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COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

VOLUME 1

CSO FACILITIES PLAN

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COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

VOLUME 1

CSO Facilities Plan

VOLUME 2

Appendices A through F

VOLUME 3

Environmental Information Document

DRAFT

COMBINED SEWER OVERFLOW

FACILITIES PLAN - PHASE 3

SEPTEMBER 30, 1989

For:

City of New Bedford
Massachusetts

By:

Camp Dresser and McKee Inc.
Boston, Massachusetts

EXECUTIVE SUMMARY

Background

This draft report presents the findings of the Phase 3 facilities plan for control of combined sewer overflow (CSOs) in New Bedford. The purpose of the report is to evaluate various alternatives for reducing the volume and frequency of CSOs. The report was prepared to comply with a Consent Decree between the City of New Bedford, the Environmental Protection Agency, the Massachusetts Department of Environmental Protection, and the Conservation Law Foundation of New England.

The report is issued as a draft for comment by the public, Technical Advisory Group, Citizen Advisory Committee, and regulatory groups. A public hearing to receive comment is scheduled on November 15, 1989. Based on comments received from the public and regulatory agencies, a final report will be submitted on or before March 1, 1990.

This Phase 3 CSO Facilities Plan was closely coordinated with the draft Phase 2 Facilities Plan for the Wastewater Treatment Plant (WWTP) submitted in August 1989 so that the most cost-effective solutions for treating both dry-weather and wet-weather flows could be developed. The wastewater collection system improvements recommended in the draft Phase 2 WWTP Facilities Plan have been incorporated into the Phase 3 CSO Facilties Plan.

Summary of Findings

The principal findings of the CSO Facilties Plan are given below.

1. CSOs degrade water quality.

Approximately 1.5 billion gallons of combined sewer overflow is generated every year by the 35 CSOs in New Bedford. Of this total, about 1.1 billion gallons enters the receiving waters as overflows while the remainder is

conveyed to the WWTP. The overflows discharge to the receiving waters as follows:

Receiving Water	Average Annual CSO Volume Spilled	Average Frequency (no./yr)
Clarks Cove	496 MG	45
Outer Harbor	33 MG	49
Inner Harbor	595 MG	50

2. The 10 largest CSOs contribute 86 percent to 90 percent of the total CSO volume for a given rainstorm.

CSO 004 contributes 40 percent and CSOs 020, 022, 023, 025, 030, 031, 036, 040, 041 contribute 42 percent of the total CSO volume.

3. Storm drains contribute significant loadings of pollutants to the receiving waters.

Approximately 1.85 billion gallons of rainfall runoff is discharged to the receiving waters from separate storm drains. The stormwater is discharged to the receiving waters as follows:

Receiving Water	Average Annual Stormwater Discharges	Average Frequency (no./yr)
Clarks Cove	286 MG	100
Outer Harbor	256 MG	100
Inner Harbor	1,308 MG	100

4. The water quality of New Bedford's CSOs is typical of other CSOs.

Results of the field sampling program confirm that the water quality of New Bedford's CSOs is comparable to that found in other cities. The results of the Phase 3 sampling confirm the sampling results from the previous two phases.

5. CSOs contribute significant loadings of coliform bacteria, floatables, and metals to the receiving waters.

CSOs also contribute to aesthetics problems by discharging floatables, oil, and grease. Metals are discharged from CSOs in potentially toxic concentrations within the mixing zones of a CSO.

6. There is a clear priority as to the order in which CSOs should be abated.

Clarks Cove is the receiving water with the highest priority for CSO abatement because of the beaches and shellfish resources in the area. The impaired uses of these resources have been related to CSO discharges. The greatest receiving water benefits can be realized by abating CSO discharges to Clarks Cove.

The Outer Harbor is the receiving water with the second highest priority. It also contains beaches and shellfish resources. These resources, however, are affected not only by New Bedford's CSOs, but also by stormwater from New Bedford and Fairhaven. Unlike Clarks Cove, where the major pollutant sources will be cleaned up once the CSOs are controlled, the Outer Harbor will continue to be affected by stormwater flowing out of the Inner Harbor, and transported across the Outer Harbor from Fairhaven.

The Inner Harbor is the receiving water with the lowest priority. Storm drains discharge over one billion gallons of runoff to the Inner Harbor every year. The Inner Harbor is an EPA Superfund site primarily because of PCB contamination in the sediment; studies are ongoing to assess various cleanup methods. Until the PCB problem is remediated, the Inner Harbor will remain closed to shellfishing. In addition, because the Inner Harbor is a major commercial waterway, it has limited value for swimming.

7. The engineering evaluations were facilitated by dividing the 35 CSOs into 6 groups.

Group 1 consists of CSOs 003 and 004 which discharge into Clarks Cove. Group 2 includes all CSOs on the west side of Clarks Point. Group 3 includes all CSOs on the east side of Clarks Point that discharge into the

Outer Harbor. Groups 4, 5, and 6 include CSOs discharging into the Inner Harbor.

8. In general, projects for the individual CSO groups are recommended over more consolidated and/or system-wide alternatives on the basis of cost and implementation.

Estimated costs of the individual storage/treatment alternatives are either lower in cost or comparable in cost to more consolidated or system-wide alternatives.

It is more cost-effective to maximize upstream storage of CSO flows for treatment at the secondary WWTP than it is to build a separate primary facility at the WWTP dedicated to system-wide CSO flows.

There are sufficient areas of open space in New Bedford where CSO flows could be stored and pumped back into the Rodney French interceptor for treatment at the WWTP. Upstream storage of CSO flows proves to be less costly than building a separate primary treatment facility at Fort Rodman to handle only CSOs. Additional space was allocated in the draft Phase 2 WWTP for building a separate CSO treatment facility. Given that it is not cost-effective to build a separate CSO facility, less land will be needed at Fort Rodman for wastewater facilities. However, smaller satellite treatment facilities are cost-effective in certain areas of the City.

9. Thirty-one sites with open space were identified within New Bedford that could be used for storage-pump back or satellite treatment facilities.

The 31 sites were screened and 13 were selected for detailed evaluations based on location, ownership, land availability, proximity to the Rodney French interceptor, elevation, present uses of the land, zoning, and other considerations. The 13 sites were then evaluated further based generally on engineering and construction-related considerations. Of the 13 sites, 6 were selected as being feasible and cost-effective, one for each of the 6 CSO groups. These 6 sites were used in the recommended plan.

10. The most cost-effective solution for the New Bedford CSO system that meets the study's receiving water objectives and implementation concerns includes separate storage-pump back facilities for CSO Groups 1 through 4, and separate satellite treatment facilities for CSO Groups 5 and 6.

During rainstorms that exceed the conveyance capacity of the wastewater collection system, combined sewer overflows for Groups 1 through 4 would be stored in underground tanks. When the rainstorm ends, the stored overflows would be pumped into the Rodney French interceptor for conveyance to the secondary WWTP, where they would be treated and discharged out the rehabilitated 60-inch outfall. Overflows from Groups 5 and 6 would be conveyed to separate treatment facilities (one for each group) where they would receive the equivalent of primary treatment. Effluent from the satellite treatment facilities would be discharged into the Inner Harbor.

11. The level of CSO control would be up to a 3-month storm, except for CSO Groups 1 and 2 where a higher level of control is warranted.

A higher level of control is provided for Groups 1 and 2 because they discharge into Clarks Cove, the receiving water with the highest priority. There, facilities designed for a control level up to a 1-year storm are recommended. For storms up to a 3-month storm, overflows from Group 1 would be stored at an underground facility located at Francis Field for later pump back to the interceptor for treatment at the WWTP. When storms larger than the 3-month storm occur and the storage facility fills up, excess flow would be diverted by an 84-inch gravity sewer and routed to the Inner Harbor for discharge. The recommended plan will essentially eliminate CSO Group 1 from discharging into Clarks Cove. This is a significant improvement because CSO Group 1 contributed 99 percent of the pollutant loadings to Clarks Cove.

It is not possible to eliminate discharges from CSO Group 2 unless sewer separation were undertaken. For CSO Group 2, storage at Hazelwood Park up to a 1-year storm (3 inches of rain) is recommended. Under the recommended plan, CSO spills into Clarks Cove would be reduced from 45 times per year, to less than once per year (or effectively zero times).

A 3-month storm (1.8 inches of rain) level of control would be provided for CSO Groups 3 through 6.

12. The recommended plan could result in the reopening of closed shellfish beds in Clarks Cove.

All New Bedford CSOs and storm drains discharging to Clarks Cove would be controlled. Overflows from Group 1 would be eliminated by providing storage up to a 3-month storm (and eventual pump back to the interceptor and WWTP), and routing flows exceeding the control level to the Inner Harbor. Overflows from Group 2 would be routed to a storage facility capable of holding volumes for up to a 1-year storm. Two storm drains near Rogers Street in Dartmouth would possibly have to be controlled to eliminate all point source discharges to the Clarks Cove. Implementing the recommended plan could allow the shellfish beds to be reopened. The Conservation Law Foundation estimates that there are 107,242 bushels of quahogs in Clarks Cove with an estimated market value of $5 million per year. Thus, the recommended plan could result in a net economic gain of $5 million to the local economy every year.

13. Water quality improvements to the Outer and Inner Harbors with CSO controls would not be as significant as in Clarks Cove because these receiving waters are also affected by other pollution sources.

In the Outer Harbor, CSOs now discharge about 49 times per year, potentially impacting East Beach as frequently. Under the recommended plan, overflows would occur on average only 4 times per year, a 92 percent reduction. However, pollution from storm drains discharging to the Inner Harbor is estimated to reach the East Beach 15 times per year. Stormwater pollution includes animal feces, trash, litter, oil, grease, and other constituents associated with urban land use that wash off the streets every time it rains. While controlling the overflows from CSO Group 3 would certainly improve water quality in the Outer Harbor, there is no guarantee that shellfish beds that are currently closed in the Outer Harbor would be reopened by implementing the recommended plan. Storm drains in New Bedford and Fairhaven would still discharge over 100 times each year, and would

likely cause violations of the coliform bacteria standard in at least portions of the Outer Harbor.

In the Inner Harbor, CSOs now spill over 50 times per year. Stormwater is as significant a source of pollution as CSOs. With the recommended plan, CSOs would spill untreated effluent only 4 times per year on average. Storm drains would continue to discharge to the receiving waters every time it rains. The receiving water benfits to be realized with CSO controls for Groups 4, 5 and 6 are marginal. Further, the PCB problem in the sediments of the Inner Harbor coupled with the fact that no swimming occurs because the Inner Harbor is a major commercial waterway suggests that its current classification as Class SB needs to be reconsidered, and possibly revised. Reclassifying the receiving water as Class SC is an alternative that should be considered before implementing CSO controls for Groups 4, 5, and 6.

14. <u>The capital cost of the recommended plan would be $182.5 million</u>.

Table 1 shows the capital and O&M costs of the recommended plan by CSO Group. These cost estimates include engineering and contingencies. There is a specific order that the 6 projects should be undertaken. Projects for Groups 1 and 2 should be undertaken first because they provide the greatest water quality benefits. CSO Group 3 should be undertaken in Stage 2 because the Outer Harbor is the receiving water with the next highest priority. Finally, Stage 3 projects should include those associated with CSO Groups 4, 5, and 6.

15. <u>Implementing the recommended plan will pose a significant financial burden to the City</u>.

The cost of wastewater treatment, which includes undertaking CSO abatement projects and construction of the secondary WWTP, is estimated to escalate from $5.8 million in 1990 to between $22 million and $52 million (1989 dollars) by the year 2005. The final cost to users will depend on how much state and federal assistance the City can obtain. If the City can obtain up to 75 percent assistance, then the average household sewer bill in the year 2000 will be 3.5 times higher than the average bill in 1990. If the

TABLE 1

COST SUMMARY OF RECOMMENDED PLAN

CSO Group	Capital Costs ($ million)	Annual O&M Costs
Group 1	$63.7	$212,800
Group 2	$15.3	$43,700
Group 3	$15.3	$38,200
Group 4	$31.8	$111,200
Group 5	$20.4	$83,500
Group 6	$36.0	$148,900
TOTAL COST	$182.5	

NOTES: (1) Capital Costs do not include land acquisition, legal and administrative costs.

(2) All costs are in 1989 dollars (ENR Construction Costs Index 4600).

City cannot obtain any federal assistance, then the average household bill in the year 2000 will be eight times higher than the 1990 average. The user rates are based on constructing CSO projects for Groups 1 and 2 in 1998, and Group 3 in 2000. CSO projects for Groups 4, 5, and 6 have the lowest priority and construction would be completed by 2015.

16. The recommended plan also includes other structural improvements and best management practices (BMPs) that the City should undertake and/or continue.

System-wide structural improvements include renovations to existing pump stations, force mains, and sewers to correct equipment or capacity deficiencies. The grit problem in the Rodney French interceptor will have to be resolved. Finally, modifications must be made to 5 existing CSO regulators. These improvements, which are identified and described in the draft Phase 2 WWTP Facilities Plan, will improve system capacity and decrease CSO discharges.

There are several BMPs that the City undertakes that improve water quality by reducing CSO discharges, including catch basin cleaning, regulator inspection, and enforcement of the Industrial Pretreatment Program. These practices should continue at their present levels of effort.

17. Sewer separation was carried through the detailed evaluations but not included in the recommended plan because it would not provide the same water quality benefits as the storage-pump back and satellite treatment facilities.

The cost to separate the entire combined system of New Bedford would be about $138.9 million broken down as follows:

Group 1	$42.4 million
Group 2	$6.7 million
Group 3	$5.3 million
Group 4	$27.7 million
Group 5	$20.6 million
Group 6	$36.2 million
Total	$138.9 million

While eliminating raw sewage flows would improve water quality, water quality problems would persist because the separated storm drains would still be discharging to the receiving waters every time it rains. Stormwater has been identified as a significant source of pollution because of the animal feces, trash, litter, oil, and grease that washes off the streets and ends up along the shoreline.

The recommended plan provides the City with an opportunity to control two of the most significant sources of pollution to its receiving waters at the same time. EPA recognizes that stormwater is a significant source of pollutant loadings nationwide, and is developing guidelines and procedures to mitigate stormwater impacts. Stormwater controls will likely be mandated at the federal level in the foreseeable future. Implementing the recommended plan will provide the City with an opportunity to be a leader in pollution control because it will be one of the first communities in the Commonwealth to recognize the water quality impacts of stormwater, and address them.

18. Alternatives to the recommended plan have been developed if the WWTP is at the Standard Times site.

If the recommended secondary WWTP site is changed from Fort Rodman to the alternate site, Standard Times, then the recommended CSO abatement plan changes slightly. The same storage sites would be used to hold combined sewer overflow. The costs of the force main to route the the flows to the new site would be negated by the cost savings associated with eliminating the added overflow/siphon pipeline. The costs of the recommended plan whether the secondary WWTP is located at Fort Rodman or Standard Times would be essentially the same.

ACKNOWLEDGMENTS

A study of this size and complexity requires contributions from many people to be successful. We hope we have not omitted anyone who helped on this study. If we did, we regret the omission.

City of New Bedford

Special thanks to Ms. Marcy Weatherbee who provided valuable guidance, direction and assistance throughout the project.

Camp Dresser & McKee Inc.

Numerous engineers and scientists from Camp Dresser & McKee Inc. put forth critical management, design, and analytic efforts. Mr. Stephen Hickox served as project director. Mr. David Noonan was acting project manager. He was ably assisted by Mr. Gary Mercer. Preliminary designs and the components of the recommended plan were developed by Mr. Steve Beyer under the direction of Mr. Vic Coletti. Dr. Thomas Hruby developed and applied the empirical wind analysis used in the receiving water evaluations. Ms. Annette Huber and Mr. Rob Kapner encoded, calibrated, and ran the SWMM and STORM models under the capable direction of Mr. David Schafer. Ms. Liz Beardsley prepared the EID guided by Mr. Phil Kennedy. Mr. David Giles and Mr. Chuck Adelsberger prepared the siting analysis. Mr. Art Zeman and Mr. John McCormack were important members of the design team. Mr. Mark Carroll and Mr. David Fredette were primarily responsible for the field monitoring program. They, along with Coral Damkroger, Fred Schneader, Art Zeman, Chuck Adelsberger, Ling Chu, and Joe Tota spent long, cold winter nights collecting important field data. Significant contributions were made by Mr. Carl Johnson, Mr. Bob Beaumont, Ms. Bernadette Kolb, Dr. Guillermo Vicens, and Mr. Howard Yamaguchi. Cost estimates for the recommended plan were prepared by Mr. George Guay. The financial analyses were conducted by Mr. John Gall and Mr. Joseph Ridge. The graphics and design drawings -- critical components in a facilities plan -- were prepared by Mr. Joe Pitta and Mr. Russ Briggs. Mr. Robert Flynn also contributed maps and graphs. The text of this report was overseen by Mr. Roland Robinson, assisted by Ms. Cathy Hooper and Ms. Joann Repoza. Receiving water dye studies were conducted by Ocean Surveys Inc. of Old Saybrook, Connecticut.

REGULATORY REVIEW

Timely contributions and oversight were made by many members of the regulatory committee. We especially appreciate the assistance, guidance and cooperation of:

Mr. Ron Lyberger	Mass DEP
Mr. Alan Slater	Mass DEP
Mr. Rich Tomczyk	Mass DEP
Mr. Steve Couto	EPA
Ms. Susan Coin	EPA
Mr. Jan Smith	CZM
Mr. Larry Gil	MDWPC
Mr. Steve Halterman	MDWPC
Dr. Russ Isaac	MDWPC

CITIZENS ADVISORY COMMITTEE

We gratefully acknowledge the time given and interest shown by the Citizens Advisory Committee. We appreciate their helpful comments.

Mr. Brad Bourque, Chairman	Shellfish Warden
Mr. Jeff Osuch	City of New Bedford
Mr. Anthony Cabral	Portugese Committee
Mr. Seth Garfield	Coalition for Buzzards Bay
Mr. Martin Manley	Harbor Development Commission
Mr. Mark Montigny	Chamber of Commerce
Ms. Rosalie Otley	North End Resident
Ms. Marilyn Pontes	South End Resident
Mr. Ralph Saulñier	City Councilor
Mr. Raymond Belanger	City Health Department

The public participation program was conducted by Ms. Regina Villa and Ms. Jeanne Murphy of Regina Villa Associates.

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LIST OF ABBREVIATIONS

CSO	-	Combined Sewer Overflow
NBC	-	Narragansett Bay Commission
O&M	-	Operations and Maintenance
EID	-	Environmental Information Document
EPA	-	Environmental Protection Agency
RIDEM	-	Rhode Island Department of Environmental Management
SWMM	-	Stormwater Management Model
STORM	-	Storage Treatment Overflow Runoff Model
BOD	-	Biochemical Oxygen Demand
BMPs	-	Best Management Practices
HCF	-	Hundred Cubic Feet
MERL	-	Marine Ecosystem Research Laboratory
RIPDES	-	Rhode Island Pollutant Discharge Elimination System
NPDES	-	National Pollutant Discharge Elimination System
CWA	-	Clean Water Act
BPWTT	-	Best Practicable Waste Treatment Technology
WWTF	-	Wastewater Treatment Facility
DWF	-	Dry Weather Flow
MGD	-	Million Gallons Per Day
MH	-	Manhole
F-Coli	-	Fecal Coliform
TSS	-	Total Suspended Solids
VSS	-	Volatile Suspended Solids
TKN	-	Total kjeldahl Nitrogen
T-coli	-	Total Coliform Bacteria
S.M.	-	Standard Methods 16th Edition, 1985
mg/l	-	Milligrams Per Liter
cols./100 mls	-	Coliforms per 100 Milliliters
Do	-	Dissolved Oxygen
Km	-	Kilometers
In.	-	Inch
ASA	-	Applied Science Associates
Inv. Elev.	-	Invert Elevation
in/hr	-	Inches Per Hour
hrs	-	Hours
lbs	-	Pounds
cfs	-	Cubic Feet per Second
S.D.	-	Standard Deviation
USGS	-	United States Geological Survey
cm/s	-	Centimeters Per Second
OF	-	Overflow
MPN/SEC	-	Most Probable Number Per Second
SS	-	Stormsewer
Vol.	-	Volume
FPS	-	Feet Per Second
M	-	Meters
ASCE	-	American Society of Civil Engineers
WRE	-	Water Resources Engineers
STP	-	Sewage Treatment Plant

LIST OF ABBREVIATIONS, (continued)

I/I	-	Inflow/Infiltration
ENR	-	Engineering News Record
Sett-S	-	Settleable Solids
ppt	-	Parts Per Thousand

1.0 INTRODUCTION

1.1 BACKGROUND

The City of New Bedford owns and operates the wastewater collection and treatment facilities in the City of New Bedford. The city's wastewater collection system currently serves about 60 percent of the city's area, and 96 percent of the population. The system consists of about 128 miles of older combined sewers in the south and central parts of the city, plus about 76 miles of separate sewers in the northern end. Wastewater is conveyed by an 11-mile main interceptor to the Fort Rodman treatment plant, where it receives primary treatment.

The focus of this report is on the 38 combined sewer outlets which activate during rainfall events and discharge a mixture of sewage and storm runoff into the three major receiving water bodies that surround the city. The location of the 38 CSOs and the 3 receiving waters are shown on Figure 1-1.

CSO planning in New Bedford has been undertaken in three phases. The Phase 1 report, entitled "Interim Summary Report on Combined Sewer Overflows-Phase 1" was completed by Camp Dresser and McKee Inc. in December 1983. The Phase 1 report recommended that dry weather discharges to the receiving waters should be eliminated before work on CSOs is addressed. Since that report was issued, the City has intensified their efforts to abate dry weather discharges in its system by repairing tide gates, cleaning and fixing regulator structures, and installing new sanitary flow pipes to the interceptor system. The Phase 1 report also identified CSOs as contributors to the degradation of water quality in the receiving waters. To mitigate the impacts of CSOs, the City has repaired all raw sewage lift pumps at the wastewater treatment plant, begun efforts to remove sediment deposits in the main interceptor, and raised weir heights at selected regulators to capture more of the CSO flow before it spills.

GLOSSARY

abatement (mitigation): reduction of adverse environmental effects such as water quality degradation.

combined sewage: wastewater from residences, commerical buildings, industrial plants, and institutions together with any groundwater and stormwater that may be present.

combined sewer: a sewer designed to cary both wastewater and stormwater runoff.

combined sewer overflow (CSO): 1) the portion of flow from a combined sewer system which discharges into a water body, usually during a rainfall event. 2) the pipe which carries this discharge.

computer modeling: a method of predicting the consequences of a set of given conditions using a mathematical representation of real physical processes by use of a cmputer.

dry weather flows: flows in sewer systems under dry weather conditions including wastewater plus some groundwater that leaks into sewer pipes.

interceptor sewer: a sewer without direct connections to buildings, which is used to collect and carry flows from one or more smaller sewers to a central point for treatment and discharge.

manhole: an opening in a sewer provided for the purpose of permitting a person to enter or leave it.

non-structural solution: an abatement strategy whuich relies on enhanced operation and maintenance practices.

receiving waters: rivers, lakes, oceans or other water bodies that receive treated or untreated wastewater.

regulator: a device in combined sewer systems which diverts wet weather flows to an overflow when the capacity of the sewer is exceeded.

runoff: that portion of rainfall, melted snow or other precipitation which flows across the ground surface to a drain, sewer, stream or other body of surface water.

sanitary sewer: a sewer designed to carry domestic sewage and industrial wastes.

sewer: any pipe used to collect and carry away wastewater and/or stormwater runoff to a treatment plant or a receiving water body.

sewer appurtenances: secondary structures connected directly to sewers such as manholes, tidegates and regulators.

storm sewer: a sewer that carries intercepted surface runoff, street wash and other waters, or drainage, but excludes domestic sewage and industrial wastes.

stormwater: excess water running off surfaces during and immediately after periods of precipitation.

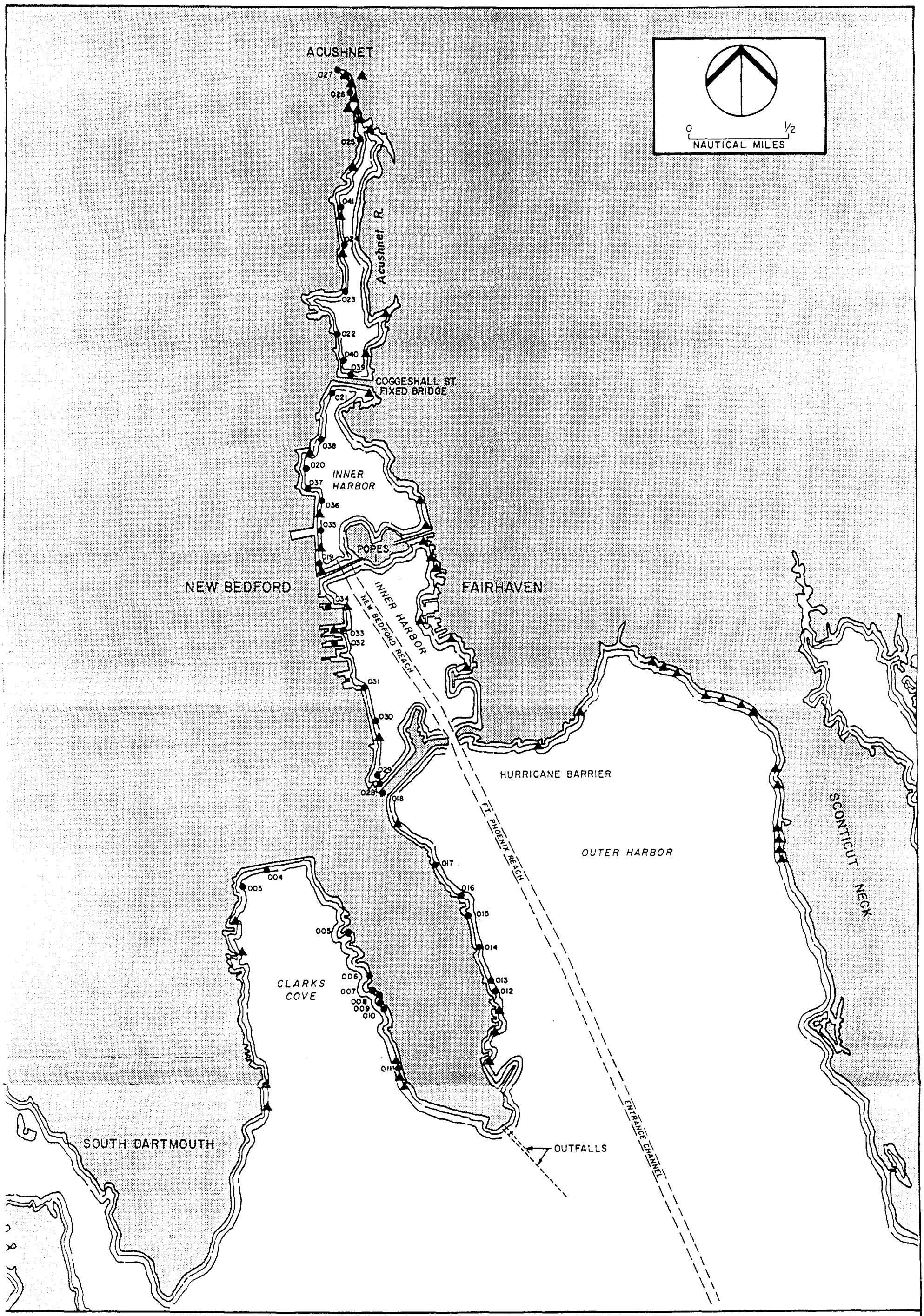
street sewer (lateral or connector sewer): the first element of a wastewater collection system used to collect wastewater from one or more buildings.

structurally intensive solution: an abatement strategy which relies on construction or repair of a wastewater system.

tidegate: a swinging gate in a combined sewer overflow which prevents seawater from entering the sewer system during high tides.

wastewater: the spent water of a community which may be a combination of the liquid and water-carried wastes from residences, commercial buildings, industrial plants and institutions, together with any groundwater, surface water and stormwater that may be present.

wet weather flows: dry weather flows plus stormwater introduced into a combined sewer.



City of New Bedford

COMBINED SEWER OVERFLOW • FACILITIES PLAN - PHASE 3

Study Area

CDM

FIGURE 1- 1

The purpose of the 1987 Phase 2 study was to supplement information developed in Phase 1 by extending the boundaries of the stormwater model (SWMM), field checking regulator dimentions, and evaluating the hydraulic response of several different designs of the main interceptor with the stormwater model. The SWMM model was extended to include the entire New Bedford wastewater collection system. The design alternatives that were evaluated for the main interceptor included complete removal of grit in the interceptor, connecting flows in the Belleville area downstream of Wamsulta Street, and connecting flows from the Sawyer Avenue area to the North End relief sewer, among others. The results of the Phase 2 study were summarized in a letter report (dated November 15, 1988) to the Massachusetts Department of Environmental Protection (then the Department of Environmental Quality Engineering).

This report summarizes the findings of the Phase 3 study for controlling combined sewer overflows in New Bedford.

1.2 PURPOSE

The Phase 3 reports builds on the foundation provided in the Phase 1 report and the Phase 2 work. The Phase 1 report identified non-structural measures that could be undertaken to reduce CSOs and improve water quality. Phase 2 refined the estimates of combined sewer overflow spilling into the receiving waters. Phase 3 involves a more detailed evaluation of a wider range of measures--both structural and non-structural--that can be formulated into a long-term recommended plan to control CSO discharges. The recommended plan includes various engineering projects to be undertaken to abate CSOs, the expected receiving water benefits, a schedule for, and the costs of implementing the plan.

The Phase 3 CSO Facilities Plan is being prepared to comply with a Consent Decree between the City of New Bedford, the Environmental Protection Agency, the Massachusetts Department of Environmental Protection and the Conservation Law Foundation. The Consent Decree calls for the submittal of a draft facilities plan on or before October 1, 1989. Following a 5-month

review period, a final facilities plan is to be submitted on or before March 1, 1990.

The Phase 3 report covers the following topics:

Section 2:	Existing Wastewater Facilities
Section 3:	Dry Weather Flows and Loads
Section 4:	Wet Weather Flows and Loads
Section 5:	Existing Receiving Water Quality
Section 6:	Receiving Water Issues and Modeling Results
Section 7:	Assessment of CSO Control Technologies
Section 8:	Siting Issues
Section 9:	Evaluation of Selected Alternatives
Section 10:	Recommended Plan
Section 11:	Financial Capability

A separate Environmental Impact Document (EID) is attached as part 2.

1.3 COORDINATION WITH WASTEWATER TREATMENT PLANT FACILITIES PLAN

The Consent Decree also requires that the City implement new wastewater facilities (including collection, treatment, and disposal). To that end, CDM prepared and submitted the Phase 2 Facilities Plan for the Wastewater Treatment Plant (WWTP) in August 1989. The recommended plan includes the installation of a secondary treatment plant at Fort Rodman that will be able to treat peak flows of 75 mgd and sewer system modifications to convey the sewage to the proposed facility. One of the CSO abatement alternatives is to store the mixture of rainwater and sewage flows in excess of the plants capacity and treat it at the WWTP when capacity becomes available. Therefore, it was necessary to closely coordinate the preparation of both the WWTP and CSO Facilities Plans so that the most cost-effective solution for treating both dry-weather and wet-weather flows could be developed. Biweekly meetings were held between the project teams to coordinate the studies and ensure that consistent plans were developed.

2.0 EXISTING WASTEWATER FACILITIES

2.1 GENERAL

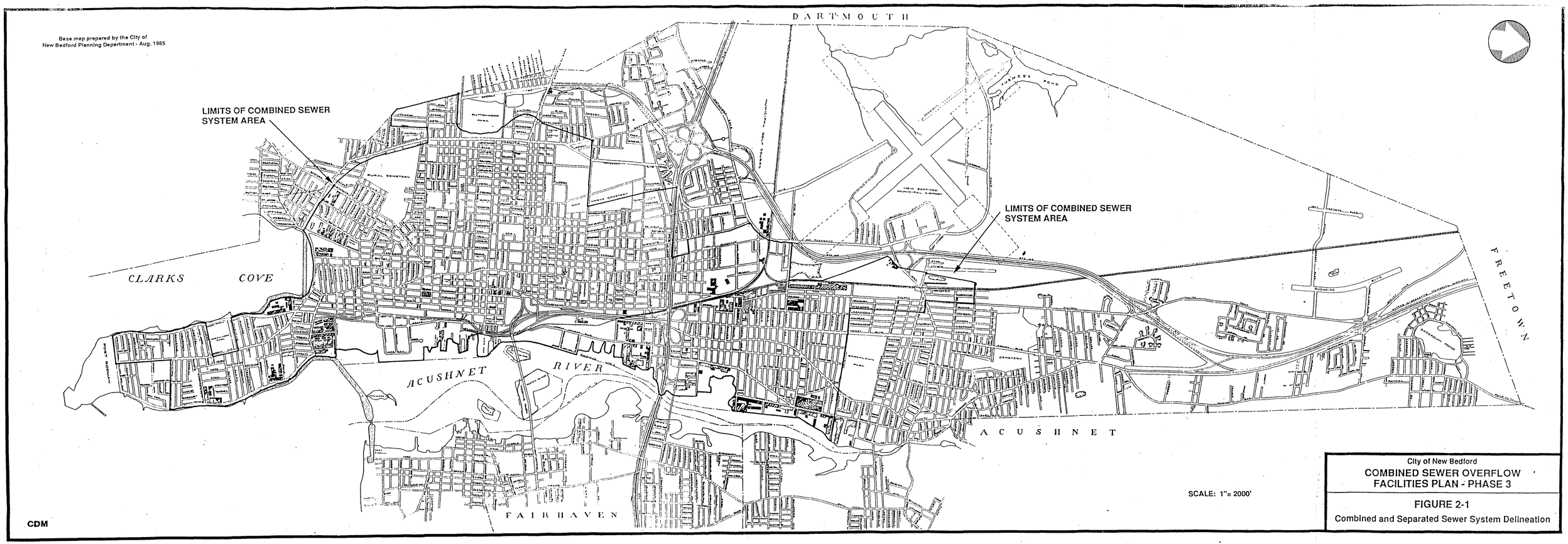
This section presents an overview of New Bedford's existing wastewater collection and treatment systems, particularly the combined sewer system. Approximately 128 out of 204 total miles of sewers (63 percent) are combined sewers. The remaining 76 miles (37 percent) are separate sewers, built within the last 50 years primarily on the north side of the city as that area developed. This section also discusses the interceptor and submain system outfalls, population and land use, and the storm drainage system.

2.2 SERVICE AREA

New Bedford's municipal wastewater collection system currently serves about 60 percent (7,742 acres) of the city's area and 96 percent of the population. In addition, the system accepts flow from about 600 dwellings in the town of Dartmouth and 60 dwellings in the Town of Acushnet. The older, combined sewer system services the central and southern sections of the city; while the newer, separated system services the north end. Figure 2-1 delineates the combined and separated sewer systems of the City of New Bedford.

Within the service area, wastewater is conveyed by the eleven-mile Rodney French interceptor (discussed in Section 2.5) to the existing Fort Rodman primary treatment plant. Major components of the existing wastewater collection system include:

- 3 major interceptors
- 7 major collector sewers
- 22 wastewater pumping stations
- 7 miles of force mains



City of New Bedford

COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 2-1

Combined and Separated Sewer System Delineation

- 67 regulators, which regulate flow to interceptors or to overflow outlets
- 38 combined sewer outlets

The major components of the existing municipal wastewater collection system are shown on Figure 2-2.

2.3 WASTEWATER TREATMENT FACILITIES

2.3.1 OVERVIEW

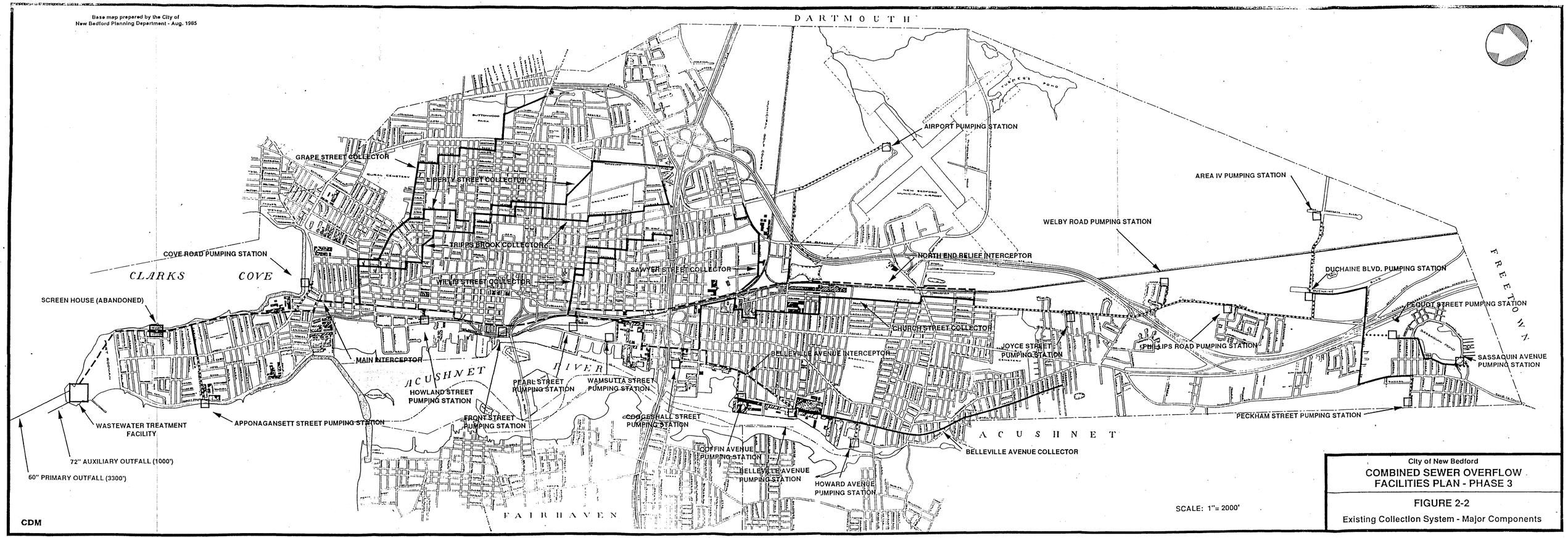
The City owns and operates a wastewater treatment plant located at Fort Rodman on the southerly end of Clarks Point. Wastewater receives primary treatment at the facility consisting of grit removal, sedimentation, and chlorination prior to disposal into Buzzards Bay. Effluent is discharged through two outfall: a 3,300-foot long 60-inch diameter pipe through which most flow is discharged and a 1,000-foot long 72-inch diameter pipe. There is no diffuser on either outfall.

The City has operated the plant since its construction in 1972. In accordance with the terms of the Consent Decree, the City is presently evaluating proposals from private contractors who would operate and maintain the plant starting in 1990.

The plant was designed to treat an average wastewater flow of 30 mgd and the headworks, screening channels, and influent pumps have a peak hydraulic capacity of about 100 mgd. The sedimentation tanks can treat up to approximately 90 to 100 mgd based on acceptable surface loading rates (3000 gpd/sf) and detention times. Currently, the average daily dry weather flow is 29.2 mgd, 26.4 mgd is treated at the existing primary WWTP and 2.8 mgd is discharged to the receiving waters by CSOs as dry weather overflows.

All wastewater is conveyed to the treatment plant by the Rodney French interceptor which has a full capacity of about 130 mgd at its downstream end, from the screenhouse to the WWTP. Upstream of this point the interceptor is limited to a capacity of 70 to 75 mgd. At present, the

Base map prepared by the City of
New Bedford Planning Department - Aug. 1985



CDM

City of New Bedford
COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 2-2

Existing Collection System - Major Components

capacity of the interceptor is further reduced by two conditions: the grit accumulations in its northerly segment between Holly Street and Willis Street and the system hydraulics at the connection from the Tripps Brook and Grape Street collectors.

Because of these constraints, the maximum flow which can be conveyed to the treatment plant without causing CSO discharges is about 60 mgd. Remedial plans currently being developed by the city will alleviate the grit-related capacity constraints and proposed modifications to the sewer system under the wastewater facilities plan will increase the hydraulic capacity of the interceptor system to about 75 mgd, which will provide adequate capacity for the projected design dry weather flow. The planned modifications are presented in Section 7.0.

2.3.2 RECENT IMPROVEMENTS

Improvements to the existing plant completed in early 1988 include:

- improvements to sludge and scum piping
- new vent piping in raw sewage pumps
- improvements to chemical feed control systems
- ventilation system improvements
- acoustical improvements in sludge and scum pumping station

2.4 INTERCEPTOR AND SUBMAIN SYSTEM

2.4.1 INTERCEPTORS

The city's collection system includes three major existing interceptors; the Rodney French interceptor, hereafter called the Main interceptor, the North End Relief interceptor and the Belleville Avenue interceptor.

Main Interceptor

The Main interceptor collects wastewater from the entire service area and transports it to the existing primary treatment plant at Fort Rodman. The interceptor is 11-miles long, and ranges in size from a 54-inch segmented block sewer at Holly Street to an 84-inch by 92-inch horseshoe-shaped reinforced cast-in-place concrete conduit terminating at the Fort Rodman treatment plant. At its downstream end, the interceptor has a full hydraulic capacity of approximately 130 mgd, however, maximum flows are limited by the hydraulic capacity in upstream reaches. At present, the interceptor can convey approximately 60 mgd to the Fort Rodman WWTP without causing CSO discharges. By surcharging the conveyance system, flows greater than 60 mgd can reach the plant, however, this will result in CSO discharges.

Belleville Avenue Interceptor

The Belleville Avenue interceptor was constructed in the late 1970s to replace the upstream end of the Main interceptor. This interceptor, approximately 2500 feet in length, originates at the terminus of the 30-inch Belleville Avenue force main near Davis Street and terminates at the Main interceptor on Holly Street. The Belleville Avenue interceptor is constructed of 42-inch PVC pipe.

North End Relief Interceptor

The North End Relief interceptor, approximately 16,000 feet in length, originates at the intersection of Worcester Street and Lynn Street and terminates at the main interceptor near Route 18 and Pearl Street. The interceptor is constructed of reinforced concrete (R.C.) pipe ranging in size from 14 to 42-inches.

2.4.2 SUBMAIN SYSTEM

General

The New Bedford wastewater collection system has many major sewer lines generally referred to as "collector sewers." The largest collector sewers, (see Figure 2-2) are the Grape Street, Liberty Street, Tripps Brook, Willis Street, Sawyer Street, Church Street, and Belleville Avenue collectors. With the exception of the Church Street collector, all are combined sewers, and as such, were designed to accommodate stormwater runoff. The Church Street collector is basically a sanitary sewer that is interconnected with storm drains; however, no wet weather overflow structures are associated with this particular collector.

Grape Street Collector

The Grape Street collector, approximately 14,200 feet in length, originates at the intersection of Kemator and Oesting Streets and terminates at the Main interceptor on Second Street. The collector ranges in size from 36-inch to 72-inch brick conduit and 42-inch by 45-inch to 50-inch by 48-inch R.C. horseshoe-shaped conduit.

Liberty Street Collector

The Liberty Street collector, approximately 12,500 feet in length, originates on Apache Court, near the Route 195/140 interchange and terminates at the Grape Street collector on Grape Street. The collector ranges in size from 20-inch to 48-inch conduit and 42-inch by 48-inch to 48-inch by 54-inch R.C. horseshoe-shaped conduit. This collector is interconnected with the Tripps Brook collector via overflow pipes in three locations; a 30-inch connection at Newton and Elm Streets, an 18-inch connection at Park and Union Streets, and a connection at Chancery and Arnold Streets.

Tripps Brook Collector

The Tripps Brook collector, approximately 15,300 feet in length, originates on Potter Street just west of Turner Street and terminates at a connection to the Grape Street collector on River Street. The collector ranges in size from 24-inch R.C. pipe to 54-inch brick conduit. Interconnections exist between the Tripps Brook at Liberty Street collectors at the locations mentioned previously.

Willis Street Collector

The Willis Street collector, approximately 8,500 feet in length, originates at the corner of Arnold and Bullock Streets and terminates at the main interceptor at Route 18 across from Willis Street. The collector ranges in size from 15-inch R.C. pipe to 42-inch by 42-inch horseshoe-shaped R.C. conduit.

Sawyer Street Collector

The Sawyer Street collector, approximately 7,500 feet in length, originates at the intersection of Malden and Walker Streets and terminates at the Main interceptor at the intersection of Front and Sawyer Streets. The collector ranges in size from 24-inch V.C. (vitrified clay) to 60-inch brick conduit. A portion of the sewer on Purchase Street is constructed of 42-inch by 45-inch and 42-inch by 42-in R.C. horseshoe-shaped conduit.

Church Street Collector

The Church Street collector, approximately 11,000 feet in length, originates at the intersection of Church and Lynn Streets and terminates at a connection to the Sawyer Street collector near Van Buren Street and the railroad tracks. The collector consists of 20-inch and 24-inch pipe.

Belleville Avenue Collector

The Belleville Avenue collector, approximately 11,500 feet in length, originates at the intersection of Ashley Boulevard and Acushnet Avenue and

terminates at the Belleville Avenue pump station. The collector ranges in size from 12-inch to 36-inch pipe.

2.5 COMBINED SEWER SYSTEM

2.5.1 GENERAL

The function of a combined sewer system is to collect sanitary flows and stormwater runoff and convey them to the interceptor system. Flow to the interceptor system is controlled by pipe capacity and regulator structures. Regulators allow flows exceeding pipe capacity to be relieved and discharged. Combined flow or dry weather flow in excess of the pipe capacity is discharged to receiving waters by the regulators through CSO pipes. In locations where the elevation of the receiving water could exceed the regulator elevation, the CSOs are equipped with tide gates to prevent backflow of receiving water into the system.

2.5.2 REGULATORS

The city's wastewater collection system incorporates 67 regulators to control the flow of dry and wet weather wastewater entering the interceptor system. Combined flows in excess of the interceptor's capacity flow through overflow pipes (CSOs) to receiving waters. There are 38 such outlets in New Bedford's system. Figure 2-3 shows the locations of these outlets in the study area. Table 2-1 provides a summary of the regulatory structures, including type, pipe size, receiving water, and elevation of the overflow weir.

There are three types of regulators in the New Bedford system: fixed weirs, high outlets, and floats with variable orifices.

Normally, these overflow regulators are installed in manholes. The fixed weir and high outlet types are set at hydraulically determined elevations above the dry weather flow pipes. The fixed weir type is usually constructed of brick or concrete. Their operation is designed to be automatic, based on the depth of flow in the collection system during storm

Base map prepared by the City of
New Bedford Planning Department - Aug. 1985

SCALE : 1"= 2000'

City of New Bedford
COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 2-3
Combined Sewer Overflows

CDM

TABLE 2-1

REGULATOR SUMMARY

Regulator Number and Location	Regulator Type	Incoming Sewer Pipe Size	Dry Weather Connection	Overflow Size	Receiving Water and Outfall	Critical Elevations on New Bedford Datum
3A Padananam & Cove	High level outlet	22"	24"	24"	Clarks Cove 003	1.58
04A Rockdale & Cove	Float	10", 10"	18"	18"	Clarks Cove 004	0.28
041A Rockdale, 20" west of Cove	High level outlet	18"	18"	24"	Clarks Cove 004	2.9
04B Orchard & Cove	Float	48"	18"	48"	Clarks Cove 004	-1.34
04C Crapo & Cove	Float	18", 15"	10"	18"	Clarks Cove 004	-2.38
04D Orchard & Rivet	Side weir wall	24"	24"	24"	Clarks Cove 004	3.1
04E Bonney & Rivet	Weir	72"	36"x12"	72"	Clarks Cove 004	4.14
04F Bonney & Rivet	Weir	24"	24"	24"	Clarks Cove 004	3.1
04G Crapo & Rivet	Side weir wall	54"	27"	54"	Clarks Cove 004	4.41
042A County & Cove	Float	Twin 24"	18"	Twin 24"	Clarks Cove 0041	1
0421 David & WRF	Weir	10", 10"	10"	12"	Clarks Cove 0042	4.6
05A Dudley & WRF	Weir	10"	15"	18"	Clarks Cove 005	3.6
06A Lucas & WRF	Weir	10"	8"	15"	Clarks Cove 005	2.58
06B Oaklawn & WRF	Weir wall	10"	8"	15"	Clarks Cove 006	?
06C Capitol & WRF	Weir	10"	18"	15"	Clarks Cove 006	3.11
06D Lucas on WRF	Weir	15", 10"	10"	15"	Clarks Cove 006	3.89
07A Capitol & WRF	Weir	18"	18"	18"	Clarks Cove 007	6.66
08A Calumet & WRF	Weir	10"	8"	12"	Clarks Cove 008	5.58
09A Aquidneck & WRF	High level outlet	10"	12"	18"	Clarks Cove 009	6.52
010A Bellevue & WRF	Side weir wall	10"	10"	12"	Clarks Cove 010	4.92
0101A Hudson & WRF	High level outlet	12"	10"	18"	Clarks Cove 0101	3.03
Freedom & ERF	Weir wall	10"	10"	10"	Clarks Cove 011	1.86 (abandoned)
012A Ricketson & ERF	Float	18"	12"	18"	Outer Harbor 012	-0.24
012B Bellevue & ERF	Float	15"	12"	18"	Outer Harbor 012	-0.24
013A Aquidneck & ERF	Float	15"	10"	15"	Outer Harbor 013	1.32
014A Apponagansett & ERF	Float	24", 12"	12"	24"	Outer Harbor 014	0.04
015A Butler & ERF	Float	15", 15"	12"	24"	Outer Harbor 015	-1.41
016A Frederick & ERF	High level outlet	18"	18"	15"	Outer Harbor 016	1.31
017A Rodney & ERF	Float	15", 15"	10"	15"	Outer Harbor 017	-1.11
017B Mott & ERF	High level outlet	15"	15"	12"	Outer Harbor 017	0.79 (abandoned)
017C David & ERF	Float	24"	12"	24"	Outer Harbor 017	-3.02
017D Ruth & ERF	Float	15"	12"	15"	Outer Harbor 017	-1.83
018A Cove & ERF	Float	18"	12"	18"	Inner Harbor 018	-1.17
018B Cove & ERF	High level outlet	12"	12"	12"	Inner Harbor 018	-0.22

TABLE 2-1 (continued)

Regulator Number and Location	Regulator Type	Incoming Sewer Pipe Size	Dry Weather Connection	Overflow Size	Receiving Water and Outfall	Critical Elevations on New Bedford Datum
030A Potomska & Second	Weir	30"	24"	30"	Inner Harbor 0182	1.93
030B South & Second	Weir	12"	12"	15"	Inner Harbor 0182	2.73
032A Russell & Second	Side weir wall	24"	24"	36"	Inner Harbor 0183	5.01
032B Madison & Second	Side weir wall	24"	24"	24"	Inner Harbor 0183	7.75
032C WAlnut & Acushnet	Side weir wall	24"	10"	30"	Inner Harbor 0183	18.45
033A School & Acushnet	Side weir wall	24"	12"	24"	Inner Harbor 0184	23.52
033B Spring & Acushnet	Side weir wall	24"	30"	12"	Inner Harbor 0184	23.89
034 Union & Acushnet	High level outlet	30"	10"	30"	Inner Harbor 0185	29.34
035A Hillman & Foster	Side weir wall	24"	12"	24"	Acushnet Estuary 0186	63.52
035B Pleasant & Maxfield	Side weir wall	24"	12"	24"	Acushnet Estuary 0186	52.26
036A Willis & Purchase	Weir	24"	24"	48"	Acushnet Estuary 0187	25.22
036B Pearl & Purchase	Side weir wall	10"	14"	15"	Acushnet Estuary 0187	23.42
037 Pearl & Route 18	High level outlet	12"	12"	10"	Acushnet Estuary 0188	9.74
020A Wamsutta & Route 18	Double side weir wall	Twin 24"	36"	30"	Acushnet Estuary 020	4.94
020B Logan & Acushnet	Double side weir	36"	15"	30", 18"	Acushnet Estuary 020	6.82
021A Washburn & N. Front	Weir wall	15"	8"	15"	Coggeshall Pump Station	9.42
021B Kenyon & N. Front	Weir wall	10"	18"	10"	Acushnet Estuary 021	9.02
022A Sawyer & N. Front	Double side weir wall	48"	18"	48"	Acushnet Estuary 022	9.68
022B Holley west of Belleville	Side weir wall	12"	6"	12"	Acushnet Estuary 022	9.25
022C Tallman west of Belleville	Side weir wall	10"	8"	10"	Acushnet Estuary 022	9.67
023A Coffin & Riverside	Weir wall	24", 24"	20", 15"	42"	Acushnet Estuary 023	0.32
024A Hathaway & Riverside	Weir wall	48"	12"	48"	Acushnet Estuary 024	-0.23
025A Howard & River	High level outlet	24"	to P.S. wetwell	18"	Acushnet Estuary 025	-0.35
027A Mill	Side weir wall	12"	12"	15"	Acushnet Estuary 027	2.64
027B Ohio & Acushnet	Side weir wall	22"	18"	20"	Acushnet River 027	17.11
027C Belleville & Mill	Side weir wall	24"	24"	24"	Acushnet River 027	16.94
028A Belleville & Belleville South	Side weir wall	12"	12"	12"	Acushnet Estuary 028	3.55
029B Belleville & Belleville North	High level outlet	36"	P.S. wetwell	36"	Acushnet Estuary 028	-0.95
029A Howard & Coggeshall	Weir	18"	18"	18"	Inner Harbor 029	5.9 (abandoned)
030A P.S. & Coggeshall	Weir	15", 10"	P.S.	24"	Acushnet Estuary 030	3.76
031A Grinnell & Second	Side weir wall	10"	24"	24"	Inner Harbor 031	1.76
031B Allen & Second	Side weir wall	36"	18"	36"	Inner Harbor 031	3.26
031C Howland St. P.S.	High level outlet	36"	36"	18"	Inner Harbor 031	2
031D Bonney & Grinnell	High level outlet	54"	24"	72"x72"	Inner Harbor 031	22.85
032A Water & Blackner	High level outlet	12", 12"	15"	24"	Inner Harbor 032	2.11 (abandoned)

events. During periods of wet weather, the depth of flow in the conduit increases as the flow rate increases. Eventually the depth of flow will increase to the point where the water surface will be higher than the fixed weir or high outlet pipe invert resulting in an overflow event. Conversely, when the flow rate subsides, overflows cease.

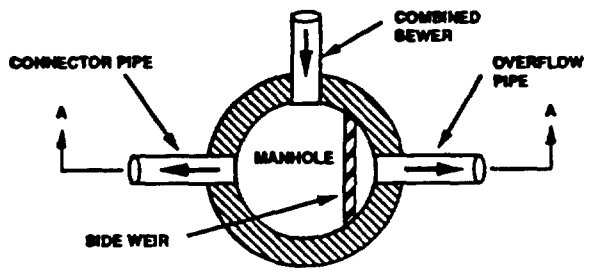
In 13 locations (see Table 2-1), the overflow regulators also include float activated mechanisms. The mechanisms consist of a float which is attached to a lever arm which in turn controls a flap gate attached to the dry weather flow pipe. These regulators function the same way as the fixed weir and high outlet types in that depth of flow governs their operation. As the depth of flow increases during wet weather, it causes the float to raise the lever arm closing the flap valve. As the flap valve shuts, it throttles the flow entering the collection system's dry weather pipe and causes it to overflow the fixed weir into the overflow pipe. Field surveys indicate that all of the float-type regulators are either inoperable or the float mechanism itself has been removed. Typical plans and sections of regulator types found in the study area are shown on Figure 2-4 through 2-6.

2.5.3 TIDE GATES

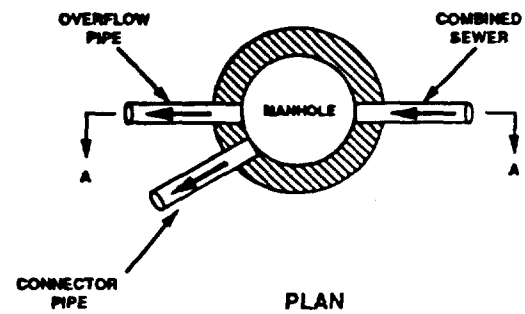
Tide gates are employed at some regulators in the City's sewer system to prevent backflow into the system from receiving waters. Generally, the tide gates are rectangular or circular flap valves installed in the structure's overflow chamber. During periods of rising tide, water backs up the outlet pipe and against the valve face causing the valve to shut. Conversely, when the depth of flow in the overflow chmaber of the regulator creates a hydrostatic pressure exceeding that of the tide, the flap valve opens allowing combined sewage to discharge to the receiving waters (see Figure 2-6).

2.6 STORM DRAINAGE SYSTEM

As mentioned previously in this section, New Bedford's older sections have combined sanitary and stormwater sewers in many areas. Stormwater enters



PLAN



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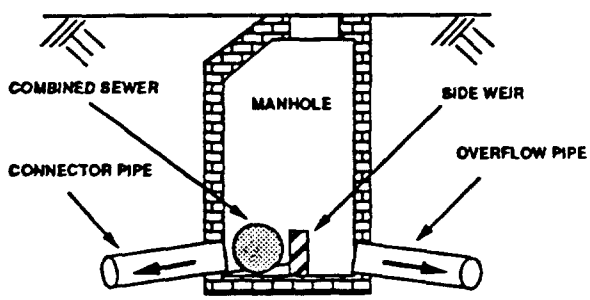
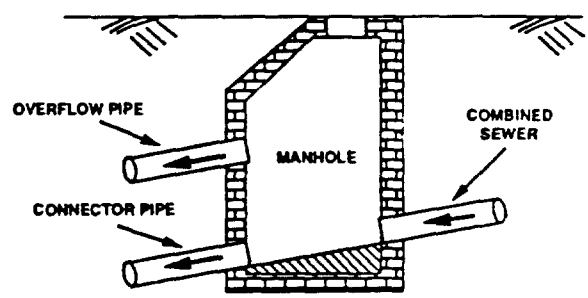


SECTION A-A

SIDE WEIR WALL



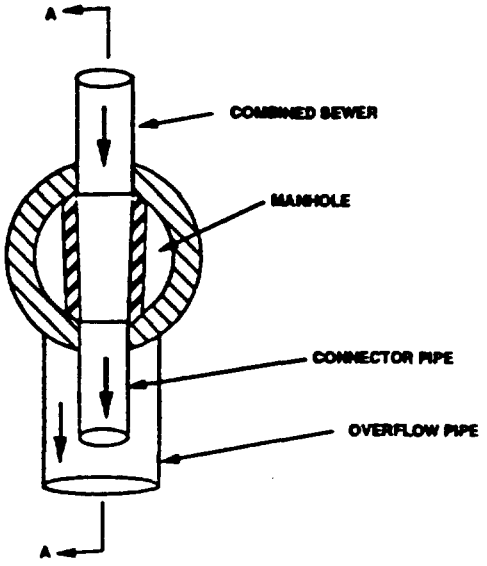
SECTION A-A

HIGH OUTLET

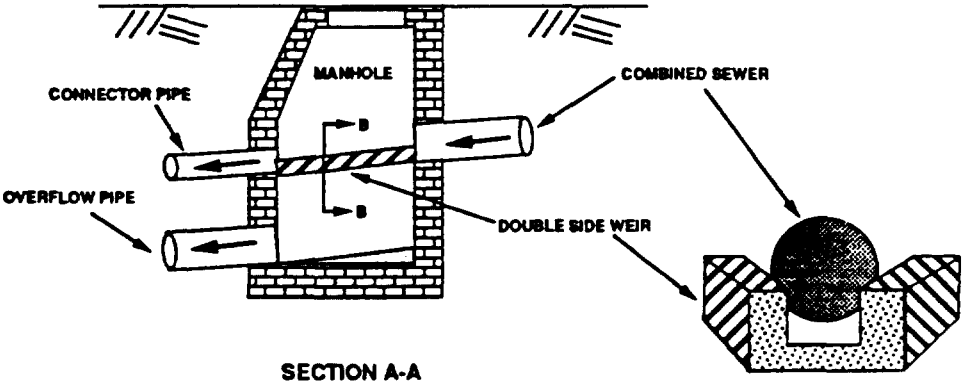
SCALE: none

CDM

City of New Bedford COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3	FIGURE 2-4 SIDE WEIR WALL AND HIGH OUTLET TYPE REGULATORS



PLAN



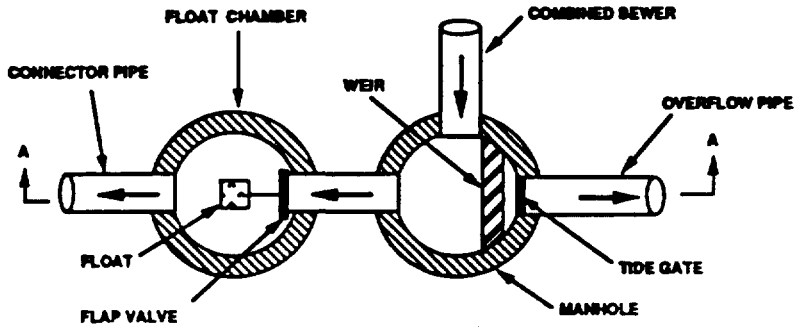
SECTION A-A

SECTION B-B

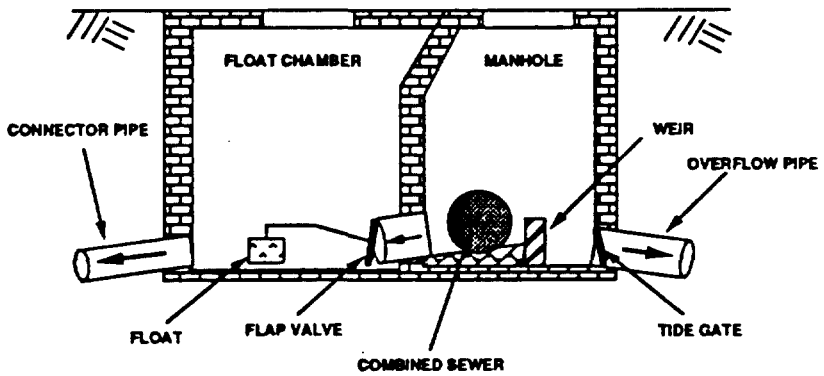
SCALE: none

CDM

City of New Bedford COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3	FIGURE 2-5 DOUBLE SIDE WEIR WALL TYPE REGULATOR



PLAN



SECTION A-A

SCALE: none

CDM

City of New Bedford COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3	FIGURE 2-6 FLOAT WITH VARIABLE ORIFICE TYPE REGULATOR

the sanitary system by means of direct catch basin connections, gutter drains, roof drains, basement drainage, and foundation drains. When it has been cost effective, the City has separated combined sewers. In addition, over the last 50 years the City has constructed only separate sanitary and stormwater conduits. The north end of the City is completely separated since this area has been developed within the last 50 years.

There are 55 separated storm drain outfalls tributary to the receiving waters. The Phase I reported that the storm drain outfalls from the Hathaway Street/Coffin Avenue area and Potomska Street/Front Street area are contaminated by sanitary sewage. Since that time, corrective action has been taken. The addition of a 15-inch sewer in the Hathaway Street/Coffin Avenue area has eliminated storm drain contamination by sanitary sewage. Also, the ongoing construction of "Dry Weather Discharge Abatement Facilities" will ultimately relieve the storm drains in the Potomska Street/Front Street area of sanitary sewage contamination by December, 1989.

3.0 DRY WEATHER FLOWS & LOADS

3.1 GENERAL

An accurate estimate of the area tributary to the combined sewer system and its population and land use is needed to estimate the flows entering the sewer system under both dry and wet weather conditions. This section addresses the determination of tributary area, population, land use, and the development of dry weather flow estimate. An extensive analysis of dry weather flow for the City of New Bedford was presented in the draft Phase 2 WWTP Facilities Plan (CDM, 1989); a summary of this analysis for average and peak dry weather flow for present and future conditions is presented below.

The second step of the process, estimating wet weather flows and comparing these flows with field-measured data, is presented in Section 4.0. Through this comparison, models of the combined sewer system network are calibrated and are then used to evaluate methods for CSO abatement.

3.2 POPULATION AND LAND USE

The tributary area to the New Bedford sewer system is well documented, and includes most of the City of New Bedford and small portions of the towns of Acushnet and Dartmouth. Figure 4-3 (in Section 4.0) shows the total area and the division of the sewer service area into 15 subbasins.

Estimates of current and future service population area are important not only in estimating the portion of flows attributable to domestic sources, but also the type of pollutants associated with the flow. Population patterns also can be used to predict future activity and associated wastewater flows.

The current service population for the New Bedford wastewater collection network is estimated at approximately 95,700, based on the 1985 Decennial Census by the Commonwealth of Massachusetts.

Several methodologies for estimating the future population served by the New Bedford sewer system were considered. The methodology adopted for use in the system design is summarized below; a more thorough description of the selection process and the methods reviewed is presented in draft Phase 2 WWTP Facilities Plan (CDM, 1989).

Future population projections were based on two parameters: the average yearly increase in the number of housing units over the past decade (0.5 percent), and the average number of persons per housing unit over the 25 year period 1960 to 1985 (a decrease from 2.5 to 2.4). Assuming that the number of housing units would continue to increase at the same average rate (0.5 percent) and that the average number of persons per housing unit would remain at 2.4 through 1995 and decrease to 2.36 by the design year 2014, the resulting population in 2014 would be 111,355. The added population is assumed to occur in developing areas. Therefore by 2014, 99 percent of the population will be served by the sewer system, rather than the current 96 percent.

Land use within the New Bedford sewer system service area, including those parts of Ascushnet and Dartmouth serviced by New Bedford, consists primarily of high-density residential and commercial/industrial uses, which together represent 75 percent of the total sewered area. Land use estimates for each tributary subbasin were originally presented in the 1983 Interim CSO Report. Since then, Geographic Information Systems (GIS), or digitized databases of geographic data, including land use, have been developed. The most current data on land use is available on these systems, through surveys conducted by the University of Massachusetts in 1985, and was adopted for this study. The land use associated with the subbasins shown in Figure 2-1 is summarized in Table 3-1, along with the current population estimates for the 3 community service area.

TABLE 3-1

POPULATION AND LAND USE BY TRIBUTARY AREA

Subbasin Number	Area[1] (acres)	Population[2] (est. 1985)	Land Use (acres)[1]					
			Commercial	Industrial	Low-Density Residential	Medium Density Residential	High Density Residential	Open Space/ Forest
1	614	11000	67	169	4	53	313	8
2	244	3100	0	35	13	11	138	47
3	1945	23400	101	349	61	205	1005	224
4	244	1300	0	68	0	52	80	44
5	138	1300	10	51	0	0	67	10
6	275	5000	90	30	0	0	154	1
7	285	3100	32	198	0	0	18	37
8	177	5000	7	34	0	0	128	8
9	213	500	22	182	0	0	0	9
10	200	2300	42	120	0	0	36	2
11	604	8600	22	202	14	82	246	38
12	118	3400	38	33	0	0	43	4
13	1263	16600	162	218	32	133	616	102
14	1234	5400	20	246	14	218	249	487
15	198	5700	1	15	0	0	182	0
Total	7752	95700	614	1950	138	754	3275	1021

(1) Land Use Analysis performed by the Remote Sensing Project, the Department of Forestry and Wildlife Management, University of Massachusetts, Amherst, 1988.

(2) From the draft Phase 2 WWTP Facilities Plan, (CDM, 1989). Based on 1985 Decennial Census by the Commonwealth of Massachusetts.

3.3 DRY WEATHER FLOWS

A detailed analysis of current and future dry weather flows is presented in the Draft Phase 2 WWTP Facilities Plan. Key aspects of that analysis are summarized below.

3.3.1 DOMESTIC FLOWS

Domestic wastewater flow is currently 64 gallons per capita per day (gpcd), based on metered water consumption data from 1983 to 1987. Assuming 90 percent of this flow is returned to the sewer system, the total residential contribution is 5.8 mgd. This rate of water use per capita is assumed to remain constant through the design year, 2014, because of current water conservation efforts and plumbing code revisions. The increase in domestic flow in the design year will result only from increased population, and is estimated to be 1.0 mgd, for a total domestic flow of 6.8 mgd.

3.3.2 INDUSTRIAL FLOWS

Water use data and industrial survey questionnaires show a current industrial flow of 5.8 mgd; the average rate of use is 6,000 gallons per acre per day (gpad). In the future, charges for water use will sharply increase, making it unlikely that new development will include industries with high water use. Therefore, it is assumed that by the year 2014, all land zoned for industrial use will be developed with industries using water at a rate of 2000 gpad. The total flow in 2014 is projected to be 8.6 mgd.

3.3.3 COMMERCIAL/INSTITUTIONAL FLOWS

The present flow from commercial and institutional users is approximately 2.3 mgd. No change is expected in this water use category based on historical records. However, future expansion of the sewer system would allow a small increase in commercial/institutional acreage. Therefore, the total projected commercial/institutional flow in the year 2014 was estimated to be 2.5 mgd.

3.3.4 INFILTRATION/INFLOW (I/I)

Infiltration occurs in older systems through leaks and cracks in sewer mains and drainage connections, and inflow through leaking tide gate structures. CDM conducted an infiltration/inflow (I/I) study in 1981 and a sewer system evaluation survey in 1983. Based on these studies, the current level of I/I is estimated to be 15.3 mgd, of which 14.0 mgd is due to infiltration and 1.3 mgd is due to tidal inflow. Through proposed rehabilitation of the system, infiltration in the design year 2014 would be reduced to 11.4 mgd, and tidal inflow to 0.7 mgd.

3.3.5 DRY WEATHER OVERFLOWS

The quantity of dry weather discharge is currently estimated to be 2.8 mgd, based on a 1987 CDM report on the elimination of dry weather discharges. All dry weather overflows are expected to be eliminated by 1994.

3.3.6 FLOW SUMMARY

Table 3-2 provides a summary of average dry weather flows for the current system and for the design year 2014, by tributary subbasin. There is no significant increase in average dry weather flow, primarily because increases in domestic, industrial, and commercial flow are offset by decreases in infiltration and inflow. Although a single value of dry weather flow is provided, in reality the flow fluctuates diurnally and seasonally. The diurnal variation (shown as a ratio of the high point to the low point) can easily be a factor of two. The seasonal variation is lower, with a ratio of high to low of not more than 1.5. Groundwater levels are the primary cause of the seasonal fluctuations because they affect the quantity of infiltration. For the purposes of this study, the average design year dry weather flow under low groundwater conditions is 27.2 mgd, and under high groundwater conditions, 38.5 mgd. This variation in flow is small compared to wet weather volumes, and is not likely to impact CSO frequency and volume. This will be explicitly evaluated in the following section.

TABLE 3-2

AVERAGE DRY WEATHER FLOWS BY TRIBUTARY AREA
FOR PRESENT AND FUTURE CONDITIONS

Subbasin Number	Present - 1987 Flows (mgd)[1] Domestic	Industrial	Commercial/ Institutional	Infiltration/Inflow	Total	Future - 2014 Flows (mgd)[1] Total
1	0.6	2.3	0.6	1.2	4.7	5.1
2	0.2	0	0.1	0.7	1.0	1.0
3	1.5	0	0.4	5.4	7.3	7.1
4	0.1	0.5	0	0.7	1.3	1.4
5	0.1	0.1	0	0	0.2	0.2
6	0.3	0.1	0.5	0.9	1.8	1.8
7	0.2	0.5	0	0.2	0.9	1.1
8	0.2	0	0	0.7	0.9	0.9
9	0	0.2	0	0.1	0.3	0.3
10	0.1	0	0.1	0.4	0.6	0.6
11	0.5	0.2	0.3	0.5	1.5	1.6
12	0.2	0.2	0	0.9	1.3	1.3
13	1.1	0.8	0.1	2.3	4.3	4.4
14	0.3	0.9	0.2	0.6	2.0	2.2
15	0.4	0	0	0.7	1.1	1.0
Total	5.8	5.8	2.3	15.3	29.2[2]	30.0

(1) From the draft Phase 2 WWTP Facilities Plan (CDM, 1989).

(2) This represents the total dry weather flow which includes the flow reaching the treatment plant (26.4 mgd) and the dry weather overflows (2.8 mgd).

No historical data exist for estimating peak flows, therefore they were determined using standard peaking factors. The selection of peaking factors is also described in the draft Phase 2 WWTP Facilities Plan; a summary of adopted peaking factors is given below. Based on the Merrimack curves generated by historical data from similar systems, a peaking factor of 2.7 was used for current domestic, commercial and institutional flows, and a factor of 2.6 was used for the future flows from these sources. A factor of 3.0 was applied to industrial flows based on the number of shifts per day, and the rate of batch discharges. The peaking factor for infiltration was determined to be 1.7 from a regression analysis on groundwater infiltration rates for New Bedford and six similar systems. Tidal inflow is assumed to be uniform, therefore a peaking factor of 1.0 was applied. The resulting 1989 peak flow (64.4 mgd) and 2014 peak flow (70.6 mgd) are shown in Table 3-3 by tributary subbasin.

3.4 DRY WEATHER LOADS

3.4.1 GENERAL

Dry weather loads can be divided into two major categories: conventional and non-conventional pollutants. Biochemical oxygen demand (BOD) and total suspended solids (TSS) are defined as conventional pollutants. Non-conventional pollutants are constituents which are potentially toxic when found at excessive concentrations.

3.4.2 CONVENTIONAL POLLUTANTS

BOD and TSS concentrations in CSOs provide one measure of potential impacts to receiving waters. High levels of BOD can cause reductions in levels of oxygen in the water, putting stress on fish and other aquatic animals and plants. TSS represents the amount of organic and inorganic solids in the water; high TSS concentrations lead to increased turbidity and accumulation of silt in the benthic layer.

The quantity of BOD and TSS in dry weather flow can be broken down according to source, such as domestic, commercial, institutional,

TABLE 3-3

PEAK DRY WEATHER FLOWS BY TRIBUTARY AREA
FOR PRESENT AND FUTURE CONDITIONS

Subbasin Number	Present - 1987 Flows (mgd)[1] Domestic	Industrial	Commercial/ Institutional	Infiltration/Inflow	Total	Future - 2014 Flows (mgd)[1]
1	1.6	6.9	1.6	2.0	12.1	13.6
2	0.5	0	0.3	1.2	2.0	2.2
3	4.0	0	1.1	8.7	13.8	14.7
4	0.3	1.5	0	1.2	3.0	3.3
5	0.3	0.3	0	0	0.6	0.7
6	0.8	0.3	1.4	1.4	3.9	4.2
7	0.5	1.5	0	0.5	2.5	2.8
8	0.5	0	0	1.2	1.7	1.9
9	0	0.6	0	0.2	0.8	0.9
10	0.3	0	0.3	0.6	1.2	1.3
11	1.4	0.6	0.8	0.8	3.6	4.0
12	0.5	0.6	0	1.4	2.5	2.6
13	3.0	2.4	0.3	3.8	9.5	10.4
14	0.8	2.7	0.5	0.9	4.9	5.5
15	1.1	0	0	1.2	2.3	2.5
Total	15.6	17.4	6.3	25.1	64.4	70.6

(1) From the draft Phase 2 WWTP Facilities Plan (CDM, 1989).

industrial, and septage. For this study, the estimated loading from domestic, commercial, and institutional sources was based on typical loading factors. The industrial component was derived from a comparison of existing TSS and BOD data from 1983 to 1987 and loadings from other sources. Another source of BOD and TSS is septage, or the pumped contents from tanks and cesspools. The loading factors of pollutants from each of these sources is presented in Table 3-4. These are average values; the factors will vary over time, but for the purposes of the CSO study the average dry weather factors are used. When the loading factors listed in Table 3-4 are multiplied by their corresponding flow or population, an overall average mass loading can be calculated for present and future conditions. The adopted current concentrations for BOD and TSS are 137 mg/l and 146 mg/l. The concentrations of BOD and TSS in the year 2014 are assumed to be 189 mg/l and 225 mg/l.

3.4.3 NON-CONVENTIONAL POLLUTANTS

Non-conventional pollutants are analyzed in this study because of their potential hazard to the ecology and economy of the receiving waters. Pollutants under this classification include heavy metals, volatile organics, semi-volatile organics (also known as acid, base, neutral compounds or ABNs), pesticides and polychlorinated biphenyls (PCBs). These pollutants are examined are on EPA's Priority Pollutant List (PPL) and Hazardous Substance List (HSL).

A wastewater sampling program was conducted in the spring and summer of 1987 at the headworks of the existing wastewater treatment plant to evaluate the presence of non-conventional pollutants. The detailed results from this program are contained in Appendix B of Volume III of the draft Phase 2 WWTP Facilities Plan. The basic approach used to determine pollutant levels is summarized below.

The dry weather average influent loadings of a particular pollutant found in the influent were calculated based on the concentration measured in each sample and the flow reading that was recorded on that dry weather day of sampling. In a number of cases, the concentrations were reported below the

TABLE 3-4

ADOPTED BOD AND TSS CONTRIBUTIONS BY COMPONENT

Component	BOD		TSS	
	Existing Average	Design Year (2014)	Existing Average	Design Year (2014)
Residential (lb/capita/day)	0.17	0.19	0.18	0.22
Commercial (mg/l)	200	200	200	225
Institutional (mg/l)	200	200	200	225
Industrial (mg/l)	260	275	260	320
Septage (mg/l)	5,000	5,000	15,000	15,000

(1) From the draft Phase 2 WWTP Plan (CDM, 1989).

detection limit of the test method. In these instances, the following procedure was followed:

- If a pollutant was detected above the sample's detection limit at least once in the samplings, then half the detection limit was used as the concentration for those samples that reported results below detection limits.
- If pollutant was never detected above the sample's detection limit, then no loadings were presented.

When estimating the future wasteloads of non-conventional pollutants, the assumption has been made that the concentrations of non-conventional pollutants in future years would remain the same as the present day. Thus, the loadings of non-conventional pollutants would increase in direct proportion to the projected increases of wastewater generated from sources other than infiltration/inflow. This approach assumes that future wastewater characteristics will be similar to existing conditions.

Existing and future concentrations adopted for this CSO study for heavy metals, ABNs, PCBs, and VOCs are presented in Tables 3-5 through 3-8.

TABLE 3-5

EXISTING AND FUTURE AVERAGE METAL CONCENTRATIONS

Constituent	Existing[1] Concentrations (mg/l)	Future (2014)[2] Concentrations (mg/l)
Antimony	0.005	0.006
Arsenic	0.002	0.003
Beryllium	0.008	0.010
Boron	0.255	0.320
Cadmium	0.002	0.002
Chromium	0.087	0.109
Copper	0.132	0.165
Cyanide	0.008	0.010
Lead	0.029	0.036
Mercury	0.001	0.001
Molybdenum	0.019	0.023
Nickel	0.069	0.087
Selenium	0.004	0.005
Silver	0.008	0.010
Thallium	0.004	0.005
Zinc	0.195	0.244

(1) Based on influent samples taken from the headworks of the New Bedford wastewater treatment plant.

(2) Increase based on proportional increases of domestic, commercial, institutional and industrial flows relative to total flow.

(3) Derived from the draft Phase 2 WWTP Facilities Plan (CDM, 1989).

TABLE 3-6

EXISTING AND FUTURE AVERAGE ANNUAL ABN CONCENTRATIONS

Constituent	Existing[1] Concentrations (mg/l)	Projected (2014)[2] Concentrations (mg/l)
Phenol	0.012	0.015
Benzyl alcohol	0.007	0.009
2-methylphenol	0.006	0.008
4-methylphenol	0.032	0.040
Benzoic Acid	0.045	0.056
4-chloro-3-methylphenol	0.006	0.008
Isophorone	0.006	0.008
1,2,4-trichlorobenzene	0.006	0.008
2-methylnapthalene	0.006	0.008
N-Nitrosodiphenylamine	0.008	0.010
Di-n-butyl phthalate	0.007	0.008
Butylbenzyl phthalate	0.006	0.008
Bis(2-ethylhexyl) phthalate	0.059	0.074
Di-n-octyl phthalate	0.006	0.008
Napthalene	0.009	0.011
Diethyl phthalate	0.007	0.009

(1) Based on influent samples taken from the headworks of the New Bedford wastewater treatment plant.

(2) Increase based on proportional increases of domestic, commercial, institutional and industrial flows relative to total flow.

(3) Derived from the draft Phase 2 WWTP Facilities Plan (CDM, 1989).

TABLE 3-7

EXISTING AND FUTURE AVERAGE PCB CONCENTRATIONS

Constituent	Existing[1] Concentrations (mg/l)	Projected (2014)[2] Concentrations (mg/l)
gamma-BHC	0.00008	0.00012
PCB-1242	0.00025	0.00032
PCB-1254	0.00033	0.00040

(1) Based on influent samples taken from the headworks of the New Bedford wastewater treatment plant.

(2) Increase based on proportional increases of domestic, commercial, institutional and industrial flows relative to total flow.

(3) Derived from the draft Phase 2 WWTP Facilities Plan (CDM, 1989).

TABLE 3-8

EXISTING AND FUTURE AVERAGE ANNUAL VOC CONCENTRATIONS

Constituent	Existing[1] Concentrations (mg/l)	Projected (2014)[2] Concentrations (mg/l)
Methylene chloride	0.005	0.006
1,2-dichloroethene	0.005	0.006
Chloroform	0.009	0.012
1,2-dichloroethane	0.005	0.006
1,1,1-trichloroethane	0.010	0.012
Trichloroethylene	0.012	0.015
1,1,2,2-tetrachloroethylene	0.007	0.010
1,1,2,2-tetrachloroethane	0.005	0.007
Toluene	0.037	0.047
Ethylbenzene	0.009	0.011
Total xylenes	0.042	0.053
2-Butanone	0.029	0.036
Acetone	0.132	0.166
Benzene	0.004	0.005
4-methyl 2-pentanone	0.006	0.008

(1) Based on influent samples taken from the headworks of the New Bedford wastewater treatment plant.

(2) Increase based on proportional increases of domestic, commercial, institutional and industrial flows relative to total flow.

(3) Derived from the draft Phase 2 WWTP Facilities Plan (CDM, 1989).

4.0 WET WEATHER FLOWS AND LOADS

4.1 INTRODUCTION

This section describes the characterization of wet weather flows and loads. The characterization of CSOs was done in several steps. First, field data were collected at various CSO outfalls during the monitoring program to determine the volume and water quality of the discharges. Numerical models of the combined system were developed to estimate the response of the system (in terms of frequency, volume, and duration of spills) to various rainfall events. The models were calibrated to the data collected during the field program. Finally, once calibrated, the models were used to determine what rainfall volume and intensity is needed to cause each CSO regulator to spill, how frequently each CSO regulator spills during the year, the volume of typical spills, and what water quality loadings to the receiving waters typically result from these spills. The water quality loadings from CSOs are then compared to the loadings from separated storm drains.

Section 4.2 summarizes previous modeling efforts of the New Bedford combined system. The field program conducted in 1989 for the land-based modeling is described in Section 4.3. Sections 4.4 and 4.5 describe the development, calibration, and application of the sewer models. The models are used to simulate the hydraulic response of New Bedford's sewer network to various rainfall events under existing and improved conditions. A summary of results is provided in Section 4.6, which includes how often each CSO regulator spills, how much typically spills, and the related loadings to the receiving waters.

4.2 PREVIOUS WORK

New Bedford's sewer system and wastewater treatment plant have been extensively studied. Previous studies provide preliminary evaluations of both the sewer system and the wastewater treatment plant (WWTP). The sewer

system was first evaluated in 1983, as presented in the Interim Summary Report on Combined Sewer Overflows, Phase 1. Phase 2 extended the land-based models using additional field data collected in 1987. The Phase 1 WWTP Facilities Plan and Screening Studies report, prepared in 1987, examined feasible designs for a new wastewater treatment plant. The draft Phase 2 WWTP Facilities Plan was completed in August 1989, and information from both WWTP reports relevant to this Phase 3 CSO report is described below.

The Interim Summary Report on Combined Sewer Overflows, Phase 1, prepared in 1983, was the starting point for the current study and modeling effort. Two models were used to describe the system: EPA's Stormwater Management Model (SWMM) and the Storage Treatment Overflow Runoff Model (STORM), originally developed by the Army Corps of Engineers' Hydraulic Engineering Center. SWMM was used primarily to represent the detailed hydraulics of the sewer system for individual rainfall events. STORM provides estimates of long-term average annual pollutant loadings. These models are described in more detail in Sections 4.4 and 4.5. Initially, only the hydraulics of the main interceptor running north-south through the City of New Bedford were explicitly modeled in EXTRAN, a subroutine of SWMM. An extensive field program was conducted to calibrate and verify the model and provide an estimate of flows reaching the treatment plant. This program evaluated not only the physical structures of the system, but the volume of water transported within the system, using depth measurements taken during several storm events.

The STORM model was developed to generate average annual pollutant loads to the receiving waters, and to estimate the frequencies of overflow events. Hydraulic data from EXTRAN and field data on pollutant concentrations at CSO discharge points were used to calibrate and verify the STORM model. Estimates of runoff volumes were made based on land use data and field observations.

Finally, in preparation for the Phase 2 study, the RUNOFF portion of the SWMM model was used to refine predictions of runoff volumes for individual events, rather than the long-term estimates STORM provides on a yearly

basis. The largest subbasin in the system, subbasin 3, was chosen for the calibration of RUNOFF. Depth of flow in several pipes was measured during 3 storm events, and subsequently used to calibrate the RUNOFF parameters.

Phase 2 of the CSO facilities plan involved data collection and extension of the land-based models. The field data from this 1987 work were incorporated in the calibration and verification for the SWMM model.

The Phase 1 WWTP Facilities Plan study, completed in 1987, and the draft Phase 2 WWTP Plan, provide the framework for the design and construction of a new secondary treatment plant for the City of New Bedford. The scope of these reports emphasize treatment plant design and the siting of the facility and outfall. Extensive analysis and field work was conducted to estimate current and future dry weather flows and pollutant loads. The dry weather conditions used in the CSO modeling were based on conditions described in these two reports. In addition, field measurements of pump capacities, sediment depths in the main interceptor, and regulator elevations contained in these reports were incorporated into this Phase 3 CSO modeling effort. The wet weather conditions used in the Phase 1 WWTP report are based on the 1983 CSO study. The Phase 3 CSO study, presently undertaken, contains more detailed analyses, superseding the wet weather analyses presented in the 1983 report and in the Phase 1 WWTP Facilities Plan report.

4.3 FIELD PROGRAM

4.3.1 OBJECTIVES

The purpose of the field investigations for the current CSO modeling was to further characterize the quantity and quality of CSO discharges, and to provide data to calibrate and verify the pollutant parameters in the STORM model. Additional field work was done to calibrate and verify the receiving water model, and is described separately in Section 6.0 and Appendix C1.

There are 35 active combined sewer overflow outfalls (CSOs) in New Bedford, and 23 separated storm drain outfalls (3 previous CSOs have had their contributing sewer pipes separated from storm drains, and are now operating as storm drain outfalls). An additional 35 storm drains contribute to receiving waters from adjacent communities. Because of budget limitations, not all 35 CSOs could be sampled. Six sampling sites were selected based on four factors: volume of overflow, proximity to resource areas, frequency of overflow, and accessibility. Outfalls discharging to each of the three receiving waters (Clarks Cove, the Inner Harbor and the Outer Harbor) were selected. The locations of the six monitored outfalls are shown in Figure 4-1, and are listed below:

Clarks Cove

Outfall 004 - Clarks Cove hurricane barrier

Outer Harbor

Outfall 016 - Frederick Street and East Rodney French Boulevard

Inner Harbor

Outfall 022 - Sawyer Street

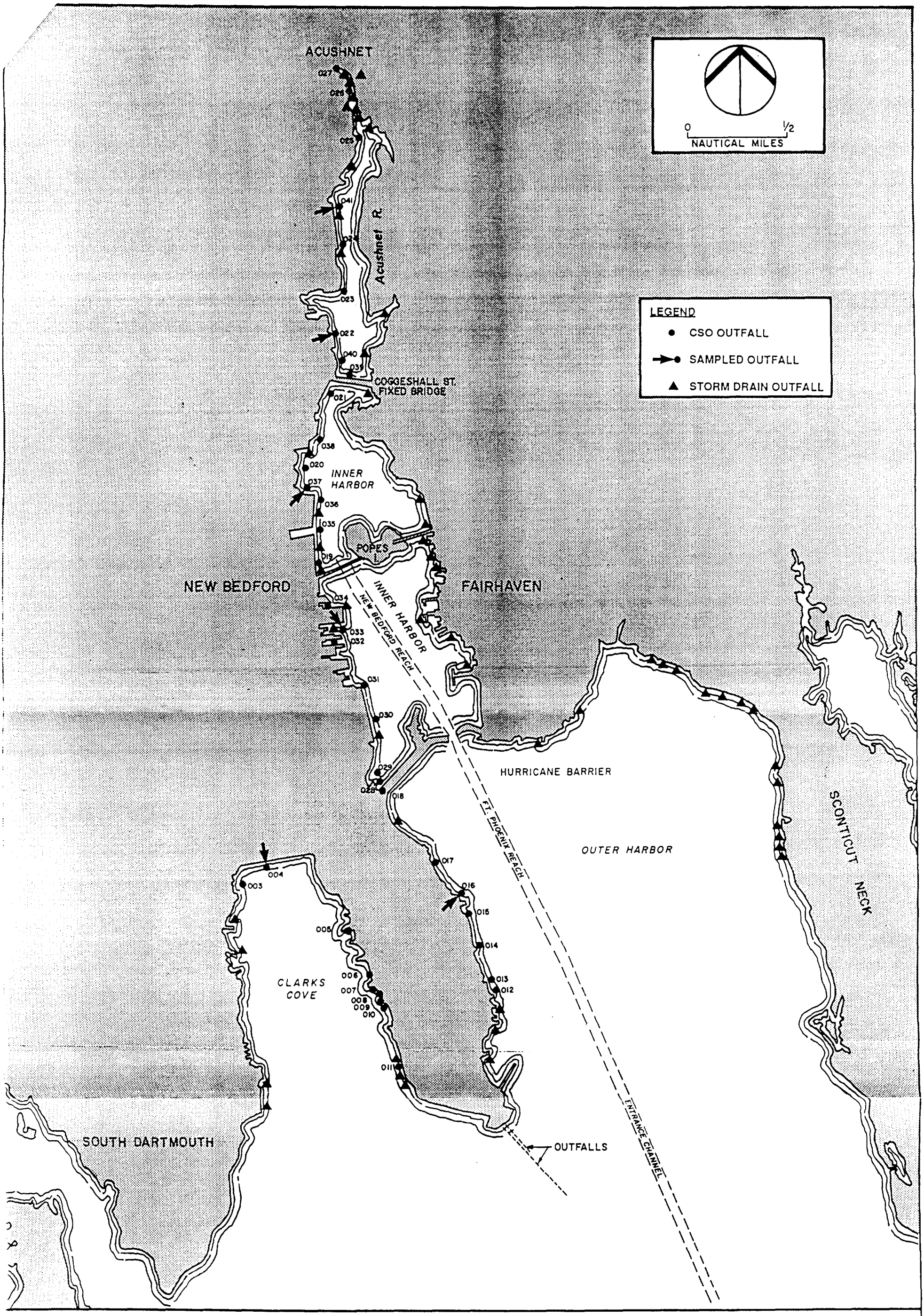
Outfall 033 - School Street and Acushnet Avenue

Outfall 037 - Pope Street

Outfall 041 - Belleville Avenue

The following data were collected at each of the six outfalls:

1. Flows during wet and dry weather conditions over a three month period (mid-January to mid-March).
2. Concentrations of conventional pollutants during up to two storm events.
3. Concentrations of metals and non-conventional parameters during one storm event.
4. Concentrations of priority pollutants during one storm event.



City of New Bedford

COMBINED SEWER OVERFLOW • FACILITIES PLAN - PHASE 3

Combined Sewer Outfalls,Stormwater Discharge Points and Monitoring Sites for the 1989 Field Program

CDM

FIGURE 4 - 1

5. Acute toxicity of a composite sample on Mysid shrimp and Sheepshead minnow.

4.3.2 FLOW MEASUREMENTS

Techniques/Equipment

Flow-measuring equipment was installed to record flow at each sampling location. In 5 of the 6 outfall locations, Montedoro-Whitney Flowloggers were installed. At the sixth outfall, the Belleville Avenue Pump Station, side-wall weirs were constructed because of limited accessibility, and Manning Ultrasonic Dippers were installed. Table 4-1 outlines the type of equipment used at each CSO outfall. In most cases flows could not be measured at the ends of the pipes because incoming tides submerged the outfalls. Consequently, flows were measured in pipes further upstream in the system to avoid backwater influences. In some cases, up to 4 separate gauges in separate pipes were needed to measure the total overflow from the CSO.

Montedoro-Whitney Flowloggers consist of a datalogger mounted directly beneath a manhole cover for easy access, and a sensor, connected to the datalogger, which is installed on the invert of the pipe. The sensor detects depth of flow based on pressure. Velocity is measured based on the movement of ionized particles over the sensor. The information is recorded and stored in the datalogger once every minute. Using Montedoro-Whitney's Datamate software, estimates of flow based on depth, velocity, and user-defined outfall parameters were computed using the Point-Velocity Flow Equation (McCullough Associates, 1979).

The Manning Ultrasonic Dippers are mounted directly upstream of a weir several feet above the average level of flow. The dippers determined depth of flow based on sound waves reflected off the surface of the water. The data are continuously logged on circular 7-day charts. Estimates of flow over the weir were calculated based on the dimensions and physical characteristics of the weir.

TABLE 4-1

EQUIPMENT USED IN FIELD STUDY FOR FLOW MEASUREMENTS

CSO	Location	Flow Measuring Device(s)	Storm drain only	CSO only	CSO and Storm Drain	Comments
004-1*	Clarks Cove	(1) Montedoro-Whitney unit			x	Configuration of the combined system necessitated the use of 4 gages installed in pipes directly tributary to the main outfall in order to measure flow.
004-2	Clarks Cove	(1) Montedoro-Whitney unit		x		
004-3	Clarks Cove	(1) Montedoro-Whitney unit		x		
004-4	Clarks Cove	(1) Montedoro-Whitney unit	x			
014	Apponagansett Pump Station	(1) Montedoro-Whitney unit		x		No dry weather flow
022	Samper Avenue	(1) Montedoro-Whitney unit		x		Dry weather flow
034	State Pier	(1) Montedoro-Whitney unit			x	Dry weather flow. Invert 3' below high tide level
037	JFK Highway	(1) Montedoro-Whitney unit		x		Dry weather flow
041-1 041-2	Belleville Avenue	(2) Manning Ultrasonic Dippers		x		2 combined sewers contribute flow to the main overflow so 2 Manning Dipper/weir setups were necessary to measure flow. Minimal dry weather flow.

004-1 = CSO No. 004 - Gauging Location No. 1

Results

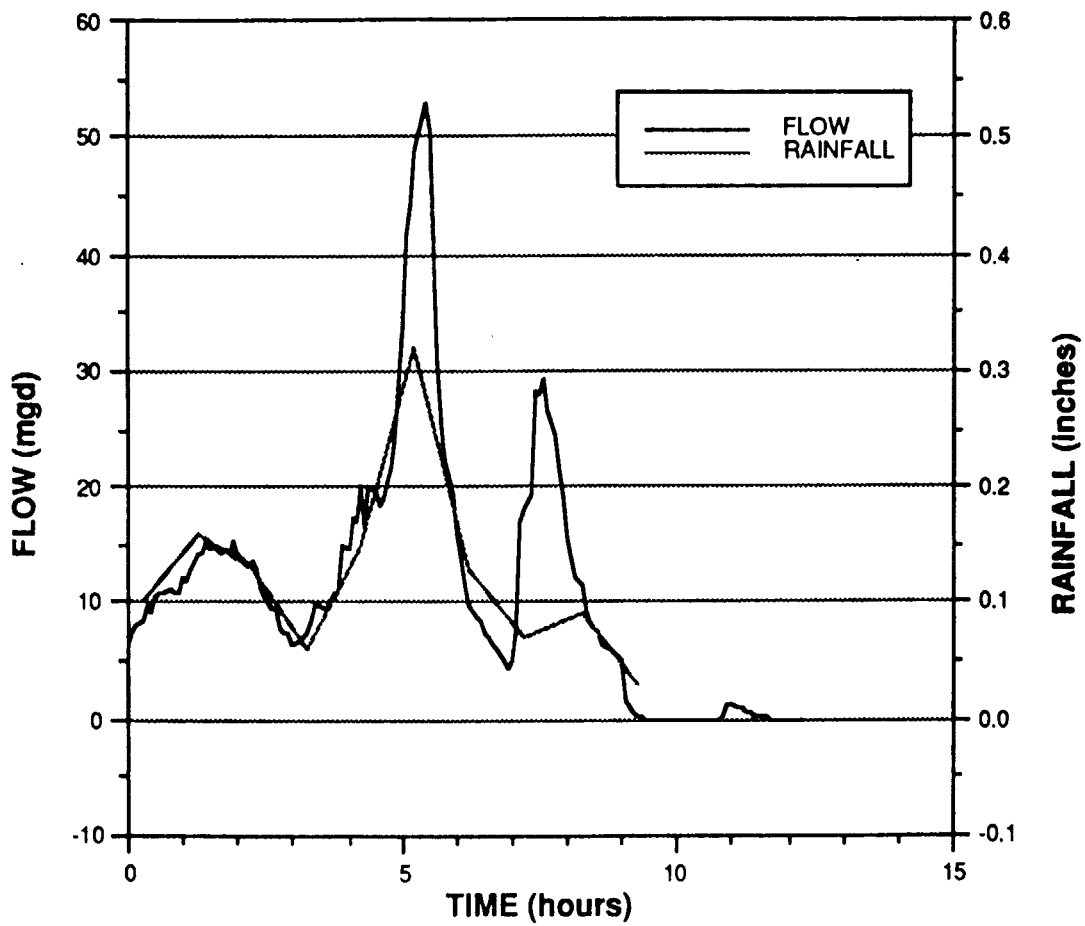
Data from the Montedoro-Whitney Flowloggers can be presented in tabular or graphical form, showing time, depth, velocity, and flow. Figure 4-2 illustrates the type of information available from the flowlogger data. This figure shows the volume of flow from CSO 022, one of the larger CSOs in the sewer network, plotted in conjunction with precipitation data from the event. Data were manually recorded from the Manning Dipper charts and similar estimates were computed for volumes of overflow from the Belleville Avenue outfall. Table 4-2 summarizes on what days flow data were obtained for the individual outfall locations. A summary of all data that were collected is presented in Appendix A.

4.3.3 WATER QUALITY SAMPLING

Water quality samples were collected at the end of the pipes at six CSO outfall locations: 004, 016, 022, 033, 037, and 041, located respectively at Clarks Cove, Frederick Street, Saugus Street, State Pier, JFK Highway, and Belleville Avenue Pump Station. For each outfall detailed information was recorded for each event, including the sampling procedure, weather conditions, measurements of pH, temperature and dissolved oxygen and visual inspection of the color and floatables in the outfall.

Samples were collected from each of the outfalls using a stainless steel bucket. Composite samples were stored in a 18.9-liter jug until the last sample was collected. The appropriate vessels for the specific parameters were then filled (see Table 4-3 for sample vessel descriptions). Water quality samples were also collected from one storm drain contributing to flows at CSO outfall 004, as shown on Figure 4-1.

Samples were analyzed for conventional pollutants (which include nutrients, BOD, TSS, coliforms, pH, and visual characteristics) by GHR Analytical. EnviroSystems, Inc. analyzed samples for non-conventional pollutants (priority pollutants as defined by EPA) and conducted acute toxicity tests. A summary of conventional pollutant concentrations is presented in Table 4-4. Table 4-5 summarizes the concentrations of non-conventional



City Of New Bedford
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FACILITIES PLAN - PHASE 3

FIGURE 4-2
VOLUME OF FLOW FROM OUTFALL 022
DURING THE MARCH 24, 1989 RAIN EVENT

TABLE 4-2

FLOW MEASUREMENTS TAKEN AT
COMBINED SEWER OVERFLOWS IN 1989

	RAIN EVENTS						
OUTFALL	1/26/89	2/3/89	2/14/89	2/15/89	2/21/89	3/18/89	3/24/89
004-1*	✓	✓	✓	✓			
004-2*			✓		✓		
004-3*	✓						✓
004-4*						✓	
014							✓
022			✓	✓	✓	✓	✓
033	✓		✓			✓	✓
037	✓		✓			✓	
041							✓

NOTE: * 4 gauges in 4 separate pipes were needed to measure flows at CSO 004.

TABLE 4-3

SAMPLE VOLUME COLLECTION REQUIREMENTS

Conventionals	Bottle Used	Volume Needed
Fecal & Total Coliform	250 ml Sterile glass	
BOD, Colors, TS, TSS, Floatables	1 liter plastic	8000 ml
Oil & Grease	500 ml glass with H_2SO_4@<2pH	3 liters
COD, Total Phosphorus, TKN, Ammonia	1 liter plastic with H_2SO_4@<2pH	3.2 quarts
Lead, Chromium	100 ml plastic with Nitric Acid	1 gal (min)
Acute Toxicity		
96-hr *Mycid* test	4 liter plastic	8 liters
96-hr *Cyprinodon* test	4 liter plastic	2 gal(min)
Priority Pollutant Scan		
Metals	2 liter glass	3600 ml
Cyanide	1 liter glass	3.6 liters
ABN	250 ml glass	1 gal (min)
Pesticides/PCBs	250 ml glass	
VOA	2 - 40 ml glass vial	

TABLE 4-4

CONCENTRATIONS OF CONVENTIONAL POLLUTANTS
IN NEW BEDFORD COMBINED SEWER OVERFLOWS

	SAMPLING				POLLUTANTS					
OUTFALL	DATE	START TIME	STOP TIME	SAMPLE TYPE	TSS (mg/l)	BOD (mg/l)	AMMONIA (mg/l)	TKN (mg/l)	P (mg/l)	TOTAL COLIFORM (MPN/100 ml)
041-1	02-14-89	647	657	Discrete	140	120	4.75	17.40	3.22	> 24000
037	02-14-89	701	705	Discrete	40	46	1.86	10.60	1.58	> 24000
004	02-14-89	847	917	Composite	140	31	1.09	4.16	0.86	> 24000
004-1	02-21-89	1950	2000	Discrete	61	40	0.81	3.06	0.86	> 24000
004-2	02-21-89	2010	2020	Discrete	190	42	0.90	3.30	0.99	> 24000
004-3	02-21-89	1915	1925	Discrete	180	44	1.10	4.50	1.53	> 24000
004-4	02-21-89	1930	1940	Discrete	70	10	0.28	0.99	0.42	> 24000
033	02-14-89	913	1045	Composite	44	27	1.86	5.63	0.69	> 24000
022	02-14-89	1005	1017	Discrete	74	80	2.65	8.76	0.45	> 24000
033	02-15-89	1230	1240	Discrete	97	160	2.44	8.49	0.38	> 24000
037	02-15-89	1400	1410	Discrete	47	93	2.47	14.80	2.12	> 24000
022	02-21-89	945	1005	Discrete	210	90	2.30	9.01	0.82	> 24000
041-1	02-21-89	1830	1840	Discrete	100	76	2.69	8.39	0.82	> 24000
016	02-21-89	1835	1845	Discrete	74	9	0.30	2.02	0.55	> 24000

TABLE 4-5

CONCENTRATION OF NON-CONVENTIONAL POLLUTANTS IN NEW BEDFORD COMBINED SEWER OVERFLOWS

Parameter	OUTFALL 004	022	033	037	04
Salinity (ppt)	0	0	7	0	0
Specific Conductance (µmhos/cm)	293	279	9996	415	63
pH	6.29	6.57	6.68	6.71	6.67
Ammonia (mg/l)	1.1	2.7	1.8	2.0	5.0
Alkalinity (mg/l)	14	40	43	20	51
Hardness (mg $CaCO_3$/l)	20	28	1260	24	56
Total Residual Chlorine (mg/l)	<0.01	<0.01	<0.01	<0.01	<0.01
Total Solids (mg/l)	180	280	7300	300	520
Total Suspended Solids (mg/l)	33	89	25	44	100
Total Organic Carbon (mg/l)	8	19	3	14	11
Aluminum, total (mg/l)	1.3	0.8	0.3	0.5	0.8
Antimony, total (mg/l)	<0.04	<0.04	<0.04	<0.04	<0.04
Arsenic, total (mg/l)	<0.01	<0.01	<0.01	<0.01	<0.01
Beryllium, total (mg/l)	<0.005	<0.005	<0.005	<0.005	<0.005
Cadmium, total (mg/l)	<0.005	<0.005	<0.005	<0.005	<0.005
Chromium, total (mg/l)	0.01	0.15	0.007	<0.01	0.01
Copper, total (mg/l)	0.07	0.09	<0.01	<0.01	0.07
Iron, total (mg/l)	3.0	1.6	0.06	0.05	1.4
Lead, total (mg/l)	0.12	0.08	0.8	1.4	2.3
Mercury, total (mg/l)	<0.0003	<0.0003	<0.05	0.07	0.11
Nickel, total (mg/l)	<0.03	0.13	<0.0003	<0.0003	<0.0003
Selenium, total (mg/l)	<0.01	<0.01	<0.03	<0.03	<0.03
Silver, total (mg/l)	<0.01	<0.01	<0.01	<0.01	<0.01
Thallium, total (mg/l)	<0.01	<0.01	<0.01	<0.01	<0.01
Zinc, total (mg/l)	0.25	0.65	0.2	0.1	0.4
Cadmium, dissolved (mg/l)	<0.005	<0.005	<0.005	<0.005	<0.005
Copper, dissolved (mg/l)	<0.02	0.04	0.03	0.03	0.03
Lead, dissolved (mg/l)	<0.05	<0.05	<0.05	<0.05	<0.05
Silver, dissolved (mg/l)	<0.01	<0.01	<0.01	<0.01	<0.01
Cyanide, total (mg/l)	<0.01	<0.01	0.02	<0.01	<0.01
Phenols, total (mg/l)	22	12	11	22	20
Acid/Base/Neutral Extractables (µg/l)	408	1038	423	661	490
Pesticides (µg/l)	nd	nd	nd	nd	nd
Volatile organics (µg/l)	32	902	340	116	636

ND - Not Detected

pollutants. These data were compared to the data from the 1983 sampling program and the results were found to be consistent between the sampling programs. Table 4-6 summarizes mean values and standard deviations for each pollutant for both storm drains and CSOs. Fewer data were collected for storm drains, so literature values are presented in Table 4-6 for a "reasonableness" check. In general, the mean concentration found in New Bedford storm drains tends to be skewed towards the 90th percentile data, with the mean BOD concentration exceeding the 90th percentile. The results from both CSO and storm drains were incorporated in the estimates of pollutant loads to receiving waters using the STORM model, as shown in Section 4.6. Acute toxicity test results are discussed in Section 6.0. Complete reports of the analyses are contained in Appendix A.

4.4 MODELING THE COMBINED SEWER SYSTEM

The objective of combined sewer modeling is to accurately represent the hydrology and hydraulics of the sewer system. Given this representation, the response of the network can be observed under various storm events and system modifications and improvements, and the quantity and quality of combined sewer overflows (CSOs) for both individual events and long-term rainfall records can be predicted. Modeling is a powerful tool in evaluating and selecting appropriate CSO abatement strategies.

4.4.1 PURPOSE

New Bedford's wastewater collection system is complex: the system surcharges in several locations even under minor rain events; tidal influences and backwater conditions frequently extend over thousands of feet of pipe, influencing both the frequency and volume of overflow; and flows are controlled at multiple locations by pumps, weirs, and orifices. An oversimplification of the system would completely misrepresent the actual system operation.

The Stormwater Management Model (SWMM), developed through funding by the EPA, is capable of modeling this level of complexity with the use of the RUNOFF and extended transport (EXTRAN) sections of the computer code. The

TABLE 4-6

AVERAGE POLLUTANT CONCENTRATIONS BASED ON FIELD DATA

POLLUTANT		CSO CONCENTRATIONS		STORM DRAIN CONCENTRATIONS		LITERATURE VALUES FOR STORM DRAINS[4]	
		Mean	Standard Deviation	Mean	Standard Deviation	Median Urban Site	For 90th Percentile Urban Site
TSS	(mg/l)	80	57	95	63	100	300
BOD	(mg/l)	53	43	21	12	9	15
Ammonia	(mg/l)	1.46	1.19	0.28[1]	—	—	—
TKN	(mg/l)	6.71	4.44	2.12	1.33	1.50	3.30
Lead	(mg/l)	0.11	0.07	0.18	0.17	0.144	0.350
Zinc	(mg/l)	0.35	0.21	0.35	0.22	0.160	0.500
Chromium	(mg/l)	0.08	0.07	0.06	0.02	—	—
Copper	(mg/l)	0.11	0.06	0.08	0.06	0.034	0.093
Iron	(mg/l)	1.80	0.79	N.T.[2]	N.T.[2]	—	—
Aluminum	(mg/l)	0.76	0.31	N.T.[2]	N.T.[2]	—	—
Total Coliform[3]	(MPN/100 ml)	1.56×10^6	—	1.48×10^5	—	8×10^3	1.86×10^5

NOTES:
(1) Based on one sampling point
(2) Pollutant not tested for in 1983 sampling program
(3) Geometric mean; sampling values ranged from 10^5 to 10^6 for CSOs and from 10^3 to 10^6 for storm drains.
(4) From "Results of the Nationwide Urban Runoff Program, Volume 1 - Final Report," U.S. Environmental Protection Agency. December, 1983.

RUNOFF subroutine simulates the rainfall/runoff process; EXTRAN routes the flows generated by RUNOFF through the interceptor network. EXTRAN is capable of handling non-steady flow, backwater conditions, surcharging, street flooding, pumps, weirs, and orifices. SWMM, therefore, was selected to enhance our understanding of the various conditions that lead to overflows. It was used to estimate what storm intensity and duration, or what level of infiltration/inflow combined with peak dry weather flow conditions, would cause an overflow to occur. From this detailed understanding of the hydraulics of the sewer system, alternative strategies for reducing and eliminating CSOs can be developed and analyzed.

4.4.2 DESCRIPTION OF SWMM

SWMM was used in New Bedford to characterize two basic processes: the hydrologic process, modeled by the RUNOFF subroutine, and hydraulic routing, modeled by the EXTRAN subroutine.

Surface runoff, as simulated in RUNOFF, is a function of:

- rainfall intensity and duration
- antecedent conditions
- percent of the area that is impervious
- topography, and
- soil characteristics

Combined sewer service areas are represented by a series of subcatchments. The program accounts for losses due to infiltration on pervious surfaces, and surface detention. SWMM accounts for changes in the timing of runoff based on drainage basin characteristics in producing inlet hydrographs for each subcatchment. The overall process is driven by measured rainfall data.

The parameters required for the simulation of a subcatchment in RUNOFF include area, width of the overland flow path, percent impervious ground area, ground slope, roughness factors (Manning's "n") for both impervious and pervious areas, and depression storages for impervious and pervious areas.

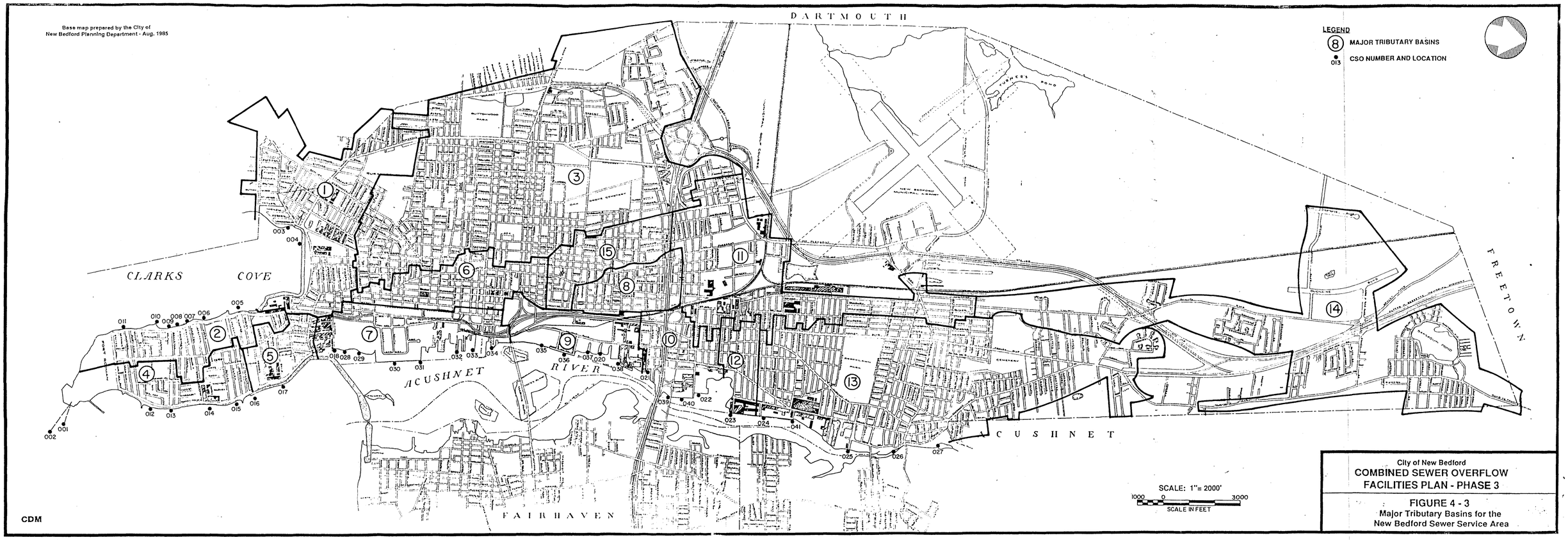
Infiltration in SWMM can be represented through the use of the Horton equation or the Green-Ampt equation. Green-Ampt was selected for this study primarily because it offers the advantage of using physically based parameters that are more readily predicted than the parameters required by the Horton equation. Parameters required by the Green-Ampt equation include hydraulic conductivity, initial moisture deficit and capillary suction. These parameters are adjusted during the calibration process.

Based on these parameters, and using assumed or actual rainfall data, RUNOFF generates hydrographs that are then used as input to EXTRAN for routing through the sewer network. EXTRAN is a dynamic flow routing code used to simulate unsteady flow in open channels and closed conduit systems. The program solves the full dynamic equations for gradually varied unsteady flow (the Saint-Venant equations) using an explicit, time step algorithm. This method allows the modeling of pressure flow, flow reversal, surcharging, and backwater conditions, all of which occur in the New Bedford system.

The sewer system is represented by a series of links (conduits) and nodes (conduit junctions or manholes). A link is defined by a length between two nodes, a shape and size of pipe, and a Manning's "n." Nodes are defined by an invert elevation and a ground elevation. Control structures or regulators can be modeled as high level outlets or as weirs with or without tide gates. Pump stations can also be modeled, with up to 3 pumping rates that can be activated either by reaching a certain elevation or depth, or by filling a given wet well volume.

4.4.3 APPLICATION TO THE NEW BEDFORD SYSTEM

The service area of the New Bedford sewer system (the tributary area to the New Bedford wastewater treatment plant) was divided into 15 subbasins, shown in Figure 4-3. The divisions were based on the areas associated with the major collectors of the New Bedford system, and by the three corresponding receiving waters for the CSOs. Each of the 15 subbasins were further subdivided into 5 to 20 subcatchments for input to RUNOFF. A summary of the average input parameters for each of the 15 major subbasins


Base map prepared by the City of
New Bedford Planning Department - Aug. 1985
DARTMOUTH
LEGEND
MAJOR TRIBUTARY BASINS
CSO NUMBER AND LOCATION
CLARKS COVE
ACUSHNET RIVER
FAIRHAVEN
ACUSHNET
FREETOWN
SCALE: 1"= 2000'
1000 0 3000
SCALE IN FEET
CDM
City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3
FIGURE 4 - 3
Major Tributary Basins for the
New Bedford Sewer Service Area

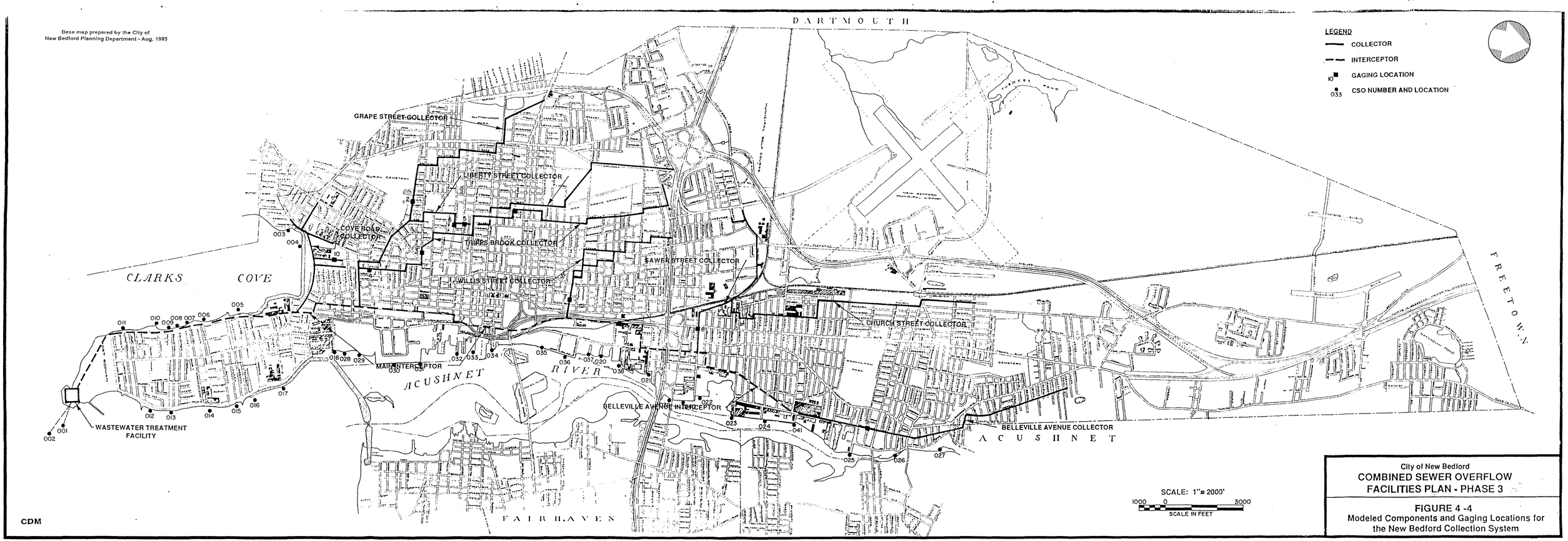
is given in Table 4-7. Note that only combined and partially combined areas (i.e., areas where roof leaders are tied into the sanitary network in separated areas) are modeled in SWMM. Area, average width, and slope of the subbasins were based on measurements from USGS quadrangle sheets (the scale is 1:25000 with a 10-foot contour interval). Percent impervious was initially estimated from land use data. From the calibration of the SWMM model for the 1983 CSO Interim Study, depression storage for pervious and impervious surfaces was found to be 0.25 and 0.06 inches, respectively; the resistance factor or Manning's "n" for pervious and impervious surfaces was estimated at 0.30 and 0.025, and Green-Ampt infiltration parameters were set at the following values:

capillary suction: 4.0 inches
infiltration rate: 0.10 in/hr
initial moisture deficit: 0.025

These values were used initially in the model, and adjusted for model calibration based on field data.

The network of pipes simulated using EXTRAN is outlined in Figure 4-4. The main sewer interceptor was modeled in the 1983 CSO report. All other pipe data were developed for this report from as-built drawings of the sewer system and field surveys. Extensive field work was conducted for the 1987 Phase 1 WWTP Facilities Plan, including an evaluation of regulators, measurement of sediment accumulation in the main interceptor, and the determination of pump station capacities. These data were used to update the main interceptor model and to configure the new models of the other 9 major collectors, including the Sawyer Street, Willis Street, Cove Road, Tripps Brook, Liberty Street, Grape Street, Belleville Avenue, North End Relief, and Church Street collectors. Schematics of these collectors are shown in Appendix D.

The majority of New Bedford's combined sewer overflows are controlled by devices such as weirs or high level outlets, known as regulators. Table 4-8 lists all system regulators found in New Bedford's system. Each is numbered with its associated CSO discharge number, and given a letter to



City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4 -4
Modeled Components and Gaging Locations for the New Bedford Collection System

TABLE 4-7

SUMMARY OF SUBBASIN INPUT PARAMETERS FOR THE RUNOFF BLOCK OF SWMM

Sub-basin	Area (acres)	Average Width (feet)	Percent Impervious	Slope (ft/ft)
1	176.2	1100	39	0.02
2	128.2	940	39	0.04
3	941.8	1850	53	0.02
4	37.0	670	35	0.04
5	64.7	950	30	0.03
6	185.6	550	55	0.03
7	43.8	950	36	0.01
8	157.2	650	64	0.06
9	34.2	930	60	0.01
10	112.2	1700	60	0.01
11	394.1	630	43	0.02
12	100.1	1300	60	0.03
13	94.8	660	55	0.02
14	separated area	N/A	N/A	N/A
15	198.2	1570	52	0.02

TOTAL COMBINED AREA: 2668.1 acres

TABLE 4-8

REGULATOR INVENTORY

Regulator Number	Location	Type	Critical Elevation*
03A	Padnaram & Cove	High level outlet	1.58
04A	Rockdale & Cove	Float	0.28
04B	Orchard & Cove	Float	-1.34
04E	Bonney & Rivet	Weir	4.14
04F	Bonney & Rivet	Weir	3.10
04G	Crapo & Rivet	Side weir	4.41
04H	Bonney & Cove	Weir	-0.32
04I	David & WRF	Weir	4.60
05A	Dudley & WRF	Weir	3.60
06A	Lucas & WRF	Weir	2.58
06C	Capitol & WRF	Weir	3.11
06D	Lucas on WRF	Weir	3.89
07A	Capitol & WRF	Weir	6.66
08A	Calumet & WRF	Weir	5.58
09A	Aquidneck & WRF	High level outlet	6.52
010A	Belleville & WRF	Side weir	4.92
011A	Hudson & WRF	High level outlet	3.03
012A	Ricketson & ERF	Float	-0.24
012B	Bellevue & ERF	Float	-0.24
013A	Aquidneck & ERF	Float	1.32
014A	Apponogansett & ERF	Float	0.04
015A	Butler & ERF	Float	-1.41
016A	Frederick & ERF	High level outlet	1.31
017A	Rodney & ERF	Float	-1.11
017C	David & ERF	Float	-3.02
017D	Ruth & ERF	Float	-1.83
018A	Cove & ERF	Float	-1.17
018B	Cove & ERF	High level outlet	-0.22
020A	Wamsutta & Rt. 18	Double side weir	4.94
020B	Logan & Acushnet	Double side weir	6.82
021A	Washburn & N. Front	Weir	9.42
021B	Kenyon & N. Front	Weir	9.02
022A	Sawyer & N. Front	Double side weir	9.68
022B	Holly west of Belleville	Side weir	9.25
022C	Tallman, west of Belleville	Side weir	9.67
023A	Coffin & Richardson	Weir	0.32
024A	Hathaway & Riverside	Weir	-0.23
025A	Howard & River	High level outlet	-0.35
026	Truro	Direct outlet	
027A	Mill	Side weir	2.64
027B	Ohio & Acushnet	Side weir	17.11

NOTE:

*Elevation, using New Bedford Datum, at which an overflow begins to occur.

TABLE 4-8 (continued)

Regulator Number	Location	Type	Critical Elevation*
027C	Belleville & Mill	Side weir	16.94
028A	Gifford	Under Repair	
029A	Water & Blackmer	Separated	
030A	Potomska & Second	Weir	1.93
030B	South & Second	Weir	2.73
031A	Grinnell & Second	Side weir	1.76
031B	Allen & Second	Side weir	3.26
031C	Howland Street P.S.	High level outlet	2.00
031D	Bonney & Grinnell	High level outlet	22.85
032A	Russell & Second	Side weir	5.01
032B	Madison & Second	Side weir	7.75
032C	Walnut & Acushnet	Side weir	18.45
033A	School & Acushnet	Side weir	23.52
033B	Spring & Acushnet	Side weir	23.89
034A	Union & Acushnet	High level outlet	29.34
035A	Hillman & Foster	Side weir	63.52
035B	Pleasant & Maxfield	Side weir	52.26
036A	Willis & Purchase	Weir	25.22
036B	Pearl & Purchase	Side weir	23.42
036C	Pearl & Route 18	High level outlet	9.74
037A	Pope	Direct outlet	
038A	Wamsutta	Separated	
039A	Howard & Coggeshall	Separated	
040A	Coggeshall P.S.	Weir	3.76
041A	Belleville & Belleville S.	Side weir	3.55
041B	Belleville & Belleville N.	High level outlet	-0.95

NOTE:

*Elevation, using New Bedford Datum, at which an overflow begins to occur.

distinguish multiple regulators associated with an outfall. Although many of the regulators were designed as a float with a variable orifice (the flow to the interceptor is controlled by a flap gate connected to a float), none of them currently operate as designed because the float has either been removed or damaged. As a result, the regulators behave as weirs or side-weirs. All regulators are therefore represented in SWMM as either high level outlets, weirs, or single or double-side weir walls. Critical elevations, or the elevation at which an overflow begins (i.e., the top of a weir or the invert of a high level outlet) are also specified.

Roughness coefficients or Manning's "n" values in the sewer pipes were assumed to range from 0.015 to 0.020 based on field inspections. The final value selected was based on the calibration process of matching field water depths to model depths.

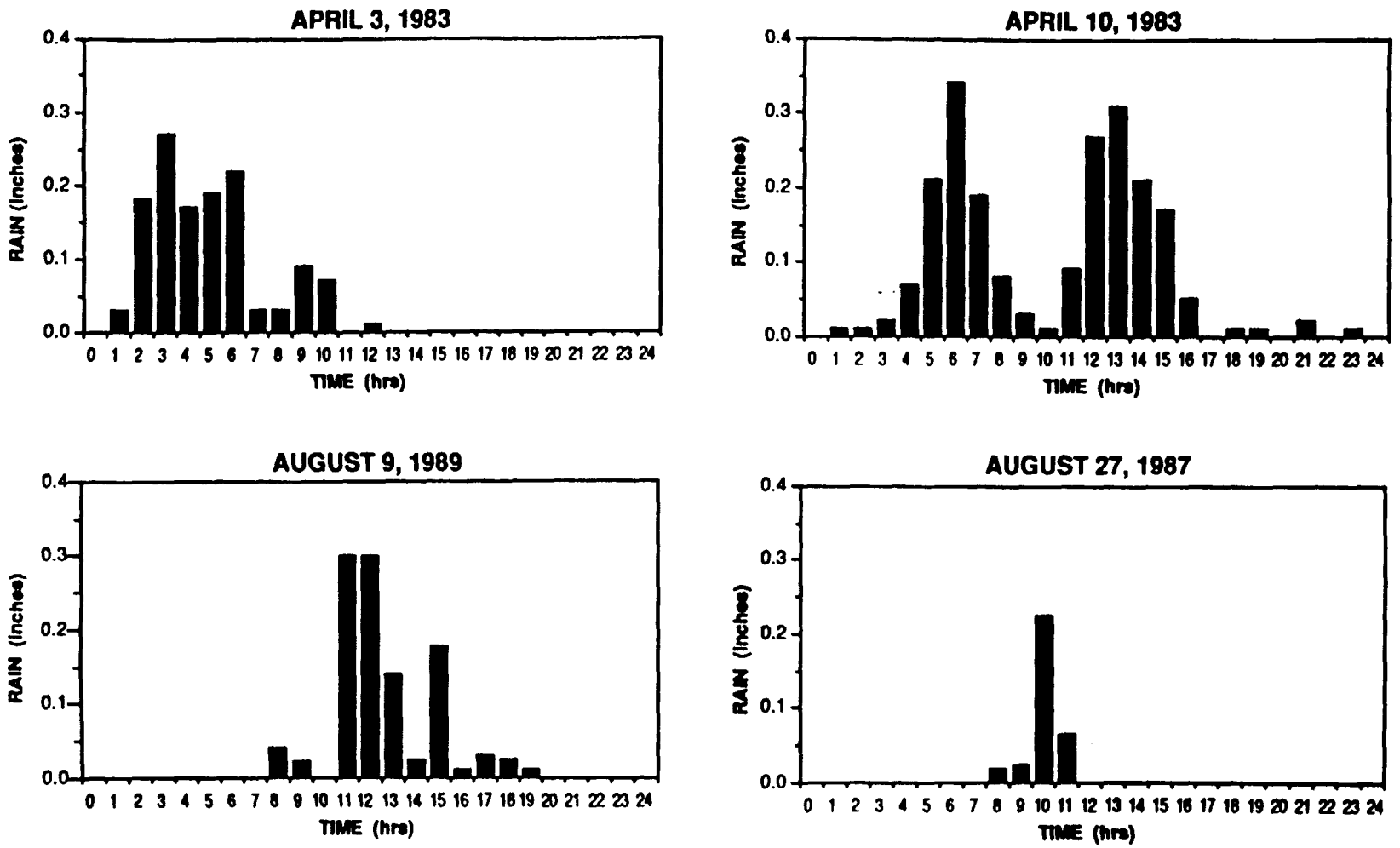
4.4.4 CALIBRATION

The main interceptor was calibrated during the 1983 study. Four storms were used to calibrate the pipe networks added to the model since 1983. The four events and the associated total rainfall for each are:

April 3, 1983	1.26 inches
April 10, 1983	2.12 inches
August 9, 1987	1.08 inches
August 27, 1987	0.33 inches

Hyetographs (or rainfall versus time plots), are shown in Figure 4-5. When these events are compared to the design events discussed in Section 4.4.6, the 8/27/87 storm is between a 1- and 2-week event (i.e., has a frequency of occurrence of somewhere between every week and every other week). The 8/9/87 storm is slightly larger than a 1-month event. The 4/3/83 storm is between a 1-month and 3-month event, and the 4/10/83 is between a 3- and 6-month event.

Because of limited number of personnel, not all sites were monitored for all events. Table 4-9 lists the ten sites that were gaged and when storm



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-5
RAINFALL HYETOGRAPHS FOR FOUR
CALIBRATION EVENTS

TABLE 4-9

SUMMARY OF FIELD DATA USED IN SWMM CALIBRATION AND VERIFICATION

Sampling Site Number*	Street Location	STORMS 4/3/83	4/10/83	8/9/87	8/27/87	12/15/87
1	Newton St. & Elm St.			x	x	
2	Grape St. & Field St.			x	x	
3	Allen St. & Cottage St.				x	
4	Thompson St. & Crapo St.	x	x			
5	Chancery St. & Maple St.	x	x			
6	Chancery St. & Hawthorne St.				x	
7	Willis St & State St.					x
8	Sawyer St. & Ashley Blvd.					x
9	Belleville Ave. & Sylvia St.					x
10	Bonney St. & Jouvette St.					x

NOTE:

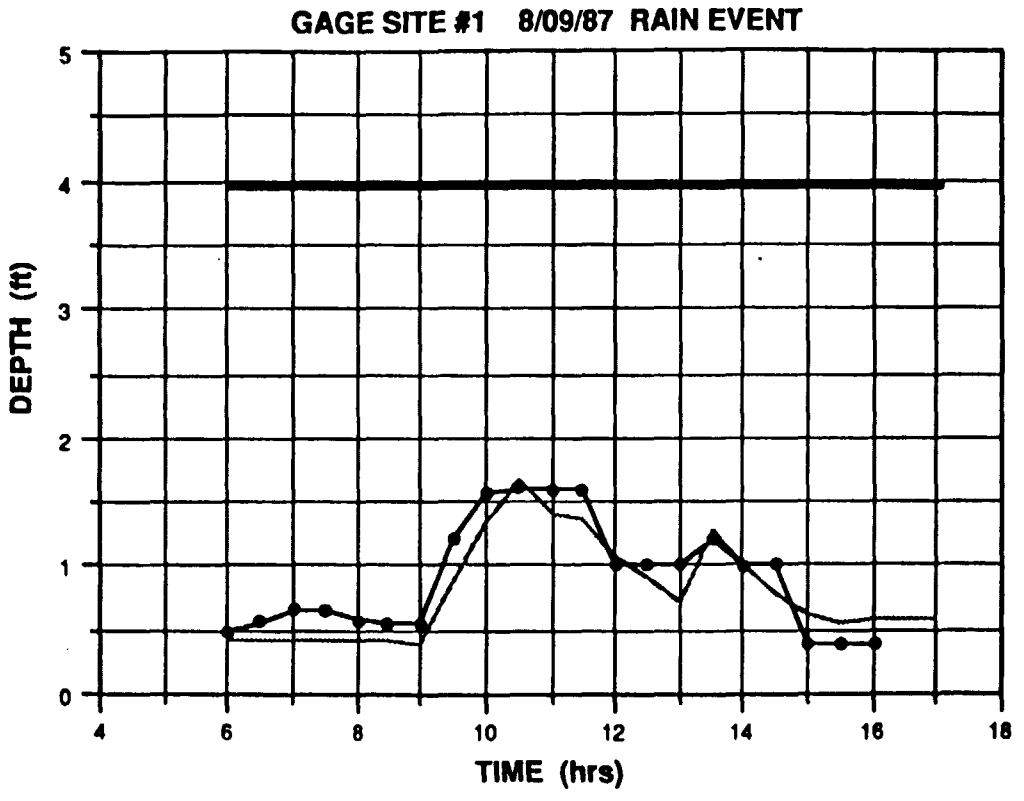
*Gage locations shown on Figure 4-4.

data were collected, including the verification storm on 12/15/87. Figure 4-4 shows the locations of the 10 gage sites.

The first step in calibrating the model was to accurately represent inflow to the system, through adjustment of the RUNOFF block parameters. This was accomplished using data collected from gages set upstream of flow control structures. For example, the gage at the intersection of Newton Street and Elm Street (site 1 in Figure 4-4) provided a measure of the runoff entering the Liberty Street and Tripps Brook collectors. A gage was also placed at the Grape Street and Field Street intersection (site 2), to calibrate the runoff for the Grape Street collector.

Based on field measured data, adjustments were made to two of the RUNOFF parameters: percent impervious, and the soil infiltration rate. Percent impervious gives a measure of the proportion of the ground surface which is impermeable. Most of the rainfall on impervious areas will become surface runoff and enter the combined sewer system, but rainfall on pervious surfaces has the opportunity to infiltrate into the soil and therefore will not enter the system. The impervious area within a subcatchment depends on several factors. A roof top is an impermeable surface, but can be included as part of the pervious area if the runoff empties onto a lawn. Similarly, if runoff from a parking lot flows to an adjacent pervious surface, the parking lot would not necessarily be considered impervious. In New Bedford, values of 30 to 65 percent impervious were determined to be appropriate during the model calibration process. The soil infiltration rate was adjusted from an initial value of 0.1 in/hr to 0.2 in/hr. Table 4-7 lists the calibrated characteristics of each subbasin required as input to the RUNOFF subroutine. Figures 4-6 and 4-7 show the results of the calibration of two of the gaged points. There is a good match between actual and simulated depths at both sites, indicating that the rainfall/runoff process is adequately represented.

The Tripps Brook and Liberty Street collectors intersect twice, providing two opportunities for the flows from the Tripps Brook collector to spill into the Liberty Street system. Gages at site 1 and site 3, at the intersection of Allen Street and Cottage Street, give a measure of the flow



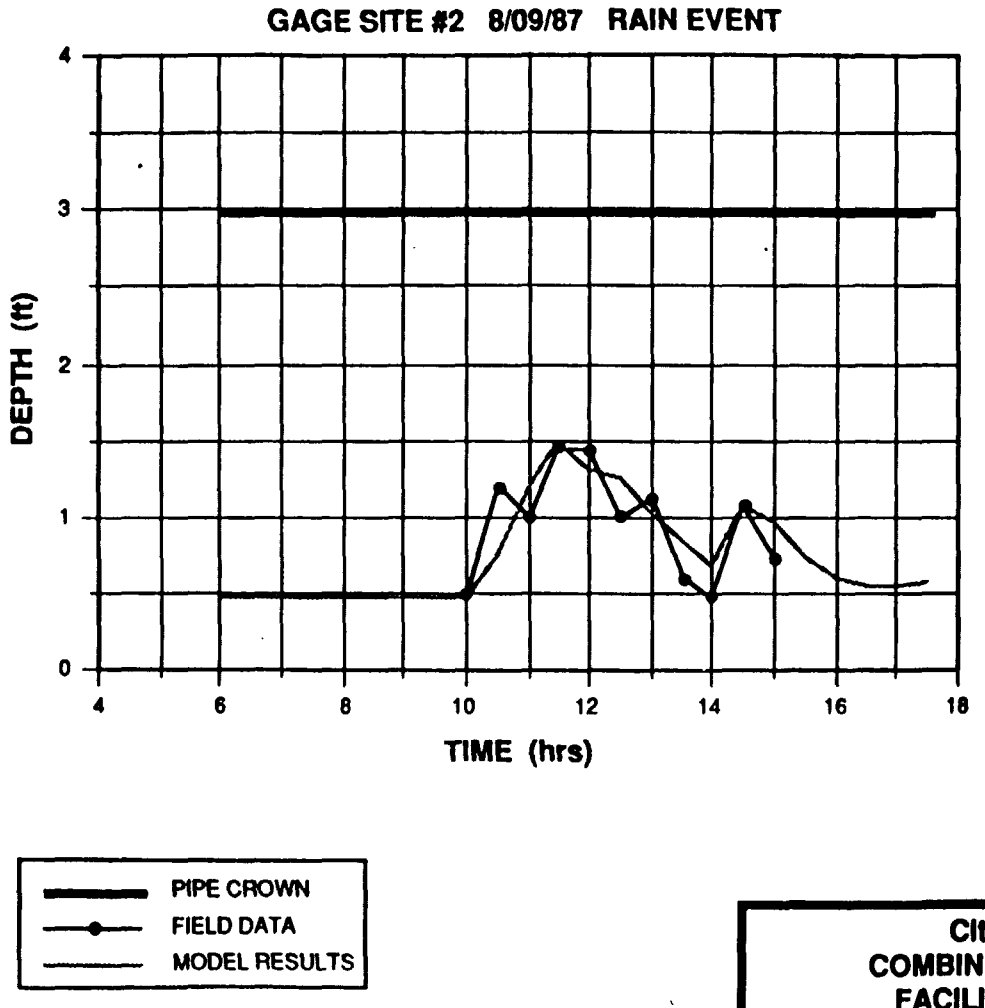
PIPE CROWN

FIELD DATA

MODEL RESULTS

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-6
SWMM CALIBRATION RESULTS:
COMPARISON OF SIMULATED VS. FIELD-MEASURED DEPTHS AT GAGING SITE 1



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-7
SWMM CALIBRATION RESULTS:
COMPARISON OF SIMULATED VS.
FIELD-MEASURED DEPTHS AT GAGING SITE 2

in the system before and after the crossovers. The gages only operated simultaneously for one event on 8/27/89. The results of the calibration are shown in Figures 4-8 and 4-9.

The Tripps Brook collector was gaged at site 4 (Thompson Street and Crapo Street) to measure the flow after regulator 31D. A plot of depths for the largest measured storm, 4/3/83, show only slight differences between the field and simulated data (Figure 4-10).

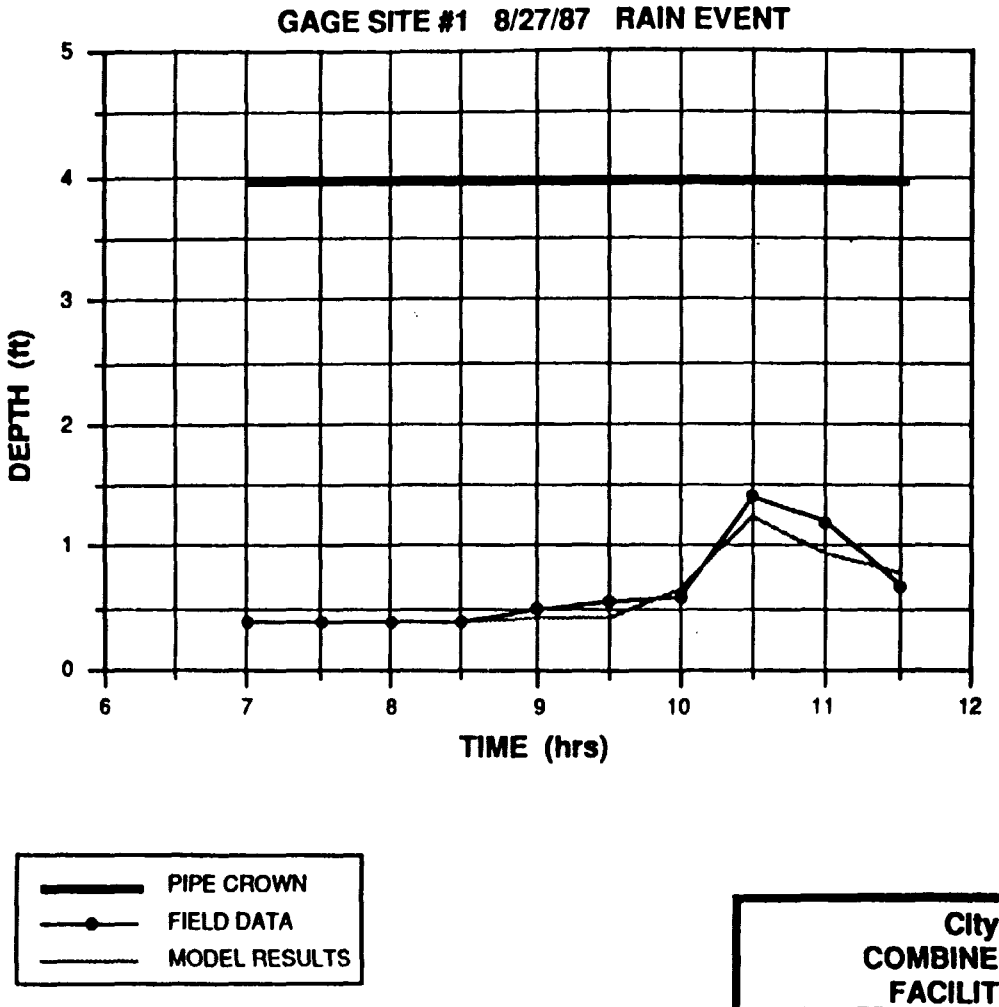
There is street flooding from the Liberty Street collector at the intersections of Chancery and Maple Streets. Two gages were set up to measure depth: one at the intersection of Chancery and Maple (site 5) and one downstream at site 6 (the Hawthorne and Chancery Streets intersection). Figure 4-11 shows a good correlation of field and simulated depths at site 5 under 3- to 6-month storm conditions. The model also accurately duplicated smaller events, as shown by Figure 4-12 at site 6.

These gages provided sufficient data to calibrate both the RUNOFF and EXTRAN blocks of SWMM. Similarities between basins allowed the parameters used in the calibrated basins to also be used in basins in which no field data were collected.

4.4.5 VERIFICATION

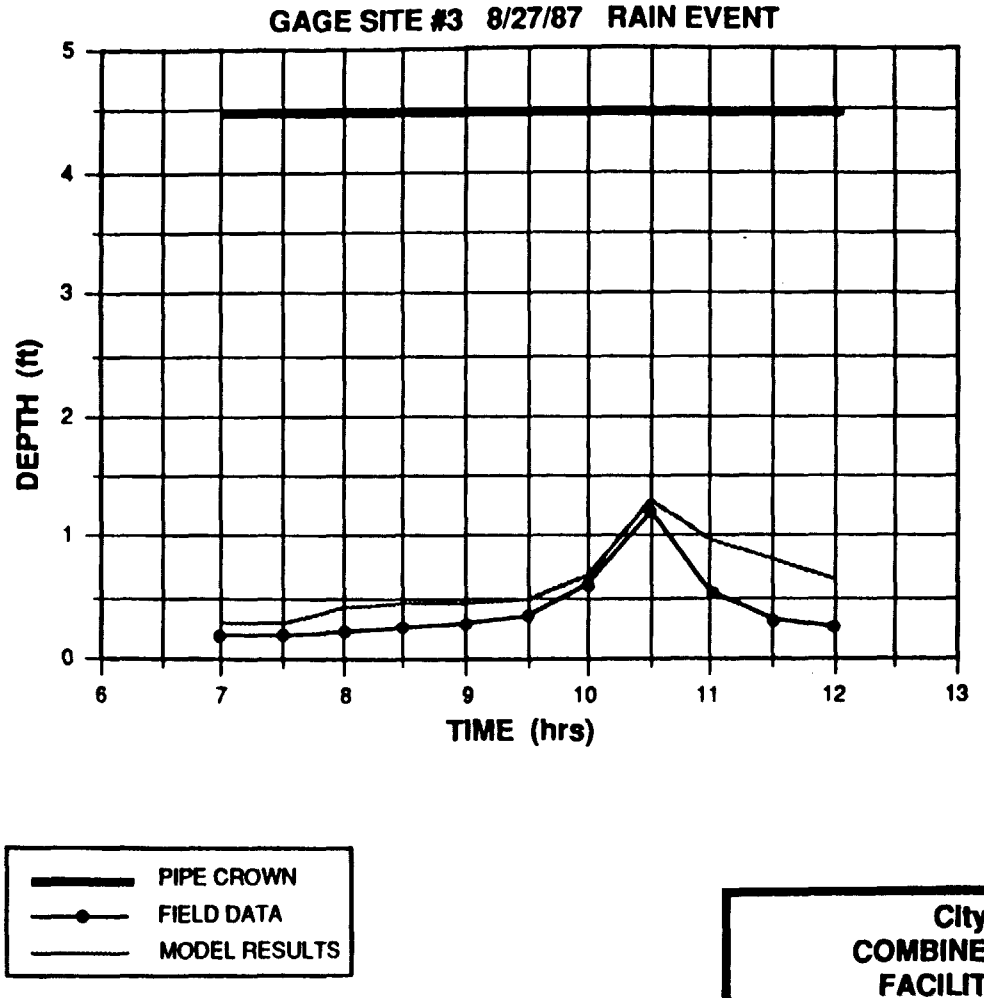
To test the assumption of applying the calibration parameters from the Tripps Brook, Liberty Street, and Grape Street collectors to the rest of the system, gages were set up in four new locations. Data were collected on 12/15/87 when 1.64 inches of rain fell (approximately a 3-month event). The hyetograph is shown in Figure 4-13.

The RUNOFF block was verified with gages placed at the intersections of Willis and State Streets (site 7), and Ashley Boulevard and Sawyer Street (site 8). Both sites measure flow upstream of any regulator in the Willis Street and Sawyer Street collectors. Figures 4-14 and 4-15 show a satisfactory match of the field and model depths.



