EXCELLENCE IN
ENVIRONMENTAL
CONSULTING
SERVICES

XCG File #3-587-01-85

THUNDER BAY POLLUTION PREVENTION AND CONTROL PLAN PHASE 2

JUNE 28, 1999

Submitted to:

THE CITY OF THUNDER BAY
500 DONALD STREET
THUNDER BAY, ONTARIO
P7E 5V3

Submitted by:

XCG CONSULTANTS LTD.
1 PORT STREET EAST, SUITE 210
MISSISSAUGA, ONTARIO
L5G 4N1

and

WARDROP ENGINEERING
595 SQUIER STREET
THUNDER BAY, ONTARIO
P7B 4A7

XCG Consultants Ltd.
1 Port St. E., Suite 201
Mississauga, ON
Canada
L5G 4N1
Tel: (905) 891-2400
Fax: (905) 891-2554
E-mail:
mississauga@xcg.com

TABLE OF CONTENTS

EXECUTIVE SUMMARY

TABLES

FIGURES

APPENDICES

ES-1. PROJECT OVERVIEW

The City of Thunder Bay, with participation and funding assistance from the Province of Ontario and the Federal Government through the Great Lakes 2000 Cleanup Fund, commissioned a study in late 1993 to investigate direct municipal discharges to water courses in the Thunder Bay urban service area. The purpose of this study is to develop a Pollution Prevention and Control Plan for the Thunder Bay urban service area.

The Study which is identified as the City of Thunder Bay Pollution Prevention and Control Plan (PPCP) was to be completed in two phases. The Phase 1 State-of-the-System was submitted on June 1, 1995. In the Phase 1 report an assessment of wastewater collection and treatment facilities was conducted in addition to an evaluation of area water resources. The Phase 1 work identified problem items and issues relating to collection system hydraulics, Combined Sewer Overflow (CSO) control, basement flooding, pollution prevention and wastewater treatment requirements.

Phase 1 also included consultation with the public in the form of an information booth at the annual fair in the City of Thunder Bay and a presentation to the RAP Public Advisory Committee.

Phase 2 of the study evaluates pollution prevention and control strategies. The end result is an implementation plan addressing short and long term control objectives and servicing needs of the City of Thunder Bay. In developing pollution prevention and control measures, community standards, regulatory requirements, collection system management, receiving water quality, and cost effectiveness are considered.

Since the works recommended from this study may be subject to the requirements of the Class Environmental Assessment Act for Municipal Water and Wastewater Projects, the study was carried out in accordance with the approved planning and design process contained within the Act. The Phase 1 Study Report met the requirements of the Phase 1 of the EA process while this report parallels Phase 2 of the Class EA process.

ES-2. STATE-OF-THE-SYSTEM SUMMARY

The following summarizes the key findings of the Phase 1 Report:

- Evidence of surcharging and wet weather infiltration and inflow into the sanitary trunk sewers of the North Ward was identified.
- Within the South Ward, surcharge conditions were identified in the Neebing Interceptor.

- A risk of basement flooding was identified in the South Ward.
- CSO and stormwater discharges were not shown to be significant sources of pollutant loadings in the area.
- Development areas beyond the Expressway require servicing.
- The lower reaches of the Kaministiquia River were found to support a relatively pollutant tolerant macroinvertebrate pollution.
- Boundary flows contribute the largest pollution loads to Lake Superior followed by industrial sources and the Water Pollution Control Plant (WPCP).
- The Thunder Bay WPCP has not historically provided adequate disinfection to meet the MOEE draft disinfection policy.
- Due to the physical condition of the old WPCP plant, it will need to be replaced to meet system requirements to the year 2016.
- The Thunder Bay WPCP may be required to proceed to secondary plant upgrading in the near future in order to meet general MOEE compliance requirements for phosphorus.

Following the completion of the Phase 1 report, two investigations were conducted. Stormwater outfall sampling in Boulevard Lake determined that urban sources were not a significant source of bacteria to Boulevard Lake and could not account for the bacteria levels observed in the Lake. In addition, inspections of the Neebing/McIntyre Interceptor revealed a significantly deflected pipe section with the potential for collapse. The City is currently considering options for the repair/rehabilitation of this section and has developed an emergency response plan.

It was also reported that an oil and grease separator upstream of the Ridgeway CSO would be flushed out during wet weather flow conditions, resulting in discharges of petroleum products into the Kaministiquia River.

ES-3. PPCP DEVELOPMENT

The PPCP consists of two components; the Short Term PPCP and the Long Term PPCP culminating in an Implementation Plan.

The development of a Short Term PPCP addresses the immediate operational concerns identified in the Phase 1 State-of-the-System. Typically, the elements of a Short Term Plan include low cost alternatives that are relatively simple to implement over a five to ten year period. As well, the Short Term Plan provides a foundation for Long Term PPCP measures.

The Long Term Plan has a planning horizon of 20 to 25 years. When developing the Long Term Plan, it must be responsive to improvements realized through the Short Term Plan initiatives. The Long Term Plan must also consider ultimate development conditions in the Thunder Bay area to ensure development can be sustained.

ES-4. SHORT TERM PPCP

The Short Term PPCP consists of low cost programs designed to be implemented over a five to 10 year period with programs forming the foundation for long term control strategies. Key elements of the Short Term PPCP address the following areas:

- Collection System Management
- CSO Control
- Basement Flooding
- Stormwater Management
- Thunder Bay WPCP
- Pollution Prevention

The recommended Short Term PPCP and costs are presented in Table ES.1 at the end of the Executive Summary. The following sub-sections discuss key elements of the Short Term PPCP.

ES-4.1 Collection System Management

The management of any collection system is extremely important as a means to make informed operational and maintenance decisions while providing a reliable service. To this end, a complete review of City Operation and Maintenance practices was conducted to identify program enhancements as well as to identify new initiatives. The goal of the review was to improve the City's ability to provide reliable services while protecting the receiving waters.

The following enhancements to current City programs are recommended (a complete summary is presented in Table ES.1 at the end of this section):

- CCTV inspection of the entire collection system over the next 10 years, including manhole inventory and inspection
- Expansion of the sewer flushing program as part of CCTV inspection
- Outfall survey and sampling program
- A 5-year flow monitoring program

Another component of collection system management is to maximize the use of existing facilities. To this end, recommendations are made for two flow diversions to provide hydraulic relief to the Neebing Interceptor, construction of a new sanitary connection for the James and Quebec Street area to eliminate the sanitary connection to the local storm sewer and extraneous flow reduction programs associated with catchbasins and river intrusion through outfalls.

As well, repairs to the Neebing/McIntyre Interceptor is recommended for immediate implementation to prevent a structural failure that could result in significant basement flooding through loss of capacity.

ES-4.2 CSO Control

As part of the CSO control program, both the physical condition and the performance of the existing regulators were considered. Basin wide, the level of CSO control is greater than 97% volumetric control exceeding the minimum MOEE CSO guideline control objective of 90% volumetric control. Only one CSO regulator, RK2 (Hardisty/Victoria), does not meet the 90% control level. Two recommendations were made with respect to CSO control. The first is to adjust regulator RK2 to increase its interception capacity to achieve a 90% level of control. The second recommendation is to replace, over time, the regulating devices servicing the Kaministiquia Interceptor.

Another component of CSO control considered is the control of floatables. Floatables control can be achieved by retrofitting existing CSO chambers with baffle plates. Removal of floatables will improve the aesthetics of the receiving water by retaining debris in the collection system.

Another element of CSO control addressed intrusion of the Neebing River through CSO chambers into the collection system when outfall gates remain open due to debris. River intrusion can effect the Neebing Interceptor capacity, potentially aggravating the risk of basement flooding. Continued inspection of outfalls following wet weather events, currently done by City staff, is essential and consideration should be given to replacing the outfall gates with a "duck bill" style of outfall as replacement is required.

ES-4.3 Basement Flooding

A risk assessment and sensitivity analysis was conducted to determine the risk of flooding in the South Ward. As part of the Short Term PPCP, it is recommended that the City and community define an acceptable level of risk, commence flow monitoring and CCTV inspection programs in identified high risk areas, and update the hydraulic analysis and risk assessment with local data. In the event the risk of basement flooding exceeds the community standard, sewer separation should be undertaken to reduce/eliminate the risk in areas not previously separated. In areas that have been separated, system storage in the form of an oversized in-line storage pipe or a tank storage configuration is recommended. As the City undertakes other elements of the Short Term PPCP, such as public education on rain leader disconnection, it is likely the risk of basement flooding will be more clearly understood and a reduction in the risk of basement flooding realized.

ES-4.4 Stormwater Management

Continued enforcement and application of the MOEE "Stormwater Management Practices Planning and Design Manual", June 1994 is important with respect to new developments, construction of new storm outfalls and in retrofitting existing outfalls. Beyond the application of these guidelines, no end-of-pipe stormwater controls are recommended.

ES-4.5 Thunder Bay WPCP

The City of Thunder Bay will be undertaking a pilot study to determine the most appropriate treatment technology. The technologies recommended for pilot testing include BAF (Biological Aerated Filters) and an optimized CAS (Conventional Activated Sludge) design. As a result of discussions with City staff in Windsor, Ontario, the City has decided to include in the pilot test the trickling filter/solids contact process.

The results from this study will be used to identify the most appropriate site specific design parameters for each technology resulting in the most cost effective and appropriate technology. As well, the pilot study will provide capital and operating costs and experience, provide a comparison of performance and support an application to MOEE for approval of any non-standard treatment technology or process design. As indicated in the Technical Memorandum (Appendix B) undertaking a pilot study to refine the design parameters, treatment technology and processes could result in savings in the neighbourhood of $4 to $6 million in capital costs.

It is also recommended that the City continue its efforts to improve phosphorus removal at the existing facility and address the suspected mixing limitations in the existing anaerobic digesters.

ES-4.6 Pollution Prevention

The objective of pollution prevention measures is to minimize the accumulation of pollutants on streets and other tributary land areas as well as to reduce the entry of pollutants into the collection systems. Typical pollution prevention measures can include, but are not limited to the following:

- Street cleaning
- Public education programs
- Recycle programs
- Fertilizer and pesticide control
- Soil erosion control
- Commercial/Industrial control
- Operation and maintenance practices
- Catchbasin Cleaning

Two recommendations are made with respect to the existing pollution prevention programs of the City. It is recommended that the City clean all catchbasins at least once per year.

The City undertakes public awareness programs to promote good practices. One area of public education for the City to address is the removal of roof downspouts. A bylaw exists, however, the City has not actively enforced downspout disconnection. Information packages for the public can lead to voluntary disconnection. The removal of rain leaders from existing developments will provide additional wet weather sewer capacity and assist in reducing the peak flows in combined and partially separated sewer systems in the South Ward.

ES-5. LONG TERM PPCP

An extension of the Short Term PPCP, the Long Term PPCP programs will ensure sustainable development with a level of collection system performance, wastewater treatment and stormwater control acceptable to the community and regulatory agencies.

The key elements of the Long Term PPCP address the same areas of concern as the Short Term PPCP, namely:

- Collection System Management
- CSO Control
- Basement Flooding
- Stormwater Management
- Thunder Bay WPCP

Table ES.2, at the end of the Executive Summary, presents the recommended Long Term PPCP programs. The following sub-sections discuss key elements of the Long Term PPCP.

ES-5.1 Collection System Management

Collection system management in the context of the Long Term PPCP is associated with the control and management of excess wet weather flows, the need for development capacity, and the need to ensure a reliable level of service.

Servicing the North Ward with the Golf Links Extension was assessed, identifying a preferred alternative that has the Golf Links Extension intercepting the River Terrace pump station and continuing through to the John Street Trunk sewer at Maple Avenue. This alternative will provide hydraulic relief for the John Street Trunk sewer. Another long term initiative is the continuation of the short term program to eliminate storm catchbasin connections in the North Ward to the sanitary sewer through the construction of new storm sewer services and connection to existing storm sewers.

ES-5.2 CSO Control

No Long Term PPCP CSO controls were found to be necessary for the City of Thunder Bay. Presently, the City has a level of CSO control higher than 90% volumetric control basin wide, the minimum control level specified in the MOEE CSO Guidelines.

ES-5.3 Basement Flooding

As part of the Short Term PPCP, the reduction or elimination of basement flooding in the combined and partially separated areas of the South Ward are addressed. As well, system improvements proposed as part of the Golf Links Trunk sewer extension to intercept a portion to John Street will provide hydraulic relief in the John Street Trunk sewer. The interception of flows above Maple Avenue from the John Street Trunk sewer, including future development flows, will reduce the possibility of basement flooding in the John and High Street area.

ES-5.4 Stormwater Control

The stream and loadings analysis undertaken in Phase 1 showed no clear evidence that stormwater represents a significant source of pollutants annually or on an event basis. The Short Term PPCP recommends continued enforcement of the provincial stormwater guidelines; there is no change for the Long Term PPCP. The Long Term PPCP does not contain any projects associated with the control or treatment of stormwater.

ES-5.5 Thunder Bay WPCP

The Short Term PPCP recommends a pilot study be undertaken to identify the most cost- effective secondary treatment technology. The City of Thunder Bay is proceeding with the pilot study program. The outcome of the program will be recommendations on the secondary treatment technology. From this recommendation the City is committed to proceed to pre-engineering, final design and construction. It is anticipated that following the final pilot study recommendation full secondary treatment will be implemented within 5 years. The preliminary cost estimates for secondary treatment are in the range of $26 million to $34 million; the costs of the upgrade will be refined with the pilot study results and completion of the pre-engineering.

As part of the WPCP upgrade, the improvements to the Kaministiquia Interceptor are recommended to improve the hydraulic capacity and simplify system operations by eliminating the WPCP old pumping station.

ES-6. IMPLEMENTATION PLAN

The proposed Implementation Plan is designed to prioritize projects to achieve the objectives of pollution control planning in meeting community standards; CSO control and stormwater guidelines; WPCP effluent requirements; Provincial Water Quality Objectives; and the objectives of RAP and the Binational program.

The Implementation Plan presented provides the initial framework to implement the PPCP programs recommended. The Plan will change and evolve through the implementation period and should be considered a living document to be revisited and revised as more and better information becomes available. The long term goals of the plan are to accomplish the following:

- Reduce urban pollutant loadings to receiving waters and to protect water resources
- Ensure reliable services
- Reduce/eliminate basement flooding
- Provide services for future developments
- To provide secondary treatment

Table ES.3 presents a prioritized list of programs in the Implementation Plan where 1 is the highest and 5 is the lowest.

Figure ES.1 presents the Implementation Plan schedule and cash flow information. The implementation period is considered to be 20 years corresponding to the planning period. The implementation period for projects associated with new services for future development may occur beyond the 20 year planning period.

The cost of each program and project has been distributed to develop a cash flow projection for the City. For some programs no new dollars are identified indicating that program funding should be from existing operational budgets. Funding for projects associated with the reduction or elimination of basement flooding has been distributed uniformly across a ten year period. A specific distribution can not be determined until specific projects have been identified through the setting of a community standard and refinement of analysis with local flow data. The costs associated with the McVicar's Creek storage facility has not been shown in the Implementation Plan. It is anticipated that the storage required beyond the initial 1,000 m^3 could be funded through development charges. Alternatively, development in this area could be limited to the existing service capacity available.

Figure ES.2a and ES.2b present the cash flow requirements for the Implementation Plan. Figure ES.2b does not include the WPCP upgrade to secondary treatment.

The Thunder Bay PPCP was carried out in accordance with the approved planning and design process contained within the Class Environmental Assessment Act for Municipal and Water and Wastewater Projects. The recommended works outlined in the Implementation Plan can be categorized as Schedule A projects requiring no public notification. The only exception to the Schedule A is likely the Golf Links Extension (Item 19) and storage at McVicar's Creek (Items 22 and 28), which would fall under Schedule B type projects requiring suitable public notification on two occasions. Early in the evaluation of the Thunder Bay WPCP, confirmation from the EA Branch was received identifying that the change to secondary treatment would be classified as a Schedule A project, given there is no change in the plant's rated capacity.

Table ES.1 Recommended Short Term PPCP and Costs

Program	Program Description	Qualitative Benefits	Cost
Collection System Management			
Operation and Maintenance			
CCTV Inspection	• Inspect and inventory collection systems over 10 year period. • Include sewer flushing • Incorporate manhole inspection and inventory • Re-inspect problem sections on a 5 year cycle until rehabilitated	• Condition assessment and information • Identification of structural defects • Assist in prioritizing rehabilitation programs • Identification of extraneous flow sources • Increased pipe capacity with cleaning • Establishes an ongoing sewer and manhole inspection program for the City • I/I reduction creates more system capacity and reduces treatment needs.	\$2.8 million over 10 years \$1.1 million years 1-5 \$1.7 million years 5-10
Manhole Inspection	• Inspect and inventory all manholes in the City • Combine with CCTV program	• Provides structural condition information • Identify extraneous flow sources • Prioritize rehabilitation projects • I/I reduction creates more system capacity and reduces treatment needs	\$0 • included in CCTV inspection program
CSO Inspection & Maintenance	• No change recommended to existing programs	• Reduce likelihood of equipment failure	\$0 • ongoing program
Storm Sewer Outfall Inspection & Maintenance	• No change to existing spring programs of inspection and maintenance • Conduct outfall survey to locate and document outfalls • Identify outfalls with dry weather seepage and estimate flow rate. • Collect dry weather seepage water quality sample for conventional and metals analysis • Inspect problem flap gates after every rainfall event	• Quantifies dry weather seepage/ extraneous flow rates • Sampling program could identify cross-connections or other pollutant sources. • Assist in the enforcement of the Sewer Use Control Bylaw • Reduce river intrusions	\$0 • Conduct with existing staff • Cost share sampling program with MOEE and Lakehead Conservation Authority
Sewer Flushing	• Coordinate existing program with CCTV inspection • Expand to 100% average in South Ward	• Increased pipe capacity • Sewer maintenance	\$0 • Included with CCTV inspection program
Pump Station Maintenance	• No changes to existing programs of inspection and maintenance	• Reduced likelihood of equipment failure	\$0 • Ongoing program
Structural			
Neebing/McIntyre Improvement	• No recommendation, study pending	• Structural stability • Reliable service	\$0 • Study pending

Table ES.1 Recommended Short Term PPCP and Costs

Program	Program Description	Qualitative Benefits	Cost
Maximize Use of Existing Facilities			
1. Diversions	• Divert excess wet weather flows from the Neebing Interceptor to the Brunswick Connector sewer and Neebing/McIntyre Interceptor • Divert flow from Neebing Interceptor to Cameron Trunk	• Provides much needed hydraulic relief to the Neebing Interceptor	• $21,000 Neebing/Brunswick • $22,000 Neebing/Cameron
2 Extraneous Flow Reduction	• Enforce By-law to remove rainwater leaders from sanitary sewers • Unable to develop or initiate a program without system inventory	•	• -
3. Catchbasin Cross-Connection	• Seal, restrict and provide new catchbasin leads to limit wet weather inflow to sanitary system • New storm sewer to disconnect 3 catchbasins	• Reduces wet weather load in sanitary sewer • Reduces treatment needs • Foundation for Long Term removal plan	• $12,000 CB sealing & flow restriction • $120,000 New storm sewer
4. Outfall Flap Gate Replacement	• Replace outfall flap gates as required with "duck bill" design • Ensure existing gate seals are in good condition	• Reduced inflow • Increase in available pipe capacity and reduction in treatment needs • Less operational and maintenance required for "duck bill"	$300,000 • Replacement of 8 outfall gates
5. James & Quebec Connection	• Construct new sanitary connector to existing sanitary system	• Removal of direct sanitary connection to storm sewer and outfall to Kaministiquia River	$93,000
Monitoring Program	• Initiate 5 year flow monitoring program • Establish at least 8 permanent monitoring stations in essential interceptors sewers • Purchase two velocity-area meters and one rain gauge	• Additional model calibration data • Qualify extraneous flow • On-line collection system information that can be used to develop operational strategies	$115,000
XP-SWMM Model	• Update model calibration with current flow data • Update model network with inspection records • Refine analysis to assess PPCP status • Expand model into local areas	• Improved information related to system hydraulic performance • May reduce the works identified through the use of better information • Ability to assess changing conditions beyond the PPCP study	$0
		CSO Control	
Ridgeway Oil/Grease Separator	• Replace existing Oil/Grease separator with a larger unit, or • Provide bypass of peak flows to prevent flushing	• Improved Oil/Grease capture • Reduction in contaminated discharges	$25,000 to $30,000
Floatables Control	• Identify sources of floatables • Retrofit CSO chambers with baffle plate for floatables control • Retrofit if floatables identified	• Reduced floatables will improve aesthetics • Source identification	$0 • No cost identified

Table ES.1 Recommended Short Term PPCP and Costs

Program	Program Description	Qualitative Benefits	Cost
CSO Regulator Replacement Program	• Replace Kaministiquia regulators with either a vortex or Hydroslide type device as required	• More reliable performance • Low cost • Reduced maintenance	$175,000 • Replaces 11 regulators $15,000 • Replaces RK2 regulators
Regulator Settings	• Adjust RK2 regulator to increase interception rate	• Achieve minimum 90% volumetric control	$0
Basement Flooding			
South Ward Basement Flooding	• Update risk assessment with local flow data and improved model calibration • Determine community standard • Replace existing combined pipes with separate pipes to achieve a desired level of control • Provide system storage in previously separated areas	• Eliminate/minimize the risk of basement flooding	$340,000 to $4.3 million over a 10 year period (2 year to 10 year level of risk)
Stormwater Control			
Stormwater Management	• Continue enforcement and application of "Stormwater Management Practices Planning and Design Manual"	• Improved stormwater quality and quantity control	-
Thunder Bay WPCP			
Pilot Study	• Initiate year long pilot study investigating treatment technologies for secondary upgrade to WPCP	• Significant savings in capital cost of secondary facility • Design parameters suited to Thunder Bay • Trained staff familiar with secondary process and operations	$300,000 to $400,000
Phosphorus Removal	• Continue with existing optimization efforts	• Improved phosphorus removal to meet effluent requirements	$0
Digester Optimization	• Improve digester mixing	• With proper mixing digester volume will be sufficient for full secondary facility	$0
Pollution Prevention			
Street Cleaning	• No change to existing program	• Removal of pollutant	$0
Catchbasin Cleaning	• Increase scope of program to 100% coverage	• Removal of pollutants before they enter storm sewer system	• Increase annual operating budget
Public Education	• No changes to existing programs • Promote downspout disconnection • Co-ordinate efforts with RAP, MNR etc. • Promote "good practices"	• Informed public • Reduce demand for water and wastewater treatment capacity	$0
Total Cost			**$9.3 million**

Table ES.2 Recommended Long Term PPCP and Costs

Program	Program Description	Benefits	Costs
Golf Links Extension	• Extend Golf Links through River Terrace pump station over to John Street Trunk sewer at Algonquin Avenue • Replace John Street Sewer between Algonquin Avenue and the Expressway	• Provides future capacity to developments beyond the Expressway • Diverts existing flow from the John Street Trunk sewer providing hydraulic relief in the upper portion	$3.3 million $1.0 million
McVicar's Creek Storage	• 8,760 m^3 of storage required for ultimate development • 1,000 m^3 storage for developments up to 2010 • McVicar's Creek Trunk sewer to receive a maximum of 2.5 average DWF • Detention time 12 hours • Storage can be staged with development	• Storage can be staged • Provides control over flows into the McVicar's Creek Trunk • More cost effective than extending the Golf Links • Cost of storage borne by developer	$5.4 million (total) $615,000 (1,000$m^3$) $0 (city)
John Street Trunk Sewer Improvement	• Twin 400 m section of sewer between Algoma and Ontario Streets	• Provides hydraulic relief in local area • Reduces hydraulic grade line and the risk of basement flooding	$740,000
North Ward Catchbasin Disconnection Program	• Construct new storm sewers to existing outlets • Disconnect catchbasins from sanitary and reconnect to new storm sewer	• Removes storm flow from sanitary sewers • Reduces wet weather response in sanitary system	$775,000
Thunder Bay WPCP Upgrade	• No recommendation • Pilot study pending	• Reduction in loadings to Lake Superior • Meet Regulatory requirements	$35 million
Kaministiquia Interceptor Improvements	• Replace 215 m of 750 mm with 1,670 mm pipe between old pump station regulator and the Main pump station • Part of WPCP upgrade	• Reduces the need for the old pump station • Improved hydraulic and simplified operations	$1.4 million
Total Cost			**$42.2 million**

Table ES.3 Program Priorities

Item	Program	Cost	Priority	Implementation Period
1.	CCTV, manhole inspection and sewer flushing program	$2.8 million	1	10 years • becomes ongoing
2.	CSO Inspection and Maintenance	existing budgets	1	ongoing
3.	Pump Station Maintenance	existing budgets	1	ongoing
4.	Neebing/Brunswick Diversion	$21,000	1	3 years
5.	Neebing/Cameron Diversion	$22,000	1	5 years
6.	North Ward Catchbasin Sealing	$12,000	1	2 years
7.	James and Quebec Connection Correction	$93,000	1	2 years
8.	Monitoring Program	$115,000	1	5 years • becomes ongoing
9.	RK2 Regulator Replacement and Adjustment	$15,000	1	2 years
10.	South Ward Basement Flooding Program	$340,000 to $4.3 million	1	10 years
11.	Stormwater Management Controls	existing budgets or developers	1	ongoing
12.	Thunder Bay WPCP Pilot Study	$300,000 to $400,000	1	initiated - 2 years
13.	Phosphorus Removal	existing budgets	1	ongoing
14.	Digester Optimization	existing budgets	1	ongoing
15.	Catchbasin Cleaning – 100% coverage annually	increase existing budget	1	ongoing
16.	Pollution Prevention Programs (street cleaning, public education)	existing budgets	1	ongoing
17.	Neebing/McIntyre Improvements	existing budgets	1	1 year
18.	Storm Sewer Outfall Inspection, Maintenance and Survey	existing budgets	2	7 years • becomes ongoing
19.	Golf Links Extension to River Terrace P.S.	$3.3 million	2	10 years
20.	Thunder Bay WPCP Upgrade to Secondary Treatment	$25 to $35 million pending Pilot Study recommendations	2	5 years
21.	North Ward Storm Sewer and Catchbasin Disconnection	$120,000	3	7 years
22.	Initial 1,000 m^3 storage @ McVicar's	$0 (developer pay)	3	7 years
23.	John Street Trunk Sewer Improvement	$740,000	3	7 years
24.	Kaministiquia Interceptor Improvements	$1.4 million	3	5 years

Table ES.3 Program Priorities

Item	Program	Cost	Priority	Implementation Period
25.	CSO Regulator Replacement Program	$175,000	4	15 years
26.	Golf Links Extension to Algonquin Avenue and Upgrade of John Street Trunk to Expressway.	$1.0 million	4	15 years
27.	Outfall Flap Gate Replacement Program	$300,000	5	25 years
28.	McVicar's Storage - 8,760 m^3 (only 7,760 m^3 required @ $4.7 million if the initial 1,000 m^3 installed)	$5.4 million	5	25+ years
29.	North Ward Catchbasin Disconnection Program	$775,000	5	25+ years

Figure ES-1
Implementation Plan

ITEM	PROGRAM	COMMENTS	COSTS	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	BEYOND
1.	CCTV, Sewerline, Manhole and Sewer Flushing Program	- Program began in 1996 - Initial 10 year program - Beyond 10 years new CCTV program required	$2,800,000	120,000	245,000	245,000	245,000	245,000	340,000	340,000	340,000	340,000	340,000	- Inspection program is ongoing, however, the annual level of effort is reduced											
2.	CSO Inspection & Maintenance	- Ongoing program - No new resources required	$0	- Ongoing program, existing budget																					
3.	Pump Station Maintenance	- Ongoing program - No new resources required	$0	- Ongoing program, existing budget																					
4.	Neebing/Brunswick Diversion	- Provides hydraulic relief and control to the Neebing Interceptor reducing the likelihood of surcharging conditions	$21,000			21,000																			
5.	Neebing/Cameron Diversion	- High level relief of the Neebing Interceptor	$22,000				22,000																		
6.	North Ward Catchbasin Sealing	- Cost effective way to disconnect CB - Reduce inflow into North Ward sanitary system	$12,000		12,000																				
7.	James and Quebec Connection Correction	- Removal of cross connection	$93,000			93,000																			
8.	Monitoring Program	- Monitoring program will provide additional model calibration data and increase of flows in the collection system - 2 meters and 1 rain gauge - 10 permanent stations	$115,000		12,000	10,000			18,600	18,600	18,600	18,600	18,600	- Ongoing program											
9.	RK2 Regulator Replacement and Adjustment	- RK2 to be replaced and adjusted in the short term - Provides City information on new regulator technology	$15,000			15,000																			
10.	South Ward Basement Flooding Program	- Program may not need to be fully implemented - Flow monitoring and system modelling should be used to re-assess need - The level of risk assumed will change the costs - No cost identified in cash flow	$4,300,000	430,000	430,000	430,000	430,000	430,000	430,000	430,000	430,000	430,000	430,000												
11.	Stormwater Management Controls	- Follow Provincial Guidelines - Ongoing	$0	- Ongoing program, existing budget																					
12.	Thunder Bay WPCP Pilot Study	- Study to be initiated in 1996 - $300,000 to $ 400,000 depending on final scope	$400,000	100,000	200,000	100,000																			
13.	Phosphorus Removal Program	- Ongoing program	$0	- Ongoing program, existing budget																					
14.	Digester Optimization	- Ongoing program	$0	- Ongoing program, existing budget																					
15.	Catchbasin Cleaning 100% Coverage	- South Ward has 50 to 60% coverage this is to be increased to 100% annually	$0	- Ongoing program, existing budget																					

Thunder Bay PPCP
6/22/99
Implementation Plan ES-1.xls

Figure ES-1
Implementation Plan

ITEM / PROGRAM	COMMENTS	COSTS	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	BEYOND
16. Pollution Prevention Programs	- Ongoing initiatives - Promote Roof Leader Disconnection	$0	- Ongoing program, existing budget																					
17. Neebing/McIntyre Interceptor Improvements	- Study pending - Requires immediate action	$0		- Pending																				
18. Storm Sewer Outfall Inspection, Maintenance and Survey	- Investigate cost sharing with other agencies - Outfall survey to be repeated on a 7 year cycle	$0		- Existing budget							- Program designed on a 7 year cycle													
19. Golf Links Extension to River Terrace P.S.	- Alignment is not set - It is assumed the extension will be phased in over 7 years - Development driven	$3,300,000	330,000	330,000	330,000	330,000	330,000	330,000	330,000	330,000	330,000	330,000												
20. Thunder Bay WPCP Upgrade to Secondary Treatment	- City commited to provide full secondary treatment within 5 to 10 years	$35,000,000				7,000,000	7,000,000	7,000,000	7,000,000	7,000,000														
21 North Ward Storm Sewer and Catchbasin Disconnection	- New storm sewer will allow 3 CBs to be disconnected reducing wet weather inflow.	$120,000						120,000																
22. McVicar's Creek 1,000 m3 Storage	- Initial storage volume required - Investigate cost sharing to fund	$615,000					615,000																	
23. John Street Trunk Sewer Improvements	- Provides hydraulic relief in High St. area - Implementation withi 5 years	$740,000							740,000															
24. Kaministiquia Interceptor improvements	- Implement inconjunction with WPCP upgrade to secondary treatment	$1,400,000					1,400,000																	
25. CSO Regulator Replacement Program	- Replacement program over 15 years to replace 10 regulators on the KAM Interceptor - Assumed that one or two regulators addressed each year of the program	$175,000					15,900	15,900	15,900	15,900	15,900	15,900	15,900	15,900	15,900	15,900	15,900							
26 Golf Links Extension to Algonquin Ave. and upgrade of John St. Trunk to Expressway	- Requires Item 20 to be completed - Development driven	$1,000,000											250,000	250,000	250,000	250,000								
27. Outfall Flap Gate Replacement Program	- Program to be implemented on an "as need" basis - costs are distributed - It is assumed all outfall gates will need to be replaced over the next 20 year period	$300,000					8,000 - RN21		20,000 - RN25, RN28, RN33			15,000 - RN24	13,000 - RN27				122,000 - RN20				122,000 - RN32			
28. McVicar's Creek 8,760 m3 Storage	- No cost identified to the City - Need for storage is development driven - Cost recovered in development charges - Approximately $4.7 million	$0																- Additional storage would be required for future developments						
29. North Ward Catchbasin Disconnection Program	- Program may not need to be fully implemented - Flow monitoring and system modelling should be used to re-assess need	$775,000																77,500	77,500	77,500	77,500	77,500	77,500	310,000
		TOTAL	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	BEYOND
CASH FLOW (x1,000 and rounded)		$51,200	$1,000	$1,200	$1,200	$8,000	$10,000	$8,300	$8,900	$8,100	$1,100	$1,100	$300	$300	$300	$300	$100	$100	$100	$100	$200	$100	$100	$300

Figure ES-2a
Cash Flow - With WPCP Upgrade

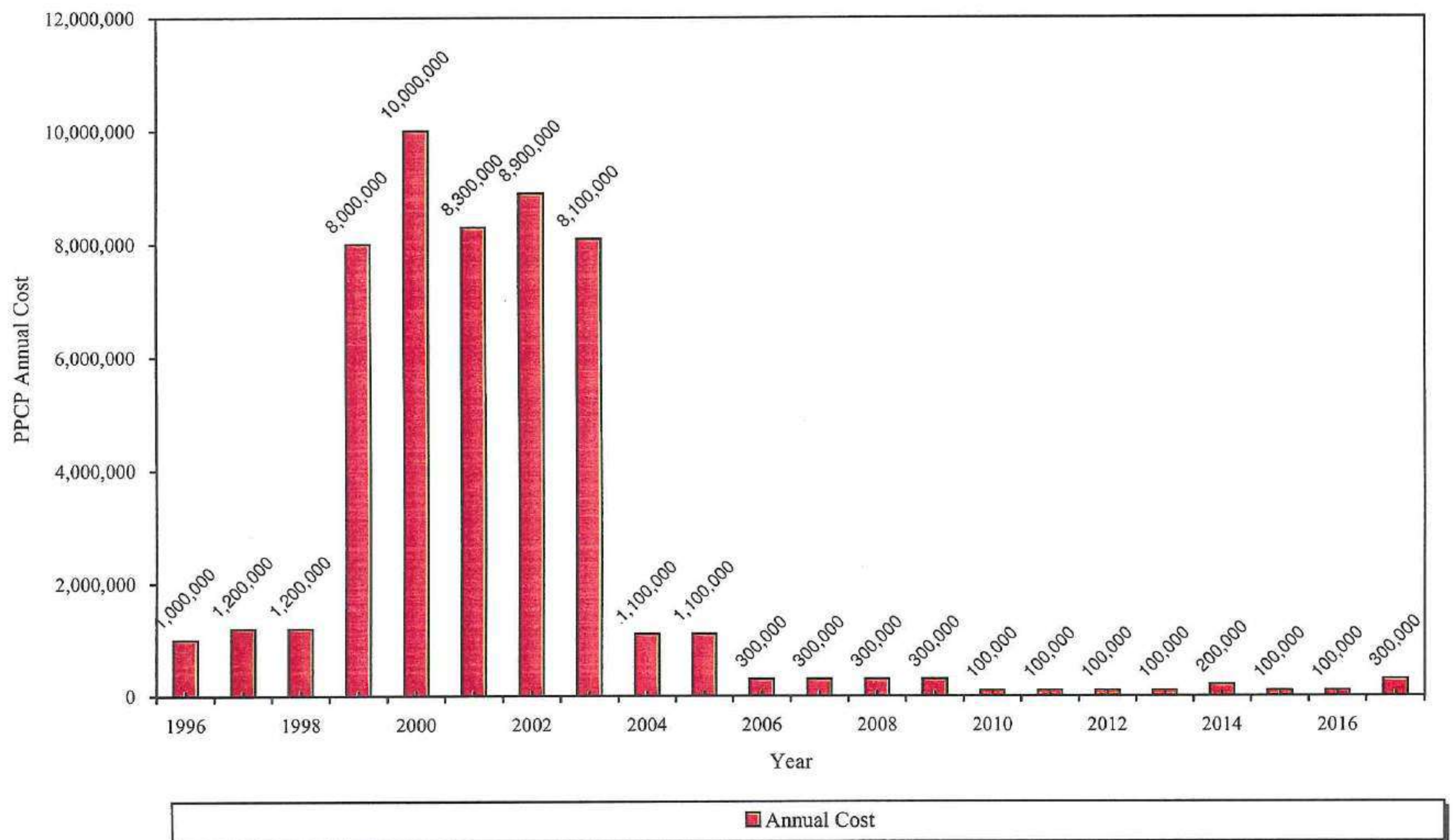
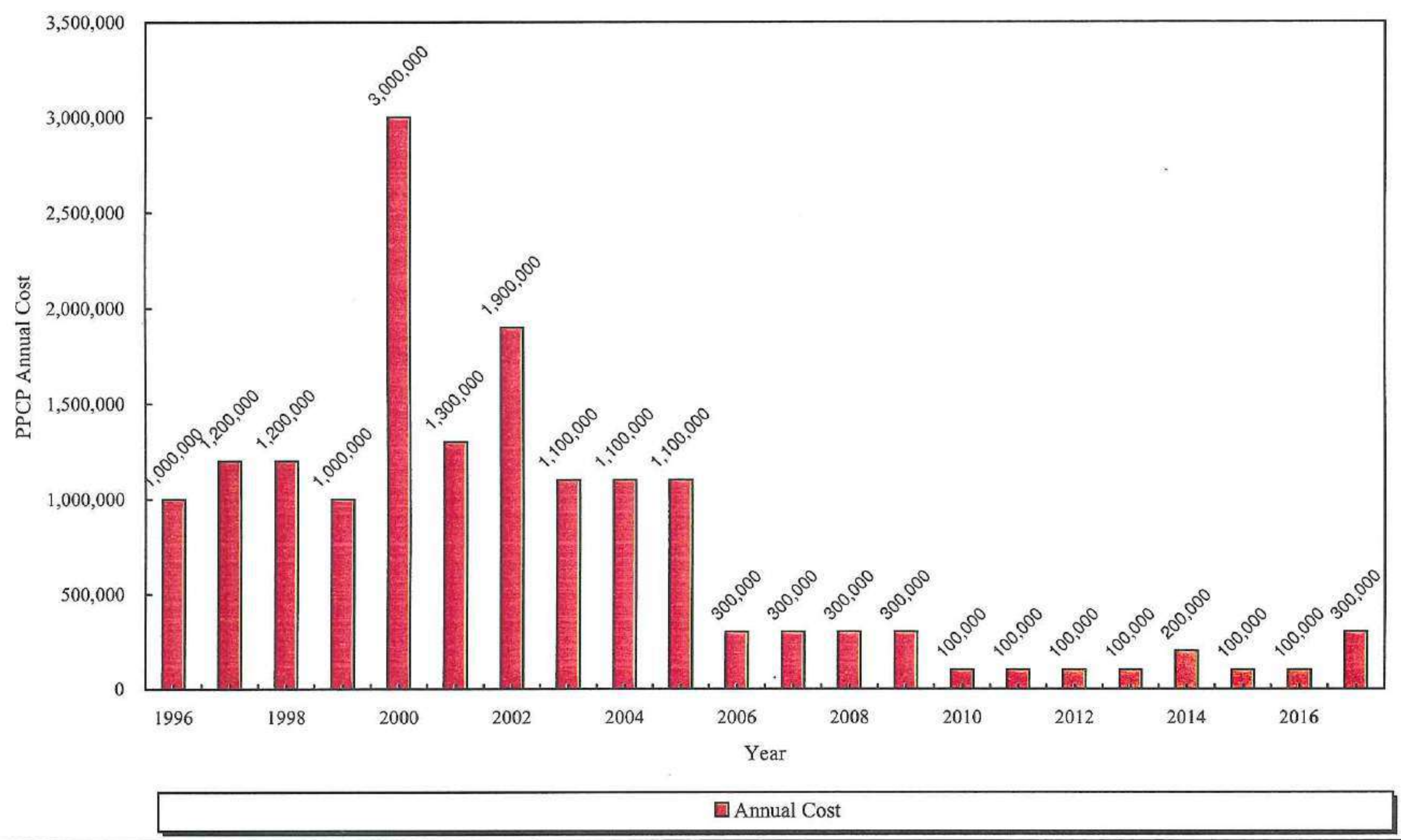


Figure ES-2b

Cash Flow - Without WPCP Upgrade



1. *INTRODUCTION*

1.1 *Background*

The City of Thunder Bay, with participation and funding assistance from the Province of Ontario and the Federal Government, commissioned a study in late 1993 to investigate direct municipal discharges to water courses in the Thunder Bay urban service area. The purpose of this study is to develop a Pollution Prevention and Control Plan for the Thunder Bay urban service area.

The Study, which is identified as the City of Thunder Bay Pollution Prevention and Control Plan (PPCP), was to be completed in two phases generally described as follows:

Phase 1

- Identification of direct discharge points to receiving waters from the City of Thunder Bay urban service area
- Determination of the quantity and quality of discharges under dry and wet weather conditions
- Environmental problem identification

Phase 2

- Evaluation of pollution prevention and control strategies for the City of Thunder Bay
- Performance, economic, environmental and social implications of control strategies
- Development of recommendations including implementation plan and cost schedule

The final Phase 1 State-of-the-System was submitted on June 1, 1995. In the Phase 1 report an assessment of wastewater collection and treatment facilities was conducted in addition to an evaluation of area water resources. The Phase 1 work identified problem items and issues relating to collection system hydraulics, CSO control, basement flooding, pollution prevention and wastewater treatment requirements. The Phase I work program was designed to complement the RAP process in problem definition and public participation. It is anticipated that the final Implementation Plan will also complement the RAP Stage 2 work underway.

Phase 2 of the study evaluates pollution prevention and control strategies with the end result being an implementation plan suited to the City of Thunder Bay.

The Phase 2 Report is divided into five sections starting with a brief summary of key findings and updates from Phase 1, followed by a discussion on components of a PPCP, development of short and long term plan alternatives and finally a recommended pollution prevention and control plan.

1.2 Environmental Assessment Process

Ultimately, the works resulting from this study may be subject to the requirements of the Class Environmental Assessment Act for Municipal and Water and Wastewater Projects. Therefore, the study is being carried out in accordance with the approved planning and design process contained within the Act.

The Phase 1 Study Report met the requirements of Phase 1 of the EA process by identifying system problems and presenting the system problems to the public. Phase 2 of the study will parallel Phase 2 of the EA process where a preferred control strategy will be developed. As well, the Phase 2 studies will establish the requirements for subsequent approvals.

Collection system improvements will tend to fall under the Schedule A or B approval process for municipal water and wastewater projects; therefore, no Environmental Study Report (ESR) will be required for these projects. Schedule A projects involve the repair, modification, reconstruction of existing facilities to provide operational, maintenance or other improvements, and require no public consultation. For Schedule B project, which include projects that extend or expand services, two points of public contact are required. The proponent may select the method of public notification that best suits the circumstances. The EA approvals branch has confirmed that upgrades to the wastewater treatment plant would fall under Schedule A given there is no change in the plant rated capacity. However, the process adopted to review plant design alternatives has followed the more comprehensive requirements of Schedule B. In all cases a "bump up" could be requested where it would be necessary to proceed with Phase 3 and 4 of the EA process and prepare an Environmental Study Report.

2. STATE-OF-THE-SYSTEM-SUMMARY

2.1 Introduction

The City of Thunder Bay, with a total population of 114,000 and a land area of approximately 323.5 square kilometres was founded in 1970 by the amalgamation of the former Cities of Port Arthur and Fort William plus portions of Neebing and McIntyre Townships. The City is located on the west shore of Lake Superior with a water front area that stretches approximately 14 kilometres.

The study area, shown in Figure 2.1, is bounded by the Ultimate Urban Service Area limits, and includes the lands extending westward from Lake Superior generally to the Thunder Bay Expressway, from the City limits in the north, south to the Kaministiquia River. The study area includes seven water courses that pass through the Ultimate Urban Service Area to Lake Superior, Kaministiquia River, McKellar River, Mission River, Neebing River, McIntyre River, Current River and McVicar Creek. All of the water courses including Lake Superior have been identified as important for fisheries either as habitat areas or as zones of passage to spawning grounds. A number of wetlands areas are located within the study area which have been designated provincially significant.

The most optimistic population projection as presented in the November 1993 "Trends and Forecast Report" prepared by the City's Planning Division suggests a peak population of approximately 120,000. The ultimate service population in the Ultimate Urban Service Area is considered to be a total of 151,750. The major growth areas are adjacent to the Thunder Bay Expressway at Arthur Street, Golf Links Road, John Street and Red River Road.

The sewage collection and treatment facilities evolved separately for each City. The North Ward (Port Arthur) system was generally developed as a separate sanitary collection system. Nevertheless by 1961, there were a number of known road catchbasins connected to the sanitary system in the downtown core area where no storm outfalls were available. The sanitary flows are intercepted by the McVicar's Creek Trunk, John Street Trunk and Port Arthur Interceptor sewers. One overflow chamber exists at the lower end of the McVicar's trunk system which has a sluice gate that remains open. Currently, there are no reported overflows at this location. An overflow at Clarke Street has been closed.

The South Ward (Fort William) system consists of combined sewers, partially separated systems and, in newer areas, separate systems. A large portion of the existing sewer system drains toward the Neebing and Kaministiquia Interceptor sewers. The ongoing City program of sewer separation involves removing the road and surface flow component from the combined sewer. However, footing drains and a significant number of roof leaders are still connected. A total of 35 combined sewer overflow chambers were originally installed, controlling inflow to the Neebing and Kaministiquia interceptors. Presently, 20 were identified to be active with the remainder either abandoned or closed.

The Thunder Bay Wastewater Pollution Control Plant (WPCP) provides primary treatment and phosphorus removal and has an average design flow capacity of 109,100 m^3/d for treatment of wastewater from the entire City. The plant effluent flows to the Kaministiquia River via a submerged outfall without diffusion.

Figure 2.1 shows the location of key sewerage facilities in the study area.

2.2 Summary Of Existing Systems

A detailed analysis of the existing collection system operation, water resources and treatment plant operation is presented in the Phase 1 - State-of-the-System Report. This section summarizes the key findings of that report.

2.2.1 Collection System

The hydraulic performance of the existing collection system was assessed with the use of an XP-SWMM Model. Appendix E contains descriptions of all modelled pipe and regulator elements, while Appendix F contains dry weather flows generated for each modelled pipe element.

2.2.1.1 North Ward

The North Ward wastewater collection system was developed as a separated system. However, there is evidence of surcharging and wet weather infiltration and inflow in the system. The following summarizes the system problems identified:

Field Inspection and Monitoring

- Extraneous flows account for approximately 25 to 30% of the dry weather flow to the treatment plant.
- There are known cross-connections (catchbasin) to the sanitary collection system.

Hydraulic Performance

- Surcharge conditions exist at the junction of John Street Trunk and the Port Arthur Interceptor, as well as at the junction of the McVicar's Creek Trunk and the Port Arthur Interceptor during wet weather conditions.

- Surcharge conditions have occurred, and can occur, upstream in the John Street interceptor restricting upstream flows and limiting the capacity for future development flows.
- City staff have reported surcharge conditions in the upstream portion of the McVicar's Creek Trunk that have resulted in outflow from the system to the Creek. Systems analysis has not identified these conditions and basement flooding has not been identified in the area.

Overflow Control

- No overflows were identified.

2.2.1.2 South Ward

The South Ward system was originally developed as a combined sewer system. The City has undertaken a separation program that will be completed within the next two years resulting in approximately 65% separation. Phase 1 identified the following problems in the South Ward collection system:

Field Inspection and Monitoring

- The majority of regulator chambers on the Kaministiquia Interceptor are in poor operating condition and in need of ongoing maintenance or replacement.
- The regulating structures on the Neebing are in reasonable operating condition. However, there is evidence that flap gates to the Neebing River can become obstructed on occasion and do not seal tightly, thus, allowing the inflow of river water into the collection system.
- The interception capacities of the regulators contributing to the Neebing Interceptor are considered high, allowing more flow into the interceptor than originally designed for, this creates the potential for surcharged conditions.
- Extraneous flows account for approximately 25 to 30% of flows to the treatment plant.
- A sanitary connection from a motel in the James and Quebec Street area was found to be connected to a storm sewer pipe discharging to the Kaministiquia River.

Hydraulic Performance

- The Neebing Interceptor surcharges during wet weather events.
- Surcharge conditions in the Neebing Interceptor restrict the outlet capacity of local collection systems leading to the potential for basement flooding.

- The potential for basement flooding still exists in 16 areas following the completion of the separation program based on systems analysis. Generally there is little risk of basement flooding for a 2 year design event, however, the risk increases significantly for a 10 year design event.

Overflow Control

- For the typical year, 1988, only one regulator was active on the Neebing system and 8 on the Kaministiquia Interceptor providing an overall volumetric control of 95% system wide. RK2 (Hardisty/Victoria) on the Kaministiquia Interceptor has a volumetric control of 75%. This is below the proposed minimum MOEE level of control of 90%. All other regulators exceed the 90% level of CSO control on an individual basis.

2.2.2 Water Resources

The stream and loading assessment varied by stream, but included beneficial uses, aquatic life status, water quality and loadings assessments. The emphasis of the study was to establish the potential impacts of CSO upon area waters. Hence, the evaluation activities focused on the Kaministiquia and Neebing/McIntyre Rivers. Nevertheless, it was of interest to examine uses, water quality and loadings in other area streams as well as Lake Superior. The loadings assessment was comprehensive for all receiving waters and included urban sources contrasted with boundary flows. Boundary flow conditions represent the water quality and quantity at the point of entry into the urban area of Thunder Bay.

The Phase 1 stream and loadings assessment identified the following:

- No evidence was found of impairment to cool and warmwater fish populations and fish migratory routes.
- The lower reaches of Kaministiquia River still have a limited diversity of biotic communities. Relatively pollutant tolerant macroinvertebrates taxa are found in the Kaministiquia River.
- The Neebing and McIntyre Rivers support a good variety of pollutant intolerant macroinvertebrate taxa.
- Provincial Water Quality Objectives (PWQO) are exceeded at the urban boundary for iron, cadmium and copper.
- Mercury concentrations were found to be high in the storm outfalls sampled, however, this may be a result of a limited database or may reflect local geology. It is believed the Mercury is naturally occurring and is not related to urban activities or sources.

- Pollutant loads from storm outfalls for dry weather seepage and stormwater represent less than 3% of the annual loads to the area receiving waters for most pollutants evaluated. The exception is mercury, which was found to exist in higher concentrations, resulting in over 95% of the annual loadings.
- Overall, the boundary flows contribute the largest pollutant loads to Lake Superior in the range of 50 to 80%. The greatest sources of BOD_5, total p, chromium and zinc (no mercury data was available) is from both urban and industrial sources accounting for approximately 70% of annual loads combined.
- Industrial sources represent the next greatest pollutant sources in the area followed by the WPCP.
- The PWQO are exceeded for bacteria at the mouth of the Neebing/McIntyre River and McVicar Creek.

2.2.3 Thunder Bay WPCP

A review of the existing status of the Thunder Bay Water Pollution Control Plant (WPCP) with respect to flow, contaminant loadings and biosolids generation projections for the design period to the year 2016 was conducted in Phase 1.

Highlights of this review are summarized below:

Current Status

- The Thunder Bay WPCP is currently operating between 65 and 79% of its average day rated capacity of 109,100 m^3/d.
- The plant has consistently met its compliance requirements for removal of BOD_5 and TSS in the last four years. The effluent phosphorus criteria have been slightly exceeded in 3 of the last six years. Although a compliance criterion does not exist for disinfection, the plant has not historically provided adequate disinfection to meet the bacteriological criterion specified in an MOEE draft disinfection policy.
- The old plant is in structurally poor condition, and equipment has become obsolete.
- Improvements to the Main Pumping Station are planned to reduce charges for energy consumption. Future improvements are necessary if the effluent sewer to the old plant is to be retired.
- The City has initiated a program to improve phosphorus removal.

Limitations to Meeting Capacity Requirements for the Year 2016

- No increase in the rated plant average day flow capacity will be required for the design period. The average raw wastewater flow projected for the year 2016 is 93,285 m^3/d compared to the current rated capacity of 109,100 m^3/d. A flow rate of 109,920 m^3/d is projected for the ultimate service population.
- The peak instantaneous flow that the collection system could deliver to the plant projected for the year 2016 (709,272 m^3/d) exceeds the actual existing main pumping station capacity of 455,000 m^3/d.
- Existing screening and grit removal processes, and the outfall, provide adequate hydraulic capacity for the peak instantaneous flows projected for the design period.
- Adequate primary treatment plant process capacity and biosolids management capacity exists for the design period. Thickening of waste activated sludge from the secondary process will be required.
- The existing chlorination system has insufficient capacity to provide adequate disinfection of wastewater at current and projected flows.
- Due to the physical condition of the old plant, its capacity will need to be replaced.

Factors Affecting Expansion and Upgrading Requirements for the Design Period

- The Sewage Treatment Plant (STP) Regulation presently being developed by the MOEE will require all Ontario primary treatment plants to upgrade to secondary treatment plants. A maximum of 15 years is being considered for initiation of the secondary plant upgrade.
- The Thunder Bay WPCP may be required to proceed with secondary plant upgrading in the near future because it cannot meet general MOEE compliance requirements for phosphorus.
- The Thunder Bay RAP will also recommend a time frame for the secondary upgrade project. RAP recommendations are currently under development.
- The City of Thunder Bay has requested comments from the local Health Unit and the MOEE with regards to the requirement of discontinuing disinfection.

The International Joint Commission recommended Lake Superior be designated a zero discharge demonstration zone, where no point source discharge of a persistent bioaccumulative toxic be permitted. In response, government agencies announced the Binational Program to Restore and Protect the Lake Superior Basin. In this program, the concept of zero discharge applies to wastewater treatment plants. This may require a higher standard of treatment than presently required by the Province of Ontario. As of yet, no specifications have been set.

2.3 Phase 1 - Updates

Two investigations were conducted following the finalization of the Phase 1 State-of-the-System Report released in June 1995. The first investigation involved additional dry and wet weather sampling of stormwater sources into Boulevard Lake, the second addressed the structural condition of the Neebing/McIntyre Interceptor sewer. A third update to Phase 1 addressing discharges from the Ridgeway CSO is discussed and incorporated into the PPCP.

2.3.1 Boulevard Lake Stormwater Outfall Sampling Program

Water quality samples were collected at storm outlets to Boulevard Lake. The objective of the sampling was to determine if urban stormwater contributes to the high bacteria levels recorded in Boulevard Lake.

Prior to initializing the sampling program a survey of the Lake perimeter was undertaken to identify stormwater outlets. Three outfalls were identified to convey stormwater generated by urban developments adjacent to the Lake. As well, three surface ditch drains were identified; however, they drained local lots and were not sampled. Figure 2.2 shows the storm outlets locations (A, B, and C) where samples were collected.

Samples were collected on one dry weather occasion where flow was evident and during two wet weather events. Samples were analyzed for fecal coliform and for one wet weather event mercury was included in the analysis.

Table 2.1 presents the analytical results.

Table 2.1 Boulevard Lake Storm Outlet Sampling

Event Date	Event Type	Fecal Coliform (#/100 mL)			Mercury (mg/L)		
		Site A	Site B	Site C	Site A	Site B	Site C
June 19	dry	<10	no flow	no flow	-	-	-
June 13	wet	<10	no flow	<10	-	-	-
July 4	wet	160[1]	no flow	<10	0.0105[2]	no flow	0.0026[2]

Notes:
1. Maximum count during event, other samples <10.
2. Event Mean Concentration (EMC).

A review of Table 2.1 indicates that the storm outlets that service urban developments are not a significant source of bacteria to Boulevard Lake. Only one sample result was greater than 100 orgs/100 mL for the July 4 event at Site A, however, the EMC is calculated to be less than 100 orgs/100 mL as other samples collected during the event were < 10 orgs/100 mL.

The mercury levels are elevated; however, this is consistent with the previous results of the storm outlet sampling. These results further substantiate the belief that the source of mercury is naturally occurring.

2.3.2 Neebing/McIntyre Interceptor Capacity Constraint

Internal inspection of the Neebing/McIntyre Interceptor revealed a segment on the William Street right-of-way between May Street and Fort William Road that is significantly deflected from the 85" diameter and potentially could collapse. The City has initiated an emergency plan to ensure continued service with bypass pumping in the event of a collapse during dry weather. Overflow to the Neebing River may be inevitable to prevent basement flooding, if wet weather was to occur during a collapse condition. The City is considering various options to structurally strengthen the deflected section that could result in a reduced diameter and pipe capacity. The repair/rehabilitation of the Neebing/McIntyre Interceptor is considered in the development of the PPCP.

2.3.3 Ridgeway CSO

An oil separator in the Ridgeway CSO is reported by CP Rail operating staff to be flushed out during wet weather periods. The result is a discharge of petroleum products to the Kaministiquia River. As well, CP Rail operations has reported medical waste, such as syringes and rubber gloves, being found in the oil separator at the Ridgeway CSO.

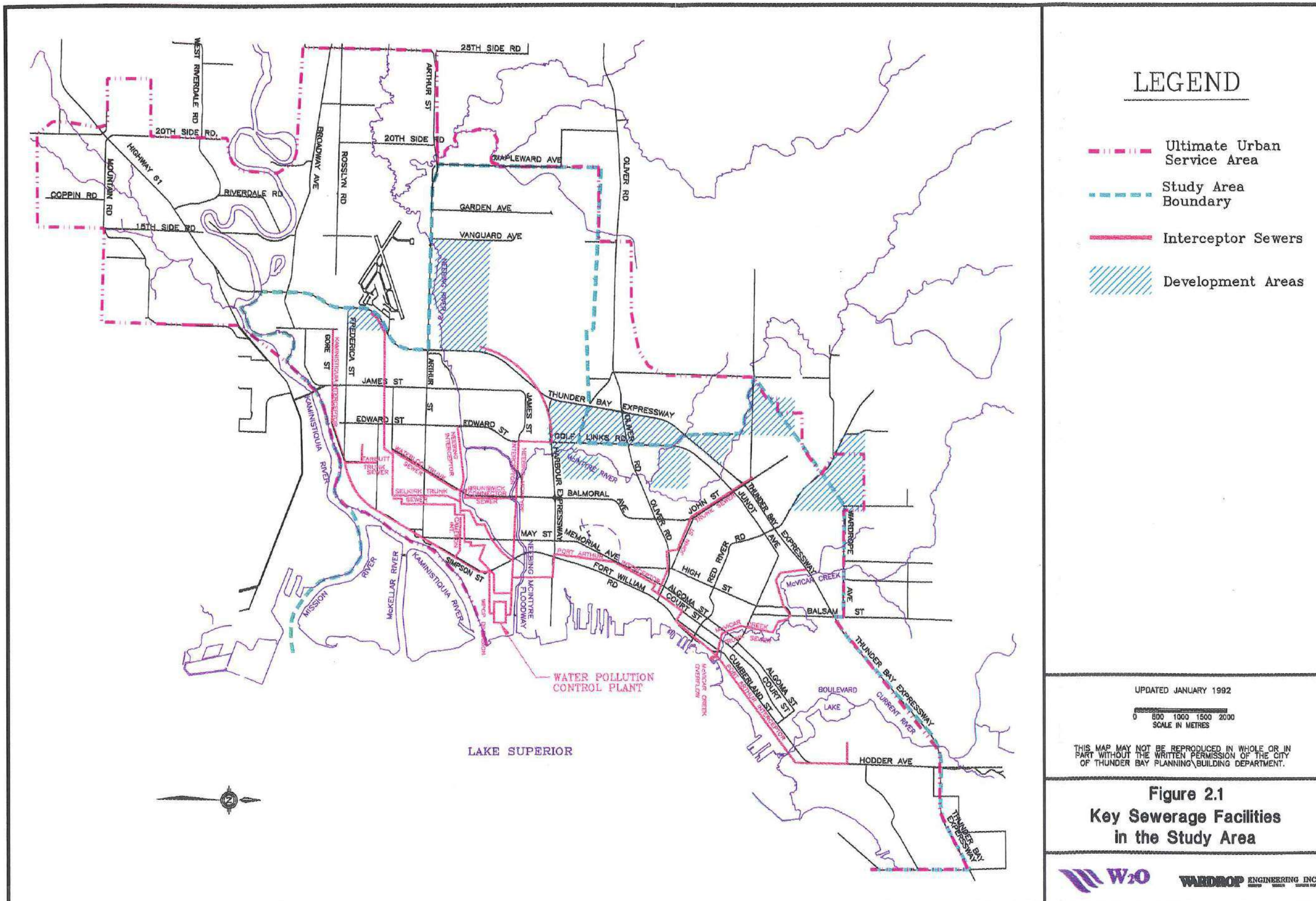


Figure 2.1
Key Sewerage Facilities in the Study Area

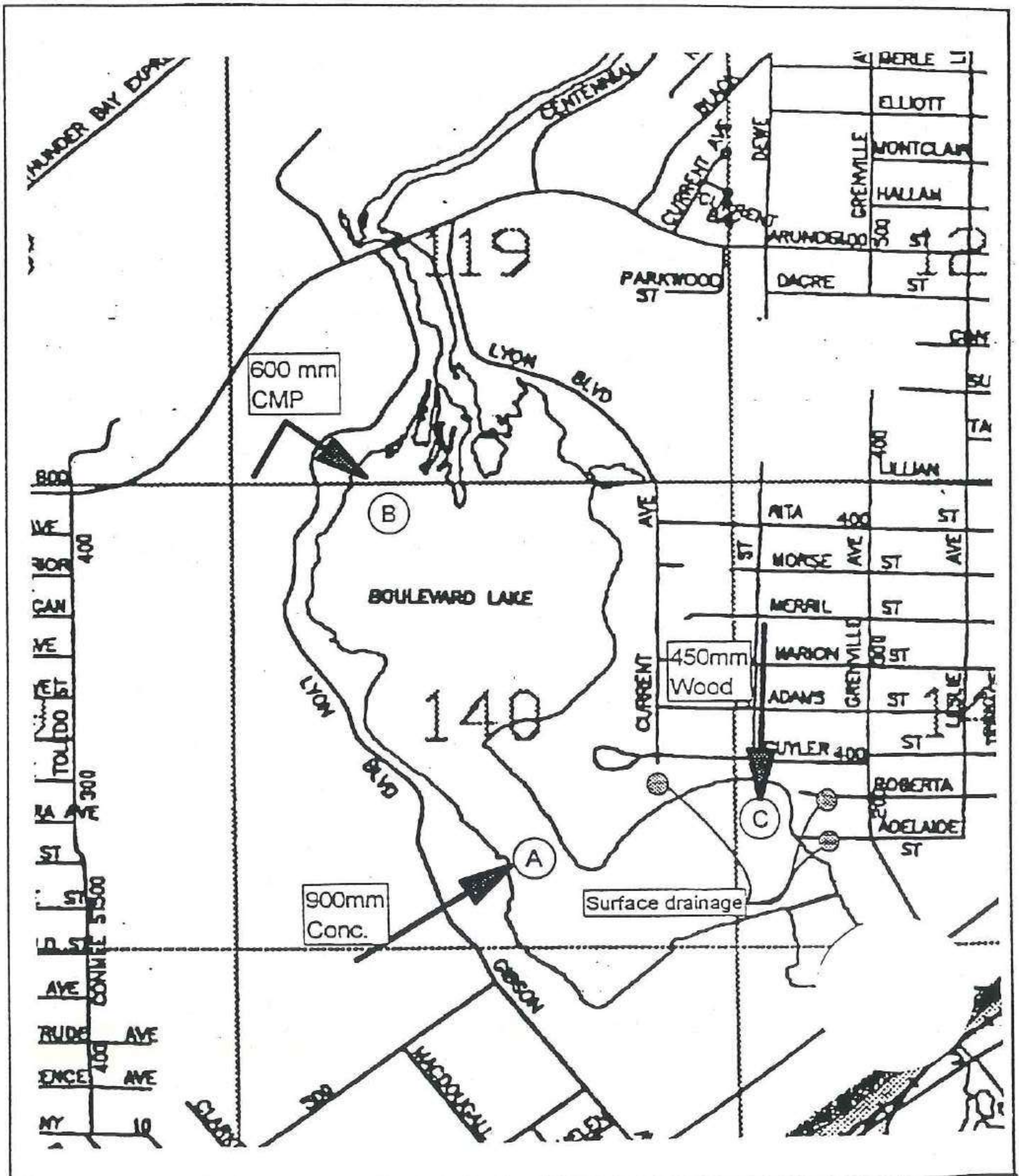


Figure 2.2
Boulevard Lake Storm Outfall Sampling Points

3. POLLUTION PREVENTION AND CONTROL PLAN DEVELOPMENT

3.1 Pollution Prevention Control Plan Objectives

The primary objective of the PPCP is to address the problems identified in Phase 1. In developing pollution prevention and control measures the following factors must be balanced and taken into consideration.

3.1.1 Community Standards

Community standards are considered to include:

- ***Public Health.*** Elimination to the maximum extent practical of harmful bacteria associated with combined sewer overflows and stormwater discharges.
- ***Basement Flooding.*** Elimination to the maximum extent practical the potential of basement flooding associated with wet weather events.
- ***Aesthetics.*** Elimination to the maximum practical extent visibly objectionable solids and floatables related to combined sewer overflow.
- ***Odour Control.*** Elimination to the maximum practical extent of undesirable odour incidents resulting from combined sewer overflows.

3.1.2 Regulatory Requirements

The following is a brief summary of the relevant regulatory requirements to be considered in developing the PPCP.

3.1.2.1 CSO Control

The MOEE policy has now been calculated as Procedure F-5-5[1] as a supporting document for Guideline F-5[2]. Procedure F-5-5 is presented in Appendix A.

---

1 Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewer Systems.

2 Levels of Treatment for Municipal and Private Sewage Treatment Works Discharging to Surface Waters.

Procedure F-5-5 has the following goals:

- Eliminate dry weather overflows
- Minimize impacts to aquatic life and human health from CSOs.
- Ensure that body contact recreational criteria at beaches will not be violated as a result of CSOs.
- Achieve as a minimum, compliance with body contact recreational water quality objectives (PWQO for E. Coli) at beaches impacted by CSOs for at least 95% of a four month period, June 1 to September 30, for an average year.

In order to satisfy the above goals set out by the Ministry, each operator with CSOs is required to develop a PPCP, meet minimum CSO controls and provide additional controls for beaches where recreational water users are impaired by CSOs or where justified by site specific receiving water quality assessments.

Based on the CSO Policy, the City has met the requirement of preparing a PPCP. Secondly, in the Phase 1 Report, no beach area or sensitive water bodies were identified to be affected by CSO discharges. Therefore, the higher standard of CSO control for body contact areas is not required.

The key requirements of the proposed CSO policy that are incorporated into the development of the PPCP includes the following minimum CSO controls:

- Eliminate CSOs during dry-weather periods except under emergency conditions.
- Establish proper operation and regulatory inspections and maintenance programs for the combined sewer system.
- Establish pollution prevention programs (e.g. source controls, public education, water conservation, street cleaning, etc.).
- Minimize solids and floatable materials
- Maximize use of existing collection systems
- Maximize the wastewater treatment plant for treatment of wet weather flows.
- During a seven month period starting within 15 days of April 1, capture and treat at a level equivalent to primary treatment the average dry weather flow plus 90% of average wet weather flow.

3.1.2.2 Stormwater Regulations

The MOEE issued the "Stormwater Management Practices Planning and Design Manual", June 1994, as a technical guide for professionals involved in the planning, design, and review of stormwater management practices. The application of these guidelines in the context of Thunder Bay is important with regards to new developments, construction of new storm outfalls and in retrofitting existing outfalls.

3.1.2.3 RAP and Binational Program

Thunder Bay was identified as one of 17 Areas of Concern (AOC) in Ontario where degraded water quality conditions are considered to impair beneficial water uses. Factors resulting in Thunder Bay being designated as an AOC include conventional pollutants, heavy metals, toxic organics, contaminated sediments, fish consumption advisories, impacts on biota and beach closures. A RAP Stage 1 study initiated in 1988 documented the existing environmental conditions and problems in the Thunder Bay AOC.

The objective of the Thunder Bay RAP is to improve water quality in the Thunder Bay drainage basin. The RAP Stage 2 report is currently being prepared and is expected in 1998.

Recent initiatives, under the direction of the Lake Superior Task Force (senior environmental and natural resource managers) and the Superior Work Group, have produced the Lake Superior Binational Program which has designated Lake Superior as a zero discharge demonstration zone for persistent bioaccumulative toxic substances. Presently, volume, II - Draft Stage 1 Lakewide Management Plan, October 1993, has identified nine critical pollutants for zero discharge as well as identifying causal pollutants which are considered candidates as critical pollutants. As well, the US E.P.A. has recommended classifying Lake Superior as a "National Resource" that would require more stringent regulations regarding discharges from storm sewers, CSOs and treatment facilities.

3.1.2.4 Sewage Treatment Plant

Secondary treatment will be required at the Thunder Bay WPCP within the next 20 years based on MOEE Draft Sewage Treatment Plant regulations and the Canada-Ontario Agreement on the Great Lakes Basin Ecosystem which specifies priority to upgrade primary plants on the Great Lakes.

A Technical Memorandum, *Evaluation of Secondary Treatment Upgrade Options*, has been prepared for the City of Thunder Bay as part of this study. The Technical Memorandum summary, presented in Appendix B, details various treatment technologies available to the City making recommendations on treatment and disinfection options. The City has initiated a Pilot Study set to start in Fall 1996 to investigate possible treatment technologies.

3.1.3 Collection System Management

In Thunder Bay, collection system management will play a significant role in the development of pollution prevention and control measures.

Elements of collection system management include:

- System reliability
- System efficiency (optimized use of existing facilities)
- Reduction/Elimination of basement flooding
- Reduction/Elimination of CSOs
- Reduction in extraneous flows
- Structural integrity
- Provision for future development flows

Each of these elements is considered in the development of the PPCP. Collection system management has the benefit of improving the overall system performance while having an ancillary benefit of reducing pollutant loadings through CSO control and stormwater management.

All analysis conducted to evaluate and assess changes to the collection system were done using the calibrated XP-SWMM system model developed in Phase 1.

3.1.4 Receiving Water Quality

Phase 1 results did not identify CSO and stormwater discharges as significant pollutant sources to the local receiving waters. However, with the improved industrial effluent quality and control, and the proposed upgrade to secondary treatment it is possible that CSO and stormwater discharges will become more evident as a source of pollutants.

3.1.5 Cost Effectiveness

The PPCP development process has been sensitive to the cost effectiveness of the proposed control measures. The City of Thunder Bay is not in the position to consider large capital facilities to address CSO and stormwater control, especially when there is little evidence of receiving quality degradation associated with CSO and stormwater in the area.

Therefore, the focus of control measures under consideration relate to low cost and effective options that will assist the City in managing their wastewater and stormwater facilities while reducing pollutant discharges.

