 in Table 11-1 combines the Green Hill Pond diversion, modifying regulators by raising weirs ½ foot, rehabilitating the Kelly Square structure, and diverting flow from Kelly Square to the Western Interceptor. On an annual basis, this is expected to reduce the number of QCSOSTF discharges from 7 to 5, annual volume from the QCSOSTF from 34 MG to 27 MG, and increase to 98 percent the amount of combined sewage treated at the UBWWTF.

Table 11-1
Summary of Potential CSO Control Improvements

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
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

Phase II will investigate the promising alternatives identified in Table 11-1 to ensure they function satisfactorily without unintended consequences. Some of the issues that need to be resolved in Phase II include:

- **Green Hill Pond Diversion** - Can the entire Green Hill Pond drainage be diverted to Lake Quinsigamond, without degrading Lake Quinsigamond water quality, and without causing flooding on Coal Mine Brook?

- **Regulator Modifications** - How high can each regulator weir be raised without causing upstream flooding. Provisions to replace weirs with a higher and longer weir have been accounted for in the Phase I cost estimate. By installing a longer weir, it can be raised to a higher elevation without causing upstream flooding.

- **Kelly Square Structure Rehabilitation** - Can reliable real time control operating protocols be instituted at Kelly Square so the gate can be closed in small storms to store flows and opened in large storms to prevent flooding? Will the old Mill Brook Conduit upstream of Kelly Square become pressurized, and if so, will it be structurally adequate to withstand the pressures?

- **Diversion to Western Interceptor at Kelly Square** - Is the Western Interceptor able to accept the increased flows, even during larger storm design events than considered in Phase I?

- **Pumping Alternatives at the QCSOSTF** - Can siting issues related to the force mains needed for expanded pumping options be resolved? Can power requirements for the pumping alternatives be met without new and expanded sources of power (the costs in Phase I assume sufficient power is available)?