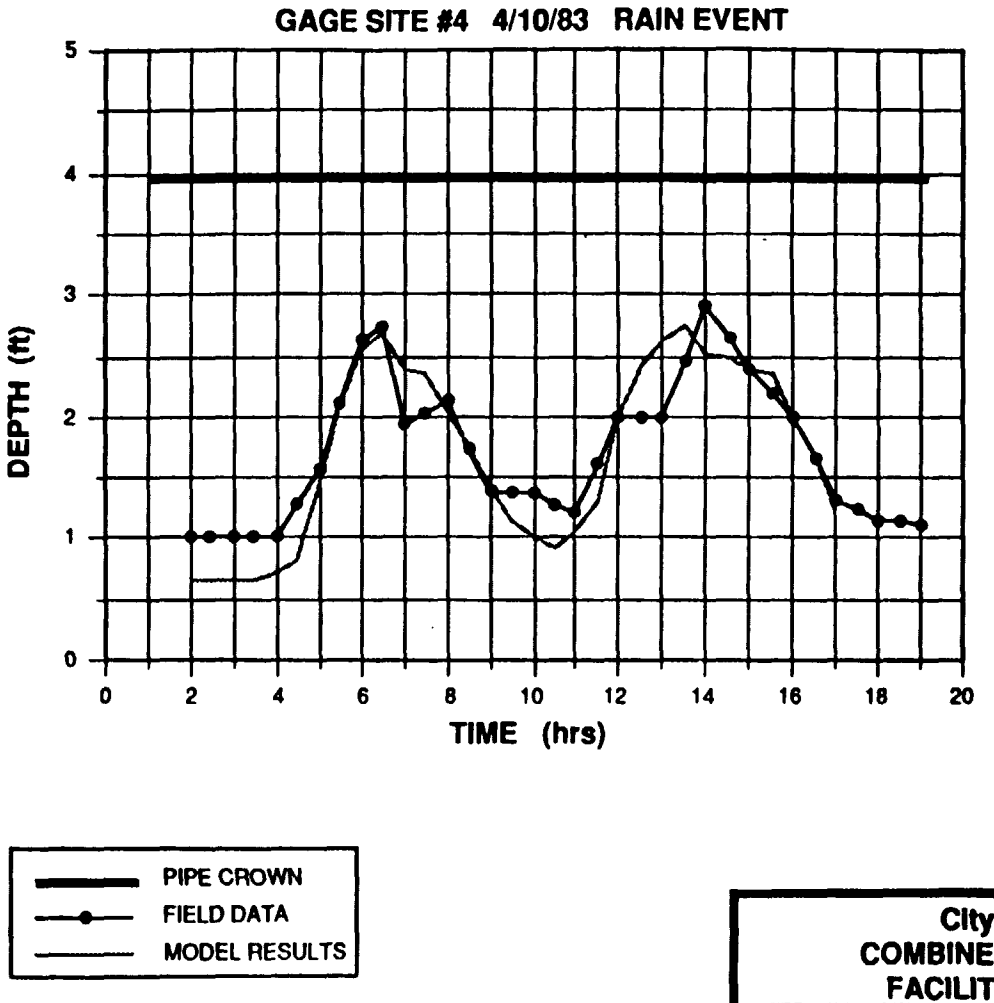
City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-8
SWMM CALIBRATION RESULTS:
COMPARISON OF SIMULATED VS.
FIELD-MEASURED DEPTHS AT GAGING SITE 1



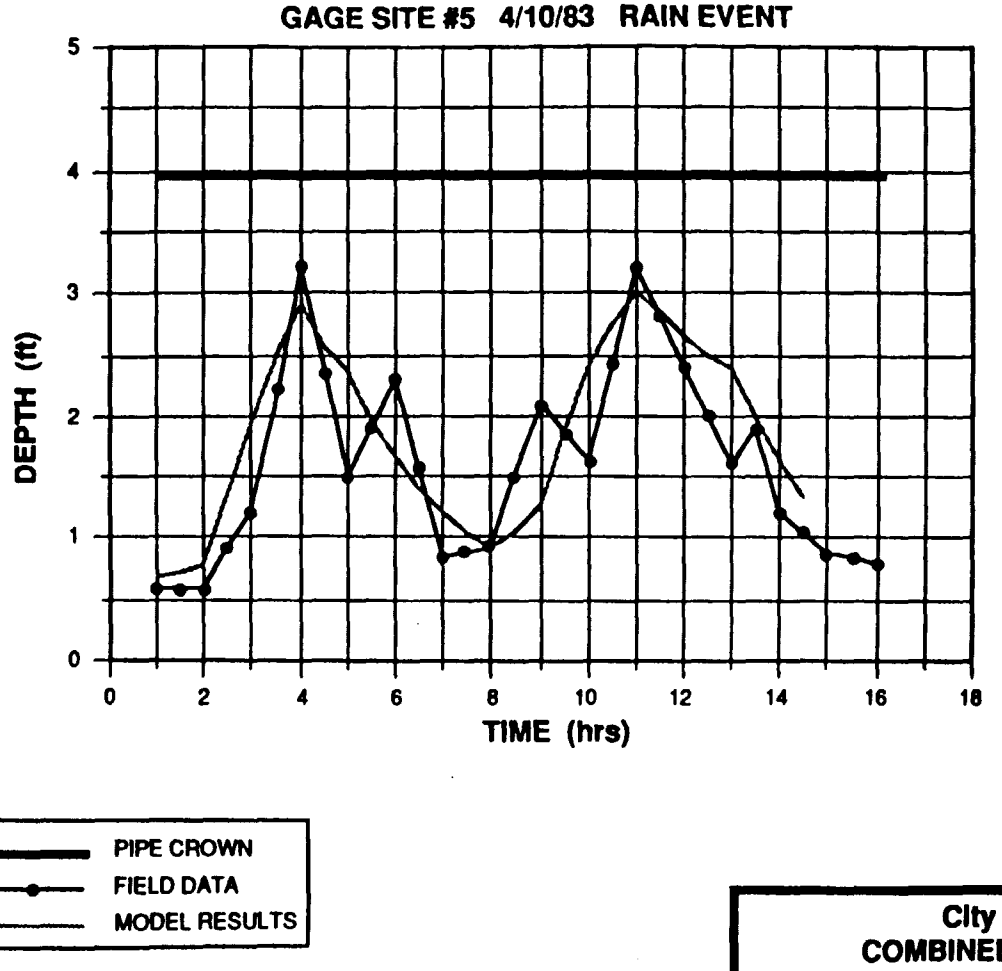
City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-9
SWMM CALIBRATION RESULTS:
COMPARISON OF SIMULATED VS.
FIELD-MEASURED DEPTHS AT GAGING SITE 3



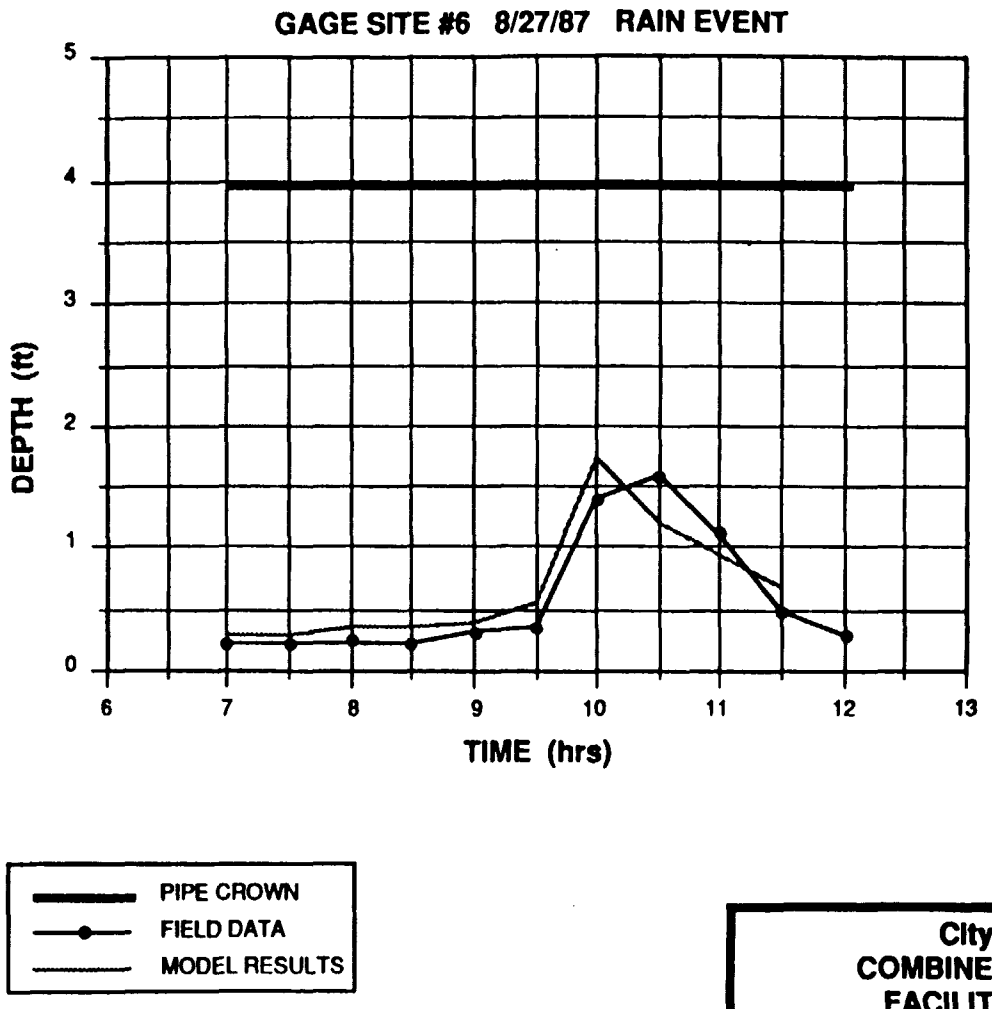
City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-10
SWMM CALIBRATION RESULTS:
COMPARISON OF SIMULATED VS.
FIELD-MEASURED DEPTHS AT GAGING SITE 4



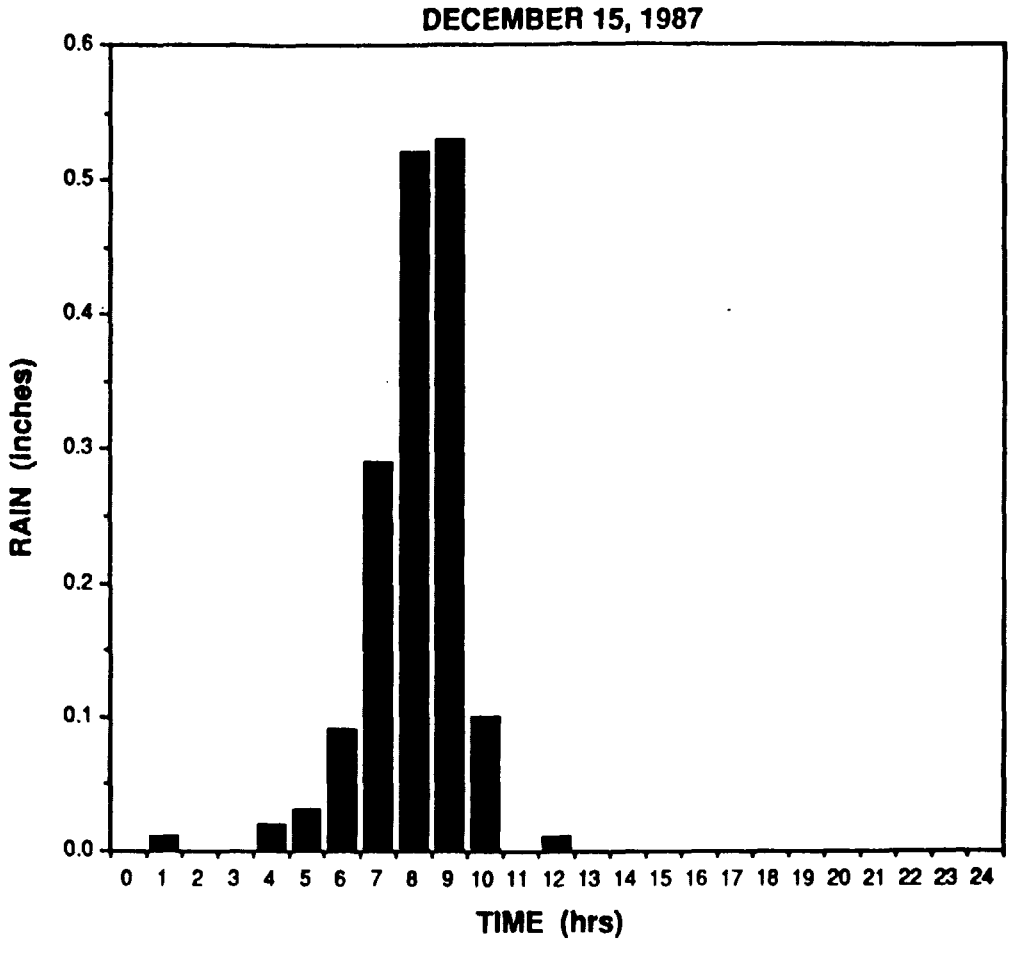
City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-11
SWMM CALIBRATION RESULTS:
COMPARISON OF SIMULATED VS. FIELD-MEASURED DEPTHS AT GAGING SITE 5



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-12
SWMM CALIBRATION RESULTS:
COMPARISON OF SIMULATED VS.
FIELD-MEASURED DEPTHS AT GAGING SITE 6



City Of New Bedford
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FACILITIES PLAN - PHASE 3

FIGURE 4-13
RAINFALL HYETOGRAPH FOR VERIFICATION EVENT

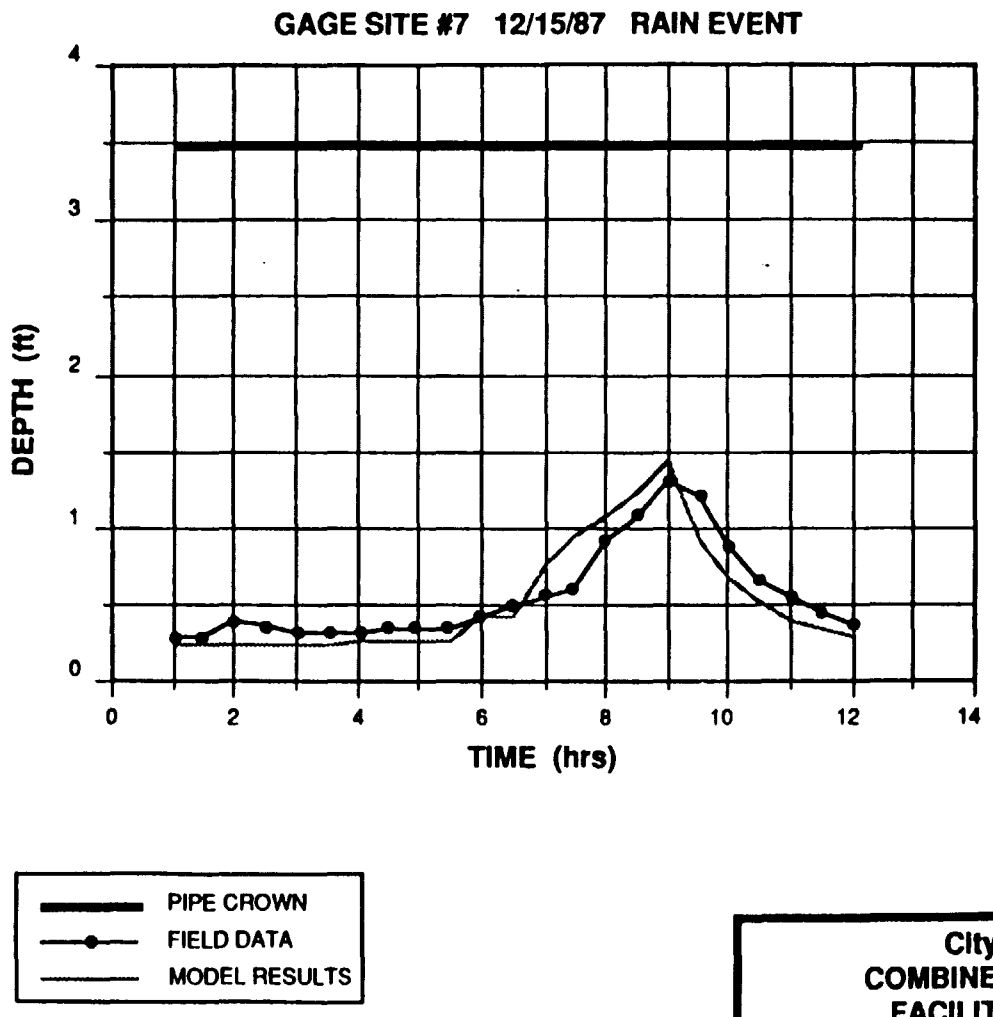

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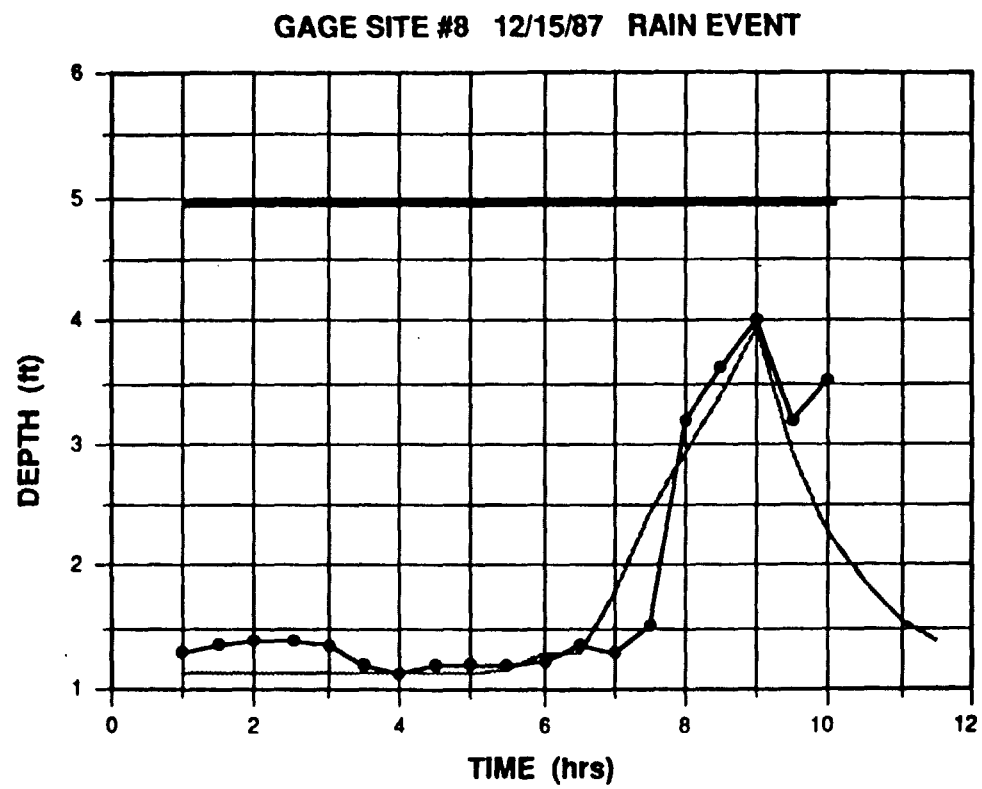
FIGURE 4-14
SWMM VERIFICATION RESULTS:
COMPARISON OF SIMULATED VS.
FIELD-MEASURED DEPTHS AT GAGING SITE 6



PIPE CROWN
FIELD DATA
MODEL RESULTS

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FIGURE 4-15
SWMM VERIFICATION RESULTS:
COMPARISON OF SIMULATED VS. FIELD-MEASURED DEPTHS AT GAGING SITE 8

Flow routing in EXTRAN was evaluated with the gaging at site 9, at the intersection of Belleville Avenue and Sylvia Street, and at site 10, on Bonney Street between Cove Road and Rivet Street.

Site 9 provided data for the Belleville Avenue Interceptor. The flow up to this point had two opportunities to overflow, at regulators 27B and 27C. The comparison of field and simulated flows is shown in Figure 4-16. The maximum range of depth that the gage could measure was exceeded at 7.5 hours, indicating surcharging, which was found in the model as well.

Finally, site 10 provided a measure of overflow volume for one of the 004 regulators -- the largest CSO in the system. Figure 4-17 illustrates the comparison between field and model depths at this site for the 12/15/87 storm. There is flow in the pipe before the storm because of a current dry weather discharge from regulator 4D.

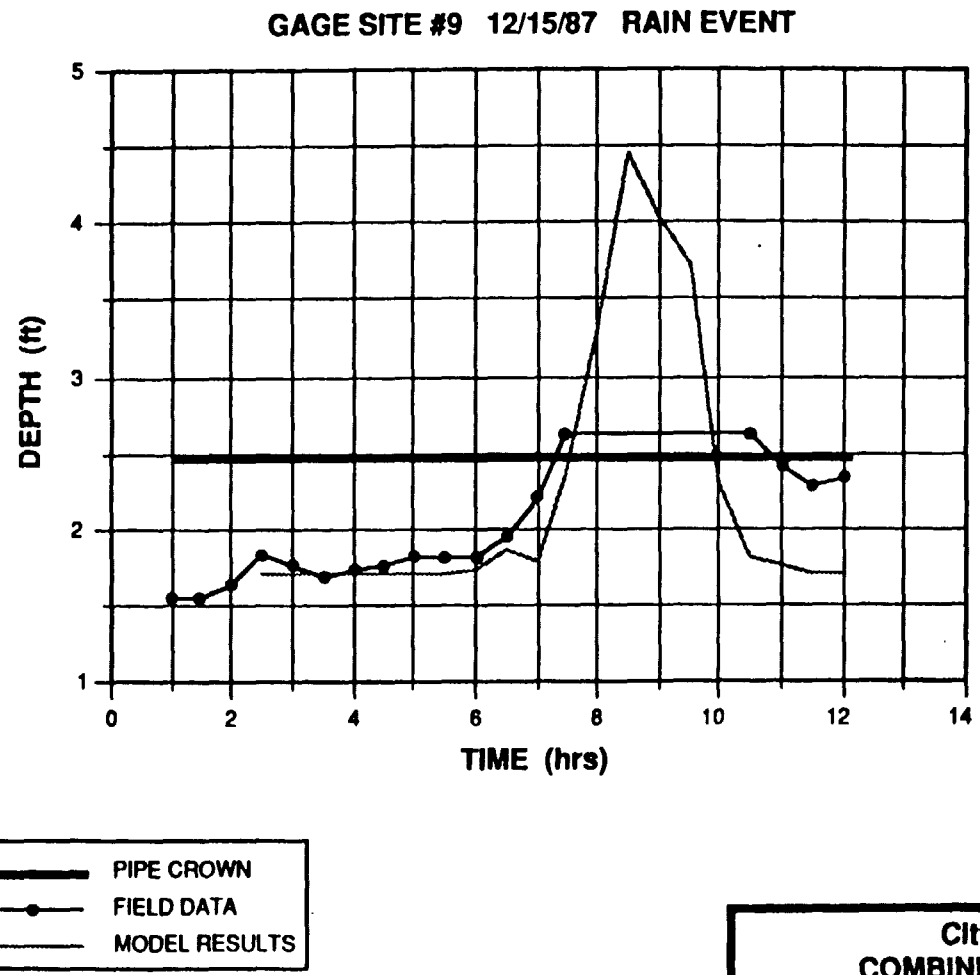
With the SWMM model calibrated and verified, the next step was to run the model for a series of design storms.

4.4.6 DESIGN STORMS

Design Storm Selection

The modeling of design storm events provides the basis for the comparison of the benefits and costs associated with alternative abatement strategies. The response of the sewer system is a function of a storm's magnitude, duration, and intensity. By reviewing the effectiveness of a given strategy under a variety of conditions we can accurately estimate the resulting costs and benefits.

The design events are actual storms selected from a 30-year hourly rainfall record (1950 to 1979) at the New Bedford gaging station (gage number 195246 from the National Climatic Data Center). Eight events were evaluated initially, each defined as an event equaled or exceeded, on the average:



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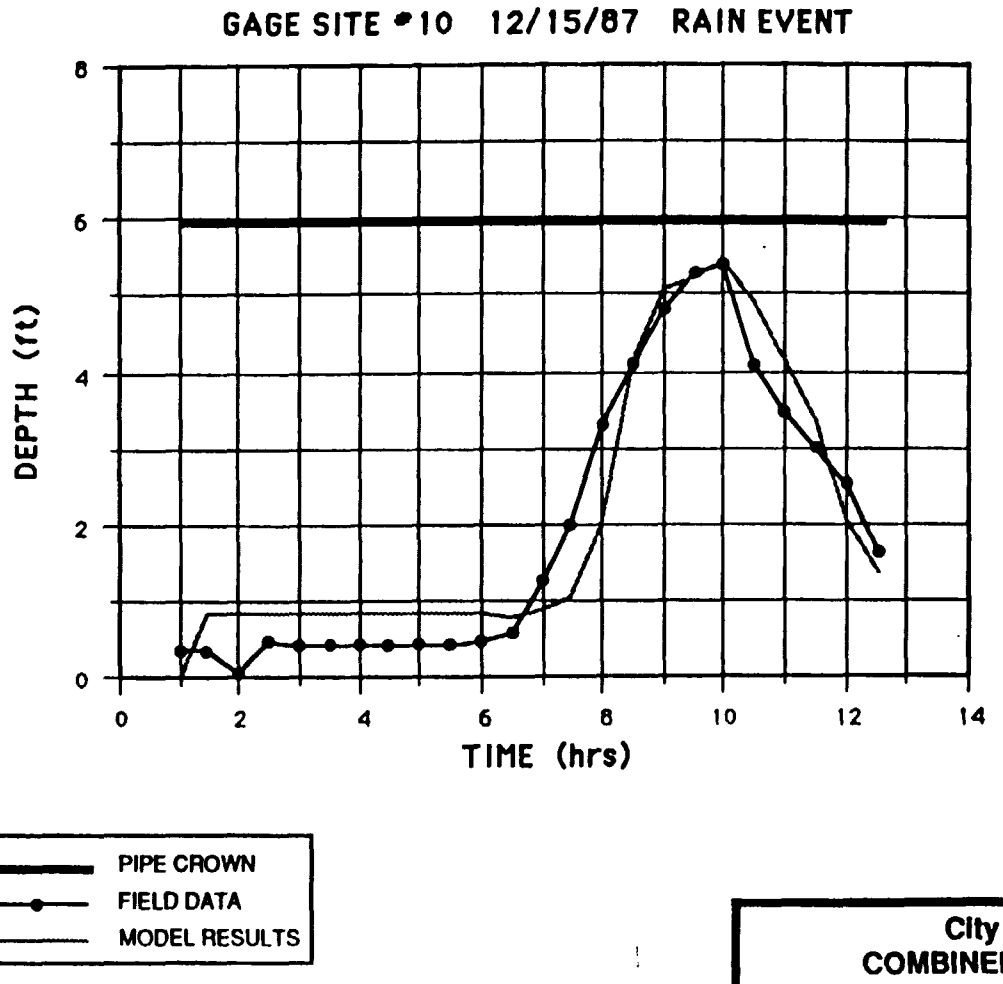
FACILITIES PLAN - PHASE 3

FIGURE 4-16
SWMM CALIBRATION RESULTS:
COMPARISON OF SIMULATED VS. FIELD-MEASURED DEPTHS AT GAGING SITE 9



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 4-17
SWMM VERIFICATION RESULTS:
COMPARISON OF SIMULATED VS. FIELD-MEASURED DEPTHS AT GAGING SITE 10

- 1-week event, exceeded 52 times a year;
- 2-week event, exceeded 26 times a year;
- 1-month event, exceeded 12 times a year;
- 3-month event, exceeded 4 times a year;
- 6-month event, exceeded twice a year;
- 1-year event, exceeded once every year;
- 2-year event, exceeded once every 2 years; and
- 5-year event, exceeded once every 5 years.

Given these definitions, each storm event in the 30-year record was identified by an interval of rainfall bounded before and after with at least 4 hours of no precipitation. Using this technique, 3280 events were identified over the 30-year period and ranked by volume. A probability of an event volume being equaled or exceeded by any other event, also called the exceedance probability, was then established for each event. The 5-year event, for example, would theoretically be equaled or exceeded six times in a 30-year record, and would have an associated exceedance probability of 0.0018 (6/3280) using the conventional CSO rainfall analysis.

This method of design storm selection is not the same as an annual series analysis, which uses the maximum annual events from a long-term record to estimate the recurrence interval of an extreme event. The exceedance probability for the 5-year event using annual series would be 0.20 (1/5) for example, rather than the 0.0018 calculated above. The annual series analysis is less appropriate, because the critical events in a CSO study typically have return periods of less than one year. Therefore, the conventional method uses <u>all</u> of the rainfall events in the analysis.

The exceedance probability for each design event is listed in Table 4-10, and is unique to the selected rainfall record length and the definition of an event (i.e., bounded by four hours of no precipitation). The exceedance probability associated with design events is not the same as the probabi

TABLE 4-10

DESIGN EVENT EXCEEDANCE PROBABILITIES

Design Event	Exceedance Probability
Five-year	0.0018
Two-year	0.0046
One-year	0.0091
Six-month	0.0183
Three-month	0.0366
One-month	0.1097
Two-week	0.2377
One-week	0.4755

lity of an overflow. For example, some overflows will discharge in a 3-month event and some will not, depending on the configuration of the regulator and other system parameters.

Rainfall events with exceedance probabilities close to a given design event were examined according to maximum intensity and duration. Given two events of the same volume of rainfall, the New Bedford combined sewer system responds with greater overflow volumes due to a short, intense storm than from a longer storm with lower maximum intensities. Therefore, storms with higher maximum intensities, as close to the exceedance probability as possible, were selected as design events. For example, the volume of rainfall in a 5-year event is 4.57 inches. Two rainfall events were considered, one with a volume of 4.57 inches and a maximum intensity of 1.25 in/hr, and the other with a volume of 4.55 inches and a maximum intensity of 0.55 in/hr. The first was selected for the 5-year design storm. Table 4-11 shows each of the design events, their associated volumes, and maximum and average intensities. Figures 4-18 and 4-19 illustrate the design storm hyetographs.

Application to the New Bedford System

To establish baseline conditions for the comparison of alternative strategies, the SWMM model was run for each of the design events for existing conditions and for the proposed improvements to the WWTP. The improved system, including upgraded pump stations, additional WWTP capacity, secondary treatment and increased pipe capacity is referred to as the "base condition." Table 4-12 gives a complete summary of overflow volumes for each of the eight design events under existing conditions. Rankings of the CSOs by overflow volume for the 1-month, 3-month and 1-year events are shown in Table 4-13. Among these events, outfall 004, located along Cove Road, dominates the volume of overflow, representing from 30 to 44 percent of the total volume; outfall 022, near Sawyer Street, follows with 12 to 14 percent of the total volume. Minimizing the overflows from these two outfalls alone would reduce the volume of CSO entering the receiving waters from 45 to 56 percent. The next six CSOs in the ranking, 020, 023, 030, 031, 036, and 040, are consistently ranked in the top eight

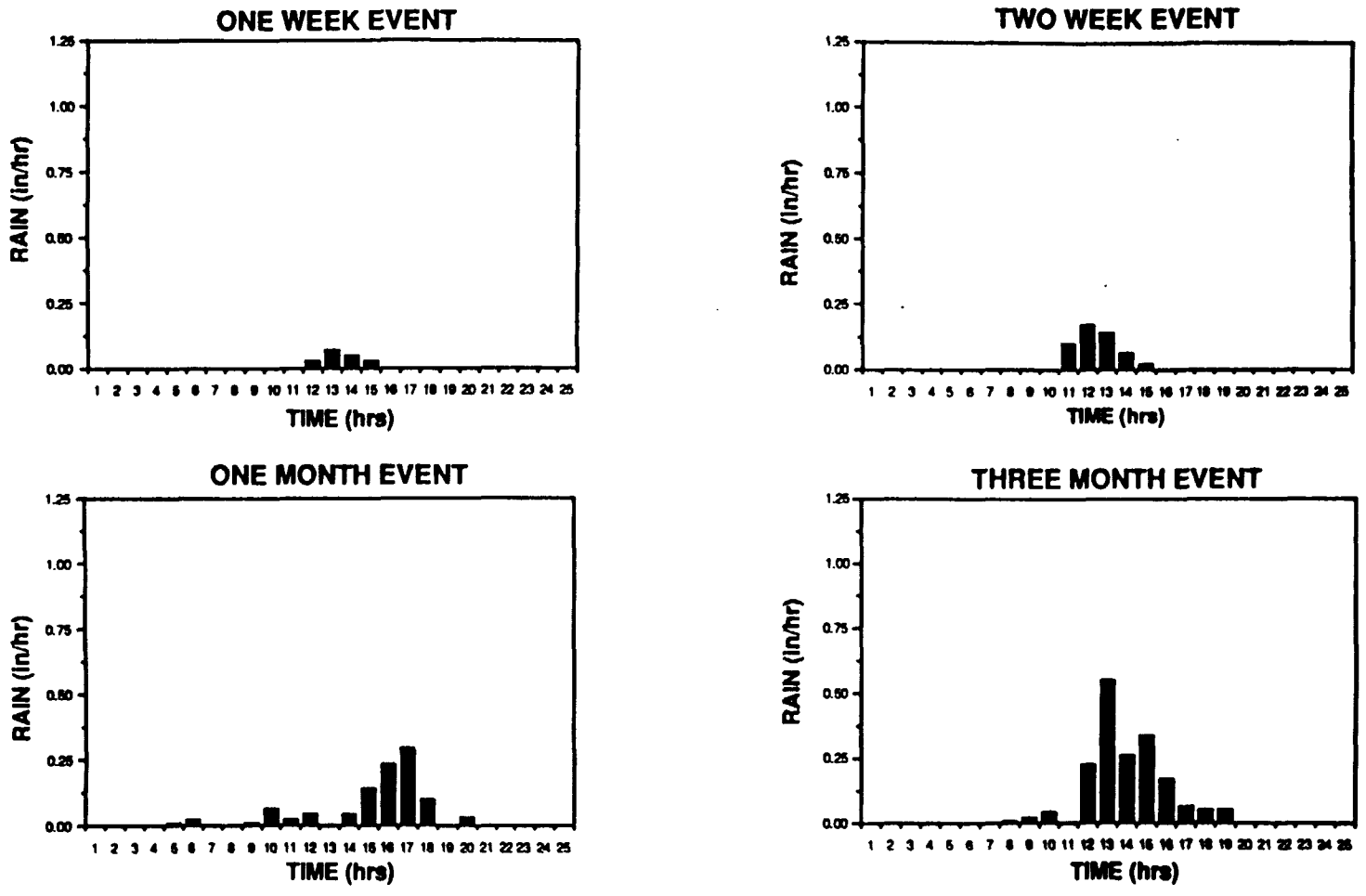


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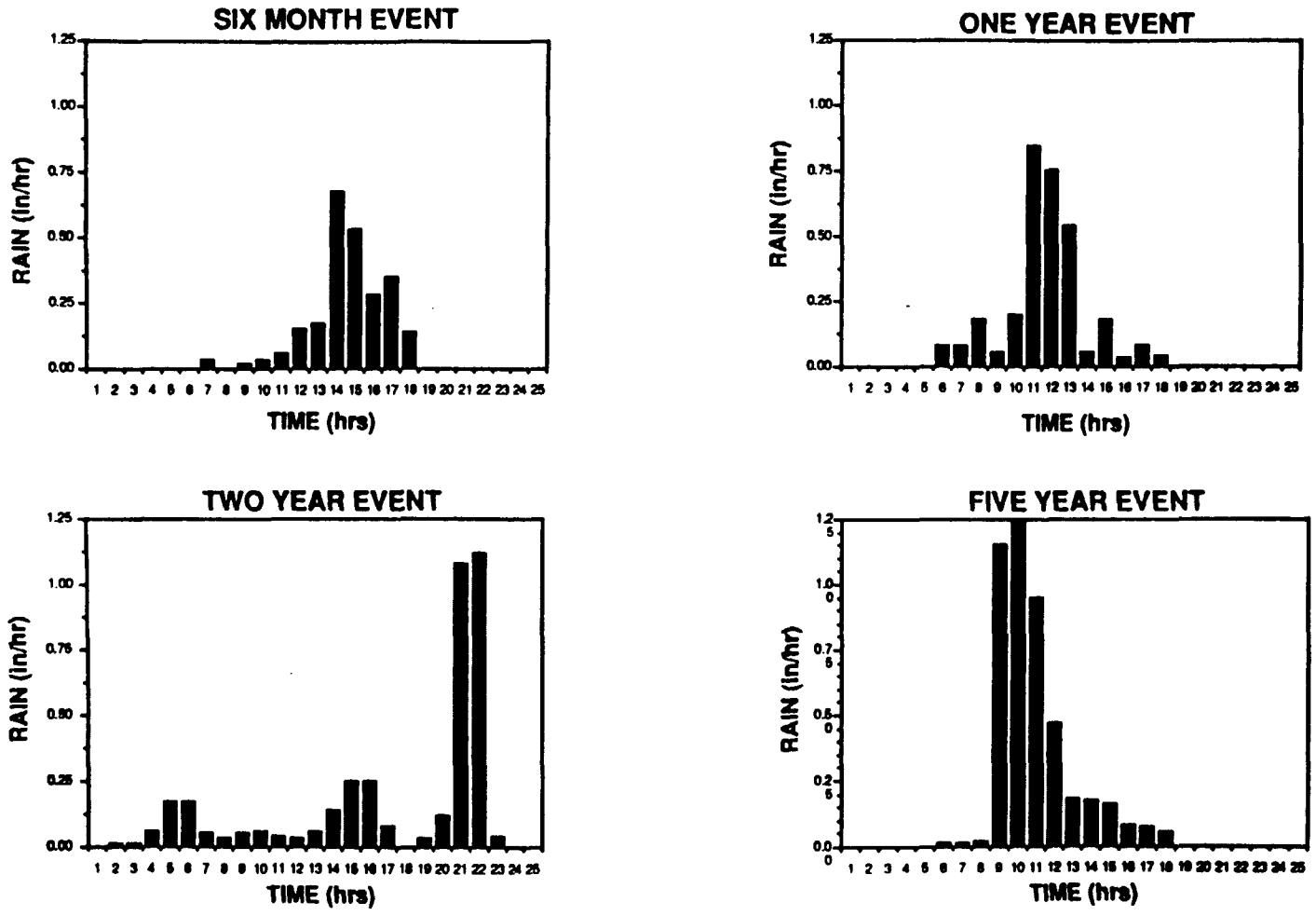
FIGURE 4-18
RAINFALL HYETOGRAPHS FOR DESIGN
EVENTS: 1 & 2 WEEK, AND 1 & 3 MONTH



City Of New Bedford
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FACILITIES PLAN - PHASE 3

FIGURE 4-19
RAINFALL HYETOGRAPHS FOR DESIGN
EVENTS: 6 MONTH, 1, 2 AND 5 YEAR

TABLE 4-11

DESIGN STORM PARAMETERS

Design Event	Date	Volume (inches)	Duration (hrs)	Maximum Intensity (in/hr)	Average Intensity (in/hr)
5-year	6/19/72	4.57	13	1.25	0.35
2-year	11/2/69	3.85	19	1.15	0.20
1-year	5/15/54	3.10	13	0.84	0.24
6-month	10/2/69	2.44	12	0.68	0.20
3-month	6/17/72	1.76	12	0.55	0.15
1-month	4/2/79	0.99	16	0.29	0.06
2-week	7/6/58	0.49	5	0.17	0.10
1-week	11/21/59	0.18	4	0.07	0.05

TABLE 4-12

COMBINED SEWER OVERFLOWS DUE TO DESIGN EVENTS

Outfall Number	OVERFLOW FROM DESIGN EVENTS (IN MG)							
	One-Week	Two-Week	One-Month	Three-Month	Six-Month	One-Year	Two-Year	Five-Year
003	0.00	0.00	0.01	0.06	0.28	0.54	0.97	1.21
004	0.30	3.96	8.59	23.12	31.28	37.73	47.29	53.77
005	0.00	0.00	0.00	0.01	0.36	0.79	1.08	1.85
006	0.00	0.00	0.02	0.33	0.47	0.74	1.08	1.46
007	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.36
008	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.14
009	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.08
010	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.04
011	0.00	0.00	0.00	0.03	0.31	0.50	0.76	1.15
012	0.01	0.04	0.09	0.20	0.35	0.48	0.64	0.89
013	0.00	0.00	0.01	0.02	0.02	0.03	0.04	0.04
014	0.01	0.05	0.10	0.22	0.38	0.53	0.71	1.00
015	0.00	0.02	0.03	0.06	0.09	0.11	0.14	0.17
016	0.00	0.00	0.01	0.01	0.02	0.02	0.03	0.03
017	0.00	0.11	0.28	0.68	1.13	1.62	2.04	2.27
018	0.00	0.08	0.17	0.58	0.90	1.43	1.91	2.73
020	0.62	1.23	2.56	4.03	6.29	8.15	9.85	9.96
021	0.00	0.00	0.03	0.14	0.27	0.34	0.49	0.75
022	0.00	1.05	2.63	6.72	11.22	15.30	19.07	27.59
023	0.17	0.70	1.46	2.77	4.35	5.86	7.48	9.82
024	0.00	0.06	0.18	0.51	0.83	0.99	1.30	1.63
025	0.04	0.16	0.33	0.68	1.04	1.40	1.80	2.36
026	0.03	0.11	0.23	0.45	0.69	0.92	1.17	1.52
027	0.01	0.04	0.08	0.17	0.27	0.39	0.57	0.97
028	0.02	0.08	0.17	0.37	0.68	1.01	1.37	2.15
030	0.00	0.12	0.55	2.06	2.84	2.98	3.77	4.25

TABLE 4-12 (continued)

Outfall Number	OVERFLOW FROM DESIGN EVENTS (IN MG)							
	One-Week	Two-Week	One-Month	Three-Month	Six-Month	One-Year	Two-Year	Five-Year
031	0.00	0.09	0.53	4.34	8.80	9.91	13.35	17.28
032	0.00	0.00	0.00	0.09	0.36	0.98	1.35	2.70
033	0.00	0.00	0.00	0.02	0.05	0.10	0.14	0.26
034	0.00	0.00	0.01	0.14	0.31	0.50	0.71	1.18
035	0.00	0.00	0.00	0.00	0.02	0.07	0.11	0.30
036	0.00	0.19	0.73	2.90	5.16	7.25	9.21	13.34
037	0.04	0.14	0.28	0.57	0.89	1.19	1.53	1.99
040	0.00	0.33	0.82	2.12	3.47	4.66	6.11	8.61
041	0.00	0.10	0.27	0.76	1.19	1.58	2.10	2.66
TOTALS:	1.25	8.66	20.17	54.16	84.32	108.13	138.40	176.51

TABLE 4-13

RANKING OF COMBINED SEWER OVERFLOWS DURING THE 1-MONTH, 3-MONTH AND 1-YEAR DESIGN EVENTS BY VOLUME OF OVERFLOW

Ranking	1-month event	3-month event	1-year event
1	004	004	004
2	022	022	022
3	020	031	031
4	023	020	020
5	040	036	036
6	036	023	023
7	030	040	040
8	031	030	030
9	025	041	017
10	017, 037	025, 017	041
11	041	018	018
12	026	037	025
13	024	024	037
14	018, 028	026	028
15	014	028	024
16	012	006	032
17	027	014	026
18	021, 015	012	005
19	035, 016, 013	027	006
20		021, 034	014
21		032	011, 034
22		015	012
23		011	027
24		013, 033	021
25		005, 016	015
26			033
27			035
28			007, 013
29			016, 003

CSOs, and represent 30 to 36 percent of the volume of overflows. The top ten CSOs in any event have a combined volume equal to 86 to 90 percent of the total.

The outfalls, as shown in Figure 4-1, have been arranged into 6 subgroups according to receiving water and their proximity to each other. The groupings are:

Grouping	Outfalls	Receiving Water	Geographic Location
1	003-004	Clarks Cove	northern edge of Clarks Cove
2	005-011	Clarks Cove	west side of Clarks Point
3	012-018	Outer Harbor	east side of Clarks Point
4	028 030-034	Inner Harbor	between the hurricane barrier and Pope's Island
5	020-021 035-037	Inner Harbor	between Pope's Island and the Coggeshall Street bridge
6	022-027 040-041	Inner Harbor	between the Coggeshall Street bridge and Tarklin Hill Road

The overflows from these groups during the design events are shown graphically in Figure 4-20. Note that CSOs 029, 038 and 039 are not included because the network of pipes leading to each is now separated, therefore only storm flows are currently discharged at these outlets.

Group 1 dominates the overflows to Clarks Cove. Groups 2 and 3 have comparatively minor volumes of overflow. The Inner Harbor receives significant volumes of CSOs from three groups (4, 5, and 6) with Group 6 dominating. These conditions are the baseline, against which potential system improvements are compared.

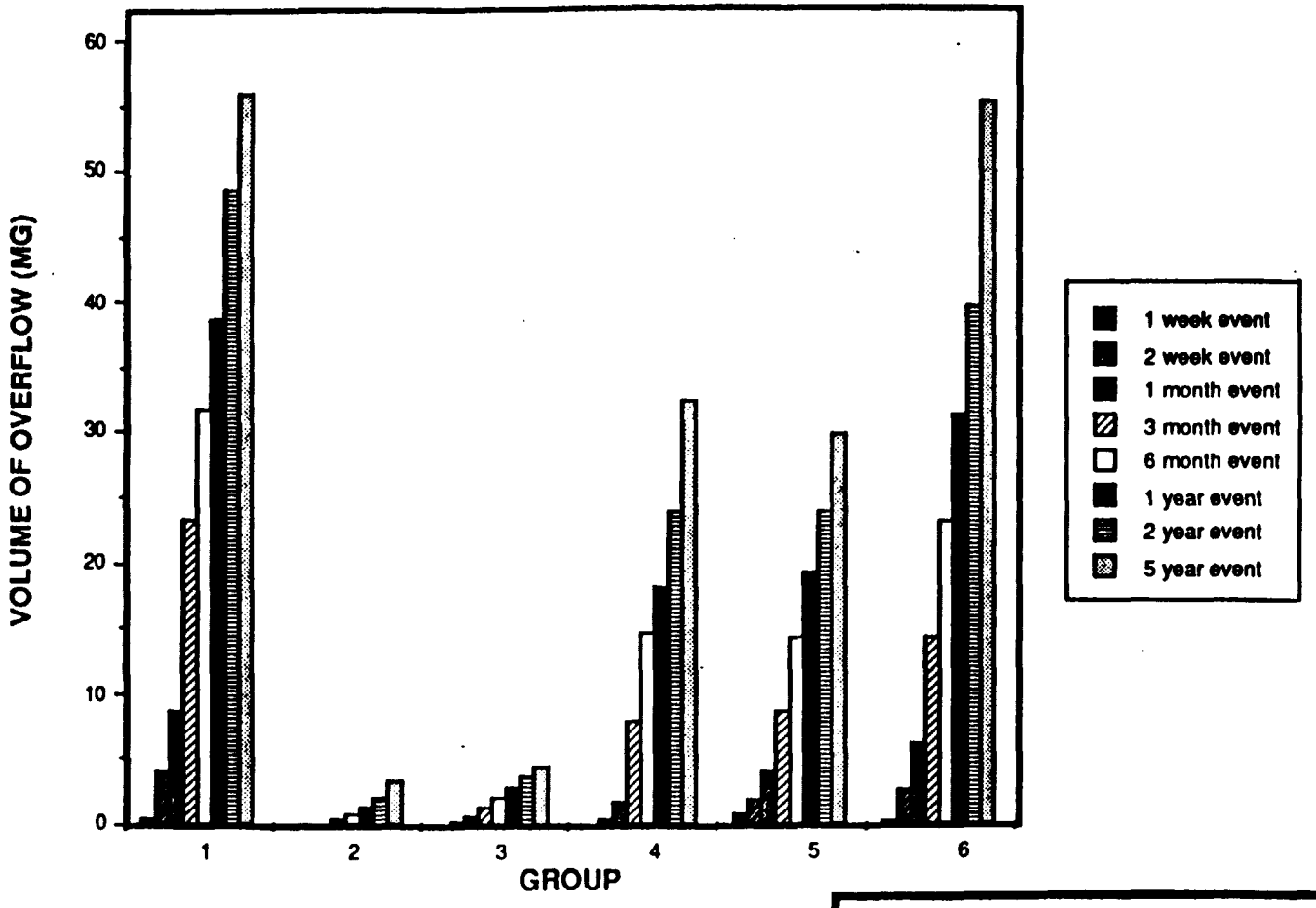

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FIGURE 4-20
RELATIVE VOLUMES OF CSOs FOR SIX MAJOR OUTFALL GROUPS DURING THE DESIGN STORMS

Estimates of flows from storm drains in New Bedford, Acushnet, Dartmouth and Fairhaven were made based on drainage area and land use characteristics. Table 4-14 compares CSO and storm drain volumes for both the 3-month and 1-year design events. Storm drain flows contribute 60 percent of the total storm-related flows to the receiving waters.

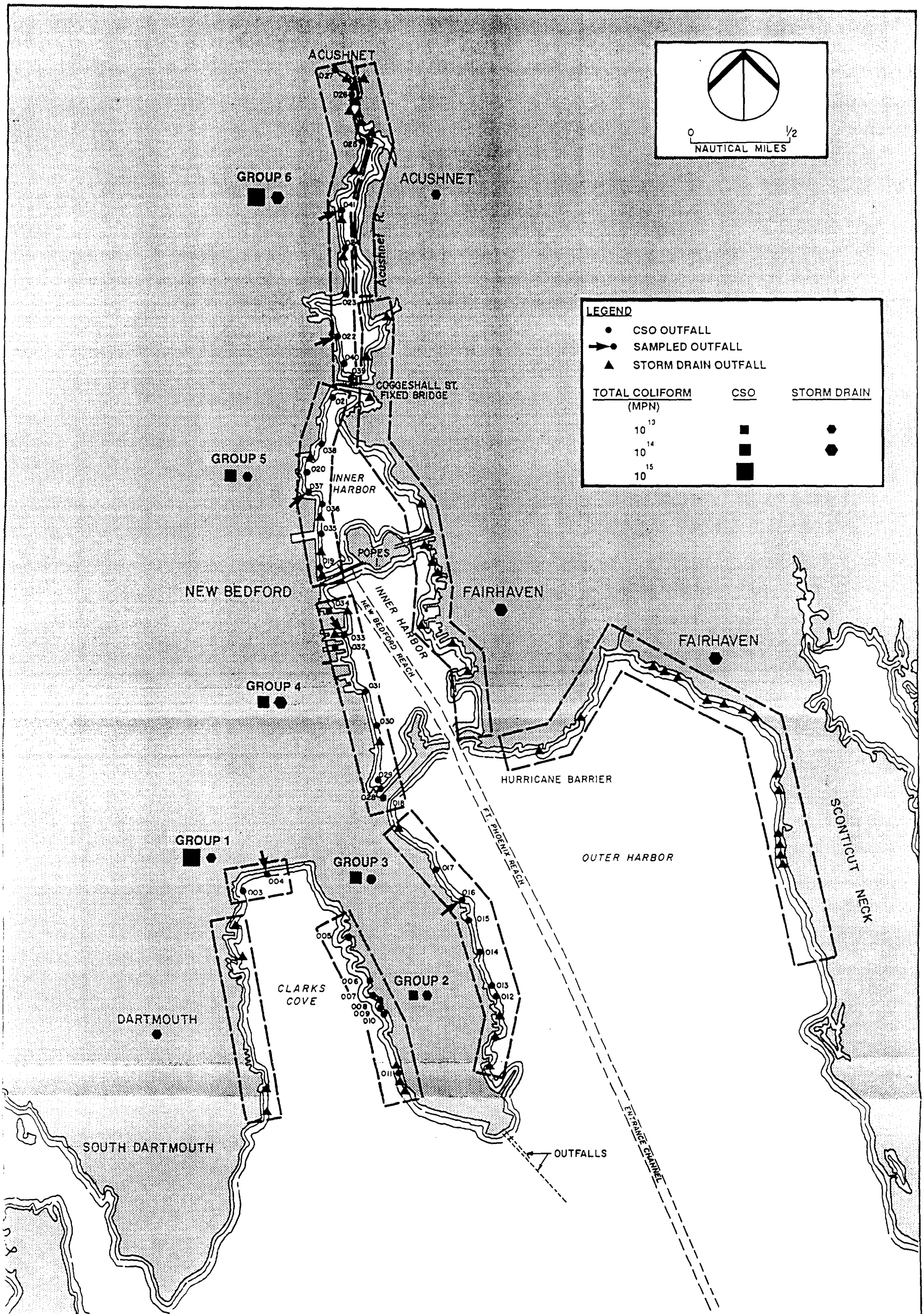
To gain some perspective on the impact on receiving waters from CSOs, and for later receiving water modeling, pollutant loads for both CSOs and storm drains were estimated on an event basis. Using the mean pollutant values presented in Table 4-6 and the flows from Table 4-14, pollutant loads for the 3-month and 1-year events were calculated by receiving water (see Table 4-15). The loads from storm drains are generally greater than CSO loads in the Inner and Outer Harbors (except for BOD, TKN, ammonia and coliform bacteria) primarily because of the significantly larger volume of flow from storm drains. CSO loads dominate in Clarks Cove for all pollutants, although the loads from storm drains alone are significant. A graphical example of the relative loads from CSOs and storm drains for existing conditions is shown for total coliform bacteria from a 3-month event in Figure 4-21.

The recommended plan for the WWTP developed by the Phase 2 Facilities Plan (known as the base condition) substantially reduces the volume of overflows, as shown by the comparison of existing and base conditions for the 3-month event in Figure 4-22. The volumes are reduced primarily in CSO Groups 1, 5 and 6, with almost no effect occurring in Group 4. The full effect of these improvements is illustrated in the annual results discussed in Section 4.6.

4.5 ANNUAL LOADINGS FROM CSOs AND STORM DRAINS

4.5.1 OBJECTIVES

The SWMM model provides a means to analyze the hydrologic and hydraulic components of the New Bedford sewer system on an event basis. A critical part of the CSO problem though, is the cumulative effect CSO discharges have on the receiving waters over long periods of rainfall record. STORM



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Total Coliform Loads from a Three Month Design Storm for CSOs and Storm Drains

CDM

FIGURE 4 - 21

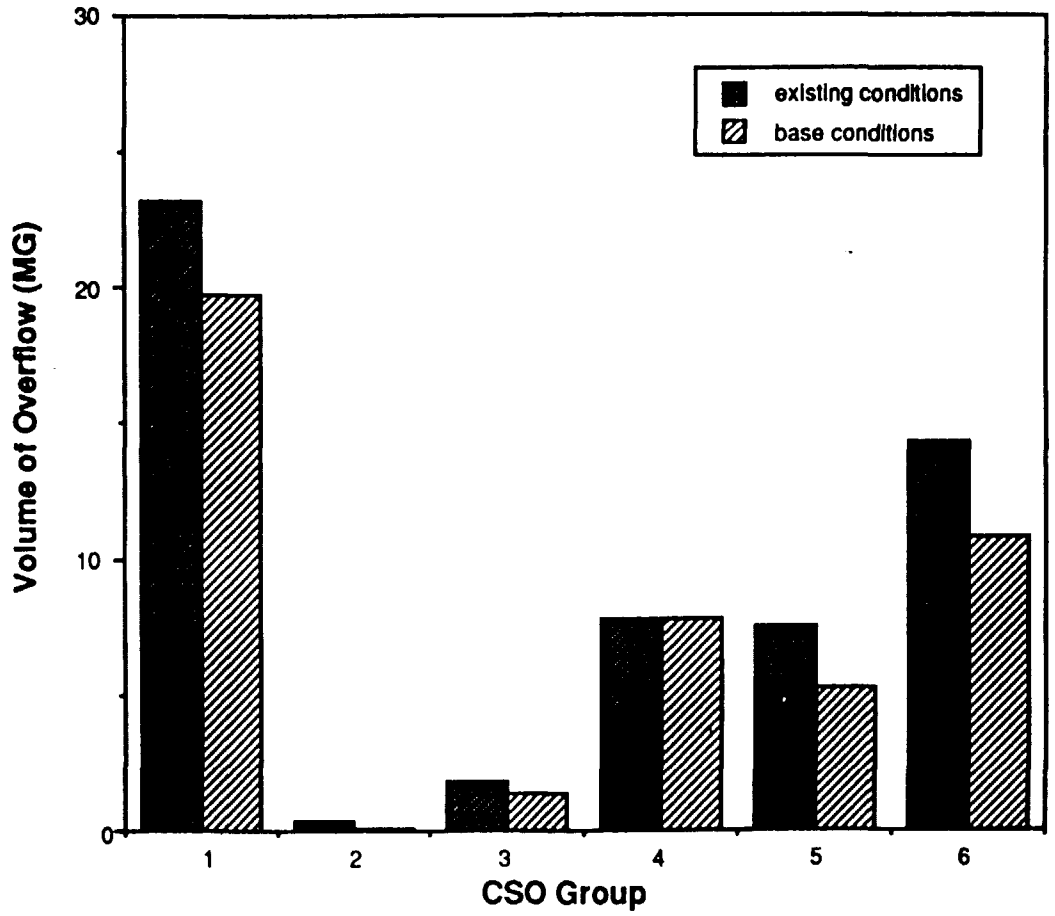

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FIGURE 4-22
COMPARISON OF CSO VOLUMES
UNDER EXISTING AND BASE CONDITIONS
FOR THE 3-MONTH DESIGN EVENT

TABLE 4-14

COMPARISON OF CSO AND STORM DRAIN VOLUMES ON AN EVENT BASIS

Volumes in million gallons

Receiving Water City/Grouping		3-month event CSOs	3-month event Storm Drains	1-year event CSOs	1-year event Storm Drains
Inner Harbor					
New Bedford	Group VI	14.32	23.60	31.44	50.87
New Bedford	Group V	7.50	8.25	16.66	13.08
New Bedford	Group IV	7.02	8.79	15.48	18.88
Acushnet		--	2.62	--	5.73
Fairhaven		--	25.08	--	54.83
Subtotal		28.84	68.34	63.58	143.39
Outer Harbor					
New Bedford	Group III	1.77	4.01	4.22	8.85
Fairhaven		--	9.00	--	19.71
Subtotal		1.77	13.01	4.22	28.56
Clarks Cove					
New Bedford	Group II	0.37	1.85	2.06	4.12
New Bedford	Group I	23.18	8.91	38.27	18.87
Dartmouth		--	4.89	--	10.70
Subtotal		23.55	15.65	40.33	33.69
Total		54.16	97.00	108.13	205.64

TABLE 4-15

CSO AND STORM DRAIN POLLUTANT LOADS BY RECEIVING WATER FOR TWO DESIGN EVENTS

Pollutant	Receiving Water	3-Month Event CSO	3-Month Event Storm Drain	1-Year Event CSO	1-Year Event Storm Drain
TSS (lbs)					
	Inner Harbor	19254	54180	42447	113680
	Outer Harber	1181	10314	2818	22643
	Clarks Cove	15772	12408	26925	26710
BOD (lbs)					
	Inner Harbor	12756	11977	28122	25129
	Outer Harbor	782	2280	1866	5005
	Clarks Cove	10417	2743	17838	5905
Ammonia (lbs)					
	Inner Harbor	352	160	775	335
	Outer Harbor	21	30	51	67
	Clarks Cove	287	37	491	78
TKN (lbs)					
	Inner Harbor	1616	1209	3560	2537
	Outer Harbor	100	231	236	505
	Clarks Cove	1318	277	2258	597
Lead (lbs)					
	Inner Harbor	26.5	102.6	58.4	215.3
	Outer Harbor	1.6	19.5	3.9	42.9
	Clarks Cove	21.7	23.5	37.0	50.7
Zinc (lbs)					
	Inner Harbor	84.3	199.6	185.3	418.8
	Outer Harbor	5.2	38.0	12.3	83.4
	Clarks Cove	68.8	45.7	117.8	98.4
Chromium (lbs)					
	Inner Harbor	19.2	34.2	42.4	71.8
	Outer Harbor	1.2	6.5	2.9	14.3
	Clarks Cove	15.7	7.8	26.9	16.9
Copper (lbs)					
	Inner Harbor	26.5	45.6	58.4	95.7
	Outer Harbor	1.6	8.7	3.9	19.0
	Clarks Cove	21.7	10.5	37.0	22.4
Total Coliform (MPN)					
	Inner Harbor	$1.71*10^{15}$	$3.82*10^{14}$	$3.76*10^{15}$	$8.03*10^{14}$
	Outer Harbor	$1.05*10^{14}$	$7.28*10^{13}$	$2.50*10^{14}$	$1.60*10^{14}$
	Clarks Cove	$1.39*10^{15}$	$8.76*10^{13}$	$2.38*10^{15}$	$1.88*10^{14}$

is a continuous simulation model that has the capability to model CSO discharges occurring over the course of a year or longer. Using STORM, long-term planning level observations about CSO frequency, volume, and pollutant loadings to receiving waters can be made.

4.5.2 DESCRIPTION OF STORM MODEL

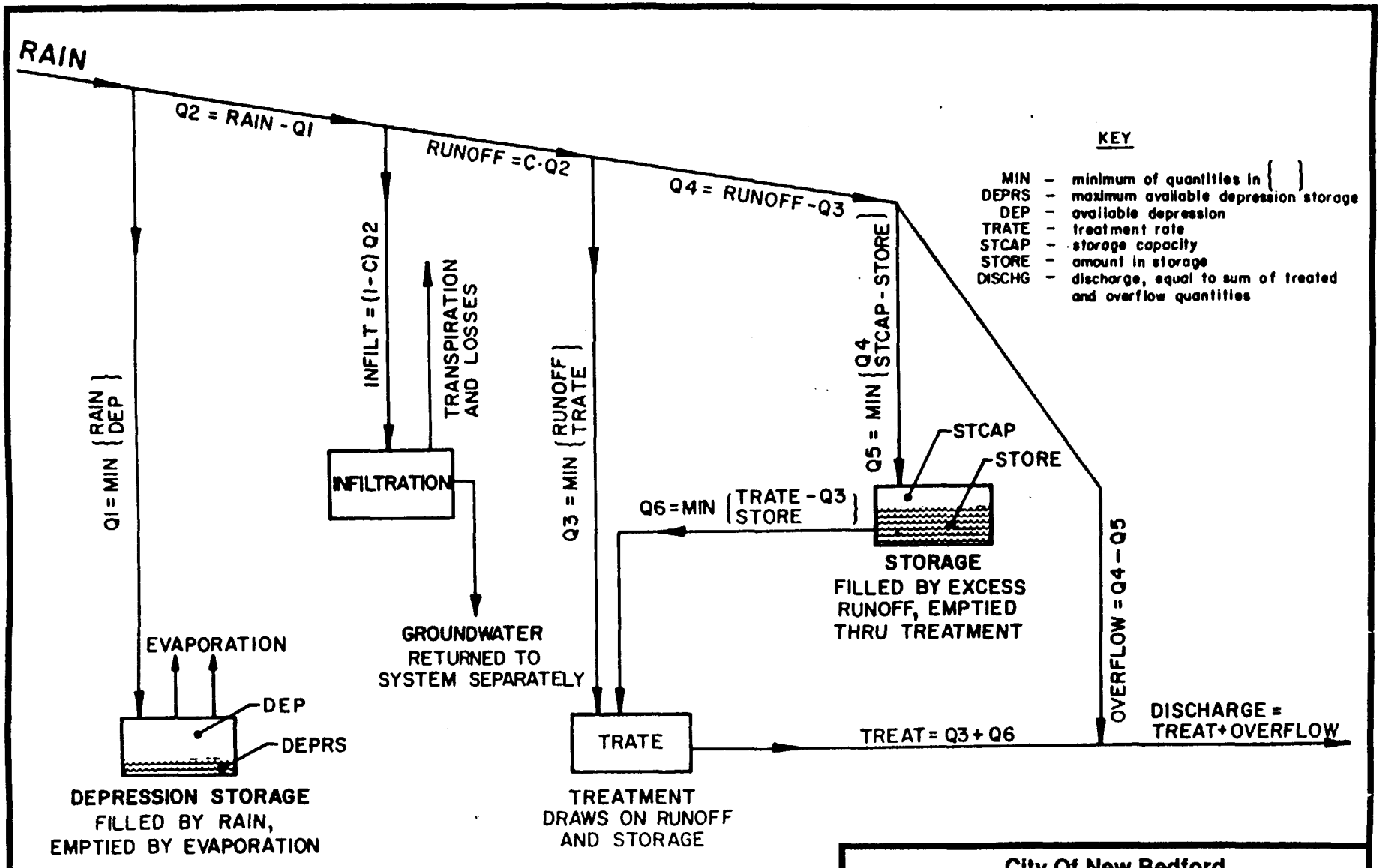
STORM continuously simulates overflow quantity and quality using surface runoff and land use-pollutant relationships. STORM simulates runoff using the modified rational method, which requires an hourly rainfall record, monthly evaporation rates, depression storages, and runoff coefficients for each subbasin. As in the RUNOFF block of SWMM, subbasins in the study area were modeled individually. Figure 4-23 shows the conceptual flow chart of the STORM model hydrology.

Overflow quantities are calculated from the interaction of the storage and treatment capacities of the sewer network, and the volumes of runoff entering the system. Treatment capacities (pipe conveyance capacities) were determined from SWMM. Overflow quantity is determined from the assumption that storm runoff and sanitary flows are completely mixed, unless the system is separated.

Output from STORM is primarily in tabular form, containing hourly to yearly summaries of overflow, treatment, and runoff volumes. The model also contains an algorithm that analyzes frequency of overflow, treatment, or runoff events. Output includes a table and a plot of the specified frequency distribution.

4.5.3 APPLICATION TO STUDY AREA

Traditionally, STORM has been used to model individual drainage basins with one treatment rate specified for the entire drainage area. In order to simulate the entire New Bedford wastewater collection system, a number of modifications were made to the STORM computer code. These changes were aimed at capturing interaction between subbasins where surcharging and backwater conditions occur. Specifically, the modifications allow the user



NOTE: EXCLUDING SANITARY FLOW

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FIGURE 4-23
SCHEMATIC OF CDM-STORM HYDROLOLGY

to save hourly records of simulated treatment and overflow quantities, and then use them as input to down-gradient runs.

The complexity of the New Bedford wastewater collection system necessitated multiple runs with STORM. In most cases, the combined sanitary and storm flows are given more than one opportunity to overflow between the time they enter the system and the time they reach the WWTP. Since only one treatment rate can be assigned within each basin, the tributary areas were broken down into smaller subcatchments, each with an associated treatment rate. Using the modified version of the model, the treated component (which does not overflow) was added to the inflow in the subsequent down-gradient run. Thus, the complete collection system was modeled by sequentially stepping through the pipe network. The wastewater that did not leave the system through overflows was accounted for by saving output time histories from previous runs.

Treatment capacities were determined from SWMM output. In order to accurately simulate frequencies of overflows, treatment rates were assigned to match flows in SWMM at the point where overflows just begin to occur. Minor adjustments were made to account for increased flows that occur during surcharge events.

4.5.4 CALIBRATION

Fundamental to simulations using STORM is the quantity of flow entering the system. STORM was calibrated for the individual subbasins by adjusting the runoff coefficients until the runoff matched what was predicted for the same area by the RUNOFF subroutine in SWMM. This was done for several rain events that range statistically from a 3-month event to a 1-year event. Table 4-16 shows the runoff coefficients for two basins that produced the desired runoff quantities for six events. The range of values results from the fact that the models use different algorithms for predicting runoff. Specifically, RUNOFF in SWMM takes into account permeability. This means that seepage is proportional to rainfall intensity in RUNOFF, while in STORM it is only accounted for by the runoff coefficient or "C" value. The same "C" value that matches runoff for an intense storm will predict too

TABLE 4-16

RUNOFF COEFFICIENTS CALIBRATED TO VARIOUS DESIGN STORMS

Design Storm	Willis St. (Basin 15) "C" Value	Sawyer St. (Basin 11) "C" Value
3-month, duration	0.52	0.44
3-month, intensity	0.57*	0.47*
6-month, duration	0.51	0.43
6-month, intensity	0.63	0.50
1-year, duration	0.50	0.43
1-year, intensity	0.67	0.53
AVERAGE VALUE	0.57	0.47

*Denotes value selected for continuous simulations of historical rainfall.

much runoff in STORM for a less intense event. This occurs because the runoff algorithm allows a lower percentage of the rainwater to seep into the permeable area. The "C" value corresponding to the more intense 3-month event was chosen, as it is representative of the range of calculated runoff coefficients.

Depression storage, which accounts for puddling of rainwater, was determined directly from the input to SWMM. Area-weighted averages were calculated based on the value used in SWMM: 0.06 and 0.25 inches respectively for impermeable and permeable areas. Monthly evaporation rates were based on Class A Pan Evaporation Data for New Bedford contained in the Climatic Atlas of the United States.

Treatment rates were assigned based on computed conveyance capacities in SWMM. As explained in Section 4.5.3, these rates were chosen to match flows in connector pipes at the point when overflows begin to occur. In certain instances, this value was increased slightly to account for the fact that pipe flows increase above design capacity when they are surcharging.

Table 4-17 shows the overflow quantities predicted by STORM versus those predicted by SWMM for each CSO outfall during the 3-month intensity event. These are also shown graphically in Figure 4-24. Differences between the predicted values are due to differences between the models and practical limitations of STORM. First, STORM is not a hydraulics model, so it cannot capture routing effects and resulting differences in times of concentration between subbasins. By accounting for all rainfall within each hourly timestep instead of adding on the time it takes the water to traverse the watershed, STORM tends to increase peak inflows. A second and related point is that STORM only models a constant treatment rate, thus it cannot capture the variable nature of capacity that is calculated in SWMM when surcharging occurs. Finally, discrepancies can also result because STORM is unable to simulate backwater effects and other interaction between pipes and subbasins.

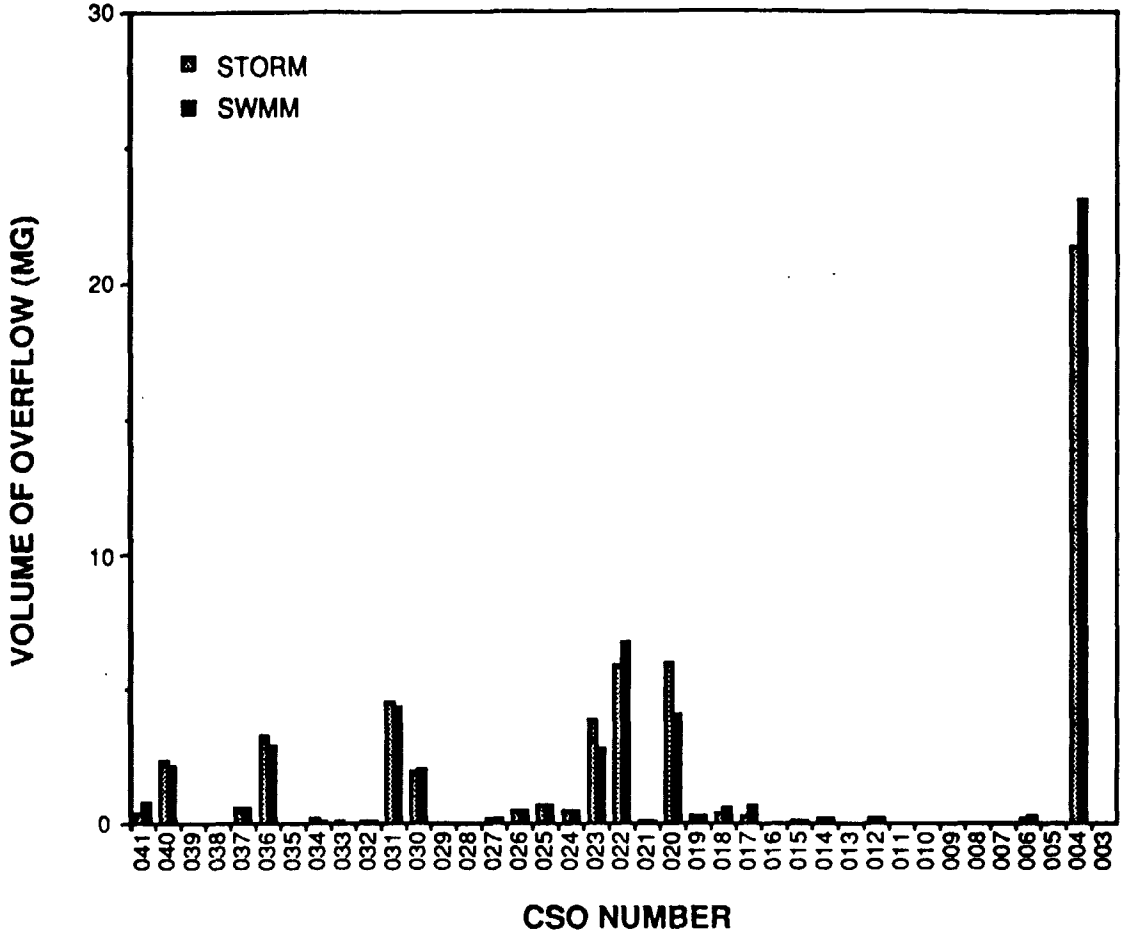

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FIGURE 4-24
COMPARISON BETWEEN SWMM AND
STORM RESULTS FOR
THE 3-MONTH DESIGN STORM

TABLE 4-17

COMPARISON OF SWMM AND STORM MODEL RESULTS FOR 3-MONTH DESIGN STORM

	OVERFLOW VOLUMES (MG)	
Outfall	SWMM	STORM
003	0.06	0.06
004	23.10	21.40
005	0.01	0
006	0.33	0.20
007	0	0
008	0	0
009	0	0
010	0	0
011	0.03	0
012	0.20	0.20
013	0.02	0.02
014	0.22	0.21
015	0.06	0.06
016	0.01	0.01
017	0.68	0.31
018	0.58	0.35
020	4.03	5.96
021	0.14	0.14
022	6.72	5.82
023	2.77	3.79
024	0.51	0.45
025	0.68	0.68
026	0.45	0.45
027	0.17	0.17
028	0.37	0.37
030	2.06	1.92
031	4.34	4.51
032	0.09	0.1
033	0.02	0.06
034	0.14	0.19
035	0	0
036	2.90	3.30
037	0.57	0.57
040	2.12	2.32
041	0.76	0.37
	54.16	53.93

In formulating the STORM input decks, priority was given to matching overflow quantities at the major outfalls. This ensured that the overall loadings to each of the three receiving waters (Inner Harbor, Outer Harbor, and Clarks Cove) were as accurate as possible. Table 4-17 confirms that the total overflow predicted by STORM is equivalent to what SWMM predicted.

4.6 FINDINGS

4.6.1 ANNUAL VOLUMES AND FREQUENCY OF CSO DISCHARGES

STORM provides a relatively quick method to simulate CSO discharges over long periods of time. This model was used to estimate average annual frequency and volume of CSO discharges at each of the 35 outfalls.

The rainfall database used for this study (the New Bedford gage from 1950 through 1988) was obtained from the National Climatic Center. One year of rainfall data was selected as input to the calibrated STORM model to simulate "average" annual discharge volumes and frequencies. The primary selection criterion for choosing a year representative of "normal" rainfall was comparing the average annual volume computed for the entire record to that of the individual year. The year 1968 has a total rainfall closest to the 38-year average and was therefore selected.

The validity of using one year of record to simulate "average" conditions was examined by conducting a simulation at a selected CSO location using six other years of record that most nearly matched the 38-year average annual rainfall. The results of this simulation showed that the computed frequency and volume of discharge for 1968 was very close to the average for the six other years. A further check was made to ensure that the selected record did not contain a disproportionate number of statistically large rain events that would skew the results.

The STORM results indicate that approximately 1,500 million gallons of runoff is generated annually in areas served by combined sewer systems. Out of this total, some 1,100 million gallons currently enter the receiving waters as overflows with the remainder being conveyed to the WWTP. The

discharges are divided roughly equally among the Inner Harbor areas and Clarks Cove, with a small portion going to the Outer Harbor. Table 4-18 shows a breakdown of the predicted volumes and frequency of discharge for each CSO outfall and totals for the three receiving waters. The total overflow is somewhat lower than the 1,500 million gallons of overflow predicted in the 1983 Phase 1 study. The difference is primarily due to a large area, in the upper reaches of the sewer system, which was previously modeled as combined but is now separated. Other small differences may result from assumptions regarding the conveyance capacities of regulators throughout the system. In this Phase 3 study, basins and structures were examined in more detail and the STORM input decks were linked closely to the SWMM results. Predicted overflow volumes are very sensitive to "treatment rates" assigned to the regulators.

Annual Pollutant Loadings

Pollutant concentrations in the CSO discharges were calculated from 1983 Phase 1 data and 1989 Phase 3 data. The data for each study period were classified into three different receiving waters: Inner Harbor, Outer Harbor, and Clarks Cove. The ranges of values reported in the 1983 data and the 1989 data were consistent with each other. No patterns for discerning quality differences between locations were observed. Since it was not possible to establish characteristic concentrations between the receiving water bodies, average concentrations were computed without regard to the locations of the discharges. For each constituent, average annual loadings were then calculated by multiplying discharge volumes by the computed concentration.

Results of the pollutant loading calculations are shown in Table 4-19. The table summarizes for each constituent the average expected annual CSO discharges into the three receiving water bodies, under present conditions.

The STORM model was also developed for the base condition (improvements recommended by Phase 2 WWTP Facilities Plan in place), and calibrated to the SWMM model. The effects of these improvements can be seen in Table 4-20, which compares annual volumes and loads for existing and base

TABLE 4-18

SIMULATED ANNUAL OVERFLOW VOLUMES AND FREQUENCY OF DISCHARGE

Receiving Water	Outfall	Overflow (MG)	Frequency (times/year)
Clarks Cove	003	0.43	17
	004	495.77	50
	005	0.16	5
	006	0.34	15
	007	0.00	1
	008	0.00	1
	009	0.00	1
	010	0.00	1
	011	0.10	5
	TOTAL	496.81 MG	
Outer Harbor	012	4.53	47
	013	0.40	45
	014	4.80	47
	015	1.60	47
	016	0.28	47
	017	10.37	49
	018	11.24	46
	TOTAL	33.22 MG	
Inner Harbor	020	150.93	50
	021	1.42	21
	022	119.38	43
	023	63.53	50
	024	5.41	37
	025	37.43	45
	026	2.73	24
	027	0.10	8
	028	8.19	46

TABLE 4-18 (cont.)

SIMULATED ANNUAL OVERFLOW VOLUMES AND FREQUENCY OF DISCHARGE[1]

Receiving Water	Outfall	Overflow (MG)	Frequency (times/year)
Inner Harbor	030	37.47	34
(continued)	031	54.14	37
	032	0.62	10
	033	0.37	10
	034	1.64	21
	035	0.01	1
	036	46.62	36
	037	13.10	50
	040	38.71	45
	041	13.66	39
	TOTAL	595.46 MG	

TOTAL ANNUAL OVERFLOW VOLUME 1125.49 MG

NOTE:

(1) This table does not include dry weather discharges, which are to be eliminated by 1990 through the Phase 2 WWTP improvements. Estimated annual volumes for existing dry weather discharges are as follows:

Outfall	Volume (MG)
003	2.63
004	58.40
015	18.25
017	3.65
020	193.45
022	73.00
028	18.25
031	32.85
034	36.50
038	3.65
041	361.35

TABLE 4-19

ANNUAL DISCHARGES FROM CSOs AND STORM DRAINS

Receiving Water	Discharge Volume	TSS	BOD	TKN	Ammonia	Lead	Zinc	Copper	Chromium	Iron	Aluminum	T. Coliform (MPN)
					(l b s / y e a r)							
CLARKS COVE												
CSO												
New Bedford	496 MG	$3.3*10^5$	$2.2*10^5$	$2.8*10^4$	$6.2*10^3$	$4.6*10^2$	$1.4*10^3$	$4.6*10^2$	$3.3*10^2$	$7.5*10^3$	$3.1*10^3$	$2.9*10^{16}$
STORMWATER												
New Bedford	201 MG	$1.6*10^5$	$3.5*10^4$	$3.5*10^3$	$4.7*10^2$	$3.0*10^2$	$5.9*10^2$	$1.3*10^2$	$1.0*10^2$	No data	No data	$1.1*10^{15}$
Dartmouth	85 MG	$6.7*10^4$	$1.5*10^4$	$1.5*10^3$	$2.0*10^2$	$1.3*10^2$	$2.5*10^2$	$5.7*10^1$	$4.3*10^1$	No data	No data	$4.8*10^{14}$
TOTAL	286 MG	$2.3*10^5$	$5.0*10^4$	$5.0*10^3$	$6.7*10^2$	$4.3*10^2$	$8.4*10^2$	$1.9*10^2$	$1.4*10^2$	No data	No data	$1.6*10^{15}$
INNER HARBOR												
CSO												
New Bedford	595 MG	$4.0*10^5$	$2.6*10^5$	$3.3*10^4$	$7.4*10^3$	$5.5*10^2$	$1.7*10^3$	$5.5*10^2$	$4.0*10^2$	$8.9*10^3$	$3.8*10^3$	$3.5*10^{16}$
STORMWATER												
New Bedford	826 MG	$6.5*10^5$	$1.4*10^5$	$1.4*10^4$	$1.9*10^3$	$1.2*10^3$	$2.4*10^3$	$5.5*10^2$	$4.1*10^2$	No data	No data	$4.6*10^{15}$
Acushnet	46 MG	$3.6*10^4$	$8.1*10^3$	$8.1*10^2$	$1.1*10^2$	$6.9*10^1$	$1.3*10^2$	$3.1*10^1$	$2.3*10^1$	No data	No data	$2.6*10^{14}$
Fairhaven	436 MG	$3.5*10^5$	$7.6*10^4$	$7.6*10^3$	$1.0*10^3$	$6.5*10^2$	$1.3*10^3$	$2.9*10^2$	$2.2*10^2$	No data	No data	$2.4*10^{15}$
TOTAL	1308 MG	$1.0*10^6$	$2.3*10^5$	$2.3*10^4$	$3.1*10^3$	$2.0*10^3$	$3.8*10^3$	$8.7*10^2$	$6.5*10^2$	No data	No data	$7.3*10^{15}$
OUTER HARBOR												
CSO												
New Bedford	33 MG	$2.2*10^4$	$1.5*10^4$	$1.8*10^3$	$4.1*10^2$	$3.0*10^1$	$9.6*10^1$	$3.0*10^1$	$2.2*10^1$	$5.0*10^2$	$2.1*10^2$	$1.9*10^{15}$
STORMWATER												
New Bedford	99 MG	$7.8*10^4$	$1.7*10^4$	$1.7*10^4$	$2.3*10^2$	$1.5*10^2$	$2.9*10^2$	$6.6*10^1$	$5.0*10^1$	No data	No data	$5.5*10^{14}$
Fairhaven	157 MG	$1.2*10^5$	$2.8*10^4$	$2.8*10^4$	$3.7*10^2$	$2.4*10^2$	$4.6*10^2$	$1.0*10^2$	$7.9*10^1$	No data	No data	$8.8*10^{14}$
TOTAL	256 MG	$2.0*10^5$	$4.5*10^4$	$4.5*10^4$	$6.0*10^2$	$3.8*10^2$	$7.5*10^2$	$1.7*10^2$	$1.3*10^2$	No data	No data	$1.4*10^{15}$

Source: Pollutant concentrations calculated from 1983 and 1989 Sampling data.

TABLE 4-20

COMPARISON OF ANNUAL LOADS FOR EXISTING AND BASE CONDITIONS

		EXISTING CONDITIONS						BASE CONDITIONS					
Receiving Water	CSO Group	Overflow Volume per Year (MG)	Overflow Frequency	Total Coliform (MPN)	TSS (lbs)	Zinc (lbs)	Copper (lbs)	Overflow Volume per Year (MG)	Overflow Frequency	Total Coliform (MPN)	TSS (lbs)	Zinc (lbs)	Copper (lbs)
Clarks Cove	1	496.2	50	2.9×10^{16}	331,275	1449	456	359.2	45	2.1×10^{16}	239,811	1,049	330
	2	0.6	15	3.5×10^{13}	401	2	1	0.3	6	1.8×10^{13}	200	1	1
Outer Harbor	3	33.2	49	2.0×10^{15}	22,165	97	30	22.4	49	1.3×10^{15}	14,955	65	21
Inner Harbor	4	102.4	46	6.0×10^{15}	68,365	299	94	100.5	46	5.9×10^{15}	67,096	294	92
	5	212.1	50	1.3×10^{16}	141,603	620	195	63.9	50	3.8×10^{15}	42,661	187	59
	6	280.9	50	1.7×10^{16}	187,536	820	258	188.4	45	1.1×10^{16}	125,780	550	173
Total		1125.4	50	6.7×10^{16}	751,345	3287	1,034	734.7	50	4.3×10^{16}	490,503	2,146	676

conditions by group. Although the frequency of discharge is not sharply reduced, the pollutant loads to the receiving water decrease by 35 percent under the base condition. This represents a significant improvement in terms of water quality, but is clearly not sufficient to solve the CSO problem.

4.6.2 STORMWATER CONTRIBUTIONS

A similar analysis was conducted for stormwater discharges, albeit on a somewhat grosser scale. Instead of calculating flows for each outfall, quantities were lumped together according to receiving water body. Three STORM input decks were defined by specifying runoff coefficients and contributing runoff areas for each of the three receiving waters. The 1-year record was used to predict annual discharge quantities under "normal" rainfall conditions.

The STORM simulations indicate that under present conditions direct stormwater discharges contribute approximately 1900 million gallons annually to the New Bedford Harbor and Clarks Cove. Three-quarters of this stormwater is discharged into the Inner Harbor with another large portion going to Clarks Cove. Table 4-19 shows the predicted discharge volumes to the respective receiving waters. The improved estimate is less than the 2150 million gallons initially estimated. The minor difference is due to differences in assigned runoff coefficients. The runoff coefficients used in the current study were selected based on the values used for the subbasins in the calibrated SWMM model. Since this analysis was very sensitive to these runoff coefficients, they were later checked against values generated using data produced by the ARC/INFO geographic information system.

In addition to quantity predictions, quality and resultant pollutant loadings due to stormwater discharges were calculated. As part of the same data collection efforts which generated the CSO sampling data, stormwater discharge quality was monitored. Table 4-19 shows the computed average concentrations for the various constituents based on the 1983 and 1989 data. Here again, since no discernible pattern was found for

distinguishing stormwater runoff quality for the different receiving waters, the overall average was used to calculate annual loadings. Table 4-19 also presents the computed loadings broken down by constituent for the Inner Harbor, Outer Harbor, and Clarks Cove. Clearly evident from this table is the fact that for most of the analyzed pollutants, predicted loadings from direct stormwater discharges are on the same scale as those predicted from CSOs.

The water quality impacts of these loadings on the receiving waters will be addressed in Sections 5.0 and 6.0 and Appendices C1 and C2.

4.7 SUMMARY

The results of the model development and application are as follows:

- The SWMM model accurately represents both inflow and routing of dry and wet weather flows, based on data from five storm events.
- Regulating structures are generally hydraulically independent of flow conditions in the interceptor, with major exceptions at regulators 22A, on Sawyer and N. Front Streets; 20A, on Wamsutta and Route 18; 31A, on Grinnell and Second Streets; and the regulators along the west side of Clarks Point under extreme surcharge conditions.
- Based on the design storm analysis, approximately 80 percent of the overflow volume is discharged by the 8 largest CSOs: 004, 022, 031, 020, 030, 040, 023, and 036.
- Phase 3 sampling data confirms results from the Phase 1 study.
- Under present conditions annual CSO discharges are approximately 600 million gallons to the Inner Harbor; 30 million gallons to the Outer Harbor; and 500 million gallons to Clarks Cove.
- Direct annual stormwater discharges total roughly 1,310 million gallons to the Inner Harbor, 260 million gallons to the Outer Harbor, and 290 million gallons to Clarks Cove.
- Direct stormwater discharges may influence receiving water quality as much as or greater than CSO discharges for certain pollutants.

SECTION 4 REFERENCES

Camp Dresser & McKee Inc. 1983. Draft Phase 1 Interim Summary Report on Combined Sewer Overflows. Prepared for the City of New Bedford.

Camp Dresser & McKee Inc. 1989. Draft Phase 2 Facilities Plan. Prepared for the City of New Bedford.

McCollough Associates Inc. 1979. Point Velocity Discharge Measurement Method for Partially Full Sewer Lines.

U.S. Environmental Protection Agency. December, 1983. Results of the Nationwide Urban Runoff Program, Volume 1 - Final Report.

5.0 EXISTING RECEIVING WATER QUALITY

5.1 OBJECTIVES OF RECEIVING WATER EVALUATIONS

This section describes the basis for the receiving water quality evaluations conducted as part of the CSO facilities plan. Receiving water quality evaluations provide an environmental basis for assessing the compliance of CSO abatement alternatives relative to regulatory policies on CSO control. The evaluations also describe the geographical extent, magnitude, frequency, and duration of water quality violations resulting from CSO discharges (under both existing and future conditions).

This section is organized as follows. Section 5.2 describes the regulatory requirements that must be met in the receiving waters. Section 5.3 provides a general description of the receiving waters, including their physical characteristics, and circulation, stratification, and sediment deposition patterns. The marine resources in the study area are described in Section 5.4. Finally, Section 5.5 assesses the water quality impacts from CSO discharges under existing conditions.

5.2 REGULATORY REQUIREMENTS

The regulatory requirements that must be addressed in evaluating the receiving water section of this facilities plan are the Commonwealth of Massachusetts water quality standards and the Ocean Sanctuaries Act. The water quality standards are currently undergoing revision by DEP. As many issues in the revised standards, particularly provisions for CSO discharges, remain unresolved, this project uses the current water quality standards and the interim CSO policy (DWPC, 1987), which describes the application of these standards to CSO discharges, as the basis for compliance evaluations.

EPA (1989) has also developed a national control strategy for CSOs. This strategy discusses several items that might be included in a state-wide strategy and sets a deadline for states to develop their strategies. Because Massachusetts already has an interim policy for CSO discharges, this policy is the basis for analysis conducted herein. Nonetheless, it is prudent to be aware of how EPA's national strategy might result in changes to the Massachusetts policy. This section, therefore, includes a discussion of EPA's national strategy and compares them to the Massachusetts interim policy for CSOs.

The Massachusetts water quality standards, Massachusetts interim CSO policy and the EPA national CSO strategy are included in Appendix F.

5.2.1 MASSACHUSETTS WATER QUALITY STANDARDS

The Massachusetts water quality standards are issued by DEP and are published in the Code of Massachusetts Regulations (CMR) Section 314. The purpose of the water quality standards is to enhance the quality and value of water resources of the Commonwealth and to secure to the Commonwealth the benefits of the federal Clean Water Act.

Water Classification

Massachusetts has established three classifications for coastal and marine waters depending on the types of recreation, aquatic life, and wildlife. Following are the definitions for each regulatory class:

> Class SA - Waters which are designated for the uses of protection and propagation of fish, other aquatic life and wildlife; for primary and secondary contact recreation; and for shellfish harvesting without depuration in approved areas.
>
> Class SB - Waters designated for the uses of protection and propagation of fish, other aquatic life and wildlife; for primary and secondary contact recreation; and for shellfish harvesting with depuration (Restricted Shellfish Areas).
>
> Class SC - Waters designated for the protection and propagation of fish, other aquatic life and wildlife; and for secondary contact recreation.

The area of study for this report embodies Class SA and Class SB waters. The Inner Harbor area is Class SB water, while the Outer Harbor and Clarks Cove are Class SA waters.

Minimum Criteria

The Commonwealth has adopted a set of minimum criteria that are applicable to all surface waters. The minimum criteria are designed to meet the objectives set forth by the federal Clean Water Act, and are given in Table 5-1. Also, Massachusetts has set forth four additional minimum numerical criteria based on the classification of the coastal and marine waters. These additional minimum criteria -- temperature, pH, dissolved oxygen, and total coliform bacteria -- are given in Table 5-2.

Toxic Substances Criteria

The present Massachusetts regulations (314 CMR 4.03) also state that:

> The Division [of Water Pollution Control] will use EPA criteria established under section 304(a)(1) of the Federal Act as guidance in establishing case-by-case discharge limits for pollutants not specifically listed in these standards, but included under the heading "Other Constituents" in 314 CMR 4.03(4), for identifying bioassay application factors and for interpretation of narrative criteria.

At this time, the state has not developed a policy on the interpretation of its narrative standard regarding the use of EPA criteria, although this issue is being addressed in the revisions to the Massachusetts water quality standards. These revisions are in part prompted by the 1987 revisions to the federal Clean Water Act that require the state to develop numerical limits for pollutants for which EPA has published criteria under Section 304. The 1987 Act does not, however, require that states adopt EPA's numerical criteria.

Since the revisions to the Massachusetts standards are presently under review, these revisions are not used in this facilities plan. However, DWPC's interim CSO policy (discussed below) states that pollutant loadings in such concentrations that cause acute toxicity in organisms passing

TABLE 5-1

MASSACHUSETTS WATER QUALITY STANDARDS

The following minimum criteria are adopted and shall be applicable to all surface waters of Massachusetts unless criteria specified for individual classes are more stringent.

Parameter	Criteria
1. Aesthetics	All water shall be free from pollutants in concentrations or combinations that: (a) Settle to form objectionable deposits; (b) Float as debris, scum or other matter to form nuisances; (c) Produce objectionable odor, color, taste or turbidity; or (d) Result in the dominance of nuisance species
2. Radioactive Substances	Shall not exceed the recommended limits of the United States Environmental Protection Agency's National Drinking Water Regulations.
3. Tainting Substances	Shall not be in concentrations or combinations that produce undesirable flavors in the edible portions of aquatic organisms.
4. Color, Turbidity, Total Suspended Solids	Shall not be in concentrations or combinations that would exceed the recommended limits on the most sensitive receiving water use.
5. Oil and Grease	The water surface shall be free from floating oils, grease and petrochemicals and any concentrations or combinations in the water column or sediments that are aesthetically objectionable or deleterious to the biota are prohibited. For oil and grease of petroleum origin the maximum allowable discharge concentration is 15 mg/l.
6. Nutrients	Shall not exceed the site-specific limits necessary to control accelerated or cultural eutrophication.
7. Other Constituents	Water shall be free from pollutants in concentration or combinations that (a) Exceed the recommended limits on the most sensitive receiving water use; (b) Injure, are toxic, or produce adverse physiological or behavioral responses in humans or aquatic life; or (c) Exceed site-specific safe exposure levels determined by bioassay using sensitive species.

TABLE 5-2

ADDITIONAL MASSACHUSETTS WATER QUALITY STANDARDS FOR MARINE WATERS

Following are additional minimum criteria applicable to coastal and marine waters.

For Class SA Waters:

Parameter	Criteria
1. Dissolved Oxygen	Shall be a minimum of 85 percent of saturation at water temperature above 77°F (25°C) and shall be a minimum of 6.0 mg/l at water temperatures of 77°F (25°C) and below.
2. Temperature Increase	None except where the increase will not exceed the recommended limits on the most sensitive water use.
3. pH	Shall be in the range of 6.5-8.5 standard units and not more than 0.2 units outside of the naturally occurring range.
4. Total Coliform Bacteria	Shall not exceed a median value of 70 MPN per 100 ml and not more than 10% of the samples shall exceed 230 MPN per 100 ml in any monthly sampling period.

For Class SB Waters:

Parameter	Criteria
1. Dissolved Oxygen	Shall be a minimum of 85 percent of saturation at water temperatures above 77°F (25°C) and shall be a minimum of 6.0 mg/l at water temperatures of 77°F (25°C) and below.
2. Temperature Increase	None except where the increase will not exceed the recommended limits on the most sensitive water use.
3. pH	Shall be in the range of 6.5-8.5 and not more than 0.2 units outside of the naturally occurring range.
4. Total Coliform Bacteria	Shall not exceed a median value of 700 MPN per 100 ml and not more than 20% of the samples shall exceed 1000 MPN per 100 ml during any monthly sampling period, except as provided in 314 CMR 4.02(1).

through the mixing zone, or pose a threat to human health are prohibited. In addition, far field (outside the mixing zone) concentrations shall not be acutely or chronically toxic to aquatic life. This is the criteria that will be used to evaluate compliance with Massachusetts water quality standards for toxic substances.

5.2.2 MASSACHUSETTS INTERIM CSO POLICY

DWPC (1987) has set forth an interim policy for Abatement of Pollution from Combined Sewer Overflows (hereinafter referred to as the interim policy) as a guidance document for the planning and implementation of abatement projects for CSOs. The Division adopted this policy to classify waterbody segments impacted by CSOs, apply the water quality criteria to protect beneficial uses, and specify a minimum level of treatment for CSO projects.

The interim policy recognizes that some excursions from receiving water criteria may be tolerated while still maintaining beneficial uses. The policy defines as a goal (to serve as the basis of design of abatement measures) that "allowable excursions" are permissible as long as they do not exceed a specific frequency or impact critical uses of the receiving water. This frequency is an extreme hydrologic event that causes a maximum frequency of exceedance of criteria of 1 percent, or approximately four days per year. The policy also gives the maximum allowable excursions to numerical criteria that will be allowed during these events; these are listed in Table 5-3. Water quality standards will be considered as met when excursions from receiving water criteria occur no more than 1 percent of the time (due to the impact of CSOs) and are within the magnitude specified. Critical uses are defined as bathing areas, shellfishing areas, water supply intakes, recreation areas, wildlife refuges, and other areas of ecologic or economic concern. All feasible alternatives are to be analyzed before a CSO discharge to a critical use area will be approved.

The policy also allows all CSOs with overlapping instream effects to be considered as a single discharge. Thus, individual discharges need not be

TABLE 5-3
WATER QUALITY CRITERIA IN THE MASSACHUSETTS INTERIM CSO POLICY

CRITERIA	ALLOWABLE EXCURSIONS FROM CRITERIA*				RECEIVING WATER ANALYSIS
	EXTENT	FREQUENCY	DURATION	MAGNITUDE	
Aesthetics	Mixing Zone	0	0	No nuisance conditions	Wind drift analyses
	Farfield**	0	0	No acute toxicity	
Dissolved Oxygen	Mixing Zone	-	-	Minimum of 3.0 mg/l	Local analysis and empirical modeling
	Farfield	1% (4 times/yr)	24 hours	Greater than 50% saturation	
Coliform Bacteria	Mixing zone	-	-	No public health problems	Contours of dilution from empirical and Battelle models
	Farfield	1% (4 times/yr)	24 hours		
Temperature	Mixing zone	-	-	No nuisance conditions	Temperature contours
	Farfield	1% (4 times/yr)	24 hours	No acute toxicity	
pH	Mixing zone	-	-	No acute toxicity	Dilution modeling and assessments of pH impacts
	Farfield	1% (4 times/yr)	24 hours	5.5-9.0, within 0.5 of ambient	
Nutrients	Mixing zone	-	-	No nuisance conditions	Nutrients and productivity measurements from WWTP Facilities Plan, Dilution Modeling
	Farfield	-	-	Total loading meets standards	
Solids, Floatables, Scum, Oil, etc.	Mixing zone	-	-	No aesthetic problem or toxicity	Wind drift analysis
	Farfield	-	-		
Toxicity	Mixing zone	-	-	No acute toxicity	Pollutant tracking from Battelle model Dilution modeling
	Farfield	-	-	No acute or chronic toxicity	

*From "Interim Policy for Abatement of Pollution of CSOs" (DWPC, 1987)
**Farfield = Area outside of mixing zone

eliminated or treated to the same degree as long as the total load of pollutants is reduced to meet water quality standards.

The interim policy also recognizes that site-specific impact analyses are necessary to evaluate the extreme hydrologic condition and resultant receiving water impacts. Consequently the policy does not specify a uniform level of treatment for CSO discharge, but rather requires a uniform analysis methodology to be applied to CSO projects. This methodology includes the following elements.

1. Inventory of existing combined sewer systems including location of intakes, bypasses, pipes, regulators, and outfalls.

2. Assessment of system performance. Identification of measures to maximize sewer system efficiency and minimize overflows. These may include correcting malfunctions, unblocking clogged lines, optimizing regulator functions, and locating unused in-line storage capacity.

3. Measurement or estimation of wastewater constituents, concentrations and wastewater flows, duration and frequency.

4. Measurement or estimation of receiving water impacts, duration and frequency. This includes identifying beneficial uses, the public interest and demand for these uses, seasonal limitations, natural limitations, and the compatibility of multiple uses of the receiving water.

5. Identification and evaluation of potential interactive/overlapping pollutant sources in impacted areas.

6. Projection of future wasteloads, flow, and impacts.

7. Development of alternatives for eliminating or mitigating impacts and the associated costs. Alternatives include source controls, collection system controls, and storage and treatment. Costs should be associated with benefits.

5.2.3 EPA STRATEGY FOR CSOs

EPA's (1989) National Control Strategy for CSOs requires that NPDES-approved states or EPA regions develop permitting strategies for CSOs no later than January 15, 1990. These strategies are to be consistent with the national strategy. It sets forth three objectives:

1. To ensure that if CSO discharges occur, they are only as a result of wet weather,

2. To bring all wet weather CSO discharge points into compliance with the technology-based requirements of the Clean Water Act and applicable State water quality standards, and

3. To minimize water quality, aquatic biota, and human health impacts from wet weather overflows.

EPA's strategy includes a discussion of several elements that may be addressed in the state strategies. These include identifying and classifying CSO discharge locations, developing priorities for those discharges that are unpermitted or do not meet permit requirements, establishing permit issuance guidelines, stating compliance schedules, establishing minimum technology-based requirements for inclusion in permits, defining additional CSO control measures, establishing monitoring programs, modifying water quality standards, and providing funding.

The EPA national strategy appears to differ from the Massachusetts interim policy in two areas: technology-based requirements and allowing excursions from water quality standards. Regarding the technology-based requirements, the EPA strategy states that:

> All permits for CSO discharges should require the following technology-based limitations as a minimum best conventional pollutant control technology (BCT)/best available technology economically achievable (BAT), established on a best professional judgment basis: (1) proper operation and regular maintenance programs for the sewer system and combined sewer overflow points; (2) maximum use of the collection system for storage; (3) review and modification of pretreatment programs to assure CSO impacts are minimized; (4) maximization of flow to the POTW for treatment; (5) prohibition of dry weather overflows; and (6) control of solid and floatable materials in CSO discharges.

While the Massachusetts interim policy includes concepts of maintenance programs and elimination of dry weather overflows, it does not embrace the "storage and treat" abatement technique as does the EPA strategy. Instead the Massachusetts policy allows that "individual discharges need not be eliminated or treated to the same degree...to allow greater flexibility to produce alternatives and the possibility of more cost-effective abatement

measures based on an optimal mix of structural and non-structural solutions."

EPA's national strategy further states that CSO "permits must be written to ensure CSO discharges do not cause violations of water quality standards. The applicability of water quality standards should not be waived under any circumstances." They allow that in limited cases adjustments to the standards may be appropriate to address the impact of pollutants in wet weather flows, but they do not include a concept similar to the 1 percent frequency of exceedance embodied in the Massachusetts policy.

5.2.4 OCEAN SANCTUARIES ACT

The receiving waters of New Bedford Harbor (excluding the Inner Harbor) are located in the Cape and Island Sanctuary. This sanctuary was established by Chapter 742 of the Massachusetts Ocean Sanctuary Act of 1971. In the project area, this sanctuary includes New Bedford Outer Harbor and Clarks Cove; the act does not apply to the Inner Harbor.

The Ocean Sancutaries Act places designated sanctuaries under the "care and control" of the Massachusetts DEM and states that they shall "be protected from any exploitation, development or activity that would seriously alter or otherwise endanger the ecology or appearance of the ocean, the seabed or subsoil thereof...." The portion of the Act that is relevant to the CSO Facilities Plan is that related to discharges from WWTPs. The Act currently prohibits any new municipal WWTP discharges in the Cape and Island Sanctuary. The category of new discharges includes any increase over the design capacity of the discharge at the time the ocean sanctuary was designated.

Because many of the abatement alternatives considered in the CSO Facilities Plan include routing former CSO discharges to New Bedford's new secondary WWTP, the application of the provisions of this Act to CSOs is integrally linked to decisions regarding the draft WWTP Phase 2 Facilities Plan (CDM, 1989; see Volume V). The application of the Ocean Sanctuaries Act provisions to both the CSO and WWTP Facilities Plans has been part of

ongoing discussion with the state's regulatory agencies. For the CSO facilities plan, it is assumed that the Ocean Sanctuaries Act will not restrict the evaluation of alternatives for the abatement of CSO discharges.

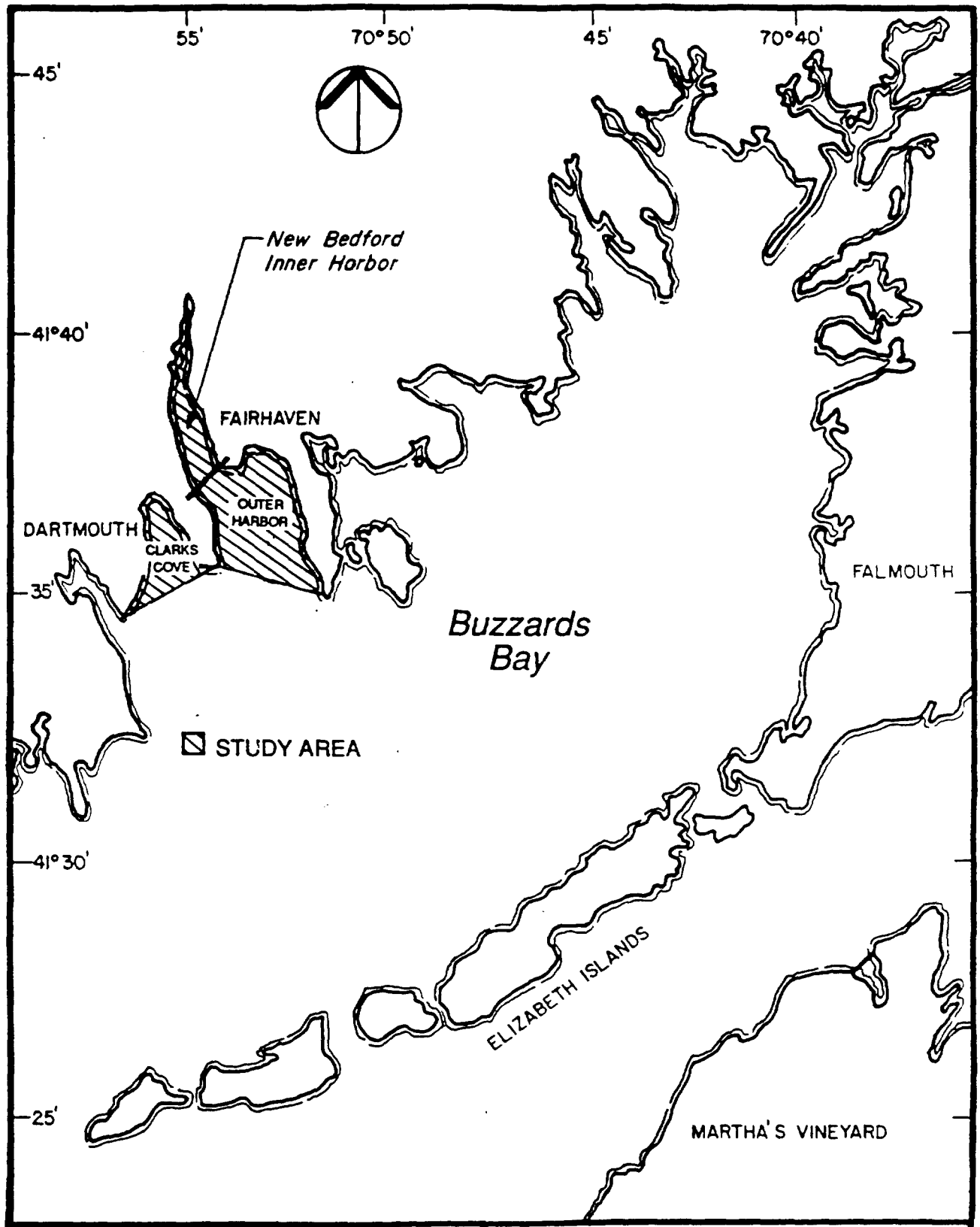
5.3 DESCRIPTION OF RECEIVING WATERS

5.3.1 GENERAL OVERVIEW

Buzzards Bay is 28-miles (45-km) long on a northeast-southwest axis, and about 10.5-miles (17-km) wide (see Figure 5-1). It is bounded on the northwest by the Massachusetts mainland, on the east by Cape Cod, on the southeast by the Elizabeth Islands, and on the southwest by Rhode Island Sound. Its principal connection to the Atlantic Ocean is through Rhode Island Sound, but there are also connections to Cape Cod Bay via the Cape Cod Canal, and to Vineyard Sound through several openings -- Woods Hole, Robinson's Hole, Quick's Hole, and other passages between the Elizabeth Islands. Numerous small streams (but no major rivers) discharge to Buzzards Bay.

New Bedford Harbor, the largest embayment within Buzzards Bay, was formed by an increase in sea level that drowned the Acushnet River valley. It is about 4.4 miles wide by 3.8 miles long (7-km by 6-km), and is located on the northwest coast of Buzzards Bay. Its principal tributary, the Acushnet River, flows into the head of the Inner Harbor, between New Bedford and Fairhaven. A gated hurricane barrier separates the Inner and Outer Harbors.

In Buzzards Bay, a semi-diurnal (twice daily) tide pattern prevails. Throughout Buzzards Bay, including New Bedford Harbor, the average tidal range is approximately 3.6 feet (1.1 m), with a spring tide of about 4.6 feet (1.4 m).



City of New Bedford COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3	FIGURE 5-1 PROJECT STUDY AREA

5.3.2 STUDY AREA FOR CSO FACILITIES PLAN

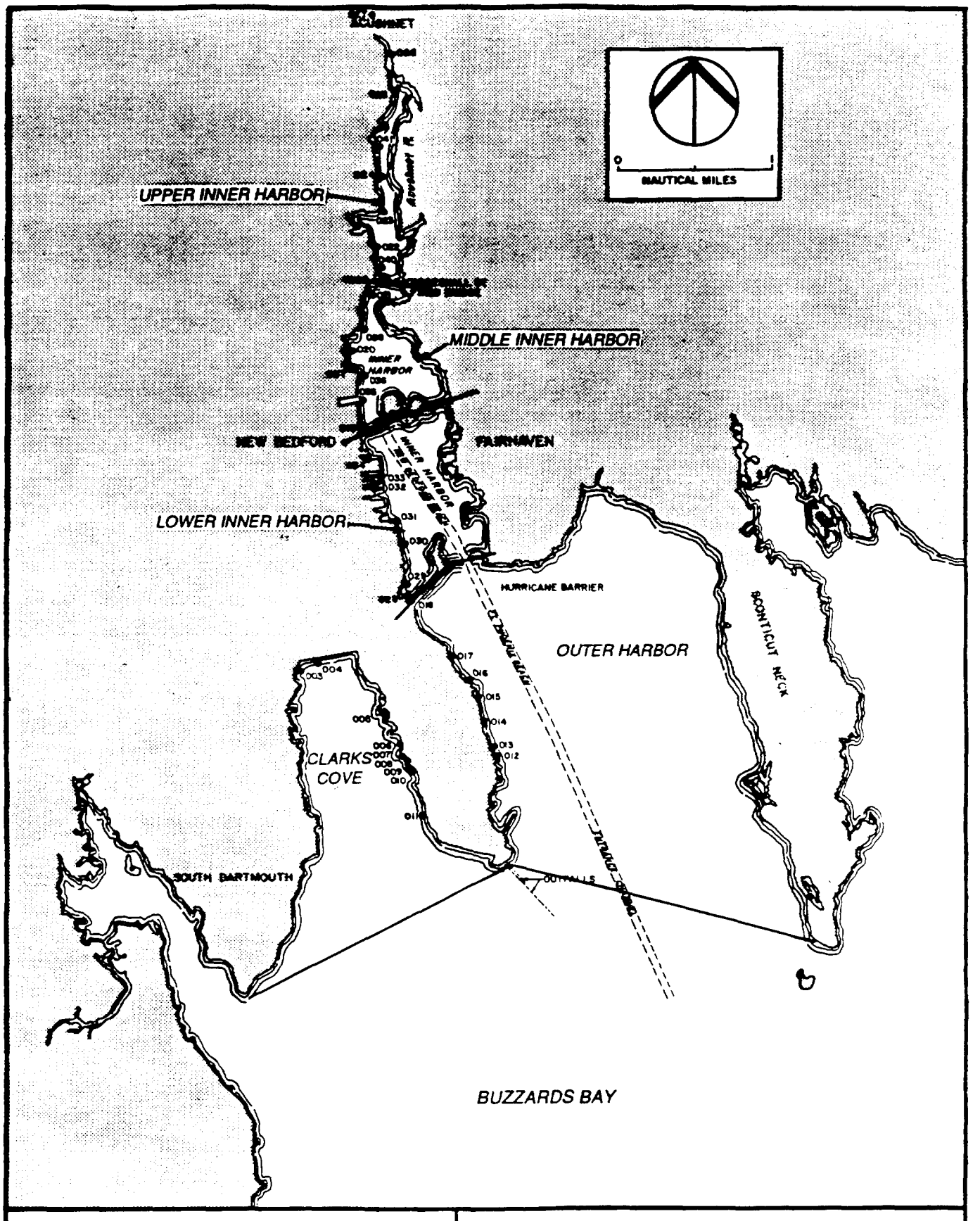
The receiving waters for New Bedford's CSOs can be separated into three distinct bodies of water: the Inner Harbor, the Outer Harbor, and Clarks Cove. They are shown on Figure 5-2, which also shows the locations of the 38 CSOs in New Bedford that are the subject of this facilities plan.

Inner Harbor

The Inner Harbor lies between the City of New Bedford and the Town of Fairhaven and extends from the tidal portion of the Acushnet River to the hurricane barrier at Fort Phoenix. For the purpose of many of the analyses performed in this facilities plan, the Inner Harbor is divided into three reaches: the upper reach (north of the the Coggeshall Street bridge), the middle reach (between the Coggeshall Street bridge and the Route 6/Popes Island bridge), and the lower reach (south of the Route 6/Popes Island bridge). There are 20 combined sewers discharging to the Inner Harbor.

Water depths in the upper Inner Harbor are quite shallow, generally less than 3 feet (1 m) except in the channel where depths average about 10 feet (3 m) at mean low water (MLW). In the middle reach of the Inner Harbor, the average water depth is about 11 feet (3.3 m), but more than half of this area is less than 4 feet (1.3 m) deep. The lower reach of the Inner Harbor averages about 13 feet (4 m) deep. Much of this area is a dredged shipping channel about 30 feet (9 m) deep, while water depths over about 60 percent of the area are less than 8 feet (2.4 m). Approximate surface areas and volumes of each of the reaches in the Inner Harbor are given below.

Reach	Surface Area acres (km^2)		Volume ft^3 (m^3)	
Upper	212	(0.85)	$34x10^6$	($0.9x10^6$)
Middle	242	(0.97)	$125x10^6$	($3.5x10^6$)
Lower	495	(1.98)	$277x10^6$	($7.7x10^6$)



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 5-2
STUDY AREAS FOR CSO FACILITIES PLAN

Outer Harbor

The Outer Harbor, as defined for the CSO facilities plan, is the water south of the hurricane barrier and north of an imaginary line extending from Clarks Point in New Bedford across the waters of Buzzards Bay to Wilbur Point on Sconticut Neck in Fairhaven. Southeast of Fort Phoenix is an area of shoals that are exposed at low tide. The average water depth is about 18 feet (5.5 m) at MLW. The volume of water in the Outer Harbor at MLW is about 877 million ft^3 (24.4 million m^3). Six combined sewers discharge to the Outer Harbor.

Clarks Cove

Clarks Cove is in the southwestern corner of New Bedford; its western shore is in the Town of Dartmouth. For this study, Clarks Cove is defined as the body of water north of an imaginary line from Ricketsons Point in South Dartmouth to Clarks Point in New Bedford. A section of the hurricane barrier was constructed around the northern arc of the cove's shoreline. The average water depth in Clarks Cove is about 15 feet (4.5 m) at MLW. The volume of water in Clarks Cove at MLW is about 611 million ft^3 (17 million m^3). Nine combined sewers discharge to Clarks Cove.

5.3.3 CIRCULATION IN THE STUDY AREA

Circulation within the study area is not well understood and only a few limited field programs have been conducted in the area. This section discusses previous field studies; studies conducted as part of this facilities plan, specifically the dye field program and numerical modeling studies, are discussed in Section 6.0.

Field programs in the Inner Harbor were performed as part of the EPA Superfund project in New Bedford; a one-month field study by Geyer and Grant (1986) examined circulation and flushing patterns. Circulation in Clarks Cove has not been previously studied. However, two dye studies performed in Clarks Cove (see Section 6.0) confirmed the general forcing factors of circulation patterns in the study area. An understanding of

circulation in the Outer Harbor is best extrapolated from field data collected as part of studies on New Bedford's WWTP outfall (CDM, 1979; CDM, 1983b; CDM, 1989).

Circulation throughout the study area is principally a result of limited but regular tidal and estuarine components, and wind-driven flows. Circulation patterns in Buzzards Bay, the irregular bathymetry and shoreline, freshwater runoff, and near-shore thermal warming are other less significant factors in the overall circulation picture.

Inner Harbor

The complex geometry of the Inner Harbor limits characterization of its hydrodynamics, particularly due to jet flow at the constrictions (e.g. bridge openings). Its only definitive features are that near-surface currents flow in the same direction as the wind, and that there is a mean estuarine flow (Geyer, 1989).

The following discussion of circulation in the Inner Harbor was adapted from the study by Geyer and Grant (1986). They placed five moorings that included continuously-recording instruments for velocity, pressure, conductivity, temperature and/or light transmission in the study area -- three in the Inner Harbor and two in the Outer Harbor (see Figure 5-3). Their study also included two drifter deployments in the Inner Harbor.

Geyer and Grant found that tides and winds are the principal contributors to Inner Harbor circulation, with a weak (about 1 cm/s) estuarine component. Tidal currents dominate in the lower and middle Inner Harbor due to the narrowness of the dredged channel (with velocities in the constrictions exceeding 50 cm/s). Elsewhere, tidal currents are weak (about 5 cm/s), while wind-driven currents of 10 cm/s in the upper water column are common. Their drifter experiments showed that the surface waters respond fairly directly to wind forcing, with an approximate transfer coefficient of 0.01 to 0.015 (a transfer coefficient relates wind speed to the the speed of the upper water surface). At depth, wind-driven currents flow opposite the wind at roughly one-half the speed of the

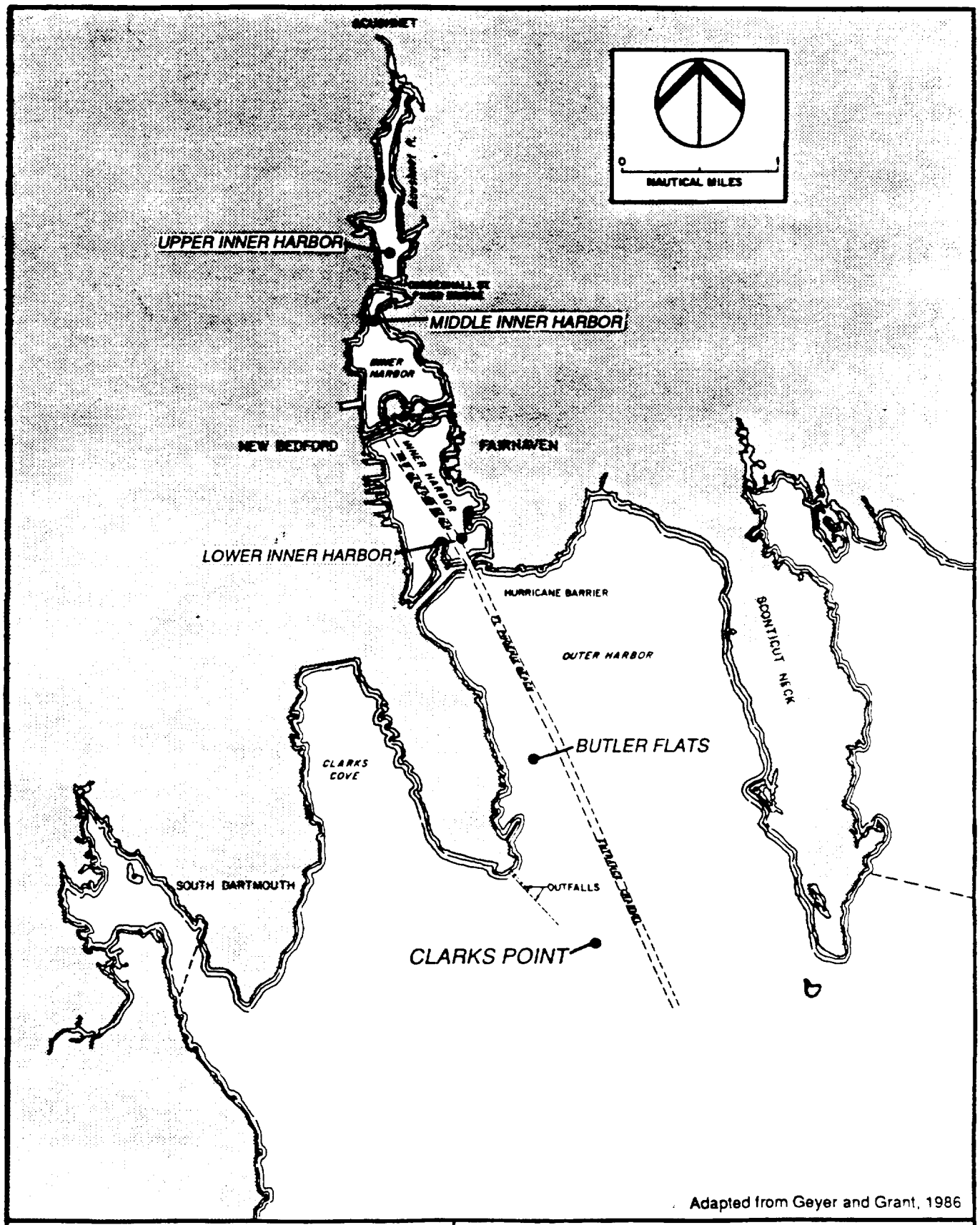


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FIGURE 5-3
1986 CURRENT METER DEPLOYMENT LOCATIONS

near-surface currents. While the tidal range in the Inner Harbor is similar to that in Buzzards Bay, the data show a small tidal lag up the Inner Harbor and some frictional damping.

Residence time in the Inner Harbor depends upon tidal flushing, wind-driven currents, and residual circulation patterns. Flushing is caused primarily by tidal flows at the Inner Harbor constrictions, and by wind-driven currents elsewhere. Wind driven currents due to either northerly or southerly winds cause net exchange among the reaches of the Inner Harbor, and between the Inner and Outer Harbors. They estimated that north to south, near-surface, wind-driven currents averaging 4 cm/s can induce 3-km net transport during a single day. Strong southerly winds can also produce this flushing due to deep return flows caused by the north-trending surface flows, although light southerly winds counteract the estuarine flow and inhibit net circulation.

Geyer and Grant also found that the greatest flushing exchange rates are in the upper Inner Harbor. Fifty percent of the upper Inner Harbor volume flows outward past the Coggeshall Bridge with each ebbing tide. The entire Inner Harbor discharges 25 percent of its volume with each tidal cycle. They estimated that one-half of the outgoing flows pass back through the Inner Harbor constrictions on incoming tides, so that the upper Inner Harbor has a 25 percent flushing rate and the entire Inner Harbor has a 12 percent net exchange over each tidal cycle.

They estimated an expected residence time within the Inner Harbor of a water parcel originating at the head of the Inner Harbor and gradually pushed southward by wind-driven currents to be about six days. They derived this estimate by assuming that wind-driven flows will move the parcel through the Inner Harbor in three days, but that net transport will be slowed at the tidal constrictions of the Inner Harbor. On average, a parcel will require two tidal cycles to pass through each Inner Harbor constriction, bringing the net residence time to six days. This value is an average and is dependent upon the presence of typical northerly or southerly winds.

Outer Harbor

Geyer and Dragos (1988) found evidence that suggests a net counterclockwise circulation in the shallow area of the Outer Harbor to the east of the shipping channel. This may be due to a strong jet motion outward through the hurricane barrier during ebb tide, and a slower "potential type" flow during flood tide with water flowing more uniformly toward the opening in the barrier. The net result of these two flows is a strong residual flow away from the opening along its axis, and a uniform return flow towards the opening. This is known as "tidal rectification," and appears in the area east of the shipping channel as a residual cyclonic eddy. The current meter data show that velocities on the overbank areas are smaller than those in the channel during ebb tide, and that during the flood tide, the currents in the shipping channel are smaller than ebb currents. Current meter data from Butler Flats mooring (see Figure 5-3) showed maximum tidal speeds on the order of 10 cm/s in the NW-SE direction, and wind-driven speeds up to 5 cm/s (Geyer and Grant, 1986).

As with the Inner Harbor, circulation in the Outer Harbor and Clarks Cove is strongly influenced by wind. Winds from the north tend to enhance flushing, winds from the south tend to retard flushing. Because of the paucity of field studies, no estimates of residence times have been made for the Outer Harbor or Clarks Cove. In the part of New Bedford Harbor just south of the study area for this project (i.e. south of Clarks Point), residence times can vary from about two days (during periods of strong wind) to as long as two weeks, depending on wind and large-scale circulation patterns in Buzzards Bay (Geyer and Dragos, 1988).

5.3.4 DENSITY STRATIFICATION IN THE STUDY AREA

Density stratification is composed by two independent phenomena: temperature and salinity. Each has a unique annual cycle that contributes to the local density structure seen within the study area. In the Outer Harbor and Clarks Cove, the salinity is always close to seawater concentrations, in line with the lack of large freshwater inputs in the study area, although surface salinity in the winter is sometimes between 26

and 30 ppt. Water temperatures range from 0 to 25°C. There is often a 1 to 2°C vertical temperature difference in the summer; in the winter, the water column is virtually isothermal.

Data for the Inner Harbor are scarce. The limited data suggest a horizontal density gradient that is a result of salinity stratification. One set of data (July 23, 1986 during a low runoff period) show a 1 ppt salinity gradient and about a 1.5°C temperature gradient from the I-95 bridge to the hurricane barrier. Vertical salinity gradients are typically 0.5 to 1 ppt in the summer but can be as high as 5 ppt following large freshwater events (Geyer, 1989). These gradients were found during high water and low water conditions on July 23. During storms, however, a strong salinity gradient is established that may last several days.

Continuously-recorded temperature data collected in the Inner Harbor during July and August 1986 (Geyer and Grant, 1986) showed distinctive diurnal heating patterns (probably due to the relatively shallow water depth) and advection of water between the three regions of the Inner Harbor. For example, the meter in the middle region of the Inner Harbor showed warmer water from the upper Inner Harbor and cooler water from the lower region.

In the Outer Harbor, vertical density stratification is weak in the winter and moderate in the summer. The degree of stratification is similar throughout the study area, with the exception of temperature/salinity fronts often seen in the vicinity of the hurricane barrier and northern reaches of the Outer Harbor (the hurricane barrier serves to separate the warmer, less saline water of the Inner Harbor from the Outer Harbor). In the winter, total vertical density stratification typically averages 0.2 to 0.5 σ_t (the relationship between density and σ_t units is as follows: a density value of 1024.5 kg/m^3 equals 24.5 σ_t). Summer stratifications are often about 1 σ_t under normal runoff conditions, but can range from near 0 to 2 σ_t. During the summer, this gradient results both from temperature and salinity variations, but in the winter it is only the result of salinity variation. The Outer Harbor is also often well mixed during winter periods of low runoff.

In four sets of readings (September 1 and 17, November 20, and December 5, 1987), there was also a small horizontal density gradient (typically <0.5 σ_t) in Clarks Cove (Geyer and Dragos, 1988). This density gradient is significantly less than that seen in the upper Outer Harbor suggesting that most freshwater enters through the hurricane barrier. Maximum vertical density differences were measured at 0.5 σ_t. Again, summertime horizontal and vertical differences were due to both salt and temperature stratification; winter density differences are primarily the result of salt stratification.

5.3.5 SEDIMENT DEPOSITION PATTERNS

The major study on sediment composition in the study area was performed by Summerhayes et al. (1977). Field programs conducted as part of the EPA Superfund project in New Bedford have updated these data; unfortunately, the Superfund data are not available at this time.

Buzzards Bay is a drowned drainage system; its basic trend is NNW-SSE with steep sides and smooth-floored troughs that have been filled in with sediments of mud and silt. One of the major troughs in the bay is the drowned valley of the Acushnet River, which forms New Bedford Harbor. Summerhayes' map of sediment composition indicates that the heterogeneous composition found in Buzzards Bay continues into New Bedford Harbor. Muddy sands and sandy muds are found along the central axis of the drowned river valley, but this is surrounded by patches of sediment ranging from gravel to sandy muds.

New Bedford Harbor and Buzzards Bay are net depositional areas due to an onshore movement of water and its sediment load in the lower water column. Before the hurricane barrier was built, deposition rates in the Inner Harbor were 1 to 2 cm/yr, with 0.5 cm/yr in the shallow areas. The construction of the barrier reduced the efficiency of tidal flushing and increased the rate of siltation by four or five times. In the Outer Harbor, deposition rates are on the order of 2 to 3 mm/yr. Accumulations tend to be larger in the deeper and sheltered areas, and less in shallow areas where storms may more easily resuspend the deposited sediments.

Biological processes, influenced by seasonal water temperatures, can affect the stability of the bottom, and thus the critical velocity required to suspend sediment. Rhoads (1987) concluded that in the Inner Harbor area the summer is characterized by bottom stability and net deposition due to an increase in critical velocities caused by benthic biological activity (sediment binding by worm tubes and plant exudations); sediment resuspension was more likely during the storms that characterize the winter season. In the Outer Harbor, resuspension during storm events is decreased due to greater water depth, but during the summer months, when near-bottom water temperatures exceed 5 to 10°C, critical velocities may actually decrease by a factor of 2 to 3 because of bioturbation (the mixing of sediment by biota).

5.4 HARBOR RESOURCES INVENTORY

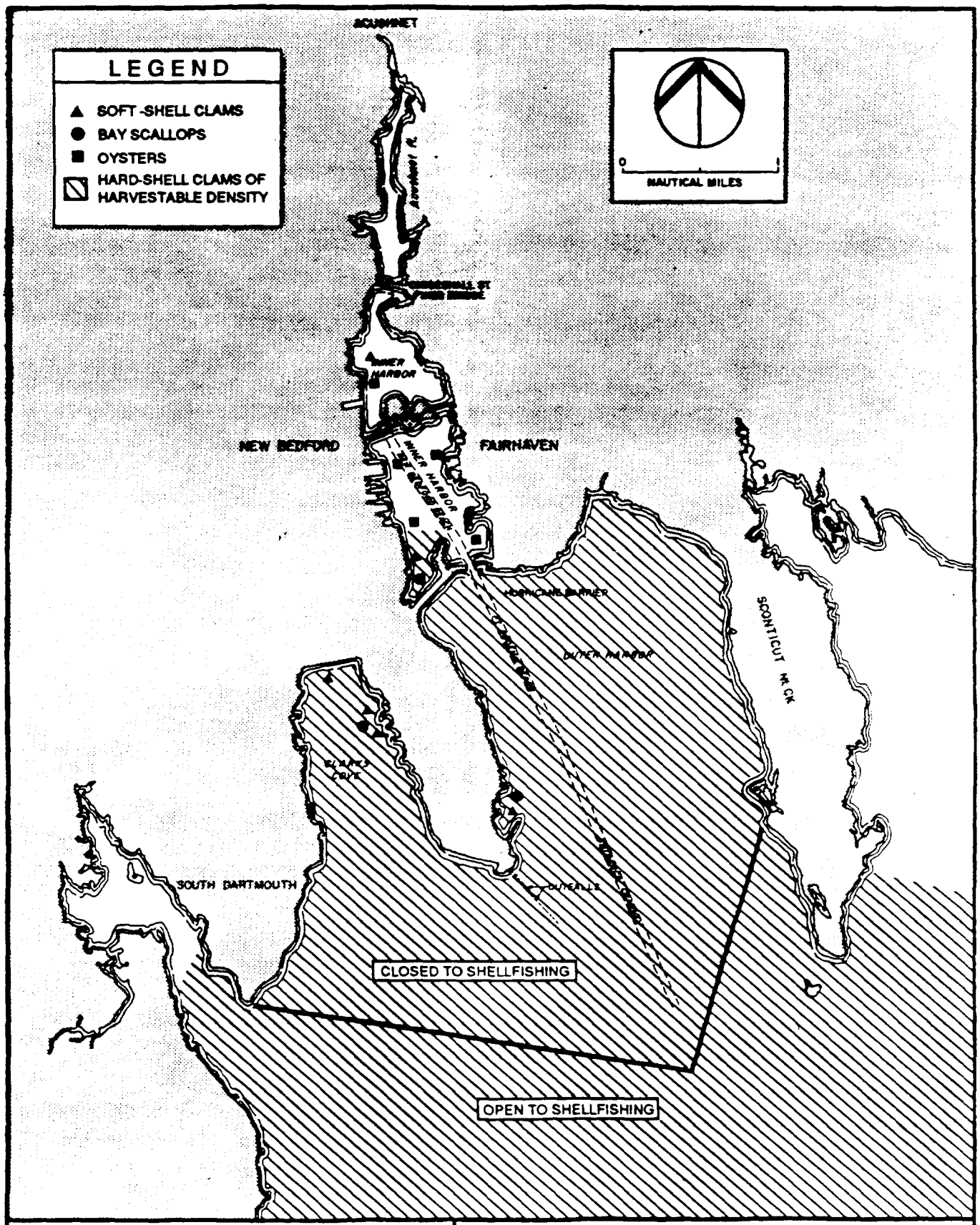
This section presents an inventory of the harbor resources in New Bedford Harbor, and as such defines the baseline conditions against which the CSO remedial alternatives can be evaluated. The specific harbor resources discussed herein are: marine shellfish and finfish resources (including the coliform and PCBs closure boundaries), recreational facilities, navigational channels and anchoring sites, and endangered species. This section also contains maps of these resources.

5.4.1 MARINE SHELLFISH AND FINFISH RESOURCES

The use of the shellfish and finfish resources in New Bedford Harbor is currently restricted by two closures. The first restricts the taking of shellfish because of bacterial contamination. The second prohibits the taking of lobster and finfish because of PCB contamination.

Closure Areas

The current boundary of the shellfish closure as of April 12, 1989 due to coliform bacterial contamination is shown in Figure 5-4. The shellfish resources to the north of the solid lines marked on the map are closed to harvesting. Loadings of coliform bacteria to the Harbor originate from CSOs, dry weather overflows, stormwater runoff, and WWTP effluent. The



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 5-4

LOCATIONS OF SHELLFISH RESOURCE AREAS AND SHELLFISH CLOSURE LINE

shellfish closures do not affect the bay scallop, lobster, or whelk resources. Massachusetts public health officials closed the Inner Harbor and the Acushnet River Estuary to all shellfishing in 1925 due to gross pollution. These water bodies have remained closed since that time. Closures in the Outer Harbor and Clarks Cove have occurred within the last decade.

The harvesting of lobster and finfish is also severely limited because of high levels of PCBs. The current restrictions on finfish and lobsters are outlined in Figure 5-5. The closure areas, which were established in 1979 by the Massachusetts Department of Public Health, prohibit all fishing activities in the area located to the north of the hurricane barrier. The next area to the south is closed to the taking of lobsters, eels, flounder, scup or tautog. The rest of the Outer Harbor and Clarks Cove is closed to the taking of lobster only.

Finfish

While New Bedford Harbor supports a large fishing fleet, no commercial finfisheries exist in the harbor or in Buzzards Bay because net fishing is prohibited.

Although sportfishing does occur in Buzzards Bay, no recent surveys are available on the species and their abundances. Sportsfishing is pursued extensively from the Dartmouth and Fairhaven shorelines, from the jetties at Clarks Point, and from boats in the Outer Harbor (CDM, 1983b). The major catches for the sport fishery in the Outer Harbor include: bluefish, *Potatomus saltatrix*; scup, *Stenotomus chrysops*; striped bass, *Morone saxatilis*; Atlantic mackerel *Scomber scombrus* (Kolek, 1979).

Areas of heavy fishing pressure for scup and tautog are rocky ledges near Wilbur Point. Other bottom-feeding fishes receive less pressure than do scup and tautog, and are principally fished from shore. While recreational fishing for bottom feeding fish is prohibited in the closure area, the closures are not enforced, and consequently appears to have had little impact on bottom-fishing activity. Bluefish are the principal gamefish in

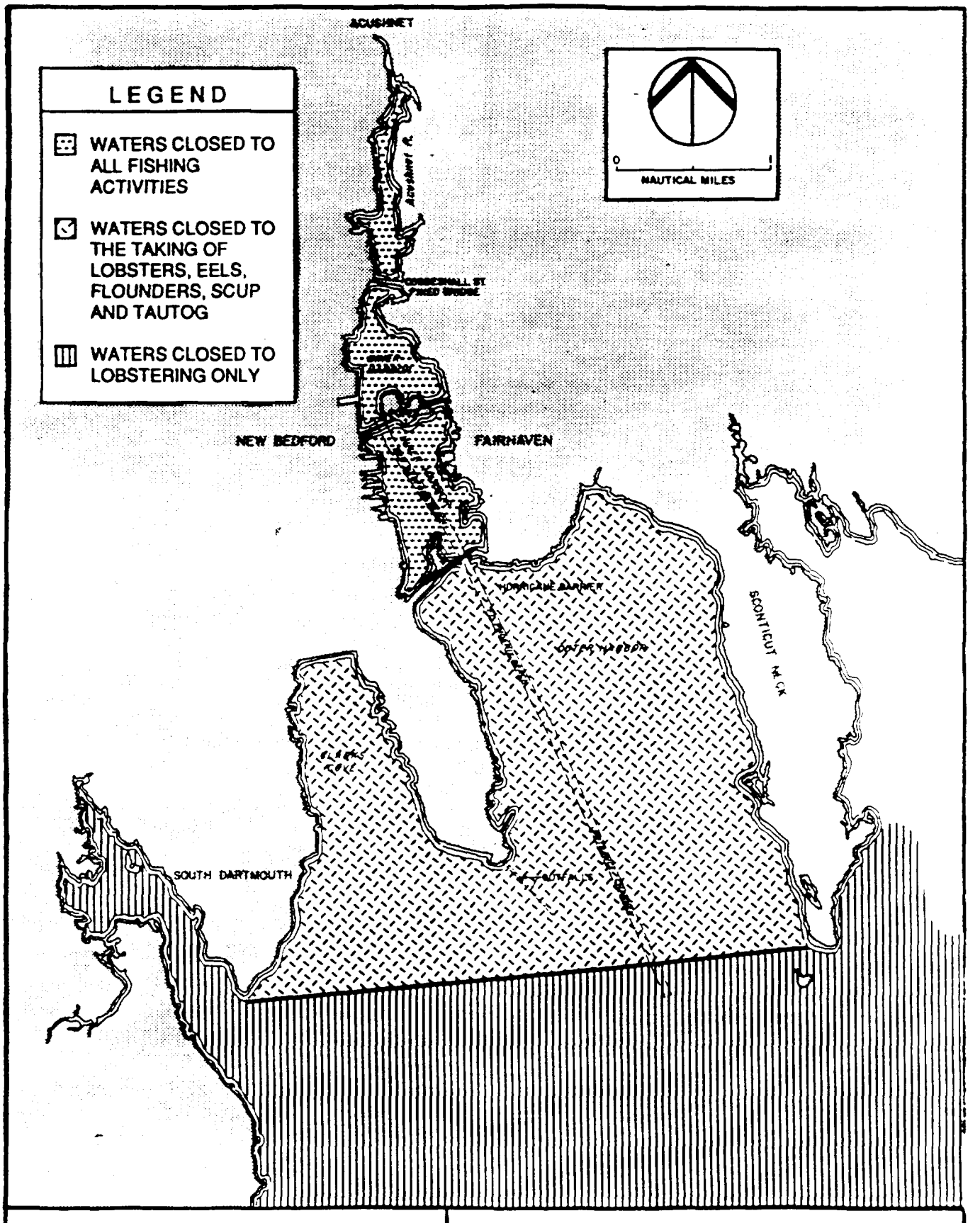


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FIGURE 5-5
AREAS SUBJECT TO PCB CLOSURES

the Outer Harbor, and are fished from boats in the Egg Island and Little Egg Island areas (CDM, 1983b).

Shellfish

The intertidal and subtidal shellfish resources in the New Bedford region were identified based on recent conversations with state biologists (M. Hickey and F. Germano) and the local shellfish warden (B. Bourque). Shellfish resources include the soft-shell clam (*Mya arenaria*), the hard-shell clam (*Mercenaria mercenaria*), black clams (*Arctica islandica*), bay scallops (*Aequipecten irradians*), oysters (*Crassostrea virginica*), whelks (locally called conch, *Busycon* spp.), and lobster (*Homarus americanus*). The boundaries of the beds of non-motile shellfish are shown in Figure 5-4.

The motile species (lobster and whelk) are found throughout the area. Prior to the closure, lobsters were regularly trapped in the Outer Harbor and because of their motility, their distribution is not limited to any bottom type or location. The taking of lobster is restricted throughout most of the study area by the closure imposed due to PCB contamination. Figure 5-5 shows that lobstering is prohibited throughout the Outer Harbor to beyond Great Ledge and Negro Ledge.

Lobster was the major species harvested commercially in the harbor before its closure in 1977. Approximately 50 commercial and 100 recreational lobstermen used the area, and the value of the fishery in the closed area was estimated to exceed $125,000/yr in 1981 (Kolek and Ceurvels, 1981). Lobstering does occur in Buzzards Bay beyond the closure boundary. No data, however, are available on the value of the present lobster resource (CDM, 1983b). The present numbers of lobster in the Harbor, however, are high (Bourque, 1989), probably because no commercial trapping has taken place since the closure. From Buzzards Bay as a whole, approximately 12 million pounds of lobsters were landed between 1967 and 1984 (Battelle, 1987). Recreational lobstering occurs just outside the study area (in the vicinity of the existing WWTP outfall) despite the closure, especially close to shore where rock rip rap provides cover for lobsters (CDM, 1983b).

The whelk fishery began only a few year ago, and it is not affected by either closure. The Outer Harbor and Clarks Cove support eight commercial fishermen who, together, are landing approximately 16,000 lbs of whelk (conch) per day (Bourque, 1989).

The hard-shelled clam is economically the most important of the bivalve shellfish species in the area. It is commercially harvested outside the closure areas. At certain times, clams are harvested from the closure area; these clams are relocated outside the boundary for depuration and harvesting at a later time. The relocation program also includes the "selling" of seed stock to other Massachusetts communities. In 1989, clams for this relocation program were primarily taken from Inner Harbor, near Palmer's Island.

The resource within the closed areas has been estimated at 520,000 bushels (Conservation Law Foundation, 1988). It is generally found in harvestable densities in the southern end of the Inner Harbor, and throughout the Outer Harbor and Clarks Cove out to about the 11-m (35-foot) depth contour. [Volume IV of the draft Phase 2 WWTP Facilities Plan (CDM, 1989) contains a discussion of the results from field surveys in the study area.] Patches of lower densities do exist within this area, such as at the terminus of the existing 60-inch WWTP discharge pipe and on the rock ledges (which are the wrong habitat), but these areas are small.

The other bivalve species (soft-shell clams, oysters, scallops) are found in the shallow bays and soft-bottom intertidal area around the harbor. The areas where the populations of these species have been found, which are within the boundaries under the jurisdiction of New Bedford (Bourque, 1989), are identified on Figure 5-4. The natural abundance of these species does not appear to be large enough to support a major commercial fishery because their habitat is limited in size. The resources, however, could support recreational harvesting if the closure resulting from coliform pollution is lifted. Soft-shell clams are usually found in the intertidal zone and shallow subtidal zone in sandy muds and fine sands. Oysters need a hard substrate to grow and at present are found mostly on

rock bulkheads and pilings in the Inner Harbor. Their distribution is limited to the intertidal zone by predation from Oyster Drills (an introduced species from Japan). Scallops are found on shallow subtidal bottoms, especially in eel grass beds. All of these habitats are not very extensive in the New Bedford area.

Although natural scallop populations seem to be sparse, the harvest of this species is currently being enhanced by the addition of hatchery-reared young in Clarks Cove. The New Bedford shellfish warden purchases young scallops (seed), grows them to a larger size in rafts, and then releases them for subsequent harvest by local shellfishermen (Bourque, 1989).

5.4.2 RECREATIONAL FACILITIES

Recreational activities are common in the coastal region of New Bedford. These activities include swimming, boating, water skiing, recreational fishing and shellfishing, surfing, and SCUBA diving. The major recreational activities in the study area are discussed below.

Inner Harbor and Acushnet Estuary

The direct public use of the Inner Harbor and Acushnet Estuary is limited to recreational boating, including the mooring of approximately 50 pleasure boats. Recreational and tourist activities take place at numerous locations directly adjacent to these waters, particularly along the historic waterfronts of New Bedford and Fairhaven.

The Inner Harbor, a major commercial resource, is the largest fishing port in the United States in terms of annual tonnage landed; more than 250 commercial fishing vessels operate from New Bedford.

Clarks Cove and Outer Harbor

The primary public uses of Clarks Cove and the Outer Harbor are swimming, boating, sport fishing, and (limited) shellfishing. Bathing beaches are located along all shores of Clarks Cove and the Outer Harbor; these

facilities are shown in Figure 5-6 and identified in Table 5-4. The 1986 person-day use of the East and West Beaches operated by the City of New Bedford was 90,630 person-days. Use of Fairhaven's Fort Phoenix public beach in 1986 was estimated at 46,941 person-days, while municipal beaches sold 1,051 beach permits (carloads) with an estimated ridership of 4 to 5 per car. The Parks Department of the Town of Dartmouth sold 4,709 beach stickers for automobiles and mopeds, with an additional 54 special permits issued to individuals (CDM, 1987).

Massachusetts DEP regulations require that a beach be closed if the total coliform concentration exceed 1000 MPN/100 ml. Although relevant to current and anticipated uses of the water bodies, the standards of the SA classification are not consistently met due to bacteriological contamination. While most of these water bodies have been closed to shellfishing, beach closings have been exceedingly rare, with a maximum closure of one day after the worst rain storm.

According to City of New Bedford public health officials, no Clarks Point beaches have been closed due to high coliform counts in the last 6 or 7 years. The Fairhaven Health Board reported no recent closures of either the Ft. Phoenix State Park Beach or the West Island Public Beach due to high coliform concentrations (Perry, 1987). Although no beaches have been closed, water quality data show that coliform concentrations increased after rain storms. The relationship between high coliform bacteria counts and rain events is examined further in Section 5.5.