3.2 PPCP Components

Developing a PPCP must address both the immediate concerns of the City, regulatory agencies and the public, as well as addressing long term objectives. The PPCP comprise two components, a Short Term PPCP and a Long Term PPCP.

3.2.1 Short Term PPCP

The development of a Short Term Plan addresses the immediate operational concerns identified in the Phase I State of the System. Typically, the elements of a Short Term Plan include low cost alternatives that are relatively simple to implement over a five year period. As well, the Short Term Plan will form a basis for Long Term PPCP measures.

3.2.2 Long Term PPCP

The Long Term Plan has a planning horizon of 20 to 25 years in which time controls or system improvements are implemented. When developing the Long Term Plan, it must be responsive to improvements realized through the short term plan initiatives. The Long Term Plan must also consider ultimate development conditions in the Thunder Bay area to ensure development can be sustained.

4. SHORT TERM POLLUTION PREVENTION AND CONTROL PLAN

4.1 Overview

The following section outlines in detail the programs and control measures proposed for the Short Term PPCP. The Short Term PPCP consists of low cost measures designed to be implemented over a five to ten year period. The Short Term programs proposed form the foundation for the long term control strategies.

The key elements of the Short Term PPCP address the following areas:

- Collection Systems Management
- CSO Control
- Basement Flooding
- Stormwater Management
- Thunder Bay WPCP
- Pollution Prevention

4.2 Collection System Management

The management of any collection system is extremely important as a means to make informed operational and maintenance decisions. To this end, the following sub-sections review existing City programs, proposing initiatives or improvements that will benefit the City from the perspective of improved system understanding, reliability, performance, CSO control and water quality.

4.2.1 Operation and Maintenance

Table 4.1 briefly summarizes the current operation and maintenance programs, identified in discussions with City staff, associated with the collection systems. Current City programs include manhole inspections on an as time permits basis, a weekly inspection of CSO regulating chambers and flap gates, an annual catchbasin cleaning program, an annual sanitary sewer cleaning and flushing program, and daily storm outfall inspection during spring thaw. The pump station program includes a comprehensive annual, monthly and weekly inspection and maintenance schedule of work. Finally a sewerline TV inspection program was recently initiated on a 20 years inspection cycle.

Operation and maintenance (O&M) programs are essential to ensure reliable services as well as a means to protect the substantial investment in existing infrastructure. Deteriorating infrastructure has some very serious consequences including reduced system reliability, potential for structural failure, interruption of services, expensive rehabilitation costs, extraneous flows and a shortened system life. Based on reviewing current O&M programs, discussions with City Staff, and observations made throughout the study, changes to O&M programs are proposed as part of the Short Term PPCP.

Table 4.1 Current Operation and Maintenance Programs

Programs	Descriptions
Manhole Inspection	• South Ward Interception manholes inspected weekly. • Inspections of other manholes are conducted in trouble areas on a weekly or monthly basis as time permits. • No City wide inspection program. • Recently included as part of the CCTV sewerline inspection contract.
CSO Regulator Inspection	• Control manholes are pressure washed and greased once per year. • Chambers and flap gates are inspected weekly or following major rainfall events.
Catchbasin Cleaning	• Catchbasins are vacuumed cleaned once per year in North Ward. • 50-60% of catchbasins in the South Ward are cleaned annually. • Major cleaning initiated in spring.
Sewer Flushing	• 100% of North Ward sanitary sewers flushed annually • 60% of South Ward sanitary sewers flushed annually
Storm Sewer Outfall	• Inspected during spring ice break up
Sewerline CCTV Inspection	• Conducted in response to sewerline problems • 20 year inspection program recently initiated by the City.
Pump Station Maintenance	• Annual break down of pump station • Monthly cleaning of components • Daily check of main pump station • Area pump station inspected three time/week • Alarm system test every visit • Draw down test monthly – SCADA

4.2.1.1 Inspection Programs

Understanding infrastructure needs is essential to make decisions related to rehabilitation programs and to knowledgeably set priorities. In order to improve the decision process it is recommended that the City implement more pro-active inspection programs. To be pro-active, the City needs to not just respond to problems as they arise, but to identify potential problem conditions in advance of failure.

The following recommendations are proposed to improve existing programs in the short term as well as to provide the foundation for continued inspection programs.

Sewerline CCTV Inspection

The City has initiated a City wide CCTV inspection program based on a 20 year cycle. In other words, over a 20 year period all sewers will be TV inspected at least once. It is recommended that the program be accelerated to provide a complete condition inventory over the next 10 years. The objective of the accelerated program is to identify potential trouble areas before they develop into failure points (i.e. collapsed sections). Presently, the City has only CCTV records in areas that have experienced recent problems and in areas associated with capital construction programs. The age of the collection system and recent experience indicates that there are sections presently in need of rehabilitation and at this time there is no way to develop or prioritize a structural rehabilitation program.

The proposed accelerated program should have the following features:

- Sewer flushing and manhole inspection in conjunction with CCTV inspection.
- Key system elements should be inspected within the first 5 years of the program. Priority areas and elements include:
  - Pipes greater than 450 mm in diameter
  - Interceptor and trunk sewers
    - Neebing Interceptor
    - Neebing/McIntyre Interceptor
    - Cameron Trunk
    - Kaministiquia Interceptor
    - Port Arthur Interceptor
    - John Street Trunk
    - McVicar's Creek Trunk
    - Brunswick Connector
  - Areas with historical basement flooding, the priority areas as indicated by the basement flooding assessment (Section 4.4).

- Pipe sections identified as problem sections should be re-inspected on a 5 year cycle until a repair is made or rehabilitation undertaken.
- The second 5 years of the program will focus on less sensitive areas. It will not be necessary to inspect new system elements that are less than 10 years old unless problems are suspected.

Figure 4.1 shows the priority areas of CCTV work for the first 5 year inspection cycle. In the first 5 years, approximately 276 km of pipe will require inspection, followed by 432 km in the second 5 years.

Manhole Inspection

Presently there is no system-wide manhole inspection program. It is recommended that a program be initiated in conjunction with the CCTV program. In areas that are identified to have trouble manholes, the local collection system should be included as part of the priority list of CCTV inspection areas.

Manhole inspection was included as part of the 1995 CCTV inspection contract and this practice should be continued in future contracts.

CSO Regulator Inspection

The existing program is suitable and no improvements are proposed.

Storm Sewer Outfall Inspection

The spring storm outfall inspection program is in place to prevent ice damage to outfall structures. No recommendations are made to improve this program. It is recommended that the City undertake a summer inspection of all storm outfalls to locate and document outfalls to the receiving streams. Documentation should include location, pictures, condition, dimensions, observations (i.e., dry weather seepage, debris, odour, etc.). This information will be a valuable resource to the City. As well, following wet weather events, outfalls with flap gates should be inspected to ensure the gate is seated properly to provide a good seal against river water intrusion.

4.2.1.2 Maintenance Programs

The objectives of maintenance programs are to prevent equipment or system failure. The City has several maintenance programs addressing CSO regulators, the collection systems, storm outfalls and pump stations.

CSO Regulator Maintenance

No recommendations are made to improve the existing maintenance program of cleaning and greasing each regulator once per year. The regulator inspection program in place is considered a maintenance function and is preventative in nature.

Storm Outfalls

Outfalls with flap gates are cleaned and greased annually. No changes to the existing maintenance program are recommended.

Sewer Flushing

The City has no program for periodic cleaning and flushing of interceptors or large diameter trunk sewers. Otherwise, 100% of the North Ward and 60% of the South Ward sewer network are flushed annually. To illustrate the importance of sewer flushing, efforts to clean the Dease Street outfall and to conduct CCTV inspection as part of the Franklin storm sewer Stage 1 project in 1994 showed the amount of debris and deposits that can result from poor maintenance. This reduction in cross sectional area from deposits will result in a reduction in the effective pipe capacity.

It is recommended that the sewer flushing program be coordinated with the sewer CCTV inspection program. Typically, sewer flushing is done prior to CCTV inspection, however, to reduce the cost it is recommended that flushing occur only when identified as a need by the CCTV inspector. As well, it is recommended that the City provide 100% coverage of the South Ward annually.

Pump Stations

In the City of Thunder Bay there are five wastewater pump stations and the main treatment plant pump station. The maintenance program for all of the stations is comprehensive and well documented with annual, monthly and daily maintenance at each station. The City has designed a complete and thorough preventative maintenance program greatly reducing the potential of mechanical, control or alarm failure.

4.2.2 Neebing/McIntyre Interceptor Improvements

Recent inspection of the Neebing/McIntyre Interceptor revealed a severely deflected section in the May Street area. There is a risk of collapse of this section and the City has developed a contingency plan in the event of failure. The City has also initiated a study to investigate possible remedial measures to stabilize the pipe to prevent failure. One of the techniques being considered is slip lining the 2,100 mm (84 in) pipe. This would have the effect of reducing the pipe diameter to approximately 1,350 mm (54 in). An evaluation was undertaken to determine if the reduced pipe diameter would affect the service capacity of the Neebing/McIntyre Interceptor and more importantly result in insufficient pipe capacity to service future growth areas. The evaluation assumes ultimate development conditions, implementation of Short Term PPCP diversions of the Neebing Interceptor, Alternative 4 is adopted for the Golf Links Extension and North Ward servicing, and the Kaministiquia Interceptor connection to the Main pump station is made. The assumptions made are discussed in following sections of the report.

A dry weather assessment revealed no capacity constraints. Figure 4.2 shows the peak HGL along the Neebing/McIntyre Interceptor with a reduced diameter from the Brunswick connection to just downstream of May Street at Minnesota Street for the 10 year design storm event. The reduced diameter does not result in a hydraulic constraint for the 10 year design storm event.

No recommendations are made on the type of repair or rehabilitation technique to correct the Neebing/McIntyre Interceptor as this will be made in another study. However, a reduction in pipe diameter will not result in a capacity constraint in the future based on the analysis completed. The Short Term PPCP recommends flow monitoring in the Neebing/McIntyre Interceptor at Syndicate to verify the initial hydraulic model simulations prior to selecting a final repair/rehabilitation program (as discussed in subsection 4.2.5 Monitoring Programs).

4.2.3 Maximize Use of Existing Facilities

4.2.3.1 Diversions

Collection system modelling was conducted as part of the Phase 1 assessment. The modelling has shown that there is little opportunity for in-system storage to manage wet weather flows except in two locations.

The Neebing/McIntyre Interceptor and the Cameron Trunk sewer are under-utilized during wet weather based on the 5 and 10 year design storm hydraulic assessment. Opportunities to take advantage of the available storage and capacity by employing simple system modifications were investigated to relieve the Neebing Interceptor during wet weather periods.

The Brunswick Connector sewer serves the area south of the Neebing River, north of Empire Avenue, and west of Waterloo Street. The Brunswick sewer was designed to eliminate the Brunswick Avenue pump station and was constructed with a common manhole at the Neebing Interceptor. A recent inspection of the common manhole revealed no manhole benching to direct flow to the Brunswick or Neebing sewers. Flow exiting the manhole (upstream drainage areas of both Neebing and Brunswick drainage areas) is split between the Neebing Interceptor and Brunswick Connector sewer.

At this location there is an opportunity to reconstruct the common manhole to divert excess wet weather flows into the Brunswick Collector and Neebing/McIntyre Interceptor providing flow control into the top end of the Neebing Interceptor.

Figure 4.3 illustrates two alternative configurations for the reconstruction of the common Neebing/Brunswick manhole. The first configuration includes a regulating gate (or equivalent) on the Neebing Interceptor exit pipe from the chamber. The gate opening would be set to allow approximately 2.5 times the Neebing Interceptor's average dry weather flow (2,600 m^3/d) to continue to flow down the Neebing Interceptor. Flow in excess of 2.5 times average dry weather flow would be diverted to the Brunswick Connector sewer. Alternatively, all dry weather flow can be directed to the Brunswick Connector sewer by installing a weir wall in the chamber in front of the Neebing Interceptor exit pipe. During wet weather flow conditions, flows would be prevented from entering into the Neebing Interceptor unless the flow level exceeded the weir wall. It is estimated that the weir wall should be no higher than 300 mm to prevent surcharging in the upstream sections of the Brunswick and Neebing collection systems. The advantage of the gate structure over the weir wall configuration is the ability to control flow into the top end of the Neebing Interceptor and the ability to return to the original configuration if desired.

In conjunction with the Brunswick/Neebing diversion, another control structure is proposed that would allow flows from the Neebing Interceptor to flow into the adjacent Cameron Trunk sewer during excess wet weather flow conditions. The diversion would be located at Cameron Street and Marks Street where the sewers are in close proximity. The Cameron Trunk sewer is at a higher elevation at this location. This would be used to ensure that flows continue down the Neebing Interceptor for smaller events so the Cameron diversion is used only in more critical events.

The connecting sewer would be on a reverse grade, and a backwater valve would be required to ensure that flows from the Cameron sewer do not flow back into the Neebing Interceptor system. Figure 4.3 also shows the configuration at the Neebing/Cameron connection.

Configuration 1 is structured to allow flows to revert back to their existing flow routes to provide additional operational flexibility in the event of a collapse in the Neebing/McIntyre Interceptor or to undertake local system maintenance. Figure 4.4 shows the hydraulic control achieved for the 10 year design storm event along the Neebing Interceptor with both diversions in place. XP-SWMM was used to assess the changes in system hydraulics and operations with the Configuration 1 diversions in place. In the evaluation a flow equivalent of 2.5 times the Neebing Interceptor average dry weather flow (2,600 m^3/d) is permitted to flow into the Neebing Interceptor from the Brunswick diversion during wet weather. Flows in excess of 2.5 times average dry weather flow are diverted to the Brunswick Connector Sewer.

Figure 4.5 shows the peak HGL in the Brunswick Connector and Neebing/McIntyre Interceptor with the two diversions in place (Configuration 1). The evaluation of the Neebing/McIntyre Interceptor accounted for ultimate development conditions and a reduced pipe diameter (85" to 54") that may result from rehabilitation of the Interceptor. The hydraulic performance for Configuration 2 is similar to Configuration 1, providing relief in the Neebing Interceptor.

4.2.3.2 Flow Control

Another method to maximize the use of existing facilities is to reduce extraneous flows. This can be accomplished through sewer rehabilitation aimed at preventing rainfall derived inflow and infiltration. However, to establish a rehabilitation program, sewer inspection and condition information is required. The CCTV inspection program proposed will provide the information required to develop and prioritize an effective rehabilitation program.

Inflow reduction or elimination represents an effective method to maximize the use of existing facilities. As identified by City Staff, 37 catchbasins are known to be connected to the sanitary system in the North Ward. A review of each catchbasin location in the field was undertaken to identify appropriate methods to remove or reduce inflow into the sanitary sewer. Alternatives considered include:

- ***Flow Slipping.*** Where road grades permit (>2%) seal catchbasin and allow flow to continue to a catchbasin connected to an existing storm sewer system, open space or water course.
- ***New Storm Sewer.*** Construct new storm sewer services.
- ***New Catchbasin Leads.*** Install new 250 mm catchbasin lead and re-connect catchbasin to an existing storm sewer system. It is assumed that a catchbasin lead will be no longer than 25 m.
- ***Inlet Control.*** Restrict the amount of inflow to sanitary system.

Table 4.2 presents a prioritized summary of recommended control measures to reduce or eliminate wet weather inflow in the North Ward sanitary collection system from catchbasin connections. The Short Term control program includes catchbasin sealing, inlet restrictions and new storm sewer segments. The removal of the remaining connected catchbasins is considered in the Long Term PPCP.

Inflow was identified from the Neebing River into the Neebing Interceptor at two locations during the field inspections (RN32: Wellington/Cumming, RN21: May/Southern). Inflow at RN32 was attributed to debris preventing the flap gate from closing, and RN21 had a poor flap gate seal.

All of the CSO outfalls on the Neebing River are susceptible to inflow from the river. The river stage fluctuates between 183 and 186 which is in the same range as the dam elevations of the Neebing regulator chambers. Therefore, if a flap gate has a poor seal, or is wedged open with debris, river inflow will likely occur.

Table 4.2 Catchbasin Flow Control

Location	Recommended Action	Required Works
Court St. and Manitou St.	Seal CB or flow restriction	Seal CB and allow flow to travel 210 m to Court St. and Bay St. (slope=2.2%)
Ambrose St. and New St.	Flow restriction	Install inlet restriction device
Dorothy St. and Carrie St.	Flow restriction	Install inlet restriction devices in two CBs
Prospect Ave. and Van Norman St.	Seal CB at South west corner	Seal CB and allow flow to travel 256 m to Hebert St. and Peter St. (slope=9.5)
Van Norman St. and High St.	Flow restriction	Install flow restriction device
Cumberland St. and Tupper St.	Flow restriction	Install inlet restriction device
Knight St. and Dawson St.	Flow restriction	Install flow restriction device
St. James St. and Court St.	Flow restriction	Install flow restriction device
Ruttan St. and Argyle St.	Seal CB	Seal CB and allow flow to travel 100 m to St. James St. and Algoma St. (slope=5.9%)
Court St. and Wolseley St.	Flow restriction	Install flow restriction device
Farrand St. and Van Horne St.	Flow restriction	Install inlet restriction device
Front St. and Wolseley St.	New storm sewer	Construction of 290 m of storm sewer connected to existing 24" storm on Front St. (lead for 3 CB)

As part of the City's inspection program the CSO chambers and outfalls are inspected every week and following large storm events. As part of this inspection inflow from the Neebing River should be identified and corrective action taken immediately. An alternative to flap gates, that the City may consider when replacing existing flap gates, is a "duck bill" design. The "duck bill" valve is a flexible rubber check valve designed to open with a minimum specified head that will close with the back pressure. The flexible material can form a seal around foreign objects that would typically cause a flap gate to wedge open. The flexible "duck bill" design will reduce maintenance and inspection costs and operational problems associated with the existing flap gates. The valves can be installed at an outlet or upstream in the outfall pipe. The design of the outfall with a "duck bill" should include a provision for a high level relief in the event the "duck bill" is obstructed by ice. The installation of "duck bill" or equivalent type structures will not change the hydraulic conditions in either system. As part of the Short Term program there is no recommendation to replace any of the flap gates. However, as the existing gates deteriorate with time, the City should consider the "duck bill" design. The existing flap gates do not worsen hydraulic conditions within the system and replacement with the "duck bill" will not change the hydraulics. Appendix C contains information on the "duck bill" valve.

4.2.4 Dry Weather Discharge

A sanitary connection to the storm sewer system was identified in the James and Quebec Street area. As part of the Short Term program a new service connection should be constructed to re-direct the sanitary flow to the sanitary collection system. Figure 4.6 shows the required connection. The correction of this connection will require new service connections to each of the buildings on the property. No other dry weather sanitary/combined discharges were identified.

4.2.5 Monitoring Programs

It is recommended that the City of Thunder Bay establish an ongoing flow monitoring program. The objectives of the flow monitoring program is three fold:

- To evaluate the effectiveness of system rehabilitation programs
- To establish continuous monitoring in key system elements.
- Collect additional flow monitoring data for improved XP-SWMM model calibration.

Flow data is a valuable measure of the performance of any collection system. It provides information related to flow rates, system capacity, the extent of inflow and infiltration, and can be used in prioritizing rehabilitation works and evaluating their effectiveness. A continuous monitoring program is proposed for the City of Thunder Bay that will provide a suitable level of information at key points throughout the collection system.

Currently the City operates three velocity-area flow meters, of which one is dedicated to Sewer-Use By-law Enforcement, leaving two for collection system monitoring. It is anticipated that the City will need to purchase at least two more velocity-area meters over the 5 year program to replace existing equipment or to monitor additional locations. It is also recommended the City purchase one rain gauge and logger to collect precipitation data. Other stations in the area operated by the Lakehead Conservation Authority can be used to augment local data. The benefit of operating a City rain gauge station is control over the set up, the location, and in accessing data. The monitoring program is designed around a 5 year cycle of monitoring key system components. As well, the program is designed to establish permanent continuous monitoring stations along the Port Arthur, Kaministiquia, Neebing and Neebing/McIntyre Interceptors.

Table 4.3 presents a summary of key monitoring locations in the sanitary and combined sewer systems assuming two flow meters are available each year.

Table 4.3 Proposed 5 Year Flow Monitoring Locations

	Location	Comments
		Year 1
1	Neebing Interceptor @ Brunswick St.	• Recommended diversion location • Calibration data for Neebing basin
2	Neebing Interceptor @ Alexandra St.	• End of Neebing interceptor • Calibration data for Neebing basin
3	Brunswick Connector @ Legion Track Dr.	• Recommended diversion location • Calibration data for Brunswick basin
4	Neebing/McIntyre Interceptor @ Upstream of Syndicate	• Related to diversion of Neebing to Brunswick • Related to interceptor rehabilitation • Establishing flows/capacity in interceptor
		Year 2
5	Victoria Ave @ Waterloo trunk	• I/I identification - assist in prioritizing CCTV
6	Arthur St. @ Waterloo trunk	• I/I identification - assist in prioritizing CCTV
7	Isabella St. @ Waterloo trunk	• I/I identification - assist in prioritizing CCTV
8	Walsh St. @ Waterloo trunk	• I/I identification - assist in prioritizing CCTV
		Year 3
9	John St. Trunk @ Algoma St.	• PPCP monitoring station #2 • Additional data - potential HGL problem area
10	John St. Trunk @ upstream of Algonquin Ave.	• Before parallel system starts • Potential HGL problem area
11	John St. Trunk @ Oliver & High St.	• Before parallel system combine at High St. • 2 meters required
		Year 4
12	McVicar's Ck Trunk @ Court St.	• PPCP monitoring station #3
13	McVicar's Ck Trunk @ Margaret St.	• I/I, capacity, calibration data
14	McVicar's Ck. Trunk @ Madeline St.	• I/I, capacity, calibration data
15	McVicar's Ck. Trunk @ Hwy 17/11	• Boundary flows, I/I, capacity, calibration data
		Year 5
16	Port Arthur Interceptor @ Allied Chemical	• PPCP monitoring station #4 - additional data, previous monitoring relatively dry period
17	Port Arthur Interceptor @ Main St.	• PPCP monitoring station #1 - additional data, previous monitoring relatively dry period
18	Port Arthur Interceptor @ John St. Trunk	• Performance of interceptor
19	Port Arthur Interceptor @ McVicar's Ck Trunk	• Performance of interceptor

The 5 year program outlined will provide an improved understanding of flows throughout the entire network and form the basis for more detailed flow investigations to identify potential areas for sewer rehabilitation, to correct I/I problems, as well as to assist in establishing priorities for CCTV inspection. The flow monitoring program should remain flexible enough to address special concerns as they arise. For example, if the City undertakes collection system rehabilitation, such as crack sealing, flow data should be collected before and after to determine the benefit of system rehabilitation.

As flow and rainfall data is collected over the 5 year program, the data can be used to improve the calibration of XP-SWMM network model. City staff have received XP-SWMM training providing them with the basic skills required to work with the model. One of the benefits of XP-SWMM is if the need arises, local consultants will be able to assist the City in model calibration and system analysis. Maintaining and improving the collection system model will enable the City to update system analysis and review PPCP plan recommendations.

In conjunction with the 5 year flow monitoring program, the City should consider establishing permanent monitoring stations in key elements of the collection system. The permanent monitoring stations can be linked into a SCADA system to simplify data acquisition. Permanent stations that monitor depth only are relatively inexpensive, can be calibrated to determine a depth-discharge relationship and can provide early warning of collection system problems. Table 4.4 presents the recommended permanent depth monitoring stations.

Table 4.4 Permanent Monitoring Stations

Location	Comments
~~Neebing/McIntyre Interceptor~~	• ~~upstream of collapsing section with alarm~~
Port Arthur Interceptor	• Establish four stations corresponding to year 5 monitoring stations (Table 4.3)
Kaministiquia Interceptor	• Establish three stations - near the WPCP, Arthur St. and James St.

4.2.6 *Sampling Programs*

A storm outfall sampling program is proposed. The objectives of the program are to quantify dry weather seepage flows and to collect water quality samples. The dry weather samples should be analyzed for conventional parameters and metals. The program may identify any existing sanitary cross-connections or industrial discharges. The sampling program can be conducted in conjunction with the survey program recommended.

It is recommended an initial survey be conducted and where sample results reveal anything unexpected, to repeat sampling at these sites to confirm initial analysis results. The outfall survey is recommended on a five year cycle. Implementation of the storm outfall survey may be possible in co-operation with the local MOEE, the Lakehead Conservation Authority, Lakehead University and/or RAP.

4.3 CSO Control

4.3.1 Regulator Technology

The float and gate CSO regulators on the Kaministiquia system were identified to be in poor physical and operational condition. It is recommended that these regulators be replaced with more reliable CSO regulator technology. Two regulator types are available that would provide more reliable discharge characteristics and require less maintenance:

- Vortex throttles
- Hydroslide

There are a number of suppliers using vortex technology. Briefly, the vortex throttle has no moving parts and operates as an orifice or simple pipe under normal flow and low head conditions. At higher heads the flow begins to vortex restricting flow through the device resulting in a controlled throughflow rate despite an increasing head. Orifice inserts can be installed in the vortex chamber to change the discharge characteristics from 75% to 175% of the nominal capacity. The inserts allow the operator to change the throughflow rate easily without replacing the vortex throttle. Figure 4.7 shows a typical vortex throttle installation and device.

The second CSO regulator type the City should consider is a Hydroslide. The hydroslide is a float controlled device. A float operated arm controls the aperture opening through the device, as the head increases in the chamber the float rises causing the control plate to decrease the throughflow opening. The result is a constant throughflow capacity. Different throughflow design rates can be achieved by simple changes in the length and movement of the control plate. Figure 4.8 shows a typical hydroslide device. Appendix C contains detailed information on both the vortex throttle and hydroslide devices.

Both CSO regulator types are suitable for replacing the existing Kaministiquia CSO regulators with minimal modification to the existing chamber structures.

As part of the Short Term PPCP CSO control program the City should consider replacing the existing Kaministiquia regulators with lower maintenance units such as vortex throttles or hydro-slides. Prior to replacing any of the regulators short term flow monitoring should be undertaken to measure and establish the range of flows. This will assist in selecting the appropriate flow regulator, will provide additional information on the flow reduction achieved through sewer separation program and verify the level of CSO control. Where possible, wet weather flows should be intercepted and transported to treatment.

Table 4.5 summarizes the Kaministiquia regulators that can be replaced with the cost of the two units proposed. It is estimated to cost between $110,000 and $175,000 to replace all regulators.

No changes are proposed for the CSO chambers associated with the Neebing Interceptor.

Table 4.5 Kaministiquia Regulator Replacement

Regulator ID	Throughflow Capacity (L/s)	Vortex Throttle Cost ($)	Hydroslide Cost ($)
RK1	59.0	12,600	8,590
RK2	42.0	10,500	7,375
RK3	175.0	16,800	9,800
RK4	18.4	10,500	6,160
RK5	79.5	14,700	6,160
RK6	20.6	10,500	7,375
RK7	34.1	10,500	7,375
RK8	36.4	10,500	7,375
RK9	40.9	10,500	7,375
RK10	181.8	16,800	9,800
RK12	173.9	16,800	9,800
Sub-Total		**140,700**	**87,210**
Plus 15% contingency + 7% GST		32,430	20,100
Total		**$173,130**	**$107,310**

Note:

1. RK2 regulator replaced as part of Short Term PPCP. Throughflow capacity increased from 21 L/s to 42 L/s.

4.3.2 CSO Regulator Settings

Basin wide the level of CSO control is greater than 90% volumetric control meeting the minimum control objective of the MOEE CSO guidelines. Only one CSO regulator, RK2 (Hardisty/Victoria), does not meet the 90% control level. Even though no improvement is necessary to meet the MOEE CSO Guidelines basin wide, a simple adjustment to the overflow weir elevation or an increase in throughflow capacity would be sufficient to increase the control of RK2 to greater than 90% control without any impact downstream on the Kaministiquia Interceptor.

It is proposed that RK2 chamber be retrofitted with either a vortex throttle or hydroslide designed to increase the dynamic throughflow capacity from 21 L/s to 42 L/s to meet the 90% control objective. This will enable the City to evaluate the regulator technology firsthand while achieving a 90% control level at RK2.

4.3.2.1 Floatables Control

Following the completion of the Phase 1 Report, it was reported by CP Rail operations staff that during wet weather the flow through an oil and grease separator was sufficient to flush the unit, resulting in discharges of petroleum products to the Kaministiquia River through the Ridgeway CSO chamber. As well, it was noted that medical waste is commonly found in the oil and grease separator.

To prevent the flushing out of the oil and grease separator, two alternatives are available. The first is to replace the separator with a larger unit that can handle the peak wet weather flows without flushing. The second alternative is to install a separator bypass that would operate only during peak wet weather periods to divert excess flows, thus, protecting the separator. Either alternative would improve the overall operation and prevent discharges to the Kaministiquia River.

Floatables were not identified as a community concern; however, there is evidence of floatables at the Ridgeway oil and grease separator and CSO. In the case of the Ridgeway CSO, it is recommended that the City undertake a program to identify the source of the medical wastes reported and to eliminate the source. Floatable control should be considered at any CSO chamber where there is a possibility of floatables being discharged to the receiving waters. In most cases, the regulating chambers can be retrofitted with a baffle plate attached to the overflow weir that will trap floatables. The baffle plate will extend below and above the weir wall height and be offset approximately 6 inches. The offset will allow flow to pass under the baffle and over the weir while trapping the floatables. It is recommended that floatable control be implemented wherever there is evidence or suspicion of a floatables issue.

4.4 South Ward Basement Flooding

Phase 1 hydraulic analysis identified that basement flooding was still possible in 16 areas of the South Ward with completion of the combined sewer separation program. Figure 4.9 shows the approximate boundaries of the sixteen drainage areas. It is recognized that some of the 16 areas identified to be at risk of flooding have been partially separated. Partial separation removes the road and surface drainage from the combined sewer pipe, but inflow from sources such as roof leaders and foundation drains will continue to contribute to flows in the sanitary pipe.

The intention of the following sub-section is to identify the areas at risk of basement flooding in the South Ward for different design storm event conditions.

The outcome of the analysis provides the City of Thunder Bay a basis on which to focus City resources in priority areas as part of a Short Term Control Program. As well, it provides a conceptual level assessment of control alternatives to address basement flooding for different levels of protection.

In focusing City resources, a combination of actual flow data in risk areas with a detailed inventory of connections to the local combined/sanitary system will enable the City to determine the most appropriate control alternative and level of protection achievable. The Short Term Program already includes as part of the recommended Monitoring Program a component covering historical basement flooding areas.

4.4.1 Basement Flooding Model

The XP-SWMM model developed in Phase 1 for post separation conditions was used as the basis for the basement flooding assessment. The Phase 1 model was refined for the basement flooding assessment with the extension of the interceptor system model into the local collection system upstream of regulators on the Kam Interceptor and Neebing/McIntyre Interceptor where basement flooding has occurred historically in the South Ward.

Extension of the network model required adjustments to model runoff parameters to reflect the nature of the local collection systems. Of particular importance was the number of rooftops suspected of being connected to the combined/sanitary system. In Phase 1 it was assumed that approximately 50% of the roofs in the original combined service area were still connected to the sanitary/combined system. A recent 1997 survey by City of Thunder Bay staff identified that the number of downspouts that discharged underground was typically less than 10%, in the range of 5% to 6%. One area was identified to have approximately 25% of downspouts discharging underground. To reflect the approximately 10% connected roofs, the percent imperviousness for each catchment area used in the model was reduced accordingly, down from the percent imperviousness based on the original 50% connected assumption. An additional area was also included beyond the 16 areas already identified in Phase 1 as being at risk. The 17th area is tributary to regular RK8 located at Ridgeway Street and Simpson Street.

A sensitivity analysis was also undertaken to understand the sensitivity of the assumption that 10% of the homes are connected to the combined/sanitary sewer. The sensitivity analysis considered a 50% and 80% level of roofs being connected to the combined/sanitary system. Percent imperviousness was recalculated for 50% and 80% level of connection. Table G-1, in Appendix G is the working table used to calculate the percent imperviousness considering 10%, 50% and 80% of the rooftops, is directly connected to the sanitary/combined system. The calculation is based on counting the number of homes in each regulator service area, determining the connected roof area and adding the connected area to other impervious areas (roads, driveways, etc.). Finally, in areas that have undergone separation the associated surface drainage (roads, driveways, etc.) area is removed from combined service area to reflect post separation conditions. The post separation percent imperviousness values for 10%, 50% and 80% level of roof connections are part of Table G-1.

To assess the risk of basement flooding the 2, 5 and 10 year design storm events were used. The design storms were developed using the Intensity Duration Frequency (IDF) curves of the AES Fort William's rain gauge. The IDF data was used to develop design storm events using a 4-hour Chicago distribution. The Chicago distribution is commonly used to design and assess collection systems with stormwater contribution. The 2, 5 and 10 rainfall hyetographs and values are also included in Appendix G.

4.4.2 Basement Flooding Assessment

The basement flooding assessment is premised on approximately 10% of the roofs in the original combined sewer area still being connected to the sanitary/combined pipe. City staff have confirmed that 10% of roofs connected would reflect the current conditions.

The refined Phase 1 XP-SWMM model for post separation conditions and 10% of the roofs being connected was used to undertake a detailed hydraulic analyses in the South Ward for the 2, 5 and 10 year design storm events to assess the level of risk associated with basement flooding. An area was considered to be at risk of basement flooding if the hydraulic gradeline (HGL) exceeded the crown of the sewer. If the HGL exceeds the crown, the homes connected to that pipe segment were considered at risk. This is a conservative approach since basement flooding occurrences depend on the elevation of the house connection relative to the HGL. However, this approach is considered appropriate for assessment purposes. Table 4.6 presents the estimated number of homes at risk of basement flooding as a result of the 2, 5 and 10 year design storm events. If any portion of a pipe segment appeared to experience surcharged conditions homes connected to the pipe were considered at risk.

Figures 4.10, 4.11 and 4.12 show the sanitary/combined sewer segments where surcharging conditions occur in the modelling for the 2, 5 and 10 year design storm events respectively.

In reviewing the areas at risk of basement flooding for the 2, 5 and 10 year design storm events, there are areas of the system where sewer separation has not occurred as well as areas where sewer separation has been completed. As the severity of the design storm goes from 2 to 10 year the level of risk increases.

4.4.3

Table 4.6 Number of Houses Potentially Experiencing Basement Flooding

Flooding Area	2 Year Design Storm Event	5 Year Design Storm Event	10 Year Design Storm Event
Area # 1	35	46	66
Area # 2	0	0	0
Area # 3	0	0	129
Area # 4	0	50	50
Area # 5	0	90	90
Area # 6	0	22	88
Area # 7	0	0	0
Area # 8	99	154	154
Area # 9	6	29	29
Area # 10	11	11	11
Area # 11	0	0	0
Area # 12	0	70	98
Area # 13	0	0	0
Area # 14	0	6	65
Area # 15	0	0	0
Area # 16	0	0	0
Area # 17	8	44	100
Total	**159**	**522**	**880**

4.4.4 Basement Flooding Control Alternatives

To control the risk of basement flooding, the following control measures were assessed:

- In-system storage; and
- Combination of storage and selective sewer separation.

Alternative controls were also considered, including a Rainfall Derived Inflow/Infiltration Program (RDI/I) and system diversions.

Municipalities have undertaken RDI/I programs to reduce the amount of stormwater entering combined or partially separated sewer systems. Infiltration and inflow reduction can practically be achieved through disconnecting roof leaders from the sanitary sewer, allowing the roof leaders to surface drain. As well, some municipalities have investigated disconnecting foundation drains and redirecting the flow with sump pumps to the surface. The disconnection of foundation drains in the context of Thunder Bay, and for most municipalities, has been found to be impractical to implement.

Diversion of flows were investigated, however, no opportunities were identified to divert flows away from areas with a risk of flooding.

An evaluation of in-system storage and the increase in conveyance capacity through sewer separation was undertaken using the XP-SWMM model refined in Phase 2 as a means to reduce/eliminate the risk of basement flooding. The analysis was conducted for the 2, 5 and 10 year design storm events. No analysis was conducted for design events greater than the 10 years event as it is unlikely a larger event could be intercepted through existing inflow points (catchbasins, roof leaders, etc.).

4.4.4.1 In-System Storage

The XP-SWMM model was used to determine the inline storage volumes necessary to control the HGL to within the pipe for the 2, 5 and 10 year design storm events.

Table 4.7 presents the general location and storage volumes required to reduce the risk of basement flooding during the 2, 5 and 10 year design storm events. The storage volumes determined represent the minimum storage necessary to control the HGL to within the pipe, thus reducing the risk of basement flooding. For each design storm event, storage was introduced at certain locations that were determined to be the most effective in controlling the HGL.

An examination of Table 4.7 revealed that total storage volumes of approximately 440 m^3, 2,800 m^3, and 5,640 m^3 would be required in the South Ward to reduce the risk of basement flooding associated with the 2, 5 and 10 year design storm events respectively. The ultimate configuration of storage is in-line and could be in the

Table 4.7 Storage Requirement to Reduce the Risk of Basement Flooding

Flooding Area	Location of Facility	2 Year Storm Event Volume (m^3)	5 Year Storm Event Volume (m^3)	10 Year Storm Event Volume (m^3)
Area #1	Christina St. between Sprague St to Tarbutt St.	20	120	270
Area #3	Empire Ave. Selkirk Ave. to Franklin St.	0	0	150
Area #4	Simpson St. Cumming St. to Miles St.	0	470	720
Area #5	Cameron St. May St. to Simpson St.	0	500	1,210
Area #6	Alberta St. Southern St. to Atlantic Ave.	0	0	130
Area #8	McMurray St. Ogden St. to Robertson St., and Robertson St. McMurray St. to McMillan St.	290	750	1,150
Area #9	Exhibition Grounds Northern Ave. & Prince Arthur Blvd.	20	270	700
Area #10	May St. Durban St. to Northern Ave.	110	190	300
Area #12	Northern Ave. Syndicate Ave. to Brodie St.	0	110	260
Area #14	Moodie St. Brunswick St. to Selkirk St.	0	70	150
Area #17	Ridgeway St. Vickers St. to Syndicate Ave.	0	320	600
Total Storage Volume (m^3)		**440**	**2,800**	**5,640**

form of a parallel pipe or in-street tank. Site conditions will dictate the most appropriate form of storage and operation.

4.4.4.2 Storage and Selective Separation

A review of areas of the South Ward at risk of basement flooding during the 2, 5 and 10 year design events reveals that the risk of basement flooding remains despite the local drainage separation. For these areas, sewer separation has not removed sufficient rainfall derived I/I flows in the sanitary sewers to reduce/eliminate the risk of basement flooding. In these areas that have undergone sewer separation in-line storage is proposed. If no separation has taken place, sewer separation (or an extension to already separated areas) is proposed through the installation of new storm sewers. Of the 17 drainage areas assessed only three areas have combined sewer areas suited to separation, namely areas 9, 10 and 12.

Table 4.8 presents a summary of proposed separation works for areas 9, 10 and 12 for the 2, 5 and 10 year design events.

Concepts of the proposed works, both storage and separation, are shown in Appendix G for each area at risk of basement flooding for the 2, 5 and 10 year design events.

4.4.5 Sensitivity of Results

A sensitivity analysis was conducted on the results of the flooding analysis to determine the sensitivity of modelling assumptions on the analysis results. As part of Phase 1, the XP-SWMM model was calibrated with available flow data collected from combined and separated drainage areas only as part of Phase 1 field activities. Flow data was not collected for any partially separated area, making it necessary to make assumptions on runoff characteristics for the partially separated areas. The following discusses the sensitivity of these assumptions using two flooding areas, Areas No. 2 and No. 3 as examples.

Each drainage area is composed of pervious and impervious surfaces. In a combined area, the percentage imperviousness is calculated by dividing the total impervious area (roof, roadway, sidewalks, etc.) by the total area. In a partially separated area, the effective percent imperviousness is calculated by dividing the total area into the connected impervious area. The connected pervious area in a partially separated system is less than that of a combined system as surface drainage from driveways and sidewalks is now directed toward a new storm sewer.

The sensitivity analysis undertaken explores the sensitivity of the assumption that 10% of the homes are connected to the combined/sanitary sewer. The percent imperviousness was recalculated assuming that 50% and 80% of the homes are connected to the combined/sanitary sewer in areas 2 and 3.

Figure 4.13 shows the surcharged segments in Areas 2 and 3 for the 10 year design storm event for 10%, 50% and 80% connections. As the level of connection increases the extent of surcharging increases in the two areas. To illustrate the impact of the connection assumption on the hydraulic gradeline Figure 4.14 shows the HGL along Empire Avenue for the three connection levels evaluated. With 10% of roof area connected, surcharge conditions are less severe, in contrast, with 80% of roof area connected, significant surcharge conditions would exist along Empire Avenue in Area No. 3.

The analysis shows that the results of the flooding analysis are highly sensitive to the percent of roofs connected to the sanitary sewer. Using the assumption of 10% there is little likelihood of flooding, but if 50% is used, there is measurable increase in risk. The sensitivity of the results demonstrate the need for local flow monitoring and the only method available to reliably determine the wet weather response in a partially separated system.

4.4.6 Cost

Preliminary costs for the storage and sewer separation alternatives are presented in Table 4.9. Detail costing information is contained in Appendix D. Storage in the form of a tank or a parallel pipe is similar in cost and is not therefore distinguished.

In reviewing the costs, storage is less costly up to a 5 year level, beyond a 5 year level implementing storage with separation in areas 9, 10 and 12 is shown to be more cost effective.

4.4.7 Recommendations

Reports of basement flooding have declined with the completion of the sewer separation program. However, system analysis still identifies areas of risk given the assumption that 10% of roof leaders are connected. Table 4.10 presents the priority areas premised on the level of risk for the 2, 5 and 10 year design storm events.

The sensitivity analysis shows that model assumptions have a significant impact on the assessment of the risk of basement flooding. It is recommended that the City undertake flow monitoring in priority areas before proceeding with corrective action to determine actual local flows in the system to re-evaluate the level of risk. As well, it is recommended that the City work with the community to define an acceptable level of risk.

In the meantime, the City must plan for additional controls to reduce the level of risk to a reasonable level acceptable to the City and the community. It is recommended that the City adopt sewer separation combined with local storage to reduce the risk of basement flooding for the following reasons:

- The installation of new storm sewers will be more conducive to existing City programs.
- Storage facilities will control peak flows at critical junctions.
- Storage facilities can be phased in as the need is identified.

Table 4.8 Sewer Separation to Reduce the Risk of Basement Flooding

Flooding Area	Location		2 Year	5 Year	10 Year
Area #9	Exhibition Grds Exhibition Grds	West of Northern West of Northern	- -	130m @ 900 142m @ 375	130m @ 1200 142m @ 525
Area #10	May St. May St. May St.	Durban & Northern West of Durban Durban to Northern	115m @ 375 235m @ 375	115m @ 450 235m @ 450	115m @ 525 235m @ 450
Area #12	Brodie St.	Durban to Northern	-	125m @ 525	125m @ 525

Table 4.9 Cost of Basement Flooding Control Alternatives

Level	Alternative Cost ($)	
	In-System Storage	Storage/Separation
2	340,000	410,000
5	2,123,000	2,130,000
10	4,260,000	3,900,000

Table 4.10 Basement Flooding Risk Priority Areas

Priority Group	Area No.	Comments
1	Areas 1, 8, 9, 10 and 17	Risk of basement flooding less than 2 years
2	Areas 4, 5, 12 and 14	Risk of basement flooding less than 5 years
3	Areas 3, and 6	Risk of basement flooding less than 10 years
4	Areas 2, 7, 11,13 15 and 16	Risk of basement flooding greater than 10 years

4.5 Stormwater Control

Continued enforcement and application of the MOEE "Stormwater Management Practices Planning and Design Manual", June 1994 is important with respect to new developments, construction of new storm outfalls and in retrofitting existing outfalls. Beyond the application of these guidelines, no end-of-pipe stormwater controls are recommended.

Stormwater source controls are addressed under pollution prevention and public education initiatives.

4.6 Thunder Bay WPCP

4.6.1 Pilot Studies

An important component of the PPCP was the assessment of the wastewater treatment plant and the issues affecting upgrading and expansion requirements for the 2016 design period and the provision for full secondary treatment. A Technical Memorandum was prepared "Evaluation of Secondary Treatment Upgrade Options", October 1995, addressing these issues (Appendix B). The memorandum recommends the City initiate a year long pilot study of short listed secondary treatment process technology at the Thunder Bay WPCP. The technologies recommended for pilot testing include BAF (Biological Aerated Filters) and an optimized CAS (Conventional Activated Sludge) design. As a result of discussions with City staff in Windsor, Ontario, the City of Thunder Bay has decided to include in the pilot test the trickling filter/solids contact process.

The results from this study will be used to identify the most appropriate site specific design parameters for each technology resulting in the most cost effective and appropriate technology. As well, the pilot study will provide capital and operating costs and experience, provide a comparison of performance and support an application to MOEE for approval of any non-standard treatment technology or process design. As indicated in the Technical Memorandum undertaking a pilot study to refine the design parameters, treatment technology and processes could result in savings in the neighbourhood of $4 to $6 million in capital costs over conventional plant design.

4.6.2 Optimization of Phosphorus Removal

The Certificate of Approval for the Thunder Bay WPCP states that the plant is in non-compliance if the annual average final effluent phosphorus concentration is in excess of 1 mg/L. Currently, the City is undertaking to improve phosphorus removal at the existing primary treatment plant to achieve compliance of less than 1 mg/L on a monthly basis.

It is recommended that the City continues their efforts and address improved removal through the pilot study.

4.6.3 Digester Optimization

It is suspected that the existing anaerobic digesters are experiencing mixing limitations that reduce their effective volume. The digesters are adequately sized for biosolids generation with the implementation of secondary treatment with proper mixing. It is recommended that the City address the question of mixing inadequacy of the anaerobic digesters.

4.7 Pollution Prevention

Pollution prevention measures include source controls and initiatives that reduce the amount of stormwater related pollutants from entering either a combined sewer or storm sewer system. Source controls and prevention measures tend not to have large capital expenses associated with them but can increase operational costs. The objective of pollution prevention measures is to minimize the accumulation of pollutants on streets and other tributary land areas as well as to reduce the entry of pollutants into the collection systems. Typical pollution prevention measures can include, but are not limited to the following:

- Street cleaning
- Public education programs
- Recycle programs
- Fertilizer and pesticide control
- Soil erosion control
- Commercial/Industrial control
- Operation and maintenance practices
- Catchbasin Cleaning

Inherent in the City's present operation and maintenance practices is pollution prevention. The O&M programs were previously reviewed and discussed and are not incorporated in the review of pollution prevention measures.

The City of Thunder Bay has initiated a number of pollution prevention measures that are reviewed in the following sub-sections including recommendations on program improvements.

4.7.1 Street and Catchbasin Cleaning

Street litter and pollutant build up can be a significant source of certain pollutants. In the City of Thunder Bay there is 17,150 km of curb that is cleaned a minimum of twice per year (spring and fall). In actual fact the frequency is greater on residential streets, and on major thoroughfares (i.e. Victoria Avenue) weekly street cleaning is conducted. In conjunction with street sweeping the City undertakes street flushing and sidewalk sweeping. No improvements to the existing street sweeping program are recommended.

The benefit of catchbasin cleaning is similar to street cleaning. Catchbasin cleaning will remove accumulated sediments and debris before they reach the combined or storm sewer systems. The catchbasins are cleaned once per year in the North Ward and approximately 50 to 60% of the catchbasins are cleaned in the South Ward annually. It is recommended that all catchbasins in the City are cleaned at least once per year and preferably twice.

4.7.2 Public Education

The City undertakes public awareness programs to promote good practices in the City of Thunder Bay. The City is involved in funding and participating in the Thunder Bay 2002 program promoting environmentally friendly lifestyle choices addressing issues such as, recycling, hazardous waste, water conservation, composting and energy efficiency. As well, the City uses mail inserts in billings to promote the Hazardous Waste Depot, Recycling Depots and to provide notification on other City related business (i.e. lawn watering restrictions).

Public education is one of the most effective pollution prevention tools available to the City. The objective of public education is to inform and educate the public on specific issues related to CSO and stormwater control. The City can be more pro-active in this area to promote "good practices" that will ensure the local water courses do not degrade. The local rivers are a valuable resource to the City and should be protected as such by drawing the link between what goes into the sewer (storm or sanitary) and what goes out. It is important for the City to coordinate public education programs with local environmental groups, RAP, MNR and MOEE.

One area of public education for the City to address is the removal of roof downspouts. A bylaw exists; however, the City has not actively enforced downspout disconnection. Information packages for the public can lead to voluntary disconnection.

4.7.3 Water Conservation

As part of the Thunder Bay 2002 program the City is actively promoting water conservation as a means to reduce water and wastewater demands. The City's involvement in this program will likely lead to cost savings through less demand.

4.7.4 Industrial Pre-Treatment and Sewer Use Bylaw

In 1990 the City of Thunder Bay participated in the provincial MISA Demonstration Project. Participation in the project led to the development of a Sewer Use Control Program and a Sewer Use Bylaw. The Sewer Use Bylaw allows the City to impose user charges based on sewage loadings to their sanitary collection system from a particular source. At present three industries are subject to sewer surcharge fee and one of the industries has a pretreatment program. The Sewer Use Bylaw also regulates discharges to the stormwater collection systems requiring industries to create a Best Management Practices (BMPs) to reduce pollutant loads into the stormwater collection systems.

Industrial pretreatment is promoted by the City through Best Management Practices (BMPs). The basis for the BMPs program is to handle wastes at the source and to promote waste reduction and to improve waste handling procedures. To date, the City has prepared a Motor Vehicle Service BMP Requirements program for service stations. The City plans to implement similar BMP plans for restaurants, photo processors and industrial laundry facilities.

The City has made significant progress in the implementation of BMPs and should continue with the existing programs and include new industries in their program. The City should identify opportunities to coordinate their initiatives with RAP or other government agencies in the implementation of BMP programs. An important component of the Sewer Use Control Bylaw and the industrial pretreatment programs is enforcement. Without enforcement through regular inspection, monitoring and sampling programs the City will not be able to ensure compliance.

4.8 Recommended Short Term PPCP Summary And Cost

Table 4.11 presents a summary of the recommended Short Term PPCP programs, their qualitative benefits, and the associated costs. Detailed costing information is provided in Appendix D.

If all components of the Short Term PPCP were implemented the cost over the next 5 to 10 years would be in excess of $9.0 million. It is not anticipated that all programs recommended in the Short Term PPCP will be implemented in a 5 to 10 year period. In fact, some of the programs identified are considered long term, however, they will start as part of the Short Term PPCP.

Table 4.11 Recommended Short Term PPCP and Costs

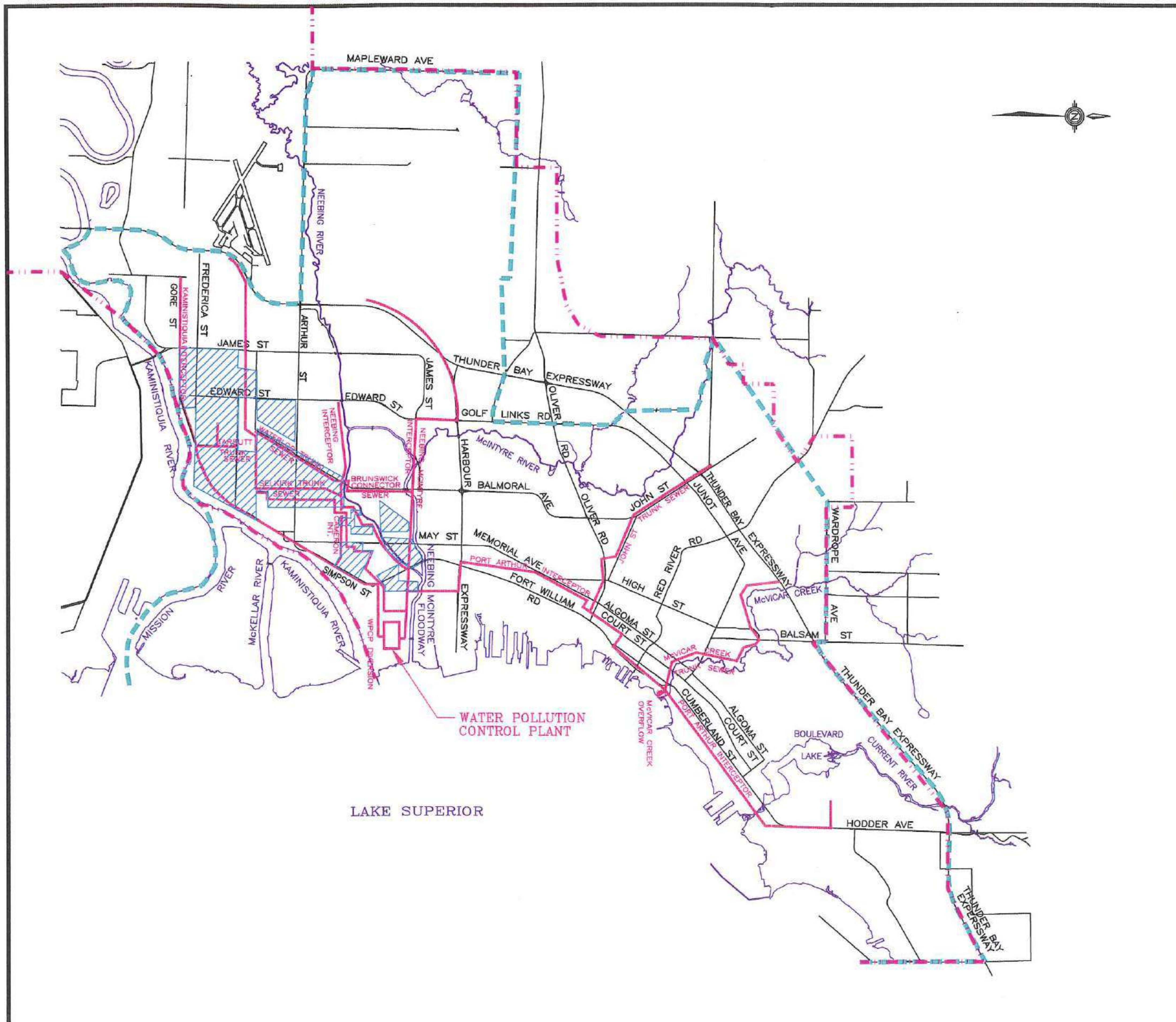
Program	Program Description	Qualitative Benefits	Cost
Collection System Management			
Operation and Maintenance			
CCTV Inspection	• Inspect and inventory collection systems over 10 year period. • Include sewer flushing • Incorporate manhole inspection and inventory • Re-inspect problem sections on a 5 year cycle until rehabilitated	• Condition assessment and information • Identification of structural defects • Assist in prioritizing rehabilitation programs • Identification of extraneous flow sources • Increased pipe capacity with cleaning • Establishes an ongoing sewer and manhole inspection program for the City • I/I reduction creates more system capacity and reduces treatment needs.	$2.8 million over 10 years $1.1 million years 1-5 $1.7 million years 5-10
Manhole Inspection	• Inspect and inventory all manholes in the City • Combine with CCTV program	• Provides structural condition information • Identify extraneous flow sources • Prioritize rehabilitation projects • I/I reduction creates more system capacity and reduces treatment needs	$0 • included in CCTV inspection program
CSO Inspection & Maintenance	• No change recommended to existing programs	• Reduce likelihood of equipment failure	$0 • ongoing program
Storm Sewer Outfall Inspection & Maintenance	• No change to existing spring programs of inspection and maintenance • Conduct outfall survey to locate and document outfalls • Identify outfalls with dry weather seepage and estimate flow rate. • Collect dry weather seepage water quality sample for conventional and metals analysis • Inspect problem flap gates after every rainfall event	• Quantifies dry weather seepage/ extraneous flow rates • Sampling program could identify cross-connections or other pollutant sources. • Assist in the enforcement of the Sewer Use Control Bylaw • Reduce river intrusions	$0 • Conduct with existing staff • Cost share sampling program with MOEE and Lakehead Conservation Authority
Sewer Flushing	• Coordinate existing program with CCTV inspection • Expand to 100% average in South Ward	• Increased pipe capacity • Sewer maintenance	$0 • Included with CCTV inspection program
Pump Station Maintenance	• No changes to existing programs of inspection and maintenance	• Reduced likelihood of equipment failure	$0 • Ongoing program
Structural			
Neebing/McIntyre Improvement	• No recommendation, study pending	• Structural stability • Reliable service	$0 • Study pending

Table 4.11 Recommended Short Term PPCP and Costs

Maximize Use of Existing Facilities			
1. Diversions	• Divert excess wet weather flows from the Neebing Interceptor to the Brunswick Connector sewer and Neebing/McIntyre Interceptor • Divert flow from Neebing Interceptor to Cameron Trunk	• Provides much needed hydraulic relief to the Neebing Interceptor	• $21,000 Neebing/Brunswick • $22,000 Neebing/Cameron
2 Extraneous Flow Reduction	• Enforce By-law to remove rainwater leaders from sanitary sewers • Unable to develop or initiate a program without system inventory		-
3. Catchbasin Cross-Connection	• Seal, restrict and provide new catchbasin leads to limit wet weather inflow to sanitary system • New storm sewer to disconnect 3 catchbasins	• Reduces wet weather load in sanitary sewer • Reduces treatment needs • Foundation for Long Term removal plan	• $12,000 CB sealing & flow restriction • $120,000 New storm sewer
4. Outfall Flap Gate Replacement	• Replace outfall flap gates as required with "duck bill" design • Ensure existing gate seals are in good condition	• Reduced inflow • Increase in available pipe capacity and reduction in treatment needs • Less operational and maintenance required for "duck bill"	$300,000 • Replacement of 8 outfall gates
5. James & Quebec Connection	• Construct new sanitary connector to existing sanitary system	• Removal of direct sanitary connection to storm sewer and outfall to Kaministiquia River	$93,000
Monitoring Program	• Initiate 5 year flow monitoring program • Establish at least 8 permanent monitoring stations in essential interceptors sewers • Purchase two velocity-area meters and one rain gauge	• Additional model calibration data • Qualify extraneous flow • On-line collection system information that can be used to develop operational strategies	$115,000
XP-SWMM Model	• Update model calibration with current flow data • Update model network with inspection records • Refine analysis to assess PPCP status • Expand model into local areas	• Improved information related to system hydraulic performance • May reduce the works identified through the use of better information • Ability to assess changing conditions beyond the PPCP study	$0
		CSO Control	
Ridgeway Oil/Grease Separator	• Replace existing Oil/Grease separator with a larger unit, or • Provide bypass of peak flows to prevent flushing	• Improved Oil/Grease capture • Reduction in contaminated discharges	$25,000 to $30,000
Floatables Control	• Identify sources of floatables • Retrofit CSO chambers with baffle plate for floatables control • Retrofit if floatables identified	• Reduced floatables will improve aesthetics • Source identification	$0 • No cost identified

Table 4.11 Recommended Short Term PPCP and Costs

Program	Program Description	Qualitative Benefits	Cost
CSO Regulator Replacement Program	• Replace Kaministiquia regulators with either a vortex or Hydroslide type device as required	• More reliable performance • Low cost • Reduced maintenance	$175,000 • Replaces 11 regulators $15,000 • Replaces RK2 regulators
Regulator Settings	• Adjust RK2 regulator to increase interception rate	• Achieve minimum 90% volumetric control	$0
Basement Flooding			
South Ward Basement Flooding	• Update risk assessment with local flow data and improved model calibration • Determine community standard • Replace existing combined pipes with separate • pipes to achieve a desired level of control • Provide system storage in previously separated areas	• Eliminate/minimize the risk of basement flooding	$340,000 to $4.3 million over a 10 year period (2 year to 10 year level of risk)
Stormwater Control			
Stormwater Management	• Continue enforcement and application of "Stormwater Management Practices Planning and Design Manual"	• Improved stormwater quality and quantity control	-
Thunder Bay WPCP			
Pilot Study	• Initiate year long pilot study investigating treatment technologies for secondary upgrade to WPCP	• Significant savings in capital cost of secondary facility • Design parameters suited to Thunder Bay • Trained staff familiar with secondary process and operations	$300,000 to $400,000
Phosphorus Removal	• Continue with existing optimization efforts	• Improved phosphorus removal to meet effluent requirements	$0
Digester Optimization	• Improve digester mixing	• With proper mixing digester volume will be sufficient for full secondary facility	$0
Pollution Prevention			
Street Cleaning	• No change to existing program	• Removal of pollutant	$0
Catchbasin Cleaning	• Increase scope of program to 100% coverage	• Removal of pollutants before they enter storm sewer system	• Increase annual operating budget
Public Education	• No changes to existing programs • Promote downspout disconnection • Co-ordinate efforts with RAP, MNR etc. • Promote "good practices"	• Informed public • Reduce demand for water and wastewater treatment capacity	$0
Total Cost			**$9.3 million**



LEGEND

Ultimate Urban Service Area

Study Area Boundary

Interceptor/Trunk Sewers

Priority CCTV Areas

NOTE:
Priority Areas & elements include:
1) areas of historical basement flooding
2) all Interceptor & Trunk Sewers
3) all pipe greater than 450mm diameter

UPDATED JANUARY 1992

0 500 1000 1500 2000
SCALE IN METRES

Figure 4.1
Priority Areas for CCTV Program

W2O WARDROP ENGINEERING INC.