The Outer Harbor and Clarks Cove are popular areas for recreational boating. Boat ramps and landings are located along the shores of both water bodies; these are shown in Figure 5-6. Most marinas and commercial piers are located in the Inner Harbor; however, pleasure boats are moored along the shorelines of Clarks Cove and the Outer Harbor.

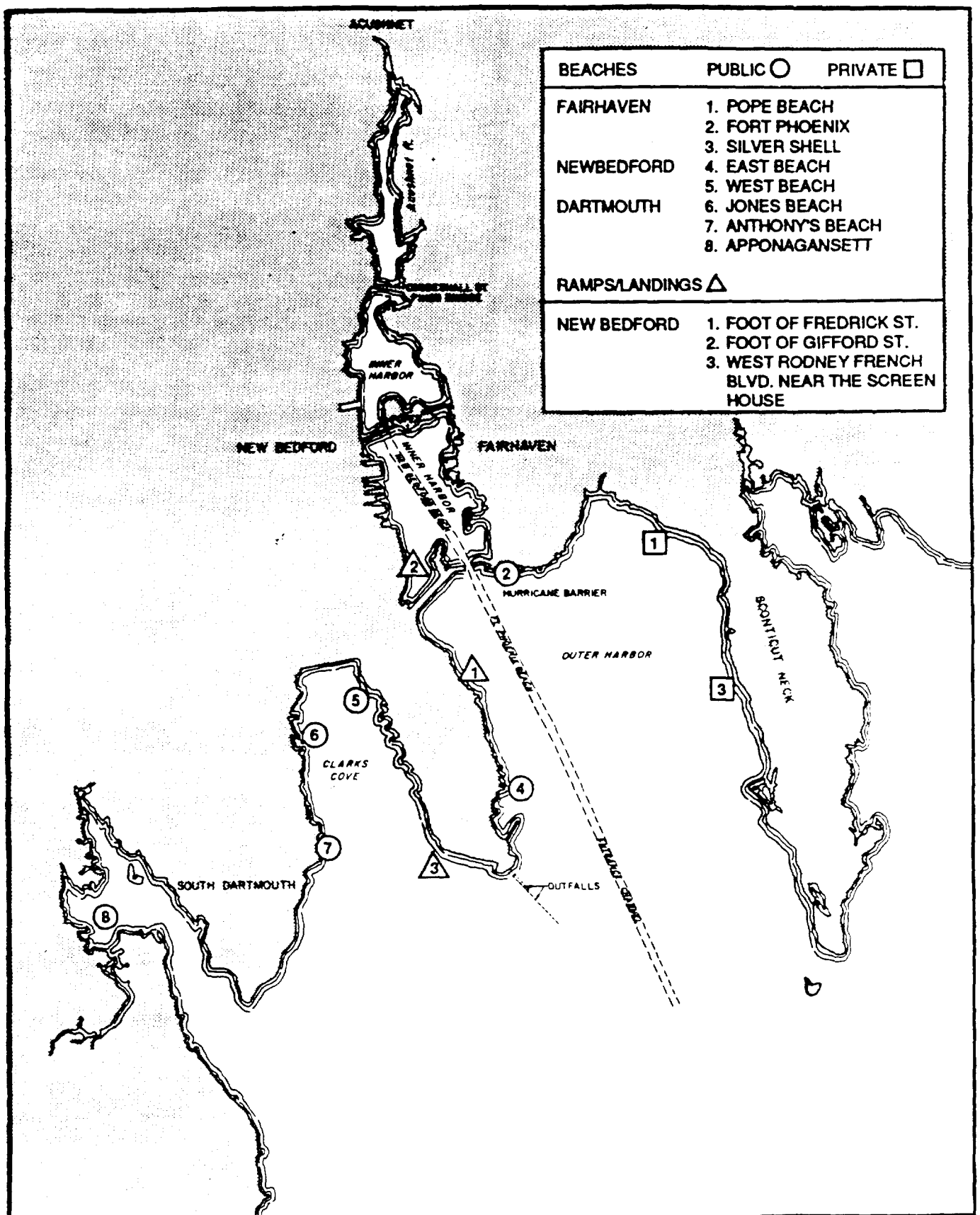
Sport fishing for scup, bluefish, striped bass, and Atlantic mackerel occurs in New Bedford's Outer Harbor. The actual number of recreational fishermen is not known. Although the taking of many shellfish species is prohibited, recreational scalloping is permitted.

TABLE 5-4

BEACHES, PUBLIC DOCKS AND LANDINGS IN NEW BEDFORD HARBOR[1]

Beaches	
Fairhaven	1. Pope Beach (private)
	2. Fort Phoenix (State/Town)
	3. West Island (public)
	4. Silver Shell (private)
New Bedford	5. East Beach (public)
	6. West Beach (public)
Dartmouth	7. Jones Beach (public)
	8. Anthony's Beach (private)
	9. Apponagansett (public)
	10. Round Hill Beach (public)
Ramps/Landings	
New Bedford	1. Foot of Frederick Street
	2. Foot of Gifford Street
	3. West Rodney French Boulevard near the screen house

[1]Locations as shown in Figure 5-6.



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 5-6
RECREATIONAL FACILITIES

5.4.3 NAVIGATION CHANNELS AND ANCHORING SITES

The major navigation channels in New Bedford Harbor are marked and maintained by the U.S. Coast Guard. The Inner Harbor is connected to Buzzards Bay by a dredged and marked channel running south-southeast from the hurricane barrier. From the mouth of the dredged area the marked channel continues to the east of Negro Ledge and west of Mosher Ledge. Another buoyed approach to the harbor is between Dumpling Rocks and the Sandspit. The navigation channels into and out of the New Bedford Harbor and Apponagansett Bay that are marked by buoys are shown in Figure 5-7.

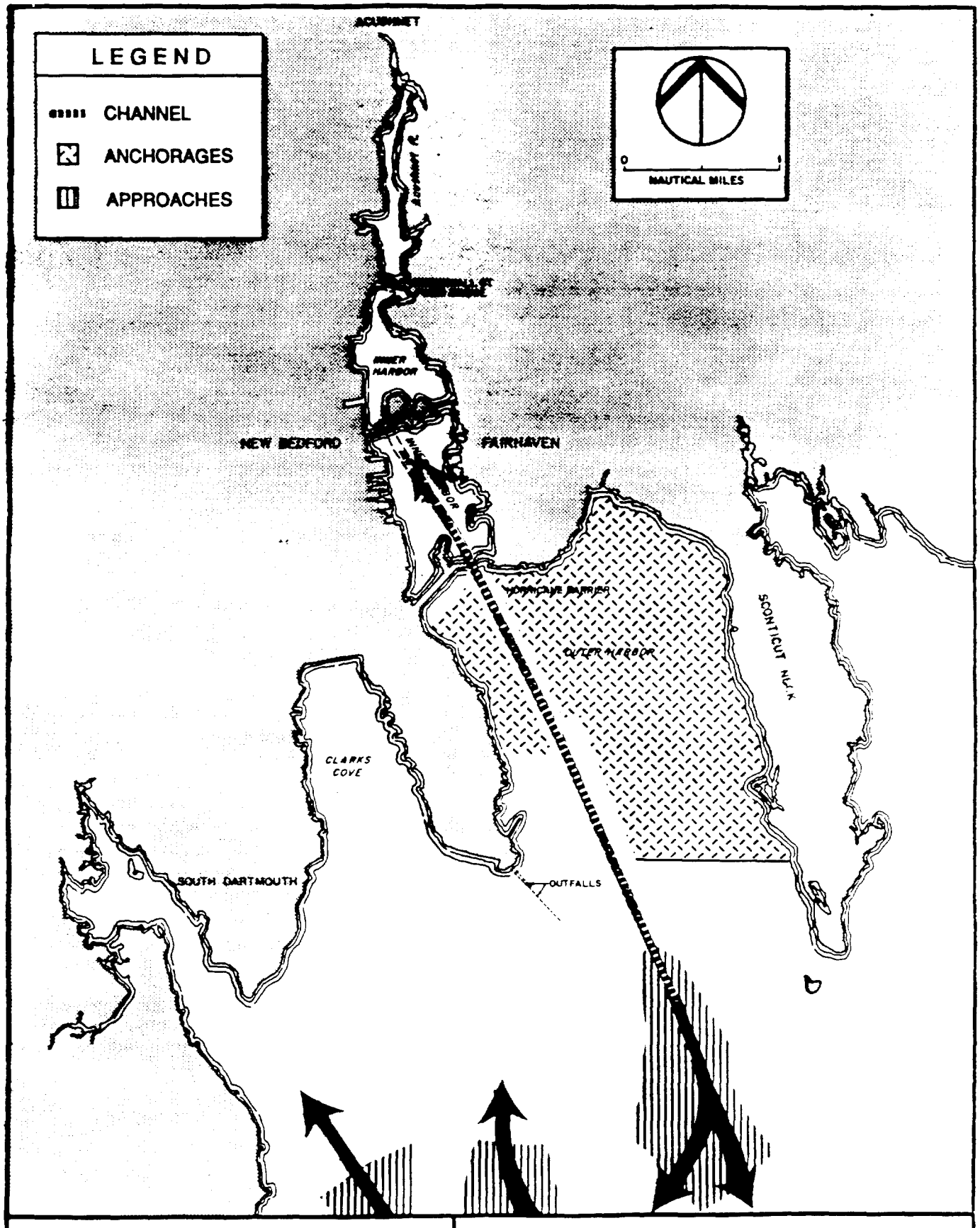
Anchorage sites may be found on either side of the marked channel in the Outer Harbor. Approximately 40 pleasure boats are moored in Clarks Cove; another 40 are anchored in the Outer Harbor.

5.4.4 ENDANGERED SPECIES

Table 5-5 lists the endangered or threatened species, as published in 44 FR 3636, that may possibly inhabit or obtain nutrients from the waters of Buzzards Bay and New Bedford Harbor.

50 CFR 17.95, 19.96 or Part 226 do not designate any part of the study area as critical habitat for these species. In fact, none of the whales listed have recently been observed or recorded within Buzzards Bay (Prescott, 1988).

Although not published on state species lists, several endangered and threatened turtle species are known to be summer residents in Buzzards Bay. These are the loggerhead sea turtle (*Caretta caretta*, threatened), Atlantic Ridley sea turtle (*Lepidochelys kempii*, endangered), and the leatherback sea turtle (*Dermochelys coriacea*, endangered). In 1987, five leatherback turtles were found stranded on shores near Dartmouth and one loggerhead turtle was hit by a boat off Bourne. Sporadic siting of Ridley turtles occurs in Buzzards Bay, but no strandings have been recorded (Prescott, 1988).



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 5-7
MAJOR NAVIGATIONAL CHANNELS AND APPROACHES

TABLE 5-5

ENDANGERED SPECIES

Common Name	Scientific Name	Population	Known Distribution	Portion of Range Where Threatened or Endangered
Sturgeon, shortnose	*Acipenser breviostrum*	N/A	USA: Atlantic Coast-US/Canada	Entire
Whale, Blue	*Balaenoptera musculus*	N/A	Oceanic	Entire
Whale, bowhead	*Balaena mysticetus*	N/A	Oceanic	Entire
Whale, finback	*Balaenoptera physalus*	N/A	Oceanic	Entire
Whale, gray	*Eschrichtius gibbosus*	N/A	Oceanic	Entire
Whale, humpback	*Megaptera novaengliae*	N/A	Oceanic	Entire
Whale, right	*Eubalaena spp.* (all species)	N/A	Oceanic	Entire
Whale, Sei	*Balaenoptera borealis*	N/A	Oceanic	Entire
Whale, sperm	*Physeter catodon*	N/A	Oceanic	Entire

No federally listed and proposed endangered or threatened species exist in the study area (letter of June 21, 1988 from G.E. Beckett, U.S. Fish and Wildlife service to G. Ruta, EPA). Buzzards Bay and some of its associated tidal inlets are a major habitat for the piping plover (Charadrius melodus), a federally endangered species. The plover nests, feeds, and rests on seven beaches near New Bedford (West Island Beach in Fairhaven, Barneys Joy, and Little Beach in Dartmouth, and Gooseberry Neck, Horseneck Beach, Acoaxet Beach, and Richmond Pond in Westport).

5.5 WATER QUALITY IMPACTS UNDER EXISTING CONDITIONS

The New Bedford area has had water quality problems for much of this century. The Inner Harbor area has been closed to shellfishing due to pollution since 1925. The discovery of PCBs in the mid 1970s led to a ban of all fishing activities in the Inner Harbor. Outer New Bedford Harbor and Clarks Cove were closed to shellfishing because of pollution in 1971. High PCB levels in 1979 led to a further ban on lobster and bottom-feeding fish. These closures are still in effect today due to recurring pollution problems.

The aforementioned toxic substances pollution problems have led to New Bedford Harbor being placed on the EPA's Superfund list because of toxic chemicals in the sediments. The Superfund list is a listing of hazardous waste sites in the United States that demand immediate remedial action.

The purpose of this section is to examine existing water quality in the study area and attempt to determine whether CSOs contribute significantly to present problems. To do this, it is first necessary to describe the primary sources of contaminants to the study area.

5.5.1 SOURCES OF CONTAMINANTS

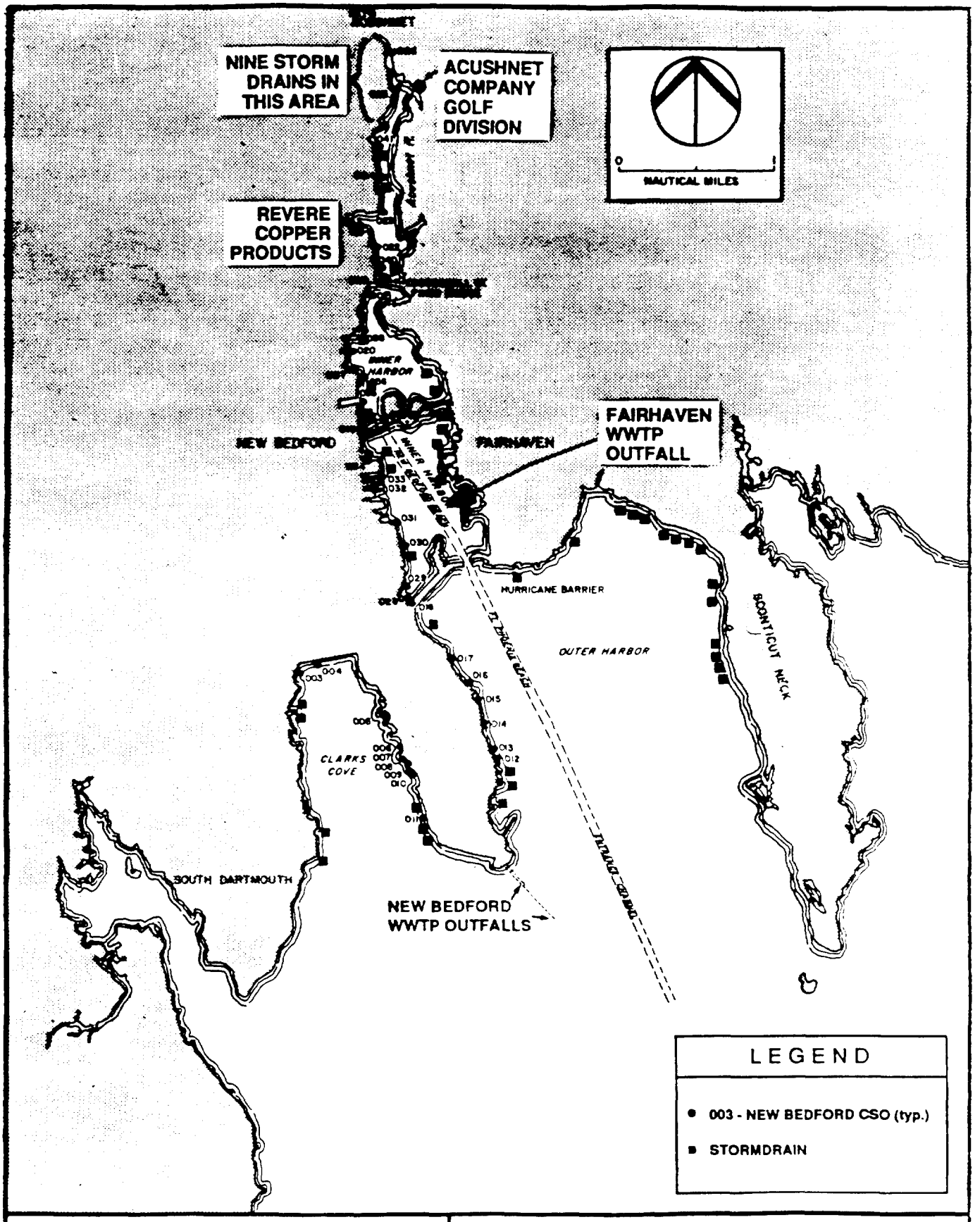
New Bedford Harbor receives wastewater from several sources along its coastline. In addition to the discharges from the New Bedford CSOs, municipal plants in the project study area are the New Bedford primary WWTP and the Fairhaven secondary WWTP. (Dartmouth's WWTP discharges through an

ocean outfall located about 3,000 feet south of Salters Point, which is about 4 miles south of the study area.) Other sources of contaminants to the study area include industrial dischargers and 55 storm drains. Figure 5-8 shows the location of these discharges. The highly contaminated sediments of the Inner Harbor are also a source of contaminants to the receiving water.

Municipal WWTPs

New Bedford WWTP. The New Bedford wastewater system serves about 60 percent of the area of the City of New Bedford, and 98 percent of the population. In addition, the system accepts flow from about 600 dwelling units in the Town of Dartmouth and 60 units in Acushnet. Wastewater is treated at the existing primary plant at Fort Rodman and is discharged into the harbor via two outfall pipes, as shown in Figure 5-8. The main outfall is a 60-inch diameter cast iron pipe that extends about 3,300 feet south-southeast from the tip of Clarks Point. It was constructed in the 1920s when the interceptor system was installed. The second outfall is an auxiliary pipe of 72-inch diameter. Constructed in 1974, it is a prestressed concrete pipe extending about 1,000 feet from the tip of Clarks Point into the Outer Harbor. The existing plant's chlorine contact tanks contain a flow-splitting weir arrangement that directs wastewater flow to the two outfall pipes. Wastewater is first discharged through the main (60-inch) outfall and then to the auxiliary (72-inch) outfall when flow exceeds the capacity of the main outfall.

The recently completed draft Phase 2 WWTP Facilities Plan (CDM, 1989) recommends that a new 75-mgd secondary treatment plant be located at Fort Rodman to the northwest of the present plant. It also recommends that the existing 60-inch outfall be rehabilitated for continued use as the discharge pipe for the WWTP. The rehabilitation will entail cleaning the pipe and installing a high density polyethylene liner pipe with a nominal pipe size of 54 inches. The new WWTP will include an effluent pump station to allow the larger flows to be discharged out the outfall pipe.



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 5-8
MAJOR WASTEWATER AND STORMWATER DISCHARGE LOCATIONS

Fairhaven WWTP. The Fairhaven WWTP provides secondary treatment for largely domestic wastewater from Fairhaven and the neighboring town of Mattapoisett. An expansion from 2.1 to 5 mgd is under construction in 1989. The effluent discharges into the lower Inner Harbor through a 9,300-foot long, 36-inch diameter outfall.

CSOs and Storm Drains

New Bedford's wastewater system has 35 active CSOs, most of which discharge during wet weather. They are shown in Figure 5-8. The combined sewer service area is about 14 km^2 (3,440 acres). Overflows from this system discharge to the Acushnet River, New Bedford's Inner and Outer Harbors, and Clarks Cove. These CSOs are the subject of this facilities plan.

The towns adjoining New Bedford have predominantly separated sewer systems. Dartmouth, Fairhaven, and Acushnet have separate sewer and storm drain systems. The Phase I CSO Facilities Plan (CDM, 1983a) identified 55 storm drains discharge to the study area (currently 58), and noted that 6 of the storm drains (2 in New Bedford and 4 in Acushnet) were contaminated by sanitary sewage. The distribution of these drains by town and by receiving water is given Table 5-6. Flows and contaminant loads of the storm drains was discussed in Sections 4.0 and 6.0.

Industrial Discharges

Buzzards Bay Wastewater Discharge Survey Data for 1982, 1983, 1985, 1986, and 1987 collected by DWPC (1988) list industrial wastewater dischargers in the study area. Two of of the discharges have significant flows and are discussed below. Three other industrial dischargers -- Acushnet Nursing Home, Acushnet Capacitor Co., and Commonwealth Electric Company -- have small effluent flow rates (less than 20,000 gpd); thus, they are not considered in this study.

The Acushnet Rubber Company, Golf Division's industrial discharge consists of wastewater from a blasting operation and non-contact cooling water. It

TABLE 5-6

STORM DRAIN LOCATIONS

Town/Receiving Water	Number of Storm Drains
DARTMOUTH	
Clarks Cove	4
NEW BEDFORD	
Clarks Cove	3
Outer Harbor	4
Lower Inner Harbor	3
Middle Inner Harbor	4
Upper Inner Harbor	6
	20
ACUSHNET	
Upper Inner Harbor	5
FAIRHAVEN	
Outer Harbor	15
Lower Inner Harbor	6
Middle Inner Harbor	3
Upper Inner Harbor	2
	26

releases less than 0.5 mgd into the Acushnet River estuary. Sanitary and process wastewater are discharged to the municipal sewer system.

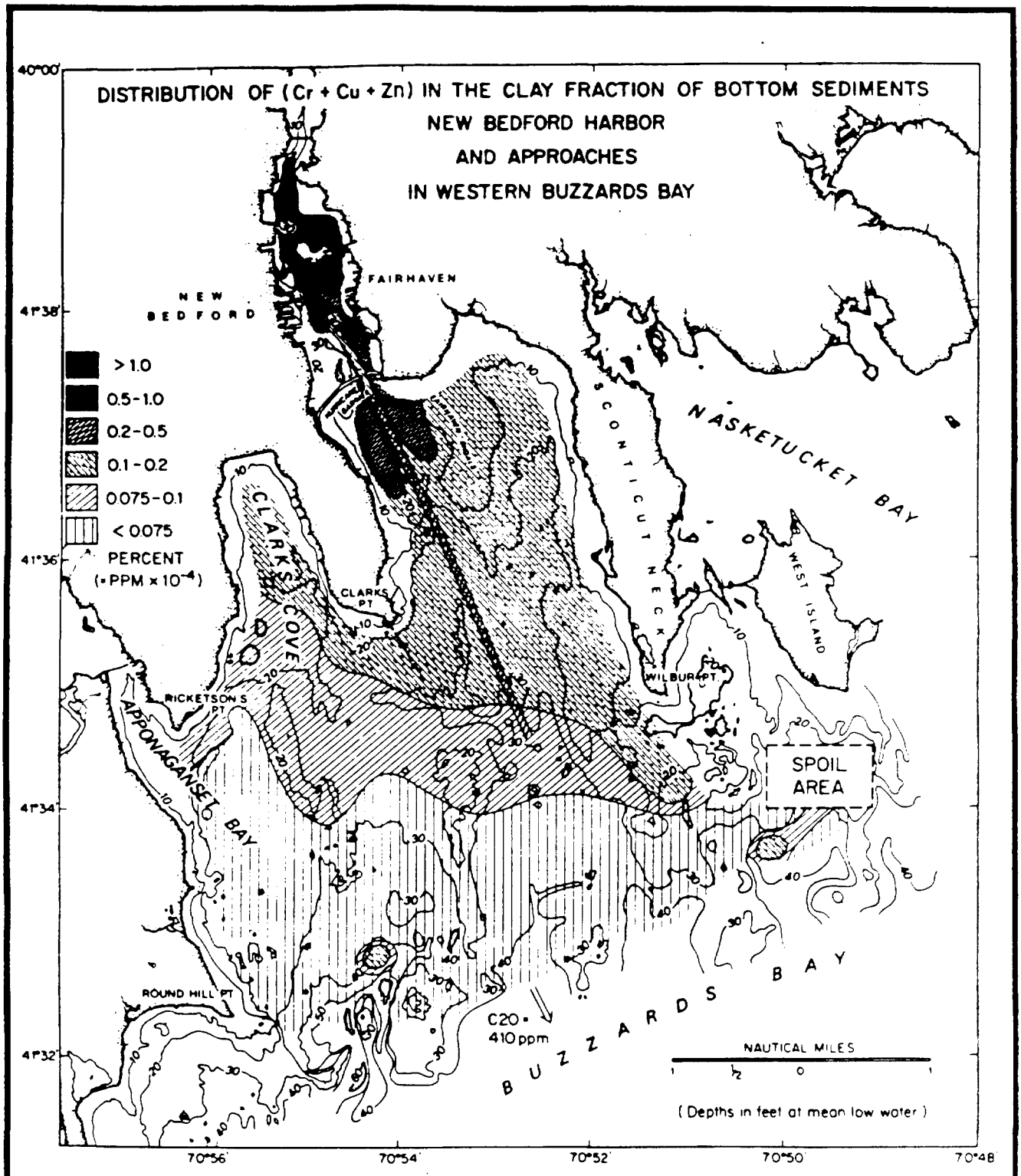
Two outfalls from the Revere Copper Products have NPDES permits to discharge treatment effluent into the Acushnet River. The larger flow (0.46 mgd) includes non-contact cooling water, storm water and treated process wastewater. The smaller flow from Revere Copper Products combines oil-laden wastewater with raw sanitary wastewater, passes it through an oil separator, and discharges through an outfall.

Contaminated Harbor Sediments

As discussed above, New Bedford Harbor is an EPA Superfund site. The primary reason for its inclusion on the Superfund list is the highly contaminated sediments, especially by PCBs. Other major contaminants of sediments that have been identified are copper, lead, chromium, arsenic, and zinc. The highest concentrations of PCBs in sediments in the study area are near the Aerovox site in the upper Inner Harbor and near the Cornell-Dubilier site on the east side of Clarks Point just south of the hurricane barrier.

Summerhayes et al. (1977) found that New Bedford sediments, especially the fine-grained ones, are contaminated with copper, chromium, lead, and zinc. They developed a relationship using the concentrations of these metals (Cu+Cr+Zn against Pb) to suggest which sediments were highly contaminated and which were less contaminated or uncontaminated. The data indicated a sharp change at about 1000 ppm Cu+Cr+Zn, which occurs just seaward of Clarks Point and trends more or less NW-SE across the Outer Harbor (Figure 5-9). This 1000 ppm contour also included most of Clarks Cove. Figure 5-9 also showed the gradient of contaminated sediments in the Inner Harbor with the highest concentrations being found in the northern end. In one location in the Inner Harbor, the Cu+Cr+Zn concentration comprised about 1.2 percent of the sediments.

More recent data on sediment contaminant levels were collected during the 1984 to 1986 surveys for the EPA Superfund project. While these data are



From: Reproduced Summerhayes et al. (1977)

City of New Bedford COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3	FIGURE 5-9 DISTRIBUTION OF (Cr + Cu + Zn) IN THE CLAY FRACTION OF BOTTOM SEDIMENTS

unpublished, some values are available; they are discussed in detail in Volume IV of the draft WWTP Phase 2 Facilities Plan (CDM, 1989). For the most part, these data confirm the trends found by Summerhayes et al., especially in terms of decreasing PCB concentrations with increasing distance from the Inner Harbor (and indeed in a downstream direction within the Inner Harbor itself).

The high levels of contaminants in the sediments in New Bedford Harbor contribute to the concentrations of the chemicals in the receiving water. This can happen in several ways; some examples are: the equilibrium partitioning of contaminants between the sediment and the water column and resuspension of bottom sediments because of wave action.

5.5.2 EXISTING WATER QUALITY

As discussed above, several sources of contaminants contribute to the degradation of water quality in the study area. Many of the State's water quality standards appear not to be met in the receiving waters of the study area, including: coliform bacteria, dissolved oxygen, toxic pollutants such as metals and PCBs, aesthetics such as floating debris and scum, and oil and grease.

Because existing CSO discharges are untreated, they are probably significant contributors to some of the water quality problems. Clearly, untreated CSO discharges can contribute to violations of state standards for aesthetics and oil and grease. Together with storm water discharges, CSOs add substantially to bacterial contamination in the receiving waters. A more detailed discussion of the possible relationship with CSO discharges and dissolved oxygen, toxics, and coliform bacteria follows.

Dissolved Oxygen

The relationship between CSO discharges and dissolved oxygen levels is more difficult to establish. Most of the DO measurements in New Bedford Harbor were taken in the vicinity of the existing WWTP discharge, which is outside of the CSO study area (DWPC, 1980; CDM, 1983; Howes and Taylor, 1989). Of

those measurements in the study area, 8 sampling stations were located in the Outer Harbor; no historical DO data could be found for the Inner Harbor or Clarks Cove.

The data in the Outer Harbor include some values (5 out of 42 profiles) that indicate violations of the state's Class SA water quality criteria of 6.0 mg/l. The lowest DO value and the date for each of the 5 profiles were: 3.5 mg/l (8/5/80); 5.2 mg/l (8/12/80); 3.9 mg/l (9/16/80); 5.1 mg/l (7/18/88); 5.5 mg/l (8/15/88). The 1980 data were collected measured in near the shipping channel east of the end of Clarks Point. One of the 1988 profiles was taken near Butler Flats, and the other was taken off of Popes Beach on Sconticut Neck. The draft Phase 2 WWTP Facilities Plan (CDM, 1989; see Volume IV, Section 4.4) includes a more detailed discussion of DO data in New Bedford Harbor.

An examination of the data in the greater Outer Harbor (CDM, 1989; Volume IV, Section 4.4) concluded that DO levels did not vary significantly with proximity to the WWTP outfall. This fact suggested that any oxygen-depleting effects of the outfall are quickly dispersed throughout the surrounding area and do not center on the outfall. The same is probably true of CSO discharges, where the BOD discharged by CSOs is well mixed in the Outer Harbor. Thus, it is difficult to separate oxygen depletions due to CSOs, storm drains, and the WWTP discharge, etc. A rough approximation, however, of the contribution of CSO discharges to DO depressions can be derived by examining the magnitudes of annual BOD loading from the various sources (see Section 4.0).

Coliform Bacteria

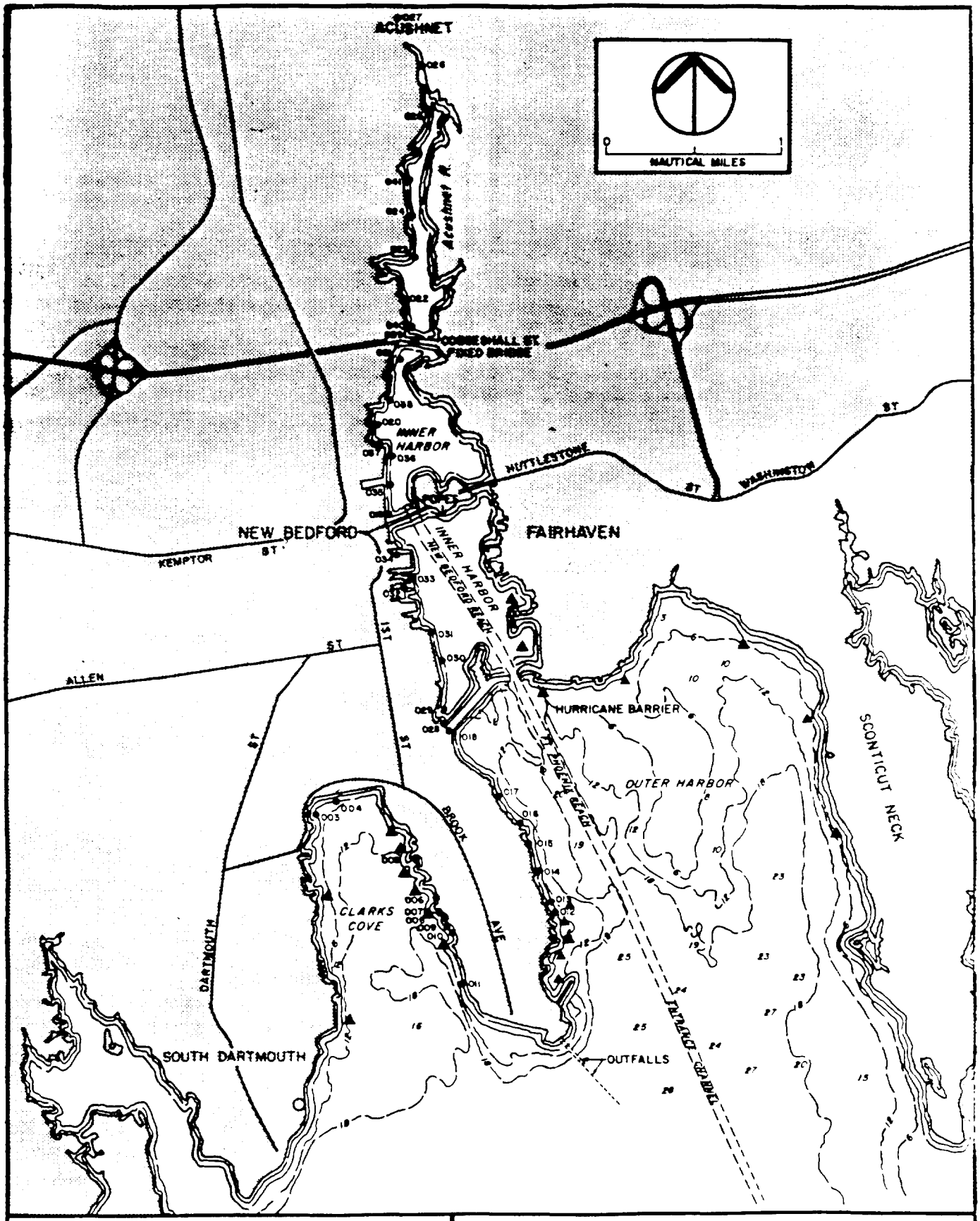
Coliform bacteria are common in sewage and are discharged in large numbers by CSOs. To assess the impact of existing CSO discharges on beach closings, existing data (post 1980) on coliform levels in the water were compiled. Primary sources for the data were files from the Board of Health in New Bedford, Fairhaven and Dartmouth; the same testing method (the MPN method) is used by each town's board. In addition, there were several

one-time, or sporadic studies done by other agencies. Coliform sampling stations, for all studies consulted, are shown on Figure 5-10.

The data were analyzed by selecting the sampling dates where high coliform counts (in excess of the state water quality criteria [70 MPN/100 ml]) were recorded. Table 5-7 is a summary of all water quality sampling during the summer conducted by New Bedford, Dartmouth and Fairhaven Board of Health officials. The New Bedford data were more extensive (5 years of data, generally sampled once a week during the summer). Samples are usually collected mid-week (Tuesday or Wednesday) in the morning or early afternoon. Sampling in Dartmouth and Fairhaven is more sporadic. This difference between the communities is shown in Table 5-7 which summarizes the total number of of samples collected by each community in a given year.

For coliform values that exceed the water quality criteria, the sampling dates were compared to rainfall records to determine if a rainfall event occurred within 72 hours of the sampling. The 72-hour period was selected based on projected die-off rates and estimates of transport. Many, if not most, of the high coliform bacterial counts were sampled within 72 hours of a rain event. Other sources of coliform bacteria, such as dry weather overflows, may account for some of the values that did not match rainfall events.

Examples of rainfall related coliform counts are given in Tables 5-8 through 5-10 for New Bedford, Dartmouth, and Fairhaven, respectively. For New Bedford (Table 5-8), only the top ten storms, in terms of rainfall quantity, (when coliform samples were collected) were included. For these ten storms, the data show that, on average, half of the 19 sampling stations exceed the state water quality criteria. The data for Dartmouth and Fairhaven are more difficult to interpret because of less frequent sampling. In general, the Dartmouth and Fairhaven data show fewer violations of coliform bacteria criteria for those samples taken within 72 hours of rainfall events. This may be attributable to the prevailing winds and/or the distance of sampling stations from CSOs and storm drains. There are, however, too few data to draw defensible conclusions.



City of New Bedford

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FIGURE 5-10
COLIFORM BACTERIA SAMPLING STATIONS

TABLE 5-7

INCIDENCE OF TOTAL COLIFORMS EXCEEDING 70 MPN/100 ml

NEW BEDFORD, MASSACHUSETTS BEACHES

Station	1980	1981	1982	1983	1984	1985	1986	1987	1988
East Beaches	5[1]/96[2]	8/96	10/88	20/96	14/104	5/104	21/104	19/96	6/80
West Beaches	7/132	24/132	29/121	29/132	29/143	24/143	17/143	6/132	10/110

DARTMOUTH, MASSACHUSETTS BEACHES

Station	1985[3]	1986	1987	1988
Anthony's Beach	1/2	0/4	0/6	0/5
Jones Beach	1/2	0/4	0/6	0/5

FAIRHAVEN, MASSACHUSETTS BEACHES

Station	1985	1986	1987	1988
Fort Phoenix	1/6	4/6	1/7	1/8
Manhattan Avenue	4/6	0/6	1/7	1/8
James St./Boulder Park	1/6	0/1		
Reservation Road	0/6	0/6	1/7	0/8
Silver Shell Beach			0/1	
Winsegansett Avenue	0/5			
Wilbur Point	2/6	1/6	0/7	1/8

Notes:

(1) Represents the number of samples exceeding 70 MPN/100 ml.

(2) Represents the total number of samples.

(3) Based on Fecal Coliform counts exceeding 14 MPN/100 ml.

TABLE 5-8

RAINFALL RELATED COLIFORM COUNTS
NEW BEDFORD, MASSACHUSETTS

Sample Date	No. of Days From Last Rainfall	Rainfall (inches)	No. of Stations >70 MPN/100 ml
8/03/88	3	0.50	5[1]/19[2]
7/20/88	0	1.84	4/19
8/12/87	3	0.91	2/19
7/22/87	2	0.43	3/19
7/29/86	1	1.33	19/19
7/02/86	0	1.73	5/19
7/10/85	1	0.66	14/19
6/27/84	2	3.62	15/19
6/29/83	1-2	2.11	15/19
7/21/81	0	0.59	10/19

NOTES:

(1) Represents the number of samples that exceeded 70 MPN/100 ml

(2) Represents the total number of samples

TABLE 5-9

RAINFALL RELATED COLIFORM COUNTS
DARTMOUTH, MASSACHUSETTS

Sample Date	No. of Days From Last Rainfall	Rainfall (inches)	No. of Stations >70 MPN/100 ml
8/25/88	1	0.68	0[1]/2[2]
7/27/88	0	2.06	0/2
6/02/88	1	0.30	0/2
8/06/87	3	0.23	0/2
6/24/87	1	0.33	0/2
6/10/87	3	0.76	0/2
8/14/86	3	0.44	0/2
7/29/85	0	1.02	1/2

NOTES:

(1) Represents the number of samples that exceeded 70 MPN/100 ml

(2) Represents the total number of samples

TABLE 5-10

RAINFALL RELATED COLIFORM COUNTS
FAIRHAVEN, MASSACHUSETTS

Sample Date	No. of Days From Last Rainfall	Rainfall (inches)	No. of Stations >70 MPN/100 ml
8/02/88	2	0.50	0[1]/4[2]
9/01/87	0	0.66	0/4
8/04/87	1	0.23	1/4
7/22/87	2	0.43	1/4
6/23/87	0	0.33	0/3
8/12/86	1	0.44	0/4
7/15/86	1-3	2.17	0/3
6/13/86	1	0.85	1/3
8/27/85	2	1.87	3/5
7/16/85	0	1.03	1/5
7/02/85	3	0.42	2/4
6/17/85	1	0.42	2/4

NOTES:

(1) Represents the number of samples that exceeded 70 MPN/100 ml

(2) Represents the total number of samples

The relationship between winds and coliform bacteria levels is addressed further in the Section 6.0.

Based on the above analysis, high coliform counts in the waters of Clarks Cove and the Outer Harbor are often measured within a few days following storm events (although not high enough to produce long-term beach closings as discussed in Section 5.4). While historical sampling data for the Inner Harbor were sparse, sampling conducted as part of this facilities plan (see Appendix B1) shows a high coliform bacteria counts during and immediately following rain events in the Inner Harbor. These results are expected as CSOs, along with dry weather overflows and storm drains, are considered to be the major sources of bacterial contamination in the study area.

Toxic Pollutants

The current water quality standards for toxic substances are qualitative. As discussed in Section 5.2, the state has not developed a policy on interpreting its qualitative standard with regard to the EPA qualitative criteria, although this issue is being addressed in the revisions to the Massachusetts water quality standards. However, DWPC's interim CSO policy (discussed below) states that pollutant loadings in such concentrations that cause acute toxicity in organisms passing through the mixing zone, or pose a threat to human health are prohibited. In addition, far field (outside the mixing zone) concentrations shall not be acutely or chronically toxic to aquatic life. There are two standard methods for examining acute and chronic toxicity: constituent by constituent analysis and whole effluent toxicity testing.

The individual constituent analysis is performed by examining diluted effluent concentrations against various EPA water quality criteria contained in the "Gold Book." Analyses of CSO effluents conducted as part of this facilities plan (see Appendix A) indicate that several constituents may exceed these water quality criteria following dilution in the receiving water. The toxic substances that have concentrations in the effluent (prior to dilution) that are higher than the EPA criteria are hexachlorobutadiene, di-n-butyl phthalate, butylbenzyl phthalate, bis

(2-ethylhexyl) phthalate, methylene chloride, copper, lead, zinc, chromium, and nickel. Of these chemicals, copper would require the highest dilutions to meet its acute and chronic water quality criteria. Analyses described in Section 6.0 indicate what levels of copper (as a worst case constituent) are expected after initial dilution, and how these levels compare to the water quality criteria at various locations in the receiving waters under both existing and future conditions.

Whole effluent toxicity testing is performed to assess the combined effects of an effluent on aquatic life. Certain aquatic organisms are exposed to varying dilution of the effluent and two concentrations are determined: the LC50, or the concentration at which 50 percent of the organisms die, and the No Observable Effect Limit (NOEL), or the concentration for which there appears to be no effect on the growth of the representative species. Whole effluent toxicity tests were also conducted as part of this facilities plan. The results of this testing and examination of expected receiving water dilutions under existing and future conditions is also found in Section 6.0.

5.6 SUMMARY

This section has described the basis for the receiving water analysis conducted in Section 6.0 by describing regulatory requirements, water quality classifications, existing harbor resources, sources of contaminants in the study area, and how CSO discharges might contribute to existing water quality problems. The discharge of human and industrial wastes over the years has resulted in the degradation of water quality in New Bedford Harbor. The shellfish resources in the Inner Harbor have been closed to harvesting for over 50 years because of high levels of bacterial contamination. More recently, all fishing activities in the Inner Harbor have been prohibited because bottom sediments are highly contaminated with toxic substances, especially PCBs. Toxic substances and bacterial contamination in the Outer Harbor and Clarks Cove have also resulted in restrictions on the harvesting of much of the shellfish and bottom-feeding finfish.

Because of the many sources of contaminants to these receiving waters, it is difficult to assess the role CSO discharges have in reducing water quality. The correlation between rainfall events and high coliform bacteria counts certainly implicates CSO discharges as a significant contributor to the problem. Untreated CSO discharges are also the probable source of much of the floating (unsightly scum and oil and grease) contamination in the study area. The relationship between the toxic substances contamination and CSO discharges, however, is unclear. The major sources of toxic substances (particularly in the sediments) appear to be of industrial origin. The EPA Superfund study in New Bedford, which is not complete, is performing a detailed examination of the toxic substances contamination issues.

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6.0 WATER QUALITY IMPACTS ANALYSIS

6.1 OBJECTIVES

This section presents the methods and findings of the water quality impact analysis. The water quality analysis employs the findings from Section 4.0 "Wet Weather Flows and Loads" and Section 5.0 "Existing Water Quality." The objectives of the water quality impact analysis are:

- To determine whether water quality standards are exceeded by CSO discharges in New Bedford under present conditions and future no-action conditions, and if so, the geographic extent, magnitude, frequency and duration of the violations;
- To determine the improvement of water quality resulting from the implementation of the recommendations in the draft Phase 2 WWTP Facilities Plan (Base Conditions);
- To assess the need and degree of CSO control in order to meet water quality standards; and
- To provide an engineering and environmental basis for judging the compliance of selected alternatives relative to EPA and DEP policies on CSO control.

To achieve these objectives, a receiving water model examining the impacts from CSO flows in New Bedford was developed based on an extensive field program. This section describes the development and validation of the model. The model is applied to examine the existing water quality in the three receiving water bodies: Clarks Cove, the Outer Harbor, and the Inner Harbor. Analytical methods are described in this section to assess the ability of CSO abatement alternatives to meet water quality criteria and other DEP and EPA objectives as discussed in Section 5.0.

6.2 WATER QUALITY CRITERIA

Section 5.0 presented the Massachusetts water quality criteria and examined the criteria that are affected by CSOs. Those that may be currently affected by CSO discharges include:

- Aesthetics
- Dissolved Oxygen
- Coliform Bacteria
- Nutrients
- Toxicity

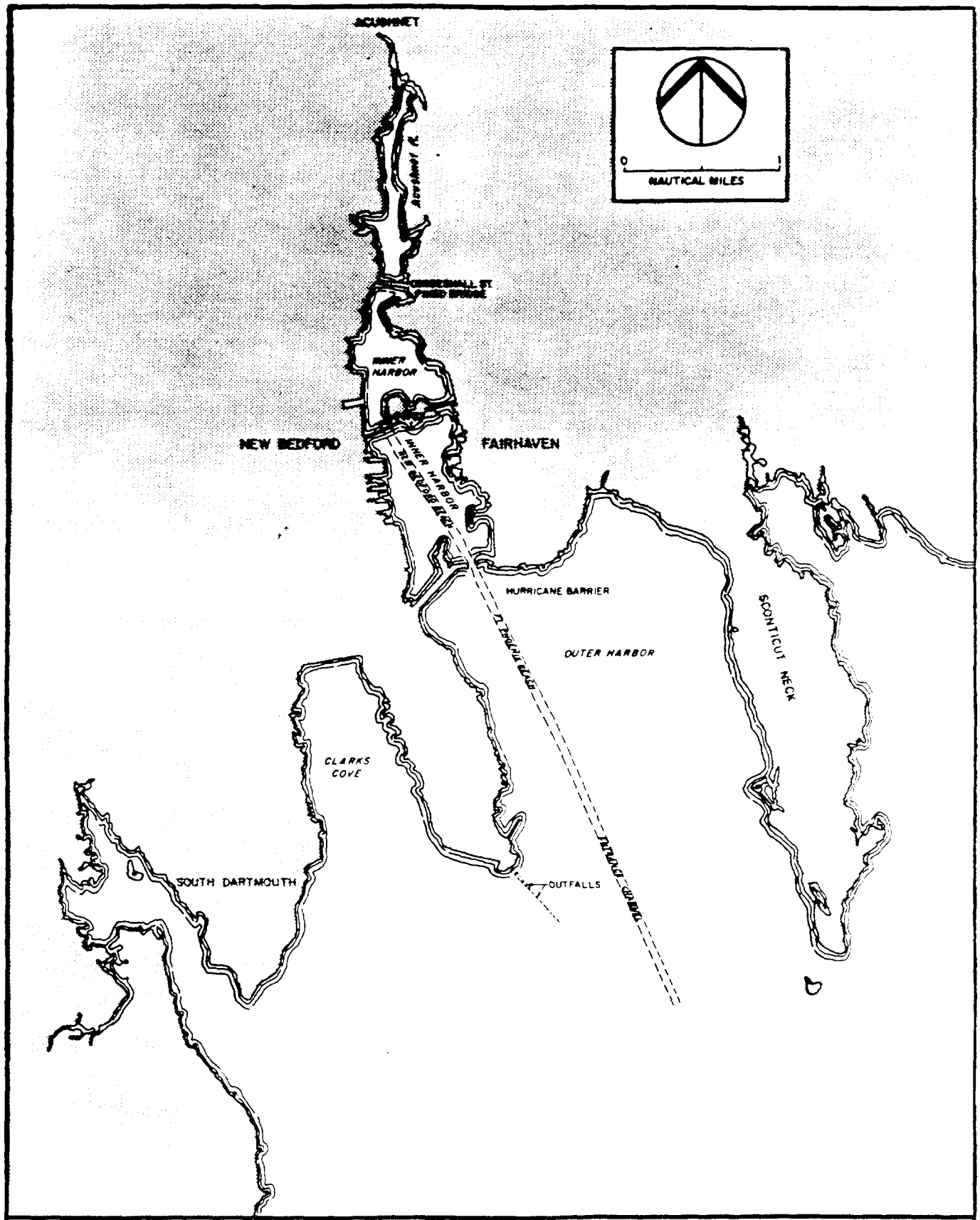
Other sources of pollution -- WWTPs and storm drains -- also contribute pollutants that may result in violation of the above criteria.

This water quality analysis identifies the contribution of the violations (by loading) from CSOs. The focus of the receiving water analysis is on coliform bacteria, toxicity, dissolved oxygen, and nutrients. The achievement of the aesthetics criteria, which includes floatables, scum, and oil, is considered a minimum goal of any CSO abatement plan. A map of the study area is presented in Figure 6-1.

6.3 RECEIVING WATER FIELD PROGRAM

Two field programs were undertaken to determine the important physical and chemical parameters in Clarks Cove, the Outer Harbor, and the Inner Harbor. The first examined the transport and dilution of various CSO discharges by using dye injection. The second program measured the concentrations of total coliform bacteria and ammonia (NH_3) in the receiving waters after a rain event.

As one component of the program, Ocean Surveys, Inc. (OSI), as a subcontractor to CDM, performed dye studies during CSO discharges at six selected CSOs. Appendix B1 contains the OSI report and findings. The dye studies provide information on the transport and dilution of CSO discharges. Four batch releases and two continuous release dye studies were performed. Batches of dye were released at CSO 033, 022, and twice at 004. Continuous releases of dye were made at CSO 037 and 014. The study included measurements of parameters that influence plume buoyancy and trajectory: water temperature, conductivity, and salinity, as well as wind speed and direction.



City of New Bedford

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FIGURE 6-1
STUDY AREA

The second component of the program examined the wet weather concentration of a non-conservative (coliform) and a relatively conservative (ammonia) constituent in the Inner Harbor. Water samples were collected at the Coggeshall Street bridge, the hurricane barrier, and the Popes Island bridge during the storm of May 11, 1989. Appendix B2 contains an analysis of the findings. This information was used to provide an estimate of dilution in the Inner Harbor.

6.4 SOURCES OF POLLUTION

New Bedford Harbor receives pollution discharges from two WWTPs, New Bedford and Fairhaven, five industrial discharges, and numerous storm drains in addition to the CSO discharges.

6.5 EMPIRICAL TRANSPORT AND DILUTION MODEL

In New Bedford, the freshwater CSO discharges are released at or near the surface in shallow bays or in an enclosed harbor. Such conditions result in the formation of freshwater lenses which sit on top of the more saline waters. The dispersion, transport, and dilution characteristics of such lenses are not well understood. A receiving water model was developed because available numerical models were judged to inadequately represent these physical processes that govern movement. A complete description of the receiving water model is contained in Appendix C1. A brief overview of the model is presented here.

6.5.1 DILUTION

CSO discharges are diluted by the mixing that occurs in the receiving water. Mixing is in turn a function of many factors such as waves, the tidal shear velocity, wind driven shear velocities, and molecular dispersion. The process of estimating dilution is simplified if the dilution is measured directly in the field under different conditions. This is the approach taken for this modeling effort. Changes in the concentrations of a conservative dye injected into CSO discharges were monitored for four to six hours to establish the rate of change in

concentration. Dye studies were done at six CSOs in different receiving waters under different storm conditions to obtain an average and range of dilutions for modeling.

6.5.2 TRANSPORT

In general the transport of a freshwater mass in the coastal environment can be considered as independent of the mixing that is taking place within the water mass. Transport is more a function of the actual water velocities rather than of mixing. For this modeling effort, the following variables were evaluated to determine their importance in the transport of a CSO discharge in New Bedford Harbor:

- wind driven surface currents;
- tidal currents;
- initial velocity of the discharge; and
- lateral dispersion.

Wind Driven Surface Currents

Wind driven surface currents in New Bedford Harbor are the most significant variable in the transport of CSO plumes. Wind transfer coefficients were determined using the results of the dye studies and the hourly wind velocities during the actual storms to estimate the speed and direction of travel of discharge events. Wind transfer coefficients determined from the collected data were as follows:

Location	Wind Transfer Coefficient
Open water (Clarks Cove and Outer Harbor)	2.5%
Along shores (Clarks Cove and Outer Harbor)	0.5% to 2.0%
Inner Harbor	<1.0%

Using a transfer coefficient of 2.5 percent and a wind of 10 mph, a surface plume would move 6 miles (9.7 km) in 24 hours in the Outer Harbor or Clarks Cove. For comparison, the distance from Clarks Point to Sconticut Neck is less than 2 miles (3.2 km). This indicates that wind driven surface currents is one mechanism that can transport a CSO discharge across the Outer Harbor.

Hourly wind data records from the New Bedford hurricane barrier are available only from 1985 on. This is too short a period for modeling the less frequent design storms. To produce a longer record, the data from the Providence airport weather station was correlated to the hourly wind record at the New Bedford hurricane barrier to produce a 37-year record. The hourly wind data for each occurrence of the 1-week, 2-week, 1-month, 3-month, 6-month, 1-year and 5-year storm was used in modeling. The Figures 6-2, 6-3, and 6-4 present the 24-hour tracks of 1-hour data with a transfer coefficient of 2.5 percent for the 1-week, 3-month, and 1-year storms respectively.

Several observations can be made from comparing the plots. First, the winds for the 1-week storm are uniformly distributed in all directions, whereas the 3-month and 1-year storms are more in the northeast and southwest directions. This reflects the storm tracks commonly found in New England, where the storms with the heaviest rain tend to be "Nor'easters" or "So'westers." A second observation is that CSO plumes resulting from the larger storms (3-month and 1-year) do not travel a greater distance than those resulting from the small storm (1-week).

Transport in the Inner Harbor is different from that found in the Outer Harbor and Clarks Cove. Wind is important in transport, but only in the direction of the main axis (north-south) of the Inner Harbor. Wind transport was assumed to occur only when the wind direction is along the main axis of the Inner Harbor.

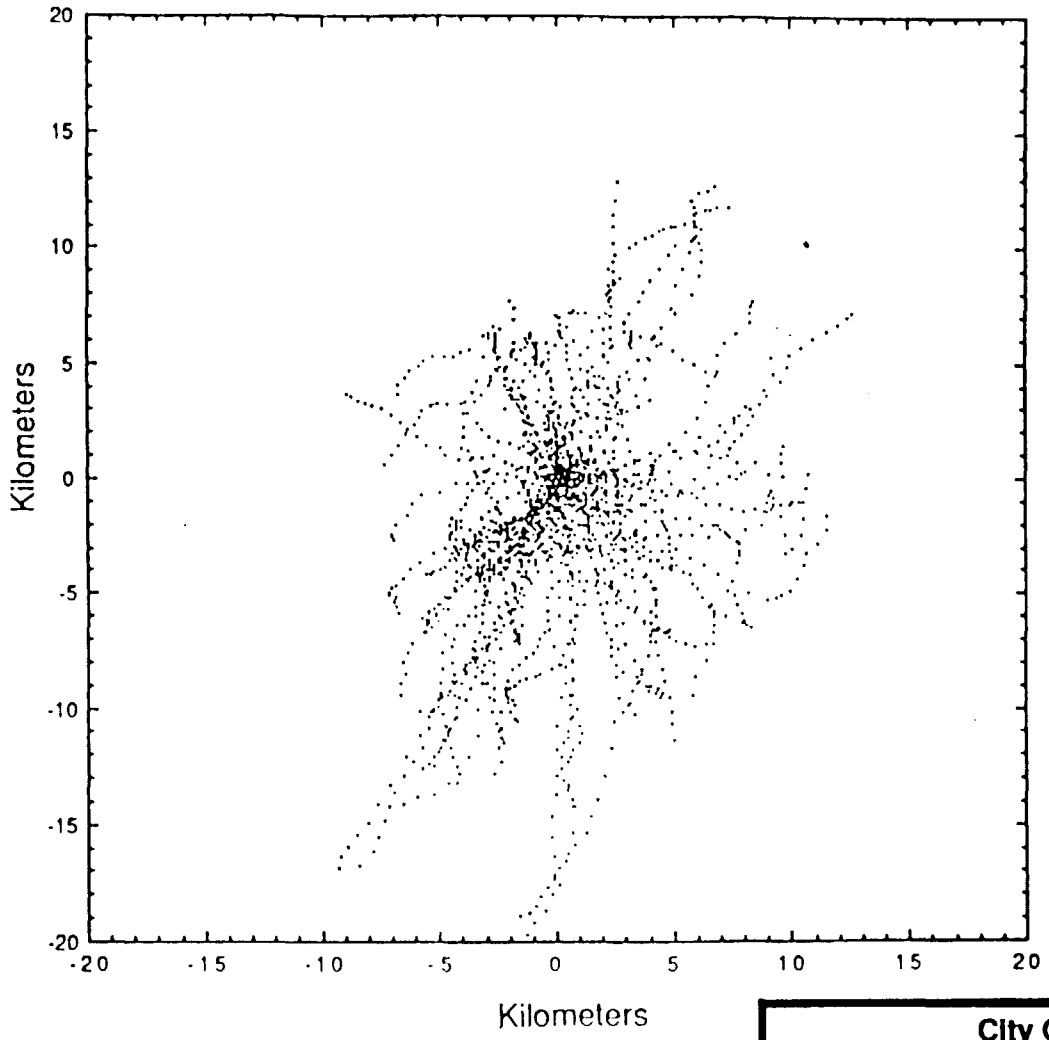

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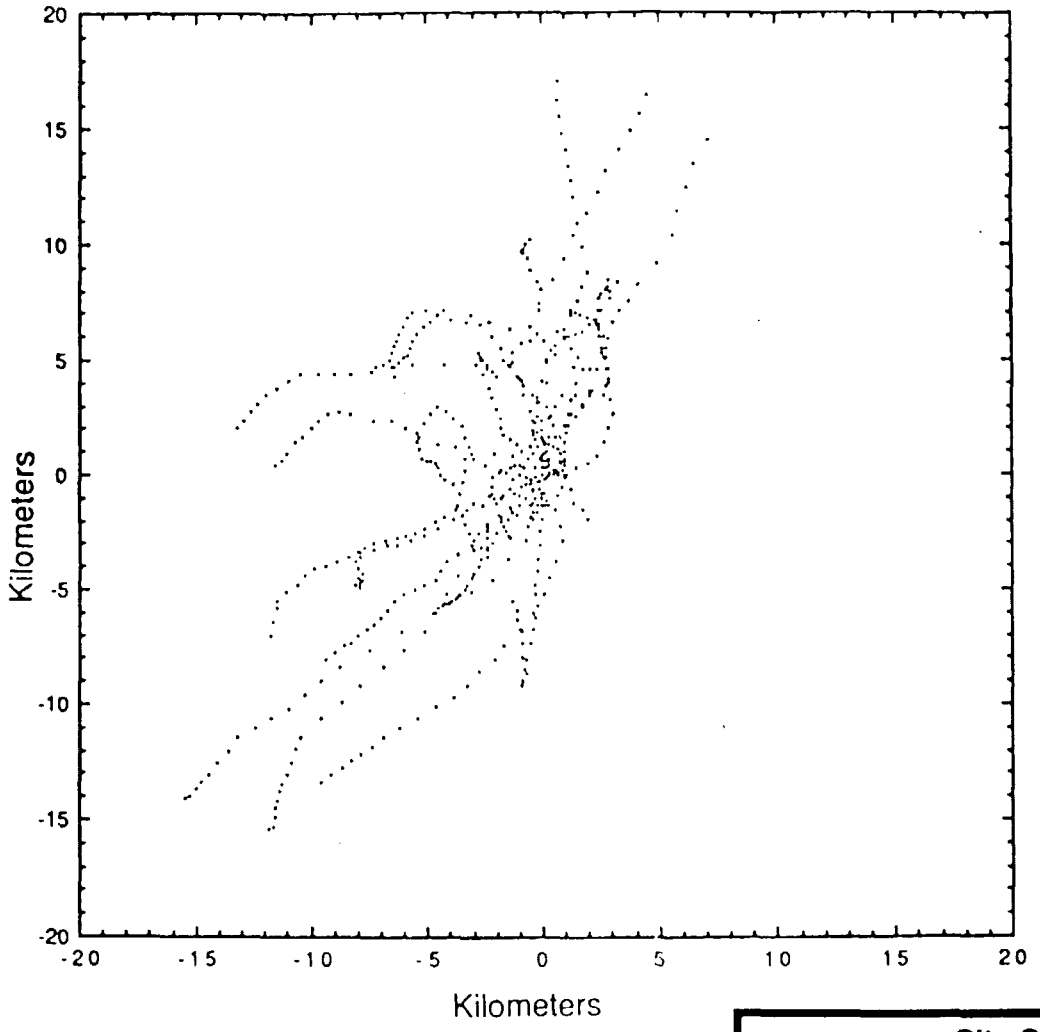
FIGURE 6-2
WINDTRACKS - 1 WEEK STORM



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FACILITIES PLAN - PHASE 3

FIGURE 6-3
WINDTRACKS - 3 MONTH STORM

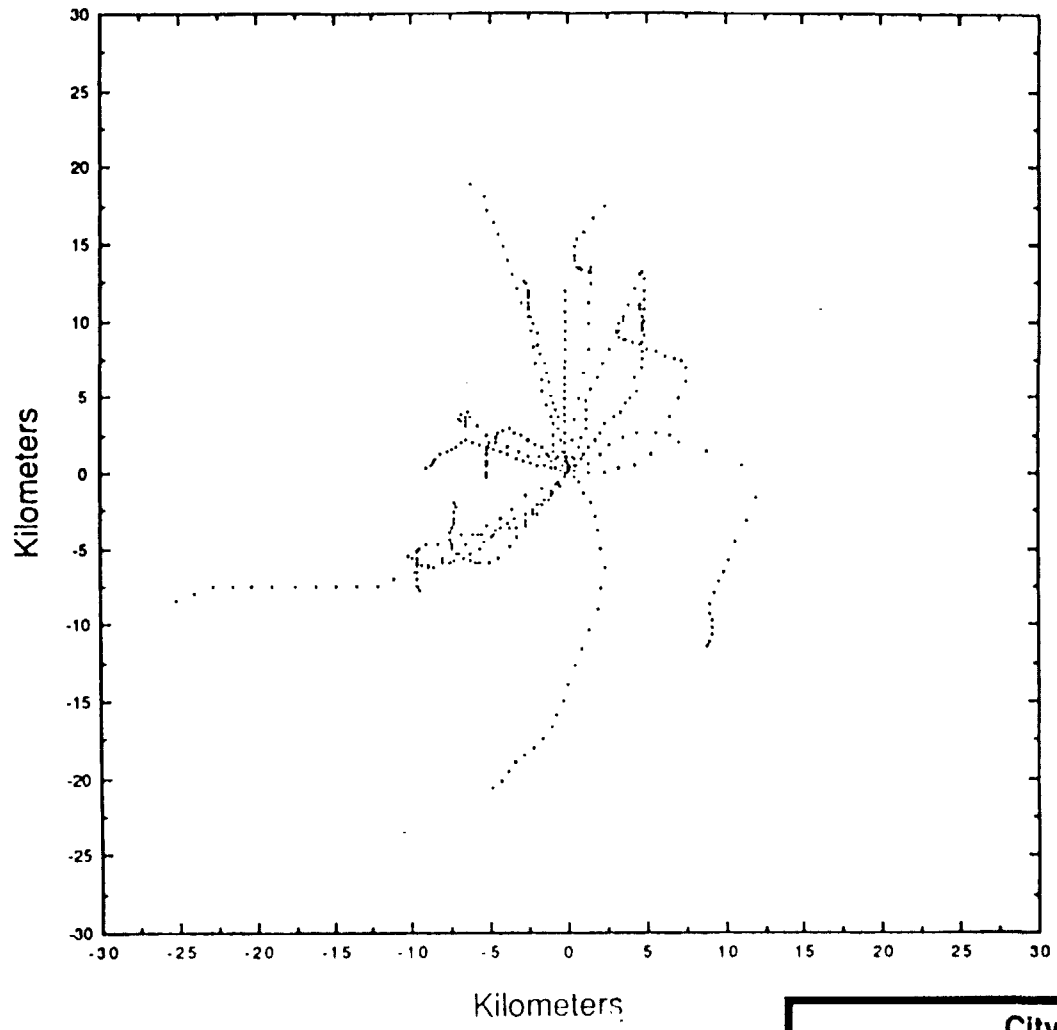

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FIGURE 6-4
WINDTRACKS - 1 YEAR STORM

Tidal Currents

Currents generated by tides are the second major factor affecting transport in coastal waters. The velocities and direction of this transport mechanism, however, are highly variable and depend on the local topography and tidal range.

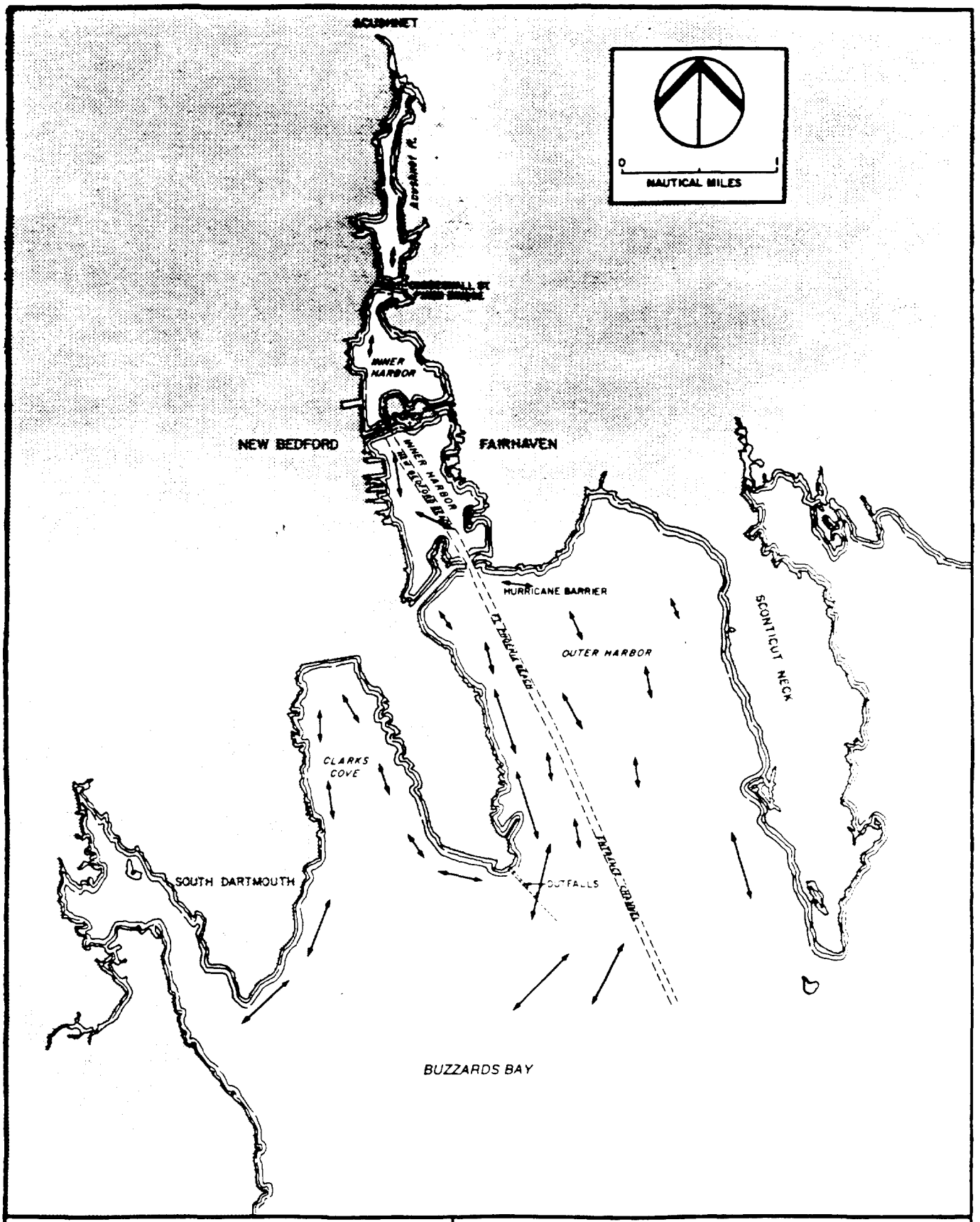
In New Bedford, the tidal transport was estimated from particles tracks generated by a three-dimensional circulation model developed by Battelle Inc. for the draft Phase 2 WWTP Facilities Plan, which was calibrated to the available current meter records. Figure 6-5 presents the tidal vectors for the three receiving waters. The analysis of the tidal transport in Clarks Cove and Outer Harbor indicates that tidal excursions range between 0.2 and 0.8 km. For the purpose of modeling tidal transport, a value of 0.5 km was used as the distance traveled in one ebb or one flood event.

In the Inner Harbor, tides were determined to be a significant transport mechanism. Geyer and Grant (1986) estimated that the tidal exchange of the Inner Harbor waters between the Inner Harbor and the Outer Harbor was determined to be 12 percent during a tidal cycle. This finding and the results from the dye studies indicate that in the Inner Harbor, tidal transport is the major north-south transport mechanism.

Initial Velocity of the Discharge

The initial momentum of the discharge will transport the plume a distance into the receiving waters. This information was used to estimate the size of the initial mixing zone by using the momentum reduction method. Figure 6-6 presents the mixing zones for each CSO. In the Inner Harbor, the CSO mixing zones cover approximately one-half of the harbor. In Clarks Cove and Outer Harbor, only CSO 004 in Clarks Cove produces a large mixing zone.

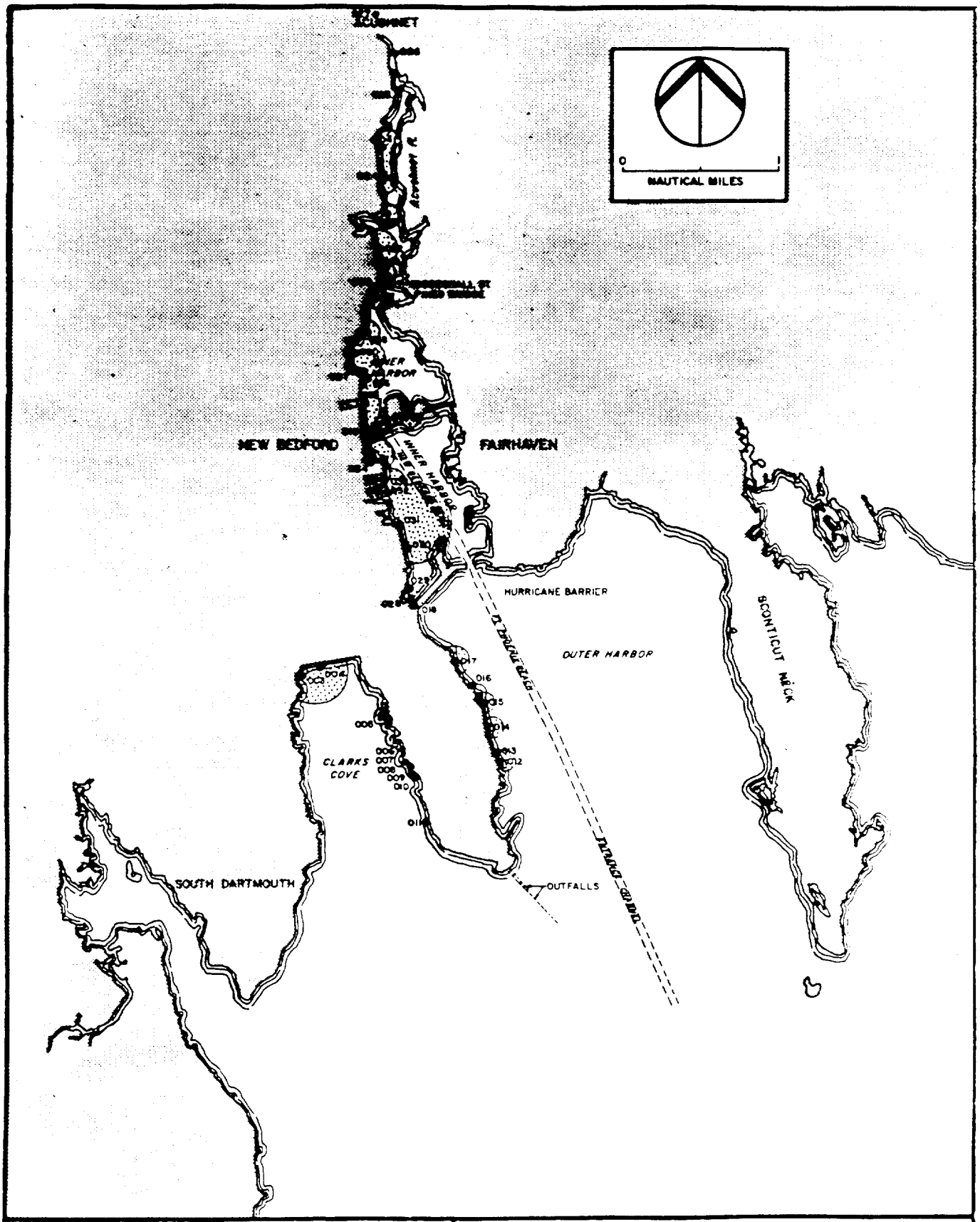
Based on this analysis, initial momentum is not a significant transport mechanism in the far-field modeling of a CSO discharge plume. The initial momentum will transport a CSO discharge plume a small distance, but will not carry it to distant beaches or shellfish beds.



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 6-5
TIDAL VECTORS



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 6-6
MIXING ZONES

Lateral Dispersion

The transport due to lateral dispersion of a CSO discharge plume was calculated using the Brook's equation (Grace, 1978) to determine the spreading of the plume. Using the dye studies, a dispersion coefficient of 12.9 ft^2/s (1.2 m^2/s) was determined for open water and 4.3 ft^2/s (0.4 m^2/s) for near shore (see Appendix B1). The lateral distance a plume would spread due to dispersion in 24 hours is 1.6 km in Clarks Cove and the Outer Harbor and 1.0 km in the Inner Harbor.

This analysis indicates that lateral dispersion is a significant transport mechanism and needs to be considered in the transport model.

Findings

Wind is the primary transport mechanism of CSO discharges in Clarks Cove and the Outer Harbor with transport from tides and lateral dispersion as secondary ones. In the Inner Harbor, transport from wind and tide are equally significant. Lateral dispersion is less significant. The initial velocity of the CSO discharge does not transport plumes very far, and is not significant for the far-field modeling. After determining the transport mechanisms, the next step was to examine how transport and dilution affects the fate of pollutants in the receiving waters.

6.5.3 FATE OF POLLUTANTS

The previous analysis describes the transport mechanisms, or how the a pollutant moves in the receiving water. This section addresses the fate of coliform bacteria and toxics in the receiving waters, which are the critical water quality concerns identified in Section 5.0.

Coliform Bacteria

Coliform bacteria are enteric bacteria that are used as an indicator of the degree of wastewater pollution in natural waters. The results of the Land Based Monitoring Program, which are presented in Appendix A, and Section

4.0 indicate that the average concentration of total coliform bacteria from a CSO discharge is 1,500,000 per 100 ml. For this analysis the fecal coliform standard will be used. Following the convention used in the Commonwealth's water quality standards, fecal coliform concentration is 0.2 (or 1/5) of the total coliform concentrations. Therefore, the average fecal coliform bacteria concentration from New Bedford CSOs is taken to be 300,000 per 100 ml. A similar calculation was performed for storm drain and for comparisons, the fecal coliform bacteria concentration from storm drains is estimated at 50,000 per 100 ml.

The water quality criteria for total coliform bacteria is 70 MPN per 100 ml for Class SA waters and 700 MPN per 100 ml for Class SB waters. For fecal coliform bacteria, the criteria are 14 MPN per 100 ml for Class SA and 140 MPN per 100 ml for Class SB. Massachusetts' beach standard is 200 MPN fecal coliform bacteria per 100 ml.

Two factors, dilution and die-off, reduce coliform bacteria concentrations in the receiving waters. Dilution is a physical process that involves the mixing of the freshwater of a CSO discharge with the saline water of the receiving water. As stated above, and described in Appendix C1, dilution is a function of time. The rate of dilution is 10 per hour for the open waters of Clarks Cove and the Outer Harbor. That is, in the first hour, 10-fold dilution would act to reduce the numbers of fecal coliform bacteria from 300,000 per 100 ml to 30,000 per 100 ml. In the second hour, a 20-fold dilution would reduce the initial fecal coliform bacteria count of 300,000 per 100 ml to 15,000 per 100 ml. A further example is, in the 24th hour, a 240 dilution would reduce the initial fecal coliform bacteria count to 300,000/240 or 1,250 per 100 ml. If the plume is pushed to shore, the dilution rate reduces to 2.5 per hour.

In addition to dilution, the coliform bacteria numbers die-off naturally. No studies were performed for this facilities plan that examined the rate of die-off of the coliform bacteria. Existing values for a die-off rate are highly variable and range from a T_{90} value (the time it takes for 90 percent of the bacteria to die) of 4 hours to 4 days. A T_{90} of 1 day was used in the modeling because it is a value commonly used in the northeast

(CDM, 1989; MWRA, 1988) and because the field data suggest a similar value. A sensitivity analysis, however, was done using a range of T_{90} values to determine how the modeling results might change.

Based on the above analysis and using an initial fecal coliform bacteria level of 300,000 per 100 ml for CSOs, the time required to achieve the fecal coliform bacteria standard after a CSO discharge was determined. A plot of time versus fecal coliform bacteria concentration which includes both a dilution (rate of 10 per hour) and die-off (T_{90} of 1 day) for Clarks Cove and the Outer Harbor is presented in Figure 6-7. The time to achieve water quality criteria for Class SA waters from a CSO discharge range from 42 to 54 hours for a fecal coliform bacteria level of less than 14 per 100 ml, and from 21 to 32 hours to achieve the beach standard of less than 200 per 100 ml. The time required to achieve water quality criteria from a storm drain discharge ranges from 23 to 34 hours for 14 per 100 ml standard and from 8 to 15 hours for the beach standard of 200 per 100 ml.

The coliform bacteria standard is violated for a longer period after a CSO discharge in the Inner Harbor than in Clarks Cove or the Outer Harbor. Though the Inner Harbor is Class SB waters, which has a higher allowable fecal coliform of 140 per 100 ml, the protected nature of the harbor results in a smaller dilution rate. Dilution rates of 1.7 per hour were calculated for the Inner Harbor compared with 2.5 - 10 per hour for the Outer Harbor and Clarks Cove. The Inner Harbor also has limited dilution ability because the volumes of freshwater discharged during the large rain events (storms equal to or greater than the 3-month storm) and within the order of magnitude of the total volume of the receiving waters. The time required to achieve water quality criteria for Class SB waters of fecal coliform bacteria standard of 140 per 100 ml is 37 hours for a 1- or 2-week storm, 44 hours for for the 3-month storm and 52 hours for a 1-year storm.

Toxics Substances

Toxic compounds include metals and priority pollutants. For the time scale of an CSO event, toxics are considered as conservative constituents. Dilution was assumed to be the only force acting to reduce the

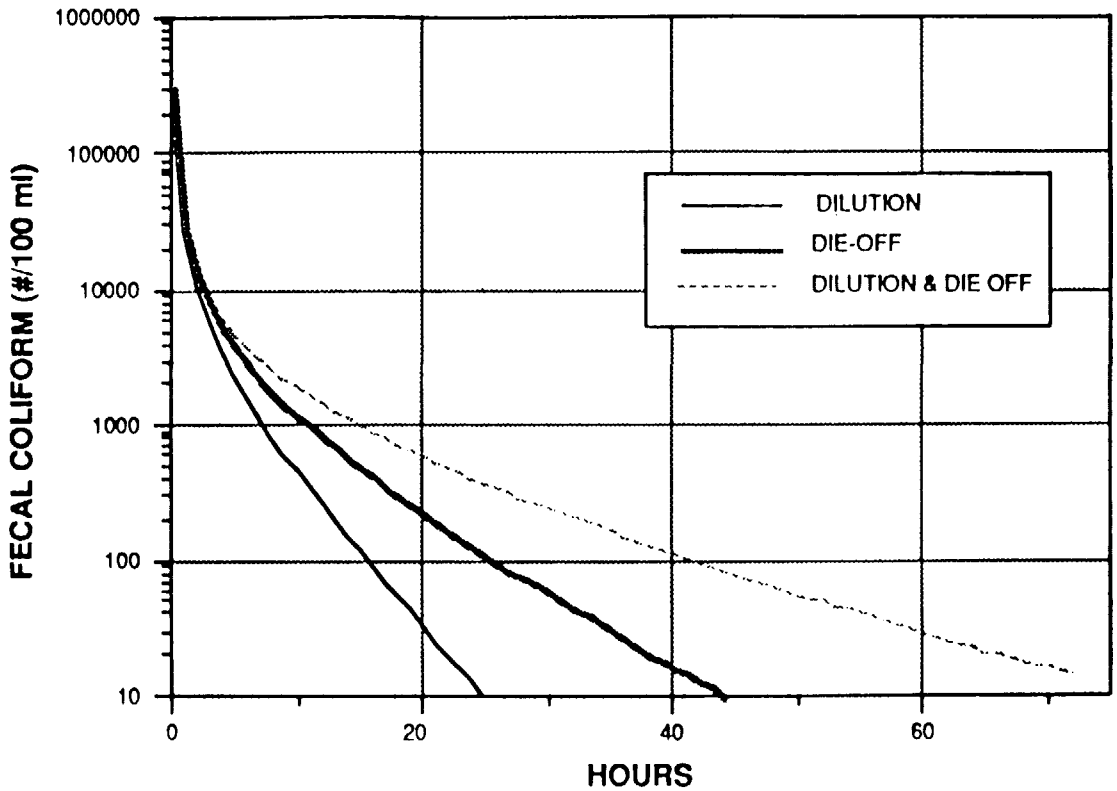


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FIGURE 6-7
DILUTION AND DIE-OFF

concentration of toxic pollutants. Using a dilution rate of 2.5 and 10 times per hour for Clarks Cove and Outer Harbor, the time necessary to achieve the required dilutions can be determined using the dilution curve on Figure 6-7.

As discussed above, the protected nature of the Inner Harbor reduces the mixing ability of the receiving water to 1.7 times per hour. The overall dilution there is also further reduced because of a background buildup of pollutants from the large volume of CSO and storm drain discharge. Once the maximum dilution (based on water volumes) is reached, further dilution is only 1.02 times per hour from tidal exchange with the Outer Harbor. The following table outline the times required for various storms:

Storm	Time Required 10 Dilutions	Time Required 30 Dilutions
1 to 3 month	6 hours	18 hours
1-year	6 hours	22 hours

6.6 MODEL PRESENTATION

The nature of wind, the major transport mechanism of the model, is stochastic not deterministic. The direction and speed of the wind changes frequently and is driven by the weather patterns. The random nature of the wind makes exact prediction of a plume track impossible. As a result, the receiving water modeling is done in terms of probability of impact. To determine the impact areas from a storm, an adaptation of the PACE model was used. A particle was released at the location of a CSO and moved in the direction of the prevailing wind with a velocity based on the wind transfer coefficient. The particle was moved in hourly increments based on the hourly wind record. The location was recorded after a specified time, depending on the time required to meet water quality standards.

The location of the particle was used as an indication of the direction and distance a discharge plume would travel during each storm on record. The width of the plume at this point was then determined by including the tidal

transport and lateral dispersion that had taken place during the specified time.

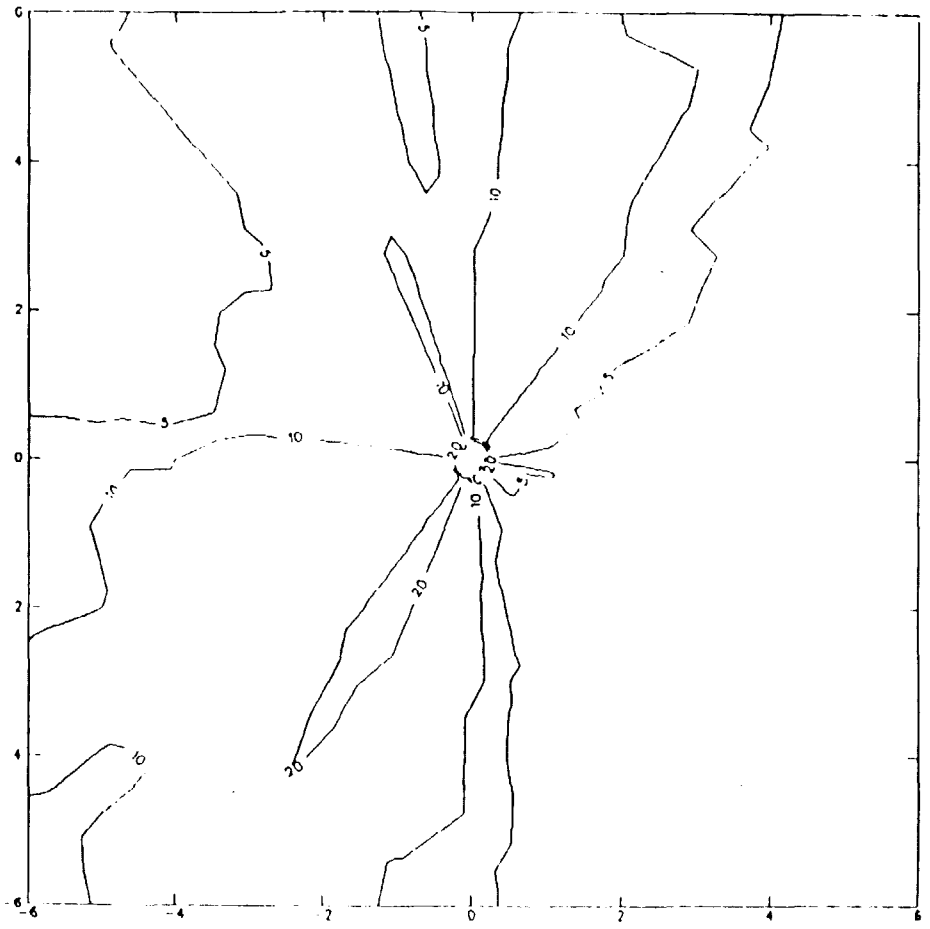
The area impacted by a discharge for each storm was taken as the "pie-shaped" wedge of water between the discharge point and the width of the plume at the last point reached by wind transport. By overlaying the wedge shape areas for all events of a design storm, the probability of an area being impacted during a given storm event was determined. For example, if a beach was "hit" by 5 plumes out of a total of 50 storms on record, it was assigned a probability of being impacted 10 percent of the time during the design storm. Figure 6-8 presents the probability of impact 21 hours following the beginning of 3-month storms. 21 hours is the duration of a violation of the beach standard for fecal coliform bacteria from a CSO discharge in Clarks Cove or the Outer Harbor. A detailed description of the stochastic modeling is given in Appendix C1.

6.7 EXISTING CONDITIONS

Using the transport and dilution model and EPA equations for dissolved oxygen dynamics, the existing conditions of the three receiving water bodies were examined for dissolved oxygen, nutrients, coliform bacteria, and toxics. To facilitate the analysis, only the significant sources of pollution are considered and described.

6.7.1 DISSOLVED OXYGEN

The impact of CSO and storm drain discharges on dissolved oxygen (DO) levels in the receiving waters was estimated by modifying the simplified far-field oxygen depletion model for coastal waters suggested by EPA (1982). A dissolved oxygen depression was calculated in the water column for the CSO discharges, which can be subtracted from a background dissolved oxygen level. A complete discussion of the model is included in Appendix C2.



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FACILITIES PLAN - PHASE 3

FIGURE 6-8
PROBABILITY OF IMPACTS
3 MONTH STORM, 21 HOURS

Clarks Cove

Under the worst case, the nadir of the dissolved oxygen sag occurs in Clarks Cove approximately 24 hours after the CSO discharges, and the maximum oxygen depression is 1.8 mg/l. A typical background dissolved oxygen concentration in Clarks Cove is 6.5 mg/l with a water temperature at 25° C. This would result in a dissolved oxygen concentration of 4.7 mg/l. This dissolved oxygen depression is similar for all storms, since the data suggest that concentrations of BOD_5 and TKN do not vary much with design storm, and the EPA equation is a function of concentration. This worst case analysis assumes the CSO discharge plume is forced against the shore, which results in reduced dilution (2.5 per hour) with the receiving water. The dissolved oxygen of the CSO discharge effluent is assumed to be 50 percent saturation or 3.3 mg/l. This assumed dissolved oxygen value is lower than typically CSO discharge values because CSO 004 has storage and pumping. The BOD would start to be exerted while the overflow is in storage and partially deplete the dissolved oxygen concentrations.

If the plume moves into the open water, dilutions with the receiving water body is greater (10 per hour) and the oxygen depression is only 0.3 mg/l or a dissolved oxygen concentration of 6.2 mg/l.

Outer Harbor

Similar calculations on dissolved oxygen levels in the Outer Harbor indicate the Outer Harbor has no significant dissolved oxygen sag after a CSO and storm drain discharge. Unlike Clarks Cove, the dissolved oxygen concentration of the CSO and storm drain discharge effluents is considered fully saturated. This results in no dissolved oxygen depression because the reaeration rate is high enough to compensate for oxygen depletion from the BOD.

Inner Harbor

The analysis of dissolved oxygen in the Inner Harbor is more complex than in the other two receiving water bodies. For the larger storms, such as

the 1-year storm, the volume discharged from CSOs and storm drains are so large that maximum dilution of the freshwater and the saline waters is achieved within nine hours. Subsequent dilutions in the Inner harbor are from tidal exchange with the Outer Harbor at a much slower rate. However, despite the limited dilution, the analysis indicates that the dissolved oxygen depression is only 0.4 mg/l for the 1-year storm at the slowest dilution rate. Storms smaller than a 1-year storm would have dissolved oxygen depressions below 0.4 mg/l. The dissolved oxygen concentration would be 6.1 mg/l.

Findings

The analyses indicates that the dissolved oxygen depression from CSO and storm drain discharges of oxygen demanding materials are small. The water quality standard of dissolved oxygen which requires DO to be greater than 50% saturation, is currently being achieved during and after a CSO overflow in all three major hydrographic regimes. No further analysis on dissolved oxygen will be performed and dissolved oxygen will not be considered as a constraint in the recommended plan.

6.6.2 NUTRIENTS

Nutrients in the CSO discharges are a concern if their loadings are sufficient to cause algal blooms which become unsightly, or cause a depression in dissolved oxygen as the phytoplankton decompose. At present there are several sources of nutrients to Clarks Cove, the Outer Harbor, and the Inner Harbor. These include the CSO discharges, storm drains, the Acushnet River, and the existing wastewater treatment plants of Fairhaven and New Bedford.

Nitrogen is the major nutrient of concern in coastal algal blooms because it is the one most commonly limiting phytoplankton growth (Nixon and Pilson, 1983). This is the case in New Bedford Outer Harbor (CDM, 1989), and probably also the case in Clarks Cove and the Inner Harbor. Although no data exist on the ratio of nitrogen to phosphorus in these latter two water bodies (the analysis used to determine which nutrient is limiting),

conditions there are similar enough to other locations in New England that nitrogen can also be assumed to be limiting. Nitrogen has been found limiting in Massachusetts Bay (MWRA, 1988), as well as in the Outer Harbor (CDM, 1989).

Under existing conditions, there does not seem to be a problem with nuisance algal blooms in any of the three hydrographic regimes in New Bedford. Nutrient balances in the Outer Harbor were discussed in the draft Phase 2 WWTP Facilities Plan (CDM, 1989). In the Outer Harbor the existing primary treated waste discharges are the major source of nitrogen, contributing approximately 1.6×10^6 lbs of nitrogen/year (CDM, 1989). The CSOs, on the other hand, contribute only 1800 lbs/year of nitrogen as TKN. Any changes in the CSO discharges will not significantly change the nutrient balance of the Outer Harbor because they are such a small contributor to the overall balance.

The amount of nitrogen discharged into Clarks Cove by New Bedford CSOs is 28,000 lbs/yr, while that coming from storm drains in Dartmouth and New Bedford is 5000 lbs/yr (see Table 4-19). These loadings are not causing any excessive stimulation of phytoplankton growth in the area. One of the stations sampled for the draft Phase 2 WWTP Facilities Plan (CDM, 1989) was located about 1 km south of the mouth of Clarks Cove. During the intensive sampling undertaken in 1987-89, however, none of the samples showed high concentrations of the phytoplankton. The biomass of phytoplankton at the mouth of the Cove and the nutrient levels in the water column were always lower than that found at the existing WWTP discharge.

Phytoplankton biomass is usually measured as a function of the amount of chlorophyll-a present in the water. Concentration above 50-80 ug/l can be considered as nuisance blooms (Smayda, 1988). Chlorophyll-a concentrations in the Outer Harbor, where nutrient levels are the highest, have never exceeded 20 ug Chl-a/l (CDM, 1989) which is well below algal bloom levels. In the Turner (1989) study of the Inner Harbor the sample on July 27, 1988 has a chlorophyll-a levels of approximately 65 ug/l which indicate a dense algal population was present. Although Turner (1989) does not report the presence of a nuisance bloom on that day, the chlorophyll-a value suggests

that the algal population was close to nuisance levels. The algal populations, however, were not consistently high. At other times Turner (1989) found chlorophyll-a values in the Inner Harbor that were similar to those in the Outer Harbor.

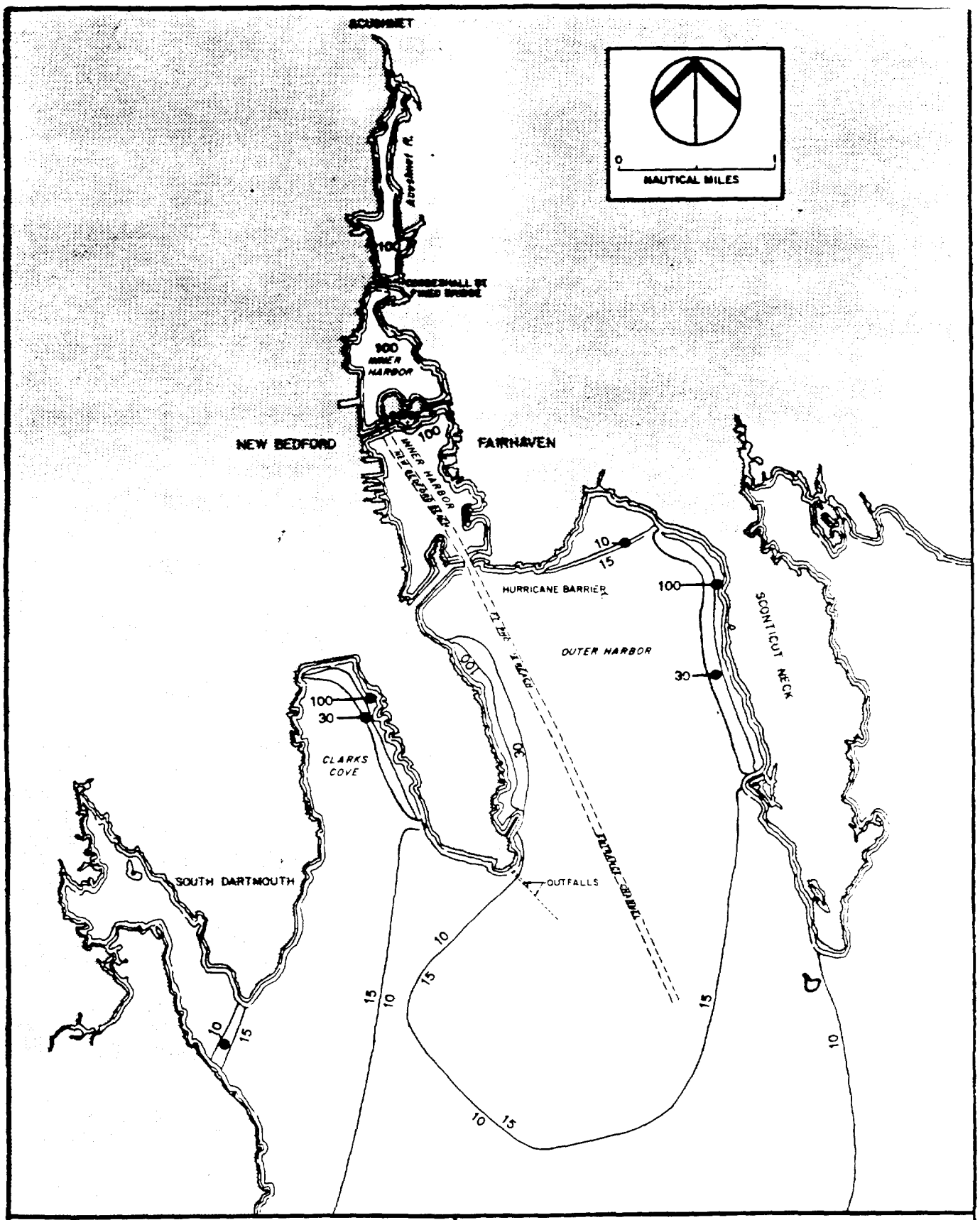
There is a fairly large annual input of nitrogen into the Inner Harbor from several sources, including 33,000 lbs of nitrogen (as TKN) from CSOs, 23,000 lbs from storm drains, and 84,000 lbs from the Fairhaven Treatment Plant that would account for the high algal biomass.

The poor flushing of the Inner Harbor will tend to keep the nutrients and phytoplankton within the harbor. All of the CSO abatement alternatives involve some reductions in nutrient loadings to the Inner Harbor. However, even if all nutrients discharged by CSOs are removed, no major changes in algal productivity or biomass are expected. Of the quantified nitrogen sources the CSOs contribute at most 24 percent (33,000 lbs) of the total annual input. The Fairhaven treatment plant on the other hand, contributes 60 percent and storm drains contribute 16 percent.

In summary, based on the review of data, nutrients levels under existing conditions are not high enough to result in algal blooms in any of the three receiving waters. This finding indicates that CSO abatement does not need to include control of nutrients.

6.7.3 FECAL COLIFORM BACTERIA

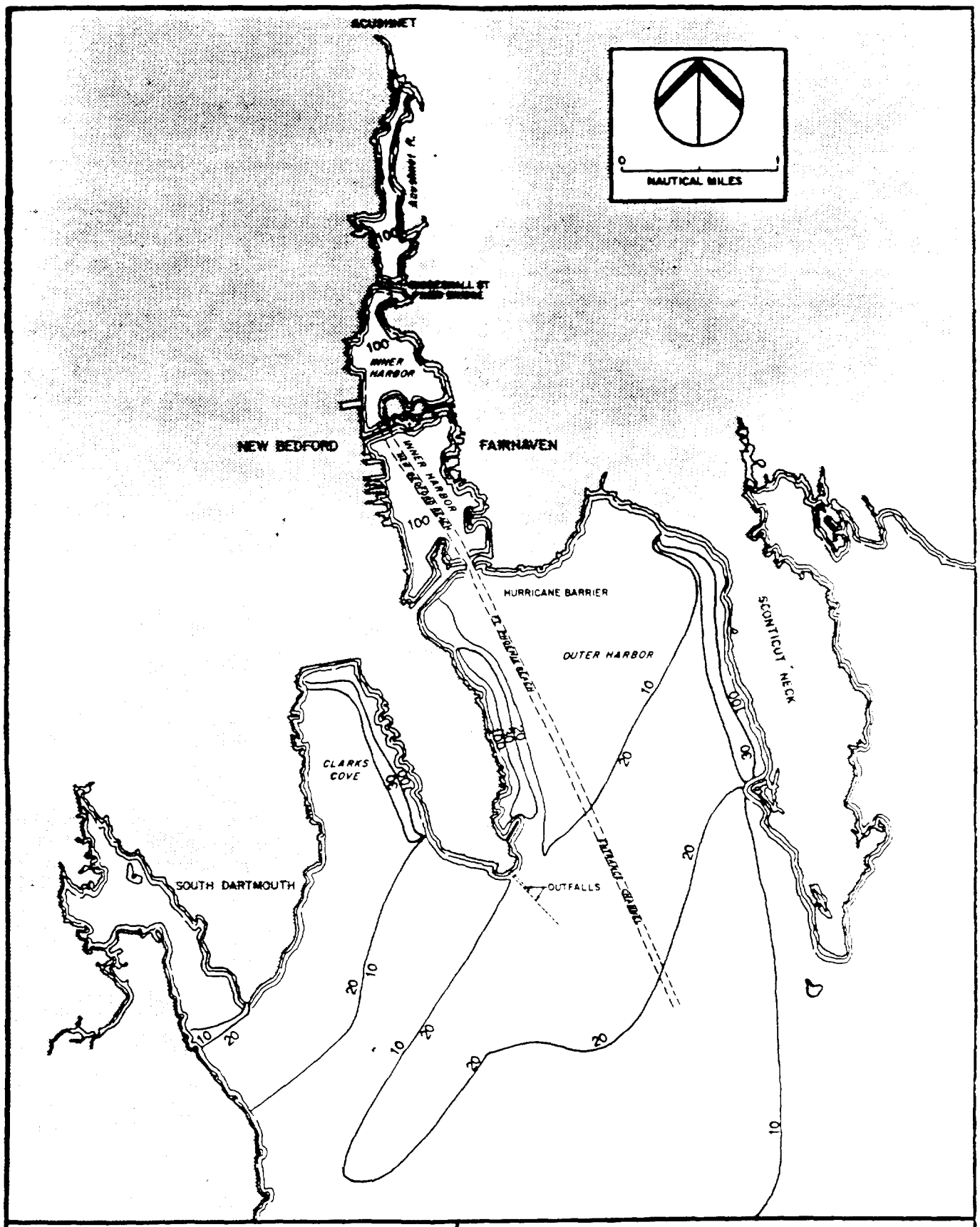
The stochastic receiving water model was used to examine the existing conditions of the fecal coliform bacteria in the three receiving water bodies. The analysis considers CSO and storm drain discharges. Figures 6-9, 6-10 and 6-11 present the probabilities of impacts from a discharge plume for the 2-week, 3-month and 1-year storms.



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 6-9
PROBABLITY OF IMPACTS
2 WEEK STORM



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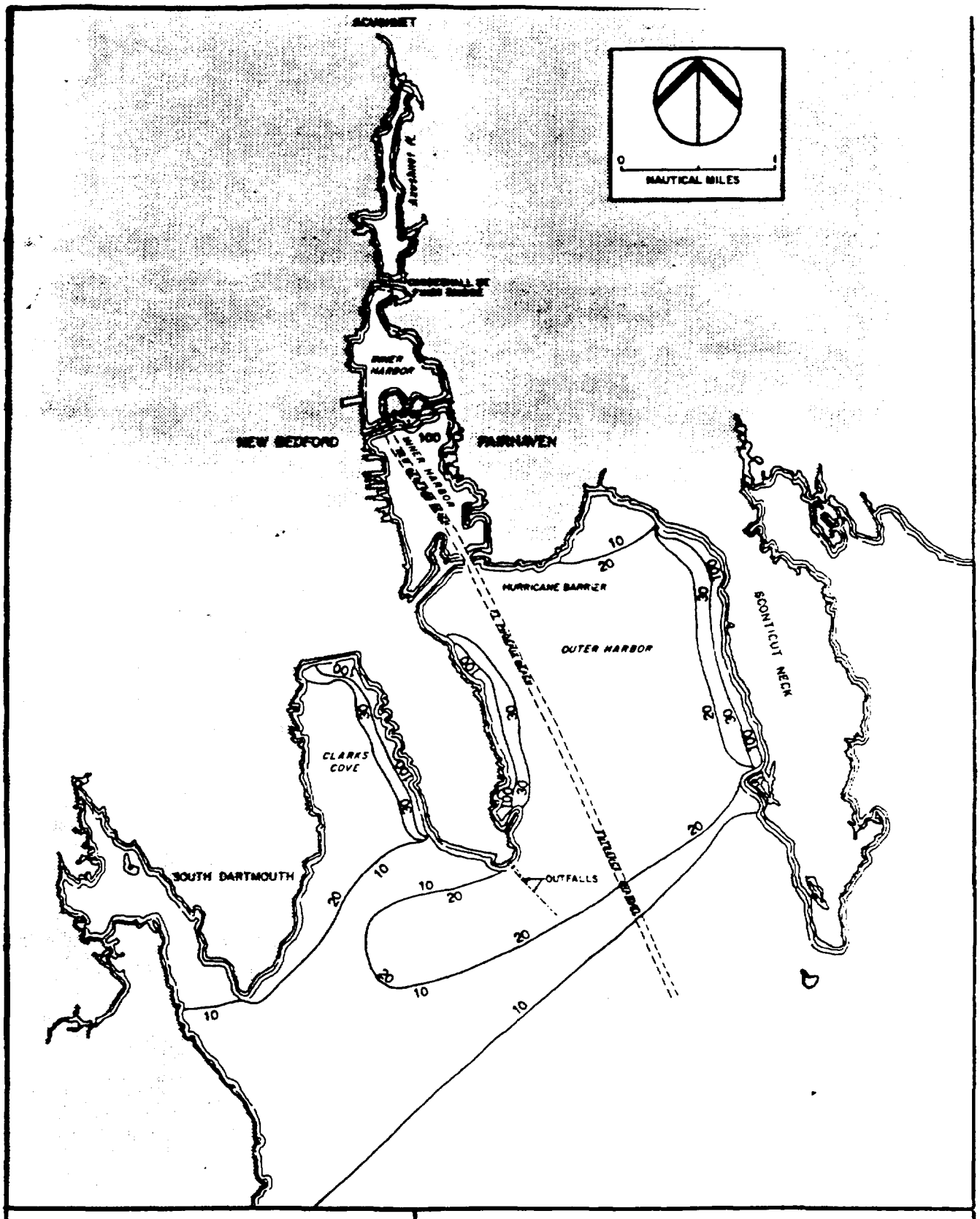
COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 6-10
PROBABLITY OF IMPACTS
3 MONTH STORM



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COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 6-11
PROBABILITY OF IMPACTS
1 YEAR STYORM

For this analysis the CSOs are grouped by geographic region as described in Section 4.0. The groupings are:

Grouping	CSO	Receiving Water	Geographic Location
1	003-004	Clarks Cove	North shore of Clarks Cove
2	005-011	Clarks Cove	West shore of Clarks Point
3	012-018	Outer Harbor	East shore of Clarks Point
4	028 030-034	Inner Harbor	Between the hurricane barrier and Popes Island
5	020-021 035-037	Inner Harbor	Between Popes Island and the Coggeshall Street bridge
6	022-027 040-041	Inner Harbor	Between the Coggeshall Street bridge and Tarklin Hill Road

As can be seen from the figures, the 1-year storm does not impact a greater area then the 3-month or the 2-week storm. This follows from earlier findings on wind driven transport mechanism. The findings there indicates that the larger storms (6-month through 5-year) do not travel greater distances than the smaller storms (3-month or less).

To facilitate assessment of CSO abatement alternatives, the probabilities presented in the above figures were converted to an estimate of annual water quality violations. Dry weather overflows were not considered part of this analysis, but rather will be addressed separately. The analysis examined only storm (rain) driven overflow events. Table 6-1 presents the annual expected fecal coliform bacteria violations of water quality standards and beach standards from CSO and storm drain discharges. In addition, the annual duration of violation in hours was determined for each water body from each CSO and storm drain source. Table 6-2 presents the expected annual duration of violation from fecal coliform bacteria for the receiving waters.

Under existing conditions, the fecal coliform standard for Class SA waters will be exceeded in Clarks Cove from the discharges of CSOs in Groups 1 and 2 for an estimated 2350 hours of the year. This is approximately 25

TABLE 6-1

EXISTING CONDITIONS
ANNUAL EXPECTED FECAL COLIFORM VIOLATION (OCCURRENCES)

Source/Resource	CSO Groups #1 & #2	CSO Group # 3	CSO Groups #4, #5, & #6	Storm Fairhaven	Storm Inner Harbor	Total CSO	Total Storm	Total All
Overflows	50	49	50	100	100			
Shellfish								
1. Clarks Cove	50	0	0	0	0	50	0	50
2. Outer Harbor (N.B.)	0	49	8	17	16	49	17	49
3. Outer Harbor (F.H.)	0	5	3	100	5	5	100	100
4. Inner Harbor	0	8	50	0	100	50	100	100
5. Buzzards Bay	8	8	6	16	11	8	16	16
Beaches (2)								
1. Smith Neck	6	5	0	0	0	6	0	6
2. S. Dartmouth	50	0	0	0	0	50	0	50
3. Clarks Point (West)	50	0	0	0	0	50	0	50
4. Clarks Point (East)	0	49	8	16	15	49	16	49
5. Sconticut Neck	0	38	1	100	0	3	100	100
6. Fort Phoenix	0	8	3	15	11	8	15	15
Volume Discharged (MG)	496	33	595	157	1340	1125	1497	2619

TABLE 6-2

EXISTING CONDITIONS
ANNUAL EXPECTED FECAL COLIFORM VIOLATION (HOURS)

Source/Resource	CSO Groups #1 & #2	CSO Group # 3	CSO Groups #4, #5, & #6	Storm Fairhaven	Storm Inner Harbor	Total CSO	Total Storm	Total All
Shellfish								
1. Clarks Cove	2350	0	0	0	0	2350	0	2350
2. Outer Harbor (N.B.)	0	2303	336	391	368	2303	391	2303
3. Outer Harbor (F.H.)	0	210	126	2900	115	210	2900	2900
4. Inner Harbor	0	336	2700	0	3400	2700	3400	3400
5. Buzzards Bay	336	336	252	368	0	336	368	368
Beaches (2)								
1. Smith Neck	126	105	0	0	0	126	0	126
2. S. Dartmouth	1350	0	0	0	0	1350	0	1350
3. Clarks Point (West)	1350	0	0	0	0	1350	0	1350
4. Clarks Point (East)	0	1323	168	128	120	1323	128	1323
5. Sconticut Neck	0	63	21	1200	0	63	1200	1200
6. Fort Phoenix	0	168	63	176	88	168	176	176
Volume Discharged (MG)	496	33	595	157	1340	1125	1497	2619

percent of the total time in a year. This implies that shellfish beds could be opened 75 percent of time, the fact that closure typical extends five days, means that if there are 50 closure events per year (Table 6-1), the beds will be closed for 250 days a year. CSO Groups 1 and 2 also cause approximately 336 hours of violations in Buzzards Bay. Though storm drains discharge to Clarks Cove, most stormwater enters the Cove by overland flow, not through a pipe. Stormwater entering by overland flow has significantly better water quality, then that entering from a pipe. The number of storm drains in Clarks Cove were considered small and not included in the analysis.

The Outer Harbor was divided into two parts for the analysis: the New Bedford side and the Fairhaven side. This was done to reflect the corporate jurisdictions in opening or closing shellfish beds and beaches. On the New Bedford side of the Outer Harbor, CSO Group 3 is estimated to cause 2303 hours of violations per year. The CSOs (CSO Groups 4,5 & 6) and storm drains in the Inner Harbor and storm drains in Fairhaven are estimated to cause from 210 to 336 hours of violations in the New Bedford side of the Outer Harbor.

Violation in the fecal coliform bacteria standard on the Fairhaven side of the Outer Harbor is estimated to be 2900 hours a year as a result of the the storm drains in Fairhaven, with lesser time periods of violations from CSOs on Clarks Point (CSO Group 3, 210 hours) and CSOs (CSO Groups 4, 5 & 6, 126 hours) and storm drains (115 hours) in the Inner Harbor.

The bacterial water quality standards in the Inner Harbor are violated from CSOs (CSO Groups 4, 5 & 6) and storm drains in the Inner Harbor. The storm drains are estimated to result in a slightly greater time of violation (3400 hours) compared to the CSOs (2700 hours). CSO Group 3 on Clarks Point contributes an estimated 336 hours of violation to the Inner Harbor.

In two of the receiving water bodies, the Fairhaven side of the Outer Harbor and the Inner Harbor, storm drains are a major source of fecal coliform bacteria and contribute a significant time period of violation.

The beach standard is also often violated throughout the Harbor. Beaches from Smith Neck (in Dartmouth) to Sconticut Neck (in Fairhaven) are estimated to exceed the beach standard of 200 fecal coliform bacteria per 100 ml for up to 1300 hours of the year.

Several conclusions can be reached from this examination of the existing conditions.

- Fecal coliform bacteria criteria are exceeded up to 40 percent of the time in New Bedford receiving waters from CSO and storm drain discharges.
- The largest impact on a resource area (shellfish or beach) is from whichever of the two pollution sources (CSO or storm drain) is nearest.
- Storm drains cause the longest time violations on the Fairhaven side of the Outer Harbor and in the Inner Harbor.
- CSO discharges at Groups 1, 2 and 3 and storm drains in Fairhaven can cause coliform violations in Buzzards Bay.

6.7.4 TOXIC SUBSTANCES

Using the receiving water model, the probability of impacting receiving waters was examined for copper (the toxic substance which requires the highest dilution) and the toxicity of the whole effluent.

Copper

To meet EPA's Gold Book copper standard for acute or chronic toxicity, a CSO discharge requires 40 dilutions (see Appendix A). The findings of the field program indicates that 4 to 16 hours will be required in Clarks Cove and the Outer Harbor, and 23 hours or more in the Inner Harbor to achieve 40 dilutions. Figure 6-12 presents the area a CSO discharge may impact during a 3-month storm prior to achieving the required 40 dilutions. In Clarks Cove and the Outer Harbor, the potential impacted area is sizable, and for the Inner Harbor all the harbor could be impacted prior to achieving the required dilutions.

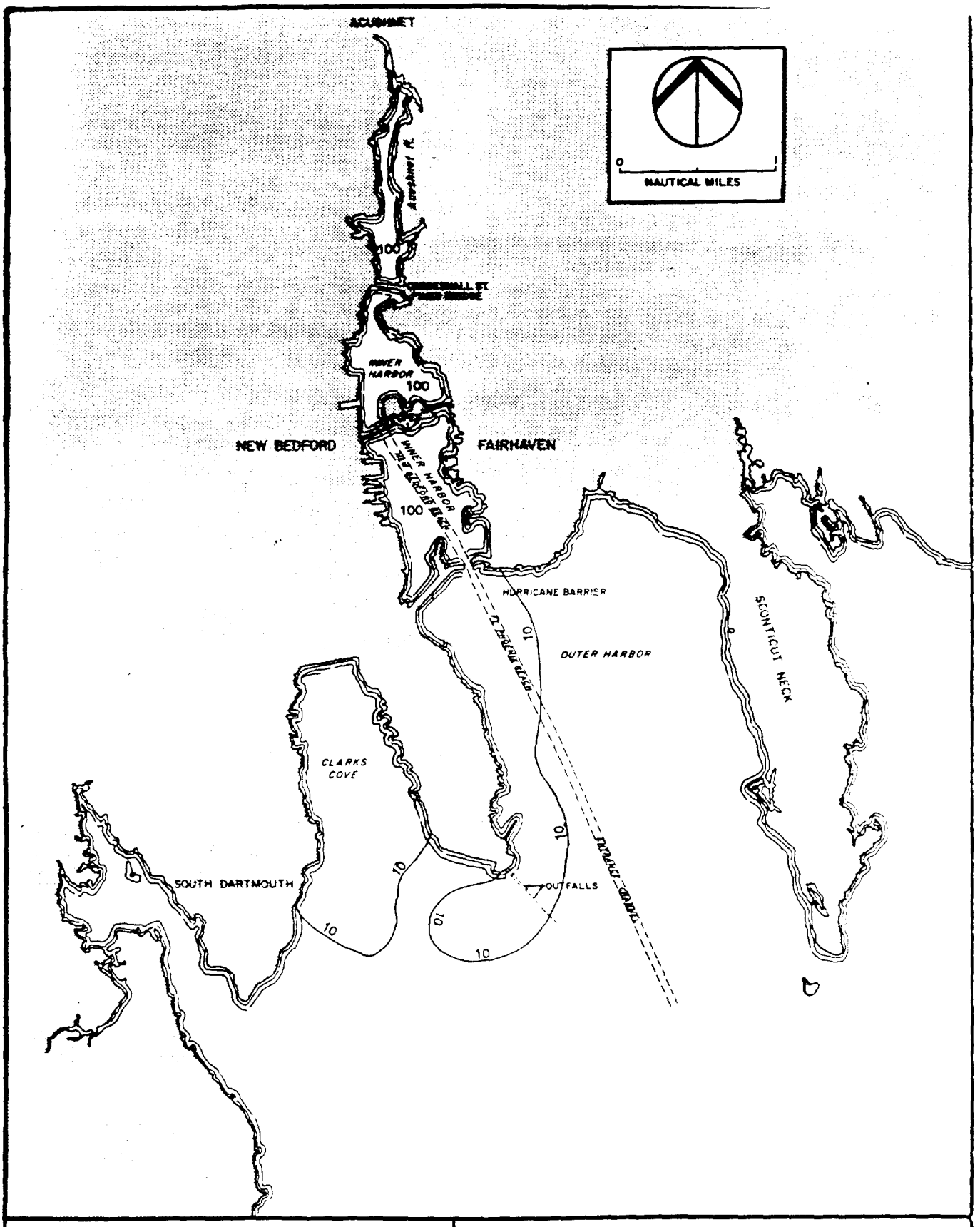

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FIGURE 6-12
PROBABLITY OF COPPER TOXICITY 4 HOURS AND 40 DILUTIONS

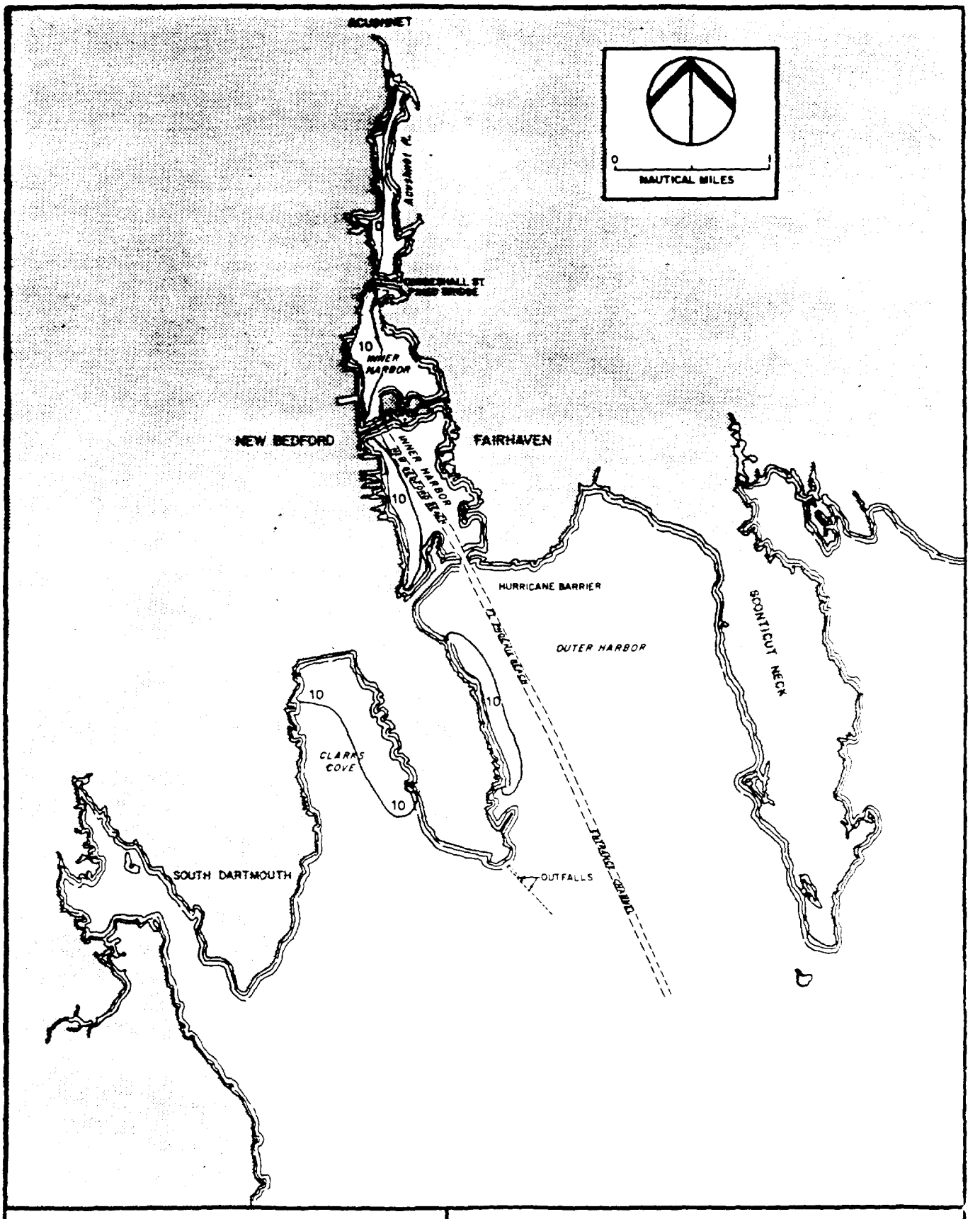
Whole Effluent Toxicity

A more appropriate test is the whole effluent toxicity test. Appendix A contains the findings of the testing performed for this study. The testing found the largest required dilution to be 8:1 for sheephead minnow (*Cyprinodon variegatus*). For Clarks Cove and the Outer Harbor 8 dilutions can be achieved in 1 to 3 hours and for the Inner Harbor 8 dilutions require 5 hours. Figure 6-13 presents the areas that may be impacted. The potential impacted area is small in Clarks Cove and the Outer Harbor, and limited to near shore in the Inner Harbor.

6.8 BASE CONDITIONS

A 75-mgd secondary treatment plant and system upgrades are proposed for the City of New Bedford, as described in the draft Phase 2 WWTP Facilities Plan (CDM 1989). The net effect will be to raise the conveyance and plant capacity from the present 60 mgd to a 75 mgd. This is improvement was modeled using the land-based models SWMM and STORM to predict what reduction in CSO discharge could be expected from the improvements. The results were then used as input into the receiving water model to predict the improvements to be expected in the water quality.

Improvements in water quality from the planned system upgrade are presented in Table 6-3. CSO Group 1 shows a small decrease in the amount of time water quality standards are violated while the remain CSO groups do not improve, even though the volume discharged decreases by 35 percent (from 1120 MG to 730 MG). The reason for this apparently insignificant improvement is the that frequency of discharge of some CSOs did decrease, but not all decreased within a CSO group. For example in CSO Group 3 on the east side of Clarks Point, CSOs 012 through 016 decreased their frequency of discharge from a range of 45 to 47 times a year to a range of 2 to 15 times a year. However, the frequency of discharge for CSO 017 remained the same at 49 times per year. The system-wide improvements do not address every CSO in the system.



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 6-13
PROBABILITY OF WHOLE EFFLUENT TOXICITY 1 HOUR & 10 DILUTIONS

TABLE 6-3

COMPARISON OF EXISTING WITH BASE CONDITIONS
ANNUAL EXPECTED FECAL COLIFORM VIOLATION BASE CONDITION

Source/Resource	EXISTING CONDITION			75 MGD 2º & SYSTEM UPGRADES		
	CSO Groups #1 & #2	CSO Group # 3	CSO Groups #4, #5, & #6	CSO Group #1 & #2	CSO Group # 3	CSO Groups #4, #5 & #6
Shellfish						
1. Clarks Cove	2350	0	0	2115	0	0
2. Outer Harbor (N.B.)	0	2303	336	0	2303	336
3. Outer Harbor (F.H.)	0	210	126	0	210	126
4. Inner Harbor	0	336	2700	0	336	2700
5. Buzzards Bay	336	336	252	294	336	252
Beaches (2)						
1. Smith Neck	126	105	0	105	105	0
2. S. Dartmouth	1350	0	0	1215	0	0
3. Clarks Point (West)	1350	0	0	1215	0	0
4. Clarks Point (East)	0	1323	168	0	1323	168
5. Sconticut Neck	0	63	21	0	63	21
6. Fort Phoenix	0	168	63	0	168	63
Volume Discharged (MG)	496	33	595	360	22	352

The improved conditions will be used as the "Base Condition" against which CSO abatement alternatives will be evaluated (see Section 9.0).

6.9 SUMMARY

The field program and analysis of the fundamental parameters indicate:

- Wind is the dominant transport mechanism in Clarks Cove and the Outer Harbor.
- Tidal transport and lateral dispersion are a secondary transport mechanism in Clarks Cove and the Outer Harbor, tidal transport and winds are major factors in the Inner Harbor.
- Dilution rates of CSO and storm drain discharges are linear with time.
- Winds associated with storms that are equal to or greater than the 3-month storm track in the northeast or southwest direction.

Application of the receiving water model to CSO and storm drain discharges indicates:

- CSO discharge plumes can travel across the Outer Harbor or Clarks Cove in 6 hours or less.
- Plumes can travel as far for the 1-week storm as for the 1-year storm.
- The small size of the Inner Harbor and the large discharge volume from CSOs and storm drains result in the entire Inner Harbor being impacted within hours of the start of a storm.
- Fecal coliform bacteria from CSO discharges can cause water quality violations for up to 54 hours in Clarks Cove, the Outer Harbor and Buzzards Bay; and up to 52 hours in the Inner Harbor.
- Fecal coliform bacteria from storm drain discharges can cause water quality violations for up to 34 hours in Clarks Cove and the Outer Harbor, and up to 29 hours in the Inner Harbor.

Examination of the results of the receiving water model indicates that under existing conditions:

- CSOs do not contribute loadings that cause an a violation of the allowable dissolved oxygen levels;

- All sources of pollution (WWTPs, industrial discharges, CSOs and storm drains) do not contribute sufficient nutrients to result in nuisance algal blooms in any of the three receiving waters;

- Whole effluent from CSOs is marginally toxic and sufficient dilution occurs in the first few hours in Clarks Cove and the Outer Harbor, and within 5 hours in the Inner Harbor;

- Copper will require 4 to 16 hours in Clarks Cove and the Outer Harbor and up to 23 hours in the Inner Harbor to achieve the dilution needed to meet its water quality standard;

- The fecal coliform bacteria standard is violated from CSOs and storm drain discharges up to 40 percent of the time in the three receiving water bodies.

- Fecal coliform bacteria from Fairhaven storm drains cause violations in the Fairhaven side of the Outer Harbor and the Inner Harbor.

Findings from examination of the Base Conditions are as follows:

- Constructing a 75-mgd secondary treatment plant and implementing the planned system upgrades will not reduce the closure time for shellfish beds and beaches.

- The planned upgrades will reduce the annual volume discharged from CSOs by 35 percent.

The development of the understanding of the fate of CSO discharges on receiving water bodies will allow an assessment in the next sections of appropriate CSO abatement technologies and the evaluation of control alternatives.

7.0 ASSESSMENT OF CSO ABATEMENT TECHNOLOGIES

7.1 GENERAL

This section describes a wide range of CSO abatement technologies and assesses their applicability for meeting the CSO control objectives for New Bedford. The purpose of this assessment is to select appropriate technologies for evaluation and comparison in Section 9.0.

A broad range of alternative strategies is available for the control of pollutants discharged from CSOs ranging from no action to total capture and treatment. The most cost effective plan will fall between these two extremes and will generally employ a combination of several CSO abatement technologies. The effectiveness of a particular CSO abatement technology depends upon, among other things, the specific water quality objectives to be met. Section 5.0 presented the water quality issues and objectives for New Bedford. In this section CSO abatement technologies will be assessed to determine their applicability in meeting these objectives.

7.2 CSO ABATEMENT TECHNOLOGY DESCRIPTIONS

Table 7-1 lists the alternative CSO abatement technologies to be analyzed in this report. The alternatives have been divided into four general categories: system management, source control, collection system control modifications, and storage and treatment. Each technology is described in general terms below. In these descriptions, technologies with limited feasibility in New Bedford are identified and eliminated from further consideration. Following these descriptions, a more in-depth analysis is presented for the most feasible technologies.

TABLE 7-1

CSO ABATEMENT TECHNOLOGIES

SYSTEM MANAGEMENT

- Catch Basin Cleaning
- Regulator Inspection and Maintenance
- Sewer Cleaning/Flushing
- Infiltration/Inflow Control
- Improvement/Replacement of Conveyance System

SOURCE CONTROL

- Commercial/Industrial Runoff Control
- Street Sweeping and Solid Waste Management
- Fertilizer/Pesticide Control
- Soil Erosion Control
- De-icing Practices
- Air Pollution Reduction
- Porous Pavement

COLLECTION SYSTEM MODIFICATIONS

- Sewer Separation
- Construction of Parallel Conveyance System
- Regulating Devices and Tide Gates
- Remote Monitoring and Control/Flow Diversion
- Polymer Injection

STORAGE AND TREATMENT

Storage Technologies

- In-Line
- Off-Line
- Surface Storage

Treatment Technologies

- Screening
- Sedimentation
- Dissolved Air Floatation
- Swirl and Helical Concentrators
- Disinfection

7.2.1 SYSTEM MANAGEMENT

Improved system management techniques can benefit receiving water quality by reducing CSO discharge volumes and first flush pollutant loads. Proper maintenance can significantly improve the performance of the existing collection system by correcting malfunctions, unblocking clogged sewers, optimizing regulator function, locating unused in-line storage capacity, sewer, catch basin and regulator cleaning, sewer replacement where necessary, and disconnection of unauthorized connections.

Catch Basin Cleaning

Catch basins are installed in collection systems to collect and convey surface runoff to the sewer system, and to capture sand, grit, and solids prior to entering the system. The sump below the outlet pipe in a catch basin captures solids and must be cleaned regularly. Catch basins can be cleaned manually or by a bucket, eductor, or vacuum. Properly maintained catch basins can reduce the quantity of solids entering the combined sewer system. New Bedford has an extensive catch basin cleaning program. The need for expansion of this program is evaluated later in this section.

Regulator Inspection and Maintenance

Optimizing the existing regulators through proper cleaning, maintenance, and adjustments is essential in maintaining proper hydraulic conditions in the system and minimizing overflows. Clogging of regulators can reduce the hydraulic capacity of the interceptor connections increasing the frequency of CSO discharge, and in severe cases cause dry weather overflows. The City has adopted a regulator inspection program in conjunction with its sewer cleaning and maintenance program. The need for expansion of this program will be evaluated later in this section.

Sewer Cleaning/Flushing

Deposition of solids is a common problem in combined sewer systems. These systems are designed to handle peak wet weather flow, therefore their

hydraulic capacity greatly exceeds flowrates that normally occur during periods of dry weather. Consequently, dry weather flow velocities are usually much lower than the design (full pipe) velocity which may cause solids to settle in the pipelines. During large storms, these solids are resuspended resulting in high pollutant concentrations during the initial period of a storm.

To avoid this "first flush" phenomenon (the resuspension of the settled solids due to a storm flow) sewers may be cleaned by mechanical means (rodding or draglines) or by flushing with water. Either technique will flush the solids through the system during dry weather when system capacity is available to convey flow to the wastewater treatment plant before a storm event flushes the solids to the receiving waters through CSOs. In severe cases of solids deposition, storm flows will not resuspend the settled material which will decrease the pipe's hydraulic capacity.

The City maintains a sewer cleaning program to minimize the effects of deposition in problem sewers. Implementation of a more extensive program is evaluated as part of a CSO abatement program later in this section.

Infiltration/Inflow Control

To maximize the collection system's capacity, it is necessary to remove the extraneous flows caused by infiltration and inflow. Infiltration is groundwater that enters the system through broken or cracked pipes, defective joints, depressed manholes, and manhole walls. Inflow results from direct connections to the system from roof leaders, cellar and yard drains, commercial and industrial drains, and malfunctioning tide gates. Since combined sewer systems are intended to carry both wastewater and stormwater, inflow cannot be entirely eliminated, but can be reduced or detained to attenuate peak flows. Infiltration can be reduced by replacing or lining defective pipes. Infiltration and inflow comprise nearly half of the average day treatment plant influent. Reductions in infiltration/-inflow are evaluated further later in this section.

Improvement/Replacement of Conveyance System

As discussed above, the replacement of old pipes can reduce the quantity of infiltration, thus increasing the conveyance capacity of the system. This technology will be assessed further in this section.

7.2.2 SOURCE CONTROL

Source control techniques can be employed either to decrease the quantity of water entering the system or minimize certain pollutants from the waste stream at their source. Generally, source control techniques have minimal capital costs but are labor intensive having high operation and maintenance costs.

Commercial/Industrial Runoff Control

The quality of CSOs can be improved through the control of runoff from commercial and industrial establishments in the drainage area. Of particular concern are gas stations and other petrochemical establishments. Oil traps or permanent oil collection systems can be employed to reduce the quantity of pollutants entering the system.

New Bedford's sewer use ordinance prohibits the discharge of gasoline and other flammable materials, and wastes that constitute a hazard to humans or animals or create a public nuisance. It places limits on specified pollutants such as metals, greases, and oils and industrial wastes. The City has an industrial waste reporting system and sampling program in place by which discharge from industrial users are monitored. Expansion of the program to include monitoring of runoff from potential sources of oils and grease or specific pollutants identified in the sampling program will be considered further. Industrial pretreatment and spill control are discussed in more detail later in this section.

Street Sweeping and Solid Waste Management

Street sweeping has become a common practice in urban areas to improve the aesthetic environment by removing litter and debris from gutters. Sweeping has an additional benefit of improving the water quality of surface runoff. The removal of dust, dirt, and debris will decrease the quantity of solids entering the system (however will not reduce BOD substantially in a combined system). Methods of street cleaning include manual, mechanical broom sweepers, vacuum sweepers, and street flushing. The objective of street flushing is to transport pollutants to interceptors during dry weather when interceptor capacity is available to convey flow to the wastewater treatment plant.

The effectiveness of street sweeping is a function of the following: sweeper efficiency, number of passes, equipment speed, pavement conditions, equipment type, fraction of street swept, litter control programs, and street parking restrictions. In practice, street parking is the greatest obstacle to an effective program.

Solid waste management consists of animal waste control and litter control. Animal excrement is a source of stormwater pollution, especially nitrogen and pathogenic organisms. Littering contributes to floatables that are discharged through CSO outlets.

Street sweeping and solid waste management alone are not sufficient to provide effective CSO abatement. However, since both serve to mitigate sources of pollution identified as issues for this study (coliform and floatables), the role of these measures in a system-wide CSO abatement program is examined in further detail later in this section.

Fertilizer/Pesticide Control

The use of fertilizers and pesticides can increase the pollutant levels primarily nutrients in stormwater runoff. Control of the use and storage of these chemicals can help to reduce this pollutant loading. The most

effective means of control is through the municipal parks department who often use and store large quantities of fertilizers and pesticides.

Results of the sampling program indicate that fertilizers and pesticides do not contribute to the wastestream of pollutants identified in CSO discharges in New Bedford. Careful use and storage of fertilizers and pesticides should be maintained, however, since they are not an apparent source of CSO pollution, additional control of fertilizers/pesticides will not be considered.

Soil Erosion Control

Construction sites are one of the primary contributors of sediment in surface runoff. Enforcement of guidelines established for erosion and sediment control at construction sites can reduce the suspended solids loading of CSO discharge. These guidelines include: maintenance of native vegetation, sediment retention basins, installation of hay bales, and protection of stockpiled earth.

Although soil erosion is currently not a significant source of CSO-related pollution identified during this study, erosion and sediment control practices should be enforced at all future construction sites. Additional controls will not be considered.

De-icing Practices

The use of salt on roadways to reduce icing during winter months presents a potential detrimental effect on the surface runoff pollutant load, particularly chloride concentrations. Storage of salt is another source of chloride contamination. Proper storage of salt requires protection from rain to prevent runoff of salt solution.

Generally, salt is mixed with sand to reduce skidding on roadways. The sand is a source of grit, which may settle in combined sewers, particularly during thaw periods.

New Bedford uses about 1,000 tons per year of salt/sand mixture for de-icing of roads. Although chlorides were not identified as a receiving water quality issue, solids deposition in the sewer system, as discussed previously, may contribute to increased CSO related pollutant loads. Accordingly, it is recommended that the City adopt measures to maintain salt and sand levels to the minimum required to provide safe conditions for vehicular traffic during snow and icing conditions. De-icing will not be considered for further study.

Air Pollution Reduction

Particulate matter in the atmosphere that ultimately settles out is another potential source of stormwater runoff contamination. The so-called "dustfall" is a result of both natural causes (fugitive dust from soils and pollen) and man-made processes (grinding and pulverizing processes, combustion, construction dust, etc.).

This source of pollution is insignificant compared to other sources, therefore further evaluation is not warranted.

Porous Pavement

The quantity of runoff that enters a combined sewer system may be reduced or attenuated with the use of porous pavement. Rainwater is able to infiltrate through the paved surface into the ground if a properly draining base course is provided. It's use is potentially more cost-effective in new developments than existing paved areas because pavement removal and regrading necessary to rebuild developed areas is expensive, in addition to being disruptive to traffic. The area of New Bedford served by combined sewers is densely developed, thus it is unlikely that use of porous pavement would be cost effective. In addition, the long term benefits of porous pavement is questionable because pore spaces eventually plug with sand, silt and other particulates and may also fill with ice in the winter season.

For the above reasons, porous pavement appears to be of questionable value in terms of cost and reliability and implementation is disruptive. No further consideration will be given to this technology.

7.2.3 COLLECTION SYSTEM MODIFICATIONS

Collection system modifications can reduce CSO discharge frequency and volume by removing inflow sources, increasing interceptor capacity and optimizing the performance of the collection system.

Sewer Separation

The separation of a combined sewer system into separate sanitary and stormwater systems is accomplished by constructing new sanitary sewers and using the existing combined sewers as storm sewers (or vice versa). It must be recognized that the separate storm water system can continue to present a water quality problem, the impacts of which must be considered when evaluating the effectiveness of sewer separation.

Separation of the combined sewer system is a viable CSO abatement strategy for New Bedford and will be evaluated in greater detail.

Construction of a Parallel Conveyance System

The volume of CSO discharge may be reduced by diverting additional flow into the intercepting sewer system during wet weather. To provide the additional conveyance capacity, additional or parallel pipelines may be required.

Construction of parallel conveyance facilities is applicable to New Bedford and is evaluated in greater detail later in this section.

Regulating Devices and Tide Gates

Flow regulators are designed to divert dry weather wastewater flow to an interceptor and to divert flows in excess of the capacity of the

interceptor to receiving waters. Three types of regulators are commonly used, static, semi-automatic dynamic, and automatic dynamic. Static regulators provide the least control but are simple in design and operation with no moving parts. Semi-automatic regulators respond automatically to changes in flow but are susceptible to malfunction due to the adverse operating environment of sewers. Automatic regulators can be controlled remotely or locally.

Tide gates are sometimes installed on overflow conduits to minimize backflow of sea water into the sewer system during high tides. Without these gates tidal waters would enter the system consuming hydraulic capacity and possibly overloading the treatment plant.

All regulators in the New Bedford sewer system which tidally affected are equipped with gates to prevent inflow. The City has initiated a program of tide gate repair and maintenance to restore all tide gates to proper working order. It is estimated that all tide gate repairs will be completed by mid-1990.

Currently there are regulators in New Bedford's combined sewer system that sometimes overflow under peak dry weather flow conditions due to hydraulic limitations of the interceptor system. These limitations are due to: (1) excessive grit deposition within certain interceptor reaches, and (2) inadequate interceptor carrying capacity which results in flows backing up into regulators and out overflows during peak flow conditions. The feasibility of modifying regulators to provide capacity for all dry weather flows will be further evaluated later in this section. A program for assessing the extent of the grit (which contains significant concentrations of PCBs) and evaluating potential remedial plans is currently being developed by the City.

Remote Monitoring and Control/Flow Diversion

The diversion of flow can sometimes reduce the volume and frequency of CSO discharge by conveying flow from one drainage basin having limited hydraulic capacity to another, which has excess capacity. Existing

pipeline capacity may be utilized either to convey flow or as in-line storage (see discussion Section 7.2.4). Components include a data gathering system to monitor rainfall, pumping rates, treatment rates and regulator positions; a central computer processing center; and an instrumentation and control system to remotely regulate pumps, gates, valves and regulators.

Field observations and hydraulic model simulations indicate that no substantial excess flow capacity is available in any segment of the system. Use of flow diversion and control will be considered further, however, in conjunction with in-line storage.

Polymer Injection

The injection of certain polymers into a collection system can effectively decrease pipe friction thereby increasing a pipe's hydraulic capacity. A literature search was performed on the use and effectiveness of polymer injection in other combined sewer systems. Most of the studies available were done between 1969 and 1977 by the EPA. According to one source, the addition of polymer into gravity sewer lines could increase the flow through the pipes, forcing more flow to the treatment plant and less toward the overflow. Polymer slurry injections into gravity sewer lines have resulted in significant hydraulic friction reductions increasing pipeline capacities up to 144 percent.

Use of polymer injection in sewers requires the construction of remote stations where polymer may be stored and injected into the pipelines. In addition, instrumentation to monitor flow and regulate polymer dosage is required.

Several problems are associated with polymer injection as a CSO abatement technology, including:

1. Polymer may coagulate the smaller solids in the wastewater into larger solids which can not be kept in suspension due to insufficient velocities and hence reduce the cross-section of the pipe and necessitating additional pipe cleaning.

2. When wastewater is pumped, the molecular structure of the polymer may break down making it ineffective in reducing fluid friction.

3. Polymers have limited storage life.

4. The properties of polymers are affected by temperature requiring adjustments in dosing rates.

5. The cost of polymers is high.

In addition to the above, use of polymers is dependent upon the capacity of downstream facilities to either treat or store the increased flow which will occur.

Due to its high cost, complexity of operation, potentially high maintenance requirements and uncertain level of effectiveness, this technology will not receive further consideration.

7.2.4 STORAGE AND TREATMENT

Storage and treatment of CSOs can be localized or centralized. Treatment facilities meet CSO abatement objectives by reducing pollutant loadings prior to discharge. Storage facilities either eliminate the need for, or reduce the capacity requirements of CSO treatment facilities by holding wastewater until conveyance and treatment capacity at a central WWTP is available.

Storage can take the form of surface storage, in-line storage, or off-line storage. The stored flows can either be treated and discharged locally or stored until capacity is available in the interceptors and wastewater treatment plant to convey the flow to the WWTP or central CSO treatment facility.

Storage Technologies

In-Line Storage. Inflatable dams or automatic gates are devices used to create in-line storage. These devices take advantage of the existing collection system capacity and serve to retain storm-related flows and reduce hydraulic peaks.

Inflatable dams are generally located in combined sewers downstream of dry weather connections. The dams, which can be remotely controlled, are normally inflated to divert dry weather flows into the interceptor and store wet weather flows in the combined sewer. If flows exceed the system capacity and upstream flooding occurs the dams are deflated to release stored flow.

The concept of inflatable dams is similar to that of remote monitoring and control systems discussed previously in that flow monitoring is essential to properly divert flow in accordance with specific flow conditions.

Automatic gates have been used more frequently in such applications and are superior to inflatable dams in terms of precision of control and durability.

Other devices employed to utilize in-line storage are stop logs and orifice type restrictions. Stop logs provide operating flexibility similar to inflatable dams but require manual insertion and removal. Orifice type restrictions can be employed to utilize storage capacity in segments of combined sewers upstream of the dry weather connections. Inflatable dams are advantageous because they are less labor intensive than stop logs which must be placed and removed manually, often under adverse weather conditions. Orifice type restrictions are subject to the same maintenance requirements as static regulators to prevent clogging with debris. All in-line storage technologies may increase the cleaning and maintenance requirements of combined sewers since the storage of flow in pipes reduces flow velocities, increasing the possibility of deposition of solids.

The total volume of storage available in New Bedford's major combined sewers is small in comparison to the CSO discharge volume generated by storms of even moderate frequency. However, due to the limited hydraulic capacity of the interceptor system, New Bedford experiences extremely high frequency of CSO discharge because overflows occur even during extremely small storms. Consequently, use of in-line storage may eliminate the need to use major storage and treatment facilities during minor storms even though the effect on the capacity requirements on these facilities may be

negligible. The resulting savings in operation and maintenance costs may justify in-line storage technologies, thus their use will be further evaluated.

Off-line Storage. Off-line storage facilities serve to contain the volume of CSOs that exceed the system's capacity for conveyance and treatment. Types of storage facilities include underground tanks, abandoned pipelines, or deep tunnels. The location of off-line storage is usually at overflow points or near dry weather or wet weather treatment facilities. Land availability is a prime concern with the use of off-line storage. These facilities can be relatively simple in design and operation and can effectively reduce the volume frequency of overflows. Off-line storage is feasible in New Bedford and is evaluated in detail later.

Surface Storage. Storage of stormwater runoff prior to entering the collection system can be accomplished through roof storage, playground storage, in natural ponds, or in man-made basins or lagoons.

Roof storage can be effective in areas having many buildings with flat roofs. However, this method of storage could present a problem of water seepage into the buildings and/or damage to the structural integrity of the building. Roof storage is most attractive for new construction in warm climates where snow will not collect on flat roofs.

Playground and recreational areas can be used to detain rainfall for a limited time to reduce peak flow in the system. Space availability, public acceptance and potential hazardous conditions are drawbacks associated with this method. In addition, use of these facilities to store runoff may interfere with their intended use and increase maintenance requirements.

Depending on existing land use and the existing natural topography, temporary stormwater detention may be implemented for runoff attenuation. Retention of stormflow in areas having porous soils will allow some or all of the detained flow to infiltrate into the soil instead of entering the sewer system.

In general, open space in densely developed urban areas such as New Bedford is limited to park and recreational land and parking areas. Because use of these areas for storage of runoff would interfere with their intended use, this technology will not receive further consideration.

Treatment Technologies

Technologies available for localized treatment of CSOs are presented below.

Screening. Screens for wastewater treatment are available in various types and sizes ranging from bar racks, to coarse and fine screens, to microstrainers. Screens are effective in removing large solids and floatables from a combined system -- 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. Screens can be installed either in-line or at off-line facilities. In-line facilities must be closely monitored and cleaned to prevent loss of hydraulic capacity which could cause flooding.

Sedimentation. Gravity sedimentation using high surface overflow rates (to conserve space) achieve 20 to 40 percent removal of BOD and TSS in CSOs. Land requirements for this technology are high and residual solids handling is an important consideration.

Dissolved Air Flotation. Dissolved air flotation (DAF) has been used in San Francisco to treat CSOs, and has proved to be relatively effective in removing up to 20 to 50 percent of the suspended solids and floatables.

Swirl and Helical Concentrators. Swirl regulators/concentrators operate as a solids liquid separator removing both suspended solids and floatables through rotationally induced forces. Swirls have been seen to remove about 50 percent of the suspended solids in a combined system. Helical concentrators are similar in design to the swirl but are more effective as an in-line device rather than off-line.