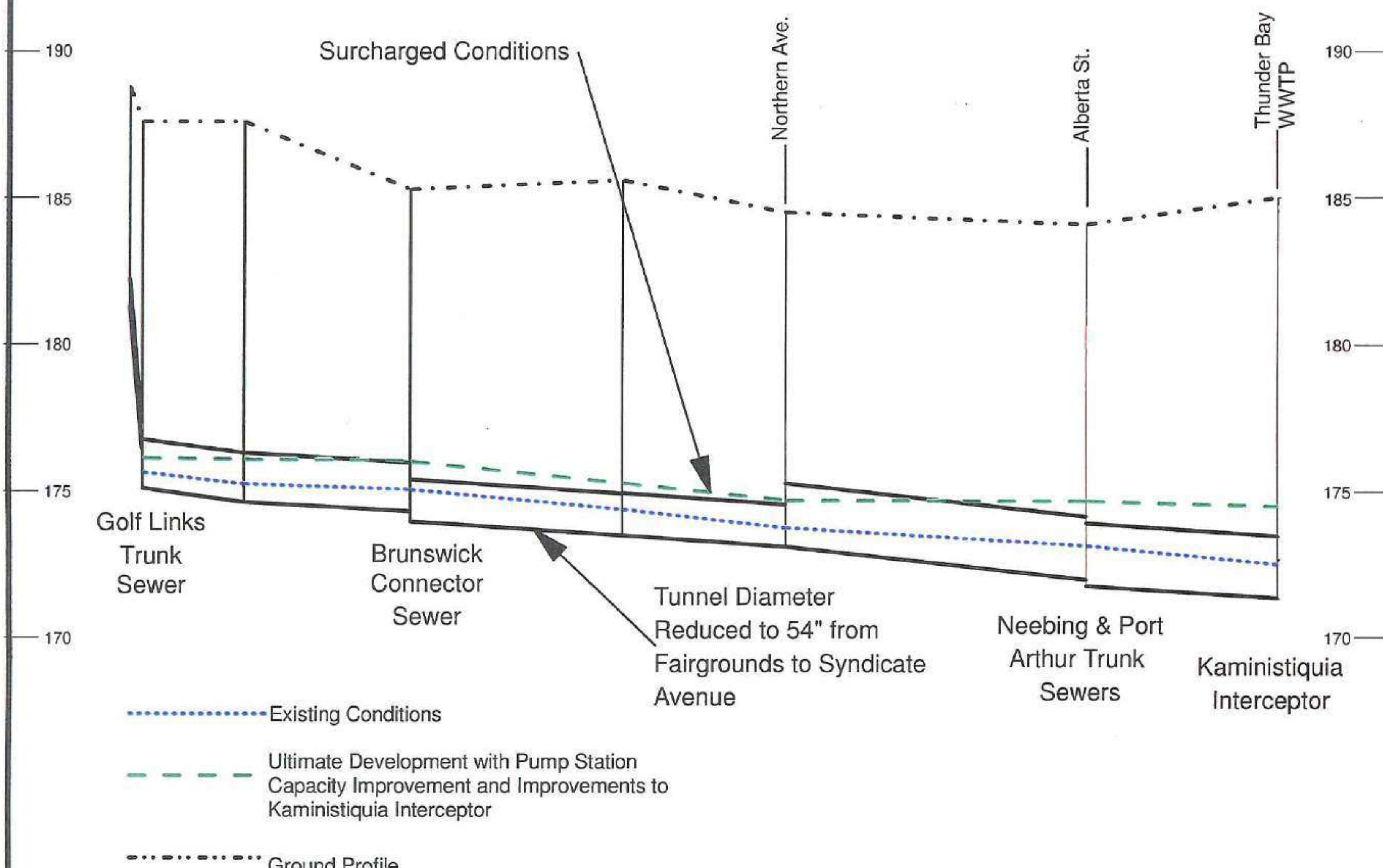


Figure 4.2
1,370mm (54") Diameter Neebing/ McIntyre Inceptor
10 Year Design STorm HGL and Ultimate Development

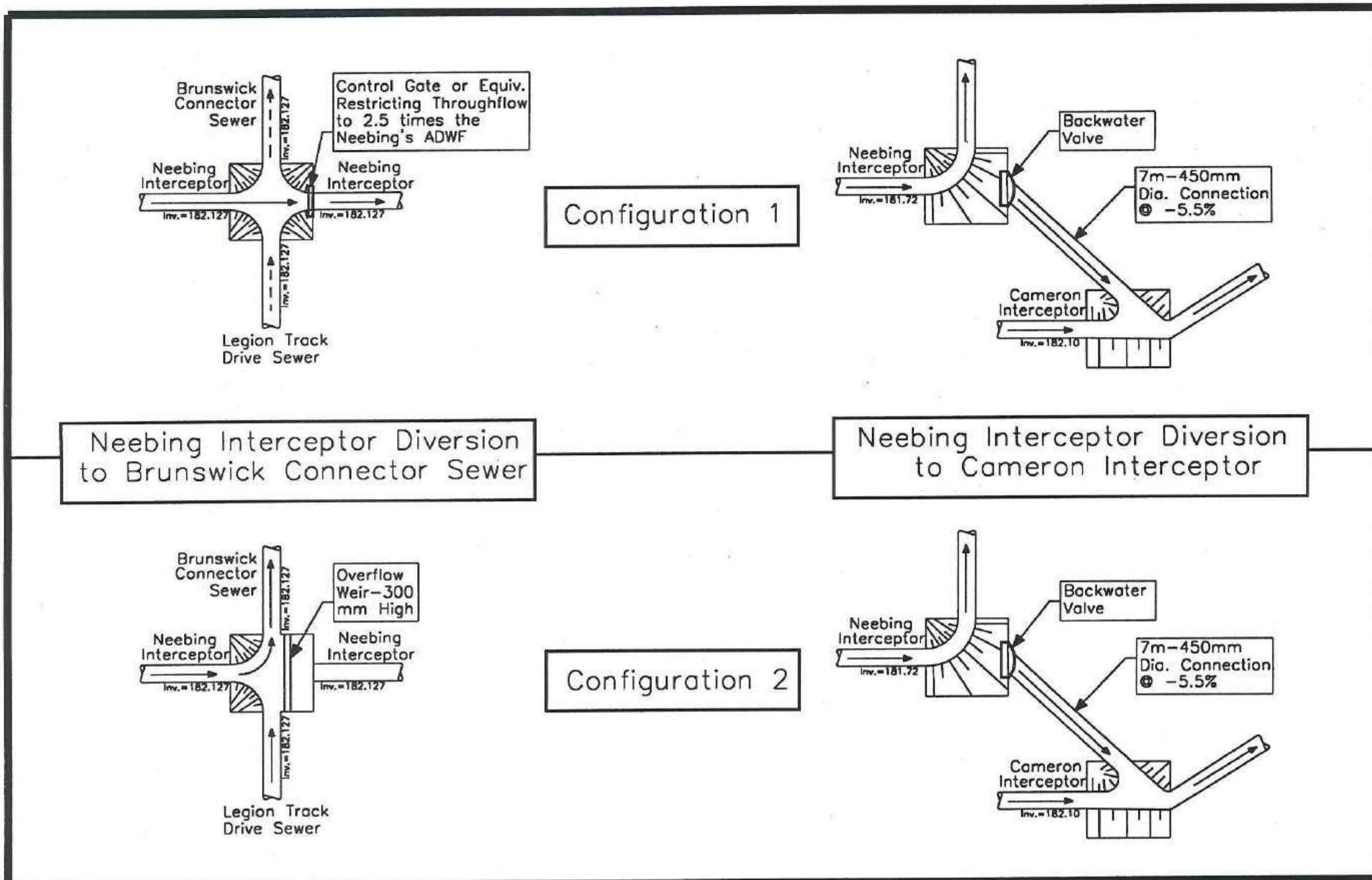


Figure 4.3
Neebing Interceptor Diversions

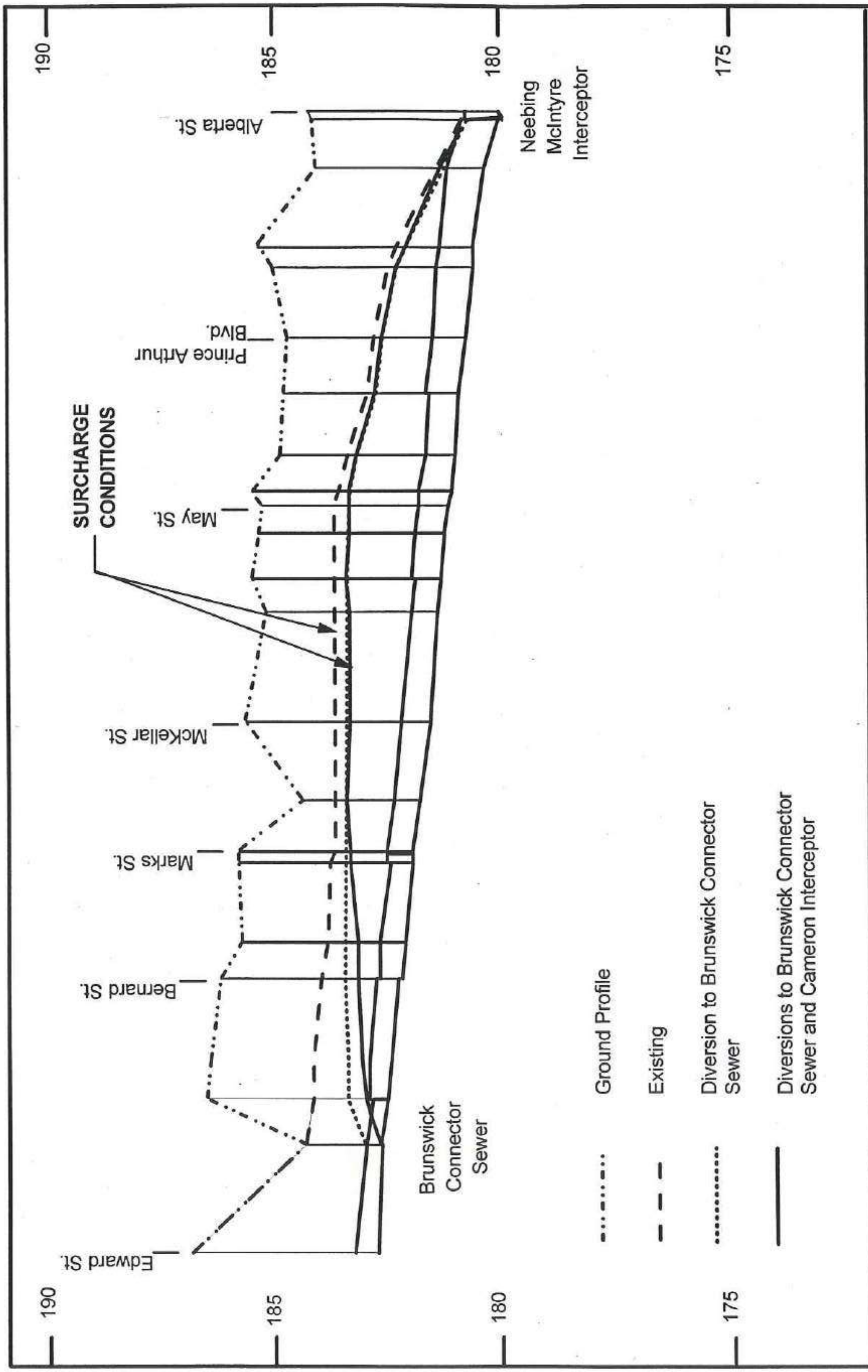


Figure 4.4
Neebing Interceptor
10 Year Design Storm Event HGL

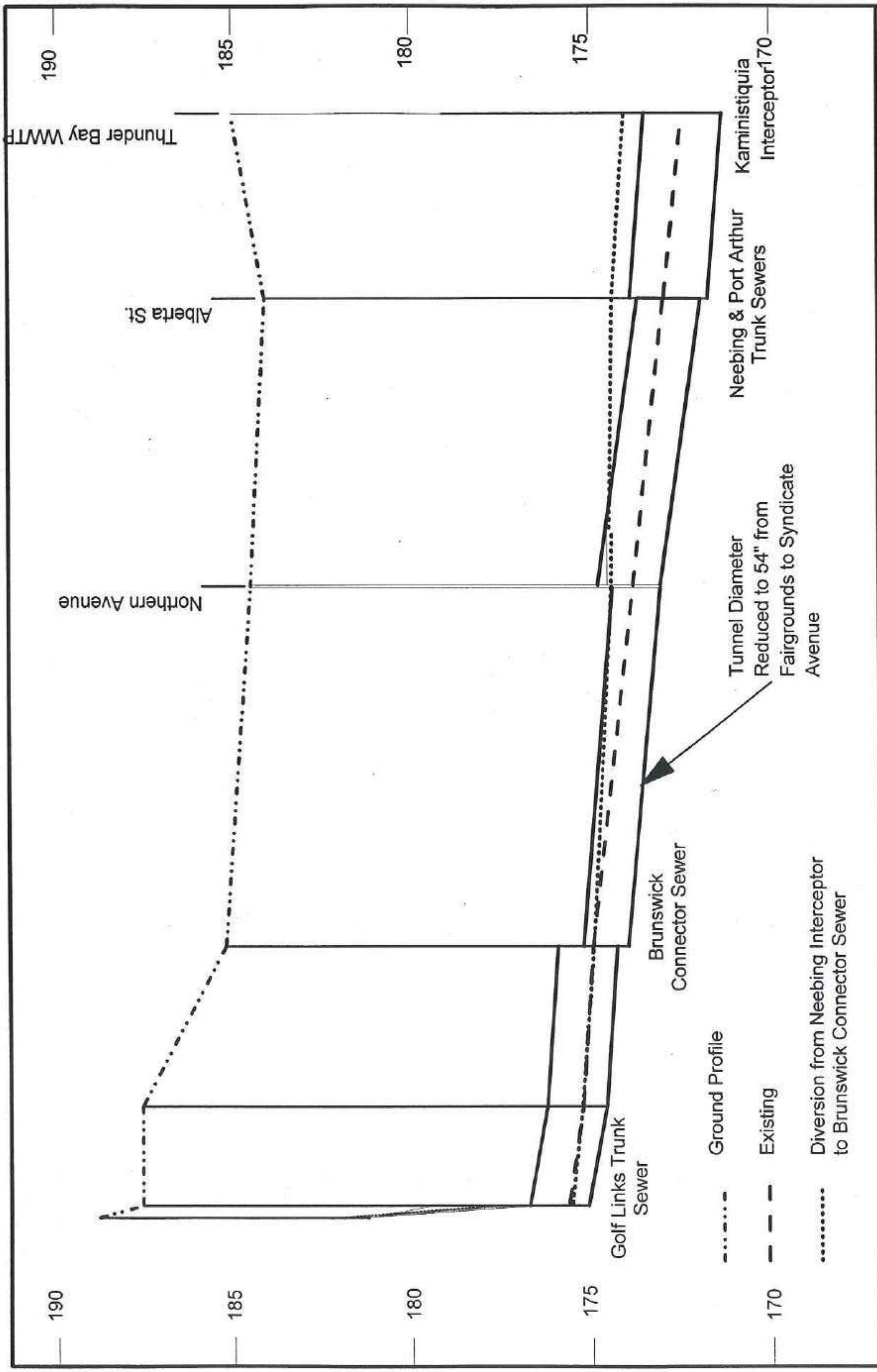


Figure 4.5
Brunswick Connector Sewer and Neebing/McIntyre Interceptor
10 Year Design Storm Event HGL

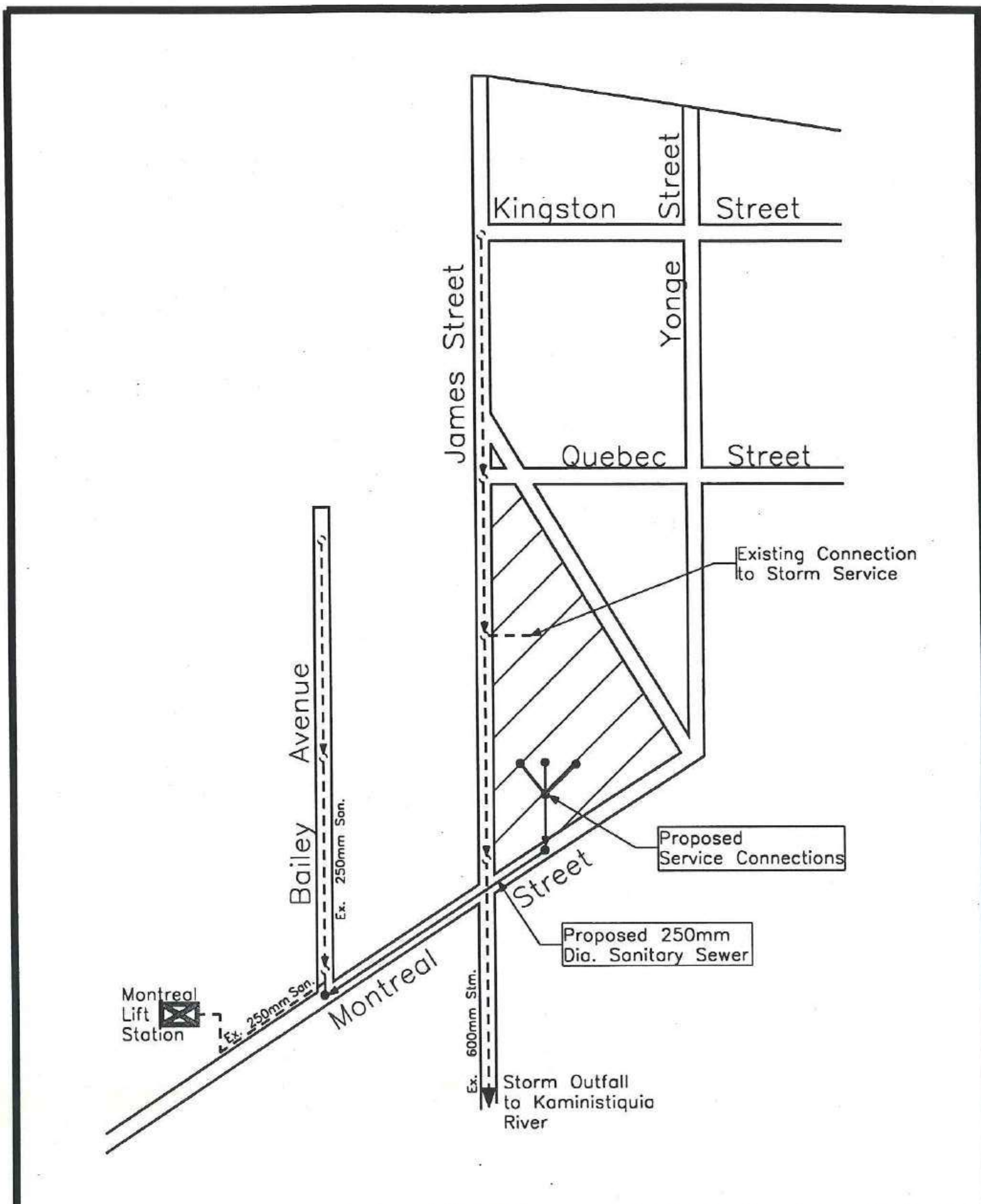
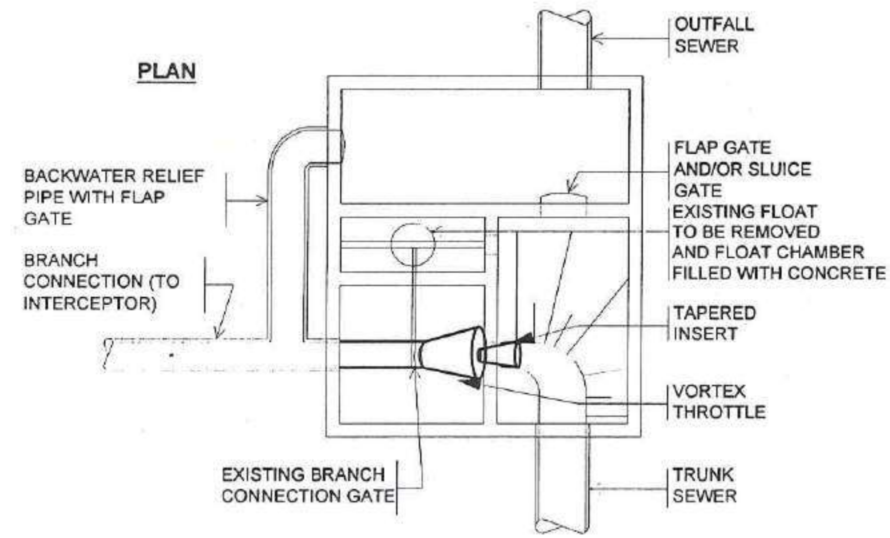


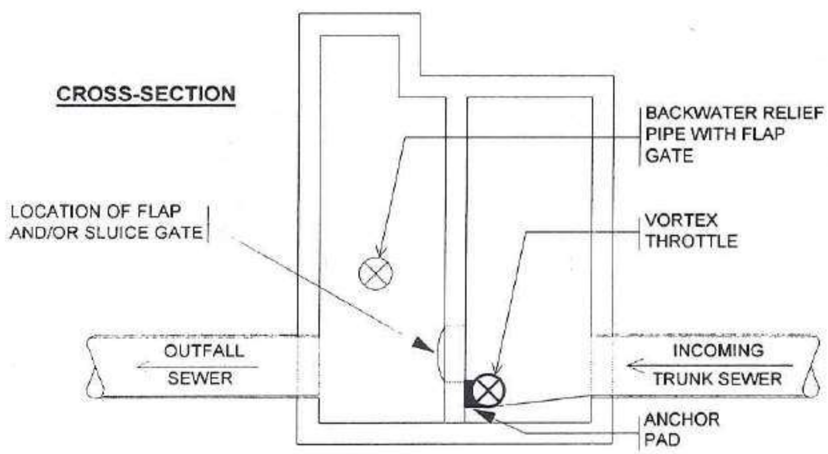
Figure 4.6
James and Quebec Street Servicing

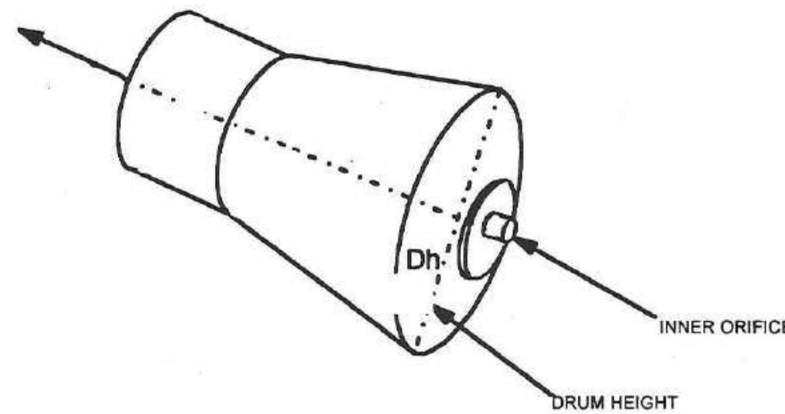
PLAN



TYPICAL INSTALLATION OF VORTEX THROTTLE IN FLOAT GATE REGULATOR

CROSS-SECTION





DETAIL OF VORTEX THROTTLE

INSTALLATION NOTES:

1. **Vortex Throttle is to be bolted to the back side of the existing wall.**
2. **A tapered insert should be installed through the regulator wall to ensure that the transition through the wall is smooth.**

Figure 4.7
Typical Vortex Installation

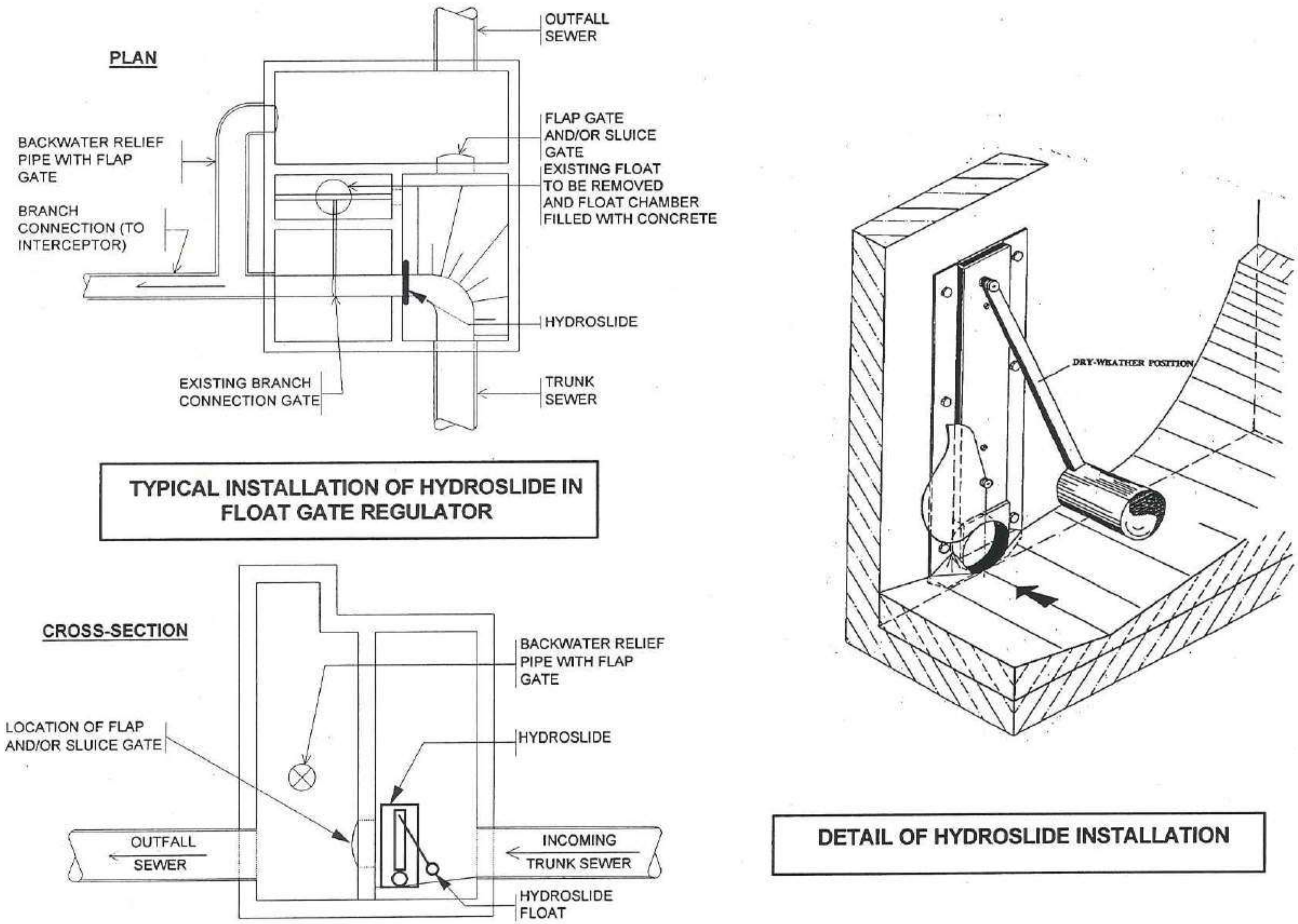


Figure 4.8
Typical Hydroslide Installation

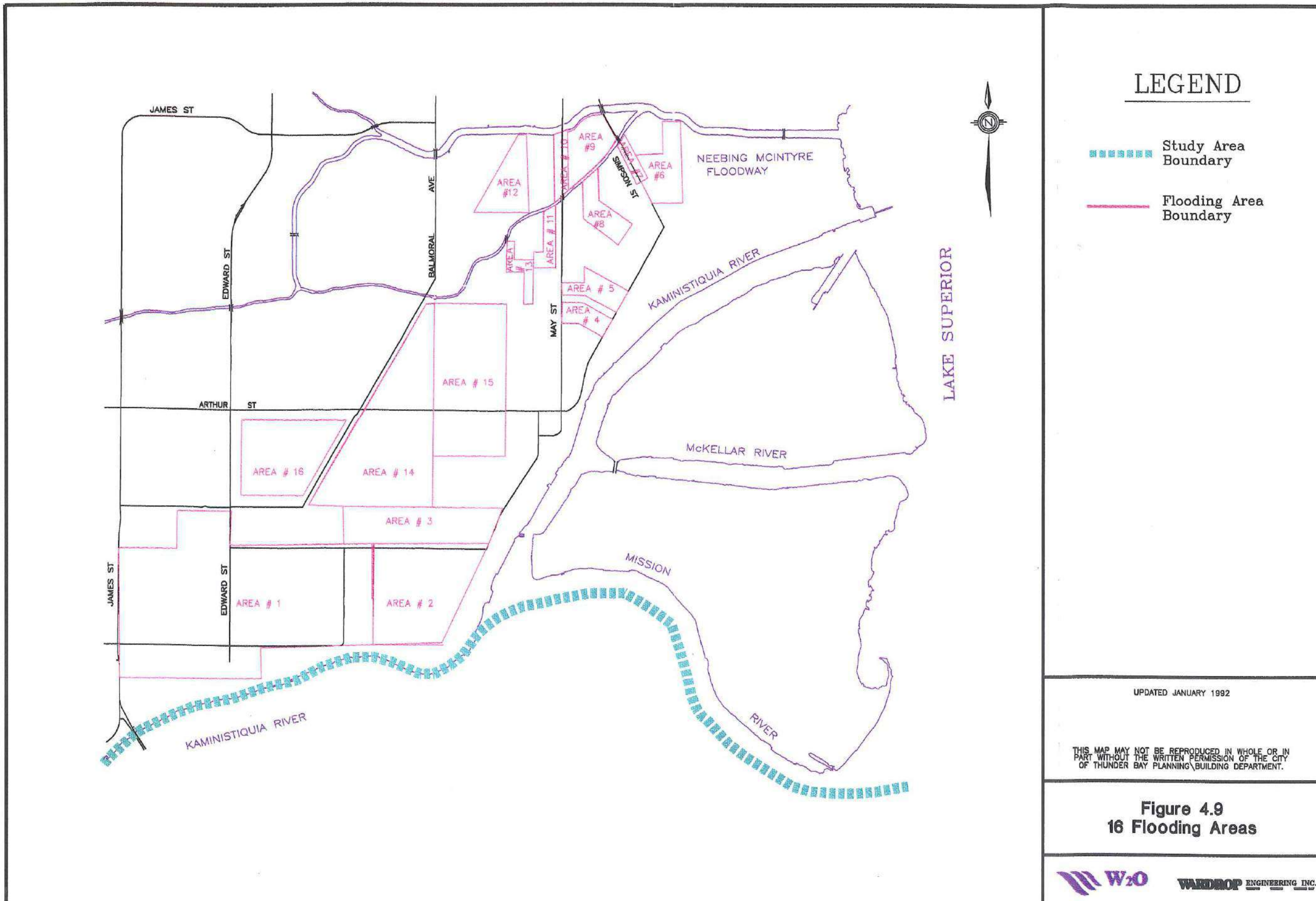


Figure 4.9
16 Flooding Areas

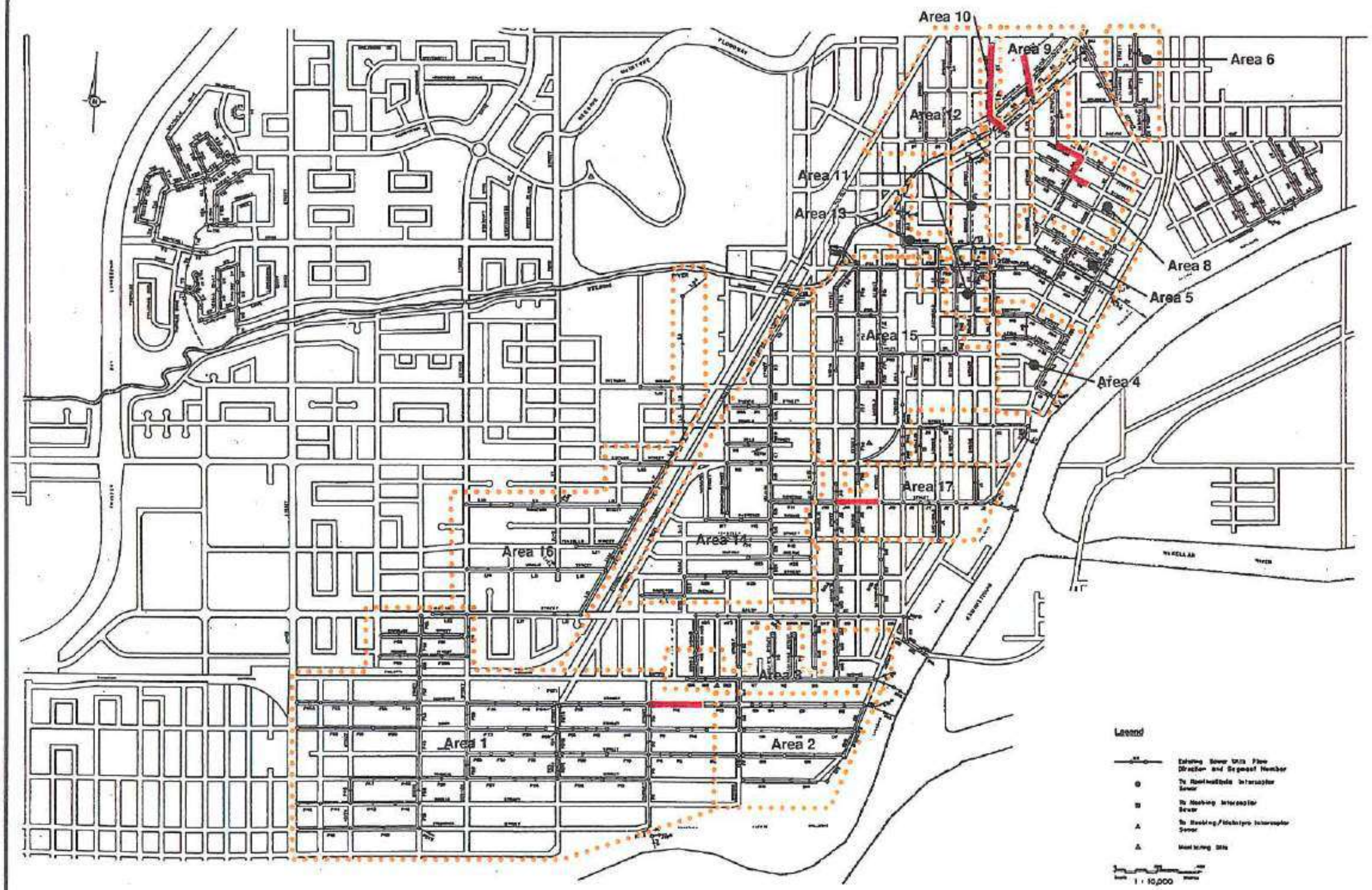


Figure 4.10 Sewer Segments with Surcharge for the 2 Year Design Event

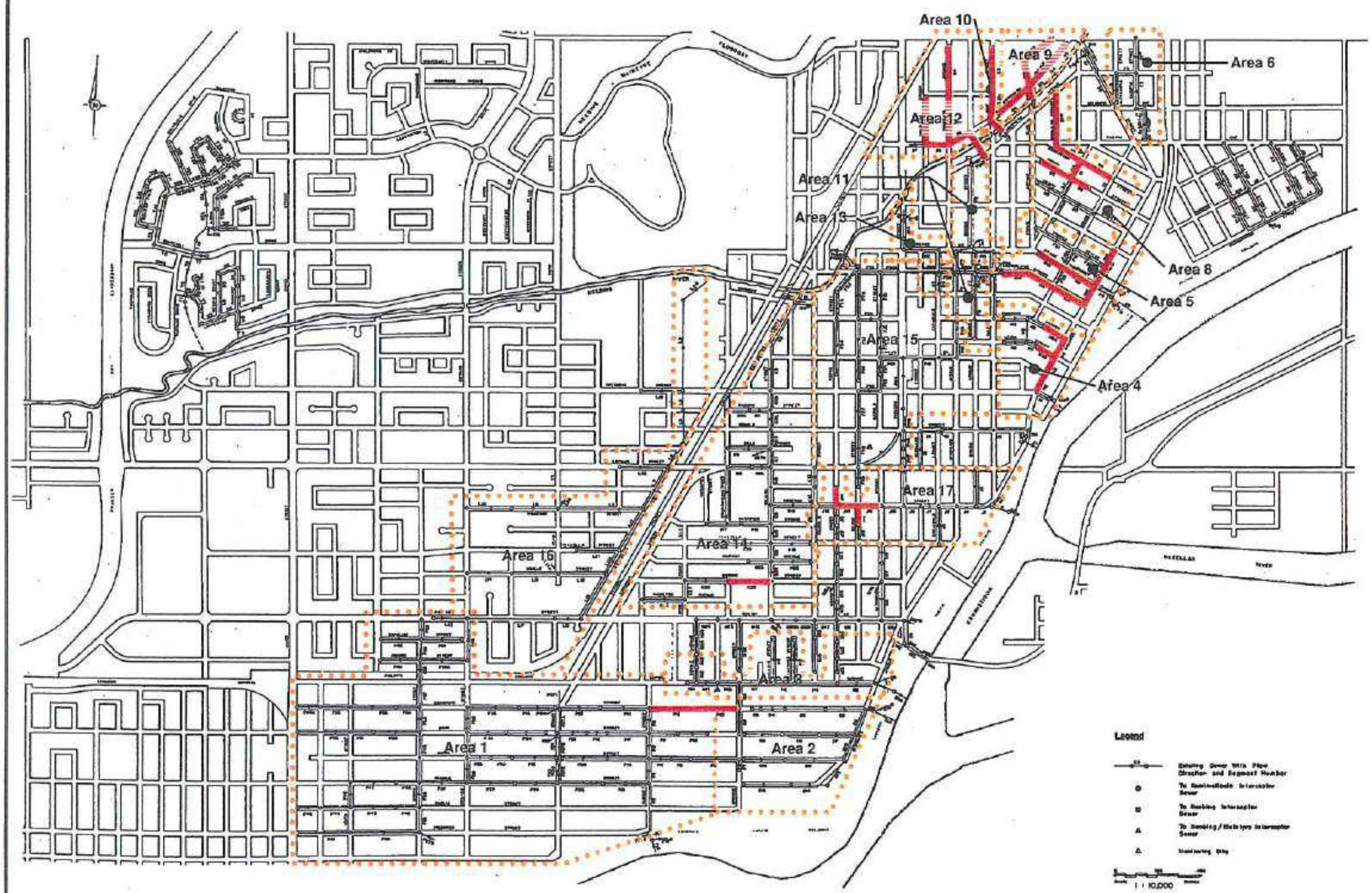


Figure 4.11 Sewer Segments with Surcharge for the 5 Year Design Event

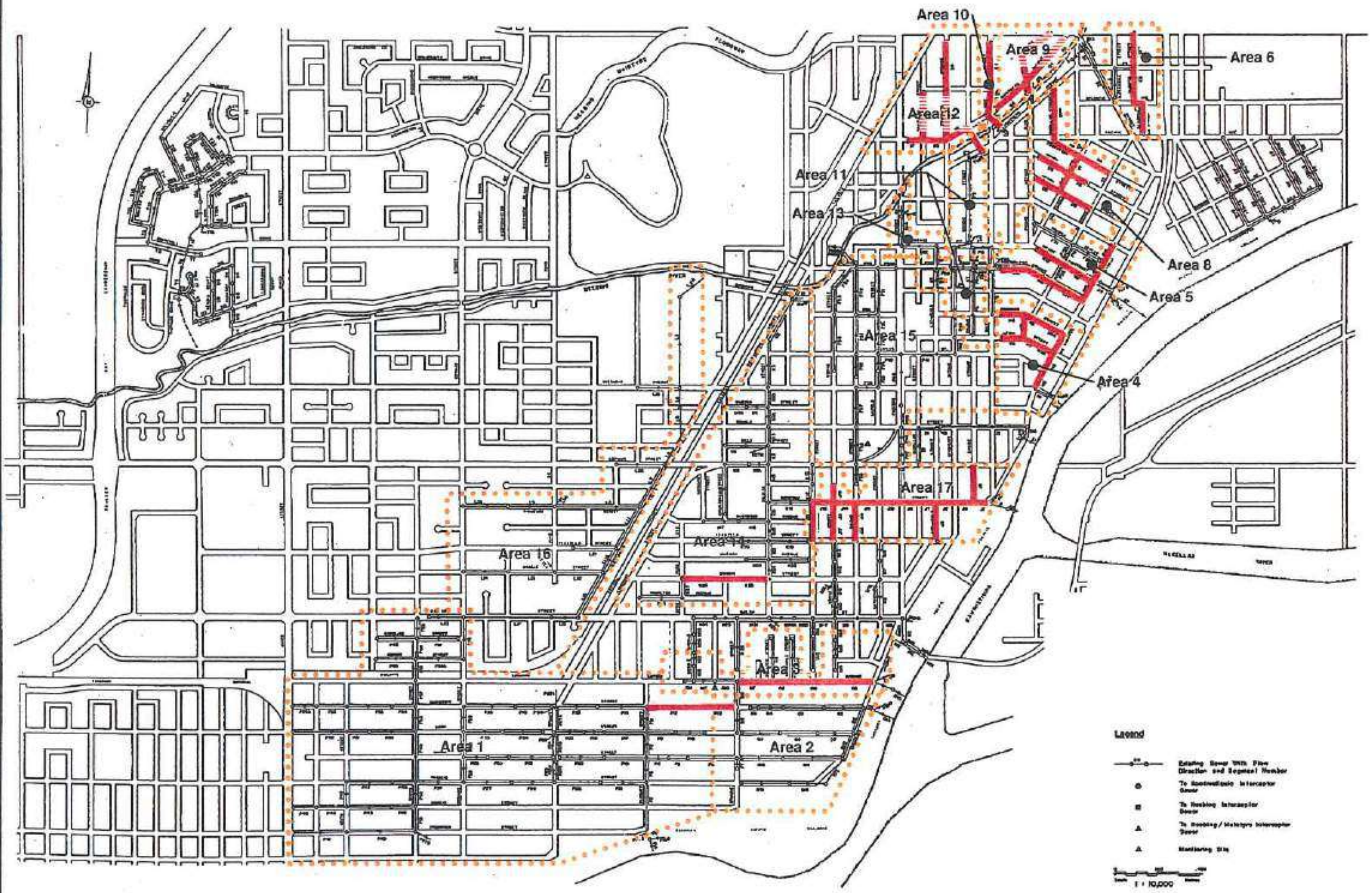


Figure 4.12 Sewer Segments with Surcharge for the 10 Year Design Event

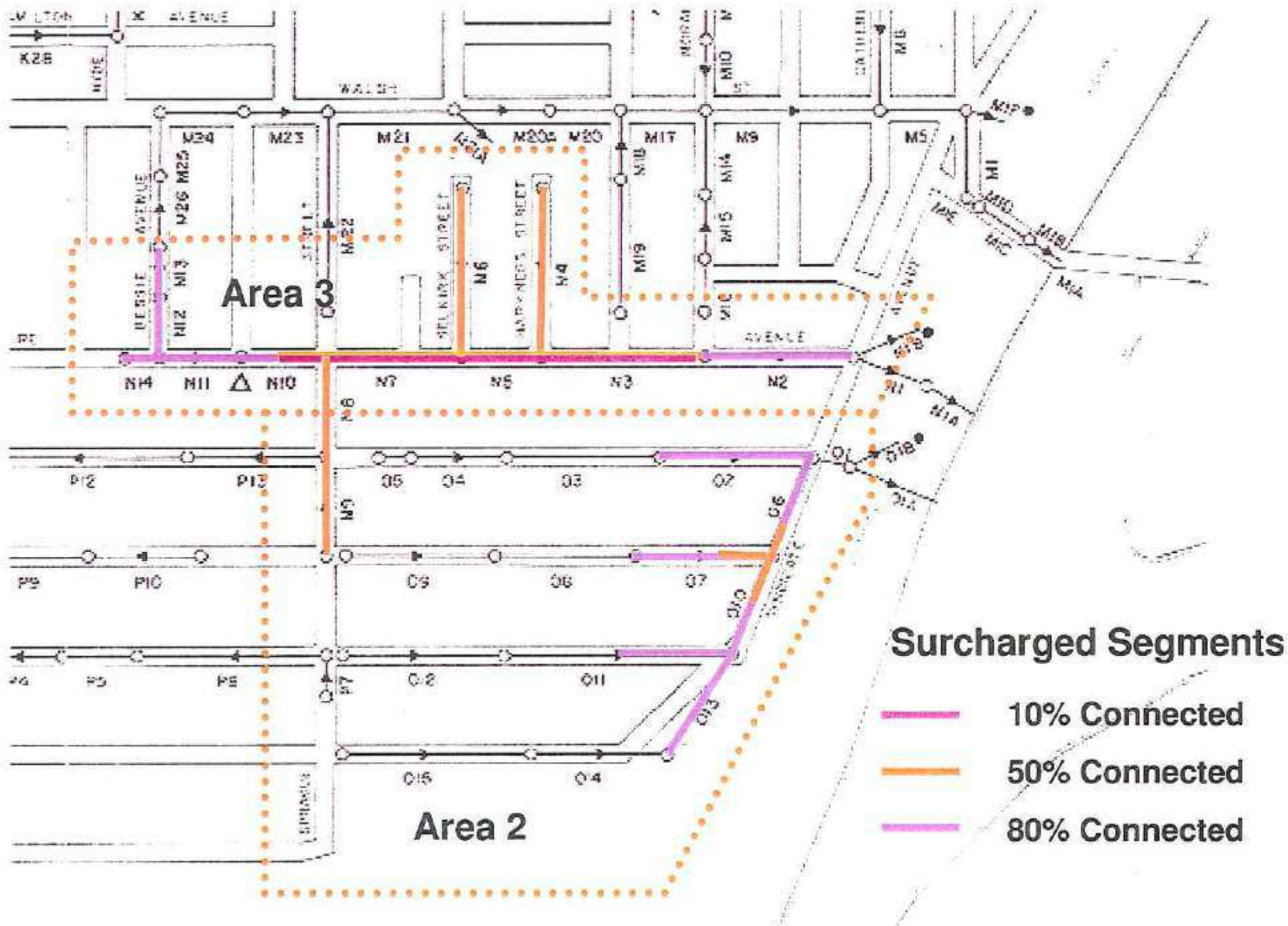


Figure 4.13 *10 Year Design Storm*
Surcharged Segments in Areas 2 and 3 for 10%, 50% and 80% Connected

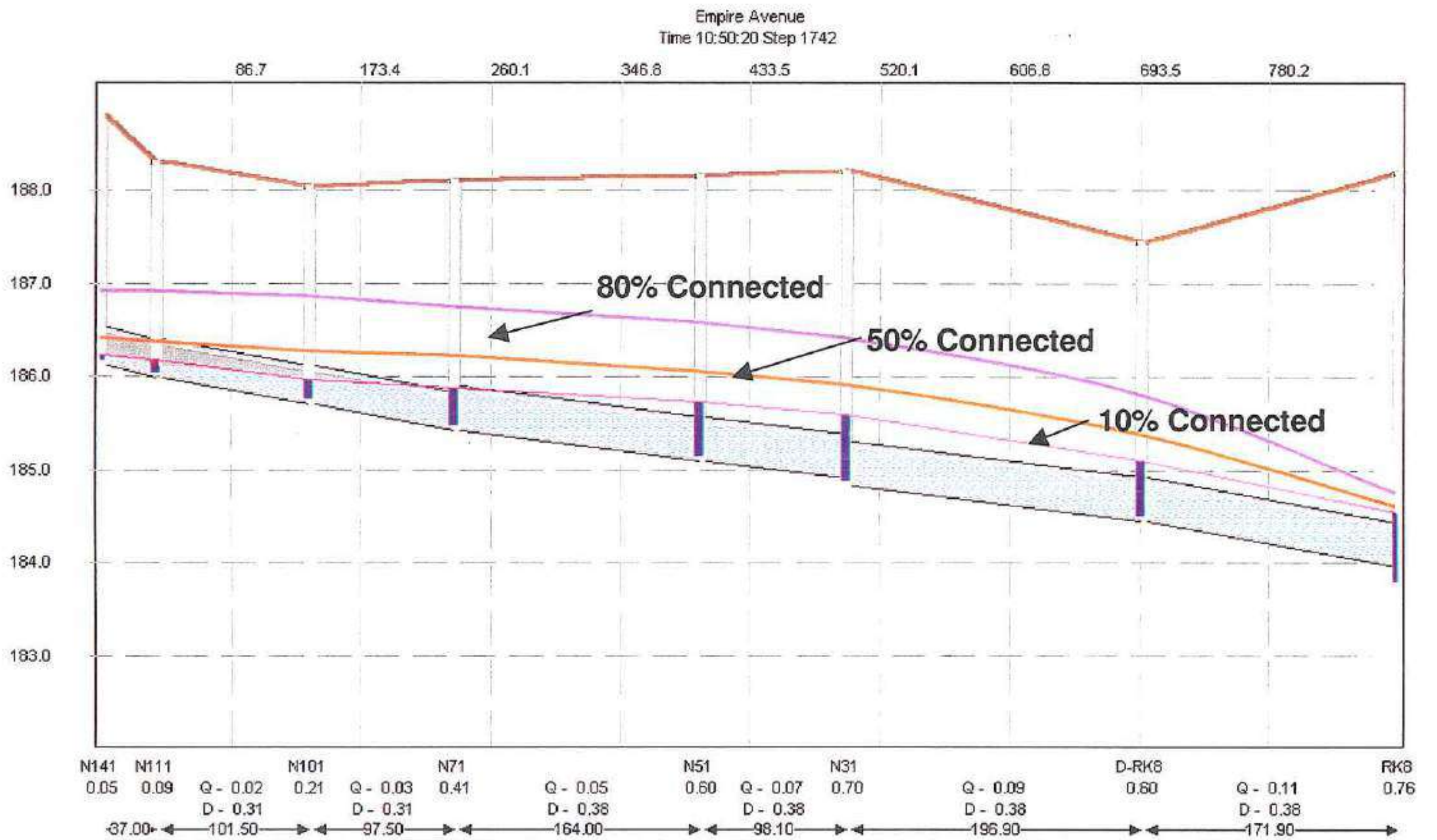


Figure 4.14 *10 Year Design Storm*
Hydraulic Grade Line on Empire Avenue for 10%, 50% and 80% Connected

5. LONG TERM POLLUTION PREVENTION AND CONTROL PLAN

5.1 Overview

The following section outlines in detail the programs and control measures proposed for the Long Term PPCP. An extension of the Short Term PPCP, the Long Term PPCP programs will ensure sustainable development with a level of collection system performance, wastewater treatment and stormwater control acceptable to the community and regulatory agencies.

Controls associated with Long Term PPCP programs tend to be more capital intensive and are to be implemented over a 20 to 25 year planning period or, in some cases, as the need or services are required (i.e. new development).

The key elements of the Long Term PPCP address the same areas of concern as the Short Term PPCP, namely:

- Collection System Management
- CSO Control
- Stormwater Management
- Thunder Bay WPCP

5.2 Collection System Management

As previously discussed in the Short Term PPCP, the management of any collection system is extremely important to ensure reliable service, to protect the significant capital investment made and to be able to make informed operational and management decisions.

Collection system management in the context of the Long Term PPCP is associated with the control and management of excess wet weather flows, the need for development capacity, elimination of basement flooding, and the need to ensure a reliable level of service. The purpose of the following sub-sections is to present the details of costs and physical characteristics of each proposed project and the evaluations that led to the recommendations.

5.2.1 Golf Links Extension and North Ward Servicing

In a 1974 Trunk Sanitary Sewer Report the configuration of sanitary trunk sewers to service future development areas were recommended. In particular, the Golf Links Extension was proposed that would connect into Neebing/McIntyre Interceptor at Golf Links Road and extend up to the Expressway, and then north-east on the Expressway to the John Street Trunk sewer and continue on to the McVicar's Creek Trunk sewer. Figure 5.1 shows the proposed location of the Golf

Links Extension sanitary trunk as envisioned in 1974.

The analysis in 1974 indicated that, with the population increase and the developments proposed for areas beyond the Expressway, the John Street and McVicar's Creek Trunk sewers would become overloaded. To prevent overloading, the Golf Links Extension sewer was designed to intercept the new development flows generated in areas beyond the Expressway and convey them to the Neebing/McIntyre Interceptor.

As part of the Long Term PPCP the proposed Golf Link Extension sewer has been re-evaluated based on both current system information and ultimate development projections. In conjunction, the hydraulic control of flows in the John Street, McVicar's Creek and Port Arthur sanitary trunk sewers have been considered in developing an overall flow control strategy for the North Ward. The calibrated XP-SWMM model developed in Phase 1 was employed to evaluate the following servicing alternatives:

Alternative 1A

- Golf Links Extension connected to the John Street sewer at the Expressway passing through the River Terrace pump station, continuing along the Expressway to McVicar's Creek Trunk.

Alternative 1B

- An alternative route considered extends the Golf Links sewer to the River Terrace pump station continuing to connect to the John Street Sewer at Algonquin Avenue, going up John Street to the Expressway and across the Expressway to the McVicar's Creek Trunk sewer.

Alternative 2

- Golf Links Extension terminating at John Street and the Expressway connected to the John Street Sewer at Algonquin Avenue and no controls on the McVicar's Creek Trunk.

Alternative 3

- Storage at the McVicar's Creek Trunk sewer at the Expressway and Golf Links Extension to John Street Trunk at Algonquin Avenue and up to the Expressway.

Alternative 4

- Golf Links Extension to John Street at Algonquin Avenue up to the Expressway, upstream storage on McVicar's Creek Trunk and improved conveyance in the High Street area of John Street Trunk.

The evaluation of each alternative was done under the following conditions:

- Ultimate development conditions and population in Thunder Bay
- 10 year design storm event for wet weather performance comparisons
- Rainfall Derived Inflow/Infiltration (RDI/I) rates for new developments were taken as similar to those in the John Street and McVicar's Creek service areas.
- 60 L/s discharge from the new Water Treatment Plant at Lillian Avenue into the Port Arthur Interceptor.
- Proposed Short Term PPCP diversions at Neebing/Brunswick and Neebing/Cameron.
- Golf Links Interceptor alignment will pass through the River Terrace pump station intercepting local flows.

Figures 5.2 to 5.5 show the servicing alternatives evaluated and primary development areas.

5.2.1.1 Existing Servicing Conditions

System analysis revealed the following under existing servicing conditions:

- No dry weather capacity constraints identified in John Street, McVicar's Creek, Port Arthur or Neebing/McIntyre sanitary trunk sewers.
- Surcharge conditions exist in the John Street Trunk sewer for the 10 year design event. The most critical sections are located north of Algonquin Avenue and in the High Street area.
- Surcharge conditions exist in the upper portion of the McVicar's Creek Trunk Sewer; however, the hydraulic grade line is at a reasonable level.
- Surcharge conditions exist in the Port Arthur interceptor between McVicar's and John Street at a reasonable level.
- No surcharge in the Neebing/McIntyre Interceptor.

Overall, there are existing wet weather capacity concerns in the John Street Trunk sewer and to a less extent in the McVicar's Creek Trunk and Port Arthur Interceptor sewers, while there are no wet weather concerns identified in Neebing/McIntyre Interceptor. Historically, basement flooding in the North Ward has been sporadic in nature, the only area that has had repeated incidents of flooding is in the High Street area on the John Street Trunk sewer. The hydraulic analysis identified this area to have a high HGL during wet weather.

City staff indicated that during severe wet weather conditions outflow is reported from the McVicar's Trunk sewer to the local Creek. System hydraulic analysis could not simulate these conditions using up to a 10 year design event. As well, City staff were unable to identify where this was reported on the McVicar's Creek system.

A study of the existing hydraulic conditions within the North Ward sewer system conducted by Wardrop Engineering Inc. predicted localized surcharge conditions within the McVicar's Creek Trunk Sewer at Manley and Margaret Streets and near Elm Street. Modelling completed as part of the Wardrop study was conducted without the benefit of flow data for calibration from the McVicar's Creek Trunk Sewer. The hydraulic model of the North Ward is based on the Wardrop work with the benefit of more calibration data to refine model calibration and provide more reliable results.

5.2.1.2 Ultimate Development Conditions

Figures 5.6 to 5.9 show the hydraulic grade lines (HGL) for the John Street Trunk, McVicar's Creek Trunk, Port Arthur Interceptor and Neebing/McIntyre Interceptor for the four servicing alternatives considered.

Alternative 1A - Golf Links to John Street Trunk and McVicar's Creek Trunk

The Golf Links Extension to John Street and McVicar's Creek Trunk sewers will intercept all new development flows and existing flows along the Golf Links alignment and beyond the Expressway. Therefore, a marginal reduction in the existing hydraulic conditions in the John Street Trunk, McVicar's Creek Trunk and the Port Arthur Interceptor is expected with existing developments diverted to the Golf Links Sewer.

The surcharging in the John Street Trunk sewer at Algonquin Avenue and High Street will persist, as will the elevated levels in McVicar's Trunk. There will be effectively no change in flows and flow levels in the Port Arthur Interceptor. No hydraulic capacity constraints were identified in the downstream Neebing/McIntyre Interceptor.

Alternative 1B - Golf Links to John Street at Algonquin Avenue and McVicar's Creek Trunk Sewer

The Golf Links Extension would intercept flows at Algonquin Avenue and John Street, and would intercept new development flows from the areas beyond the Expressway and existing flows generated in the area north of Algonquin Avenue tributary to the John Street Trunk sewer. Intercepting flows at John Street and Algonquin Avenue removes the hydraulic constraint in the John Street Trunk sewer at Algonquin Avenue and reduces the HGL along the remainder of the John Street

Trunk. Intercepting flow at Algonquin Avenue will require replacement of a section of the John Street Trunk between Algonquin Avenue and the Expressway. The replacement is necessary to convey new flows from development areas beyond the Expressway to the Golf Links connection point at Algonquin Avenue. The present John Street sewer capacity above Algonquin Avenue is insufficient to convey future flows. In the High Street area surcharged conditions will still continue to occur due to poor hydraulic conditions.

Overall, the Golf Links extension to John Street Trunk sewer at Algonquin Avenue will provide relief to hydraulic constraints in the upper sections of the Trunk as well as provide sanitary services for future development. There will be a marginal improvement in the hydraulic performance of the McVicar's Creek Trunk as all new development flows and existing flows beyond the Expressway are intercepted by the Golf Links Extension. The Port Arthur Interceptor HGL is marginally reduced with the diversion of existing and future flows from John Street Trunk at Algonquin Avenue.

Alternative 2 - Golf Links to John Street

The same hydraulic benefits as Alternative 1B will be realized for the John Street Trunk sewer if the Golf Links is extended to John Street at Algonquin and continues up to the Expressway. Future development areas upstream of the Expressway and McVicar's Creek would be serviced through the McVicar's Creek Trunk sewer. Under dry weather conditions there are no capacity issues; however, during the 10 year design event there is surcharging in the upper sections of McVicar's Creek Trunk (Figure 5.7). The additional flows will also increase the HGL in the Port Arthur Interceptor downstream of the McVicar's Creek connection (Figure 5.8).

Alternative 3 - Golf Links to John Street, Storage at McVicar's Creek Trunk

The same hydraulic benefits as Alternative 1B will be realized for the John Street Trunk sewer with the Algonquin Avenue connection. For this alternative 8,760 m^3 of storage is placed at the top end of the McVicar's Creek Trunk sewer at the Expressway to control the inflow into the existing trunk sewer from new development areas during wet weather conditions. The storage facility is designed to allow a maximum of 2.5 times average dry weather flow (12,900 m^3/d), to continue to flow into the McVicar's Creek Trunk and to retain excess flows for up to 12 hours. This alternative has the flexibility to be implemented as development occurs, for example, only 1,000 m^3 of storage would be required for the developments proposed to 2010.

Alternative 3 would provide a similar level of control over flows in the John Street, McVicar's Creek and Port Arthur Interceptor as Alternative 1B and would provide some flexibility in implementation.

Alternative 4 - Alternative 3 and Improved Conveyance at High Street

None of the previous alternatives were able to control the HGL in the John Street Trunk Sewer at High Street. An analysis of this section of the John Street Trunk sewer reveals that the changes in the pipe grades and slopes do not result in good hydraulic performance. To improve the hydraulics conditions, a 400 m section of John Street Trunk sewer was twinned between High Street and Algoma Street with a 1,350 mm pipe (54 inches). This was found to eliminate the surcharge conditions in the area (Figure 5.6).

5.2.1.3 Cost

The cost of each alternative was prepared and is presented in Table 5.1. Detail cost information are included in Appendix D.

5.2.1.4 Recommendation

Alternative 4 is recommended as the most cost-effective way to provide servicing for future development while providing hydraulic relief for the John Street Trunk, McVicar's Creek Trunk and Port Arthur Interceptor sewers. Implementation of Alternative 4 can be done in stages as development proceeds. The implementation of storage to manage flows into McVicar's Creek Trunk sewer can be staged to meet the development pressures. As indicated, population and flow projections indicate that only 1,000 m^3 of storage is required to meet the needs to the year 2010. The Short Term PPCP flow monitoring program will assist the City in determining the need for implementing Alternative 4.

5.2.2 North Ward Catchbasin Cross Connections

The Short Term PPCP recommended a removal program of catchbasins connected to the sanitary sewers in the North Ward. The catchbasins removed did not require any new services or construction of storm sewers. The objective of removing the catchbasin connections to the sanitary sewers is to reduce the stormwater flows in the sanitary collection systems. As a continuation of the Short Term disconnection program Table 5.2 summarizes the remaining catchbasins connected, the work required to remove the connection and redirect flow to an existing storm outlet and the associated cost. It is estimated that the program would cost approximately $775,000.

Table 5.1 Golf Links Extension and North Ward Servicing Alternatives

Alternative	Cost ($)	Comments
1. Golf Links to John and McVicar's Creek	$13,700,000	Services new development areas Provides flow control into John and McVicar's Trunk sewers Does not address HGL in John Trunk sewer at High Street No change in Port Arthur Interceptor No capacity constraints in Neebing/McIntyre
2. Golf Links to John	$4,100,000	Services development areas to John Street only Surcharging conditions will exist in the McVicar's Creek Trunk and in the John Trunk at High Street HGL increases in Port Arthur Interceptor No capacity constraints in Neebing/McIntyre
3. Golf Links to John, 8,760 m^3 storage at McVicar's Creek (with 1,000 m^3 storage to 2010)	$9,500,000 ($4,750,000)	Services all new development areas Provides flow control into John and McVicar's Trunk sewers Does not address HGL in John Trunk sewer at High Street Storage can be implemented in stages with development HGL decreases in Port Arthur Interceptor No capacity constraints in Neebing/McIntyre
4. Alternative 3 and Twinned Section of John Street Trunk (with 1,000 m^3 storage to 2010)	$10,300,000 ($5,500,000)	Services all new development areas Provides flow control into John and McVicar's Trunk sewers Removes surcharging conditions in John Trunk sewer at High Street Storage can be implemented in stages with development HGL decreases in Port Arthur Interceptor No capacity constraints in Neebing/McIntyre

Table 5.2 Long Term PPCP Catchbasin Disconnection Program

Location	Description	Cost ($)
Queen St. and Winnipeg	Construction of 180 m of storm sewer connected to existing 36" storm on Queen St.	45,900
Rockwood Ave. and John St.	Connect to existing storm on Rockwood Ave.	1,785
Crown St. and Oliver Rd.	Construction of 195 m of storm sewer connected to existing 18" storm on Oliver Rd.	49,725
Ontario St. between Oliver Rd. and Cornwall Ave.	Construction of 168 m of storm sewer connected to existing 30" storm on Oliver Rd.	42,840
Machar Ave. and Cornwall Ave.	Construction of 250 m of storm sewer connected to existing 18" stub at John St. and Machar Ave.	63,750
Court St. and Cornwall Ave.	Construction of 60 m of storm sewer connected to existing 15" storm on Johnson Street	15,300
Court St. and Ambrose St.	Construction of 65 m of storm sewer connected to existing 15" storm on Wilson St.	16,575
Court St. and Lincoln St.	Construction of 65 m of storm sewer connected to existing 10" storm on Pearl St.	16,575
College St. between St. Patricks Square and Hebert St.	Construction of 100 m of storm sewer connected to existing 15" storm on River Rd.	25,500
Hebert St. and High St.	Construction of 240 m of storm sewer connected to existing 10" storm on Red River Rd.	61,200
Algoma St. and Van Norman St.	Construction of 110 m of storm sewer connected to existing 12" storm on Algoma St.	28,050
College St. and Dawson St.	Construction of 210 m of storm sewer connected to existing 24" storm on Tupper St.	53,550
Melvin Ave. and Dobie St.	Construction of 70 m of storm sewer connected to existing 15" storm on River Rd.	17,850
St. James St. and Front St.	Construction of 290 m of storm sewer connected to existing 24" storm on Front St.	73,950
Elm St. and River St.	Construction of 75 m of storm sewer connected to existing 18" storm on River Rd.	19,125
Van Horne St. and Front St.	Construction of 290 m of storm sewer connected to existing 24" storm on Front St.	73,950
Birch St. and Parsons Ave.	Construction of 230 m of storm sewer connected to existing 18" storm on High St.	58,650
Clavet St. and Court St.	Construction of 30 m of storm sewer connected to existing 10" storm on Elizabeth St.	7,650
Cumberland St. and Beck St.	Construction of 105 m of storm sewer connected to existing 24" storm on MacDougall St.	26,775
Front St. and Wolseley St.	Construction of 290 m of storm sewer connected to existing 24" storm on Front St.	73,950
Total		**$772,650**

5.2.3 Kaministiquia Interceptor Improvements

In Phase 1 no performance concerns were identified in the Kaministiquia Interceptor. The implementation of Short Term PPCP measures with respect to CSO Control and the diversion of flow from the Neebing Interceptor to the Cameron system, and ultimately the Kaministiquia Interceptor, will not have a significant affect on the Kaministiquia Interceptor.

The improvement to the Kaministiquia Interceptor identified is related to the upgrade at the wastewater treatment plant. Presently, the Kaministiquia Interceptor, a 1,670 mm (66") diameter pipe, flows into a regulating chamber at the treatment plant that has a 750 mm diameter (30") throughflow pipe to the Main pumping station. Excess flows are diverted to the old pump station and primary treatment facility. The old pump station is both in poor physical and mechanical condition and represents a maintenance problem for the City. Minimizing or eliminating the need for the old pump station would be beneficial to the City by consolidating the treatment plant pumping at the Main pump station. Figure 5.10 schematically shows the connection of the Kaministiquia Interceptor to the wastewater treatment plant.

It is proposed to replace the 215 m of 750 mm pipe with a 1,670 mm diameter pipe. The new section would connect to the Neebing/McIntyre Interceptor 2,100 mm (84") influent pipe at the drop structure outside of the Main pump station.

An evaluation of connecting the Kaministiquia Interceptor to the Neebing/McIntyre Interceptor at the Main pump station was undertaken to determine if there would be any negative effects. The 10 year design storm event was used for the assessment. The evaluation was undertaken for the existing development conditions as well as the ultimate development conditions including the Golf Links Extension - Alternative 4, and the Neebing diversions.

Figure 5.11 shows the maximum HGL along the Kaministiquia Interceptor from the Cameron Interceptor to the Main pump station and along the Neebing/McIntyre Interceptor respectively.

The replacement of 1,670 mm diameter pipe will not affect the operation of the Neebing/McIntyre Interceptor and will eliminate the need for the old pump station. The existing diversion to the old pump station must be maintained until the Main pump station is upgraded as part of the WPCP upgrade to secondary wastewater treatment. The cost of the Kaministiquia Interceptor improvement is estimated to be $1.4 million (Appendix D for details).

5.3 *CSO Control*

No Long Term PPCP CSO controls were found to be necessary for the City of Thunder Bay. Presently, the City has a level of CSO control higher than the minimum of 90% volumetric control basin wide.

5.4 *Stormwater Control*

The stream and loadings analysis undertaken in Phase 1 showed no clear evidence that stormwater represents a significant source of pollutants annually or on an event basis. The Short Term PPCP recommends continued enforcement of the provincial stormwater guidelines; there is no change for the Long Term PPCP. The Long Term PPCP does not contain any projects associated with the control or treatment of stormwater.

5.5 *Thunder Bay WPCP*

The Short Term PPCP recommends a pilot study be undertaken to identify the most cost-effective secondary treatment technology. The City of Thunder Bay is proceeding with the pilot study program. The outcome of the program will be recommendations on the secondary treatment technology. From this recommendation the City is committed to proceed to pre-engineering, final design and construction. It is anticipated that following the final pilot study recommendation full secondary treatment will be implemented within 5 years.

A pollutant loadings assessment was undertaken in Phase 1 quantifying the relative contribution of various sources.

A pollutant loadings assessment was undertaken in Phase 1 quantifying the annual loadings from various pollutant sources to the receiving waters. Table 5.3 presents a comparison of annual loadings for BOD_5, TSS and TP for the treatment plant with the upgrade to secondary treatment. Table 5.3 includes the existing conditions as well as the future loadings, using the plant average design flow of 109,100 m^3/d and the design objective and compliance criteria concentrations.

In reviewing Table 5.3, it is evident that despite the increase in flow there will be an overall reduction in the annual loadings of BOD_5 and TSS to the Kaministiquia River. There will be an increase in the annual load of TP to the Kaministiquia River.

Table 5.3 Thunder Bay WPCP Loadings

Condition	Flow (m^3/d)	BOD_5 (kg/year)	SS (kg/year)	TP (kg/year)
Existing (21, 46, 1.08)[1]	80,550	2,381,400	1,352,400	31,750
Future Design (15, 51, 1)[1]	109,100	597,300	597,300	39,800
Future Compliance (25, 25, 1)[1]	109,100	995,550	995,550	39,800

Notes:
1. Concentrations for BOD_5, SS and TP in mg/L.

The preliminary cost estimates for secondary treatment are in the range of $26 million to $34 million; the costs of the upgrade will be refined with the pilot study results and completion of the pre-engineering.

5.6 Recommended Long Term PPCP and Costs

Table 5.4 presents a summary of the recommended Long Term PPCP programs, qualitative benefits and associated costs.

Table 5.4 Recommended Long Term PPCP and Costs

Program	Program Description	Benefits	Costs
Golf Links Extension	• Extend Golf Links through River Terrace pump station over to John Street Trunk sewer at Algonquin Avenue • Replace John Street Sewer between Algonquin Avenue and the Expressway	• Provides future capacity to developments beyond the Expressway • Diverts existing flow from the John Street Trunk sewer providing hydraulic relief in the upper portion	\$3.3 million \$1.0 million
McVicar's Creek Storage	• 8,760 m^3 of storage required for ultimate development • 1,000 m^3 storage for developments up to 2010 • McVicar's Creek Trunk sewer to receive a maximum of 2.5 average DWF • Detention time 12 hours • Storage can be staged with development	• Storage can be staged • Provides control over flows into the McVicar's Creek Trunk • More cost effective than extending the Golf Links • Cost of storage borne by developer	\$5.4 million (total) \$615,000 (1,000m^3) \$0 (city)
John Street Trunk Sewer Improvement	• Twin 400 m section of sewer between Algoma and Ontario Streets	• Provides hydraulic relief in local area • Reduces hydraulic grade line and the risk of basement flooding	\$740,000
North Ward Catchbasin Disconnection Program	• Construct new storm sewers to existing outlets • Disconnect catchbasins from sanitary and reconnect to new storm sewer	• Removes storm flow from sanitary sewers • Reduces wet weather response in sanitary system	\$775,000
Thunder Bay WPCP Upgrade	• No recommendation • Pilot study pending	• Reduction in loadings to Lake Superior • Meet Regulatory requirements	\$35 million
Kaministiquia Interceptor Improvements	• Replace 215 m of 750 mm with 1,670 mm pipe between old pump station regulator and the Main pump station • Part of WPCP upgrade	• Reduces the need for the old pump station • Improved hydraulic and simplified operations	\$1.4 million
Total Cost			**\$42.2 million**

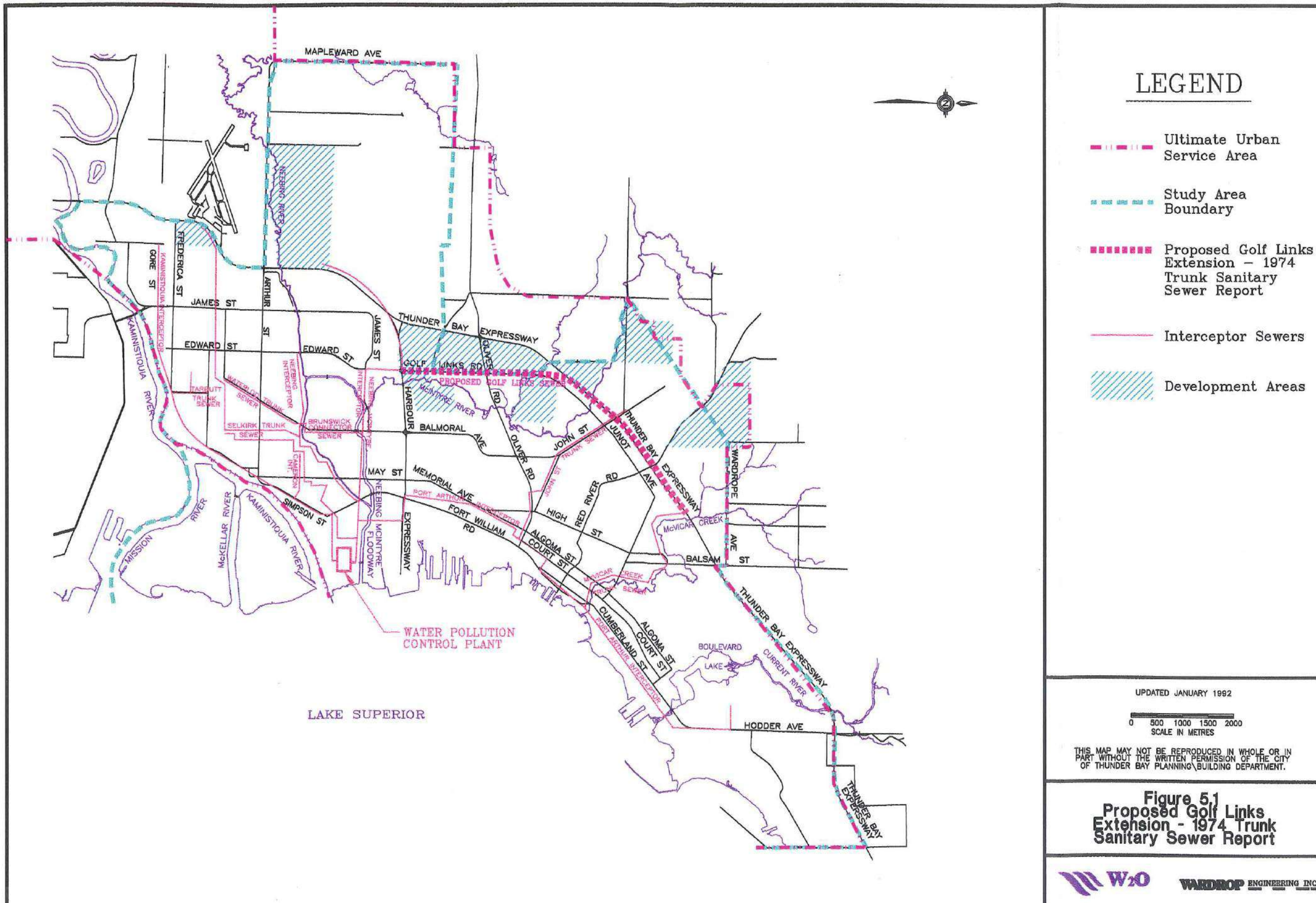
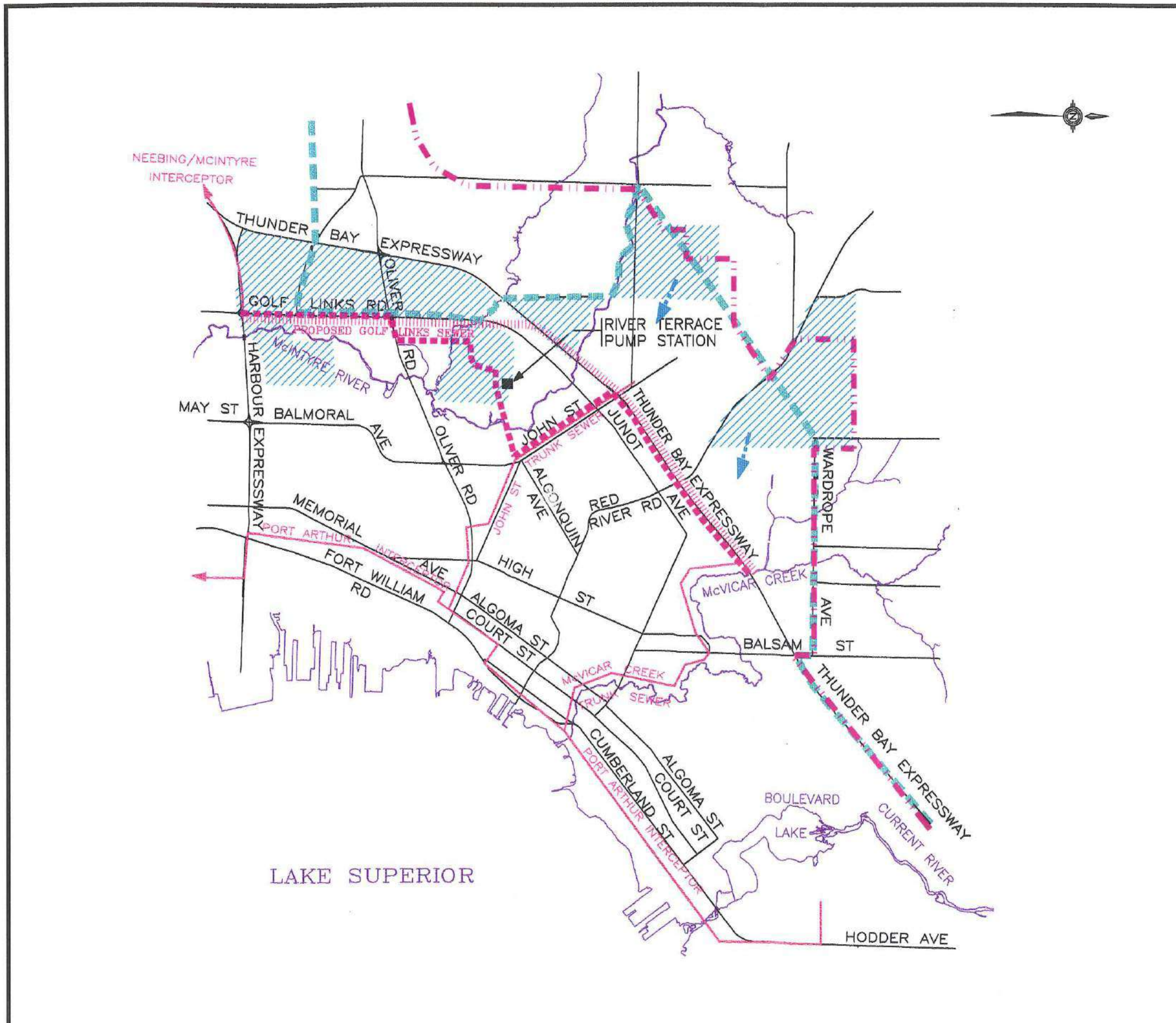


Figure 5.1 Proposed Golf Links Extension - 1974 Trunk Sanitary Sewer Report



LEGEND

- Ultimate Urban Service Area
- Study Area Boundary
- Interceptor Sewers
- Development Areas
- Alternative 1A: Golf Links to McVicars Creek
- Alternative 1B: Golf Links to McVicars Creek

UPDATED JANUARY 1992

0 500 1000 1500 2000
SCALE IN METRES

**Figure 5.2
North Ward Servicing
Alternative 1A and 1B**

W2O WARDROP ENGINEERING INC.