Disinfection. Disinfection is utilized to destroy pathogenic microorganisms. A number of disinfection technologies are available including chlorine, sodium hypochlorite, ozone, and ultraviolet radiation. The most common method of disinfection is dosing with chlorine, although recently its apparent toxicity to aquatic life has been questioned. For this reason dechlorination is often required under certain conditions. In the marine environment, dechlorination has not always proven effective. The whole effluent toxicity test of the dechlorinated effluent for the draft Phase 2 WWTP Facilities Plan, indicates no decrease in toxicity for the dechlorinated effluent.

Table 7-2 compares the treatment technologies with respect to their effectiveness in meeting CSO abatement objectives for New Bedford. These technologies are further investigated in the following section. No treatment technology alone is adequate to meet all water quality objectives identified in this study. However, various combinations of treatment technologies may be employed to meet CSO abatement goals. This concept is discussed further in Section 9.0.

7.3 TECHNOLOGY INVESTIGATIONS

Based on the preceding discussions, the CSO abatement technologies that warrant further investigation are listed in Table 7-3. In this section, these technologies are investigated in greater detail to determine their possible role in a CSO abatement program for New Bedford. Those technologies which, upon further review, appear to be potentially significant and cost effective in reducing CSOs are fully evaluated in Section 9.0.

7.3.1 SYSTEM MANAGEMENT

Catch Basin Cleaning

A catch basin is a chamber or well, usually located at the curbline of a street, which allows surface water to enter a sewer or drain. A sump at the base of the structure traps grit and sediment below the outlet pipe and

TABLE 7-2

DESIGN CRITERIA AND EFFECTIVENESS OF CSO TREATMENT TECHNOLOGIES

TECHNOLOGY	POLLUTANTS REMOVED	ENERGY REQUIREMENTS	ABILITY TO INTERGATE WITH CSO STORAGE	OPERATING EXPERIENCE FOR HIGH FLOW RATES
Screening	Large solids	negligible	necessary to precede storage	extensive
Sedimentation	20 to 40 percent of BOD and TSS and portion of toxics associated with settleable material	low energy requirements for sludge removal	design of sedimentation facilities can include CSO storage	extensive
Dissolved Air Floatation	20 to 50 percent of BOD and TSS, floatables, and portion of toxics associated with floatable material	moderate to high energy requirements for dissolved air and pumping underflow	limited	limited
Swirl and Helical Bend Concentrators	Up to 50 percent of BOD and TSS and portion of toxics associated with solids removed	moderate to high energy requirements to pump flow into swirl and to pump underflow	not compatible	limited to flows up to 50 mgd

TABLE 7-3

FEASIBLE CSO ABATEMENT TECHNOLOGIES FOR NEW BEDFORD

SYSTEM MANAGEMENT

- Catch Basin Cleaning
- Regulator Inspection and Maintenance
- Sewer Cleaning/Flushing
- Infiltration/Inflow Control
- Improvement/Replacement of Conveyance System

SOURCE CONTROL

- Industrial Pretreatment and Spill Control
- Street Sweeping

COLLECTION SYSTEM MODIFICATIONS

- Sewer Separation
- Construction of Parallel Conveyance System
- Regulating Devices and Tide Gates

STORAGE AND TREATMENT

Storage Technologies

- In-Line
- Off-Line

Treatment Technologies

- Screening
- Sedimentation
- Dissolved Air Flotation
- Swirl and Helical Concentrators
- Disinfection

retains water which submerges a trap on the outlet. The purpose of the sump is to minimize sewer clogging by preventing coarse debris from entering the sewer system and to reduce odors from sewers by providing a water seal. However, studies have shown that water purged from catch basins during a storm has a high pollutant content that contributes to first-flush loadings.

Proper maintenance and cleaning of catch basins can improve the quality of storm runoff by maximizing the solids retention efficiency of the basin. The proper frequency for catch basin cleaning depends on several local parameters, such as sump capacity, quantity of accumulated street solids, antecedent dry periods, meteorological conditions, street cleaning methods, surrounding land use, topography, and the erodability of the soils subject to washoff. Based on responses to a comprehensive national survey, median catch basin cleaning frequency is once per year (APWA, 1973).

Studies have indicated a wide range of pollutant loadings related to catch basin discharge. Survey results of liquid samples from San Francisco's catch basins showed the following average pollutant characteristics: BOD_5-110 mg/l, total nitrogen-8 mg/l, total phosphorous-0.2 mg/l. A 1980 EPA-sponsored study reported the following typical pollutant concentrations for urban surface runoff: total solids-496 mg/l, total suspended solids-415 mg/l, BOD_5-20 mg/l, COD-115 mg/l, total nitrogen-10 mg/l.

Catch basins remove particulates and floatables from the waste stream before discharging to the sewer system. Floatables can be effectively removed if the outlet is equipped with a suitable trap. Their effectiveness in controlling other pollutants depends on the size distribution of the particulates in the wastestream and the design of the sump. A 1977 EPA-sponsored study reported that removal efficiencies between 40 and 80 percent for particulates over a 0.25 to 2.0 mm diameter range. Removal efficiencies for particles below this range were insignificant except at extremely low inflow rates. It was also found that removal efficiencies were unaffected by solids deposition of less than approximately half the sump depth but decreased dramatically at depths

exceeding half the sump depth. The performance evaluation was based on basins having a sump depth four times the outlet pipe diameter.

The 1980 EPA study reported that between 50 and 60 percent of pollutants in street runoff have particle sizes below 0.25 mm. This would imply that between about 20 and 50 percent of pollutant loadings from surface runoff could be removed in catch basin sumps.

For catch basins meeting the dimensional requirements set forth in the EPA study, catch basin cleaning should be scheduled to prevent solids deposition exceeding half of the sump depth. The standard catch basin in New Bedford's combined sewer system has a 12-inch diameter outlet pipe and a 24-inch sump depth, or half of the recommended depth to diameter ratio. Based on the reported results of the above referenced EPA studies, the removal efficiencies for New Bedford's catch basins are low due to their shallow sump depths. This implies that the catch basins are designed to serve their intended function of removing floating and dense materials from the wastestream, but are not effective in terms of runoff-related pollution control.

The City has initiated a catch basin inspection and cleaning program as recommended by CDM in the Wastewater Collection System Inspection Report prepared for New Bedford in 1987. Routine inspection resulted in cleaning of 1,041 catch basins during 1987. It is recommended that the City continue inspection and cleaning practices as necessary to minimize the required cleaning of sewers, however, expansion of the program is not warranted since reduction in runoff-borne pollution would be minimal.

Regulator Inspection and Maintenance

New Bedford has adopted a regulator inspection and maintenance program in conjunction with the other recommendations from the 1987 Wastewater Collection System Inspection Report. All regulators are inspected twice monthly and minor repairs are made and cleaning is performed as necessary. In addition, a priority list of major repairs has been formulated and implementation has commenced. Completion of all major repair projects is

expected by early 1990. Present monitoring efforts require two staff on a part time basis to inspect all regulators twice monthly. Inspection reports are maintained for all regulators to monitor their operation and repair records. The primary benefit of regulator monitoring is the prevention of maintenance related dry weather discharges by CSOs.

Wet weather operation of regulators is of minor importance because all of the semi-automatic regulators in New Bedford's sewer system have been converted to a fixed orifice type regulators. Experience in many combined sewer systems has shown that semi-automatic regulators designed to vary flow rate entering interceptors are impractical due to their high incidence of mechanical failure. For this reason, it is recommended that all former semi-automatic regulators continue to operate as fixed orifices.

The current inspection and maintenance practices of the City appear adequate to maintain the static type of regulators in the system. Expansion of the regulators' inspection/maintenance programs is not warranted.

Sewer Cleaning/Flushing

The City has implemented a sewer maintenance program as recommended in the 1987 Wastewater Collection System Inspection Report. The program includes regular inspection of major collectors and interceptors, cleaning of sewers on an as-needed basis and recording inspection and maintenance records. The Department of Public Works maintains a rodding machine and hydraulic jet rodder for cleaning of sewer lines.

The most severe sedimentation problem is approximately 6,000 feet of Rodney French interceptor between Holley Street and Willis Street. Due to the presence of toxic material (PCBs) in the grit, the City can not flush this section of the sewer without controlling and capturing the material. The City is currently developing a program for Phase I and Phase II Site Assessments in accordance with the Massachusetts Contingency Plan which will establish a plan for disposal of the grit.

Review of the maintenance reports indicates that the major collectors remain relatively clean and free of debris. The majority of sedimentation and blockage has been reported to be in several of the smaller collectors and generally sewers confined to particular problematic areas. Although these areas require periodic maintenance, they are not a CSO-related problem. Consequently, no additional sewer maintenance procedures are warranted.

Infiltration/Inflow Control

It is estimated that over half of the existing average dry weather wastewater flow reaching the wastewater treatment plant in New Bedford is infiltration and inflow (I/I). Of the estimated 15.3 mgd I/I, 1.3 mgd is attributed to tidal inflow and the remaining 14.0 mgd to infiltration and other inflow sources. The design (year 2014) flows for the secondary wastewater treatment facilities are based on implementation of I/I reduction measures and include flows of 0.7 mgd for tidal inflow and 11.4 mgd from infiltration. Excluding the tidal inflow, this represents a 19 percent reduction in I/I.

DEP guidelines for performing Phase I Sewer System Evaluation Surveys estimate maximum infiltration removal rates of 50 percent for individual sewer segments and manholes. Allowing for infiltration from building services and unidentified sources, the maximum overall removal efficiency is in the range of 30 to 40 percent. These assumptions are supported by experience in performing sewer system rehabilitation projects.

To assess the benefit of I/I removal, the impact on CSO discharge volume was determined for two conditions:

1. Wastewater facilities design flow which includes an I/I reduction of 19 percent from current levels.
2. Expanded I/I removal program which includes an I/I reduction of 40 percent from current levels.

Average annual CSO discharge volumes were estimated for each condition using the STORM model.

Costs for sewer system rehabilitation for I/I removal were developed from the Phase I Sewer System Evaluation Survey conducted for New Bedford by CDM in 1984. Based on the costs of I/I removal developed in that report, the relationship between cost and I/I removal was extrapolated up to a removal rate of 40 percent as shown on Figure 7-1.

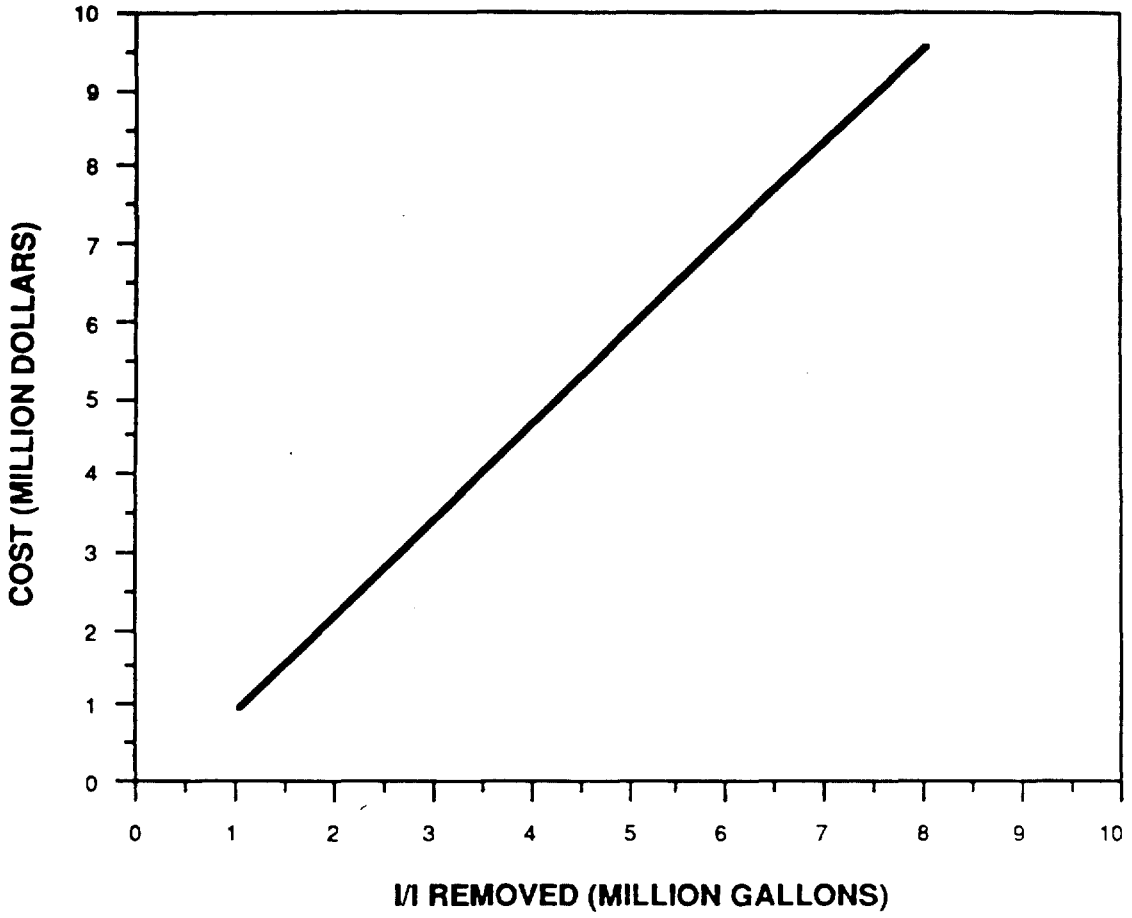
The results of the I/I reduction analysis are shown on Table 7-4. The average annual CSO discharge volume for the design case (Base Conditions) is 709 million gallons at an estimated system rehabilitation cost of 2.74 million dollars. Expanding the I/I removal program to achieve 40 percent removal reduces the annual CSO discharge to 689 million gallons at a system rehabilitation cost of 6.67 million dollars. The additional reduction in CSO discharge of 20 million gallons requires an incremental cost increase of 3.93 million dollars, or $.20/gal. This cost exceeds the incremental cost of other CSO reduction strategies. For this reason, expansion of the I/I removal program is not warranted on the basis of CSO abatement.

Improvement/Replacement of Conveyance System

Improvements to the sewer system were recommended in the wastewater facilities plan to increase the system capacity to convey peak dry weather flow to the wastewater treatment plant. Included were the upgrading of several pumping stations, modification of several regulator structures, and the replacement of the interceptor on Cove Road. The recommended sewer system improvements are listed on Table 7-5.

In addition to the above improvements, additional system modification may be recommended as part of the grit removal plan currently being developed by the City. Presently, the capacity of the main interceptor is limited by accumulation of grit upstream of Willis Street. Due to the presence of hazardous materials in the sediment, its disposal must comply with the provisions of the Massachusetts Contingency Plan. The replacement of segments of the pipeline may be included under the final recommendations.

The I/I removal plan recommended in the Phase III SSES includes the rehabilitation of miles of pipelines by lining, grouting of joints, or



City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN-PHASE 3

FIGURE 7-1
COST vs. I/I REMOVAL

TABLE 7-4

INFILTRATION/INFLOW REDUCTION IMPACTS

ALTERNATIVE	I/I REDUCTION	ANNUAL VOLUME CSO DISCHARGE (MG)	COST	INCREMENTAL UNIT COST ($/GAL.REM.)
No Action	None	735	0	
Meet Design Flow Criteria For Secondary Wastewater Treatment Facilities	46% of Tidal Inflow 19% of Remaining I/I	709	$2,740,000	.12
Expand I/I Reduction Program	46% of Tidal Inflow 40% of Remaining I/I	689	$6,670,000	.20

TABLE 7-5

RECOMMENDED SEWER SYSTEM IMPROVEMENTS
DRAFT PHASE 2 WWTP FACILITIES PLAN

IMPROVEMENTS REQUIRED FOR BOTH
FORT RODMAN AND STANDARD TIMES SITES

Cove Road Pumping Station Replacement

New Ruth Street Pumping Station

Abandon Apponogansett Street Pumping Station

Cove Road Interceptor Replacement

Ruth Street P.S. Interceptor

Modify Five Existing Regulators

Main Interceptor Partial Replacement Or Grit Removal

Cove Road P.S. Force Main

Ruth Street P.S. Force Main

Influent Sewer To Treatment Facility From Main Interceptor

Outfall Connection to Treatment Facility

IMPROVEMENTS REQUIRED FOR
STANDARD TIMES SITE ONLY

Sewer Connection From Main Interceptor To Plant Influent Sewer

West Clarks Point Pumping Station

West Clarks Point Force Main

West Clarks Point Collector Sewers

Connection From Effluent Conduit (Converted Main Interceptor)
To Outfall

pipeline replacement. Based on the results of the analysis presented in the preceding section on I/I removal, no additional pipeline replacement is warranted to reduce I/I related CSO discharge.

7.3.2 SOURCE CONTROL

Commercial/Industrial Runoff Control

New Bedford's Industrial Pretreatment Program (IPP) was approved by the Massachusetts DEQE in 1985. Permits were issued for all industrial users by 1987.

The IPP provides for sampling twice annually and inspection once annually of each industrial user by the City. One of the two samplings is performed without the prior knowledge of the user. Categorical industrial users discharging in excess of 25,000 gal/day to the municipal sewer system are required to sample and analyze effluent weekly, bi-weekly, or monthly and report the results to the City on a monthly basis. The results of the IPP are reported to DEP semi-annually by the City in accordance with the requirements of the NPDES permit for the municipal sewer/treatment facilities.

There are presently twenty-six significant industrial users and forty-four non-significant industrial users discharging to the municipal sewer system (significant users are those industries that discharge in excess of 25,000 gal/day). Sixteen of the significant industrial users are categorical users.

The City's monitoring and enforcement efforts have been successful in maintaining pollutants within the limits established in the sewer use ordinance and individual industrial discharge permits. Notices of violation and compliance schedules issued by the City in response to violations have resulted in subsequent compliance by industrial users. Since it appears that present monitoring is adequate to meet the requirements of current discharge limits reduction of industrial borne

pollutants from CSOs would require reduction of the existing discharge limits.

The local discharge limits for metals under the New Bedford Sewer Use Ordinance are shown on Table 7-6. Of the metals listed, lead, zinc, copper and chromium were detected in CSO discharges. Lead concentrations in CSO discharge samples were found to be about one-fifth of the discharge limit, while all other metals in samples were found to be in concentrations less than ten percent of the discharge limit.

Revision of the local discharge limits was recommended in the draft Phase 2 WWTP Facilities Plan on the basis of water quality impacts of the proposed treatment plant outfall. No further revisions are warranted on the basis of CSO-related water quality issues.

Street Sweeping

The City maintains a sweeping program to remove dust, dirt, and litter from streets. Sweeping is performed between April and November using three mechanical sweepers. Sweepings are disposed of at the City Landfill at no cost. The most recent annual operating budget for sweeping was $100,000.

In 1988, approximately 4,100 miles of street were swept, an average of 15.8 passes annually for the City's 260 miles of streets. The frequency is about once every two weeks during the sweeping period.

An EPA study (1977) reported street sweeping effectiveness to be a function of the following:

- Sweeper Efficiency
- Cleaning Frequency
- Number of Passes
- Equipment Speed
- Pavement Conditions

TABLE 7-6

LOCAL LIMITS FOR WASTEWATER DISCHARGES TO THE SEWERAGE SYSTEM

Constituent	Limit mg/l
Arsenic	1.40
Codmium	1.20
Chromium	7.00
Copper	4.50
Cyanide	1.90
Lead	.60
Mercury	.01
Nickel	4.10
Silver	1.20
Zinc	4.20

- Equipment Type
- Public Awareness

The rate of dust and dirt accumulation is related to the land use within a particular area. In addition, sand accumulates on streets due to de-icing practices during winter. As noted earlier, New Bedford uses about 1,000 tons annually of sand/salt mixture for street de-icing. Estimated daily dust and dirt accumulation are shown on Table 7-7. These estimates are based on the following mean accumulation rates for each land use type found in EPA-sponsored studies:

LAND USE TYPE	RATE (lbs/curb-mile/day)
Single Family	45
Multiple Family	110
Commercial/Industrial	200
Open Space	70

As shown on the table, it is estimated that about 23,000 pounds or 11.5 tons of dust and dirt accumulate on the City's streets within the combined sewer area daily.

The fraction of total dirt, dust, and sand that is washed off streets by rainfall is a function of the particle size distribution. It is estimated that generally about 60 percent of dust and dirt and that 40 percent of sand would be conveyed by runoff. The resulting annual volume of solids that would be washed off of streets is about 2,900 tons in the absence of street sweeping. Based on the results of sampling performed during this study the estimated annual TSS loading discharged through CSOs is 751,000 pounds or 375 tons implying that the volume of solids actually entering the sewer system is significantly less.

As noted earlier, the efficiency of sweeping depends on several variables. Two items, equipment type and frequency of cleaning, significantly impact the effectiveness of sweeping. Mechanical sweepers such as that used by the City are relatively inefficient in removing fine material smaller than

TABLE 7-7

NEW BEDFORD STREET DUST AND DIRT ACCUMULATION

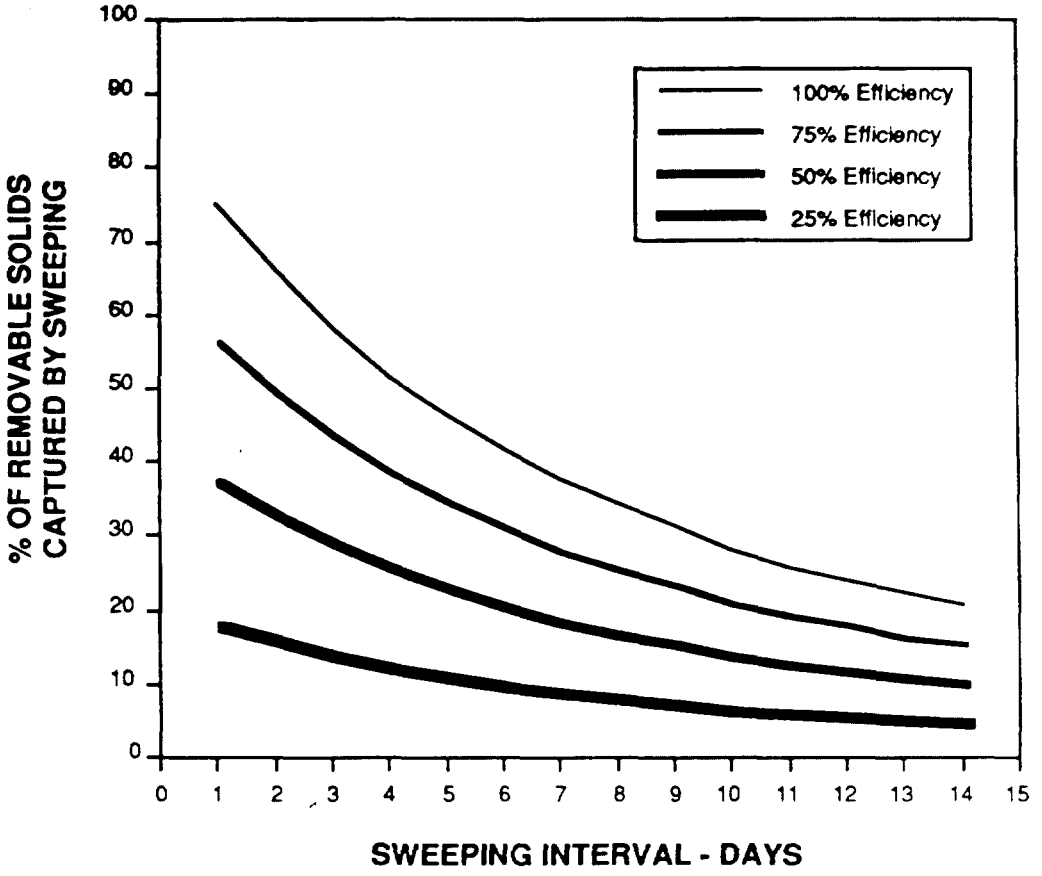
	AREA (ACRES)					DAILY DUST & DIRT ACCUMULATION (LBS)				
Subbasin Number	Low Density Res.	High Density Res.	Ind.& Comm.	Open Space	Percent Combined Sewers	Low Density Res.	High Density Res.	Ind.& Comm.	Open Space	Total
NB1	57	313	236	8	28.7	48	665	913	9	1635
NB2	24	138	35	47	52.5	39	532	242	118	931
NB3	266	1005	450	224	46.1	371	3422	2780	484	7057
NB4	52	80	68	44	15.2	24	89	134	33	280
NB5	0	67	61	10	47.1	0	236	389	24	649
NB6	0	154	120	1	68.0	0	776	1101	3	1880
NB7	0	18	230	37	15.4	0	22	470	28	520
NB8	0	122	41	8	88.3	0	798	483	33	1314
NB9	0	0	204	9	16.0	0	0	443	5	448
NB10	0	36	162	2	56.0	0	148	1222	5	1375
NB11	96	246	224	38	65.2	190	1182	1961	118	3451
NB12	0	43	71	4	84.7	0	266	806	14	1086
NB13	165	616	380	102	7.5	36	340	389	38	803
NB14	232	239	266	487	0	0	0	0	0	0
NB15	0	188	16	0	97.1	0	1352	208	0	1560
	892	3265	2564	1021		708	9828	11,541	912	22,989

0.4 mm. A great portion of the pollutional potential is associated with the fine solids fraction of the street surface contaminants. The removal rates of a mechanical sweeper have been reported to vary from 10 to 60 percent, with 25 percent as a probable average. With a parked cars problem, the actual removal rate may be lover.

Vacuum type street sweepers, which are widely used in Europe and South America, are becoming popular in the United States. A leading manufacturer has estimated that about 10,000 of their units are in use worldwide, about 2,000 in U.S. and 300 in Canada. This type of sweeper has attained 95 percent efficiency under testing conditions. However, actual performance would probably not exceed 50 percent due to parked vehicles and traffic Each unit costs about $95,000, about $25,000 more than a mechanical sweeper.

The average annual precipitation for New Bedford is about 45 inches with a mean inter-event interval of about 4 days. Based on this information, Figure 7-2 was developed relating cleaning frequency to solids removal for various cleaning efficiencies. As shown on the figure, only about five percent of the removable solids are swept at the current bi-weekly frequency. Because sweeping in winter is impractical due to snow banks and freezing, it is estimated that the maximum annual volume of solids that could be swept which would otherwise be conveyed by runoff is 1500 tons. The resulting impact of sweeping is removal of about 75 tons of runoff-conveyed solids and a reduction of CSO-related TSS loadings by 11 tons.

Doubling the street sweeping effort would only increase the effectiveness to about 9 percent of removable solids reducing CSO related TSS loading by 7.5 tons annually. Replacement of existing sweepers with vacuum type sweepers and expanding to weekly sweeping would reduce CSO-related TSS loadings by 31 tons annually. Table 7-8 summarizes the costs and benefits in terms of reduced solids loadings associated with CSO discharges for the existing and expanded street sweeping programs.



City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN-PHASE 3

FIGURE 7-2
STREET CLEANING FREQUENCY vs. EFFICIENCY

TABLE 7-8

STREET SWEEPING COSTS AND IMPACTS ON CSO SOLIDS LOADINGS

Alternative	ANNUAL TSS LOADING DISCHARGED BY CSO'S (lbs)	ADDITIONAL COST[1] Capital	O&M[2]	Total
Maintain Present Sweeping Schedule	751,000	0	0	0
Expand Sweeping Program to Weekly Frequency	734,000	$260,000	$728,000	$ 988,000
Expand Sweeping Program to Weekly Frequency and Repalce Equipment	690,000	$620,000	$558,000	$1,178,000

(1) Cost over current expenditures

(2) O&M present worth over 20-year planning period

Because of its limited efficiency increased street sweeping will not substantially decrease CSO-related solids loadings in New Bedford receiving waters. In addition, sweeping has little impact on coliform loadings. For these reasons, expanding the City's sweeping programs is not warranted as a CSO abatement.

7.3.3 COLLECTION SYSTEM MODIFICATIONS

Sewer Separation

Sewer separation is defined as the division of an existing combined sewer system into non-interconnected sanitary and storm sewer systems. The sanitary sewer system is tributary to a wastewater treatment facility, and the storm sewer system discharges, without treatment, directly to receiving waters. Complete sewer separation will eliminate CSOs. However, heavily urbanized areas such as New Bedford can generate a significantly polluted stormwater runoff, and the negative impacts on receiving water quality may still be significant.

Theoretically, the quality of stormwater originating from rainfall should be higher than that of CSOs. In urban areas, however, stormwater runoff often becomes contaminated by sources such as excrement from domestic animals and organic loadings originating in industrial areas spread by vehicular traffic. It is possible for stormwater runoff, during certain storm events, to contain high coliform concentrations during and immediately following a storm event at the point of discharge. In addition, stormwater from highly developed urban areas generally contains substantial amounts of floatables. Oils from motor vehicles are typically collected in drainage systems from roadway and parking area runoff as well as paper and other debris.

Analysis of samples taken from separate storm drains during this study's sampling program have indicated that stormwater runoff contains significant concentrations of solids and metals and relatively small coliform concentrations. Accordingly, the separation of combined sewers by itself would not meet this study's objectives relative to mitigation of

floatables. Separation may partially meet certain objectives in conjunction with certain system management practices, such as street sweeping and catch basin cleaning.

The separation of all or portions of the system would result in substantial construction-related problems. The service connections from buildings in combined areas often serve the dual purpose of conveying both sanitary flow and drainage from foundation sumps, roof leaders, and similar sources to the combined sewer. Effective separation of these sources would require modifications to plumbing in buildings, which is generally very difficult and often entails extensive renovations. For the City of New Bedford to assume the expense of the building's plumbing separation would be costly, and may raise legal problems as well. To require such separation at the owner's expense would obviously produce many complaints, and would be very difficult to enforce.

System-wide separation has further disadvantages, primarily cost, time requirements, disruptions to traffic, and possibly other utilities. Large numbers of people are directly affected by full separation projects due to the need to excavate in virtually every street having a combined sewer. As a result, public response by residents, businesses, and commuters affected by excavation in streets is negative for a project of this magnitude.

The City has completed design for the separation of a substantial portion of the combined sewer system, however, no significant separation projects have been implemented. The estimated cost of complete separation of New Bedford's sewer system is $95 million including construction, engineering and contingencies. System-wide separation would entirely eliminate CSO discharges at a unit cost of $0.085 per gallon per year. This unit cost is comparable to other cost-effective CSO abatement strategies, however, the elimination of CSO discharge will cause a larger increase in stormwater discharge. As noted previously, stormwater-related pollutants, primarily solids and floatables, would continue to discharge to New Bedford receiving waters.

Due to its limited effectiveness, implementation problems relating to removal of connections, disadvantages in terms of traffic and utility disruption and construction impacts such as noise and dust system-wide separation is not recommended. Separation will be considered on a case by case basis for individual CSO outlets for which conveyance, storage, or treatment costs may be excessive.

Construction of Parallel Conveyance System

Providing additional system conveyance capacity in the form of new interceptors and/or pumping facilities primarily is contingent upon the availability of downstream conveyance and treatment facility capacity. System hydraulics, land/easement availability, and construction considerations also must be assessed.

New Bedford's wastewater is conveyed to the wastewater treatment facility by the main interceptor south from the Belleville Avenue force main to the south end of Clarks Point. The collector sewers generally flow southeasterly to the main interceptor which parallels the Acushnet River on the east side of the City. The full hydraulic capacity of the main interceptor at its downstream end is about 130 mgd, however, capacity constraints upstream limit the maximum flow rate to about 60 mgd without CSO discharges occurring. Recommendations in the draft Phase 2 WWTP Facilities Plan will increase the main interceptor hydraulic capacity to convey the design peak dry weather flow (71 mgd) to the treatment facility.

Because the secondary treatment facilities will have no more capacity than the interceptor system, no parallel conveyance pipelines are feasible unless built in conjunction with storage and/or treatment facilities. CSO abatement plans incorporating parallel conveyance with storage and treatment will be evaluated further, and discussed later in Section 9.0.

Regulating Devices and Tide Gates

The draft Phase 2 WWTP Facilities Plan recommended regulator modifications necessary to convey all dry weather flows to the treatment plant. After

implementation of these modifications, the segment of the main interceptor between River Street and the abandoned screen house (approximately 8,500 feet) will be able to convey about 69 mgd with a full pipe capacity (with CSO discharges) of 74 mgd. Due to the limited capacity available in the interceptor, regulator modifications that would divert additional flow into the interceptor are not feasible. Regulator modifications designed to utilize in-line storage will be further evaluated later in this section.

7.3.4 STORAGE AND TREATMENT

In-Line Storage

In-line storage can be a viable CSO abatement technology only if the existing sewer system has sufficiently large pipelines to provide significant storage volume. In order to assess the feasibility of in-line storage for New Bedford, all of the major combined collector sewers were analyzed under various storm conditions. The results of this analysis are shown on Table 7-9 and summarized below.

Table 7-9 shows the available storage volume and the total flow volume for each of the sewer major collector sewers for storms having 2-week and 3-month intervals. For the 2-week storm, the system-wide storage volume available is about half of the total combined sewage and runoff flow volume over the duration of the event. For the 3-month storm, the system-wide storage available is only about 5 percent of the total flow volume. Theoretically, if all available storage capacity were to be utilized for every storm event, the average annual reduction in CSO discharge volume is estimated to be 215 million gallons, a reduction of approximately 30 percent. However in practice, it is likely that CSO discharges could be reduced by about only 75 percent of this value or 160 million gallons annually.

Because the available storage is minimal in relation to the storage required for larger (low frequency) storms, use of in-line storage will have a negligible impact on the capacity requirements of downstream treatment/storage facilities. In-line storage does, however, have benefits

TABLE 7-9

AVAILABLE STORAGE VOLUME AND TOTAL CSO VOLUMES FOR
MAJOR COLLECTOR SEWERS - TWO WEEK STORM AND THREE MONTH STORM
(ALL VOLUMES IN MILLION GALLONS)

	TWO WEEK STORM		THREE MONTH STORM	
SEWER COLLECTOR	AVAILABLE STORAGE VOLUME	TOTAL CSO VOLUME	AVAILABLE STORAGE VOLUME	TOTAL CSO VOLUME
Willis Street	0.54	1.14	0.45	4.91
Grape Street	0.35	0.88	0.15	4.60
Liberty Street	0.88	2.68	0.08	13.63
Tripps Brook	1.11	2.03	0.68	10.34
Belleville Avenue	0.04	0.45	0.01	1.83
Sawyer Street	0.39	1.07	0.25	4.52
Church Street	1.01	0.82	0.85	3.52
TOTAL	4.32	9.07	2.47	43.35

in reducing operation and maintenance requirements for off-site storage facilities by reducing the number of storm events for which they must operate.

Figure 7-3 shows the major collector sewers that may be used for providing in-line storage as part of a system-wide CSO abatement plan. Flow would be retained in the collector sewer by the installation of static controls (orifices and weirs) and modification of existing regulators to utilize the available storage. Because of its apparent potential as a cost-effective CSO abatement strategy, the use of in-line storage will be further evaluated in conjunction with other storage and treatment technologies.

Off-Line Storage

Off-line CSO storage and eventual pumpback to the interceptor system is one of the most widely used and effective methods for CSO mitigation. It involves the capture and storage of CSOs in tanks (usually large underground concrete tanks or tunnels). The stored volume is pumped into the interceptor system and to the WWTP when flow capacity is available. This type of CSO mitigation eliminates overflows generated by storms up to a specific intensity and/or duration.

Most large combined sewer systems require off-line storage or satellite CSO treatment systems to eliminate or significantly reduce CSO-related pollution. Off-line storage facilities have demonstrated their effectiveness in controlling stormwater and combined sewer overflows. Many regional plans include storage or combinations of storage alternatives as an integral part of the overall CSO abatement process.

Storage facilities can be designed to hold the settled solids with periodic cleaning by dredging, mechanical chain and flight scrapers, or other means. When the solids are held in storage, some primary treatment (sedimentation) may be provided, thus reducing solids loadings at the WWTP. The suspended solids in the stored mixed flow will settle due to quiescent conditions. The settled solids can be handled by the following options:

- Collect and pump to the interceptor as a concentrated slurry to be handled at the WWTP during the event.
- Collected, stored and pumped to the interceptor as a concentrated slurry to be handled at the WWTP after the event.
- Collected and dewatered at the storage site then transported to the sludge processing facilities.
- Resuspended in the stored mixed flow during the pump back period for transportation to and handling at the WWTP.

Excessively long detention times can result in the settled solids becoming anaerobic with the resulting gas production acting to resuspend solids and cause offensive odors. Accordingly, prompt flow evacuation is necessary and may require that supplemental treatment capacity be provided at the WWTP to provide adequate flow capacity.

Off-line storage will be evaluated during the detailed evaluation phase. Facilities for each overflow and/or groupings of overflows will be sized and cost estimated at various CSO control levels in order to determine the optimum level of CSO control.

Screening

Screening is typically used in conjunction with storage or treatment systems for CSO abatement. For this study, it would serve to primarily reduce floatables. Bar screens are almost always installed at the entrance to storage or 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 1-1/2" bar spacing. Finer screens with 1/2" to 1" spacings are located just downstream. A double screen setup also has less tendency to be blocked than one fine screen.

In lieu of stationary fine bar screens, traveling woven wire mesh screens are sometimes used. These types of screens offer higher removal efficiencies due to the mesh design, however, operation and maintenance requirements are extremely high. Since the media are cleaned by applying a high velocity water jet spray, handling and disposal of this sidestream

would greatly increase the complexity of the operation, as well as the required building size, operational requirements and, consequently, costs. In addition, the head loss through this unit is two or three times that of a stationary unit. Accordingly, this method of fine screening is not considered applicable for use at CSO storage or treatment facilities.

In addition to its use for preliminary treatment of combined wastewater at treatment and storage facilities, screening alone will be evaluated as a physical treatment process for CSO discharges where a higher level of treatment may not improve receiving water quality.

Sedimentation

Sedimentation reduces solids loadings from CSO by gravitational settling and removal of suspended solids. It will also reduce metals and BOD loadings to a lesser degree because a fraction of each of these are associated with particles that can be removed by gravitational settling.

Several features of sedimentation make it advantageous to other technologies as a CSO abatement strategy.

- Sedimentation basins can also provide storage capacity which, if used properly, can reduce the volume treated and discharged to the receiving waters.
- High rate sedimentation provides detention times in the range of one to two hours which also allows for disinfection in the basins.

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, it will be considered in developing CSO abatement plans for New Bedford.

Dissolved Air Flotation

Dissolved air flotation (DAF) uses the formation of small air bubbles on suspended particulate matter to float the particulate matter for removal. After the combined sewage is saturated with dissolved air under pressure, depressurization releases the air from the solution as small bubbles. Oil, grease, and other floatables can also be removed. Small and light suspended matter can be removed more efficiently and quickly than by sedimentation. Chemical addition (generally polymer) is usually used to improve removal efficiency. Operating costs are relatively high due to the pumping costs to pressurize the water and compressed air and chemical requirements. The process is also sensitive to operational control.

DAF has been used primarily for processing sludges in municipal and industrial water and wastewater treatment applications. Due to the relatively high operating costs and sensitivity to operational control associated with this process, other less costly and complex technologies have been developed that have replaced the use of DAF in many of these applications.

In addition to its high cost and operational complexity, DAF is unproven on a large scale basis for CSO abatement and the process is not amenable to operational demands that may be encountered in CSO applications such as start-up on short notice and highly variable flow rates.

For the above reasons, DAF is not considered to be a feasible treatment process for CSOs and will not be included in our evaluation of CSO abatement plans.

Swirl and Helical Bend Concentrators

Swirl and helical bend concentrators regulate both the quantity and quality of combined sewage. Solids separation is caused by the inertia differential resulting from a non-linear path of flow travel. The flow is separated into an overflow, and a concentrated low volume of wastewater that is intercepted for treatment at a treatment plant (underflow).

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

1. The unit performs the dual function of both regulating flow to the interceptor system and treating CSO discharge.
2. Land area requirements are low compared to 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.

Based on the above, it appears that swirl or helical bend concentrators could potentially provide, a low cost and efficient technique to successfully regulate and treat combined wastewater. The use of these units is, however, subject to several 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 will require use of either high rate disinfection or construction of contact tanks to provide adequate detention time for bacteria kill.
3. The configuration of the swirl concentrator results in large loss in hydraulic head between the influent combined sewer and the underflow pipe.

Items 1 and 3 above could result in the need for storage and pumpback facilities in conjunction with a concentrator. Interceptor and treatment capacity must be available for the underflow during a storm event. If underflow rates exceed the available interceptor capacity or sufficient grade is not available, storage of the underflow may be required with

pumpback following the storm that may negate the advantage of relatively small land requirements associated with these facilities.

A further concern regarding swirl/helical bend concentrators is the lack of information on removal efficiencies from actual on-line facilities. As a result, there is little reliable full scale operational data to confirm the basis of design for concentrator units. A recent technical paper reported several observations regarding swirl concentrators in the United States. Performance results for a prototype unit in Syracuse, New York indicated favorable results for both BOD and TSS removal. The average storm flows during the test period, however, were less than 30 percent of the design flow used to size the unit making it impossible to verify predicted efficiencies. It was also reported that observations of a full scale concentrator in Lancaster, Pennsylvania appeared to exhibit excessive turbulence during peak flow periods. Finally, a swirl unit in Presque Isle, Maine is reported to be operating satisfactorily over the full range of its design flows. It was noted that the hydraulic loading rate for this unit is considerably more conservative than that prescribed in the EPA design handbook (approximately 1/3 of the loading rate for the Lancaster and Syracuse concentrators).

Swirl/helical bend concentrators will receive further consideration in this study. The uncertainty concerning the efficiency of their solids removal capabilities will be reflected in establishing preliminary design parameters and our recommendations.

Disinfection

The disinfection agents used for a wastewater and stormwater treatment include chlorine, hypochlorite (calcium and sodium), chlorine dioxide, and ozone. These four potential disinfection agents have some common characteristics; all are oxidizing agents, corrosive to equipment, and are highly toxic to both microorganisms and other life. Shown in Table 7-10 are the characteristics of principal CSO disinfection agents. Ultraviolet has been used as disinfection agent, but has proven ineffective in the turbid mixed flows of a CSO discharge.

TABLE 7-10

CHARACTERISTICS OF PRINCIPAL COMBINED SEWER OVERFLOW DISINFECTION AGENTS (1)

Characteristic	Chlorine	Hypochlorite	Chlorine Dioxide	Ozone	Bromine Chloride
Stability	Stable	6 Month Half-Line	Unstable	Unstable	Unstable
Reacts with Ammonia to Form Chloramines	Yes	Yes	No	No	Yes
Destroys Phenols	At High Concentrations	At High Concentrations	Yes	Yes	At High Concentrations
Produces a Residual	Yes	Yes	Short Lived	No	Short Lived
Affected by pH	More Effective at pH 7.5	More Effective at pH 7.5	Slightly	No	More Effective Between pH 6-10
Hazards	Toxic	Slightly Toxic	Toxic, Explosive	Toxic	Toxic

(1) From "Urban Stormwater Management and Technology Update and User's Guide," U.S. EPA, September 1977. Table 121.

Selection of a CSO disinfection system should be based on the following considerations:

- CSO's are highly variable both in quantity and quality. Thus, any disinfection system must have the capability to meet these fluctuations.
- Chlorine, chlorine dioxide, and ozone are all dangerous gasses that must be carefully handled by competent operators. Lesser hazards are associated with hypochlorite, which requires bulk storage and is somewhat unstable unless diluted.
- The coliform group of indicator organisms have a relatively low chlorine resistance when compared to such pathogens as entericviruses and protozoan cysts. Thus, chlorine or other disinfectant dosages may be effective with coliforms but not with viruses and cysts. Higher dosage levels may be required.

An important consideration in the selection of a disinfection system is the capacity and location of the treatment facility. Use of toxic gases is undesirable in densely populated areas and small-scale facilities that are only monitored on a periodic schedule. Common source disinfection systems should be considered for CSO facilities located near wastewater treatment facilities for dry weather flow.

The presence of shellfish resource areas in Clarks Cove and the Outer Harbor makes chlorination (and dechlorination) a questionable CSO abatement technology for these two receiving water bodies. Disinfection will be considered in the Inner Harbor.

7.4 ASSESSMENT OF TECHNOLOGIES

This section presents descriptions of various CSO abatement technologies and an assessment of their effectiveness and limitations in general and as they pertain to New Bedford's CSO abatement objectives and sewer system limitations. This assessment forms the basis upon which system-wide CSO abatement plans will be developed. Our conclusions are as follows:

System Management

The City has adopted recommendations made in previous studies on the management of the sewer system. After completion of the tide gate rehabilitation program, improved system management will have essentially eliminated all maintenance-related CSO discharges with the exception of those caused by the existing grit in the main interceptor. The City should maintain existing system maintenance practices.

Source Control

The City's present monitoring and enforcement efforts of the Industrial Pretreatment Program appear to be adequate. Due to its limited effectiveness in meeting CSO abatement objectives, no additional street sweeping is warranted.

Collection System Modifications

Except for sewer separation, modifications to the sewer system are contingent upon use of other technologies to provide storage or treatment capacity necessary for their implementation. Construction of parallel sewers and flow regulating devices and modification of existing regulators will be evaluated in conjunction with storage and treatment technologies in developing system-wide CSO abatement plans.

Due to its limited effectiveness in meeting receiving water quality objectives, sewer separation will only be considered on a limited basis.

Storage and Treatment

Both in-line and off-line storage will be included in developing system-wide CSO abatement plans. Treatment technologies that will be considered in various combinations are screening, sedimentation, swirl/helical concentrators, and disinfection. The level of treatment to be provided will be based on the receiving water quality requirements on a site by site basis.

7.5 SCREENING OF ALTERNATIVES

CSO technologies have been described and their application investigated in the previous section. The findings of the investigations indicate that system management, source control, and collection system modifications will be part of the recommended plan, but are not sufficient without storage and treatment. This section examines the appropriateness of the various storage and treatment alternatives for each of the three receiving water bodies: Clarks Cove, Outer Harbor, and Inner Harbor.

The three receiving water bodies have different water quality classifications and hence have different levels of CSO abatement may be warranted for each. Clarks Cove and Outer Harbor are designated Class SA. Both water bodies contain viable shellfish resources and beaches. These water bodies have high beneficial uses and the Division of Water Pollution Control Interim CSO guidelines state the Division's intent to avoid or minimize impacts on critical use areas. The Inner Harbor is designated Class SB, which is considered fishable/swimmable, with restricted harvesting of shellfish. Currently, because of the high levels of PCBs in the sediment in addition to the high coliform counts, the Inner Harbor is closed to shellfishing. The Inner Harbor's major use is limited to commercial and recreational boating.

Storage and treatment abatement technologies are evaluated by their ability to achieve water quality standards in the analysis that follows. Appropriate technologies are recommended for further analysis in Section 9.0.

Analysis in Section 6.0 indicates that CSO discharges probably contribute to violations for the following water quality criteria: aesthetics, fecal coliform bacteria, and toxic substances. Individual metals, such as copper and lead, would exceed the Other Constituents criteria using EPA "Gold Book" values. However, whole effluent testing performed for this study indicates the toxic area would not exceed the mixing zone, an acceptable limit. The analysis presented in Section 6.0 indicates that dissolved oxygen and nutrients criteria are currently not being violated.

7.5.1 CLARKS COVE

Under the improved Base Conditions (modifications to the sewer system to increase its capacity recommended in the draft Phase 2 WWTP Facilities Plan), Clarks Cove receives CSO discharges totaling 360 MG per year from CSOs Groups 1 and 2. CSO 004 in Group 1 discharges approximately 45 times a year, which results in high fecal coliform bacteria levels much of the year.

To achieve the standards for aesthetics and fecal coliform bacteria, screening and disinfection would be needed. Screening would remove floatables and disinfection would reduce fecal coliform bacteria loadings to acceptable levels. However, disinfection, whether accomplished by using chlorine, hypochlorite, chlorine dioxide, ozone and bromine chloride, is toxic to shellfish larvae. Though the existing WWTP effluent is chlorinated it has no apparent negative effects on the harbor shellfish resource. However, the effect of a chlorinated effluent located at the shoreline, an area with limited dilution, may have an adverse impact on shellfish larvae. Dechlorination facilities could be constructed but are expensive and studies indicate that the dechlorinated effluent may still be toxic. Based on this finding, disinfection will not be part of the recommended CSO abatement plan for in Clarks Cove.

Sedimentation and swirl and helical concentrators reduce solids in a CSO discharge. Removal of solids reduce the discharge loadings of BOD and metals and hence reduce the toxicity of the discharge. However, the CSO discharge still occurs and to meet coliform standards, disinfection would be required. The disinfection requirement eliminates sedimentation and swirl and helical concentrators from further consideration in Clarks Cove.

Sewer separation is a viable method to eliminate CSOs. However, as discussed in Section 7.3.3, the stormwater runoff carries a significant pollutant load, particularly fecal coliform bacteria, to the receiving water. The resulting improvement in the receiving water quality from the removal of the sanitary discharge while increasing the storm drain discharge would be insufficient to achieve Class SA standards. Though

sever separation is an acceptable method for CSO abatement, Clarks Cove would not attain the designated water quality standard.

The remaining CSO abatement technology to consider is off-line storage. The Commonwealth's water quality standard for fecal coliform bacteria will be examined to determine the required storage volume, or the frequency of overflows allowed. The fecal coliform bacteria standard is 14 bacteria per 100 ml, which in the CSO Interim Policy has been interpreted to be allowed to be exceeded 4 times a year for a duration of 24 hours each. An accumulated allowable excursion time of 96 hours is permitted per year. The land based model STORM (Section 4.0) and the receiving water model developed in Section 6.0 were used to determine the frequency of overflows and the duration of violations in the receiving water. The model findings indicate that the 6-month level of storage has the fecal coliform bacteria criteria being violated for 94 hours (2 events - 47 hours per event) in Clarks Cove from CSO Groups 1 and 2. The 3-month storage level, on the other hand, shows the water quality standard is violated for 188 hours (4 events - 47 hours per event), which is over the allowable 96 hours. Storage levels of less than 6-months can only be considered if the outflows from CSO Groups 1 and 2 are re-directed to another receiving water body, such as the Inner Harbor.

In summary, CSO Groups 1 and 2 in Clarks Cove require storage and treatment to achieve the water quality standards. The required level of abatement is at least up to a 6-month storm. If storage capacity to control the 6-month storm is not viable because of land constraints, then the CSO overflows could be re-directed to the Inner Harbor where it can be shown, a lower level of control is warranted.

7.5.2 OUTER HARBOR

The Outer Harbor under the Base Condition, receives CSO discharges totaling 22 MG a year from CSO Group 3. CSO 017 discharges approximately 49 times a year, which results in high fecal coliform bacteria levels much of the year.

As in Clarks Cove, the Outer Harbor contains a large shellfish resource and numerous beaches. The presence of shellfish will preclude the use of disinfection as a CSO control, and hence, as in Clarks Cove, storage will be considered as the appropriate technology for further considerations.

The receiving water model was used to evaluate the appropriate level of storage. For this analysis the Outer Harbor was divided into the Fairhaven side and the New Bedford side. The corporate boundary is the dividing line. This was done to compare the areas affected by storm drains located in Fairhaven, CSOs and storm drains in the Inner Harbor, and CSO Group 3. Table 7-11 presents the findings from the analysis. The water quality impacts from storm drains in Fairhaven and the Inner Harbor preclude water quality improvements for storage levels greater than the 3-month storm for CSO Group 3. The findings also suggest that the Outer Harbor will not achieve Class SA water quality standards until both storm drains and CSOs are controlled. Based on this finding, a 3-month level of storage is recommended as the minimum level of control for the CSO Group 3.

7.5.3 INNER HARBOR

The Inner Harbor will receive CSO discharges totaling 353 MG per year from CSO Groups 4, 5 and 6 under Base Conditions. CSO 020 and 037 discharge approximately 50 times a year, which results in high fecal coliform bacteria levels much of the year.

Unlike the other two receiving water bodies, the Inner Harbor has a limited shellfish population which is located between Route 6 (Popes Island) Bridge and the hurricane barrier, this is the receiving water for CSO Group 4. North of the Route 6 Bridge at CSO Groups 5 and 6, no viable shellfish resource are known to exist. An important issue in the Inner Harbor is that storm drains in Fairhaven side and New Bedford side will preclude attainment of the Class SB water quality standards for aesthetics and fecal coliform bacteria, even if all CSOs are eliminated. The target of CSO control in the Inner Harbor is to reduce the loadings of pollutants and improve water quality. Analysis of the appropriate CSO technologies will be examined by CSO group in the Inner Harbor.

TABLE 7-11

ANNUAL EXPECTED FECAL COLIFORM VIOLATION (HOURS)

OUTER HARBOR

Source/Resource	1-MONTH STORAGE (HOURS)				3-MONTH STORAGE (HOURS)			
	CSO Group #3	CSO Groups #4, #5, & #6	Fairhaven Storm	Inner Harbor Storm	CSO Group #3	CSO Groups #4, #5, & #6	Fairhaven Storm	Inner Harbor Storm
Outer Harbor (New Bedford)	564	336	391	368	188	336	391	368
Outer Harbor (Fairhaven)	42	126	2900	115	17	126	2900	115

Group 4

The receiving water designation for CSO Group 4 is Class SB and a viable shellfish resource is present. These shellfish are, however, affected by PCB contamination in the Inner Harbor; at least one sample taken in the lower Inner Harbor had PCB levels in hard-shell clam tissue above the FDA action limit (Battelle, unpublished). To effectively regulate aesthetics, fecal coliform bacteria and toxics, and protect the shellfish resource, storage is the only appropriate CSO control. A second reason for examining storage is that the lower portion of the Inner Harbor influences the water quality of the Outer Harbor, via the shipping channel and gates in the hurricane barrier. The level of storage chosen for Group 4 should match the 3-month level of control used for CSO Group 3 in the Outer Harbor.

Group 5 and 6

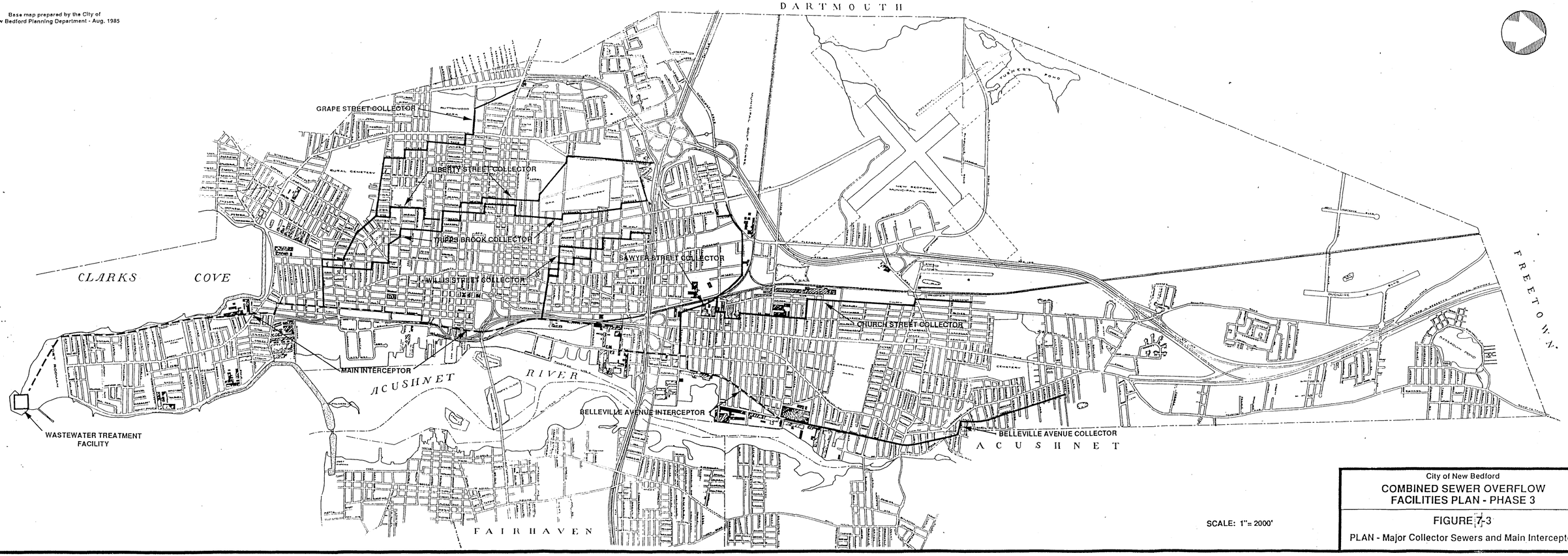
Improvement of the water quality in the middle and upper portions of the Inner Harbor can be attained by screening and disinfection. Sedimentation and swirl and helical concentrators should also be considered for these two CSO groups. The use of storage should be evaluated in light of the cost of storage and the benefits of improved water quality.

7.6 SUMMARY

Storage and treatment options were analyzed to determine the appropriateness of each in the attainment of the designated water quality standards. The findings are summarized as follows:

- CSO Groups 1 and 2 in Clarks Cove - A 6-month level of storage or the re-direction of the CSOs is required to meet Class SA standards. Disinfection is not recommended in Clarks Cove because of the viable shellfish resource.
- CSO Group 3 in the Outer Harbor - A 3-month level of storage is recommended. Storage levels greater than 3 months are not warranted because of the uncontrolled storm drains in Fairhaven side of the Outer Harbor and the storm drains in the Inner Harbor. The Outer Harbor will not attain Class SA water quality standards.

Base map prepared by the City of
New Bedford Planning Department - Aug. 1985



SCALE: 1"= 2000'

CDM

City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 7-3

PLAN - Major Collector Sewers and Main Interceptor

- CSO Group 4 in the Inner Harbor - The same level of storage as CSO Group 3 (3-month). This portion of the Inner Harbor will not attain Class SB water quality standards because of storm drain loadings.

- CSO Groups 5 and 6 in the Inner Harbor - Screening and disinfection is recommended at a minimum. Sedimentation is recommended to reduce the loadings of toxics. These portions of the Inner Harbor will not attain Class SB water quality standards even with total elimination of all CSOs.

SECTION 7.0 REFERENCES

American Public Works Association. 1973. Survey of Practices as to: Street Cleaning, Catchbasin Cleaning, Snow and Ice Control.

Pisano, W. and H. Brombach. 1987. Recent West German Experience with Vortex Solids Separators Provides New Motivation for Application of Swirl Concentrators for Control of Combined Sewer Overflow Pollution. Presented at National Water Pollution Control Federation Conference, October 1987.

U.S. Environmental Protection Agency. 1977. Municipal Environmental Research Laboratory. Urban Stormwater Management and Technology: Update and Users Guide.

8.0 SCREENING OF SITING ALTERNATIVES

8.1 INTRODUCTION

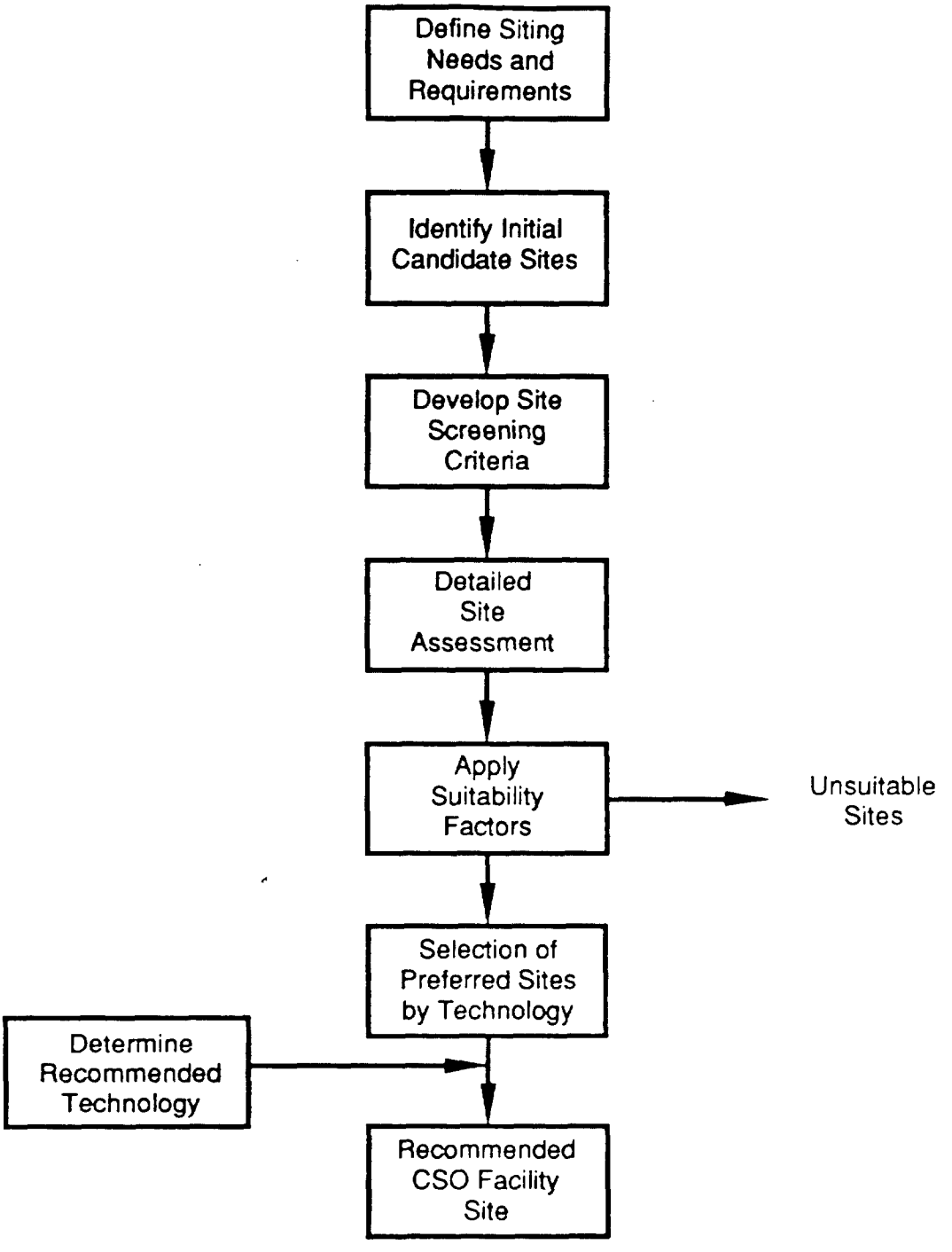
The evaluation of alternative sites for the various CSO abatement facilities consists of defining siting needs and requirements, identifying initial candidate sites based on set of minimum requirements, screening initial sites to eliminate unsuitable sites by applying specific siting criteria, and conducting a detailed site evaluation in which preferred sites are identified. Figure 8-1 presents the screening methodology flow chart. This section presents the process used to identify, evaluate, and select the preferred alternative site locations for the CSO abatement facilities. The siting criteria include environmental, technical, and socioeconomic considerations.

8.2 SITING NEEDS AND REQUIREMENTS

As identified in Figure 8-2, 38 combined sewer overflows exist in New Bedford. Presently, CSO 029 has been abandoned and CSOs 038 and 039 have been separated from the combined system. The remaining combined sewers discharge to the following receiving waters: CSOs 003 through 011 discharge to Clarks Cove, CSOs 012 through 017 discharge to the Outer Harbor, and CSOs 018, 020 through 028, 030 through 037, 040 and 041 discharge to the Inner Harbor. CSOs 001 and 002 are the outfalls for New Bedford's existing primary wastewater treatment plant.

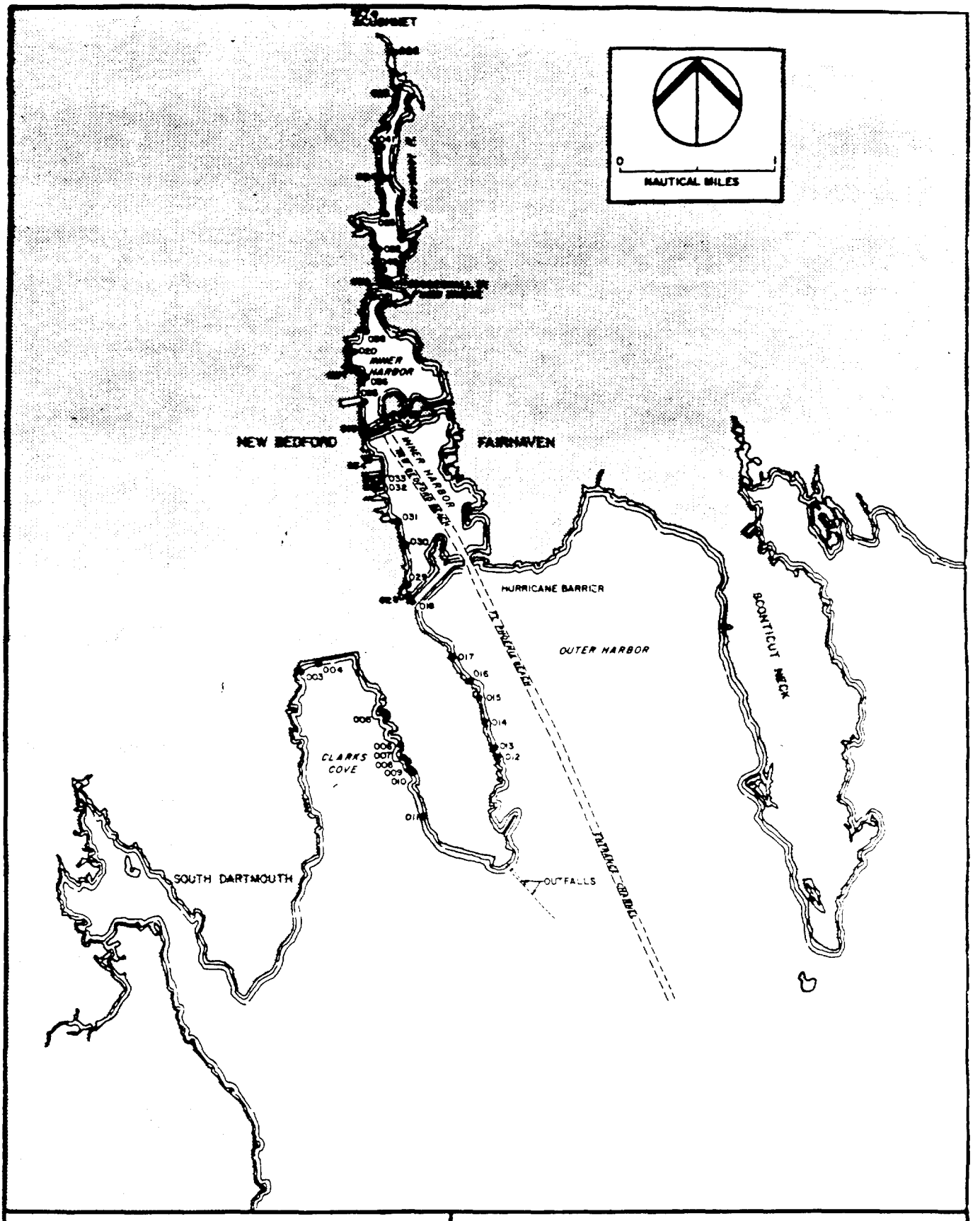
CDM separated the CSOs into six groups for the purpose of sizing abatement facilities. The groupings are based on the reach of shoreline where the CSOs discharge and their proximity to the main sewage interceptor. The CSO groupings are shown in Figure 8-3.

The siting criteria for CSO abatement facilities were selected based on the initial screening of alternatives in Section 7.5. The recommendations made in that section include:



City Of New Bedford
COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

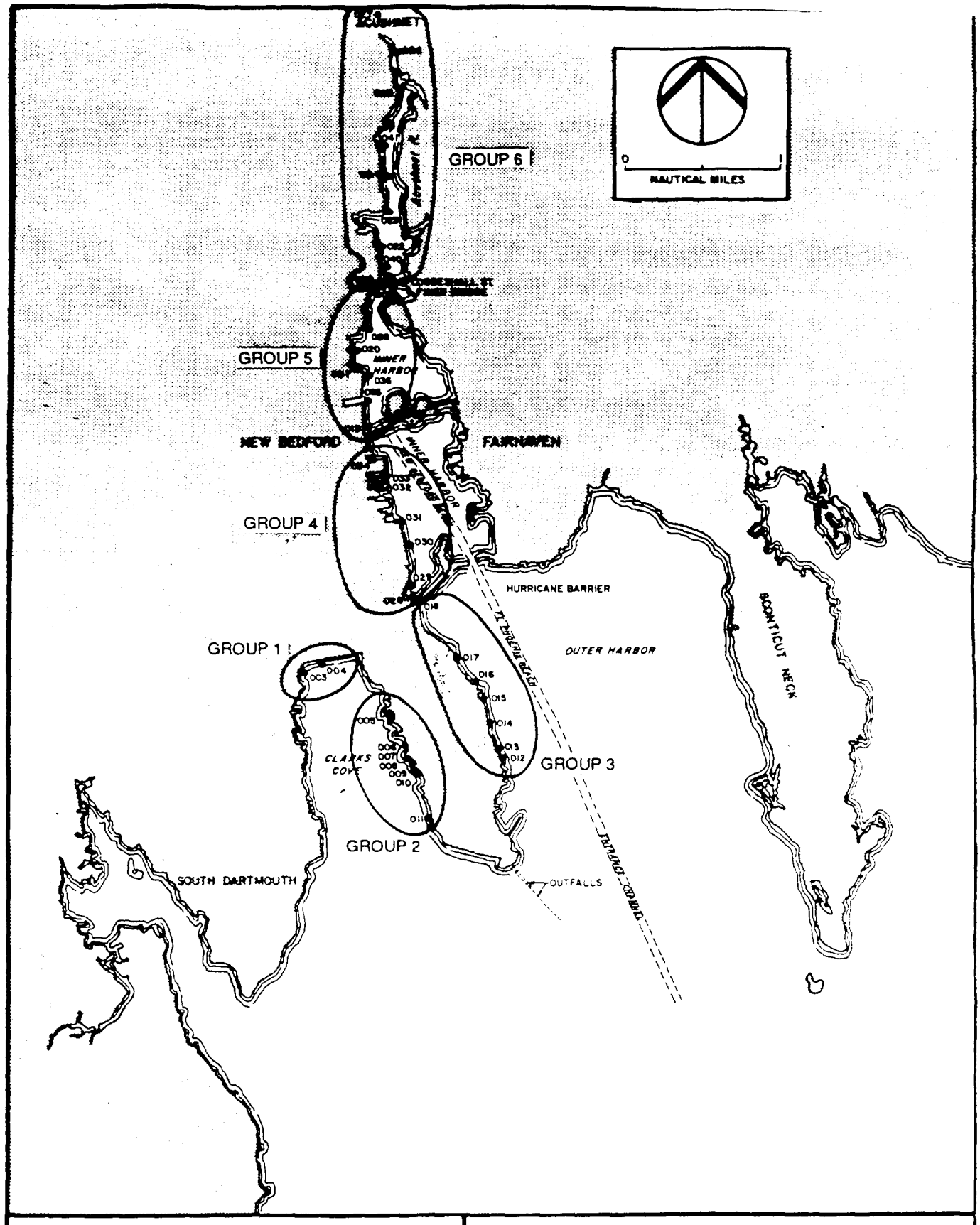
FIGURE 8-1
SITE SCREEENING METHODOLOGY FLOW CHART



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 8-2
COMBINED SEWER OVERFLOW (CSO) LOCATIONS



City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN – PHASE 3

FIGURE 8-3
CSO GROUPINGS

- Group 1 - 6-month as a minimum level of storage and pump back to the interceptor or a lesser level of storage and the re-direction of CSO Group 1 overflow to the Inner Harbor.

- Group 2 - 6-month level of storage and pump back to the interceptor.

- Group 3 - 3-month level of storage and pump back to the interceptor.

- Group 4 - 3-month level of storage and pump back to the interceptor.

- Group 5 - Screening and disinfection as a minimum, sedimentation should be considered for reduction of toxic substances, with discharge to the Inner Harbor.

- Group 6 - Screening and disinfection as a minimum, sedimentation should be considered for reduction of toxic substances, with discharge to the Inner Harbor.

The most land-intensive CSO abatement alternative is a storage facility. The area required for such a facility is dependent upon the tributary area of the CSO, the design storm under consideration, and the effective depth of the storage facility. A 12-foot sidewater depth was assumed for determination of the surface area required for storage for various design storms. However, initial sizing of CSO Group 1 and the consolidated option storage and pump back facilities indicated that a greater sidewater depth is warranted. A 18-foot sidewater depth is used for the storage facilities for Group 1 and the consolidated option (Group 1 and 4).

A screening facility can be constructed on a site with an area between 0.5 to 1.0 acres, depending on the design storm. This includes a buffer around the facility as well as a small parking area. The facility would consist of a single story structure approximately 60-foot by 60-foot containing the coarse and fine mechanical bar screens.

Satellite sedimentation facilities with screening and disinfection were sized with 12-foot sidewater depth.

Although storage/pump back and sedimentation/disinfection facilities may require a large amount of area, the majority of the facility would be located underground and the site could be returned to its previous land use

subsequent to construction. Primarily, the impacts to the site would be due to construction. A single building, approximately 60-feet by 60-feet, containing the headworks and pumps would be required.

The required area for storage/pump back and screening/disinfection were determined from analysis of the results of the SWMM and STORM models as described in Section 4.0. Storage volumes were selected to contain the predicted CSO discharge volume, and sedimentation tank volumes were selected based peak loadings. A buffer distance of 100 feet was included in the minimum area.

The siting needs and minimum required area for each CSO Group abatement facility are presented in Table 8-1.

8.3 IDENTIFICATION OF INITIAL CANDIDATE SITES

The initial candidate sites were selected for evaluation based on the ability to meet one or more of the following criteria:

- Sites should be in the vicinity of the CSO group, preferably near the center of the grouping
- Sites should be vacant or without structures
- Sites should be publicly-owned
- Sites should have sufficient acreage to accommodate the selected abatement technology with a 100-foot buffer zone around the perimeter

Using aerial photographs, assessors maps of the City of New Bedford, field visits, and the above criteria, 31 initial sites were selected for the various proposed CSO abatement facilities. Figure 8-4 shows the location of the 31 initial candidate sites with respect to the 6 CSO groupings. Table 8-2 presents a brief description of the 31 sites including ownership, land area, zoning, and current land use.

TABLE 8-1

SITING NEEDS FOR ALTERNATIVE CSO ABATEMENT TECHNOLOGIES

CSO Group	Technology	3-Month Minimum Required Area (Acres)[1]
Group 1 (003-004)	Storage Facility	8
Group 2 (005-011)	Storage Facility	1
Group 3 (012-018)	Storage Facility	2
Group 4 (028, 030-034)	Storage Facility	5
Group 5 (020, 021, 035-037)	Treatment Facility	2
Group 6 (022-027, 040, 041)	Treatment Facility	3
Group 1 & Group 4 (Consolidated Option)	Storage Facility	13

(1) Area requirements based on control of 3-month design storm and 12-foot sidewater depth for Groups 2 through 6, Group 1 and the consolidated option (Groups 1 and 4) has an 18-foot sidewater depth. Note that less area would be required if a deeper tank were constructed for Groups 2 through 6.

TABLE 8-2

DESCRIPTION OF ALTERNATIVE SITES FOR CSO ABATEMENT FACILITIES

SITE NO.	CSO GROUP	LOCATION	OWNERSHIP	LAND AREA (ACRES)	ZONING	CURRENT LAND USE
1	1	City-owned land between Cove Road and Hurricane Barrier	Public	15.00	Residence "A"	Athletic Field Flood Storage
2	2	Hazelwood Park at West Rodney French Boulevard and Lucas Street	Public	23.05	Residence "A"	Park Recreation
3	2	Brock Avenue and Coral Street	Public	8.71	Residence "A"	Baseball Fields Park
4	2	Fort Rodman property south of West Rodney French Boulevard and South Rodney French Boulevard	Public	22.39	Residence "A"	Open Space Athletic Field Parking
5	2	Brock Avenue and Hudson Street	Private	1.61	Residence "A" Mixed-Use Business	Open Space Parking Area
6	3	East Rodney French Boulevard and Apponagansett Street	Private	9.52	Residence "B"	Open Space Parking Lot
7	3	Brock Avenue and Freedom Boulevard	Public	17.84	Residence "A"	Athletic Field Open Space
8	3	North of Hurricane Barrier on East Rodney French Boulevard and north of Frederick Street	Private	3.67	Residence "B"	Open Space
9	3	Roosevelt Junior High School property at Frederick Street and Cleveland Street	Public	2.30	Residence "B"	Open Space Athletic Field

TABLE 8-2, continued

DESCRIPTION OF ALTERNATIVE SITES FOR CSO ABATEMENT FACILITIES

SITE NO.	CSO GROUP	LOCATION	OWNERSHIP	LAND AREA (ACRES)	ZONING	CURRENT LAND USE
10	3	Cleveland Street and David Street	Private	4.10	Industrial "B"	Open Space Recreation
11	4	Gifford Street and Front Street	Private	18.43	Industrial "B"	Open Space Recreation Parking
12	4	MacArthur Drive and Potomska Street, east of Front Street	Private	10.46	Industrial "B"	Open Space Radio Facility
13	4	Conway Street and Hassey Street	Private	6.45	Waterfront Industrial and Working Waterfront Overlay	Open Space Parking Storage Facility
14	4	MacArthur Drive east of Walnut Street at Leonard's Wharf	Private	5.33	Waterfront Industrial and Working Waterfront Overlay	Parking Fuel Storage
15	4	MacArthur Drive at Union Street near State Pier	Private	0.67	Waterfront Industrial and Working Waterfront Overlay	Open Space
16	5	Herman Melville Boulevard at Antonio L. Costa Avenue	Private	4.76	Waterfront Industrial and Working Waterfront Overlay	Parking Open Space Supply Storage
17	5	Herman Melville Boulevard and Hervey Tichon Avenue	Public (leased)	3.16	Waterfront Industrial and Working Waterfront Overlay	Open Space Parking Supply Storage

TABLE 8-2, continued

DESCRIPTIONS OF ALTERNATIVE SITES FOR CSO ABATEMENT FACILITIES

SITE NO.	CSO GROUP	LOCATION	OWNERSHIP	LAND AREA (ACRES)	ZONING	CURRENT LAND USE
18	5	Clasky Park at Purchase Street and Pearl Street	Public	7.20	Residence "A"	Park Recreation
19	5	Wamsutta Street and Herman Melville Boulevard	Public (leased)	16.38	Waterfront Industrial and Working Waterfront Overlay	Open Space Boat Storage
20	5	North Front Street and Wamsutta Street	Private	4.10	Industrial "B" and Working Waterfront Overlay	Parking Warehouse Facility
21	5	Belleville Avenue and Washburn Street	Public	7.22	None apparent	Open Space Gas Station Pump House Parking
22	6	Coggeshall Street and Mitchell Street	Private	5.20	Mixed-Use Business	Parking
23	6	Sawyer Street near Mitchell Street	Private	3.54	Industrial "B"	Open Space
24	6	East end of Sawyer Street at Acushnet River	Private	7.76	Industrial "B"	Recreation Parking Open Space

TABLE 8-2, continued

DESCRIPTIONS OF ALTERNATIVE SITES FOR CSO ABATEMENT FACILITIES

SITE NO.	CSO GROUP	LOCATION	OWNERSHIP	LAND AREA (ACRES)	ZONING	CURRENT LAND USE
25	6	Riverside Playground at Coffin Avenue and Belleville Avenue	Public	3.67	Industrial "B"	Open Space Recreation
26	6	Riverside Avenue between Belleville Avenue and Hathaway Street	Private	4.84	Industrial "B"	Parking Open Space
27	6	Brooklawn Park at Brooklawn Street and Ashley Boulevard	Public	76.50	Residence "A"	Recreation Open Space
28	6	River Road and Howard Avenue	Private	1.56	Industrial "B"	Parking Open Space
29	1	Cove Road and Orchard Street	Private	2.96	Industrial "B"	Commercial Business
30	5	Herman Melville Boulevard at Wamsutta Street	Public/Private	12.55	Working Waterfront Overlay and Industrial "B"	Railroad yard
31	4	Gomes School at Potomska Street and South Second Street	Public	3.61	Industrial "A" Mixed-Use Business	Open Space Recreation

8.4 SITE SCREENING CRITERIA

Following the development of the initial candidate site list, more specific screening criteria were developed to eliminate inappropriate sites from the list and to select preferred sites.

The surface and subsurface conditions vary widely between locations. However, coastal regions tend to have soils that are more uniform in formation and therefore, generalizations can be made. Thus surface and subsurface conditions were not used as an initial screening criterion. Detailed subsurface investigation should be performed during the detailed site evaluation process.

Each site was rated as most suitable, suitable, or least suitable as defined below for the following criteria.

8.4.1 LOCATION

The site should preferably be located near the geographic center of a CSO outfall grouping and be accessible to the main sewage interceptor so as to minimize the required piping to the abatement facility. For this screening, a site is considered most suitable if it is within 1000-ft of the center of a CSO grouping. This distance will allow the overflows to flow by gravity to the abatement facility. A site is considered suitable if it is between 1000-feet and 2000-feet. This distance may allow the overflows to flow by gravity or require a pump station and force main. A site is considered least suitable if it is located more than 2000-feet from the center of a CSO grouping. This distance will require a pump station and force main to convey the overflows to the abatement facility.

8.4.2 OWNERSHIP

The ownership of a site is important to the implementation of a recommended plan, since a legal landtaking can be time consuming and increase the total project cost significantly. Also multiple ownership of a site is considered an obstacle to implementation. A site was considered most

suitable if it is owned by the City of New Bedford, suitable if the site is owned by another local agency or by the Commonwealth of Massachusetts, and least suitable if it is privately owned. The City administration has expressed a preference for not using privately owned land for public purposes.

8.4.3 AREA

The site should be large enough to accommodate the proposed abatement facility including a 100-ft buffer zone. The area required for selected CSO abatement facilities was presented previously. Deeper storage tanks can be constructed on smaller sites to accommodate the design storage volume, however, construction cost and construction difficulty increase with increasing depth. The site should also have sufficient area for construction staging. The larger the site, the greater the opportunity to buffer the site with natural or landscaped screening to mitigate potential odors, noise, and visual impacts during construction and operation of the facility. The most suitable site has ample area for construction staging, the facility and a buffer. A suitable site has sufficient area for the facility but little or no room for construction staging or a buffer. The least suitable site would require atypical design and construction methods to fit the facility on the site.

8.4.4 ELEVATION

Each site was evaluated based on the average elevation relative to National Geodetic Vertical Datum of 1929. Site elevation is considered most suitable if the average site elevation is less than 20 feet. The CSO overflows can flow by gravity to a facility at a site elevation less than 20 feet. A site is considered suitable if the average elevation is between 20 feet and 40 feet. A facility at this elevation will require a pump station to convey the overflows to it. A site is least suitable if the site is above 40 feet. At this elevation, the overflows will require a larger and more costly pump station.

8.4.5 ZONING

The City of New Bedford Comprehensive Zoning regulations do not specifically consider the zoning requirements for CSO or wastewater treatment facilities. However, the zoning regulations do allow the siting of "municipal facilities" as necessary in any area. Zoning was used as the indicator of the preferred type of development in an area. For this study, a site is considered most suitable if it is zoned for industrial activity. A site is considered suitable when it is in a retail, commercial, or office area or in a non-residential zone. Least suitable sites are in residential areas. A detailed description of each relevant zoning classification is presented in Section 8.5.

8.4.6 SITE ACCESS AND TRAFFIC

Access to the site should be adequate for the passage of construction vehicles and operation and maintenance equipment without significantly disrupting residential or recreational areas. Sites which have access via existing roadways are preferred to those requiring a new right-of-way and road construction. Special consideration was given to wastewater disinfection facilities since the chemicals required for the disinfection must be delivered to the site.

Current traffic patterns were considered when determining potential traffic congestion due to construction. A site is considered most suitable if access is easily attainable without creating traffic delays or interfering with public safety. A site is suitable if there is a potential for traffic delays due to construction; and a site is considered least suitable if construction and/or operation would cause adverse impacts on any of the main streets in the City.

8.4.7 CURRENT LAND USE

The existing land use, proposed land use, and the impact of the proposed abatement facility on future land use were evaluated. A vacant site or site containing a parking lot is considered the most suitable, a site with

at least a 50 percent vacancy is considered suitable, and sites with facilities occupying more than 50 percent of the total area or a designated open space or parkland with athletic facilities is considered least suitable.

8.4.8 SURROUNDING LAND USE

The surrounding land use should be compatible to the proposed facility to minimize the impact of the facility on the community. Vacant sites located in industrial areas adjacent to predominantly industrially-zoned and vacant areas were considered most suitable. Industrial, commercial, or business-owned areas with some vacant land or residentially-zoned vacant property is considered suitable. The least suitable sites are surrounded by developed residentially-zoned areas or fully utilized business property. Consideration of future development plans and property values were also evaluated.

The above criteria and ranking methodology are presented in Table 8-3.

8.5 CITY OF NEW BEDFORD COMPREHENSIVE ZONING

City of New Bedford Comprehensive Zoning Regulations define ten zoning districts and provide for "Special Flood Hazard Areas." Following are excerpts from the zoning regulations that describe zoning districts relevant to the site evaluations.

8.5.1 RESIDENCE "A", "B" AND "C" DISTRICTS

Within a Residence "A", "B" or "C" district, no premises shall be used and no building or structure shall be erected which is intended or designed to be used, in whole or in part for industry, trade, manufacture or commerce.

Public service buildings are allowed in these districts if constructed to conform to and harmonize with the buildings in the district, provided they are not used for manufacture, trade, "storage" or garage.

TABLE 8-3

ABATEMENT FACILITY

SITING CRITERIA AND EVALUATION METHODOLOGY

Criteria	Most Suitable (+)	Suitable (0)	Least Suitable (-)
Location	Located within 1,000 ft from the center of a CSO group	Located between 1,000 ft and 2,000 ft from a CSO group	Located more than 2,000 ft from a CSO group
Ownership	Owned by the City of New Bedford	Owned by other local or state agency	Privately owned
Area	Sufficient area for conventional facility with sufficient buffer and staging area	Sufficient area for facility, limited staging area and buffer zone	Atypical design and construction required, no staging area or buffer zone
Elevation	Average elevation is less than 20 feet (NGVD)	Average elevation is between 20 and 40 feet	Average elevation is greater than 40 feet
Zoning	Industrially-zoned district	Business district	Residentially-zoned district
Access/Traffic	Easily accessible, no traffic delays anticipated	Potential for traffic delays	Adverse impacts on traffic flow and/or public safety
Current Land Use	Vacant land	Vacant land with some structures some structures	Structures over more than 50% of the land or designated open space with recreational facilities
Surrounding Land Use	Vacant land, industrially-zoned	Areas of light industry or commercial areas with some vacant land	Predominantly residential areas near sensitive receptors

8.5.2 BUSINESS DISTRICTS

Within the city, there shall be two business districts: one shall encourage mixed residential/business and commercial development, and one shall allow business/commercial development only. They shall be known as the Mixed-Use Business district and the Planned Business district, respectively.

Within the Mixed-Use Business district, no building or premise shall be used and no structure shall be erected which is intended or designed to be used, in whole or in part, for other than one or more of the following specified purposes:

(1) Any use permitted in Residence "A", "B" or "C" districts

(2) Stores, markets and restaurants

(3) Stables

(4) Filling stations and public parking lots

(5) Garages

(6) Any manufacturing or industrial

(7) Billboards

Within the Planned Business district, no residential use shall be permitted and no building or premise shall be used, and no building or structure shall be erected which is intended or designed to be used, in whole or in part, for other than one or more of the following specified purposes:

(1) Stores, markets or restaurants

(2) Filling stations or public parking lots

(3) Garages

(4) Billboards

8.5.3 INDUSTRIAL "A" DISTRICTS

Within any Industrial "A" district, no building or premise shall be used and no building or structure shall be erected which is intended or designed to be used, in whole or in part, for residential purposes or for other than the following specified purposes:

(1) Stores, markets or restaurants

(2) Filling stations or public parking lots

(3) Wholesale establishments, storage or warehousing facilities

(4) Light manufacturing

(5) Billboards

8.5.4 INDUSTRIAL "B" DISTRICTS

Within the Industrial "B" district, any use otherwise lawful shall be permitted except a building or structure intended or designed to be used, in whole or in part, for residential purposes or for use as a fish fillet plant or fish processing plant. However, a fish processing plant shall be allowed in Industrial "B" zoned land within the Working Waterfront Overlay district, defined later in this section.

8.5.5 INDUSTRIAL "C" DISTRICTS

Within any Industrial "C" district, no building or premise shall be used and no building or structure shall be erected which is intended or designed to be used, in whole or in part, for other than the following purposes:

(1) Light and heavy manufacturing

(2) Storage and warehousing in conjunction with manufacturing

(3) Billboards

8.5.6 WATERFRONT INDUSTRIAL DISTRICT

Within any Waterfront Industrial district, no building or premises shall be used and no building or structure shall be erected which is intended or designed to be used, in whole or in part, for other than the following specified purposes:

(1) Transformer station, pumping station, gas regulation station and other public utility uses

Twelve other Waterfront Industrial categories were cited in the Comprehensive Zoning Regulations, however, only the aforementioned category pertains to this analysis.

8.5.7 SPECIAL FLOOD HAZARD AREAS

The special flood hazard areas are established as an overlay district and include all special hazard areas designated as zones A, A1, A9, A11, V9 and V14 on the New Bedford Flood Insurance Rate Maps (FIRM) as amended, effective January 5, 1984.

The purpose of this division is to provide for adequate minimum standards and procedures for the construction of new residential and non-residential structures and existing structures that are substantially improved, which are located within special flood hazard areas, so that the construction or improvement of such structures will be eligible for insurance under the national flood insurance program by conforming to recognized construction techniques designed to offer flood protection and minimize flood losses.

Utilities in Special Flood Hazard Areas Shall Receive Special Consideration, as follows:

(1) All vents of sewage systems shall be constructed of solid pipe, terminating at least one foot above the base-flood elevation.

(2) Manhole covers on individual or public sewer systems below the base-flood elevation, shall have watertight covers to the manhole frames.

(3) Sewer service piping shall be waterproofed, from the main sewage disposal system to either above the base-flood elevation, or the interior of a waterproofed structure.

(4) All construction materials for sewer service or disposal systems shall be water resistant.

(5) Any underground tank for the storage of fuels or other liquids, located below the base-flood elevation, shall be designed and weighted to prevent flotation when empty.

No materials or chemicals which would create hazard to life by flotation, release or contact, by solution or chemical reaction through contact with water, creating harmful effects, fire or explosion, shall be stored or used in manufacturing in any special flood hazard area.

8.6 DETAILED SITE ASSESSMENT

Thirty-one sites were screened for the siting of abatement facilities. All of the potential sites are located within the corporate limits of the City of New Bedford.

Each of the 31 alternative sites were evaluated based on the siting criteria presented in Section 8.4. The procedure applied gave site a "+" rating if under the criteria the site was most suitable, a "0" if the site is suitable and a "-" if the site is least suitable. The results of the evaluation were summed for each site. This allowed sites to be compared numerically.

Legal and permitting considerations for the sites that remain after the initial site screening are discussed in the Environmental Impact Document (EID). The extent of the permitting required must be determined for each remaining site with regard to floodplains, tidelands and wetlands as well as any required building permits.

The evaluation of each initial candidate site is discussed below.

SITE 1 (CSO GROUP 1 and Consolidated Option)

Location

The site is located just north of Clarks Cove. A cresent-shaped parcel referred to as Francis playground, the site is bounded by Cove Road to the north, east, and west and by the hurricane barrier to the south. The center of the site is located within 500 feet of CSO 004 which is the focal concern of CSO Group 1.

Site 1 is also being evaluated for the possible location of consolidated storage facility for CSO Groups 1 and 4. The site is about 3000 feet from the optimal location for a consolidated storage facility.

Ownership

The site is owned and maintained by the City of New Bedford.

Area

The area of this cresent-shaped parcel is not available on the City's assessor's map, however, CDM has estimated the area to be approximately 15 acres. However, due to the cresent shape of the parcel, the usable area may be only 50% of the total area. 50 percent of 15 → 7.5 < 8 not suitable?

Elevation

Based on the United States Geological Survey (USGS) topographic map, the elevation over the entire area of the site is less than 10 feet National Geodetic Vertical Datum (NGVD).

Zoning

The site is zoned as a Residence "A" district.

Site Access and Traffic

The site is accessible from Cove Road at any point along the north, east, or west bounds. Cove Road is a heavily trafficked road that can readily accommodate construction equipment. However, traffic from the site could cause some delays.

Current Land Use

The land is grassy open space with one pump station located on the eastern end of the site and another adjacent to the hurricane barrier on the west side of the site. The parcel is also used as a flood storage area and has a basketball court and soccer field.

Surrounding Land Use

The zoning is mixed north of Cove Road and consists of Industrial "B" between Orchard Street and Bonney Street, Industrial "A" between Bonney

Street and Crapo Street, Mixed-Use Business between Crapo Street and County Street, and a combination of Mixed-Use Business and Residence "C" northeast of the parcel.

SITE 2 (CSO GROUP 2)

Location

The site is Hazelwood Park on the west side of Clarks Point. Hazelwood Park is bounded to the north by homes on Valentine Street, to the south by homes on Lucas Street and by West Rodney French Boulevard to the west. Brock Avenue bounds the site to the east. The site is approximately 1,000 feet from the center of CSO Group 2.

Ownership

The site is owned by the City of New Bedford and maintained by the New Bedford Park Department.

Area

According to the City assessor's map, the site is approximately square and has an area of 23.05 acres.

Elevation

Based on the USGS topographic map, elevations at Hazelwood Park range from 10 feet to 45 feet. The average site elevation is estimated to be 25 feet.

Zoning

The site is zoned as a Residence "A" district.

Site Access and Traffic

The site is accessible from either West Rodney French Boulevard to the west or Brock Avenue to the east. Both access routes are moderately traveled and can accommodate construction equipment.

Current Land Use

Hazelwood Park is a public park. There are eight tennis courts, two basketball courts and four buildings constructed on the property. Two access roads enter the park from West Rodney French Boulevard and one of the roads extends through the park to the Brock Avenue at the east end.

Surrounding Land Use

Homes surround Hazelwood Park to the north, south and east. Beaches along the Clarks Cove shoreline abut West Rodney French Boulevard. All the surrounding area is zoned as a Residence "B" district.

SITE 3 (CSO GROUP 2)

Location

The site is referred to as Victory Park and located near the east-west center of Clarks Point. Homes on Bellevue Street bound the site to the north, homes on Nautilus Street bound the property to the west, homes along Abalone Street bound the property to the south, and Brock Avenue bounds the site to the east. The site is divided into north and south parcels by Coral Street. The site is about 1,200 feet from the center of the CSO Group 2.

Ownership

The site is owned and maintained by the City of New Bedford.

Area

The site is composed of two parcels of land. According to the New Bedford assessor's map, the parcel north of Coral Street is 4.82 acres and the parcel south of Coral Street is 3.89 acres. Therefore, the total site area is approximately 8.71 acres.

Elevation

Based on the USGS topographic map, the site elevation ranges from 30 feet to 35 feet. The average site elevation is about 32 feet.

Zoning

The site is zoned as a Residence "A" district.

Site Access and Traffic

The site is accessible from Brock Avenue which abuts the east side. Also, Coral Street, which divides the site into two parcels, is an access route to each parcel. Brock Avenue is a road that experiences moderate traffic volume and can accommodate construction equipment.

Current Land Use

The land is used for athletic fields. There are two baseball fields on the south parcel and one on the north parcel.

Surrounding Land Use

The land to the south, east and west of Victory Park is zoned as a Residence "A" district. North of the site is zoned Residence "B". The north, south, and west sides of the site are residential areas, while east of Brock Avenue is a facility owned by the City of New Bedford (Site 7).

SITE 4 (CSO GROUP 2)

Location

The site is located on the southern region of Clarks Point on the west portion of Fort Rodman. The site is bounded by South Rodney French Boulevard to the north, Clarks Cove to the west, Buzzards Bay to the south and the Camp Kennedy military complex to the east. The site is about 3,000 feet from the center of CSO Group 2.

Ownership

The site is owned by the City of New Bedford.

Area

According to the City assessor's map, the site consists of one large parcel with a land area of 22.39 acres.

Elevation

Based on the USGS topographic map, site elevations range from 5 to 20 feet. The average site elevation is approximately 15 feet.

Zoning

The site is zoned as a Residence "A" district.

Site Access and Traffic

The Fort Rodman site is accessible from South Rodney French Boulevard. The road has light to moderate traffic volume and can accommodate construction equipment.

Current Land Use

The land is vacant and composed of about a 60% grass and tree area on the central and western portions. A soccer field and a parking area are located on the east-southeast regions of the site.

Surrounding Land Use

The area to the east and north of the site is zoned Residence "A". Homes are located north of the site opposite South Rodney French Boulevard. East of the site are barrack buildings used for educational programs, the U.S. Army base, and the historic district, including Fort Tabor. To the west is Battery Milliken, a World War II gun emplacement.

SITE 5 (CSO GROUP 2)

Location

The site is located on the southwest part of Clarks Point, just north of Fort Rodman. The site is bound by Hudson Street to the north, Brock Avenue to the east, homes on Point Street to the west and homes on South Rodney French Boulevard to the south. The site is approximately 2,200 feet from the center of CSO Group 2.

Ownership

The site consists of three parcels of land owned by two people.

Area

The site consists of three parcels totaling 1.61 acres according to the City assessor's map.

Elevation

Based on the USGS topographic map, the site elevations range from 25 feet to 30 feet. The average site elevation is approximately 27 feet.

Zoning

The site is zoned about 50% Residence "A" and 50% Mixed-Use Business.

Site Access and Traffic

The site is accessible from Brock Avenue. The area experiences light to moderate traffic volume and can accommodate construction equipment.

Current Land Use

The land is vacant, about 50% is grass and trees and about 50% is a dirt parking area.

Surrounding Land Use

The surrounding area (with the exception of a small area to the north of the site and west of Brock Avenue which is zoned Mixed-Use Business) is zoned as a Residence "A" district. The area in the vicinity of the site is residential.

SITE 6 (CSO GROUP 3)

Location

The site is located on the east side of Clarks Point. Apponagansett Street bounds the site to the north, a residential area bounds the south, homes on Fern Street bound the west, and East Rodney French Boulevard bounds the site to the east. The site is about 1,100 feet from the center of CSO Group 3.

Ownership

The site is privately owned and is comprised of three parcels of land. All three parcels are owned by one person.

Area

According to the City of New Bedford assessor's map, three parcels of area 1.58, 4.57, and 3.37 acres combine to form a total site area of 9.52 acres. The site is rectangular in shape with the long dimension in the east-west direction.

Elevation

Based on the USGS topographic map, elevations at the site range from 10 to 30 feet. The average site elevation is approximately 20 feet.

Zoning

The site is zoned Residence "B".

Site Access and Traffic

The site is accessible from East Rodney French Boulevard. Traffic is moderate in this area and would not be severely impacted by the presence of construction equipment.

Current Land Use

Of the three parcels that comprise the site, only one parcel, an east-west rectangle portion in the northeast corner is being utilized. This parcel is a paved parking area with some open space to the west and south of the parking lot. The remainder of the site is open space covered with trees.

Surrounding Land Use

The land surrounding the site to the south and west is residential and east of the site is New Bedford's Outer Harbor. North of the site, opposite Apponagansett Street, is an area zoned Industrial "B" from East Rodney French Boulevard west to Swan Street and Residence "B" from Swan Street west to Fern Street.

SITE 7 (CSO GROUP 3)

Location

The site is located near the center of Clarks Point. The site is bound to the north by Rickertson Street, to the south by Portland Street, to the east by Lighthouse Lane, and to the west by Brock Avenue. The site is divided into north and south parcels by Freedom Boulevard. The site is located about 2,900 feet from the center of CSO Group 3.

Ownership

The site is owned and maintained by the City of New Bedford.

Area

According to the City assessor's map, the parcel north of Freedom Boulevard is 12.09 acres and the parcel south of Freedom Boulevard is 5.75 acres. Therefore, the total site area is about 17.84 acres.

Elevation

Based on the USGS topographic map, the average site elevation is about 35 feet.

Zoning

The zoning is Residence "A" for both parcels.

Site Access and Traffic

The site is accessible from Brock Avenue which extends the length of Clarks Point. Brock Avenue is a moderately traveled road that can accommodate construction equipment. Traffic patterns would not be affected.

Current Land Use

There is a water storage facility on the southwest quadrant of the parcel north of Freedom Boulevard. The south parcel has a soccer field on the west half that extends from Freedom Boulevard to Portland Street.

Surrounding Land Use

The area to the south, east, and west of the site is zoned Residence "A", and the area to the north is zoned Residence "B". West of the site, opposite Brock Avenue, is a recreational area with 3 baseball fields (Site 3). East of the south parcel is the City of New Bedford Public Health Facility. North and south of the site are residential areas.

SITE 8 (CSO GROUP 3)

Location

The site is located on the east side of Clarks Point and bound by East Rodney French Boulevard to the east and Cleveland Street to the west. The site is bound by homes on Rodney Street to the north and by the hurricane barrier to the south. The site is about 1,500 feet from the center of CSO Group 3.

Ownership

The site consists of two privately owned parcels. Both parcels are owned by one person.

Area

The site is rectangular in shape with the long dimension in the east-west direction. The City assessor's map does not indicate an area for this land, however, CDM estimates it to be about 3.67 acres.

Elevation

Based on the USGS topographic map, site elevations range from 8 to 20 feet. The average site elevation is about 15 feet.

Zoning

The site is zoned Residence "B".

Site Access and Traffic

The site is accessible from East Rodney French Boulevard which is a moderately trafficed road. East Rodney French Boulevard can accommodate construction equipment without any disruption of traffic.

Current Land Use

A fenced in parking area is located on the east end of the site. The remainder of the site is undeveloped and covered with trees.

Surrounding Land Use

The area in the vicinity of the site is zoned Residence "B". Residential areas are to the north and south, a Junior High School (Site 9) is opposite the site on Cleveland Street.

SITE 9 (CSO GROUP 3)

Location

The site is located near the east-west center of Clarks Point. The site is bound by Cleveland Street to the east, Frederick Street to the south, a Junior High School to the west, and multi-unit housing to the north. The site is located about 1,800 feet from the center of CSO Group 3.

Ownership

The site is Roosevelt Junior High School which is owned by the City of New Bedford.

Area

The site is part of Roosevelt Junior High School property. The area of the Junior High School property is 5.05 acres, however, due to onsite structures, the available area is approximately 2.30 acres which includes an area currently occupied by a trailer-like addition to the original structure.

Elevation

Based on the USGS topographic map, the average site elevation is about 20 feet.

Zoning

The site is in a district zoned Residence "B".

Site Access and Traffic

The site is in a neighborhood that has many multi-unit structures requiring occupants to park on the street. The best access is from Frederick Street off East Rodney French Boulevard. Construction equipment would probably disrupt normal traffic patterns in the area.

Current Land Use

The site has a structure which is an addition to the original construction of the Junior High School. The structure is located at the western most edge of the site. There are also two playing fields, one each at the northeast and southeast corners of the site.

Surrounding Land Use

The region in the vicinity of the school is zoned Residence "B". Residential areas lay to the north and southeast, open space (Site 8) is to the east and public school facilities are west and southwest of the site.

SITE 10 (CSO GROUP 3)

Location

The site is located on the northern portion of Clarks Point. The site is bound by David Street to the north, Mott Street to the south, and Cleveland Street to the west. An industrial facility abuts the site to the east. The center of the site is about 2,500 feet from the center of CSO Group 3.

Ownership

The site is privately owned by one person.

Area

The site consists of one large parcel and five 0.1-acre parcels. The large parcel is a fraction of a large lot that extends from East Rodney French Boulevard west to Cleveland Street. The area of the site was estimated from aerial photographs to be about 4.06 acres.

Elevation

Based on the USGS topographic map, site elevations range from 12 to 20 feet. The average site elevation is about 18 feet.

Zoning

The site is zoned Industrial "B".

Site Access and Traffic

The site is accessible from David Street to the north, Mott Street to the south and Cleveland Street to the west. David Street is best for access because Mott Street and Cleveland Street border residential neighborhoods. Traffic is light, therefore, access from David Street should accommodate construction equipment with no expected disruption to traffic.

Current Land Use

The land is used for open space and recreation. Facilities include a baseball field and playground. The site has considerable open space.

Surrounding Land Use

The land use in the vicinity of the site is mixed. East and north of the site are industrial complexes. South and west are residential areas.

SITE 11 (CSO GROUP 4 AND CONSOLIDATED OPTION)

Location

The site is referred to as the Standard Times site and is located at the southern-most region of the Inner Harbor near the shoreline adjacent to the hurricane barrier. The site is bound by open space to the north, Gifford Street to the south and the Inner Harbor to the east. The west side is bound by Front Street which has some industry located just off the street. The site is located about 2,700 feet from the center of CSO Group 4.

Site 11 is one of the two sites (see Site 1) under consideration for the location of a consolidated storage facility for CSO Groups 1 and 4. The site is about 500 feet from the optimal location for the consolidated facility.

Ownership

According to City of New Bedford assessor's mapping, the site consists of three parcels of land. All the parcels are privately owned.

Area

The total area of the three parcels is about 18.43 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned Industrial "B".

Site Access and Traffic

The site is accessed from Front Street to the west, Blackmer Street to the north and Gifford Street to the south. Area traffic volume is moderate, and all the roads can accommodate construction equipment.

Current Land Use

Site use includes paved and grass parking areas, industry, and open space. The south 200 feet of the site contains paved parking for the length of Gifford Street. Industry and parking combine to cover a 150-foot deep portion of the site for the length of Front Street. The north, central, and eastern portions of the property are playing fields and open space.

Surrounding Land Use

The area north and south of the site is zoned Industrial "B" and a district west of John F. Kennedy Memorial Highway is zoned Mixed-Use Business. North of the site are a radio station tower and fish processing facilities.

South of the site are commercial and industrial buildings including Stride Rite and Cape Cod Sports. West of Front Street are vacant land and commercial property.

SITE 12 (CSO GROUP 4)

Location

The site is located just north of the hurricane barrier at the intersection of Front Street and Potomska Street. The site is bound by businesses to the north and west, the Inner Harbor to the east, and open space to the south. The site is approximately 1,500 feet from the center of CSO Group 4.

Ownership

The site is privately owned.

Area

According to the City of New Bedford assessor's map, the site consists of one parcel with an area of about 10.46 acres.

Elevation

Based on the USGS topographic map, site elevations are less than 10 feet.

Zoning

The site is zoned Industrial "B".

Site Access and Traffic

According to the assessor's map, the site has no frontage on either Wright Street (about 225 feet to the north) or Front Street (about 200 feet west). However, a road has been installed from the intersection of Potomska Street and MacArthur Drive to the northwest of the site. The road has a moderate traffic load and can accommodate construction equipment.

Current Land Use

The land is almost entirely open space with a radio tower and associated building located on the west-central region of the site.

Surrounding Land Use

The area north of the site is zoned a Waterfront Industrial district and a Working Waterfront Overlay district. West and south of the site is zoned Industrial "B". Industrial complexes are located north and west of the site. The Acushnet River is to the east and open space to the south.

SITE 13 (CSO GROUP 4)

Location

The site is centered north to south between Route 6 and the hurricane barrier. The site is bound by industrial facilities to the north, Conway Street to the south, and Cape Street to the west. The Inner Harbor bounds the site to the east. The site is located at the center of CSO Group 4.

Ownership

The site is privately owned by a corporation.

Area

According to the City assessor's map, the area of the site is 6.45 acres.

Elevation

Based on the USGS topographic map, site elevation is less than 10 feet.

Zoning

The site is zoned a Working Waterfront Overlay district and a Waterfront Industrial district.

Site Access and Traffic

The site is accessed by traveling east on Conway Street off MacArthur Drive. The road is lightly traveled and can accommodate construction equipment with no disruptions to traffic.

Current Land Use

The site is apportioned approximately 25% open space, 25% buildings, and 50% parking space.

Surrounding Land Use

Surrounding land is zoned Working Waterfront Overlay and Waterfront Industrial to the north, east and west. The south area is zoned as Industrial "B" and Working Waterfront Overlay. Industrial facilities are located to the north, south, and west of the site.

SITE 14 (CSO GROUP 4)

Location

The site is located south of State Pier at the east end of Walnut Street. MacArthur Drive bounds the site to the north and west. The site is bound by industrial property to the south and the Inner Harbor to the east. The site is approximately 1,800 feet from the center of CSO Group 4.

Ownership

The site consists of four parcels of land all privately-owned by a corporation.

Area

The site consists of four parcels with a total area of about 5.33 acres. Structures are present that make some of the land unusable.

Elevation

Based on the USGS topographic map, the site elevation ranges from 5 to 10 feet.

Zoning

The site is zoned as a Working Waterfront Overlay district and a Waterfront Industrial district.

Site Access and Traffic

The site is accessible from MacArthur Drive to the north and west. MacArthur Drive has moderate to heavy traffic and can accommodate construction equipment.

Current Land Use

The land is occupied by a fuel storage tank to the east, a building to the west. Paved parking areas are dispersed over the site.

Surrounding Land Use

Land to the north and south of the site is zoned Working Waterfront Overlay and Waterfront Industrial. Land to the west, west of John F. Kennedy Memorial Highway, is zoned Residence "C". Parking lots and industry comprise the area land use north and south of the site.

SITE 15 (CSO GROUP 4)

Location

The site is a grassy median strip just south of State Pier and located between John F. Kennedy Memorial Highway and MacArthur Drive. The site is bounded by Union Street to the north and an industrial complex to the south. The site is about 2,600 feet from the center of CSO Group 4.

Ownership

The site consists of 2 parcels privately owned.

Area

According to the City assessor's map, the site has a total area of 0.67 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned a Waterfront Industrial district and a Working Waterfront Overlay district.

Site Access and Traffic

The site is accessible from MacArthur Drive to the east. The road is heavily traveled and construction could cause minor delays. MacArthur Drive can accommodate the construction equipment.

Current Land Use

The site is an undeveloped, grassy area.

Surrounding Land Use

The area west of John F. Kennedy Memorial Highway and the piers to the east are zoned for Mixed-Use Business. Areas north and south of the site are zoned Working Waterfront Overlay and Waterfront Industrial districts.

SITE 16 (CSO GROUP 5)

Location

The site is located north of New Bedford and Fairhaven Bridge (Route 6) at the Inner Harbor shoreline. Herman Melville Boulevard bounds the site to the west and industrial complexes bound the site to the east. The site is located about 1,250 feet from the center of CSO Group 5.

Ownership

The site is privately owned.

Area

According to the City assessor's map, the site consists of one parcel with an area of 4.76 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned as a Waterfront Industrial district and a Working Waterfront Overlay district.

Site Access and Traffic

The site is accessible from Herman Melville Boulevard to the west. The road is lightly traveled and can accommodate construction equipment. No traffic delays are expected during construction.

Current Land Use

The site is a large open lot. There are scattered parking areas on the site, apparently overflow parking from industry to the north and south.

Surrounding Land Use

The area to the north is a combination of industrial buildings (50%), paved parking (25%), and open space. South of the site is a large industrial facility that extends the length of the south side. West of the site, across Herman Melville Boulevard, is a railroad yard. East of the site is the Inner Harbor.

SITE 17 (CSO GROUP 5)

Location

The site is due east of Pearl Street and north of Hervey Tichon Avenue. Herman Melville Boulevard bounds the site to the west, Hervey Tichon Avenue to the south, and the Inner Harbor to the north and east. The site is located approximately 600 feet from the center of CSO Group 5.

Ownership

The site consists of two parcels owned by the City of New Bedford. Two industries lease the parcels from the City.

Area

According to the City assessor's map, the site consists of two parcels with a total land area of about 3.89 acres. After discounting unusable portions containing structures, the usable area is about 3.16 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned as a Waterfront Industrial district and a Working Waterfront Overlay district.

Site Access and Traffic

The site is accessible from Herman Melville Boulevard to the west and Hervey Tichon Avenue to the south. Both roads are lightly traveled and can accommodate construction equipment.

Current Land Use

The land is open space with some parking. Industry is present on the east side of the site.

Surrounding Land Use

The area north and south of the site is zoned as Working Waterfront Overlay and Waterfront Industrial. East of the site is the Inner Harbor and west of the site is the railroad yard zoned Working Waterfront Overlay and Industrial "B". Industry is currently established north and south of the site.

SITE 18 (CSO GROUP 5)

Location

The site is known as Clasky Park and is located north of Pearl Street and south of Pope Street between County Street to the west and Purchase Street to the east. The site is located approximately 2,100 feet from the center of CSO Group 5.

Ownership

The site is owned and maintained by the City of New Bedford.

Area

According to the City assessor's map, the site consists of one parcel having 2 sections because Pleasant Street passes through the site in the north-south direction. The area to the west of Pleasant Street is about 5.58 acres and the area to the east is about 1.62 acres. Therefore, the total site area is about 7.2 acres.

Elevation

Based on the USGS topographic map, the site elevation ranges from 40 to 100 feet. The average site elevation is approximately 70 feet.

Zoning

The site is zoned as a Residence "A" district.

Site Access and Traffic

The site is located in the middle of a thickly settled area. The site may be accessed from Pleasant and Purchase Streets to the east, Pope Street to the north, Pearl Street to the south, and County Street to the west. The area is thickly settled and construction equipment may cause delays in ordinarily moderate to severe traffic.

Current Land Use

The land is grassy, open space with some scattered trees.

Surrounding Land Use

The site is zoned Residence "A" and Residence "B" to the north, south, and west. Across Purchase Street to the east of the site is an area zoned Industrial "A". Buildings to the north, south, and east are single- and multi-family residential units. Industrial buildings are located across Purchase Street to the east.

SITE 19 (CSO GROUP 5)

Location

The site is located east of Herman Melville Boulevard where it intersects with Wamsutta Street. The site is bounded by the Inner Harbor to the north, east, and south. Herman Melville Boulevard bounds the site to the west. The site is located about 500 feet from the center of CSO Group 5.

Ownership

The site is owned by the City of New Bedford and leased to a shipping company.

Area

The site consists of one parcel of land with a total area of 16.38 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned as a Waterfront Industrial district and a Working Waterfront Overlay district.

Site Access and Traffic

The site is accessible from Herman Melville Boulevard. The area has a light to moderate traffic load and can accommodate construction equipment.

Current Land Use

The land is used for marine industry. The present land use is boat maintenance and storage. A dock extends into the river from the northwest point of the land. The site has considerable area that is undeveloped.

Surrounding Land Use

The area north and west of the site is zoned as a Working Waterfront Overlay district and Industrial "B" districts. South of the site is zoned as Waterfront Industrial and Working Waterfront Overlay districts. Industry exists to the south and west. An abandoned industrial structure exists to the north and the Inner Harbor is to the east.

SITE 20 (CSO GROUP 5)

Location

The site is located north of Wamsutta Street and east of North Front Street. The parcel is bounded by North Front Street to the west, an industrial complex to the north, and the Inner Harbor to the east and south. The site is located about 800 feet from the center of CSO Group 5.

Ownership

The site is privately owned.

Area

The site consists of one parcel of land with a total area of 4.1 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned as an Industrial "B" district and a Working Waterfront Overlay district.

Site Access and Traffic

The site is accessible from North Front Street. Traffic is light to moderate and can accommodate construction equipment. No disruption in traffic patterns will occur.

Current Land Use

The site is a vacant industrial site covered 30% by one structure, 60% by paved parking and 10% miscellaneous open space.

Surrounding Land Use

The area north and west of the site is zoned Industrial "B". North of the site is also zoned as a Working Waterfront Overlay district. South of the site is zoned as a Working Waterfront Overlay district and a Waterfront Industrial district.

SITE 21 (CSO GROUP 5)

Location

The site is divided into 2 parcels. The first is to the south of Interstate 195 (I-195) and bounded by Belleville Avenue to the west, the Washburn Street off-ramp from I-195 to the east, Washburn Street to the south, and I-195 to the north. The second parcel is bound by Coggeshall Street to the north, I-195 to the south, Belleville Avenue to the west, and the Coggeshall Street on-ramp to I-195 to the east. The site is located about 2,600 feet from the center of CSO Group 5.

Ownership

According to the City assessor's map, the City of New Bedford owns and maintains the south parcel. No ownership is indicated for the north parcel, however, it is presumed by the New Bedford planning office to be owned by the Commonwealth of Massachusetts.

Area

The area of the parcel south of I-195 is about 2.05 acres, and the area of the parcel to the north of I-195 is about 5.17 acres. Therefore, the total acreage associated with this site is approximately 7.22 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

No apparent zoning according to the Building Zone Map of the City of New Bedford.

Site Access and Traffic

The parcel south of I-195 is accessible from either Washburn Street or Belleville Avenue. The parcel north of I-195 is accessible from either Coggeshall Street or Belleville Avenue. Traffic is moderate to heavy in this vicinity, however, construction will not hinder traffic patterns.

Current Land Use

The south parcel is primarily open space with billboards at the west end of the site and a basketball court in the south-central portion of the site. The north parcel has a pump house at the northwest corner, a business structure at the north boundary adjacent to the pump house, and a parking area at a north-central portion of the property. The remainder of the site is grassy, open space.

Surrounding Land Use

The area north of the site is zoned Industrial "A" and Mixed-Use Business. West of the site is zoned Industrial "A", and south of the site is both a

Working Waterfront Overlay district and an Industrial "B" district. The area north and south of the site is highly developed with business and industry. West of the site are multi-family dwellings.

SITE 22 (CSO GROUP 6)

Location

The site is bounded by Coggeshall Street to the south, Mitchell Street to the west, Saugus Street to the north, and the Inner Harbor to the east. The site is located about 3,500 feet from the center of CSO Group 6.

Ownership

The site consists of two parcels of land, one parcel is privately owned and the other is owned by the Commonwealth.

Area

According to the City assessor's map, the total area of the parcel is 16.86 acres. However, because of the many existing business complexes, the available area is about 5.21 acres.

Elevation

Based on the USGS topographic map, the site elevation ranges from 10 to 14.

Zoning

The site is zoned as a Mixed-Use Business district.

Site Access and Traffic

Access to the site is from either Coggeshall Street to the south, Sawyer Street to the north, or Mitchell Street to the west. The roads have a heavy traffic load and can accommodate construction equipment. No traffic delays due to construction are foreseen.

Current Land Use

The land is vacant and used for parking by area businesses.

Surrounding Land Use

The area to the north of the site is zoned Industrial "B" and the area west of the site is zoned Industrial "A". North of the site is a combination of open space and commercial establishments. West of the site are multi-family residential structures. South of the site is open space (Site 21) and east of the site is the Inner Harbor.

SITE 23 (CSO GROUP 6)

Location

Site 23 is bounded by Sawyer Street to the south, Belleville Avenue to the west, and industrial buildings to the north and east. The parcel is located about 2,800 feet from the center of CSO Group 6.

Ownership

The site is privately owned.

Area

According to the City assessor's map, the site is a single parcel of land with an area of 3.54 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned as an Industrial "B" district.

Site Access and Traffic

The site is accessible from Sawyer Street to the south. Traffic volume is relatively light in the area and construction should not interfere with normal traffic flow.

Current Land Use

The parcel is vacant except for an approximate 125-foot wide portion of the site along the entire western boundary.

Surrounding Land Use

The area to the north and east of the site is zoned as an Industrial "B" district. West of the site is Mixed-Use Business and south of the site is a combination of Mixed-Use Business and Industrial "A".

SITE 24 (CSO GROUP 6)

Location

Site 24 is located to the north of Sawyer Street at its' eastern end. The site is bounded by the Inner Harbor to the north and east. The site is approximately 2,700 feet from the center of CSO Group 6.

Ownership

The site is owned by the City of New Bedford and maintained by the Parks Department.

Area

According to the City assessor's map, the total land area is 7.76 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned as an Industrial "B" district.

Site Access and Traffic

The site is accessible from Sawyer Street. Traffic volume is light in this area and should not be adversely affected by construction equipment, which can be accommodated by Sawyer Street.

Current Land Use

The site is open space with a soccer field located in the middle and a parking lot along the south boundary. Currently, this site is utilized by the Inner Harbor Superfund cleanup effort.

Surrounding Land Use

The area to the west is zoned Industrial "B" and the area south of the site is zoned Mixed-Use Business. Industrial complexes are west and south of the site.

SITE 25 (CSO GROUP 6)

Location

The site, referred to as Riverside Playground, is bound by Coffin Avenue to the north, Belleville Avenue to the west, a parking area to the south and the Inner Harbor to the east. The site is approximately 1,700 feet from the center of CSO Group 6.

Ownership

Riverside Playground is owned and maintained by the City of New Bedford.

Area

According to the City assessor's map, the site consists of one parcel with an area of 3.67 acres.

Elevation

Based on the USGS topographic map, the site elevation ranges from 5 to 15 feet. The average site elevation is approximately 10 feet.

Zoning

The site is zoned as an Industrial "B" district.

Site Access and Traffic

The site is accessible from Coffin Avenue to the north and Belleville Avenue to the west. The roads can accommodate construction equipment and should not adversely impact traffic.

Current Land Use

The site is used as a park and has recreational facilities including a playground, playing fields, basketball courts, and tennis courts.

Surrounding Land Use

The area north of the site is zoned Residence "C", Mixed-Use Business, and Industrial "B". West of the site is zoned Mixed-Use Business and south of the site is Industrial "B". North of the site on Coffin Avenue are several small businesses, and some multi-family housing. East of the site is the Inner Harbor, west of the site is multi-family housing and south of the site is an industrial complex.

SITE 26 (CSO GROUP 6)

Location

The site is a triangular plot of land bounded by Belleville Avenue to the west, Riverside Avenue to the east and Hathaway Street to the south. Belleville Avenue and Riverside Avenue merge to form the northern property bound. The site is approximately 500 feet to the center of CSO Group 6.

Ownership

The site consists of five parcels of land that are privately owned.

Area

According to the City assessor's map, the site's five parcels have a total land area of 4.84 acres.

Elevation

Based on the USGS topographic map, the average site elevation is about 15 feet.

Zoning

The site is zoned industrial, with half the site zoned Industrial "A" and half zoned Industrial "B".

Site Access and Traffic

The site is accessible from either Hathaway Street, Riverside Avenue, or Belleville Avenue. The area has a heavy traffic load and construction activity could adversely impact the normal traffic volume. All the roads can accommodate construction equipment.

Current Land Use

The land is approximately 50% open space and 50% parking space. The parking is being utilized by businesses to the east of the site.

Surrounding Land Use

North, east, and south of the site is zoned Industrial "B". West of the site is primarily zoned Mixed-Use Business. Industrial facilities are north and east of the site. South of the site is a salvage yard, parking area, and several buildings. West of the site is a scattering of business enterprises.

SITE 27 (CSO GROUP 6)

Location

The site, referred to as Brooklawn Park, is bounded by Brooklawn Avenue to the north, Ashley Boulevard to the west, Irvington Street to the south and Acushnet Avenue to the east. The site is located about 3,700 feet from the center of CSO Group 6.

Ownership

Brooklawn Park is owned and maintained by the City of New Bedford.

Area

According to the City assessor's map, the site consists of two parcels of land with a total area of 76.5 acres.

Elevation

Based on the USGS topographic map, the site elevation ranges from 35 to 80 feet. The average site elevation is about 50 feet.

Zoning

The site is zoned as a Residence "A" district.

Site Access and Traffic

The access points to the site are Ashley Boulevard to the west and Acushnet Avenue to the east. The area has a heavy traffic load but would not be adversely impacted during construction.

Current Land Use

The site is open space and parkland. Site facilities include maintenance structures, soccer and baseball fields, tennis and basketball courts, open fields, trails, wooded areas, and a fountain area on the east grounds.

Surrounding Land Use

The area surrounding Brooklawn Park is zoned Residence "B". Homes front the roads that bound the park.

SITE 28 (CSO GROUP 6)

Location

The site is located off River Road at the east end of Howard Avenue. The parcel is rectangular in shape, and is bounded by River Road to the west, the Inner Harbor to the east, an industrial facility to the south and open space to the north. The site is located about 3,800 feet from the center of CSO Group 6.

Ownership

The site consists of eight parcels of land. The ownership is private.

Area

According to the City assessor's map, the area of the eight parcels is approximately 1.56 acres. The eight parcels that comprise the site vary in shape and size.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned an Industrial "B" district.

Site Access and Traffic

Site access is from Howard Avenue to the south or River Road to the west. The area has a lightly traffic load and would not be adversely impacted by construction. The roads can accommodate construction equipment.

Current Land Use

The land is a paved parking area.

Surrounding Land Use

The area north and south of the site is zoned as an Industrial "B" district. Zoning west of the site is Mixed-Use Business and Residence "B". A vacant lot leading to an industrial facility abuts the property to the north. An industrial complex is on the south side of the site and parking lots are located to the southwest and west of the site. Northwest of the site is a residential area and the Inner Harbor is to the east.

SITE 29 (CSO GROUP 1)

Location

The site is north of the hurricane barrier at Clarks Cove. The land is triangular-shaped and bounded by Orchard Street to the west, Cove Road to the south, and parking areas to the north and east. The site is about 400 feet from the center of CSO Group 1.

Ownership

The site consists of three parcels of land. The land is privately owned by three parties.

Area

According to the City assessor's map, the site consists of 3 parcels with a total land area of 2.96 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned as an Industrial "B" district.

Site Access and Traffic

Access to the site is from Cove Road to the south or Orchard Street to the west. The area has a heavy traffic load but will not be adversely impacted by construction. Both roads can accommodate construction equipment.

Current Land Use

There is a structure erected on each of the three parcels that comprise the site.

Surrounding Land Use

The area surrounding the site is zoned Industrial "B". West of the site, across Orchard Street, are several businesses. North and east of the site are industrial complexes with open space and multiple parking areas. South of the site, across Cove Road, is the west end of the hurricane barrier.

SITE 30 (CSO GROUP 5)

Location

The site is bounded by Herman Melville Boulevard to the east, railroad tracks to the west and south, and businesses to the north along Wamsutta Street. The site is located about 600 feet from the center of CSO Group 5.

Ownership

The site consists of four parcels. Ownership is divided between the City of New Bedford and two businesses.

Area

According to the City assessor's map, the four parcels of land have a combined area of 12.55 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site is zoned as a Working Waterfront Overlay district and an Industrial "B" district.

Site Access and Traffic

The site is accessible from Herman Melville Boulevard to the east. The area has light to moderate traffic loads and can accommodate construction equipment. No adverse impacts should occur during construction.

Current Land Use

The land is a railroad yard.

Surrounding Land Use

The zoning is Industrial "B" to the north, Industrial "A" and Mixed-Use Business to the west, and Working Waterfront Overlay and Waterfront Industrial to the south and east. Industry exists on all surrounding areas of the site.

SITE 31 (CSO GROUP 4)

Location

The site is bounded by Gomes School to the north, Potomska Street to the south, and South Second Street to the west. John F. Kennedy Memorial Highway bounds the site to the east. The site is approximately 1,800 feet from the center of CSO Group 4.

Ownership

The site consists of one parcel of land owned by the City of New Bedford. The site is part of the Gomes School which abuts the property to the north.

Area

According to the City assessor's map, the site has a land area of 3.61 acres.

Elevation

Based on the USGS topographic map, the site elevation is less than 10 feet.

Zoning

The site has two zoning districts. The northern region, about 30% of the total site area, is zoned Industrial "A". The remaining 70% is zoned Mixed-Use Business.

Site Access and Traffic

The site is accessible from South Second Street or Potomska Street. South Second Street is a one-way street. Potomska Street provides the most practical access to the site, however, may cause traffic delays due to the heavy traffic load. Potomska Street can accommodate construction equipment.

Current Land Use

The site has recreational facilities including a basketball court, tennis court, playing field, playground, and paved parking area. About 50% of the parcel is an open, grass field.

Surrounding Land Use

Zoning is Mixed-Use Business to the north, south, and west of the site. East of the site, opposite John F. Kennedy Highway, is an area zoned Working Waterfront Overlay and Industrial "B". Gomes School abuts the property to the north, residential areas are to the west and south, and a bus terminal is located opposite John F. Kennedy Highway.

The thirty-one alternative sites were evaluated based on siting criteria described in Section 8.4. The results of the evaluation have been summarized and are presented in Table 8-4.

8.7 SELECTION OF PREFERRED SITES

The site evaluation process (discussed in Section 8.6 and presented in Table 8-4) provides a mechanism for screening the 31 alternative sites. The screening process is designed to reduce the number of sites, or to select preferred sites, to be considered for more detailed engineering, environmental, and socioeconomic analysis. The results of the evaluation do not indicate the absolute best site for location of CSO abatement facilities. Instead, the evaluation matrix indicates which sites should be further considered and which sites are inappropriate. Ultimate selection of a site for inclusion in the recommended plan can only be made after detailed investigations are undertaken, and site-specific conditions are considered.

As mentioned, preferred sites were selected based on site evaluation presented in Table 8-4. Thirty-one alternative sites were identified as possible locations for an abatement facility for one of the six CSO Groups. Also, several of the 31 sites having considerable land area were considered for location of a consolidated storage facility to control CSO design volumes for Groups 1 through 4.

Each site is compared with other alternative sites being considered for each CSO Group. The method yields one to three preferred sites for each CSO Group. Among the criteria, site location, ownership and area are most important when selecting a site because deficiencies in these areas (i.e., bad location, private ownership, and insufficient area) indicate fundamentally unsuitable sites. Following are the site comparisons according to CSO Group.

TABLE 8-4

SITE EVALUATION MATRIX

Site No.	Location	Ownership	Area	Elevation	Zoning	Access/ Traffic	Current Land Use	Surrounding Land Use	Summary
	Siting Criteria								
CSO Group 1									
✓ 1	+	+	+	+	-	0	-	0	2+
29	+	-	-	+	+	+	-	0	1+
CSO Group 2									
✓ 2	+	+	+	0	-	+	-	-	1+
3	0	+	+	0	-	+	-	-	0
✓ 4	-	+	+	0	-	+	+	0	2+
5	-	-	+	0	0	+	+	-	0
CSO Group 3									
6	0	-	+	0	-	+	+	-	0
7	-	+	+	0	-	+	-	-	1-
✓ 8	0	-	+	+	-	+	0	-	0
✓ 9	0	+	+	0	-	0	-	-	1-
10	-	-	+	+	+	+	-	-	0
CSO Group 4									
12	0	-	+	+	+	+	0	0	3+
✓ 13	+	-	+	+	+	+	0	0	4+
14	0	-	+	+	+	+	0	0	3+
15	-	-	-	+	+	0	0	0	1-
✓ 31	0	+	+	+	+	0	-	-	2+
CSO Group 5									
16	0	-	+	+	+	+	+	0	4+
✓ 17	+	+	+	+	+	+	0	0	6+
18	-	+	+	-	-	0	+	-	1-
✓ 19	+	+	+	+	+	+	0	0	6+
✓ 20	+	-	+	+	+	+	0	0	4+
21	-	+	+	+	+	+	0	0	4+
30	+	0	+	+	+	+	0	-	4+

TABLE 8-4, continued

SITE EVALUATION MATRIX

Site No.	Location	Ownership	Area	Elevation	Zoning	Access/ Traffic	Current Land Use	Surrounding Land Use	Summary
	Siting Criteria								
CSO Group 6									
22	-	0	-	+	+	+	+	0	2+
23	-	-	-	+	+	+	0	+	1+
24	-	+	+	+	+	+	-	0	3+
✓ 25	0	+	0	+	+	+	-	-	2+
✓ 26	+	-	+	+	+	-	+	0	3+
27	-	+	+	-	-	+	-	-	2-
28	-	-	-	+	+	+	+	0	1+
CSO Group 1 and Group 4 (Consolidated Storage Facility)									
1	-	+	+	+	-	0	-	0	0
✓ 11	+	-	+	+	+	+	-	0	3+

NOTES:

+ Most Suitable
0 Suitable
- Least Suitable
✓ Selected for Detailed Evaluation

8.7.1 CSO GROUP EVALUATION

CSO Group 1

Two sites, Site 1 and Site 29, were evaluated as possible locations for CSO Group 1 abatement facilities. Group 1 sites are located north of the hurricane barrier at Clarks Cove. Abatement facilities would be constructed to control design overflows from CSOs 003 and 004. Both sites were comparable regarding the summary totals with Site 1 (2+) rated slightly higher than Site 29 (1+). Site 1 is preferable because of its large size and public ownership.

Therefore, Site 1 was selected as the preferred site to locate abatement facilities for CSO Group 1.

CSO Group 2

Sites 2, 3, 4 and 5 were considered for CSO Group 2 abatement facilities. Group 2 sites are located on the west side of Clarks Point. Abatement facilities would be constructed to control design overflows from CSO outfalls 005 through 011.

The summary rating from the evaluation matrix indicates that all sites within this grouping appear to be comparable. However, further analysis of siting criteria indicates that Site 5 (0) is rated "least suitable" because of location and ownership. Site 2 (1+) is rated "most suitable" for location, ownership, and area; and Site 3 (0) was only rated "suitable" for location. Sites 2 and 4 are considered preferable because of ownership, area and current land use.

Therefore, Site 2 and Site 4 were selected as preferred sites for CSO Group 2.

CSO Group 3

Sites 6 through 10 were considered for CSO Group 3 abatement facilities. Group 3 sites are located on the east side of Clarks Point. Abatement facilities would be constructed to control overflows from CSOs 012 through 018.

At first glance, the evaluation matrix summary does not indicate any singular preferred site. All sites within the Group 3 except Site 7 (1-) and Site 9 (-1) have an equivalent rating (0). A closer examination of the criteria indicates that Site 9 is the only alternative site which has no rating lower than "suitable" for the three most critical siting criteria, location, ownership and area. Therefore, it was carried forward for further consideration.

From the remaining Group 3 site alternatives, Sites 6 (0) and 8 (0) have identical ratings for each critical criteria with only one "least suitable" rating while Site 10 has two "least suitable" ratings. Sites 6 and 8 were evaluated further by considering the next most-critical criteria: elevation, since high site elevation can markedly increase construction costs. Site 8 was rated slightly better than Site 6 because it is lower in average elevation.

Therefore, Site 8 and Site 9 were selected as the preferred sites for CSO Group 3.

CSO Group 4

Sites 11 through 15 and Site 31 were considered for CSO Group 4 abatement facilities. Group 4 sites are located in East New Bedford, north of the hurricane barrier, and south of Route 6. Abatement facilities would be constructed to control design overflows from CSOs 028 and 030 through 034 in the Inner Harbor.

Site 13 (4+) is considered a preferred site because of the high rating and it is the only site rated "most suitable" for location. From Sites 12

(3+), 14 (3+) and 31 (2+), Site 31 is preferable because it is rated "suitable" or better for location, ownership, and area, the three critical criteria.

Therefore, Site 13 and Site 31 were selected as the preferred sites for CSO Group 4.

CSO Group 5

Sites 16 through 21 and Site 30 were considered for CSO Group 5 abatement facilities. Group 5 sites are located in East New Bedford, north of Route 6, and south of Interstate 195. Abatement facilities would be constructed to control design overflows from CSOs 020, 021 and 035 through 037 in the Inner Harbor.

For Group 5, the two highest rated sites, Site 19 (6+) and Site 17 (6+), are the most preferred sites. Other highly rated sites include 16 (4+), 20 (4+), 21 (4+) and 30 (4+). From these four, only Sites 20 and 30 are rated "most suitable" for location. However, Site 20 is considered a more preferable site than Site 30 because Site 30 has some unresolved environmental issues associated with the railroad yard.

Therefore, Site 17, Site 19, and Site 20 were selected as the preferred sites for CSO Group 5.

CSO Group 6

Sites 22 through 28 were considered for CSO Group 6 abatement facilities. Group 6 sites are located in East New Bedford, north of Interstate 195. Abatement facilities would be constructed to control design overflows from CSOs 022 through 027, 040 and 041 in the Inner Harbor.

All sites except Site 27 (2-) can initially be considered good sites. However, Site 25 (2+) is preferred because it has only "suitable" or better ratings for location, ownership, and area. From the remaining

possibilities, Site 26 (3+) is preferred because it is the only site with a "most suitable" rating for location.

Therefore, Site 25 and Site 26 were selected as the preferred sites for CSO Group 6.

CSO Group 1 and Group 4 (Consolidation Option)

Sites considered for location of a consolidated storage facility for CSO Groups 1 and 4 require a minimum land area of 13.0 acres. Based on land area, 2 sites (Sites 1 and 11) of the 8 sites considered for Groups 1 and 4 initially appear to be usable. From the two sites, Site 11 (3+) is the only site with a "most suitable" rating for location, which is considered to be the most important siting criteria for construction of a centrally-located consolidated storage facility.

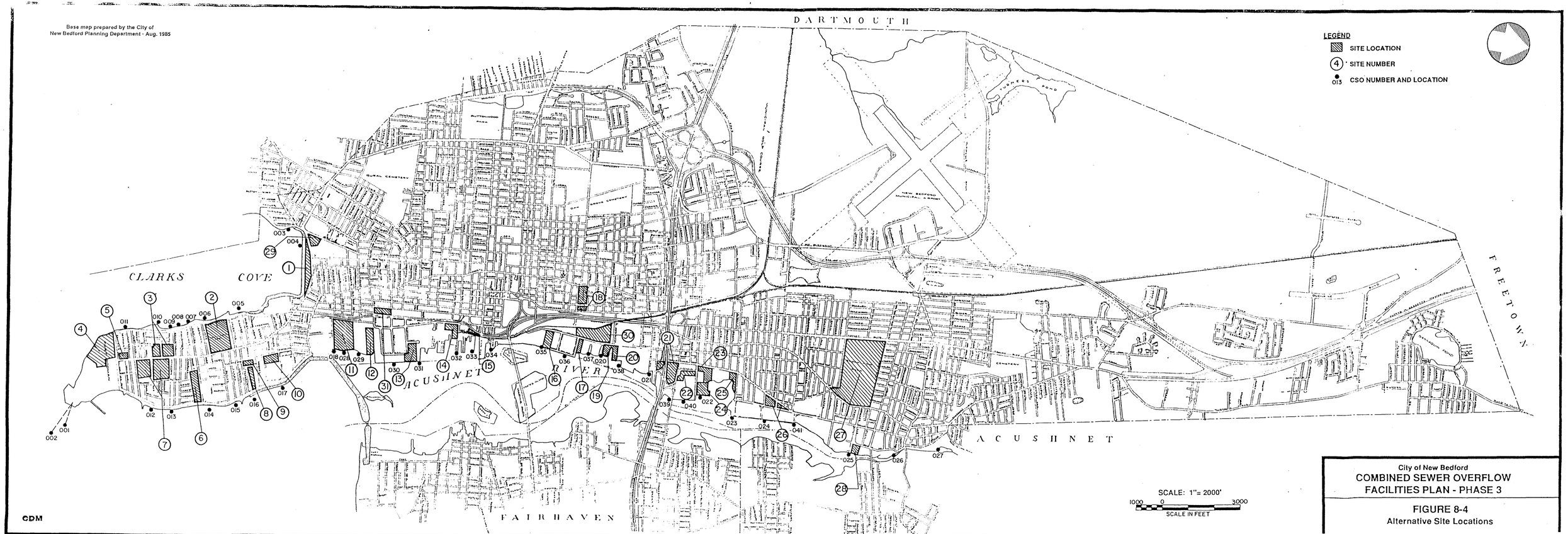
The methodology used to rate site location for the consolidated storage option were the same as those outlined in Section 8.4. However, the optimal siting location for the consolidation option is not the same as the optimal location for Group 1 or 4 alone. Therefore, for the two sites considered for consolidation, the Site Evaluation Matrix may show a different location rating under the consolidation evaluation section than under the site's particular group.

Therefore, Site 11 was selected as the preferred site for the consolidated storage of Groups 1 and 4 overflows.

8.8 SUMMARY

Alternative sites were evaluated to select appropriate sites for locating CSO abatement facilities for more detailed analysis. The methodology used involved the initial ranking of the sites based on eight siting criteria. Sites with suitable ranking were then evaluated in more detail. Location, ownership, and area were considered to be more important than the other five criteria, with location considered the most important criteria.

From the two-step evaluation process, 13 sites were selected to be considered for more detailed analysis in later sections. The thirteen sites selected were Sites 1, 2, 4, 8, 9, 11, 13, 17, 19, 20, 25, 26, and Site 31.



City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 8-4
Alternative Site Locations

9.0 EVALUATION OF SELECTED ALTERNATIVES

9.1 GENERAL

In Section 7.0, available technologies for mitigation of CSO impacts were identified and assessed as to their applicability for New Bedford. Those technologies that warrant further, more detailed evaluation were then selected. In this section, the results of detailed evaluation of selected alternatives is presented. The recommended CSO abatement plan for the City is identified at the end of this section, and discussed in detail in Section 10.0.

This section is organized into five major parts:

- alternative evaluation criteria
- evaluation methodology
- discussion of alternatives for each grouping of CSO outlets
- system-wide CSO abatement alternatives
- recommendations

9.2 ALTERNATIVE EVALUATION CRITERIA

9.2.1 PLANNING PERIOD

The planning period is the time period over which a wastewater treatment system is evaluated for cost-effectiveness. Pursuant to the Clean Water Act (Public Law 92-500) and EPA guidelines on cost-effectiveness, the planning period for this study is defined as 20 years, beginning with the initial operation of a wastewater treatment system.

The planning period delineates the time period for population, land use development and wastewater flow projections. Service life, useful life and

construction staging for combined sewer overflow control facilities are addressed below.

9.2.2 DISCOUNT RATE

Cost estimates have been prepared for several types of CSO control facilities to provide a means of comparing various concepts and establishing expected budgetary expenditures. Alternatives can be selected for further, more detailed evaluation depending on the cost and efficiency of the facilities.

The estimates have been prepared in accordance with EPA guidelines, including construction and annual facility operation and maintenance costs. The present worth economic analysis is based upon a 20-year planning period and an 8.625 percent interest rate.

9.2.3 EVALUATION CRITERIA

CSO abatement facilities must be selected to meet receiving water quality objectives defined in Section 5.0. CSO mitigation plans employing the alternatives selected for further consideration in Section 7.0 were evaluated based on the following criteria:

1. Impact on Receiving Water Quality
2. Estimated Costs
   - o Capital
   - o Operation and Maintenance
3. Siting Constraints
4. Complexity and Other Operational Considerations
5. Environmental Impacts
   - o Construction-Related
   - o Long-Term
6. Reliability

Considerations other than costs and water quality impacts which are more difficult to quantify are addressed in the discussion of each strategy

evaluated. Environmental impacts and mitigative measures, both long term and construction related are identified in Part II, the Environmental Impact Document (EID).

9.2.4 RECEIVING WATER PRIORITIES

The three receiving waters impacted by New Bedford's CSOs were prioritized based on impaired uses, and social and economic value. Swimming and shellfish harvesting potential essentially dictated the following priority ranking:

1. Clarks Cove
2. Outer Harbor
3. Inner Harbor

The purpose of ranking each receiving water is twofold:

- To determine CSO mitigation alternatives that meet the water quality objectives of each receiving water; and
- To assist in prioritizing components of system-wide CSO mitigation projects.

Receiving water issues and objectives were discussed in detail in Section 5.0. Basically, Clarks Cove is of highest priority since it has the highest use potential from both recreational and shellfish harvesting standpoints, resulting in the highest social and economic value of the three receiving waters. Several CSOs, including CSO 004 which discharges the largest volume of CSO to any New Bedford receiving water, substantially impact swimming areas and shellfish beds. The physical characteristics of the Cove, including wind, tidal action and its enclosed configuration, tend to magnify CSO impacts.

The Outer Harbor has similar impaired uses, and high swimming and shellfish harvesting potential, second only to Clarks Cove. The six CSOs that discharge directly into this receiving water have relatively small drainage areas resulting in lower CSO volumes and resulting impacts, as compared to Clarks Cove. Also, the physical characteristics of the Outer Harbor are

more conducive to dampening impacts from the six CSOs along the shore, and from those discharging into the Inner Harbor and Acushnet Rivet which eventually flow into the Outer Harbor through the hurricane barrier. Resulting CSO impacts are dampened slightly as compared to Clarks Cove due to wind and tidal effects, and the Outer Harbor's larger mixing volume.

The Inner Harbor is ranked third since its uses (existing and potential) are more limited than either Clarks Cove or the Outer Harbor. It's present use is, for the most part, limited to boating - both pleasure and commercial. CSOs have a relatively high impact to the quality of water in the Inner Harbor (and Acushnet River) due to its physical characteristics and lack of substantial water movement (i.e., flushing action). Storm runoff also impacts water quality since annual pollutant loadings are comparable to CSOs in the Inner Harbor. Under certain wind and tide conditions, pollutant discharges into the Inner Harbor also impact certain high use areas in the Outer Harbor. This is discussed in greater detail in Sections 5.0 and 6.0. In general however, CSOs have minimal impact on current uses of the Inner Harbor and its social and economic value.

9.3 METHODOLOGY

9.3.1 GENERAL

The methodology employed to develop and evaluate CSO mitigation alternatives and select a recommended plan is a step-by-step and somewhat iterative process. The major steps are as follows:

1. Establish CSO groupings.

2. Based on receiving water priority and objectives (Section 5.0), select applicable CSO storage and treatment technologies for each grouping from those identified in Section 7.0.

3. Identify site availability/constraints within each group and select potential CSO facility site(s). The two alternative secondary wastewater treatment plant (WWTP) sites (Fort Rodman and Standard Times) will be given highest consideration for applicable CSO groupings.

4. Determine physical facilities required to consolidate CSOs within each grouping.

5. Perform cost-benefit analyses on selected CSO mitigation technologies and determine cost-effective CSO control level for each grouping.

6. Develop CSO mitigation alternatives for each grouping which reflects the optimum combination of storage/treatment at the cost-effective control level, including required consolidation facilities (i.e., sewers, diversion structures, etc...).

7. Estimate costs associated with complete elimination of CSOs for each grouping to compare with the cost to reduce CSOs to the cost-effective control level from Step 6. Sewerage system separation shall be the technology employed for elimination of CSOs.

8. Develop system-wide CSO abatement alternatives reflecting feasible CSO storage/treatment centralization options. The two WWTP sites will be given highest consideration for CSO storage and/or treatment.

9. Determine the cost effective CSO control level and optimum combination of storage/treatment for each system-wide alternative.

10. Estimate the present worth cost of all alternatives, including complete separation.

11. Assess and rank each alternative based on the criteria given in Section 9.2.3.

12. Select the most feasible and cost-effective CSO abatement alternative for each CSO grouping, and system-wide. The cost for complete separation (i.e., CSO elimination) will be compared with each alternative to clearly indicate the cost difference between the cost-effective CSO control level and complete CSO elimination.

13. Develop recommended plan and prioritize each project within the plan based on receiving water priority and impacts, and cost-effectiveness.