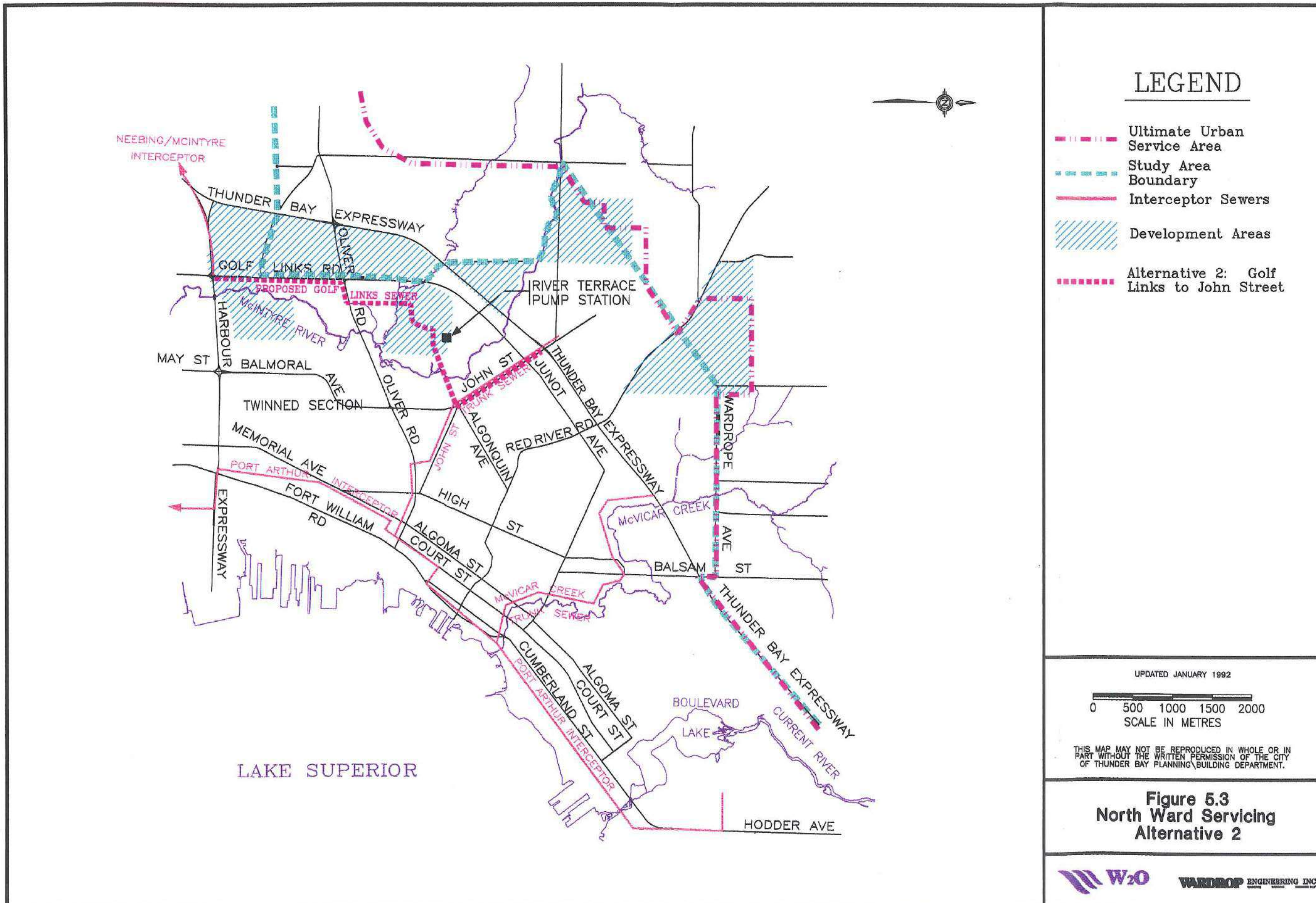


Figure 5.3
North Ward Servicing
Alternative 2

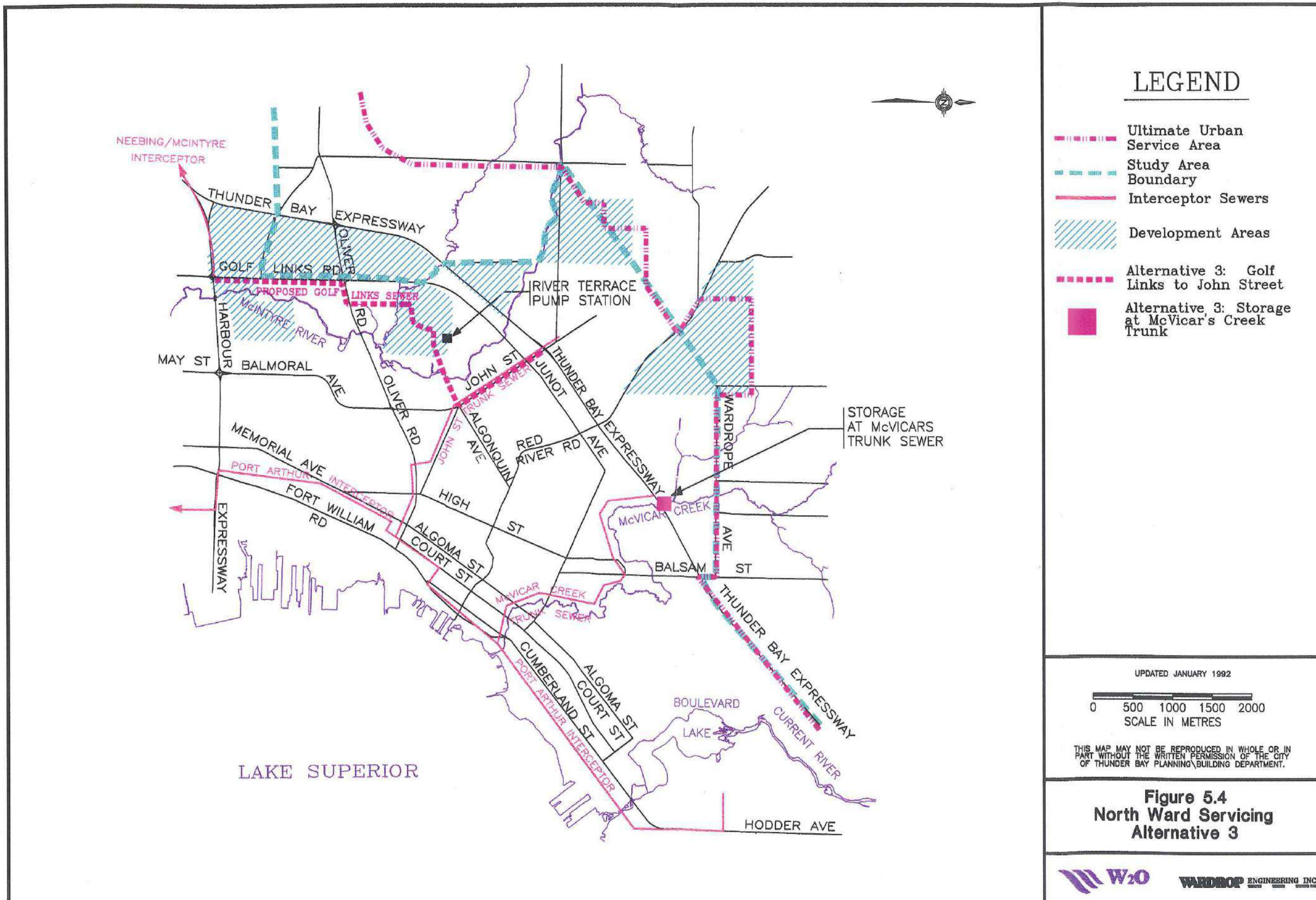


Figure 5.4
North Ward Servicing
Alternative 3

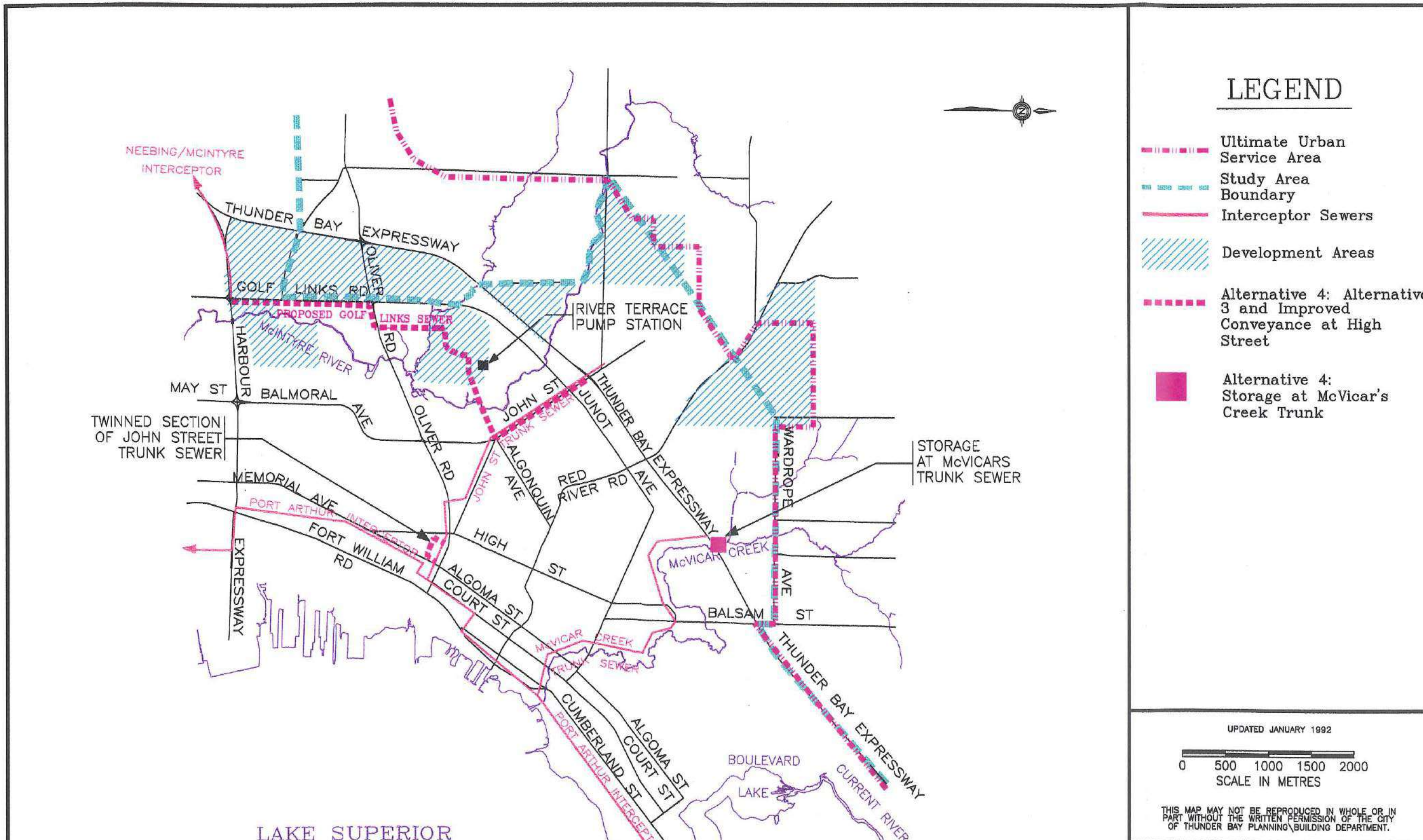


Figure 5.5
North Ward Servicing
Alternative 4

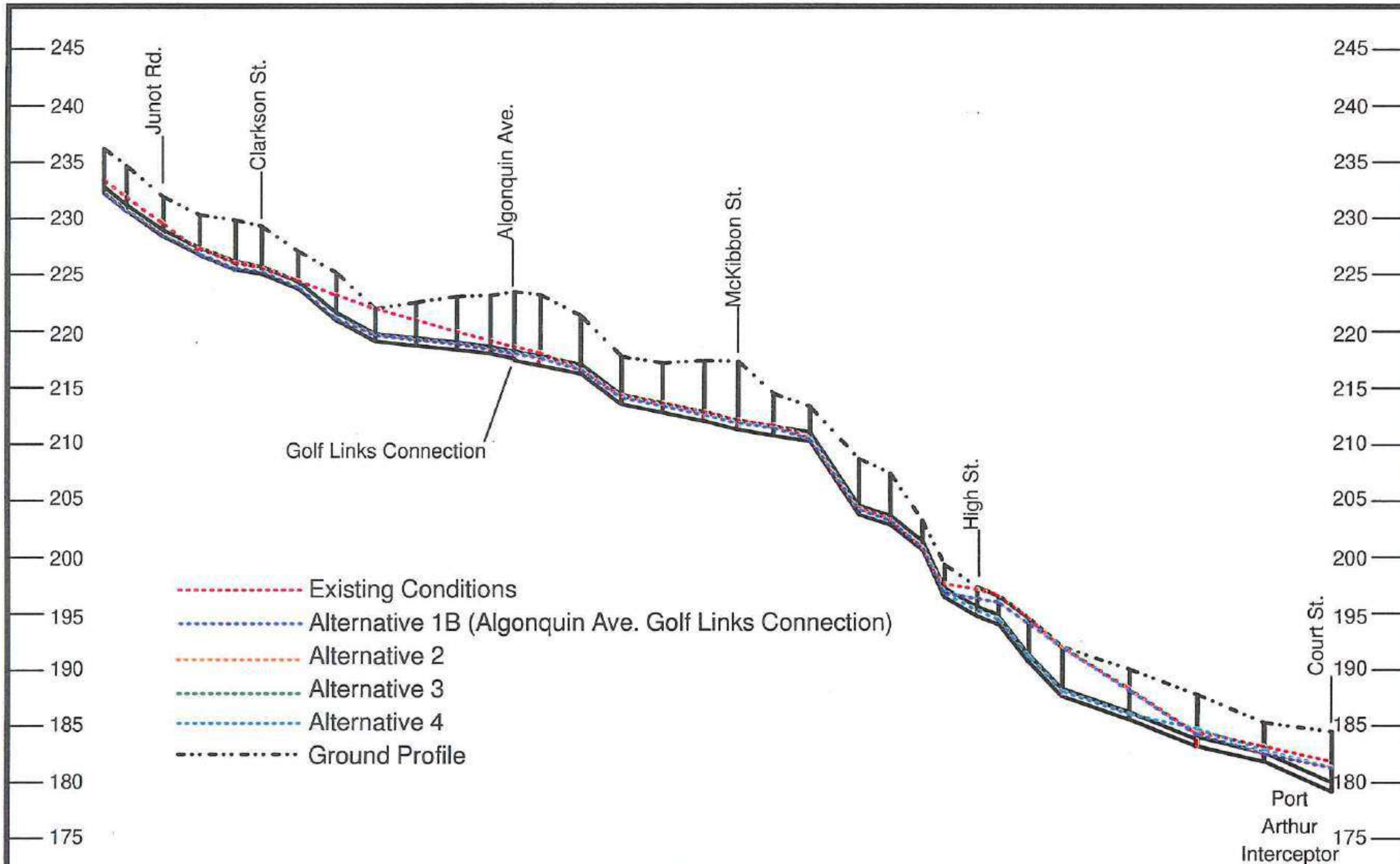


Figure 5.6
John Street Trunk Sewer Servicing Alternatives
10 Year Design Storm Event HGL

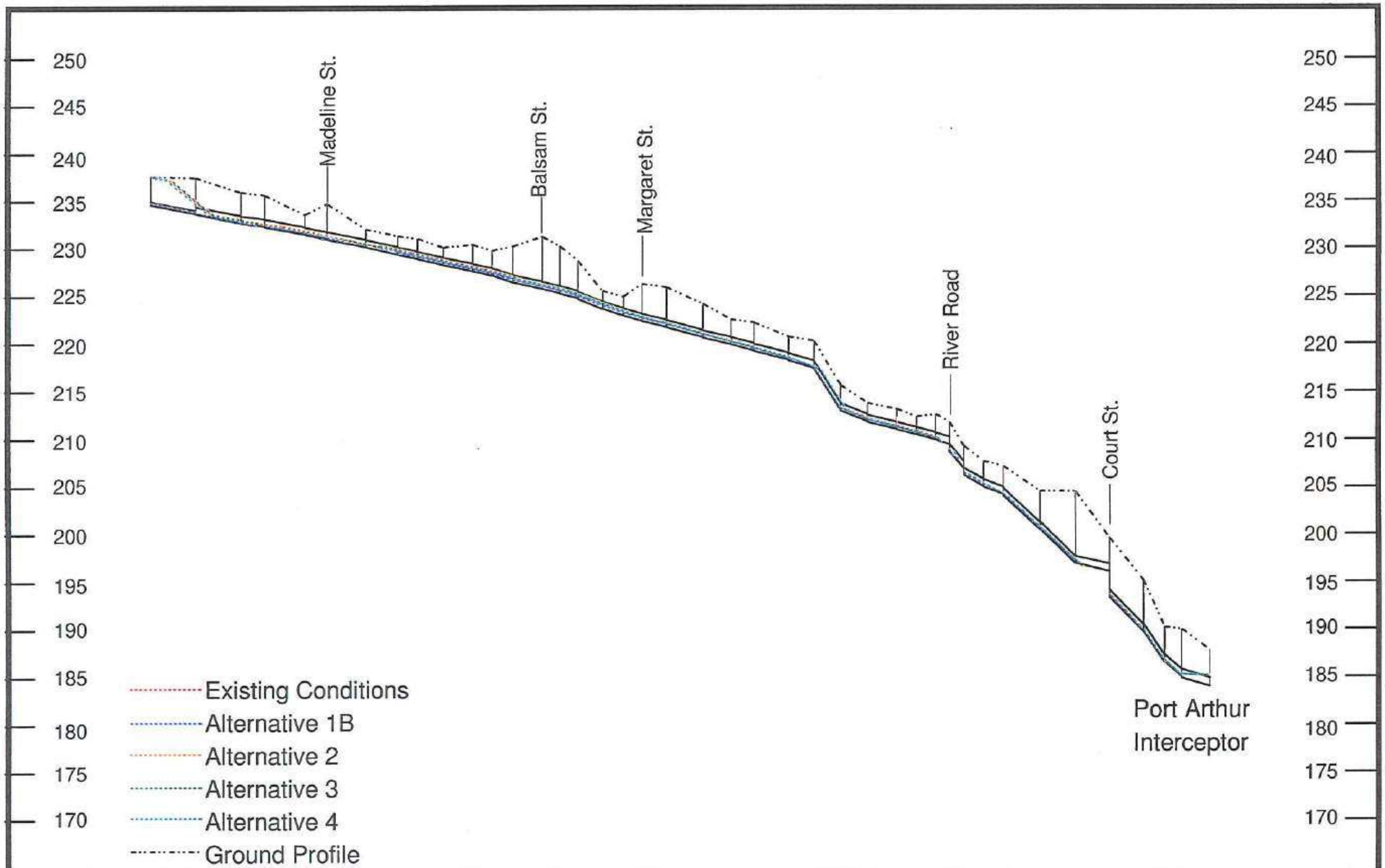


Figure 5.7
McVicar's Creek Trunk Sewer Servicing Alternatives
10 Year Design Storm Event HGL

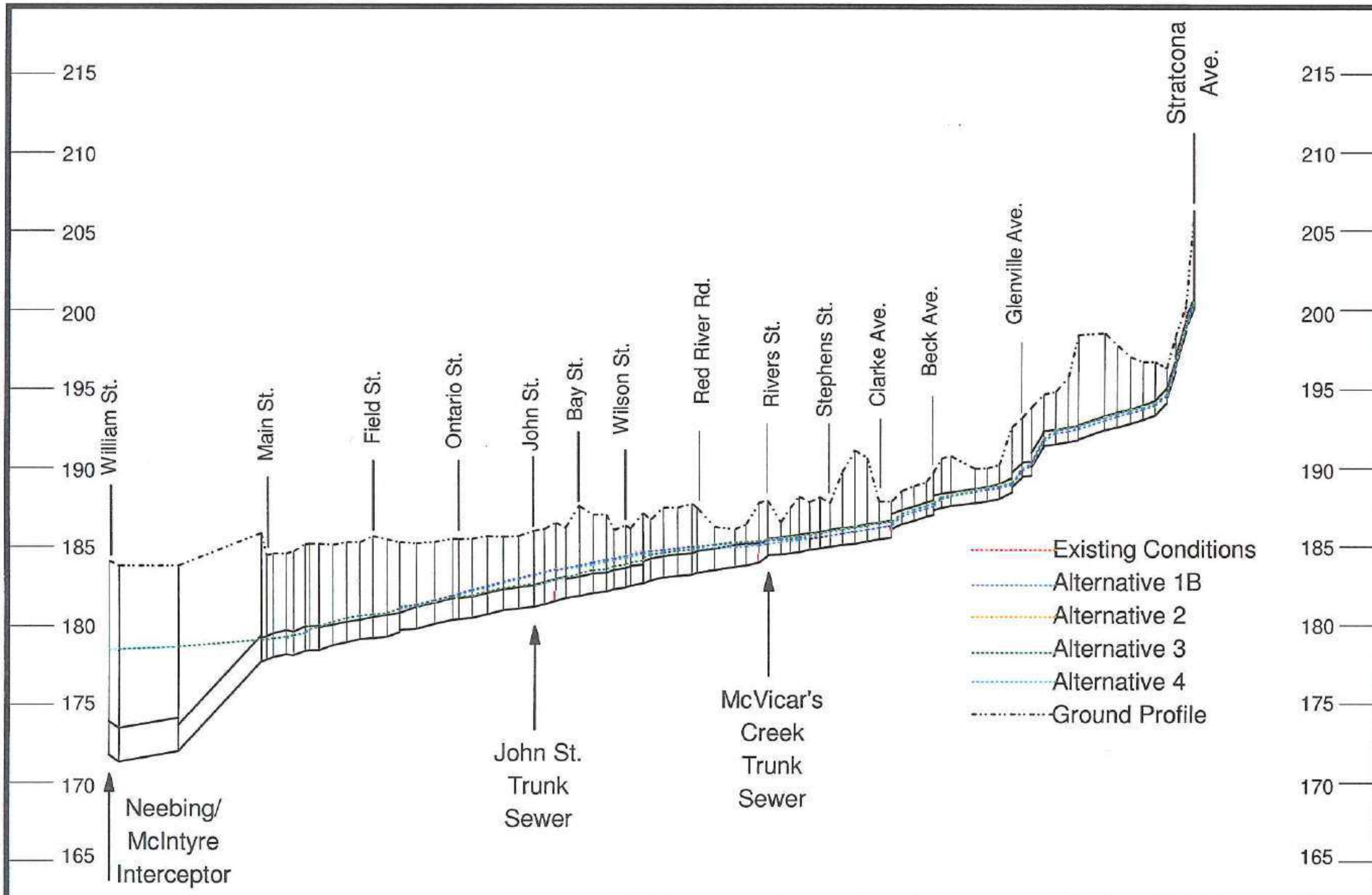


Figure 5.8
Port Arthur Interceptor Servicing Alternatives
10 Year Design Event HGL

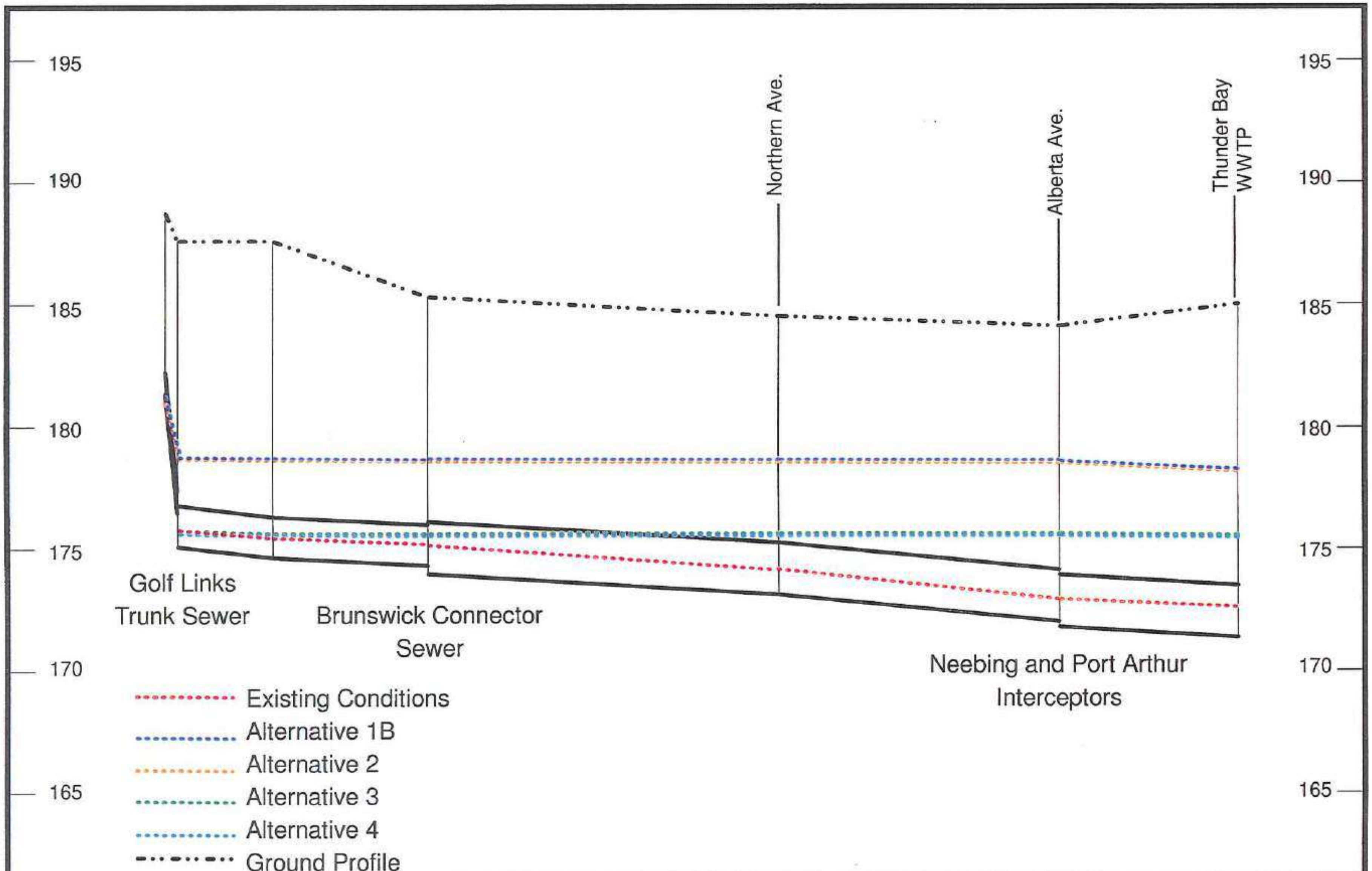


Figure 5.9
Neebing/ McIntyre Interceptor Servicing Alternatives
10 Year Design Event HGL

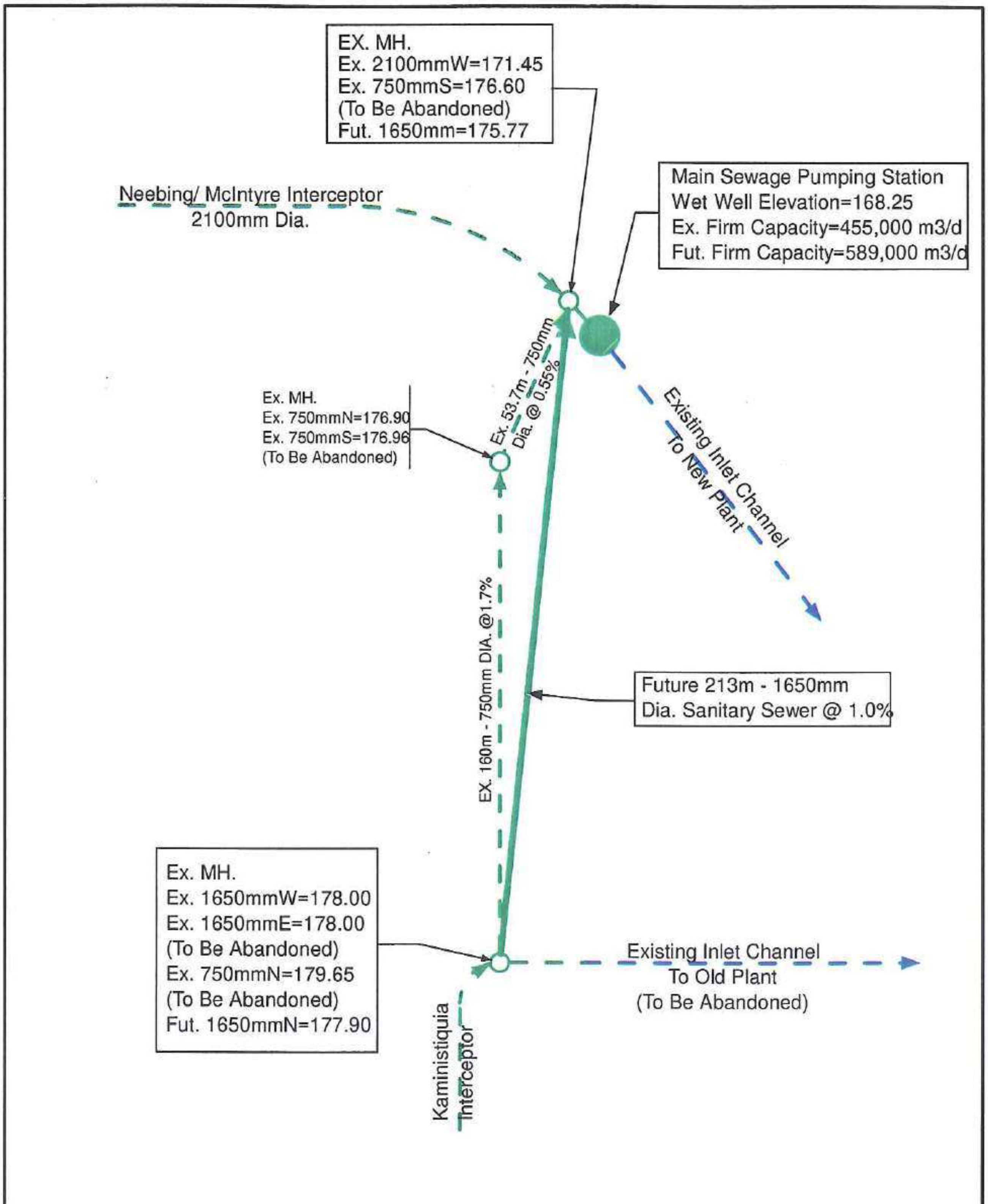


Figure 5.10
Kaministiquia Interceptor Connection
To Main Pump Station

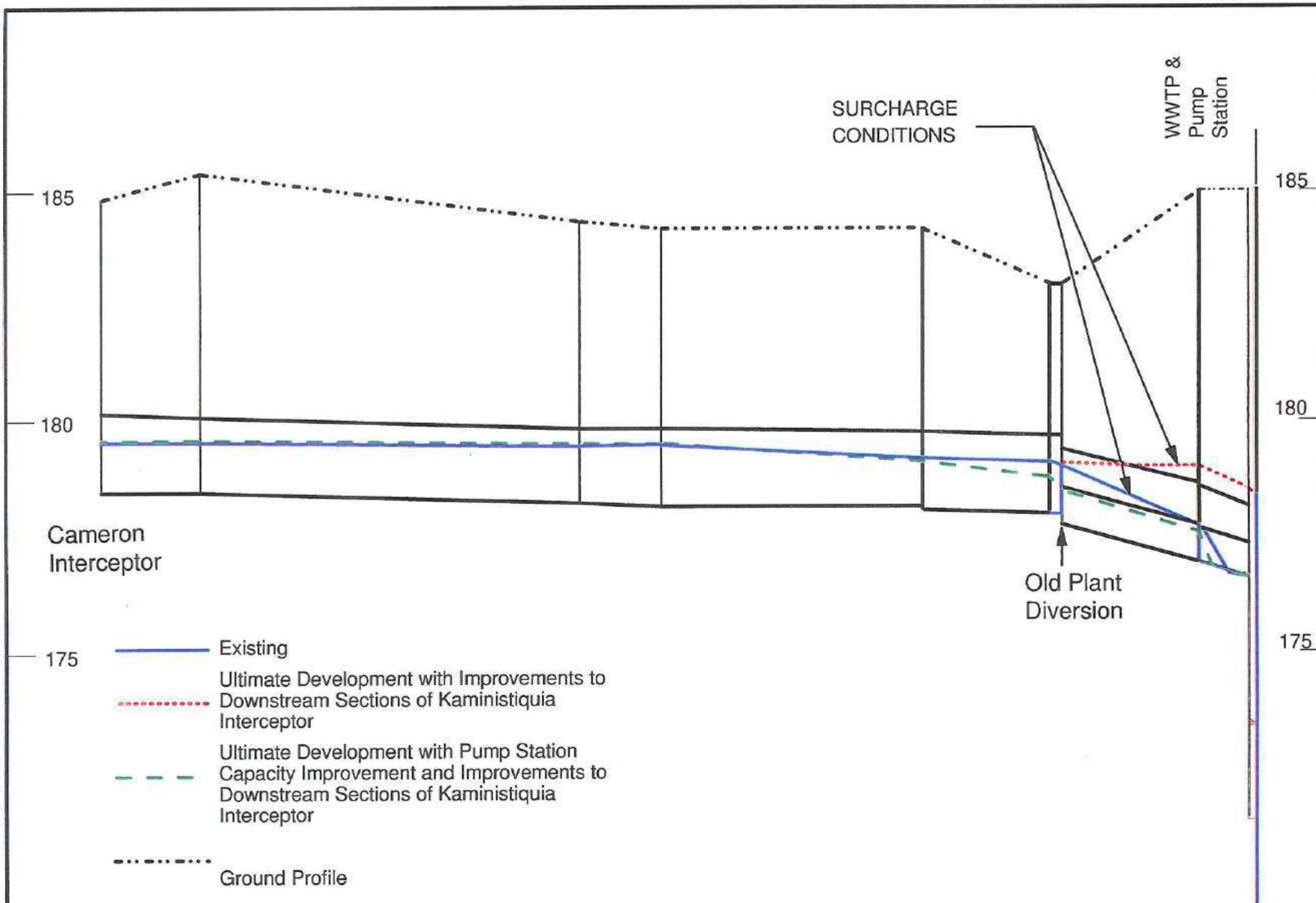


Figure 5.11
Kaministiquia Interceptor
10 Year Design Storm HGL with Pump Station Improvements

6. *IMPLEMENTATION PLAN*

6.1 *Overview*

The following section provides a summary of Short and Long Term PPCP initiatives and programs, outlining an Implementation Plan and schedule for the City of Thunder Bay. The proposed Implementation Plan is designed to prioritize projects to achieve the objectives of pollution control planning in meeting community standards; CSO control and stormwater guidelines; WPCP effluent requirements; Provincial Water Quality Objectives; and the objectives of RAP and the Binational program.

The Implementation Plan presented provides the initial framework to implement the PPCP programs recommended. The Plan will change and evolve through the implementation period and should be considered a living document to be revisited and revised as more and better information becomes available. The long term goals of the plan are to accomplish the following:

- Reduce urban pollutant loadings to receiving waters and to protect water resources
- Ensure reliable services
- Reduce/eliminate basement flooding
- Provide services for future developments
- To provide secondary treatment

6.2 *Prioritization*

All recommendations have been prioritized based upon a detailed review of each program. The review process used to prioritize programs considered the following factors:

Public Contact

Projects that would reduce/eliminate public contact with sewage were given a high priority. This reflects the potential public health concerns associated with urban discharges.

Collection System Management

Initiatives associated with system operations are given a higher priority. Infrastructure information is needed for the City to make informed decisions with regards to rehabilitation as well as to ensure reliable services.

Cost Effectiveness

Projects that are lower cost providing immediate results are given higher priority. Typically, short term programs are designed to provide immediate benefits for minimal cost; long term programs tend to involve capital expenditures.

CSO Control

CSO control was not identified as a major source of pollutant loadings and therefore projects related to CSO control do not have a high priority.

Pollution Prevention

Programs that address pollution prevention are given a high priority. Prevention is a very cost-effective way of reducing pollutant discharges. Pollution prevention includes educational programs, stormwater controls, etc.

To prioritize programs five levels of priorities have been established, 1 being the highest and 5 being the lowest. The Short Term PPCP and Long Term PPCP summaries are presented in Table 6.1 indicating the assigned priority, the projected costs and implementation period based on the program review and the above factors.

6.3 Schedule And Cash Flow

Figure 6.1 presents the Implementation Plan schedule and cash flow information. The implementation period is considered to be 20 years corresponding to the planning period. The implementation period for projects associated with new services for future development may occur beyond the 20 year planning period.

The costs of each program and project have been distributed to develop a cash flow projection for the City. For some programs no new dollars are identified indicating that program funding should be from existing operational budgets. Funding for projects associated with the reduction or elimination of basement flooding has been distributed uniformly across a ten year period. A specific distribution can not be determined until specific projects have been identified through the setting of a community standard and refinement of analysis with local flow data. The costs associated with the McVicar's Creek Trunk storage facility have not been shown in the Implementation Plan. It is anticipated that the storage required beyond the initial 1,000 m^3 could be funded through development charges. Alternatively, development in this area could be limited to the existing service capacity available.

Figure 6.2a and 6.2b present the cash flow requirements for the Implementation Plan. Figure 6.2b does not include the WPCP upgrade to secondary treatment.

The Thunder Bay PPCP was carried out in accordance with the approved planning and design process contained within the Class Environmental Assessment Act for Municipal Water and Wastewater Projects. The recommended works outlined in the Implementation Plan can be categorized as Schedule A projects requiring no public notification. The only exception to the Schedule A is likely the Golf Links Extension (Item 19) and storage at McVicar's Creek (Items 23 and 28), which would fall under Schedule B type projects requiring suitable public notification on two occasions. Early in the evaluation of the Thunder Bay WPCP, confirmation from the EA Branch was received identifying that the change to secondary treatment would be classified as a Schedule A project, given there is no change in the plant's rated capacity.

Table 6.1 Program Priorities

Item	Program	Cost	Priority	Implementation Period
1.	CCTV, manhole inspection and sewer flushing program	$2.8 million	1	10 years • becomes ongoing
2.	CSO Inspection and Maintenance	existing budgets	1	ongoing
3.	Pump Station Maintenance	existing budgets	1	ongoing
4.	Neebing/Brunswick Diversion	$21,000	1	3 years
5.	Neebing/Cameron Diversion	$22,000	1	5 years
6.	North Ward Catchbasin Sealing	$12,000	1	2 years
7.	James and Quebec Connection Correction	$93,000	1	2 years
8.	Monitoring Program	$115,000	1	5 years • becomes ongoing
9.	RK2 Regulator Replacement and Adjustment	$15,000	1	2 years
10.	South Ward Basement Flooding Program	$340,000 to $4.3 million	1	10 years
11.	Stormwater Management Controls	existing budgets or developers	1	ongoing
12.	Thunder Bay WPCP Pilot Study	$300,000 to $400,000	1	initiated - 2 years
13.	Phosphorus Removal	existing budgets	1	ongoing
14.	Digester Optimization	existing budgets	1	ongoing
15.	Catchbasin Cleaning – 100% coverage annually	increase existing budget	1	ongoing
16.	Pollution Prevention Programs (street cleaning, public education)	existing budgets	1	ongoing
17.	Neebing/McIntyre Improvements	existing budgets	1	1 year
18.	Storm Sewer Outfall Inspection, Maintenance and Survey	existing budgets	2	7 years • becomes ongoing
19.	Golf Links Extension to River Terrace P.S.	$3.3 million	2	10 years
20.	Thunder Bay WPCP Upgrade to Secondary Treatment	$25 to $35 million pending Pilot Study recommendations	2	5 years
21.	North Ward Storm Sewer and Catchbasin Disconnection	$120,000	3	7 years
22.	Initial 1,000 m^3 storage @ McVicar's	$0 (developer pay)	3	7 years
23.	John Street Trunk Sewer Improvement	$740,000	3	7 years
24.	Kaministiquia Interceptor Improvements	$1.4 million	3	5 years

Table 6.1 Program Priorities

Item	Program	Cost	Priority	Implementation Period
25.	CSO Regulator Replacement Program	$175,000	4	15 years
26.	Golf Links Extension to Algonquin Avenue and Upgrade of John Street Trunk to Expressway.	$1.0 million	4	15 years
27.	Outfall Flap Gate Replacement Program	$300,000	5	25 years
28.	McVicar's Storage - 8,760 m^3 (only 7,760 m^3 required @ $4.7 million if the initial 1,000 m^3 installed)	$5.4 million	5	25+ years
29.	North Ward Catchbasin Disconnection Program	$775,000	5	25+ years

Figure 6-1
Implementation Plan

ITEM	PROGRAM	COMMENTS	COSTS	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	BEYOND
1.	CCTV, Sewerline, Manhole and Sewer Flushing Program	- Program began in 1996 - Initial 10 year program - Beyond 10 years new CCTV program required	$2,800,000	120,000	245,000	245,000	245,000	245,000	340,000	340,000	340,000	340,000	340,000	- Inspection program is ongoing, however, the annual level of effort is reduced											
2.	CSO Inspection & Maintenance	- Ongoing program - No new resources required	$0	- Ongoing program, existing budget																					
3.	Pump Station Maintenance	- Ongoing program - No new resources required	$0	- Ongoing program, existing budget																					
4.	Neebing/Brunswick Diversion	- Provides hydraulic relief and control to the Neebing Interceptor reducing the likelihood of surcharging conditions	$21,000			21,000																			
5.	Neebing/Cameron Diversion	- High level relief of the Neebing Interceptor	$22,000				22,000																		
6.	North Ward Catchbasin Sealing	- Cost effective way to disconnect CB - Reduce inflow into North Ward sanitary system	$12,000		12,000																				
7.	James and Quebec Connection Correction	- Removal of cross connection	$93,000			93,000																			
8.	Monitoring Program	- Monitoring program will provide additional model calibration data and increase of flows in the collection system - 2 meters and 1 rain gauge - 10 permanent stations	$115,000		12,000	10,000			18,600	18,600	18,600	18,600	18,600	- Ongoing program											
9.	RK2 Regulator Replacement and Adjustment	- RK2 to be replaced and adjusted in the short term - Provides City information on new regulator technology	$15,000			15,000																			
10.	South Ward Basement Flooding Program	- Program may not need to be fully implemented - Flow monitoring and system modelling should be used to re-assess need - The level of risk assumed will change the costs - No cost identified in cash flow	$4,300,000	430,000	430,000	430,000	430,000	430,000	430,000	430,000	430,000	430,000	430,000												
11.	Stormwater Management Controls	- Follow Provincial Guidelines - Ongoing	$0	- Ongoing program, existing budget																					
12.	Thunder Bay WPCP Pilot Study	- Study to be initiated in 1996 - $300,000 to $ 400,000 depending on final scope	$400,000	100,000	200,000	100,000																			
13.	Phosphorus Removal Program	- Ongoing program	$0	- Ongoing program, existing budget																					
14.	Digester Optimization	- Ongoing program	$0	- Ongoing program, existing budget																					
15.	Catchbasin Cleaning 100% Coverage	- South Ward has 50 to 60% coverage this is to be increased to 100% annually	$0	- Ongoing program, existing budget																					

Figure 6-1
Implementation Plan

ITEM	PROGRAM	COMMENTS	COSTS	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	BEYOND
16.	Pollution Prevention Programs	- Ongoing initiatives - Promote Roof Leader Disconnection	$0	- Ongoing program, existing budget																					
17.	Neebing/McIntyre Interceptor Improvements	- Study pending - Requires immediate action	$0		- Pending																				
18.	Storm Sewer Outfall Inspection, Maintenance and Survey	- Investigate cost sharing with other agencies - Outfall survey to be repeated on a 7 year cycle	$0		- Existing budget							- Program designed on a 7 year cycle													
19.	Golf Links Extension to River Terrace P.S.	- Alignment is not set - It is assumed the extension will be phased in over 7 years - Development driven	$3,300,000	330,000	330,000	330,000	330,000	330,000	330,000	330,000	330,000	330,000	330,000												
20.	Thunder Bay WPCP Upgrade to Secondary Treatment	- City commited to provide full secondary treatment within 5 to 10 years	$35,000,000				7,000,000	7,000,000	7,000,000	7,000,000	7,000,000														
21	North Ward Storm Sewer and Catchbasin Disconnection	- New storm sewer will allow 3 CBs to be disconnected reducing wet weather inflow.	$120,000						120,000																
22.	McVicar's Creek 1,000 m3 Storage	- Initial storage volume required - Investigate cost sharing to fund	$615,000					615,000																	
23.	John Street Trunk Sewer Improvements	- Provides hydraulic relief in High St. area - Implementation withi 5 years	$740,000							740,000															
24.	Kaministiquia Interceptor improvements	- Implement inconjunction with WPCP upgrade to secondary treatment	$1,400,000					1,400,000																	
25.	CSO Regulator Replacement Program	- Replacement program over 15 years to replace 10 regulators on the KAM Interceptor - Assumed that one or two regulators addressed each year of the program	$175,000					15,900	15,900	15,900	15,900	15,900	15,900	15,900	15,900	15,900	15,900	15,900							
26	Golf Links Extension to Algonquin Ave. and upgrade of John St. Trunk to Expressway	- Requires Item 20 to be completed - Development driven	$1,000,000											250,000	250,000	250,000	250,000								
27.	Outfall Flap Gate Replacement Program	- Program to be implemented on an "as need" basis - costs are distributed - It is assumed all outfall gates will need to be replaced over the next 20 year period	$300,000					8,000 - RN21		20,000 - RN25, RN28, RN33			15,000 - RN24	13,000 - RN27				122,000 - RN20				122,000 - RN32			
28.	McVicar's Creek 8,760 m3 Storage	- No cost identified to the City - Need for storage is development driven - Cost recovered in development charges - Approximately $4.7 million	$0																- Additional storage would be required for future developments						
29.	North Ward Catchbasin Disconnection Program	- Program may not need to be fully implemented - Flow monitoring and system modelling should be used to re-assess need	$775,000																77,500	77,500	77,500	77,500	77,500	77,500	310,000
			TOTAL	1996	1997	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	BEYOND
	CASH FLOW (x1,000 and rounded)		$51,200	$1,000	$1,200	$1,200	$8,000	$10,000	$8,300	$8,900	$8,100	$1,100	$1,100	$300	$300	$300	$300	$100	$100	$100	$100	$200	$100	$100	$300

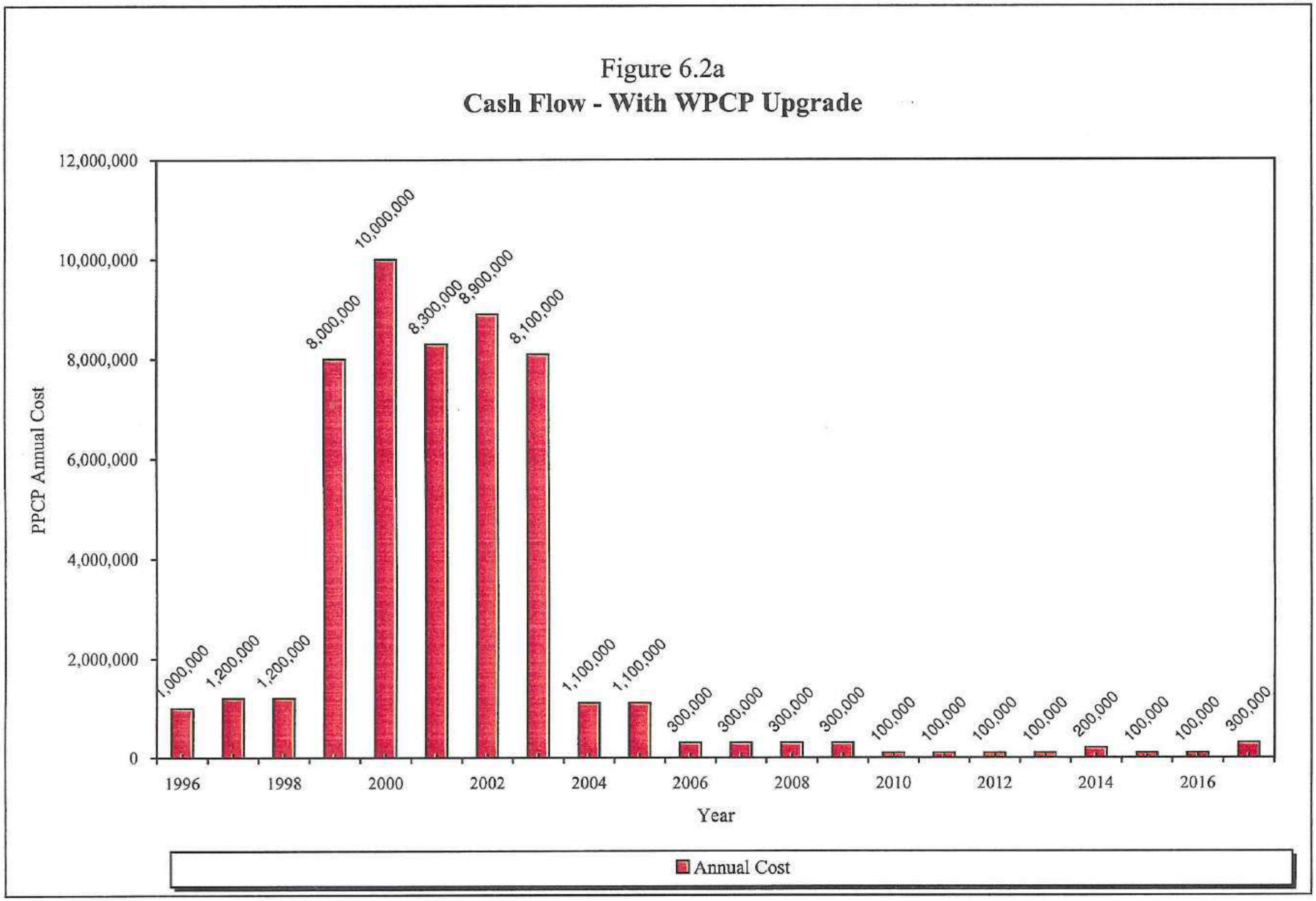
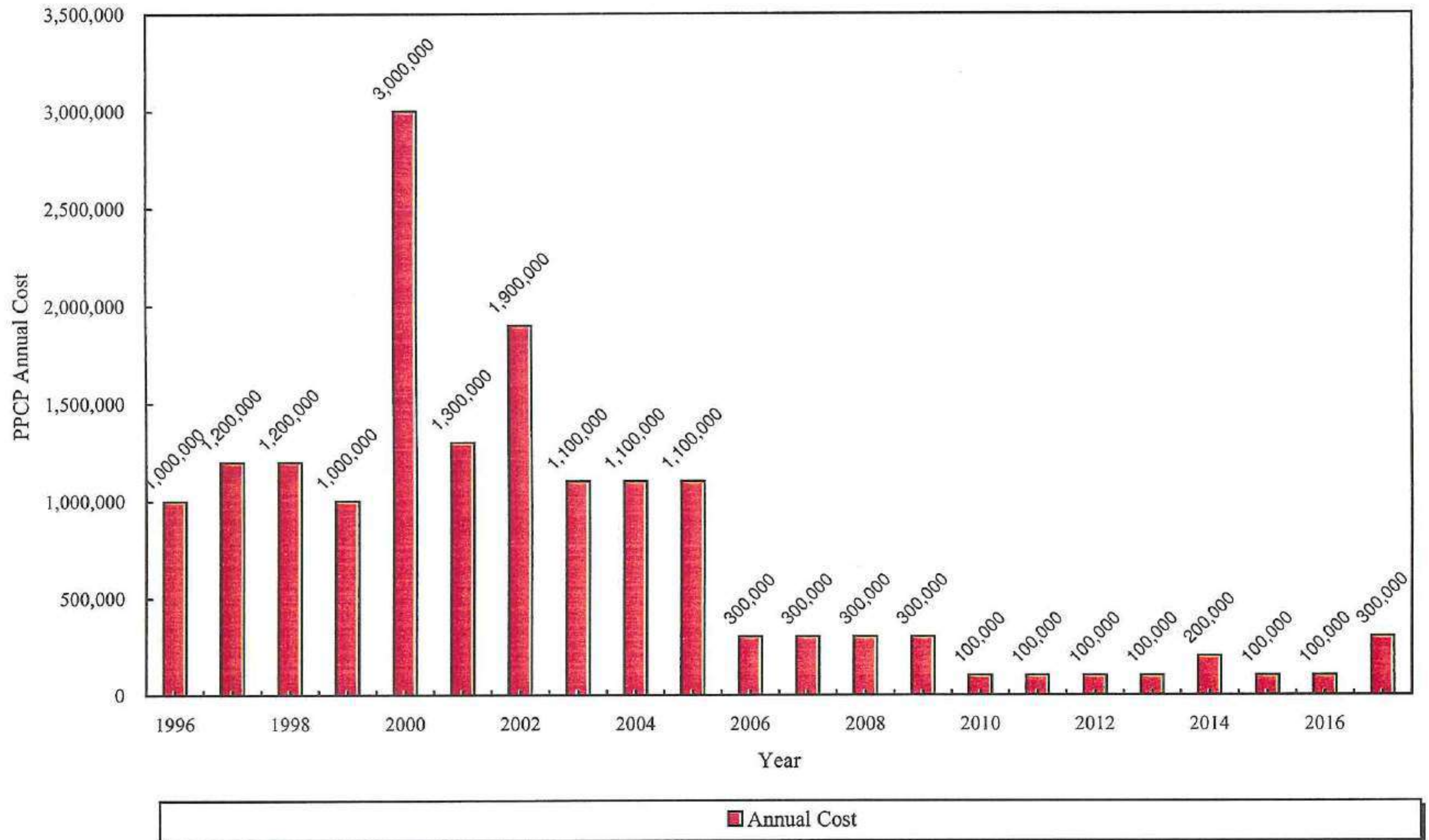


Figure 6.2a
Cash Flow - With WPCP Upgrade
PPCP Annual Cost
12,000,000
10,000,000
8,000,000
6,000,000
4,000,000
2,000,000
0
1,000,000
1,200,000
1,200,000
8,000,000
10,000,000
8,300,000
8,900,000
8,100,000
1,100,000
1,100,000
300,000
300,000
300,000
300,000
100,000
100,000
100,000
100,000
200,000
100,000
100,000
300,000
1996
1998
2000
2002
2004
2006
2008
2010
2012
2014
2016
Year
Annual Cost

Figure 6.2b
Cash Flow - Without WPCP Upgrade



APPENDIX A
MOEE CSO PROCEDURE F-5-5

PROCEDURE F-5-5

DETERMINATION OF TREATMENT REQUIREMENTS FOR MUNICIPAL AND PRIVATE COMBINED AND PARTIALLY SEPARATED SEWER SYSTEMS

1. RATIONALE

Procedure F-5-5 is a supporting document for Guideline F-5 "Levels of Treatment for Municipal and Private Sewage Treatment Works Discharging to Surface Waters".

A Combined Sewer System (CSS) is a wastewater collection system designed to convey both sanitary wastewater and stormwater runoff through a single-pipe system to a sewage treatment works. During dry weather, it conveys sanitary wastewater. During a precipitation event (rainfall or snowmelt) the capacity of the CSS and/or treatment facility may be exceeded by the total wastewater flow. This results in the occurrence of a combined sewer overflow (CSO) which is an untreated mixture often containing high levels of floatables, pathogenic microorganisms, suspended solids, oxygen-demanding organic compounds, nutrients, oil and grease, toxic contaminants and other pollutants. The CSOs represent a potential health hazard and can have adverse effects on aquatic life, recreational uses and water supplies. The goals of this Procedure are to:

(a) eliminate the occurrence of dry weather overflows

(b) minimize the potential for impacts on human health and aquatic life resulting from CSOs

(c) achieve as a minimum, compliance with body contact recreational water quality objectives (Provincial Water Quality Objectives (PWQO) for Escherichia coli (E. coli)) at beaches impacted by CSOs for at least 95% of the four-month period (June 1 to September 30) for an average year.

2. DEFINITIONS

A "combined sewer system (CSS)" is a wastewater collection system which conveys sanitary wastewaters (domestic, commercial and industrial wastewaters) and stormwater runoff through a single-pipe system to a Sewage Treatment Plant (STP) or treatment works. Combined sewer systems which have been partially separated and in which roof leaders or foundation drains contribute stormwater inflow to the sewer system conveying sanitary flows are still defined as combined sewer systems in this Procedure.

A "combined sewer overflow (CSO)" is a discharge to the environment from a combined sewer system that usually occurs as a result of a precipitation event when the capacity of the combined sewer is exceeded. It consists of a mixture of sanitary wastewater and stormwater runoff and often contains high levels of floatables, pathogenic microorganisms, suspended solids, oxygen- demanding organic compounds, nutrients, oil and grease, toxic contaminants and other pollutants.

An "overflow event" occurs when there is one or more CSOs from a combined sewer system, resulting from a precipitation event. An intervening time of twelve hours or greater separating a CSO from the last prior CSO at the same location is considered to separate one overflow event from another.

"Dry weather flow" is sewage flow resulting from both:

(i) Sanitary wastewater (combined input of industrial, domestic and commercial flows); and

(ii) Infiltration and inflows from foundation drains or other drains occurring during periods with an absence of rainfall or snowmelt.

"Wet weather flow" is the combined sewage flow resulting from:

(i) Sanitary wastewater; and

(ii) Infiltration and inflows from foundation drains or other drains resulting from rainfall or snowmelt; and

(iii)Stormwater runoff generated by either rainfall or snowmelt that enters the combined sewer system.

A "regulator" is any structure that in dry weather permits the passage of all flows to treatment and in wet weather permits discharge to an outfall or relief sewer of all flows in excess of some specific flowrate.

An "average year" refers to:

(i) the long term average of flow based on using simulation of at least twenty years of rainfall data and/or

(ii) a year in which the rainfall pattern (e.g. intensity, volume and frequency) is consistent with the long-term mean of the area; and/or

(iii)a year in which the runoff pattern resulting from the rainfall (e.g. rate, volume and frequency) is consistent with the long-term mean of the area.

A "swimming and bathing beach" is a strip of shoreline with the physiographic, climatic, access, and ownership attributes necessary to accommodate significant water contact and non-contact recreation under favourable aquatic conditions.

3. SEPARATE VERSUS COMBINED SEWERS

The Ministry "Guidelines for the Design of Sanitary Sewage Systems, July 1985" states that *"All new sewer construction within the Province of Ontario should be of the 'separate' type, with all forms of storm and groundwater flow being excluded to the greatest possible extent. New 'combined' sewer systems will not be approved.* "

However, existing combined sewers may undergo rehabilitation or be replaced by new combined sewers provided the municipality or operating authority has met the Ministry requirements as set out in this document.

4. MINISTRY REQUIREMENTS FOR MUNICIPAL & PRIVATE COMBINED

SEWER SYSTEMS

To meet the goals of this Procedure each municipality or operating authority of a combined sewer system will be expected to:

(a) develop a Pollution Prevention and Control Plan (PPCP) as outlined in Section 5;

(b) meet minimum CSO controls as outlined in Section 6; and

(c) provide additional controls

- for beaches impaired by CSOs where water quality is not meeting the PWQO for E. coli as outlined in Section 9
- where required by receiving water quality conditions as specified in Procedure B-1-1 "Water Management - Policies, Guidelines, Provincial Water Quality Objectives of the Ministry of Environment and Energy, July 1994".

The site-specific nature and impacts of CSOs are recognized in this Procedure. There is flexibility for selecting controls for local situations.

5. POLLUTION PREVENTION AND CONTROL PLAN (PPCP)

A Pollution Prevention and Control Plan (PPCP) should be developed to meet the goals of the Procedure by:

- outlining the nature, cause and extent of pollution problems;
- examining alternatives and proposing remedial measures; and,
- recommending an implementation program.

Water quality problems may be caused primarily by combined sewer overflows or by a combination of sources including CSOs. Where the pollution problem is due to a combination of sources, the discharges will be investigated and prioritized based on the relevant significance of the various discharges. In some cases the receiving water quality and pollutant transport mechanisms will be assessed in the PPCP.

To address the impact of CSOs the components of the PPCP shall include:

(a) characterization of the combined sewer system (CSS);

- Monitoring, modelling and other appropriate means shall be used to characterize the CSS and the response of the CSS to precipitation events. The characterization shall include the determination of the location, frequency and volume of the CSOs as well as the concentrations and mass of pollutants resulting from CSOs. Through this process the existence and severity of suspected deficiencies will be confirmed. Records shall be kept for combined sewer systems including the following:

- location and physical description of CSO outfalls in the collection system, emergency overflows at pumping stations, and bypass locations at STPs;

- location and identification of receiving water bodies for all combined sewer outfalls;
- combined sewer system flow and STP treatment capacities; present and future expected peak flow rates during dry weather and wet weather;
- capacity of all regulators; and
- location of cross-connections.

- Operational procedures shall be developed for combined sewer systems including the following:

- combined sewer maintenance programs; and,
- regulator inspection and maintenance programs.

(b) an examination of non-structural and structural CSO control alternatives that may include:

- source control;
- inflow/infiltration reduction;
- operation and maintenance improvements;
- control structure improvements;
- collection system improvements;
- storage technologies;
- treatment technologies;
- sewer separation.

(c) an implementation plan with cost estimates and schedule of all practical measures to eliminate dry weather overflows and minimize wet weather overflows.

- The implementation plan should show how the minimum CSO prevention and control requirements and other criteria in this Procedure are being achieved.

6. MINIMUM COMBINED SEWER OVERFLOW (CSO) CONTROLS

The minimum CSO controls consist of the following :

(a) Eliminate CSOs during dry-weather periods except under emergency conditions.

- Each municipality shall demonstrate that the combined sewer system, including the regulators, and associated treatment facilities are adequate for the transmission and treatment of all peak dry weather flows from the service area.

- An emergency condition would exist when e.g. basement flooding, damage to equipment at treatment works or pumping stations, or treatment process washout was occurring or was imminent.

(b) Establish and implement Pollution Prevention programs that focus on pollutant reduction activities at source e.g. reduced use of potential pollutants like fertilizer and pesticides in parks; public education programs on e.g. anti-littering and illegal dumping of used motor oil and other materials into catchbasins; water conservation to reduce dry weather sanitary flow and hence CSOs; street cleaning to reduce CSO floatables; roof-leader disconnection and installing rain barrels to reduce flows into the sewer system; education/assistance for industries to minimize the use/discharge of pollutants; and enforcement of municipal by-laws or regulations.

(c) Establish and implement proper operation and regular inspection and maintenance programs for the combined sewer system in order to ensure continued proper system operation.

(d) Establish and implement a floatables control program to control coarse solids and floatable materials e.g. by reducing the amount of street litter that enters the catchbasins and the CSS; by removing debris from CSOs at the outfalls using measures such as trash racks and screens; and by removing floatables from the surface of the receiving water after a CSO occurs.

(e) Maximize the use of the collection system for the storage of wet weather flows which are conveyed to the Sewage Treatment Plant for treatment when capacity is available e.g. by adjusting regulator settings.

(f) Maximize the flow to the Sewage Treatment Plant for the treatment of wet weather flows e.g. by removing obstructions to flow.

> - The secondary treatment capacity should be utilized as much as possible for treating wet weather flows with the balance of flows being subject to primary treatment. Measures to increase the wet weather hydraulic capacity at the Sewage Treatment Plant (e.g. Step Feed operation) should be investigated.

(g) During a seven-month period commencing within 15 days of April 1, capture and treat for an average year all the dry weather flow plus 90% of the volume resulting from wet weather flow that is above the dry weather flow. The volumetric control criterion is applied to the flows collected by the sewer system immediately above each overflow location unless it can be shown through modelling and on-going monitoring that the criterion is being achieved on a system-wide basis. No increases in CSO volumes above existing levels at each outfall will be allowed except where the increase is due to the elimination of upstream CSO outfalls. During the remainder of the year, at least the same storage and treatment capacity should be maintained for treating wet weather flow. The treatment level for the controlled volume is described in Section 7.

7. LEVEL OF TREATMENT

The treatment processes of the sewage treatment plants should be optimized to minimize the pollutant loadings under wet weather conditions. The Pollution Prevention and Control Planning study should evaluate the operation of the Sewage Treatment Plant under wet weather conditions in consultation with Ministry Regional staff. This may lead to wet weather-specific operating conditions which may produce lower overall pollutant loadings.

During wet weather, the minimum level of treatment required for flows above the dry weather flow (as specified in sections 6 and 9) from combined sewer systems is primary treatment or equivalent. The effluent guideline for primary treatment is 30% carbonaceous biochemical oxygen demand (BOD5) removal and 50% total suspended solids (TSS) removal for an average year during the seven month period as specified in section 6(g). The baseline for the calculation of the average pollutant removal is the influent passing the headworks of the treatment facility under wet weather conditions.

The dry weather flow from combined sewer systems is subject to the process effluent concentration criteria of the STP whether they are primary treatment plants or secondary treatment plants. During wet weather, for secondary treatment plants, the flows through the secondary treatment capacity will be subject to the process effluent concentration criteria of the STP. The flows in the STP which bypass the secondary treatment will be subject to a minimum level of primary treatment.

The treatment of wet weather flows from combined sewer systems may occur at the central Sewage Treatment Plant or at other locations such as satellite treatment facilities. Satellite treatment facilities may be built to treat wet weather flows where there are space limitations or limited capacity in the collection system to get the wet weather flows to the STP. There are a number of satellite treatment technologies some examples of which are vortex separators, high-rate sedimentation, dissolved air flotation and high-rate filtration. Satellite treatment facilities when used to treat wet weather flows from combined sewer systems are subject to the minimum level of primary treatment requirements specified above. In addition, for satellite treatment facilities the effluent concentration for total suspended solids should not exceed 90 mg/l for more than 50 % of the time for an average year during the seven-month period as specified in section 6 (g).

8. EFFLUENT DISINFECTION

Effluent disinfection is required where the effluent affects swimming and bathing beaches and other areas where there are public health concerns. The local Medical Officer of Health identifies public health concerns such as e.g. whether recreational beaches are safe for swimming.

The interim effluent quality criterion for disinfected combined sewage during wet weather is a monthly geometric mean not exceeding 1000 E. coli per 100 ml. This criterion may be modified by the Regional staff of the Ministry on a case-by-case basis due to site-specific conditions.

In cases where chlorination is used as the disinfection process, subsequent dechlorination of the sewage works effluents shall be used to minimize the adverse effects of chlorine residuals on public health and the aquatic environment where necessary.

All bypasses at the Sewage Treatment Plant should be subjected to the disinfection process where available in order to reduce the bacterial loadings at discharge.

9. BEACH PROTECTION

Additional controls above the minimum CSO controls (section 6) are required for swimming and bathing beaches affected by CSOs and consist of the following :

(a) There should be no violation of the body contact recreational water quality objective (Provincial Water Quality Objectives (PWQO)) for E. coli of 100 E. coli per 100 ml. based on a geometric mean at swimming and bathing beaches as a result of CSOs for at least 95% of the four-month season (June 1 to September 30) for an average year.

(b) Controlling to not more than two overflow events per season (June 1 to September 30) for an average year in a combined sewer system with the combined total duration of the CSOs at any single CSO location being less than 48 hours and ensuring that the controlled combined sewage which does not overflow receives a level of treatment (as specified in section 7) plus disinfection (as specified in section 8) is deemed to satisfy section 9(a). An additional overflow event per season may be allowed if the proponent can demonstrate that section 9(a) will still be satisfied and the combined total duration of the CSOs at any single CSO location will be less than 48 hours.

10. MONITORING

Monitoring of wastewater flows and overflows should be undertaken at locations within the sewer system for the purposes of assessing upgrading requirements and determining compliance with Ministry requirements. The nature of monitoring programs shall be specified in the Pollution Prevention and Control Plan or as determined by the Ministry through its Regional staff. The responsibility for providing monitoring shall rest with the municipality or operating authority of the combined sewer system.

11. NEW SANITARY CONNECTIONS TO COMBINED SEWER SYSTEMS

When and where significant combined sewer system deficiencies exist, the Regional Office of the Ministry shall require that the provision of sanitary servicing for additional development tributary to the deficient system be curtailed to prevent aggravation of the problem until the necessary upgrading, as outlined by a Pollution Prevention and Control Plan is carried out in keeping with the requirements of this Procedure. Some development is allowed as upgrading proceeds, conditional upon its progress. The staged upgrading should at a minimum provide for the transmission and treatment of all flows from the additional development.

This provision applies to significant development i.e. not to simple, one lot infill cases.

12. NEW STORM CONNECTIONS TO COMBINED SEWER SYSTEMS

New storm drainage systems shall not be permitted to connect to existing combined systems if that increases the gross area serviced by the combined sewer system except where evaluations indicate that circumstances allow no other practical alternative. The evaluations must be documented as part of a Pollution Prevention and Control Plan.

"Piece-meal" construction on existing combined sewer systems will be permitted only with overriding justification such as for the purpose of relocation (e.g., to accommodate underground utilities, subway structures, new buildings and pedestrian tunnels, etc.) or for the purpose of capacity improvement (e.g., to relieve basement flooding or to provide emergency additional conveyance capacity to treatment works to reduce overflows) or for rehabilitating deteriorated sewer conditions.

13. ENFORCEMENT

Procedure F-5-5 will be used to:

(a) review applications for approval to ensure that the proponent is in compliance with the Procedure prior to the issuance of a Certificate of Approval.

(b) assist regional staff in setting minimum requirements in preparing Control Orders to bring systems into compliance with the Procedure.

(c) assist enforcement staff in evaluating a combined sewer system operator's due diligence when investigating violations of the Environmental Protection Act and/or the Ontario Water Resources Act.

Any deviation or relaxation from this Procedure should be reviewed by the Regional Director and the Director, Program Development Branch.

APPENDIX B
TECHNICAL MEMORANDUM WPCP UPGRADE OPTIONS

Appendix B
TECHNICAL MEMORANDUM
THUNDER BAY WPCP UPGRADE OPTIONS

1.0 *INTRODUCTION*

An important component of the Thunder Bay Pollution Prevention and Control Plan involves an assessment of the need to expand and upgrade the existing Thunder Bay WPCP to provide full secondary treatment. A number of alternatives have been identified as possible options for upgrading the existing facility from primary treatment to secondary treatment. The objective of this part of the study was to identify the most cost-effective secondary treatment alternative.

Initial phases involved identification of projected flows, concentrations, and effluent quality. Based on the projections, a preliminary review of a number of possible secondary treatment technologies was performed. The treatment alternatives were screened to identify the preferred treatment upgrade alternative. A similar analysis was performed for the selection of the effluent disinfection process. The biosolids handling facility was reviewed to determine the impact of secondary treatment.

Following selection of the preferred secondary treatment upgrade alternatives, a more detailed design analysis was performed for each. This included process sizing, conceptual site layout, and capital and operating cost estimations.

2.0 *DESCRIPTION OF EXISTING FACILITIES*

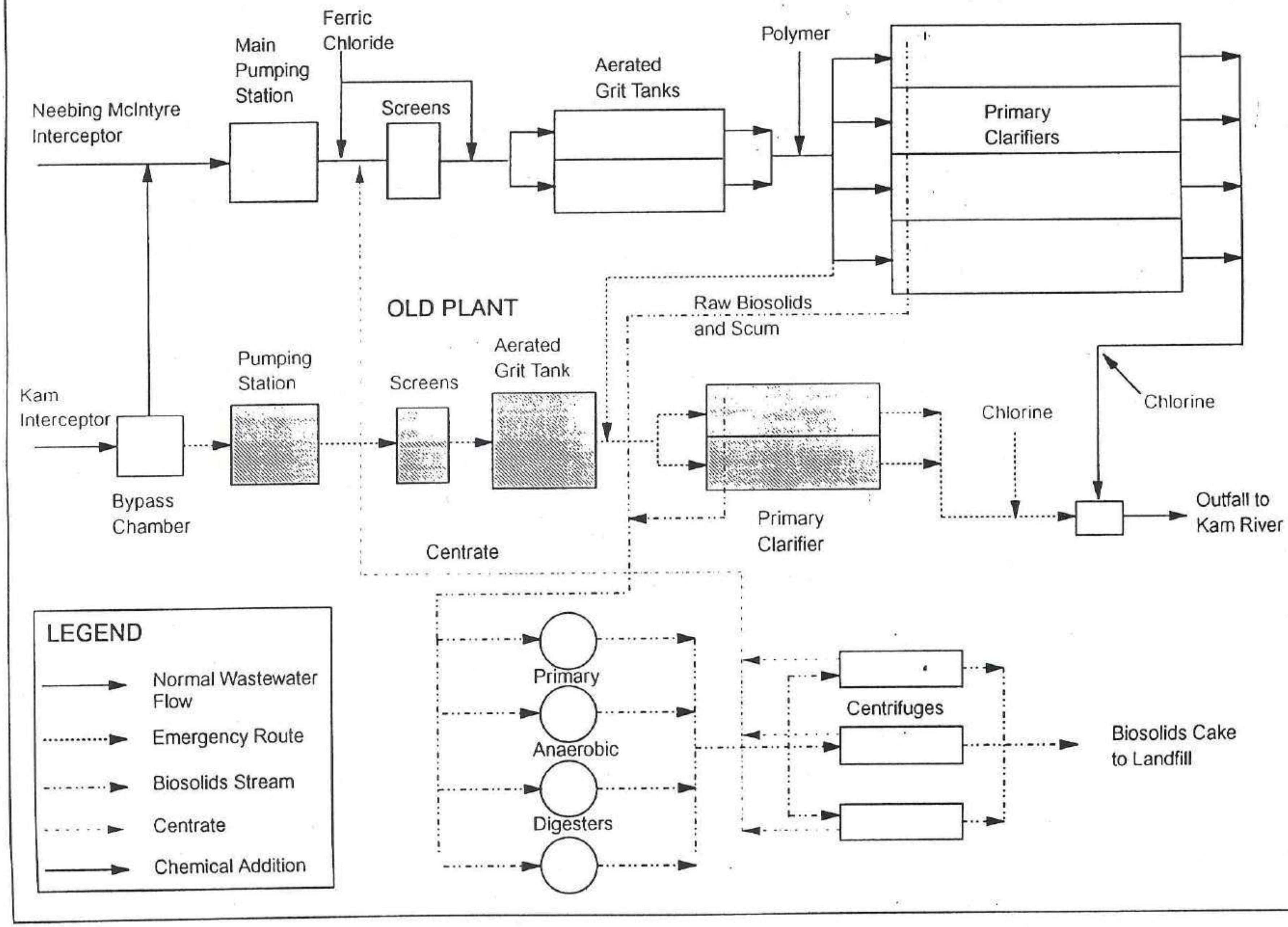
The "Thunder Bay Pollution Prevention and Control Plan Phase 1 - State-of-the-System Report" described the design, operation and performance of the existing Thunder Bay WPCP in detail. These descriptions are summarized in this sub-section. Figure 1 presents a flow schematic of the existing facilities.

2.1 *Site Layout*

The Thunder Bay WPCP consists of two separate primary treatment trains. The original facility or Old Plant is rated for average flows of 27,600 m^3/d and is used only for treatment of peak wet weather flows. Currently, all raw wastewater flow is directed to the newer facility with an rated capacity of 81,500 m^3/d. The rated average flow capacity of the overall facility is 109,100 m^3/d.

To the east of the existing primary clarifiers, a total land area of approximately 23,000 m^2 is available for the secondary facility. This area consists of about 10,000 m^2 to the east of the New Plant primary clarifiers, and 13,000 m^2 to the east of the Old Plant primary clarifiers. There is also land available to the west of the existing site.

FIGURE 1: PROCESS FLOW SCHEMATIC OF THE THUNDER BAY WPCP



2.2 *Liquid Treatment*

Two separate drainage areas feed the Thunder Bay WPCP. Wastewater from the North Ward flows to the main pumping station before being pumped directly to the New Plant. Wastewater from the South Ward flows to a diversion chamber on-site, where it is normally directed to the main pumping station. Under high flow conditions, a portion of wastewater entering the diversion chamber is directed to the Old Plant.

Wastewater pumped to the new plant is dosed with ferric chloride to precipitate phosphorus and then passes through four parallel automatically cleaned bar screens. Screened wastewater is treated in two parallel grit tanks. Polymer is added to grit tank effluent to enhance phosphorus removal and suspended solids settling.

Degritted wastewater is directed to four parallel rectangular primary settling tanks through an aerated distribution channel. Clarified effluent is chlorinated during the period April 15 to November 15, before discharge through an outfall to the Kaministiquia River. There is no chlorine contact chamber. Contact time for disinfection is provided in the outfall pipe.

During high flow periods, the flow is diverted to the old pumping station and headworks when the Kaministiquia Interceptor 760 mm (30 in) bypass surcharges. Wastewater pumped from the old pumping station to the new primary clarifiers is not dosed with chemicals for phosphorus removal.

2.3 *Biosolids Treatment*

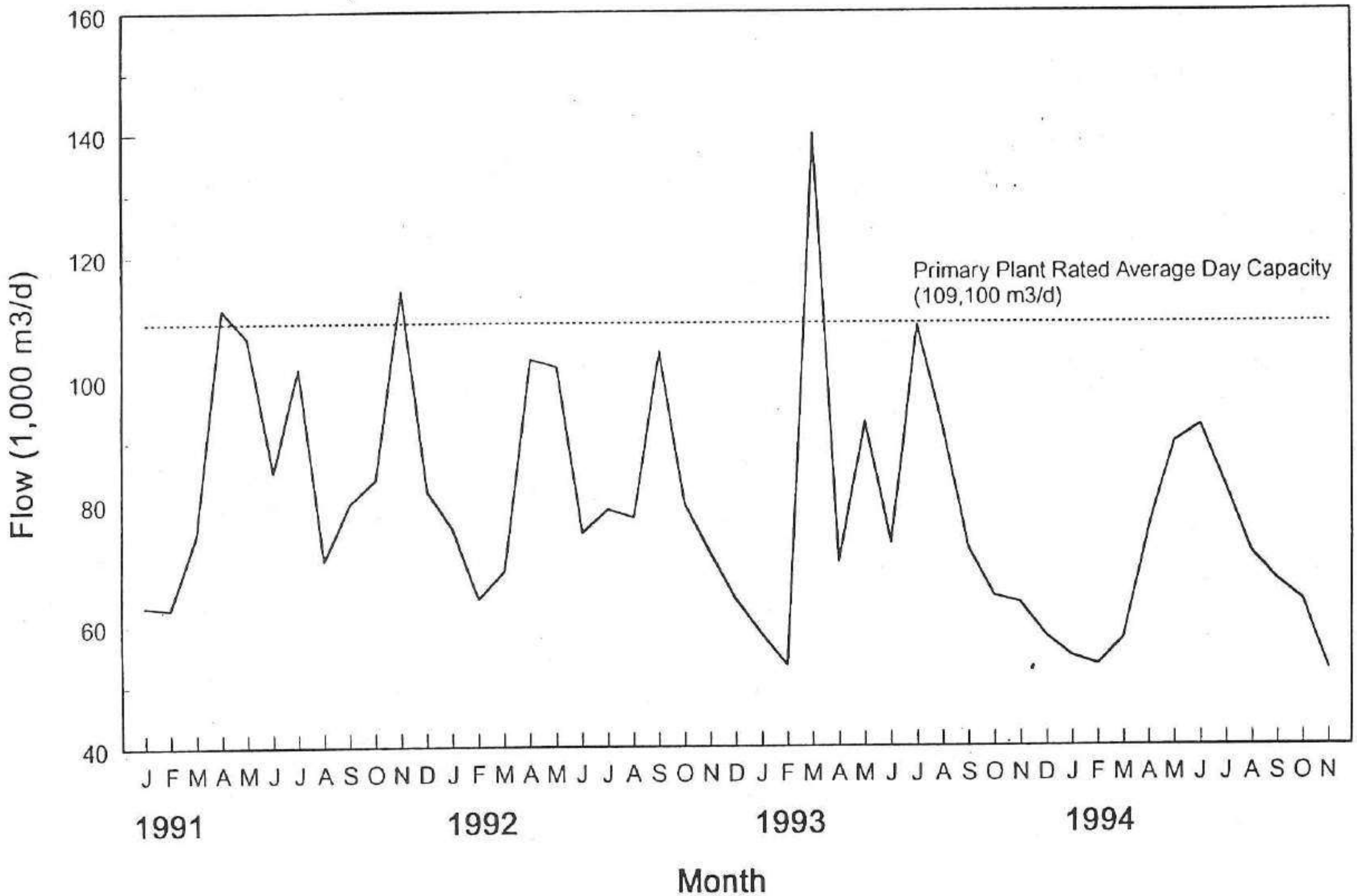
Raw biosolids generated at the Thunder Bay WPCP are treated in four primary anaerobic digesters providing a total volume of 4,886 m^3. Digested biosolids are dewatered in two of three available parallel centrifuges (ie. 2 operating, 1 stand-by), each with a capacity of 6 L/s. Centrifuge centrate is returned to the head of the plant before the screens. The biosolids cake is transported off-site for landfill disposal.

3.0 *DESIGN BASIS*

3.1 *Flows*

Figure 2 presents the monthly average flows over the historic period from 1991 to November 1994. Results show that flows increase during wet-weather periods, indicating

Figure 2
Monthly Average Flows
January 1991 to November 1994



infiltration from extraneous flow sources. Over the period, the rated average day plant capacity was exceeded during three months. Annual average flows over the period are presented in Table 1.

Table 1
Historic Flows to the Thunder Bay WPCP

Year	Annual Average Flow (m^3/d)
1991	86,340
1992	80,420
1993	71,880
1994[1]	69,300

Note:

1. Based on January to November average.

The highest annual average flow over the period occurred during 1991. These flows are equivalent to 809 L/cap.d, assuming a constant population of 106,745. This rate includes residential, commercial, and industrial wastewater generation plus contributions from extraneous flows in the existing collection system.

Based on projected population growth to the year 2016, flow projections were estimated. The results are presented in Table 2. In addition to growth estimates, the City plans to discharge backwash water from Bare Point to the sewer system. Backwash water flow is anticipated to be as high as 10,368 m^3/d.

Average forecast flows to the year 2016 are estimated at 103,653 m^3/d. Thus, the existing primary facility which is rated for average day flows of 109,100 m^3/d will be able to handle the projected flows, and thus will not require expansion or changes to the existing Certificate of Approval rated capacity. However, the Old Plant which is over 30 years old has mechanical equipment that is obsolete and is not structurally sound. Replacement of the Old Plant headworks and primary clarifier capacity will be required. The secondary treatment upgrade will be designed to provide treatment for the existing rated capacity.

Flow projections may be reduced through water conservation initiatives and extraneous flow removal from the collection system. Conservation initiatives can be carried out through public education and plumbing requirements for new homes. Changes in plumbing code requirements will reduce flow through implementation of low flow toilets, water saving shower heads, etcetera. These reductions could reduce the size requirements of hydraulically limited unit processes such as secondary clarifiers, but will have no impact on the size of organically limited processes such as biological reactors. The net effect of such measures on the size and cost of the secondary facility at Thunder Bay will be minimal.

Table 2
Forecasted Average Day Flows to the Thunder Bay WPCP

Description	Population	Per Capita Flow (L/cap.d)	Total Flow (m^3/d)
Existing Population	106,745	809	86,340
Population growth to year 2016	13,255	524[1]	6,945
Water Treatment Plant (WTP) Backwash Water	-	-	10.368
Total for year 2016	120,000	---	98,469

Note:

1. Based on 80% of the average annual water consumption rate of 542 L/cap.d, or 434 L/cap.d, plus 90 L/cap.d for infiltration in new sewers recommended by MOEE guidelines. Average domestic water consumption included residential, commercial, and industrial water use, calculated from water records from 1983 - 1987.

3.2 *Contaminant Concentrations and Loadings*

Raw sewage characteristics and primary effluent quality over the period from 1991 to 1993 are presented in Table 3.

Historic data indicates excellent primary clarifier performance with average BOD_5 and TSS removals of 57% and 79% respectively. For the period 1991 to 1993, the maximum monthly average primary effluent BOD_5 concentration was 118 mg/L, and for TSS was 102 mg/L. Forecast raw wastewater loadings to the Thunder Bay WPCP in the design year 2016 are presented in Table 4.

Table 3
Raw Sewage and Primary Effluent Concentrations over the Period 1991 to 1993

Period	BOD_5 (mg/L)			TSS (mg/L)			TP (mg/L)		
	Raw Waste-water	Primary Effluent	% removal	Raw Waste-water	Primary Effluent	% removal	Raw Waste-water	Primary Effluent	% removal
1991	166	67	60%	246	53	79%	3.86	1.00	74%
1992	162	69	57%	232	40	83%	4.00	0.99	75%
1993	173	81	53%	190	46	76%	4.15	1.08	74%
Average	167	72	57%	223	46	79%	4.00	1.02	75%
Max Monthly Average	249	118	--	359	102	--	8.5	1.48	--

Table 4
Forecasted Raw Wastewater Contaminant Loadings to the Thunder Bay WPCP

Description	Population	Total Flow (m^3/d)	BOD_5 Loading (g/cap.d)	BOD_5 Conc. (mg/L)	TSS Loading (g/cap.d)	TSS Conc. (mg/L)	TP Loading (g/cap.d)	TP Conc. (mg/L)
Existing Population	106,745	86,340	139(1)	172	199(1)	246	3.7(1)	4.6
Population Growth	13,255	6,945	80(2)	153	90(2)	172	4.0(2)	7.6
WTP Backwash Water	-	10,368	-	-	-	-	-	-
Total	120,000	103,653	132	154	187	216	3.7	4.4

Notes:

1. Maximum annual contaminant loading reported for period 1988 to 1993.
2. Typical unit contaminant loadings for domestic wastewater.

The results in Table 4 indicate the forecast raw wastewater contaminant concentrations are similar to current raw wastewater concentrations.

3.3 *Secondary Plant Peak Capacity*

XP-SWMM, EXTRAN BLOCK was used to predict secondary plant peak flows during the wettest month of a wet year (June, 1984). Modelling of storm events during June 1984 indicated that a secondary plant peak factor of 1.4 would result in less than 3% of the total plant wastewater volume bypassing the secondary facility during the wettest month of a wet year. The small volume (< 3%) by-passing the secondary plant during the wet weather period would still receive primary treatment followed by disinfection.

For a more conservative secondary plant design, the peak factor of 1.4 will be assumed to be the maximum total daily flow to the secondary treatment facility. Peak instantaneous flows will be higher and a peak factor of 1.8 will be applied for the design. Typically, secondary plant design peak factors are greater than around 2.0. The lower peak factor in the case of Thunder Bay is a result of extensive modelling and monitoring which indicates that a lower peak factor will be adequate. Also, since the wettest month of a wet year (June, 1984), an extensive sewer separation program has been completed aimed in the South Ward combined sewer area at reducing wet weather flow sources.

3.4 *Secondary Treatment Facility Loading Summary*

The summary of the design loading parameters for the secondary treatment facility are presented in Table 5. The design parameters for the secondary facility assume continued chemical addition to the primary clarifiers. If this were discontinued for practise of simultaneous precipitation (addition within the secondary process) then loading forecasts would most likely increase, influencing secondary treatment process design.

3.5 *Effluent Criteria*

Critical to the selection of the secondary treatment alternative is the ability to meet the effluent criteria and compliance objectives. Preliminary discussions with the MOEE Thunder Bay Regional Office and the Thunder Bay Remedial Action Plan (RAP) Committee have indicated that upgrading to meet current MOEE Procedure F-5-1 and F-8-1 would be appropriate based on the current receiving water quality. The effluent compliance criteria for the design of the Thunder Bay WPCP secondary treatment facility are outlined in Table 6.

Table 5
Design Loading for Secondary Treatment Upgrade

Description	Average	Maximum Day	Peak Instantaneous
Flow	109,100 m^3/d	152,740 m^3/d[1]	196,380 m^3/d[2]
BOD_5 load	8,400 kg/d	15,400 kg/d	---
BOD_5 concentration	77 mg/L[3]	-	---
TSS concentration	76 mg/L[4]	-	---

Notes:

1. Based on secondary plant peak day flow factor of 1.4.
2. Based on secondary plant peak instantaneous flow factor of 1.8.
3. Based on conservative primary clarifier BOD_5 removal of 50%.
4. Based on conservative primary clarifier TSS removal of 65%.
5. Pro-rated maximum month BOD_5 from 1991 to 1994 times secondary plant peak day factor.

Table 6
Thunder Bay Secondary Effluent Criteria

Parameter	Design Objective	Compliance Criteria
BOD_5	15 mg/L (annual average)	25 mg/L (annual average)
TSS	15 mg/L (monthly average)	25 mg/L (annual average)
TP	1.0 mg/L (monthly average)	1.0 mg/L (monthly average)

4.0 *PRELIMINARY REVIEW OF SECONDARY TREATMENT TECHNOLOGIES*

A review of a number of secondary treatment alternatives was conducted, including:

- Activated Sludge Processes (Conventional, High Rate, Step Feed, Pure Oxygen)
- Rotating Biological Contactor (RBC)
- Trickling Filter (TF)
- Trickling Filter/Solids Contact (TF/SC)
- Biological Aerated Filters (BAF)

- Deep Shaft Technology
- Biological Phosphorus Removal Processes (A/O, UCT/VIP).

Screening criteria included the ability of the process to meet effluent limits, land area requirements, and relative capital and operating costs.

Screening of alternatives was performed by weighting a number of important considerations in the selection of a secondary treatment process. Screening parameters and weighting were developed by W2O, with input from the staff at the City of Thunder Bay and the Project Steering Committee.

The most important considerations in the upgrade selection were identified as the ability to meet compliance criteria, land area requirements, and costs (capital and operating). The importance of each of these is crucial to the selection of the optimum secondary treatment option.

Based on the screening criteria, the preferred secondary treatment upgrade alternatives would be biological aerated filters (BAF) and convention activated sludge (CAS). BAF processes provide exceptional effluent quality, with the smallest land requirements. Also in a BAF, the biological treatment and solids separation occurs in one unit process rather than two.

The conventional activated sludge process is the most common secondary treatment process in Ontario of all those reviewed. A well operated CAS can consistently achieve good effluent quality. CAS also provides operational flexibility to maintain effluent quality during seasonal variations in flow, load and temperature. The step feed option of the CAS was ranked similar to the CAS process and should be considered for Thunder Bay due to the flexibility that it provides under wet weather conditions.

5.0 *DETAILED EVALUATION OF SELECTED SECONDARY TREATMENT ALTERNATIVES*

The following section presents a summary of the detailed evaluation conducted. Details of the analysis are presented in the Thunder Bay WPCP Evaluation of Secondary Treatment, Upgrade Options Technical Memorandum. Based on the screening of a number of secondary treatment alternatives, two processes were selected for detailed evaluation. The selected processes were the biological aerated filter (BAF) and the conventional activated sludge (CAS) process. The evaluation included process sizing, conceptual site layout, and preliminary capital and operating cost estimates. The recommendations based on the evaluation are presented in Section 7.0.

Three commercially available biological aerated filters were evaluated: BIOCARBONE, BIOFOR and BIOSTYR.

Capital costs include equipment, instrumentation, materials, engineering fees, and installation. These costs are based on a conceptual level of design and utilize cost curves and standard cost estimation techniques. These costs are generally considered to be accurate to within -25% to +40%. Costs will be affected by site specific factors such as soil conditions, condition of existing hardware, etc. Wardrop Engineering developed capital cost estimates for the BIOFOR. Capital equipment costs for the BIOCARBONE and BIOSTYR processes were also obtained. The additional process equipment costs for these processes were assumed to be similar to those for the BIOFOR. Table 7 presents the capital cost estimates for the BAF secondary treatment options.

The operating and maintenance costs at the existing Thunder Bay primary WPCP will increase with the addition of a secondary treatment process. The major operating and maintenance costs typically consist of electrical energy costs, chemical costs, and labour. Electrical energy costs will increase due to the additional aeration and pumping requirements. Chemical costs at the facility are mainly associated with the pre-precipitation of phosphorus down to an effluent residual of approximately 1 mg/L. Effluent limits for phosphorus with secondary treatment will remain at 1 mg/L (see Section 3.5) and thus chemical costs should not change significantly with an upgraded facility. In fact, chemical dosage may be reduced as biological phosphorus requirements during secondary treatment will reduce the degree of phosphorus removal required in the primary treatment stage. Labour requirements are difficult to estimate for any treatment process, and in this case will depend on the additional staff needed to maintain and operate the new system. Sludge management costs will also increase due to the increased sludge production from the secondary facility to be stabilized, thickened, and disposed. O & M costs will not differ significantly for either the BAF or CAS secondary treatment option.

Table 7 indicates that the BIOSTYR and BIOFOR are the least expensive BAF options, while the BIOCARBONE is about $6 million more expensive.

Other impacts from the BAF process include the possibility of odours associated with it, especially in the case of the BIOCARBONE process where the exiting air is contacting raw sewage and may remove some volatile organics. In the BIOFOR and BIOSTYR processes, odour problems should not be significant since the exiting air has last contacted treated water. Odour control, if required, can be accomplished through use of a bio-scrubber. The scrubber must be sized to treat all process air, and thus must have a firm installed capacity for treating 24,000 m^3/hr.

Table 7
Estimated Capital Costs for BAF Secondary Treatment Upgrades to the Thunder Bay WPCP (Cost in 000's $)

Description	BIOCARBONE	BIOFOR	BIOSTYR
Process Equipment, instrumentation and installation	17,555	12,782	12,965
Concrete/Piles/Excavation	4,800	5,568	3,970
Piping	290	290	290
Electrical	613	613	613
Chlorination	70	70	70
Other	650	650	650
Sub-total	23,978	19,973	18,558
Contractor Overhead & Profit (10%)	2,398	1,997	1,856
Contingencies (15%)	3,597	2,995	2,784
Engineering Fees (15%)	3,597	2,995	2,784
Total	$33,570	$27,960	$25,982

In addition, discussions with West Windsor PCP staff indicated that there were numerous operational problems with the pilot scale BIOCARBONE unit tested. These related to unequal flow distributions within the filter resulting in uneven solids buildup. This created air pockets within the filter, and odours developed due to low D.O. conditions in some parts of the filter.

Later pilot studies with a BIOFOR unit at the West Windsor PCP indicated no operational difficulties, little or no odours, and excellent effluent quality similar to the BIOCARBONE unit reported earlier. Pilot studies on the BIOSTYR began in September, 1995 at the West Windsor PCP.

The conventional activated sludge process (CAS) is one of the most common secondary treatment processes in Canada. The process provides reliable effluent quality and flexibility to deal with a wide variety of environmental conditions.

Two CAS designs are presented within this section. One is based on MOEE Design Guidelines for a conventional activated sludge process. A second case is presented for a high rate aeration tank design based on site specific loading conditions. This results in a less conservative design approach; however, pilot studies may be required to confirm these design parameters to the satisfaction of MOEE Approvals Branch.

Wardrop Engineering developed capital cost estimates for both CAS designs. These costs are based on a conceptual level of design and utilize cost curves and standard cost estimation techniques. These costs are generally accurate to within -25% to +40%. The capital cost estimations are presented in Table 8.

The operating and maintenance costs at the Thunder Bay WPCP will increase with the addition of a secondary treatment process due to increased electrical energy, labour, and sludge management requirements. These increases are difficult to estimate, but are not anticipated to differ significantly from those for BAF secondary treatment.

6.0 *DISINFECTION*

Disinfection is provided to reduce the amount of pathogenic organics in the effluent from the facility. Three disinfection upgrade alternatives have been considered: chlorination, chlorination/ dechlorination, and ultraviolet (UV) irradiation. Other disinfection alternatives are available such as ozonation, but these have not been reviewed due to limited experience with these technologies in Ontario. Ozonation was originally used for disinfecting water supplies. For ozonation, the strong oxidizing agent, ozone, is introduced into the wastewater where it is believed to kill bacteria through disintegration of the cell wall.

Table 9 presents the capital, operating and maintenance (O & M), and life cycle cost estimates for the disinfection alternatives reviewed. All O & M cost estimates are based on year-round disinfection. For the UV system, capital costs were estimated based on costs determined for a facility treating similar quantities of wastewater as Thunder Bay WPCP. Cost estimates include the contact basin, UV lamps, instrumentation, and jibs (removal of lamps). Operating and maintenance costs include that associated with energy costs, labour, lamp replacement, and chemical cleaning costs. Energy costs were estimated using approximate electricity costs at Thunder Bay WPCP ($0.07/kWh). The other O & M costs were estimated using typical requirements as outlined in WERF (1995) which compared UV disinfection to chlorination. In addition, conversion to alum addition for phosphorus removal may reduce sludge mass generation rates, decreasing sludge handling costs. However, this reduction is estimate at only 5% and thus has been considered negligible in the life cycle costing analysis.

Table 8
Estimated Capital Costs for CAS Secondary Treatment Upgrades to the Thunder Bay WPCP (Cost in 000's $)

Description	High Rate Aeration	MOEE Aeration
Process Equipment, instrumentation and installation	9,372	9,372
Concrete/Piles/Excavation	12,225	14,956
Piping	550	550
Electrical	739	739
Chlorination	70	70
Other	725	745
Sub-total	23,681	26,432
Contractor Overhead & Profit (10%)	2,368	2,643
Contingencies (15%)	3,552	3,965
Engineering Fees (15%)	3,552	3,965
Total	$33,153	$37,005

The costs of the chlorine contact tank for the chlorination/dechlorination option was estimated using 1986 CAPDET cost curves prorated to 1994 dollars using the ENR construction cost index. Dechlorination facilities equipment requirements were estimated using values reported in WERF (1995). Operating costs for chlorination/dechlorination are mainly associated with the purchase of chemicals, although some labour is involved in residual monitoring and contact basin cleaning. Chlorine costs were based on existing unit costs at the Thunder Bay WPCP ($0.75/kg).

Table 9
Estimated Life Cycle Costs for Various Disinfection Alternatives.

Disinfection System	Capital Cost Estimate[1] ($)	O & M Costs ($/annum)	Life Cycle Costs[2] ($/annum)
Chlorination	$163,000	$155,000	$168,100
Chlorination/Dechlorination	$2,113,000	$185,000	$354,500
UV Disinfection	$2,600,000	$130,000	$338,520

Notes:

1. Capital cost estimates include contingencies (15%) and engineering (15%).
2. Life cycle costs based on 20 years and 5% interest.

The large difference in going from chlorination to chlorination/dechlorination is mostly associated with the need for a chlorine contact basin. As well, all equipment associated with dechlorination is required (evaporators, scrubbers, scales, etc.).

Life cycle costs indicate that chlorination is the cheapest disinfection alternative, mainly due to the minimal capital expenditures required.

Chlorination is currently practised at the facility, and thus staff are already familiar with the process. Chlorine injectors, in addition to those already existing at the Thunder Bay WPCP, will be required to meet effluent disinfection requirements. As mentioned earlier, the outfall sewer provides adequate chlorine contact time for disinfection.

Based on cost, performance, safety and experience in Ontario, chlorination is the preferred disinfection alternative for the Thunder Bay WPCP. The existing outfall at the Thunder Bay WPCP provides adequate disinfection contact time at design peak flows. However, additional chlorine injectors will be required to meet the disinfection demand during peak flow periods.

7.0 *RECOMMENDATIONS*

Process design requirements were developed for three commercially available biological aerated filters and two conventional activated sludge process designs. Based on the expected effluent quality, ease of operation, and capital and operating costs for the BAF processes, they were recommended as the secondary treatment upgrade option at the Thunder Bay WPCP.

BAF processes can typically achieve effluent BOD_5 and TSS concentrations of less than 10 mg/L each. The processes are also simple to operate with backwash cycles that can be set either on a timed basis or based on exceedence of an allowable headloss through the filter bed.

Due to the possible operational difficulties and odour problems reported to occur with the BIOCARBONE process, it is recommended that either the BIOFOR or BIOSTYR process be considered for implementation. The BIOFOR process is currently installed in full-scale facilities in Quebec and operating successfully. The BIOSTYR process is a relatively new process being developed and marketed by the same company that markets the BIOCARBONE process. Additional investigation of this process should be performed before implementation. Information from the BIOSTYR pilot studies at West Windsor PCP will be useful.

A gravity thickener has been included for concentrating the BAF backwash solids. Results of an investigation into the peak factor for the Thunder Bay WPCP indicates that a peak design factor of 2.0 is adequate for the facility. Based on the historic peak factor, it appears that the existing primary clarifiers would operate at peak surface overflow rates (SOR) similar to MOEE Design Guidelines with waste activated sludge co-thickening. Thus, separate thickening equipment is not necessary for the conventional activated sludge option based on the historic peak flow factor.

All of the process designs tend to be conservative and capital cost savings could likely be realized through pilot testing. For example, all BAF designs were based on maximum sustained daily BOD_5 loads. Pilot testing will confirm process performance of the BAF units with Thunder Bay WPCP wastewater and climatic conditions, and help to obtain MOEE approval for the final design. Also, this will allow investigation into the impact of the backwash solids on the primary clarifiers when co-thickening is practised.

If pilot testing is to be performed on the BAF units at the Thunder Bay WPCP, it is recommended that a pilot scale conventional activated sludge (CAS) process be run in parallel with the BAF units. This will allow for direct comparison of the effluent quality from each process. Also, this will allow for an optimal CAS design in case it is decided to be implemented at the Thunder Bay WPCP for secondary treatment.

Chlorination is recommended as the method of disinfection at the Thunder Bay WPCP. The is the cheapest option on a life cycle basis. However, if the City of Thunder Bay wishes to discontinue disinfection through chlorination, UV irradiation would be the preferred alternative.

8.0 *REFERENCES*

CH2M Hill, 1994. *Pilot Plant Evaluation of UV Disinfection For the Skyway WPCP.* Prepared for the Regional Municipality of Niagara.

EPA (U.S.), 1982. *Technology Assessment of the Deep Shaft Biological Reactor.* EPA-600/2-82-002.

Gore & Storrie Ltd., 1988. *City of Thunder Bay Water Pollution Control Plant Optimization Study.*

Grady, C.P.L Jr., Lim, H.C., 1980. *Biological Wastewater Treatment: Theory and Applications.* Marcel Dekker Inc.

Metcalf & Eddy Inc., 1991. *Wastewater Engineering: Treatment, Disposal, Reuse.* McGraw Hill inc.

Meunier, C., 1995. John Meunier Inc., Montreal. Personal Communication.

MOEE, 1984. *Guidelines for the Design of Water Treatment Plants and Sewage Treatment Plants.*

MOEE, 1993. *Evaluation of Innovative Alternatives for the West Windsor PCP Upgrade.* ISBN 0-7778-0821-8.

Pollock, D., 1995. Deep Shaft Technology Inc., Alberta. Personal Communication.

Romano, L., 1995. City of Windsor. Personal communication.

WEF, 1991. *Design of Municipal Wastewater Treatment Plants.*

APPENDIX C
REGULATOR TECHNOLOGY

HISTORICAL PERSPECTIVE: USE OF VORTEX FLOW THROTTLES AS FLOW CONTROLLERS IN SEWERAGE SYSTEMS

William C. Pisano, Ph.D.* and Prof.-Dr.-Ing. Hans Brombach**

*Principal, Havens and Emerson, Boston
**President, UFT, Bad Mergentheim & Prof. Stuttgart University, Germany

Presented at

AWPCA/WEF National Specialty Conference
"Collection Systems - Operation and Maintenance"

June 27-30, 1993 at Tucson, Arizona

HISTORICAL PERSPECTIVE : USE OF VORTEX FLOW THROTTLES AS FLOW CONTROLLERS IN SEWERAGE SYSTEMS

William C. Pisano, P.E., Ph.D. * and Prof.-Dr.-Ing. Hans Brombach **

* Principal, Havens & Emerson, Boston

** President, UFT, Bad Mergentheim & Prof. Stuttgart University , Germany

1. FORWARD

This paper summarizes the practice in Western Europe and North America of using static flow controllers within sewerage systems to provide remote "self-inducing" flow throttling using vortex principles. There exist nearly 8000 such installations with capacities from 0.5 to 300 cfs. The hydraulic principles governing the unique bi-stable flow characteristics of vortex throttles are reviewed to understand the flow characteristics that can be created by geometric manipulation of vessel dimensions. Controllers with no feedback were initially developed to provide an inexpensive alternative to mechanical or electrically driven controllers. Demand for adaptive feedback flexibility in German storm overflow facilities has lead to the development of motor driven knife gates (electrically controlled with microcomputers and sensors) preceding the vortex throttle (200 installations). In a further development, a self contained "TURBO" within the vortex throttle operates an oil cylinder pump which modulates the preceding knife valve, thereby eliminating the need for external power source (50 installations).

2. HISTORY : VORTEX VALVE TECHNOLOGY

The roots of vortex valve technology date back many years. In the late 1920's, a non-return valve, or vortex diode, was invented in Germany in order to reduce the danger of uncontrolled blow-off of hot steam in the case of pipe fractures (Thoma, Prof. D., 1928 and Heim R., 1929). This invention was left without any practical application until the sixties.

It was then re-discovered in the US and was modified and used for control functions in which maximum reliability was required, such as the control of rocket motors and emergency cooling circuits for nuclear power stations. In Germany, so-called low-pressure vortex amplifiers have been developed since about 1970 for use in hydraulic civil engineering practice (Brombach, 1975). The first practical applications began in Germany in 1978 (Quadt and Brombach). Today, in Western Europe and America, there are more than 3000 German type vortex devices of different types and size in operation.

At the same time, independent experiments were conducted in Denmark with vertically arranged vortex throttles for use in catch basins. Later Danish conical in-line types were developed and subsequently improved in Britain. Today there are about 5000 such devices in operation in Europe and America, mostly the catchbasin vertical type.

2. SIMPLIFIED EXPLANATION OF VORTEX FLOW CONTROLLERS

Vortex devices work exclusively with flow effects. There are no moving parts and there is no external energy supply. The flow effects are three-dimensional and determined by fluid acceleration. To date, there is no satisfactory quantitative description of the flow processes by means of a mathematical model.

The operating principles of this technology can be demonstrated by considering the simplified model shown in Fig. 1 (Brombach a, 1984). A large container is connected via a channel (C) to a cylindrical vessel (V). The vessel (V) has a centralized hole at the bottom, of diameter (Do) or of area (Ao). If the pipe (C) is routed so that the water under constant head (H) enters the vessel (V) perfectly radially, e.g. via a ring pipe with small radial nozzles, (see Fig. 1A), the result is a sink flow in the direction of the outlet orifice. The outflow jet is constricted so that the flow rate becomes,

Q = Ao*M*SQUARE ROOT (2g*H).

If the outlet is of sharp-edged design (orifice plate), the loss coefficient, M, is about equal to 0.6.

Fig. 1B shows the same setup, but the inlet nozzles are arranged tangentially. The difference is striking. There is a free vortex generated under these conditions. The nearer a water particle comes to the center line of the vessel (V), the greater its peripheral velocity becomes. The centrifugal force becomes so great that a vortex core is formed through the outlet hole and allows the water to flow out in the form of a hollow jet. This effect can be observed in a bath tub vortex. The flow obeys the same above function, but the flow coefficient drops to a value of about 0.15. This value can be reduced further to a limit of about 0.11 by additional refinements not shown in Fig. 1, which reflect characteristics of the most advanced equipment.

A remarkable feature of this vortex flow is that each disturbance of the flow, e.g. due to entrained solids or air pockets, leads to a weakening of the centrifugal force and thus to an increase of the flow. This unusual behavior, also known as the self cleansing effect, has obvious advantages in flow control of waste streams with solids and debris. This phenomena can be dramatically observed by placing an object or even "stepping" into a throttle in full vortex operation. The restriction will immediately result in a flow switch to higher orifice flow. Vortex flow is instantly resumed once the object is removed.

Although the two experiments in Fig. 1 differ only slightly in the arrangement of the inlet nozzles, the flow is very different as the flow rate is reduced to about 1/4 as a result of the tangential supply of water. Expressed differently, it is possible under vortex flow conditions to obtain the same flow as a simple orifice, but the area (Ao) of the outlet nozzle will be 4 times the size of the comparable simple orifice (or the diameter twice as large as comparable simple orifice). If it were possible to switch automatically between the two flow states, the result would be flow valves whose flow rates are adjustable in the ratio of 1:4.

The earliest German flow throttle depicted in Fig. 2A is simply a cylindrical disc fed tangentially with a central orifice outlet at the bottom of the unit. An air vent on top of the unit permits air to be exhausted or drawn in to stabilize the vortex action. The device instantly goes into the vortex mode and the characteristic curve is aliken to a pump curve. Through experimentation , it was found that by inclining the device around its axis (see Fig. 2B), the resulting controller could combine both types of orifice and vortex flow principles into a single stage - discharge characteristic. Thus, it is possible to have a controller switch to and fro, from orifice to vortex and back, by changing only the pressure head of the inlet of the device. The name "vortex valve" derives from the device's ability to vary flow resistance as a function of inlet pressure (Brombach b,1984).

The resistance coefficients of vortex throttle can be favorably compared to the mechanical B&B float-operated regulator which vary from 0.95 (100% shutter opening) down to 0.725 (5% shutter opening) (Brown & Brown, 1960). It is for this reason that vortex valves can throttle discharge with much larger aperture openings in comparison to a standard orifice.

3. TYPES OF CONICAL IN-LINE VORTEX THROTTLES

Over the last 10 years, there have been three general types of conical in- line (inflow and outflow along same axis) types of vortex devices used. The first derives from Denmark. The second is a British variation of the Danish device. The third is from independent German roots.

In 1977, J. Mosbaek Johannessen together with C. Maegaard of Denmark, developed and an in-line vortex device called the "conical" (due to the cone shaped configuration) to generate in-system storage within the Northwest Interceptor, Cleveland, Ohio. (See Fig. 3-A). The intake of this prototype device is a through a "slot "along the side including (not shown) a portion of the backplate .

This prototype was later altered (see Fig. 3-B) for general commercial applications and has an inlet slot cut along the longitudinal axis of a truncated cone with an internal orifice located at the outlet back plate. Incoming flow impacts the back plate and then enters the device through the inlet slot. The vertical height of the slot is about 40% of the orifice diameter in the back of the unit. A family of stage discharge curves is generated by altering the internal orifice dimension. The large end of the truncated cone is a fixed multiple (twice) of the internal orifice.

Johannessen later changed the conical design, as shown in Fig. 3-C, due to high incidence of reported debris cloggage in the inlet slot, turbulent entrance conditions (lowered effectiveness of unit in vortex mode), deviations of rated flow capacity due to variability of field installation (deviation of inlet slot from vertical alignment would alter the flow characteristic), and, reduction of the structural rigidity of the cone due to the inlet slot along the cone length.

The conical device shown in Fig. 3 -C includes a truncated cone section, a circular plate at the inlet end with a cylindrical inlet at 45 degrees to the back plate, an oval plate at the outlet end and a cylindrical outlet. While a 45 degree inlet is not optimal for creating maximal throttling (this occurs when the inlet feed into vortex chamber is perpendicular to axis of rotation), the configuration does have a distinct advantage for passage of long objects (timbers). On the other hand , a device with maximal braking (perpendicular intake) implies maximal orifice opening to pass normal debris.

Families of stage/discharge curves for this conical configuration can be generated by simultaneously altering the dimension of the truncated cone large end and the internal orifice located at the truncated cone small end. The opening of the inlet section is either equal to or greater than the internal orifice. The outlet discharge pipe is always equal to or larger than the internal orifice. The commercial technology discharge range for the Johannessen conical vortex valve is 0.3 cfs to about 20 cfs. The coefficient of discharge in the vortex braking mode for Johannessen's device range from 0.18 up to 0.37, depending upon the particular geometric configuration. Actual field measurements indicate that the coefficients should be higher (Pisano, 1985).

About eleven years ago, Smisson of the U.K. modified the original Danish design by inclusion of "Concorde" inlet box (see Fig. 3-D) attached onto the inlet slot to improve flow conditions into the device (reduce inlet turbulence and provide a greater inlet opening).

Over the last decade, Brombach, Germany developed the conical type shown in Fig 2 -B. As described earlier, the conical vortex valve consists of an inclined housing (the lower generating line of the conical part of the housing is horizontal) and a tangential inlet pipe joining into the housing. The size of the central exit port is variable by means of exchangeable orifices. The housing has a hinged cover as well as vents (an important stabilizing improvement).

Positive venting is important to eliminate surges during initial start-up (air is actually exhausted during the rising stage) and for supplying the vortex with an air source to ensure smooth stabilized vortex action. (See Fig. 4-A). Venting enhances reproducibility and accuracy, particularly for low to intermediate head stages. Trapped air in the upper portion of the cone can result in hysteresis of the flow characteristic. Small devices can draw air supply from the outlet of the device, but with turbulence. Maximal throttling occurs with a smooth well-defined and centrally aerated vortex. This fact is contrary to popular belief that the vortex valve is really a "turbulence throttle". The opposite is true.

About five years, Brombach found that the hydraulic braking effect can be further enhanced by eliminating the flat cover plate (see Fig. 2-B) and replacing it with a smooth dish head shapes (see Fig. 4A/B for new shape. Besides improving the braking effect, the cover is lighter and structurally stronger (flat back plates on large devices designed to withstand high pressure head could weigh in excess of 500 pounds, and can bend and deform under severe back pressure surges).

Flow characteristics of the German vortex valves are the product of the following five geometrical parameters: a) inlet diameter, b) outlet diameter, c) housing diameter, d) angle of inclination, and e) vessel shape ; and the pressure on the inlet side. If the valve does not discharge freely, but against a back-pressure, there is a further parameter. Systematic variation of the geometrical parameters results in an entire family of vortex valves. Fig 4-B shows some of the effect of the angle of inclination on the flow characteristics while Fig 4-C depicts possible characteristics by varying the outlet aperture relative to fixed inlet diameter and angle of inclination. Figure 4-D depicts a typical (1990's) vortex throttle installation with gated bypass for ease in maintenance.

In practical applications, slide valves and connecting pipes are often positioned upstream of the valve itself, and the dimensioning of the inner orifice accounts for these additional hydraulic resistances. All of the German vortex valves used for combined sewer regulation are constructed with hinged hatches allowing for maintenance and permitting replacement of adjustable plastic and/or steel orifice inserts for altering the units stage/discharge characteristic. A wide range of different flow characteristic curves can be accomplished through the replacement of inserts allowing for capacity expansion/reduction.

4. VORTEX THROTTLE DESIGN CONSIDERATIONS

Fig. 5 shows a typical vortex valve stage/discharge characteristic. The lower portion of the curve increases (rising head) to a defined peak (orifice characteristic), "kick-back" occurs in the transition zone (energy re- arrangement from orifice to vortex mode), and then the curve gradually rises when in the vortex mode. On falling head, the characteristic curve is essentially the same, showing small hysteresis in the vicinity of the "kick-back" zone. The amount of hysteresis is unimportant with high performance valves (M, loss coefficient, less than 0.2) and minimum entrance losses, but may be considerable with long and bended loss inlet lines and less efficient devices (M greater than 0.2).

This characteristic permits a variety of combined sewer regulator design possibilities. First, the operating head range for normal maximum dry weather flows plus allowances for maximum infiltration are chosen to be on the rising portion of the unsubmerged weir portion of the curve to avoid adverse backhead (which could result in upstream deposition). This dry weather range is chosen to be well to the left of the "switchback" or "kickback" point in order to ensure that the unit will not be "hung" when in a receding flow condition. Second, the maximum orifice or peak flow condition is often chosen to coincide with estimated flow at "first flush" conditions. Third, overflow spill weirs should be set when the throttle is in vortex mode. Since the switchback point can be erratic, operating levels should be above this level. Fourth, this region of the vortex curve can also be used to control flow at the maximum storm condition so that discharge at maximum head could be the same or even less than the peak "first-flush" flow occurring at low head conditions. The downstream WWTP would receive at most no more than either of these two hydraulic limits. Last, system- wide utilization of these controllers can apportion interceptor capacity to carry only "first-flush" to the WWTP from each of the contributing sewer sheds, and then allow only the clearer flows during storm conditions to overflow.

5. MAJOR NORTH AMERICAN VORTEX INSTALLATIONS

Fig. 6 depicts a regulator chamber containing a vortex valve whose flow characteristic can be halved by opening the cover and replacing a new insert. This device is one of 13 new regulators (design capacity: 2 to 15 cfs, and head range: 9 to 23 feet) controlling combined sewer flow to a new 3+ mile tunnel (Marigot Project) with pumpage to a new WWTP located on the southerly side of the island of Laval, adjacent to Montreal, Canada. Regulator inserts allow for changing flow characteristics (staged infiltration reduction), and changes in pump station and WWTP hydraulic conditions. This project is the largest in North America in terms of number of units and technology scale .

The decision to use vortex throttles rather than motorized sluice gates was based on the desire not to provide electrical power to the chambers, and the need for no-moving part self-actuated controllers. The design was "fast - tracked" without long term flow measurements, and infiltration levels are known to be high (there is a continuing program to reduce these levels). Since the head differential between low head (dry weather flow) and maximum flow at peak head conditions is substantial (for some installations in excess of 20 feet), the "kick-back " portion of the vortex throttle characteristic curve (see Figure 5) was strongly "warped". This meant that if a major storm occurred during high infiltration conditions, and if infiltration levels were much greater than estimated, then on a receding flow hydrograph the vortex device would not "move" out of the "kick back" condition down to normal low orifice flow (operating at much lower head). Mechanical chainfalls were attached to the roof of the chambers for ease in opening the heavy plated covers for orifice insert change, but also for removing the vortex throttles if the infiltration "kickback" problem ever arose. This condition has not been observed. The vortex throttle configuration shown in Fig. 6 could be improved if the inlet pipe to the vortex chamber were both extended, and placed instead on the right hand side of the vortex drum, allowing discharge to the tunnel with fewer bends, fittings, and associated head loss.

The largest singular installation (capacity) in North America is located in Quebeç City ,Canada. The combined sewer regulation chamber consists of 2- 8 foot diameter 3/8" stainless steel vortex drums fed by 48" lines with piped outlets. See Fig. 7-A. The design condition on the configuration is 169 cfs under a head of about 12 feet. The characteristic function for the installation is depicted in Fig. 7-B. At four feet of head, a discharge of about 145 cfs was desired with a gradual increase up to the design limit. The small ratio of vortex drum size to inlet diameter permitted the gradual and nearly vertical flow change from orifice to vortex flow conditions. The units were put into operation in 1986 and have operated satisfactorily.

The largest US system of vortex flow controllers for combined sewer regulation is in Saginaw, Michigan. In 1984, 9 of the City's 34 regulation chambers (West side with service area of about 5000 acres) were modified to include vortex throttles (type shown in Fig. 3-C with capacities varying from 1 cfs-20 cfs) (Pisano, 1988). This program was funded through the 108 Great Lakes Demonstration Grant Program. These devices were to replace mechanical float-operated throttles, and were segmented (bolt-together sections) to permit ease in installation. Regulator chamber cross and spill weirs were modified to maximize potential in-line transient storage. A post evaluation program showed few operational problems. Based on the success of the 1984 program, in 1986 the City using its own funds modified the balance of the City's (East Side) regulation chambers. All work was competed by the end of 1987. A total of 12 additional chambers were outfitted with German type vortex throttles (again segmented) . Roughly 6+ MG of in-line transient storage was created. The overall total of 21 units is the largest system wide configuration in the US. The City has reported in the spring of 1988 that the wet sludge processed during rainfall events has increased from 75+ tons/day to approximately 90 tons/day (Pisano, 1988). Expenditures were about $3 M dollars.

Operating experiences in Saginaw have been favorable. Cloggage in small units (less than 0.5 cfs) does occur. Experience indicates that critical aspect of design is the entrance condition to the intake of the vortex throttle. It should be contoured and tapered to accelerate flow into the inlet to limit deposition and cloggage during low flow conditions. Structural problems with welded seams on one large Danish-type segmented device (20 cfs) were eliminated by later retro-fitting centrally-located air vents on the vortex drum's back plate. The City reports far less maintenance requirements than noted earlier for mechanical float - operated systems.

The largest German -type vortex valves in the US were installed in Columbus, Ohio in 1986 (a battery of 3 devices: 35 cfs, 20 cfs and 15 cfs at design head of 12.5 feet). The flow characteristics (on falling stage) of these three devices were field - verified. Similar sized German -type devices were installed in St. Paul, MN. in 1987. The largest Danish /British variation vortex throttle (55 cfs) was also installed in 1987 in St. Paul, MN.

6. VORTEX FLOW REGULATORS WITH ELECTRONIC FEEDBACK

In the last six years, there has arisen a need for feedback throttled control at German storage installations, particularly for small, downstream flow - sensitive situations. The central idea is to sense indirect flow rates through pressure sensors attached to the housing of the vortex controller, and then send this data to a microprocessor computer with various programmable operations to electronically drive a motorized knife valve situated on the intake to the vortex throttle. Pressure inlet conditions on the vortex throttle can be adjusted and hence the throttle's discharge. Discharge is continuously adjustable through positioning the slide plate in front of the throttle. The only moving parts are the valve actuator and the slide plate. See Fig. 8 for typical layout. The advantage of the vortex throttle is the relatively large aperture opening for flow and debris passage in comparison to a pinch valve or a sluice gate. This advantage is significant when the design requirement is to throttle very small flow rates containing debris and solids. Two hundred such configurations have been installed, and the operational experience has been good provided that competent technicians are servicing the facility.

Typically, the vortex throttle in such installations is horizontally placed such that the flow characteristic is monotonic in shape. Discharge can be indirectly determined as the pressure readout (tap and sensor located on the housing of the device) can be determined. This dish-head shaped vortex valve can throttle very small flows (0.5 cfs) at nominal heads (4-6 feet) with aperture openings on the order of 5-6". The technology range for this type of vortex throttle installation has ranged from 0.5 cfs up to 8 cfs. An 8 cfs vortex throttle has an inlet diameter of about 16" and the diameter could be as large as 80". For higher flows, the setup becomes too large and other alternatives are more attractive.

Programmable microprocessors have been devised to accomplish an extremely wide and versatile set of functions, ranging from environmental sensing of the components themselves (with shut-off), to repetitive commands to pass and unlodge debris, to normal feedback flow attenuation and shut "off-on" operations. Functions are standardized and "clip -in" to standard frames. Slide valves with a consistently high degree of quality control have been a problem. Control panels are usually outside the control dry pit and are typically provided with all the necessary receptacles for lights, alarms, heating etc.

7. "TURBO" VORTEX THROTTLE TYPE

Three years ago, Brombach developed a new throttle with adjustable feedback control not requiring an electrical source or components (see Fig. 9). A simplified schematic of the new "TURBO" regulating throttle is depicted in Fig. 10. Flow enters the vortex valve tangentially. In the upper section of the vortex chamber there is a light, flat turbine wheel made of synthetic material. A screw shaft leads to a oil pump through the hinged dish head cover. The pump is supplied with hydraulic oil from a reservoir above the sluice gate. Since the vortex chamber is partially filled during dry weather flow, the turbine wheel is dry and motionless. As the inflow increases during a storm, the swirling flow in the chamber engages the turbine wheel causing the oil pump to push hydraulic oil into the oil actuator which is housed in the head flange of the hydraulic cylinder. The actuator activates a low pressure hydraulic constant rate cylinder driving a moveable knife gate. This gate throttles the inflow to the vortex chamber just enough that the turbine speed matches the preset design flow. Should the flow fall below the design flow then the turbine wheel becomes still and a rebound spring opens the sluice gate.

The system is a "true" flow regulator with closed feedback loop since the vortex valve and turbine wheel serve simultaneously as a flow meter and source for the throttling energy. The dynamic behavior of the TURBO's regulating cycle was optimized using a combination of laboratory and mathematical simulation techniques. The TURBO stabilizes remarkably in less than a minute even under extreme conditions, i.e., surges from zero to design flow conditions. This rapid dynamic stability is essential for regenerating the regulating cycle during clogging. The system has unusual passive reserves. Should the oil hydraulic controller fail, i.e., oil in cylinder was not maintained, then the vortex valve still serves as an "emergency brake". If the sluice gate hits a rock while closing, then a relief valve within the actuator allows the gate to rebound. All moveable parts remain in the oil stream. Jamming, freezing or corrosion do not occur.

During dry weather flow the TURBO flow characteristic is slightly S-shaped (see Fig.11) and is determined by the hydraulic operation of the vortex throttle. As the flow rises, the actuator and the sluice gate operation maintain a constant discharge independent of head. A nominal design flow characteristic can be can be adjusted in two ways. First, the exterior adjustment knob on the actuator can be manually changed to adjust the internal bypass valve within the actuator. This change causes the turbine wheel to turn faster (or slower) to change the sluice gate position. Small flow adjustments can be accomplished in this manner (see Fig. 11). Larger flow adjustments can also be easily accomplished. In the center outlet of the vortex throttle there is a loose plastic orifice plate which determines design discharge. A given design discharge can be changed by simply exchanging the orifice plate. The TURBO design discharge can be thus modified in the ratio of 1:4 using a combination of both adjustments. The practical technology range of the TURBO is about 4 cfs (12" intake to vortex vessel). Today, there are over 50 installations in Germany for controlling outflow from storm overflow tanks.

Please refer all correspondence to William C. Pisano, Havens & Emerson, 8 Fanueil Hall Marketplace, Boston, Ma. 02109. Phone : 617-248-0765.

REFERENCES

1. Thoma, D. (1928), Vorrichtung zur Behinderung des Ruckstroemens, Deutsche Patentschrift 507713.
2. Heim, R. (1929) Versuche zur Ausbildung der Thomaschen Rueckstrombremse. Mitteilungen des Hydraulischen Institits der TH Muenchen, Heft 3, Verlag R. Oldenbourg, Muenchen/Berlin.
3. Brombach, H. (1975) Vortex Amplifiers for Low Control Pressure Ratios. Proc of the 7th Cranfield Fluidics Conference, BHRA, Cranfield, England.
4. Quadt, K. and Brombach H., (1978) Practical Experiences from Vortex Throttles Controlling Storm Overflow Tanks. Korrespondenz Abwasser, Vol. 1.
5. Brombach H. et al. (1984 a), Latest Developments in Sewer Flow Control Using The Vortex Principle, WPCF, New Orleans.
6. Brombach, H. (1984b) Vortex Flow Controllers in Sanitary Engineering, Trans. ASME Jour Dynamic Systems Measurements and Control, Vol. 106.
7. Brown & Brown Inc. (1960),Automatic Sewage Regulators, Bul. 83A.
8. Pisano, W.(1985) In-system Storage Controls for Reduction of Combined Sewage Overflow in Saginaw, Mi. EPA-905/2-85-001-A.
9. Pisano, W. (1988b) In-system Storage and Flow Regulation Enhances Wet Weather Waste Treatment For the City of Saginaw, Mi., International Joint Commission, Windsor, Canada.

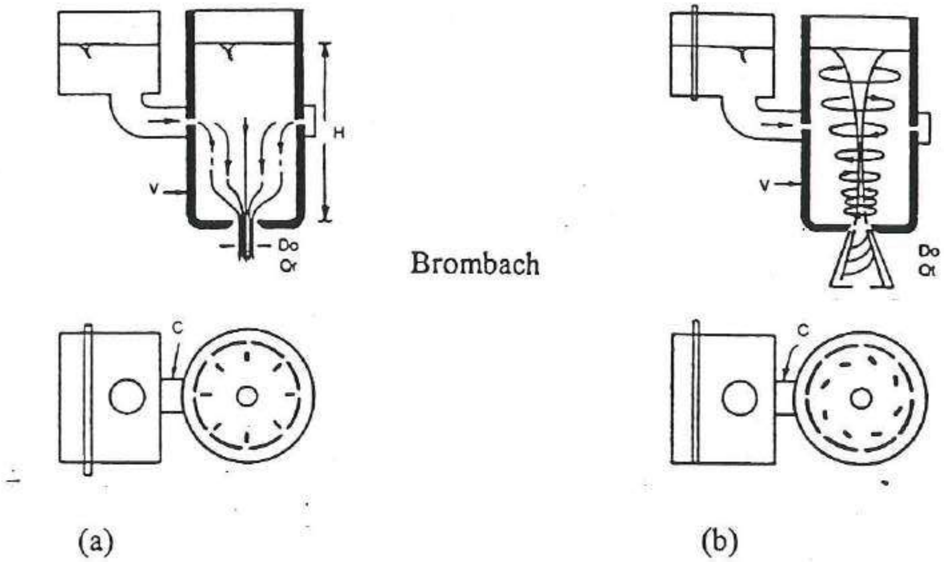


Figure 1 Simplified mathematical rationale for vortex valve:
(a) radial sink flow and (b) free vortex flow.

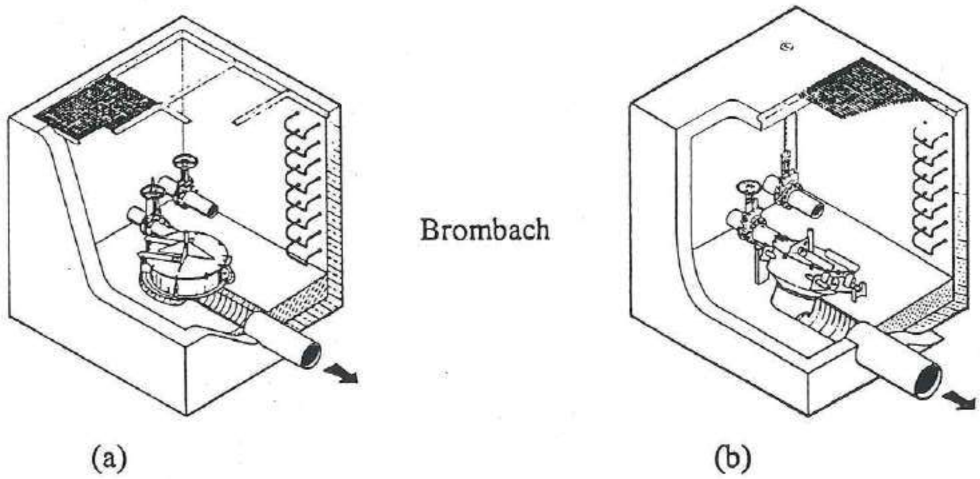


Figure 2 Typical German vortex flow throttles:
(a) horizontal and (b) inclined.

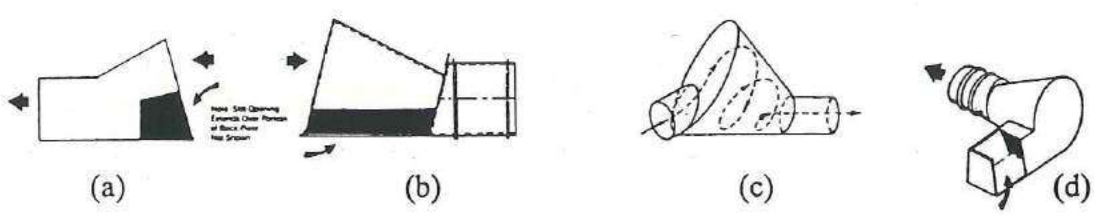


Figure 3 Danish-type conical vortex throttles: (a) prototype at Cleveland, Ohio; (b) slotted opening; (c) intake at 45° to solid backplate; and (d) modified box intake on slotted unit.

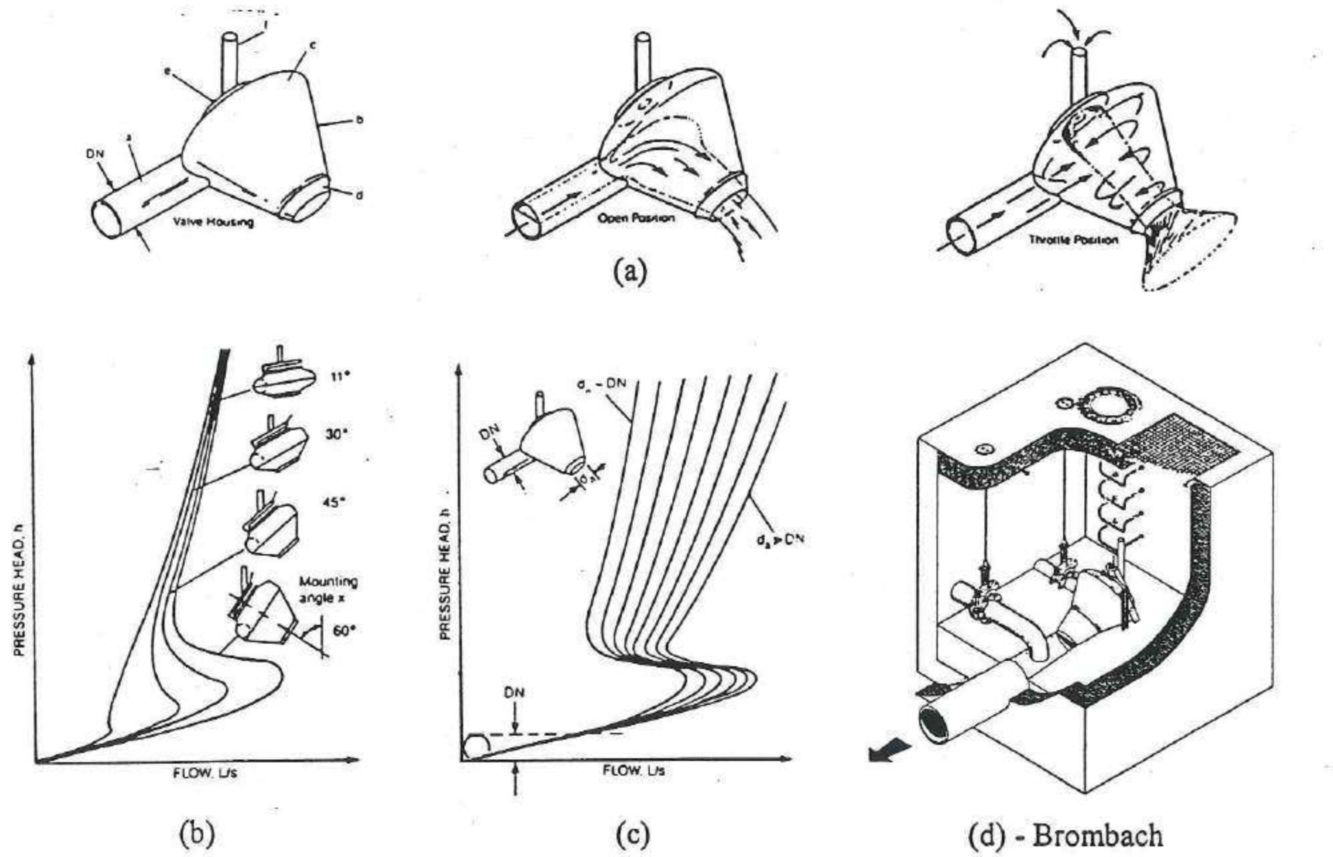


Figure 4 Characteristics for advanced German-type flow throttle; (a) structural and operation; (b) effect of mounting angle on flow curve; (c) effects of outlet aperture change relative to inlet diameter fixed; and (d) typical 1990's installation.
Note: DN - nominal width of valve; a - horizontal feed pipe; b - swirl chamber; c - domed valve cover; d - exchangeable aperture; e - inspection port; f - swirl core ventilation.

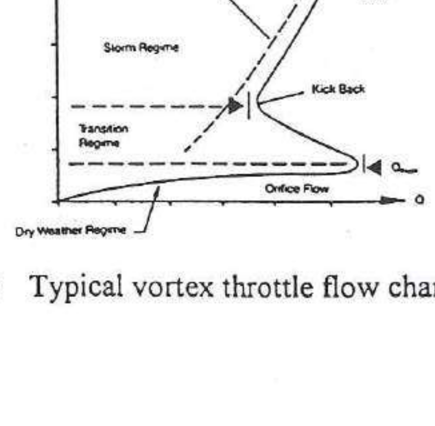
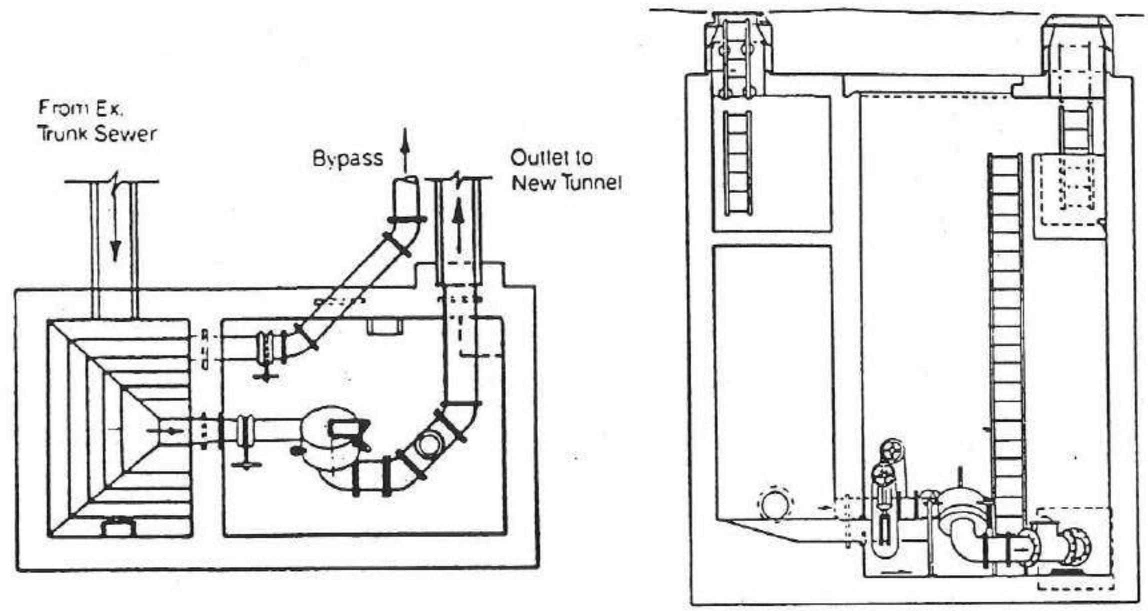
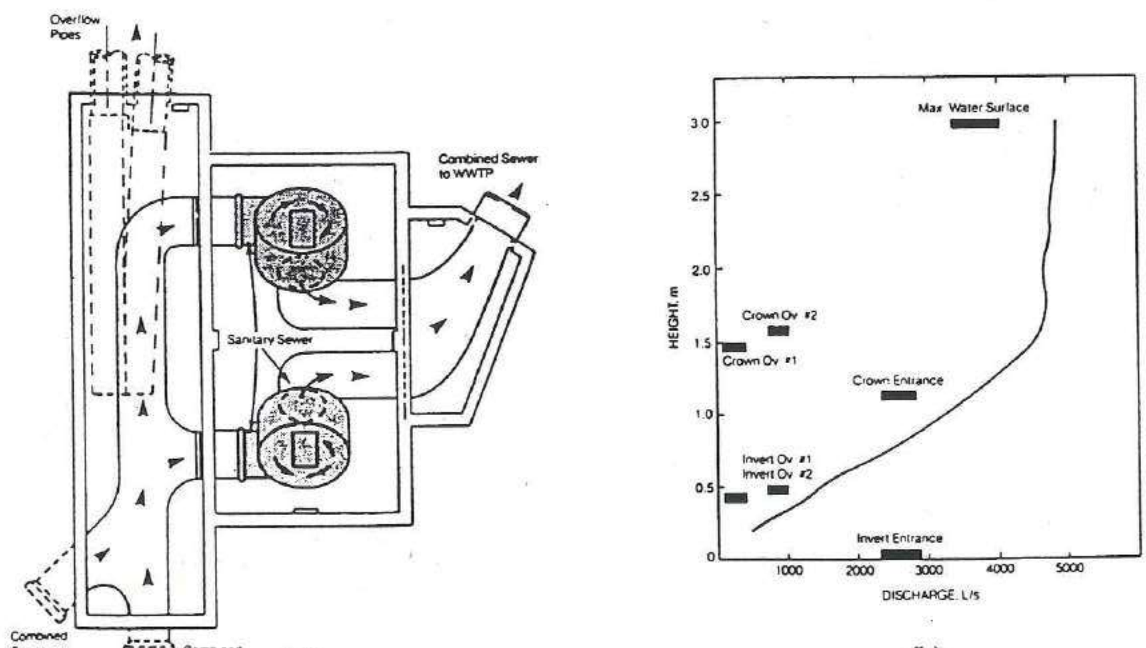


Figure 5 Typical vortex throttle flow characteristic.



(a) (b)

Figure 6 Typical vortex flow throttle regulation chambers for Marigot Project, Laval, Canada; (a) plan view (b) side view.



(a) (b)

Figure 7 Combined sewer regulation chamber for Quebec City, Canada; (a) top view and (b) discharge characteristics.