14. Develop implementation schedule for the total program.

Steps 1 through 12 will be discussed in detail in the following portions of this section. Steps 13 and 14, the Recommended Plan, is presented in Section 10.0.

9.3.2 CSO GROUPINGS

The 35 CSOs have been consolidated into 6 groupings as illustrated on Figure 9-1 and summarized in Table 9-1. The number of each group reflects its order of priority relative to receiving water priority (Section 9.2.4) and ultimately the implementation of CSO mitigation strategies. The following considerations entered into the selection of CSOs for each grouping:

- receiving water priority, issues and objectives
- physical proximity of CSOs
- system hydraulic limitations relative to CSO consolidation
- availability of sites for CSO storage/treatment facilities

9.3.3 CSO ABATEMENT TECHNOLOGIES

Table 9-2 summarizes the CSO control alternatives identified in Section 7.0 which may be applicable in New Bedford. While all alternative technologies may be viable from a system-wide perspective, not all may be appropriate on a group-by-group basis. In this section, the alternative technologies are assessed as to their applicability for each of the six CSO groupings.

9.3.3.1 Collection System Modifications

Sewer Separation. As discussed earlier in this section, sewerage system separation will be evaluated for each CSO group. It is considered an appropriate technology for complete elimination of CSOs. This evaluation will enable comparison with storage and/or treatment options which will mitigate CSO impacts to an extent reflecting the most cost-effective control level (i.e., the selected design storm).

Costs for separation can be directly compared with costs for storage/treatment options, however receiving water impacts and benefits cannot. Storage/treatment options would serve to mitigate pollution contributed by both components of CSOs - storm runoff plus sanitary sewage.

TABLE 9-1

CSO GROUPINGS

Receiving Water	CSO Group Number	CSO Numbers	Tributary Area (acres)	Estimated Annual CSO Volume (MG)
Clarks Cove	1	003, 004	814.7	496.2
	2	005 thru 011	128.2	0.6
		SUBTOTAL	942.9	496.8
Outer Harbor	3	012 thru 018, 028	101.7	41.4
		SUBTOTAL	101.7	41.4
Inner Harbor (and Acushnet River)	4	030 thru 034	532.7	94.2
	5	035 thru 037, 020, 021	395.6	212.1
	6	040, 041, 022 thru 027	695.2	280.9
		SUBTOTAL	1623.5	587.2
		SYSTEM-WIDE TOTALS	2668.1	1125.4

TABLE 9-2

CSO CONTROL ALTERNATIVES SELECTED FOR DETAILED EVALUATION

COLLECTION SYSTEM MODIFICATIONS

- Sewer Separation
- Construction of Parallel Conveyance Systems in Conjunction with System-Wide CSO Treatment Facilities
- Regulator Modifications in Conjunction with System-Wide CSO Treatment Facilities

STORAGE

- In-Line
- Off-Line

TREATMENT

- Screening
- Sedimentation
- Swirl and Helical Bend Concentrators
- Disinfection

Separation would eliminate the sewage portion, however storm runoff would continue to discharge to all New Bedford receiving waters untreated each time it rains. In Section 6.0, it was demonstrated that if all CSOs were eliminated (i.e., total system separation), storm-generated flows and pollutant loadings to all New Bedford receiving waters would still be significant. In Section 7.0, pollution associated with storm runoff was further discussed.

From these discussions, it is apparent that separation is a more viable alternative for Groups 5 and 6 than for Groups 1 through 4. Inner Harbor and Acushnet Rivet uses are significantly less sensitive to storm water impacts under the separation option. For Groups 5 and 6, separation can be compared more appropriately with CSO treatment and, to a lesser extent, with storage/pumpback from a receiving water impacts standpoint. Clarks Cove and Outer Harbor uses, however, are very sensitive, especially to aesthetic impacts from floatables contained in storm runoff. Accordingly, receiving water impacts (benefits) of separation cannot be compared with the benefits resulting from storage/pumpback of CSOs. Reducing both components of CSOs to fewer discharges per year or less would result in substantially more beneficial impacts to the uses of Clarks Cove and the Outer Harbor as compared to separation. As stated above, separation eliminates the sewage portion of CSOs, however storm water discharges and associated pollutant loadings would continue to enter these high priority waters each time it rains. Separation will not be recommended for Groups 1 through 4, however it will be evaluated for comparison purposes only with CSO storage/pumpback.

In addition to total system separation for each grouping, <u>partial separation</u> (i.e., separation of limited areas within CSO groupings) was also evaluated, as appropriate, in combination with storage/treatment options. Where partial separation could eliminate or significantly reduce the size and cost of conveyance pipelines and/or storage/treatment facilities, evaluation was considered to be warranted. In general, the cost-effectiveness of partial separation is related to the physical layout of the existing sewer system and resulting pipelines necessary to convey flow to the storage or treatment facility. Based on an assessment of

potential cost savings for the six CSO groupings, partial separation was only considered for Group 6.

Construction of Conveyance Sewers. For most CSO abatement plans, new sewers are typically required to either (1) consolidate CSOs for conveyance to a storage/treatment facility, or (2) increase existing system interceptor conveyance capacity to the WWTP. For CSO abatement alternatives that are evaluated for each grouping, required new sewers will generally consist of CSO consolidation conduits. For system-wide CSO abatement alternatives, CSO consolidation conduits are also necessary. In addition, additional system interceptor conveyance capacity is also required for those system-wide alternatives that involve CSO treatment at the WWTP. Details of required conveyance sewers for each alternative evaluated are discussed later in this section.

Regulating Devices. As discussed in Section 7.3.3, the potential use of regulating devices as a CSO control technology is limited in New Bedford due to the capacity constraints of the existing interceptor system. As a result, this alternative was evaluated for only two applications.

1. To increase system capacity in conjunction with system-wide CSO control alternatives which include parallel conveyance systems to augment interceptor capacity.
2. To maximize use of in-line storage by regulating flow in major collector sewers.

Evaluation of the first application will be discussed in Section 9.5. The second will be discussed in Section 9.4 and was investigated only for those groupings having major collector sewers. Included are Group 1 (Tripps Brook, Liberty Street and Grape Street collectors), Group 5 (Willis Street collectors) and Group 6 (Belleville Avenue , Willis Street and Church Street collectors).

9.3.3.2 Storage and Treatment

As indicated in Section 9.2.4, storage of CSOs with eventual pumpback into the interceptor system is the appropriate technology for Groups 1

through 4. This will serve to reduce CSO discharge frequency into the higher priority receiving waters to four occurrences per year or less (i.e., the selected design storm for storage/pumpback facilities will have a minimum 3-month recurrence frequency as discussed later in this section). The primary advantage of this technology as it relates to the higher priority receiving waters is that treatment of stored CSOs will occur at the new secondary treatment plant, with treated effluent discharging into offshore waters. There would be no local CSO treatment and effluent discharge to near shore, high use (priority) receiving waters. Group 4 also warrants this technology since under certain wind and tide conditions, CSO discharges can impact high use areas in the Outer Harbor (Section 6.0).

Local CSO treatment is not appropriate for Groups 1 through 4 since the treated effluent would still contain levels of fecal coliform bacteria and other pollutants that may adversely impact uses. The equivalent of primary wastewater treatment plus disinfection would be employed for CSO treatment. Although solids, BOD and bacteria would substantially be reduced by this technology, remaining solids in the treated effluent may contain heavy metals and toxic compounds. In addition, the chlorinated effluent may impact fish and shellfish larvae unless de-chlorination is practiced or another disinfectant is used. Aesthetics would also be affected by local CSO treatment system discharges, an important consideration due to the high social and economic value of these receiving waters.

Storage/pumpback and CSO treatment are both appropriate technologies for Groups 5 and 6. Receiving waters include the Inner Harbor and Acushnet Rivet where major uses are limited to boating (pleasure and commercial). CSO discharges from these groups have little to no impact on the higher priority waters beyond the hurricane barrier (Outer Harbor). Also, remaining pollutants contained within treated CSO discharges would have no impact on current uses of the river or Inner Harbor. Accordingly, local CSO treatment and effluent discharge is, in addition to storage/pumpback, an appropriate technology for Group 5 and 6.

In-Line Storage. As noted in the discussion of regulating devices (Section 9.3.3.1), in-line storage was evaluated for CSO Groups 1, 5 and 6.

Off-Line Storage/Pumpback. Off-line storage/pumpback was evaluated for all CSO groupings to the maximum extent possible. Since pumpback rates for storage are limited by the capacity of downstream conveyance and treatment facilities, storage at all CSO groupings is not feasible under certain combinations of design storm and treatment capacity. Table 9-3 shows a matrix of total CSO volume and maximum storage volume which can be pumped back in an 18-hour period for various design storms and conveyance/treatment capacities. As shown, the total CSO volume exceeds the maximum volume which can be pumped back under several combinations of design storm and system capacity. In those cases, treatment is required at CSO groups 5 and/or 6.

Treatment

As noted earlier in this section, treatment of CSOs is only applicable to CSO Groups 5 and 6. Due to the limited potential uses of the Inner Harbor, screening plus disinfection will be evaluated as a feasible combination of treatment technologies, in addition to high rate sedimentation with screening and disinfection. Swirl and helical bend concentrators were also considered as an alternative to sedimentation basins, but were found to be more costly and less practical.

Screening. While screening alone does not achieve significant levels of solids reduction it does, however, reduce amounts of objectionable floatables from CSOs. It's most significant disadvantage as compared to sedimentation basins is that the storage volume in the basins serves to dampen peaks in flow rate. Consequently, if pumping is required in conjunction with screening facilities, pumps must have greater capacities in order to meet peak flow rates. Another advantage sedimentation basins offer is contact time for disinfection. Screening

TABLE 9-3

TOTAL DISCHARGE VS. MAXIMUM STORAGE FOR WWTP CAPACITIES [1]

	WWTP Capacity Including Secondary Treatment And CSO Facilities (MGD)							
	75		100		130		155	
Design Storm	Total CSO Volume	Maximum Storage[2]	Total CSO Volume	Maximum Storage[2]	Total CSO Volume	Maximum Storage[2]	Total CSO Volume	Maximum Storage[2]
1-Month	15	28	10	44	4	54	1	69
3-Month	47	28	38	44	29	54	23	69
6-Month	79	28	70	44	60	54	52	69
12-Month	103	28	90	44	78	54	69	69

NOTES:

(1) CSO Discharge And Maximum Storage Volume in Million Gallons

(2) Maximum volume that can be pumped back into the system and to the WWTP over an 18 hour period

facilities alone would require some type of detention to provide sufficient contact time.

Sedimentation. Sedimentation at hydraulic loading rates of up to 1500 gal/day/ft^2 were evaluated. As noted in the preceding discussion, the storage volume provided in sedimentation basins serves to dampen peak influent flow rates and provides contact time for disinfection. In performing optimum tank size analyses, the trade-off in pumping facility costs versus sedimentation basin cost was evaluated. As a result, hydraulic loading rates may be lower than the maximum 1,500 gal/day/ft2 (providing a higher level of treatment) for the most cost-effective sized facility.

Swirl/Helical Bend Concentrators. Upon further, more detailed evaluation, swirl or helical bend concentrators are not suitable for use in New Bedford. Because the underflow from a concentrator is designed to discharge directly to the interceptor system, it must either replace an existing flow regulator, or provision must be made to store and pumpback the underflow. The configuration of New Bedford's interceptor system makes replacement of existing regulators with swirl or helical bend concentrators impractical due to the lack of available land and incompatibility with the surroundings along the main interceptor. In addition, swirl concentrators do not provide adequate detention time, and would not substantially reduce effluent pumping requirements as do sedimentation basins. For these reasons, swirl concentrators were found not to be cost-effective at either CSO Group 5 or Group 6.

Disinfection. The method of disinfection evaluated was sodium hypochlorite in commercially available solutions (generally 15% strength). Liquid chlorine compound solution is considered the most appropriate disinfection method due to the hazards associated with the handling and storage of chlorine gas. Because chlorine gas is highly toxic, its use is not recommended in a CSO facility since it would be staffed only on a part-time basis. Sodium hypochlorite would be injected directly into the influent channel of a treatment facility.

For disinfection in conjunction with screening facilities only, adequate chlorine contact time would require a chlorine contact chamber while sedimentation basins provide detention times sufficient for in-basin chlorine contact.

Due to concern over the potential impacts of chlorine residual on the aquatic environments, dechlorination has been included in all treatment alternatives. Two alternative methods of dechlorination are available, sulfur dioxide (SO_2) and sodium bisulfate ($NaHSO_3$). Sulfur dioxide is a gas and would be purchased in compressed gas cylinders while sodium bisulfate is a liquid solution which is available at strengths up to 38%. For CSO applications, the sodium bisulfate alternative is most appropriate for the same reasons that the use of sodium hypochlorite was selected for disinfection.

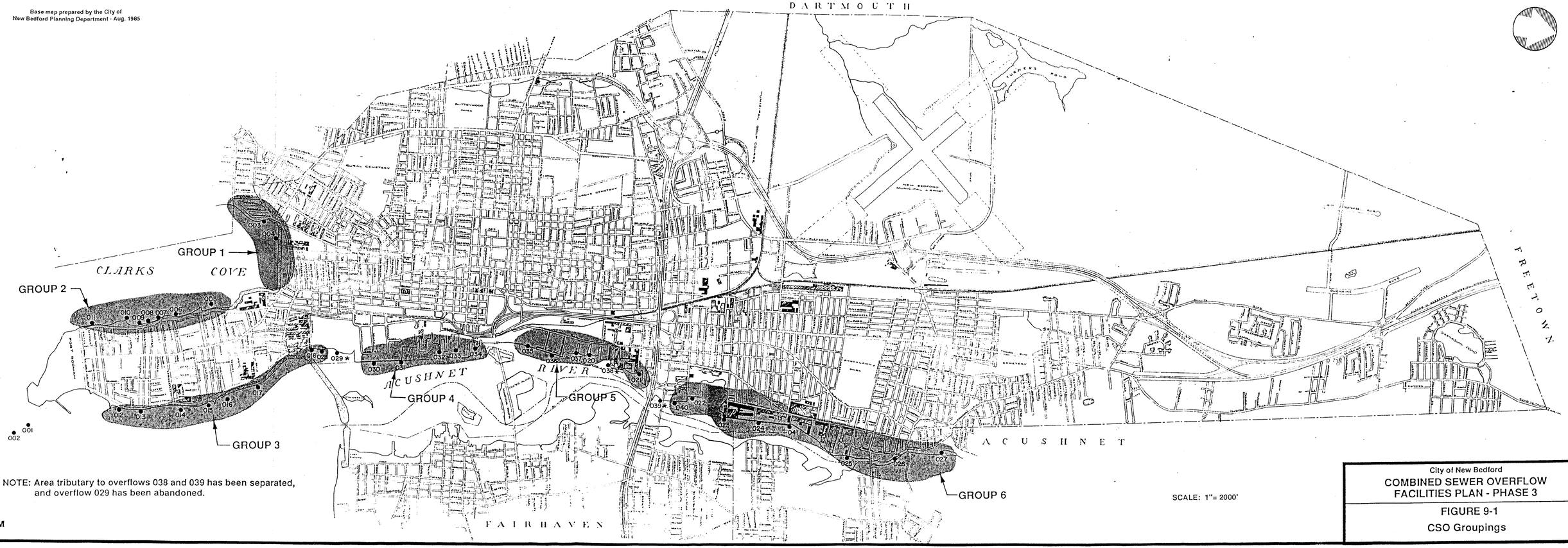
9.3.4 PRELIMINARY COST ESTIMATES

Cost estimates developed for the evaluation of alternatives were based on curves relating capital cost to capacity for the components of each alternative plan. Figure 9-2 shows the cost curves which include the following items:

1. Storage Basins
2. Sedimentation Basins
3. Swirl/Helical Bend Concentrators
4. Pumping Facilities
5. Disinfection Facilities

Information from a number of sources was utilized in developing the curves. This included actual construction costs of existing CSO facilities, unit costs for individual treatment and transport facilities developed from actual construction costs, EPA guidelines for estimating costs of CSO treatment facilities, and estimated costs for individual treatment unit processes. Cost curves are for current price levels and reflect an approximate ENR index of 4,600.

Base map prepared by the City of
New Bedford Planning Department - Aug. 1985



* NOTE: Area tributary to overflows 038 and 039 has been separated, and overflow 029 has been abandoned.

SCALE: 1"= 2000'

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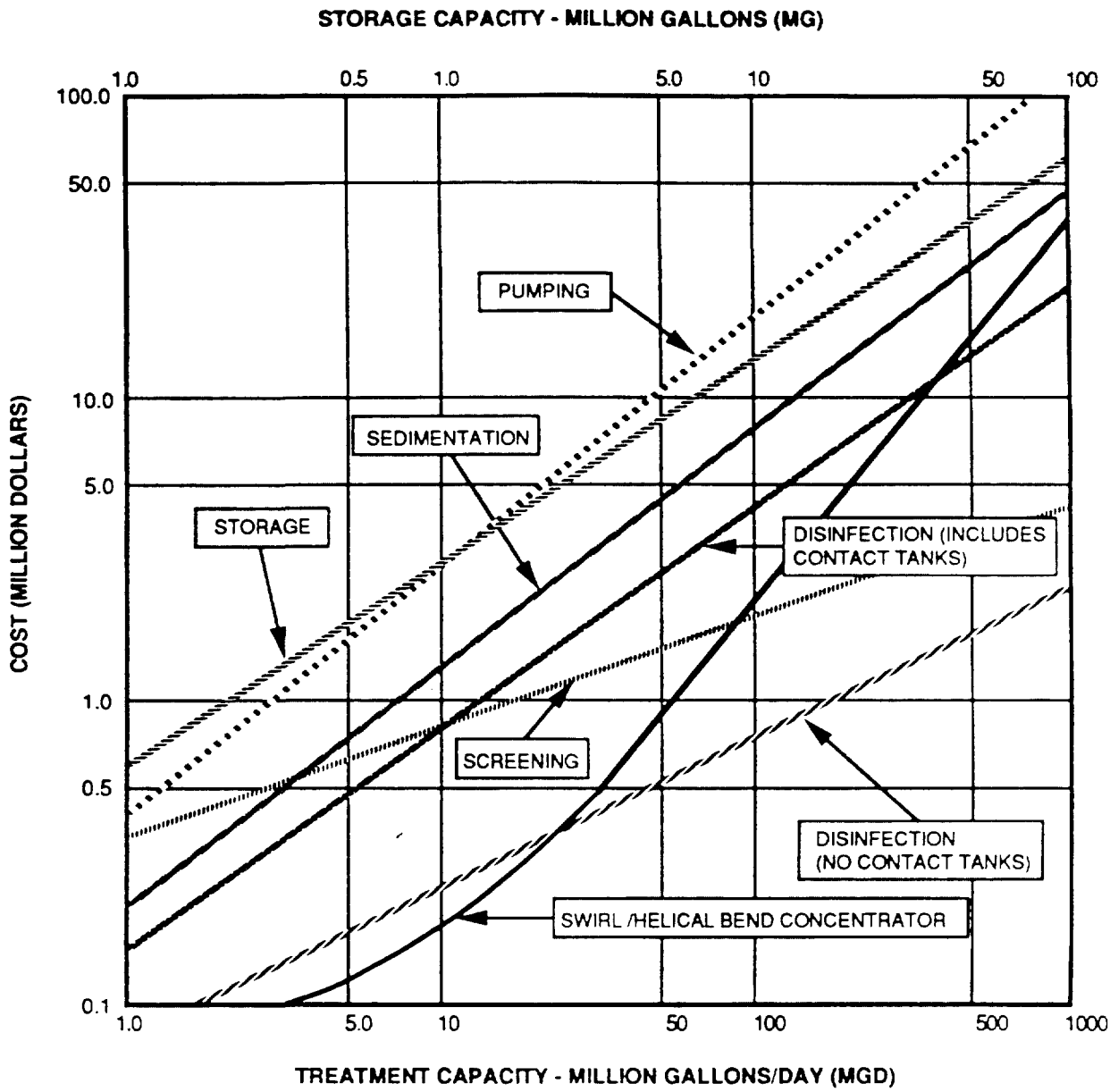
FACILITIES PLAN - PHASE 3

FIGURE 9-1
CSO Groupings



* COSTS REFLECT CURRENT PRICE LEVELS - ENR = 4600

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FIGURE 9-2
COST CURVES FOR ALTERNATIVE
CSO ABATEMENT STRATEGIES

9.3.5 ANALYSIS OF STORAGE/TREATMENT OPTIONS

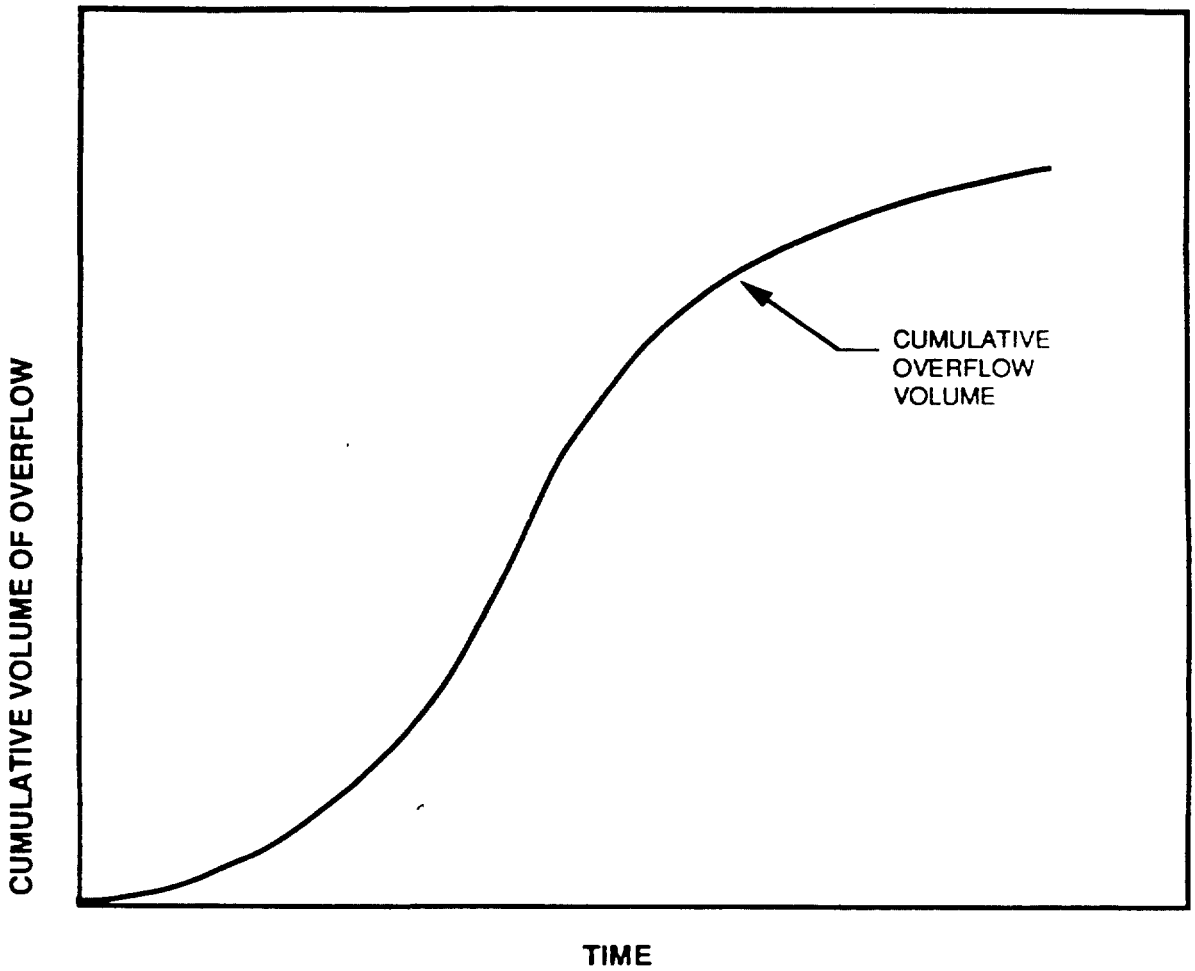
Trade-off analyses, conducted to determine the optimum combination of storage and/or treatment for CSO Groups 5 and 6, were performed utilizing a mass diagram. A mass diagram represents the cumulative wet weather influent volume to a CSO treatment or storage facility at a particular site over time, and can be developed directly from a design storm hydrograph generated by the SWMM model. A typical mass diagram is shown on Figure 9-3. This diagram allows the following to be readily determined:

- maximum required rate of CSO treatment with no storage provided (boundary condition);
- maximum required CSO storage volume with no treatment provided (boundary condition);
- CSO storage volume required at any selected treatment rate.

The slope of mass diagram represents the rate of CSO discharge at any time during a storm event. The boundary conditions thus occur at the end of the storm (slope equals zero; treatment rate equals zero; required storage volume is maximum) and at the inflection point of the curve (slope is maximum; treatment rate is maximum required storage volume is zero). The storage volume required for any treatment rate between these two conditions can be determined directly from the mass diagram.

During a storm event, the rate of CSO discharge increases as the rainfall intensity increases. Storage becomes necessary when the CSO discharge rate exceeds the capacity of the treatment facility (graphically, this is represented by the point of tangency of the treatment rate line and the mass diagram below the inflection point).

The actual storage volume required depends on how a storage/treatment facility is designed to operate. If maximum capture and transport of CSOs for centralized treatment is desired (i.e., minimize effluent discharge from satellite treatment facilities), all available storage at the facility is first filled prior to initiating CSO treatment processes with local effluent discharge. For this option, the storage volume required is the



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COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-3
TYPICAL MASS DIAGRAM

total CSO volume at the tangency point above the inflection point on the curve. If maximum local treatment is desired (i.e., minimize storage requirements and, in turn, facility size by discharging from each treatment cell (sedimentation basin) as it reaches its hydraulic capacity), the storage volume is the sum of the individual sedimentation basin volumes (cells) determined by the difference between the mass diagrams and the treatment rate line of the preceding basins (cells). Figure 9-4 illustrates the two concepts on a typical mass diagram.

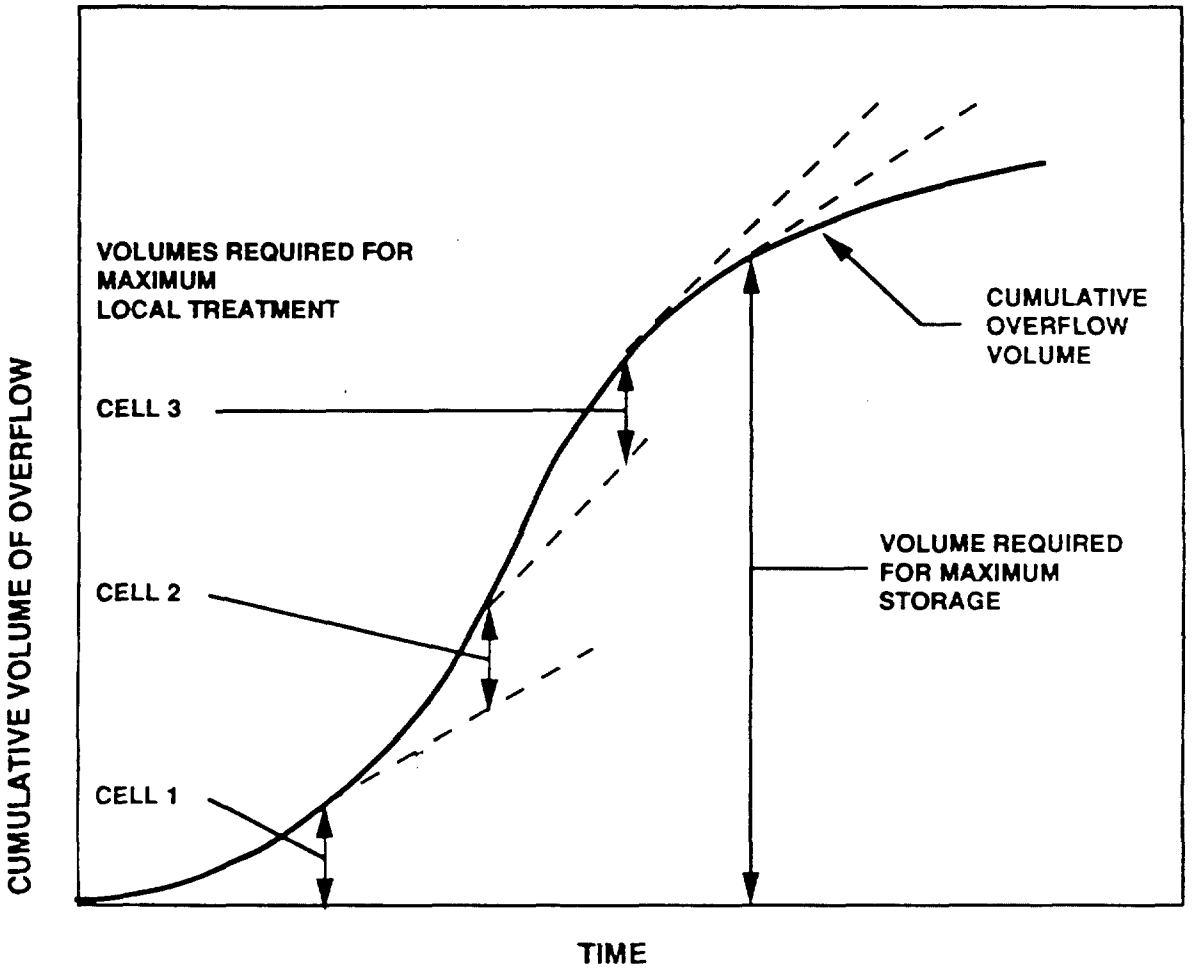
Analyses for CSO Groups 5 and 6 were made employing both concepts. As a result of these analyses, the first concept (maximum storage) was selected for the following reasons:

- Maximum use of storage is consistent with the concept of maximum utilization of the wastewater treatment plant.
- There is no significant cost savings associated with maximizing treatment capacity primarily since pumping of the treated effluent to receiving waters is required. The cost savings of the smaller storage/treatment facility is essentially offset by the cost for larger capacity effluent pumping facilities.

By applying the unit costs discussed previously, the optimum combination of storage and treatment capacity can be determined at a given site for a selected design storm.

9.3.6 DESIGN STORM SELECTION (COST-BENEFIT ANALYSIS)

The design storm for determining required sizes/capacities of CSO abatement facilities shall have a minimum recurrence interval of 3-months in accordance with regulatory agency requirements (Section 7.4). In concept, the capacity of CSO abatement facilities sized for this storm event would be exceeded no more than 4 times per year. However, this may not necessarily reflect the most cost-effective control level. Under certain situations, a less-frequent storm having a 6-month or 1-year recurrence interval may be warranted. Accordingly, the optimum design storm for CSO abatement alternatives was selected by performing a cost-benefit analysis. This analysis, performed in compliance with regulatory agency requirements



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COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-4

STORAGE-TREATMENT CONCEPTS

discussed in Section 1.0, is intended to identify the CSO control level that results in the greatest benefit for the least amount of dollars spent. It is commonly referred to as the "knee-of-the-curve" analysis. It entails the following tasks:

1. Apply the model EXTRAN (a component of the SWMM model) for several synthetic design storms. The number of synthetic storms selected depends on the minimum required to define the shape of the cost-benefit curve. For this study the 1 month, 3 month, 6 month and 12 month frequency storms - each 6 hours in duration were selected. The 1-month storm was included only to better define the shape of the curve.

2. From the results of the EXTRAN, determine required sizes and/or capacities of the selected CSO abatement facilities for each storm.

3. Estimate the present worth cost of the facilities for each design storm.

4. Apply the model STORM using the long-term rainfall record incorporating the storage volume and/or treatment rate provided by the CSO abatement facilities. Four STORM runs were required for this study, one for each size/capacity (representing each design storm).

5. From the results of STORM, determine CSO frequency (average number of overflows per year) and/or CSO volumes for each facility size/capacity.

6. Plot results and generate a curve which illustrates present worth costs for various levels of CSO mitigation.

A typical curve is shown on Figure 9-5. The optimum, or most cost-effective CSO control level occurs at the "knee" of the curve where the incremental reduction in CSO frequency begins to decrease with increasing costs. For this example, the optimum CSO control level, from an annual overflow volume standpoint, is 4 MG, which is the control level provided by a facility designed for a 3-month frequency storm.

9.4 CSO GROUP ALTERNATIVES

In Section 8.0, potential sites were identified for CSO storage and/or treatment facilities, and criteria was presented for their evaluation. Based on these criteria, the 31 sites were screened as to their

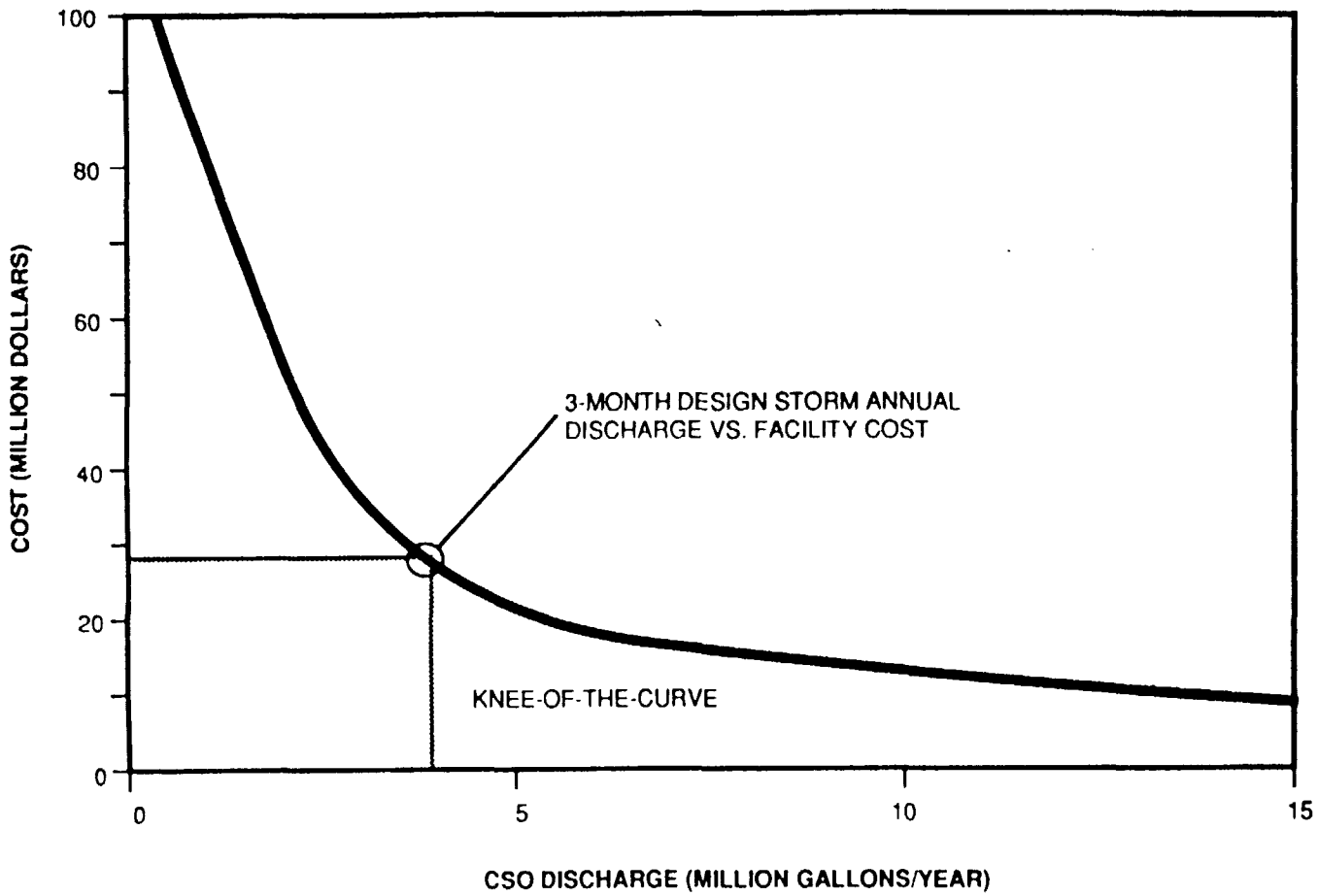

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FIGURE 9-5
TYPICAL COST-BENEFIT CURVE

applicability for each of the CSO groups. This screening process reduced the number of viable sites to 13 which were included in these detailed evaluations.

In Section 9.3.3, selected CSO abatement technologies were evaluated for their applicability to each CSO group. It was determined that not all technologies were viable for each group due to receiving water impacts, configuration of the sewer system, or system-wide capacity constraints.

The preceding analyses provide a framework of feasible siting and technology combinations for each CSO group from which CSO abatement alternatives can be identified and evaluated in detail. These evaluations include storage/treatment options for each group, as well as options involving the consolidation of groups into larger, more centralized storage/treatment facilities. This section will present the alternatives evaluated, and describe the major conveyance facilities required in conjunction with the storage/treatment facilities. In addition, the selected sites will be evaluated further in terms of their technical compatibility with the alternatives.

9.4.1 DESCRIPTION OF ALTERNATIVES

The following sections present brief descriptions of each alternative evaluated for each CSO Group. The detailed evaluation of these alternatives is presented in Section 9.4.2.

9.4.1.1 Group 1

Four major alternatives were evaluated for CSO Group 1.

1. Storage/Pumpback
2. Storage/Pumpback - Overflows Diverted to Inner Harbor
3. Sewer System Separation
4. Consolidation with Group 4 (see section 9.4.1.7).

In addition, in-line storage was evaluated in conjunction with Alternatives 1, 2 and 4.

Alternative 1 - Storage/Pumpback. Under this alternative, CSOs would be conveyed to a storage facility at Site 1, which is located between the Hurricane Barrier and Cove Road as shown on Figure 9-6. Required major facilities include:

- A storage facility with pumping station
- CSO consolidation pipelines in Cove Road
- A force main to convey stored CSOs to the interceptor/WWTP

As shown on Figure 9-6, the force main route differs for the two WWTP sites. For the Fort Rodman site, the force main would extend east in Cove Road to the main interceptor. For the Standard Times site, the force main would extend northeast in Cove Road, Water Street, Cove Street and North Front Street to the WWTP.

Alternative 2 - Storage/Pumpback-Overflows Diverted to the Inner Harbor. This alternative is similar to Alternative 1 except that the storage/pumpback facility would be smaller. In addition, a conveyance pipeline would be required to convey flows from events which exceed storage facility design storms away from Clarks Cove and to the lower priority Inner Harbor. The pipeline would extend east in Katherine Street and Blackmer Street to Front Street, then east in an existing drainage easement to the Inner Harbor. The required location of this conduit results in an area of about 80 acres where excess flows cannot be diverted. This area is generally located south of the conduit (i.e., between the conduit and the storage facility). CSOs from this area would continue to flow to the storage facility even when its capacity is reached. To prevent overflows to Clarks Cove under these extreme conditions, the pumpback system would be used to pump excess flows to the diversion conduit for transport to the Inner Harbor. This would require a force main from the storage facility to the diversion conduit, in addition to the pumpback force main. The facilities required for this alternative are shown on Figure 9-7.

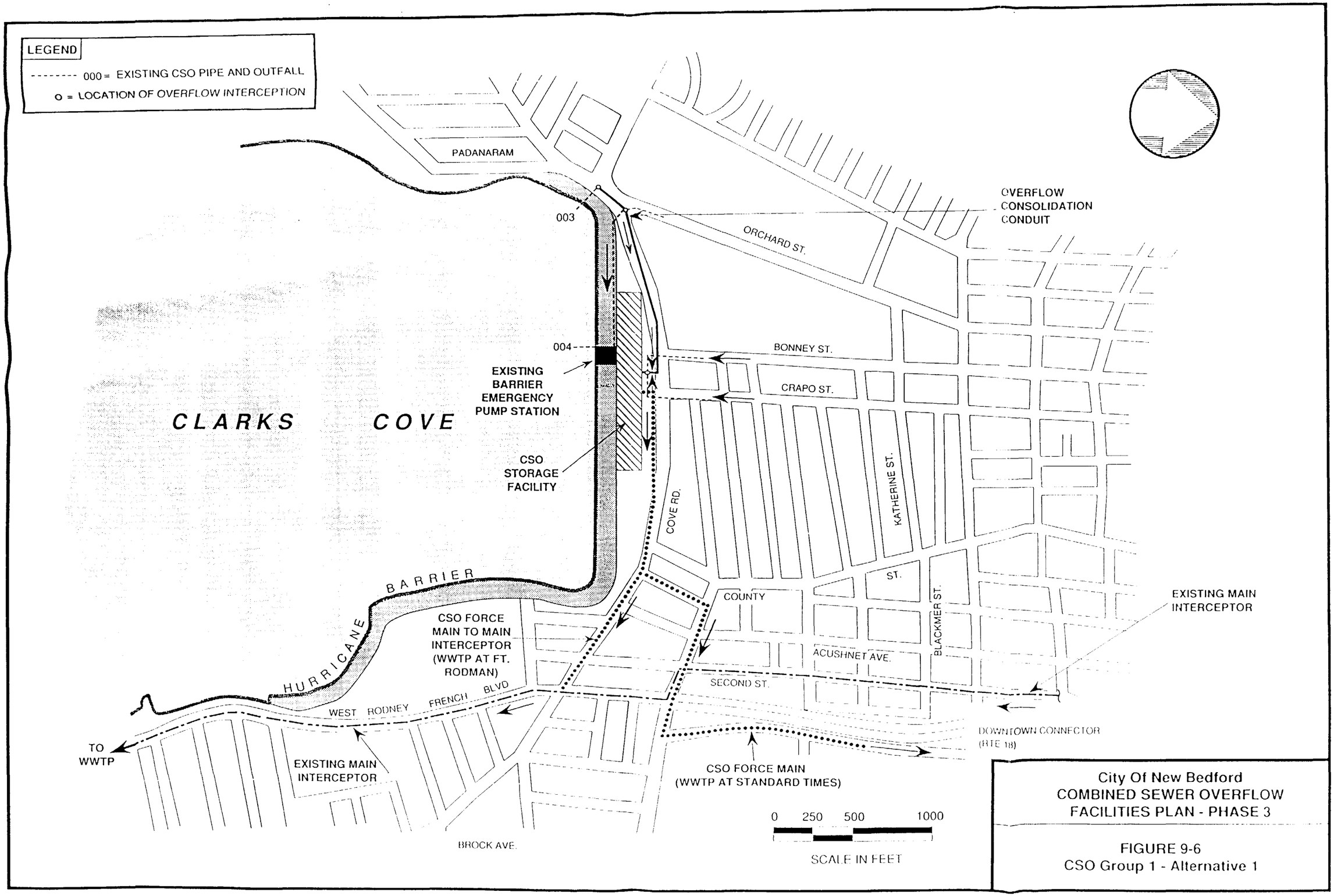

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FIGURE 9-6
CSO Group 1 - Alternative 1

Alternative 3 - System Separation. As noted in Section 9-3 this alternative will not meet water quality objectives for Clarks Cove, but is included for comparative purposes. The tributary area for each CSO group is shown on Figure 9-8. The area requiring separation in Group 1 is approximately 815 acres.

Alternative 4 - Consolidation with Group 4. This alternative is discussed in section 9.4.1.7.

In-Line Storage. Use of in-line storage in conjunction with Alternatives 1,2, and 4 requires that flow regulating devices be installed in the major collector sewers. The major collectors are the Grape Street, Liberty Street and Tripps Brook collectors and are shown on Figure 9-9.

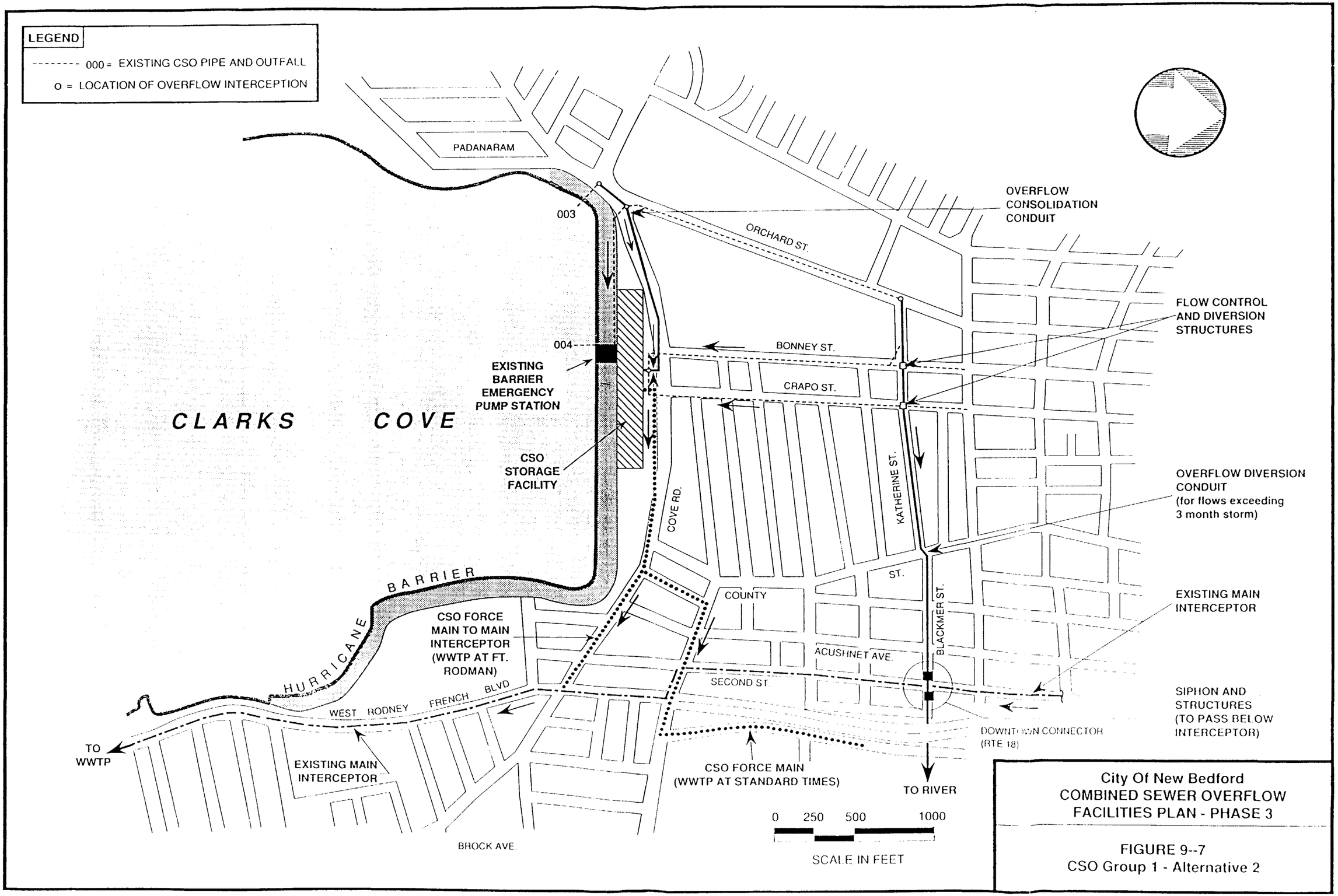
9.4.1.2 Group 2

Three alternatives were evaluated for CSO Group 2.

1. Storage/Pumpback
2. Sewer System Separation
3. Consolidation with Group 1 (See section 9.4.1.7)

Alternative 1 - Storage/Pumpback. In Section 8.0, Site 2 (Hazelwood Park) and Site 4 (Fort Rodman) were identified as the two preferred sites for CSO abatement facilities for Group 2. On the basis of technical feasibility, Site 2 is superior to Site 4, and was selected. The primary advantage of Site 2 is that the pipelines required to convey CSOs to Site 4 and the storage facility at Site 4 would be substantially deeper, more costly and more complex to construct than those required for Site 2. The depth of excavation has significant cost and constructibility implications due to the shallow water table and presence of rock in the area.

Under this alternative, CSOs would be conveyed south from David Street and north from Hudson Street to a storage/pumpback facility located at Hazelwood Park. Pumpbacked flow would discharge directly into the Main



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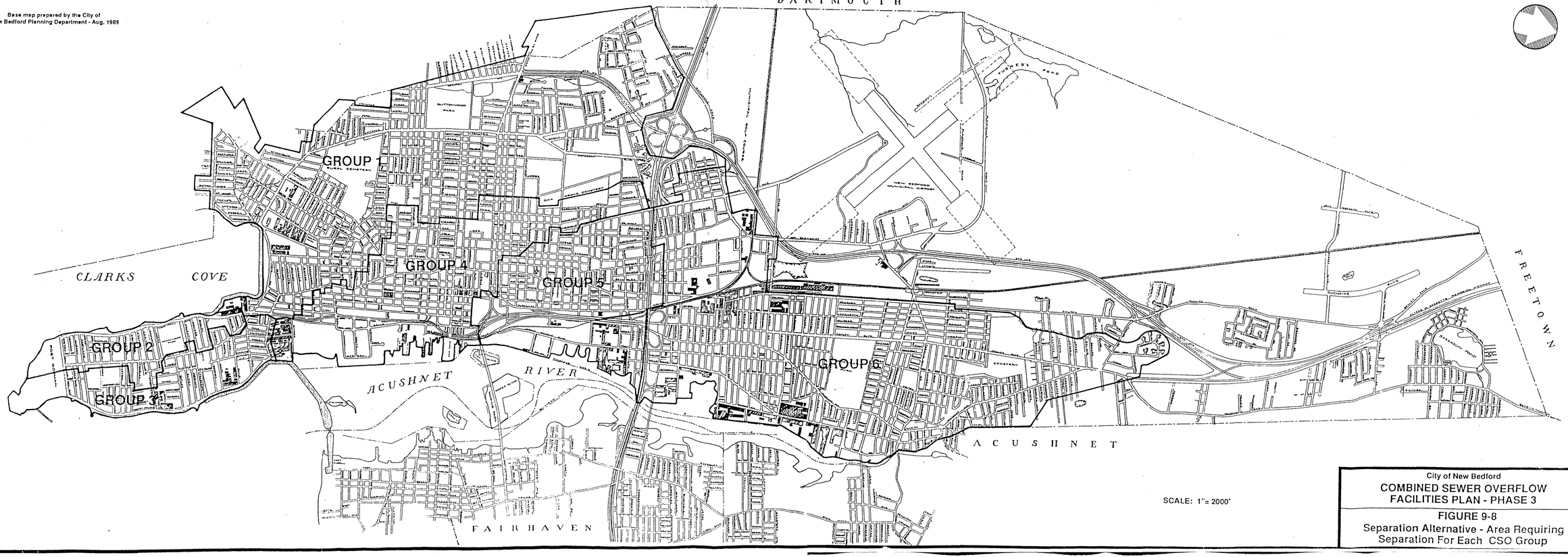
FACILITIES PLAN - PHASE 3

FIGURE 9--7
CSO Group 1 - Alternative 2

Base map prepared by the City of
New Bedford Planning Department - Aug. 1985



SCALE: 1"= 2000'

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COMBINED SEWER OVERFLOW
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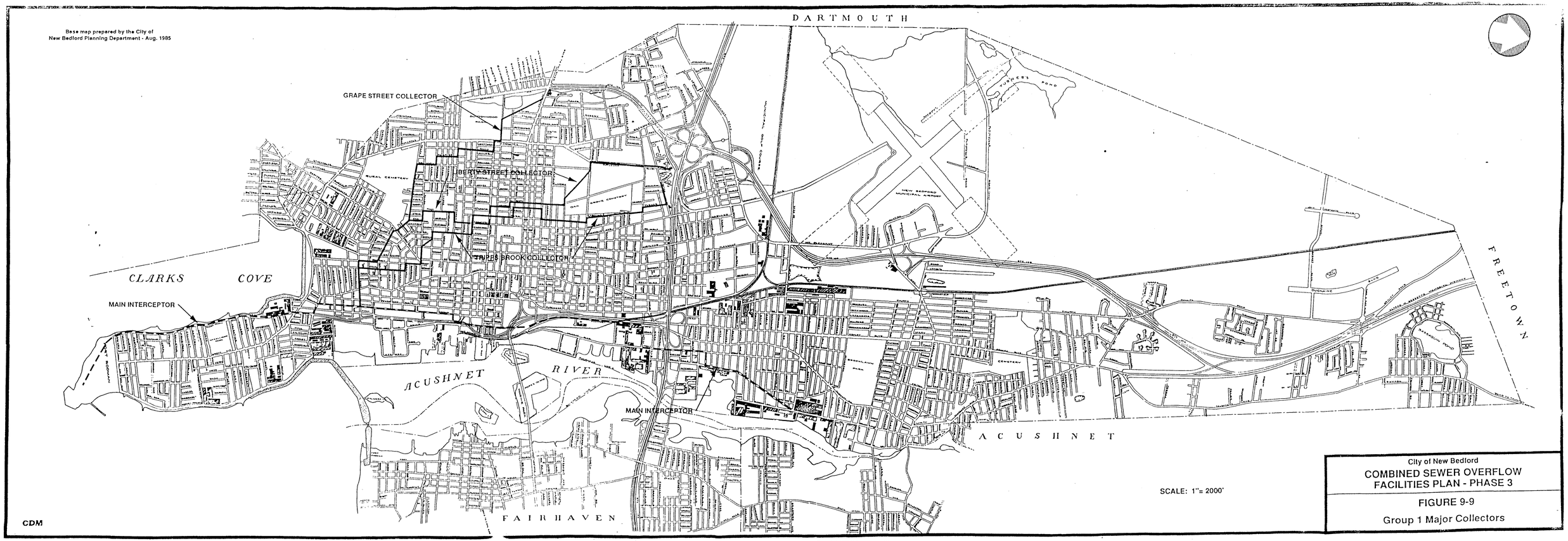
FIGURE 9-8
Separation Alternative - Area Requiring
Separation For Each CSO Group



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COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 9-9

Group 1 Major Collectors

Interceptor if the WWTP is located at Fort Rodman. If the plant is located at Standard Times, the force main would extend north to the Standard Times site. The facilities required for this alternative are shown on Figure 9-10. As noted in Section 9.3.3, the facility must be designed for a 12-month storm to meet water quality objectives for Clarks Cove.

Alternative 2 - Sewer System Separation. As was noted for Group 1, separation of Group 2 will not meet receiving water quality objectives for Clarks Cove, however it was evaluated for comparison with other alternatives. The area requiring separation for Group 2 contains approximately 128 acres, as indicated on Figure 9-8.

Alternative 3 - Consolidation with Group 1. This alternative is discussed in Section 9.4.1.7.

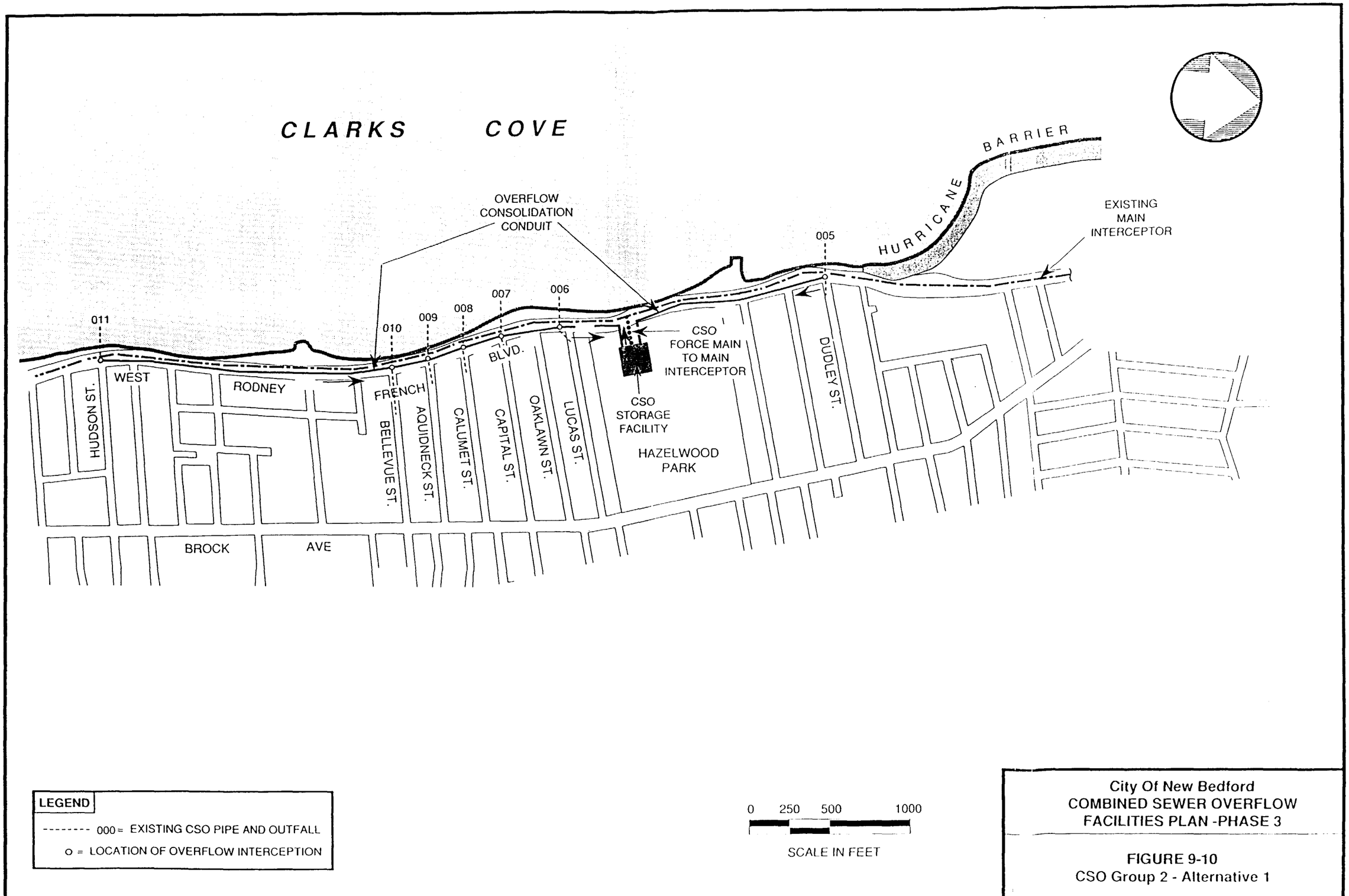
9.4.1.3 Group 3

Four alternatives were evaluated for CSO Group 3.

1. Storage/Pumpback
2. Sewer System Separation
3. Consolidation with Group 4 (See section 9.4.1.7)
4. Consolidation with Groups 1 and 4 (See section 9.4.1.7)

In Section 8.0, Sites 8 and 9 were selected as the preferred sites for CSO abatement facilities for Group 3. Since both sites are located on Frederick Street, the site selection has no significant impact on the piping required to intercept and convey CSOs to the facility. However, Site 8 is preferable because it is larger, and the excavation for the facility would be much less extensive than at Site 9. Consequently, Site 8 is selected on the basis of cost and constructibility.

Alternative 1 - Storage/Pumpback. Under this alternative, CSOs would be conveyed south from Gifford Street and North from Ricketson Street to the



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COMBINED SEWER OVERFLOW
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FIGURE 9-10
CSO Group 2 - Alternative 1

storage facility. Pump backed flow would be conveyed by force main either to the Main interceptor if the WWTP is located at the Fort Rodman site, or directly to the plant if it is located at the Standard Times site. The required facilities are shown on Figure 9-11.

Alternative 2 - Sewer System Separation. Under this alternative, no storage or conveyance facilities are required since the entire combined sewer system would be separated. The area requiring separation for Group 3 is shown on Figure 9-8 and contains approximately 102 acres.

Alternative 3 - Consolidation with Group 4. This alternative is discussed in Section 9.4.1.7.

Alternative 4 - Consolidation with Groups 1 and 4. This alternative is discussed in Section 9.4.1.7.

9.4.1.4 Group 4

Two alternatives were evaluated for Group 4.

1. Storage/Pumpback
2. Sewer System Separation

As noted in the preceding sections, several alternatives involving consolidation of Group 4 is discussed in Section 9.4.1.7.

In Section 8.0, Sites 13 and 31 were identified as the preferred sites for CSO abatement facilities in Group 4. Further evaluation reveals that Site 13 (Conway and Hassey Streets) is most preferable because it minimizes the depth of the facility and conveyance pipes. As noted in the discussion of site selection for Group 2, the depth of the facility and conveyance pipes impacts both the cost and complexity of construction. For these reasons, Site 13 was selected.

Alternative 1 - Storage/Pumpback. Under this alternative, CSOs would be conveyed south from CSO outlet pipe 034 in MacArthur Drive, and north from

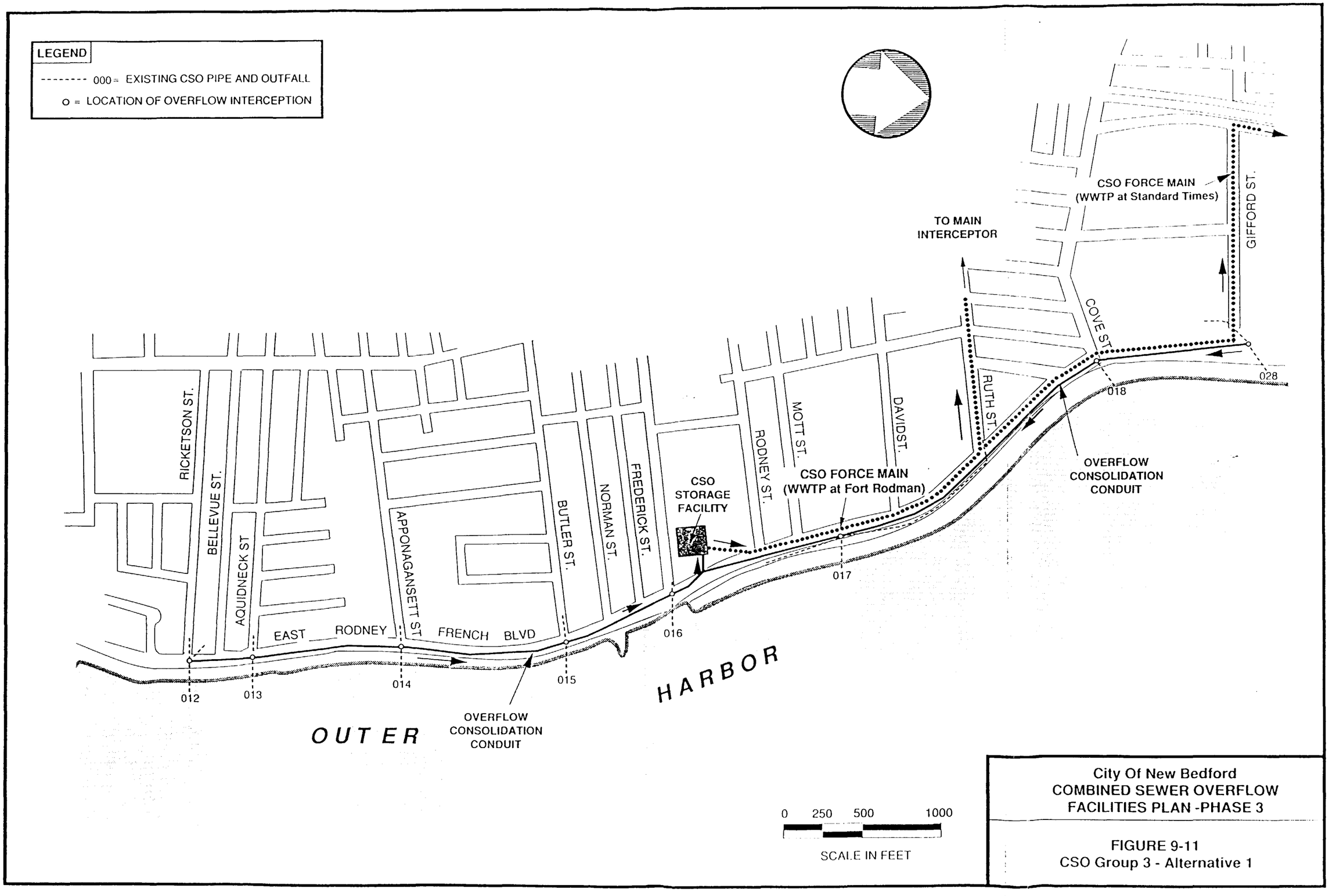

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FIGURE 9-11
CSO Group 3 - Alternative 1

South Street to a storage facility at Site 13. Pumpbacked flow would be conveyed either across the Downtown Connector to the Main interceptor if the treatment plant is at the Fort Rodman site, or directly to the plant if it is located at the Standard Times site. The facilities required for this alternative are shown in Figure 9-12.

Alternative 2 - Sewer System Separation. The area that would require separation for Group 4 contains approximately 533 acres (Figure 9-8).

9.4.1.5 Group 5

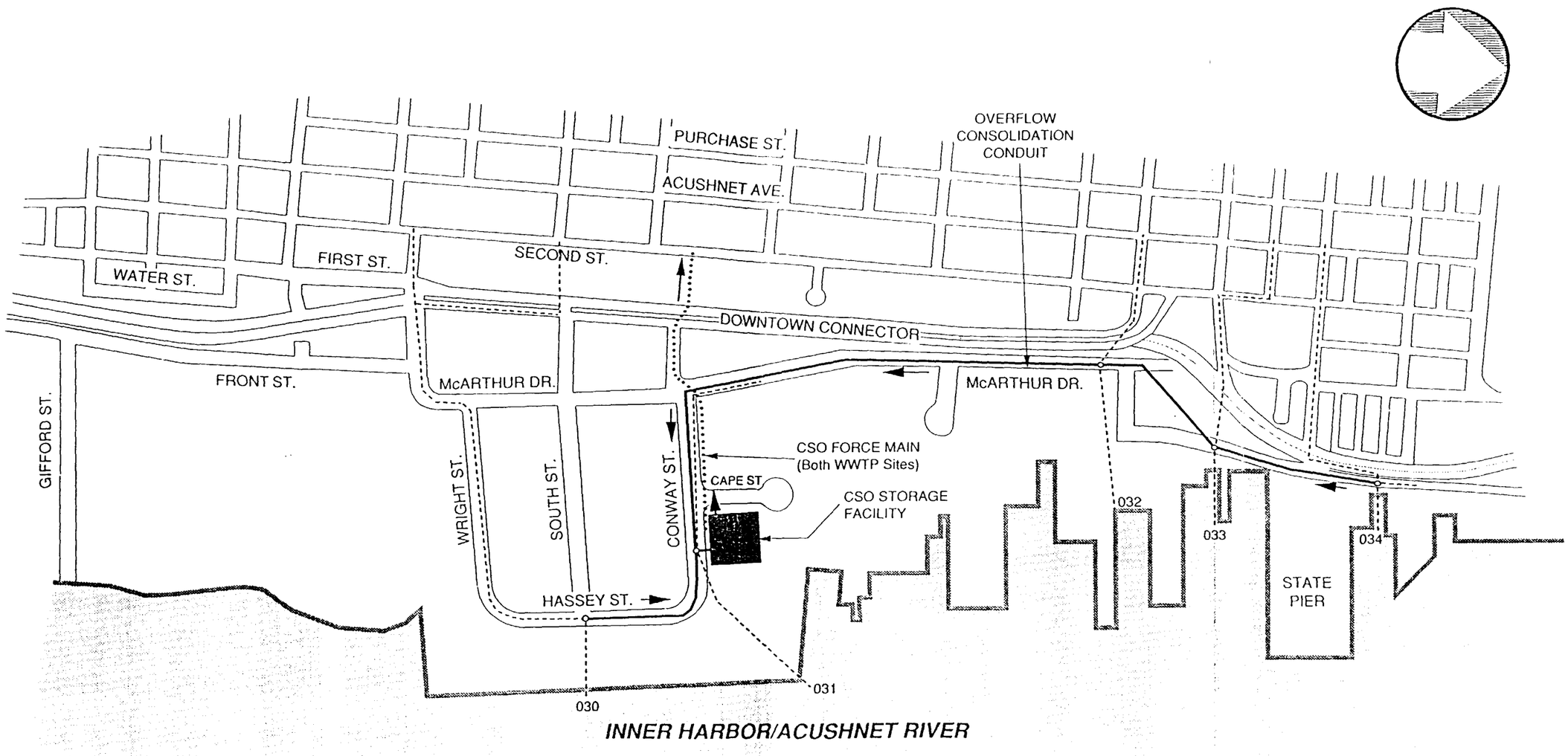
In Section 9.3.3.2, it was shown that storage/treatment alternatives for CSO Groups 5 and 6 are limited by the conveyance and treatment capacity of the interceptor system and wastewater treatment plant. Information provided in Table 9-3 indicates that storage/pumpback is not feasible for either Group 5 or 6 unless centralized CSO treatment is provided at the new secondary WWTP. Consequently, storage for Group 5 was evaluated only in conjunction with the system-wide CSO abatement alternatives discussed in Section 9.5.

Three major alternatives were evaluated for Group 5.

1. Treatment
2. Separation
3. Consolidation with Group 6 (See section 9.4.1.7)

In addition to these alternatives, the potential for use of available in-line storage was evaluated in conjunction with Alternatives 1 and 3.

In Section 8.0, Sites 17, 19 and 20 were selected as the preferred sites for CSO abatement facilities for Group 5. For technical reasons, Sites 19 and 20 are preferable to Site 17 because the depth of the conveyance sewers and the treatment facility would be shallower at those sites. Site 19 was selected over Site 20 because it has a greater buffer from its surroundings (Site 20 is adjacent to an existing building). In addition, Site 19 is the



LEGEND

-------- 000 = EXISTING CSO PIPE AND OUTFALL

O = LOCATION OF OVERFLOW INTERCEPTION

0 250 500 1000

SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-12
CSO Group 4 - Alternative 1

preferable site on the basis of the screening criteria established in Section 8.0.

Alternative 1 - Treatment. Under this alternative, flow would be conveyed south from Washburn Street in Belleville Avenue, Kilburn Street and North Front Street to the treatment facility, and north on Herman Melville Boulevard from CSO pipe 035 to the facility. Treated effluent would be discharged directly to the Inner Harbor. The facilities required for this alternative are shown on Figure 9-13.

Alternative 2 - Sewer System Separation. The area requiring separation for Group 5 contains approximately 396 acres (Figure 9-8).

Alternative 3 - Consolidation with Group 6. This alternative is discussed in section 9.4.1.7.

In-Line Storage. Use of in-line storage in conjunction with Alternatives 1 and 3 requires that a flow regulating device be installed in the Willis Street Collector, as shown on Figure 9-14.

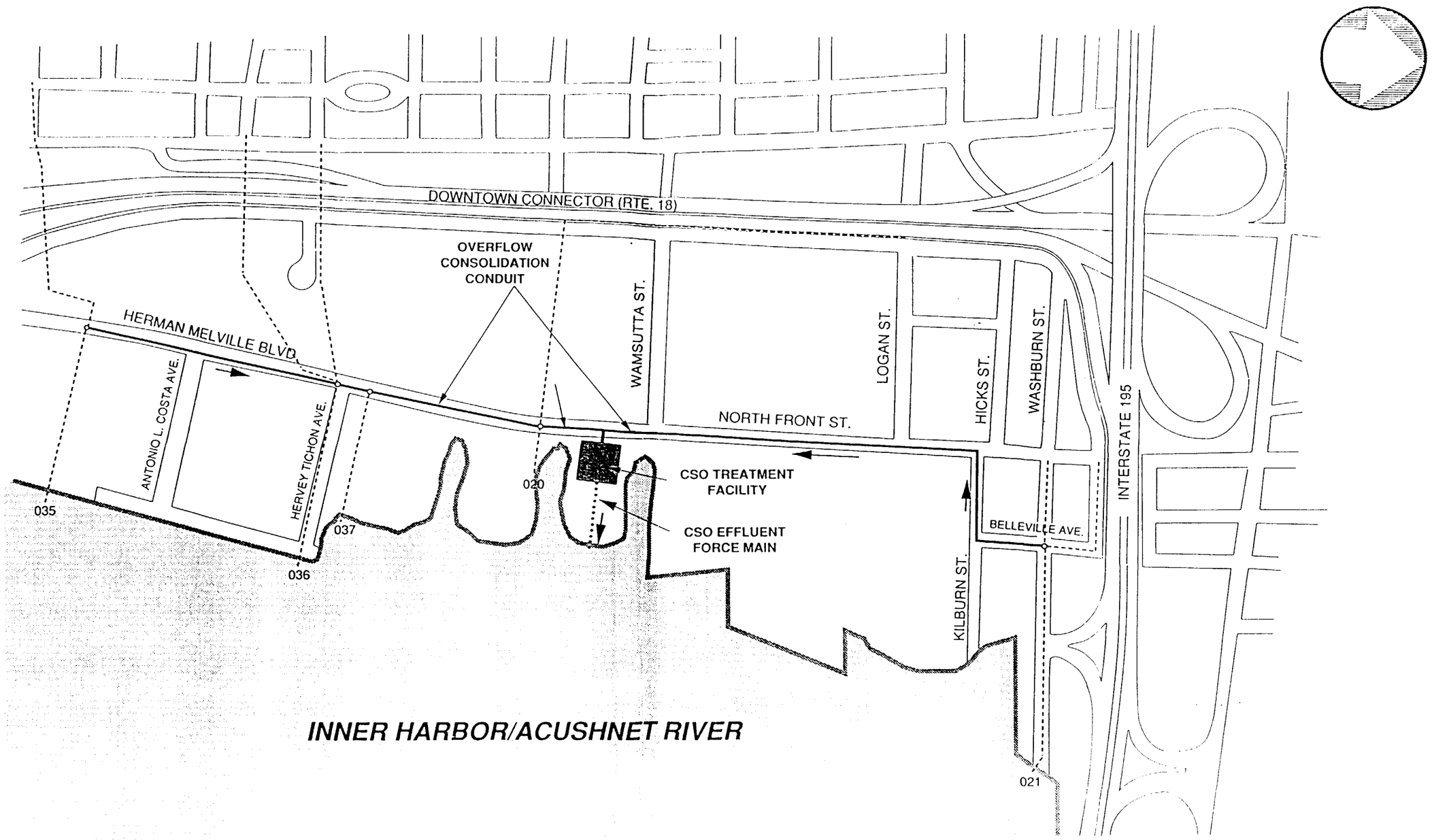
9.4.1.6 Group 6

Due to the capacity constraints of the WWTP, Group 6 is subject to the same limitations as Group 5, (i.e., storage is not a viable alternative.) Consequently, all alternatives for Group 6, except system separation, include CSO treatment.

Three alternatives were evaluated for CSO Group 6.

1. Treatment
2. System Separation
3. Consolidation with Group 5 (See section 9.4.1.7)

In addition to the above, partial separation of the system and in-line storage were evaluated in conjunction with Alternatives 1 and 3.



LEGEND

-------- 000 = EXISTING CSO PIPE AND OUTFALL

O = LOCATION OF OVERFLOW INTERCEPTION

0 250 500 1000

SCALE IN FEET

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COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-13
CSO Group 5- Alternative 1

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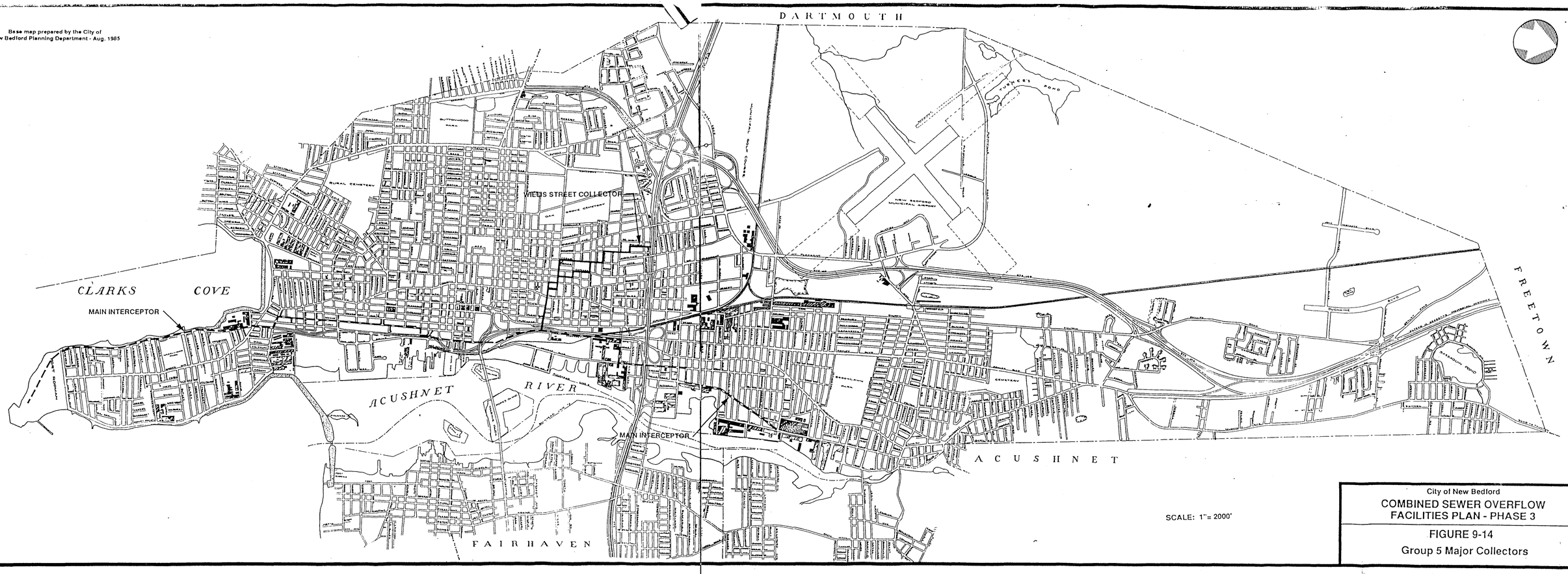


CDM

City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-14
Group 5 Major Collectors

In Section 8.0, Sites 25 and 26 were selected as the preferred sites for CSO Group 6. Further evaluation reveals that Site 26 is preferable because the facility and conveyance pipelines are shallower and less costly than if facilities were located at Site 25.

Alternative 1 - Treatment. Under this alternative, flow would be conveyed north from Coggeshall Street in Mitchell Street and Belleville Avenue to Coffin Avenue and north in Riverside Avenue to the treatment facility. Flow would be conveyed south either from Mazeppa Street (no partial separation) or Howard Avenue (partial separation) south to the facility. Treated effluent from the facility would be conveyed across Belleville Avenue and discharged into the Acushnet River. The required facilities are illustrated on Figure 9-15.

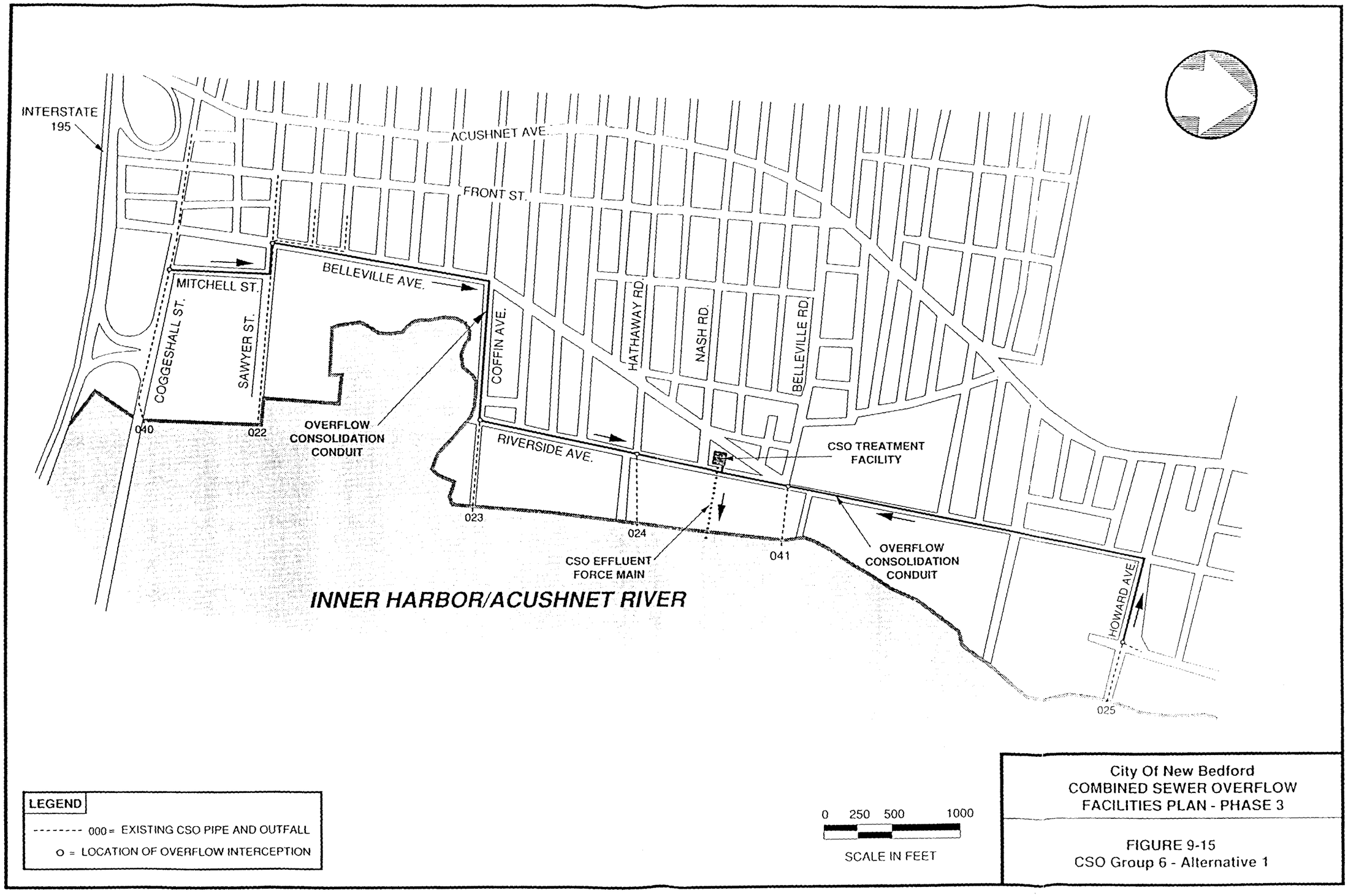
Alternative 2 - Sewer System Separation. The area requiring separation for Group 6 contains approximately 695 acres (Figure 9-8).

In-Line Storage. Use of in-line storage in conjunction with Alternatives 1 and 2 requires that flow regulating devices be installed in the Belleville Avenue, Church Street and Sawyer Street collectors, as shown on Figure 9-16.

9.4.1.7 Consolidated CSO Group Alternatives

As mentioned throughout preceding discussions, several alternatives that involve consolidation of CSO groups were developed and evaluated. Logical combinations of groups were selected based on system hydraulics, relative location, site availability and other engineering and construction related considerations. These alternatives are discussed in the following portions of this section and are illustrated on Figures 9-17, 9-18 and 9-19. The evaluation of these alternatives, as well as all individual group options, is presented in Section 9.4.3.

Groups 1 and 4. Under this alternative, overflows from Group 1 would be intercepted at Orchard Street, Bonney Street and Crapo Street and conveyed east in Katherine Street and Blackmer Street to a consolidated facility



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-15
CSO Group 6 - Alternative 1

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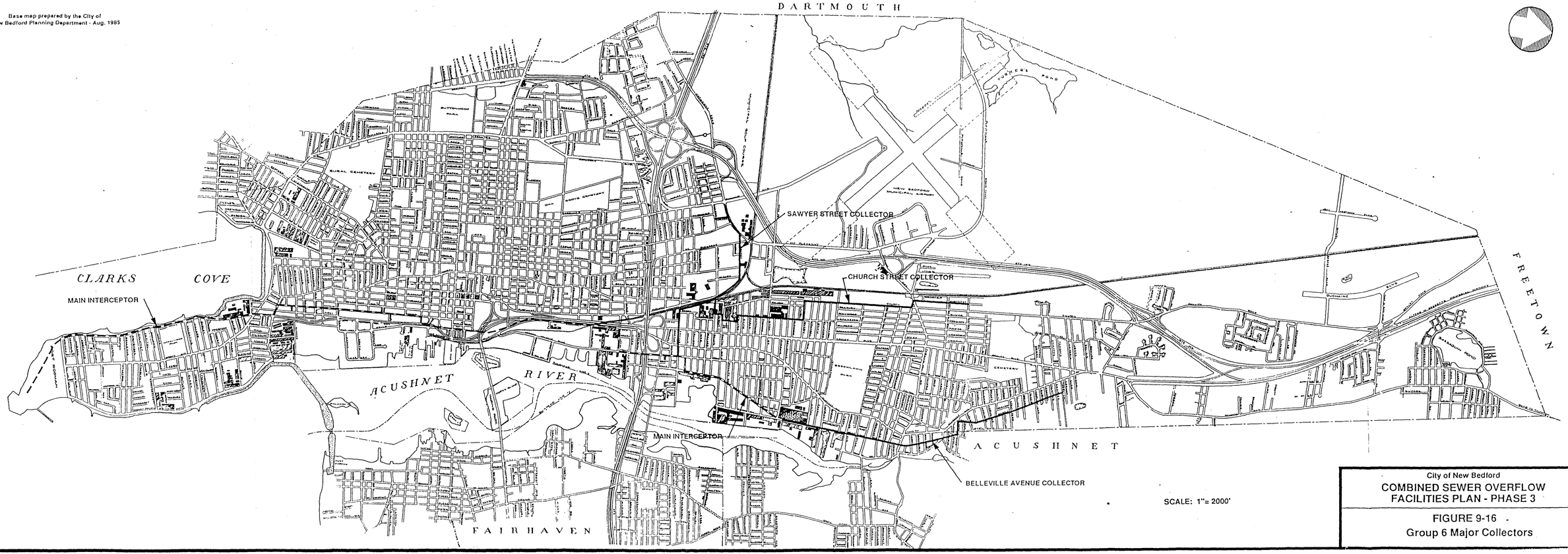
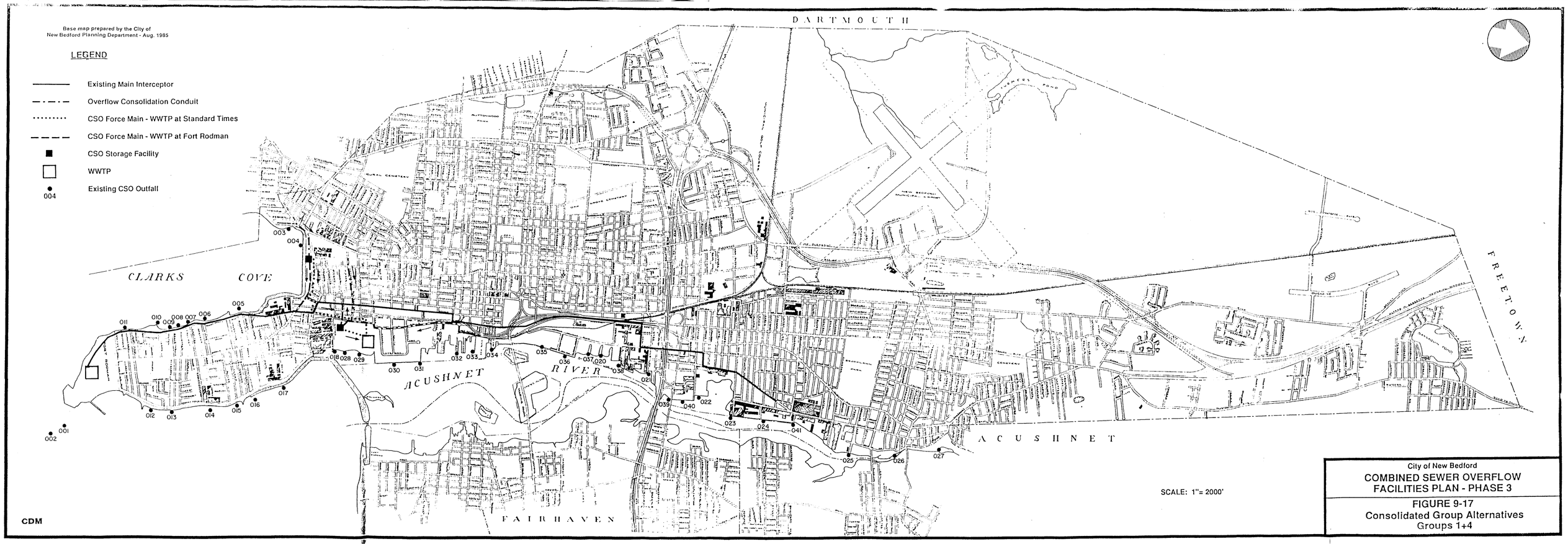


SCALE: 1"= 2000'

CDM

City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-16
Group 6 Major Collectors


Base map prepared by the City of
New Bedford Planning Department - Aug. 1985
LEGEND
Existing Main Interceptor
Overflow Consolidation Conduit
CSO Force Main - WWTP at Standard Times
CSO Force Main - WWTP at Fort Rodman
CSO Storage Facility
WWTP
Existing CSO Outfall
004
DARTMOUTH
CLARKS COVE
ACUSHNET RIVER
ACUSHNET
FAIRHAVEN
FREETOWN
001
002
003
004
005
006
007
008
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SCALE: 1"= 2000'
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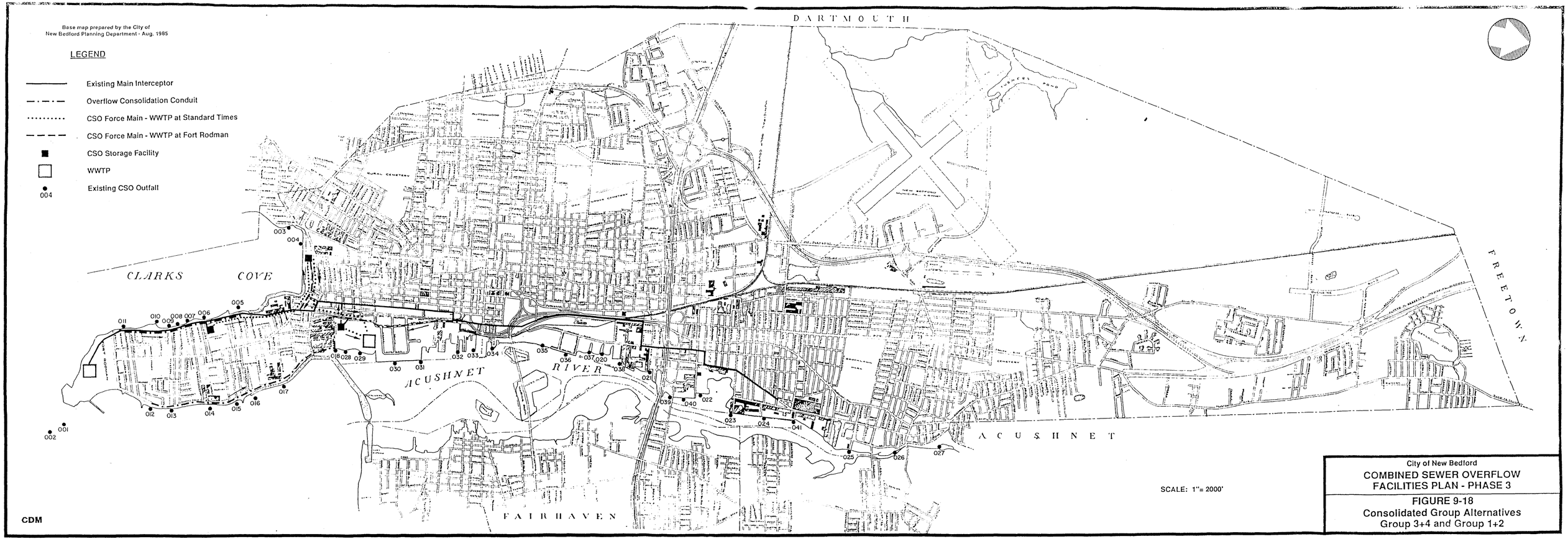
City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3
FIGURE 9-17
Consolidated Group Alternatives
Groups 1+4


Base map prepared by the City of
New Bedford Planning Department - Aug. 1985
LEGEND
Existing Main Interceptor
Overflow Consolidation Conduit
CSO Force Main - WWTP at Standard Times
CSO Force Main - WWTP at Fort Rodman
CSO Storage Facility
WWTP
Existing CSO Outfall
004
DARTMOUTH
FREETOWN
FAIRHAVEN
CLARKS COVE
ACUSHNET RIVER
ACUSHNET
001
002
003
004
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SCALE: 1"= 2000'
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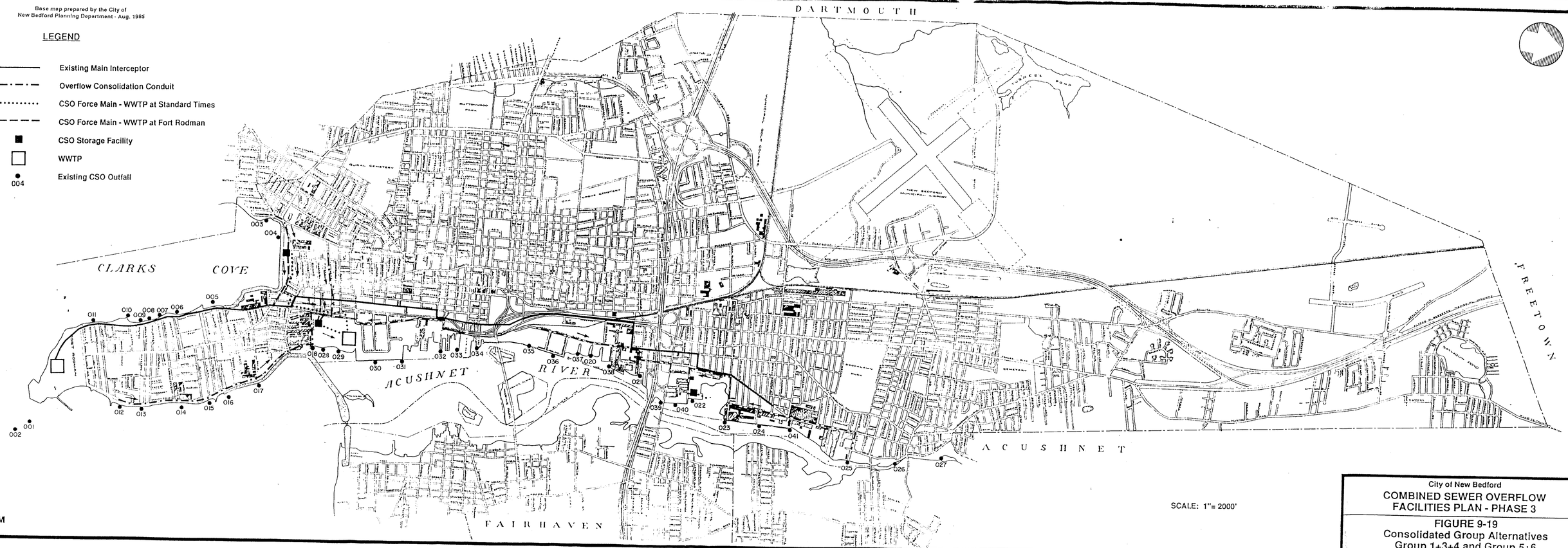
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3
FIGURE 9-18
Consolidated Group Alternatives
Group 3+4 and Group 1+2

Base map prepared by the City of
New Bedford Planning Department - Aug. 1985

LEGEND

- Existing Main Interceptor
- Overflow Consolidation Conduit
- CSO Force Main - WWTP at Standard Times
- CSO Force Main - WWTP at Fort Rodman
- CSO Storage Facility
- WWTP
- 004 Existing CSO Outfall



SCALE: 1"= 2000'

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City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-19
Consolidated Group Alternatives
Group 1+3+4 and Group 5+6

located at the Standard Times site. This would be a storage/pumpback facility since CSO treatment and effluent discharge from this location would impact Outer Harbor receiving water uses. Because the area south of the regulators in Rivet Street would not be conveyed to this consolidated facility, a separate storage/pumpback facility would be provided for CSOs from this area. As noted in Section 9.3, water quality objectives in Clarks Cove will not be met by separation. Consequently, this is not a feasible option for this area.

Also under this alternative, CSOs from Group 4 would be intercepted in MacArthur Drive and Front Street, and conveyed south to the facility at Standard Times. Depending on the location of the new wastewater treatment facility, the volume of this consolidated CSO facility would either be pumped to the existing main interceptor for transport to the new treatment facility if located at Fort Rodman, or pumped directly to the headworks of the treatment facility if it is located at the Standard Times site.

Groups 2 and 1. Under this alternative, CSOs from Group 2 would be intercepted and conveyed north (CSOs 006 to 011) and south (CSO 005) in West Rodney French Boulevard to a storage/pumpback facility located at the Hazelwood Park site. Pumpback from this facility would be conveyed north to a second storage/pumpback facility located in an area adjacent to the hurricane barrier on the south side of Cove Road. Depending on the location of the new wastewater treatment plant, pumpback from this second storage facility will either enter the existing main interceptor for transport to the new treatment facility if located at Fort Rodman, or to be conveyed directly to the headworks of the treatment facility if it is located at the Standard Times site.

Groups 3 and 4. Under this option, CSOs from Group 3 would be intercepted and conveyed north in East Rodney French Boulevard, along an existing drain easement and west in Gifford Street to a consolidated storage/pumpback facility for Groups 3 and 4 located at the Standard Times site. Overflows from Group 4 would also be intercepted and conveyed south in MacArthur Drive and Front Street to the facility. Depending on the location of the new wastewater treatment plant, stored CSOs will either be pumped to the

existing main interceptor for conveyance to the new treatment facility located at Fort Rodman, or pumped to the headworks of the new treatment plant if it is located at Standard Times.

Groups 1,3 and 4

For this alternative, overflows would be intercepted and conveyed north in East Rodney French Boulevard (Group 3), along an existing drain easement and west in Gifford Street to a consolidated storage/pumpback facility for Groups 1,3 and 4 located at the Standard Times site. Overflows would also be intercepted and conveyed south in MacArthur Drive and Front Street (Group 4) as well as at Orchard Street, Bonney Street and Crapo Street (Group 1) and conveyed east in Katherine Street and Blackmer Street to the facility. As mentioned previously, the area south of the regulators in Rivet Street would not be conveyed to this facility. Accordingly, a separate storage/pumpback facility will be required under this arrangement (as discussed for Groups 1 and 4). Pumpback to the system would occur as previously explained for the other options where the consolidated facility is at Standard Times.

Groups 5 and 6

Under this alternative, overflows from Group 6 would be intercepted and conveyed west in Howard Avenue, south in Belleville Avenue and Riverside Avenue, west in Coffin Avenue, and east in Sawyer Street to a consolidated treatment facility for Groups 5 and 6 located on Site 24 at the end of Sawyer Street. Overflows from Group 5 would also be intercepted and conveyed north in Herman Melville Boulevard, east in Kilburn Street, north in Belleville Avenue, east in Coggeshall Street and north in Mitchell Street where a connection with overflows from the north occurs at the intersection of Mitchell Street and Sawyer Street. Treated effluent from the facility would discharge directly to the Inner Harbor. As noted in Section 9.4.1.6, partial separation of Group 6 will be evaluated under this alternative.

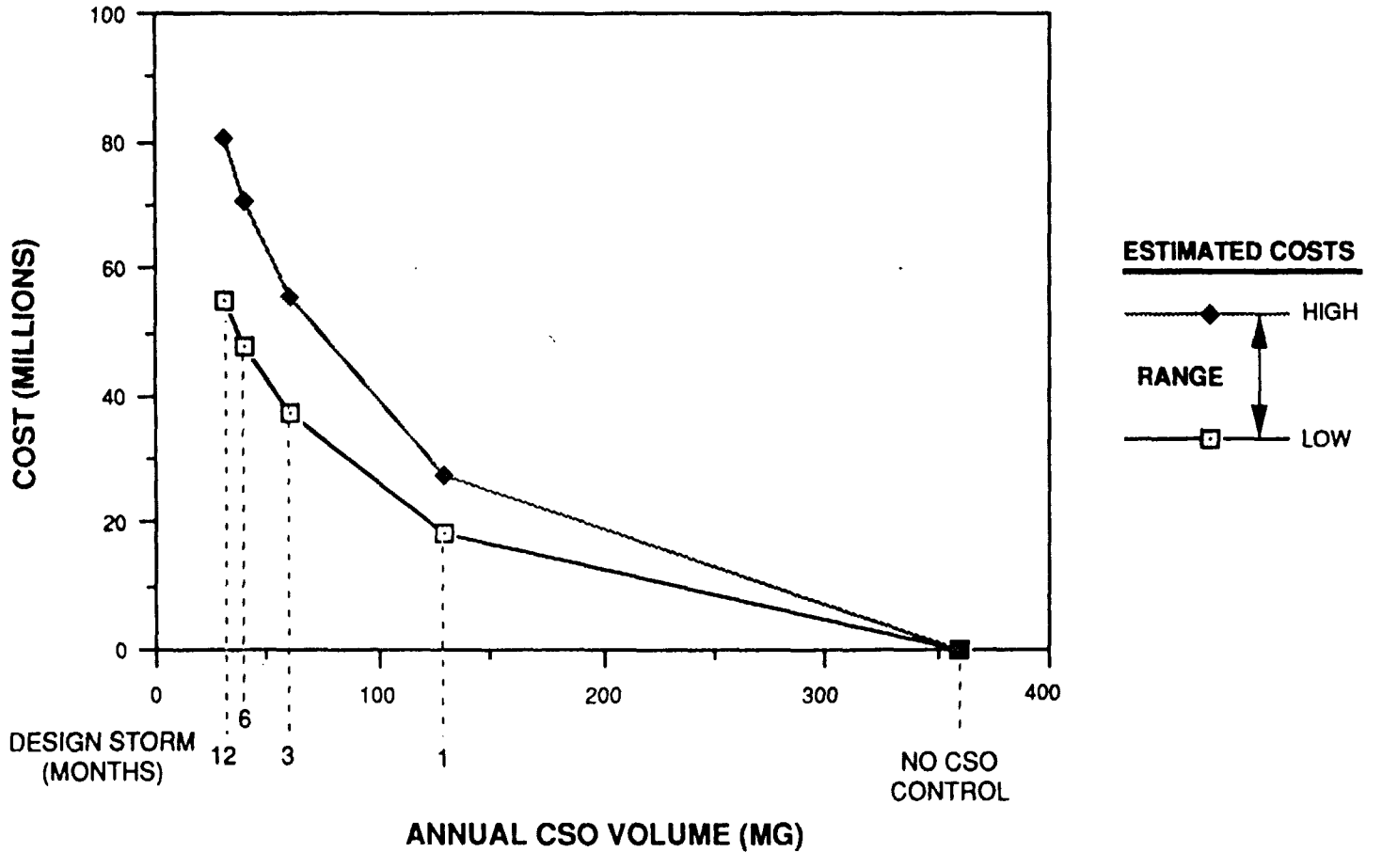
9.4.2 DESIGN STORM SELECTION

Estimated costs were developed for the group CSO abatement alternatives for each design storm to determine the most cost effective level of CSO control. Cost curves presented in Section 9.3.4 were used for this analysis. The curves are based on average construction conditions and assume maximum structure depths of thirty feet.

Because site conditions in New Bedford vary and may significantly impact actual construction costs, alternative costs were also estimated assuming a structure depth of forty feet and more complex construction conditions. The higher costs are often associated with consolidated storage facilities designed for large (low frequency) storm events because site limitations necessitate deeper structures to provide the required storage volume. This procedure resulted in upper and lower limits of estimated costs for each alternative, to assure that the resulting range of costs are as representative as possible.

Figures 9-20 through 9-25 are curves that relate estimated cost to CSO discharge for five levels of CSO control for each of the six groupings. The levels of CSO control evaluated are the 12-, 6-, 3-, and 1-month design storms, as well as no control (i.e., baseline conditions). In all cases, it is apparent that the most cost-effective control level or "knee" of each curve is between a 1-month and 3-month frequency storm. Consequently, detailed evaluations and cost analyses are warranted for the 3-month storm event for each group alternative, since this is the minimum level of CSO control required based on CDM's interpretation of Massachusetts DEP Interim CSO policy.

This cost-effective level of control is appropriate for CSO groups 3,4,5 and 6. However, a higher level of control is warranted for Groups 1 and 2 based on receiving water impacts. As stated numerous times and as discussed in Section 9.2.4, Clarks Cove is of primary concern since it has the highest use potential from recreational and shellfish harvesting compared to the other two receiving water bodies. It has the highest value to the City as compared with the Outer Harbor and Inner Harbor. A



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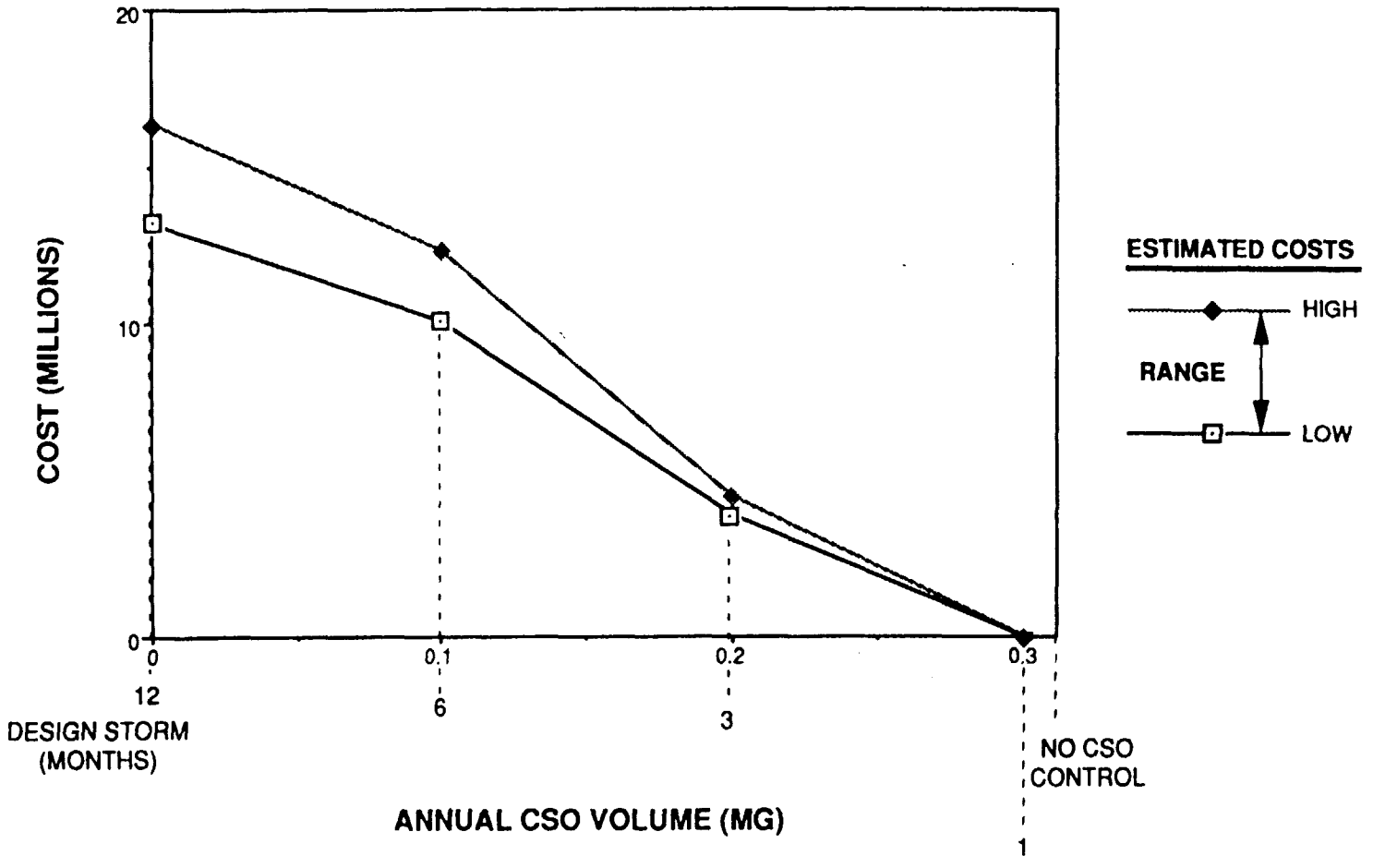
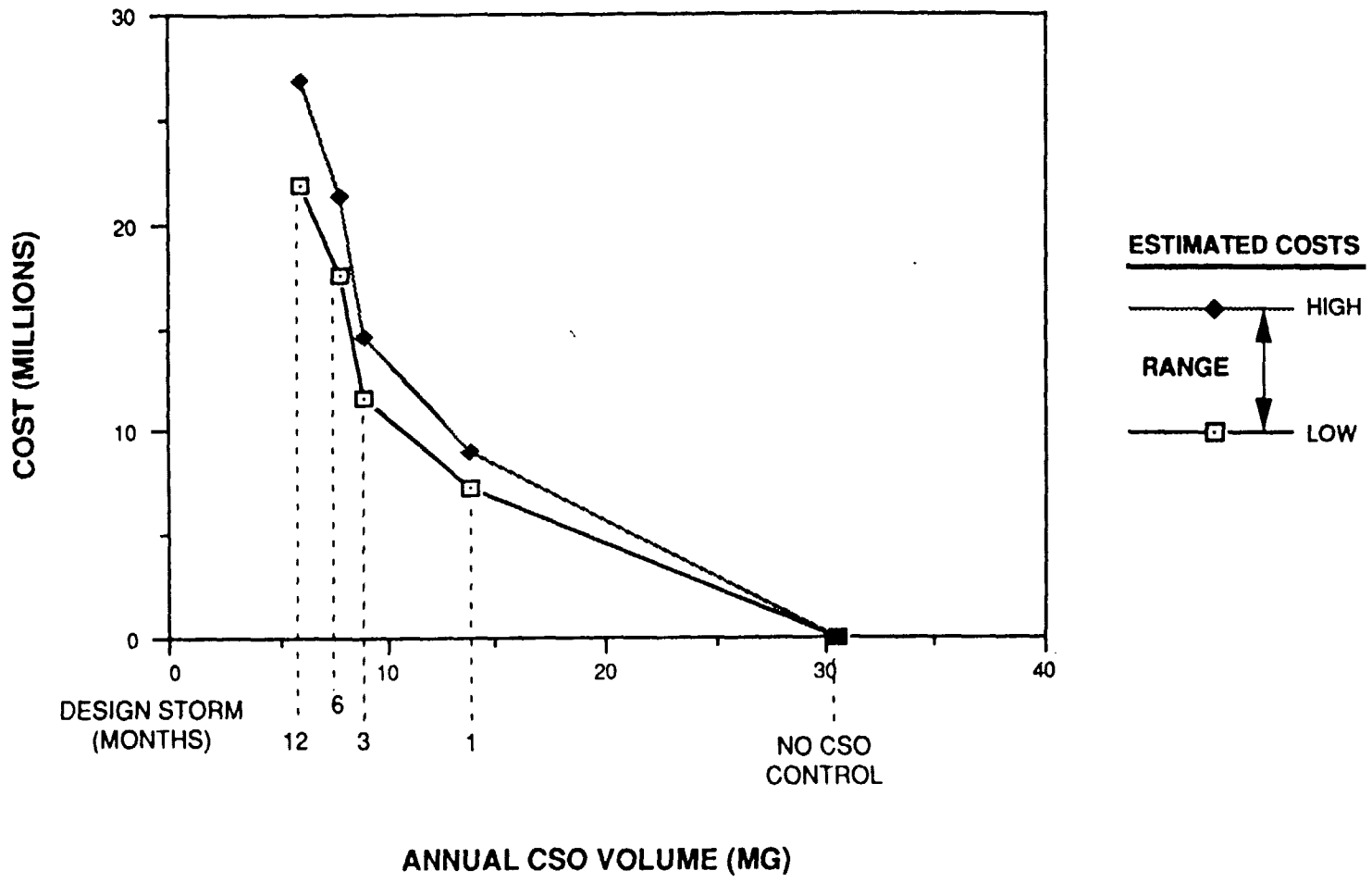
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-20
COST-BENEFIT CURVES
CSO GROUP 1



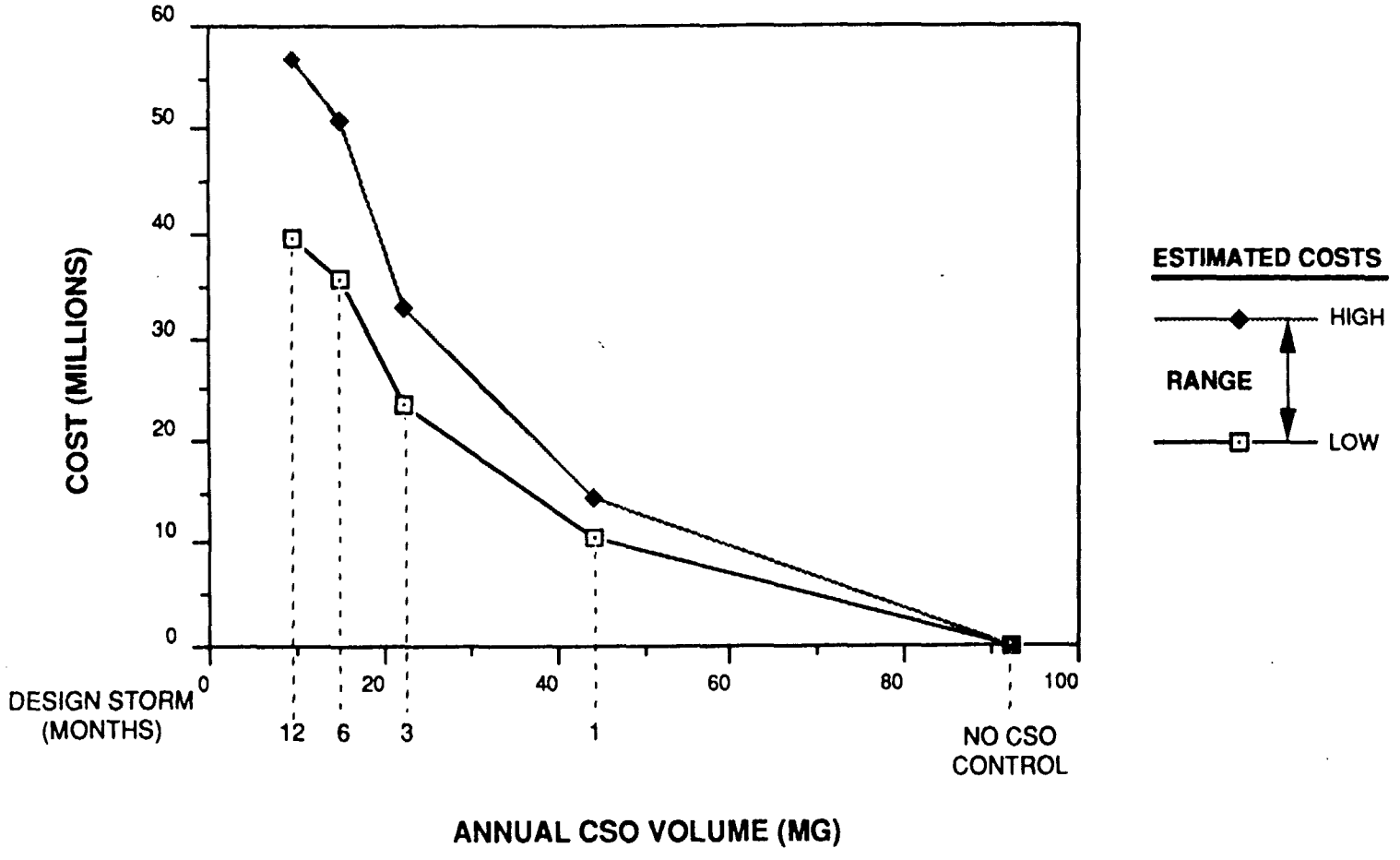
City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-21
COST-BENEFIT CURVES
CSO GROUP 2



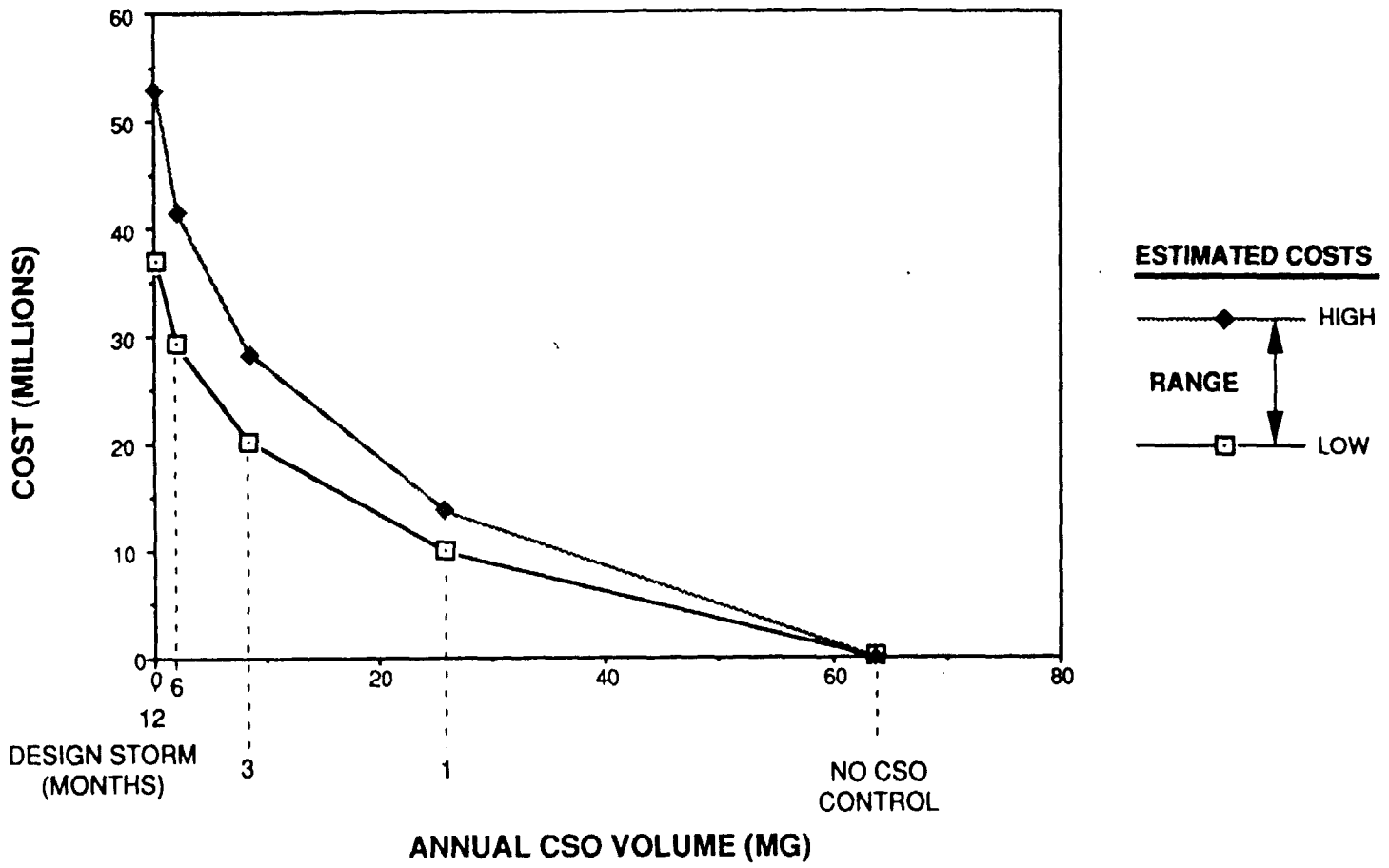
City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-22
COST-BENEFIT CURVES
CSO GROUP 3



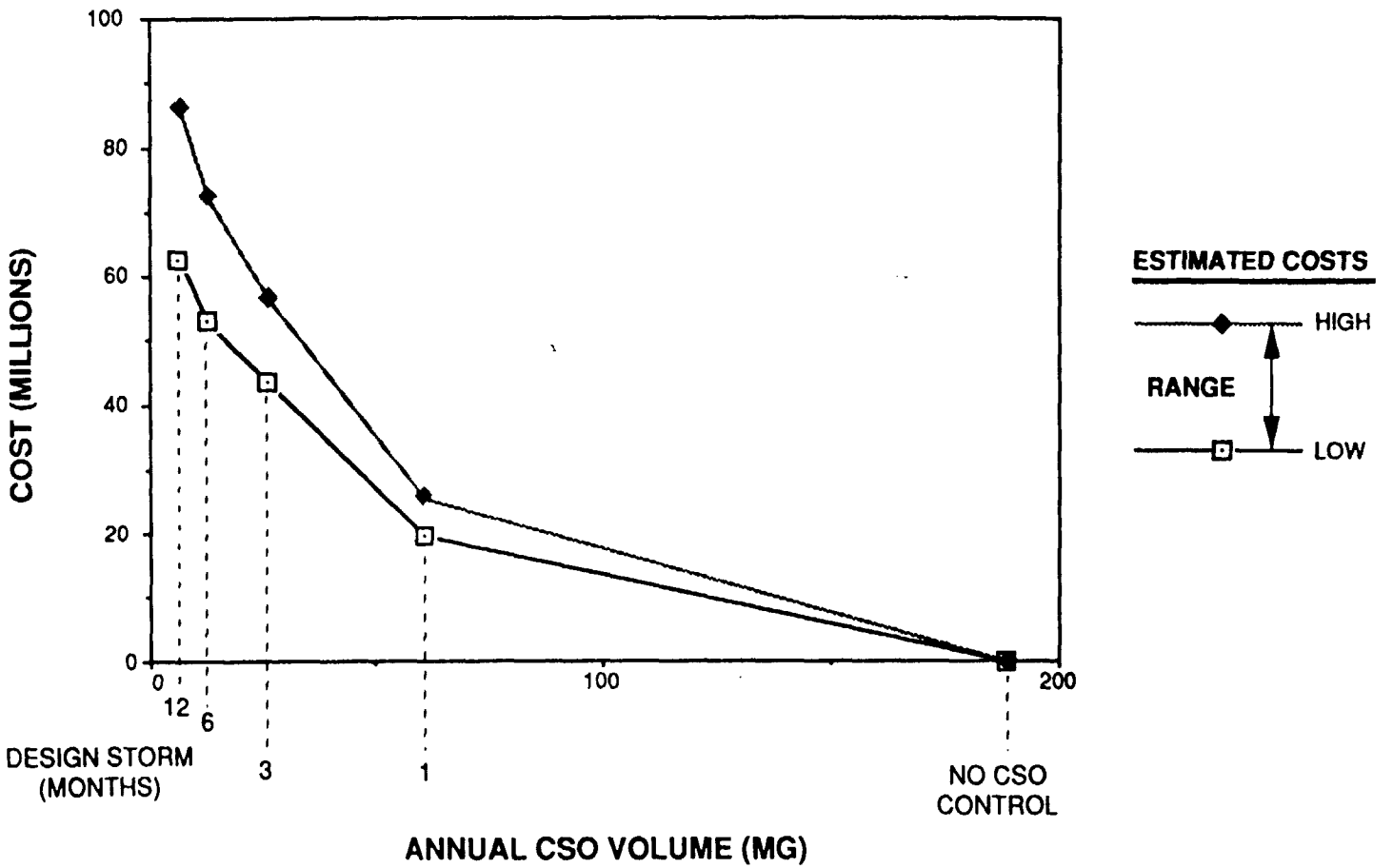
City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-23
COST-BENEFIT CURVES
CSO GROUP 4



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-24
COST-BENEFIT CURVES
CSO GROUP 5



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-25
COST-BENEFIT CURVES
CSO GROUP 6

higher level of CSO control, beyond the 3-month storm "cost-effective" level, is required to assure maximum restoration of impaired uses in this vital area. Accordingly, the minimum level of control selected for Groups 1 and 2 is the 12-month frequency storm. The feasibility of complete CSO elimination (other than separation - see Section 9.3.3.1) was also assessed.

9.4.3 EVALUATION OF GROUP ALTERNATIVES

In this section, each of the alternatives identified in Section 9.4.1 is evaluated on the basis of the criteria established in Section 9.2. Table 9-4 summarizes the results. All required major facilities and estimated costs to provide storage or treatment for CSOs for each group and each group consolidation alternative is presented. Estimated costs for complete separation are also shown (for comparison purposes only for Groups 1 through 4).

Facility sizing (design storm) is based on the criteria established in Section 9.4.2. All facilities except those discharging to Clarks Cove are designed to treat or store CSO's up to a 3-month storm. The facilities which discharge to Clarks Cove are sized to accommodate up to a 12-month storm. Complete elimination of CSOs to Clarks Cove was also evaluated for applicable alternatives:

In the following sections, sizes, capacities and estimated costs for the facilities associated with each alternative are described. Individual CSO Groups are presented first, followed by the alternatives employing consolidation of CSO Groups. The descriptions are based on the WWTP being located at Fort Rodman. Impacts of the plant being located at Standard Times are discussed at the end of each description.

9.4.3.1 Group 1

Alternative 1 - Storage/Pumpback (Figure 9-6). Under this alternative, the storage facility would be designed for a 12-month frequency storm to meet water quality objectives for Clarks Cove, and would require a volume of

TABLE 9-4

ESTIMATED COSTS[1] OF CSO GROUP ALTERNATIVES

Individual Group Alternatives[2]			Capital[3]					
Group	Alternative	Facility (capacity)	Storage/Treatment	Piping[5]	Total	Operation & Maintenance[4]	Total[3]	Separation[3]
1	1	Storage (33.1 MG)	76.9	1.1	78.0	2.8	80.8	42.4
1	2	Storage (19.9 MG)	50.4	13.3	63.7	1.7	65.4	42.4
2	1	Storage (1.7 MG)	11.3	4.0	15.3	0.3	15.6	6.7
3	1	Storage (1.5 MG)	10.6	4.7	15.3	0.3	15.6	5.3
4	1	Storage (7.6 MG)	26.8	5.0	31.8	0.9	32.7	27.7
5	1	Treatment (34 mgd)	15.9	4.5	20.4	0.7	21.1	20.6
6	1	Treatment (54 mgd)	21.0	15.0	36.0	1.2	37.2	36.2
Consolidated Group Alternatives[2]								
Groups		Facility (capacity)						
1 + 4		Storage (27.5 MG)	76.4	22.1	98.5	2.3	100.8	70.1
		Storage (4.6 MG)						
2 + 1		Storage (20.9 MG)	61.1	18.8	79.9	0.2	80.1	49.1
		Storage (1.1 MG)						
3 + 4		Storage (9.1 MG)	32.6	14.2	46.8	1.1	47.9	33.0
1 + 3 + 4		Storage (29.0 MG)	77.9	26.0	105.9	2.5	108.4	75.4
		Storage (4.6 MG)						
5 + 6		Treatment (150 mgd)	44.7	28.7	73.4	3.3	76.7	56.8

NOTES: (1) In millions of dollars - present day price levels (September, 1989), ENR = 4600
(2) Except for separation which is shown at far right
(3) Includes engineering and contingencies, assumed to be 30% of capital cost
(4) Present worth cost
(5) Includes CSO consolidation conduits, new regulators and other required structures, and force mains

33.1 MG. Results of evaluations indicate this alternative is not feasible or cost-effective based on hydraulic limitations of the system, and constraints of the site available for the facility. Unless additional treatment capacity were to be made available at the wastewater treatment plant, the estimated pumpback time for this facility alone, using all available system capacity, would be over twenty hours. This utilization of all available system conveyance and treatment capacity would prevent timely pumpback from other group storage facilities. This does not conform with the other preferred group storage options. The option of providing additional system capacity is discussed in Section 9.5 (System-Wide Alternatives).

The storage tank (sidewater) depth required for a 33.1 million gallon storage facility within the limits of the available site is about thirty feet, which will result in a deep excavation and high construction costs. Complex and costly construction techniques would be necessary for excavation, bracing and dewatering. Other facilities required under this alternative are approximately 350 feet of 84-inch influent pipe in Cove Road, 1,000 feet of 30-inch pipe to convey flow from CSO 003, and 1,400 feet of 54-inch force main.

Table 9-4 summarizes the costs for this alternative based on the WWTP at Fort Rodman. As shown, the total estimated cost is approximately $80.8 million. If the WWTP were to be at Standard Times, the force main length would increase to approximately 3,600 linear feet increasing the cost of this alternative to about $82.4 million.

Alternative 2 - Storage/Pumpback-Overflows Diverted to Inner Harbor (Figure 9-7). For this option, the storage facility would be designed for a 3-month frequency storm and have a capacity of 19.9 MG. The overflow pipe, which would convey flows in excess of the storage/pumpback facility capacity to the Inner Harbor, would be 84-inch in diameter and approximately 3,600 feet long. A siphon, consisting of two, 48-inch barrels and inlet and outlet structures, would be required to cross the existing main interceptor at the intersection of Second and Blackmer Streets. In addition, the overflow pipe would cross the Downtown

connector, which would probably require either a jacked or tunneled installation. Approximately 350 feet of 84-inch pipeline will be required to consolidate and convey flow from Bonney Street and Crapo Street into the facility. The force main would be approximately 1,400 feet long and would be 48-inch in diameter. Approximately 1,000 feet of 30-inch pipe would be required to convey flow from CSO 003 to the facility.

Under the concept of this alternative, flows that exceed the 3-month design storm capacity of the storage/pumpback facility would be diverted to the Inner Harbor through the 84-inch conduit described above. However, the required location of this conduit results in an area of about 80 acres where excess flows cannot be diverted. This area is generally located south of the conduit. CSOs from this area would continue to flow to the storage facility even when it's capacity is reached. To prevent overflows to Clarks Cove under these extreme conditions, the pumpback system would be used to pump excess flow to the 84-inch diversion conduit for transport to the Inner Harbor. Under this arrangement, a second force main would be required from the storage facility to the 84-inch conduit (1500 feet of 36-inch diameter pipe located in Bonney Street). Automatically controlled valves would be used to control the discharge from the storage facility. Under extreme events when the storage volume is filled, but flow continues to enter the facility, the pumps would convey flow to the 84-inch diversion conduit. Once the storm event ends, and flow into the facility stops, pumps would then convey flow back into the system (i.e., pumpback) by proper operation of the control valves. This proposed arrangement would serve to essentially eliminate CSO discharges to Clarks Cove from Group 1, except for extreme events (i.e., storms with much less frequent recurrence intervals than the 12-month storm).

Table 9-4 summarizes the estimated costs for this alternative based on the WWTP at Fort Rodman. As shown, the total cost is estimated to be $65.4 million. If the WWTP were to be located at Standard Times, the pumpback force main length would increase to 3600 feet and the length of the overflow pipeline would increase by 800 feet. The additional cost of these items is approximately $2.9 million for a total cost of $68.3 million if the WWTP is located at Standard Times.

This alternative would provide a higher level of CSO control than Alternative 1, since it would (essentially) provide elimination of CSOs from Group 1 into Clarks Cove (except for extreme events with frequencies much greater than a 12-month storm). Since it is considerably lower in cost, it is preferred over Alternative 1.

Alternative 3 - System Separation (Figure 9-8). As discussed earlier in this section, separation is not an appropriate CSO abatement option for Group 1. However, the estimated cost to separate the Group 1 combined sewer system is shown on Table 9-4 for comparison only with other alternatives. As shown, the cost for separating Group 1 is estimated to be $42.4 million.

In-Line Storage. Use of in-line storage to reduce capacity requirements and operation and maintenance costs of treatment facilities was evaluated.

The total storage available in the three major collector sewers in CSO Group 1 is approximately 1.0 MG during a 3-month storm. If all of this volume could be used, the required capacity of the storage tanks would be reduced by 3 percent for Alternative 1, and 5 percent for Alternative 2. The cost savings for either facility would be approximately $1.3 million if available pipe storage is utilized.

In order to use the storage available in the collector sewers, it would be necessary to install flow controls devices at several locations along each pipeline. The number required and locations are dependent upon the length and elevation drop of the collector sewers. Generally, in-line storage is most feasible in larger pipelines having relatively flat slopes because fewer controls are necessary under this condition than for smaller, steeper pipelines.

The drainage area included in Group 1 is relatively large, encompassing approximately 943 acres or 35 percent of the total combined sewer area in New Bedford. As a result, the sewer system in this group is extensive, and the total length of the three major collectors is approximately 8.0 miles. However, the vertical drop of the collector network is 87 feet from the

upstream end of the Tripps Brook collector to the Main interceptor. Because of the length and elevation drop of the system, several flow control devices would be required to effectively utilize the available pipe storage capacity. Aside from the capital cost of these improvements, these facilities would require substantial maintenance to prevent pipeline clogging and resulting system back-ups, and significantly increase system complexity and operator attention needs. In fact, the number would be so extensive that occasional back-ups and hydraulic restrictions would most likely occur in spite of adequate maintenance practices. For these reasons in-line storage is not recommended for Group 1.

9.4.3.2 Group 2

Alternative 1 - Storage/Pumpback (Figure 9-10). For this alternative, the storage facility would be designed for a 12-month frequency storm to meet water quality objectives for Clarks Cove, and would require 1.7 MG of capacity. The pipelines necessary to convey flow to the facility would include approximately 250 feet of 42-inch, 1,500 feet of 30-inch, and 3,550 feet of 24-inch pipe. The force main to convey stored CSOs back into the interceptor would be 12-inch diameter and about 100 feet long.

Table 9-4 summarizes the costs for this alternative based on the WWTP at Fort Rodman. As shown the total estimated cost is approximately $15.6 million. If the WWTP were to be at Standard Times, the force main length would increase to about 5,600 feet, increasing the total estimated cost to approximately $16.9 million.

Alternative 2 - System Separation (Figure 9-8). Although separation is not feasible for Group 2, its estimated cost is shown on Table 9-4 for comparison to other alternatives. As shown, the cost for separating Group 2 is estimated to be $6.7 million.

9.4.3.3 Group 3

Alternative 1 - Storage/Pumpback (Figure 9-11). The storage/pumpback facility would be designed for a 3-month storm with a capacity of 1.45 MG.

Pipelines required to convey flow to the facility include 2,000 feet of 18-inch, 1,600 feet of 24-inch and 2,700 feet of 30-inch pipe in East Rodney French Boulevard between Cove Street and Ricketson Street. The force main which would convey stored flow to the system would be 12-inch in diameter, extending north and west in East Rodney French Boulevard and David Street to the main interceptor.

Table 9-4 summarizes the costs for this alternative based on the WWTP at Fort Rodman. As shown, the total estimated cost for this alternative is approximately $15.6 million. If the WWTP were to be at Standard Times, the force main length would increase by approximately 1,600 feet, increasing the total estimated cost to $16.0 million.

Alternative 2 - System Separation (Figure 9-8). As noted in Section 9.3.3 separation is not a feasible CSO control technology for Group 3. The estimated cost to separate the combined sewer system is shown on Table 9-4 only for comparison to other alternatives. As shown, the cost for separating Group 3 is estimated to be $5.3 million.

9.4.3.4 Group 4

Alternative 1 - Storage/Pumpback (Figure 9-12). The storage/pumpback facility would be designed for a 3-month storm with a capacity of 7.6 MG. The pipelines required to convey flow to the facility would include approximately 1,100 feet of 15-inch pipe, 3,950 feet of 24-inch pipe and 150 feet of 66-inch pipe in MacArthur Drive, Conway Street and Hassey Street. The force main which would convey the stored flow back to the interceptor would be 30-inch diameter and approximately 1,300 feet long extending west from the facility, crossing the Downtown connector, and connecting to the main interceptor at Second Street.

Table 9-4 summarizes the costs for this alternative based on the WWTP at Fort Rodman. As shown, the total estimated cost for this alternative is approximately $32.7 million. The plan is identical if the WWTP were to be located at Standard Times, therefore, costs are the same.

Alternative 2 - System Separation (Figure 9-8). As noted in Section 9.3.3, separation is not a feasible CSO control technology for Group 4. The estimated cost to separate the combined sewer system is shown on Table 9-4 only for comparison to other alternatives. As shown, the cost for separating Group 4 is estimated to be $27.7 million.

9.4.4.5 Group 5

Alternative 1 - Treatment (Figure 9-13). Two treatment alternatives were evaluated for Group 5: 1) screening and disinfection and; 2) sedimentation, screening and disinfection. Both were evaluated for a 3-month storm.

The evaluation determined that option 2 would be less costly than 1. The most cost effective size sedimentation basin, determined by analysis using a mass diagram (Section 9.3.5), has a volume of 2.0 MG with a treatment capacity of 34 MG. The 2 MG of storage enables the facility to store flow during the peak flow period, reducing treatment and pumping capacity requirements from the storm peak of 53 MG to 34 MG. The estimated capital cost of the facility is $15.9 million.

Screening and disinfection was determined to be more expensive due to the need to provide chlorine contact chambers which are not required for sedimentation basins, and the additional pumping capacity required to meet peak storm flow rates. The estimated capital cost for a screening and disinfection facility is $20.4 million, or $4.5 million more than a sedimentation, screening and disinfection facility.

In addition to cost, there are more beneficial receiving water impacts achieved with sedimentation as compared to screening. Sedimentation provides a higher level of treatment than screening alone, removing solids which would be discharged in the effluent of a screening/ disinfection facility. Reduction of solids loadings to the Inner Harbor will have beneficial aesthetic impacts in addition to providing some reduction in loadings of metals and BOD. Although the current uses of the Inner Harbor are severely limited due to existing water quality, current efforts to

remove PCBs in sediments and future projects which may eliminate other sources of upstream pollution could substantially improve the Inner Harbor's water quality in the future. If there are improvements which could restore presently impaired uses of the Inner Harbor, benefits from the higher level of treatment would be realized.

There are relatively minor disadvantages of sedimentation as compared to screening which include greater land and maintenance requirements. However, the larger capacity pumping facilities, which would be required to accommodate peak flows for a screening facility, would also have significant land and maintenance requirements.

Due to its lower cost and greater receiving water quality benefits, sedimentation plus disinfection was selected as the treatment alternative for Group 5.

Other facilities required in conjunction with the sedimentation/ disinfection facility, include pipes to consolidate the CSO outlets and convey flow to the facility, and an effluent pipe to discharge treated flow to Inner Harbor. The conveyance pipelines include approximately 1,100 feet of 15-inch, 2,300 feet of 18-inch and 950 feet of 60-inch pipe. The effluent pipe would extend east from the facility to the Inner Harbor terminating near the existing CSO 037.

Table 9-4 summarizes the costs for this alternative based on the WWTP at Fort Rodman. As shown, the total estimated cost is $21.1 million. There would be no change to this alternative if the WWTP were to be located at Standard Times.

Alternative 2 - System Separation (Figure 9-8). Separation of the combined sewer system in Group 5 was evaluated as an alternative to treatment. The estimated cost of complete separation, as shown on Table 9-4, is $20.6 million.

One advantage of separation over treatment facilities is that operation and maintenance requirements are much less extensive. In addition to

eliminating the operation and maintenance of a treatment facility, existing regulators that also require periodic maintenance would be abandoned.

As noted previously in this study, separation of the combined system eliminates only those sources of pollutant loads which are related to the sanitary portion of combined sever overflows. Pollutant loads associated with stormwater would continue to impact receiving waters. Although stormwater related impacts do not significantly affect current Inner Harbor uses, they may in the future if water quality improves from the results of the PCB removal efforts and other potential projects. In addition, separation can be difficult and extremely disruptive due to the need to excavate in essentially all streets where there are existing combined sewers. Identification and removal of all inflow sources can also be difficult to accomplish. Other concerns related to separation are discussed in Section 7.0.

In-Line Storage. The utilization of in-line storage to reduce capacity requirements and operation and maintenance costs of the treatment facility was also evaluated. Results show that the major collector sewer tributary to Group 5, the Willis Street collector, has less than 0.5 MG of storage capacity available under a 3-month storm event. This volume reflects about half of the CSO discharge from a 2-week storm event.

The use of this available storage would have little effect on the size of the sedimentation basins. The 2 million gallon volume is required to treat flows which are sustained for several hours after the peak flow has passed for the design storm. As a result, the additional storage which would be provided by the pipe would have no effect on basin design capacity.

The use of in-line storage would also have negligible impacts on operation and maintenance costs. Since the storage available in the pipe cannot accommodate CSOs flow for even minor rain events, its use would have no appreciable impact on the frequency of operation of the treatment facilities.

For these reasons, in-line storage is not recommended for Group 5.

9.4.3.6 Group 6

Alternative 1 - Treatment (Figure 9-15). The same treatment alternatives evaluated for Group 5 were evaluated for Group 6, with similar results. Treatment by sedimentation plus disinfection was found to be less costly than screening alone plus disinfection for the same reasons as presented for Group 5. The sedimentation basins would provide 3.2 MG of storage and would have a capacity of 54 mgd. The estimated cost for this facility is $21.0 million as compared to $31.7 million for a screening and disinfection facility with capacity to meet a peak flow of 98 mgd. These flow rates and resulting facility costs include flow reductions which will occur as a result of the separation of the tributary area to CSO outlets 026 and 027 (this evaluation is presented under Alternative 2, later in this section).

Other facilities required for flow conveyance are approximately 2,750 feet of 24-inch, 650 feet of 36-inch, 850 feet of 42-inch, 3,800 feet of 78-inch and 700 feet of 84-inch pipe to consolidate the CSO outlets in Group 6, and 350 feet of 66-inch pipe to discharge treated effluent across Riverside Avenue to the Inner Harbor.

The total cost of this alternative is summarized on Table 9-4 based on the WWTP at Fort Rodman. As shown, the total estimated cost of this alternative including capital and operation and maintenance is $37.2 million. The cost of this alternative is the same if the WWTP were to be located at Standard Times.

Alternative 2 - System Separation (Figure 9-8). Total separation of the combined sewer system was evaluated for Group 6 as an alternative to treatment. The estimated cost of separating the entire Group 6 combined system is $36.2 million as shown on Table 9-4. The advantages and disadvantages of sewer system separation in comparison to CSO treatment are the same for Group 6 as discussed for Group 5.

Partial separation of the Group 6 tributary area was found to be cost-effective. Separation of the area tributary to CSO 026 and 027 would be less costly than conveying flow from this area for treatment at the

proposed Group 6 facility on Nash Road. The tributary area is about 39 acres and is located at this northern extremity of New Bedford's combined sewer system. Conveying flow from this area to the treatment facility would require about 3,100 ft. of 24-inch pipe. This gravity conduit would lower the elevation of the treatment facility by about 5 feet, substantially increasing construction complexity and costs for the facility and all CSO consolidation piping.

The estimated construction cost for separating 026 and 027 is $1.56 million. The alternative of conveying flow from this area to the treatment facility would increase piping costs alone by $2.10 million. This does not include the additional cost of the deeper treatment facility which would be substantial. Based on this evaluation, separation of the 39 acres tributary to CSOs 026 and 027 is recommended. As discussed earlier in this section, separation is a viable and acceptable option for CSO abatement for Group 6 considering the limited uses and value of the receiving water.

In-Line Storage. Use of in-line storage to reduce capacity requirements and operation and maintenance costs of treatment facilities was evaluated. Of the three major collector sewers tributary to CSO Group 6, only the Sawyer Street and Church Street collectors were evaluated. The Belleville Avenue Collector has no known combined sewers directly connected (i.e. all tributary combined flow is regulated upstream), thus cannot be utilized for storage.

The total storage available in the two collector sewers for a 3-month event is approximately 1.1 MG. The size of the sedimentation basins for Group 6 is required to accommodate flows which are sustained for several hours after the peak flow, in the same manner as the sedimentation basins for Group 5. In order to achieve any appreciable reduction in capacity requirements, the facility would require over 7 MG of total storage. As a result, in spite of the significant volume of storage available in the two collector sewers, there would be little impact on the capacity requirements of the treatment facilities.

Analysis of the 1-month storm event indicates that if all available storage in the two collectors were utilized, about 40 percent of the total flow volume for Group 6 could be stored in-line and that nearly all storage would be provided for a 2-week event. This suggests that there may be potential operation and maintenance savings if in-line storage is used in conjunction with the treatment facility. Utilizing this storage effectively is not possible however, since most of the storage available is in the higher elevation Church Street Collector but most of the inflow is in the lower Sawyer Street Collector. System elevations and hydraulics prevent the utilization of available pipeline storage in both collectors. As a result, no significant volume can be stored in-line.

9.4.3.7 Consolidated CSO Group Alternatives

Groups 1 and 4 (Figure 9-17). Under this alternative, flows in excess of the capacity of the storage/pumpback facility at the Standard Times site (which would be designed for a 3-month storm) would discharge to the Inner Harbor. The storage volume required is 26.1 MG, which would require a site in the range of 6 to 10 acres, depending on the sidewater depth of the facility. Due to the large land requirements, the preferred site is Standard Times. The pipelines required to consolidate CSOs in Group 4 and convey flow to the facility would be approximately 1,100 feet of 18-inch, 1,840 feet of 36-inch, 950 feet of 72-inch and 1,350 feet of 78-inch pipes. The pipeline necessary to convey flow from CSO Group 1 would be approximately 2,500 feet of 84-inch pipe which would intercept and convey flow east in Katherine Street and Blackmer Street. Also required would be a siphon crossing of the Main interceptor at the intersection of Blackmer Street and Second Street, and a jacked or tunneled crossing of the Downtown Connector. The stored volume would be pumped back to the system through 1,000 feet of 54-inch force main to the Main interceptor, crossing under the Downtown connector.

In order to accommodate CSOs from the south portion of CSO Group 1, a separate storage/pumpback facility would be required at Site 1. The facility would be designed to store CSOs up to a 12-month storm, and have a required storage volume of 4.6 MG and a pumping capacity of 7.4 mgd. The

force main would be 1,400 feet of 20-inch pipe. Approximately 1,000 feet of 30-inch pipe is also required to convey flow from CSO 003 to this facility.

Table 9-4 summarizes the costs for this alternative if the WWTP is at Fort Rodman. As shown, the total cost is estimated to be $100.8 million. If the WWTP was also located at Standard Times, several of the components would differ as follows:

- The effluent force main from the consolidated facility would be no greater than 200 feet.
- The siphon crossing of the Main interceptor would be eliminated since the Main interceptor would be diverted into the WWTP at Rivet Street.
- The length of 20-inch pumpback force main from the Site 1 storage facility would increase to 4,200 feet.

The decrease in cost resulting from the above is $20 million, for a total of $98.8 million if the WWTP is at Standard Times.

It should be noted that the cost of the consolidated storage facility may also be greater if the WWTP is located at Standard Times. The WWTP will be constructed prior to any CSO storage project, thus would occupy most of the site. This would result in design and construction-related constraints for this relatively large CSO facility, possibly resulting in additional cost.

Groups 2 and 1 (Figure 9-18). Under this concept, an evaluation was made to determine the most cost effective size of the Group 2 facility which would provide a combined pumping capacity and storage volume to accommodate flows from a 12-month storm. The peak flow rate and total volume for a 12-month storm for Group 2 are 19.2 mgd and 1.67 MG, respectively. The optimum combination of pumping capacity and storage volume was determined to be 1.1.MG of local storage with a pumping capacity of 5.0 mgd to convey flows to the second facility at Group 1.