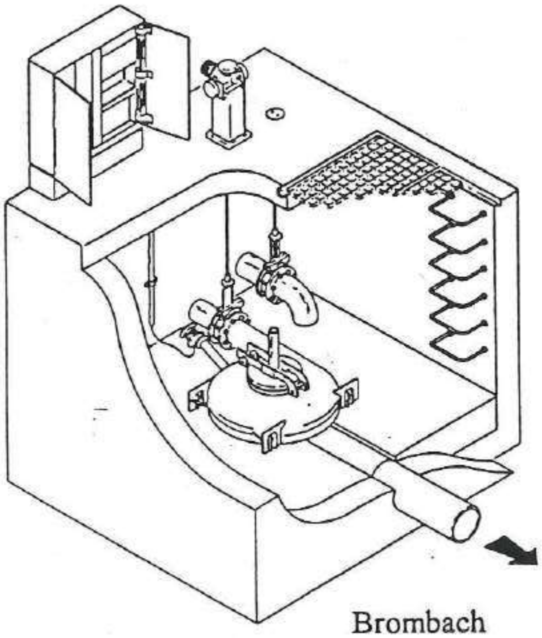


Figure 8 Typical vortex controller with electronic controls

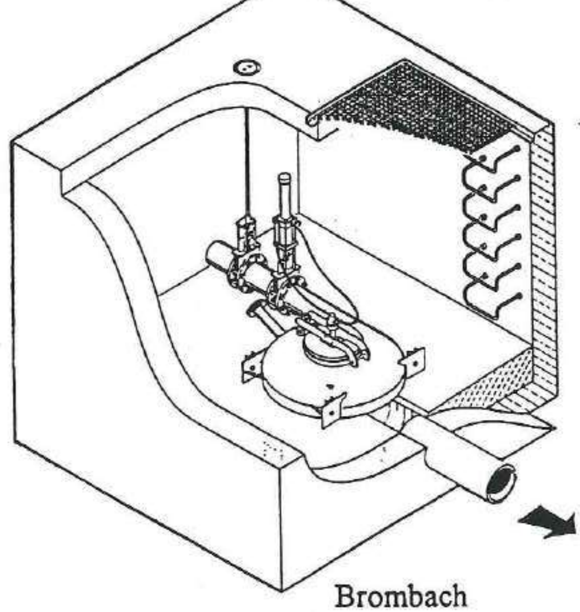


Figure 9 "Turbo" vortex controller

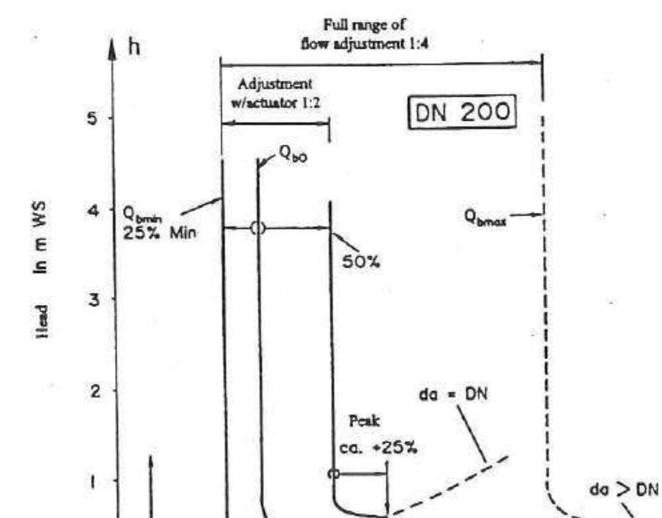


Figure 11 Discharge curves for "Turbo" - vortex.

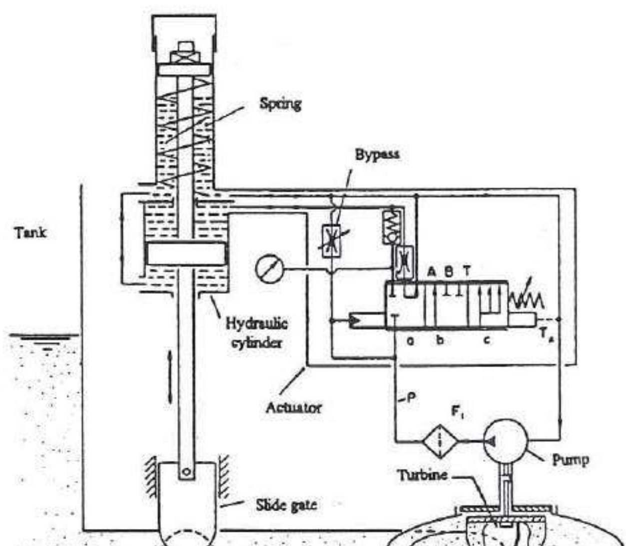


Figure 10 "Turbo" regulating cycle (oil hydraulic circuit diagram)

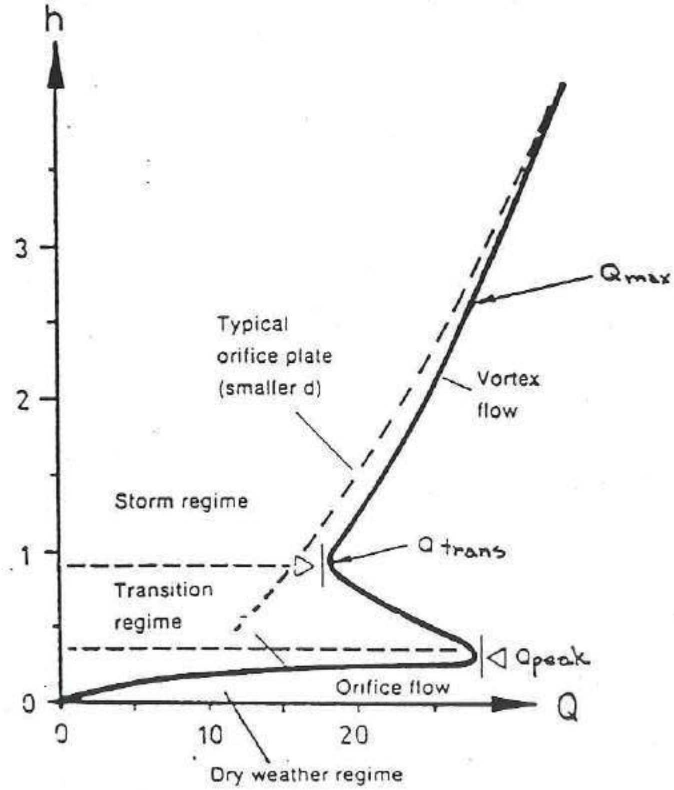
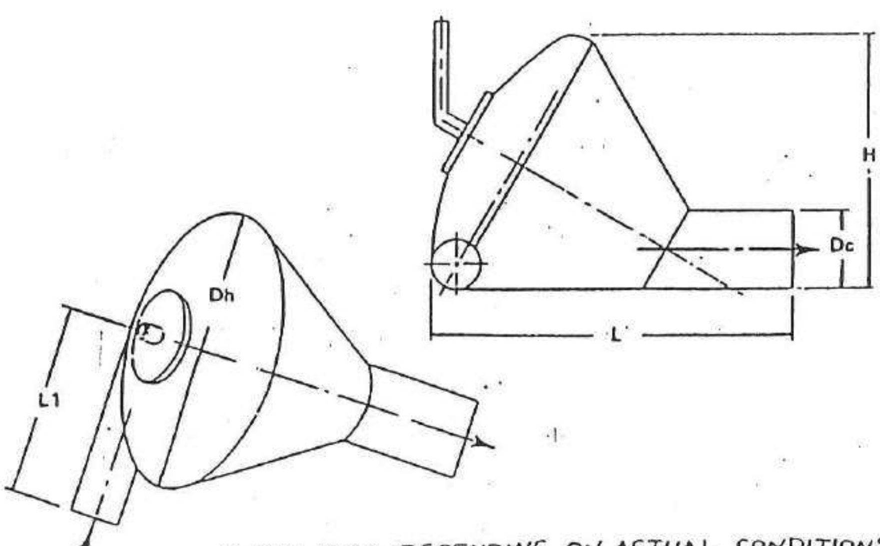


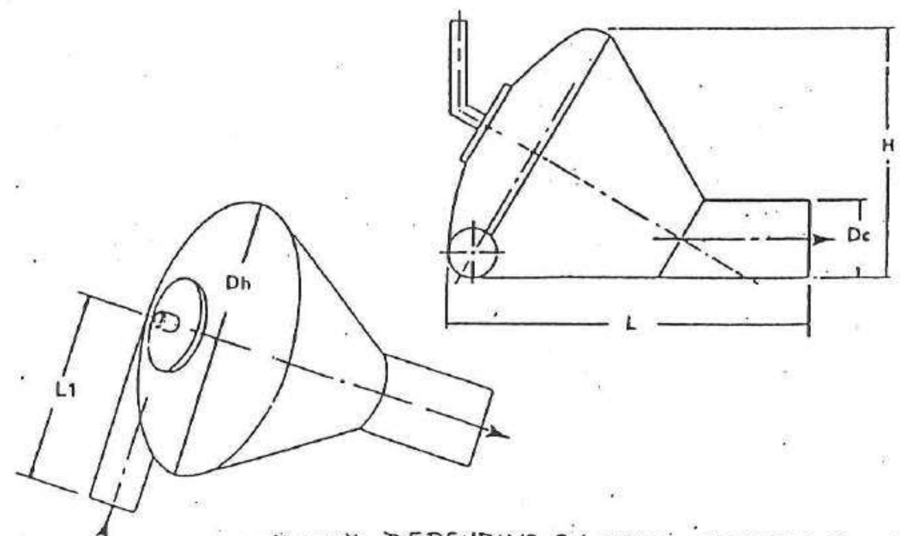
Figure 1 Typical HYDROVEX Vortex Valve Discharge Rating Curve

	MODEL SELECTION & DIMENSIONING									
	IHV 60/2.5					IHV 60/3				
Dn	Dh	L*	H	Dc*	L1	Dh	L*	H	Dc*	L1
100 mm	250	410	215	150	225	300	460	260	150	250
4 in.	10	16,5	8,7	6	9	12	18,3	10,4	6	10
125 mm	315	455	270	200	280	375	520	325	200	315
5 in.	12,5	18,3	10,8	8	11,3	15	20,8	13	8	12,5
150 mm	375	500	325	250	340	450	575	390	250	375
6 in.	15	20	13	10	13,5	18	23	15,6	10	15
200 mm	500	620	435	300	450	600	720	520	300	500
8 in.	20	24,8	17,3	12	18	24	28,8	20,8	12	20
250 mm	625	725	540	375	565	750	850	650	375	625
10 in.	25	29	21,7	15	22,5	30	34	26	15	25
200 mm	750	830	650	450	675	900	980	780	450	750
12 in.	30	33,3	26	18	27	36	39,3	31,2	18	30
400 mm	1000	1040	865	600	900	1200	1240	1040	600	1000
16 in.	40	41,5	34,6	24	36	48	49,5	41,5	24	40
500 mm	1250	1250	1080	750	1125	1500	1500	1300	750	1250
20 in.	50	50	43,3	30	45	60	60	52	30	50



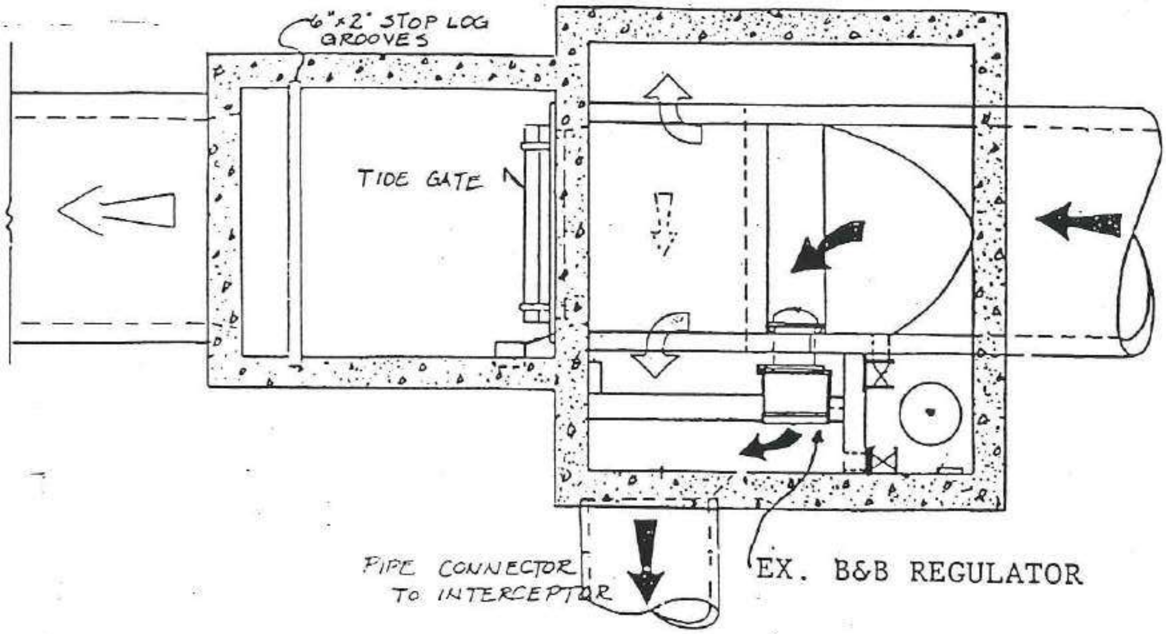
* MAY VARY, DEPENDING ON ACTUAL CONDITIONS

	MODEL SELECTION & DIMENSIONING					
	IHV 60/4					
Dn	Dh	L*	H	Dc*	L1	
100 mm	400	560	345	150	300	
4 in.	16	22,3	13,9	6	12	
125 mm	500	645	435	200	375	
5 in.	20	25,8	17,3	8	15	
150 mm	600	790	520	250	450	
6 in.	24	31,5	20,8	10	18	
200 mm	800	920	695	300	600	
8 in.	32	36,8	27,7	12	24	
250 mm	1000	1100	865	375	750	
10 in.	40	44	34,6	15	30	
300 mm	1200	1280	1040	450	900	
12 in.	48	51,3	41,6	18	36	
400 mm	1600	1640	1385	600	1200	
16 in.	64	65,5	55,4	24	48	
500 mm	2000	2000	1735	750	1500	
20 in.	80	80	69,3	30	60	

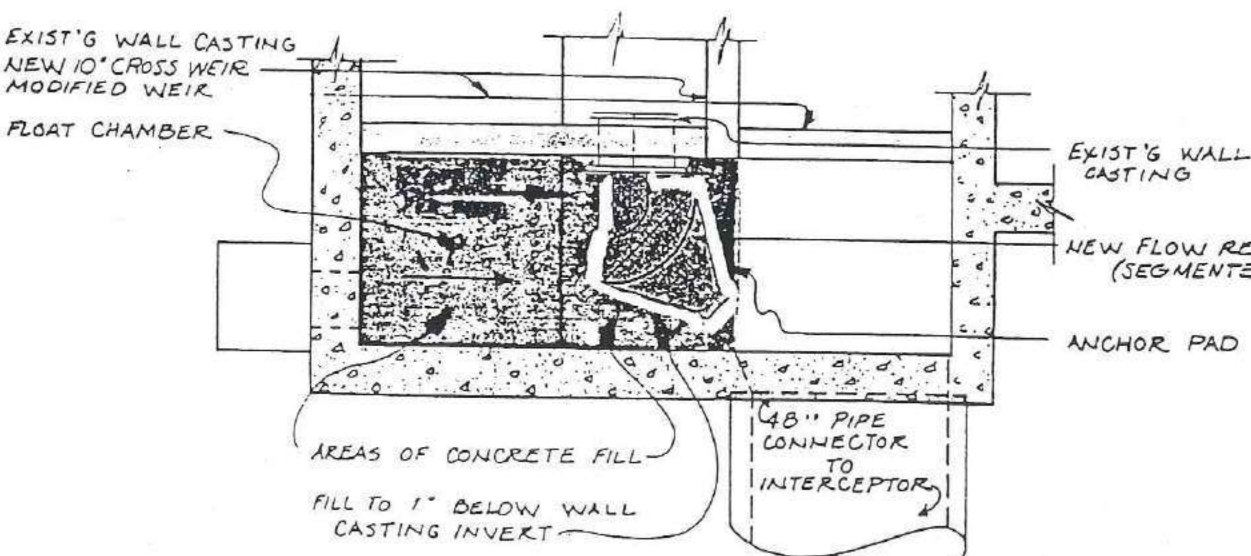


* MAY VARY, DEPENDING ON ACTUAL CONDITIONS.

Figure 2. IHV 60 Dimensions



EXISTING TYPE "B" CHAMBER



MACKINAW STREET

REGULATOR NO. 16 SCALE: 3/8" = 1'-0"

Figure 3 TYPICAL PHASE I BMP REGULATION CHAMBER MODIFICATIONS

GNA

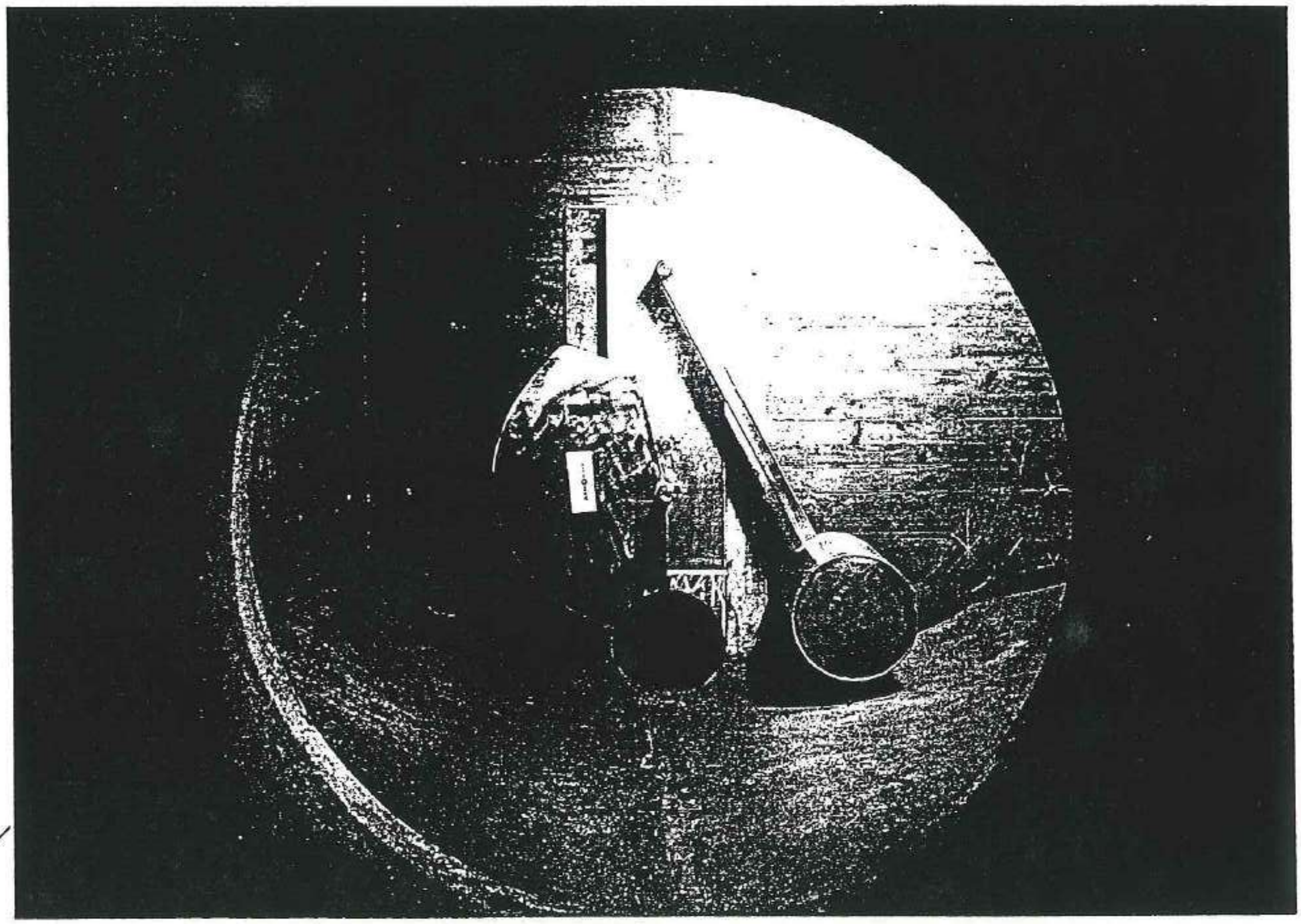
Grande, Novac & Associates Inc.

Engineering solutions for all your CSO needs

HYDROSLIDE®

CONSTANT FLOW REGULATING DEVICE

The GNA HYDROSLIDE is a proven technique for regulating sewer flows. Flows from as little as 5 l/s (80 gpm) may be regulated using valves with a nominal flow diameter of 200 mm (8 in). The float activated mechanism of the HYDROSLIDE regulator is designed to maintain a constant discharge without the use of external energy sources. The flow area is adjusted to perfectly match any increase or decrease in the upstream water level.



OPERATING PRINCIPLES

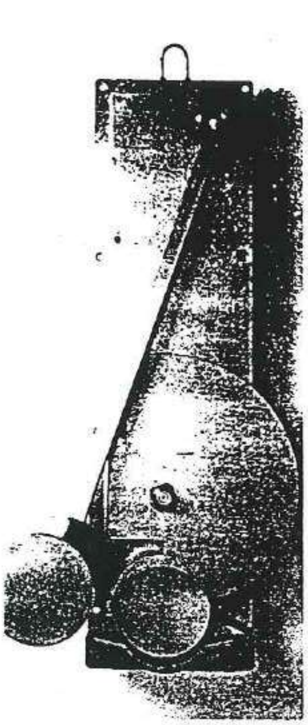
The remains in the neutral or fully open position during dry weather flow. As the inflow at the control point increases and exceeds the capacity of the fully opened unit, the increasing water level causes the float to rise, which in turn causes the flow area to be adjusted so that a constant discharge is maintained.

The flow rate is maintained constant for head variations of up to twelve (12) times the nominal valve inlet diameter. For a unit with a nominal diameter of 200 mm (8 in), the upstream water level may vary from 0 to 2.4 m (0 to 7.9 ft) while maintaining a constant discharge.

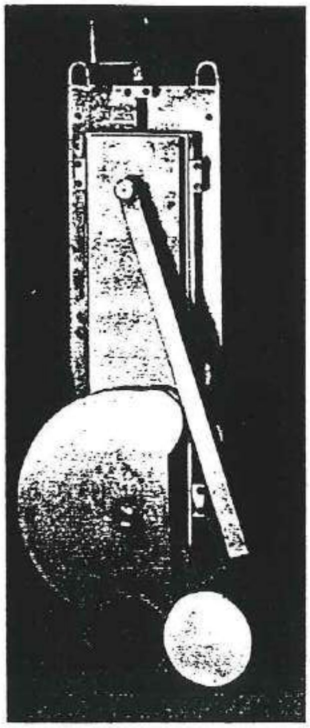
The flow passage area is largest at the most critical time: during dry weather flow, at the beginning of a storm event when the sewer lines are being flushed and at the end of a storm event when the sewer lines are emptying, thereby reducing the chances and frequency of blockage.

The flow regulator is easy to install in standard chambers through standard circular openings. The installation procedure is simple and is supplied with every unit.

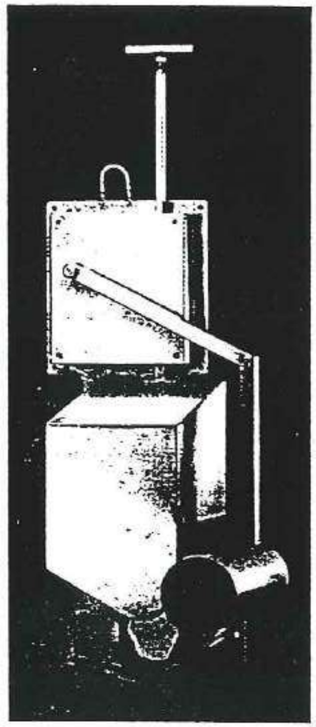
HYDROSLIDE STANDARD



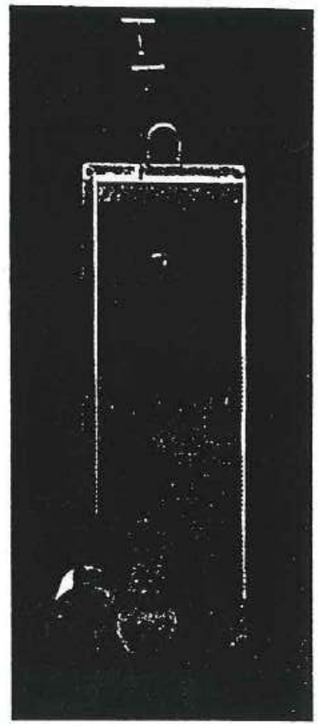
HYDROSLIDE VARIO

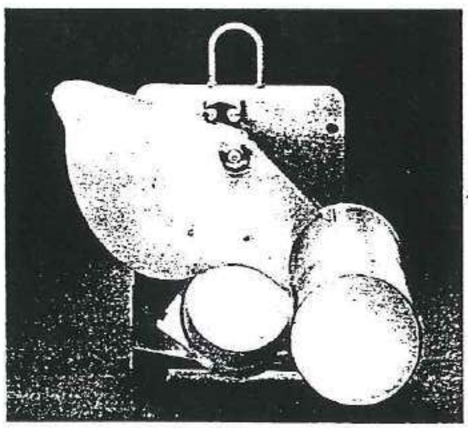


HYDROSLIDE SELFCLEAN



HYDROSLIDE VERTICAL





HYDROSLIDE MINI

- STANDARD unit for discharges between 35 and 2000 l/s (1.25 to 70 cfs).
- VARIO unit which allows the user to change the design flow by ± 30 %. The unit is designed to handle flows ranging from 35 to 2000 l/s (1.25 to 70 cfs).
- SELFCLEAN unit which is designed to open automatically and completely should a blockage occur. This unit is designed for flows ranging from 5 to 60 l/s (0.18 to 2 cfs).
- VERTICAL unit for discharges ranging from 10 to 60 l/s (0.35 to 2 cfs). This unit is designed in such a way as to enable the valve to be opened manually, from the top of the manhole, should blockage occur.
- MINI unit designed for small head variations. Design flows may vary from 15 to 60 l/s (0.53 to 2 cfs).

ADVANTAGES OF HYDROSLIDE

- constant discharge over a variable head range
- largest possible opening during critical flow periods
- flow area greater than vortex flow regulators during dry weather periods
- robust, stainless steel 304 construction
- no head loss
- no constriction or diversion of water passage
- easy and inexpensive to install in new or existing chambers
- no in-situ adjustment or testing
- applicable for all types of fluid control

For more information contact either our local representative or our Head Office:

GNA Grande, Novac & Associates Inc.

7528, Côte-de-Liesse, Montréal, Québec H4T 1E7

Tel.: (514) 737-9584 • Fax: (514) 737-6916

HYDROSLIDE

GNA Grande, Novac and Associates Inc

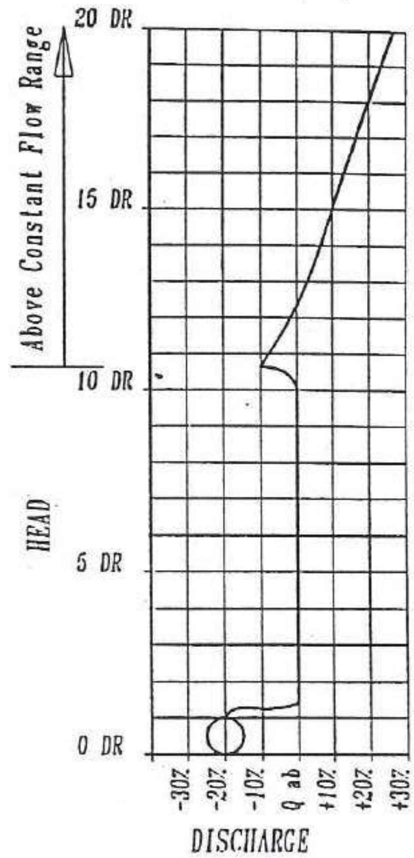
HYDROSLIDE REGULATOR for flows greater than (≥) 38.0 l/s

These regulators are designed and fabricated to adapt to standard inlet pipe diameters.

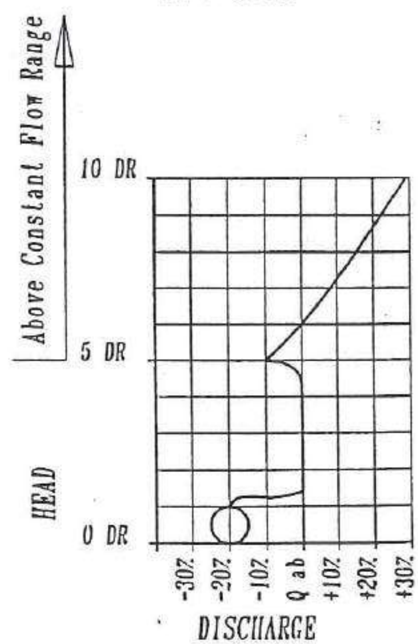
The flow curves for both these types of regulators are shown to the right of this table.

Q ab (l/s)	REGULATOR MODEL TYPE-S	REGULATOR MODEL TYPE-N
38	DR 250/210-S	DR 250/210-N
43	DR 250/220-S	DR 250/220-N
48	DR 250/230-S	DR 250/230-N
53	DR 250/240-S	DR 250/240-N
59	DR 250/…S	DR 250/…N
65	DR 300/260-S	DR 300/260-N
71	DR 300/270-S	DR 300/270-N
78	DR 300/280-S	DR 300/280-N
85	DR 300/290-S	DR 300/290-N
93	DR 300/…S	DR 300/…N
100	DR 350/310-S	DR 350/310-N
109	DR 350/320-S	DR 350/320-N
118	DR 350/330-S	DR 350/330-N
127	DR 350/340-S	DR 350/340-N
136	DR 350/…S	DR 350/…N
146	DR 400/360-S	DR 400/360-N
157	DR 400/370-S	DR 400/370-N
167	DR 400/380-S	DR 400/380-N
178	DR 400/390-S	DR 400/390-N
190	DR 400/…S	DR 400/…N
to	to	to
1900	DR 1000/…S	DR 1000/…N

Please note that the model number defines the diameter of the bolt circle for the standard flange size and that the number after the / represents the diameter of the opening.



DR /...S



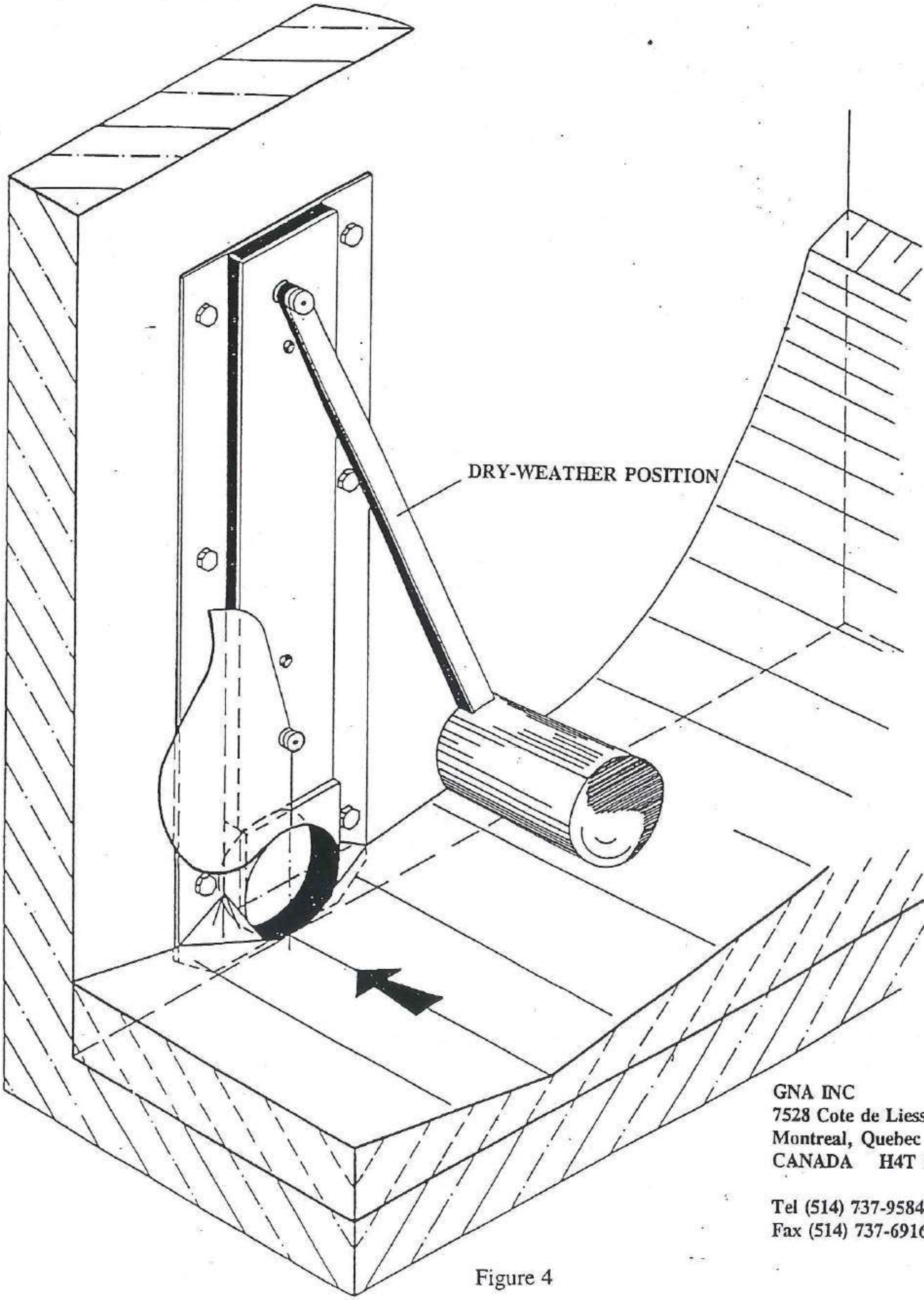
DR /...N

Figure 5

GNA Inc. 7528 Cote de Liesse, Montreal, Quebec, CANADA H4T 1E7 Tel:(514) 737-9584 Fax:(514) 737-6916

HYDROSLIDE

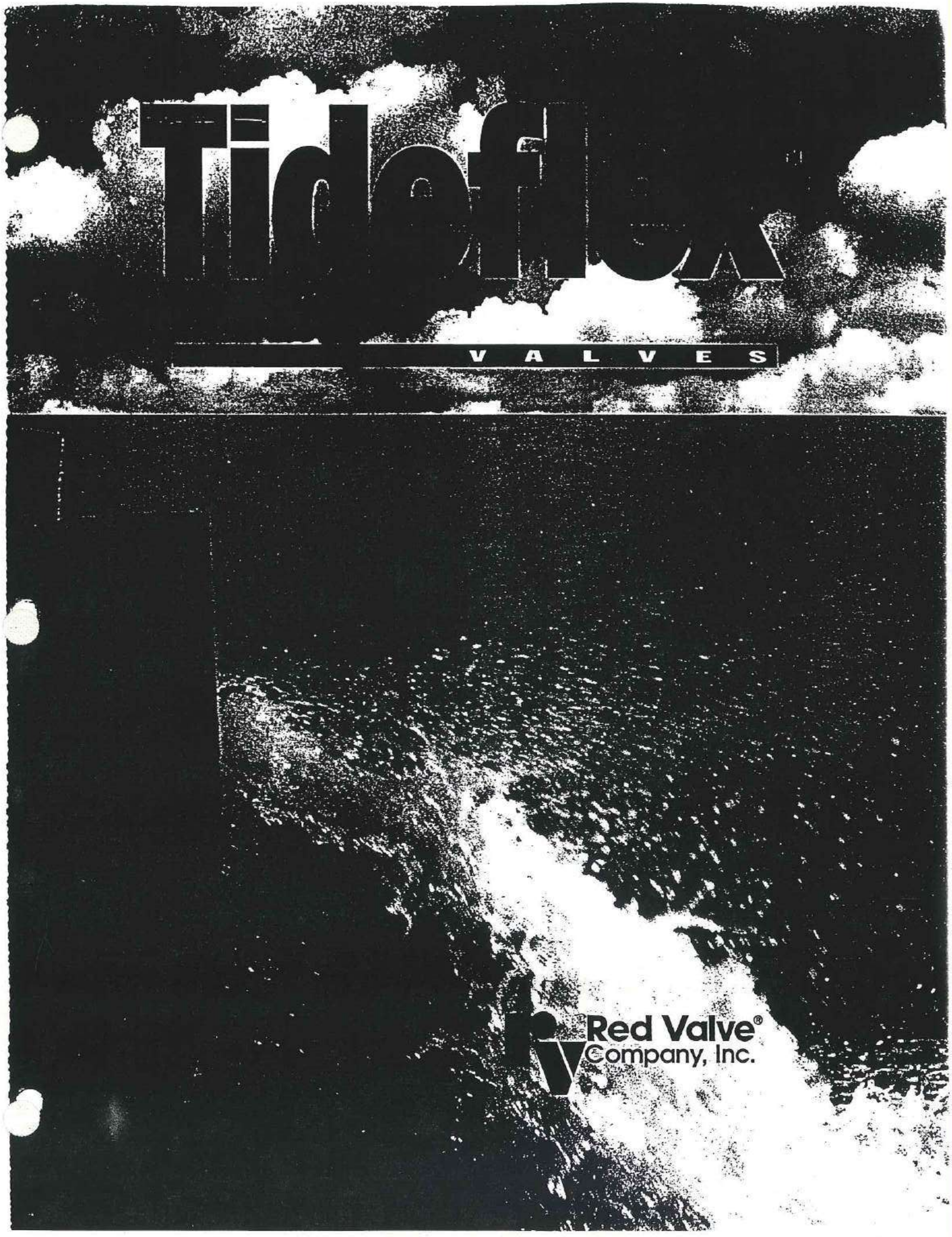
GNA Grande, Novac and Associates Inc



GNA INC
7528 Cote de Liesse
Montreal, Quebec
CANADA H4T 1E7

Tel (514) 737-9584
Fax (514) 737-6916

Figure 4


Tideflex
VALVES
Red Valve®
Company, Inc.

[redacted]

Specify Red Valve's Tideflex™ Check Valve for Absolute Backflow Prevention

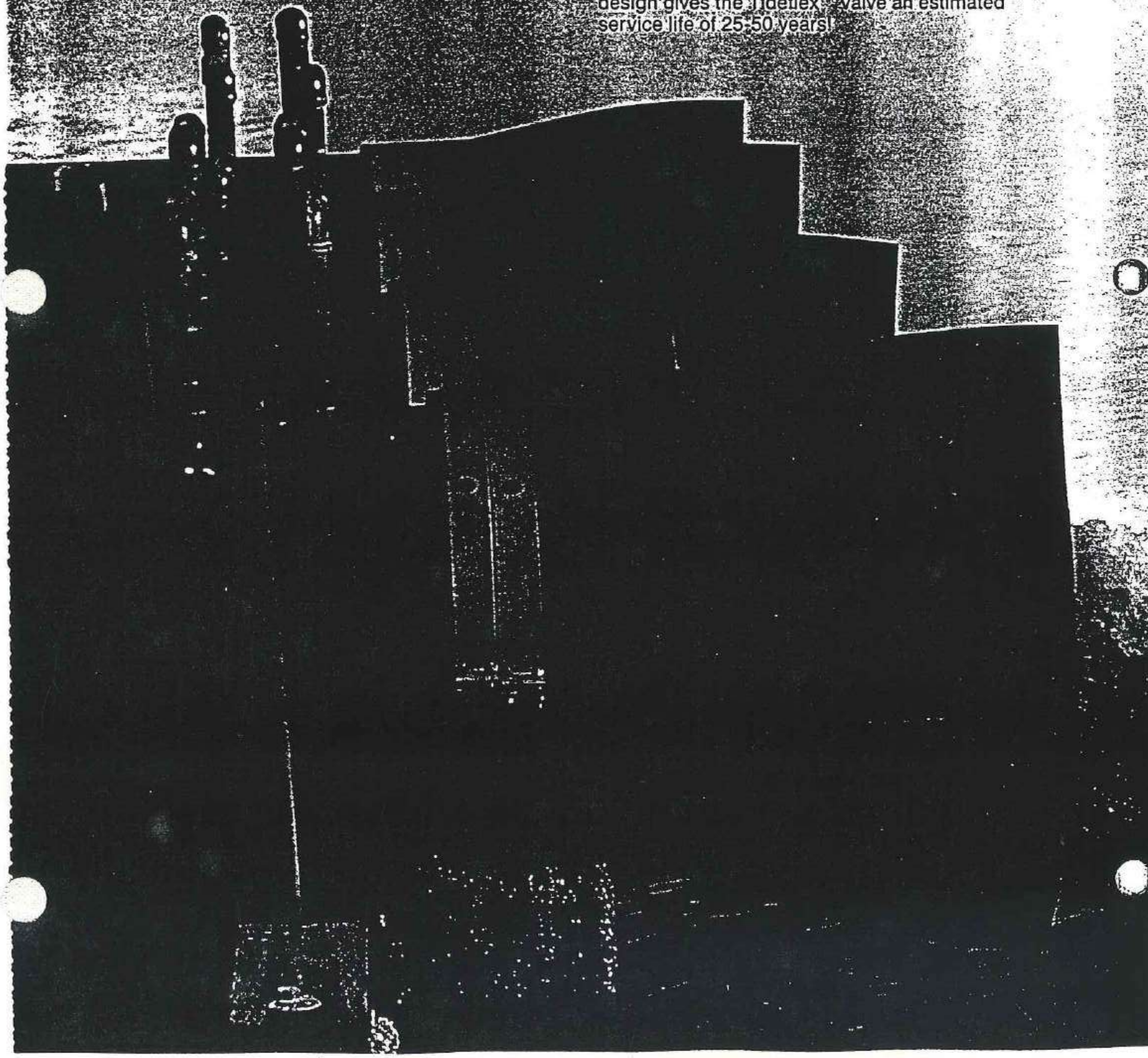
No other check valve is so effective! The Tideflex's™ uniquely simple, custom-engineered all-rubber design eliminates costly backflow from oceans, C.S.O., rivers or storm water and is the ideal valve for effluent diffuser systems.

When you specify Red Valve's patented Tideflex™ Check Valve, you get a proven record of maintenance-free backflow prevention. The Tideflex™ Valve's unique custom-engineered elastomer/fabric design produces results no other check valve can match.

Maintenance Free! Totally Passive Operation

Faulty mechanical moving parts with their inherent problems of corrosion, friction and wear are replaced by the Tideflex™ Valve's revolutionary engineered elastomer construction.

The Tideflex™ Valve's 100% elastomer construction eliminates corrosion problems from salt water and industrial waste water. Its exceptionally durable design gives the Tideflex™ Valve an estimated service life of 25-50 years!



The flexible Tideflex™ Valve elastomer will seal tightly around debris upon reverse flow. A major disadvantage of traditional check valves is that debris will lodge in the seating and prevent the valve from closing. The Tideflex™ Valve's inherent flexibility virtually eliminates the seating problems associated with traditional flap gates and check valves.

Lowest Headloss of any Check Valve

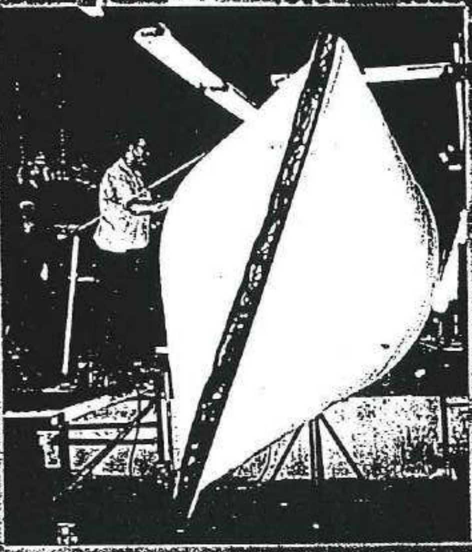
The Tideflex™ Valve's extremely low headloss is especially beneficial in low lying areas, permitting drainage with very low head pressure.

During gravity flow conditions up to a velocity of 2.3 feet per second, there is absolutely **no headloss in a Tideflex™ Valve.** What's more, the Tideflex™ Check Valve **does not reduce the outflow** from the outfall line **until the discharge end of the pipe is completely full.**

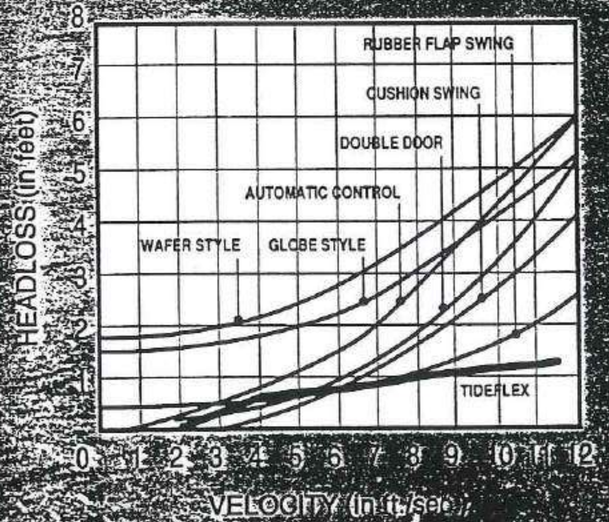
The Tideflex™ Valve is sensitive enough to open with **as little as 1" of water!** Additionally, the Tideflex™ is able to withstand up to 50 feet of back pressure and the turbulent waters of high tides, seasonal storms and flash floods.

Custom-built to Your Specifications

Every Tideflex™ Check Valve is custom-built to your flow specifications, in sizes ranging from ½" to 90" in diameter. Red Valve can build a Tideflex™ Valve to your exact requirements.

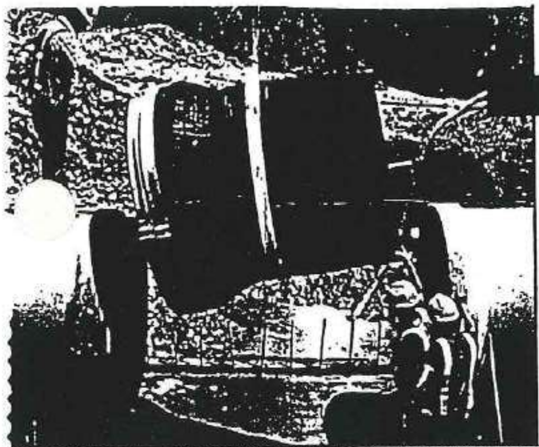


Tideflex™ Valves Are *Readily* Available in a Wide Range of Sizes—From ½" to 90" and performance criteria.



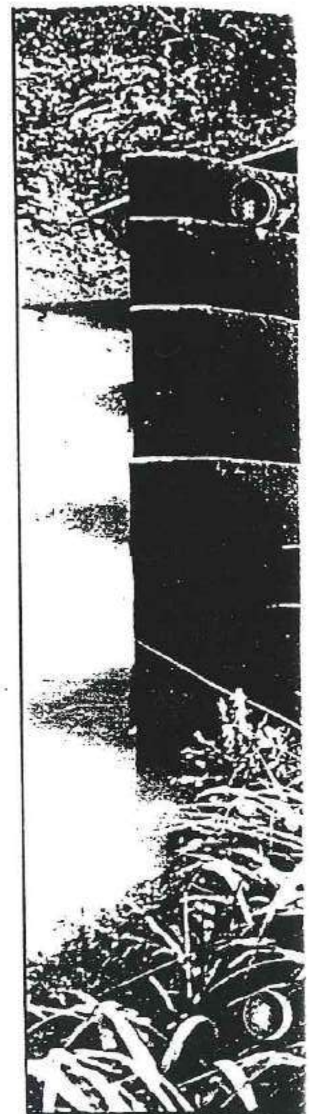
Flow tests conducted by Utah State University Water Research Laboratories.

Red Valve will provide headloss flow-charts for your specific application requirements.



WATER AND WASTEWATER PIPELINES

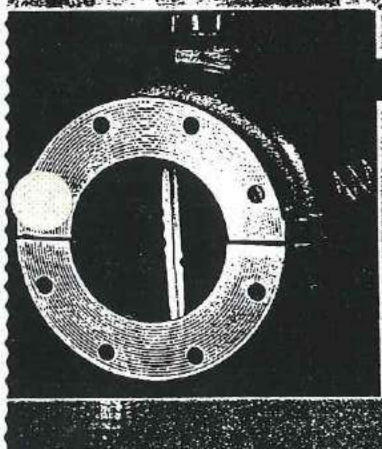
Large and small diameter Tideflex™ Valves are used in water and slurry pipelines where absolute backflow prevention is critical. The valve's maintenance-free design makes it the perfect choice for pipeline applications.



FLOOD CONTROL

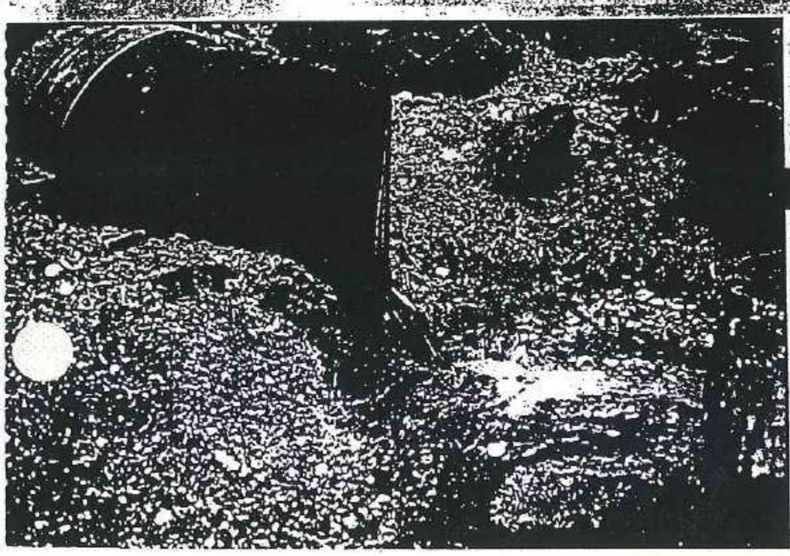
Tideflex™ Check \
extensively for floc
applications includ
levees, locks and (
systems, city storn
systems, highways
runways and large
industrial complex

Tideflex™ Applications: Simply Versatile!



WASTEWATER TREATMENT PLANTS

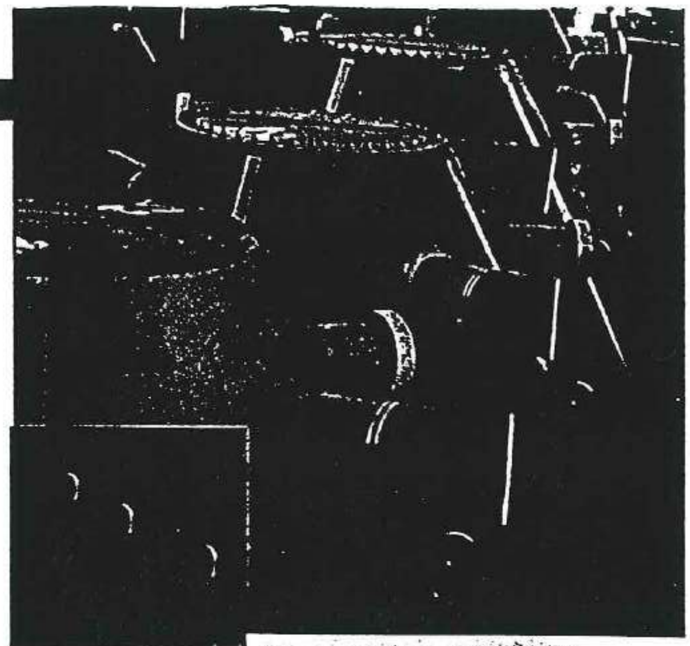
Red Valve's new Series 39 inline check valve uses patented Tideflex™ technology. There are no mechanisms, parts, hinges, discs or metal seats to freeze, corrode or bind. The unique elastomer "duckbill" sleeve seals on solids resulting in a silent non-slamming valve. The all-rubber construction eliminates maintenance requirements of traditional check valves. Large diameter inline check valves are an excellent choice for water, sewage and industrial pipelines to prevent backflow.



DRAINAGE AND OUTFALL LINES

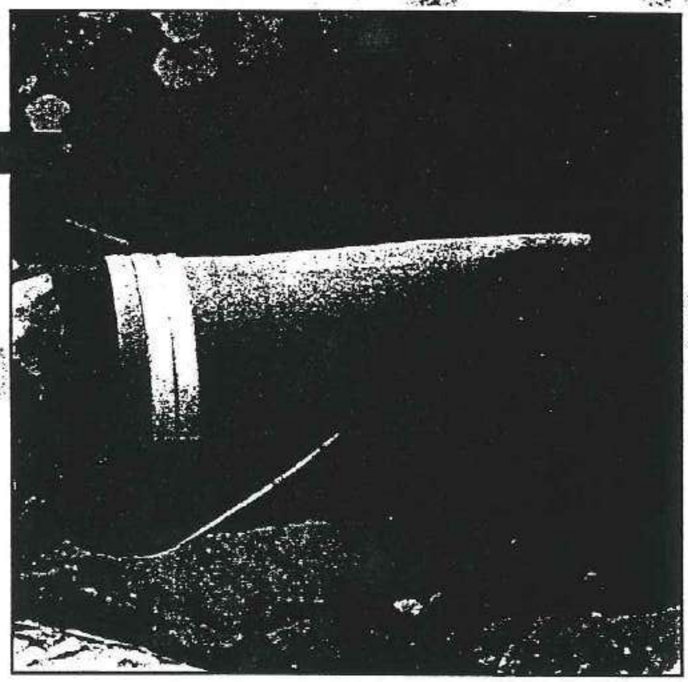
Tideflex™ Check Valves have become a frequently specified solution for commercial and residential areas where complete, dependable backflow prevention is necessary. The valve's maintenance-free, passive operation provides years of trouble-free service -- even when the valve is partially buried!

EFFLUENT DIFFUSER SYSTEMS



The Tideflex™ Valve's simple, all-rubber, maintenance-free design is the ideal choice for today's effluent diffuser systems. Silt, rocks, debris or tidal sand will not impede operations, and saltwater cannot enter the diffuser pipeline. Tideflex™ Check Valves are manufactured with varying degrees of headloss to compensate for pressure drop and maintain optimum discharge velecity of diffusion.

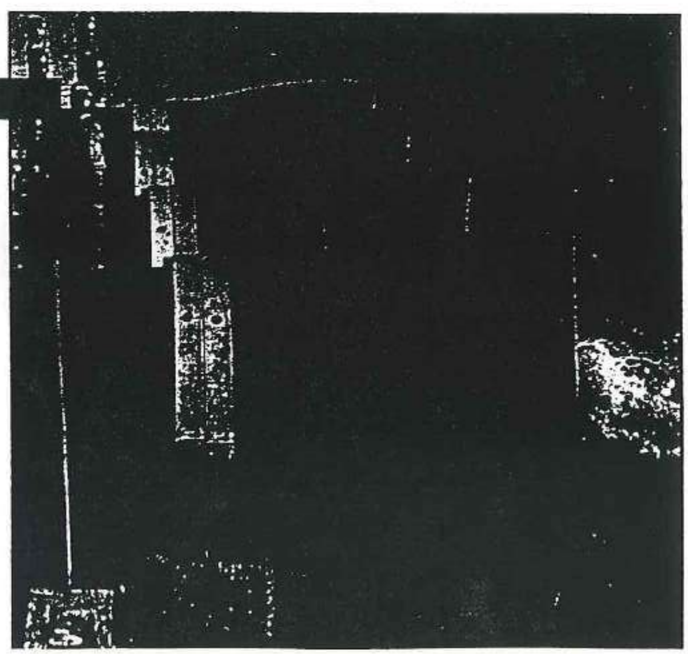
EFFLUENT DISCHARGE



As a safer, cleaner environment continues to be of paramount importance, Red Valve's Tideflex™ Valve continues to do the job. Effluent discharge from Wastewater Treatment plants are an ideal application. Ecosystems are protected by the Tideflex™ Valve's ability to diffuse effluent and prevent backflow.

are used
ol. Typical
tion basins,
nd fish bypass
collection
ng lots, airport
and

STORMWATER RUNOFF



Tideflex™ is the valve of choice of coastal and inland municipalities for stormwater and CSO systems. The Tideflex's™ low headloss characteristic is an important feature especially in low lying areas. Tideflex™ has no hinges or seats to warp or corrode. It's maintenance free!

TIDEFLEX™ PE

E.P.A. Tests Call Tideflex™ an "Excellent Solution."

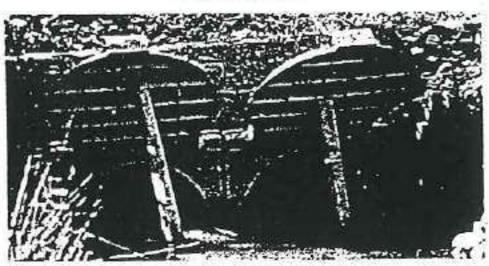
The Environmental Protection Agency's (E.P.A.) recent test results proved Red Valve's patented Tideflex™ Check Valve to be an excellent solution to eliminate maintenance costs and operational failures with traditional flap gate valves.

According to the report:

"Problems with malfunctioning flap gates, like frozen hinge pins, accumulation of debris, worn seats, misalignment, warpage and corroded parts and costs of maintenance crews are eliminated with the Tideflex™ Valve."

Today, thousands of patented Tideflex™ Valves and diffuser valve systems are operating maintenance-free worldwide. These valves have successfully withstood severe winter freezes, typhoons, hurricanes and flooding, minimizing damage to wetlands, beaches and residential areas, eliminating hydraulic surges to waste water treatment plants and saving municipalities millions of dollars in maintenance and treatment costs.

PROBLEM



These traditional flap gate valves were held open by telephone poles to eliminate loud clanging noises and allow for better outflow. Unfortunately, they no longer prevented backflow into the city's water treatment plant.

SOLUTION

Tideflex™ all rubber check valves were installed, and eliminated the noise as well as completely preventing the backflow problem. Simply Revolutionary!

Function

The Tideflex™ Valve is manufactured of flexible elastomer material reinforced with synthetic fabric much like an automobile tire. Neoprene construction with a special EPDM cover for ozone protection is furnished as a standard. Pure Gum Rubber, Hypalon, Butyl, Buna-N, EPDM and Viton are also available, and come with standard EPDM covers.

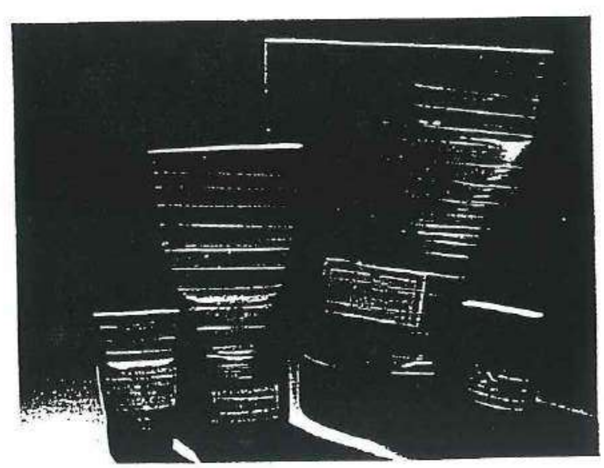
Forward hydraulic pressure opens the valve automatically without any additional energy source and reverse hydraulic pressure seals the valve automatically. The Tideflex™ Valve is simple to install. Two metal bands easily connect it to the O.D. of a pipeline.

By engineering the elastomer fabric matrix in varying degrees of flexibility, each Tideflex™ Valve is customized to your exact application to open with minimum specified head pressure and withstand maximum specified back pressure.

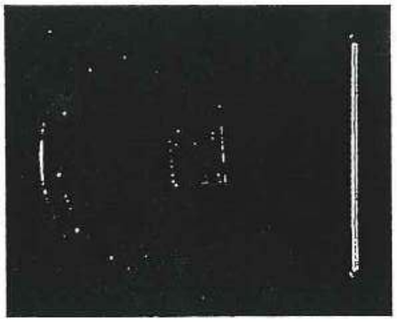
This versatile design of the Tideflex™ Valve also allows it to be used as a vacuum breaker on pipelines and pressure vessels to prevent closing.

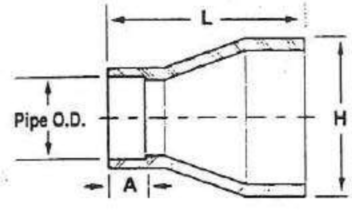
The inherent cushioning action of the Tideflex™ Valve's elastomer design completely eliminates noise. The valve's heavy-duty construction makes it vandal-proof and reduces the likelihood of children entering a pipeline.

A number of other custom-designed check valves like Red Valve's Series 33 are available for in-line service.

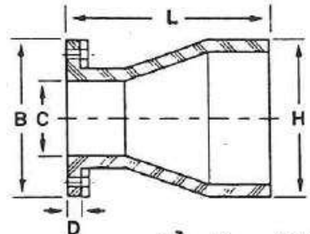


RFORMANCE





Tideflex™



Series 35

In some applications and installations, a slip-over pipe Check Valve is not feasible because of an existing flange in the piping system or an existing flange cemented in the outfall piping system vault. In these cases, the Series 35 Flanged Check Valve is the solution.

The Red Valve Series 35 Check Valve is manufactured identically to the Tideflex™ Check Valve, with the addition of an integral elastomer flange as part of the valve. The standard flange size drilling conforms to ANSI B16.5, & ANSI B16.47 class 150 standards. All other domestic and international flange standards, as well as customer specified flange dimensions, are also available. The Series 35 is furnished complete with 3/8" thick galvanized steel or stainless steel retaining rings.

TIDEFLEX™			SERIES 35					
MATING PIPE*		CUFF SLIP-ON LENGTH A	ANSI FLANGE SIZE	FLANGE O.D. B	INSIDE DIAMETER C	FLANGE THCKNESS D	MAXIMUM LENGTH L	MAXIMUM HEIGHT H
MIN. PIPE O.D.	MAX. PIPE O.D. (less than)							
3/8"	5/8"	1/2"	1/2"	3-1/2"	1/2"	1/2"	2-1/2"	1-1/4"
5/8"	7/8"	1"	3/4"	3-7/8"	3/4"	1/2"	3"	1-1/2"
7/8"	1-1/8"	1"	1"	4-1/4"	1"	1/2"	3"	1-1/2"
1-1/8"	1-3/8"	1"	1-1/4"	4-5/8"	1-1/4"	1/2"	5-3/4"	2-3/4"
1-3/8"	1-3/4"	1"	1-1/2"	5"	1-1/2"	1/2"	5-3/4"	3-5/8"
1-3/4"	2-3/8"	1"	2"	6"	2"	1/2"	5-3/4"	3-5/8"
2-3/8"	2-7/8"	1"	2-1/2"	7"	2-1/2"	1/2"	7-1/2"	4-5/8"
2-7/8"	3-7/8"	1-1/2"	3"	7-1/2"	3"	3/4"	9"	5-3/8"
3-7/8"	4-7/8"	2"	4"	9"	4"	3/4"	12"	7"
4-7/8"	5-7/8"	2"	5"	10"	5"	3/4"	15-1/4"	8-7/8"
5-7/8"	7-7/8"	2"	6"	11"	6"	1"	15-5/8"	10-3/8"
7-7/8"	9-3/4"	2"	8"	13-1/2"	8"	1"	16-1/2"	13"
9-3/4"	11-7/8"	3"	10"	16"	10"	1"	21-1/2"	16-7/8"
11-7/8"	13-3/4"	4"	12"	19"	12"	1"	26-1/2"	20-1/8"
13-3/4"	15"	4"	14"	21"	14"	1"	25-3/8"	21-1/2"
15"	17-1/4"	7"	16"	23-1/2"	15-1/4"	1"	27-1/2"	22-1/4"
17-1/4"	19"	8"	18"	25"	17-1/2"	1-1/2"	30"	26-3/4"
19"	21"	8-1/2"	20"	27-1/2"	19-1/4"	1-1/2"	32-3/8"	32-1/2"
21"	23-3/4"	8"	22"	29-1/2"	21-1/4"	1-1/2"	35-1/2"	32-1/2"
23-3/4"	29"	8"	24"	32"	24"	1-1/2"	40-1/2"	37"
29"	31-1/2"	10"	30"	38-3/4"	29-1/2"	1-1/2"	43"	49-1/2"
31-1/2"	35-1/4"	12"	32"	41-3/4"	32"	1-1/2"	51-3/8"	46"
35-1/4"	42"	12"	36"	46"	35-1/4"	1-1/2"	54"	58"
42"	48"	12"	42"	53"	42"	2"	60-1/4"	72-1/2"
48"	60"	10"	48"	59-1/2"	48"	2"	59"	77-1/2"
60"	72"	12"	60"	73"	60"	2"	72"	96-3/4"
72"	84"	16"	72"	86-1/2"	72"	2"	95"	102"
84"	90"	18"	84"	99-3/4"	84"	2"	92"	110-1/2"
90"	96"	16"	-	-	-	-	100-1/4"	118-1/2"

• Dimensions are subject to change due to customized construction. Contact Engineering for Dimensional Certification.

* Steel, Concrete, and Ductile Iron pipe O.D.s vary. Tideflex™ dimensions are based on actual pipe O.D., and therefore it is important to verify pipe O.D. for proper sizing.

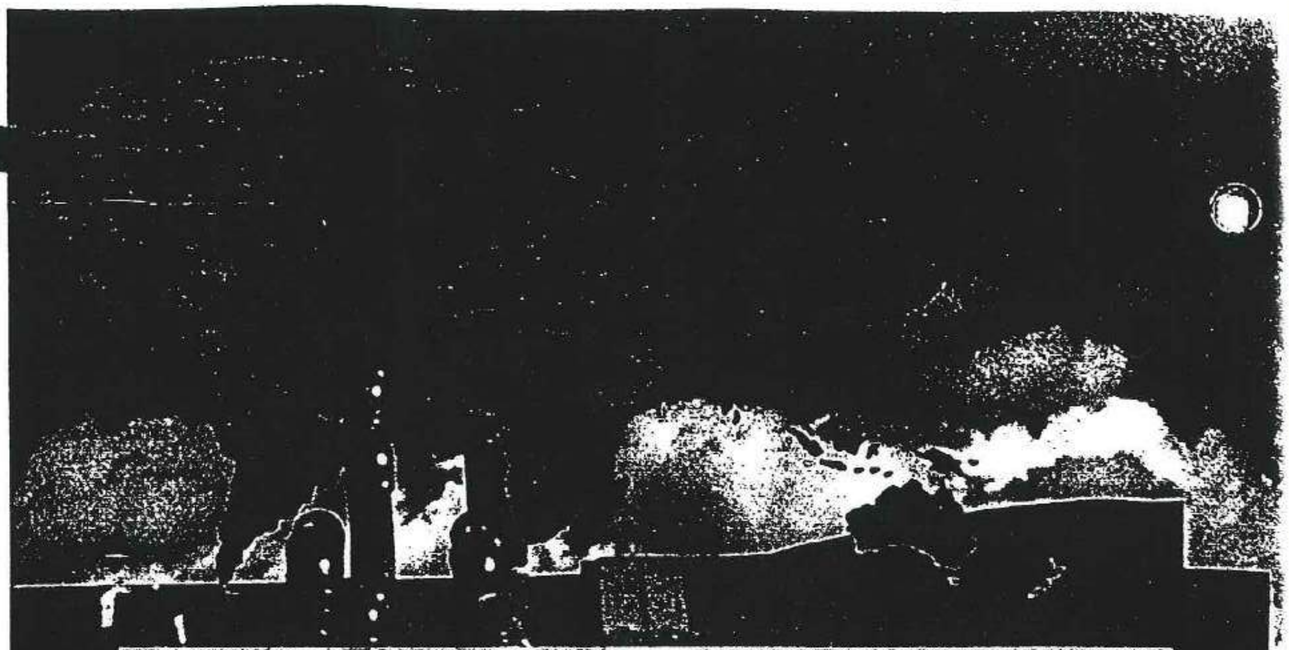
Red Valve Company, Inc.

700 N. Bell Ave.

Pittsburgh, PA 15106

(412) 279-0044

FAX (412) 279-7878



WORLD HEADQUARTERS
Pittsburgh, PA

Manufacturing Plants
Charlotte, NC
London, England

Technical Center
Carnegie, PA

Regional Sales Office
Houston, TX
Salt Lake City, UT
Orlando, FL
Philadelphia, PA

CANADIAN REPRESENTATIVES
Calgary, AB
Dartmouth, NS
Montreal, PQ
Regina, SK
Sudbury, ON
Downsview, ON
Toronto, ON
[illegible]
Winnipeg, [illegible]

U.S. REPRESENTATIVES
Anchorage, AK
Atlanta, GA
Baltimore, MD
Baton Rouge, LA
Billings, MT
Boston, MA
Buffalo, NY
Charlotte, NC
Chicago, IL
Cincinnati, OH
Cleveland, OH
Corpus Christi, TX
Dallas, TX
Denver, CO
Detroit, MI
Hartford, CT
Honolulu, HI
Houston, TX
Indianapolis, IN
Kansas City, KS
Los Angeles, CA
Milwaukee, WI
Minneapolis, MN
[illegible]
New Orleans, LA
[illegible], NE
[illegible], PA
Pittsburgh, PA
[illegible]
Richmond, VA
[illegible], MO
St. Paul, MN
Salt Lake City, [illegible]
San Francisco, CA
[illegible]
Tulsa, OK
[illegible]

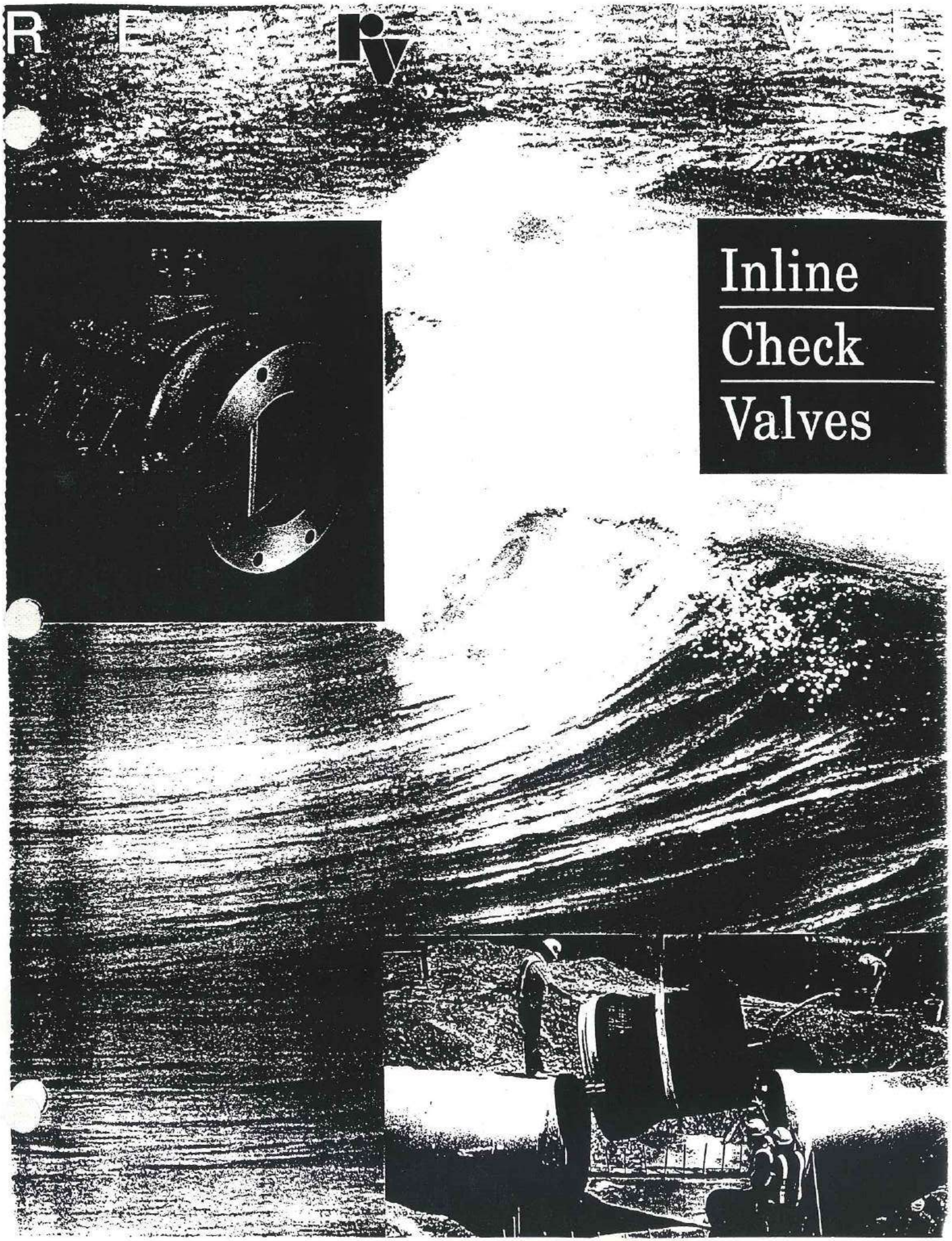
INTERNATIONAL REPRESENTATIVES

EUROPE
Austria
Belgium
Denmark
England
Finland
France
Germany
Ireland
Netherlands
Norway
Romania
Spain
Sweden

PACIFIC
Australia
China
Hong Kong
Indonesia
Korea
New Zealand
Philippines
Singapore
Thailand

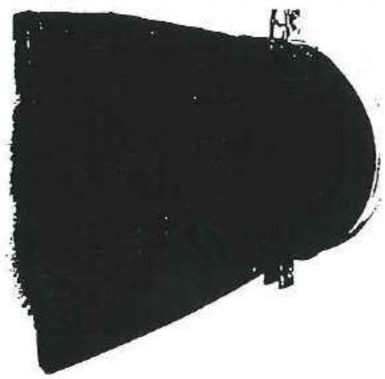
AMERICAS
[illegible]
[illegible]
[illegible]
[illegible]
Peru
Trinidad
Venezuela

[illegible] in this catalog is provided in good [illegible] to modify or improve its design [illegible] imply any guarantee or warranty for any [illegible] herein. All orders [illegible] and are subject to final [illegible] by Red Valve.