The force main extending north in West Rodney French Boulevard would be 18-inch in diameter and approximately 5,200 feet long. The pipelines required to consolidate CSOs would be identical to Alternative 1 (storage/pumpback) for Group 2 alone. Conveyance of Group 2 flows by gravity to the second storage facility at Group 1 was not a feasible alternative due to the excessive excavation depths which would be required.

The addition of 5 mgd in flow rate to the Group 1 facility, as sized for Group 1 Alternative 1 or 2, is relatively insignificant and would not necessitate changes to the size or conceptual layout of that facility.

Table 9-4 summarizes the costs for this alternative based on the WWTP at Fort Rodman. As shown, the total estimated cost is $80.1 million. There would be no change to this alternative if the WWTP were to be located at Standard Times.

Groups 3 and 4 (Figure 9-18). This consolidation alternative would employ storage at the Standard Times site similar to the alternative for consolidating Groups 1 and 4 evaluated previously. The required storage volume for a 3-month design storm is 9.1 MG. The pumpback facilities would have a capacity of 14.5 mgd. The piping required to consolidate and convey flow from Group 3 includes 1,200 feet of 18-inch pipe, 4,450 feet of 24-inch pipe, 800 feet of 30-inch pipe and 2,000 feet of 42-inch pipe. The piping required to consolidate and convey flow from Group 4 would be identical to that required for consolidating Groups 1 and 4 as discussed previously. The force main to convey stored flow to the Main interceptor would be 1,000 feet of 30-inch pipe.

The Group 3 consolidation conduit under this alternative is relatively deep due to large flows and length of piping. The pipe would enter the storage facility at a relatively deep elevation. To avoid deep and costly excavations for the entire storage facility, pumping of CSOs from Group 3 into the storage facility proved to be cost-effective. It was also found to be less costly if this pumping station was located at the storage facility and combined with the pumpback system, than located on

Clarks Point near the Group 3 CSOs. This arrangement would require only one pumping station that would serve two purposes.

Table 9-4 summarizes the estimated costs for this alternative for the WWTP at Fort Rodman. As shown, the estimated cost is $47.9 million. If the WWTP were to be located at Standard Times, the effluent force main length would be no greater than about 200 feet, reducing the estimated cost by $0.3 million, or a total of $47.6 million.

Groups 1,3 and 4 (Figure 9-19). Under this alternative, CSOs from these three groups would be consolidated and conveyed to a single storage/pumpback facility located at Standard Times.

The facilities required to consolidate and convey CSOs would be the same as these described for Groups 1 and 4 and Groups 3 and 4. The storage/pumpback facility would be designed for a 3-month storm, and have a capacity of 29.0 MG. The pumpback facilities would require a capacity of 46 mgd. The force main required to convey the stored CSOs back to the Main interceptor would consist of 1,000 feet of 54-inch pipe. As discussed under the alternative of consolidating Groups 1 and 4, construction of this relatively large storage/pumpback facility may be more difficult and costly if the WWTP is located at Standard Times.

Table 9-4 summarizes the estimated costs for this alternative for the WWTP located at Fort Rodman. As shown, the total cost of this alternative is $108.4 million. If the WWTP were to be located at the Standard Times site, the same changes would apply as described for the consolidated facility for Groups 1 and 4. These changes in the system piping would reduce the cost by $2.0 million, resulting in a total of $106.4 million.

Groups 5 and 6 (Figure 9-19). Under this alternative, the same two treatment options that were evaluated for the two groups separately were considered for this consolidated alternative; screening and disinfection, and sedimentation, screening and disinfection.

For the consolidated facility, screening and disinfection was determined to be less costly than sedimentation, because the depth of the influent CSO conveyance pipe would require that the treatment facility be extremely deep. The estimated capital cost for a screening facility with pumping capacity to meet peak flows for the 3-month storm is $44.7 million. The cost of a sedimentation facility at the elevation required by the influent piping is estimated to be $47.2 million. The screening and disinfection facility would have a flow capacity of 150 mgd while the sedimentation basins would require a lower capacity of 96 mgd due to their volume.

The piping required to consolidate and convey flow from all CSOs within the two groups to the facility would be as follows; 1,100 feet of 15-inch pipe, 2,950 feet of 24-inch pipe, 1,350 feet of 36-inch pipe, 1,200 feet of 42-inch pipe, 3,550 feet of 60-inch pipe, 3,750 feet of 78-inch pipe, 800 feet of 84-inch pipe and 1,000 feet of 96-inch pipe. Due to the length of pipe required, pipe excavations would be relatively deep near the facility, approaching 30 feet.

Table 9-4 summarizes the estimated costs for this alternative for the WWTP at Fort Rodman. As shown, the estimated total cost for the screening/disinfection option is $76.7 million. This alternative would be identical if the WWTP were to be located at Standard Times therefore there is no cost difference between the two WWTP sites.

9.5 SYSTEM-WIDE ALTERNATIVES

9.5.1 GENERAL

In Section 9.4, CSO abatement alternatives were evaluated for each of the six CSO groupings. These alternatives include local storage or treatment facilities for each grouping, as well as required CSO consolidation pipelines. Each storage alternative is based on the concept that stored flows from all CSO facilities would be pumped back into the system and receive secondary treatment at the new WWTP. Under this concept, the maximum total flow to the plant, including both average daily wastewater

flow and CSOs, would be 75 mgd, which is its peak design capacity. The average daily design flow of the plant is 30 mgd, resulting in 45 mgd of plant capacity available for treatment of combined flows during wet weather conditions.

In developing plans which would consolidate groupings into a single treatment or storage facility, strong consideration was given to centralized CSO treatment at the new secondary WWTP. In this section, alternatives employing the concept of providing additional treatment capacity for CSOs at the plant (i.e., greater than 75 mgd) are evaluated. This additional capacity would consist of primary treatment only.

The use of centralized treatment facilities for both wastewater and CSOs typically has several potential advantages over plans employing satellite CSO treatment facilities including:

- Centralized facilities are easier, less costly and more efficient to operate and maintain.
- Economies of scale may be realized by construction of a single large facility in place of several smaller facilities.
- Upstream CSO storage requirements may be significantly reduced.
- Effluent would be discharged through an offshore outfall.

9.5.2 ALTERNATIVES

The extent of additional WWTP capacity for CSO treatment considered for these analyses ranges from zero (i.e., CSOs would receive treatment through the new secondary WWTP which would be limited to the currently proposed peak design flow of 75 mgd) to 80 mgd (primary treatment capacity in addition to the plant's 75 mgd of peak secondary flow capacity). Intermediate CSO treatment capacities of 25 mgd and 50 mgd were also evaluated to determine the most cost-effective capacity of the centralized CSO facility. The upper limit of 80 mgd would enable all CSO's up to a 12 month design storm to be stored at upstream facilities and pumped back through the system and to the WWTP within an 18 hour period.

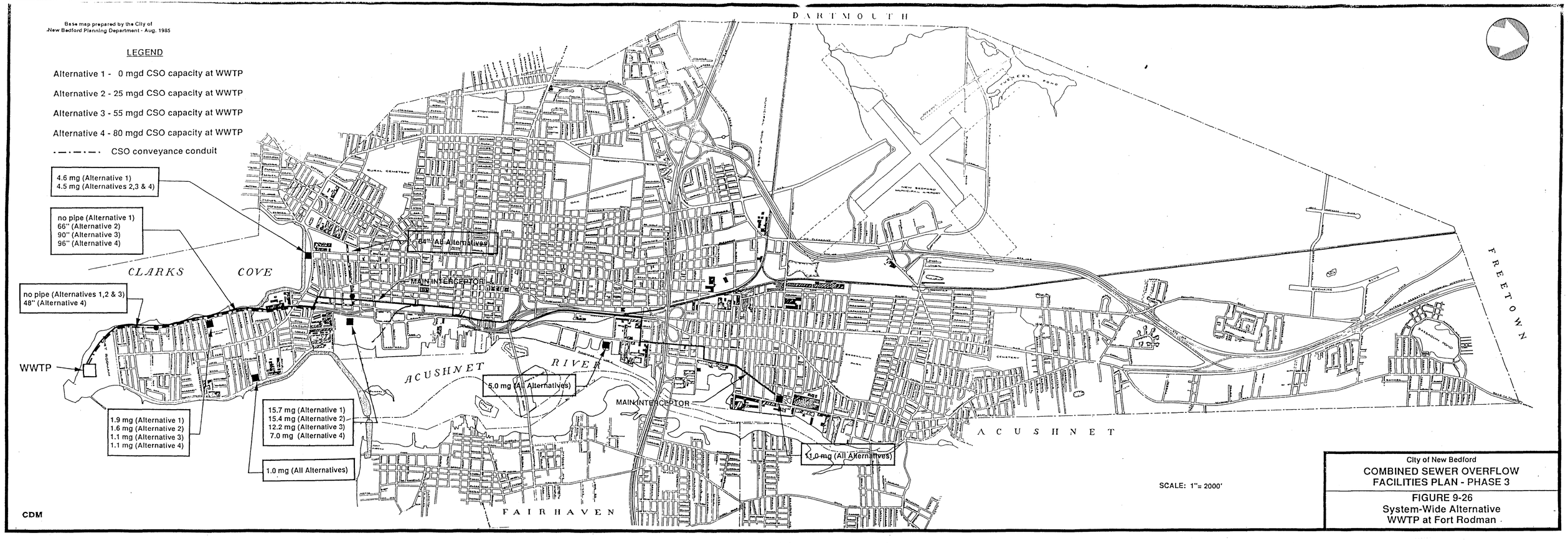
The two alternative WWTP locations were considered for the centralized CSO treatment facility; Fort Rodman, located at the south end of Clarks Point and; Standard Times, located north of the hurricane barrier on the west shore of the Inner Harbor. The locations of these sites and required CSO conveyance facilities are shown on Figures 9-26 and 9-27. The size of the conveyance facilities required for each level of CSO treatment capacity is indicated on each figure.

Under both system-wide alternatives, CSOs would be diverted from Group 1 to the existing main interceptor at Blackmer Street. A new regulating structure will be required on the main interceptor at this location. When interceptor capacity is exceeded during certain storm events, excess combined flow would be diverted and conveyed to a storage/-pumpback facility located at the Standard Times site. Also with regard to Group 1, about 80 acres of combined sewered area would not be handled by this arrangement, as discussed in Section 9.4.3.1, under the Group 1, Alternative 2 evaluation. A separate storage/pumpback facility is required for this area, as shown on Figures 9-26 and 9-27. Its operation would be as described for Group 1, Alternative 2 (Section 9.4.3.1).

9.5.3 RELATIONSHIP BETWEEN CENTRAL FACILITY CAPACITY AND UPSTREAM STORAGE/TREATMENT REQUIREMENTS

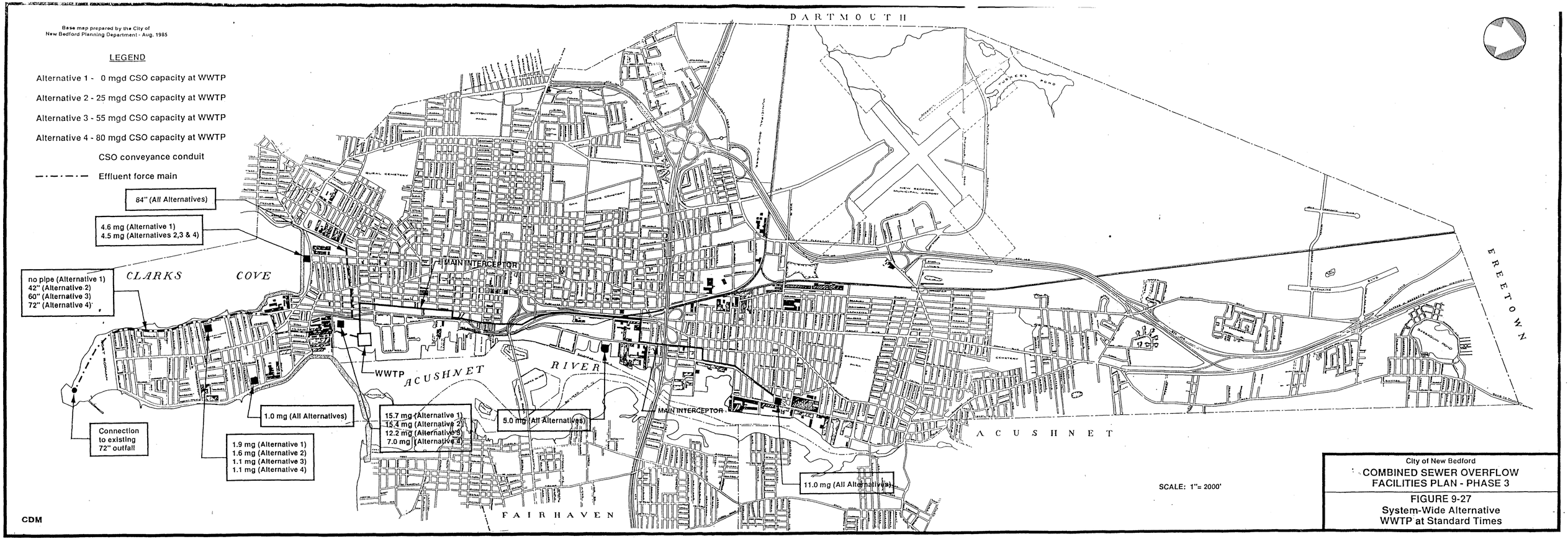
In combined sewer systems, flows which exceed the capacity of the interceptor sewers and treatment system discharge directly to receiving waters through CSO outfalls. By increasing the capacity of the interceptor sewers and treatment system, CSO discharges can be reduced, and in some cases eliminated. This also serves to decrease the capacity requirements of local storage or treatment facilities at CSO outlets.

In order to determine the capacity required for upstream storage/treatment facilities under various central CSO treatment scenarios, the system was analyzed under 1-, 3-, 6-, and 12-month design storm conditions for the four levels of CSO treatment capacity at the WWTP (0, 25, 50 and 80 mgd). The model SWMM was applied for both the Fort



City of New Bedford
COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 9-26
System-Wide Alternative
WWTP at Fort Rodman


Base map prepared by the City of
New Bedford Planning Department - Aug. 1985
LEGEND
Alternative 1 - 0 mgd CSO capacity at WWTP
Alternative 2 - 25 mgd CSO capacity at WWTP
Alternative 3 - 55 mgd CSO capacity at WWTP
Alternative 4 - 80 mgd CSO capacity at WWTP
CSO conveyance conduit
Effluent force main
84" (All Alternatives)
4.6 mg (Alternative 1)
4.5 mg (Alternatives 2,3 & 4)
no pipe (Alternative 1)
42" (Alternative 2)
60" (Alternative 3)
72" (Alternative 4)
Connection
to existing
72" outfall
1.9 mg (Alternative 1)
1.6 mg (Alternative 2)
1.1 mg (Alternative 3)
1.1 mg (Alternative 4)
1.0 mg (All Alternatives)
15.7 mg (Alternative 1)
15.4 mg (Alternative 2)
12.2 mg (Alternative 3)
7.0 mg (Alternative 4)
5.0 mg (All Alternatives)
11.0 mg (All Alternatives)
WWTP
MAIN INTERCEPTOR
MAIN INTERCEPTOR
CLARKS COVE
ACUSHNET RIVER
ACUSHNET
DARTMOUTH
FAIRHAVEN
FREETOWN
SCALE: 1"= 2000'
CDM
City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3
FIGURE 9-27
System-Wide Alternative
WWTP at Standard Times

Rodman and Standard Times siting alternatives. Results of these evaluations will determine the optimum combination of upstream storage/treatment and additional WWTP capacity.

Table 9-5 shows the total volume discharged at each design CSO grouping for each storm event and WWTP capacity option. The volumes shown are based on the plant being located at Fort Rodman. Results indicated that the WWTP location has no significant impact on the overflow volumes at each CSO grouping. Accordingly, there would be no significant difference in the capacity requirements for upstream CSO storage/treatment facilities on the basis of WWTP siting.

As shown on the table, the only locations where CSO discharge volume is significantly affected by system capacity for storms exceeding the 1 month interval are at CSO Group 4, and the Inner Harbor at the confluence of the Main interceptor and the CSO Group 1 conveyance pipeline. This is due to the fact that the peak CSO flow, system-wide, far exceeds all system conveyance and treatment capacity levels which could realistically and economically be provided. As a result, only a relatively small portion of the total system overflow volume can be intercepted and treated by a centralized system.

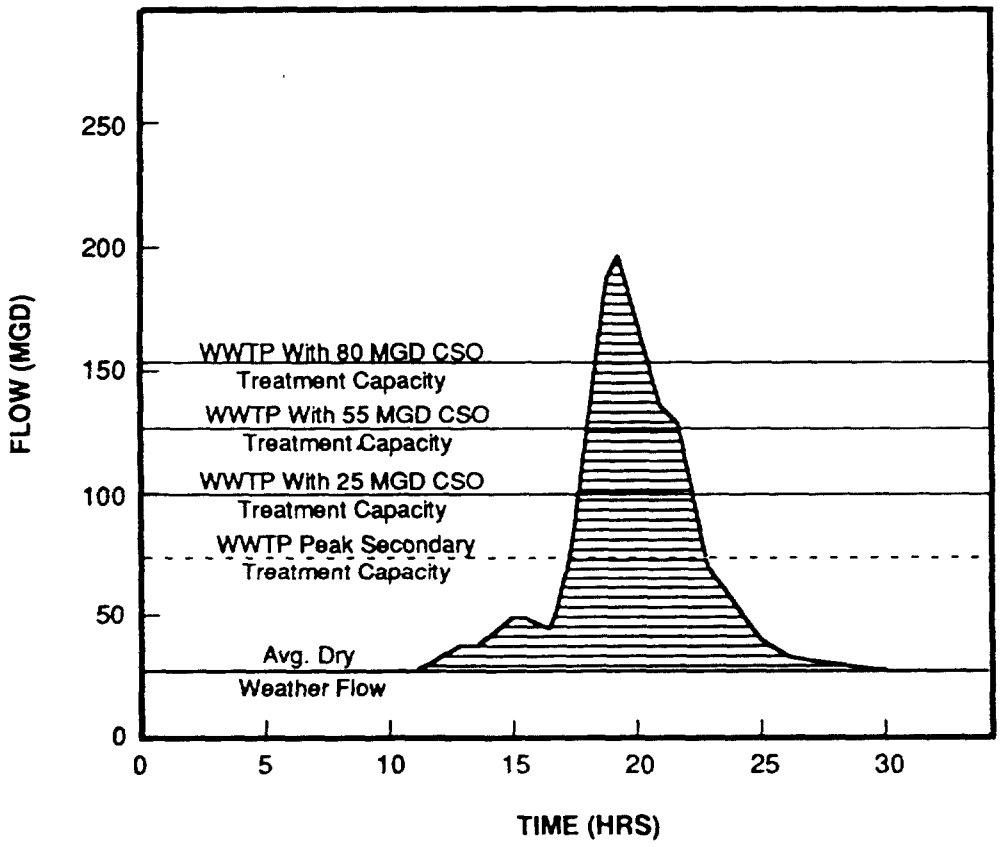
This is illustrated on Figures 9-28 through 9-31 which show system hydrographs for the 1, 3, 6 and 12 month storms. The horizontal lines show the four levels of CSO conveyance and treatment capacity evaluated. The total volume which must be either stored or treated at upstream facilities is represented by the area between the hydrograph and the treatment capacity. The figures clearly show that for all events over the 1 month storm, the provision of conveyance and treatment capacity for CSOs at the WWTP would provide only marginal benefits in terms of reducing upstream storage/treatment requirements.

TABLE 9-5

WWTP CAPACITY VS. CSO DISCHARGE VOLUME BY CSO GROUP (MG)

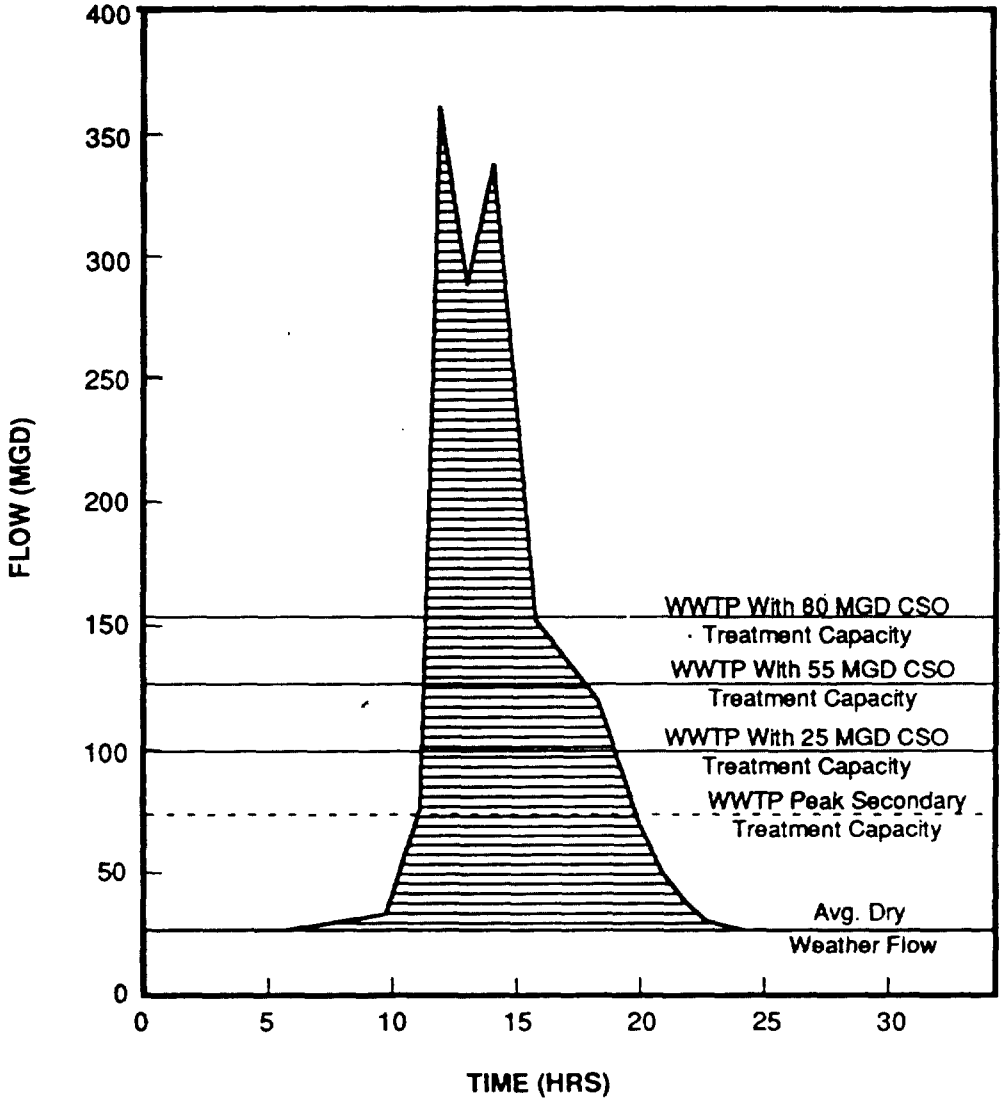
Location	1-Month Storm				3-Month Storm				6-Month Storm				12-Month Storm			
	75 MGD	100 MGD	130 MGD	155 MGD	75 MGD	100 MGD	130 MGD	155 MGD	75 MGD	100 MGD	130 MGD	155 MGD	75 MGD	100 MGD	130 MGD	155 MGD
Group 1	.0	.0	.0	.0	1.1	1.1	1.1	1.1	3.0	2.8	3.0	3.0	4.6	4.5	4.5	4.5
Group 2	.0	.0	.0	.0	.4	.1	.0	.0	1.4	.7	.4	.4	1.9	1.6	1.0	1.1
Group 3	.3	.3	.3	.3	1.0	1.0	1.0	1.0	2.1	2.1	2.1	2.1	2.8	2.8	2.9	2.9
Group 4	3.0	1.2	.8	.8	9.3	5.7	4.1	4.7	16.4	12.4	9.0	10.0	22.1	17.2	13.1	14.6
Group 5	1.8	1.8	1.8	1.8	5.2	5.2	5.2	5.2	9.7	9.6	9.6	9.6	13.7	13.7	13.6	13.6
Group 6	2.8	2.8	2.8	2.8	10.8	10.8	10.8	10.8	19.2	19.2	19.2	19.2	25.9	25.9	25.9	25.9
Main Int.[1] & Group 1	5.3	4.0	1.0	0.0	15.7	15.4	12.2	7.0	23.6	24.7	23.3	14.7	26.4	27.3	26.0	16.2

NOTE: (1) Overflows at the confluence of the Main Interceptor and the redirected Group 1 overflows.



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 9-28
Hydrograph for 1-Month Storm



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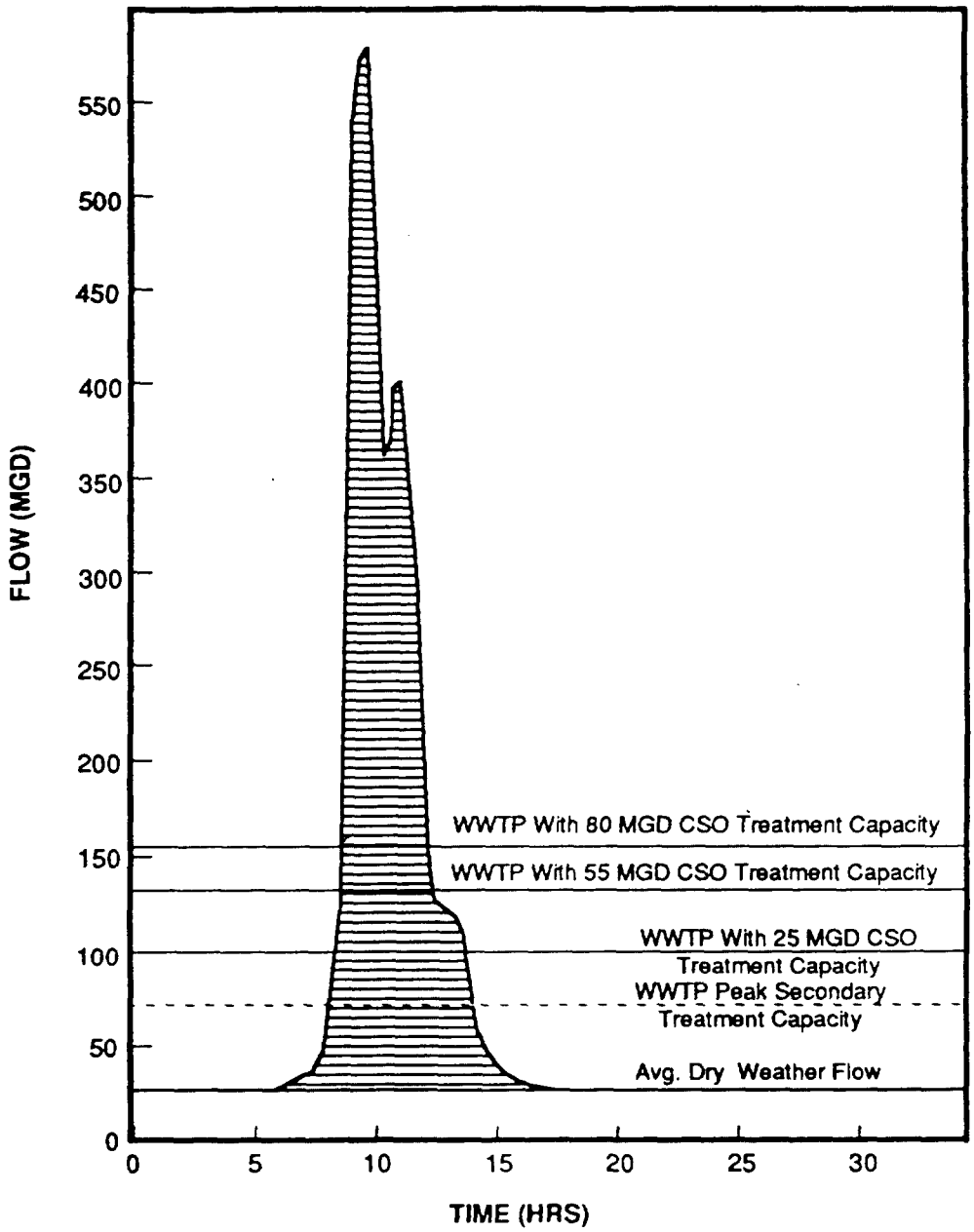
FACILITIES PLAN - PHASE 3

FIGURE 9-29
Hydrograph for 3-Month Storm



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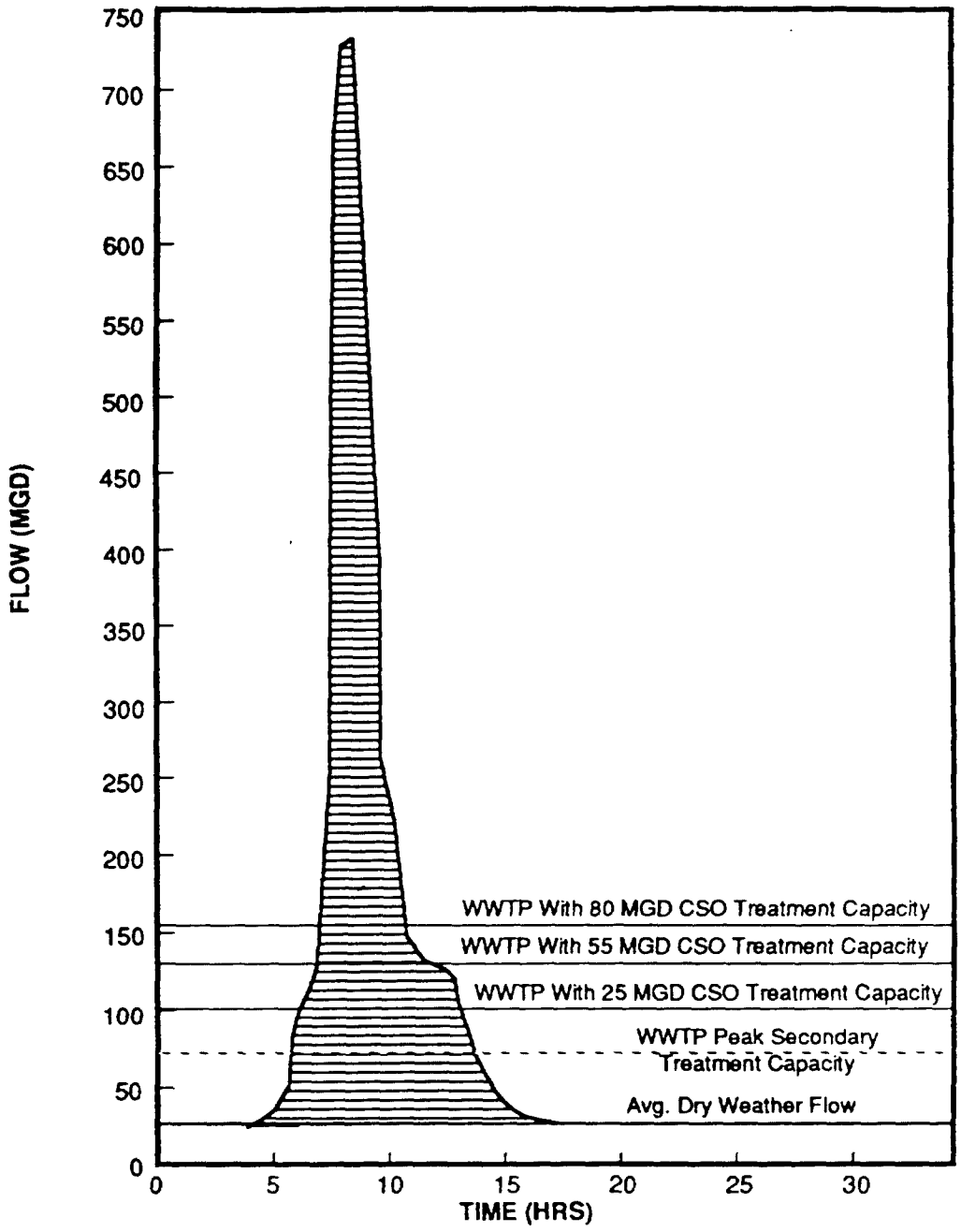
FACILITIES PLAN - PHASE 3

FIGURE 9-30
Hydrograph for 6-Month Storm



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COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 9-31
Hydrograph for 12-Month Storm

9.5.4 EVALUATION OF SYSTEM-WIDE ALTERNATIVES

System-wide alternatives were developed based on the same receiving water priorities and minimum level of CSO control as the group alternatives developed and evaluated previously in this section that is:

- CSO treatment is not a viable alternative for CSOs discharging to Clarks Cove or the Outer Harbor, but is viable for discharge to the Inner Harbor.
- The maximum frequency of CSO discharge to Clarks Cove is once per year on an average annual basis. The maximum frequency is four times per year for the Outer and Inner Harbor.

As shown in the preceding section, centralized CSO treatment is only marginally effective in reducing upstream CSO storage or satellite treatment requirements. Detailed evaluations indicate that if the WWTP capacity is increased to provide primary CSO treatment, all upstream satellite storage and treatment facilities (as required under the individual group alternatives) would still be required. Only the capacity of three satellite facilities would be reduced.

Figures 9-26 and 9-27 show the facilities required under the two alternative WWTP site options for system-wide CSO alternatives. Also shown are the capacity requirements for facilities needed to convey CSOs to the WWTP and treated effluent to the existing outfall at Fort Rodman for each of the alternative treatment capacities evaluated. In addition, the storage requirements at each CSO grouping and for each alternative WWTP capacity are also indicated.

As shown on Figure 9-26, the relief conduits required to augment the capacity of the Main interceptor for the three alternatives which provide padditional CSO treatment capacity at the WWTP range in size from 66-inch diameter for 28 mgd CSO capacity to 96-inch for 80 mgd CSO capacity (if the WWTP is located at Fort Rodman). Similarly, Figure 9-27 shows that the size of the effluent force main required to convey treated effluent from the WWTP to the existing 72-inch outfall ranges from 42-inch for 25 mgd CSO

capacity, the 72-inch for 80 mgd CSO capacity (if the WWTP is located at Standard Times).

Table 9-6 summarizes estimated costs for each WWTP capacity alternative for the two WWTP sites. The costs were developed utilizing the lowest cost combination of CSO group consolidation and feasible storage/treatment options. As indicated, the most cost-effective alternative is to provide no additional CSO treatment capacity at the WWTP. Under this scenario, only the available peak flow capacity of the secondary WWTP (75 mgd peak - 30 mgd average = 45 mgd available for treating combined flows) would be available. The cost of providing additional capacity is similar for both WWTP site options for low CSO capacities since the conveyance pipe cost for the Fort Rodman option and the force main cost for the Standard Times options are similar. For higher WWTP capacities, relief piping is require for the segment of the Main interceptor downstream of the screenhouse, resulting in a substantial increase in conveyance pipe cost. The fact that costs increase as WWTP capacity increases is expected since upstream satellite facilities are required irrespective of WWTP capacity. The need for upstream storage or treatment also eliminates potential operation and maintenance advantages of system-wide alternatives which may be achieved by centralized treatment facilities.

There are no cost or operation and maintenance advantages for centralized CSO treatment under the system-wide alternatives, as compared to individual CSO group alternatives. One potential advantage of the system-wide options, however, is that offshore CSO discharge would be maximized as opposed to discharge at local (satellite) facilities. As shown in Table 9-3, as the capacity of centralized treatment facilities increases, the volume of CSO discharge which can be stored and pumped back within an acceptable time period also increases. In order to provide for treatment of <u>all</u> CSOs at a centralized facility, approximately 100 mgd of total WWTP capacity (25 mgd CSO capacity) would be required, in addition to storage facilities at Groups 5 and 6 (as opposed to individual treatment facilities). The total cost for providing storage at Groups 5 and 6 would be considerably higher than treatment, resulting in a project cost of approximately $210 million for the WWTP at Fort Rodman. The only benefit

TABLE 9-6

ESTIMATED COSTS[1] OF SYSTEM-WIDE ALTERNATIVES

Alternatives	WWTP CAPACITY[2] 75 mgd[3]	100 mgd	130 mgd	155 mgd
Secondary WWTP at Fort Rodman				
Satellite Facilities	138.4	131.7	121.4	113.3
Consolidation/Conveyance Conduits	41.8	47.7	54.0	68.1
WWTP Expansion (Primary CSO Treatment)	- 0 -	13.3	24.3	32.0
Effluent Force Main	- 0 -	0.8	0.9	1.0
Operation & Maintenance[4]	5.7	5.7	6.0	6.0
TOTAL	185.9	199.2	206.6	220.4
Secondary WWTP at Standard Times				
Satellite Facilities	138.4	131.7	121.4	113.3
Consolidation/Conveyance Conduits	44.1	44.1	44.1	44.1
WWTP Expansion (Primary CSO Treatment)	- 0 -	13.3	24.3	32.0
Effluent Force Main	- 0 -	8.9	13.5	15.2
Operation & Maintenance[3]	5.7	5.7	6.0	6.0
TOTAL	188.2	203.7	209.3	210.6

Notes: (1) In millions of dollars - present day price levels (September 1989), ENR = 4600. All capital costs include an allowance of 30% for engineering and contingencies.
(2) Only 75 mgd receives secondary treatment, excess receives primary treatment.
(3) Design capacity of proposed new secondary WWTP.
(4) Present worth cost.

resulting from this scenario is eliminating treated CSO effluent discharges to the Inner Harbor, since they would be stored and pumped back to the system for treatment at the centralized facility (with offshore effluent discharge). Considering that the Inner Harbor is of lowest priority, this additional cost to achieve a slightly higher level of CSO control is not warranted.

The major disadvantage that centralized treatment has in comparison to individual CSO group alternatives is the limited flexibility in terms of staging it implementation. At a minimum, any system-wide plan would require that the relief conveyance and effluent force main pipes, and the WWTP improvements including sedimentation basins, screening, disinfection and pumping equipment be constructed with the initial phase of the project. For New Bedford, this would require a minimum up-front investment of approximately $109 million to construct facilities for the higher priority CSO Groups 1 and 2, and provide 25 mgd CSO capacity at the WWTP. Under the individual CSO group alternatives, the up-front cost for Group 1 and 2 CSO abatement would be only $79 million.

Based on these evaluations, it is apparent that system-wide alternatives have no advantages over individual group alternatives based on cost, implementation issues (i.e., ability to stage projects) and receiving water impacts.

9.6 SUMMARY AND RECOMMENDATIONS

Recommended alternatives are presented in Table 9-7. They were selected based on the results of detailed evaluations as presented in previous sections, and on the evaluation criteria presented in Section 9.2.3. As shown, recommendations include a separate CSO abatement plan for each of the six CSO groups that reflect the selected individual group alternative. In general, individual group alternatives are recommended over consolidated group alternatives and system-wide alternatives on the basis of cost and implementation. Estimated costs of the individual group alternatives are either lower or comparable to costs of consolidated group or system-wide alternatives. Equally important, the individual group alternatives provide

TABLE 9-7

SUMMARY OF RECOMMENDATIONS

Recommended CSO Abatement Alternatives

CSO Group	Alternative	Design Storm	Major Components
1	2	>12-month[1]	Storage (19.9 MG)/Diversion Conduit to Inner Harbor
2	1	12-month	Storage (1.7 MG)
3	1	3-month	Storage (1.5 MG)
4	1	3-month	Storage (7.6 MG)
5	1	3-month	Treatment (34 mgd)
6	1	3-month	Treatment (54 mgd)/Partial Separation

Notes: (1) Group 1 recommendations would essentially result in complete CSO elimination except for extreme storm events with recurrence intervals much less frequent than a 12-month storm.

maximum flexibility since the six discrete projects can be implemented separately based on their relative priority and on available funding. Receiving water impacts resulting from implementation of the recommended individual group alternatives are identical to consolidated group or system-wide alternatives since all were evaluated under the same levels of CSO control.

The following table compares the estimated total cost of each consolidated group alternative, to the combined cost of the applicable individual group alternatives. For example, the estimated cost of the Groups 1 and 4 consolidated alternative is $100.8 million. The estimated cost of the individual Group 1 plus the individual Group 4 alternatives is $98.1 million ($65.4 million + $32.7 million as shown in Table 9-4).

Groups	Consolidated Group Alternatives Estimated Cost ($ million)	Combined Individual Group Alternatives Estimated Cost ($ million)
1 + 4	$ 100.8	$ 98.1
2 + 1	80.1	81.0
3 + 4	47.9	48.3
1 + 3 + 4	108.4	113.7
5 + 6	76.7	58.3

As shown, individual group alternative costs are comparable to all consolidated group options except for 5 + 6 where the individual group costs are significantly lower.

The total combined cost of the six individual recommended projects is comparable to the system-wide alternatives assuming no additional WWTP capacity (discussed in Section 9.5 with costs shown in Table 9-6). The flexibility to more effectively stage recommendations is the distinct advantage individual group alternatives have over the system-wide options, as discussed above.

10.0 RECOMMENDED PLAN

10.1 OVERVIEW

This section describes the recommended plan as developed from the detailed screening of CSO abatement alternatives presented in Section 9.0. The recommended plan is comprised of non-structural and structural components as discussed in Section 10.2. A summary of the major structural components of the recommended plan is presented in Table 10-1 and shown on Figure 10-1.

There are 6 structural CSO projects that make up the recommended plan, one for each of the 6 CSO groups. The recommended plan for CSO Groups 1 through 4 includes a separate storage facility for each group. During rainstorms, combined sewer overflow would be stored in underground tanks once the main sewer lines and interceptors are full. After the rainstorm, combined sewer overflow in the storage tanks would be pumped into the Rodney French interceptor where it would be routed to the secondary wastewater treatment plant (WWTP), treated, and discharged from the rehabilitated 60-inch outfall. The recommended plan for CSO Groups 5 and 6 includes satellite treatment facilities. CSOs in these groups would be consolidated and brought to separate treatment facilities, where they would receive the equivalent of primary treatment. Effluent from the treatment plants would be discharged into the Inner Harbor. Detailed descriptions of these projects are provided in Section 10.2.

The level of CSO control would be--at a minimum--up to a 3-month storm (which occurs, on average, 4 times per year). A higher level of control is provided for Clarks Cove, however, because it is the highest priority receiving water. There, a control level up to a 1-year storm has been developed. For storms up to a 3-month storm, overflows from CSO Group 1 would be stored for later pump back into the interceptor for treatment. When storms larger than the 3-month storm occur and the storage facility fills up, excess flow would be captured in another pipe and routed to the

TABLE 10-1

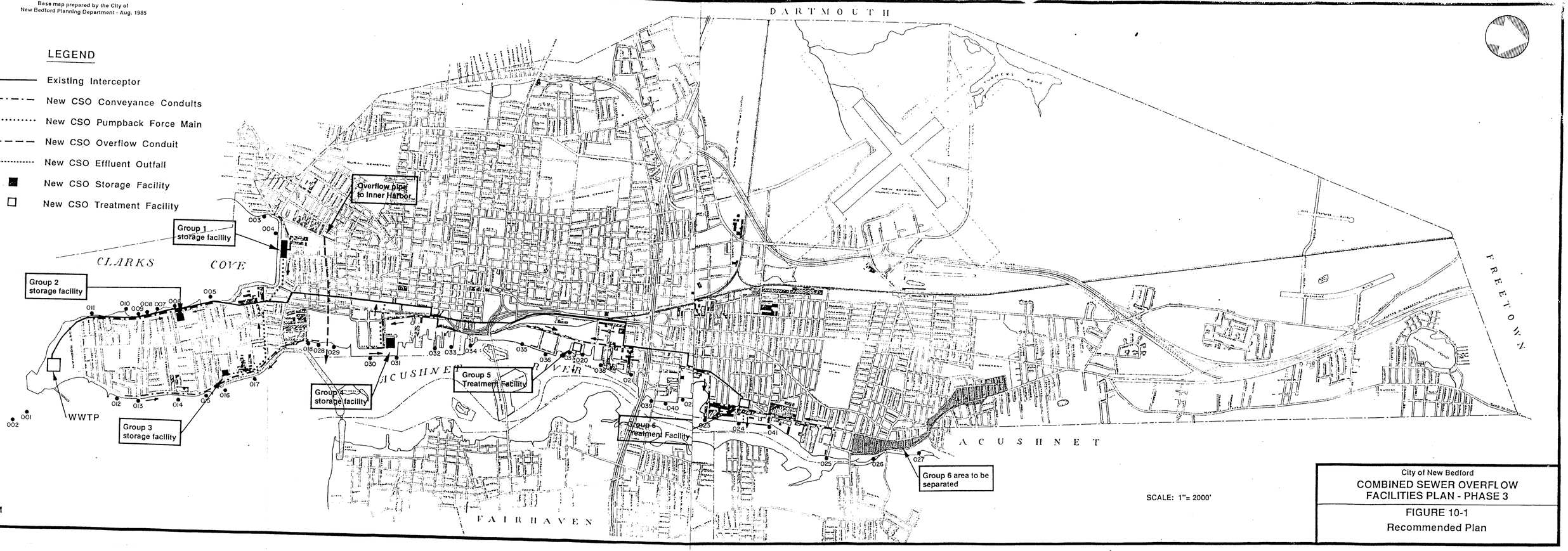
STRUCTURAL COMPONENTS OF RECOMMENDED PLAN

CSO Group	Recommended Alternative	Level of Control	Where is Treated Effluent Discharged	How often and where will untreated overflows occur?
1	Separate storage facility with pumpback to secondary wastewater treatment plant (WWTP)	3-month storm	Outer Harbor	4 times per year to Inner Harbor
2	Separate storage facility with pumpback to WWTP	12-month storm	Outer Harbor	Less than 1 per year to Clarks Cove
3	Separate storage facility with pumpback to WWTP	3-month storm	Outer Harbor	4 times per year to Outer Harbor
4	Separate facility with pumpback to WWTP	3-month storm	Outer Harbor	4 times per year to Inner Harbor
5	Satellite treatment facility	3-month storm	Inner Harbor	4 times per year to Inner Harbor
6	Satellite treatment facility	3-month storm	Inner Harbor	4 times per year to Inner Harbor

Base map prepared by the City of
New Bedford Planning Department - Aug. 1985

LEGEND

- Existing Interceptor
- New CSO Conveyance Conduits
- New CSO Pumpback Force Main
- New CSO Overflow Conduit
- New CSO Effluent Outfall
- New CSO Storage Facility
- New CSO Treatment Facility



SCALE: 1"= 2000'

CDM

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FACILITIES PLAN - PHASE 3

FIGURE 10-1
Recommended Plan

Inner Harbor where it would be discharged. The recommended plan, therefore, includes provisions for eliminating CSO Group 1 from discharging into Clarks Cove. It is not possible to eliminate discharges from CSO Group 2 unless sewer separation was undertaken. Sewer separation for CSO Groups 1 through 5 was not included in the recommended plan, as discussed in Section 10.4. Under the recommended plan, the number of times CSOs would spill into Clarks Cove would be reduced from 45 times per year, to less than once per year. Detailed discussions of the levels of control and the water quality benefits of the recommended plan are provided in Section 10.3.

The recommended plan has been developed based on several important assumptions regarding siting, engineering, and policy issues. The plan has been purposefully designed to be flexible, to facilitate implementation, to ease the cost burden on the city, and to respond to any subsequent changes in siting decisions. The recommended plan was developed based on the following considerations.

- The Secondary Wastewater Treatment Plant Will be Located at Fort Rodman - In May 1989, Mayor Bullard recommended the Fort Rodman site as the preferred site for the 75-mgd secondary wastewater treatment plant (WWTP) based on cost and engineering considerations. Part of the recommended plan for CSO abatement includes provisions for building storage facilities to hold combined sewer overflows for later treatment at the plant. The storage facilities have been designed to bleed flows back into the existing interceptor system for handling at the secondary treatment plant for 18 to 20 hours after storms are over.

- Alternative Backup Plans Have Been Prepared If The WWTP is Located at the Standard Times Site - If the recommended WWTP site is changed to the alternate site, Standard Times, then the recommended CSO abatement plan changes slightly. The same storage sites would be used to hold combined sewer overflow, but additional piping would be needed to route the flows to the new WWTP site. Because the cost of the additional piping would be negated by the cost savings from eliminating the siphon, the overall additional cost to the recommended plan would be negligible. The pipeline costs associated with the sewer system modifications recommended in the draft Phase 2 WWTP Facilities Plan.

- The Plan Maximizes Use of the WWTP - The recommended plan calls for routing a maximum volume of combined sewer overflow to the secondary

WWTP, thereby taking maximum advantage of the high degree of treatment provided by the secondary WWTP.

- Components of the Recommended Plan Have Been Prioritized So That The Greatest Receiving Water Quality Benefits Are Realized First - The recommended plan is composed of 6 CSO projects that have been prioritized in the order that they should be undertaken. The highest priority projects involve CSO Groupings 1 and 2, which discharge into Clarks Cove (the highest priority receiving water in New Bedford). Projects for CSO Group 3, which affects the next most important receiving water, the Outer Harbor, should be undertaken next. Finally, projects for CSO Groups 4, 5 and 6, which discharge to the Inner Harbor (the receiving water with the lowest priority) should be built last.

- The Recommended Plan Could Result in the Reopening of Closed Shellfish Beds In Clarks Cove - All New Bedford CSOs and storm drains discharging to Clarks Cove would be controlled. Overflows from CSO Group 1 would be eliminated by providing storage up to a 3-month design storm level (and eventual pump back to the interceptor for treatment at the WWTP), and routing flows exceeding the control level to the Inner Harbor. Overflows from CSO Group 2 would be routed to a storage facility capable of holding overflow volumes for up to a 1-year storm. Storm drains located near Rogers Street in Dartmouth would also possibly have to be controlled because they contribute high coliform loadings to Clarks Cove. Controlling the CSOs in New Bedford and the storm drains in Dartmouth would allow the shellfish beds to be reopened. The reopening of closed shellfish beds in Clarks Cove could result in an annual $5 million economic gain to the local economy.

- The Recommended Plan Is Composed of Discrete Individual Projects That Can Be Undertaken Individually or Together - To ease the economic burden on the City, the structural components of the recommended plan have been broken down into 6 discrete projects--one for each of the 6 CSO Groups. This approach provides flexibility in that each project can be taken in turn as funds become available, rather than imposing a tremendous financing burden on the City's limited resources by having to do one or more large-scale projects all at once. As mentioned above, the 6 projects have been prioritized so as to achieve the greatest water quality benefits.

An alternative to the 6 discrete plans has also been developed. Instead of constructing separate storage facilities for CSO Groups 1 and 4, they could be consolidated at one site. The most suitable site for consolidation is the privately-owned Standard Times site because it is centrally located within the CSO Groups, is close to the main interceptor, and is sufficiently large to hold the estimated combined sewer overflow volumes. A centralized storage facility for Groups 1 and 4 would cost about the same as providing separate storage facilities for Groups 1 and 4. The

alternative of providing centralized storage at the Standard Times site would provide the same water quality benefits as the recommended plan. The main disadvantage of the consolidation plan is that it would require a somewhat larger construction project all at once in order to be effective. The larger storage facility could not be cost-effectively built in small increments. In addition, monies would be spent on Group 4, which is a lower priority than Groups 2 or 3.

It is not possible to store overflows from CSO Groups 2 and 3 at the same centralized storage facility because of system capacity constraints. There is no cost-effective way to pump overflows back up to Standard Times and still provide the same level of water quality protection provided by the recommended plan.

10.2 COMPONENTS OF RECOMMENDED PLAN

The recommended plan consists of both structural and non-structural components. The structural components include 6 discrete CSO abatement projects such as storage-pump back and satellite treatment facilities, and general system-wide improvements such as raising weir heights and removing grit that has accumulated in pipes. The non-structural components include Best Management Practices (BMPs) such as regulator inspections and catch-basin cleaning, and continued enforcement of the Industrial Pretreatment Program.

10.2.1 ABATEMENT PROJECTS BY CSO GROUP

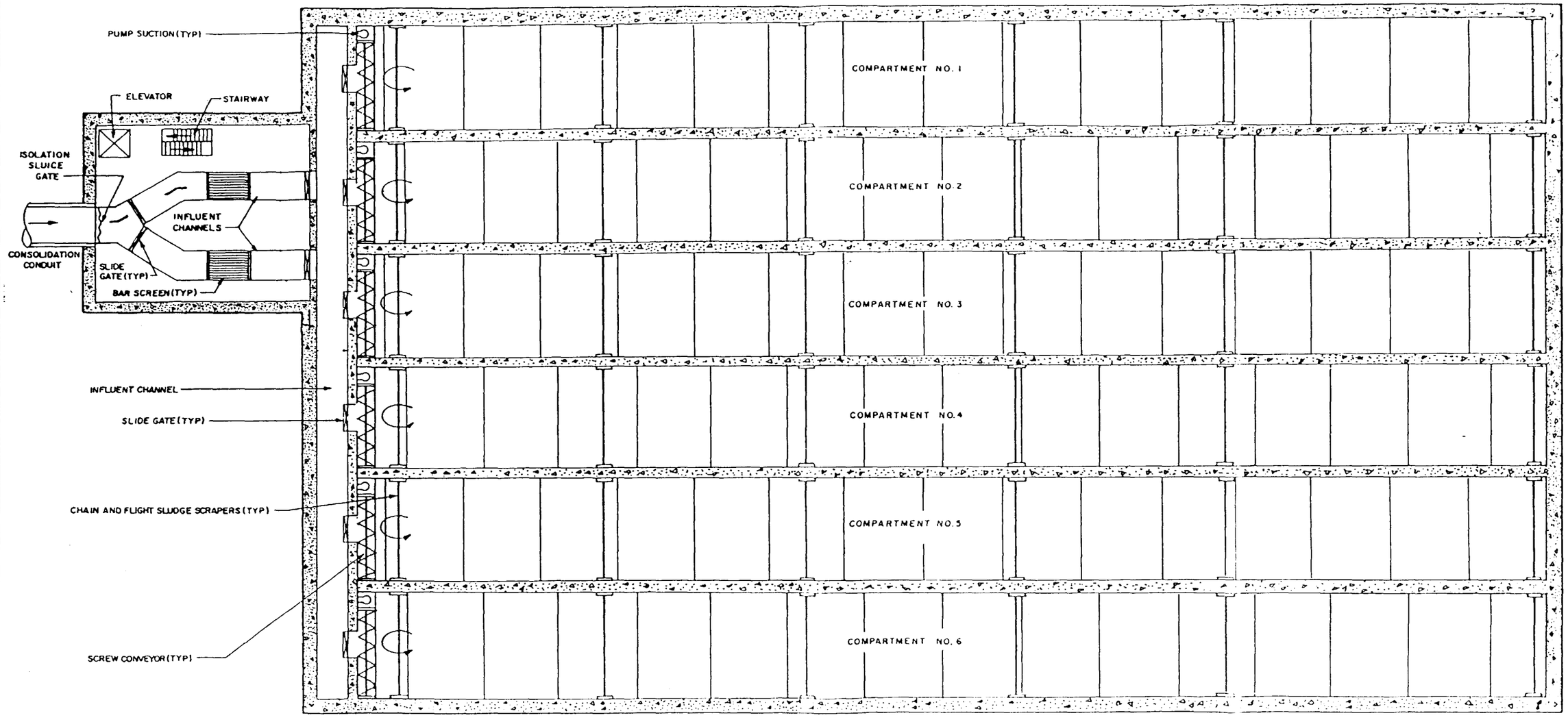
The 35 CSOs in New Bedford were consolidated into 6 groups to facilitate engineering evaluations and implementation. The recommended plan includes 6 discrete projects--one for each group as shown on Figure 10-1. Detailed layouts and descriptions of each project are provided later in this section. First, a discussion of storage-pump back and satellite treatment systems is warranted.

Description of Storage-pump Back and Satellite Treatment Systems

The primary component of each project is either a storage-pump back facility (Groups 1 through 4) or a CSO treatment facility (Groups 5 and 6). As discussed in Section 9.0, the selected CSO treatment technique for Groups 5 and 6 is screening followed by sedimentation and disinfection. This type of system has many similar features to a storage-pump back system. Both require screens, underground basins with sludge collection equipment, pumps, and a superstructure. The basins for a storage-pump back facility are much larger, however, since the entire CSO volume from the design storm must be contained. The treatment facility sedimentation basins are sized on a rate of flow basis to provide a specific overflow rate and detention time. The resulting basin volumes are much smaller. The size/capacity of pumping facilities, however, are greater for the treatment system since the design flow rate must be pumped to the receiving waters. The pump back rate for a storage-pump back system is relatively low since the stored volume is "bled" into the system over a longer period of time. The other major difference between these two types of CSO abatement technologies is disinfection. The treatment facility would have chemical storage and feed facilities to provide disinfection of the effluent using sodium hypochlorite, as well as provisions for dechlorination to eliminate the chlorine residual.

Figures 10-2 and 10-3 illustrate typical facilities required for a storage-pump back system. Similar facilities would be required for a treatment system.

As shown in these figures, flow from a consolidation conduit would enter the channel room and flow through one of two influent channels, each containing a mechanically-cleaned bar screen. Two channels are proposed in the event one of the bar screens is out of service for maintenance or repair. Manually-operated slide gates installed in each channel would direct flow through the selected channel. Each bar screen would consist of a mechanically-cleaned unit with the drive unit and screenings hopper located on the above floor level. The recommended bar spacing is 1.5 inches, which would serve to remove large solids and protect pumping units.



PLAN

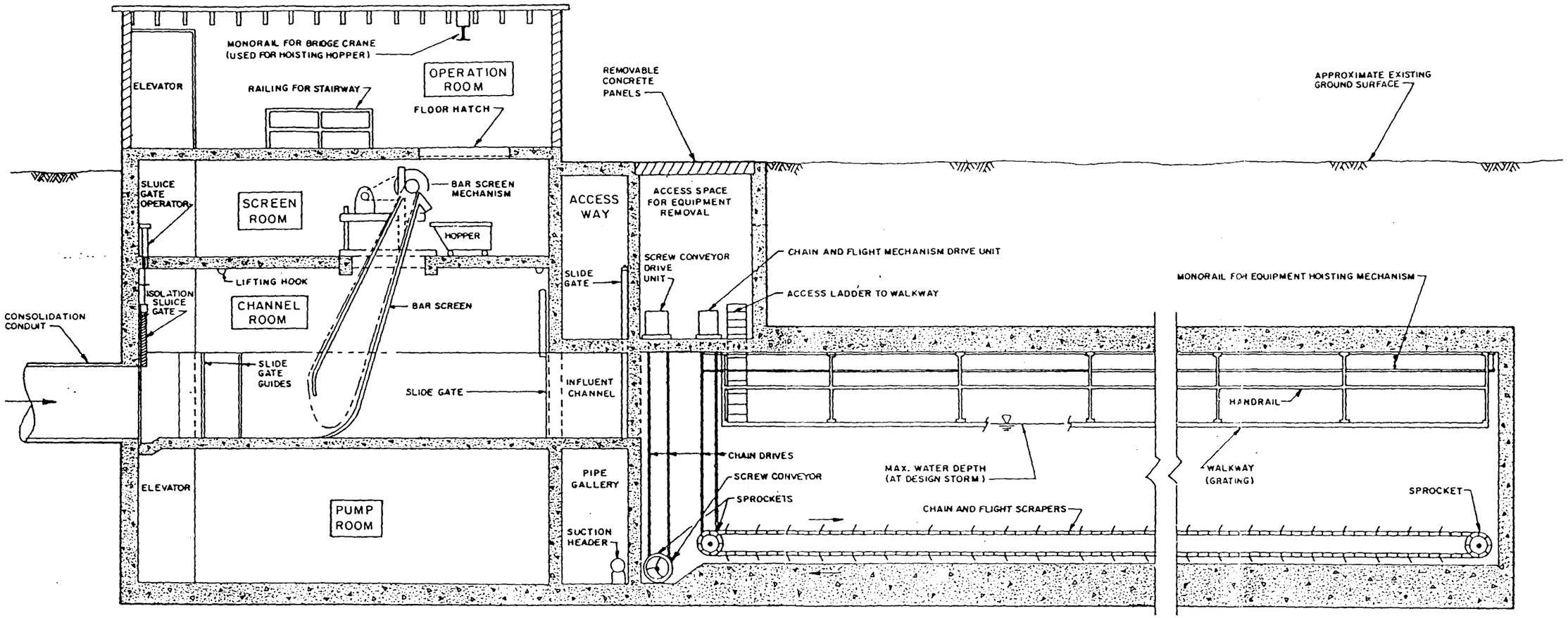
City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 10-2

Storage/Treatment Facility- Conceptual Plan

CDM



SECTION

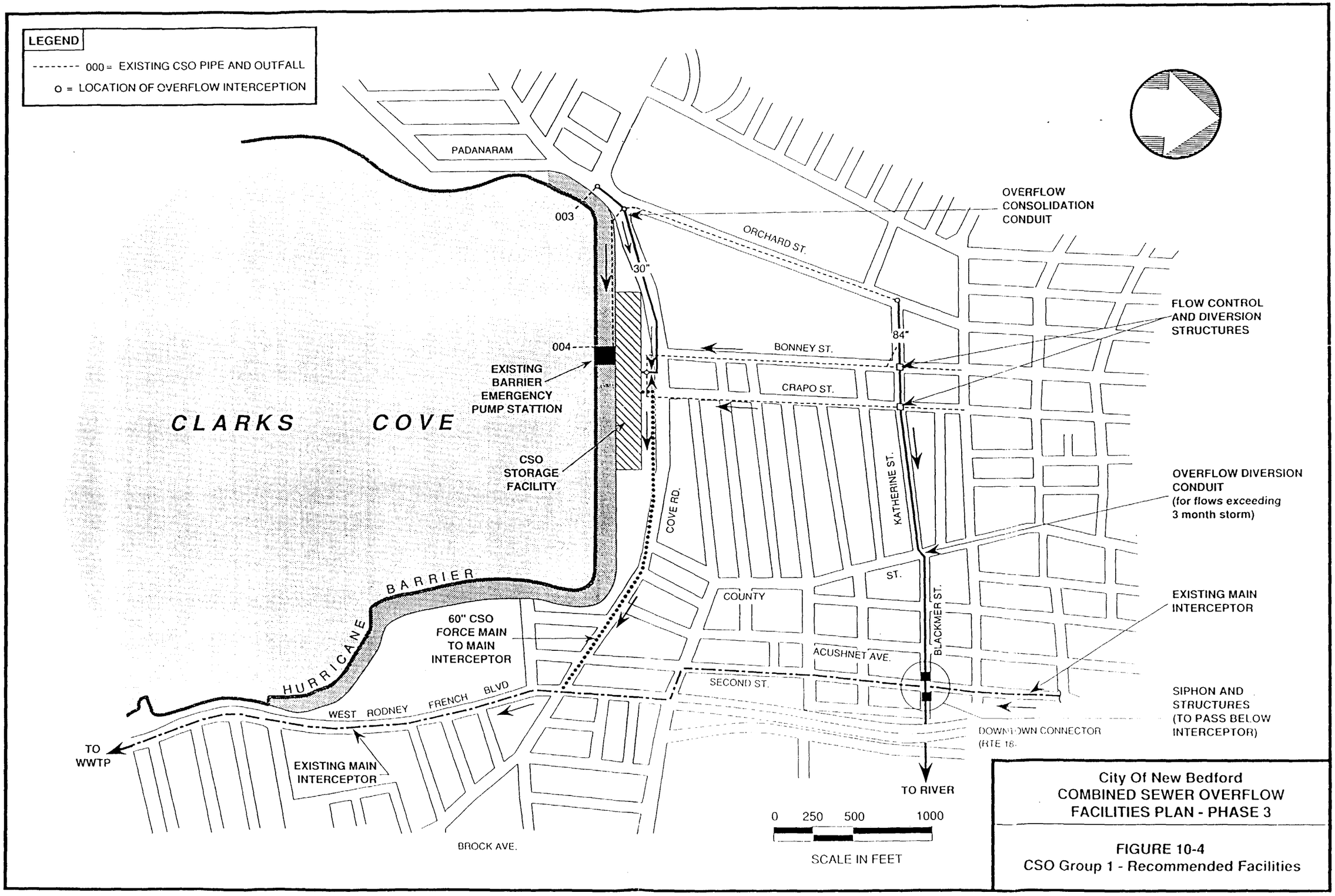
CDM

City of New Bedford

COMBINED SEWER OVERFLOW FACILITIES PLAN - PHASE 3

FIGURE 10-3

Storage/Treatment Facility- Sectional View



City Of New Bedford
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FIGURE 10-4
CSO Group 1 - Recommended Facilities

The screenings would be discharged from the rakes into a storage hopper located on the above floor level. A hopper would be provided for each bar screen. A monorail and bridge crane system located in the superstructure would be used to hoist the hoppers vertically through a floor hatch to the superstructure floor, and horizontally through doors to a truck access ramp. The hoppers would be emptied into a dump truck outside of the building to avoid operating the vehicle inside of the structure where potentially explosive gases may be present.

After the waste stream has passed through the screening unit, it would enter the influent channel for discharge into any one of the tank compartments. For a treatment system, a disinfectant (sodium hypochlorite) would be added to the waste stream here so that adequate contact time would be obtained in the sedimentation basins. Flow between and within the tank compartments would occur through a rectangular opening in the side wall which would act as a side overflow weir. The bottom elevation of the openings would be set 0.5 feet below the maximum effective depth in the tank.

Under normal operation of either a storage-pump back or treatment system, the slide gates controlling flow to compartments 2 through 6 would be closed, and the slide gate for compartment 1 open. Flow from the influent channel would enter compartment 1, and eventually overflow to compartment 2 if the CSO volume exceeds the compartment 1 capacity. As the volume increases, flow would enter the remaining compartments in series until the maximum water depth is reached in all compartments. This flow scheme could also be reversed by allowing flow to enter compartment 6 and flow in series to compartment 1. Both schemes would be practiced to equalize the use of sludge removal equipment. If flow continues for a storage-pump back system (i.e., if the storm CSO volumes exceed the design storm CSO volumes), the isolation sluice gate located in the channel room would automatically close, surcharging consolidation conduits and causing flow to discharge to the receiving water through existing CSO outfalls. For a treatment system, once all sedimentation basins are filled, treated effluent would overflow a weir located at the end of each basin (not shown on Figures 10-2 and 10-3)

and enter a wet well. Effluent pumps would then convey the treated waste stream to receiving waters.

As shown on Figures 10-2 and 10-3, sludge removal equipment for each compartment includes chain and flight scrapers that serve to transport settled solids into a trough located at the inlet end of the tank where a crew conveyor would convey them to a sump. The sludge collection mechanism drive units would be located on platforms above the inlet end of each compartment. Access to these units would be through the walkway located above the influent channel. Each platform would also contain an access ladder leading to a walkway installed along a side wall of each tank compartment. This walkway would be used for visual inspection of the sludge removal equipment. A monorail equipped with a hoisting mechanism would be installed along the ceiling of each tank compartment and would be used for removing sludge removal equipment in the event major maintenance, repair work, or replacement is required. Access to the tank for removal of the equipment would be provided at the inlet end of each compartment through removable concrete panels.

Odor control equipment has been included in the preliminary layout and cost estimate of all facilities. As currently conceived, an air exchange unit would exhaust air from the entire structure including the tank, pass it through activated carbon contained in plastic cells, and vent it through the roof of the structure. Fresh air would be drawn into the tank compartments and operations rooms through vents. A sill cock and hose would also be provided for washing down the screens between operations in order to reduce odors from accumulations.

The design of the storage facilities for Groups 1 through 4 was based on the criterion that the stored volumes would be pumped back into the Rodney French interceptor 18 to 20 hours after the rainstorm ended. The pump back rate will vary since it cannot exceed the difference between the design capacity of the sewer system (and WWTP) and the dry weather flow rate (which changes hourly). The difference can vary on a daily basis from 48 mgd during low groundwater periods, to 36 mgd during high groundwater periods. Recommended practice is to empty storage facilities prior to the

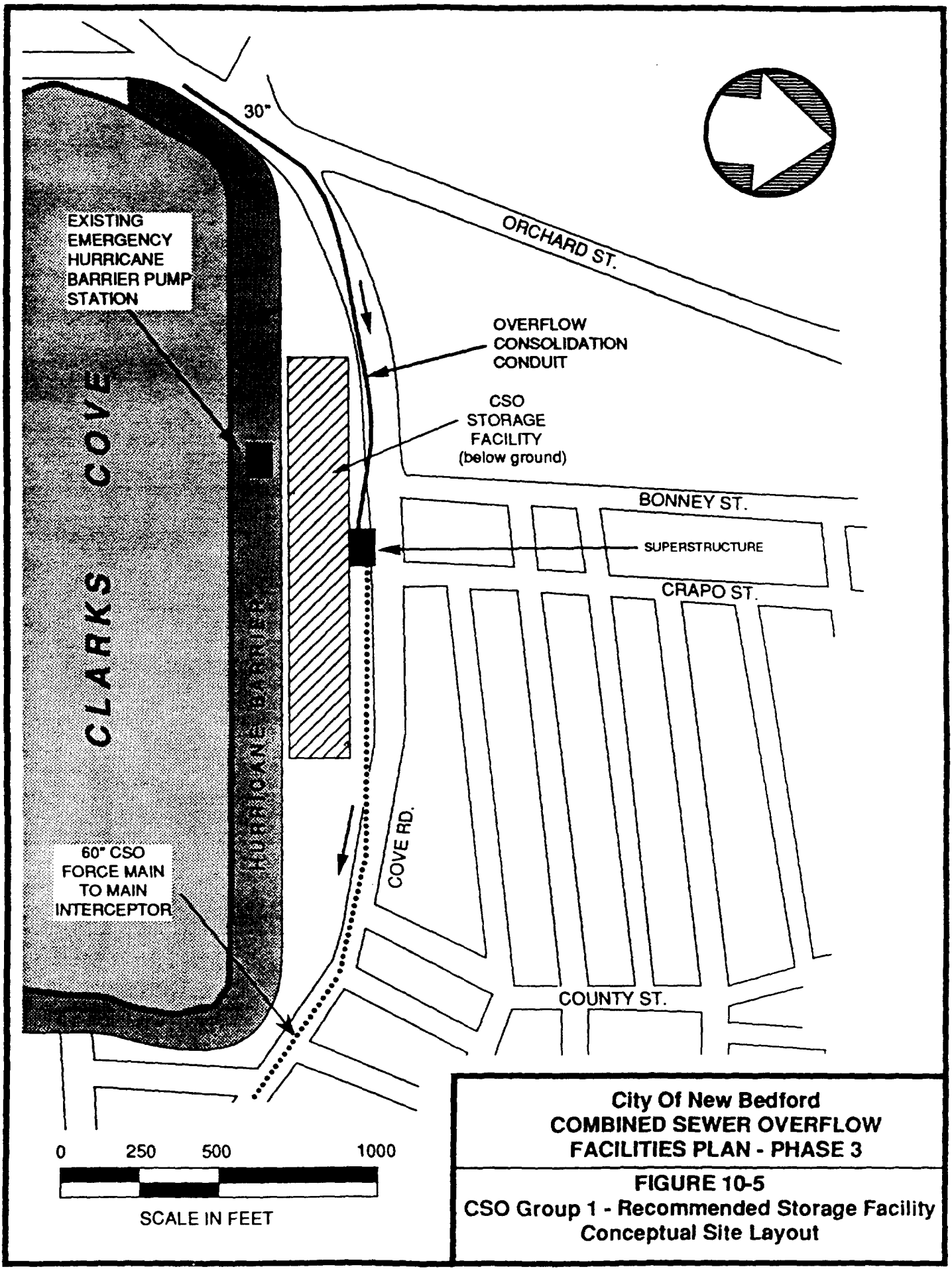
next rainstorm. For this study, the maximum acceptable pump back time was set at 18 to 20 hours after a rainstorm. Based on this criterion and the maximum pumping rates, it was determined that only flows from CSO Groups 1 through 4 could be stored and pumped back into the Rodney French interceptor for design storm events.

CSO Group 1

The recommended alternative for Group 1 consists of a storage facility that can contain combined sewer overflows generated by a 3-month frequency storm and associated piping as shown on Figure 10-4. A site layout is presented in Figure 10-5. Combined sewer overflow stored in the tanks would be pumped backed to the WWTP for treatment and discharge. The storage facility would be located in the field between the hurricane barrier and Cove Road. The storage tanks would be below ground and covered to maintain the present uses of the site, which include recreation (soccer fields) and flood storage. A structure (30 feet by 50 feet) would be located above ground to provide access to the screening facilities. The storage capacity of the facility would be 19.9 million gallons, and would have a pump back system designed for a capacity of 32 million gallons per day (mgd). The system would include six pumps: five with a capacity of 5,600 gpm each, and one for backup.

Approximately 1,400 feet of 48-inch diameter force main would be needed to link the storage facility to the Rodney French interceptor. The force main would extend east from the storage facility and connect to the main interceptor at the intersection of Cove Road and West Rodney French Boulevard, as shown on Figure 10-4.

The piping needed to consolidate CSO outlets 003 and 004 consists of approximately 1,000 feet of 24-inch gravity sewer extending east from the intersection of Pandanaram Street and Cove Road to the storage facility. The storage facility is close to where CSO 004 overflows so that new piping needs are minimal.



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-5
CSO Group 1 - Recommended Storage Facility
Conceptual Site Layout

As shown on Figure 10-4, a second pipeline is included in the recommended plan for Group 1. Because Clarks Cove has the greatest social and economic value to the City of three receiving waters, and its restoration will enhance the City's economic future, it warrants the highest degree of CSO control (i.e. elimination). The second pipeline would be used to capture overflows from the recommended storage facility when storms larger than the 3-month storm occur. Instead of having these flows discharge directly to Clarks Cove, they would be captured and routed via 3,600 feet of 72-inch pipe to the Inner Harbor where they would be discharged. The pipeline would cross the Rodney French interceptor near Blackmer Street. A siphon consisting of twin 48-inch diameter pipes would be required for the crossing. An inlet structure and an outlet structure would be built at the ends of the 100-foot long siphon pipes. Because the siphon would be placed beneath the Rodney French interceptor, significant rock excavation would be required.

This second pipeline increases the estimated cost of the abatement plan for Group 1. The capital cost to achieve a 3-month level of control (i.e., reduce the frequency of discharge from 45 times per year to 4 times per year) is estimated to be $57.2 million. This equates to a unit cost of $1.27 million per overflow day eliminated. The capital cost to build the second pipeline, which would further reduce the overflow frequency to zero discharges per year, would be $6.5 million for a total cost of $63.7 million. The unit cost for this additional level of control is $1.63 million per overflow day eliminated.

The added pipeline would be used on average four times per year. When storms larger than the 3-month storm occur and the storage facility fills up, an electronic signal would be sent to close a valve near Rivet and Bonney Streets, thereby preventing any more flow north of Katherine Street from reaching the storage facility. Instead, the flow would be redirected into the added pipeline and discharged to the Inner Harbor. Flow from the drainage area south of Rivet Street would still reach the storage facility. A small pump would be needed at the storage facility to prevent the collection system south of Rivet Street from backing up when the storage facility is full. If the storage facility is full and it is still raining,

it would be pumped out to a specified level to provide more capacity until the rainstorm ends. Flow would be pumped directly into the added pipeline/siphon and discharged to the Inner Harbor.

A summary of the conceptual design criteria and functional features of the recommended facilities for CSO Group 1 is presented in Table 10-2.

CSO Group 2

The recommended alternative for CSO Group 2 consists of a storage-pump back facility designed to hold up to a 12-month storm. The storage facility would be located in Hazelwood Park as shown on Figure 10-6. A site layout is presented in Figure 10-7. Because the overflows from CSO Group 2 are relatively small, the storage tanks need to contain only 1.7 million gallons. As with CSO Group 1, the storage facility would be built below grade so as not to interfere with continued recreational use of the park. Odor control would also be included. The screening structure (20 feet by 30 feet) at the influent to the tank would be located above grade. The pump back system would contain 3 pumps: two pumps would have a combined capacity to meet the design pump back rate of 1.3 mgd, and the third would serve as a backup. The storage facility is located close enough to the main interceptor in West Rodney French Boulevard that the force main length needed to connect would only be about 100 feet.

Consolidating CSOs 005 through 011 will require a total of 5,300 feet of pipe: the pipe would consist of a 15-inch gravity sewer extending south from David Street to the storage facility, and north from Hudson Street to the storage facility. Both pipelines would be located in West Rodney French Boulevard. A new regulator would be required at each intersection where consolidation sewers connect with the existing CSO outfalls.

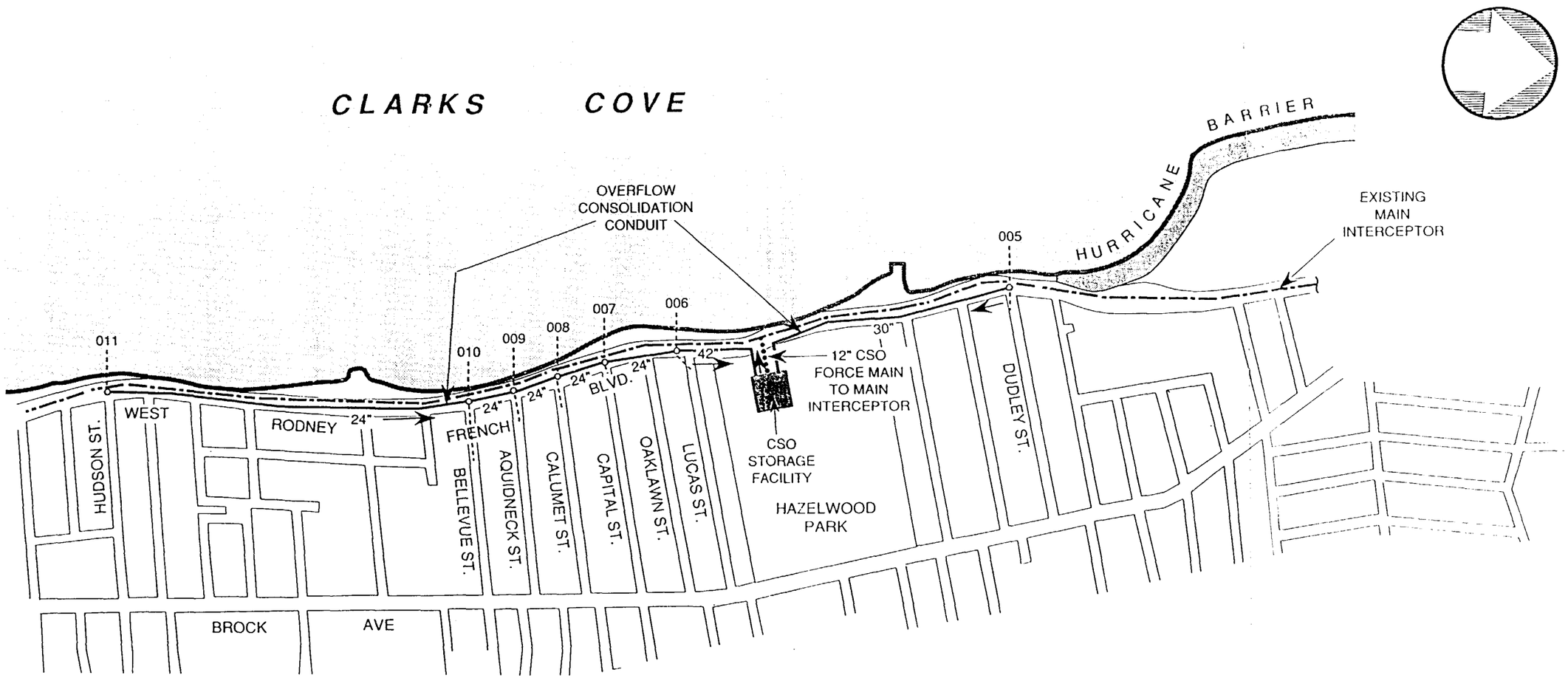
A summary of the conceptual design criteria and functional features for the recommended facilities for CSO Group 2 is presented in Table 10-3.

Note that a 12-month level of control is recommended for Group 2. As mentioned previously, Clarks Cove is the receiving water with the highest

TABLE 10-2

CSO GROUP 1 STORAGE FACILITY
CONCEPTUAL DESIGN CRITERIA AND FUNCTIONAL FEATURES

Facility Type	Storage
Design Storm Frequency	3-month, 6-hour
Design Volume (mil. gal.)	19.9
Screen Type and Spacing (in.)	Catenary; 1.0
Estimated Annual Screenings Volume (cu. ft.)	900
Number of Tank Compartments	16
Tank Dimensions - Each Compartment (ft.)	40 x 230
Sidewater Depth (ft.)	18
Pumpback System	6 Pumps @ 5,600 gpm
Sludge Collection	Chain and Flight Scrapers and Screw Conveyors
Sludge Disposal	Pump with Return Storage
Estimated Total Annual Waste Stream Volume (mil. gal.)	299



LEGEND

-------- 000 = EXISTING CSO PIPE AND OUTFALL

o = LOCATION OF OVERFLOW INTERCEPTION

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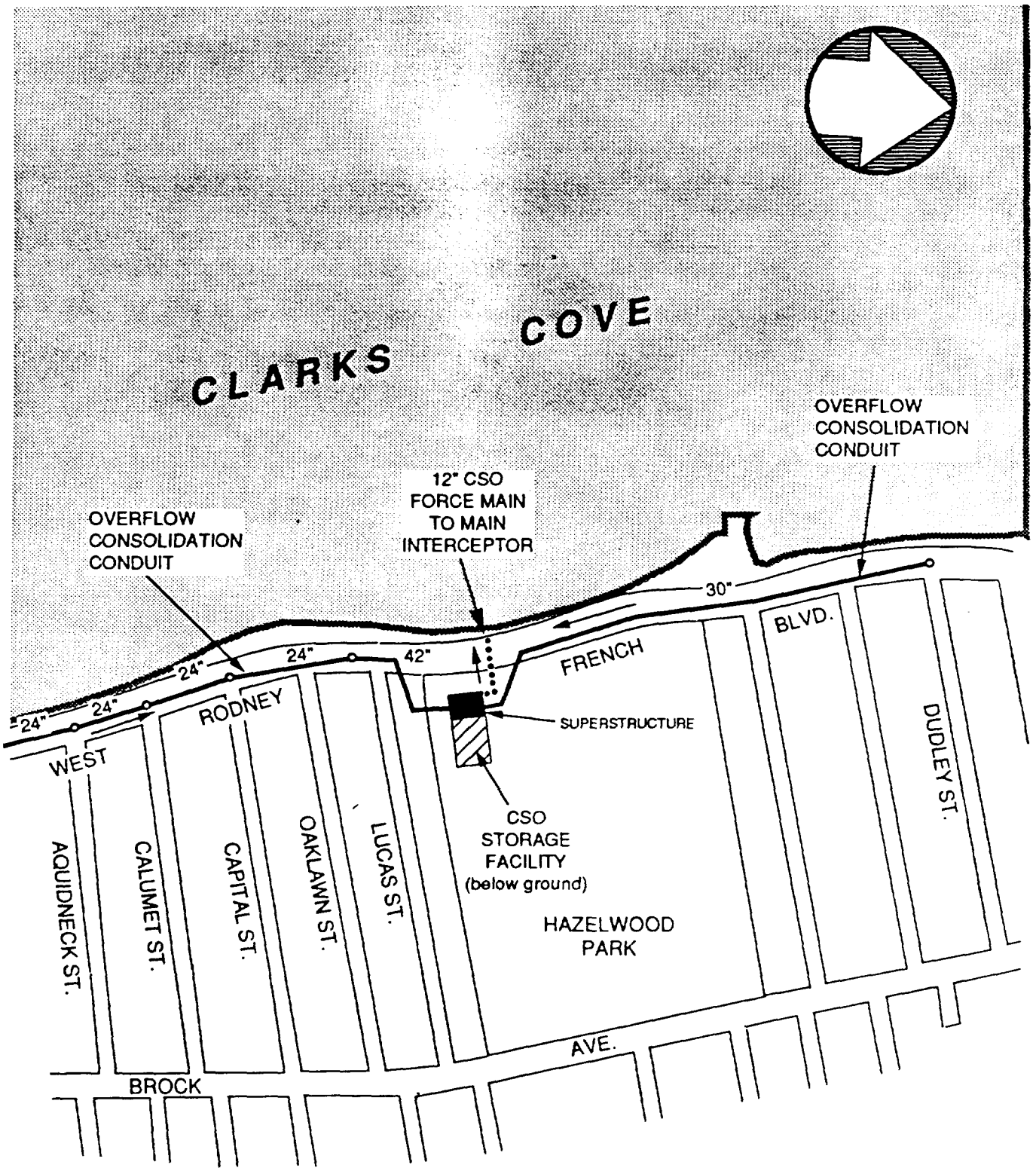
SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN -PHASE 3

FIGURE 10-6
CSO Group 2 - Recommended Facilities



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SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-7
CSO Group 2 - Recommended Storage Facility
Conceptual Site Layout

TABLE 10-3

CSO GROUP 2 STORAGE FACILITY
CONCEPTUAL DESIGN CRITERIA AND FUNCTIONAL FEATURES

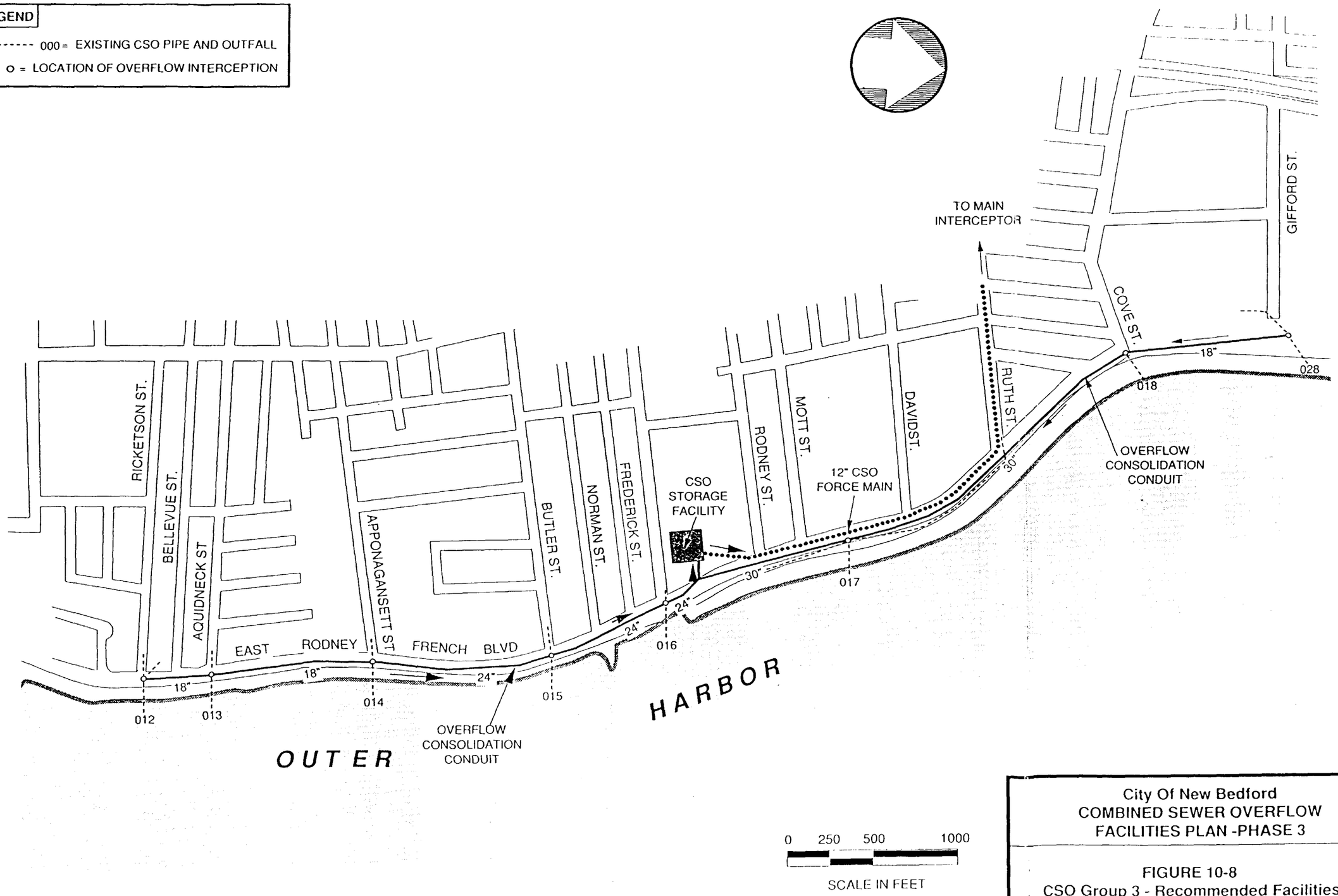
Facility Type	Storage
Design Storm Frequency	12-month, 6-hour
Design Volume (mil. gal.)	1.66
Screen Type and Spacing (in.)	Catenary; 1.0
Estimated Annual Screenings Volume (cu. ft.)	1
Number of Tank Compartments	3
Tank Dimensions - Each Compartment (ft.)	40 x 155
Sidewater Depth (ft.)	12
Storage Pumping System	3 Pumps @ 900 gpm
Sludge Collection	Chain and Flight Scrapers and Screw Conveyors
Sludge Disposal	Pump with Return Storage
Estimated Total Annual Waste Stream Volume (mil. gal.)	.34

priority. Because overflows from Group 1 would be "eliminated," a comparable level of control is warranted for Group 2, which also discharges into Clarks Cove. Under the recommended plan, overflows from Group 2 would occur on average less than once per year.

CSO Group 3

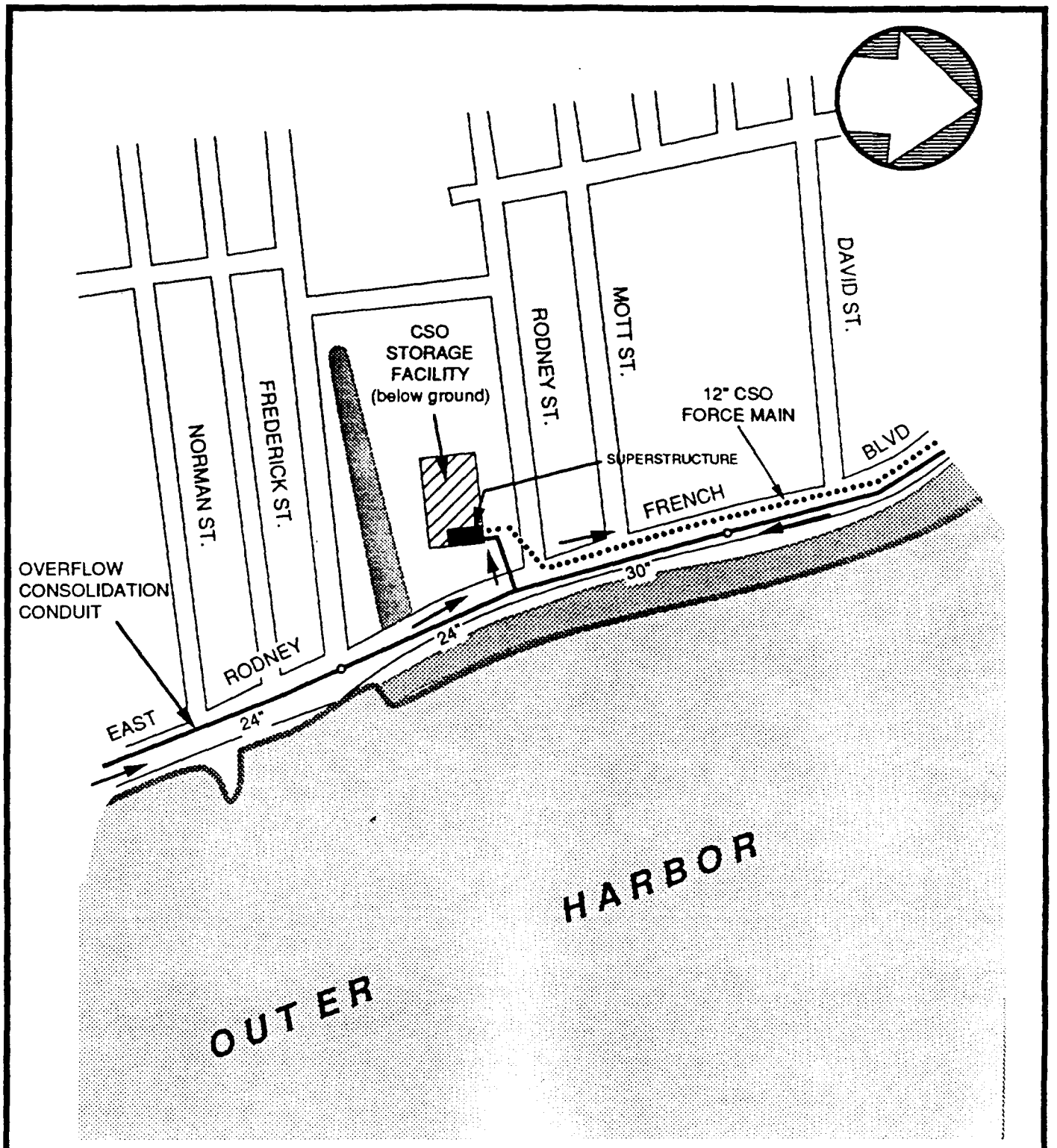
The recommended alternative for CSO Group 3 consists of a storage-pump back facility designed to hold the volume of combined sewer overflow generated by a 3-month storm. Overflows from storms larger than the 3-month storm would be discharged to the Outer Harbor at the existing overflows. The storage facility would be located north of the hurricane barrier abutting East Rodney French Boulevard as shown on Figure 10-8. A site layout is presented in Figure 10-9. The tanks would have a capacity of 1.45 million gallons and would be below grade. Because the invert of influent pipe is deep, the storage tanks must also be deep. A second reason for the tanks to be deep is because of the small parcel of land being proposed for use. The large volume of combined sewer overflow generated by a 3-month storm can only be placed underground on the small land parcel by placing storage tanks in a deep excavation. A small structure (25 feet by 40 feet) would be located above grade to provide access to the screening room. The pump back facility would include 3 pumps: two pumps would have a capacity of 900 gpm each, and a third would serve as backup.

A 12-inch diameter force main would extend from the facility north beneath East Rodney French Boulevard to David Street, and west beneath David Street to the main interceptor at West Rodney French Boulevard. Gravity sewers would extend south from Cove Street and north from Ricketson Street to the storage facility. From Cove Street, approximately 1,200 feet of 18-inch and 1,800 feet of 24-inch pipe would be needed. From Ricketson Street, approximately 1,800 feet of 24-inch and 900 feet of 30-inch pipe would be needed. The pipelines would be located in East Rodney French Boulevard. A new regulator would be required at each intersection where consolidation sewers connect to the existing CSO outfalls.



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN -PHASE 3

FIGURE 10-8
CSO Group 3 - Recommended Facilities



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SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-9
CSO Group 3 - Recommended Storage Facility
Conceptual Site Layout

A summary of the conceptual design criteria and functional features of the recommended facilities for CSO Group 3 is presented in Table 10-4.

CSO Group 4

The recommended alternative consists of a storage-pump back facility designed to contain up to a 3-month storm. Overflows from storms larger than the 3-month storm would be discharged to the Inner Harbor at the existing overflows. The storage facility would be located on Hassey Street just east of Cape Street, as shown on Figure 10-10. A site layout is presented in Figure 10-11. The tanks would have a capacity of 7.63 million gallons and would be below grade. Due to the depth of the influent pipe, the tanks must also be relatively deep because the proposed parcel of land to be used is small. A small structure (30 feet by 60 feet) would be located above grade to provide access to the screening room. The pump back facility would have a design capacity of 12.21 mgd and would include 3 pumps: two would have a capacity of 4,300 gpm each, which would be sufficient to meet the design pump back rate, and one would serve as a backup.

The storage facility would connect to the Rodney French interceptor at Second Street. Approximately 1,300 feet of 30-inch diameter force main would be needed.

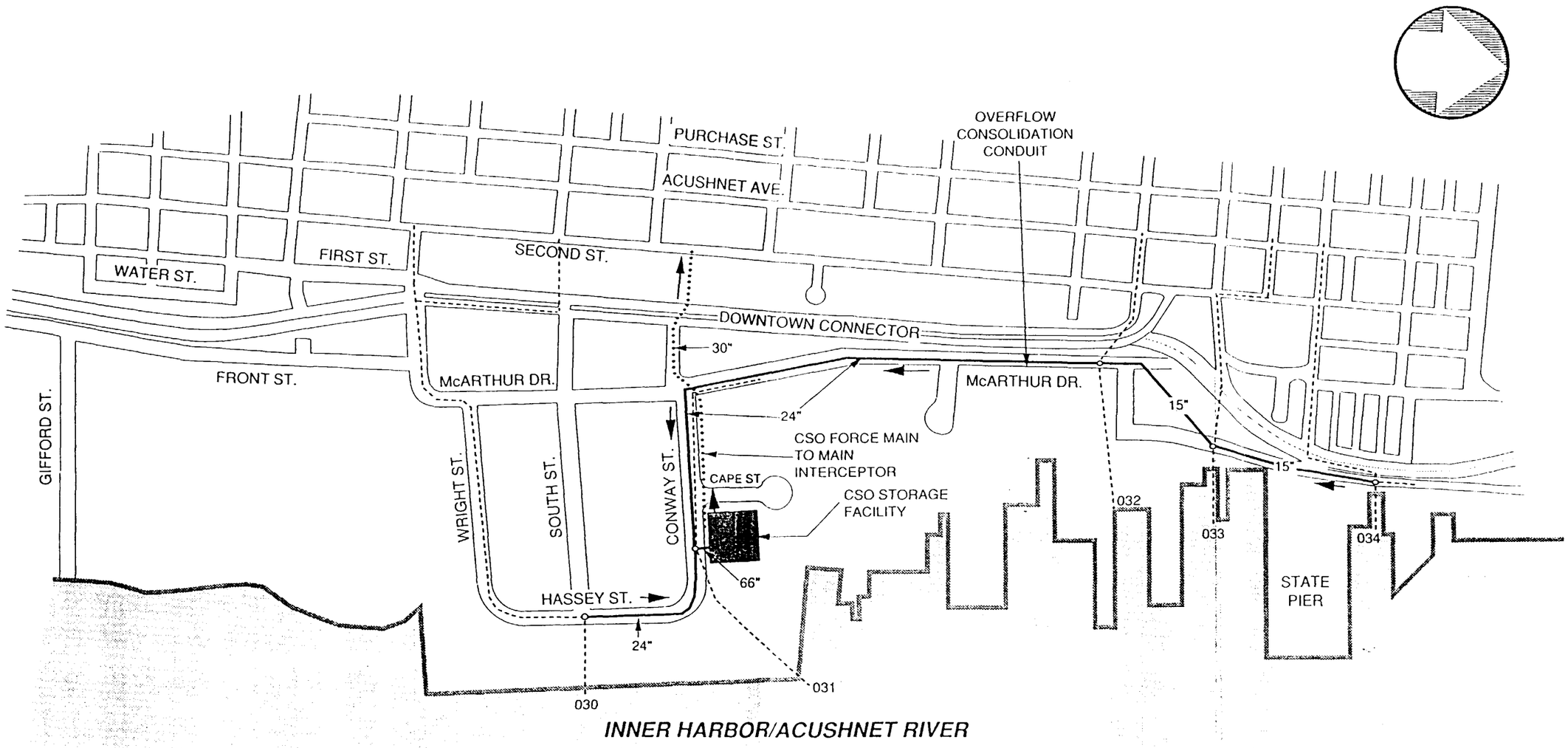
The piping needed to consolidate CSOs 030 through 034 consists of approximately 700 feet of 24-inch gravity sewer extending north from South Street along Hassey Street to the storage facility. Consolidation would also require approximately 1,100 feet of 15-inch gravity sewer, 3,250 of 24-inch gravity sewer, and 150 feet of 66-inch gravity sewer extending south beneath McArthur Drive and Conway Street to the storage facility.

A summary of the conceptual design criteria and functional features for the facilities recommended for Group 4 is presented in Table 10-5.

TABLE 10-4

CSO GROUP 3 STORAGE FACILITY
CONCEPTUAL DESIGN CRITERIA AND FUNCTIONAL FEATURES

Facility Type	Storage
Design Storm Frequency	3-month, 6-hour
Design Volume (mil. gal.)	1.45
Screen Type and Spacing (in.)	Catenary; 1.0
Estimated Annual Screenings Volume (cu. ft.)	65
Number of Tank Compartments	3
Tank Dimensions - Each Compartment (ft.)	40 x 140
Sidewater Depth (ft.)	12
Storage Pumping System	3 Pumps @ 800 gpm
Sludge Collection	Chain and Flight Scrapers and Screw Conveyors
Sludge Disposal	Pump with Return Storage
Estimated Total Annual Waste Stream Volume (mil. gal.)	21.7



LEGEND

-------- 000 = EXISTING CSO PIPE AND OUTFALL

o = LOCATION OF OVERFLOW INTERCEPTION

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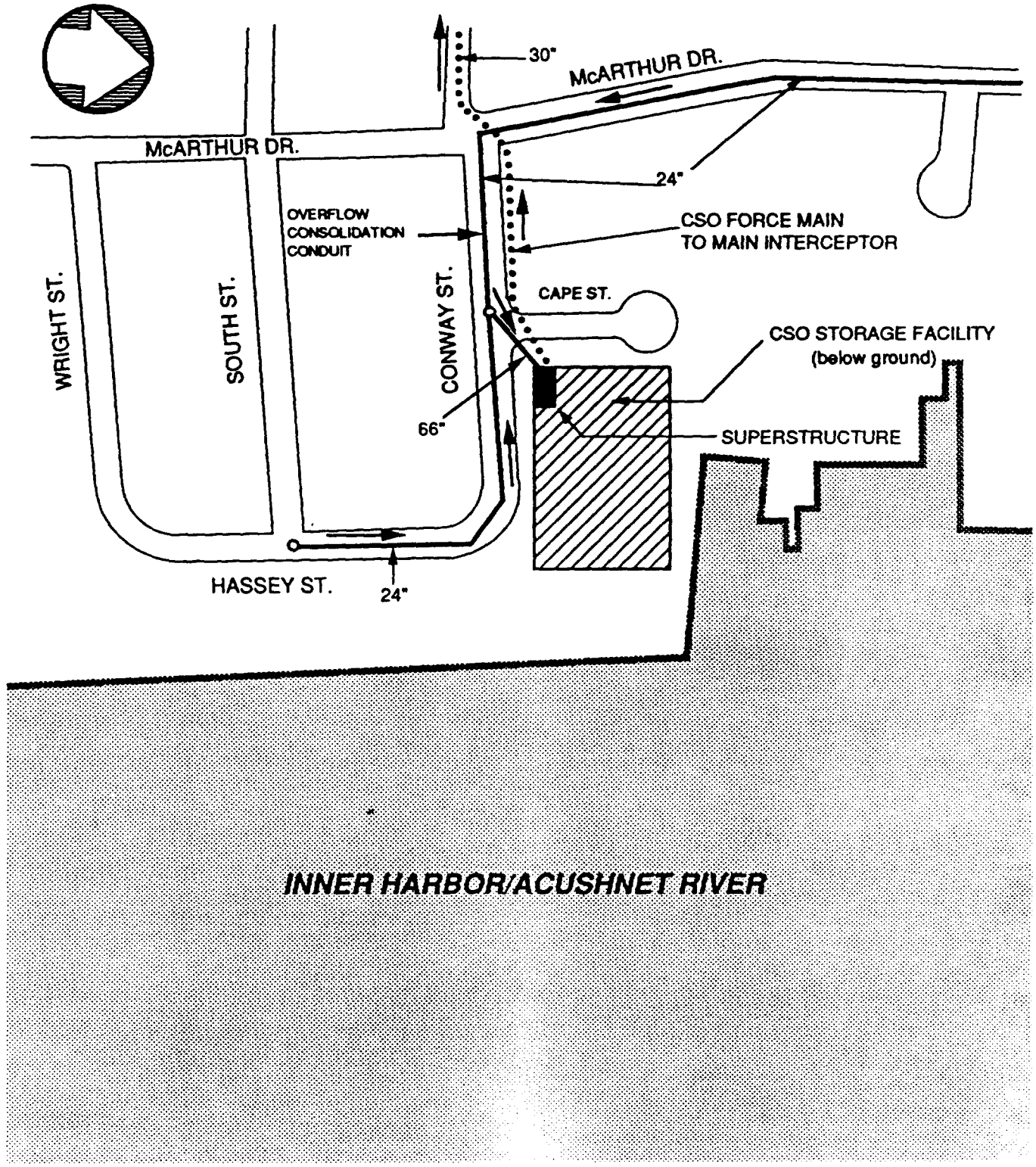
SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-10
CSO Group 4 - Recommended Facilities



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SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-11
CSO Group 4 - Recommended Storage Facility
Conceptual Site Layout

TABLE 10-5

CSO GROUP 4 STORAGE FACILITY
CONCEPTUAL DESIGN CRITERIA AND FUNCTIONAL FEATURES

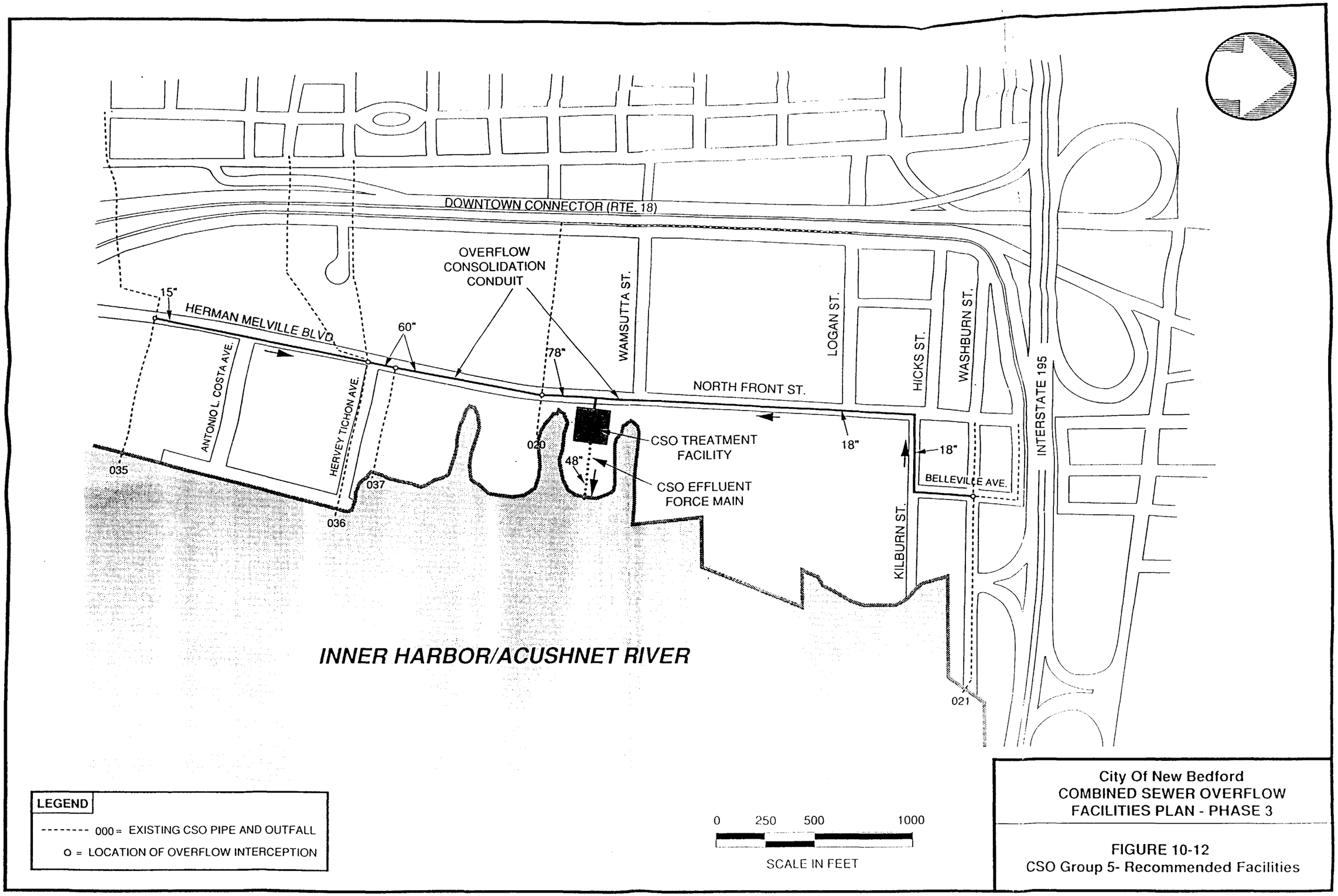
Facility Type	Storage
Design Storm Frequency	3-month, 6-hour
Design Volume (mil. gal.)	7.63
Screen Type and Spacing (in.)	Catenary; 1.0
Estimated Annual Screenings Volume (cu. ft.)	210
Number of Tank Compartments	10
Tank Dimensions - Each Compartment (ft.)	40 x 215
Sidewater Depth (ft.)	12
Storage Pumping System	3 Pumps @ 4,300 gpm
Sludge Collection	Chain and Flight Scrapers and Screw Conveyors
Sludge Disposal	Pump with Return Storage
Estimated Total Annual Waste Stream Volume (mil. gal.)	70.1

CSO Group 5

Under the design storm conditions, the combined sewer overflow volumes pumped back to the Rodney French interceptor from the four separate storage tanks for CSO Groups 1 through 4 is all that can be pumped back within 18 to 20 hours after a rainstorm. Although additional storage sites and capacity are available, there is no means of conveying the additional stored volumes to the Rodney French interceptor within the allotted time. Because it is not possible to store and treat any flow from CSO Groups 5 or 6 at the secondary WWTP, Groups 5 and 6 have to have separate satellite treatment facilities.

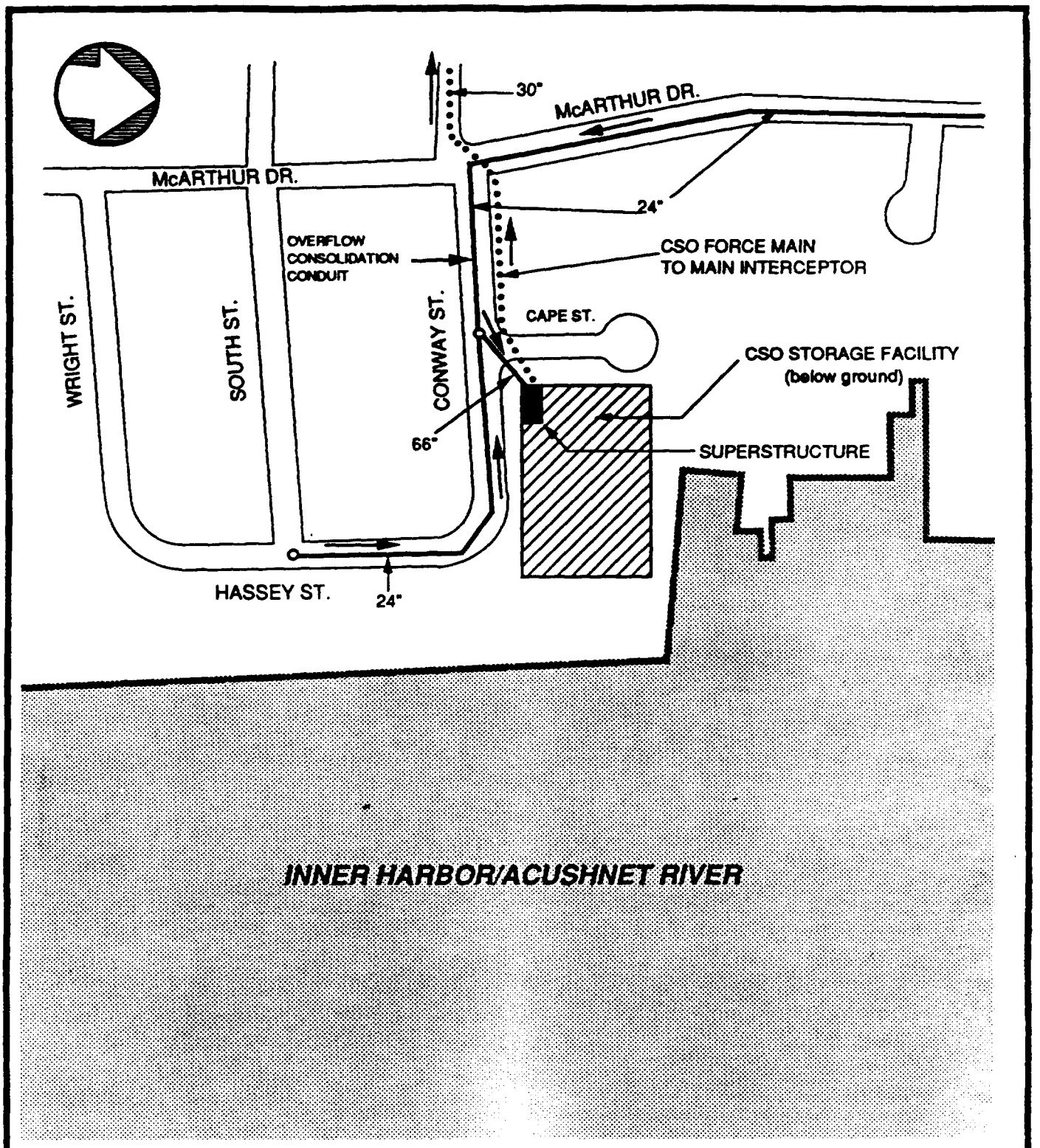
The recommended alternative for CSO Group 5 consists of a treatment facility designed for a 3-month storm. The treatment facility would be located south of Wamsutta Street and east of Herman Melville Boulevard, as shown on Figure 10-12. A site layout is presented in Figure 10-13. The facility would include sedimentation basins capable of holding up to 2 million gallons and designed for a peak treatment capacity of 34 mgd. Because storage is available in the sedimentation basins, the required effluent pumping rate is slightly lower than the peak 3-month storm influent rate. The facility would be located below grade and covered. The effluent pump system would consist of 4 pumps with a capacity of 7,900 gpm each; three would have a combined capacity to meet the design flow, and the fourth would serve as backup. The effluent would be piped for discharge to the Inner Harobr at CSO 037. A small pumping system would be needed to pump settled solids to the existing Wamsutta Street Pumping Station. This pump system would include 3 pumps, each capable of pumping 100 gpm.

The piping needed to consolidate the CSOs in Group 5 would extend south from the intersection of Washburn Street and Belleville Avenue, Kilburn Street and North Front Street, and north from CSO 035 along Herman Melville Boulevard to the storage facility (see Figure 10-8). The piping would include approximately 1,100 feet of 15-inch gravity sewer, 950 feet of 60-inch gravity sewer, and 300 feet of 78-inch gravity sewer extending north along Herman Melville Boulevard to the storage facility. Also needed would be 2,300 feet of 18-inch gravity sewer extending south along



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-12
CSO Group 5- Recommended Facilities



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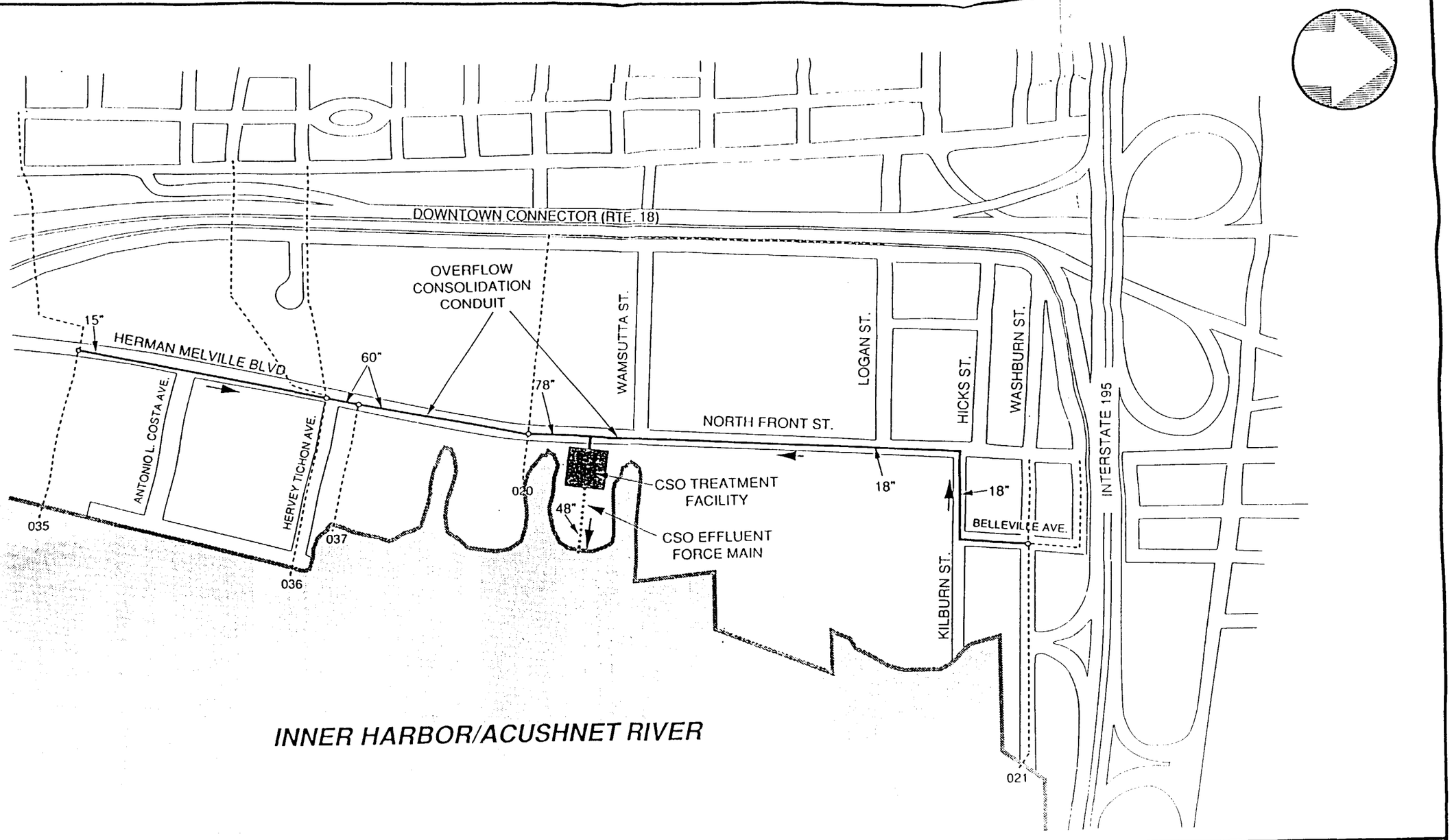
SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-11
CSO Group 4 - Recommended Storage Facility
Conceptual Site Layout



LEGEND

-------- 000 = EXISTING CSO PIPE AND OUTFALL

o = LOCATION OF OVERFLOW INTERCEPTION

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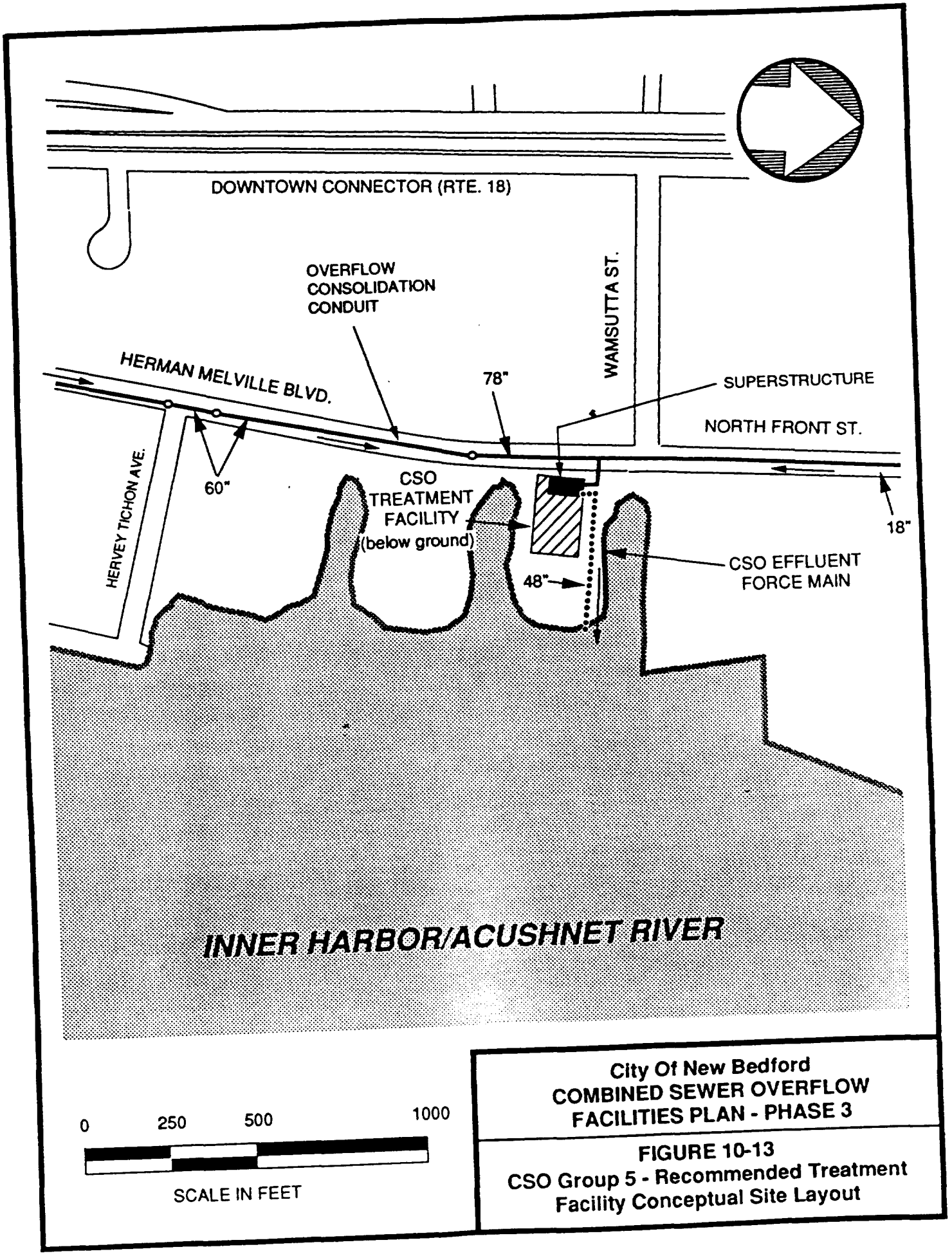
SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-12
CSO Group 5- Recommended Facilities



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-13
CSO Group 5 - Recommended Treatment
Facility Conceptual Site Layout

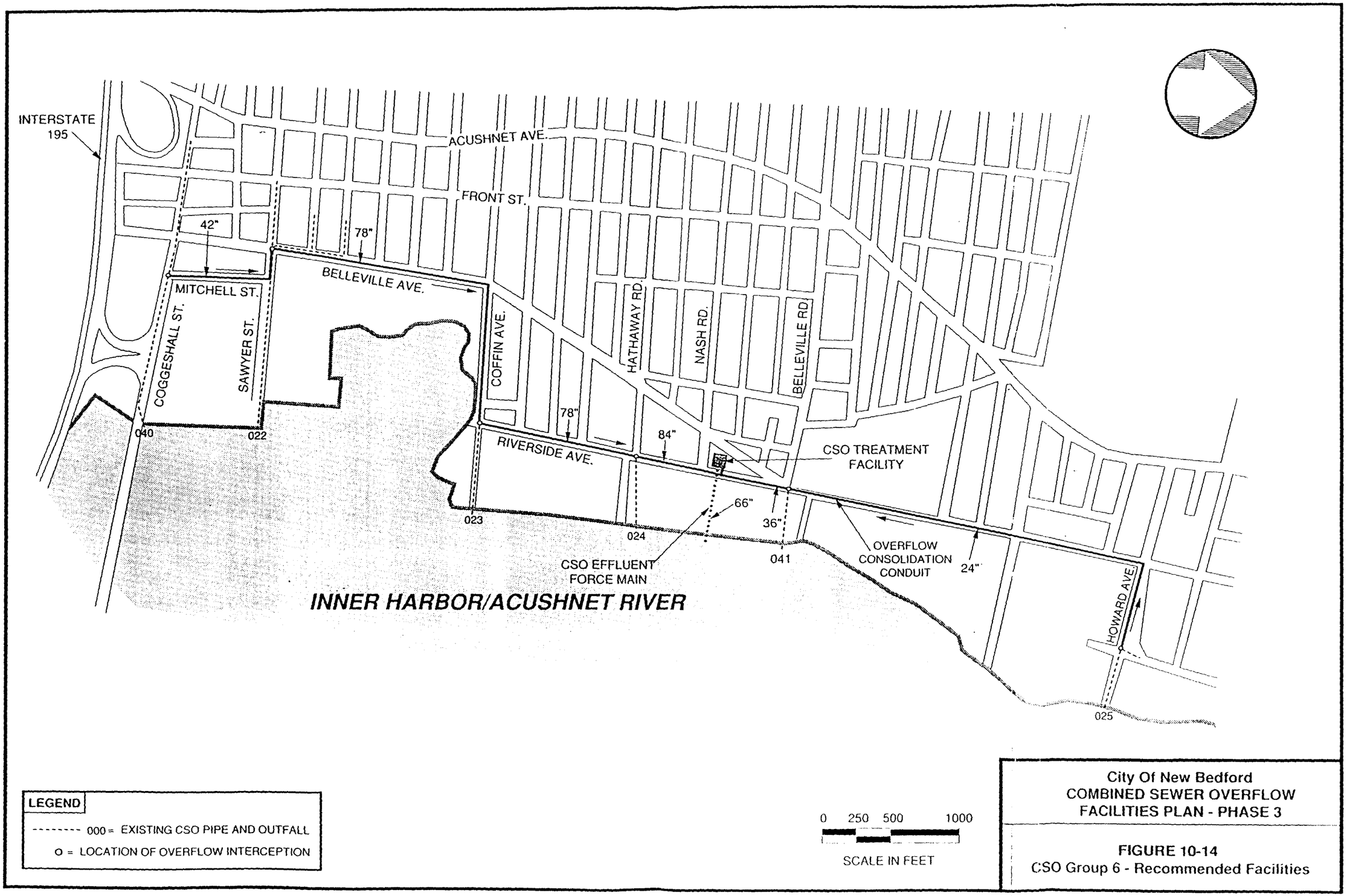
Belleville Avenue, Kilburn Street and North Street. A new regulator would be required at each intersection where a consolidation sewer connects with an existing CSO outfall.

A summary of the conceptual design criteria and functional features for the recommended CSO Group 5 facilities is presented in Table 10-6.

CSO Group 6

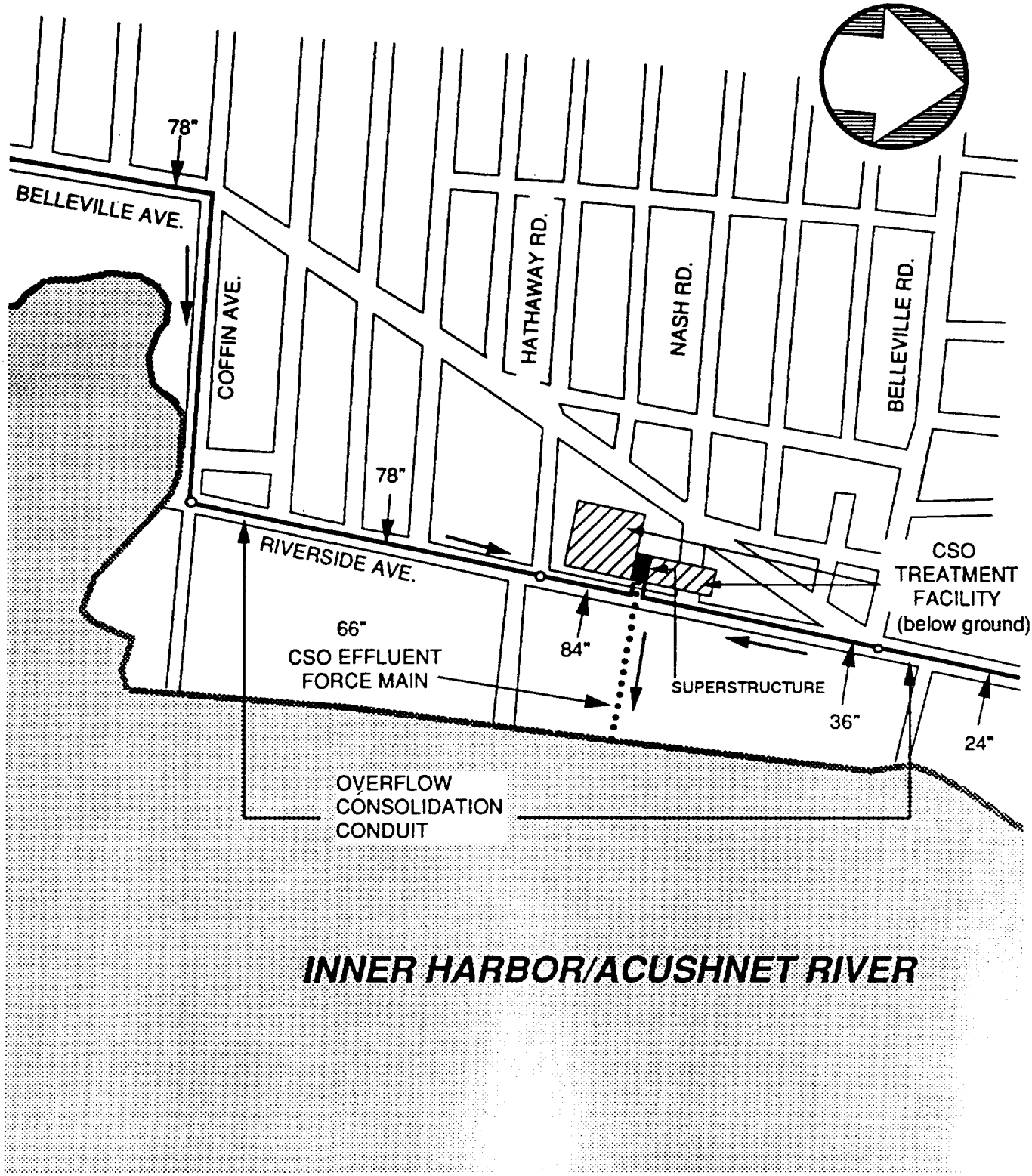
The recommended alternative for CSO Group 6 consists of a treatment facility designed for a 3-month storm located on the north side of Nash Road at its intersection with Riverside Avenue in an area currently used as public parking lots for local businesses. This facility and related components are shown in Figure 10-14. A site layout is presented in Figure 10-15. The facility would be sized for a peak treatment capacity of 54 mgd and have a total tank volume of 3.23 million gallons. Similar to the Group 5 facility, this facility takes advantage of available storage in the sedimentation basin to reduce the required treatment rate. The storage facility would be located below grade and covered. The effluent pump system would consist of 4 pumps with a capacity of 1,200 gpm each: three would have a combined capacity to meet the design flow, and one would serve as a backup. The effluent pipe from the facility, which would be about 350-feet long and 66-inches in diameter, would cross Riverside Avenue and discharge to the Inner Harbor east of the Belleville Pump Station. A small pumping system would be needed to pump settled sludge to the existing Belleville Pumping Station. This pumping system would contain 3 pumps with approximately 100 gpm of capacity each.

As indicated on Figure 10-9, the piping necessary to consolidate the CSOs would extend from the intersection of Coggeshall Street and Mitchell Street, west from the intersection of Mitchell Street and Beetle Street, north from the intersection of Beetle Street and Belleville Avenue, east from the intersection of Belleville Avenue and Coffin Avenue, and north from the intersection of Coffin Avenue and Riverside Avenue to the treatment facility. Additional piping would extend west from the regulator at Howard Avenue Pumping Station, and south from the intersection of Howard



City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-14
CSO Group 6 - Recommended Facilities



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SCALE IN FEET

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-15
CSO Group 6 - Recommended Treatment Facility Conceptual Site Layout

TABLE 10-6

CSO GROUP 5 TREATMENT FACILITY
CONCEPTUAL DESIGN CRITERIA AND FUNCTIONAL FEATURES

Facility Type	Treatment
Design Storm Frequency	3-month, 6-hour
Design Capacity (mgd)	34
Screen Type and Spacing (in.)	Catenary; 1.0
Estimated Annual Screenings Volume (cu. ft.)	170
Number of Tank Compartments	3
Tank Dimensions - Each Compartment (ft.)	40 x 190
Sidewater Depth (ft.)	18
Effluent Pumping System	4 Pumps @ 7,900 gpm
Sludge Collection	Chain and Flight Scrapers and Screw Conveyors
Sludge Disposal	Pump to Interceptor
Estimated Total Annual Waste Stream Volume (mil. gal.)	55.4

Avenue and Belleville Avenue to the treatment facility. A new regulator would be required at each intersection where a consolidation sewer connects with an existing CSO outfall.

The piping would require approximately 850 feet of 42-inch gravity sewer, 3,800 feet of 78-inch gravity sewer, and 700 feet of 84-inch gravity sewer extending from Mitchell Street to Riverside Avenue. Approximately 2,950 feet of 24-inch gravity sewer and 650 feet of 36-inch gravity sewer would be needed between Howard Avenue and Riverside Avenue.

Also included under Group 6 recommendation is separation of the area tributary to outfalls 026 and 027 (Figure 10-14). This area is about 39 acres and is located at the northern extremity of the combined sewer system. As discussed in Section 9.0, partial separation is a viable and acceptable option for CSO abatement for Group 6 considering the limited use and economic value of the receiving water.

Separation for this area proved cost-effective as compared to the conveyance and treatment of CSOs from 026 and 027 at the proposed Nash Road treatment facility. The cost of increasing the size and capacity of the facility was greater than the cost of separating the 39 acres of combined sewers (see Section 9.0). This was primarily due to the additional depth of the facility (about 5 feet) and required piping necessary to convey CSOs from 026 and 027 to the facility by gravity.

The conceptual design criteria and functional features of the recommended facilities for CSO Group 6 is presented in Table 10-7.

10.2.2 SYSTEM-WIDE STRUCTURAL COMPONENTS

In addition to the 6 discrete CSO abatement projects, several improvements that will be made to the wastewater collection system to increase pipe capacities and decrease CSO spills. These improvements are identified, described, and costed in the draft Phase 2 WWTP Facilities Plan submitted in August 1989. The improvements will directly enable more flow to reach the Rodney French interceptor and the WWTP. The improvements include

TABLE 10-7

CSO GROUP 6 TREATMENT FACILITY
CONCEPTUAL DESIGN CRITERIA AND FUNCTIONAL FEATURES

Facility Type	Treatment
Design Storm Frequency	3-month, 6-hour
Design Capacity (mgd)	54
Screen Type and Spacing (in.)	Catenary; 1.0
Estimated Annual Screenings Volume (cu. ft.)	370
Number of Tank Compartments	6
Tank Dimensions - Each Compartment (ft.)	40 x 150
Sidewater Depth (ft.)	12
Effluent Pumping System	4 Pumps @ 12,000 gpm
Sludge Collection	Chain and Flight Scrapers and Screw Conveyors
Sludge Disposal	Pump with Interceptor
Estimated Total Annual Waste Stream Volume (mil. gal.)	123

renovations to existing pumping stations, force mains, and sewers to correct equipment or capacity deficiencies. The existing Rodney French interceptor will have to be cleaned because there are grit deposits in various sections. In addition, modifications will be made to five existing CSO regulators.

The recommended plan for CSO abatement is based on the assumption that all of these improvements will have been made by the time the CSO facilities are constructed. As described in Section 4.0, the "Base Conditions" used to determine CSO spill volumes under both the no-action and recommended plans assume that the grit in the main interceptor has been addressed, and the CSO regulators have been modified, as discussed in Section 7.0. A detailed discussion of the recommended system improvements can be found in the draft Phase 2 WWTP Facilities Plan.

10.2.3 NON-STRUCTURAL COMPONENTS

Improved sewer system maintenance practices can increase system efficiency, decrease CSO discharges, and improve water quality. The City undertakes several best management practices (see Section 7.0). Because these practices improve water quality by reducing CSO discharges and/or loads, they have been incorporated into the recommended plan. The best management practices are summarized briefly below.

Catch Basin Cleaning

The city has initiated a catch basin inspection and cleaning program as recommended by CDM in the "Wastewater Collection System Inspection Report" prepared in 1987. Routine inspection resulted in cleaning of 1,041 catch basins during 1988. The city should continue inspection and cleaning practices as necessary to minimize the accumulation of sand, grit, and solids in the sewer system.

Regulator Inspection

New Bedford has adopted a regulator inspection and maintenance program in conjunction with the other recommendations from the 1987 Wastewater Collection System Inspection Report. All regulators are inspected twice monthly. At that time, minor repairs are made and cleaning is performed as necessary. In addition, a priority list of major repairs has been formulated and implementation has commenced. Completion of all major repair projects is expected by early 1990. Present monitoring efforts require two staff on a part time basis to inspect all regulators twice monthly. Inspection reports are maintained for all regulators to monitor their operation and repair records.

The primary benefit of regulator monitoring is the prevention of maintenance-related dry weather discharges through CSOs. Wet weather operation of regulators is of minor importance because all of the semi-automatic regulators in New Bedford's sewer system have been abandoned and operate as fixed orifice type regulators. Experience in many combined sewer systems has shown that semi-automatic regulators designed to vary flow rate entering interceptors are impractical due to their high incidence of mechanical failure. For this reason, it is recommended that all former semi-automatic regulators continue be permanently altered to operate as fixed orifices.

The present level of maintenance by the City should be continued.

Infiltration/Inflow Control

It is estimated that over half of the average dry weather wastewater flow reaching the New Bedford WWTP is infiltration and inflow (I/I). Of the estimated 15.3 mgd I/I, 1.3 mgd is attributed to tidal inflow and the remaining 14.0 mgd to infiltration and other inflow sources. The design (year 2014) flows for the secondary WWTP are based on implementation of I/I reduction measures, and include flows of 0.7 mgd for tidal inflow and 11.4 mgd for other sources. Excluding the tidal inflow, this represents a 19 percent reduction in I/I. As discussed in Section 7.0, I/I reductions

greater than 19 percent are not warranted on the basis of CSO control alone.

Commercial/Industrial Runoff Control

New Bedford's Industrial Pretreatment Program (IPP) was approved by the Massachusetts DEQE in 1985. Permits were issued for all industrial users by 1987.

The IPP provides for sampling twice annually and inspection once annually of each industrial user by the City. One of the two samplings is performed without the knowledge of the user. Categorical industrial users discharging in excess of 25,000 gal/day to the municipal sewer system are required to sample and analyze effluent weekly, bi-weekly or monthly, and report the results to the City monthly. The City reports the results of the IPP semi-annually in accordance with the requirements of the NPDES permit for the municipal sewer/treatment facilities.

There are currently 26 significant industrial users and 44 non-significant industrial users discharging to the municipal sewer system. Current enforcement efforts should be continued.

Regulating Devices and Tide Gates

The draft Phase 2 WWTP Facilities Plan recommended regulator modifications necessary to convey all dry weather flows to the treatment plant. After these modifications, the downstream segment of the main interceptor will have a capacity of 75 mgd. Due to the limited capacity available in the interceptor, regulator modifications that would divert additional flow into the interceptors are not feasible.

10.3 WATER QUALITY BENEFITS

This section presents anticipated improvements to receiving water quality that would result from implementing the recommended plan. The general goal of CSO control in New Bedford is to reduce the frequency and extent of

violations of the water quality standards and restore currently impaired uses. This can be done by eliminating the discharge, decreasing its quantity, improving overflow quality through treatment, or redirecting a CSO to a less sensitive receiving water. The recommended plan for New Bedford includes alternatives that result in significant reductions in the frequency and extent of CSO discharges by decreasing CSO quantity and improving discharge quality. The option of eliminating discharges via sewer separation, though carried through the detailed analysis, was not included in the recommended plan for Groups 1 through 5 for the reasons discussed in Section 10.4. Redirecting overflows into less sensitive receiving waters is another option that was carried through the detailed analysis and eventually recommended for CSO Group 1.

The benefits to receiving waters resulting from implementing the recommended plan are discussed separately by receiving water. The reason for this is that the water quality improvements from CSO abatement must be placed in context with other pollution sources within each receiving water that may continue to negatively affect receiving water quality.

Implementing the recommended plan will result in direct improvements to New Bedford's receiving water quality, especially in Clarks Cove. The water quality benefits are sizable because the number of times that CSOs spill will decrease significantly, and the level of treatment provided to CSOs will improve significantly. Table 10-7 summarizes the water quality benefits to be achieved in terms of reduced frequencies of occurrence and reduced loadings to the receiving waters of implementing the recommended plan.

In Clarks Cove, under the Base Condition (i.e., existing conditions but with the improvements to the collection system assumed to be in place), CSOs spill as frequently as 45 times per year, discharging almost 500 million gallons of combined sewer overflow annually. The recommended plan will provide essentially total CSO control for Clarks Cove. Group 1, which discharges 99 percent of the CSO volume to Clarks Cove, would be eliminated. Negligible amounts of overflow would remain from the relatively small Group 2 area where CSOs would be controlled up to a 1-year

storm. This will reduce the number of times CSOs spill from 45 times per year to less than once per year. While the level of control provided for CSO Group 1 is up to a 3-month storm, overflows from storms larger than the 3-month storm would be routed to and discharged into the Inner Harbor, effectively eliminating any combined sewer overflow to Clarks Cove from Group 1. Because of the small CSO volumes associated with Group 2, control to the 1-year storm level can be accommodated with the recommended storage facility. CSO discharges to Clarks Cove from Group 2 would then only occur under the most extreme rainstorms.

On an average annual basis, this means that the pollutant loadings associated with CSOs would also be reduced effectively to zero as shown in Table 10-8. It is necessary to qualify this discussion by stating that a storm larger than the 1-year storm could still cause CSOs in Group 2 to spill, but because these storms occur once every 2 years or once every 5 years, they cannot be statistically included in the annual values.

In the Outer Harbor, CSOs spill on the average 49 times per year, discharging as much as 33 million gallons into the receiving waters annually. Under the recommended plan, spills would occur on average only 4 times per year, a 92 percent reduction over the Base Condition. The volume spilled under the recommended plan would be 3.6 million gallons, almost a 90 percent reduction. Corresponding reductions could also be expected in the loadings of coliform bacteria, total suspended solids, and metals.

CSOs annually discharge 595 million gallons into the Inner Harbor, spilling an average of 50 times per year. Under the recommended plan, these CSOs would discharge untreated effluent only 4 times per year, a 92 percent reduction. The volume of untreated CSO discharge would be reduced to 119 million gallons. Note that the overflow pipe from CSO Group 1 would be relocated so that it would discharge to the Inner Harbor instead of Clarks Cove about 4 times per year. This is consistent with the level of CSO control provided for the other CSO Groups discharging to the Inner Harbor. The added flow routed from Group 1 to the Inner Harbor would be about 28 million gallons per year. The amount of stormwater discharged to the Inner Harbor is over 1 billion gallons per year.

TABLE 10-8

FREQUENCY OF OVERFLOWS, OVERFLOW VOLUMES, AND LOADINGS OF CSOs UNDER BASELINE CONDITIONS AND THE RECOMMENDED PLAN

		~~BASELINE~~ EXISTING CONDITIONS						WITH RECOMMENDED PLAN					
Receiving Water	CSO Group	Overflow Volume per Year (Mg)	Overflow Frequency	Total Coliform (MPN)	TSS (lbs)	Zinc (lbs)	Copper (lbs)	Overflow Volume per Year (MG)	Overflow Frequency	Total Coliform (MPN)	TSS (lbs)	Zinc (lbs)	Copper (lbs)
Clarks Cove	1	496.2	45	2.9×10^{16}	331,275	1449	456	0	0	0	0	0	0
	2	0.6	28	3.5×10^{13}	401	2	1	0.026	<1**	1.54×10^{12}	17	0	0
Outer Harbor	3	33.2	49	2.0×10^{15}	22,165	97	30	6.0	4	3.5×10^{14}	4,006	18	6
Inner Harbor	4	102.4	46	6.0×10^{15}	68,365	299	94	36.0 (28.2)***	4 (4)***	2.1×10^{15} (1.7×10^{15})***	24,034 (18,827)***	105 (82)***	33 (26)***
	5	212.1	50	1.3×10^{16}	141,603	620	195	6.1 (48.6)*	4 (26)*	3.6×10^{14} (3.7×10^{11})*	4,072 (19,468)*	18 (85)*	6 (29)*
	6	280.9	50	1.7×10^{16}	187,536	820	258	13.4 (96.3)*	4 (26)*	7.9×10^{14} (7.3×10^{11})*	8,946 (38,575)*	39 (169)*	12 (57)*
Total untreated		1125.4	50	6.7×10^{16}	751,345	3287	1,034	61.5	4	3.6×10^{15}	41,075	180	57
treated		–	–	–	–	–	–	(144.9)*	(26)*	(1.1×10^{12})*	(58,043)*	(254)*	(86)*

*The recommended plan for Groups 5 and 6 includes treatment up to the three month design event. Events larger go untreated and are shown above; the volumes that are treated are shown in parentheses with their associated annual volumes and loadings.

**Minor CSO discharges would remain from storms larger than the 12-month storm, but statistically these amounts are negligible. The frequency is 0.30/year.

***The part of the Group 4 volumes and loads attributable to the redirection of Group 1 overflows to the Inner Harbor are in parentheses.

The difference in volume spilled between the Base Condition and the recommended plan is the amount of combined sewer overflow that would receive treatment. For CSO Groups 1 through 4, secondary treatment at the WWTP would be provided up to a 3-month storm except for Group 2 (1-year storm). For CSO Groups 5 and 6, primary treatment at two satellite facilities would be provided.

Table 10-9 presents the number of times each year that the CSO groups and storm drains from different communities reach various beaches and shellfishing areas in New Bedford, Dartmouth, and Fairhaven. Table 10-10 presents the same information under the recommended plan. The improvements in water quality can be measured by the reduction in occurrences.

As shown by Table 10-9, under the Base Condition CSO Groups 1 and 2 discharge 45 times per year on average, causing plumes to reach the West Beach and South Dartmouth beach 45 times per year. Ten percent of the time (or 5 times per year), wind and tide conditions are such that the plumes can reach Smith Neck. As shown in Table 10-10, under the recommended plan, CSO plumes would be controlled up to a 1-year storm. Thus, the number of times CSO plumes would reach these same resources would be less than once per year, or effectively zero.

The recommended plan would improve water quality in Clarks Cove to the point where the shellfish beds that are now closed could be reopened. All point discharges to Clarks Cove from New Bedford would be controlled. Two storm drains near Rodgers Street in Dartmouth contribute coliform loadings to Clarks Cove, and would possibly have to be controlled to eliminate all point sources from the Cove. The Conservation Law Foundation estimates that there are 107,242 bushels of quahogs in the closed areas of Clarks Cove with an estimated market value of $5 million per year. Implementing the recommended plan for CSO Groups 1 and 2 could result in a net economic gain of $5 million every year because the Clarks Cove shellfish beds could be allowed to reopen.

In the Outer Harbor, CSOs now discharge about 49 times per year, affecting the East Beach as many times. Eight times per year, winds and tides will

TABLE 10-9

THE NUMBER OF TIMES PER YEAR CSOs AND STORM DRAINS AFFECT SHELLFISH AREAS AND BEACHES UNDER BASELINE CONDITIONS

	CSO Groups #1 & #2	CSO Group # 3	CSO Groups #4, #5 & #6	Separate Storm Drains Fairhaven	Separate Storm Drains Inner Harbor
Frequency of Spills (number/year)	45	49	50	100	100
Location					
Shellfish (1)					
1. Clarks Cove	45	0	0	0	0
2. Outer Harbor (N.B.)	0	49	8	17	16
3. Outer Harbor (F.H.)	0	5	3	100	5
4. Inner Harbor	0	8	50	0	100
5. Buzzards Bay	7	8	6	16	11
Beaches (2)					
1. Smith Neck	5	5	0	0	0
2. S. Dartmouth	45	0	0	0	0
3. Clarks Point (West)	45	0	0	0	0
4. Clarks Point (East)	0	49	8	16	15
5. Sconticut Neck	0	3	1	100	0
6. Fort Phoenix	0	8	3	15	11

TABLE 10-10

THE NUMBER OF TIMES PER YEAR CSOs AND STORM DRAINS AFFECT SHELLFISH AREAS AND BEACHES UNDER RECOMMENDED PLAN

	CSO Groups #1 & #2	CSO Group # 3	CSO Groups #4, #5 & #6	Separate Storm Drains Fairhaven	Separate Storm Drains Inner Harbor
Frequency of Spills (number/year)	<1	4	4	100	100
Location					
Shellfish (1)					
1. Clarks Cove	<1	0	0	0	0
2. Outer Harbor (N.B.)	0	4	<1	17	16
3. Outer Harbor (F.H.)	0	<1	<1	100	5
4. Inner Harbor	0	<1	4	0	100
5. Buzzards Bay	<1	<1	<1	16	11
Beaches (2)					
1. Smith Neck	<1	<1	0	0	0
2. S. Dartmouth	<1	0	0	0	0
3. Clarks Point (West)	<1	0	0	0	0
4. Clarks Point (East)	0	4	<1	16	15
5. Sconticut Neck	0	<1	<1	100	0
6. Fort Phoenix	0	<1	<1	15	11

move CSO plumes from Group 3 over to the beach at Fort Phoenix in Fairhaven, and 3 times out to Sconticut Neck. These same resources are affected as significantly by loadings from storm drains in New Bedford and Fairhaven. Stormwater pollution from the Inner Harbor is estimated to reach the East Beach 15 times per year, and the Fort Phoenix beach 11 times per year. Under the recommended plan, CSO discharges from Group 3 would be decreased to only 4 times per year. However, the impacts of stormwater discharges from the Inner Harbor and Fairhaven would continue to affect local beaches as frequently as 15 times per year.

In the same manner, there is no guarantee that shellfish beds that are now closed in the Outer Harbor could be reopened by implementing the recommended plan. Storm drains would still discharge over 100 times each year and would likely cause violations of the coliform bacteria standard in at least portions of the Outer Harbor.

In the Inner Harbor, CSOs now spill 50 times per year. Stormwater is as significant a source of pollution as the CSOs. With the recommended plan, CSOs would spill untreated effluent only 4 times per year on the average. Storm drains would continue to discharge to the Inner Harbor every time it rains. The receiving water benefits in the Inner Harbor to be realized with CSO controls for Groups 4, 5, and 6 are minimal. The animal feces, trash, litter, oil, grease and other contaminants that washes off the streets and discharges from storm drains will continue to cause water quality problems in the Inner Harbor, and cause shellfish beds to remain closed. The PCB problem in the sediments of the Inner Harbor, coupled with the fact that no swimming occurs because the Inner Harbor is a major commercial waterway suggests that its current classification as a Class SB needs to be reconsidered, and possibly revised. Even with CSO controls, the water quality criteria presently mandated for the Inner Harbor cannot be achieved. Reclassifying the receiving water as Class SC is an alternative that should be considered before implementing CSO controls.

10.4 WHY SEWER SEPARATION IS NOT RECOMMENDED FOR GROUPS 1 THROUGH 5

Sewer separation was carried throughout the detailed engineering evaluations because it is the only alternative that "eliminates" combined sewer overflows to New Bedford's receiving waters. The combined sewer system would be separated into its two components (wet weather runoff and dry weather sewage flow) by building two separate pipelines. Sewage flows would be connected directly to the sewer system where they would be routed to the WWTP, treated, and discharged out the ocean outfall. Runoff from rainstorms would flow into separate storm drains and be discharged to the receiving waters at existing outfalls located at the ends of streets. The cost to separate the entire combined system would be $138.9 million (including a 30 percent allowance for engineering and contingencies). This is broken down as follows:

CSO GROUP	COST TO SEPARATE ($ million)
1	$42.4
2	$6.7
3	$5.3
4	$27.7
5	$20.6
6	$36.2
TOTAL:	$138.9

Sewer separation would prevent raw sewage from flowing into the receiving waters. While eliminating sewage flows would improve water quality, water quality problems would persist because the separated storm drains would still be discharging to local waters every time it rains. Stormwater has already been identified as a significant source of pollution. Thus, even though the sewer system was separated from the storm drain system, significant quantities of animal feces, litter, trash, oil, grease, and other constituents associated with urban land use would wash off the streets and end up along the shoreline every time it rains. In New England, it rains about once every 3 or 4 days, or 90 to 120 times per year.

Instead of sewer separation for CSO Groups 1 through 5, the recommended plan includes storage pump back and satellite treatment systems that can control all flow (including stormwater) for up to a 3-month storm. The exception to this is for CSO Group 2 where controls up to a 1-year storm are recommended to protect Clarks Cove as discussed in Section 10.3. Thus, instead of eliminating sewage flows but continuing to have storm drains spill over 100 times each year, the recommended plan includes provisions for controlling all flow so that spills occur only during extreme rainstorms which are expected to happen, on average, only 4 times per year.

The recommended plan provides the City of New Bedford with an opportunity to control two of the most significant sources of pollution to its receiving waters at the same time. EPA recognizes that stormwater is a significant source of pollutant loadings nationwide, and is developing guidelines and procedures to mitigate stormwater impacts. Stormwater controls will likely be mandated at the federal level in the foreseeable future. Implementing the recommended plan will provide the City with the opportunity to be a leader in pollution control because it will be one of the first communities to recognize the water quality impacts of stormwater and address them.

Note that sewer separation is recommended for a small area within CSO Group 6. Because it is not cost-effective to route CSOs from this sub-area to the proposed Group 6 CSO treatment facility, therefore sewer separation was recommended, as discussed in Section 10.2.1.

10.5 PROJECT COST AND STAGING

This section presents the capital, operation, and maintenance costs for the recommended facilities, as well as the order in which the 6 projects should be undertaken. The financial analysis and implementation schedule is presented in Section 11.0.

10.5.1 ESTIMATED PROJECT COSTS

A construction cost estimate was developed for the recommended facilities based on the conceptual plans. These estimates are intended to provide information required for prioritizing, planning, and budgeting purposes and accordingly, reflects a higher level of precision than cost estimates developed for the evaluation of alternatives. Specifically, the following estimates was developed based on conceptual facility layouts that include preliminary design of all major process equipment and consideration of site conditions that may be encountered. Cost curves developed for use in the evaluation process were based on information from a broad range of projects having differing site conditions and levels of complexity.

As shown on Table 10-11 project cost estimates include construction, engineering and contingencies, and operation and maintenance (O&M). Engineering and contingencies are estimated at 30 percent of construction cost. O&M costs reflect present worth of annual O&M at current prices with no adjustment for inflation. Construction costs are shown separately for each of the 6 storage/treatment facilities and associated conveyance facilities on Tables 10-12 through 10-17. The total estimated cost of the recommended plan is $182.5 million.

10.5.2 STAGING OF PROJECTS

The structural components of the recommended plan include 6 discrete projects that can be phased over time as funds become available to the City. The 6 projects include one for each of the 6 CSO groups. Each of the 6 projects can be undertaken separately from the others. Each project calls for either its own storage-pump back or satellite treatment facility.

There is a specific order that the 6 projects should be undertaken. They are prioritized by the water quality benefits they will provide. The projects that provide the greatest water quality benefits are scheduled first. The 6 projects have been divided into 3 construction stages.

TABLE 10-11

COST SUMMARY

	CAPITAL COST(1)	OPERATION AND MAINTENANCE COSTS
GROUP 1	$63,700,000	$212,800
GROUP 2	$15,300,000	$43,700
GROUP 3	$15,300,000	$38,200
GROUP 4	$31,800,000	$111,200
GROUP 5	$20,400,000	$83,500
GROUP 6	$36,000,000	$148,900
TOTAL COST:	$182,500,000(2)	

(1) Does not include land acquisition, legal and administrative costs.
(2) All costs are September 1989 costs.

TABLE 10-12

ESTIMATED PROJECT COSTS FOR GROUP 1

CAPITAL COSTS

• PIPING		
Gravity Sewers		$6,927,000
Force Mains		$796,000
• CSO Sludge Collection/Storage Facility		$31,850,000
• Pumping/Screening Facility (40 mgd)		$4,287,000
• Site Improvements (Allowance)		$306,000
• Bypass Sewer & Siphon		
Gravity Sewer and bypass		$2,027,000
Twin 48-inch Siphon		$2,450,000
	Subtotal	$48,643,000
	Use	$49,000,000
	30% Engineering & Contingency	$14,700,000
	Total Capital:	$63,700,000

TABLE 10-13

ESTIMATED PROJECT COSTS FOR GROUP 2

CAPITAL COSTS		
• CSO Sludge Collection/Storage Facility (1.7 mg)		$7,597,000
• Pumping/Screening Facility (2.7 mgd)		$1,013,000
• Site Improvements (Allowance)		$95,000
• Gravity Sewers		$3,076,000
• Force Main		$19,000
	Subtotal	$11,800,000
	30% Engineering & Contingency	$3,500,000
	Total Capital Cost:	$15,300,000

TABLE 10-14

ESTIMATED PROJECT COSTS FOR GROUP 3

CAPITAL COSTS

• CSO Sludge Collection/Storage Facility (1.5 mg)		$7,023,000
• Pumping/Screening Facility (2.3 mgd)		$1,022,000
• Site Improvements (Allowance)		$96,000
• Gravity Sewers		$3,129,000
• Force Main		$530,000
	Subtotal	$11,800,000
	30% Engineering & Contingency	$3,500,000
	Total Capital Cost:	$15,300,000

TABLE 10-15

ESTIMATED PROJECT COSTS FOR GROUP 4

CAPITAL COSTS		
• CSO Sludge Collection/Storage Facility (7.7 mg)		$18,291,000
• Pumping/Screening Facility (12.4 mgd)		$2,146,000
• Site Improvements (Allowance)		$187,000
• Gravity Sewers		$3,471,000
• Force Main		$405,000
	Subtotal	$24,500,000
	30% Engineering & Contingency	$7,300,000
	Total Capital Cost:	$31,800,000

TABLE 10-16

ESTIMATED PROJECT COSTS FOR GROUP 5

CAPITAL COSTS

• CSO Sludge Collection/Storage Facility (2.05 mg)		$8,108,000
• Pumping/Screening Facility (34.1 mgd)		$3,992,000
• Site Improvements (Allowance)		$156,000
• Gravity Sewers		$3,405,000
• Force Main		$39,000
	Subtotal	$15,700,000
	30% Engineering & Contingency	$4,700,000
	Total Capital Cost:	$20,400,000

TABLE 10-17

ESTIMATED PROJECT COSTS FOR GROUP 6

CAPITAL COSTS		
• CSO Sludge Collection/Storage Facility (3.25 mg)		$10,881,000
• Pumping/Screening Facility (51.8 mgd)		$5,098,000
• Site Improvements (Allowance)		$186,000
• Gravity Sewers		$11,111,000
• Force Main		$424,000
	Subtotal	$27,700,000
	30% Engineering & Contingency	$8,300,000
	Total Capital Cost:	$36,000,000

Stage 1: Projects for CSO Groups 1 and 2

As discussed in Section 5.0, Clarks Cove is the receiving water with the highest priority because of the beaches and shellfish resources in the area, and the impaired uses of these resources that result from CSO discharges. The greatest receiving water benefits can be realized, therefore, by addressing CSOs discharging to Clarks Cove. To this end, projects to consolidate, store, and pump back CSOs in Groups 1 and 2 should be undertaken first. Of the 2 groups, CSO Group 1 is a higher priority because it discharges 99 percent of the total CSO volume to Clarks Cove, and is responsible for most of the receiving water quality problems. When funding becomes available, it should be used to implement the abatement plan for CSO Group 1. To provide a comparable level of control, the abatement program for CSO Group 2 should be undertaken soon thereafter.

Stage 2: Projects for CSO Group 3

The receiving water of second highest priority is the Outer Harbor. The Outer Harbor contains beaches and shellfish beds. These resources, however, are affected not only by New Bedford's CSOs, which will be abated with the recommended plan, but also by stormwater from New Bedford and Fairhaven, which will continue to cause water quality problems in both the Inner and Outer Harbors. Unlike Clarks Cove, where the major pollutant source will be cleaned up once the CSOs are controlled, the Outer Harbor will continue to be affected by both stormwater flowing out of the Inner Harbor, and moved by wind across the Outer Harbor from Fairhaven. Nonetheless, there are water quality benefits to be realized by CSO abatement to Group 3. After the Stage 1 projects are undertaken, available funds should be directed at implementing the abatement plan for CSO Group 3.

Stage 3: Projects for CSO Groups 4, 5, and 6

These CSO abatement projects are of lowest priority because the receiving water impacts that result from them are marginal. As discussed in Section 10.3, stormwater in the Inner Harbor is as significant a pollutant source

as CSOs. Until flows from storm drains are controlled in the Inner Harbor, there is little to be gained in water quality improvement by controlling CSOs. Further, the PCB problem that exists in the Inner Harbor will preclude future shellfishing activities. Because the Inner Harbor is a major commercial waterway, it will never be used for swimming. The CSO abatement projects for Groups 4, 5, and 6 would be included in Phase 3 and would be the last to be implemented.

10.6 ALTERNATE PLANS

The CSO recommended plan was developed with the assumption that the secondary WWTP would be located at Fort Rodman. The draft Phase 2 WWTP Facilities Plan identified Fort Rodman as the preferred site because of its engineering-related and cost-related advantages. The decision needs to be finalized. Further, a policy decision was made by the City administration not to use private land for public purposes, unless as a last resort, thereby excluding the Standard Times sites from preferred consideration. If conditions change, and it is decided that the Standard Times site can be used for public purposes, it would be possible to locate the WWTP there. Further, if the Standard Times could be used for public purposes and the WWTP was still located at Fort Rodman, then the Standard Times site could be used for centralized CSO storage for CSO Groups 1 and 4 instead of the 4 discrete storage locations considered in the recommended plan. Groups 2 and 3 could not be stored at the same consolidation facility because it would not be possible to pump the flows into the Rodney French interceptor before the next storm occurred. Each of these alternatives is discussed in greater detail below. A summary of the important issues regarding each alternative is presented in Table 10-18.

10.6.1 IF SECONDARY WWTP IS AT STANDARD TIMES SITE

If the secondary WWTP is located at the Standard Times site instead of Fort Rodman, the recommended plan for CSO abatement would not change considerably. Storage-pump back would still be used for Groups 1 through 4, and satellite treatment would still be used for Groups 5 and 6. The same sites specified in the recommended plan would be used regardless of

TABLE 10-18

COMPARISON OF COST OF ALTERNATIVE PLANS

Recommended Plan	Capital Cost (million dollars)
WWTP at Fort Rodman	
o Separate Storage-Pumpback Facilities for Groups 1-4	$126.10
o Satellite Treatment at Groups 5 and 6	$56.40
Total:	$182.50
WWTP at Standard Times	
o Separate Storage-Pumpback Facilities for Groups 1-4	$125.00
	$56.40
Total:	$181.40
Standard Times Site Used For Centralized Storage	
o Consolidated Storage for Groups 1 and 4	$98.50
o Separate Storage-Pumpback for Groups 2 and 3	$30.60
o Satellite Treatment for Groups 5 and 6	$56.40
Total:	$185.50

where the WWTP is located. Captured flows from the storage facilities at Groups 1 through 4 would be pumped to the WWTP at Standard Times site where they would be treated, routed through the interceptor to Clarks Point, and discharged out of the rehabilitated outfall.

The only difference in the recommended plan for CSO control would be the addition of force mains as shown on Figure 10-10. For Group 1, an additional 800 feet of 48-inch force main would be needed. For Group 2, an additional 5,500 feet of force main would be needed, and Group 3 would require an additional 1,600 feet of force main.

Cost

If the WWTP is located at the Standard Times site, the cost of the recommended plan for CSO abatement does not change appreciably. The cost of the added force mains would be negated by the cost savings for eliminating the siphon associated with the Group 1 abatement plan.

Water Quality Benefits

If the WWTP is located at the Standard Times site, the water quality benefits of the recommended plan for CSO control would be the same as those discussed in Section 10.2.4.

Implementation

There would be no changes in the implementation of the recommended plan--either in terms of prioritization or staging--if the WWTP were located at Standard Times.

10.6.2 IF STANDARD TIMES SITE IS USED FOR CSO STORAGE

If the Standard Times site can be used for CSO storage, the recommended plan would be modified somewhat. Instead of separate storage facilities for CSO Groups 1 and 4, they could be consolidated into a larger storage facility (see Section 9.0). CSO Groups 2 and 3 would still require

separate storage facilities. Satellite treatment facilities would still be used to treat CSO Groups 5 and 6. The same level of CSO abatement control would be provided under either plan.

Although a centralized storage facility at Standard Times could be made large enough to accommodate the added volumes contributed by these two groups, there is a problem in conveying the flow to Standard Times. Flows from Groups 2 and 3 would have to be pumped back to the consolidated storage facility. Even with centralized storage at Standard Times, separate storage facilities would be required for Groups 2 and 3. Otherwise, enormous pumps would be required to enable all flows from all storms up to a 1-year storm to be handled at Standard Times. The tremendous pumping costs make storing at least a portion of the CSO volume at a separate facility nearer to Groups 2 and 3 more cost-effective. If separate facilities would be needed at Groups 2 and 3 anyway, the overall cost advantages of using Standard Times for centralized storage diminish.

Cost

If the Standard Times site could be used for CSO storage, the costs of the recommended plan would be about the same. Minimal cost savings would result from one less site, fewer pumps, and one less building. The cost advantages would be offset by the need to still have separate storage facilities at Groups 2 and 3, and satellite treatment facilities for CSO Groups 5 and 6.

Water Quality Benefits

If the Standard Times site is used for CSO storage for Groups 1 and 4, then the water quality benefits would be the same as those discussed for the recommended plan.

Implementation

The disadvantage to using centralized storage at the Standard Times site is that it will require a larger, more capital-intensive project all at once

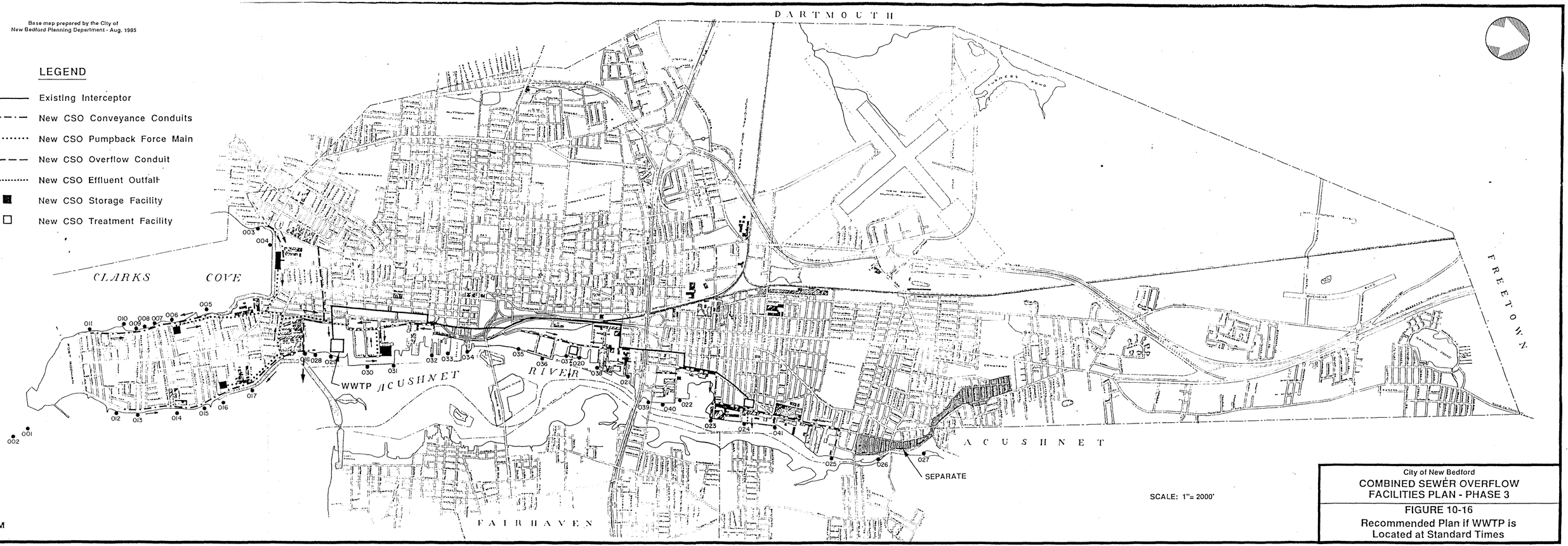
to be effective, compared to the recommended plan, which includes 6 smaller discrete projects that could be constructed over time as funds become available, as discussed in Section 10.5. To be cost-effective and efficient, it would be necessary from the start to build the storage facility large enough to hold the CSO from Groups 1 and 4. To be most effective, projects to consolidate the CSOs in each of the two groups should be built together, as should the necessary pipelines to route flows from the consolidated CSOs to the Standard Times storage facility. Of course, consolidating the CSOs and routing the flows to the centralized storage facility could be undertaken one group at a time as funds become available, instead of undertaking them together. However, there would be storage capacity that would be built and available, but remain unused unless and until both groups are consolidated and connected.

In addition, as discussed in Section 10.5, CSO Group 4 is of a much lower priority than either Group 2 or 3. If and when funds become available for CSO construction, the greatest water quality benefits are to be realized by building the recommended facilities for the other groups first.

Base map prepared by the City of
New Bedford Planning Department - Aug. 1985

LEGEND

- Existing Interceptor
- New CSO Conveyance Conduits
- New CSO Pumpback Force Main
- New CSO Overflow Conduit
- New CSO Effluent Outfall
- New CSO Storage Facility
- New CSO Treatment Facility



SCALE: 1"= 2000'

CDM

City of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 10-16
Recommended Plan if WWTP is
Located at Standard Times

11.0 FINANCIAL CAPABILITY AND IMPACT ANALYSIS

11.1 INTRODUCTION

This section examines the costs associated with financing, building, and operating of both the recommended secondary WWTP and CSO abatement projects on the City of New Bedford and its residents. Recommendations for the secondary WWTP were presented in the draft Phase 2 WWTP Facilities Plan submitted in August 1989. This facilities plan has developed recommendations for the abatement of combined sewer overflows. The fiscal impacts of these projects are considered together because the recommendations from these facilities plan will be implemented over the next decade or so.

The construction of the recommended wastewater facilities and CSO abatement facilities will entail a major financial commitment. In real 1989 dollars, the new facilities related to the secondary WWTP are estimated to cost approximately $240 million. Costs for the CSO abatement facilities are estimated to cost approximately $180 million. The general goal of CSO control in New Bedford is to reduce the frequency and extent of violations of the water quality standards.

Once the facilities are constructed, the City will also incur increased costs associated with operating the new facilities. The funds to construct and operate these facilities, if not derived from grants, must be recovered through users charges and/or through increased taxes.

The purpose of this section is twofold: (1) to evaluate the City's financial capability to construct and operate the wastewater facilities envisioned in the recommended plan, and (2) to assess the impact of the plan on the system's rate payers and the City's taxpayers.

The approach to this analysis consists of four steps: (1) reviewing and identifying current operating and financial conditions in New Bedford, (2)

estimating capital and operating costs of the new facilities with an associated funding plan, (3) projecting future expenditures and revenues, and (4) comparing and evaluating the incremental effects that these additional revenue requirements will have on the City and system users.

The remainder of this section is organized as follows:

11.2 Current Budgetary and Financial Conditions
11.3 Capital Costs and O&M Projections
11.4 Financing Assumptions
11.5 Quantitative Analysis
11.6 Fiscal Impacts
11.7 User Impacts
11.8 Incremental Impact of CSO Abatement Projects
11.9 Findings And Conclusions

11.2 CURRENT BUDGETARY AND FINANCIAL CONDITION

This section reviews the City's current budgetary and fiscal position including general fund expenditures, revenues, and legal constraints.

11.2.1 GENERAL FUND EXPENDITURES

Municipal expenditures totaled approximately $113 million in the City's 1988 fiscal year (July 1 to June 30). Table 11-1 shows 1988 and 1989 expenditures by major category. FY 1988 expenses were approximately 3 percent higher than the previous year. The most recent FY 1989 estimate, based on 10 months actual and 2 months of estimated data, is $124.0 million, which is approximately 10 percent above FY 1988.

The pattern of expenditures displayed in Table 11-1 is typical of Massachusetts communities. The largest category is public schools, which accounts for almost 45 percent of New Bedford's 1988 budget. The budgets for the police and fire departments are the second major category (almost 15 percent of the budget). Public works is also a significant category

TABLE 11-1

MUNICIPAL EXPENDITURES

($ in thousands)

Department	FY 1988	FY 1989 (Estimate)
Assessor	281.9	370.1
Auditor	193.7	212.2
City Council	201.5	224.9
Fire Department	8,142.0	7,912.4
Legal Department (City Solicitor)	451.8	331.9
Mayor's Office	168.9	175.3
Parks and Recreation	1,205.0	1,264.4
Police Department	8,685.5	9,327.2
Public Schools	50,414.8	35,223.2
Public Works	9,762.6	11,247.6
Treasurers Office	202.4	350.3
Treasurers Office--Debt Service	6,156.4	5,520.8
Water Department	3,384.5	3,212.7
All Other	23,797.0	28,670.0
TOTAL	113,048.0	124,043.0

Fiscal year (July 1 to June 30)

accounting for 8.6 percent of the budget; it is likely to increase over time as the wastewater system is upgraded and improved.

The recently approved budget for 1990 is $116.2 million, which is a 6.3 percent decrease compared to FY 89 estimates.

As of June 30, 1989, the City has approximately $20 million in debt outstanding, or approximately $200 per capita. The presently outstanding debt of the City will be retired by the year 1998. As of June 30, 1989, outstanding debt related to the wastewater system was $595,000.

At present, overlapping debt allocated to New Bedford totals approximately $650,000, or approximately $6.50 per capita. This includes the debt of the Regional Transit District, the regional Solid Waste District, and Bristol County. Overlapping debt will increase significantly in the near future, as the Greater New Bedford Regional Refuse Management District embarks upon a $60 million capital program. New Bedford's share of that debt is nearly 80 percent.

11.2.2 REVENUES AND PROPERTY TAX BASE

New Bedford, like all other Massachusetts communities, uses property tax as its major source of own-source revenues. Proposition 2 1/2 (Prop 2 1/2) severely limits the City's ability to increase revenues from that source. Prop 2 1/2 is a citizen-backed initiative that restricts the property tax rate and the growth in property tax revenues for all Massachusetts communities. This constraint will grow in importance as growth in state aid levels off and declines, at least in the short term.

The total value of real property for tax purposes was nearly $1.35 billion in FY 1988. (An assessment is not yet available for FY 1989.) This represents a very slight increase over the 1986 value of $1.3 billion. Long-term projections developed by the Bank of Boston anticipate a 4.6 percent growth in the property base in nominal dollar terms. When inflation is factored out, this means that the City's tax base is not

expected to show real growth. Table 11-2 shows the current distribution of the tax assessment.

As can be seen, the property base is dominated by residential property (>70 percent). Commercial property accounted for 13.8 percent; industrial property, 10.3 percent; and personal property, 4.9 percent.

Under the terms of Prop 2 1/2, municipalities cannot set their property tax rate higher than $25 per $1000 of assessed value. The "levy ceiling," or maximum revenues available from the property base, is calculated by multiplying the maximum rate times the community's tax base, valued at full market value. In New Bedford, the ceiling is approximately $34 million, based on the 1988 assessment. Prop 2 1/2 also restricts the growth in property tax revenues to 102.5 percent of the previous year's receipts, plus 2.5 percent of the new property added to the tax base--this is known as the "levy limit." This makes it difficult for communities to keep up with inflation, unless significant amounts of new property are added to the tax rolls. Prop 2 1/2 limits can be overridden by a two-thirds vote of the City Council.

In 1988, New Bedford levied $32.3 million in property taxes. The City uses a classified structure with residential properties taxed at a rate of $19.09 per $1,000 in assessed values; all other property classes are taxed at a rate of nearly $36 per $1,000 of assessed value. The weighted average tax rate is $24.34 per $1,000; this is very close to the maximum allowed rate of $25 per $1,000 mandated by Proposition 2 1/2.

In addition to the property tax, New Bedford generates general funds from three principal sources: state aid, excise and other local receipts, and interest earnings. Table 11-3 summarizes projected FY 1989 revenues by source.

The City's dependence on state aid is very clear from this table: Nearly 54 percent of the City's revenues are state aid payments. The principal source of the City's own source revenues, the property tax, accounts for less than 30 percent of revenues. Other local sources include

TABLE 11-2

PROPERTY ASSESSMENTS, FISCAL YEAR 1988

($ in millions)

Residential	$ 953.8
Commercial	185.6
Industrial	138.9
Personal	65.8
Total	$1,344.1

TABLE 11-3

PROJECTED REVENUES BY SOURCE AVAILABLE FOR BUDGETED OPERATIONS, 1989

($ in millions)

Revenue Source	Dollars	Percent of Total
Net State Aid	$66.5	53.6%
Property Tax Receipts	31.5	25.4
Excise and Fees	16.4	13.2
Other Local Sources	9.6	7.7
Total	$124.0	100.0%

approximately $8 million of free cash that is essentially a one-time adjustment and will not be repeated in future years.

City reliance on state aid has steadily increased over the last 6 years. In 1983, state aid totaled $39.2 million. From 1983 to 1989, state aid has increased by $37.7 million, while the City's expenditures increased $48 million. State aid has financed more than 75 percent of the City's growth in spending over the 1983 to 1989 time period. A central issue facing the City is the future amount and reliability of state aid payments.

Prop 2 1/2 is likely to become a major factor in the future fiscal management of New Bedford. As the City begins to incur large expenditures for constructing and operating the recommended wastewater facilities, and if state aid begins to taper off, the constraints of Prop 2 1/2 will be felt.

11.2.3 THE WASTEWATER DIVISION

The Wastewater Division of the Public Works Department is responsible for the collection and treatment of wastewater in New Bedford. The total budget for the division was nearly $2.7 million in FY 1988. This budget includes wages for employees, utilities, chemicals, and contracted services. (Note: The division contracts out certain services such as engineering and laboratory testing.) This budget does not include certain non-wage costs of employment (e.g., health insurance and pensions) and indirect overhead expenses (e.g., general governmental management).

Wastewater Division operations, maintenance, and replacement (OM&R) expenses have been steadily increasing from $1.4 million in FY 1985 to an estimated amount exceeding $4.4 million in FY 1990. These figures do not include planning and design expenditures related to the secondary WWTP and the CSO abatement projects. In FY 1990, the City will be required to enter into engineering planning and design contracts with a total value of approximately $12.7 million. Approximately $4.5 million will be expended in FY 1990. For purposes of this analysis, it is assumed that the entire

$12.7 million is capitalized into plant capital costs, although the City may choose to pay for them with current revenues.

Wastewater expenses are partially recovered through user fees. The City charges wastewater customers a connection fee that is presently set at $23 per year, plus a commodity charge of $0.34 per 1000 gallons of water consumed. A typical household pays an average of $57 per year under this system. Industrial customers are assessed the same connection fee, but are charged a surcharge for suspended solids and high-strength discharges. The commodity charge for industrial customers is $0.31 per 1,000 gallons of sewage discharged to the system.

Under existing intermunicipal agreements, the City also provides treatment for wastewaters generated in small sections of the Towns of Dartmouth and Acushnet. In addition to sewage that flows to the plant through the collection system, the plant receives septage, which comes from septic systems in the unsewered areas of New Bedford and Acushnet.

The current rate structure generates approximately $2.3 million in revenues. Wholesale charges result in an additional $100,000 in revenues. Currently, the City is considering a rate proposal that would increase the commodity charge for households to $0.89 per 1,000 gallons of water consumed. The typical single family would face an annual bill of approximately $112 per year. The proposal, if accepted, would generate total revenues of nearly $5.3 million, and is designed to recover some of the planning and design costs associated with the wastewater facilities.

11.2.4 GENERAL FUND PROJECTIONS

The City's fiscal capacity is sensitive to a wide range of factors, which include, but are not limited to, expansion of the tax base, growth in state aid, sewer and non-sewer capital needs, and inflation. In developing long-term (1990-2005) revenue and expenditures projections for the City, the following assumptions were made:

- State aid will remain constant in real dollar terms (e.g., the state will continue to provide $63 million in general purpose funds).
- Municipal operating expenditures will remain constant when adjusted for inflation.
- The value of the property tax base will remain constant.
- Non-sewer related borrowings for capital improvements will average $2 million a year.
- Sewer system capital expenditures are limited to those associated with the secondary WWTP and CSO abatement facilities.

These assumptions are obviously artificial. Their primary limitation is that revenues, expenditures, and the tax base are not likely to remain constant in real dollar terms. In fact, the Governor recently announced a cut in local aid that will, if effective, reduce payments to New Bedford in 1990 by approximately $3.5 million. In light of the reduction in state aid, New Bedford's 1990 budget projects expenditures to be 10 percent below 1989 expenditures. In the longer term, it is assumed that the Commonwealth will stabilize its fiscal position and that state aid will return to its levels of the mid-1980s. However, despite their limitations, these assumptions provide a useful basis for comparing alternative financing plans for the wastewater and CSO abatement facilities. (Projections of City expenditures and revenues are done on an accrual basis to simplify the analytical process.)

11.3 CAPITAL COSTS AND O&M PROJECTION

The recommended plan for the WWTP facilities includes the construction of a 30-mgd secondary WWTP to be located at Fort Rodman, sludge disposal facilities located near the airport, neighborhood improvements and relocations at Fort Rodman, and various collection system modifications. The recommended CSO abatement plan includes construction of discrete CSO abatement projects that will greatly enhance New Bedford's ability to handle sewer wastes during periods of extended rainfall. A complete description of the recommended WWTP facilities can be found in the draft Phase 2 WWTP Facilities Plan. This section provides a summary of the CSO abatement projects, which are presented in more detail in Section 10.2.

Figure 11-1 is a construction schedule for both the WWTP and CSO abatement facilities.

The CSO abatement projects have been broken into six discrete elements (by CSO Group) to facilitate engineering evaluations. The recommended alternative for CSO Group 1 consists of a storage facility designed to hold flows up to a 3-month storm. Flow would be pumped to the Rodney French interceptor for treatment at the secondary WWTP once rainstorms are over. A second pipeline has been added to capture overflows from storms larger than the 3-month storm. These overflows would be routed via a gravity sewer to the Inner Harbor where they would be discharged. The added pipeline would be used, on average, only 4 times per year.

The recommended alternative for CSO Group 2 is a storage-pump back facility designed to hold up to a 12-month storm. This higher level of control is warranted to provide additional protection for Clarks Cove, the receiving water with the highest priority. The storage tank would hold up to 1.7 million gallons, and the tank would also be pumped to the Rodney French interceptor following the end of the rainstorm.

The recommended alternative for CSO Group 3 is a storage-pump back facility designed to hold up to a 3-month storm. CSO Group 3 storage tanks will have a capacity of 1.45 million gallons and would be below grade. Overflows collected in these tanks would also be conveyed to the secondary WWTP.

The recommended plan for CSO Group 4 consists of a storage-pumpback facility designed to hold a 3-month storm. The tanks would have a capacity of 7.6 million gallons and would be below grade. Again, the overflows in these tanks would be routed to the secondary WWTP.

The CSO volumes pumped back to the interceptor to the WWTP from the storage tanks from CSO Group 1 through 4 will take up all of the available capacity of the WWTP and the interceptor to the WWTP. Because capacity is not available to treat any flow from CSO Groups 5 or 6, each of these groups must have its own treatment facility.

PROJECT	ESTIMATED COST($Mil)	CONSTRUCTION SCHEDULE 1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000	to 2015
Secondary Treatment													
1. Plant, Outfall and Sludge	$200.6			XXXXXXXXXXXXXXXXXXXXXXXXXXXX									
2. Interceptor Grit Project	$10.0			XXXXXXXXXXXXXXXXXX									
3. Collection System Improvements	$34.3												
a. Pump Station Modifications								XXXXXXXXXXXXXXXXX					
b. East Clark's Pt. PS							XXXXXXXXXXXXXXXXX						
c. Cove Road PS					XXXXXXXXXXXXXXXXXXXX								
d. North End Phase 2								XXXXXXXXXXXXXXXXX					
e. Sewer Extensions									XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX				
4. Sewer Rehabilitation	$4.8			XXXXXX									
	$248.7												
CSO Project													
Group 1	$63.7									XXXXXXXXXXXXXXXXXX			
Group 2	$15.3									XXXXXXXXXXXXXXXXXX			
Group 3	$15.3											XXXXXXXXXXXX	
Group 4	$31.8												XXXXXXXX
Group 5	$20.4												XXXXXXXX
Group 6	$36.0												XXXXXXXX
	$182.5												
Rates (@ 45% SRF)Ave House													
Secondary Treatment		$140	$140	$140	$140	$152	$147	$502	$534	$546	$546	$546	$571-$571
Treatment plus CSOs		$140	$140	$140	$140	$152	$147	$502	$534	$546	$546	$665	$690-$714

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 11-1
CONSTRUCTION SCHEDULE FOR
SECONDARY WWTP AND CSO
ABATEMENT FACILITIES

The recommended alternative for CSO Group 5 consists of a treatment facility designed for a 3-month storm. The facility would include a storage tank capable of holding up to 2 million gallons with a peak treatment capacity of 34 mgd. CSO Group 6 would also be designed for a 3-month storm, and have a peak treatment capacity of 54 mgd and a total tank volume of 3.23 million gallons.

The recommended CSO plan would be implemented in three construction stages. The first stage would involve CSO Groups 1 and 2, which discharge water into Clarks Cove, the highest priority receiving water. Construction of Groups 1 and 2 would be commence in 1998 and be completed in 1999. Construction of Group 3, which discharges into New Bedford's Outer Harbor, would commence in 2000 and be completed in 2001. The construction of these first three groups should be classified as "high priority" and should be constructed as soon as is practical.

Groups 4, 5, and 6 discharge to the Inner Harbor, the lowest priority receiving water. Construction of the abatement projects for these CSO Groups is, consequently, a lower priority; it is recommended they be constructed by 2015. The prioritization these individual projects has been designed to provide the greatest water quality benefits in the most cost-effective manner.

11.3.1 CAPITAL COSTS

The total capital costs for constructing CSO facilities are estimated to be $182.5 million (in 1989 dollars). Table 11-4 presents costs for each component.

As shown on Table 11-4, the implementation of the recommended plan is spread over a long term period. Those projects that have the most significant impact on water quality are scheduled to be undertaken first beginning in 1998. These include Groups 1 and 2 with a total capital cost of $79 million. The Group 3 abatement project is scheduled to commence construction during the year 2000, with a capital cost of $15.3 million.

TABLE 11-4

CSO PROJECT CAPITAL COSTS

(1989 $ in millions)

CSO Group	Total Cost	Construction Period	Start Date
Group 1	$ 63.7	24 months	1998
Group 2	15.3	24 months	1998
Group 3	15.3	24 months	2000
Group 4	31.8	24 months	2013
Group 5	20.4	24 months	2013
Group 6	36.0	24 months	2013
TOTAL:	$ 182.5		

Groups 4, 5, and 6 offer significantly less environmental benefits. Consequently, we recommend that their construction be delayed; the projects would be completed by 2015, and would cost $88.2 million (1989 dollars).

In developing an assumed financial plan for CSO projects, we have assumed that the City will chose to capitalize interest incurred during construction. For Groups 1 and 2, capitalized interest will increase the amount to be financed by as much as $10.1 million. For Group 3, capitalized interest will total as much as $2.0 million.

To finance the projects anticipated to be undertaken in the 1989-2005 time period, the City will be required to borrow between $94 and $106 million (1989 dollars).

11.3.2 OPERATIONS AND MAINTENANCE

The development of the CSO abatement projects will add to the O&M costs for New Bedford's wastewater system. As seen in Table 11-5, O&M costs are estimated to total $638,300 per year (1989 dollars) when all projects are constructed. However, the City will incur only $295,000 per year during the 1990-2005 time period because of the recommended project sequencing.

11.3.3 SUMMARY

From the two preceding sections, it is clear that the City will be facing significant increases in the expenses associated with the recommended wastewater facilities. In the year 2005, total revenue requirements associated with the Wastewater Division may exceed $50 million (1989 dollars). Of that total, costs associated with the CSO abatement projects are estimated to exceed $11 million.

The rate at which the City incurs these expenditures is, in part, discretionary, depending on the financing plan adopted by the City. The City may also wish to adjust its rate structure prior to the expenditures, to dampen the rate shock associated with significant increases in revenue

TABLE 11-5

ANNUAL OPERATION AND MAINTENANCE COST

(1989 $)

CSO Group	Cost	Year First Incurred
Group 1	$212,800	1998
Group 2	43,700	1998
Group 3	38,200	2002
Group 4	111,200	2015(e)
Group 5	83,500	2015(e)
Group 6	148,900	2015(e)
TOTAL	$638,300	

NOTE: (e) - estimated

requirements. The rate at which increased expenditures will accrue is discussed in later sections.

11.4 FINANCING ASSUMPTIONS

This section presents alternative scenarios for raising the capital required to construct and operate the facility. It also provides background information and analysis necessary to assist the City in choosing the funding methods most suitable to its particular circumstances. In order to fully evaluate funding options, the City must take into account its fiscal capabilities and limitations, the impact of increased user charges on sewer customers, possible financial assistance from federal and state programs, and applicable EPA rules and regulations.

11.4.1 SOURCES OF FINANCING

The City has four potential sources of funds to meet the capital costs of the WWTP and CSO abatement plans: assistance from state and federal governments (via grants or loans), borrowing in the public markets, privatization, and retained cash. Cash financing is assumed not practical and this alternative is ignored for purposes of this analysis. Privatization was discussed in the draft Phase 2 WWTP Facilities Plan (Section 12.0 of Volume V). For the purpose of this report, privatization is not discussed. Since, as is shown below, the amount of money available through grant programs is small, this funding source is also not considered in the fiscal impact analysis.

Therefore, the analysis of fiscal impact will concentrate on the remaining two options, assistance from state and federal government SRF programs, and borrowing in the public markets. The new SRF program is the principal source of federal and state assistance as the Construction Grants Program is phased out. Public markets will lend monies to New Bedford for this project, if they judge that the City will be able to repay the loans.

Both the SRF and the public market will provide capital to the City in the form of a loan that must be repaid. The City will have two primary sources of revenue to service that debt: user charges and ad valorem tax receipts.

Federal Grant Assistance

New Bedford is eligible for federal grant monies through the reserve fund for abatement of CSO pollution. If the City is to receive such funding, it will be necessary for the City to demonstrate that "significant usage of the water for shellfishing and swimming will not be possible without correction of the combined sewer overflows." We believe that the City can satisfy that requirement using the findings of this report.

Section 201(n)(2) of the Federal Water Pollution Control Act authorizes this special fund for abatement of CSO pollution in marine bays and estuaries. This statute authorizes annual expenditures of $200 million at the federal level. Massachusetts' share of the federal monies amounts to $6,867,600 a year (0.034338 of total federal monies available). Assuming New Bedford qualifies for the grant money and assuming that they received 50 percent of the total funds available through this program, they would receive about $3.4 million per year. The grant specifies a range of between 45 percent to 75 percent funding for construction-related costs for CSO projects.

These grant monies ($3.4 million) are small compared to the total financial needs of $180 million to complete the CSO abatement projects; the grant would not affect the projected, fiscal impact of the CSO abatement projects.. Thus, for purposes of this document and ease of exposition, the grant monies available under this program will be ignored.

Federal and State Assistance through the SRF Program

Traditionally, local governments have relied on federal and state assistance when building WWTPs and associated infrastructure. However, federal financial contributions to national municipal water pollution abatement programs are now in a state of transition. The Construction

Grants Program is being phased out and replaced with a revolving loan program capitalized with seed grants from the federal government.

Massachusetts is presently developing its SRF program. The Great and General Court enacted Chapter 275, which establishes the program under State law. Massachusetts is currently negotiating with EPA regarding its SRF program. For purposes of this document, we assume that the program has been implemented as described by Chapter 275.

The SRF, when implemented, will provide Massachusetts communities with a single source of capital for wastewater projects. Thus, a community could receive loans to cover all project costs, both eligible and ineligible. To receive funds from this source, communities will be required to submit their projects to DEM for review, and all submitted projects will be ranked. The priority system to be used in the SRF will be very similar to that used by the Construction Grants Program. Given the environmental benefits of the New Bedford plant, its legal status as a national municipal policy (NMP) project, and its court-ordered schedule, New Bedford will likely rank high on DEM's priority list. Chapter 275 authorizes sufficient funding, such that New Bedford could receive 100 percent of its funding needs for the treatment facilities and CSOs from the SRF, assuming the projects are properly sequenced.

The proposed state SRF program will provide assistance to all funded projects equivalent to a project receiving a 45 percent grant. This is accomplished by providing a deeply subsidized loan; in the current market, the effective loan interest rate will be approximately 1.5 percent. For hardship communities, assistance equivalent to a maximum of a 75 percent grant will be available. This subsidy will be delivered by providing the recipient with a 0 percent interest rate loan and a state grant equal to approximately 25 percent of the project's cost. State legislation imposes a cap on the assistance that one municipality can receive in a single year; the cap is set at 50 percent of the loans made in a year. Based on fiscal and economic stress data, New Bedford will most likely qualify for hardship status and thus will qualify for a higher level of assistance.

Public Markets

New Bedford's alternative source of capital funding is to issue bonds in the public market. Given the amount required, this will certainly require a public offering, rather than a private placement. In the past, Massachusetts communities were limited to general obligation (G.O.) financing (i.e., the security for the bonds is provided by a pledge of the community's full faith taxing powers and not the revenue stream of a specific project). A City is judged by the markets based on the perception of how easily it could make good on that pledge. New Bedford, given its fiscal and economic position, will be viewed as a relatively high risk. (The City's current bond rating is BAA, suggesting its relative fiscal position.) However, the need to borrow large amounts in the public markets, combined with the fiscal problems of the state could result in a down grading of that rating and an increase in the cost of borrowing.

Chapter 275 authorizes Massachusetts' cities and towns to issue revenue bonds for wastewater projects. Thus, the City could support debt for this facilities plan with user-fee charges and not rely on its G.O. pledge. This would partially eliminate the pressure such debt would place on the City's tax base and credit rating. The cost of such a revenue bond issue could vary significantly depending on the credit structure of the financing. Specifically, bond holders will closely scrutinize the independence of the wastewater utility in rate setting.

For analytical purposes, we assume that New Bedford would be able to meet its capital needs through the public markets. Such debt is assumed to carry an interest rate of 8.5 percent.

Debt Repayment

Once the City has determined the sources of funds to finance the project, it must decide how to service the debt. The City has basically two sources of revenues that will be available for debt service and O&M expenses. The first is the user charges assessed residents and businesses that discharge into the wastewater system. Alternatively, the City could pledge ad

valorem tax revenues. Of course, the City could rely upon a mixed system with both user fees and *ad valorem* receipts.

Since the City will rely on outside capital to construct this facility (borrowed funds will be utilized), the City will be required to legally pledge revenues to support debt service. This pledge may be the City's general obligation pledge or a pledge of system revenues. The pledge will be given either as part of a bond offering statement or a loan agreement between the City and the state's SRF.

EPA rules governing the SRF program require that the loan recipient "dedicate a source of revenues" to secure the loan. Current EPA regulations further require that a grant or SRF loan recipient adopt a user fee system for OM&R expenses. This system must fully recover all OM&R expenses from users, with costs allocated proportionately based on discharges. For purposes of securing an SRF loan, a dedicated source of revenue may mean user fees, *ad valorem* tax receipts, or any other revenue source controlled by the loan recipient.

In 1989, New Bedford generated approximately 60 percent of its wastewater system revenue requirements ($2.3 million) through user fees. Substantial increases will be required to meet the anticipated increase in OM&R expenses; recovering some or all of the debt service from this source will force user fees even higher.

As a municipality, New Bedford is empowered to assess and collect taxes from residents and property owners within its jurisdiction. The City can, subject to the restrictions of Prop 2 1/2, choose to pay some or all of the debt service associated with the treatment facility with general tax receipts. The allocation of costs between the users of the system and the residents of the City is both a policy and a political issue. There are no absolutely correct allocations.

11.5 QUANTITATIVE ANALYSIS

To assess the financial capability of the City and the impact of implementing the facilities plan, a fiscal impact model was developed to allow variations in a number of assumptions and parameters. The model extrapolates current general municipal expenditures and revenues based on the assumptions described in Section 11.2.4. Future costs associated with the treatment plant and CSO abatement projects are then factored in. One of the most important, if not the most important, parameters is the amount of outside assistance available to the City. Given the current uncertainties, three basic funding scenarios were evaluated:

- The City finances the recommended plan entirely on its own credit. In this case, we assume the City will issue 20-year bonds with an interest rate of 8.5 percent. (Worst Case)
- The City obtains 45 percent grant equivalency assistance from the state through the SRF. (Middle Case)
- The City obtains 75 percent grant equivalency from the state through the SRF. (Best Case)

Two alternatives for cost recovery within each funding scenario were evaluated: (1) all future revenue requirements will be met through user charges, or (2) all incremental revenue requirements will be met through the property tax. Evaluating these two extremes permitted a clear assessment of the fiscal impacts of the recommended facilities on the City and residents. The actual cost recovery system may fall between these two extremes.

The local impact model consists of three primary modules that forecast annual City expenditures and revenues through the year 2005. In the first module, the annualized costs of constructing and operating the plant are calculated and incorporated into the Wastewater Division's projected budget. This module calculates debt service based on the funding scenario and assumptions regarding inflation and capitalized interest. The second module forecasts expenditures and non-property tax receipts for the City, taking into account historical spending patterns and non-sewer related capital expenditures. The third module determines the impact of the

forecasted spending patterns and revenue requirements on the property tax levy and wastewater user charge system.

A variety of parameters related to population, household income, and water consumption were also projected. These parameters permit an assessment of the relative impact of funding scenarios on typical New Bedford households and tax payers.

As described elsewhere, this analysis is done in constant dollar-value terms (i.e. with no inflation.) We have further assumed that the City's tax base, state grants, and household income show no real growth. In general, these are considered conservative assumptions, since they tend to provide a worst case result. The two assumptions regarding the future path of state aid are arguably optimistic, which would cause the estimates of property tax needs and rates to be understated. Thus, the worst case analysis is an important reference for this evaluation.

It is also assumed that the cost of constructing the plant in real terms will not increase with time. Many analysts believe that construction costs in New England will grow more rapidly than inflation because of the anticipated high level of construction activity will cause shortages of labor and certain material. Thus, this may also cause underestimation of the future debt service requirements facing the City.

11.6 FISCAL IMPACTS

This section presents the results for the three primary funding scenarios in terms of their impact on the fiscal health of the City. This analysis of fiscal impact examines the cumulative impact on the City and its residents of both the secondary WWTP and its associated facilities and the CSO abatement projects. Together these projects require capital expenditures of nearly $430 million (1989 dollars). We examine the change in parameters such as property tax rate, excess levy required, excess levy as a percent of total levy, sewer rate, and wastewater division budget as a percent of total City budget.

Two sets of results are presented corresponding to the alternative cost recovery assumptions. The first (Table 11-6) assumes that all wastewater revenue requirements are raised through user fees. The second (Table 11-7) places nearly all of the future costs on the City's tax base. (In this case, the City is assumed to continue to generate rate revenue under the same rate structure that was in place in 1989; this generates approximately $2.3 million.)

Clearly, the greater the amount of state and federal assistance, the less severe the fiscal impact experienced by the City. When previously issued debt and indirect expenses are factored in, the total cost of the Wastewater Division's 1989 budget is estimated to be $5.3 million. This represented less than 5 percent of the City's total expenditures in 1989. If the City finances the treatment plant and CSO abatement projects in the public market, wastewater system expenses will increase to approximately $51 million (1989 dollars) by the year 2005. This will represent almost 31 percent of the City's projected budget. Wastewater expenditures, under that scenario, are projected to increase at annual average rate of 14.6 percent, net of inflation. Under the high SRF assistance scenario, the City's wastewater expenditures in the year 2005 are estimated to total over $22 million, or 16.2 percent of municipal expenditures.

New Bedford residents will be carrying a much larger debt burden. As discussed in Section 11.2.1, debt per capita, as of June 30, 1989, was less than $210. By the year 2002, when all debt for the treatment plant and the first three CSO abatement projects has been issued, that ratio will increase to over $4,500. This figure includes only the treatment plant and CSO abatement projects; overlapping debt will add approximately $750, for a total debt per capita of over $5,000 (1989 dollars).

The actual cost of debt to the City and its residents will depend on the debt service costs associated with it. Table 11-6 presents results of the fiscal impact analysis assuming all revenues are derived from user fees. This table shows that the availability of assistance from the SRF will make a significant difference. The City's expenditures on an accrual basis, for wastewater, are projected to begin increasing in 1995 as the City begins

TABLE 11-6

FISCAL IMPACT OF SECONDARY WWTP AND CSO ABATEMENT PROJECTS

USER FEE SCENARIO

Impact Measure	Capital Funding Scenario New Bedford	SRF--45%	SRF--75%
Sewer Use Fee--1990 ($ per 000 gals)	$1.33	$1.33	$1.33
Sewer Use Fee--1995	1.52	1.39	1.35
Sewer Use Fee--2000	10.77	6.33	4.81
Sewer Use Fee--2005	11.74	6.79	5.10
Total Division Budget--1990	$5,820.3	$5,820.3	$5,820.3
Total Division Budget--1995	6,679.4	6,107.9	5,915.0
Total Division Budget--2000	47,159.4	27,721.3	21,084.9
Total Division Budget--2005	51,432.0	29,737.0	22,347.1
Total Division Budget as % of Total--1990	4.7%	4.7%	4.7%
Total Division Budget as % of Total--1995	5.5%	5.0%	4.9%
Total Division Budget as % of Total--2000	29.0%	19.4%	15.4%
Total Division Budget as % of Total--2005	30.7%	20.4%	16.2%

NOTE: Dollar figures are in thousands, unless stated otherwise.

TABLE 11-7

FISCAL IMPACT OF SECONDARY WWTP AND CSO ABATEMENT PROJECTS

PROPERTY TAX SCENARIO

Impact Measure	Capital Funding Scenario New Bedford	SRF--45%	SRF--75%
Tax Rate--1990	$23.34	$23.34	$23.34
Tax Rate--1995	$23.77	$23.37	$23.23
Tax Rate--2000	$52.81	$39.00	$34.29
Tax Rate--2005	$56.17	$40.75	$35.50
In excess of Levy--1990 ($ in millions)	0	0	0
In Excess of Levy--1995	0	0	0
In Excess of Levy--2000	39.15	19.70	13.07
In Excess of Levy--2005	43.87	22.17	14.78
In Excess of Levy as % of Total--1990	0%	0%	0%
In Excess of Levy as % of Total--1995	0%	0%	0%
In Excess of Levy as % of Total--2000	52.7%	35.9%	27.1%
In Excess of Levy as % of Total--2005	55.5%	38.7%	29.6%

repayment of certain loans incurred for project construction. However, a large increase occurs in 1996, when the plant goes on line. CSO Groups 1 and 2 will be financed in 1996 and will affect 1998 rates; Group 3 projects will affect rates in 2002. By the year 2005, City residents will be paying a sewer use fee of $5.10 per 1000 gallons, assuming high SRF assistance. That will represent a 283 percent increase over 1990 rates. If the City is unable to obtain SRF assistance the sewer use fee in 2005 is projected to exceed $11.74 per 1000 gallons.

Obviously, these projected rate increases are associated with large increases in revenue requirements. The total Wastewater Division budget, including non-wage costs of employment such as health insurance, pensions and indirect overhead, is projected to increase from $5.8 million in 1990 to $22.35 million in 2005 under the high SRF case, and to $51.43 million under the New Bedford financing scenario. Two factors stand out from this analysis:

- New Bedford will be required to increase wastewater expenditures by at least $16.5 million over the period from 1990 to 2005. To put this in perspective, in 1990 the City's locally-generated revenues will total less than $43 million. The major local source of revenues, the property tax, has an effective limit of approximately $35 million and currently generates approximately $29.7 million.

- Assistance from the SRF can significantly reduce the fiscal burden on the City. If the City obtains funding under the high SRF scenario, total revenue requirements will grow by $16.5 million from 1990 to 2005. If funding is received under the low SRF scenario, revenue requirements will increase by almost $24 million. However, if the City funds the project in the public market, revenue needs will increase by $45.6 million. The difference between the high SRF and the City funding scenarios represents an annual per capita cost of $300.

The preceding analysis assumed that all future revenue requirements were met by increasing user charges. The City has the option of meeting revenue needs by raising ad valorem tax revenues. Table 11-7 presents results assuming that the property tax is the revenue source. This does not change the overall revenue requirements, but it does affect how the burden is allocated and felt.

Table 11-7 demonstrates the fiscal stress that implementation of the facilities plan can have on the City. If the City chose to finance the recommended facilities with general revenues, the tax rate would need to be increased. The resulting tax rate would exceed the Prop 2 1/2 limit, and an override or exclusion would be required as early as 1995. By the year 2000, if the tax base secures debt, an override or exclusion will be required regardless of financing source.

11.7 USER IMPACTS

The proposed facilities plan will also directly affect the citizens and businesses located in New Bedford. The actual impact on any particular individual will depend on:

- The cost recovery system (user charges or property taxes) that is used by the City.
- A household's consumption pattern for water.
- The value of the property owned by a person.

Table 11-8 summarizes the impact on individuals of the three funding scenarios. For this table, it was assumed that the City's revenue requirements are met through user charges, and that the median family of four consumes approximately 288 gallons of water per day, generates about 90 percent of water consumption as wastewater (260 gallons), and has an income in 1989 of $28,000. The estimated impacts for all future projections are based on 1989 dollars.

The table demonstrates the impact the WWTP and CSO abatement facilities will have on the average resident of New Bedford. In the high SRF funding case, the average household sewer bill in the year 2000 will be about 3.5 times higher than the average bill in 1990. If New Bedford finances the plant, the average household bill in the year 2000 will be 8 times higher than the 1990 average. The average annual sewer use charge in the year 2000 is likely to range somewhere between these amounts, or from $507 to slightly under $1200. These amounts represents between 1.78 percent and 3.98 percent of average household income in the year 2000.

TABLE 11-8

IMPACT ON USERS

Impact Measure	Capital Funding Scenario New Bedford	SRF--45%	SRF--75%
Average Sewer Bill--1990	$140.00	$140.00	$140.00
Average Sewer Bill--1995	$160.00	$147.00	$142.00
Average Sewer Bill--2000	$1132.00	$665.00	$506.00
Average Sewer Bill--2005	$1234.00	$714.00	$536.00
Sewer Bill as % of Household Income--1990	0.49%	0.49%	0.49%
Sewer Bill as % of Household Income--1995	0.56%	0.52%	0.50%
Sewer Bill as % of Household Income--2000	3.98%	2.34%	1.78%
Sewer Bill as % of Household Income--2005	4.34%	2.51%	1.89%
Taxes + Sewer as % of Income--1990	3.74%	3.74%	3.74%
Taxes + Sewer as % of Income--1995	3.78%	3.73%	3.72%
Taxes + Sewer as % of Income--2000	7.24%	5.60%	5.04%
Taxes + Sewer as % of Income--2005	7.65%	5.82%	5.20%

The impact will be more severe for hardship households in New Bedford. In 1990, the poverty level is estimated to be approximately $12,000 (1989 dollars). Therefore, in 2000, the average household below the poverty level will face sewer bills of between approximately 7 percent and 11 percent of income based on 1989 dollars. This is a significant burden.

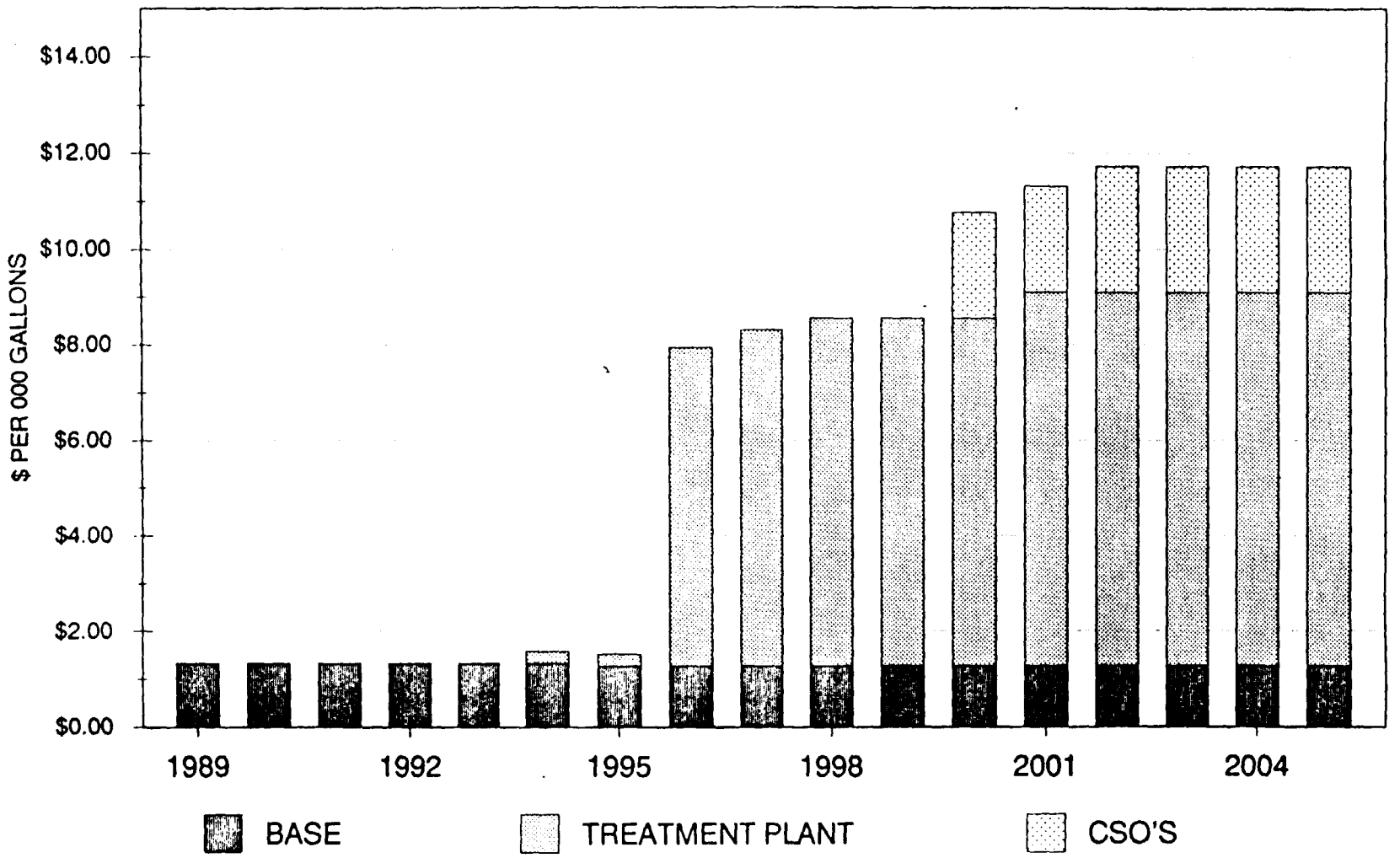
11.8 INCREMENTAL EFFECT OF CSO ABATEMENT PROJECTS

The preceding sections (11.7 and 11.8) have dealt with the cumulative impact of the WWTP and CSO abatement projects on the City and its residents. The purpose of this section is to briefly highlight the impact due only to the CSO abatement projects. During this study's planning horizon, CSO facilities will require the City to finance nearly $100 million in construction, as compared to the nearly $300 million for treatment facilities.

Figures 11-2 and 11-3 illustrate the effects these capital programs have on the sewer rates that the City will be required to assess. Figure 11-2 is based on the assumption that New Bedford must finance all projects on its own credit; Figure 11-3 is based on the assumption that the maximum amount of SRF assistance (75 percent assistance) is obtained.

Under the worst case scenarios in the year 2005, a sewer rate of $11.74 per 1000 gallons would be required to recover all costs. Approximately $1.26, or less than 11 percent is attributable to the costs of operating and maintaining the current primary treatment facilities. The cost of construction, operating and maintaining the facilities recommended in the secondary treatment plant would require $7.85 or 67 percent of the rate. CSO facilities will add $2.63 or 22 percent to the rate.

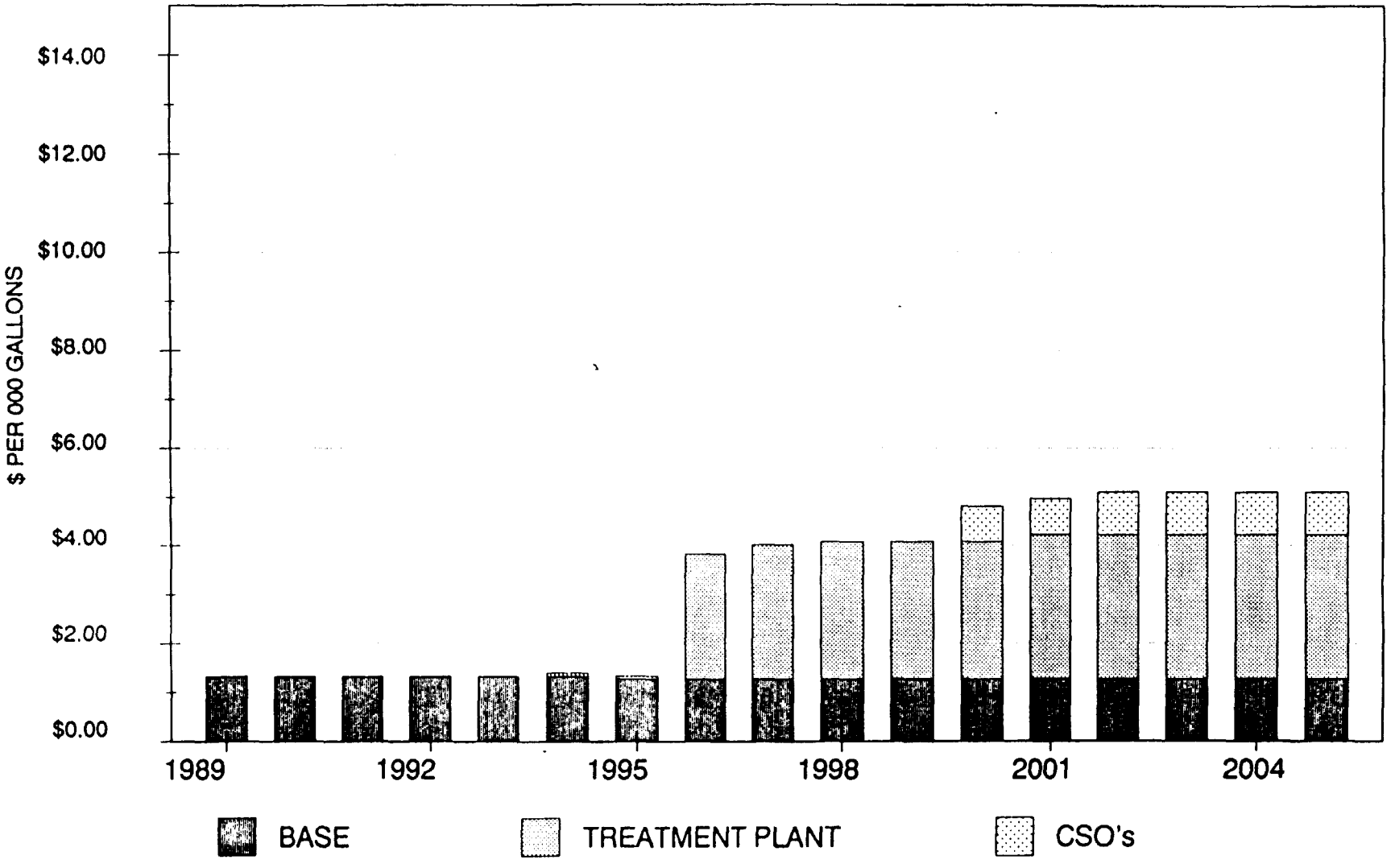
Figure 11-3 shows that under maximum SRF assistance the required rate is less than half of that required in the worst case ($5.10 versus $11.74). A higher portion of the maximum SRF rate is attributable to existing facilities (24.7 percent). Secondary treatment facilities would remain the dominant cost item accounting for almost 60 percent of the rate. CSOs add $0.87 to the rate, or 17 percent of the total.



NEW BEDFORD FINANCES ALL IMPROVEMENTS

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 11-2
IMPACT OF CAPITAL PLANS
ON SEWER RATES



ASSUMES 75% SRF ASSISTANCE

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

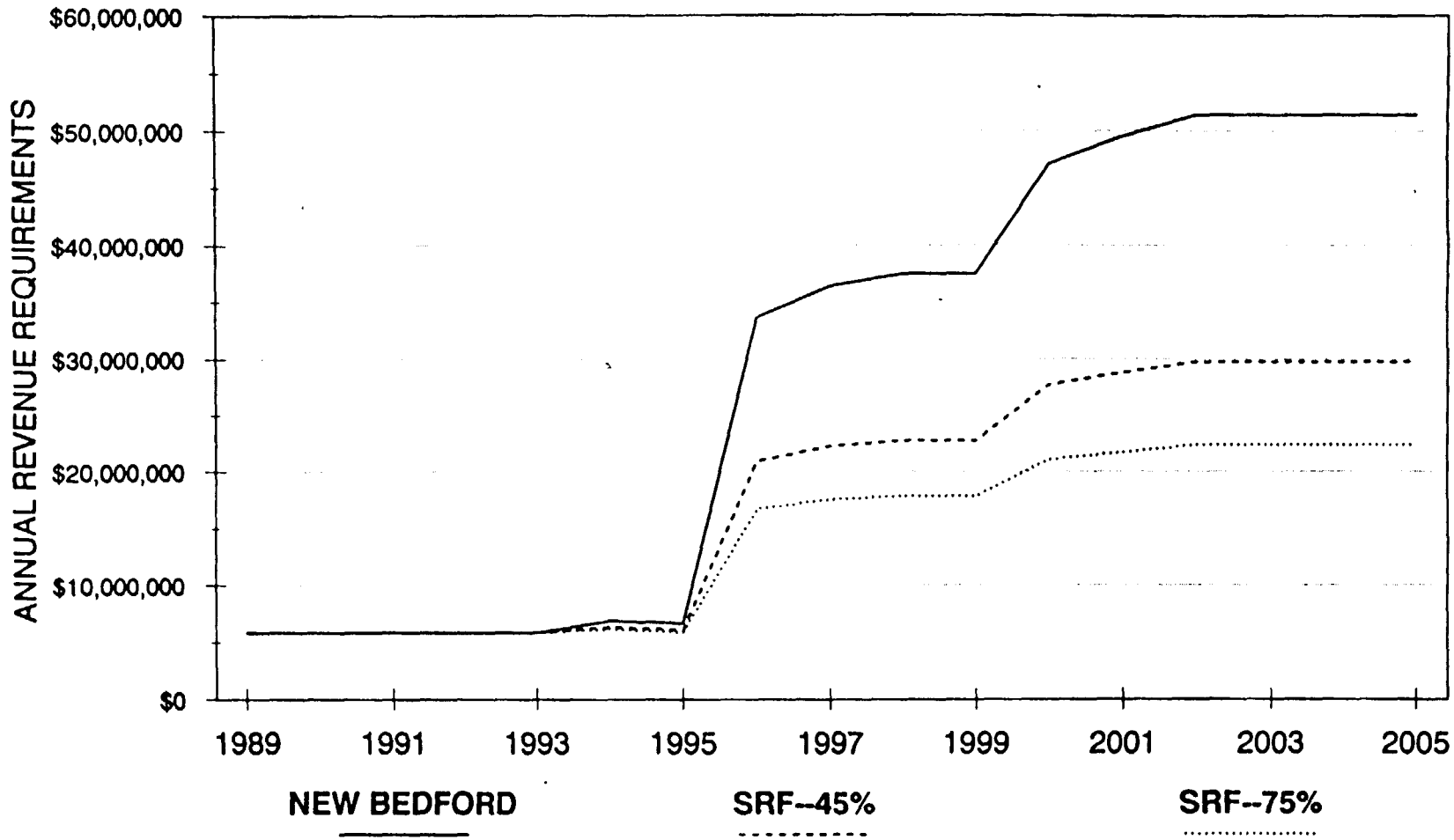
FIGURE 11-3
IMPACT OF CAPITAL PLANS
ON SEWER RATES

11.9 FINDINGS AND CONCLUSIONS

The implementation of the recommended facilities represents a major financial burden to the City of New Bedford. As seen in Figure 11-4, the cost of wastewater treatment will escalate from $5.8 million in 1990 to between $22 and $52 million (1989 dollars) by the year 2005. Wastewater services will account for between 16.5 and 32 percent of municipal expenditures in 2005, as compared to 4.7 percent in 1990. This represents a radical shift in municipal expenditures. Clearly, the SRF can ameliorate, but not eliminate, the adverse fiscal impact.

The City, given current state law, has little choice but to finance the recommended facilities with user fees. This will cause the sewer rate to increase from $1.33 per 1000 gallons to between $5.10 and $11.75 by the year 2005. The average family of 4 in New Bedford will be devoting as much as 4.4 percent of income to wastewater treatment and collection; in 1990, the comparable figure is less than 0.5 percent. Debt per capita will range between $4,000 and $5,000.

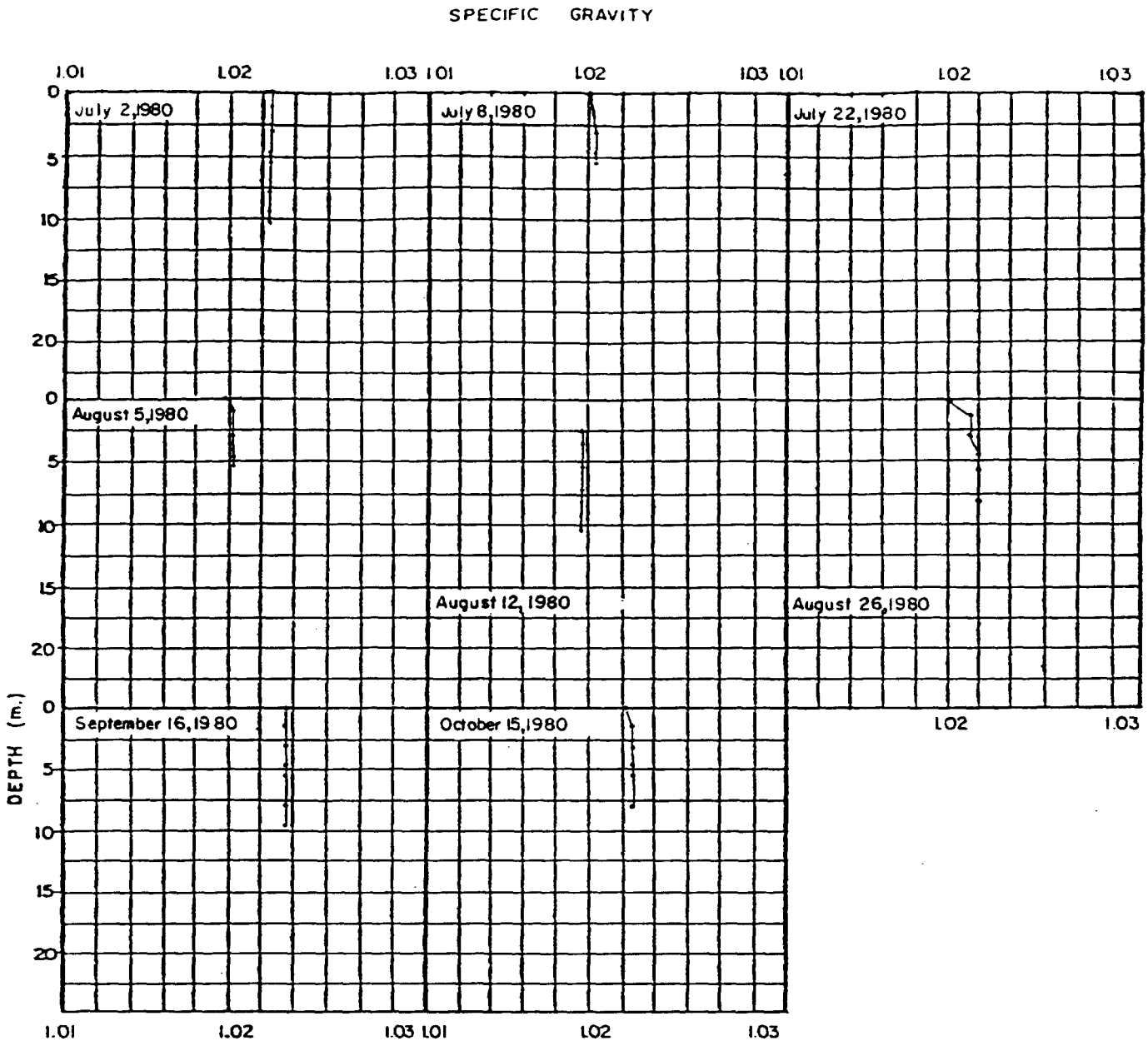
We anticipate that the implementation of these projects will have a significant adverse impact on the fiscal position of New Bedford. By a wide range of measures, including those suggested in EPA's Financial Capability Handbook, implementation of this plan will adversely impact the City's fiscal health. For many indicators, the City will move from average to weak; in other cases the City's position was already considered weak and the wastewater projects place greater strain. The weakening of the City's fiscal position will occur even with the 75 percent grant equivalency that may be available from the Massachusetts State Revolving Fund.



VALUES ARE IN CONSTANT 1989 $

City Of New Bedford
COMBINED SEWER OVERFLOW
FACILITIES PLAN - PHASE 3

FIGURE 11-4
IMPACT OF FUNDING SOURCE
ON REVENUE REQUIREMENTS



NEW BEDFORD HARBOR STUDY
SPECIFIC GRAVITY PROFILES
STATION B

FIGURE 10

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NEW BEDFORD HARBOR

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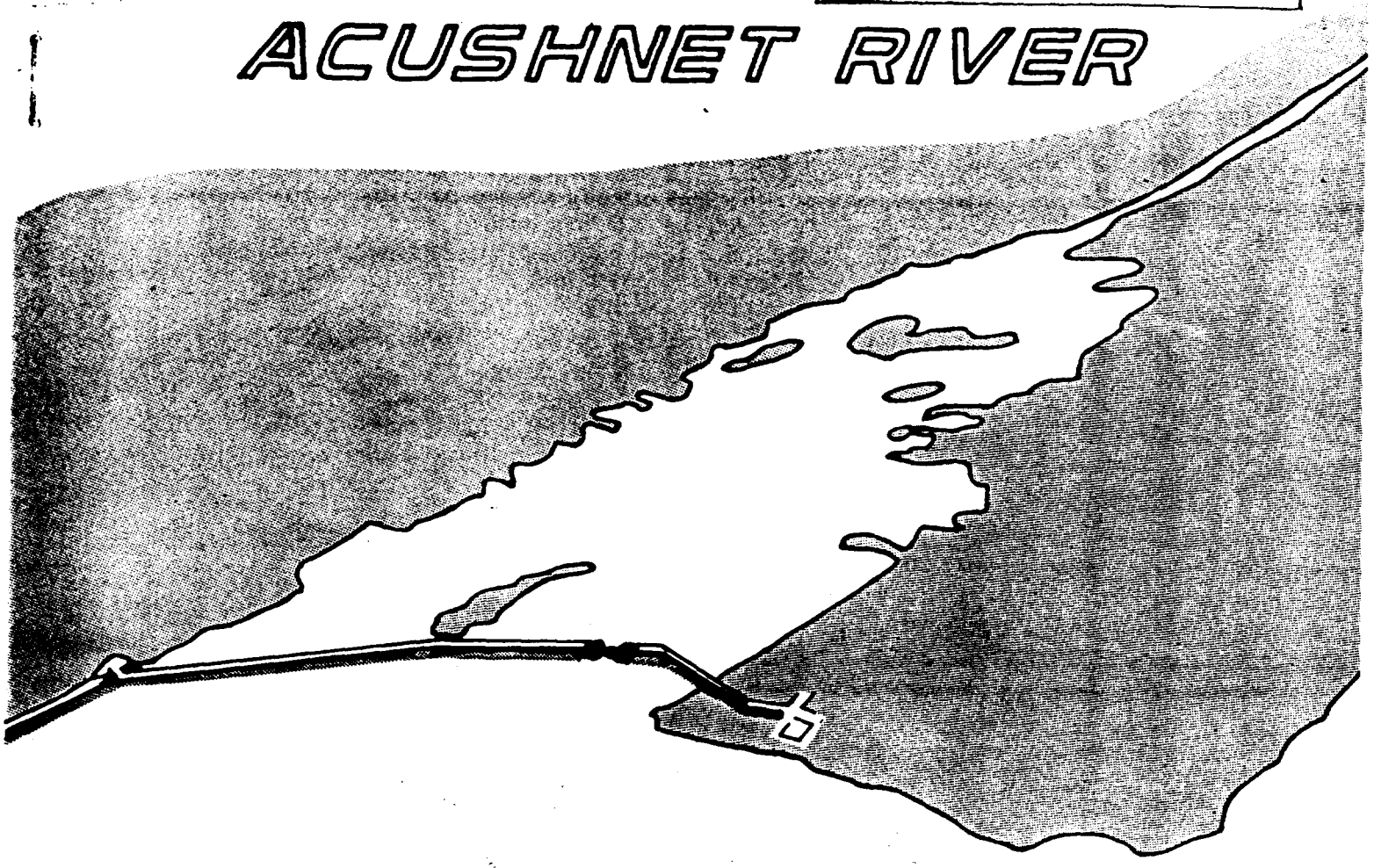
ACUSHNET RIVER



WATER QUALITY STUDY 1971

Water Resources Commission

Division Of Water Pollution Control

Thomas C. McMahon Director

Q.VIII.e.144

ACUSHNET RIVER - NEW BEDFORD HARBOR

WATER QUALITY STUDY 1971

PREPARED BY

WATER QUALITY MANAGEMENT SECTION

DIVISION OF WATER POLLUTION CONTROL

MASSACHUSETTS WATER RESOURCES COMMISSION

BOSTON

JANUARY, 1972

Publication Number: 6046

Approved By: Alfred C. Holland
Purchasing Agent

DEDICATION

This report is dedicated to the memory of Manuel Brown, Fairhaven Shellfish Warden, who passed away last Fall. Manny will be remembered fondly by all of us who knew him and worked with him during the survey. His willingness to help in any way he could as well as his good humor were a great help to the survey.

FOREWORD

Water Pollution problems in the Acushnet River and New Bedford Harbor were studied extensively in the summer of 1971 by the Southeast Regional Office and the Water Quality Management Section of the Division of Water Pollution Control. Increasing awareness of these problems by the local citizenry brought about a great deal of interest and cooperation, particularly from the Town of Fairhaven and the City of New Bedford.

Following a series of meetings and preliminary investigations, the survey crew arrived in New Bedford on July 12. Field lab facilities were set up at the new Fairhaven Sewage Treatment Plant. Five photosynthesis stations were established employing the light and dark bottle technique. Assistance was provided by Romeo Mosakowski, Shellfish Warden for the City of New Bedford. Sampling began early the following morning as six locations on the Acushnet River and five on the outer harbor (beyond the Hurricane Barrier) were sampled four times over a 24-hour period. The outer harbor stations were sampled at two depths, one foot below the surface and one foot above the bottom. Dissolved oxygen was measured for each sample at the field lab. The four samples from each location were composited into one and conveyed to the Lawrence Experiment Station of the Massachusetts Department of Public Health for analysis. Procedures set forth in Standard Methods for the Examination of Water and Wastewater (13th edition, 1971, American Public Health Association, New York) were followed in all tests. The sampling was repeated on July 15.

During the first week of sampling, most of the industries which discharge wastes to the harbor were closed. They reopened on July 19. The sampling that week therefore concentrated on the inner harbor. Two new photosynthesis stations were established in the area of the industries and the two original stations in the harbor behind the Hurricane Barrier were continued. Fairhaven Shellfish Warden Manuel Brown assisted Division personnel in this study. Three locations in the inner harbor were sampled. East, west and middle points at each location, top and bottom were sampled four times daily on July 20 and 22. Testing procedures were the same as the first week. Samples of bottom sediment were collected from all stations on July 21. Results of the survey were verified and placed in tabular form by engineers of the Water Quality Management Section.

Later in the summer, personnel from the Southeast Regional Office, assisted by U.S. Coast Guard Reserves, conducted a shoreline survey of the harbor in which all discharges were located and identified. Details and results of that survey were prepared for this report by the regional office.

ACUSHNET RIVER - NEW BEDFORD HARBOR

WATER QUALITY STUDY

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COMMONWEALTH OF MASSACHUSETTS

WATER POLLUTION CONTROL

HISTORICAL BACKGROUND

The need for governmental control to safeguard the waters of the Commonwealth was first recognized by the Massachusetts General Court in 1886. In that year, a State Division of Sanitary Engineering was established and given the responsibility to examine and advise on public health problems relating to water. In 1945, the control of water pollution was strengthened with the enactment of legislation which authorized the Massachusetts Department of Public Health to adopt water pollution regulations.

The Federal Government became deeply committed to the abatement of water pollution in 1956 when Congress passed the Federal Water Pollution Control Act. This Act provided for limited financial aid by the Federal Government to municipalities constructing pollution abatement facilities. Amendments to the Act increased the amount of aid available to 50 percent of the construction cost, provided that the State agreed to pay not less than 25 percent and had established Water Quality Standards for all its waterbodies and an enforcement program for reaching and maintaining these standards. The standards were subject to the approval of the Secretary of the Interior, who was given the authority to adopt standards for any State which failed to do so. An additional 5 percent grant was made available to any project which was consistent with the master plan of a regional planning agency for pollution control.

MASSACHUSETTS CLEAN WATERS ACT

In order to establish a strong water pollution control program in the Commonwealth in coordination with the provisions of the Federal Act, the General Court passed the Massachusetts Clean Waters Act in 1966. This Act provided for the establishment of a Division of Water Pollution Control under the Water Resources Commission. The Division was given comprehensive duties and responsibilities including the establishment of Water Quality Standards, the classification of all waters in the Commonwealth, and the establishment of a schedule for the implementation of the standards. To effect compliance with this schedule, the Division was authorized to issue orders to violators or to institute criminal proceedings. The Act further made it unlawful for any person to discharge into the waters of the Commonwealth any substances which would cause or contribute to a condition in contravention of the water quality standards, or to make or permit an outlet for the discharge of wastes without obtaining a permit from the Division. The Act has been strengthened each year through legislative amendments.

WATER QUALITY STANDARDS

After public hearings in the Spring of 1967, the Division of Water Pollution Control established water quality standards for all interstate, intrastate, and coastal waters in the Commonwealth. Included in the standards were present and proposed use classifications for each waterbody, critical limits of the amounts of various pollutants allowable in the waters for each use classification, and a plan for the implementation and enforcement of the standards. In August, 1967, the standards were approved by the Secretary of the Interior, making Massachusetts the third state in the union to have such standards approved.

ORGANIZATION OF THE DIVISION OF WATER POLLUTION CONTROL

The main office of the Division of Water Pollution Control is located in the Leverett Saltonstall Building in Boston. This office consists of an administrative staff, a legal counsel, and operational sections to administrate the broad functions of the program. These sections reflect the major areas of concern in the area of water pollution control. Their duties are as follows:

CONSTRUCTION GRANTS - In an era of rapidly rising construction costs, financial assistance is imperative for municipalities planning pollution abatement facilities. Legislation enacted concurrently with the Massachusetts Clean Waters Act provided a 150 million dollar bond issue to provide such aid. This amount was increased by 250 million dollars by legislative action in 1970. In order to be eligible for aid, a project must be approved by the Division. The Construction Grants Section reviews engineering reports, final plans and specifications for municipal treatment facilities and applications for grants for the following purposes:

1) Comprehensive planning of wastewater treatment facilities.

2) Preparation of final plans and specifications for wastewater treatment facilities.

3) Construction of wastewater treatment facilities.

4) Prefinancing of Federal grants for wastewater treatment facilities.

ENFORCEMENT - The schedule for implementation of the Water Quality Standards established dates for reaching the several steps of a pollution abatement program for each of some 550 known municipal and industrial polluters. The Enforcement Section is responsible for effecting compliance with this schedule through cooperative action where possible and legal measures when necessary.

INDUSTRIAL WASTES - The large number as well as the diversity of the industries discharging wastes to the waters of the Commonwealth requires critical review of reports and plans for industrial treatment facilities. In reviewing proposed designs, engineers of the Industrial Wastes Section meet with the industries and their consulting engineers to insure installation of the most efficient pollution abatement facilities. Additional activities of this section include inspection of newly-operating industrial treatment plants, processing of application for certification of treatment facilities as eligible for tax benefits provided under the General Laws, assisting in locating new sources of pollution and establishing implementation schedules, evaluation of monthly operating reports from existing industrial waste treatment plants, and directing pilot plant studies on the treatment of industrial wastes.

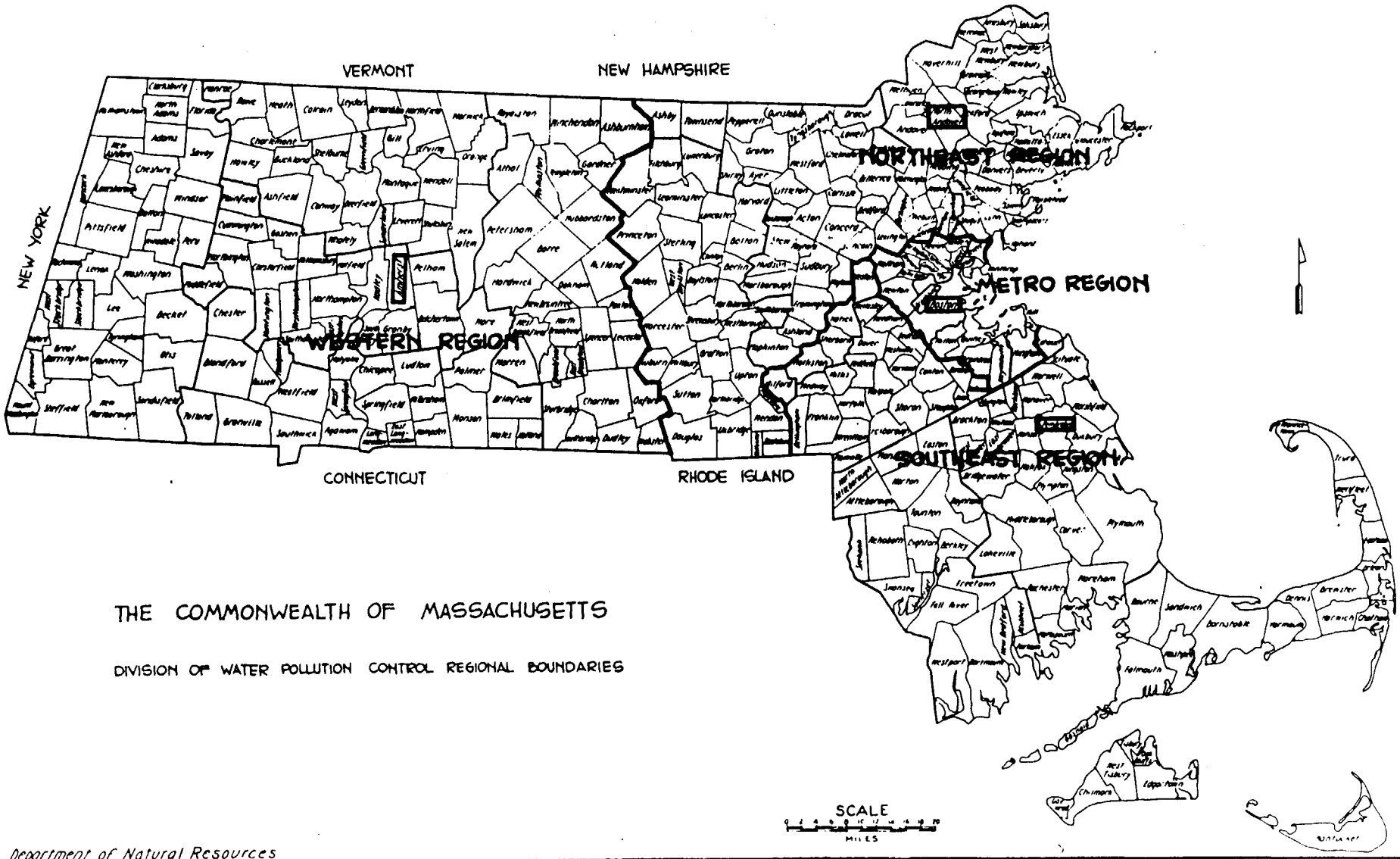
OIL POLLUTION - Increased concern with the detrimental effects of oil pollution led to the passage of the Massachusetts Oil Pollution Act of 1968, which was further strengthened in 1970. This Act provided for the immediate clean-up of all oil spills and the licensing by the Division of all marine oil terminals and waste oil collectors. The Oil Pollution Section initiates the clean-up of oil spills when required to protect water quality, finances the operation, then recovers funds from the source of the spill. The section also maintains a continuing program of research to develop and evaluate new methods for oil pollution abatement.

OPERATION AND MAINTENANCE - In order to effectively abate water pollution, a treatment plant must be operated properly. The Operations and Maintenance Section attempts to insure proper operation through plant inspections, review of monthly operating records, plant personnel training programs, and, in extreme cases, direct orders from the Division and refusal to approve applications for permits for new sewers. The mandatory certification of plant operators should upgrade the quality of plant operation throughout the Commonwealth.

RESEARCH AND TRAINING - The Massachusetts Clean Waters Act authorized 10 million dollars over a 10-year period for a research and demonstration program. This program is designed to provide increased knowledge in the areas of analysis and abatement of water pollution. Research grants are made to consulting engineers, universities, and research institutions to conduct particular projects. Training activities of this section include treatment plant operator training, in-service training of Division personnel, and a scholarship intern program for engineering students.

WATER QUALITY MANAGEMENT - In order to properly solve water pollution problems, it is necessary to consider entire river basins rather than particular waste discharges. The waters of the Commonwealth are divided into 17 major river and coastal basins. The Water Quality Management Section conducts surveys of these basins and analyzes and publishes the results. Where sufficient data is available, a river is analyzed by a computer model. Such models simulate the level of dissolved oxygen and other parameters along the entire length of a stream, enabling section personnel to determine the degree of treatment required at each discharge to maintain the desired level. This section also develops basin plans which present alternative solutions to pollution problems in a particular basin.

REGIONAL OFFICES - The Division maintains four regional offices which have responsibility for particular river basins. The Regional Office locations and regional boundaries are indicated on the attached map. Besides assisting the seven sections of the central office in their functions, the regional offices investigate complaints of water pollution, conduct field investigations of discharges, visit construction sites to supervise progress on pollution abatement projects, inspect and sample wastewater treatment plants to provide precise data on their efficiency of operation, conduct local operator training courses, and provide liason between local groups and the Division. Regional personnel acquire an intimate knowledge of local problems which is invaluable to the Division's planning.



THE COMMONWEALTH OF MASSACHUSETTS

DIVISION OF WATER POLLUTION CONTROL REGIONAL BOUNDARIES

Department of Natural Resources

MEASURES OF WATER POLLUTION

The term "water pollution" has acquired many connotations. Literally, the word pollute means "render impure;" thus, in this sense, any water containing matter other than its chemical constituent of two parts hydrogen to one part oxygen would be considered polluted. Such "pure" water, however, is never found in natural bodies; the ecological balance in a waterbody is dependent on the presence of other material. In this report, water pollution refers to a condition which is in contravention of the Water Quality Standards. Pollution degrades the physical, chemical, and bacterial quality of a waterbody and can make it unsightly, malodorous, and a health hazard, its uses sharply limited. Pollution occurs mainly through the discharge of wastes from homes and industries. The various types of pollution are: (1) oxygen-demanding, such as originates from domestic sewage and certain industrial wastes, (2) toxic materials as in some industrial wastes, (3) radioactive, (4) thermal, (5) bacterial, (6) oil, and (7) physical. Stormwater runoff from both urban and rural areas can also add pollutants to a waterbody.

The extent of pollution in a particular waterbody is determined by measuring certain chemical and biological constituents and properties. Chemical constituents, such as dissolved oxygen, phosphates, and metals, are generally measured in milligrams per liter (mg/l); since the unit weight of water is 1.0 grams per milliliter, milligrams per liter are roughly equivalent to parts per million for a solution which is mostly water.

Dissolved Oxygen (D.O.) refers to the uncombined oxygen in water which is available to aquatic life. Since this oxygen is consumed more rapidly in the decomposition of wastes, the D.O. gives an instantaneous picture of the condition of a waterbody. Time of day and temperature of the water are important in interpreting D.O. Levels. Temperature affects the amount of oxygen which water can contain. Time of day is related to the effects of algae. Algae consume oxygen through respiration throughout the day and night. During daylight hours, they add oxygen through photosynthesis. D.O. levels are therefore generally highest during the afternoon and lowest just before sunrise.

Biochemical Oxygen Demand (B.O.D.) measures the amount of oxygen required by bacteria to decompose organic matter. The B.O.D. is gradually exerted, consisting of two stages. In the first stage, carbonaceous matter is stabilized while nitrogenous substances are broken down in the second stage. The second stage (nitrification) usually begins after seven days. The ultimate, or total, B.O.D. from both stages may require an incubation period of 30 days or more. Through recurrent use, the 5 day B.O.D. has become the standard test used in water quality analysis.

Chemical Oxygen Demand (C.O.D.) refers to the amount of oxygen required to chemically oxidize waste material. Since some of the organic matter in a waste cannot be decomposed by microorganisms but can be broken down by chemical oxidation, the C.O.D. is generally greater than the B.O.D.

The C.O.D. is especially useful in analyzing a waste that contains a great deal of non-biodegradable matter.

Total Solids measures all solids in water including suspended and dissolved, organic and inorganic. They are measured by evaporating the water from a sample of known volume and weighing the residue remaining. The residue then can be ignited in a laboratory furnace to determine the organic portion. The loss on ignition is considered organic and the remaining residue, known as fixed solids, is considered to be inorganic.

Suspended Solids are those which can be removed by passing the water through a filter. The remaining solids are called dissolved solids. Suspended solids provide a good measure of the efficiency of a sewage treatment plant; primary treatment should remove about 50 percent of the suspended solids while secondary treatment should remove about 90 percent.

Coliform Bacteria are found in abundance in the intestinal tract of warm-blooded animals. They are not harmful in themselves, but their presence indicates that pathogenic bacteria also may be present. Since they can be detected by relatively simple test procedures, coliforms are used to indicate the extent of bacterial pollution from sewage. Bacterial tests usually measure the fecal coli and the total coli. Fecal coli make up about 90 percent of the coliforms discharged in fecal matter. Non-fecal coli may originate in soil, grain, or decaying vegetation.

pH measures the hydrogen ion concentration on an inverse logarithmic scale ranging from 0 to 14. pH values under 7 indicate more hydrogen ions and therefore more acidic solutions; pH values over 7 indicate less hydrogen ions and therefore more alkaline solutions. A pH of 7 indicates a neutral solution. Alkalinity is a quantitative measure of the alkaline materials present while acidity is a quantitative measure of acidic materials.

Nutrients are compounds which act as fertilizers for aquatic organisms. Small amounts are necessary to the ecological balance of a waterbody, but excessive amounts can upset the balance by causing excessive growths of algae and other aquatic plants. Sewage discharged to a waterbody usually contains large amounts of carbon, nitrogen, and phosphorus. The concentration of carbonaceous matter is reflected in the B.O.D. test. Additional tests are run to determine the concentrations of nitrogen and phosphorus.

Phosphorus appears in waterbodies in combined forms known as ortho- and poly- phosphates and organic phosphorus. The majority of the phosphorus contained in domestic sewage and industrial wastes comes from detergents. Additional phosphorus may enter a waterbody in agricultural runoff where fertilizers are used.

Nitrogen in the form of organic nitrogen decomposes into ammonia nitrogen, nitrite nitrogen and nitrate nitrogen. Since each decomposition reaction is dependent on the preceding one, the progress of decomposition can be determined in terms of the relative amounts of these four

forms of nitrogen.

Ammonia Nitrogen is present in sewage and is also generated from the decomposition of organic nitrogen. It can also be formed when nitrites and nitrates are reduced. Ammonia is particularly important since it has high oxygen and chemical demands and is also toxic to fish.

Nitrite Nitrogen is the oxidation product of ammonia. It has a fairly low oxygen demand and is rapidly converted to nitrate. The presence of nitrite nitrogen usually indicates that active decomposition is taking place.

Nitrate Nitrogen is important since it is the end product in the aerobic decomposition of nitrogenous matter. Nitrogen in this form is readily available to plants.

Turbidity is the measure of the clarity of a water sample. It is expressed in Jackson Standard Units which are related to the scattering and absorption of light by the water sample.

Color is determined by visual comparison of a sample with known concentrations of colored solutions and is expressed in standard units of color. Certain waste discharges may turn water to colors which cannot be defined by this method; in such cases, the color is expressed qualitatively rather than numerically.

Specific Conductance yeilds a measure of a water sample's capacity to convey an electric current. It is dependent on temperature and the concentration of ionized substances in the water. Distilled water exhibits specific conductance of 0.5 to 2.0 micromhos per centimeter while natural waters show values from 50 to 500 micromhos per centimeter.

The above parameters are measured in most water quality surveys. Other constituents such as metals or radioactivity are measured in areas where particular problems are known to exist. Microscopic examinations are conducted on most surveys to measure the amount of algae and other microorganisms present. Additional samples of the river bottom are usually collected in order to determine the types of deposits present. Decomposition of organic suspended matter which settles to the bottom will exert an oxygen demand on the water.

Two types of samples are collected for analysis: grab and composite. A grab sample is an instantaneous sample collected to show conditions at a particular time. Composite samples are collected over a period of time at specific intervals, giving a better picture of the overall water quality situation for the time covered.

Certain levels of the above parameters occur naturally in waterbodies. Since these levels vary among the different ponds, streams, and coastal waters, the following tables are presented for the sake of reference. Table A summarizes the numerical limits for certain parameters as specified by the Massachusetts Water Quality Standards. Table B lists levels found in unpolluted reaches of various Massachusetts waters.

TABLE A: SPECIFIED LEVELS OF CERTAIN PARAMETERS

MASSACHUSETTS WATER QUALITY STANDARDS

	Dissolved Oxygen	pH	Coliform Bacteria	Total Phosphate	Ammonia Nitrogen
Class A	Not less than 75% of saturation for 16 hours of any 24 hour period and never less than 5 mg/l.	As naturally occurs.	Not to exceed an average of 50 per 100 ml. for any monthly period.	No Standard (As naturally occurs)	No Standard (As naturally occurs)
Class B	Same as above	6.5-8.0	Not to exceed an average value of 1000 for any mo th nor 2400 in more than 20% of samples collected.	Not to exceed an average of 0.05 mg/l during any month.	Not to exceed an average of 0.5 mg/l as N during any month.
Class C	Not less than 5 mg/l during at least 16 hours of any 24 hour period and never less than 3.0.	6.0-8.5	None in concentrations that would impair usages assigned to this class.	Same as above.	Not to exceed an average of 1.0 mg/l as N during any month.
Class D	Not less than 2.0 mg/l	6.0-9.0	Same as above.	No standard.	No standard.
Class SA	Not less than 6.5 mg/l	6.8-8.5	Not to exceed a median value of 70 and not more than 10% of samples over 230.	Not to exceed an average of 0.07 mg/l as P during any month.	Not to exceed an average of 0.2 mg/l as N during any month.
Class SB	Not less than 5.0 mg/l	6.8-8.5	Not to exceed a median value of 700 and not more than 10% of samples over 2300.	Same as above.	Same as above.
Class SC	Not less than 5 mg/l during at least 16 hours of any 24 hour period and never less than 3 mg/l.	6.5-8.5	None in concentrations that would impair any uses assigned to this class.	Same as above.	Not to exceed an average of 1.0 mg/l during any month.

TABLE B: SELECTED ANALYSES OF UNPOLLUTED WATERS

	Lake Quinsigamond Worcester	Swift River below Quabbin Reservoir	Parker River Byfield	Charles River Hopkinton
Dissolved Oxygen, mg/l	11.2-12.5	7.4-9.2	7.2-8.4	6.2-7.4
5-Day B.O.D., mg/l	0.8	1.8	1.9	0.7
Suspended Solids, mg/l	1.5	5	4	0
pH	6.7	6.5	7.6	6.4
Alkalinity, mg/l	18	8	37	8
Total Coliform per 100ml.	28	1300	300	170
Fecal Coliform per 100ml.	5	14	-	-
Color, Std. Units.	28	25	68	35
Turbidity, Std. Units	2	4	2	2
Ammonia as N., mg/l	.04	0.10	0.04	0.0
Nitrite as N, mg/l	-	0.000	0.006	0.0
Nitrate as N, mg/l	-	0.0	0.1	0.0
Total Phosphate as P, mg/l	.04	.07	0.16	0.03

BASIN DESCRIPTION

Probably no other city in Massachusetts is more involved with the ocean than New Bedford. Long the center of the whaling trade, the City's leading industry remains fishing. New Bedford Harbor is busy with fishing and lobster boats as well as pleasure craft. Fish processing plants line the shores. Many reminders of the City's seafaring heritage can be seen at the Whaling Museum and other sites along the Moby Dick Trail.

New Bedford Harbor is essentially the tidal portion of the Acushnet River. The river begins in northern Acushnet at the outfall from New Bedford Reservoir and flows generally south to the New Bedford City Line. Here, at the dam by Acushnet Sawmill, the river turns tidal and continues south to Popes Island and the Route 6 bridge. This point is considered to be the boundary between the river and the harbor. Approximately 1.3 miles below the bridge, the entrance to the harbor is marked by a stone dyke. Erected by the Corps of Engineers, this 4000 foot wide barrier provides flood protection for the City. During severe storms, the 157 foot wide navigational opening can be closed. This barrier has been a center of controversy as local residents feel that it inhibits the tidal action in the harbor, thereby preventing pollutants from being dispersed into the open sea.

Beyond the barrier, the harbor broadens out into Buzzards Bay. To the east is Sconticut Neck in Fairhaven. To the west is a penninsula which is part of New Bedford. Fort Rodman lies at the tip of this penninsula. This will be the site of the New Bedford Sewage Treatment Plant. At the present time, sewage is pumped from here to a point half a mile off shore. On the west side of the penninsula is Clark Cove. A pumping station on shore frequently discharges sewage into this cove.

A dairy farm and several industries discharge wastes to the Acushnet River in Acushnet. Several industries in New Bedford and Fairhaven discharge wastes to the river and the harbor without treatment. These include several fish processing plants as well as textile mills. New Bedford also has the combined sewer problems which are so common in the older cities in the State. These sewers carry both wastes from homes and industries and storm water. During storms, they overflow to the river and the harbor. Fairhaven, meanwhile, has recently completed an extended aeration sewage treatment plant. The effluent from this plant is discharged inside the barrier.

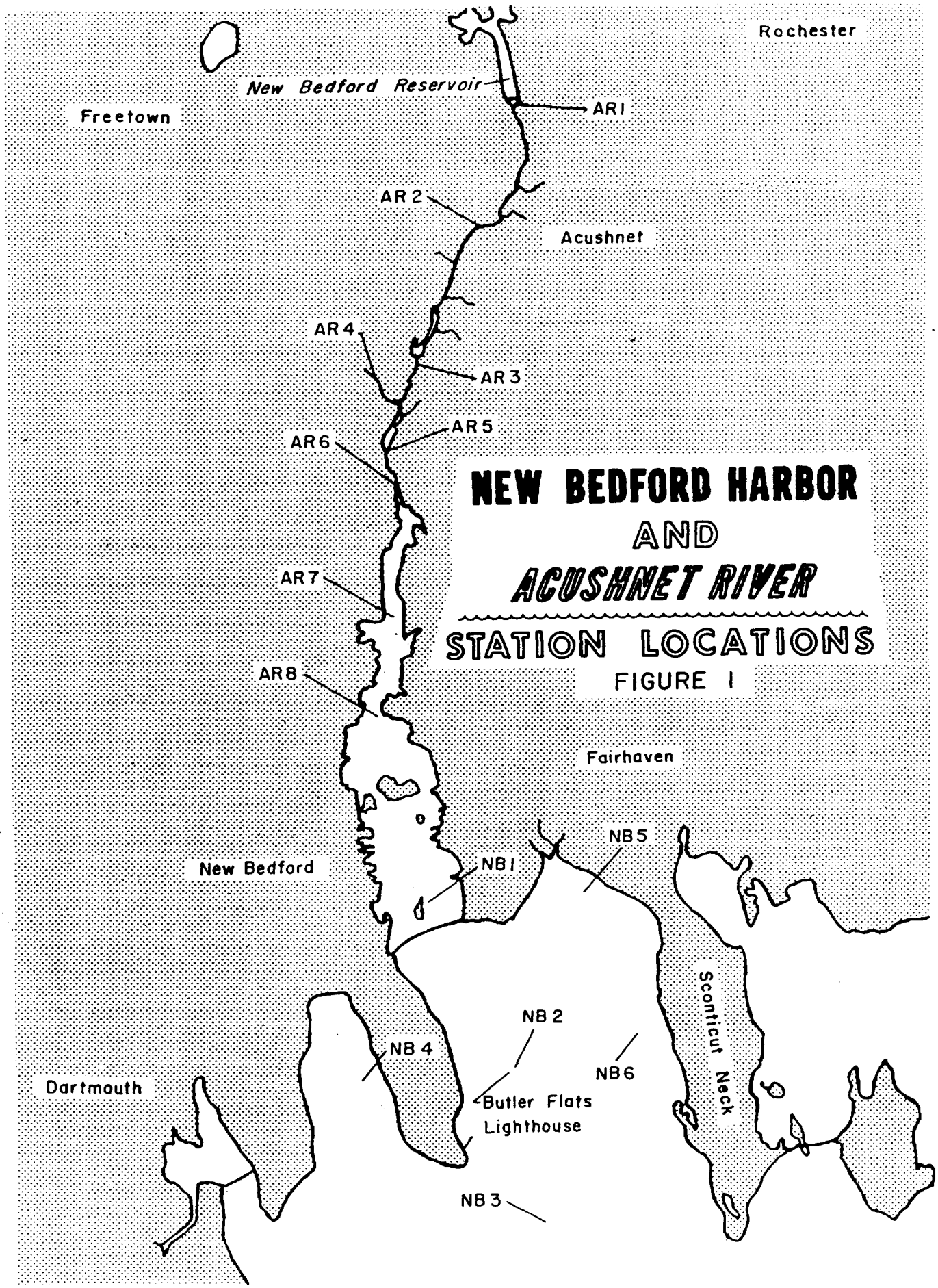
TABLE 1

DEFINITION OF SAMPLING STATIONS

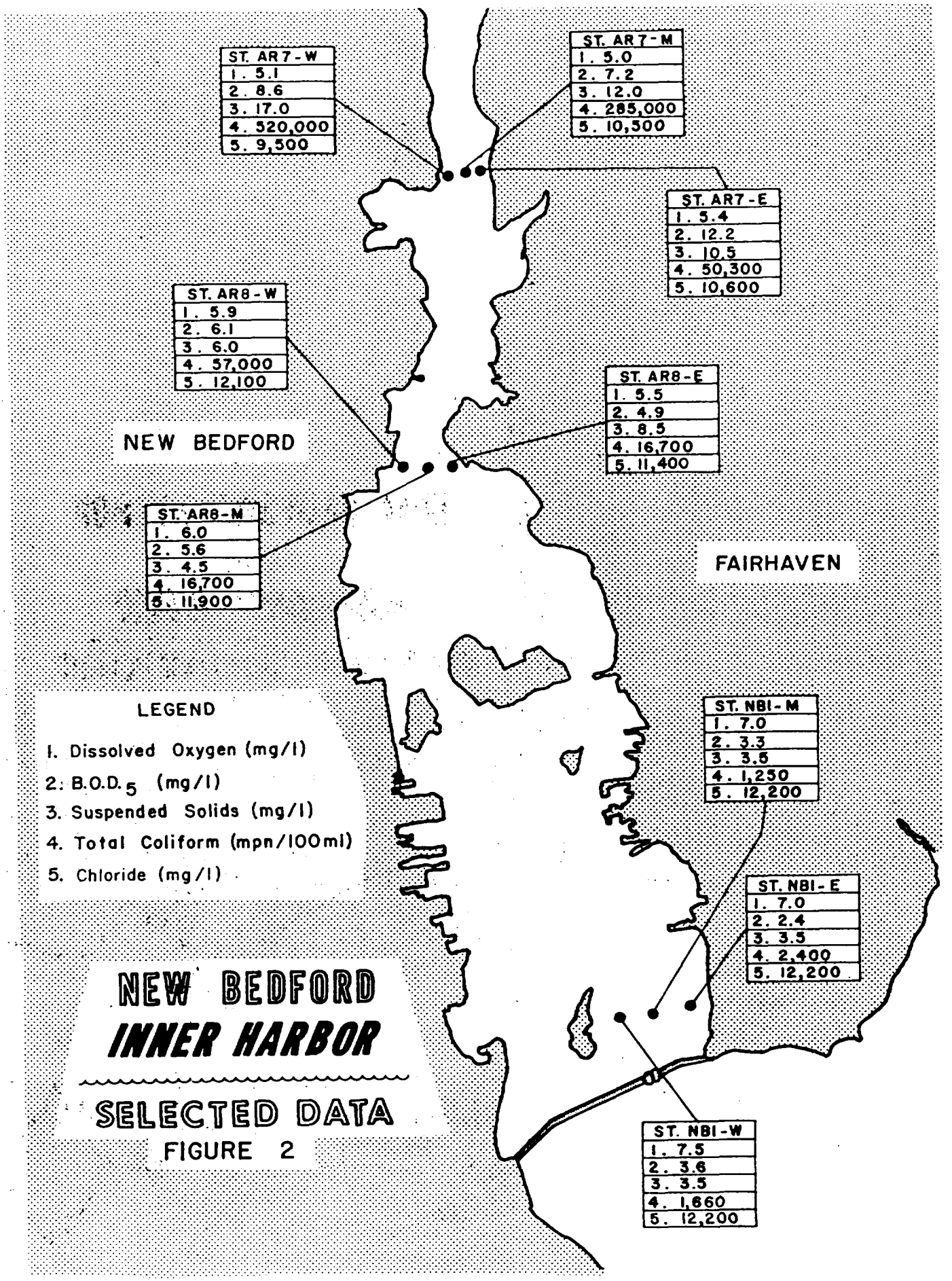
ACUSHNET RIVER - NEW BEDFORD HARBOR

STATION NUMBER	LOCATION	RIVER MILE
AR1	New Bedford Reservoir at dam, Achushnet	8.2
AR2	Achusnet River at dirt road bridge off Middle Road, Acushnet	6.8
*AR3	Acushnet River at Hamlin Road, Acushnet	5.5
AR4	Achusnet River Tributary at Middle Road, Coury Heights, Acushnet	5.0 + .4
AR5	Acushnet River at Pond Outlet above Acushnet Sawmill, Acushnet	4.6
AR6	Acushnet River at Main Street bridge, Acushnet-New Bedford City Line	4.5
AR7	Acushnet River, opposite Coffin Avenue, Fairhaven-Acushnet-New Bedford City Line	3.1
AR8	Acushnet River, opposite radio station WBSM Tower, Fairhaven-New Bedford City Line	2.1
NB1	New Bedford Harbor - Inside Hurricane Barrier, Fairhaven-New Bedford City Line	.2
NB2	New Bedford Harbor at Butler Flats Lightship, New Bedford	
NB3	New Bedford Harbor at New Bedford Sewer Outfall, New Bedford	
NB4	Clark Cove opposite sewage pumping station, New Bedford	
NB5	Nasketucket Bay opposite Pope Beach, Fairhaven	
NB6	New Bedford Harbor off Silver Shell Beach, Fairhaven	

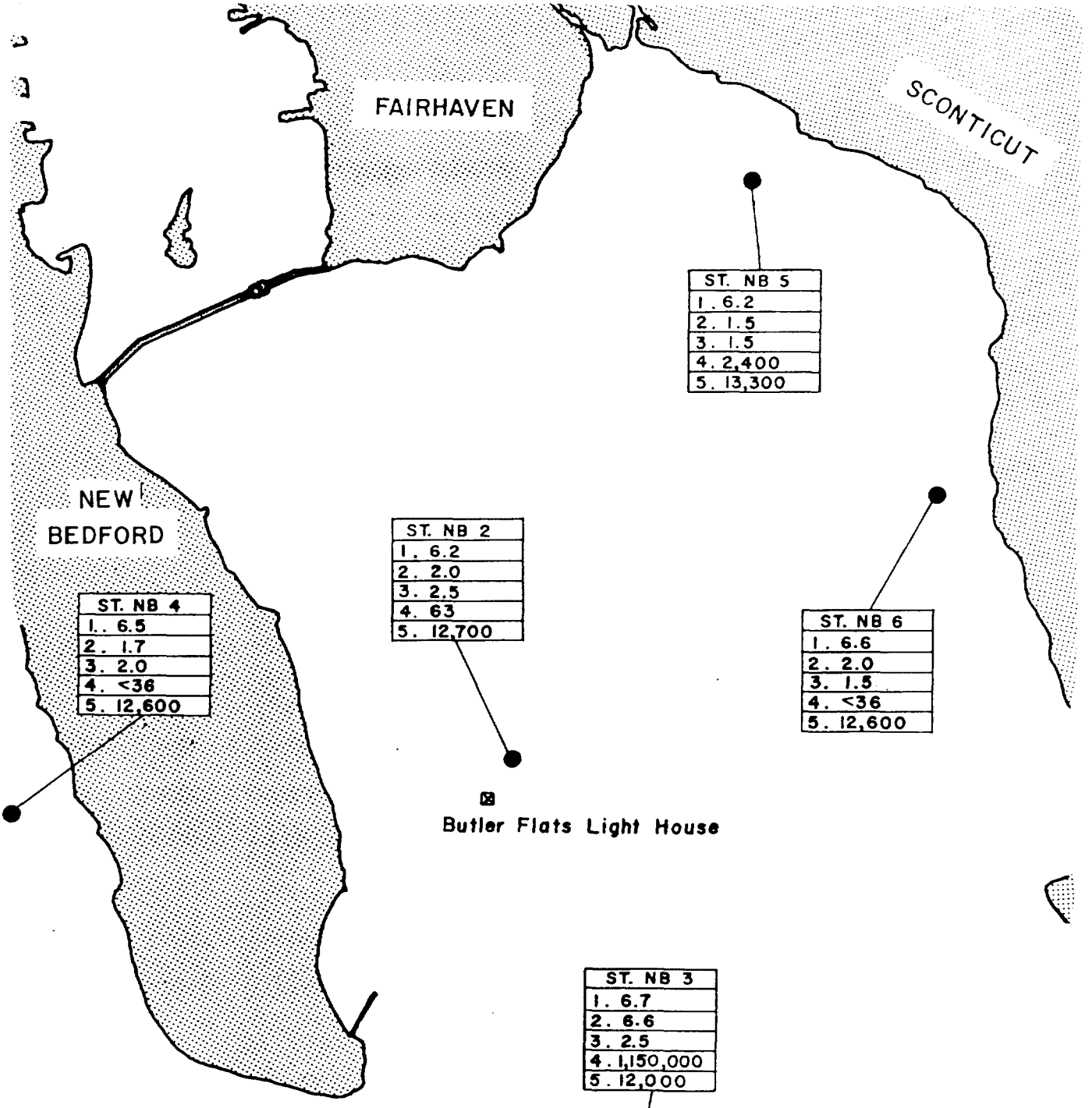
* East and West Channels were both sampled



NEW BEDFORD HARBOR
AND
ACUSHNET RIVER
STATION LOCATIONS
FIGURE 1



NEW BEDFORD
INNER HARBOR
SELECTED DATA
FIGURE 2



NEW BEDFORD
OUTER HARBOR

SELECTED DATA

FIGURE 3

LEGEND

1. Dissolved Oxygen (mg/l)
2. $B.O.D._5$ (mg/l)
3. Suspended Solids (mg/l)
4. Total Coliform (mpn/100ml)
5. Chloride (mg/l)

TABLE 2

TIME, TEMPERATURE, DISSOLVED OXYGEN

ACUSHNET RIVER 1971

	JULY 13				JULY 15			
STATION	RUN 1	RUN 2	RUN 3	RUN 4	RUN 5	RUN 6	RUN 7	RUN 8
AR1	*03:55	10:05	20:35	-	04:00	10:00	19:45	23:00
	** 66	72	75	-	68	72	71	69
	*** -	7.2	7.5		7.5	7.3	7.4	7.2
AR2	04:05	10:15	20:45	-	04:10	10:10	20:00	23:05
	64	64	71	-	-	68	68	-
	2.8	3.1	4.3		3.2	4.1	5.0	4.3
AR3-East	04:20	10:30	21:00	-	04:15	10:25	-	-
	65	70	73	-	66	70	-	-
	2.1	5.5	5.9	-	2.7	3.9	-	-
AR3-West	04:20	10:30	21:00	-	04:20	10:25	20:15	23:10
	65	70	73	-	66	70	71	68
	0.7	3.6	6.5	-	0.3	4.5	6.3	5.0
AR4	04:30	10:45	21:10	-	04:25	10:30	20:30	23:20
	60	68	70	-	62	62	66	64
	2.4	2.1	1.2	-	0.4	0.5	0.0	0.0
AR5	04:40	10:50	21:25	-	04:35	10:35	20:40	23:25
	66	68	72	-	66	68	-	68
	1.4	1.9	1.6	-	1.2	1.2	3.3	2.5
AR6	04:45	10:55	21:30	-	04:45	10:40	20:55	23:30
	64	70	68	-	65	67	68	68
	4.1	6.7	4.2	-	3.8	6.5	5.4	5.7

* Time
** Temperature, degrees Fahrenheit
*** Dissolved Oxygen, milligrams per liter

TABLE 3

TIME, TEMPERATURE, DISSOLVED OXYGEN

NEW BEDFORD OUTER HARBOR

	JULY 13				JULY 15			
STATION	RUN 1	RUN 2	RUN 3	RUN 4	RUN 5	RUN 6	RUN 7	RUN 8
NB2-Top	*05:35	11:30	16:30	23:05	8:00	11:30	16:40	-
	** -	67	72	70	65	67	67	-
	*** 1.4	7.2	7.4	6.6	6.8	6.9	7.4	-
NB2-Bottom	05:40	11:30	16:30	23:10	8:00	11:30	16:45	-
	64	67	70	71	66	67	66	-
	1.4	6.4	5.8	7.0	6.5	6 8	7.4	-
NB3-Top	05:50	11:40	16:40	0:00	08:10	11:45	17:00	-
	66	66	71	70	65	66	68	-
	5.9	6.9	7.1	7.3	6.7	6.5	6.3	-
NB3-Bottom	05:50	11:45	16:50	0:05	08:15	11:45	17:05	-
	64	65	70	68	65	66	66	-
	7.0	1.9	5.9	7.0	7.0	6.9	7.4	-
NB4-Top	06:10	11:55	17:05	-	08:20	12:00	17:25	-
	66	67	71	-	66	68	68	-
	7.2	7.4	4.0	-	6.2	6.7	7.6	-
NB4-Bottom	06:10	11:55	17:05	0:30	08:20	12:00	17:25	-
	64	67	70	70	66	68	66	-
	6.1	6.6	7.8	7.3	6.1	6.1	7.2	-
NB5-Top	06:45	12:10	17:30	-	08:35	12:15	17:55	-
	66	67	72	-	66	68	-	-
	6.8	7.4	7.2	-	7.1	7.3	7.6	-
NB5-Bottom	06:45	12:10	17:40	-	08:35	12:15	18:00	-
	66	66	71	-	66	66	68	-
	3.0	7.8	8.5	-	6.9	7.3	7.6	-
NB6-Top	06:30	12:20	17:50	-	08:45	12:25	18:05	-
	-	66	70	-	66	68	68	-
	2.4	7.2	8.2	-	7.0	7.1	8.0	-
NB6-Bottom	06:30	12:25	17:50	-	08:45	12:25	18:10	-
	65	67	70	-	66	69	68	-
	3.4	7.3	8.1	-	7.1	7.2	8.0	-

*Time
** Temperature, degrees fahrenheit
*** Dissolved oxygen, milligrams per liter

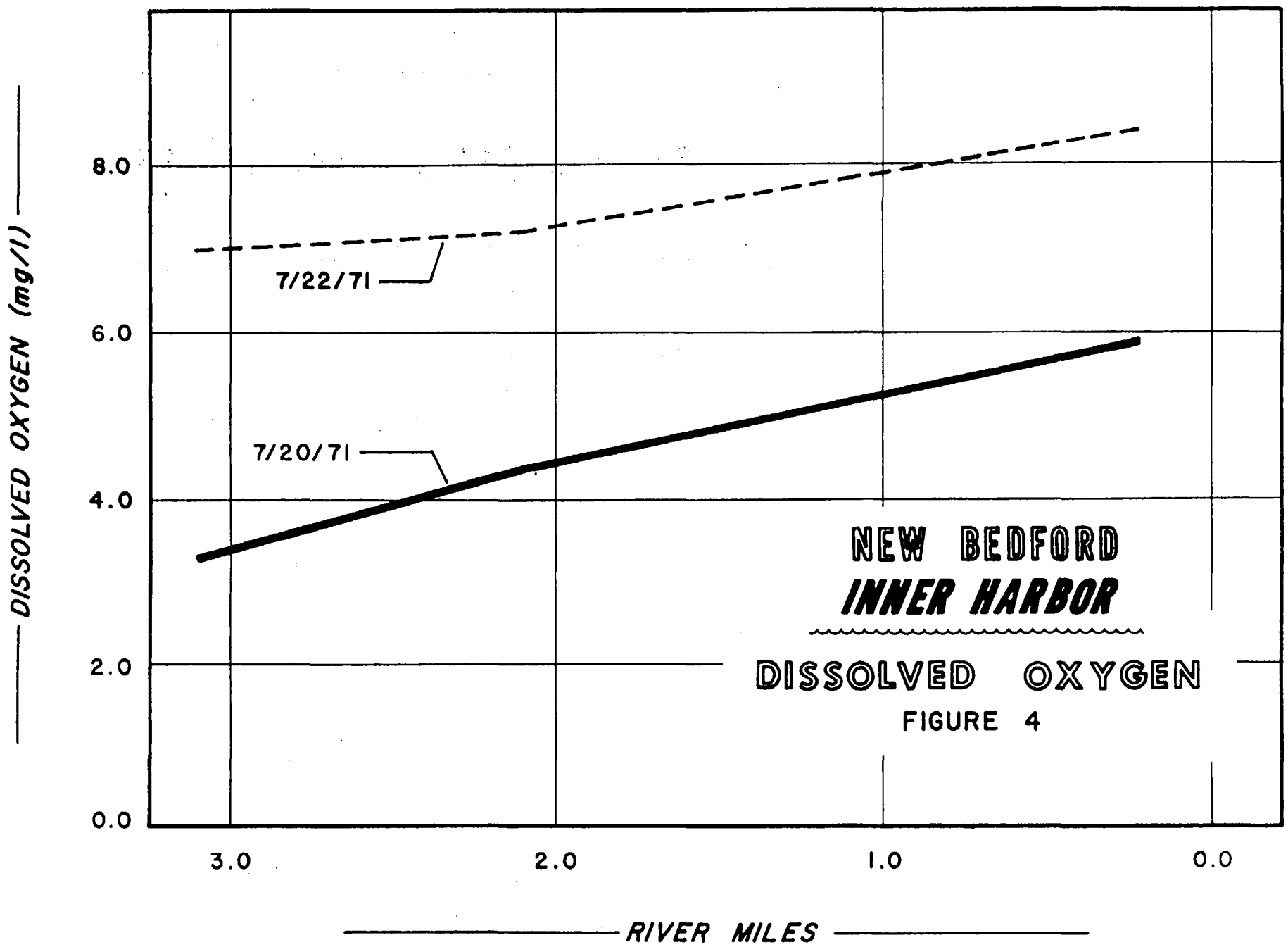

DISSOLVED OXYGEN (mg/l)
8.0
6.0
4.0
2.0
0.0
7/22/71
7/20/71
3.0
2.0
1.0
0.0
RIVER MILES
NEW BEDFORD
INNER HARBOR
DISSOLVED OXYGEN
FIGURE 4

TABLE 4

TIME, TEMPERATURE, DISSOLVED OXYGEN

NEW BEDFORD INNER HARBOR

	JULY 20				JULY 22			
STATION	RUN	RUN 2	RUN 3	RUN 4	RUN 5	RUN 6	RUN 7	RUN 8
AR7-East Top	*05:20	10:55	17:00	22:35	05:00	10:30	18:15	23:30
	** 68	70	74	72	68	72	75	75
	*** 2.2	3.6	6.1	3.4	7.1	6.9	7.6	8.1
AR7-West Bottom	05:25	10:55	17:00	22:35	05:05	10:30	18:15	23:30
	69	70	74	72	70	69	75	72
	1.1	3.6	3.9	3.8	3.3	3.8	7.3	5.8
AR7-Middle Top	05:15	10:45	17:05	22:35	04:55	10:25	18:05	0:00
	70	70	73	-	69	73	75	75
	2.3	2.3	4.9	3.3	6.6	4.3	8.8	7.7
AR7-Middle Bottom	05:20	10:55	17:05	22:35	04:55	10:30	18:05	0:05
	69	70	73	72	70	72	75	75
	2.0	2.2	4.6	2.2	3.2	2.7	9.3	6.1
AR7-West Top	05:10	10:40	17:15	22:30	04:50	10:25	-	23:45
	68	70	74	72	69	72	-	74
	3.8	2.9	4.3	2.8	6.8	4.4	9.4	6.2
AR7-West Bottom	05:15	10:45	17:15	22:30	04:30	10:25	18:00	23:45
	69	70	74	72	72	72	75	72
	1.7	1.1	3.8	2.3	2.7	4.3	9.4	5.5
AR8-East Top	05:00	10:35	16:45	22:20	04:45	10:15	17:55	23:10
	69	70	74	72	70	70	75	72
	2.9	2.4	5.9	4.0	6.7	4.5	10.9	6.7
AR8-East Bottom	05:05	10:35	16:45	22:20	04:45	10:20	17:55	23:10
	70	69	73	72	71	71	75	72
	2.1	2.7	5.8	2.9	4.2	4.0	10.7	6.7
AR8-Middle Top	04:55	10:30	16:50	22:15	04:40	10:15	17:50	23:40
	68	70	74	72	70	70	75	72
	7.1	2.6	6.5	3.9	4.8	5.3	10.8	6.6
AR8-Middle Bottom	05:00	10:30	10:50	22:15	04:40	10:15	17:50	23:40
	70	70	73	72	70	70	75	74
	3.0	2.3	3.4	3.1	3.5	4.8	10.0	5.6
AR8-West Top	04:50	10:25	16:55	22:10	04:35	10:10	17:35	23:35
	70	69	74	72	69	71	75	72
	4.1	2.4	7.0	3.7	4.6	6.4	10.7	8.1
AR8-West Bottom	04:55	10:25	16:55	22:10	04:35	10:15	17:40	23:35
	70	69	74	72	70	72	75	74
	2.4	2.3	4.5	2.5	4.7	4.8	10.8	6.8
NB1-East Top	04:45	10:15	16:20	22:00	04:10	10:05	17:30	23:20
	68	69	74	72	70	72	74	72
	5.2	5.8	6.8	5.1	7.3	7.7	10.5	7.2
NB1-East Bottom	04:45	10:15	16:15	22:00	04:15	10:05	17:30	23:20
	69	69	74	72	70	72	73	72
	5.1	5.8	5.2	5.8	5.3	7.4	7.4	7.4
NB1 Middle Top	04:30	10:10	16:30	22:00	04:05	10:00	17:20	23:15
	68	70	73	72	70	72	73	72
	5.4	5.1	7.2	5.7	6.8	7.6	9.5	9.0
NB1 Middle Bottom	04:35	10:10	16:30	22:00	04:05	10:00	17:20	23:15
	69	69	73	72	70	70	73	72
	5.4	5.0	7.1	5.7	6.8	6.0	8.3	9.0
NB1 West Top	04:25	10:10	16:30	21:50	03:55	09:55	17:15	23:25
	68	69	72	72	70	71	76	72
	5.7	5.4	7.8	5.6	6.9	8.4	11.0	9.2
NB1-West Bottom	04:25	10:05	16:30	21:50	04:00	09:55	17:15	23:25
	69	68	72	72	70	70	73	71
	4.0	5.0	4.6	5.0	6.6	5.2	6.4	5.5

* Time
** Temperature, degrees fahrenheit
*** Dissolved oxygen, milligrams per liter

TABLE 5

TEMPERATURE, DISSOLVED OXYGEN SUMMARY

ACUSHNET RIVER, NEW BEDFORD OUTER HARBOR

STATION	WATER TEMPERATURE, °F MAX	MIN	AVG	DISSOLVED OXYGEN, mg/l MAX	MIN	AVG
AR1	75	66	70	7.5	7.2	7.4
AR2	71	64	67	5.0	2.8	3.8
AR3-East	73	65	69	5.9	2.1	4.0
AR3-West	73	65	69	6.5	0.3	3.8
AR4	70	60	65	2.4	0.0	0.9
AR-5	72	66	68	3.3	1.2	1.9
AR6	70	64	67	6.7	3.8	6.0
NB2 Top	72	65	68	7.4	1.4	6.2
NB2 Bottom	71	64	67	7.4	1.4	5.9
NB3 Top	71	65	67	7.3	5.9	6.7
NB3 Bottom	70	64	66	7.4	1.9	6.2
NB4 Top	71	66	68	7.6	4.0	6.5
NB4 Bottom	70	64	67	7.8	6.1	6.7
NB5 Top	72	66	68	7.6	6.8	6.2
NB5 Bottom	71	66	67	8.5	3.0	7.0
NB6 Top	70	66	68	8.2	2.4	6.6
NB6 Bottom	70	65	68	8.1	3.4	6.8

TABLE 6

TEMPERATURE, DISSOLVED OXYGEN SUMMARY

NEW BEDFORD INNER HARBOR

	EAST			MIDDLE			WEST		
STATION	MAX	MIN	AVG	MAX	MIN	AVG	MAX	MIN	AVG
AR7 Top	* 75	68	72	75	69	72	74	66	71
	** 8.1	2.2	5.4	8.8	2.3	5.0	9.4	2.8	5.1
Bottom	75	69	71	75	69	72	75	69	72
	7.3	1.1	3.6	9.3	2.0	4.0	9.4	1.1	3.8
AR8 Top	75	69	72	75	68	73	75	69	72
	10.9	2.4	5.5	10.8	2.6	6.0	10.7	2.4	5.9
Bottom	75	69	72	75	70	72	75	69	72
	10.7	2.1	4.9	10.0	2.3	4.5	10.8	2.3	4.8
NB1 Top	74	68	71	73	68	71	76	68	71
	10.5	5.1	7.0	9.5	5.1	7.0	11.0	5.4	7.5
Bottom	74	69	71	73	69	71	73	68	71
	7.4	5.1	6.2	9.0	5.0	6.7	6.6	4.0	5.3

* Temperature, degrees fahrenheit
** Dissolved Oxygen, milligrams per liter

TABLE 7

5 DAY BIOCHEMICAL OXYGEN DEMAND

ACUSHNET RIVER, NEW BEDFORD OUTER HARBOR

STATION	JULY 13	JULY 15	AVERAGE
AR1	2.0	1.4	1.7
AR2	1.6	1.0	1.3
AR3	2.8	2.8	2.8
AR4	16.0	152.0	84.0
AR5	2.2	3.6	2.9
AR6	2.8	4.6	3.7
NB2 Top	2.4	1.6	2.0
NB2 Bottom	1.6	3.4	2.5
NB3 Top	6.0	7.2	6.6
NB3 Bottom	2.6	0.6	1.6
NB4 Top	1.8	1.6	1.7
NB4 Bottom	1.4	1.2	1.3
NB5 Top	1.4	1.6	1.5
NB5 Bottom	1.6	2.0	1.8
NB6 Top	2.0	2.0	2.0
NB6 Bottom	2.0	1.8	1.9

RESULTS IN MILLIGRAMS PER LITER

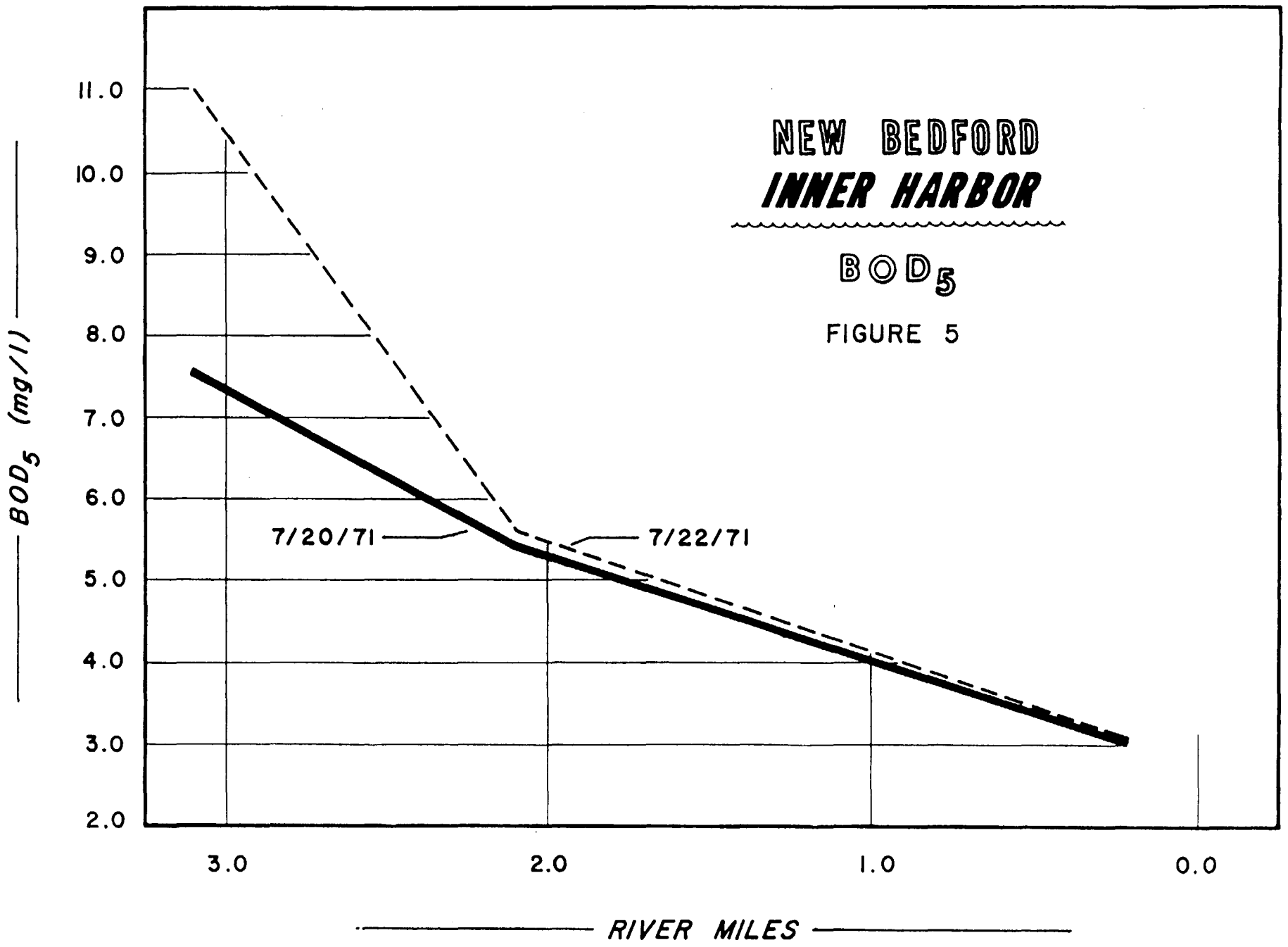

NEW BEDFORD
INNER HARBOR
BOD5
FIGURE 5
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
BOD5 (mg/l)
7/20/71
7/22/71
3.0
2.0
1.0
0.0
RIVER MILES

TABLE 8

5 DAY BIOCHEMICAL OXYGEN DEMAND

NEW BEDFORD INNER HARBOR

		EAST	MIDDLE	WEST
AR7 Top	I	8.4	5.6	8.8
	II	16.0	8.7	8.4
	AVG.	12.2	7.2	8.6
AR7 Bottom	I	7.2	6.4	5.6
	II	7.8	7.2	7.2
	AVG.	7.5	6.8	6.4
AR8 Top	I	4.0	6.0	6.4
	II	5.8	5.2	5.8
	AVG.	4.9	5.6	6.1
AR8 Bottom	I	5.6	3.2	4.8
	II	6.2	2.6	4.0
	AVG.	5.9	2.9	4.4
NB1 Top	I	3.2	3.0	3.0
	II	1.6	3.6	4.2
	AVG.	2.4	3.3	3.6
NB1 Bottom	I	2.6	3.2	2.4
	II	1.0	1.0	2.5
	AVG.	1.8	2.1	2.4

Results in milligrams per liter

DATES: I - July 20 II - July 22

TABLE 9

LONG TERM B.O.D. STUDY

ACUSHNET RIVER - NEW BEDFORD HARBOR

STATION	2 DAY	5 DAY	7 DAY	14 DAY	21 DAY
AR1	0.6	2.0	2.4	2.8	4.2
AR2	0.4	1.6	2.0	3.2	3.8
AR3	0.6	2.8	3.2	6.8	8.4
AR4	14.0	16.0	26.0	108.0	126.0
AR5	1.2	2.2	2.8	3.8	4.8
AR6	1.8	2.8	3.4	5.4	6.0
AR7 ET	3.6	8.8	9.6	10.0	11.0
EB	1.6	5.6	6.4	8.4	9.6
MT	2.8	5.6	6.0	7.6	9.2
MB	3.2	6.4	6.6	6.8	8.8
WT	5.2	8.4	9.2	10.0	13.0
WB	3.6	7.2	7.2	8.8	10.0
AR8 ET	2.4	6.4	8.0	10.0	11.0
EB	2.4	4.8	6.4	6.8	8.0
MT	2.8	6.0	6.4	8.8	9.2
MB	1.6	3.2	4.0	5.2	6.4
WT	1.2	4.0	5.6	6.0	6.0
WB	3.2	5.6	5.6	6.4	8.4
NB1 ET	1.6	3.0	3.4	3.8	5.2
EB	1.4	2.4	2.6	3.8	4.8
MT	1.6	3.0	3.0	3.4	4.4
MB	1.0	3.2	3.2	3.8	4.0
WT	2.0	3.2	3.2	3.4	4.6
WB	1.2	2.6	3.8	4.2	5.0
NB2 T	0.7	1.2	1.2	1.7	1.8
B	0.8	1.6	1.8	2.0	2.2
NB3 T	2.4	6.0	7.0	7.8	8.4
B	0.4	1.3	1.6	2.4	2.6
NB4 T	0.8	1.8	2.2	2.8	3.0
B	0.4	1.4	1.4	2.6	2.8
NB5 T	0.6	1.4	2.0	2.4	2.8
B	0.6	1.6	2.0	2.6	2.8
NB6 T	0.8	2.0	2.4	3.0	3.0
B	0.6	2.0	2.0	2.8	3.0

Results in milligrams per liter

TABLE 10

SUSPENDED SOLIDS, pH, ALKALINITY

ACUSHNET RIVER - NEW BEDFORD OUTER HARBOR

	JULY 13			JULY 15			AVERAGE	
STATION	SUSP. SOL. mg/l	pH	ALK. mg/l	SUSP. SOL. mg/l	pH	ALK. mg/l	SUSP. SOL. mg/l	ALK. mg/l
AR1	2.0	6.9	12	6.0	6.9	8	4.0	10
AR2	3.0	6.5	15	1.0	6.5	11	2.0	13
AR3	6.0	6.6	19	19.0	6.5	12	12.5	16
AR4	26.0	7.8	255	33.0	7.2	160	29.5	208
AR5	2.0	6.5	21	2.0	6.5	20	2.0	20
AR6	1.0	6.7	25	6.0	6.7	14	3.5	20
NB2 T	4.0	8.1	116	1.0	8.2	110	2.5	113
NB2 B	10.0	8.1	114	1.0	8.2	115	5.5	114
NB3 T	2.0	7.9	115	3.0	7.8	105	2.5	110
NB3 B	49.0	8.0	115	1.0	8.3	110	25.0	112
NB4 T	1.0	8.2	110	3.0	8.1	110	2.0	110
NB4 B	2.0	8.1	115	7.0	8.2	110	4.5	112
NB5 T	2.0	8.1	100	1.0	8.2	105	1.5	102
NB5 B	8.0	8.1	110	2.0	8.1	110	5.0	110
NB6 T	1.0	8.1	90	2.0	8.2	105	1.5	98
NB6 B	16.0	8.2	90	6.0	8.2	115	11.0	102