Inline Check Valves

In Line Check Valves



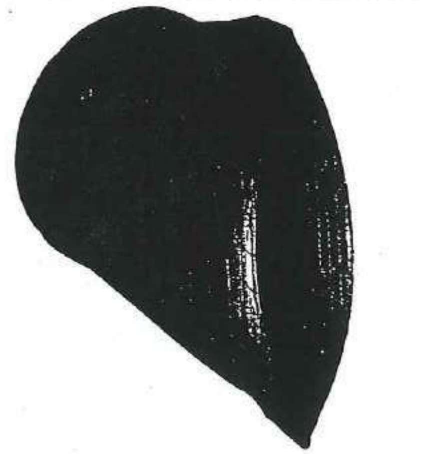
The design of Duckbill Check Valves employs Red Valve's elastomer and fabric Tideflex™ technology.

A technically advanced Duckbill Check Sleeve, manufactured with flexible elastomer material reinforced with special synthetic fabrics, is vulcanized into a duckbill shape with elastomer memory.

Forward hydraulic pressure opens the valve and on reverse flow hydraulic pressure seals the valve automatically.

Red Valve Duckbill Check Valves provide these benefits:

- Close on and around entrapped solids
- Lowest headloss
- No mechanical parts to fail
- No seats to warp, corrode, or fail
- Install in a vertical or horizontal pipeline
- Elimination of chatter or noise problems



Finite Element Analysis Programming is used in Design of Duck Bill Check Sleeve

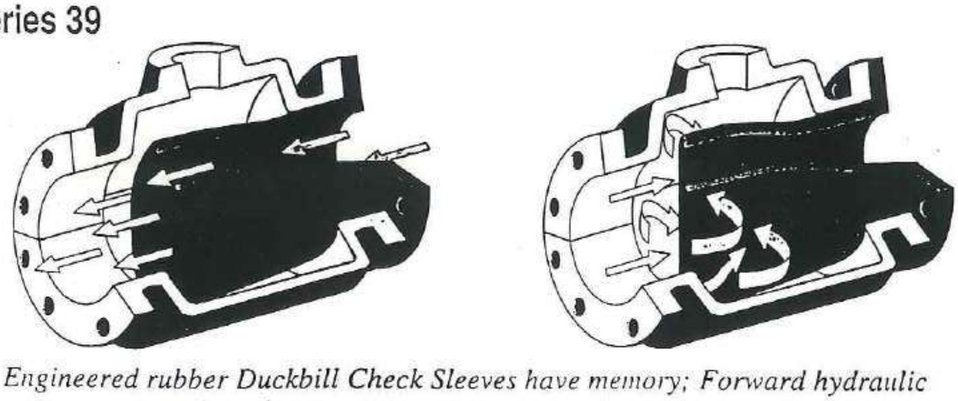
Red Valve Company has engineered a complete line of Check Valves in sizes 1/2" through 84".

Plant operators and maintenance personnel have learned to live with swinging gate, lever arm, flap, and swing check valve designs which require continuous maintenance of hinges, seats, pins, loud clanging noises, or continuous check valve chatter. Red Valve Check Valve's 100% elastomer Duckbill Check Sleeve design eliminates these problems:

- Corrosion and mechanical failure problems
- Seating problems
- Slamming and loud noise problems
- Chatter problems
- Debris and hangup problems
- Mechanical parts or maintenance
- Positioning problems

These products are designed to your flow specifications. When ordering, please specify head pressure and maximum back pressure of system.

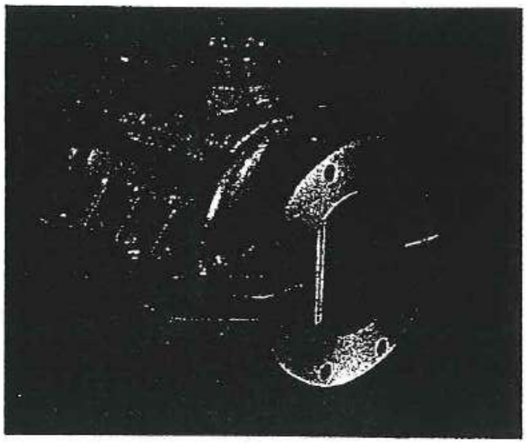
Series 39



Engineered rubber Duckbill Check Sleeves have memory; Forward hydraulic pressure opens the valve; reverse pressure seals the valve and prevents backflow.

Series 39

- Elastomer check valve resists abrasion and provides absolute backflow prevention
- Seals on entrapped solids
- No hinges or seats to bind or freeze – a maintenance-free design
- Can be mounted in any position
- Silent, non-slamming; eliminates chatter



The Red Valve Series 39 Slurry Check Valve is designed to handle abrasive slurries, sewage, sludge, and other difficult services. The heart of the Series 39 Check Valve is a fabric reinforced elastomer check sleeve that provides thru-flow at minimum pressure drop across the valve at all times. Forward pressure opens the valve automatically, reverse pressure seals the valve.

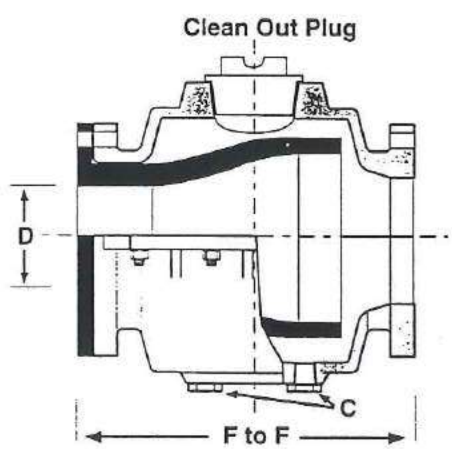
Wear and deterioration caused by continuous operation of abrasive slurries is minimized because of the inner rubber check valve. There are no mechanical parts such as hinges, discs, or metal seats which can freeze, corrode, or bind valve operation. The unique elastomer check sleeve will seal on solids. This valve's operation is silent and non-slamming.

Valve body is cast iron. Epoxy coating or rubber lined body are available. The Series 39 Inline Check Valve is provided with an inspection port and two clean out ports.

Face-to-Face dimensions meet ANSI B16.10 specs. The valve has thru-drilled flange holes. When ordering, advise line pressure and back pressure.

Materials of Construction

- Cast Iron ASTM A126 Body
- Check Sleeves Available in Pure Gum Rubber, Neoprene, Hypalon, Chlorobutyl, Polyurethane, Buna-N, Viton, and EPDM
- ANSI Class 125/150
- Epoxy Coating or Rubber Lined Body Available



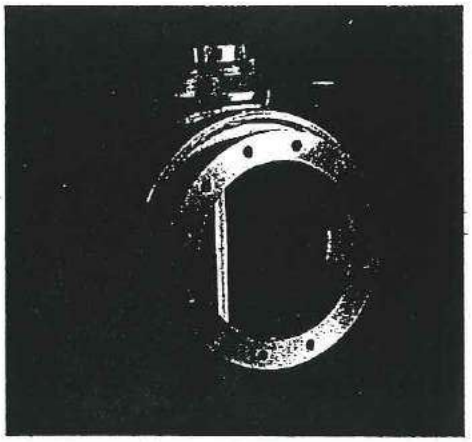
Dimensions Series 39

Size	Length F to F*	Clean Out Plug Diameter	Flush Connection C	Max. Back Pressure (psi)
4"	11½"	2"	1"	150
6"	14"	4"	1"	150
8"	19½"	4"	1"	125
10"	24½"	4"	1"	100
12"	27½"	4"	1"	75
14"	31"	4"	1"	75
16"	34"	4"	1"	50

* Higher backpressure designs available – Consult factory.

Series 39F

- Fabricated large diameter check valve design
- Seals on entrapped solids
- No hinges to bind or freeze – a maintenance free design
- Can be mounted in any position
- Silent, non-slamming



4

Materials of Construction

- Check Valve Sleeves Available in Pure Gum Rubber, Neoprene, Hypalon, Chlorobutyl, Polyurethane, Buna-N, Viton, and EPDM
- Fabricated Steel ASTM A285 Grade C or Stainless Steel 304SS ASTM A240 or 316SS ASTM A240
- Epoxy Coating Available
- ANSI Class 125/150 Flanges

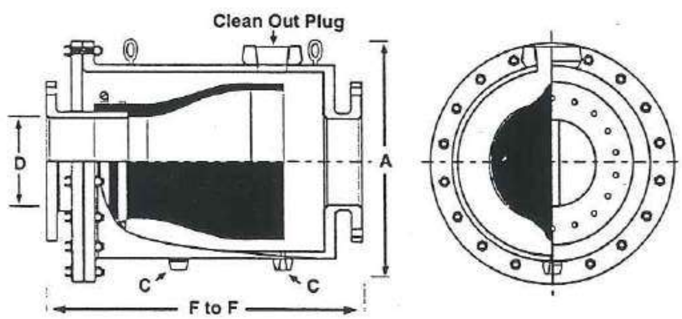
The Red Valve Series 39F Fabricated Slurry Check Valve is designed to handle abrasive slurries, sewage, sludge, and other difficult services. The heart of the check valve is a standard Tideflex™ Check Sleeve that provides thru-flow at minimum pressure drop across the valve at all times. Forward pressure opens the valve automatically, reverse pressure seals the valve.

Wear and deterioration caused by continuous operation of abrasive slurries is minimized because of the inner rubber check valve. There are no mechanical parts such as hinges, discs, or metal seats which can freeze, corrode, or bind valve operation. The Series 39's unique elastomer check sleeve will seal on solids. This valve's operation is silent and non-slamming.

The steel fabricated valve body is designed to permit easy installation or replacement of a standard Tideflex™ Check Valve.

Epoxy coating is available. The Series 39F Fabricated Inline Check Valve is provided with an inspection port and bottom flush ports.

Face-to-Face dimensions meet ANSI B16.10 specs. The valve has tapped flange holes. When ordering, advise line pressure and back pressure.



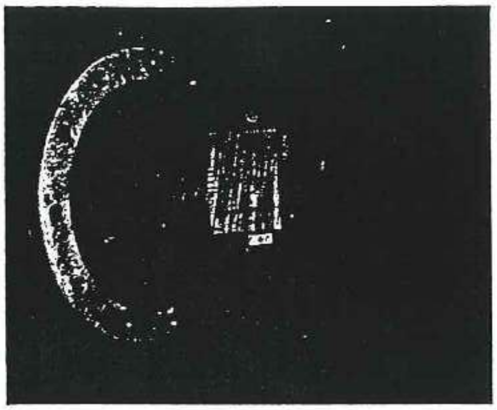
Dimensions Series 39 Fabricated Body

Size D	Length F to F*	O.D. A	Clean Out Plug Dia.	Flush Port Dia. C	Max. Back Pressure (psi)
18"	38½"	44"	6"	1"	50
20"	38½"	46"	6"	1"	50
24"	51"	55"	6"	1"	50
30"	60"	66"	6"	1"	50
36"	77"	77"	6"	1"	50
42"	80"	90"	6"	1"	25
48"	90"	102"	6"	1"	25
54"	101"	114"	6"	1"	25
60"	105"	126"	6"	1"	25
72"	118"	150"	6"	1"	25

* Higher backpressure designs available – Consult factory.

Series 35

- 100% elastomer construction eliminates maintenance
- Will not warp or freeze open or shut
- Eliminates backflow, seals on entrapped solids
- Custom built to customer specifications



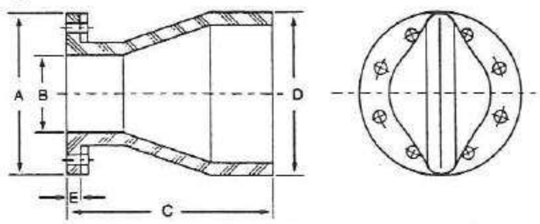
Materials of Construction

- Sleeves Available in Pure Gum Rubber, Neoprene, Hypalon, Chlorobutyl, Polyurethane, Buna-N, Viton, EPDM, and Food Grade Rubbers
- Galvanized Steel or Stainless Steel Backup Rings

The Red Valve Series 35 Check Valve is manufactured identically to the Tideflex™ Check Valve, with the addition of an integral elastomer flange as part of the valve. The flange size drilling conforms to ANSI B16.10, Class 150#. It is also available with DIN, 2632, and other standards. The Series 35 Check Valve is furnished complete with steel back-up rings for installation.

In some applications and installations, a slip-over pipe Check Valve (TF-2) is not feasible because of an existing flange in the piping system or an existing flange cemented in the outfall piping system vault. In these cases, the Series 35 Check Valve is the solution.

The Red Valve Series 35 Check Valve is simple in design, with only one part - the all-rubber duck bill check sleeve. There are no seats or interference fits to corrode or freeze valve operation, making the Series 35 virtually maintenance free. The Series 35 seals completely around solids, making it ideal for fly ash, raw sewage, sludge, lime, mining slurries, and many other abrasive and corrosive slurries.

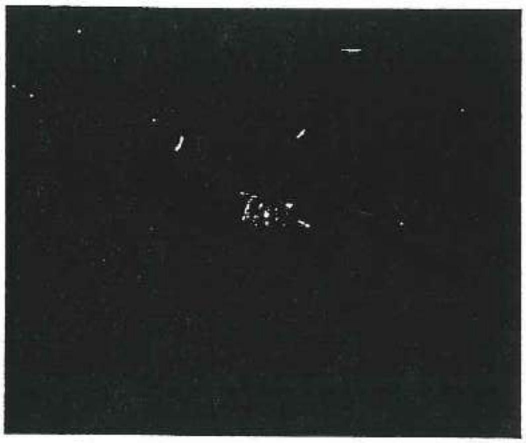


Dimensions Series 35 Flanged Check Valve

Valve Size	Flange O.D. A	Inside Diameter B	Length C	Height of Bill D	Flange Thickness E
1"	4¼"	1"	3"	2"	½"
1½"	5"	1½"	5½"	3"	½"
2"	6"	2"	6"	4"	½"
2½"	7"	2½"	7½"	5"	½"
3"	7½"	3"	9"	6"	¾"
4"	9"	4"	12"	7"	¾"
5"	10"	5"	15"	9"	¾"
6"	11"	6"	16"	11"	1"
8"	13½"	8"	17"	13"	1"
10"	16"	10"	22"	17"	1"
12"	19"	12"	26"	21"	1"
14"	21"	14"	27"	23"	1"
16"	23½"	15¼"	28"	27"	1"
18"	25"	17¼"	30"	30"	1½"
20"	27½"	19¼"	32"	33"	1½"
24"	32"	24"	40"	40"	1½"
28"	36½"	27¼"	41½"	46"	1½"
30"	38¾"	29¼"	43"	50"	1½"
36"	46"	35¼"	54"	59"	1½"
42"	53"	41½"	60"	69"	2"
48"	59½"	47½"	69"	78"	2"
54"	66¼"	53"	79½"	88"	2"
60"	73"	59"	82"	98"	2"
72"	86½"	71"	95"	117"	2"
78"	93"	77"	97"	127"	2"
84"	99¾"	83"	102"	137"	2"

Series 37

- Installs between pipe flanges, eliminates valve body
- Unique one-piece elastomer check sleeve design is maintenance-free
- Silent, non-slamming
- Closes on entrapped solids



Red Valve's new Series 37 Flanged In-line Check Valve is a simple, reliable, cost effective method of backflow prevention. The revolutionary design of the Series 37 is similar to the design of the patented Tideflex® Check Valve. The Series 37 is designed to be installed between two mating flanges, eliminating the need for a valve body.

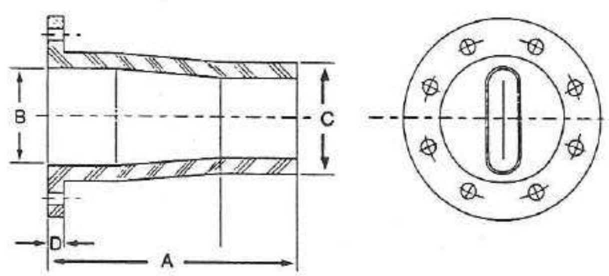
The pressure drop of the Series 37 is increased because of the smaller i.d. required to fit the check valve in the line.

The in-line Series 37 Check Valve is simple in design. There is only one moving part, the maintenance-free rubber check sleeve. Sliding, rotating, swinging, and spring parts are eliminated. There are no seats to corrode or packing to maintain; the valve is maintenance-free. The Series 37 is a passive design, requiring no external source of air or electricity to operate, thus reducing operating costs.

The Series 37 Check Valve can be ordered in a variety of elastomers. Flanges conform to ANSI B16.1 Class 125 specifications. Special custom designs or metric flanged models are also available.

Materials of Construction

- Sleeves Available in Pure Gum Rubber, Neoprene, Hypalon, Chlorobutyl, Polyurethane, Buna-N, Viton, and EPDM
- ANSI Class 125 Flanges



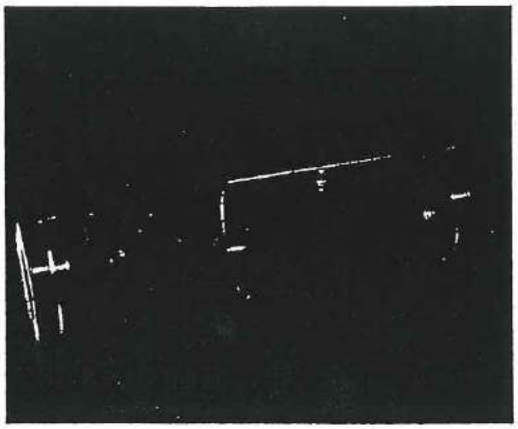
Dimensions Series 37 Flanged In-Line Check Valve

Valve Size	Length A*	Inside Diameter B	O.D. of Valve C	Flange Thickness D	Working Pressure (psi)
2"	6"	1 1/4"	1 7/8"	3/8"	125
3"	7 1/4"	2 1/4"	2 7/8"	3/8"	125
4"	13"	3"	3 7/8"	3/8"	100
6"	16"	5"	5 7/8"	3/8"	75
8"	18"	6 5/8"	7 5/8"	1/2"	75
10"	20"	8 5/8"	9 5/8"	1/2"	75
12"	21"	10 5/8"	11 5/8"	1/2"	75
14"	22"	11 1/2"	12 3/4"	5/8"	50
16"	24"	13 1/2"	14 3/4"	3/4"	50

* Larger sizes available upon request.

Series 2633

- Eliminates check valve chatter
- Seals 100% on reverse flow
- Can be mounted in any position
- Ideal for pneumatic systems



Materials of Construction

- Steel, Stainless Steel, or PVC Body
- Steel, Stainless Steel, or PVC End Connections
- Check Sleeves Available in Pure Gum Rubber, Neoprene, Hypalon, Chlorobutyl, Polyurethane, Buna-N, Viton, and EPDM

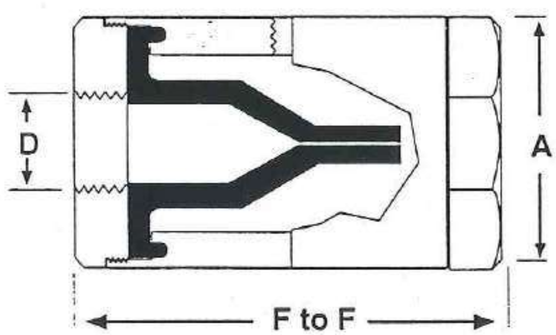
The Series 2633 is a simple in-line check valve for threaded end pipelines, and is manufactured on the same principle as Red Valve's revolutionary all rubber Tideflex® Check Valves.

The Series 2633 is simple in design: a body, two endcaps, and the working element, a special elastomer check sleeve. In the open position the sleeve creates a wide, free passage proportional to the flow in the pipeline. On flow reversal the "duck bill" shaped sleeve closes slowly and completely.

The silent, non-slamming Series 2633 Check Valve design eliminates water hammer and has low headloss. It contains no levers, packing, springs, or interference fits to corrode or freeze. The only replacement part is the simple, rugged elastomer check sleeve, making this valve virtually maintenance-free!

This small and simple in-line check valve is ideal for liquids, gases, powders, slurries, instrument or plant air, and in any environment where there is a need to prevent backflow. 7

The Series 2633 is manufactured in sizes 1/2" to 3". The check sleeve can be ordered in a variety of elastomers to match specific service conditions.



Dimensions Series 2633 Small Diameter In-Line Check Valve

Valve Size D	Length F to F	Body O.D. A	Maximum Back Pressure (psi)	Weight Steel (lbs.)
1/2"	3 1/2"	2 1/8"	125	2
3/4"	4"	2 1/4"	125	3
1"	4 1/2"	2 3/4"	125	3
1 1/2"	6 1/2"	3 3/4"	100	8
2"	7 1/2"	4"	75	14
3"	8 1/2"	5"	75	18

A Complete Line Of Quality Products . . . Built To Beat Slurries

Pinch Valves

Red Valve is the world's largest producer of Type A, Manual, and Control Pinch Valves in sizes 1/8" – 120".

Control Valves

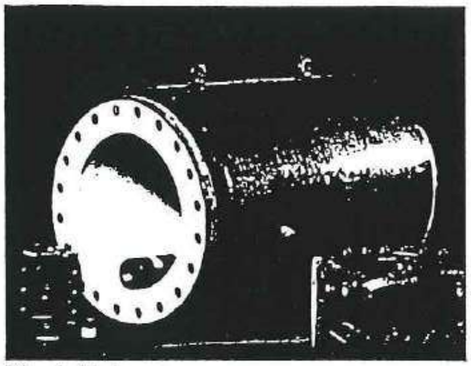
Pinch Valves offer the best solutions for slurry control with optional Slurry Cone® Sleeves.

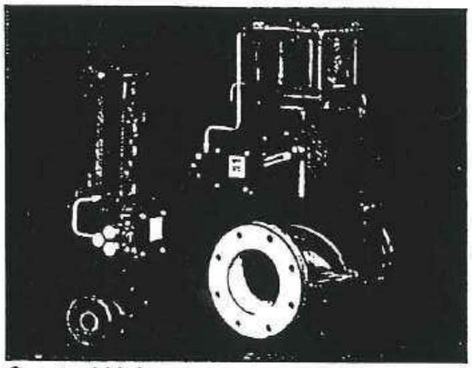
Knife Gate Valves

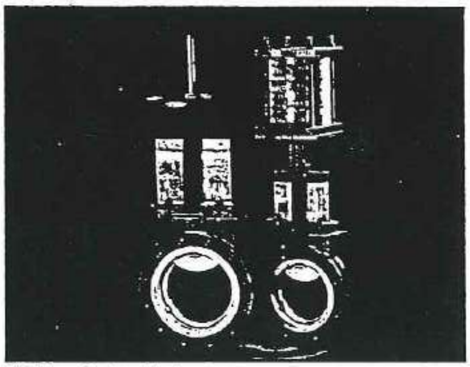
The Series G Cast Iron Body Knife Gate Valve and our new Flexgate Slurry Knife Gate Valve are built to perform in the toughest applications.

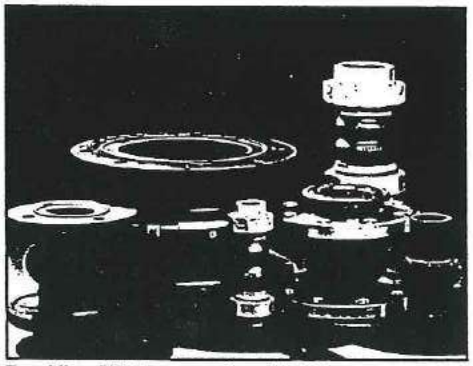
Redflex™

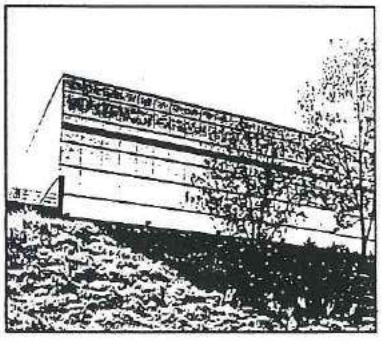
Red Valve offers a complete election of Redflex™ Expansion Joints and custom fabricated rubber pipe and fittings.


Pinch Valves


Control Valves


Knife Gate Valves


Redflex™ Expansion Joints


Red Valve corporate headquarters outside Pittsburgh, Pennsylvania

Red Valve® Company, Inc.
700 North Bell Avenue
P.O. Box 548
Carnegie, PA 15106
(412) 279-0044
FAX (412) 279-7878

Red Valve's quality product line includes a wide variety of Pinch Valves, Control Valves, Knife Gate Valves, Check Valves, Pressure Sensors, and Redflex™ expansion joints for power plants, mining operations, chemical plants, industrial, and municipal applications.

United States Environmental Protection Agency
Research and Development

Risk Reduction Engineering Laboratory Cincinnati OH 45268

EPA/600/S2-89/020 Feb. 1990

EPA

Project Summary

Development and Evaluation of a Rubber "Duck Bill" Tide Gate

Peter A. Freeman, Angelika B. Forndran, and Richard Field

A unique 54 in. diameter "duckbill rubber tide gate (RTG) was designed, fabricated, and installed in a typical New York City tide gate chamber. The operation of the RTG was observed over two years. The RTG was very effective in preventing the inflow of tidal waters and generally showed equal or improved performance compared to a typical flap gate. Hydraulically, the RTG was supposed to open to release storm flows at a positive difference in upstream head of six in. and to remain closed preventing inflow at a downstream positive head up to eight ft during high tide. Minor inflow was observed when debris was introduced into the RTG, and capability of self-cleaning was exhibited. Inflow would be significantly greater if similar size debris was lodged in the conventional flap-type gate. The maintenance crews observed no incident where the manual removal of debris was required. The existing chamber required minor modifications for the installation of the RTG. The method of adapting the RTG to an existing tide gate frame is critical to ensuring the reliability of the installation. The RTG was exposed on occasions to gale force winds and heavy rainfall during the two years of operation in New York City.

This Project Summary was developed by EPA's Risk Reduction Engineering Laboratory, Cincinnati, OH, to announce key findings of the research project that is fully documented in a separate report of the same title (see Project Report ordering information at back).

Introduction

Tide gates are a necessary component of municipal combined sewer systems, which discharge overflows into receiving waters whose surface elevations vary due to tidal or seasonal effects. In principle, these perform a check valve function, allowing excess flow mainly from storm events to discharge into receiving waters, while preventing back flow or leakage into the combined sewer system. Leakage can cause significant problems to the treatment process and associated hardware, due both to the presence of dissolved salts or other substances, as well as a waste of treatment plant capacity

The conventional flap tide gate operates by swinging outward (toward the receiving body of water) when the upstream flow exceeds the capacity of the regulator controlling flows to the interceptor (normally during storm conditions). The water level upstream of the tide gate rises to whatever level is necessary to offset the weight of the tide gate and the water level downstream of the gate. When there is no upstream flow, the gate sits firmly against the frame and does not permit backflow. Properly operating tide gates do not permit tidal inflow (backflow).

In New York City there are three types of such tide gates: (1) Pontoon gates which consist of hollow wrought iron flaps mounted on cast iron frames; (2) Timber gates predominantly made of three in. thick Greenheart timbers, and (3) Cast iron gates which are generally less than 48 in. high. A recently completed regulator improvement program study revealed that these gates were functionally adequate to prevent tidal inflow and mit excessive storm outflow. The sign life is 20 years. Some existing gates are as old as 30 years. Improperly functioning tide gates permit inflow in varying degrees. Malfunctioning gates accumulate debris, have worn seats, have corroded parts allowing entry of water, have become misaligned, and/or are warped. Inflow occurs as the tide rises above the invert of the outfall sewer. Inflow may be reduced when increasing downstream static head tends to seal the gate.

One investigation determined that maximum inflow occurs at about two-thirds high tide level when debris, warpage, or mis-alignment causes incomplete closure of the gate. Another problem which was identified is that the hinge pins tend to become frozen. Particularly in the dual hinge pin design, the intended function is lost when the lower pin is frozen. One recommended solution is to replace the existing pins with slightly undersized stainless steel pins. Pontoon type gates tend to deteriorate due to graphitization of cast iron components and corrosion/erosion of the wrought iron flaps. As a result, timber tide gates are recommended over pontoons.

The EPA has recognized the operational and economical problems of conventional tide gates. Based on these, improvements are required in tide gate technology as follows:

1. The ability to both open and close tightly in the presence of water borne debris must be greatly improved, both to prevent collection system surcharging and flooding, and also to reduce the cost of existing treatment efficiency by interfering with settling and anaerobic digestion processes and contributing to corrosion of plant equipment.
2. The reliability of tide gates must be greatly improved to relieve the requirement for frequent surveillance and maintenance, and the corresponding cost to the municipality.
3. Extended tide gate operating lifetimes are required to reduce recurring capital equipment costs.

Procedure

The subject program was set up to explore a novel approach to the tide gate problem. This approach offers considerable promise in achieving the desired performance discussed previously. The oposed concept was based on a type check valve designed and currently manufactured by the Red Valve Co., Inc.* of Carnegie, PA. (RV). This unit consists of a flexible tube which tapers to flattened sections with two or more sets of sealing lobes. Forward hydraulic head opens the lobes, to release flow. Reverse hydraulic head collapses the lobes together, to prevent reverse flow (leakage). The duckbill part of these valves is typically constructed of rubber-impregnated fabric, in the manner of an auto tire. This concept is shown in Figure 1. At the time of the program start, RV manufactured these units in diameters up to 12 in. It was the principal design task of this program to extrapolate this configuration to the 54 in. diameter required to release storm flow at the selected tide gate site.

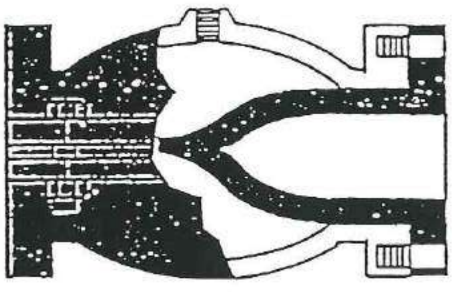


Figure 1. *Flanged end red check valve.*

This approach is attractive for municipal tide gate use in a number of ways. Mechanical moving parts, with their attendant problems of corrosion, friction, and wear, are replaced by flexible structures of environmentally stable elastomeric materials. The basic check valve action is performed without abrupt changes in flow direction, so that there is a minimum tendency to entrap debris. If debris is entrapped, the flexibility of the unit permits it to conform closely to the shape of the debris, minimizing the leakage flow under reverse hydraulic head conditions.

Manufacturing costs should remain consistent with advancement in technology in the tire industry.

Specifically, the program objectives were:

1. To identify and select a site which reasonably represented a typical tide gate location and permitted a demonstration of an RTG.
2. To install the RTG to the selected dimensions of 54 in. in diameter with hydraulic head flow characteristics similar to those of the conventional flap gate.
3. To install the RTG in a typical metropolitan combined sewer outfall, replacing a conventional flap gate with a minimum of site modification. This would demonstrate the feasibility of retrofitting into existing outfalls.
4. To evaluate the performance of the RTG, as so installed, under typical service conditions, for a period of at least 18 months. During this period, comparison was to be made with conventional flap gates as to incidences of malfunction (failure to open or close, leakage, etc.), necessary surveillance, servicing, hydraulic characteristics, and capital cost required for replacement or new installations.

The program was initiated in late 1981. The project team selected a combined sewer regulator site (Regulator #11) at 89th St. and East End Ave in Manhattan, at which a typical timber flap gate was to be replaced by the RTG. The site configuration is shown in Figure 2. RV selected an initial configuration with four sealing lobes, in a "cross" arrangement. A quarter-scale model was construct and successfully tested. The full-sc prototype unfortunately was unsuccessful, as the additional weight of the sealing lobes caused them to sag and seat in a random manner, allowing large gaps and leakage flows with reverse hydraulic head. The four-lobed arrangement was abandoned in favor of a vertically oriented, two-lobed configuration. An experimental two-lobed unit, shown in Figure 3, was completed in October 1983. Flow limitations at the RV test facility prevented full-flow hydraulic performance calibrations, so a procedure was generated to determine RTG flow area vs differential hydraulic head under static (no flow) conditions. This procedure showed that the RTG was marginally too stiff (too much hydraulic differential head was required to achieve the desired flow area). The final unit was constructed, given limited testing, and delivered to the New York City Department of Environmental Protection in December 1983.

Site modifications undertaken by NYCDEP were minimal. After removal of the existing flap gate, hinge brackets, and sealing frame, a stainless steel adapter plate was installed. The adapter makes the transition from the existing rectangular opening on a 15 degree slo tidegate chamber wall to the 54

*Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

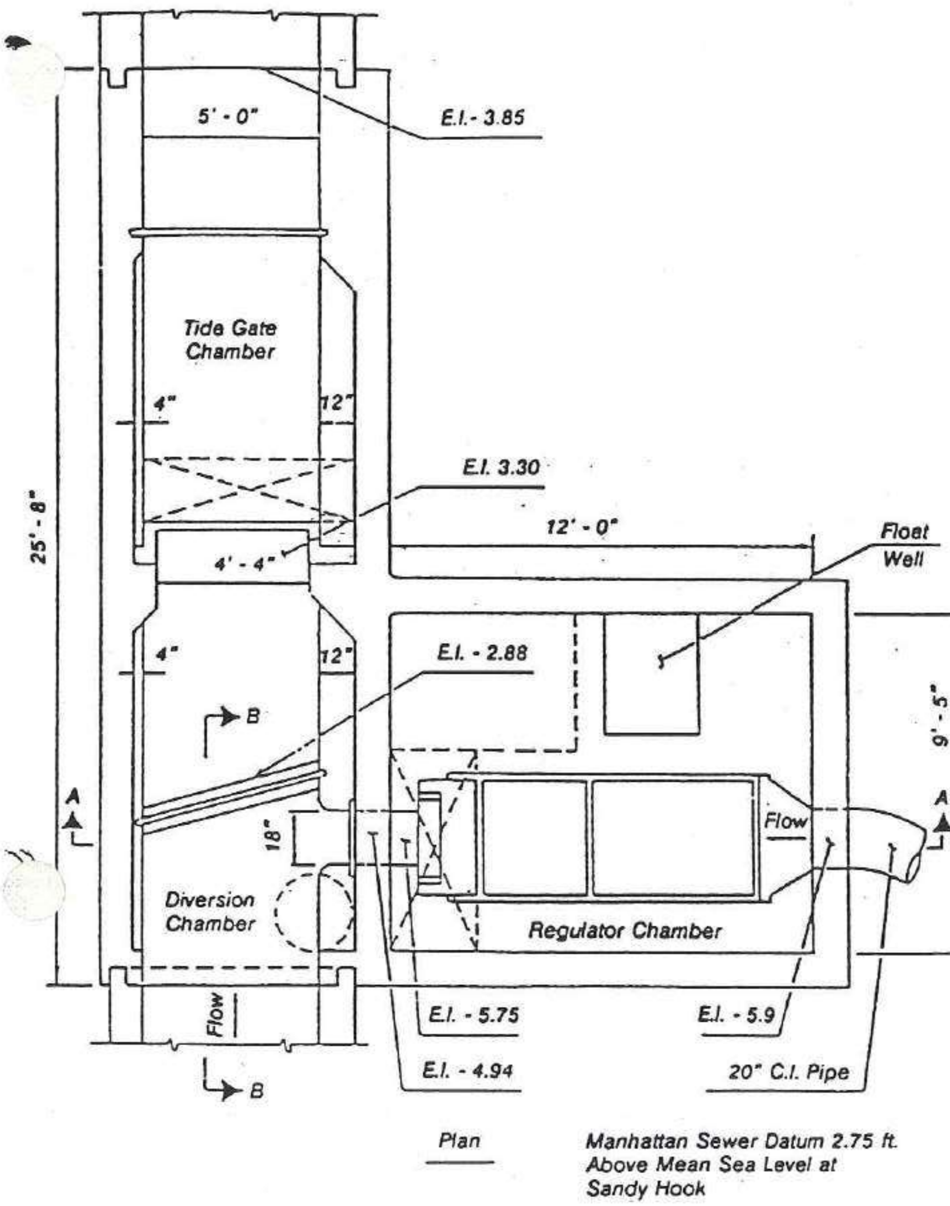


Figure 2. Wards Island - WPCP Regulator No. 11 (Plan).

circular /vertical opening required by the RTG. The adapter plate is shown in Figure 4. A clamping ring holds the RTG in position on the adapter plate stub. The RTG installation required about 2 days to complete, and was placed in service on August 11, 1984.

Results and Discussions

Upon reaching service status, the RTG was included in the normal inspection routine by NYCDEP regulator inspection maintenance crews. A special inspection sheet format was generated to assist them in making observations of the RTG's performance under various service conditions. Initial inspections were performed weekly starting October 25, 1984. The interval between inspections was increased to 2 weeks, and then 4 weeks after 8 months. A total of 18 months were observed as part of this project.

All inspection sheets indicated negligible or no leakage or inflow, even though there was in nearly all cases a reverse differential hydraulic head on the RTG, even at most low tide conditions. The inspection sheets also indicated that the RTG was normal (clean), and that no trapped debris was observed. A condition of an RTG with entrapped debris was simulated by inserting a 12-in. length of lumber (4 in. x 4 in.) into the RTG discharge section. A leakage flow of about 50 gpm occurred at a reverse hydraulic head of 2 ft. This simulated debris was later washed out of the RTG with the next occurrence of forward hydraulic head, indicating an excellent capability for self cleaning.

The principal observed difficulties with the RTG were occasional instances where hydraulic forces occurring during storm events moved the RTG on its mounting. On July 26, 1985, the RTG came loose from the adapter plate. It was reinstalled by the regulator maintenance crew in 7 hours, during which ~~techniques were~~ improvised for handling the heavy (800 lb) unit within the cramped confines of the tide gate chamber. This event prompted recommendations for suspension and handling facilities to be built into the tide gate chamber overhead, and the requirement for "pinning" the RTG to the adapter plate stub, in addition to the clamping ring.

These recommendations appear particularly desirable for future, larger RTG installations.

A rough, *in situ* hydraulic flow calibration of the RTG was performed during August and September, 1985. Continuous depth-of-flow measurements were made in the trunk sewer upstream of the regulator, and downstream of the RTG. These, plus the known hydraulic characteristics of the trunk sewer and regulator, were used to compute standard hydraulic relationships based on Manning's equation. The resulting flow calibration was relatively linear with increasing hydraulic head, as attributed to the fact that the RTG flow area is itself a function of hydraulic head differential. The unavailability of data from the storm events occurring during this period, plus some instrumentation failures, did not permit the generation of a complete flow calibration; however, a reasonable extrapolation of the obtained results indicated that the RTG's maximum flow capacity comfortably exceeds the maximum runoff flow rate from the selected drainage area without surcharging the trunk sewer.

A comparison of the hydraulic performance of the RTG and the flap-gate it replaced showed that the RTG starts to release flow at a lower hydraulic head differential for all conditions of downstream submergence. This difference

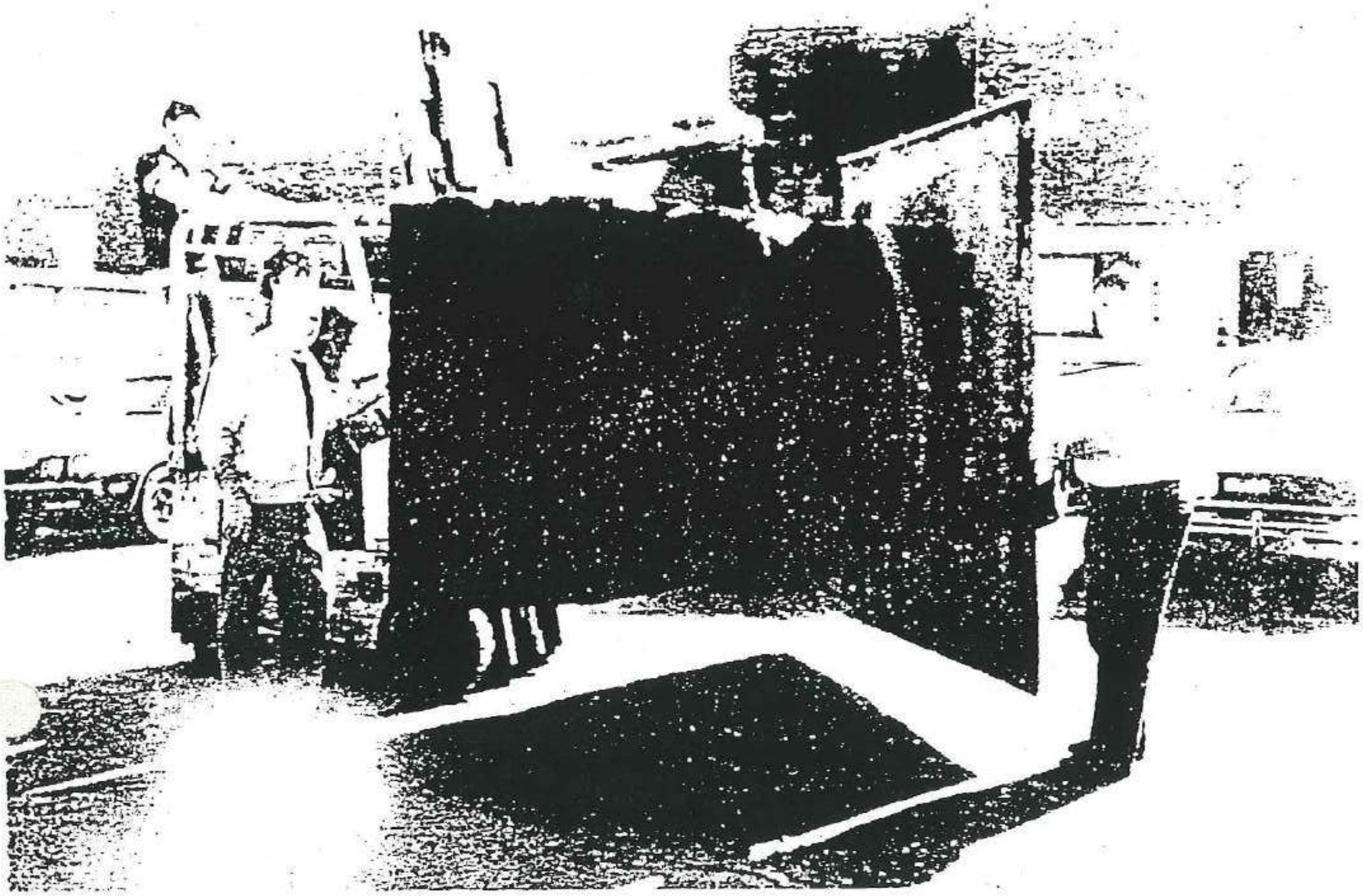


Figure 3. Two-lobed being prepared for plant testing.

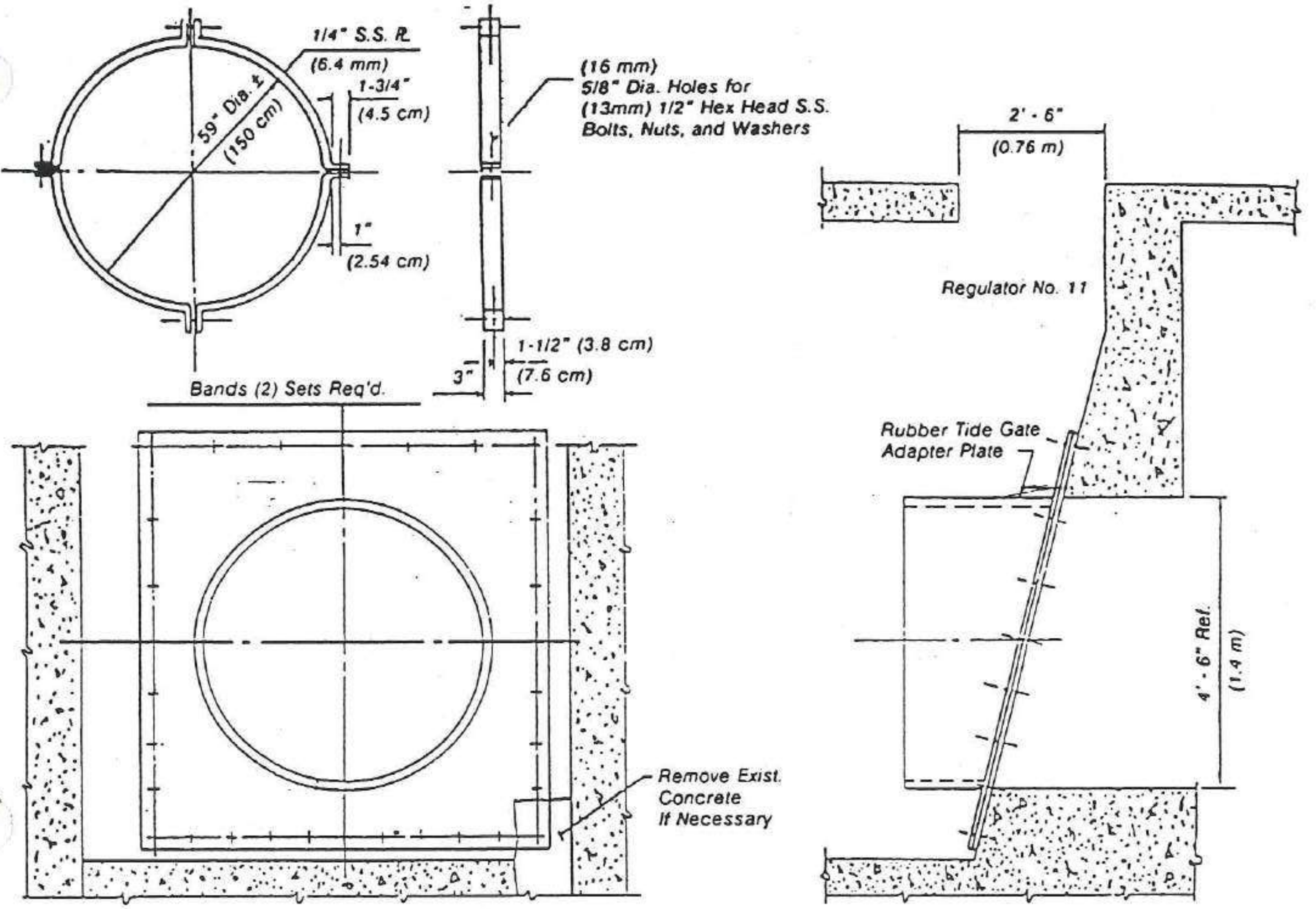


Figure 4. Adapter plate design details (fitting to frame).

occurs since the flap-gate is ballasted with lead to ensure closing under high tide conditions. The flap-gate has a higher maximum flow capability than the RTG (see Figures 5 and 6). Both units release more at less differential hydraulic head with increasing downstream submergence. The lower maximum flow capability of the RTG indicates a requirement for careful estimation of peak storm flows or oversizing, in selecting an RTG for a particular tide gate location.

Comparative costs for RTG and conventional flap-gates are given in Figure 7. These costs are manufacturer's costs only. Installation costs are dependent on location and ranged from $5,000 to $15,000 for retrofit with a RTG, while the more predictible flap gate replacement cost is approximately $9,000.

Factors to consider in estimating costs are the related savings due to:

(a) Operation and maintenance of tide gate system.

(b) Preventing inflows and treatment upsets caused by settling, digestion, and hydraulic overloading.

(c) Corrosion protection from industrial wastewaters. Structural limitation for each gate location, e.g., chamber modification, adapter plates must also be considered.

Conclusions

The basic conclusion from this program is that the rubber tide gate (RTG) is a practical, cost-effective alternative to the typical flap-type tide gate.

The RTG showed significant improvement over the flap-type tide gate in terms of leakage inflow, entrapment of debris, capability to self-clean, and susceptibility to marine fouling during 18 months of observed operation.

The RTG required virtually no labor-intensive surveillance or maintenance during routine inspection. Maintenance was required to reattach the rubber sleeve onto the adapter plate.

The design used in this prototype installation for attaching the RTG onto a smooth adapter plate using clamping rings was not sufficient to hold the RTG in place during the heavy storm and tidal action.

Non-stainless steel metals or stainless steel hardware not of type 316 will corrode in the brackish environment and cause failure of the installation by permitting the RTG to slip during storm and tidal action.

The RTG material consisting of neoprene over vulcanized rubber has shown no signs of any surface deterioration due

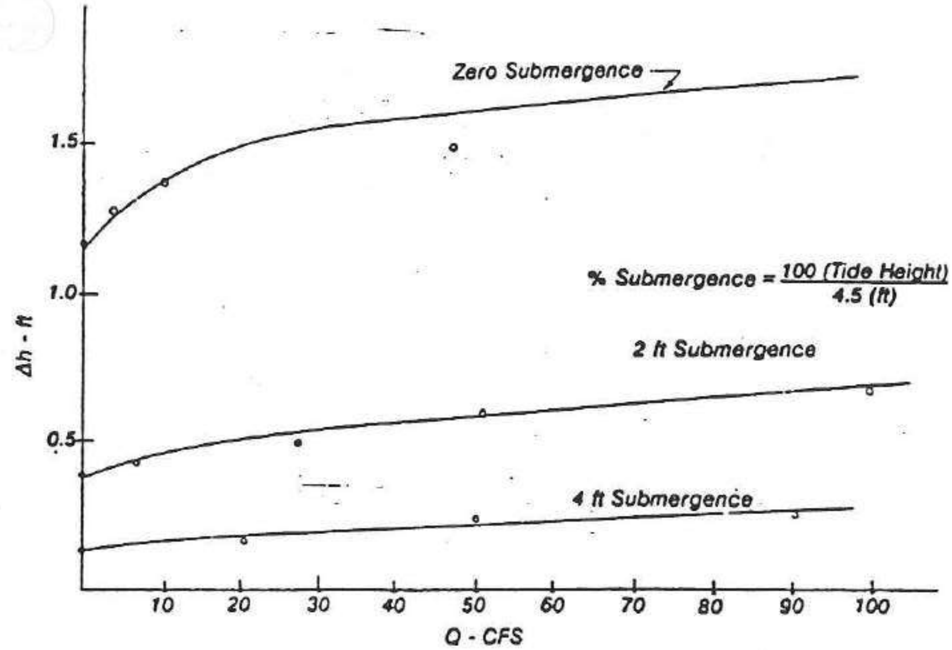


Figure 5. Estimated hydraulic performance of conventional tide gate.

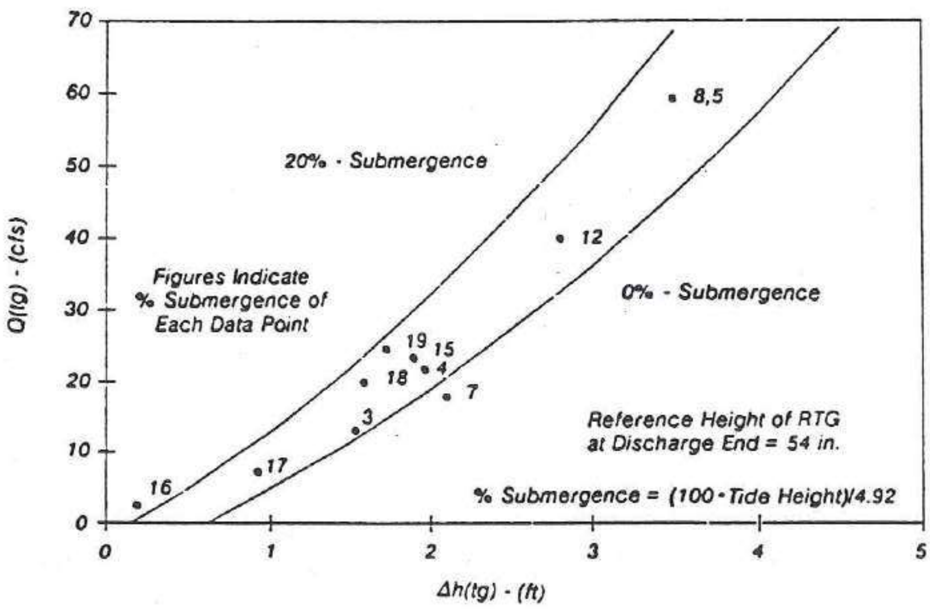


Figure 6. Estimated RTG flow characteristic.

to either tidal saltwater, wastewater constituents, or temperature fluctuation over) 18 months of observed operation. e RTG is expected to have a lifespan of 20 years or more, which is comparable to conventional tide gates. Smaller industrial installations of this type check valve are currently in operation up to 15 years.

There was no record of any backup flooding during storms or measurable tidal inflow when submerged at high tide during the observed operation. The maximum flow capacity through the RTG for any size tide gate is less than that for a flap-type gate. However, the maximum available RTG flow for this specific installation is estimated at 120 cfs, which, based on historical rainfall data, is adequate for the particular drainage area. Generically, a slight lessening of maximum outflow capacity does not cause any measurable decrease in the way of flood protection because the return storm frequency design concept is based on a stochastic phenomenon. Hydraulic comparisons between the RTG and conventional gates are developed in the final report. The release flow of the RTG starts at a lower differential hydraulic head when compared to a flap gate.

Debris caught in the RTG will cause tidal inflow to occur. however, no debris was discovered in the RTG during inspections. Inserted debris washed out without intervention by the maintenance crew and was measured to cause a relatively small inflow of 50 gpm during high tide.

A survey of municipal installations since 1984 indicates costs for RTGs are comparable to timber tide gates. Factors to consider in pricing an RTG versus a timber flap gate are equipment, installation and operation and maintenance needs for the specific location. For an equivalent area of about 25 square feet, hardware cost for flap gates averages $19,000 in New York City and $24,000 for RTGs in other municipal installations. Installation costs vary greatly, averaging about $9,000 for timber tide gates in NYC and ranging from $5,000 to $15,000 for RTGs.

Recommendations

Operational experience with the prototype rubber tide gate (RTG) indicated that some design modifications for the installation of the RTG are recommended as follows:

The RTG attachment to the adapter plate should be modified to provide a positive restraint against axial movement. The prototype installation in this project had a friction arrangement only which proved to be inadequate under heavy storm hydraulic loading and tidal action.

The adapter plate and all related hardware, should be made of stainless steel type 316 for corrosion resistance in the brackish water environment.

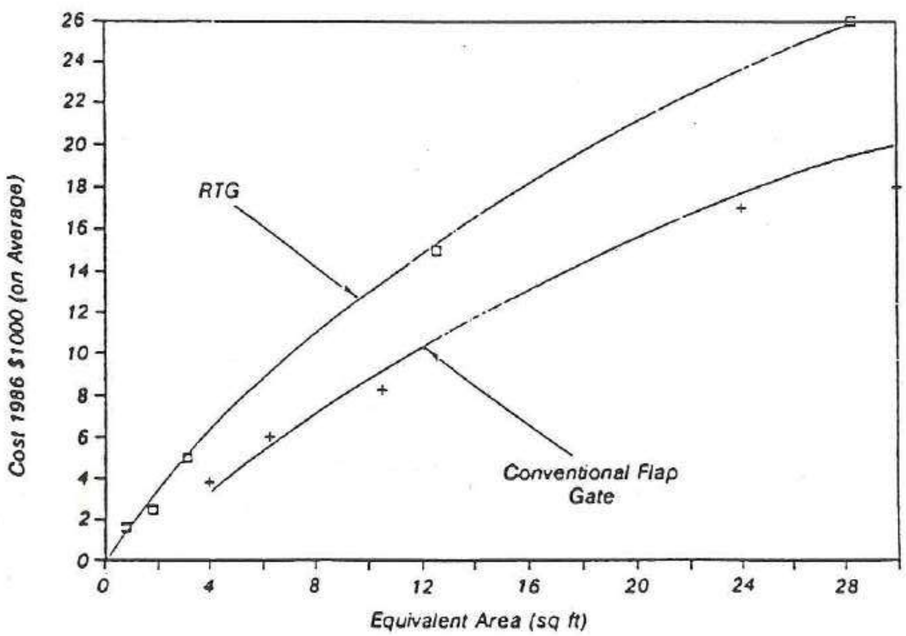
The RTG design should be modified for suspension near its discharge end to relieve cantilever loading on the mounting flange and adaptc - ile. Two larger units 84 in. and 72 in currently being fabricated by RV, will have holes through the top end of the lip to facilitate attachment to the tide gate chamber ceiling.

The liquid level upstream of the RTG decreases and flow capacity increases as the cross-sectional area of the RTG increases. Therefore to alleviate flooding (from an elevated upstream flow profile) during intense storms, it is important to maximize discharge area. A probable modification would be to make an oversized adaptor plate to accommodate the largest practicable and workable RTG.

The modifications to existing tide gate chambers should include provisions for overhead suspension of the RTG to facilitate installation and/or servicing since the weight of large sized units exceeds manual lifting capability when working in the confines of typical tide gate chambers.

It is recommended that the 54 in-diameter RTG at the current site remain in operation subject to routine O&M procedures. Observations should continue to monitor durability of material, reliability of performance, and consistency of low maintenance requirements over time.

Interested municipalities should continue to monitor NYCDEP's continuous



gure 7. *Comparison between costs of RTG and conventional flap gate.*

experience with this unit during which the RTG costs, maintenance requirements, and hydraulic performance will be more precisely established over time. Further developments of this technology should include:

- Establishing design criteria for new installations. New chambers would have cost-saving benefits such as (a) design for attachment which does not require an adaptor plate, (b) access manhole over discharge end which permits direct observation from street surface, (c) appropriately dimensioned access chimney and overhead suspension or trolley system as required for installation and removal of RTG.
- Establishing comparative costs between RTG retrofitting and repairing existing traditional flap gates. These costs would include savings from reduced surveillance and maintenance and savings in wastewater processing from reduced tidal inflow.
- Establishing protocol for repairs and maintenance. This would identify the type of damage the RTG might sustain, methods of patching and repair that are suitable, and type of training and tools required by maintenance crews servicing multiple installations.
- Establishing life expectancy of the rubber/neoprene in a sewer/outfall environment. This would involve some outfall materials testing investigations.

Peter A. Freeman is with Peter A. Freeman Associates Inc., Berlin, MD 21811; Angelika B. Forndran is with the New York City Department of Environmental Protection, Wards Island, NY 10035; and the EPA author ***Richard I. Field*** *(also the EPA Project Officer, see below) is with the Risk Reduction Engineering Laboratory, Edison, NJ 08837.*

The complete report, entitled "Development and Evaluation of a Rubber 'Duck Bill' Tide Gate," (Order No. PB 89-188 379/AS; Cost: $15.95, subject to change) will be available only from:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
Telephone: 703-487-4650

The EPA Project Officer can be contacted at:

Risk Reduction Engineering Laboratory—Cincinnati
U.S. Environmental Protection Agency
Edison, NJ 08837

United States
Environmental Protection
Agency

Center for Environmental Research
Information
Cincinnati OH 45268

BULK RATE
POSTAGE & FEES PAID
EPA
PERMIT No. G-35

Official Business
Penalty for Private Use $300

EPA/600/S2-89/020

APPENDIX D
DETAIL COSTING

CCTV INSPECTION

13-Feb-98

Description:

- 10 year program
- Sewer flushing
- Manhole inspection
- Priority areas: pipes >450 mm diam., basement flooding areas, trunk sewers and Interceptors inspected in the first 5 years.
- Balance of system inspected between 5 and 10 years

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	*1-5 years*				
	Priority areas	109,000	l.m.	$ 3.25	$ 354,250
	Sanitary/combined	167,000	l.m.	$ 3.25	$ 542,750
2.	*5-10 years*				
	Sanitary	200,000	l.m.	$ 3.25	$ 650,000
	Storm	232,000	l.m.	$ 3.25	$ 754,000
	SUBTOTAL				$ 2,301,000
	Plus 15 % Contigency				$ 345,150
	SUBTOTAL				$ 2,646,150
	G.S.T. @ 7%				$ 185,231
	TOTAL				$ 2,831,381

NEEBING/BRUNSWICK DIVERSION

13-Feb-98

Description:

- Divert flows from Neebing Interceptor to Brunswick Connector sewer
- Improve benching
- Install control gate
- temporary pumping required

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Cast in place manhole	1	each	5,000	$ 5,000
2.	Gate & control	1	each	3,500	$ 3,500
3.	Benching	1	lump sum	1,000	$ 1,000
4.	Temporary pumping	1	lump sum	2,500	$ 2,500
5.	Site Restoration	1	lump sum	5,000	$ 5,000
	SUBTOTAL				$ 17,000
	Plus 15 % Contigency				$ 2,550
	SUBTOTAL				$ 19,550
	G.S.T. @ 7%				$ 1,369
	TOTAL				$ 20,919

NEEBING/CAMERON DIVERSION

13-Feb-98

Description:

- Divert flows from Neebing Interceptor to Cameron sewer
- Improve benching
- Install control gate
- Connection sewer length

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Manhole	1	each	5,000	$ 5,000
2.	Gate & control	1	each	3,500	$ 3,500
3.	Benching	1	lump sum	1,000	$ 1,000
4.	Connection sewer - 450mm diam.	7	l.m.	500	$ 3,500
5.	Site Restoration	1	lump sum	5,000	$ 5,000
	SUBTOTAL				$ 18,000
	Plus 15 % Contigency				$ 2,700
	SUBTOTAL				$ 20,700
	G.S.T. @ 7%				$ 1,449
	TOTAL				$ 22,149

COST ESTIMATE SHEET

CATCHBASIN FLOW CONTROL - SEALING/RESTRICTIONS 13-Feb-98

Description:

- Seal CBs to allow flow to cascade to downstream CBs connected to storm sewers
- Install flow restrictions to limit inflow from CB to sanitary system
- Table 4.2 in Phase 2 report

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL	
1.	Seal CBs	10	each	500	$	5,000
2.	Flow restrictor	4	each	1,000	$	4,000
	SUBTOTAL				$	9,000
	Plus 15 % Contigency				$	1,350
	SUBTOTAL				$	10,350
	G.S.T. @ 7%				$	725
	TOTAL				$	11,075

COST ESTIMATE SHEET

CATCHBASIN FLOW CONTROL - NEW STORM SEWE

13-Feb-98

Description:

- Construct new storm sewer that will pick up 3 CBs
- Table 4.2 in Phase 2 report

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	300 mm storm pipe	290	l.m.	310	$ 89,900
2.	Reconnect 3 CBs	1	lump sum	1,500	$ 1,500
3.	Site Restoration	1	lump sum	5,000	$ 5,000

SUBTOTAL	$ 96,400
Plus 15 % Contigency	$ 14,460
SUBTOTAL	$ 110,860
G.S.T. @ 7%	$ 7,760
TOTAL	$ 118,620

OUTFALL GATE REPLACEMENT PROGRAM

13-Feb-98

Description:

- Replace outfall flap gates with "duck bill" check valves
- Provide high level relief
- 8 outfalls affected by levels in the Neebing River
- Cost relfect an installed unit
- Check valves can be installed internally, cost estimate reflect external units
- Larger units require support structure

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL	
1.	RN20 - 1375 mm	1	lump sum	99,400	$	99,400
2.	RN21 - 762x1016 mm	1	lump sum	6,160	$	6,160
3.	RN24 - 914x1219 mm	1	lump sum	11,620	$	11,620
4.	RN25 - 762x1016 mm	1	lump sum	6,160	$	6,160
5.	RN27 - 914 mm	1	lump sum	10,430	$	10,430
6.	RN28 - 381 mm	1	lump sum	4,060	$	4,060
7.	RN32 - 2134 mm	1	lump sum	99,400	$	99,400
8.	RN33 - 381 mm	1	lump sum	4,060	$	4,060
	SUBTOTAL				$	241,290
	Plus 15 % Contigency				$	36,194
	SUBTOTAL				$	277,484
	G.S.T. @ 7%				$	19,424
	TOTAL				$	296,907

JAMES & QUEBEC CONNECTION CORRECTION

13-Feb-98

Description:

- Sanitary system of Motel units are connected to local storm sewer
- Provide new service connection to each building
- New sanitary pipe required to outlet at Bailey Ave, and Montreal St.