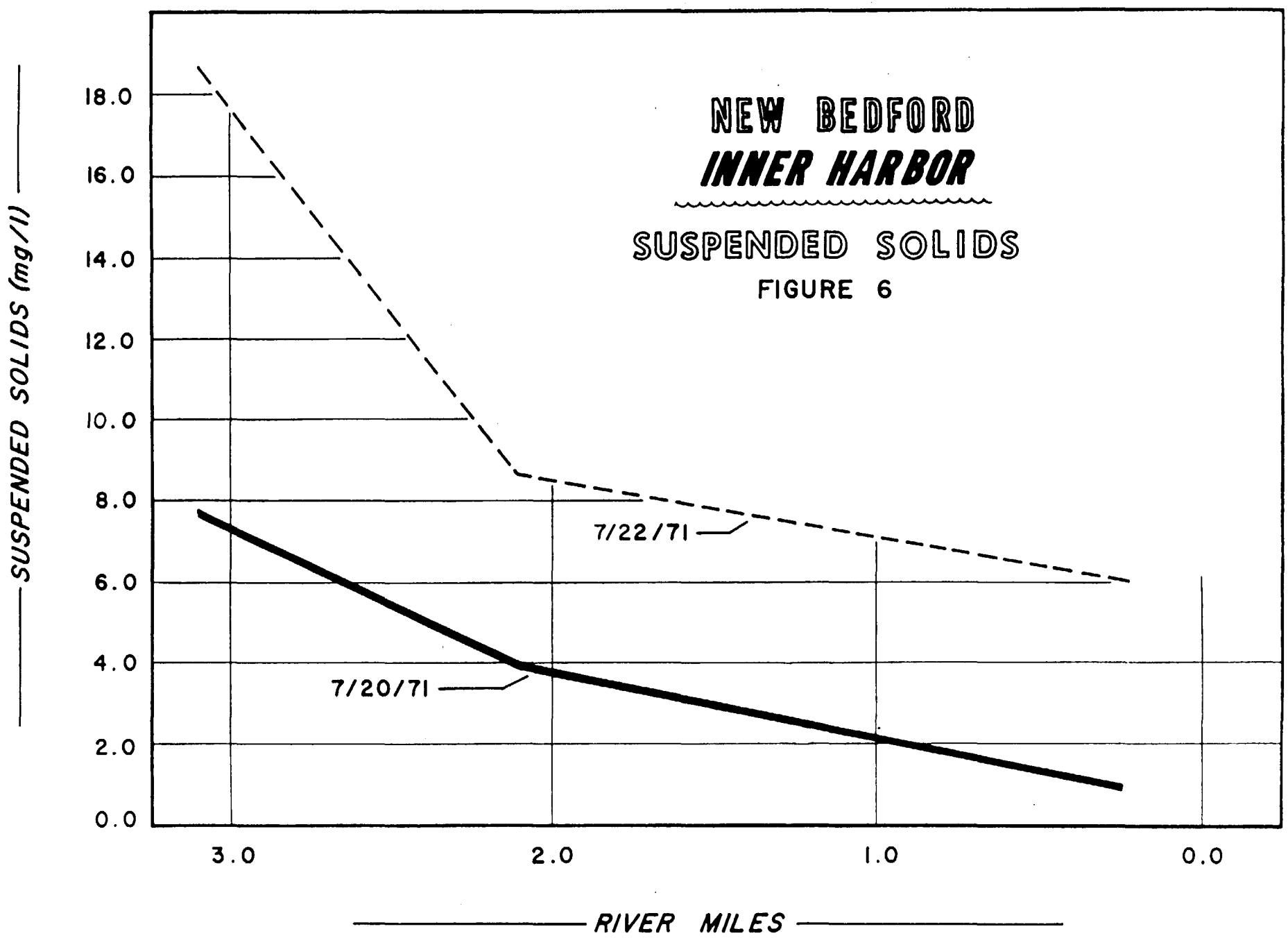

NEW BEDFORD
INNER HARBOR
SUSPENDED SOLIDS
FIGURE 6
SUSPENDED SOLIDS (mg/l)
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
7/22/71
7/20/71
3.0
2.0
1.0
0.0
RIVER MILES

TABLE 11

SUSPENDED SOLIDS, pH, ALKALINITY

NEW BEDFORD INNER HARBOR

STATION	JULY 20 SUSP. SOL. mg/l	JULY 20 pH	JULY 20 ALK. mg/l	JULY 22 SUSP. SOL. mg/l	JULY 22 pH	JULY 22 ALK. mg/l	AVERAGE SUSP. SOL. mg/l	AVERAGE ALK. mg/l
AR7 ET	6.0	6.5	85	15.0	7.8	89	10.5	87
EB	47.0	6.8	91	34.0	8.0	94	40.5	92
MT	7.0	7.2	107	17.0	7.9	79	12.0	93
MB	10.0	7.4	102	43.0	7.9	97	26.0	100
WT	10.0	7.0	95	24.0	7.1	81	17.0	88
WB	16.0	7.3	90	62.0	7.9	82	39.0	86
AR8 ET	8.0	7.8	101	9.0	7.7	95	8.5	98
EB	27.0	7.8	96	5.0	7.9	90	16.0	93
MT	3.0	7.1	101	6.0	8.0	91	4.5	96
MB	1.0	7.5	100	12.0	7.6	106	6.5	103
WT	1.0	7.7	101	11.0	7.9	95	6.0	98
WB	8.0	7.8	104	54.0	8.0	91	8.0	98
NB1 ET	1.0	7.3	102	6.0	8.0	94	3.5	98
EB	4.0	7.9	103	55.0	8.0	90	30.0	96
MT	1.0	8.0	102	6.0	8.2	95	3.5	98
MB	2.0	8.0	111	19.0	7.9	106	10.5	108
WT	1.0	7.3	105	6.0	7.5	109	3.5	107
WB	102.0	7.8	105	6.0	8.0	94	55.0	98

TABLE 12

COLIFORM BACTERIA, MPN PER 100 ML

ACUSHNET RIVER, NEW BEDFORD OUTER HARBOR

	JULY 13		JULY 15	
STATION	TOTAL	FECAL	TOTAL	FECAL
AR1	<36	<36	150	91
AR2	210	210	91	36
AR3 EAST	4600	430	9300	2400
AR3 WEST	240,000	9300	240,000	24,000
AR4	2,400,000	1,100,000	4,600,000	430,000
AR5	<36	<36	430	36
AR6	240,000	9300	75,000	24,000
NB2T	<36	<36	91	36
NB2B	<36	<36	73	36
NB3T	1,100,000	1,100,000	1,200,000	1,200,000
NB3B	1,100,000	1,100,000	2300	2300
NB4T	<36	<36	<36	<36
NB4B	<36	<36	<36	<36
NB5T	4600	4600	230	<36
NB5B	15,000	4300	<36	<36
NB6T	36	<36	<36	<36
NB6B	<36	<36	<36	<36

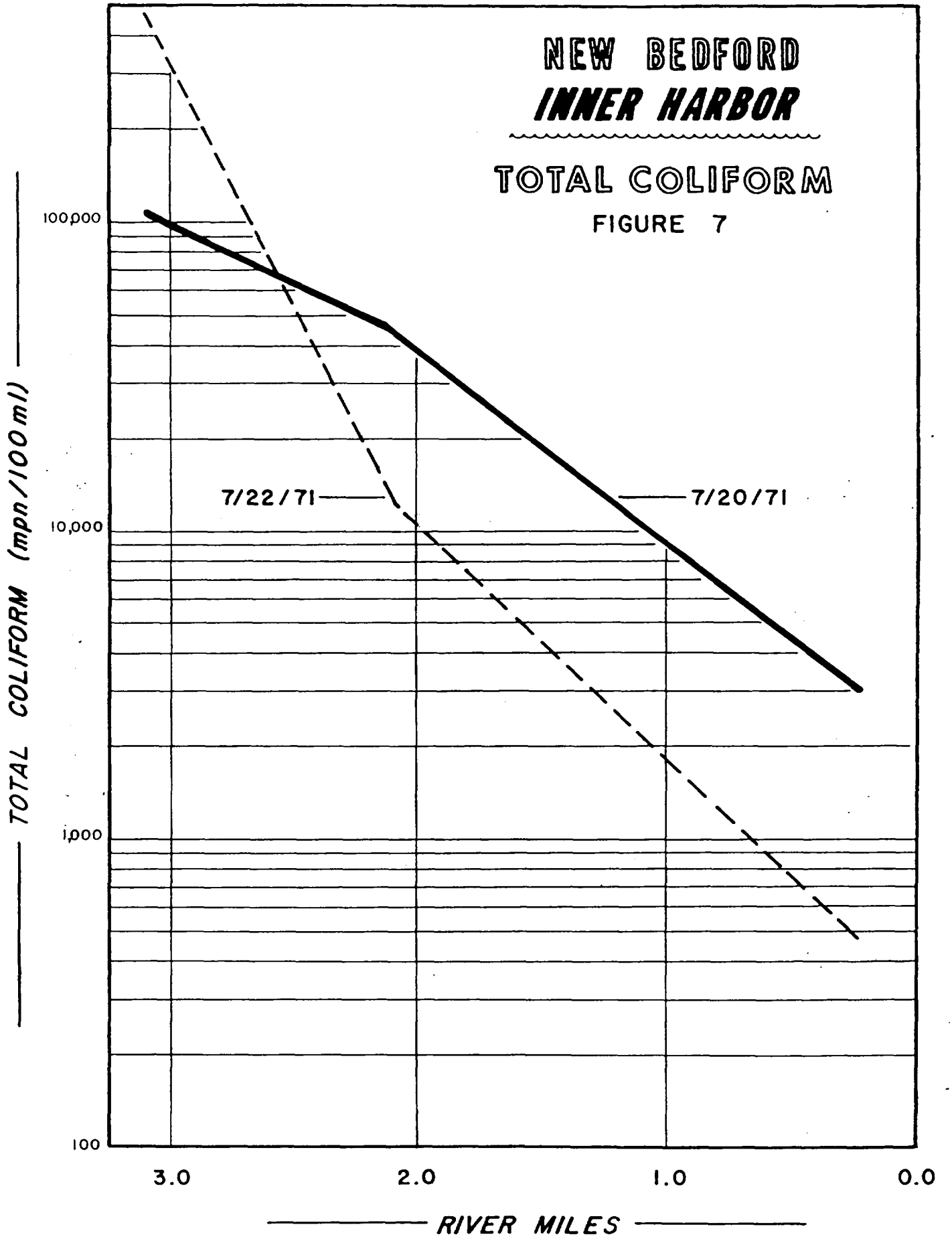

NEW BEDFORD
INNER HARBOR
TOTAL COLIFORM
FIGURE 7
100,000
10,000
1,000
100
TOTAL COLIFORM (mpn/100 ml)
7/22/71
7/20/71
3.0
2.0
1.0
0.0
RIVER MILES

TABLE 13

COLIFORM BACTERIA, MPN PER 100 ML

NEW BEDFORD INNER HARBOR

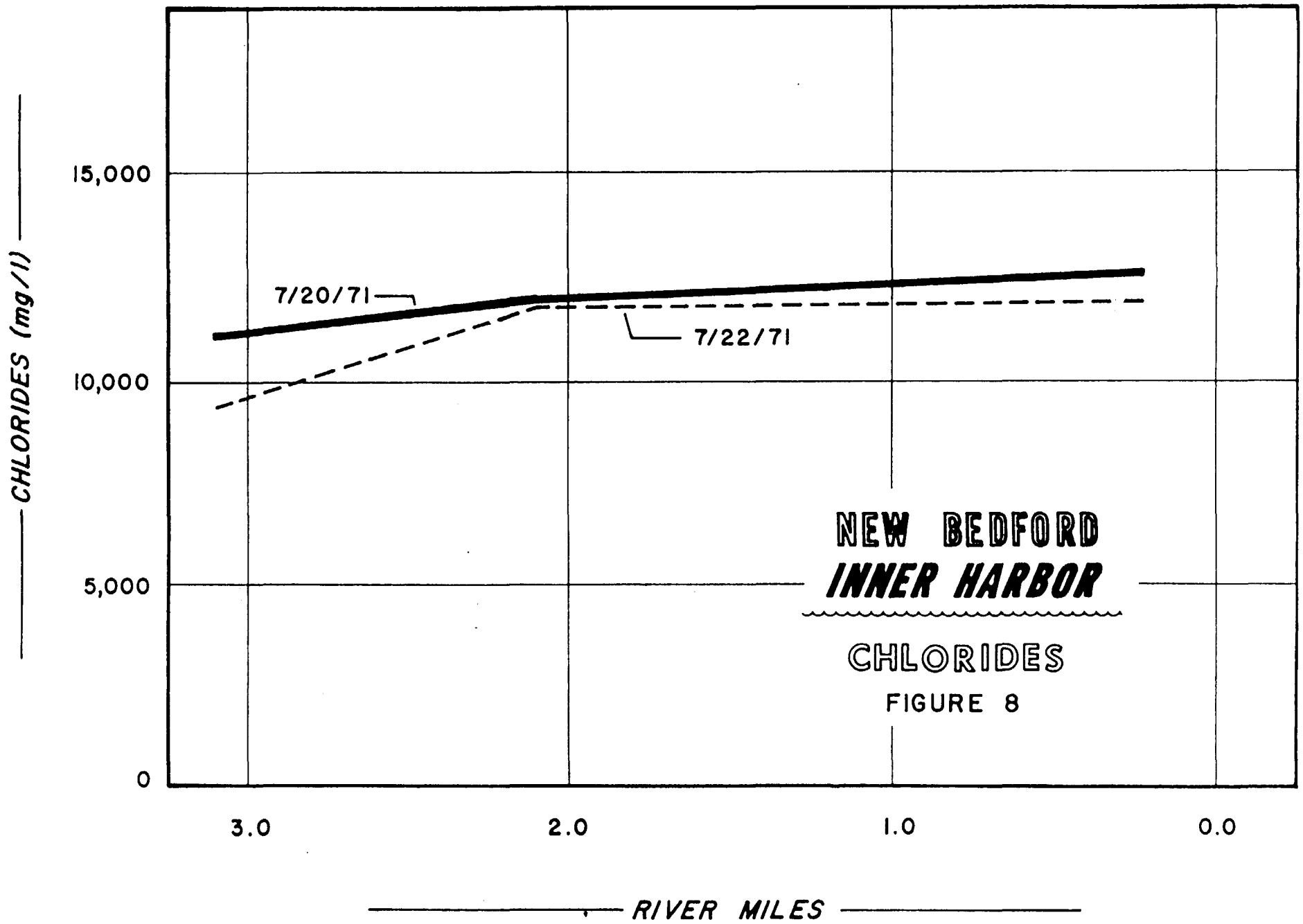
	JULY 20		JULY 22	
STATION	TOTAL	FECAL	TOTAL	FECAL
AR7 ET	93,000	4300	7500	4300
EB	24,000	4300	4300	2300
MT	110,000	15,000	460,000	240,000
MB	9300	2300	9300	2300
WT	110,000	46,000	930,000	430,000
WB	46,000	9300	4300	4300
AR8 ET	24,000	4300	9300	4300
EB	4300	1500	430	<36
MT	9300	2300	24,000	9300
MB	24,000	2300	230	36
WT	110,000	4300	4300	2300
WB	7500	4300	2300	930
NB1 ET	4300	1500	430	<36
EB	430	230	91	<36
MT	2400	230	91	<36
MB	230	91	4300	430
WT	2400	930	930	930
WB	930	230	430	<36

TABLE 14

TOTAL PHOSPHATE, CHLORIDES, CHLOROPHYLL A

ACUSHNET RIVER, NEW BEDFORD OUTER HARBOR

STATION	PO_4 mg/l	CHLORIDES mg/l	PO_4 mg/l	CHLORIDES mg/l	CHLOROPHYLL A mg/l
AR1	0.05	20	0.05	7	0.010
AR2	0.07	17	0.06	17	0.005
AR3	0.22	34	0.15	19	0.013
AR4	17.00	61	30.90	31	0.013
AR5	0.40	20	0.83	21	0.013
AR6	0.20	150	0.25	800	0.005
NB2 T	0.10	13,000	0.06	12,400	0.007
NB2 B	0.11	12,800	0.06	12,400	0.013
NB3 T	0.27	12,000	0.26	12,000	0.001
NB3 B	0.21	13,000	0.06	12,600	0.002
NB4 T	0.08	12,600	0.08	12,600	0.002
NB4 B	0.09	12,800	0.08	12,600	0.002
NB5 T	0.09	14,000	0.09	12,600	0.005
NB5 B	0.09	13,400	0.09	12,400	0.004
NB6 T	0.09	12,800	0.09	12,400	0.005
NB6 B	0.09	13,400	0.08	12,600	0.005



NEW BEDFORD
INNER HARBOR
CHLORIDES
FIGURE 8

TABLE 15

TOTAL PHOSPHATE, CHLORIDES

NEW BEDFORD INNER HARBOR

STATION	JULY 20 PO_4 mg/l	JULY 20 CHLORIDES mg/l	JULY 22 PO_4 mg/l	JULY 22 CHLORIDES mg/l
AR7 ET	0.27	11,000	0.23	10,200
EB	0.25	11,800	0.14	11,800
MT	0.25	11,200	0.20	9800
MB	0.24	11,200	0.10	10,600
WT	0.29	11,000	0.53	8000
WB	0.29	11,400	0.29	11,400
AR8 ET	0.23	11,200	0.24	11,600
EB	0.23	11,600	0.25	12,400
MT	0.15	11,800	0.22	12,000
MB	0.22	12,200	0.21	12,000
WT	0.22	12,600	0.17	11,600
WB	0.22	12,400	0.16	12,200
NB ET	0.15	12,400	0.13	12,000
EB	0.12	12,200	0.12	12,400
MT	0.11	12,400	0.14	12,000
MB	0.13	12,400	0.13	12,000
WT	0.13	12,600	0.15	11,800
WB	0.07	12,800	0.12	12,000

TABLE 16

COLOR, TURBIDITY, SPECIFIC CONDUCTANCE

ACUSHNET RIVER, NEW BEDFORD OUTER HARBOR

	JULY 13			JULY 15		
STATION	COLOR Std. Units	TURBIDITY Std. Units	SPEC. COND. Micromhos/cm	COLOR Std. Units	TURBIDITY Std. Units	SPEC. COND. Micromhos/cm
AR1	120	1	90	120	1	170
AR2	110	2	100	100	1	110
AR3	100	2	110	90	1	100
AR4	110	7	660	-	-	140
AR5	110	1	120	110	1	110
AR6	100	1	720	100	1	1450
NB2 T	13	1	44,000	5	1	47,000
NB2 B	15	3	43,000	10	1	48,000
NB3 T	15	2	43,000	20	1	45,000
NB3 B	15	6	40,000	5	1	44,000
NB4 T	10	0	45,000	10	1	47,000
NB4 B	5	0	40,000	15	1	47,000
NB5 T	15	1	42,000	15	1	46,000
NB5 B	10	1	2800	10	1	47,000
NB6 T	10	1	43,000	15	1	44,000
NB6 B	15	2	46,000	15	1	46,000

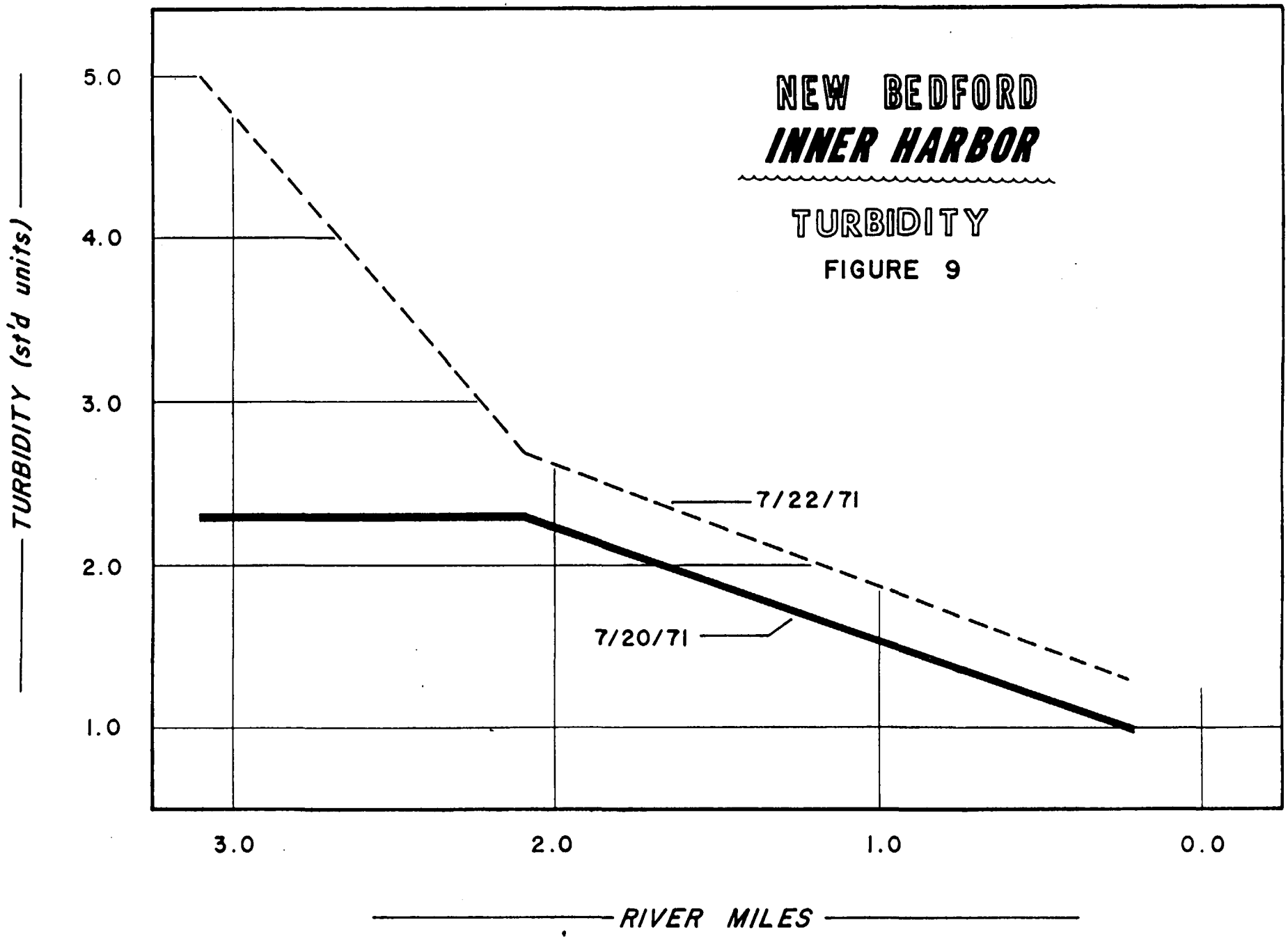

NEW BEDFORD
INNER HARBOR
TURBIDITY
FIGURE 9
5.0
4.0
3.0
2.0
1.0
TURBIDITY (st'd units)
7/22/71
7/20/71
3.0
2.0
1.0
0.0
RIVER MILES

TABLE 17

COLOR, TURBIDITY, SPECIFIC CONDUCTANCE

NEW BEDFORD INNER HARBOR

	JULY 20			JULY 15		
STATION	COLOR Std. Units	TURBIDITY Std. Units	SPEC. COND. Micromhos/cm	COLOR Std. Units	TURBIDITY Std. Units	SPEC. COND. Micromhos/cm
AR7 ET	45	2	**44,000**	50	2	37,000
EB	50	2	40,000	50	4	41,000
MT	40	2	44,000	50	3	33,000
MB	35	3	46,000	50	4	41,000
WT	30	3	37,000	45	10	29,000
WB	35	3	40,000	50	6	41,000
AR8 ET	70	3	40,000	30	3	41,000
EB	35	5	46,000	25	2	43,000
MT	40	2	46,000	35	2	37,000
MB	25	2	43,000	30	3	37,000
WT	35	2	40,000	40	3	40,000
WB	35	2	46,000	40	3	37,000
NB1 ET	15	1	42,000	20	1	40,000
EB	15	2	45,000	30	7	42,000
MT	12	1	46,000	15	2	42,000
MB	15	1	48,000	20	3	41,000
WT	12	1	43,000	20	1	39,000
WB	20	11	47,000	20	1	41,000

TABLE 18

NITROGEN CONCENTRATIONS

ACUSHNET RIVER, NEW BEDFORD OUTER HARBOR

	JULY 13			JULY 15	
STATION	Kj-N	NH_3-N	NO_3-N	NH_3-N	NO_3-N
AR1	1.2	0.01	0.0	0.00	0.2
AR2	0.7	0.07	0.2	0.05	0.0
AR3	1.4	0.20	0.0	0.10	0.0
AR4	25.7	22.00	0.0	12.00	0.0
AR5	1.3	0.10	0.0	0.24	0.1
AR6	1.2	0.10	0.1	0.07	0.1
NB2 T	0.1	0.20	0.0	0.30	0.0
NB2 B	0.4	0.30	0.0	0.25	0.0
NB3 T	0.1	0.38	0.0	0.50	0.0
NB3 B	0.1	0.30	0.0	0.20	0.0
NB4 T	0.1	0.14	0.0	0.20	0.0
NB4 B	0.1	0.20	0.0	0.15	0.0
NB5 T	0.1	0.10	0.0	0.20	0.0
NB5 B	0.1	0.12	0.0	0.18	0.0
NB6 T	0.1	0.07	0.0	0.10	0.0
NB6 B	0.1	0.10	0.0	0.10	0.0

Results in milligrams per liter

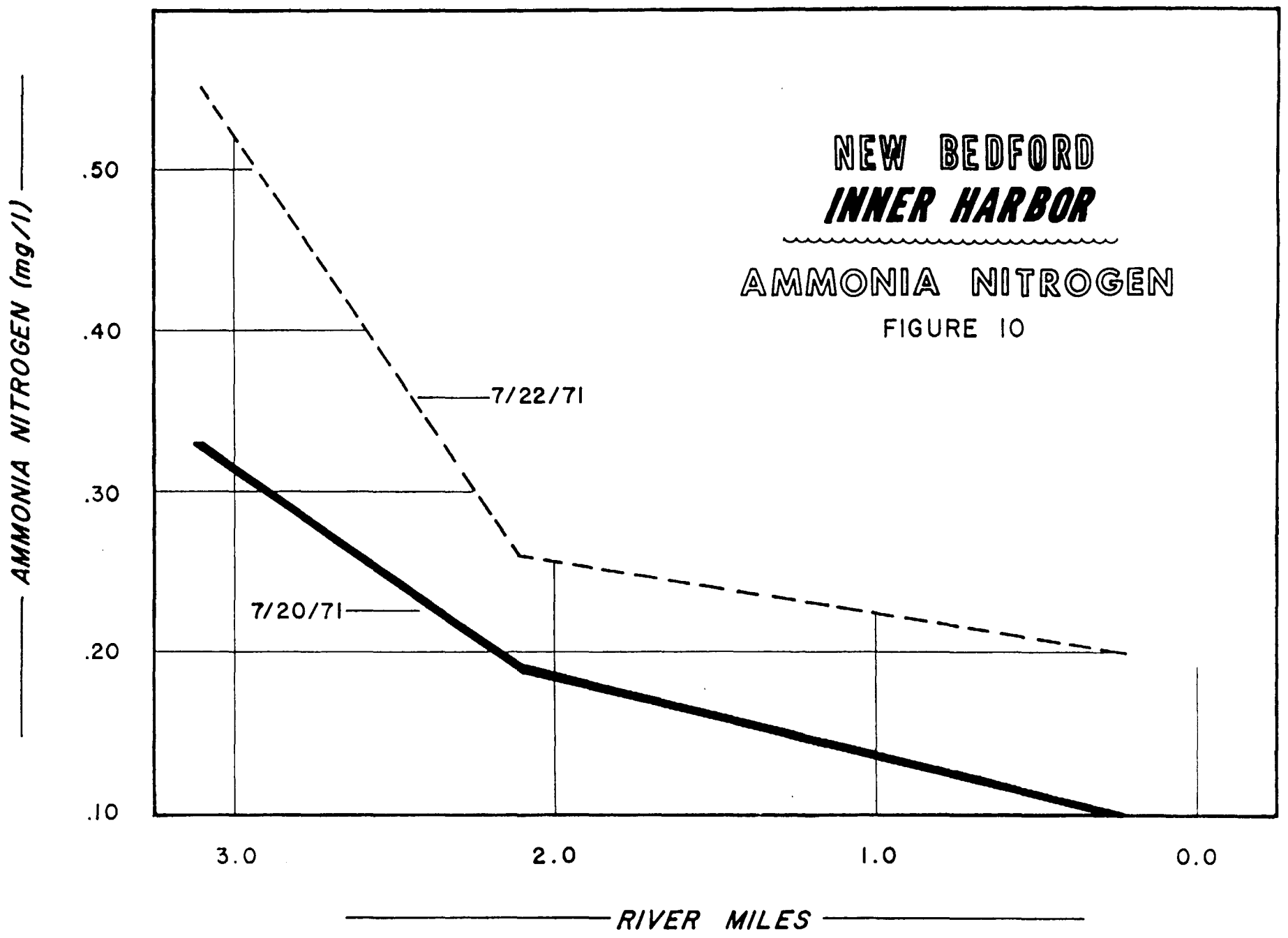

NEW BEDFORD
INNER HARBOR
AMMONIA NITROGEN
FIGURE 10
7/22/71
7/20/71
.50
.40
.30
.20
.10
AMMONIA NITROGEN (mg/l)
3.0
2.0
1.0
0.0
RIVER MILES

TABLE 19

NITROGEN CONCENTRATIONS

NEW BEDFORD INNER HARBOR

		JULY 20			JULY 22	
STATION		Kj-N	NH_3-N	NO_3-N	NH_3-N	NO_3-N
AR7	ET	2.5	0.30	0.0	0.35	0.0
	EB	1.9	0.40	0.0	0.30	0.0
	MT	1.8	0.30	0.0	0.50	0.0
	MB	0.9	0.30	0.0	0.30	0.0
	WT	3.1	0.40	0.0	0.80	0.0
	WB	0.7	0.30	0.0	0.30	0.0
AR8	ET	1.3	0.20	0.0	0.29	0.0
	EB	1.9	0.15	0.0	0.24	0.0
	MT	1.1	0.18	0.0	0.30	0.0
	MB	1.4	0.20	0.0	0.22	0.0
	WT	1.8	0.20	0.0	0.20	0.0
	WB	0.3	0.20	0.0	0.20	0.0
NB1	ET	0.4	0.10	-	0.18	0.0
	EB	0.3	0.10	-	0.17	0.0
	MT	0.5	0.10	-	0.17	0.0
	MB	0.7	0.10	-	0.23	0.0
	WT	0.5	0.10	-	0.25	0.0
	WB	0.8	0.10	-	0.30	0.0

Results in milligrams per liter

TABLE 20

MICROSCOPICAL EXAMINATION

ACUSHNET RIVER, NEW BEDFORD OUTER HARBOR

STATION	ALGAE BLUE-GREEN	GREEN	FLAGELLATES	DIATOMS	CRUSTACEA	ROTIFER	AMORPHOUS MATTER
AR1	0	175	100	40	0	0	400
AR2	0	20	20	150	0	0	1000
AR3 E	50	40	0	175	2	0	800
AR3 W	30	115	0	50	0	0	900
AR4	0	0	0	0	0	200	1500
AR5	30	70	0	800	0	0	500
AR6	30	20	0	400	0	0	500
NB2 T	0	0	0	0	2	0	400
NB2 B	0	0	0	0	0	0	300
NB3 T	0	0	0	0	0	0	500
NB3 B	0	0	0	25	0	0	500
NB4 T	0	0	0	0	2	0	400
NB4 B	0	0	0	0	0	0	500
NB5 T	0	0	0	50	0	0	400
NB5 B	0	0	0	0	0	0	400
NB6 T	0	0	0	0	1	0	300
NB6 B	0	0	0	0	3	0	400

Samples collected July 13-15, 1971
Results in Aerial Standard Units per Cubic Centimeter

TABLE 21

MICROSCOPICAL EXAMINATION

NEW BEDFORD INNER HARBOR

STATION	BLUE GREEN	ALGAE GREEN	DIATOMS	CRUSTACEA	NEMATODES	AMORPHOUS MATTER
AR 7 ET	0	0	0	0	0	4100
EB	0	0	0	0	0	4500
MT	0	0	0	0	0	6000
MB	0	0	0	0	0	5000
WT	0	0	0	6	0	7000
WB	40	75	0	5	0	1800
AR 8 ET	0	0	0	0	0	2000
EB	0	0	10	1	0	3500
MT	10	0	0	1	0	2200
MB	0	0	0	1	1	1250
WT	0	0	0	1	0	2000
WB	0	0	4	2	0	1400
NB 1 ET	0	0	10	0	0	1500
EB	0	0	0	0	0	500
MT	0	0	0	0	0	500
MB	0	0	0	0	0	700
WT	0	0	0	0	0	600
WB	0	0	0	0	0	500

Samples collected July 20-22, 1971

Results in Aerial Standard Units per Cubic Centimeter

TABLE 22

BIOLOGICAL EXAMINATION OF SEDIMENT

ACUSHNET RIVER, NEW BEDFORD HARBOR

Station	Sampled	Physical Character	Macroorganisms #30 Sieve	% Organic Debris (volume)	% Inorganic Debris (volume)
AR1	7/14	Gravel	None Living	19.0	56.0
AR2	7/14	Muck	1 Arthropod	8.0	2.0
AR3	7/14	Muck	5 Arthropods	19.0	9.0
AR4	7/14	Sand	None Living	1.0	30.0
AR5	7/14	Muck	None Living	36.0	22.0
AR6	7/14	Muck	None Living	42.0	6.0
AR7E	7/21	Muck	5 Littorina	18.0	2.0
AR7M	7/21	Muck	1 Annelid	18.0	2.0
AR7W	7/21	Muck	3 Littorina 3 Annelids	9.0	1.0
AR8E	7/21	Muck	2 Annelids 2 Littorina	4.0	1.0
AR8M	7/21	Muck and Stones	1 Littorina	16.0	2.0
AR8W	7/21	Muck	3 Annelids	2.0	8.0
NB1E	7/21	Muck	1 Annelid 1 Littorina	15.0	2.0
NB1M	7/21	Muck	None Living	13.0	1.0
NB1W	7/21	Muck	None Living	<1.0	0.0
NB2	7/21	Gravel and Shell	1 Anomiidae 2 Limpets 1 Littorina	<1.0	63.0
NB3	7/21	Muck	None Living	20.0	1.0
NB4	7/21	Muck and Shell	1 Limpet 1 Littorina 1 Annelid	<1.0	23.0
NB5	7/21	Sand	None Living	0.0	28.0
NB6	7/21	Muck	1 Annelid	<1.0	15.0

TABLE 23

SPECIAL ANALYSIS OF SEDIMENT

NEW BEDFORD HARBOR

Heavy Metal Concentrations in Milligrams Per Killigram

Passing Number 30 Sieve

STATION	MERCURY	CADMIUM	LEAD	ZINC	NICKEL	COPPER	CHROMIUM	ARSENIC
AR7E	1.90	76.0	320.0	1700.0	100.0	1920.0	960.0	5.2
AR7M	3.10	53.0	310.0	1040.0	72.0	1620.0	920.0	5.2
AR7W	3.30	0.9	560.0	2300.0	180.0	2540.0	1380.0	5.2
AR8E	1.70	40.0	320.0	1070.0	110.0	2520.0	1280.0	3.2
AR8M	3.80	40.0	290.0	600.0	81.0	1680.0	1210.0	9.2
AR8W	2.70	24.0	310.0	1200.0	550.0	7250.0	3200.0	14.0
NB1E	0.90	1.9	410.0	95.0	6.8	1930.0	110.0	0.8
NB1M	1.70	0.7	11.0	35.0	3.6	36.0	21.0	0.0
NB1W	1.70	18.0	150.0	430.0	39.0	610.0	310.0	5.2
NB2	0.75	0.4	31.0	410.0	37.0	32.0	18.0	0.6
NB3	7.70	43.0	510.0	1170.0	36.0	760.0	250.0	8.2
NB4	1.80	0.8	38.0	59.0	6.9	40.0	27.0	1.6
NB5	0.85	0.1	3.4	5.5	1.5	5.0	5.1	0.0
NB6	0.21	0.9	20.0	50.0	4.5	59.0	27.0	0.6

TABLE 24

PHOTOSYTHESIS SUMMARY

ACUSHNET RIVER - NEW BEDFORD HARBOR

STATION NUMBER	LOCATION	DEPTH	NUMBER OF OBSERVATIONS	AVERAGE PHOTOSYNTHESIS	AVERAGE RESPIRATION
1	Outer Harbor off Butler Flats Lights	1.0	8	1.0	0.4
		8.0	8	0.3	0.5
		18.0	8	0.2	0.4
2	Inner Harbor opposite power plant	1.0	14	1.1	0.2
		5.0	14	0.5	1.3
		30.0	10	0.4	1.0
3	Acushnet River above Popes Island	1.0	14	2.3	1.1
		4.0	13	1.2	1.5
		17.0	14	0.3	1.6
3A	Acushnet River above Coggeshall St. Bridge	1.0	6	4.9	2.8
		3.0	6	1.6	2.6
		9.0	6	1.4	2.8
3B	Acushnet River below Wood St. Bridge	1.0	6	2.5	2.1
		2.0	6	0.6	2.1
4	Acushnet River at Hamlin Pond	1.0	8	2.1	3.2
		4.0	8	0.4	2.8
5	New Bedford Reservoir Below Lake Street	1.0	8	0.3	0.1
		7.0	8	0.6	0.8

TABLE 25

PHOTOSYNTHESIS DATA JULY 12 - 13

NEW BEDFORD HARBOR

STATION	DEPTH (FEET)	TEMPERATURE (^{o}F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
1	1.0	70	5.2	5.7	4.7	1.0	0.5
				5.7	4.6	1.1	0.6
	8.0	70	5.0	3.5	4.3	-0.8	0.7
				3.6	4.0	-0.4	1.0
	18.0	70	5.1	4.3	4.2	0.1	0.9
				4.3	4.7	-0.4	0.4
2	1.0	72	3.7	3.3	3.6	-0.3	0.1
				2.7	--	-0.9	--
	5.0	72	5.5	4.2	2.9	-1.3	2.6
				3.0	--	0.1	--
	30.0	72	3.5	--	--	--	--
				--	--	--	--
3	1.0	75	3.5	5.0	2.2	2.8	1.3
				4.3	--	2.1	--
	4.0	75	3.1	2.4	1.9	0.5	1.2
				2.3	--	0.4	--
	17.0	75	3.2	2.6	2.2	0.4	1.0
				1.9	--	-0.3	--

TABLE 26

PHOTOSYNTHESIS DATA JULY 12 - 13

ACUSHNET RIVER

STATION	DEPTH (FEET)	TEMPERATURE (^{o}F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
4	1.0	75	8.1	5.7	4.9	0.8	3.2
				7.0	4.5	2.5	3.6
	4.0	75	4.6	3.5	2.2	1.3	2.4
				1.7	2.3	-0.6	2.3
5	1.0	79	6.9	6.9	6.6	0.3	0.3
				7.3	7.2	0.1	-0.3
	7.0	79	4.3	4.8	4.6	0.2	-0.3
				4.5	4.5	0.0	-0.2

TABLE 27

PHOTOSYNTHESIS DATA JULY 13 - 14

NEW BEDFORD HARBOR

STATION	DEPTH (FEET)	TEMPERATURE (°F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
1	1.0	70	6.0	6.3	5.9	0.4	0.1
				--	5.7	0.6	0.3
	8.0	70	5.4	5.7	5.1	0.6	0.3
				5.7	5.0	0.7	0.4
	18.0	70	5.2	5.6	5.6	0.0	-0.4
				5.4	5.1	0.3	0.1
2	1.0	74	4.1	4.5	3.0	1.5	1.1
				4.3	3.2	1.1	0.9
	5.0	74	4.3	3.5	5.4	-1.9	-1.1
				3.5	1.3	2.2	3.0
	30.0	74	4.3	3.7	2.6	1.1	1.7
				3.9	2.6	1.3	1.7
3	1.0	75	3.0	3.9	2.7	1.2	1.8
				--	2.9	1.0	2.0
	4.0	75	3.6	3.3	0.0	3.3	3.6
				4.7	2.3	2.4	1.3
	17.0	74	3.1	2.6	1.0	1.5	2.1
				--	0.0	1.6	3.1

TABLE 28

PHOTOSYNTHESIS DATA JULY 13 - 14

ACUSHNET RIVER

STATION	DEPTH (FEET)	TEMPERATURE (^{o}F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
4	1.0	74	4.6	2.2	0.3	1.9	4.3
				1.5	0.0	1.5	4.6
	4.0	73	4.4	2.1	1.2	0.9	3.2
				1.4	1.3	0.1	3.1
5	1.0	80	7.2	7.5	7.4	0.1	-0.2
				8.0	7.1	0.9	0.1
	7.0	78	7.2	7.7	7.0	-0.7	0.2
				6.9	7.0	-0.1	0.2

TABLE 29

PHOTOSYNTHESIS DATA JULY 14 - 15

NEW BEDFORD HARBOR

STATION	DEPTH (FEET)	TEMPERATURE (°F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
1	1.0	71	--	7.1	6.1	1.0	--
				7.0	--	0.9	--
	8.0	71	6.7	6.7	6.5	0.2	0.2
				6.6	6.1	0.5	0.6
	18.0	71	6.6	6.2	6.0	0.2	0.6
				6.4	6.3	0.1	0.3
2	1.0	72	4.8	4.2	4.2	0.0	0.6
				4.1	3.9	0.2	0.9
	5.0	72	4.8	4.2	3.8	0.4	1.0
				--	4.0	0.2	0.8
	30.0	72	5.3	5.3	3.6	1.7	1.7
				5.3	3.8	1.5	1.5
3	1.0	73	3.1	5.1	2.5	2.6	0.6
				5.8	2.3	3.5	0.8
	4.0	73	4.4	2.9	2.2	0.7	3.7
				--	--	--	--
	17.0	73	4.3	3.0	2.4	0.6	3.7
				3.2	3.2	0.0	4.3

TABLE 30

PHOTOSYNTHESIS DATA JULY 14 - 15

ACUSHNET RIVER

STATION	DEPTH (FEET)	TEMPERATURE (°F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
4	1.0	--	4.4	5.5	1.2	4.3	3.2
				6.6	1.4	5.2	3.0
	4.0	--	4.4	1.5	1.5	0.0	2.9
				1.6	1.6	0.0	2.8
5	1.0	80	7.4	7.2	6.7	0.5	0.7
				7.2	6.7	0.5	0.7
	7.0	78	7.1	6.9	4.8	2.1	2.3
				6.6	2.0	4.6	5.1

TABLE 31

PHOTOSYNTHESIS DATA JULY 15 - 16

NEW BEDFORD HARBOR

STATION	DEPTH (FEET)	TEMPERATURE (^{o}F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
1	1.0	71	6.5	7.4	6.0	1.4	0.5
				7.3	--	1.3	--
	8.0	71	6.7	7.5	6.0	1.5	0.7
				6.8	6.5	0.3	0.2
	18.0	71	6.6	7.0	6.0	1.0	0.6
				6.5	5.9	0.6	0.7
2	1.0	72	4.7	4.7	3.9	0.8	0.8
				5.0	4.3	0.7	0.4
	5.0	72	4.6	4.4	3.6	0.8	1.0
				4.4	3.9	0.5	0.7
	30.0	72	4.9	4.0	3.8	0.2	1.1
				3.8	--	0.0	--
3	1.0	73	4.2	6.8	3.5	3.3	0.7
				6.4	3.3	3.1	0.9
	4.0	73	4.2	4.4	2.2	2.2	2.0
				4.2	3.0	1.2	1.2
	17.0	73	3.9	3.1	3.0	0.1	0.9
				3.3	3.1	0.2	0.8

TABLE 32

PHOTOSYNTHESIS DATA JULY 15 - 16

ACUSHNET RIVER

STATION	DEPTH (FEET)	TEMPERATURE (^{o}F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
4	1.0	74	4.8	2.9	2.5	0.4	2.3
				3.0	3.0	0.0	1.8
	4.0	74	2.0	0.0	0.0	0.0	2.0
				1.3	0.0	1.3	2.0
5	1.0	80	6.8	6.8	7.0	-0.2	-0.2
				7.6	7.5	0.1	-0.7
	7.0	78	6.9	6.7	7.3	-0.6	-0.4
				6.4	7.1	-0.7	-0.2

TABLE 33

PHOTOSYNTHESIS DATA JULY 19 - 20

NEW BEDFORD INNER HARBOR

STATION	DEPTH (FEET)	TEMPERATURE (°F)	INITIAL D.O. (mg/1)	LIGHT D.O. (mg/1)	DARK D.O. (mg/1)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
2	1.0	70	5.6	5.8	5.2	0.6	0.4
				6.2	4.7	1.5	0.9
	5.0	70	6.0	5.2	4.6	0.6	1.4
				--	5.0	0.2	1.0
	30.0	70	5.5	5.7	5.7	0.0	-0.2
				5.5	5.7	-0.2	-0.2
3	1.0	72	3.0	3.9	2.7	1.2	0.3
				2.4	3.5	-1.1	-0.5
	4.0	72	4.0	5.0	2.1	2.9	1.9
				2.7	2.6	0.1	1.4
	17.0	72	4.5	2.7	2.9	-0.2	1.6
				2.6	2.4	0.2	2.1
3A	1.0	70	3.8	1.7	1.3	0.4	2.5
				4.7	1.1	3.6	2.7
	3.0	70	3.2	1.9	0.6	1.3	2.6
				1.7	0.8	0.9	2.4
	9.0	70	3.7	1.5	0.8	0.7	2.9
				1.0	0.3	0.7	3.4
3B	1.0	70	4.0	1.0	0.0	1.0	4.0
				1.8	0.0	1.8	4.0
	2.0	70	3.6	0.3	0.0	0.3	3.6
				0.3	0.0	0.3	3.6

TABLE 34

PHOTOSYNTHESIS DATA JULY 20 - 21

NEW BEDFORD INNER HARBOR

STATION	DEPTH (FEET)	TEMPERATURE (°F)	INITIAL D.O. (mg/l)	LIGHT D.O. (mg/l)	DARK D.O. (mg/l)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
2	1.0	70	5.0	7.0	4.1	2.9	0.9
				7.4	4.1	3.3	0.9
	5.0	70	4.8	4.4	4.4	0.0	0.4
				4.0	4.3	-0.3	0.5
	30.0	70	5.1	3.2	4.3	-1.1	0.8
				4.1	4.7	-0.6	0.4
3	1.0	70	3.6	5.8	2.3	3.5	1.3
				6.1	3.4	2.7	0.2
	4.0	70	4.0	3.4	2.8	0.6	1.2
				3.0	3.2	-0.2	0.8
	17.0	70	3.4	1.9	2.2	-0.3	1.2
				2.3	2.5	-0.2	0.9
3A	1.0	70	2.0	4.0	0.0	4.0	2.0
				7.3	0.0	7.3	2.0
	3.0	70	1.6	0.1	0.0	0.1	1.6
				0.0	0.0	0.0	1.6
	9.0	70	1.6	0.0	0.0	0.0	1.6
				0.0	0.0	0.0	1.6
3B	1.0	70	1.2	3.1	1.2	1.9	0.0
				5.0	0.0	5.0	1.2
	2.0	70	0.9	0.0	0.0	0.0	0.9
				0.8	0.0	0.8	0.9

TABLE 35

PHOTOSYNTHESIS DATA JULY 21 - 22

NEW BEDFORD INNER HARBOR

STATION	DEPTH (FEET)	TEMPERATURE (°F)	INITIAL D.O. (mg/l)	LIGHT D.O. (mg/l)	DARK D.O. (mg/l)	LIGHT - DARK (PHOTOSYNTHESIS)	INITIAL - DARK (RESPIRATION)
2	1.0	74	2.5	6.3	6.0	0.3	-3.5
				8.7	5.0	3.7	-2.5
	5.0	74	5.6	5.4	2.7	2.7	2.9
				--	2.8	2.6	3.2
	30.0	74	4.6	8.0	--	--	--
				0.0	--	--	--
3	1.0	74	3.6	4.6	1.6	3.0	2.0
				5.2	1.6	3.6	2.0
	4.0	74	3.7	3.2	3.0	0.2	0.7
				3.2	3.0	0.2	0.7
	17.0	74	3.4	3.1	3.0	0.1	0.4
				3.4	3.2	0.2	0.2
3A	1.0	74	3.8	8.7	0.0	8.7	3.8
				5.6	0.0	5.6	3.8
	3.0	74	3.8	3.0	0.0	3.0	3.7
				4.2	0.0	4.2	3.7
	9.0	74	3.7	4.7	0.0	4.7	3.7
				2.6	0.0	2.6	3.7
3B	1.0	74	1.8	2.6	0.0	2.6	1.8
				2.8	0.0	2.8	1.8
	2.0	74	1.7	0.0	0.0	0.0	1.7
				2.1	0.0	2.1	1.7

TABLE 36

WEEKLY PHOTOSYNTHESIS SUMMARIES

STATIONS 2 AND 3

		WEEK OF JULY 12, 1971			WEEK OF JULY 19, 1971		
STATION	DEPTH	NO. OBSERVATIONS	AVG. PHOTO.	AVG. RESP.	NO. OBSERVATIONS	AVG. PHOTO.	AVG. RESP.
2	1.0	8	0.4	0.7	6	2.0	-0.5
	5.0	8	0.1	1.1	6	1.0	1.6
	30.0	6	1.0	1.5	4	-0.5	0.2
3	1.0	8	2.3	1.2	6	2.2	0.9
	4.0	7	1.5	2.0	6	0.6	1.1
	17.0	8	0.5	1.9	6	0.0	1.1

SHORELINE SURVEY

Following the sampling program on the Acushnet River and New Bedford Harbor it was determined that a shoreline survey to locate all pertinent source discharges was necessary. Engineers of the Southeast Regional Office of the Division of Water Pollution Control, in cooperation with the Water Quality Section and the United States Coast Guard Reserves, conducted the survey August 24 through September 1, 1971. Through the efforts of Lt. Comm. John J. Fitzgerald, Commanding Officer ORTUPS 01-82045, the United States Coast Guard allowed the training ship Courier to be assigned to the New Bedford Area. An orientation period was held Tuesday morning August 24, aboard the Courier, following which, group assignments were made. Shoreline and boat patrols were utilized to locate the outfalls.

The survey revealed 130 pipe and waste source locations. Discharges varied from no flow from some sources to an average of 3 million gallons per day from sanitary sources and 1 million gallons per day from industrial sources. The combined results of the river sampling program and the shoreline survey established that the inner New Bedford Harbor is receiving pollution loads which render the waters unsuitable for all water usage.

Members of the Southeast Regional Office are starting an effective program of pollution abatement in this area by conducting inplant investigations of all industrial discharges. The data obtained from these investigations will be used to evaluate the need for pretreatment prior to connection to the municipal sewage system or for full treatment with discharge. In addition, the data will assist Division personnel in the evaluation and

review of plans for private industry and the plans being formulated by a number of consulting firms in the updating of the sewerage system for the City of New Bedford and Towns of Acushnet and Fairhaven. The present status of the pollution abatement program by these municipalities is as follows:

1. Fairhaven - The town has recently completed a new secondary sewage treatment plant, pumping stations, and interceptors to convey all wastes to the treatment plant providing secondary treatment with chlorination prior to discharge to the Inner New Bedford Harbor. One outfall has not been connected; however, the town has been directed by this Division to proceed with this connection prior to releasing funds for final payment.

2. Acushnet - The Town of Acushnet is not meeting their implementation dates. Funds have not been appropriated for the construction of sewers to tie into the City of new Bedford. Private corporations and the town storm drain are discharging inadequately treated wastes to the Acushnet River and Inner New Bedford Harbor causing contravention of the water quality standards. Robert Childs Associates, consultants, have recommended to the Town to appropriate funds and proceed with final planning for connection to New Bedford.

3. New Bedford - The City of New Bedford, located on the westerly bank of the Acushnet River and Inner Harbor, is presently constructing a 130 MGD primary sewage treatment plant with discharge into the Outer New

Bedford Harbor through an outfall located of Ft. Rodman. The western bank of the Acushnet River is heavily industrialized and a major problem exists in this area because of separated industrial waste discharges and approximately 46 combined sewage overflow points. Urban renewal projects under construction or in planning stages in the heavy populated shoreline areas include a program of sewer separation which will assist in eliminating some sanitary wastes now combined with storm drainage entering the Harbor. Two major problem areas are located at the Sawyer Street Pumping Station and Coffin Avenue Pumping Station. The City has engaged the engineering firm of Tibbets Engineering, New Bedford, to prepare an engineering plan to correct major overflows into the harbor. The engineers have been limited to a ceiling of $15,000,000 in overall construction costs for planning, whereas complete separation is estimated to cost $90,000,000. The Sawyer Street Station presently in dry weather has a constant sewage overflow. Stop logs are being installed at the present time in an effort to divert this overflow into an 18-inch sanitary sewer. Under consideration for correcting the Sawyer Street problem is the construction of a pumping station and interceptor designed for a dry weather flow of 30 million gallons per day. Flows in excess of this, which is estimated to be in the vicinity of 70 million gallons a day, will be diverted to a micro filter, then chlorinated prior to discharge into the Inner Harbor. Sludge from the micro filter would be backwashed into the sanitary sewer.

There would be stations similar to Sawyer Street located at Coffin Avenue, State Pier, and upstream from Route 6.

Other major problems, are industrial wastes discharges (presently being investigated), oil discharges by vessels and town storm drainage systems and the organic bottom deposits existing within the Inner Harbor. Appended are listings of pipe discharges, complaints, industries, and status of field investigations to date.

The City of New Bedford has been requested by the Division on advice by the Southeast Regional Office to formulate an industrial waste survey of waste discharged to the City of New Bedford sewerage system. The City will then enforce its sewer use ordinance requiring pretreatment of industrial wastes. The Southeast Regional Office will cooperate with the City in the programming of this industrial waste survey.

It is the opinion the the Regional Office that a permanent local authority - namely, the New Bedford Harbor Commission - be given funding and enforcement authority to coordinate local supervision of a water pollution abatement program within the harbor. This Commission would oversee complaints and delegate authority to the various conservation and pollution abatement groups in the vicinity, such as the Special Committee on Pollution, Counselor David R. Nelson, Chairman, for follow-up to the individual complaints and problems. Investigation would be conducted by the Division personnel on the fish processing plants and of sanitary and oil discharges from the New Bedford and Fairhaven fishing fleet should be coordinated through the Commission due to the political-economic situation existing in the fishing industry in Massachusetts today.

Two major fish kills involving menhaden and striped bass occurred in New Bedford during 1970, an indication of the degree of pollution existing in the Inner Harbor. Observations by Division personnel noted oil slicks

existing within the inner harbor in the vicinity of the New Bedford and Fairhaven fishing fleet dock areas. Debris and floating solids including fish gurrey is visible each and every day. Oil spills from New England Petroleum have been reported in the past. Thermal pollution does not seem to be a problem in respect to New Bedford Gas and Electric's generating station. However, the overall affect of thermal pollution from industrial wastes discharges to the inner harbor will have to be evaluated. Past fish kills have been attributed to the combination of the sanitary sewerage and industrial waste pollution depleting the available dissolved oxygen.

NEW BEDFORD HARBOR POLLUTION SURVEY

SOUTH DARTMOUTH

Clark's Cove

1. 2-3" CI and 1-4" A- Land drains behind white house at end of William St. - No Flow

2. 1-36" RC - North of Mohel residence - Slight Flow

3. 1-12" RC - 100 yards south of oil yard - Possible Sewage -Low flow

4. 1-6" CI - yard drain from oil yard at foot of Rodgers Street - No flow - Appears abandoned

NEW BEDFORD

Clark's Cove

5. 2-24" CI - 25 yards North of end of Rodgers Street - Clear flow

6. 5-4" CI - Traver's Seafood - Padnaram St. - One (1) minor flow

7. 1-60" RC - End of dike - 50 yards north of Traver's Seafood - No flow

8. 1-18" CI - 10 feet to South of jetty - Opposite Woodlawn St. - Small Clear Flow

9. 1-18" CI - 5 feet to north of jetty opposite Dudley St. - No flow

10. 1-18" CI - Located in retaining wall - 200 ft. north of Hazelwood Park Entrance - End of pipe visible - No flow

11. 1-18" CI - 100 yards north of Lucas Street -- End of pipe not visible - No Flow

12. 1-18" CI - Opposite Oaklawn Street - End of pipe not visible - No flow

13. 1-8" VC - Opposite Cattle St. - No flow

14. 1-60" CI - Located at foot of Coral St. - North of pumping station - No flow

15. 1-18" CI - 100 yards North of Bonito St. - No flow

16. 1-18" VC - South of Seymour St. - No flow

NEW BEDFORD

Outer Harbor

17. New Bedford Outfall Sewer - Point of discharge not visible

17. a. Storm drainage to hidden box - 200 yards from first large pier extension - No flow

18. Storm drainage to hidden box - Just inside Ft. Rodman Gate - No flow

19. 1-18" CI - Located beneath Rip-Rap and below water at end of jetty South of Seymour St. - Cloudy Discharge.

20. 1-24" CI - 100 ft. north of Portland St. - No Flow

21. 1-12" CI - Storm drain opposite first street north of Portland St. - No flow

22. 1-36" CI - Storm drain at end of jetty opposite Ricketson St. - No flow

23. 1-12" VC - and 1-24" VC - Opposite Aquidneck St. - No flow but possible combined overflow

24. 1-24" CI - Storm drain - 400 feet north of Aquidneck St.

25. 1-12" CI - Located 5 feet to the South of #26 - No flow

26. 1-18" and 4-12" - 100 ft. north of pump house - 18" Water Temperature 100° - The 12" lines appeared abandoned

27. 1-12" VC - 300 ft. South of #28 - Flow approximately 30 -40 gpm Temperature 95°

28. 1-24" CI - At end of small jetty across from Butler St. - Submerged - Slight discoloration in vicinity of pipe

29. 1-48" VC - Located outside of dike opposite chimney stack of Cornel Dubilier - Cloudy discharge - Floating Solids

30. 1-8" RC and 48" concrete culvert - Corner of Rodney French Blvd. and Cove Street - No flow but known to be sewer overflow

Inner Harbor

31. Gifford St. - Sewer outfall - small flow - Discolored

32. 1-24" CS - Blackmer St. outfall

NEW BEDFORD

Inner Harbor

33. Ell. Vee Dee Filet House - Noted entrails- Oil - Suds - and cloudy discharge under dock

34. South of New Bedford Gas & Electric by Ell Vee Dee Filet House

35. 2-4 ft. RC - Culverts - New Bedford Gas & Electric - No Discharges noted

36. 4" to 12" multiple pipes - Inlet North of Culverted discharge - New Bedford Power and Light - Light green suspended solids noted

37. Gem Filet House - Small Operation appears shut down

38. 1-60" RC - North of Gem Filet near Campanella Construction Trailer - Oil rainbows - Sewage & fish entrails noted

39. Seaview Filet Company - Homer's Wharf - wash down into harbor

40. New Bedford Fish Products - South of State Pier - Wash down into Harbor

41. Multiple 6" and 8" CI - Located at head wall of inlet to Pier 2 - No discharge but oil noted in water

42. 1-8" CI - Pier 3 area - Minor flow - Oil films noted in water

43. Cast Iron Pipe - New Bedford Ice Co. - Clean Water Flow

44. 1-18" CI - Crystal Ice Co. - Clear Water Flow

45. 1-12" VC - located 150 ft. from West end of Fish Island (South Side)- Appeared abandoned

46. 2-6" CI - West end of Fish Island (North Side) Yard drain adjacent to Route #6 Bridge - No flow

47. Yard drains from Glen Oil Co. - No flow

48. Fish Island - Unloading of fish wash down - Discharge into Harbor

49. 1-18" RC - Street drain at foot of Route #6 Bridge

50. 1-18" RC - Street drain for Route 6 Bridge (North Side)

51. 1-18" CI - Discharge from Quaker Oats flowing 1/3 full

52. 2 ft. wide runoff trough and two 2" P - No flow

53. 1-48" RC - North of Quaker Oats - Herman Melville Blvd.-No Flow

NEW BEDFORD

Inner Harbor

54. 1-36" RC - 1500 ft. North of #53 - Herman Melville Blvd. Flowing Sewage

55. 1-6 ft. box culvert - 200 yds. South of Revere Copper & Brass - Herman Melville Blvd. - Large volume of sewage

56. 1-12" and 1-24" CI and 2-36" RC - Opposite Wamsutta St.- small clear flow

57. 2-12" RC - and one storm culvert - South side of Revere Copper & Brass - Pipes not flowing but culvert was - Rainbow films noted

58. a. Revere Copper & Brass - 6" VC - 3 feet below platform

    b. 1-18" VC - 150 ft. South of North end of pier

    c. 1-24" RC - 3 feet below platform- All were submerged outfalls

59. 1-10" VC and 1-8" VC - No flow in 10 inch - Sewage and cooling water in 8" - Both outfalls submerged

60. 1-10" CS - To rear of Alpine Co. - No flow

61. 1-12" RC - Coggeshall St. - Bridge drainage - No Flow

62. 1-12" CI - Coggeshall St. - Bridge Drainage- No Flow

63. 1-60" RC - Sawyer St. - Outfall - Oil - Sewage and industiral waste noted - Heavy Flow

64. Storm drain with hidden box - Behind North end of Fairhaven Corp - No flow

65. 1-6"P - Industrial - Not flowing

66. Multiple 1" and 4" CI - Isotronics, Inc. - Flowing from back wall of building into river

67. 1-5 ft. open culvert - End of Coffin Ave. - High volume of raw sewage

68. 1-10" CI - Located 5 ft. North of #67 - No flow

69. 1-4" CI - 50 ft. North of Coffin Ave. - American Press Co. - No Flow

NEW BEDFORD

Inner Harbor

70. Multiple 4" P - Behind Cameo Curtain & Draperies Co. - Riverside Ave.

71. 1-72" Culvert - Manomet St. Outfall - Raw Sewage

72. 1-36" CI - 200 ft. north Manomet St. - No Flow

73. 3-12" CI - Behind New Bedford Rayon Co - 100 yds. North of Manomet St. - Minimal flow - Oil noted in discharge channel

74. 1-24" CI - Behind Acushnet Process Plant D - No flow

75. 1-60" RC - Box Culvert - Sewage and oil noted - North of Acushnet Process Plant D

76. 1-48" RC - Box culvert behind Tibbets Engineering Co - Directly behind large white stack - No flow

77. Multiple 6" and 12" VC - Behind Fiber Leather Corp. - Intermittent discharge - Oil and much debris and settled matter noted in vicinity of discharge.

78. 3-24" CI - South end of Acushnet Building C - Slight Flow

79. 1-10" VC - Behind Acushnet Building C - 20 yds. south of #80 - No flow

80. 1-6" VC - Behind Acushnet Building C - 20 yds. South of 81 - No flow

81. 1-4 ft. X 4 ft. concrete duct - South end of Aerovox Corp. - Steady sewage flow

82. 2-10" VC - Directly behind Aerovox - 25yds. South of #83 - Slight Sewage Flow

83. 2 ft. Concrete Duct - North end of Aerovox - 40 yds. South of #84 - Steady Flow of clear water

84. 4 ft. box culvert - concrete - 15 ft. to the south of #85 - Steady water flow - Some oil

85. 1-18" VC - Beyond Fence at end of Acushnet Drive - Steady flow - some oil

NEW BEDFORD

Inner Harbor

86. 1-18" RC - Drain from Wood St. Bridge - North Side

87. 1-12" VC - Drainage from Wood St. Bridge - North Side

88. 1-60" RC - 150 yds. North of Wood St. Bridge - Raw Sewage

ACUSHNET

Inner Harbor

89. 1-2" and 2-6" CI - Foundation wall of Bear Wheel & Brake Co. - No Flow in 6" pipes - 2" pipe appears to be oil discharge

90. Storm drain to hidden box - Slight Flow across river from Reliable Homes

91. 1-12" RC - Drainage from Wood St. Bridge - Acushnet side

92. Unknown pipe buried under debris - North edge of Wood St. Bridge

93. Acushnet Processing - Golf Ball Division

a. Oil seeping out of Rip-Rap Acushnet Process Co. - Golf ball Division - 20 ft. South of Red Building

b. 1-10" VC - Acushnet Golf Ball Division - Located at foot of first brick building south of bridge

c. 1-8" VC - Acushnet Golf Ball Division - 100 yds. South of Bridge - Pipe Flowing half full - Temp. 100°

d. 1-4" CI - 10 yds. South of #93c - No flow

e. 3-12" VC - All Pipes flowing - Black oil noted in two of them

f. 1-24" VC - Slight flow of cloudy grayish solutions

g. Oil draining into marsh grass at edge of dumping pit

94. Septic tank discharge - Acushnet Rest Home - North Main St.

95. Drainage ditch from Warren Bros.

FAIRHAVEN

Inner Harbor

96. 1-3 ft. x 8 ft. storm culvert - located opposite Magnolia Ave. - Storm drainage

FAIRHAVEN

Inner Harbor

ditch - Low flow - Trace of oil

97. 1-18" CS - foot of Veranda Ave. - 1-24" RC - drainage from Coggshall Bridge

98. 1-12" RC - 100 yds. from East End of Coggshall Bridge - South side - No Flow

99. 1-12" RC - Storm drain behind sea wall end of street south of radio towers - No flow

100. 1-4" VC - Land drain adjacent to white house at end of Pilgrim St. - No flow

101. 1-36" RC - 20 ft. North of Route 6 Bridge - No flow

102. 1-12" RC - Between Stop & Shop and Eugene's Steak House Pope's Island - Half submerged

103. Oil visible on embankment - Possibly waste crank case oil behind John Dugan Auto Service Center

104. 3-4" VC - only 1 pipe flowing --- Sewage

105. 1-10" VC - Yard drain - in sea wall South of WBSM TV and Radio Station - No Flow

106. 2-6" CI - Storm drains at end of Skipper Motor Inn - No Flow

107. Storm water pumping Station across from Park Motors on Middle St.

108. 1-10" VC - Storm drain adjacent to West End Corp. corner of Washington & Water Sts. - No Flow

109. 2-4" CI - Rear of Texaco Marine Products (Green building) No Flow

110. 9-4" CI - Located under wharf - McClean's Seafood Inc - 6 flowing - 3 dry - Clear Flow

111. 1-4" CI - Behind New Bedford Seafood Corp - Clear flow

112. 1-10" VC - Located at North side between first and second piers - No Flow

113. 1-24" RC - Storm drain - 100 yds. North of Hathaway Machine Co. - South Side - No Flow

114. 1-18" RC - Storm drain at foot of Church St. - No Flow

115. Fairhaven outfall - Could only see the boil - No flowing matter noted.

FAIRHAVEN

Inner Harbor

116. 1-24" CI - Located at end of pier on Fairhaven Marine Inc. - No Flow

117. Outer Harbor - 48" Culvert through dike east of Ft. Phoenix- Waste discharge from Atlas Tack Co.

118. 1-12" RC - 100 yds. South of Mack's Soda Bar - Clear Flow

119. 1-4" CI - South of 15 Bayview Rd. - Appears to be house drain - minimal flow

120. 1-8" CI - South of #119 - Minimal Flow

121. 1-18" CS - Redrock Beach - abandoned

122. 1-12" RC - Foot Bonney Street - No Flow

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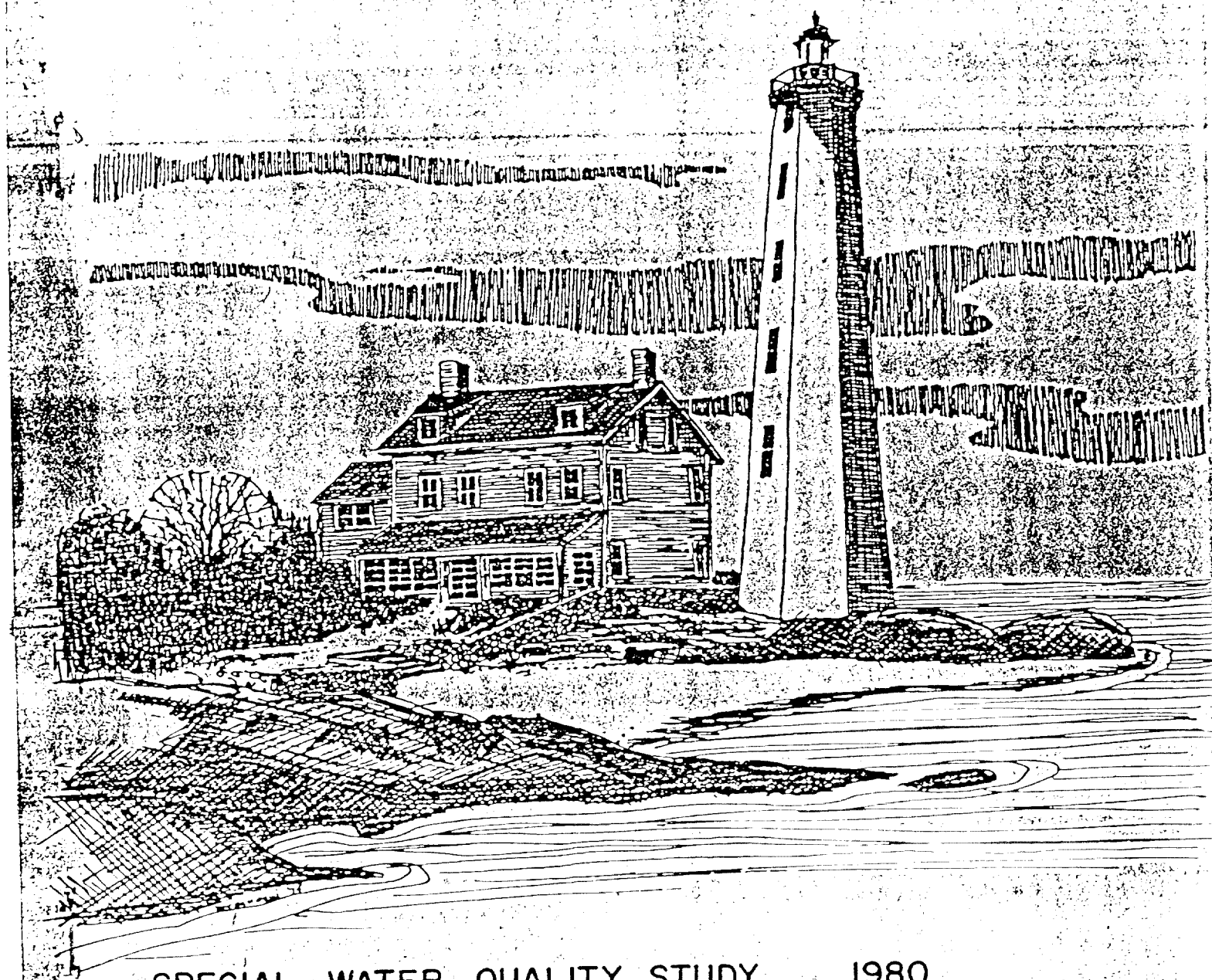
BUZZARDS BAY
OUTER NEW BEDFORD HARBOR



SPECIAL WATER QUALITY STUDY 1980

massachusetts department of environmental quality engineering
DIVISION OF WATER POLLUTION CONTROL
thomas c. mcmahon, director

SPECIAL WATER QUALITY STUDY

BUZZARDS BAY AND OUTER NEW BEDFORD HARBOR

1980

DEPARTMENT OF ENVIRONMENTAL QUALITY ENGINEERING

DIVISION OF WATER POLLUTION CONTROL

TECHNICAL SERVICES BRANCH

WESTBOROUGH, MASSACHUSETTS

JANUARY 1982

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INTRODUCTION

This report contains data from water quality surveys conducted during the summer and fall of 1980. The study was coordinated by the Massachusetts Division of Water Pollution Control with assistance from Dr. William Bannister of the University of Lowell; Dr. William Curby of Lahey Clinic; Al Davis and C.H. Clifford of Woods Hole Oceanographic Institute; and Dr. David Kan of Massachusetts Maritime Academy. The purpose of the study was to provide additional water quality data for the assessment of the City of New Bedford's application for a 301(h) waiver. Section 301(h) of the 1977 amendments to the Clean Water Act makes provision for the waiver of secondary treatment of wastewater discharged from publicly owned treatment works into marine waters under specified conditions. With a 301(h) waiver, the outfall from the New Bedford STP would be extended out into Buzzards Bay, into deeper water; without a 301(h) waiver, the New Bedford STP would be upgraded to provide secondary treatment, with the outfall remaining at the present location.

A total of eight surveys were undertaken during the period of July 2, 1980 to October 15, 1980. Five stations in the vicinity of the proposed extended outfall site, and two stations in the area of the present sewage treatment plant outfall were sampled. Temperature, salinity, and dissolved oxygen measurements were taken at regular depth intervals. Samples of the surface and bottom waters were taken for chemical analysis including, but not limited to: BOD_5, chlorides, total phosphorus, total Kjeldahl nitrogen, nitrate nitrogen, and suspended solids. Analyses for PCBs and metals were run on bottom sediment samples taken from four locations.

Sample analyses were performed at the Lawrence Experiment Station, Massachusetts Department of Environmental Quality Engineering. All were run in accordance with A.P.H.A. Standard Methods for the Examination of Water and Wastewater (14th ed.) and EPA Oceanographic Sampling and Analytical Procedures Manual, 1979, procedure A-4.

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LOCATIONS OF SAMPLING STATIONS
NEW BEDFORD HARBOR

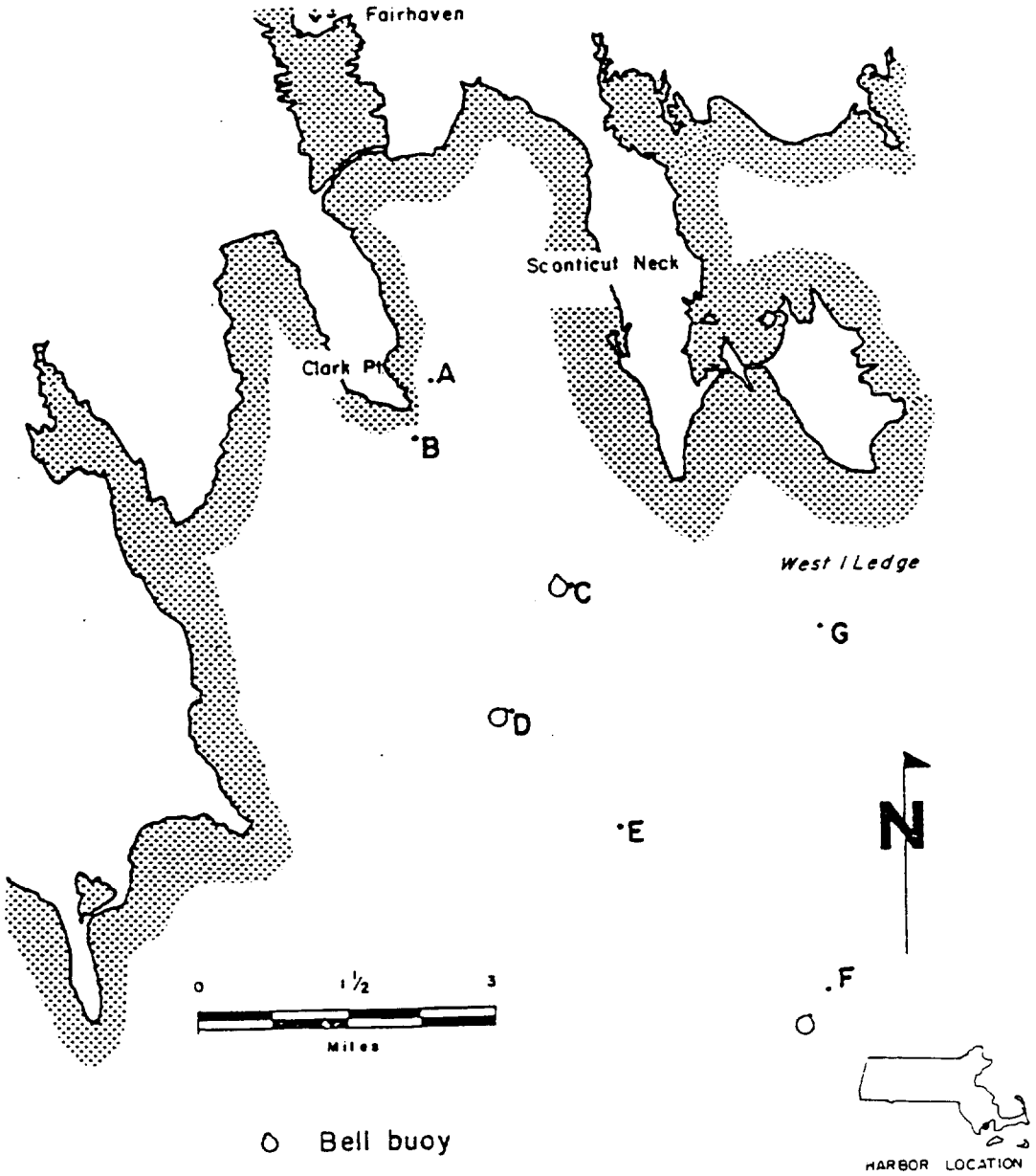


FIGURE I

TABLE 1

NEW BEDFORD SPECIAL WATER QUALITY STUDY

LOCATION OF SAMPLING STATIONS

STATION	LORAN T.D.	LATITUDE - LONGITUDE
A	14196.1 43999.2	41° 35' 56" 70° 53' 23"
B[1]	14199.6 43994.9	41° 34' 88" 70° 53' 35"
C	14195.0 43987.1	41° 34' 00" 70° 52' 03"
D	14203.1 43981.3	41° 33' 00" 70° 52' 69"
E[2]	14201.6 43973.4	41° 32' 03" 70° 51' 81"
F	14194.3 43961.8	41° 30' 76" 70° 49' 75"
G	14180.8 43982.2	41° 33' 83" 70° 49' 58"

[1]Site of New Bedford Sewage Treatment Plant outfall

[2]Proposed outfall site

TABLE 2

NEW BEDFORD SPECIAL WATER QUALITY STUDY

COLIFORM DATA (COLIFORM/100 ml)*

STATION	COUNT	DATE
A	150	7/2/80
B	112	7/2/80
C	2-8	7/2/80
D	1	7/2/80

TABLE 3

NEW BEDFORD SPECIAL WATER QUALITY STUDY

FECAL COLIFORM DATA (COLIFORM/100 ml)*

STATION	COUNT	DATE
A	140	7/2/80
B	55	7/2/80
C	3	7/2/80
D	1/500 ml	7/2/80

*Analyses by Lahey Clinic

TABLE 4

NEW BEDFORD SPECIAL WATER QUALITY STUDY

CHLOROPHYLL-a (mg/m^3)

STATION	CHLOROPHYLL-a
A - surface	5.81
bottom	--
B - surface	3.74
bottom	--
C - surface	4.98
bottom	--
D - surface	4.15
bottom	--
E - surface	1.90
bottom	--
F - surface	2.49
bottom	--
G - surface	3.32
1.0 m.	2.49
2.5 m.	3.32
4.0 m.	3.32
5.5 m.	3.32
7.0 m.	2.91
8.5 m.	3.12
10.0 m.	1.76
11.5 m.	2.17
13.0 m.	1.76

DATE: 8/12/80

TABLE 5

NEW BEDFORD SPECIAL WATER QUALITY STUDY

AMMONIA - NITROGEN (mg/l)

STATION	DATE 8/12/80	8/26/80	9/16/80
A - surface	0.02	0.00	0.02
bottom	0.05	0.03	0.03
B - surface	0.46	0.10	1.10
bottom	0.10	0.02	0.05
C - surface	0.08	0.02	0.02
bottom	0.04	0.02	0.02
D - surface	0.05	0.02	0.02
bottom	0.05	0.04	0.01
E - surface	0.24	0.07	0.01
bottom	0.08	0.04	0.05
F - surface	0.08	0.03	--
bottom	0.04	0.01	0.01
G - surface	0.08	0.03	0.02
bottom	0.07	0.01	0.01

TABLE 6

NEW BEDFORD SPECIAL WATER QUALITY STUDY

pH (STANDARD UNITS)

STATION	DATE 8/26/80	9/16/80
A - surface	7.9	7.7
bottom	8.0	7.8
B - surface	7.9	7.4
bottom	7.7	7.8
C - surface	7.9	7.9
bottom	7.8	7.9
D - surface	8.0	8.0
bottom	7.8	7.9
E - surface	8.1	8.0
bottom	7.8	8.0
F - surface	7.9	--
bottom	7.8	7.8
G - surface	7.9	8.0
bottom	7.8	8.0

TABLE 7

NEW BEDFORD SPECIAL WATER QUALITY STUDY

NITRATE-NITROGEN (mg/l)

STATION	DATE 7/2/80	7/8/80	7/22/80	8/12/80	8/26/80	9/16/80
A - surface	0.05	--	0.01	0.02	0.00	0.00
bottom	0.03	--	0.02	0.05	0.00	0.00
B - surface	0.03	0.03	0.03	0.46	0.00	0.00
bottom	0.03	0.01	0.03	0.10	0.00	0.00
C - surface	0.01	0.01	0.02	0.08	0.00	0.00
bottom	0.02	0.03	0.00	0.04	0.00	0.00
D - surface	0.04	0.02	0.81	0.05	0.00	0.00
bottom	0.03	0.90	0.01	0.05	0.00	0.00
E - surface	0.65	0.01	0.05	0.24	0.00	0.00
bottom	0.05	0.01	0.05	0.08	0.00	0.00
F - surface	0.02	0.03	0.03	0.08	0.00	0.00
bottom	0.02	0.03	0.03	0.04	0.00	0.00
G - surface	0.03	0.00	0.02	0.08	0.00	0.00
bottom	0.00	0.03	0.03	0.07	0.00	0.00

TABLE 8

NEW BEDFORD SPECIAL WATER QUALITY STUDY

KJELDAHL-NITROGEN (mg/l)

STATION	DATE 7/2/80	7/8/80	7/22/80	8/12/80	8/26/80	9/16/80
A - surface	0.3	--	0.4	0.5	0.8	0.6
bottom	0.4	--	0.7	0.2	1.4	0.6
B - surface	0.3	1.1	0.8	0.6	0.7	2.4
bottom	0.2	0.4	0.4	0.2	1.2	0.5
C - surface	0.2	0.2	0.6	0.2	0.5	0.3
bottom	0.2	0.3	0.6	0.2	0.7	0.6
D - surface	0.2	0.4	0.3	0.3	0.5	0.4
bottom	0.1	0.3	0.2	0.2	1.2	0.4
E - surface	0.2	0.3	0.3	0.2	0.6	0.5
bottom	0.2	0.2	0.3	0.1	1.1	0.5
F - surface	0.1	0.3	0.4	0.1	0.7	--
bottom	0.2	0.3	0.3	0.0	1.1	0.5
G - surface	0.1	0.2	0.2	0.1	0.5	0.6
bottom	0.1	0.3	0.0	0.1	0.3	0.5

TABLE 9

NEW BEDFORD SPECIAL WATER QUALITY STUDY

TOTAL PHOSPHORUS AS P (mg/l)

STATION	DATE 7/2/80	7/8/80	7/22/80	8/12/80	8/26/80	9/16/80
A - surface	0.02	--	0.01	0.14	0.12	0.33
bottom	0.01	--	0.00	0.14	0.20	0.15
B - surface	0.02	0.57	0.03	0.29	0.13	0.51
bottom	0.02	0.04	0.01	0.13	0.10	0.14
C - surface	0.00	0.02	0.00	0.12	0.09	0.14
bottom	0.01	0.02	0.00	0.12	0.09	0.14
D - surface	0.00	0.02	0.01	0.14	0.09	0.15
bottom	0.00	0.04	0.00	0.09	0.08	0.11
E - surface	0.01	0.02	0.00	0.09	0.08	0.15
bottom	0.00	0.02	0.00	0.09	0.08	0.10
F - surface	0.00	0.02	0.01	0.09	0.07	--
bottom	0.00	0.02	0.04	0.08	0.07	0.09
G - surface	0.00	0.04	0.01	0.10	0.12	0.12
bottom	0.00	0.04	0.01	0.09	0.04	0.11

TABLE 10

NEW BEDFORD SPECIAL WATER QUALITY STUDY

CHLORIDES (mg/l)

STATION	DATE 7/2/80	7/8/80	7/22/80	8/12/80	8/26/80	9/16/80
A - surface	18,270	--	18,540	18,000	18,000	17,200
bottom	18,270	--	18,270	18,000	18,000	17,200
B - surface	18,000	16,660	16,660	17,000	18,000	16,400
bottom	18,540	18,270	17,730	18,000	18,000	17,600
C - surface	18,270	18,270	18,270	18,000	18,000	17,600
bottom	18,270	18,270	18,810	17,000	18,000	17,600
D - surface	18,540	18,270	18,810	18,000	18,000	17,200
bottom	18,270	18,270	18,270	18,000	18,000	17,600
E - surface	18,270	18,540	18,270	18,000	--	17,600
bottom	18,270	18,270	18,270	18,000	18,000	17,600
F - surface	18,270	18,270	18,270	18,000	18,000	--
bottom	18,270	18,270	18,270	18,000	18,000	17,200
G - surface	18,270	18,270	18,270	18,000	17,600	17,200
bottom	18,270	18,270	18,270	18,000	17,600	17,200

TABLE 11

NEW BEDFORD SPECIAL WATER QUALITY STUDY

5-DAY BIOCHEMICAL OXYGEN DEMAND (mg/l)

STATION		DATE 7/2/80	7/8/80	7/22/80	8/12/80	8/26/80	9/16/80
A -	surface	1.5	--	2.2	2.4	4.3	2.0
	bottom	1.6	--	1.8	2.4	1.6	0.8
B -	surface	2.1	>7.0	>7.2	6.3	4.4	9.6
	bottom	1.7	2.8	1.6	1.8	1.8	0.8
C -	surface	1.0	1.7	1.8	2.4	4.2	1.0
	bottom	1.4	1.7	1.7	1.2	1.6	0.4
D -	surface	1.0	1.4	2.0	2.4	2.8	1.4
	bottom	0.8	3.2	1.0	1.5	1.2	0.6
E -	surface	3.4	1.6	1.4	1.5	4.8	0.8
	bottom	0.8	2.4	1.1	0.9	1.0	1.0
F -	surface	0.8	1.9	0.9	1.2	3.8	--
	bottom	1.0	1.6	1.4	0.9	3.0	0.8
G -	surface	1.9	1.7	1.2	1.2	1.4	0.8
	bottom	1.3	1.6	1.0	1.2	1.8	0.4

TABLE 12

NEW BEDFORD SPECIAL WATER QUALITY STUDY

SUSPENDED SOLIDS (mg/l)

STATION	DATE 7/2/80	7/8/80	7/22/80	8/12/80	8/26/80	9/16/80
A - surface	33	--	25	17	2.0	23
bottom	33	--	26	17	5.0	21
B - surface	36	112	50	24	18	35
bottom	40	35	34	13	4.5	22
C - surface	41	31	40	18	74	22
bottom	32	36	37	10	24	29
D - surface	39	28	35	15	17	32
bottom	33	35	38	70	10	23
E - surface	33	26	36	16	6.5	24
bottom	42	35	38	14	9.5	25
F - surface	32	37	34	14	5.5	--
bottom	31	26	33	5.5	4.5	20
G - surface	36	41	38	16	6.5	37
bottom	38	37	36	11	5.5	23

TABLE 13

NEW BEDFORD SPECIAL WATER QUALITY STUDY

POLYCHLORINATED BIPHENYL (WET WEIGHT μg/kg)

DATE: OCTOBER 15, 1980

STATION AND SEDIMENT DEPTH*	POLYCHLORINATED BIPHENYL AROCLOR NUMBER 1248	POLYCHLORINATED BIPHENYL AROCLOR NUMBER 1260	PERCENT MOISTURE
A - 0-2 cm.	323	88	61.14
A - 2-4 cm.	250	264	56.24
A - 4-8 cm.	333	531	56.16
A - 8-12 cm.	45	114	50.90
B - surface grab	3,592	22,787	29.69
E - 0-2 cm.	62	0	60.63
E - 2-4 cm.	222	0	55.24
E - 4-8 cm.	196	0	37.28
E - 8-12 cm.	367	0	33.07
F - 0-2 cm.	103	0	64.20
F - 2-4 cm.	86	0	52.13
F - 4-8 cm.	86	0	49.00
F - 8-12 cm.	234	0	45.53

*Samples at Stations A, E and F were taken with a Soutar box corer.
Sample at Station B was taken with an Eckman dredge.

TABLE 14

NEW BEDFORD SPECIAL WATER QUALITY STUDY

METALS* (mg/kg)

STATION AND SEDIMENT DEPTH	PERCENT VOLATILE SOLIDS	COPPER	LEAD	ZINC	CHROMIUM	CADMIUM	NICKEL	ARSENIC	MERCURY
A - 0-2 cm.	2.9	148	97	191	182	48	27	8.8	0.250
A - 2-4 cm.	2.7	190	98	180	140	4.2	25	9.1	0.105
A - 4-8 cm.	2.8	227	116	191	147	3.1	23	6.2	0.085
A - 8-12 cm.	2.5	183	100	185	100	1.2	19	7.5	0.274
E - 0-2 cm.	2.1	15	36	73	28	0.7	15	6.5	0.102
E - 2-4 cm.	1.8	14	31	61	24	7.0	16	6.2	0.082
E - 4-8 cm.	1.1	11	25	50	21	0.0	10	5.4	0.097
E - 8-12 cm.	1.1	9	19	48	15	8.0	8	5.3	0.642
F - 0-2 cm.	2.4	12	28	57	24	0.0	13	10.2	0.298
F - 2-4 cm.	1.9	14	29	68	28	0.8	20	10.0	0.400
F - 4-8 cm.	1.9	12	30	62	29	0.8	16	9.0	0.074
F - 8-12 cm.	1.9	13	30	65	31	0.0	16	7.4	0.074

*Samples taken with a Soutar box corer.

TABLE 15

NEW BEDFORD SPECIAL WATER QUALITY STUDY

TIDAL DATA FOR NEW BEDFORD (EST)

DATE	HIGH TIDE (HRS)	LOW TIDE (HRS)
7/2/80	1049	1607
7/8/80	1637	0952
7/22/80	1537	0810
8/5/80	1520	0827
8/12/80	0920	1520
8/26/80	0740	1321
9/16/80	1229	1740
10/15/80	1154	1715

TABLE 16

NEW BEDFORD SPECIAL WATER QUALITY STUDY

DISSOLVED OXYGEN, TEMPERATURE, AND SALINITY

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
7/2/80	A - surface	20.0	32.3	8.5
	1.5	20.5	32.3	8.2
	3.0	20.1	32.2	8.4
	4.6	20.1	32.2	8.2
	6.1	20.0	32.2	8.0
	7.6	19.8	32.3	7.7
	9.1	19.7	32.1	---
	10.7	19.7	31.9	---
7/8/80	A - surface	---	34.0	11.1
	1	---	34.0	---
	2	---	34.0	7.6
	3	---	34.0	---
	4	---	34.0	7.6
	5	---	34.0	---
	6	---	34.2	---
	7	---	34.0	---
	8	---	34.0	---
8/5/80	A - surface	26.0	32.2	7.6
	1.5	26.0	32.3	---
	2.0	---	---	7.5
	3.0	25.8	32.4	---
	4.0	---	---	6.5
	4.6	24.4	32.3	---
	6.0	---	---	6.8
	6.1	23.9	32.2	---
	7.6	23.0	32.2	---
	8.0	---	---	3.5
8/12/80	A - surface	24.3	32.1	6.4
	1.5	24.2	32.1	---
	2.0	---	---	5.2
	3.0	24.3	32.2	---
	4.0	---	---	6.1
	4.6	24.1	32.2	---
	6.0	---	---	6.0
	6.1	24.2	32.2	---
8/26/80	A - surface	23.0	32.2	9.3
	1.0	---	---	9.0
	1.5	23.0	32.2	---
	3.0	22.2	32.2	8.5
	4.6	22.2	32.1	---
	5.0	---	---	6.9
	6.1	21.7	32.2	---
	7.6	21.4	32.2	---

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
9/16/80	A - surface	20.5	32.8	3.9
	1.5	20.5	32.7	---
	2.0	---	---	4.1
	3.0	20.5	32.7	---
	4.0	---	---	3.9
	4.6	20.5	32.8	---
	6.0	---	---	4.3
	6.1	20.3	32.8	---
	7.6	19.8	33.0	---
10/15/80	A - surface	14.2	32.3	7.8
	1.5	14.2	32.3	---
	2.0	---	---	7.9
	3.0	14.1	32.3	---
	4.0	---	---	7.6
	4.6	13.9	32.4	---
	6.0	---	---	7.8
	6.1	13.8	32.3	---
7/2/80	B - surface	19.8	31.8	9.6
	1.5	19.8	32.1	8.1
	3.0	19.9	32.1	9.1
	4.6	19.9	32.2	8.1
	6.1	19.8	32.2	8.4
	7.6	19.6	32.1	8.0
	9.1	19.1	32.2	6.2
	10.7	19.0	32.2	6.3
7/8/80	B - surface	19.8	30.1	6.7
	1.5	19.7	32.1	---
	2.0	---	---	7.8
	3.0	19.8	32.0	---
	4.0	---	---	7.5
	4.6	19.7	32.1	---
	6.0	---	---	8.3
	6.1	19.7	33.1	---
7/22/80	B - surface	---	30.0	6.6
	1.0	---	30.5	---
	2.0	---	34.0	6.0
	3.0	---	34.0	---
	4.0	---	34.0	7.2
	5.0	---	34.0	---
	6.0	---	34.0	6.7
	7.0	---	34.0	---
8/5/80	B - surface	25.9	31.6	6.8
	1.5	26.1	32.3	---
	2.0	---	---	7.2
	3.0	26.1	32.3	---
	4.0	---	---	7.7

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
8/5/80	B - 4.6	24.6	32.2	---
	6.0	---	---	6.6
	6.1	23.5	32.2	---
8/12/80	B - surface	24.1	31.3	6.1
	1.5	24.0	31.8	---
	2.0	---	---	6.9
	3.0	23.8	32.2	---
	4.0	---	---	6.0
	4.6	24.0	32.2	---
	6.0	---	---	6.0
	6.1	24.0	32.2	---
	7.6	23.9	32.1	---
8/26/80	B - surface	22.9	30.8	8.8
	1.5	22.8	32.2	---
	2.0	---	---	9.2
	3.0	22.4	32.2	---
	4.0	---	---	7.6
	4.6	21.9	32.1	---
	6.1	21.3	32.3	---
	7.6	21.2	32.3	---
9/16/80	B - surface	20.7	32.8	6.8
	1.5	20.5	32.8	---
	2.0	---	---	6.2
	3.0	20.4	32.8	---
	4.0	---	---	7.2
	4.6	20.4	32.8	---
	6.0	---	---	7.1
	6.1	20.3	32.9	---
	7.6	20.3	33.0	---
	9.1	20.2	33.0	---
10/15/80	B - surface	15.0	30.7	7.5
	1.5	14.6	32.4	---
	2.0	---	---	7.9
	3.0	14.5	32.4	---
	4.0	---	---	7.4
	4.6	14.5	32.4	---
	6.0	---	---	7.8
	6.1	14.5	32.4	---
	7.6	14.5	32.4	---
7/2/80	C - surface	19.5	32.2	8.5
	1.5	19.5	32.1	8.7
	3.0	19.5	32.2	8.1
	4.6	19.5	32.1	8.3
	6.1	19.5	32.1	7.9
	7.6	19.4	32.2	8.4
	9.1	19.4	32.2	8.1
	10.7	19.5	32.2	8.3

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
7/2/80	C - 12.2	19.4	32.3	7.2
	13.7	19.1	32.3	---
7/8/80	C - surface	19.8	30.1	6.7
	1.5	19.7	32.1	---
	2.0	---	---	7.8
	3.0	19.8	32.0	---
	4.0	---	---	7.5
	4.6	19.7	32.1	---
	6.0	---	---	8.3
	6.1	19.7	33.1	---
7/22/80	C - surface	---	33.2	7.6
	1.0	---	33.5	---
	2.0	---	33.5	7.6
	3.0	---	34.0	---
	4.0	---	33.8	9.0
	5.0	---	33.5	---
	6.0	---	34.2	7.2
	7.0	---	34.0	---
	8.0	---	34.8	6.9
	9.0	---	33.8	---
	10.0	---	34.0	---
8/5/80	C - surface	24.9	32.3	7.4
	1.5	24.3	32.3	---
	2.0	---	---	6.0
	3.0	24.2	32.3	---
	4.0	---	---	6.3
	4.6	23.4	32.2	---
	6.0	---	---	6.9
	6.1	23.2	32.3	---
	7.6	23.0	32.2	---
	8.0	---	---	6.9
	9.1	22.6	32.1	---
8/12/80	C - surface	23.4	32.3	6.6
	1.5	23.6	32.2	---
	2.0	---	---	6.7
	3.0	23.6	32.2	---
	4.0	---	---	7.0
	4.6	23.5	32.2	---
	6.0	---	---	6.5
	6.1	23.5	32.3	---
	7.6	23.4	32.3	---
	8.0	---	---	6.3
	9.1	23.4	32.3	---
	10.7	23.4	32.5	---
8/26/80	C - surface	22.9	32.3	7.0
	1.5	22.4	32.3	---
	2.0	---	---	7.0

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
8/26/80	C- 3.0	21.9	32.2	---
	4.0	---	---	7.5
	4.6	21.9	32.3	---
	6.0	---	---	7.2
	6.1	21.7	32.3	---
	7.6	21.6	32.2	---
	8.0	---	---	7.6
	9.1	21.5	32.2	---
	10.7	21.3	32.2	---
9/16/80	C -surface	20.3	32.8	7.5
	1.5	20.3	32.8	---
	2.0	---	---	7.8
	3.0	20.3	32.8	---
	4.0	---	---	7.5
	4.6	20.3	32.8	---
	6.0	---	---	7.7
	6.1	20.4	32.9	---
	7.6	20.3	32.9	---
	8.0	---	---	7.5
	9.1	20.3	32.9	---
	10.7	20.3	32.9	---
	12.2	20.3	32.9	---
7/2/80	D -surface	19.2	32.2	8.0
	1.5	19.4	32.2	9.0
	3.0	19.4	32.2	9.3
	4.6	19.4	32.3	8.1
	6.1	19.4	32.3	8.3
	7.6	19.5	32.4	8.1
	9.1	19.4	32.2	8.3
	10.7	19.5	32.2	8.8
	12.2	19.5	32.2	8.0
	13.7	19.4	32.2	---
7/8/80	D -surface	19.0	32.0	8.0
	1.5	19.0	31.9	---
	2.0	---	---	7.8
	3.0	19.0	32.0	---
	4.0	---	---	7.8
	4.6	19.0	32.0	---
	6.0	---	---	7.7
	6.1	19.0	32.0	---
	7.6	18.9	32.0	---
	8.0	---	---	7.7
	9.1	18.7	32.0	---
7/22/80	D -surface	---	33.2	---
	1.0	---	33.8	---
	2.0	---	34.0	7.2
	3.0	---	34.0	---
	4.0	---	34.0	8.0
	5.0	---	34.0	---

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
7/22/80	D - 6.0	---	34.0	6.9
	7.0	---	34.0	---
	8.0	---	34.0	7.0
	9.0	---	34.0	---
	10.0	---	34.0	---
	11.0	---	34.0	---
8/5/80	D - surface	24.5	32.3	---
	1.5	23.9	32.2	---
	3.0	23.9	32.3	---
	4.6	23.7	32.3	---
	5.0	---	---	---
	6.1	23.6	32.3	---
	7.0	---	---	6.6
	7.6	23.3	32.2	---
	9.1	22.9	32.3	---
	10.7	22.3	32.2	---
	12.2	22.0	32.1	---
8/12/80	D - surface	23.1	32.1	6.7
	1.5	23.0	32.2	---
	2.0	---	---	6.7
	3.0	23.1	32.1	---
	4.0	---	---	6.7
	4.6	23.0	32.2	---
	6.0	---	---	6.7
	6.1	23.0	32.2	---
	7.6	22.9	32.2	---
	8.0	---	---	6.6
	9.1	22.8	32.2	---
	10.7	22.6	32.3	---
8/26/80	D - surface	21.9	32.3	7.7
	1.0	---	---	8.1
	1.5	22.1	32.3	---
	3.0	21.8	32.2	7.1
	4.6	21.4	32.2	---
	5.0	---	---	7.1
	6.1	21.3	32.2	---
	7.0	---	---	7.4
	7.6	20.9	32.2	---
	9.0	---	---	6.7
	9.1	20.8	32.2	---
	10.7	20.6	32.3	---
9/16/80	D - surface	20.3	32.9	7.5
	1.5	20.3	32.9	---
	2.0	---	---	3.7
	3.0	20.3	32.9	---
	4.0	---	---	4.0
	4.6	20.3	32.9	---

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
9/16/80	D - 6.0	---	---	3.6
	6.1	20.3	32.9	---
	7.6	20.3	32.9	---
	8.0	---	---	3.6
	9.1	20.3	32.9	---
	10.7	20.3	32.9	---
	12.2	20.3	32.9	---
7/2/80	E - surface	20.0	32.3	8.7
	1.5	18.9	32.3	9.6
	3.0	18.8	32.2	8.6
	4.6	18.5	32.2	9.0
	6.1	18.2	32.4	8.5
	7.6	18.1	32.5	8.8
	9.1	17.9	32.5	8.5
	10.7	17.9	32.3	9.1
	12.2	17.9	32.4	8.4
	13.7	17.8	32.4	8.7
	15.2	17.3	32.4	---
7/8/80	E - surface	18.9	32.0	8.1
	1.5	18.9	32.0	---
	2.0	---	---	8.0
	3.0	18.9	32.0	---
	4.0	---	---	8.0
	4.6	18.9	31.9	---
	6.0	---	---	7.6
	6.1	18.9	32.0	---
	7.6	18.9	32.0	---
	8.0	---	---	8.2
	9.1	18.9	32.0	---
	10.0	---	---	7.9
	10.7	18.8	32.0	---
	12.0	---	---	8.4
	12.2	18.7	32.0	---
	13.7	18.7	32.1	---
	14.0	---	---	8.3
7/22/80	E - surface	22.5	32.8	7.2
	1.0	---	33.0	---
	2.0	---	33.0	7.6
	3.0	---	33.0	---
	4.0	---	33.8	8.3
	5.0	---	34.2	---
	6.0	22.5	34.0	7.2
	7.0	---	34.0	---
	8.0	21.5	34.0	7.2
	9.0	---	33.8	---
	10.0	20.0	33.8	7.3
	11.0	---	33.5	---
	12.0	---	33.8	7.0
	13.0	21.0	34.2	---

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
8/5/80	E - surface	24.1	32.3	7.0
	1.5	24.0	32.3	---
	3.0	22.8	32.1	---
	4.0	---	---	6.8
	4.6	22.1	32.0	---
	6.0	---	---	6.9
	6.1	22.0	32.2	---
	7.6	21.6	32.2	---
	8.0	---	---	6.4
	9.1	21.6	32.1	---
	10.0	---	---	6.4
	10.7	21.5	32.1	---
	12.0	---	---	6.3
	12.2	21.5	32.1	---
	13.7	21.5	32.2	---
	15.2	21.3	31.7	---
8/12/80	E - surface	22.6	31.7	7.1
	1.0	---	---	7.0
	1.5	22.4	31.7	---
	2.5	---	---	6.8
	3.0	22.4	32.0	---
	4.0	---	---	6.9
	4.6	22.4	31.2	---
	5.5	---	---	6.9
	6.1	22.4	31.8	---
	7.0	---	---	6.8
	7.6	22.2	31.9	---
	8.5	---	---	7.0
	9.1	22.1	31.8	---
	10.0	---	---	6.7
	10.7	21.7	31.9	---
	11.5	---	---	7.4
	12.2	20.8	31.8	---
	13.0	---	---	6.8
	13.7	20.5	31.9	---
8/26/80	E - surface	21.2	32.1	6.9
	1.0	---	---	7.8
	1.5	21.3	32.2	---
	3.0	21.2	32.3	8.0
	4.6	20.9	32.3	---
	5.0	---	---	7.4
	6.1	20.2	32.3	---
	7.0	---	---	7.1
	7.6	20.1	32.3	---
	9.0	---	---	7.1
	9.1	20.0	32.3	---
	10.7	20.0	32.3	---
	11.0	---	---	7.0
	12.2	20.0	32.3	---

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
9/16/80	E - surface	20.1	32.5	7.9
	1.0	---	---	7.9
	1.5	20.0	32.5	---
	3.0	20.1	32.5	8.1
	4.6	20.1	32.4	---
	5.0	---	---	7.6
	6.1	20.0	32.4	---
	7.0	---	---	7.5
	7.6	20.0	32.4	---
	9.0	---	---	7.8
	9.1	20.0	32.4	---
	10.7	20.0	32.4	---
	11.0	---	---	7.7
	12.2	20.0	32.4	---
	13.7	20.0	32.5	---
	15.2	20.2	32.6	---
10/15/80	E - surface	15.0	32.3	7.8
	1.5	15.0	32.4	---
	2.0	---	---	8.2
	3.0	15.0	32.4	---
	4.0	---	---	6.9
	4.6	15.0	32.4	---
	6.0	---	---	7.8
	6.1	15.0	32.4	---
	7.6	15.0	32.4	---
	8.0	---	---	7.7
	9.1	15.0	32.3	---
	10.0	---	---	7.9
	10.7	15.0	32.3	---
	12.0	---	---	7.8
	12.2	15.0	32.4	---
	13.7	15.0	32.4	---
7/2/80	F - surface	18.0	32.1	8.4
	1.5	18.0	32.1	8.6
	3.0	17.8	32.2	8.3
	4.6	17.7	32.2	8.6
	6.1	17.5	32.2	8.2
	7.6	17.0	32.2	8.5
	9.1	16.8	32.2	8.5
	10.7	16.7	32.2	8.5
	12.2	16.7	32.2	8.5
	13.7	16.6	32.2	8.4
	15.2	16.6	32.3	---
	16.8	16.6	32.3	---
	18.3	16.5	32.2	---
7/8/80	F - surface	19.2	32.0	7.5
	1.5	19.1	32.0	---
	2.0	---	---	7.5

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
7/8/80	3.0	19.1	32.0	---
	4.0	---	---	7.7
	4.6	19.0	32.0	---
	6.0	---	---	7.7
	6.1	19.0	32.0	---
	7.6	19.0	32.0	---
	8.0	---	---	8.1
	9.1	19.0	32.0	---
	10.0	---	---	7.4
	10.7	19.0	32.0	---
	12.0	---	---	7.8
	12.2	18.9	32.0	---
	13.7	18.6	32.0	---
	14.0	---	---	7.9
	15.2	18.2	31.9	---
	16.0	---	---	8.0
7/22/80	F - surface	22.5	34.0	---
	1.0	---	34.5	---
	2.0	22.5	34.5	7.4
	3.0	---	34.5	---
	4.5	22.5	34.2	7.9
	5.0	---	34.0	---
	6.0	22.0	34.2	8.6
	7.0	---	34.2	---
	8.0	21.0	33.8	7.4
	9.0	---	33.2	---
	10.0	21.7	33.0	7.6
	11.0	---	33.2	---
	12.0	21.0	34.0	8.1
	13.0	---	33.2	---
	14.0	20.5	34.5	---
	16.0	20.5	---	6.8
8/5/80	F - surface	23.6	32.2	7.2
	1.5	23.7	32.3	---
	3.0	23.1	32.3	7.0
	4.6	22.2	32.2	---
	5.0	---	---	6.9
	6.1	22.2	32.2	---
	7.6	21.9	32.1	---
	9.0	---	---	7.3
	9.1	21.9	32.2	---
	10.7	21.3	32.2	---
	11.0	---	---	7.1
	12.2	20.9	32.2	---
	13.0	---	---	6.4
	13.7	20.8	32.1	---
	15.0	---	---	6.5
	15.2	20.7	32.1	---
	16.8	20.9	32.1	---

TABLE 16 (CONTINUED)

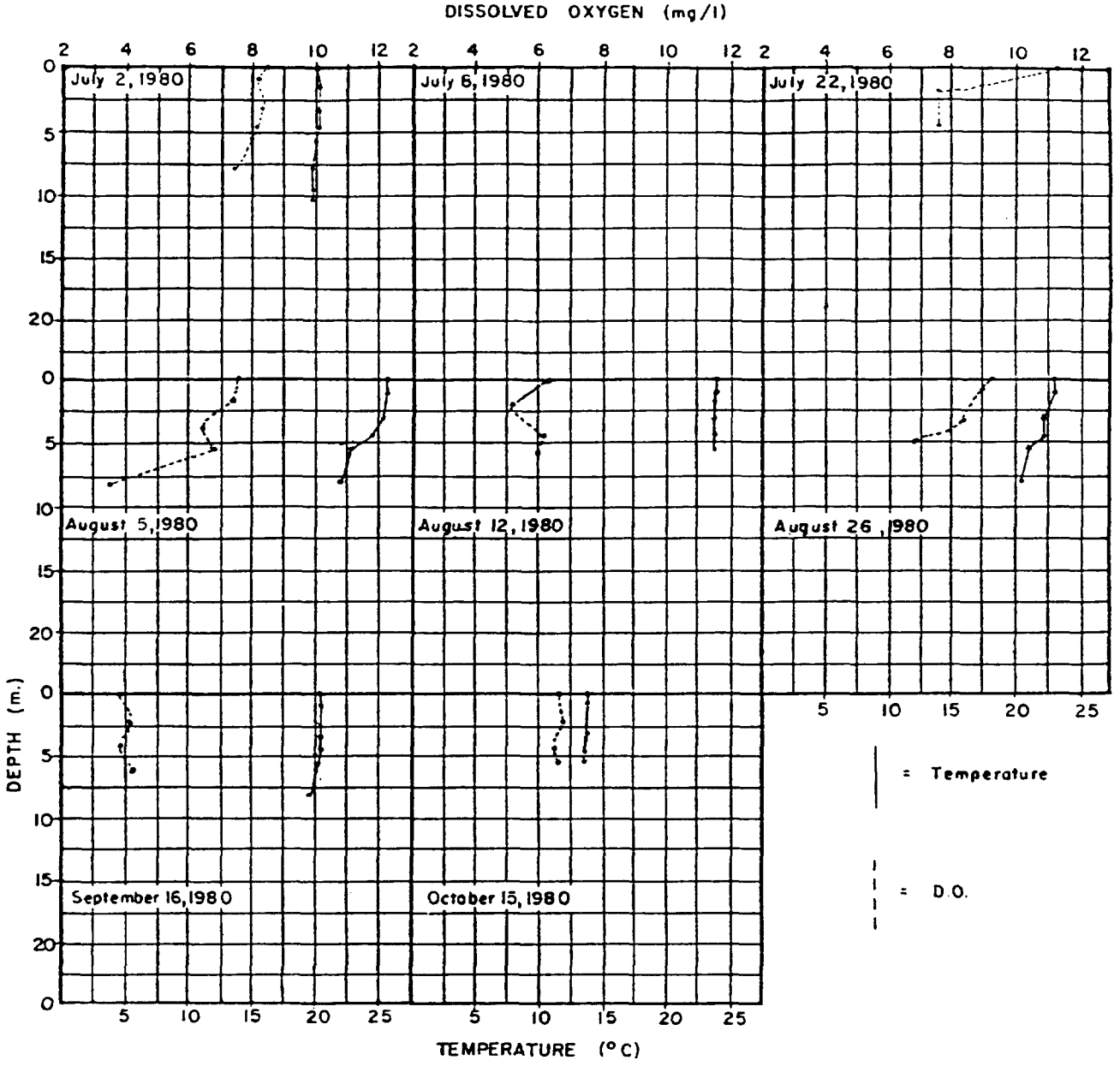
DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
8/12/80	F - surface	21.7	32.0	7.4
	1.5	21.8	32.0	---
	2.0	---	---	7.6
	3.0	21.8	32.0	---
	4.0	---	---	7.4
	4.6	21.6	32.2	---
	6.0	---	---	7.6
	6.1	21.0	32.2	---
	7.0	---	---	7.6
	7.6	20.4	32.1	---
	9.0	---	---	7.7
	9.1	18.8	32.1	---
	10.7	18.7	32.1	---
	11.0	---	---	7.0
	12.2	18.7	32.2	---
	13.0	---	---	7.4
	13.7	18.7	32.3	---
	15.0	---	---	7.3
	15.2	18.5	32.2	---
8/26/80	F - surface	20.6	32.2	7.7
	1.0	---	---	7.7
	1.5	20.0	32.2	---
	3.0	19.5	32.3	7.7
	4.6	19.4	32.3	---
	5.0	---	---	7.5
	6.1	19.4	32.2	---
	7.0	---	---	7.6
	7.6	19.4	32.3	---
	9.0	---	---	8.2
	9.1	19.4	32.1	---
	10.7	19.4	32.2	---
	11.0	---	---	7.7
	12.2	19.3	32.2	---
	13.0	---	---	7.0
	13.7	19.2	32.3	---
	15.0	---	---	7.4
	15.2	19.1	32.3	---
9/16/80	F - surface	19.7	34.2	7.9
	1.5	19.7	34.2	7.9
	3.0	19.7	34.1	7.8
	4.6	19.7	33.9	7.9
	6.1	19.7	33.6	7.7
	7.6	19.8	33.5	---
	9.0	---	---	7.9
	9.1	19.8	33.5	---
	10.7	19.8	33.6	---

TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
9/16/80	11.0	---	---	7.9
	12.2	19.6	34.0	---
	13.0	---	---	7.7
	13.7	19.7	33.3	---
	15.0	---	---	7.7
	15.2	19.5	33.3	---
	16.8	19.7	33.3	---
	18.3	19.6	33.3	---
10/15/80	F - surface	14.8	32.4	---
	1.5	14.8	32.4	---
	3.0	14.8	32.4	7.7
	4.6	14.8	32.4	---
	5.0	---	---	7.8
	6.1	14.8	32.4	---
	7.0	---	---	7.6
	7.6	14.9	32.4	---
	9.0	---	---	7.9
	9.1	14.8	32.4	---
	10.7	14.8	32.4	---
	11.0	---	---	7.7
	12.2	14.8	32.4	---
	13.0	---	---	7.7
	13.7	14.8	32.5	---
	15.0	---	---	7.7
	15.2	14.8	32.5	---
7/2/80	G - surface	19.2	32.2	7.8
	1.5	19.7	32.5	---
	4.6	19.7	32.2	---
	6.1	19.8	32.2	---
	7.6	19.8	32.1	---
	9.1	19.7	32.2	---
	10.7	19.7	32.1	8.3
	12.2	19.7	32.1	---
7/8/80	G - surface	19.8	32.0	7.4
	1.5	19.8	31.9	---
	2.0	---	---	7.1
	3.0	19.8	31.9	---
	4.0	---	---	7.0
	4.6	19.8	31.9	---
	6.1	19.7	31.9	---
	7.3	19.8	31.9	---
7/22/80	G - surface	23.0	35.0	7.5
	1.0	---	35.0	---
	2.0	23.0	35.0	6.8
	3.0	---	35.0	---
	4.0	22.5	34.5	7.3

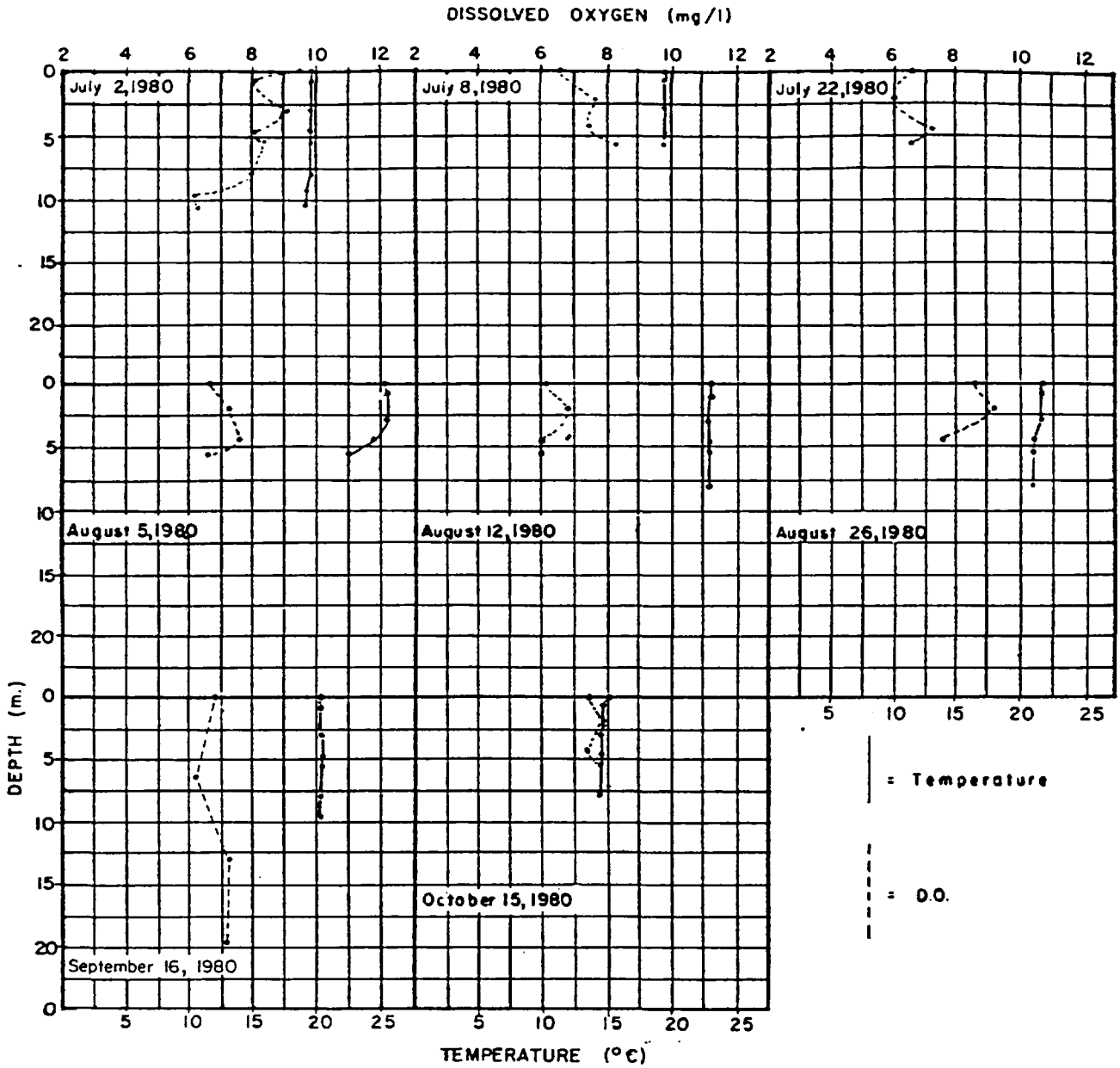
TABLE 16 (CONTINUED)

DATE	STATION AND DEPTH (m)	TEMPERATURE (°C)	SALINITY (°/00)	DISSOLVED OXYGEN (mg/l)
7/22/80	5.0	---	35.0	---
	6.0	22.5	35.0	6.3
	7.0	---	35.5	---
	8.0	22.0	35.2	6.2
8/5/80	G - surface	24.0	32.3	6.0
	1.5	23.8	32.3	---
	2.0	---	---	6.1
	3.0	23.6	32.2	---
	4.0	---	---	5.9
	4.6	23.6	32.3	---
	6.0	---	---	6.2
	6.1	23.3	32.2	---
	7.6	23.3	32.0	---
8/12/80	G - surface	23.1	32.1	6.5
	1.5	23.3	32.2	6.4
	2.0	---	---	6.3
	3.0	23.2	32.2	---
	4.0	---	---	6.2
	4.6	22.8	32.5	---
	5.0	---	---	6.0
	6.1	23.1	32.5	---
	7.6	22.9	32.2	---
	9.1	22.9	32.0	---
8/26/80	G - surface	21.6	32.4	7.7
	1.0	---	---	6.2
	1.5	21.4	32.4	---
	3.0	21.5	32.4	7.5
	4.6	21.5	32.4	---
	5.0	---	---	7.9
	6.1	21.4	32.4	---
	7.6	21.4	32.3	---
9/16/80	G - surface	19.9	33.0	7.0
	1.5	19.9	32.8	---
	2.0	---	---	7.9
	3.0	19.9	32.8	---
	4.0	---	---	6.7
	4.6	30.0	32.9	---
	6.0	---	---	7.1
	6.1	20.0	32.9	---
	7.6	19.9	32.9	---
	8.0	---	---	7.3
	9.1	19.9	32.9	---
	10.0	---	---	6.9



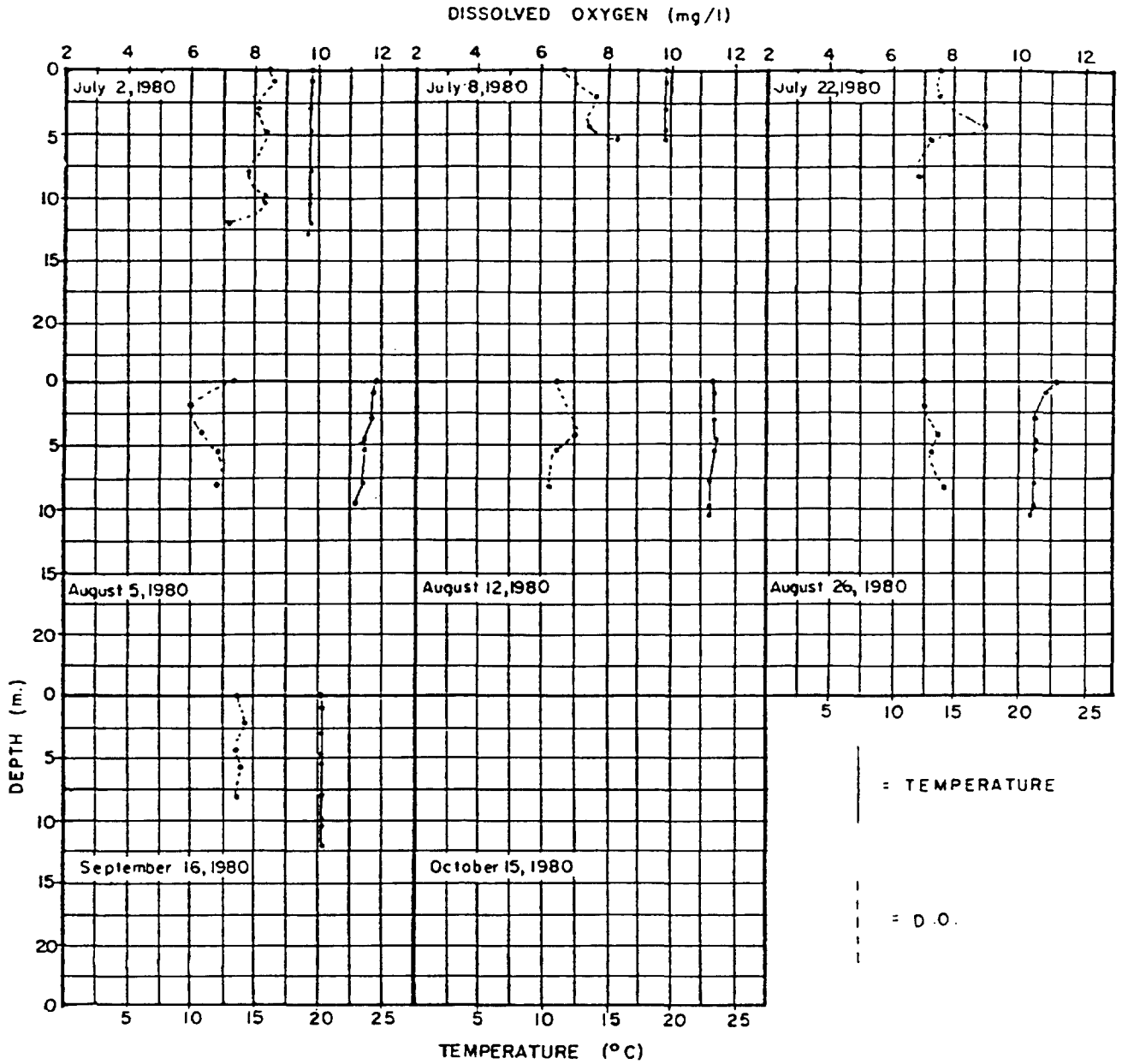
NEW BEDFORD HARBOR STUDY
DISSOLVED OXYGEN AND
TEMPERATURE PROFILES
STATION A

FIGURE 2



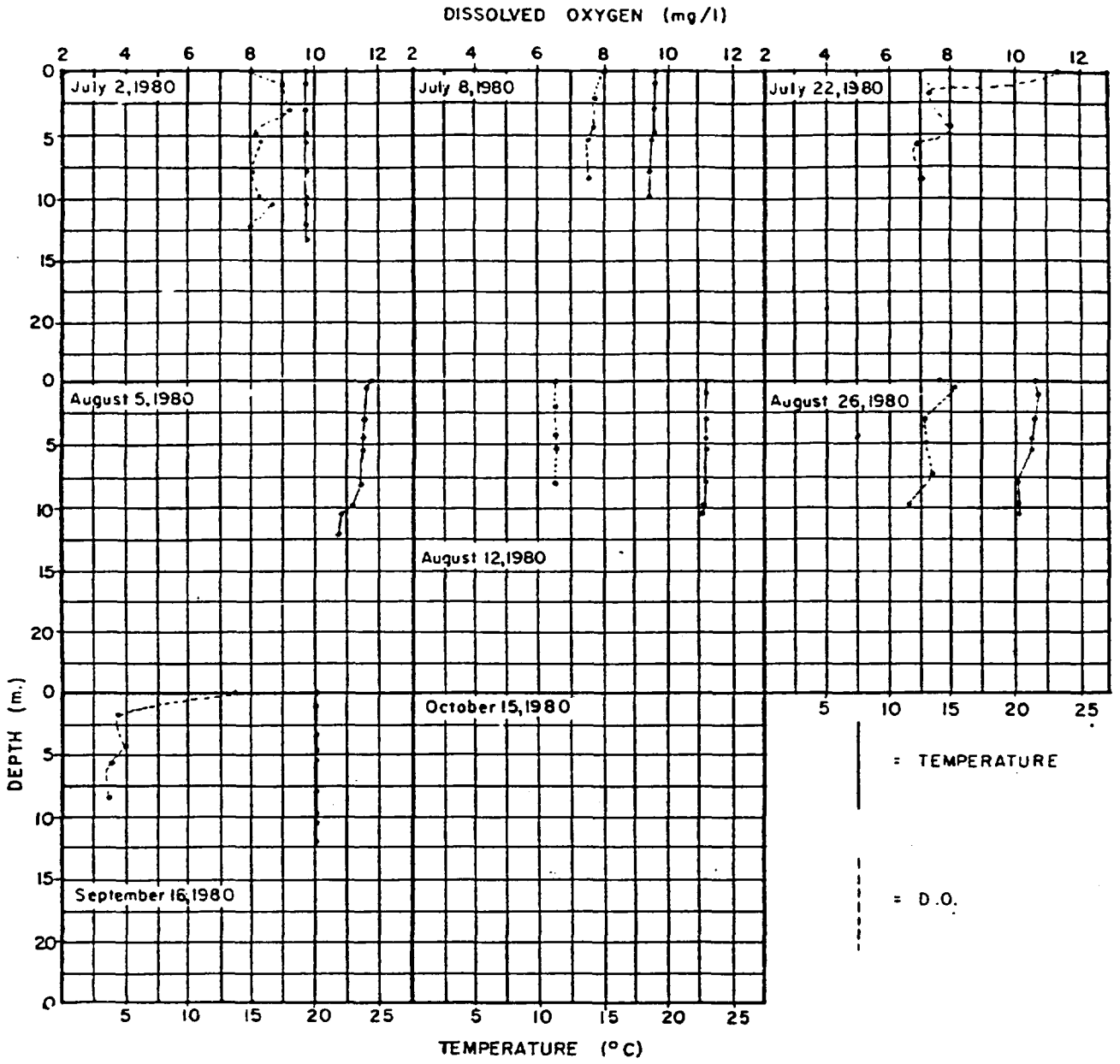
NEW BEDFORD HARBOR STUDY
DISSOLVED OXYGEN AND
TEMPERATURE PROFILES
STATION B

FIGURE 3



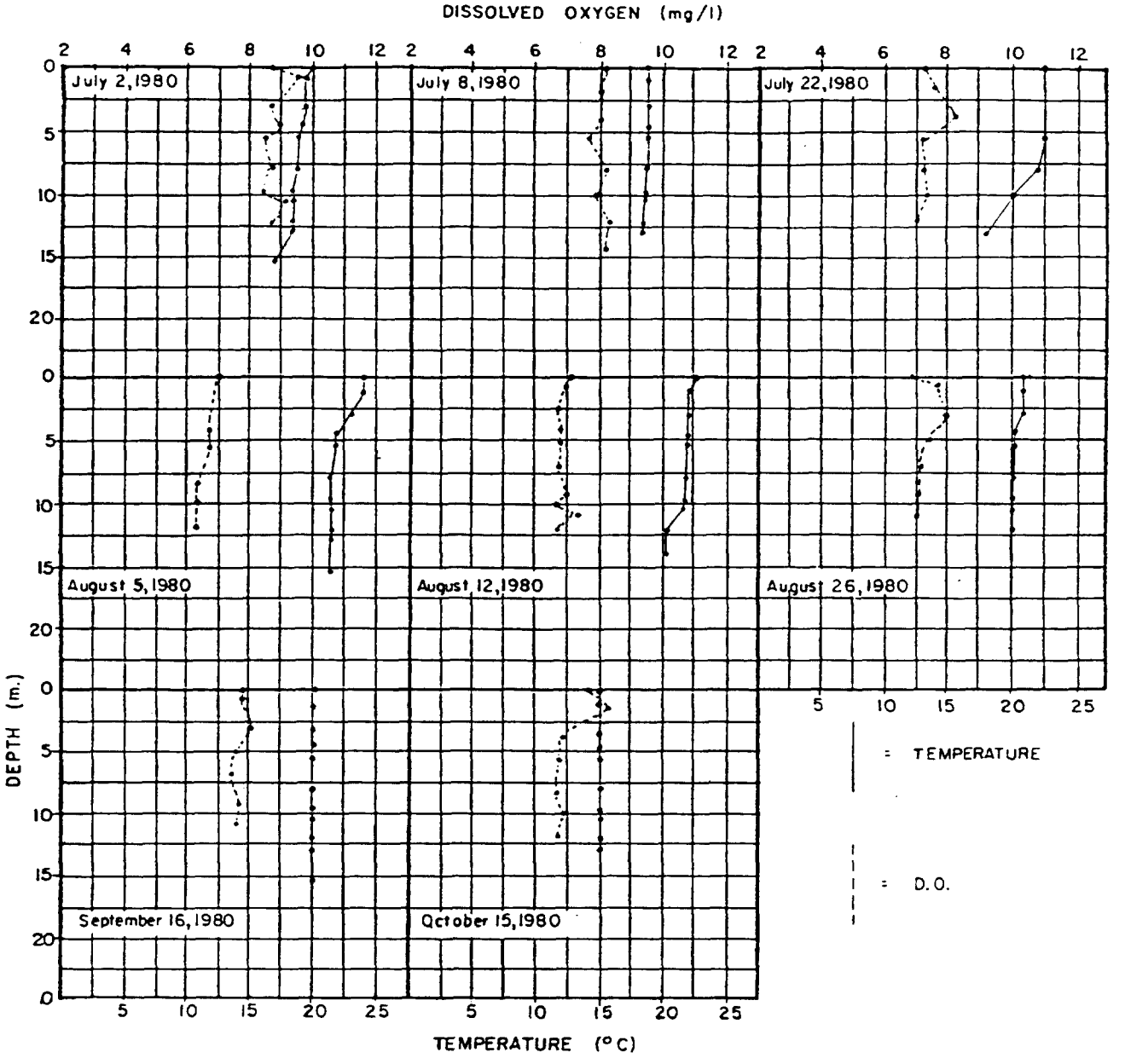
NEW BEDFORD HARBOR STUDY
DISSOLVED OXYGEN AND
TEMPERATURE PROFILES
STATION C

FIGURE 4



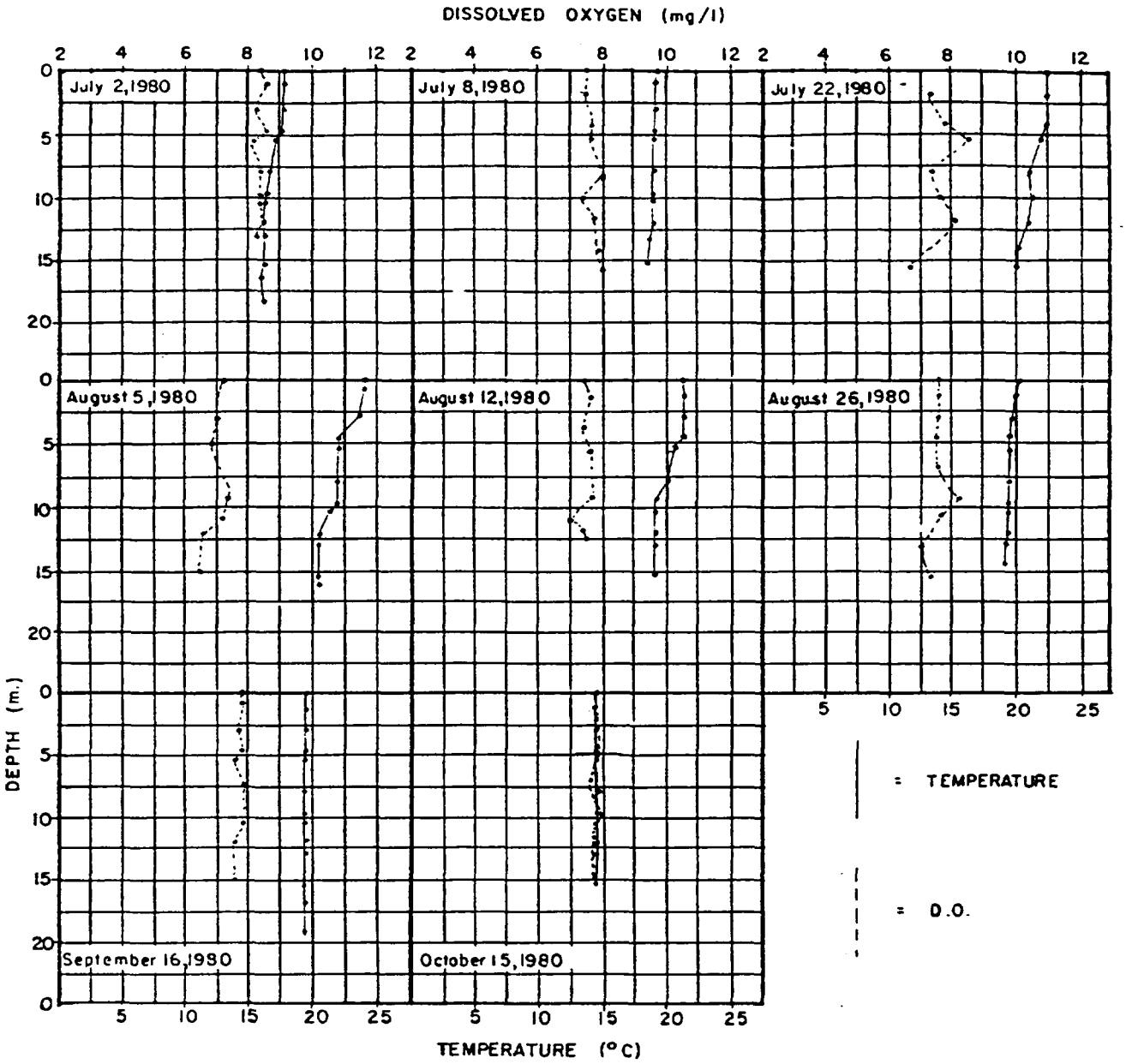
NEW BEDFORD HARBOR STUDY
DISSOLVED OXYGEN AND
TEMPERATURE PROFILES
STATION D

FIGURE 5



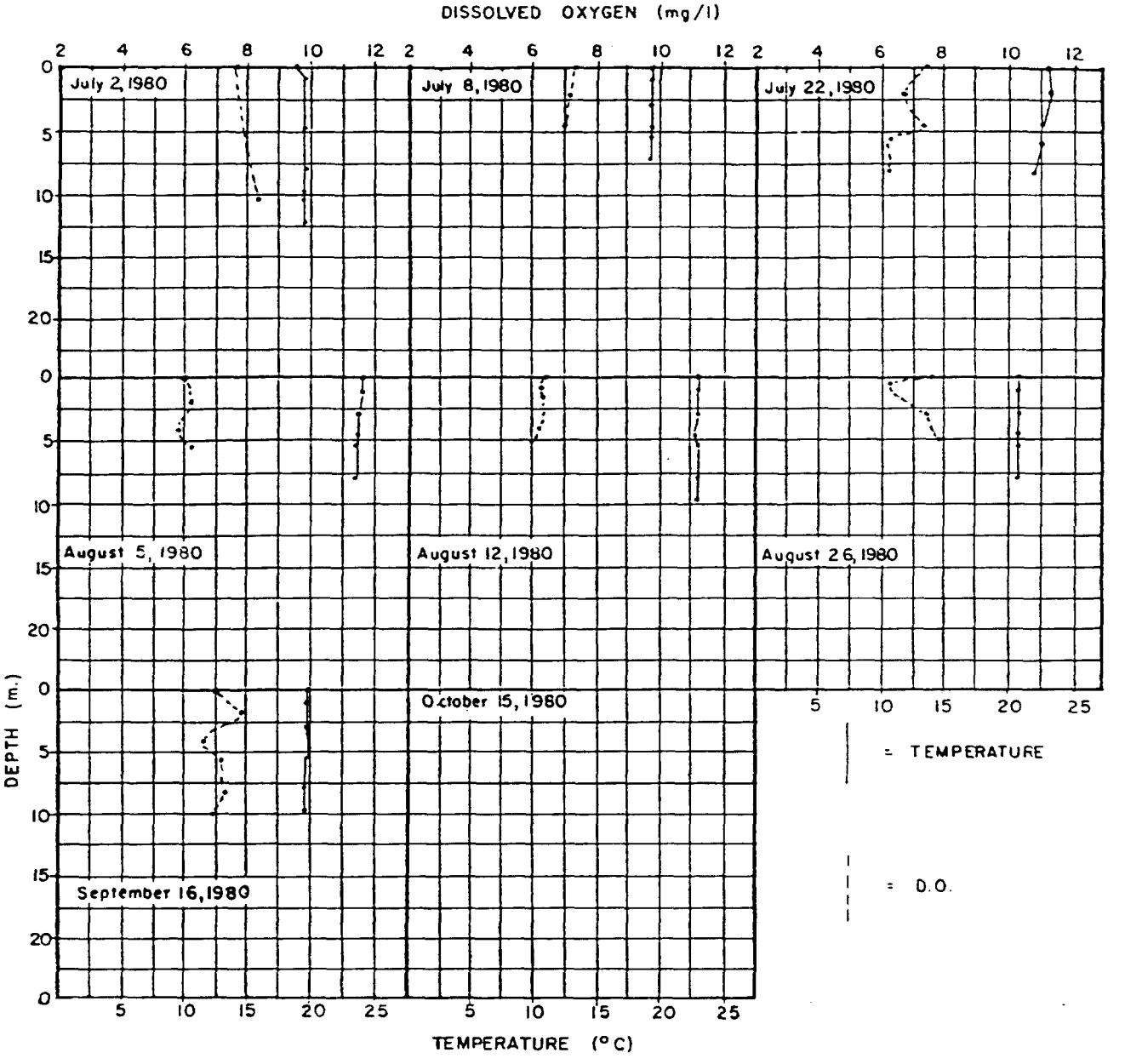
NEW BEDFORD HARBOR STUDY
DISSOLVED OXYGEN AND
TEMPERATURE PROFILES
STATION E

FIGURE 6



NEW BEDFORD HARBOR STUDY
DISSOLVED OXYGEN AND
TEMPERATURE PROFILES
STATION F

FIGURE 7



NEW BEDFORD HARBOR STUDY
DISSOLVED OXYGEN AND
TEMPERATURE PROFILES
STATION G

FIGURE 8

TABLE 17

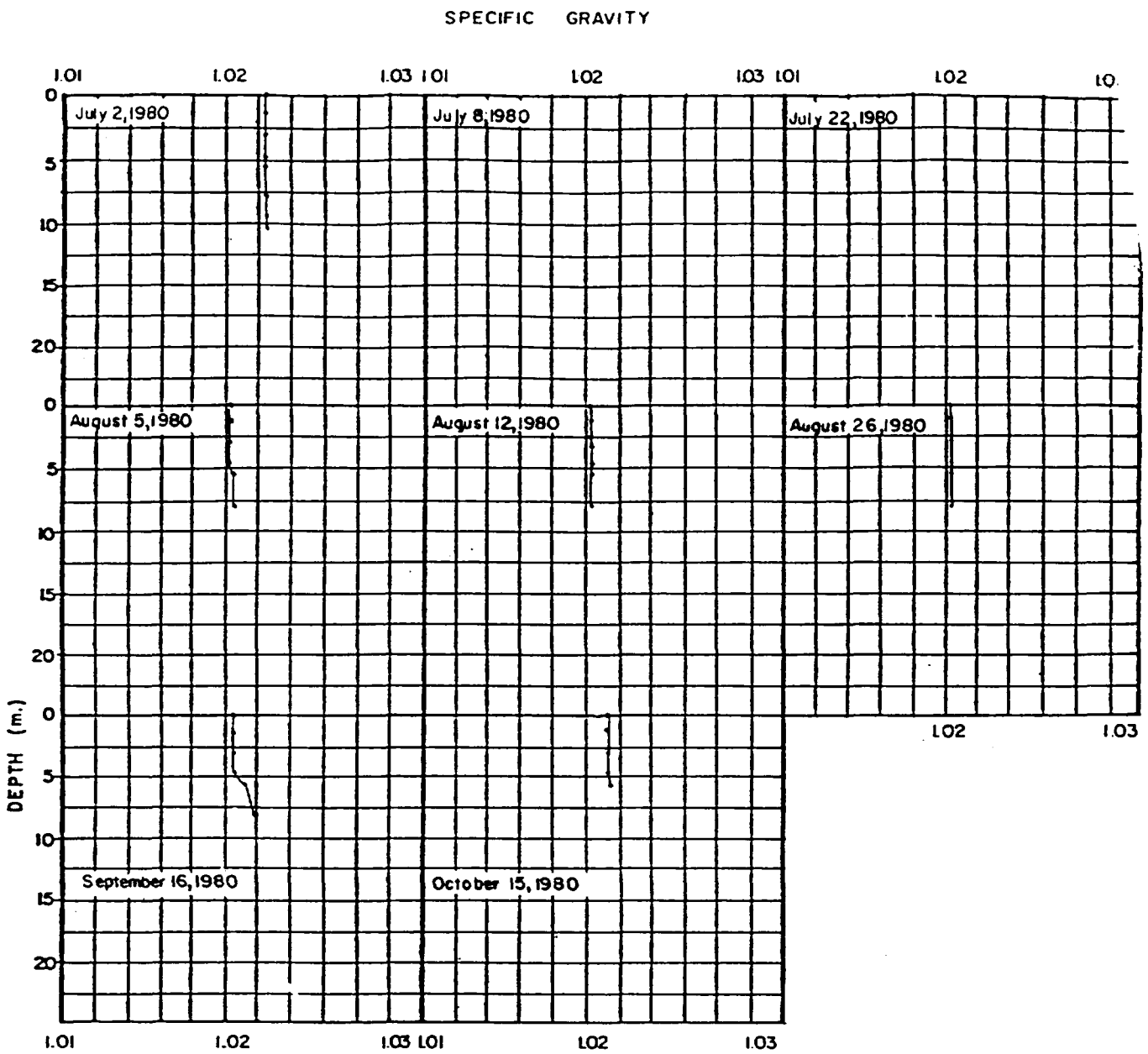
NEW BEDFORD SPECIAL WATER QUALITY STUDY

APPROXIMATE SPECIFIC GRAVITY DATA

DATE	STATION AND DEPTH (m)	APPROXIMATE SPECIFIC GRAVITY
7/2/80	A - surface	1.02250
	1.5	1.02250
	3.0	1.02250
	4.6	1.02250
	6.1	1.02250
	7.6	1.02260
	9.1	1.02260
	10.7	1.02260
8/5/80	A - surface	1.02110
	1.5	1.02110
	3.0	1.02110
	4.6	1.02110
	6.1	1.02150
	7.6	1.02170
8/12/80	A - surface	1.02100
	1.5	1.02110
	3.0	1.02110
	4.6	1.02110
	6.1	1.02110
8/26/80	A - surface	1.02190
	1.5	1.02190
	3.0	1.02200
	4.6	1.02200
	6.1	1.02220
	7.6	1.02220
9/16/80	A - surface	1.02300
	1.5	1.02300
	3.0	1.02300
	4.6	1.02300
	6.1	1.02370
	7.6	1.02400
10/15/80	A - surface	1.02360
	1.5	1.02360
	3.0	1.02360
	4.6	1.02360
	6.1	1.02370
7/2/80	B - surface	1.02240
	1.5	1.02250
	3.0	1.02250
	4.6	1.02250
	6.1	1.02250
	7.6	1.02250
	9.1	1.02250
	10.7	1.02250

TABLE 17 (CONTINUED)

DATE	STATION AND DEPTH (m)	APPROXIMATE SPECIFIC GRAVITY
7/3/80	B - surface	1.02098
	1.5	1.02250
	3.0	1.02250
	4.6	1.02250
	6.1	1.02290
8/5/80	B - surface	1.01990
	1.5	1.02110
	3.0	1.02110
	4.6	1.02120
	6.1	1.02130
8/12/80	B - surface	1.01970
	1.5	1.01990
	3.0	1.01990
	4.6	1.01990
	6.1	1.01990
	7.6	1.01990
8/26/80	B - surface	1.02020
	1.5	1.02170
	3.0	1.02170
	4.6	1.02200
	6.1	1.02200
	7.6	1.02200
9/16/80	B - surface	1.02370
	1.5	1.02370
	3.0	1.02370
	4.6	1.02370
	6.1	1.02370
	7.6	1.02370
	9.1	1.02370
10/15/80	B - surface	1.02215
	1.5	1.02365
	3.0	1.02365
	4.6	1.02365
	6.1	1.02365
	7.6	1.02365



NEW BEDFORD HARBOR STUDY
SPECIFIC GRAVITY PROFILES
STATION A

FIGURE 9