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL	
1.	250 mm sanitary pipe	175	l.m.	250	$	43,750
2.	Manholes	3	each	3,000	$	9,000
3.	Connection at Bailey	1	lump sum	1,000	$	1,000
4.	Site restoration	1	lump sum	5,000	$	5,000
5.	Install 125 mm service connections	100	l.m.	150	$	15,000
6.	Building plumbing	1	lump sum	2,000	$	2,000
	SUBTOTAL				$	75,750
	Plus 15 % Contigency				$	11,363
	SUBTOTAL				$	87,113
	G.S.T. @ 7%				$	6,098
	TOTAL				$	93,210

FLOW MONITORING PROGRAM

13-Feb-98

Description:

- Additional and/or replacement velocity-area meters
- Up to 10 permanent monitoring stations telemetered (Table 4.4)
- SCADA system not included
- rain gauge

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL	
1.	Rain gauge and logger	1	each	1,800	$	1,800
2.	Velocity-Area meter (ISCO or equivalent)	2	each	8,000	$	16,000
3.	10 Depth Sensors installed for permanent stations	10	each	5,000	$	50,000
4.	Telemetering 10 stations	10	each	2,500	$	25,000
	SUBTOTAL				$	92,800
	Plus 15 % Contigency				$	13,920
	SUBTOTAL				$	106,720
	G.S.T. @ 7%				$	7,470
	TOTAL				$	114,190

CSO REGULATOR REPLACEMENT PROGRAM - Vorte 13-Feb-98

Description:

- Two types of regulator technologies, Vortex an Hydroslide.
- Regulators to be retrofitted into Kaministiquia regulators
- Installed cost
- Vortex designed around a range of possible flows
- Inserts can change to operating charateristics

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL	
1.	RK1 - 200IHV60/4	1	each	12,600	$	12,600
2.	RK2 - 150IHV60/4	1	each	10,500	$	10,500
3.	RK3 - 300IHV60/4	1	each	16,800	$	16,800
4.	RK4 - 150IHV60/4	1	each	10,500	$	10,500
5.	RK5 - 250IHV60/2.5	1	each	14,700	$	14,700
6.	RK6 - 150IHV60/4	1	each	10,500	$	10,500
7.	RK7 - 150IHV60/4	1	each	10,500	$	10,500
8.	RK8 - 150IHV60/4	1	each	10,500	$	10,500
9.	RK9 - 150IHV60/4	1	each	10,500	$	10,500
10.	RK10 - 300IHV60/4	1	each	16,800	$	16,800
11.	RK12 - 300IHV60/4	1	each	16,800	$	16,800
	SUBTOTAL				$	**140,700**
	Plus 15 % Contigency				$	21,105
	SUBTOTAL				$	161,805
	G.S.T. @ 7%				$	11,326
	TOTAL				$	173,131

COST ESTIMATE SHEET

CSO REGULATOR REPLACEMENT PROGRAM - Hydroslide 13-Feb-98

Description:

- Two types of regulator technologies, Vortex an Hydroslide.
- Regulators to be retrofitted into Kaministiquia regulators
- Installed cost
- Hydroslides are adjusted by changing float arm length

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL	
1.	RK1 - DR250	1	each	8,590	$	8,590
2.	RK2 - DR220	1	each	7,380	$	7,380
3.	RK3 - DR390	1	each	9,800	$	9,800
4.	RK4 - DR200N	1	each	6,160	$	6,160
5.	RK5 - DR200N	1	each	6,160	$	6,160
6.	RK6 - DR220	1	each	7,380	$	7,380
7.	RK7 - DR220	1	each	7,380	$	7,380
8.	RK8 - DR220	1	each	7,380	$	7,380
9.	RK9 - DR220	1	each	7,380	$	7,380
10.	RK10 - DR390	1	each	9,800	$	9,800
11.	RK12 - DR390	1	each	9,800	$	9,800
	SUBTOTAL				$	**87,210**
	Plus 15 % Contigency				$	13,082
	SUBTOTAL				$	100,292
	G.S.T. @ 7%				$	7,020
	TOTAL				$	107,312

GOLF LINKS - Alternative 1 13-Feb-98

Description:

- Extension of Golf Links to John St. and McVicars Creek
- Passes through River Terrace P.S. up to the Expressway

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	To John Street & Exp. 1200 mm pipe installed	4,985	l.m.	1,400	$ 6,979,000
2.	John to McVicars 1200 mm pipe installed including rock ex.	2,809	l.m.	1,400	$ 3,932,600
3.	Manholes	28	each	3,900	$ 109,200
4.	Road Restoration	1	lump sum	30,000	$ 30,000
5.	Connections	1	lump sum	20,000	$ 20,000

SUBTOTAL	**$ 11,070,800**
Plus 15 % Contigency	$ 1,660,620
SUBTOTAL	$ 12,731,420
G.S.T. @ 7%	$ 891,199
TOTAL	$ 13,622,619

COST ESTIMATE SHEET

GOLF LINKS - Alternative 2 13-Feb-98

Description:

- Extension of Golf Links to John St.@ Maple Ave.
- Replace John St. Trunk between Maple and Expressway

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	To John Street & Exp.				
	525 mm	1,000	l.m.	400	$ 400,000
	600 mm	720	l.m.	500	$ 360,000
	750 mm	2,785	l.m.	700	$ 1,949,500
	900 mm	580	l.m.	900	$ 522,000
3.	Manholes	30	each	3,500	$ 105,000
4.	Road Restoration	1	lump sum	20,000	$ 20,000
5.	Connections	1	lump sum	2,000	$ 2,000
	SUBTOTAL				**$ 3,358,500**
	Plus 15 % Contigency				$ 503,775
	SUBTOTAL				$ 3,862,275
	G.S.T. @ 7%				$ 270,359
	TOTAL				$ 4,132,634

GOLF LINKS - Alternative 3 13-Feb-98

Description:

- Golf Links to John Street & Expressway through River Terrance P.S. and Maple Ave.
- 8,760 m3 storage at the top of McVicars Creek

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	To John Street & Exp.				
	525 mm	1,000	l.m.	400	$ 400,000
	600 mm	720	l.m.	500	$ 360,000
	750 mm	2,785	l.m.	700	$ 1,949,500
	900 mm	580	l.m.	900	$ 522,000
3.	Manholes	30	each	3,500	$ 105,000
4.	Road Restoration	1	lump sum	20,000	$ 20,000
5.	Connections	1	lump sum	2,000	$ 2,000
6.	8,760 m3 Storage installed with hardware	8,760	cu.m.	500	$ 4,380,000

SUBTOTAL	$ 7,738,500
Plus 15 % Contigency	$ 1,160,775
SUBTOTAL	$ 8,899,275
G.S.T. @ 7%	$ 622,949
TOTAL	$ 9,522,224

COST ESTIMATE SHEET

GOLF LINKS - Alternative 4

13-Feb-98

Description:

- Golf Links to John Street through River Terrance P.S. and Maple to Exp.
- 8,760 m3 storage at the top of McVicars Creek
- Twin section of John St. trunk between Ontario and Algoma Streets

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	To John Street				
	525 mm	1,000	l.m.	400	$ 400,000
	600 mm	720	l.m.	500	$ 360,000
	750 mm	2,785	l.m.	700	$ 1,949,500
	900 mm	580	l.m.	900	$ 522,000
3.	Manholes	30	each	3,500	$ 105,000
4.	Road Restoration	1	lump sum	20,000	$ 20,000
5.	Connections	1	lump sum	2,000	$ 2,000
6.	8,760 m3 Storage installed with hardware	8,760	cu.m.	500	$ 4,380,000
7.	Twin 400 m section of John St. Trunk 1,300 mm diam.	400	l.m.	1,500	$ 600,000
	SUBTOTAL				$ 8,338,500
	Plus 15 % Contigency				$ 1,250,775
	SUBTOTAL				$ 9,589,275
	G.S.T. @ 7%				$ 671,249
	TOTAL				$ 10,260,524

2 YEAR BASEMENT FLOODING - STORAGE & SEPARATION

21-Jun-99

Description:

- Local storage
- Storage as local tank or parallel pipe
- Table 4.9

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Area 1	20	cu.m.	625	$ 12,500
2.	Area 8	290	cu.m.	625	$ 181,250
3.	Area 9	20	cu.m.	625	$ 12,500
4.	Area 10 - Separation				$ 127,385
	SUBTOTAL				$ 333,635
	Plus 15 % Contingency				$ 50,045
	SUBTOTAL				$ 383,680
	G.S.T. @ 7%				$ 26,858
	TOTAL				$ 410,538

5 YEAR BASEMENT FLOODING - STORAGE & SEPARATION

21-Jun-99

Description:

- Local storage
- Storage as local tank or parallel pipe
- Table 4.9

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Area 1	120	cu.m.	625	$ 75,000
2.	Area 4	470	cu.m.	625	$ 293,750
3.	Area 5	500	cu.m.	625	$ 312,500
4.	Area 8	750	cu.m.	625	$ 468,750
5.	Area 9 - Separation				$ 149,330
6.	Area 10 - Separation				$ 134,365
7.	Area 12 - Separation				$ 51,875
8.	Area 14	70	cu.m.	625	$ 43,750
9.	Area 17	320	cu.m.	625	$ 200,000
	SUBTOTAL				$ 1,729,320
	Plus 15 % Contingency				$ 259,398
	SUBTOTAL				$ 1,988,718
	G.S.T. @ 7%				$ 139,210
	TOTAL				$ 2,127,928

10 YEAR BASEMENT FLOODING - STORAGE & SEPARATION 21-Jun-99

Description:
- Local storage
- Storage as local tank or parallel pipe
- Table 4.9

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Area 1	270	cu.m.	625	$ 168,750
2.	Area 3	150	cu.m.	625	$ 93,750
3.	Area 4	720	cu.m.	625	$ 450,000
4.	Area 5	1,210	cu.m.	625	$ 756,250
5.	Area 6	130	cu.m.	625	$ 81,250
6.	Area 8	1,150	cu.m.	625	$ 718,750
7.	Area 9 - Separation				$ 227,930
8.	Area 10 - Separation				$ 137,785
9.	Area 12 - Separation				$ 51,875
10.	Area 14	150	cu.m.	625	$ 93,750
11.	Area 17	600	cu.m.	625	$ 375,000
	SUBTOTAL				$ 3,155,090
	Plus 15 % Contingency				$ 473,264
	SUBTOTAL				$ 3,628,354
	G.S.T. @ 7%				$ 253,985
	TOTAL				$ 3,882,338

2 YEAR BASEMENT FLOODING - STORAGE

21-Jun-99

Description:

- Local storage
- Storage as local tank or parallel pipe
- Table 4.9

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Area 1	20	cu.m.	625	$ 12,500
2.	Area 8	290	cu.m.	625	$ 181,250
3.	Area 9	20	cu.m.	625	$ 12,500
4.	Area 10	110	cu.m.	625	$ 68,750

SUBTOTAL	$ 275,000
Plus 15 % Contingency	$ 41,250
SUBTOTAL	$ 316,250
G.S.T. @ 7%	$ 22,138
TOTAL	$ 338,388

5 YEAR BASEMENT FLOODING - STORAGE

21-Jun-99

Description:

- Local storage
- Storage as local tank or parallel pipe
- Table 4.9

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Area 1	120	cu.m.	625	$ 75,000
2.	Area 4	470	cu.m.	625	$ 293,750
3.	Area 5	500	cu.m.	625	$ 312,500
4.	Area 8	750	cu.m.	625	$ 468,750
5.	Area 9	270	cu.m.	625	$ 168,750
6.	Area 10	190	cu.m.	625	$ 118,750
7.	Area 12	70	cu.m.	625	$ 43,750
8.	Area 14	70	cu.m.	625	$ 43,750
9.	Area 17	320	cu.m.	625	$ 200,000
	SUBTOTAL				$ 1,725,000
	Plus 15 % Contingency				$ 258,750
	SUBTOTAL				$ 1,983,750
	G.S.T. @ 7%				$ 138,863
	TOTAL				$ 2,122,613

10 YEAR BASEMENT FLOODING - STORAGE

21-Jun-99

Description:

- Local storage
- Storage as local tank or parallel pipe
- Table 4.9

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Area 1	270	cu.m.	625	$ 168,750
2.	Area 3	150	cu.m.	625	$ 93,750
3.	Area 4	720	cu.m.	625	$ 450,000
4.	Area 5	1,210	cu.m.	625	$ 756,250
5.	Area 6	130	cu.m.	625	$ 81,250
6.	Area 8	1,150	cu.m.	625	$ 718,750
7.	Area 9	700	cu.m.	625	$ 437,500
8.	Area 10	300	cu.m.	625	$ 187,500
9.	Area 12	160	cu.m.	625	$ 100,000
10.	Area 14	150	cu.m.	625	$ 93,750
11.	Area 17	600	cu.m.	625	$ 375,000
	SUBTOTAL				$ 3,462,500
	Plus 15 % Contingency				$ 519,375
	SUBTOTAL				$ 3,981,875
	G.S.T. @ 7%				$ 278,731
	TOTAL				$ 4,260,606

COST ESTIMATE SHEET

KAM INTERCEPTOR IMPROVEMENTS

13-Feb-98

Description:

- Replace 750 mm pipe with 1670 mm diam. pipe
- Main pump station improvements would be part of WPCP upgrade
- tunnel section

ITEM No.	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	TOTAL
1.	Tunnel and install 1670 mm pipe	215	l.m.	5,000	$ 1,075,000
2.	Access shaft	1	lump sum	50,000	$ 50,000
2.	Connections	2	each	2,000	$ 4,000
3.	Temporary pumping	1	lump sum	3,000	$ 3,000
	SUBTOTAL				$ 1,132,000
	Plus 15 % Contigency				$ 169,800
	SUBTOTAL				$ 1,301,800
	G.S.T. @ 7%				$ 91,126
	TOTAL				$ 1,392,926

APPENDIX E
XP-SWMM MODEL DATA

Regulator Data

Location	Regulator Node ID	Type	Pipe Diameter (m)	Cross-Sect Area (m2)	Pipe Depth (m)	Discharge Coefficient	Manning's Roughness n	Invert Elevation at Upstream (m)	Invert Elevation at Downstream (m)
Syndicate & Southern	RN25	Circ Side	0.24	0.05	0.00	0.6	0.0033	183.6050	183.6019
Gore & Stanley	RK12	Circ Sump	0.38	0.11	0.00	0.6	0.0045	185.6552	185.6522
Tarbutt and Frederica	RK10	Circ Side	0.45	0.16	0.45	0.6	0.0051	183.2120	183.2090
Syndicate & Christina	RK9	Rect Side	0.13	0.02	0.13	0.6	0.0023	183.5700	183.5670
Syndicate & Empire	RK8	Rect Sum	0.13	0.02	0.13	0.6	0.0023	183.6581	183.6550
Syndicate & Walsh	RK7	Rect Side	0.13	0.02	0.13	0.6	0.0023	183.5000	183.4969
Syndicate & Duncan	RK6	Rect Side	0.13	0.02	0.13	0.6	0.0023	185.0000	184.9969
Hardisty & Ridgeway	RK5	Rect Side	0.19	0.04	0.19	0.6	0.0029	183.1600	183.1570
Hardisty & May	RK4	Rect Side	0.10	0.01	0.10	0.6	0.0019	183.0380	183.0349
Hardisty & Viscount	RK3	Rect Side	0.10	0.01	0.10	0.6	0.0019	181.4700	181.4669
Hardisty & Victoria	RK2	Rect Side	0.14	0.02	0.14	0.6	0.0025	182.7100	182.7070
Marks & Neebing River	RN28	Circ Side	0.24	0.05	0.00	0.6	0.0033	181.8700	181.8669
Brunswick & Cumming	RN33	Circ Side	0.23	0.04	0.00	0.6	0.0033	183.2200	183.2169
Hardisty & Dease	RK1	Rect Side	0.30	0.11	0.30	0.2	0.0123	183.0600	183.0569
Vickers & Finlayson	RN27	Circ Side	0.24	0.05	0.00	0.6	0.0033	182.7360	182.7329
May & Southern	RN21	Circ Side	0.24	0.05	0.00	0.6	0.0033	182.9730	182.9700
Brodie & Southern	RN24	Circ Side	0.24	0.05	0.00	0.6	0.0033	182.9560	182.9529
Simpson & Neebing River	RN20	Circ Side	0.24	0.05	0.00	0.6	0.0033	183.0650	183.0620

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
KAMINISTIQUIA INTERCEPTOR							
	KAM23	KAMMH13	RRK12	Circular	396	1.68	Gore St. and Stanley Avenue
	577	D-RK12	RK12	Circular	100	1.68	Gore St. and Stanley Avenue
	320	RK12	RK11OVF	Circular	100	1.52	RK12 Overflow
	KAM22	RRK12	KAMMH11	Circular	186	1.68	Gore St. between Stanley Ave and James St.
	KAM21	KAMMH11	RRK10	Circular	2303	1.68	Gore St. between James Street and Tarbutt Street
	312	RK10	RK10OVF	Circular	107	2.13	Overflow sewer for Regulator RK10
	KAM20	RRK10	KAMMH2	Circular	596	1.68	Gore St. between James and Tarbutt St.
	KAM19	KAMMH2	RRK9	Circular	646	1.68	Along Kaministikwia R. between Tarbutt and Sprague
	1468	O13	O10	Circular	126	0.53	Francis Street and Syndicate Avenue
	1470	O10	O6	Circular	135	0.53	Brock Street and Syndicate Avenue
	1474	O15	O14	Circular	230	0.31	Sprague Street and Francis
	1475	O14	O13	Circular	191	0.38	Sprague Street and Francis
	1478	O12	O11	Circular	194	0.31	Sprague Street and Brock Street
	1479	O11	O10	Circular	275	0.38	Sprague Street and Brock Street
	1482	O9	O8	Circular	183	0.31	Sprague Street and Mary Street
	1484	O8	O7	Circular	173	0.38	Sprague Street and Mary Street
	1485	O7	O6	Circular	163	0.38	Sprague Street and Mary Street
	1471	O6	O2	Circular	138	0.53	Mary Street and Syndicate Avenue
	1490	O5	O4	Circular	40	0.31	Sprague Street and Christina Street
	1492	O4	O3	Circular	116	0.31	Sprague Street and Christina Street
	1493	O3	O2	Circular	183	0.38	Sprague Street and Christina Street
	301	O2	RK9	Circular	183	0.38	Christina Street and Syndicate Avenue
	304	RK9	RK9OVF	Rectangle	100	0.76x0.56	Overflow sewer for Regulator RK9
	KAM18	RRK9	RRK8	Circular	124	1.68	along Kaministikwia R. between Sprague and Christina
	1514	N131	N121	Circular	66	0.31	Bessie Avenue and Empire Avenue
	1515	N121	N111	Circular	66	0.31	Bessie Avenue and Empire Avenue
	1517	N141	N111	Circular	37	0.31	Bessie Avenue and Empire Avenue
	1511	N111	N101	Circular	102	0.31	Bessie Avenue and Sprague Street
	1509	N101	N71	Circular	98	0.31	Bessie Avenue and Sprague Street
	1506	N91	N81	Circular	120	0.31	Sprague Street and Mary Street
	1507	N81	N71	Circular	126	0.31	Sprague Street and Christina Street
	1503	N71	N51	Circular	164	0.38	Sprague Street and Empire Avenue

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	2000	N61	N51	Circular	202	0.31	Selkirk Street and Empire Avenue
	1499	N51	N31	Circular	98	0.38	Franklin Street and Empire Avenue
	1497	N41	N31	Circular	201	0.31	Harkness Street and Empire Avenue
	1495	N31	D-RK8	Circular	197	0.38	Norah Street and Empire Avenue
	294	D-RK8	RK8	Circular	172	0.38	Norah Street and Empire Avenue
	296	RK8	RK8OVF	Circular	100	0.38	Overflow sewer for Regulator RK8
	KAM17	RRK8	KAMNODE2	Circular	21	1.68	Syndicate Ave. between Christina and Empire Ave.
	KAM16	KAMNODE2	KAMNODE1	Circular	149	1.27	Syndicate Ave. between Empire and Walsh St.
	KAM15	KAMNODE1	RRK7	Circular	182	1.68	Syndicate Ave. between Empire and Walsh St.
	281	D-RK7	RK7	Rectangle	100	0.91x0.66	Syndicate Avenue and Vickers Street
	283	RK7	RK7OVF	Rectangle	100	0.91x0.66	Overflow sewer for Regulator RK7
	KAM14	RRK7	RRK6	Circular	334	1.68	Syndicate Ave. between Empire and Walsh St.
	273	D-RK6	RK6	Circular	200	0.3	Ridgeway Street and May Street
	275	RK6	RK6OVF	Circular	11	0.31	Overflow sewer for Regulator RK6
	KAM13	RRK6	RRK5	Circular	399	1.68	Syndicate Ave. between Walsh and Duncan St.
	264	D-RK5	RK5	Rectangle	100	0.91x0.76	Ridgeway Street and Brodie Street
	267	RK5	RK5OVF	Rectangle	100	0.91x0.76	Overflow sewer for Regulator RK5
	KAM12	RRK5	RRK4	Circular	75	1.37	Syndicate Ave. between Duncan and Brodie St.
	257	D-RK4	RK4	Rectangle	100	0.96x0.76	Ridgeway Street and May Street
	259	RK4	RK4OVF	Rectangle	100	0.96x0.76	Overflow sewer for Regulator RK4
	KAM11	RRK4	RRK3	Circular	321	1.37	Hardisty St. between Brodie and May St.
	248	D-RK3	RK3	Rectangle	100	1.47x0.91	Donald Street and Simpson Street
	251	RK3	RK3OVF	Rectangle	100	1.47x0.91	Overflow sewer for Regulator RK3
	KAM10	RRK3	RRK2	Circular	220	1.37	Hardisty St. between May and Miscount St.
	1558	H7	h7a	Circular	171	0.31	Leigh Street south of Simpson Street
	1559	h7a	h3	Circular	68	0.51	Leigh Street south of Simpson Street
	1556	H9	H7	Circular	117	0.31	Leigh Street and May Street
	1560	H7	h5	Circular	110	0.31	Leigh Street south of Simpson Street
	1550	h6	h5	Circular	116	0.31	Cumming Street south of Simpson Street
	1552	h5	h5a	Circular	157	0.31	Cumming Street south of Simpson Street
	1553	h5a	h4	Circular	14	0.61	Cumming Street and Simpson Street
	1543	h4	h3	Circular	96	0.71	Leigh Street and Simpson Street
	1546	h3	h2	Circular	96	0.71	Miles Street and Simpson Street
	1547	h2	D-RK2	Circular	91	0.71	Victoria Avenue and Hardisty Avenue
	237	D-RK2	RK2	Circular	85	0.76	Victoria Avenue and Simpson Street

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	239	RK2	RK2OVF	Circular	100	0.91	Overflow Sewer for Regulator No. RK2
	KAM9	RRK2	RRK1	Circular	575	1.37	Hardisty St. between Victoria and Dease St.
	1527	R12	R11	Circular	96	0.56	Finlayson Street and Simpson Street
	1528	R11	D-RK1	Circular	110	0.71	Dease Street and Simpson Street
	1532	R10	R9	Circular	163	0.31	Dease Street and McMurray Street
	1530	R9	D-RK1	Circular	169	0.51	Dease Street and Simpson Street
	1540	R16	R15	Circular	114	0.31	Cameron Street and May Street
	1538	R15	R14	Circular	134	0.38	Cameron Street and McKenzie Street
	1536	R14	R13	Circular	168	0.51	Cameron Street and McKenzie Street
	1534	R13	D-RK1	Circular	96	0.71	Simpson Street and Dease Street
	231	D-RK1	RK1	Circular	92	0.76	Dease Street and Simpson Street
	k1ovf	RK1	RK1OVF	Circular	100	0.91	Overflow Sewer for Regulator No. RK1
	KAM8	RRK1	KAMMH1	Circular	6	1.68	Hardisty St. between Dease and Heron St.
	KAM7	KAMMH1	KAMMH28A	Circular	476	1.68	Hardisty St. between Heron and Robertson St.
	KAM6	KAMMH28A	KAMMH29	Circular	108	1.68	Hardisty St. between Robertson and Rowand St.
	KAM5	KAMMH29	KAMMH33	Circular	409	1.68	Hardisty St. between Rowand and Pacific Ave.
	KAM4	KAMMH33	KAMMH31	Circular	88	1.68	Pacific Ave. between Hardisty and McLaughlin St.
	KAM3	KAMMH31	KAMMH32	Circular	282	1.68	McLaughlin St. between Pacific and Atlantic Ave
	KAM2	KAMMH32	219	Circular	138	1.68	Atlantic Ave. between McLaughlin and McBain St.
	KAM1	219	tbwwtp	Circular	14	1.68	McBain and Atlantic to WWTP
	KAM01	tbwwtp	k-am	Circular	149	1.67	WWTP
	KAM02	k-am	WMMH1	Circular	54	1.67	to Treatment Plant Pump Station
	TO-PUMP	WMMH1	prepump	Circular	8	2.13	Pump Station
	STP-OUT	pump-o	KAM	Circular	3425	2.74	Treatment plant bypass
CAMERON INTERCEPTOR							
	CAM9	CAMMH5	CAMMH4A	Circular	95	1.05	Cameron St. between Marks St. and Archibald St.
	CAM8	CAMMH4A	CAMMH3A	Circular	24	1.05	Cameron St. between Archibald and Brodie St.
	CAM7	CAMMH3A	CAMMH2B	Circular			Cameron St. between Brodie and May St.
	344	CAM-SYND	CAMMH1	Circular	6	0.45	Syndicate Street and Cumming Street
	CAM6	CAMMH2B	CAMMH1	Circular	353	1.07	Cameron St. between May and Mckenzie St.
	CAM5	CAMMH1	CAMMH2A	Circular	89	1.07	Cameron St. between May and Mckenzie St.
	338	CAM-BRODI	CAMMH2A	Circular	5	0.45	Brodie Street and Cumming Street
	CAM4	CAMMH2A	CAMMH3	Circular	70	1.07	Mckenzie St. between Cameron and Dease St.

	SWIM	U/S NODE	D/S NODE	PIPE	Length	DIAM.	LOCATION
	ID No.	ID	ID	TYPE	(m)	(m)	
	332	CAM-MAY	CAMMH3	Circular	14	0.45	May Street and Cumming
	CAM3	CAMMH3	CAMMH4	Circular	412	1.07	Dease between Mckenzie and Simpson St.
	CAM2	CAMMH4	CAMMH2	Circular	184	1.07	Dease St. between Simpson and Hardisty
	CAM1	CAMMH2	KAMMH1	Circular	64	1.07	Dease St. between Simpson and Hardisty
NEEBING McINTYRE							
	WM7	WMMH7	WMMH6	Circular	41	0.91	William St. between Ford St. and Neeb-McInt Floodway
	WM6	WMMH6	WMMH5	Circular	316	1.68	end of William St. to end of Franklin St.
	WM5	WMMH5	WMMH4	Circular	517	1.68	End of Franklin St. to Forest and Syndicate Ave
	WM4	WMMH4	WMMH3	Circular	1170	2.13	William St. between Syndicate ave. and Northern ave.
	WM3	WMMH3	www	Circular	937	2.13	William St. between Northern Ave and near Alberta St.
	WM2	www	WMMH1	Circular	595	2.13	From Alberta and William to near Sewage treatment Pl.
NEEING INTERCEPTOR							
	N23	NMH23	MN22	Circular	284	0.38	Neebing R. and Tarbutt St. to Brunswick Connector
	N22	NMH22	NMH21	Circular	121	0.38	Brunswick Connector to Cumming St and Brunswick St.
	N21	NMH21	NMH20	Circular	101	0.46	Cumming St. between Brunswick and Wellington St.
	N20	NMH20	NMH19	Circular	215	0.46	Cumming St. between Wellington and Franklin St.
	606	D-RN33	RN33	Circular	100	0.38	Dease Street and Franklin Street
	609	RN33	RN33-O	Circular	55	0.38	RN33 Overflow
	N19	NMH19	NMH18	Circular	97	0.53	Cumming at Franklin to Norah St.
	K28	K28-S	K27-S	Circular	205	0.31	Hamilton St. and Tarbutt St.
	K27	K27-S	K26-S	Circular	91	0.31	Hamilton Ave. and Hyde Park Ave.
	K26	K26-S	K25-S	Circular	196	0.38	Hyde Park Ave. and Moodie Street
	K25	K25-S	K24-S	Circular	206	0.46	Moodie Street
	K24	K24-S	K21-S	Circular	90	0.91	Selkirk Street and Moodie Street
	K21	K21-S	K18-S	Circular	91	0.91	Murray Avenue and Selkirk Street
	K18	K18-S	K15-S	Circular	91	1.22	Isabella Street and Selkirk Street
	K15	K15-S	K10-S	Circular	91	1.22	Ridgeway Street and Selkirk Street
	K10	K10-S	K7-S	Circular	181	1.45	Selkirk Street
	K7	K7-S	K5-S	Circular	91	1.45	Arthur Street and Selkirk Street
	K5	K5-S	K3A-S	Circular	91	1.45	Sills Street and Selkirk Street
	K3A	K3A-S	K3B-S	Circular	91	1.83	Donald Street and Selkirk Street
	K4	K4-S	K3B-S	Circular	97	0.31	Rankin Street and Brunswick Street

	SWIM	U/S NODE	D/S NODE	PIPE	Length	DIAM.	LOCATION
	ID No.	ID	ID	TYPE	(m)	(m)	
	K3B	K3B-S	K3-S	Circular	91	1.83	Rankin Street and Selkirk Street
	K3	K3-S	K2-S	Circular	218	1.98	Victoria Street and Selkirk Street
	K2	K2-S	RN30	Circular	229	1.98	Wellington Street and Selkirk Street
	RN30-O	RN30	NMH19	Circular	10	0.2	Curmming Street and Wellington Street
	468	RN30	RN32-O	Rectangle	31	1.98x1.24	Overflow for RN30
	N18	NMH18	NMH17	Circular	212	0.53	Along Norah St. to Cameron at Marks St.
	N17	NMH17	NMH16	Circular	28	0.61	Along Marks St. West of Cameron
	N16	NMH16	NMH15	Circular	137	0.61	Along Neebing R. between Marks and Harold St.
	624	D-RN28	RN28	Circular	100	0.38	Finlayson Street and Neebing River
	452	RN28	RN28-O	Circular	24	0.38	RN28 Overflow
	N15	NMH15	NMH14	Circular	209	0.61	Along Neebing R. between Harold St. and McKellar
	1261	G4	G3	Circular	110	0.76	Leith Street and Archibald Street
	1259	G3	G2	Circular	98	0.76	Cameron Street and Archibald Street
	1257	G2	G2A	Circular	82	0.91	Dease Street and Archibald Street
	1255	G2A	D-RN27	Circular	122	0.91	Dease Street and Vickers Street
	587	D-RN27	RN27	Circular	18	0.61	Vickers Street west of Dease Street
	445	RN27	RN27-O	Circular	87	0.38	RN27 Overflow
	1233	D-RN26	NMH13	Circular	90	0.3	Finlayson Street and McKellar Street
	N14	NMH14	NMH13	Circular	290	0.61	between McKellar at Pruden St. and West of Finlay Sq.
	N13	NMH13	NMH12	Circular	90	0.61	Along Neebing from West of Finlay Sq. to Syndicate
	589	D-RN25	RN25	Rectangle	100	0.76x0.51	Finlayson Street and Syndicate Avenue
	433	RN25	RN25-O	Rectangle	52	0.76x0.36	RN25 Overflow
	N12	NMH12	NMH11	Circular	120	0.69	Southern Ave between Syndicate and Brodie St.
	1331	F11	F10	Circular	104	0.76	May Street and Dease Street
	1329	F10	F31	Circular	91	0.76	Brodie Street and Dease Street
	1327	F4	F31	Circular	95	0.74	Brodie Street and Cameron Street
	1323	F31	D-RN24	Circular	20	0.74	Brodie Street between Cameron Street and Finlayson Street
	1325	F8	F7	Circular	88	0.67	Syndicate Avenue between Cameron Street and Dease Street
	1317	F7	D-RN24	Circular	73	0.91	Brodie Street between Cameron Street and Southern Avenue
	597	D-RN24	RN24	Rectangle	202	1.17x0.86	Brodie Street and Southern Avenue
	421	RN24	RN24-O	Rectangle	67	1.22x0.86	RN 24 Overflow
	N11	NMH11	NMH10	Circular	76	0.69	Southern Ave between Brodie and May St.
	1280	S41	s3	Circular	252	0.31	Syndicate Avenue west of Durban Street
	1281	s3	S2	Circular	239	0.31	Syndicate Avenue west of Durban Street
	1287	S7	S5	Circular	122	0.38	Pacific Avenue between Vickers Street and McKellar Street

	SWIM	U/S NODE	D/S NODE	PIPE	Length	DIAM.	LOCATION
	ID No.	ID	ID	TYPE	(m)	(m)	
	1285	S6	S5	Circular	240	0.31	Pacific Avenue between McKellar Street and Syndicate Avenu
	1283	S5	S2	Circular	110	0.38	Northern Avenue between Syndicate Avenue and Brodie Stre
	1277	S2	7B-1	Circular	104	0.38	Brodie Street and Southern Avenue across Neebing River
	1231	7B-1	NMH10	Circular	125	0.38	Brodie Street and Southern Avenue across Neebing River
	N10	NMH10	NMH9	Circular	37	0.69	Southern Ave between Brodie and May St.
	593	D-RN21	RN21	Rectangle	100	0.79x0.51	May Street and Pacific Avenue
	413	RN21	RN21-O	Rectangle	63	0.79x0.51	RN21 Overflow
	N9	NMH9	NMH8	Circular	98	0.69	Southern Ave between May and Wiley St.
	1275	E3	D-RN22	Circular	235	0.2	May Street west of Durban Street
	591	D-RN22	RN22	Circular	114	0.2	May Street west of Durban Street
	RN22-O	RN22	NMH8	Circular	16	0.2	May Street at Southern Avenue
	N8	NMH8	NMH7	Circular	160	0.69	Southern Ave between Wiley and Prince Arthur Blvd
	N7	NMH7	NMH6	Circular	146	0.76	Southern Ave between Arthur and McMillan St.
	1289	Q8	7B-2	Circular	219	0.25	Fairgrounds west of Northern Avenue
	1295	Q5	Q4	Circular	92	0.25	Fairgrounds west of Northern Avenue
	1293	Q4	Q3	Circular	92	0.25	Fairgrounds west of Northern Avenue
	.1291	Q3	7B-2	Circular	95	0.25	Fairgrounds west of Northern Avenue
	1299	Q7	Q6	Circular	167	0.25	Fairgrounds west of Northern Avenue
	1297	Q6	7B-2	Circular	142	0.25	Fairgrounds west of Northern Avenue
	1215	7B-2	NMH6	Circular	130	0.25	Northern Avenue to Southern Avenue under Neebing River
	N6	NMH6	NMH5	Circular	183	0.76	Southern Ave between McMillan St. and Minnesota St.
	1302	D3	D2	Circular	168	0.31	McKenzie Street and Robertson Street
	1309	D5	D4	Circular	168	0.31	Ogden Street and Prince Arthur Blvd.
	1311	D6	D4	Circular	168	0.31	Ogden Street and McMurray Street
	1307	D4	D2	Circular	99	0.38	McMurray Street from Ogden to Robertson
	1303	D2	7A	Circular	141	0.38	Robertson Street and McMilan Street
	1223	7A	NMH5	Circular	252	0.38	McMilan Street and Southern Avnue
	N5	NMH5	NMH4	Circular	64	0.76	Minnesota at Southern to just west of Southern
	1334	B2	D-RN20	Circular	177	1.37	Simpson Avenue and Atlantic Avenue
	595	D-RN20	RN20	Circular	95	1.37	Simpson Avenue and Southern Avenue
	389	RN20	RN1OVF	Rectangle	49	1.37x1.12	RN20 Overflow
	N4	NMH4	NMH3	Circular	200	0.76	west of Southern, crossing Simpson St to William St.
	1346	C4	C3	Circular	157	0.2	Pacific Avenue and Alberta Street
	1344	C3	C2	Circular	34	0.2	Atlantic Avenue at Alberta Street
	1342	C2	C1	Circular	159	0.25	Alberta Avenue and Southern Avenue

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	1340	C6	C5	Circular	147	0.25	Alexandra Avenue and Atlantic Avenue
	1338	C5	C1	Circular	86	0.25	Southern Avenue from Alexandra Avenue to Alberta Street
	1336	C1	NMH2	Circular	163	0.31	Southern Avenue to Neebing Interceptor
	N3	NMH3	NMH2	Circular	127	0.76	Along Neebing-McIntyre between Simpson & Alexandra
	ATH1	NMH2	www	Circular	18	0.76	Along Neebing-McIntyre between Alexandra & Athabaska
BRUNSWICK CONNECTOR SEWER							
	BC9	BCMH9	BCMH8	Circular	40	1.22	Connection to Neebing Interceptor
	BC8	BCMH8	BCMH7	Circular	109	1.22	West of Neebing Interceptor
	BC7	BCMH7	BCMH6	Circular	33	1.22	Crossing Neebing River
	BC6	BCMH6	BCMH5	Circular	305	2.16	Between Neebing R and Neebing-McIntyre Floodway
	BC5	BCMH5	BCMH4	Circular	169	1.53	Between Neebing R and Neebing-McIntyre Floodway
	BC4	BCMH4	BCMH3	Circular	26	1.22	Between Neebing R and Neebing-McIntyre Floodway
	BC3	BCMH3	BCMH2	Circular	80	1.52	Between Neebing R and Neebing-McIntyre Floodway
	BC2	BCMH2	BCMH1	Circular	274	1.52	Between Neebing R and Neebing-McIntyre Floodway
	BC1	BCMH1	WMMH4	Circular	150	2.16	Between Neebing R and Neebing-McIntyre Floodway
DEASE TRUNK SEWER							
	DT4	DTMH4	DTMH3	Circular	566	0.61	West of Neebing River from Ford St. to Brunswick Conn.
	DT3	DTMH3	DTMH2	Circular	107	0.61	West of Neebing River from Ford St. to Brunswick Conn.
	DT2	DTMH2	DTMH1	Circular	6	0.76	West of Neebing River from Ford St. to Brunswick Conn.
	DT1	DTMH1	BCMH7	Circular	25	0.76	West of Neebing River from Ford St. to Brunswick Conn.
FORD DRIVE TRUNK SEWER							
	F3	FMH3	FMH2	Circular	479	0.91	Along Ford Dr. from Redwood to William St.
	F2	FMH2	FMH1	Circular	298	0.91	Along Ford Dr. from Redwood to William St.
	F1	FMH1	WMMH6	Circular	8	0.91	Along Ford Dr. from Redwood to William St.
McVICARS CREEK TRUNK SEWER							
	201n	401n	402n	Circular	91	0.76	McVicars Creek and Thunder Bay Expressway
	202n	402n	404n	Circular	211	0.76	McVicars Creek and Hinton Ave
	204n	404n	406n	Circular	215	0.76	McVicars Creek and Harrison St.
	206n	406n	407n	Circular	112	0.76	McVicars Creek and Hogarth St.
	207n	407n	408n	Circular	189	0.76	McVicars Creek and Rockwood Ave.

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	208n	408n	409n	Circular	105	0.76	McVicars Creek and Madeline St.
	209n	409n	410n	Circular	181	0.76	McVicars Creek and Theresa St.
	210n	410n	412n	Circular	153	0.76	McVicars Creek and Bruce St.
	212n	412n	413n	Circular	92	0.76	McVicars Creek and Brent St.
	213n	413n	415n	Circular	121	0.76	McVicars Creek and High St.
	215n	415n	417n	Circular	138	0.76	McVicars Creek and Bryan St.
	217n	417n	418n	Circular	92	0.76	McVicars Creek and Hanley St.
	218n	418n	419n	Circular	97	0.76	McVicars Creek and Glayte (primrose?) St.
	219n	419n	421n	Circular	140	0.76	Glayte St. and Balsam St.
	221n	421n	422n	Circular	83	0.76	Glayte St. and College (near Sep School)
	222n	422n	423n	Circular	85	0.76	Glayte St. and Hartvirsen St.
	223n	423n	424n	Circular	116	0.76	McVicars Creek and Elm St.
	224n	424n	426n	Circular	99	0.76	McVicars Creek and Blaquier (at end)
	226n	426n	427n	Circular	89	0.76	McVicars Creek and Margaret St.
	227n	427n	428n	Circular	115	0.76	McVicars Creek and Hourigan Crescent
	228n	428n	430n	Circular	173	0.76	McVicars Creek and Hourigan Crescent
	230n	430n	431n	Circular	132	0.76	McVicars Creek and Doris St.
	231n	431n	432n	Circular	108	0.76	McVicars Creek and Manion Place
	232n	432n	434n	Circular	163	0.76	McVicars Creek and Bentwood Drive
	234n	434n	436n	Circular	121	0.76	McVicars Creek and Briarwood Drive
	236n	436n	437n	Circular	125	0.76	McVicars Creek Parallel Autumnwood Drive
	237n	437n	438n	Circular	128	0.76	McVicars Creek Parallel Autumnwood Drive
	238n	438n	439n	Circular	140	0.76	McVicars Creek at Sunset Bay
	239n	439n	440n	Circular	92	0.76	McVicars Creek Parallel Farrand
	240n	440n	441n	Circular	90	0.76	McVicars Creek Parallel Farrand near a church
	241n	441n	442n	Circular	65	0.76	McVicars Creek and River St.
	242n	442n	443n	Circular	68	0.76	McVicars Creek Parallel Regent St.
	243n	443n	445n	Circular	92	0.76	McVicars Creek and Jean St.
	245n	445n	446n	Circular	94	0.76	McVicars Creek Just East of Jean St.
	246n	446n	447n	Circular	173	0.76	McVicars Creek near Dawson
	248n	447n	449n	Circular	168	0.76	McVicars Creek and Algoma St.
	250n	449n	451n	Circular	160	0.76	McVicars Creek and Court St.
	251n	451n	453n	Circular	160	0.76	McVicars Creek and Nugent
	252n	453n	454n	Circular	99	0.76	McVicars Creek and Bendell St.
	254n	454n	455n	Circular	81	0.76	McVicars Creek just East of Benell

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	256n	455n	458n	Circular	134	0.91	McVicars Creek and Front St.
JOHN STREET TRUNK SEWER							
	546n	147n	146n	Circular	68	0.51	John St. and Carl St.
	544n	146n	144n	Circular	108	0.51	John St. and Thunder Bay Expressway
	542n	143n	142n	Circular	68	0.51	John St. and Fairbank Cres.
	541n	142n	141n	Circular	109	0.51	John St. and Junot or Golf Links Rd
	540n	141n	140n	Circular	108	0.51	John St. parallel Sequoia
	539n	140n	139n	Circular	106	0.61	John St. parallel Sequoia
	538n	139n	138n	Circular	77	0.61	John St. and Clarkson St.
	537n	138n	137n	Circular	109	0.61	John St. parallel to Juniper and Evans
	536n	137n	136n	Circular	113	0.61	John St. and Anten St.
	535n	136n	135n	Circular	117	0.61	John St. and Phillips or Maple
	534n	135n	134n	Circular	120	0.61	John St. parallel Juniper where Cedar crosses it
	533n	134n	133n	Circular	121	0.61	John St. parallel Juniper where Cedar crosses it
	532n	133n	132n	Circular	100	0.61	John St. and Alder St.
	530n	132n	130n	Circular	70	0.61	John St. and Algonquin Ave.
	529n	130n	129n	Circular	78	0.76	John St. and Kenogami Ave
	526n	129n	126n	Circular	119	0.76	John St. and Empress Ave.
	525n	126n	125n	Circular	122	0.76	John St. and Windemere Ave.
	524n	125n	124n	Circular	122	0.76	John St. and Hodge St.
	523n	124n	123n	Circular	125	0.76	John St. and Marlborough
	522n	123n	122n	Circular	100	0.76	John St. and Faircrest St.
	521n	122n	121n	Circular	105	0.76	John St. and Ray Blvd.
	520n	121n	120n	Circular	108	0.76	Ray Blvd. and Hartland St.
	518n	120n	118n	Circular	146	0.76	Ray Blvd. and Oliver Road
	517n	118n	117n	Circular	93	0.76	Ray Blvd. and Hill St.
	516n	117n	116n	Circular	94	0.76	Ray Blvd. and Winnipeg Ave.
	515n	116n	115n	Circular	66	0.76	Ray Blvd. east of Winnipeg Ave.
	514n	115n	114n	Circular	97	0.76	High Street and Oliver Road
	1129	130n	197n	Circular	95	0.38	Frankwood Avenue and Oliver Road
	1145	197n	129n	Circular	46	0.38	Frankwood Avenue and Oliver Road
	1114	197n	196n	Circular	89	0.38	Frankwood Avenue and Oliver Road
	1116	196n	195n	Circular	89	0.38	Ryde Avenue and Oliver Road

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	1118	195n	194n	Circular	89	0.38	McBean Avenue and Oliver Road
	1120	194n	193n	Circular	88	0.38	Ray Blvd and Oliver Road
	1122	193n	191n	Circular	859	0.38	Ray Blvd and Oliver Road
	1124	191n	190n	Circular	47	0.38	Rupert Street and Oliver Road
	1131	190n	123n	Circular	46	0.38	Rupert Street and Oliver Road
	1130	190n	122n	Circular	103	0.38	Hill Street and Oliver Road
	1133	122n	189n	Circular	79	0.38	Hill Street and Oliver Road
	1135	189n	187n	Circular	184	0.38	Winnipeg Street and Oliver Road
	1137	187n	186n	Circular	96	0.46	Winnipeg Street and Oliver Road
	1139	186n	184n	Circular	98	0.46	High Street and Oliver Road
	1141	184n	183n	Circular	86	0.46	High Street and Oliver Road
	1143	183n	182n	Circular	74	0.3	High Street and Oliver Road
	1144	182n	114n	Circular	162	0.3	Ray Blvd. and High St.
	511n	114n	111n	Circular	64	0.76	John St. and High St.
	510n	111n	110n	Circular	90	0.61	John St. and Banning St.
	509n	110n	108n	Circular	100	0.61	John St. and Ontario St.
	1242	108n	107n	Circular	200	0.61	John St. and Secord St.
	1243	107n	105n	Circular	200	0.61	John St. and Algoma St.
	1244	105n	103n	Circular	200	0.76	John St. and Machar Ave.
	1246	103n	478n	Circular	200	0.76	John St. and Memorial Court
MAIN INTERCEPTOR							
	701n	301n	303n	Circular	75	0.61	Lillian St. east of Strathcona Ave
	703n	303n	304n	Circular	107	0.61	Lillian St. east of Strathcona Ave
	704n	304n	305n	Circular	84	0.61	Lillian St. east of Strathcona Ave
	705n	305n	306n	Circular	116	0.91	CN and CP railway and Lillian Ave
	706n	306n	307n	Circular	120	0.91	CN and CP railway between Lillian Ave and Grenville
	707n	307n	308n	Circular	123	0.91	CN and CP railway between Lillian Ave and Grenville
	708n	308n	309n	Circular	131	0.91	CN and CP railway between Lillian Ave and Grenville
	709n	309n	310n	Circular	126	0.91	CN and CP railway between Lillian Ave and Grenville
	710n	310n	311n	Circular	262	0.91	CN and CP railway between Lillian Ave and Grenville
	711n	311n	312n	Circular	98	0.91	CN and CP railway between Lillian Ave and Grenville
	712n	312n	313n	Circular	123	0.91	CN and CP railway between Lillian Ave and Grenville
	713n	313n	314n	Circular	121	0.91	CN and CP railway between Lillian Ave and Grenville

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	714n	314n	315n	Circular	122	0.91	CN and CP railway and Grenville Ave
	715n	315n	316n	Circular	92	0.91	CN and CP railway south of Grenville, parallel Marine
	716n	316n	317n	Circular	103	0.91	CN and CP railway south of Grenville, parallel Marine
	717n	317n	318n	Circular	123	0.91	CN and CP railway south of Grenville, parallel Marine
	718n	318n	319n	Circular	122	0.91	CN and CP railway south of Grenville, parallel Marine
	719n	319n	320n	Circular	122	0.91	CN and CP railway south of Grenville, parallel Marine
	720n	320n	321n	Circular	238	0.91	CN and CP railway and Beck St.
	721n	321n	322n	Circular	97	0.91	CN and CP railway and Beck St. Perpendic. to railway
	722n	322n	323n	Circular	75	0.91	east of CN and CP railway at McDougall St.
	723n	323n	324n	Circular	75	0.91	east of CN and CP railway at McDougall St.
	724n	324n	325n	Circular	122	0.91	east of CN and CP railway at McColloch St.
	725n	325n	326n	Circular	122	0.91	east of CN and CP railway at Nelson St.
	726n	326n	328n	Circular	104	0.91	east of CN and CP railway at Clarke
	364n	363n	361n	Circular	200	0.61	Clarke Avenue and Cumberland Avenue
	362n	361n	328n	Circular	61	0.91	CN At Clarke Avenue
	366n	361n	OVF1n	Circular	40	0.91	Clarke Avenue Overflow
	728n	328n	329n	Circular	120	1.07	east of CN and CP railway at Clavet St.
	729n	329n	330n	Circular	120	1.07	east of CN and CP railway at Munro St.
	730n	330n	331n	Circular	123	1.07	east of CN and CP railway at Fitzgerald St.
	731n	331n	332n	Circular	121	1.07	east of CN and CP railway at Stephens St.
	732n	332n	333n	Circular	126	1.07	east of CN and CP railway at Egan St.
	733n	333n	334n	Circular	98	1.07	east of CN and CP railway at Angus
	734n	334n	335n	Circular	101	1.07	east of CN and CP railway at McIntyre St.
	735n	335n	336n	Circular	108	1.07	east of CN and CP railway at Vanhom St.
	736n	336n	337n	Circular	89	1.07	east of CN and CP railway at Wolseley St.
	737n	337n	338n	Circular	92	1.07	CN and CP railway and St. James St.
	739n	338n	340n	Circular	129	1.07	Front St. and River St.
	740n	340n	458n	Circular	88	1.07	Front St. and McVicars creek
	1570	458n	mcovf	Circular	200	0.76	Overflow at McVicars Creek
	258n	458n	459n	Circular	129	1.37	Front St. and Graham St.
	259n	459n	460n	Circular	113	1.37	Front St. and Villa St.
	260n	460n	462n	Circular	202	1.37	Front St. and Camelot St.
	262n	462n	463n	Circular	171	1.37	Front St. and Van Norman
	263n	463n	464n	Circular	55	1.37	Front St. and Waverly St.
	264n	464n	465n	Circular	141	1.37	Front St. and Lorne St.

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	265n	465n	466n	Circular	141	1.37	Front St. and Park St.
	266n	466n	468n	Circular	129	1.37	Front St. and Pearl St.
	268n	468n	470n	Circular	75	1.37	Wilson and Cumberland
	270n	470n	471n	Circular	124	1.22	Cumberland and Manitou
	271n	471n	472n	Circular	49	1.22	between Cumberland and Late St.
	272n	472n	473n	Circular	115	1.22	Late St. at Front
	273n	473n	474n	Circular	76	1.22	Bay St. and Vigars St.
	274n	474n	475n	Circular	134	1.22	Vigars St. South of Bay
	275n	475n	476n	Circular	134	1.22	Vigars St. and Cornwall Ave.
	276n	476n	477n	Circular	134	1.22	Vigars St. and John St.
	277n	477n	478n	Circular	103	1.37	Queen St. between Front and Memorial
	278n	478n	479n	Circular	111	1.37	Court at Queen
	279n	479n	480n	Circular	102	1.37	Court St between Queen and Fort William Road
	280n	480n	481n	Circular	155	1.37	Court St. and Fort William Road
	281n	481n	482n	Circular	152	1.37	Court St. South of Fort William Road
	282n	482n	483n	Circular	154	1.37	Court St. and Lisgar St.
	283n	483n	484n	Circular	152	1.37	Court St. and Spofford
	284n	484n	485n	Circular	133	1.37	Court St and Memorial St.
	285n	485n	486n	Circular	64	1.37	Memorial St. and First Ave.
	286n	486n	487n	Circular	178	1.37	Memorial St. and Second Ave.
	287n	487n	488n	Circular	178	1.37	Memorial St. and Third Ave.
	288n	488n	489n	Circular	165	1.37	Memorial St. and Fourth Ave.
	289n	489n	490n	Circular	125	1.52	Memorial St. and Field St.
	290n	490n	491n	Circular	142	1.52	Field St. and Central
	291n	491n	492n	Circular	128	1.52	Field St. south of Central
	292n	492n	493n	Circular	131	1.52	Field St. south of Central
	293n	493n	494n	Circular	135	1.52	Field St. parallel Eira St.
	294n	494n	495n	Circular	143	1.52	Field St. parallel Eira St.
	295n	495n	496n	Circular	90	1.52	Field St. and CN Railway
	296n	496n	497n	Circular	46	1.52	Field St. and CN Railway
	297n	497n	498n	Circular	114	1.52	Field St. and Main St.
	298n	498n	499n	Circular	74	1.52	Main St. and Fort William Rd.
	299n	499n	1500	Circular	128	1.52	Main St. east of Fort William Rd.
	1300	1500	1501	Circular	67	1.52	Main St. east of Fort William Rd.
	1301	1501	1502	Circular	48	1.52	Main St. east of Fort William Rd.

	SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
	347n	1502	1600	Circular	823	1.68	Main St. east of Fort William Rd.
	349n	1600	pump	Circular	590	2.13	Main St. east of Fort William Rd.
	914n	pump	www	Circular	100	2.13	Main St. east of Fort William Rd.
LEGION TRACK DRIVE TRUNK SEWER							
	564	mood	mood-ovf	Circular	5	0.31	Waterloo St. and Moodie
	lt6	m1	isa	Circular	64	0.69	Waterloo St. and Isabella St.
		d-isa	isa	Circular	298	0.69	Waterloo St. and McGregor
	550	d-rid	ridge	Circular	415	0.38	Waterloo St. and Ridgeway St.
	548	ridge	ridovf	Circular	2	0.31	Waterloo St. and Ridgeway St.
	546	ridge	r1	Circular	415	0.38	Waterloo St. and Ridgeway St.
	lt5	isa	r1	Circular	111	0.91	Waterloo St. and Begin Street
	lt4	r1	a1	Circular	116	0.91	Waterloo St. and Begin Street
	508	arthur	a1	Circular	508	0.69	Waterloo St. and Arthur St.
	lt3	a1	sills	Circular	265	1.22	Waterloo St. and Sills St.
	lt2	sills	lt1	Circular	64	0.69	Between Donald and Rankin South of Waterloo
	1	vict	lt1	Circular	64	0.69	Victoria and Legion Track Dr
	527	lt1	BCMH9	Circular	533	1.22	West of Victoria on Legion Track Dr.
TARBUTT STREET TRUNK SEWER							
	1463	P62	P60	Circular	192	0.31	Caroline Street and Leland Avenue
	1465	P61	P60	Circular	199	0.31	Caroline Street and Edward Street
	1452	P63	P60	Circular	95	0.31	Walsh Street and Brown Street
	1454	P60	P58	Circular	101	0.38	Gordon Street and Brown Street
	1461	P59A	P58	Circular	111	0.31	Gordon Street and Edward Street
	1459	P59	P58	Circular	188	0.31	Gordon Street and Leland Avenue
	1456	P58	P57	Circular	103	0.38	Empire Avenue and Brown Street
	1457	P57	P53	Circular	115	0.38	Brown Street and Christina Street
	1446	P56	P55	Circular	141	0.38	Heath Street and Christina Street
	1448	P55	P54	Circular	215	0.38	Heath Street and Christina Street
	1449	P54	P53	Circular	109	0.38	Heath Street and Christina Street
	1440	P52	P51	Circular	142	0.31	Heath Street and Mary Street

SWIM ID No.	U/S NODE ID	D/S NODE ID	PIPE TYPE	Length (m)	DIAM. (m)	LOCATION
1442	P51	P50	Circular	142	0.31	Heath Street and Mary Street
1443	P50	P49	Circular	142	0.38	Heath Street and Mary Street
1415	P53	P49	Circular	120	0.46	Brown Street and Mary Street
1413	P49	Brown	Circular	201	0.76	Brown Street and Francis Street
1417	P38	Brown	Circular	226	0.76	Brown Street and Ameila Street
1432	P45	P44	Circular	103	0.31	James Street and Ameila Street
1433	P44	P43	Circular	142	0.31	Heath Street and Ameila Street
1436	P41	P40	Circular	249	0.31	Heath Street and Frederica Street
1437	P40	P39	Circular	323	0.38	Heath Street and Frederica Street
1419	P39	P38	Circular	115	0.76	Frederica Street and Ameila Street
1426	P43	P47	Circular	61	0.31	Heath Street and Ameila Street
1428	P43	P42	Circular	215	0.38	Brown Street and Ameila Street
1429	P42	P38	Circular	107	0.38	Brown Street and Ameila Street
1423	P47	P46	Circular	218	0.31	Heath Street and Francis Street
1424	P46	Brown	Circular	121	0.76	Brown Street and Francis Street
1253	Brown	Edward	Circular	225	0.76	Brown Street and Francis Street
1398	P31	P30	Circular	210	0.31	Ford Street and Brock Street
1400	P30	P29	Circular	105	0.38	Edward Street and Brock Street
1401	P29	P28	Circular	105	0.38	Edward Street and Francis Street
1409	P16	P35	Circular	244	0.25	Christina Street and Edward Street
1410	P35	P32	Circular	121	0.61	Mary Street and Edward Street
1407	P32	P28	Circular	121	0.76	Francis Street and Edward Street
1404	P34	P33	Circular	186	0.31	Mary Street and Edward Street
1406	P33	P32	Circular	184	0.33	Mary Street and Edward Street
1387	P28	Edward	Circular	121	0.76	Francis Street and Edward Street
1251	Edward	Ford	Circular	225	0.76	Francis Street and Edward Street
1382	P18	P24	Circular	137	0.31	Mary Street and Ford Street
1384	P24	1383	Circular	121	0.31	Brock Street and Ford Street
1385	1383	Ford	Circular	122	0.31	Francis Street and Ford Street
FRA1	Ford	TARMH1	Circular	421	1.75	Francis Street and Tarbutt Street
1380	P16	P15	Circular	209	0.31	Christina Street and Ford Street
1369	P15	P14	Circular	213	0.31	Christina Street and Ford Street
1370	P14	Christina	Circular	229	0.38	Christina Street and Tarbutt Street
1360	P13	P12	Circular	152	0.31	Sprague Street and Christina Street
TAR5	P12	Christina	Circular	256	0.38	Christina Street and Tarbutt Street

	SWIM	U/S NODE	D/S NODE	PIPE	Length	DIAM.	LOCATION
	ID No.	ID	ID	TYPE	(m)	(m)	
	1375	P18	P17	Circular	99	0.31	Mary Street and Ford Street
	1358	P10	P9	Circular	132	0.31	Mary Street and Sprague Street
	1356	P9	Mary	Circular	116	0.38	Mary Street and Tarbutt Street
	1376	P17	Mary	Circular	195	0.31	Mary Street and Ford Street
	TAR4	Christina	Mary	Circular	121	0.76	Tarbutt Street between Christina Street and Mary Street
	1363	P20	P19	Circular	236	0.31	Brock Street from Ford to Tarbutt Streets
	1364	P19	Brock	Circular	205	0.38	Brock Street from Ford to Tarbutt Streets
	1349	P7	P6	Circular	66	0.31	Sprague Street at Brock Street
	1351	P6	P5	Circular	206	0.31	Brock Street from Sprague to Tarbutt
	1353	P5	P4	Circular	100	0.38	Brock Street from Sprague to Tarbutt
	1354	P4	Brock	Circular	102	0.38	Brock Street from Sprague to Tarbutt
	TAR3	Mary	Brock	Circular	116	0.91	Tarbutt Street from Mary to Brock Streets
	TAR2	Brock	TARMH1	Circular	127	1.37	Tarbutt Street from Brock to Francis Streets
	1248	TARMH1	Amelia	Circular	120	2.13	Tarbutt Street from Francis to Ameila Streets
	1249	Amelia	RK10	Circular	110	2.13	Tarbutt Street from Ameila to Frederica Streets

APPENDIX F
MODELLED DRY WEATHER FLOWS

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
McVicar's Creek Trunk Sewer										
201n	401n	402n	3182	7182	2250	4010	3.42	3.10	7696	12414
202n	402n	404n	3182	7182	2250	4010	3.42	3.10	7696	12414
204n	404n	406n	3182	7182	2250	4010	3.42	3.10	7696	12414
206n	406n	407n	3182	7182	2250	4010	3.42	3.10	7696	12414
207n	407n	408n	4667	8667	3300	5060	3.27	3.02	10799	15261
208n	408n	409n	4667	8667	3300	5060	3.27	3.02	10799	15261
209n	409n	410n	4667	8667	3300	5060	3.27	3.02	10799	15261
210n	410n	412n	4667	8667	3300	5060	3.27	3.02	10799	15261
212n	412n	413n	4667	8667	3300	5060	3.27	3.02	10799	15261
213n	413n	415n	5556	9556	3928	5688	3.20	2.97	12578	16917
215n	415n	417n	5556	9556	3928	5688	3.20	2.97	12578	16917
217n	417n	418n	5556	9556	3928	5688	3.20	2.97	12578	16917
218n	418n	419n	5556	9556	3928	5688	3.20	2.97	12578	16917
219n	419n	421n	5556	9556	3928	5688	3.20	2.97	12578	16917
221n	421n	422n	7081	11081	5006	6766	3.10	2.91	15527	19691
222n	422n	423n	7081	11081	5006	6766	3.10	2.91	15527	19691
223n	423n	424n	13843	17843	9787	11547	2.81	2.70	27534	31203
224n	424n	426n	13843	17843	9787	11547	2.81	2.70	27534	31203
226n	426n	427n	13843	17843	9787	11547	2.81	2.70	27534	31203
227n	427n	428n	13843	17843	9787	11547	2.81	2.70	27534	31203
228n	428n	430n	13843	17843	9787	11547	2.81	2.70	27534	31203
230n	430n	431n	13843	17843	9787	11547	2.81	2.70	27534	31203
231n	431n	432n	13843	17843	9787	11547	2.81	2.70	27534	31203
232n	432n	434n	13843	17843	9787	11547	2.81	2.70	27534	31203
234n	434n	436n	13843	17843	9787	11547	2.81	2.70	27534	31203
236n	436n	437n	13843	17843	9787	11547	2.81	2.70	27534	31203
237n	437n	438n	13843	17843	9787	11547	2.81	2.70	27534	31203
238n	438n	439n	13843	17843	9787	11547	2.81	2.70	27534	31203
239n	439n	440n	13843	17843	9787	11547	2.81	2.70	27534	31203
240n	440n	441n	13843	17843	9787	11547	2.81	2.70	27534	31203
241n	441n	442n	13843	17843	9787	11547	2.81	2.70	27534	31203
242n	442n	443n	13843	17843	9787	11547	2.81	2.70	27534	31203

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
243n	443n	445n	13843	17843	9787	11547	2.81	2.70	27534	31203
245n	445n	446n	13843	17843	9787	11547	2.81	2.70	27534	31203
246n	446n	447n	13843	17843	9787	11547	2.81	2.70	27534	31203
248n	447n	449n	13843	17843	9787	11547	2.81	2.70	27534	31203
250n	449n	451n	13843	17843	9787	11547	2.81	2.70	27534	31203
251n	451n	453n	13843	17843	9787	11547	2.81	2.70	27534	31203
252n	453n	454n	13843	17843	9787	11547	2.81	2.70	27534	31203
254n	454n	455n	13843	17843	9787	11547	2.81	2.70	27534	31203
256n	455n	458n	15964	19964	11287	13047	2.75	2.65	31050	34616
John Street Trunk Sewer										
546n	147n	146n	2917	11717	2062	5934	3.45	2.89	7121	17127
544n	146n	144n	2917	11717	2062	5934	3.45	2.89	7121	17127
542n	143n	142n	2917	11717	2062	5934	3.45	2.89	7121	17127
541n	142n	141n	2917	11717	2062	5934	3.45	2.89	7121	17127
540n	141n	140n	2917	11717	2062	5934	3.45	2.89	7121	17127
539n	140n	139n	2917	11717	2062	5934	3.45	2.89	7121	17127
538n	139n	138n	2917	11717	2062	5934	3.45	2.89	7121	17127
537n	138n	137n	2917	11717	2062	5934	3.45	2.89	7121	17127
536n	137n	136n	2917	11717	2062	5934	3.45	2.89	7121	17127
535n	136n	135n	2917	11717	2062	5934	3.45	2.89	7121	17127
534n	135n	134n	4720	13520	3337	7209	3.27	2.82	10907	20356
533n	134n	133n	4720	13520	3337	7209	3.27	2.82	10907	20356
532n	133n	132n	4720	13520	3337	7209	3.27	2.82	10907	20356
530n	132n	130n	4720	13520	3337	7209	3.27	2.82	10907	20356
529n	130n	129n	9229	18029	6525	10397	2.99	2.70	19504	28048
526n	129n	126n	9229	18029	6525	10397	2.99	2.70	19504	28048
525n	126n	125n	9229	18029	6525	10397	2.99	2.70	19504	28048
524n	125n	124n	9229	18029	6525	10397	2.99	2.70	19504	28048
523n	124n	123n	9229	18029	6525	10397	2.99	2.70	19504	28048
522n	123n	122n	9229	18029	6525	10397	2.99	2.70	19504	28048
521n	122n	121n	9229	18029	6525	10397	2.99	2.70	19504	28048
520n	121n	120n	9229	18029	6525	10397	2.99	2.70	19504	28048

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
518n	120n	118n	9229	18029	6525	10397	2.99	2.70	19504	28048
517n	118n	117n	9229	18029	6525	10397	2.99	2.70	19504	28048
516n	117n	116n	9229	18029	6525	10397	2.99	2.70	19504	28048
515n	116n	115n	9229	18029	6525	10397	2.99	2.70	19504	28048
514n	115n	114n	9229	18029	6525	10397	2.99	2.70	19504	28048
511n	114n	111n	9229	18029	6525	10397	2.99	2.70	19504	28048
510n	111n	110n	11085	19885	7837	11709	2.91	2.65	22807	31087
509n	110n	108n	11085	19885	7837	11709	2.91	2.65	22807	31087
1242	108n	107n	11085	19885	7837	11709	2.91	2.65	22807	31087
1243	107n	105n	11085	19885	7837	11709	2.91	2.65	22807	31087
1244	105n	103n	12026	20826	8503	12375	2.87	2.63	24442	32605
1246	103n	478n	12026	20826	8503	12375	2.87	2.63	24442	32605
Port Arthur Interceptor										
701n	301n	303n	12557	12557	8878	8878	2.86	2.86	25353	25353
703n	303n	304n	12557	12557	8878	8878	2.86	2.86	25353	25353
704n	304n	305n	12557	12557	8878	8878	2.86	2.86	25353	25353
705n	305n	306n	12557	12557	8878	8878	2.86	2.86	25353	25353
706n	306n	307n	12557	12557	8878	8878	2.86	2.86	25353	25353
707n	307n	308n	12557	12557	8878	8878	2.86	2.86	25353	25353
708n	308n	309n	12557	12557	8878	8878	2.86	2.86	25353	25353
709n	309n	310n	12557	12557	8878	8878	2.86	2.86	25353	25353
710n	310n	311n	12557	12557	8878	8878	2.86	2.86	25353	25353
711n	311n	312n	12557	12557	8878	8878	2.86	2.86	25353	25353
712n	312n	313n	12557	12557	8878	8878	2.86	2.86	25353	25353
713n	313n	314n	12557	12557	8878	8878	2.86	2.86	25353	25353
714n	314n	315n	12557	12557	8878	8878	2.86	2.86	25353	25353
715n	315n	316n	19951	19951	14106	14106	2.65	2.65	37430	37430
716n	316n	317n	19951	19951	14106	14106	2.65	2.65	37430	37430
717n	317n	318n	19951	19951	14106	14106	2.65	2.65	37430	37430
718n	318n	319n	19951	19951	14106	14106	2.65	2.65	37430	37430
719n	319n	320n	19951	19951	14106	14106	2.65	2.65	37430	37430

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
720n	320n	321n	19951	19951	14106	14106	2.65	2.65	37430	37430
721n	321n	322n	19951	19951	14106	14106	2.65	2.65	37430	37430
722n	322n	323n	19951	19951	14106	14106	2.65	2.65	37430	37430
723n	323n	324n	19951	19951	14106	14106	2.65	2.65	37430	37430
724n	324n	325n	19951	19951	14106	14106	2.65	2.65	37430	37430
725n	325n	326n	19951	19951	14106	14106	2.65	2.65	37430	37430
726n	326n	328n	19951	19951	14106	14106	2.65	2.65	37430	37430
728n	328n	329n	19951	19951	14106	14106	2.65	2.65	37430	37430
729n	329n	330n	19951	19951	14106	14106	2.65	2.65	37430	37430
730n	330n	331n	19951	19951	14106	14106	2.65	2.65	37430	37430
731n	331n	332n	19951	19951	14106	14106	2.65	2.65	37430	37430
732n	332n	333n	19951	19951	14106	14106	2.65	2.65	37430	37430
733n	333n	334n	19951	19951	14106	14106	2.65	2.65	37430	37430
734n	334n	335n	19951	19951	14106	14106	2.65	2.65	37430	37430
735n	335n	336n	19951	19951	14106	14106	2.65	2.65	37430	37430
736n	336n	337n	19951	19951	14106	14106	2.65	2.65	37430	37430
737n	337n	338n	19951	19951	14106	14106	2.65	2.65	37430	37430
739n	338n	340n	19951	19951	14106	14106	2.65	2.65	37430	37430
740n	340n	458n	41670	45670	29461	31221	2.34	2.30	68910	71851
1570	458n	mcovf	41670	45670	29461	31221	2.34	2.30	68910	71851
258n	458n	459n	41670	45670	29461	31221	2.34	2.30	68910	71851
259n	459n	460n	41670	45670	29461	31221	2.34	2.30	68910	71851
260n	460n	462n	41670	45670	29461	31221	2.34	2.30	68910	71851
262n	462n	463n	41670	45670	29461	31221	2.34	2.30	68910	71851
263n	463n	464n	41670	45670	29461	31221	2.34	2.30	68910	71851
264n	464n	465n	44229	48229	31270	33030	2.31	2.28	72375	75281
265n	465n	466n	44229	48229	31270	33030	2.31	2.28	72375	75281
266n	466n	468n	44229	48229	31270	33030	2.31	2.28	72375	75281
268n	468n	470n	44229	48229	31270	33030	2.31	2.28	72375	75281
270n	470n	471n	49891	53891	35273	37033	2.27	2.23	79909	82749
271n	471n	472n	49891	53891	35273	37033	2.27	2.23	79909	82749
272n	472n	473n	49891	53891	35273	37033	2.27	2.23	79909	82749
273n	473n	474n	49891	53891	35273	37033	2.27	2.23	79909	82749

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
274n	474n	475n	49891	53891	35273	37033	2.27	2.23	79909	82749
275n	475n	476n	49891	53891	35273	37033	2.27	2.23	79909	82749
276n	476n	477n	49891	53891	35273	37033	2.27	2.23	79909	82749
277n	477n	478n	61917	62691	43776	49408	2.18	2.17	95412	107448
278n	478n	479n	63123	75923	44628	50260	2.17	2.10	96933	105606
279n	479n	480n	63123	75923	44628	50260	2.17	2.10	96933	105606
280n	480n	481n	63123	75923	44628	50260	2.17	2.10	96933	105606
281n	481n	482n	63123	75923	44628	50260	2.17	2.10	96933	105606
282n	482n	483n	63123	75923	44628	50260	2.17	2.10	96933	105606
283n	483n	484n	63123	75923	44628	50260	2.17	2.10	96933	105606
284n	484n	485n	67657	80457	47834	53466	2.15	2.08	102611	111178
285n	485n	486n	67657	80457	47834	53466	2.15	2.08	102611	111178
286n	486n	487n	67657	80457	47834	53466	2.15	2.08	102611	111178
287n	487n	488n	67657	80457	47834	53466	2.15	2.08	102611	111178
288n	488n	489n	67657	80457	47834	53466	2.15	2.08	102611	111178
289n	489n	490n	67657	80457	47834	53466	2.15	2.08	102611	111178
290n	490n	491n	67657	80457	47834	53466	2.15	2.08	102611	111178
291n	491n	492n	67657	80457	47834	53466	2.15	2.08	102611	111178
292n	492n	493n	67657	80457	47834	53466	2.15	2.08	102611	111178
293n	493n	494n	70296	83096	49699	55331	2.13	2.07	105883	114393
294n	494n	495n	70296	83096	49699	55331	2.13	2.07	105883	114393
295n	495n	496n	70296	83096	49699	55331	2.13	2.07	105883	114393
296n	496n	497n	70296	83096	49699	55331	2.13	2.07	105883	114393
297n	497n	498n	70296	83096	49699	55331	2.13	2.07	105883	114393
298n	498n	499n	70296	83096	49699	55331	2.13	2.07	105883	114393
299n	499n	1500	70296	83096	49699	55331	2.13	2.07	105883	114393
1300	1500	1501	70296	83096	49699	55331	2.13	2.07	105883	114393
1301	1501	1502	70296	83096	49699	55331	2.13	2.07	105883	114393
347n	1502	1600	70296	83096	49699	55331	2.13	2.07	105883	114393
349n	1600	pump	70296	83096	49699	55331	2.13	2.07	105883	114393
914n	pump	www	70296	83096	49699	55331	2.13	2.07	105883	114393
Kaministiquia Interceptor										

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
KAM23	KAMMH13	RRK12	0	0	0	0	4.50	4.50	0	0
KAM22	RRK12	KAMMH11	3182	3182	2250	2250	3.42	3.42	7696	7696
KAM21	KAMMH11	RRK10	6232	6232	4406	4406	3.16	3.16	13901	13901
KAM20	RRK10	KAMMH2	6882	6882	4865	4865	3.11	3.11	15149	15149
KAM19	KAMMH2	RRK9	6882	6882	4865	4865	3.11	3.11	15149	15149
KAM18	RRK9	RRK8	7531	7531	5325	5325	3.08	3.08	16378	16378
KAM17	RRK8	KAMNODE2	8393	8393	5934	5934	3.03	3.03	17979	17979
KAM16	KAMNODE2	KAMNODE1	8393	8393	5934	5934	3.03	3.03	17979	17979
KAM15	KAMNODE1	RRK7	8393	8393	5934	5934	3.03	3.03	17979	17979
KAM14	RRK7	RRK6	10117	10117	7153	7153	2.95	2.95	21098	21098
KAM13	RRK6	RRK5	10121	10121	7155	7155	2.95	2.95	21105	21105
KAM12	RRK5	RRK4	11354	11354	8027	8027	2.90	2.90	23277	23277
KAM11	RRK4	RRK3	13316	13316	9415	9415	2.83	2.83	26646	26646
KAM10	RRK3	RRK2	13674	13674	9668	9668	2.82	2.82	27250	27250
KAM9	RRK2	RRK1	14536	14536	10277	10277	2.79	2.79	28693	28693
KAM8	RRK1	KAMMH1	16923	16923	11965	11965	2.73	2.73	32609	32609
KAM7	KAMMH1	KAMMH28A	16923	16923	11965	11965	2.73	2.73	32609	32609
KAM6	KAMMH28A	KAMMH29	16923	16923	11965	11965	2.73	2.73	32609	32609
KAM5	KAMMH29	KAMMH33	16923	16923	11965	11965	2.73	2.73	32609	32609
KAM4	KAMMH33	KAMMH31	16923	16923	11965	11965	2.73	2.73	32609	32609
KAM3	KAMMH31	KAMMH32	16923	16923	11965	11965	2.73	2.73	32609	32609
KAM2	KAMMH32	219	16923	16923	11965	11965	2.73	2.73	32609	32609
KAM1	219	tbwwtp	18382	18382	12996	12996	2.69	2.69	34950	34950
Cameron Interceptor										
CAM9	CAMMH5	CAMMH4A	1591	1591	1125	1125	3.66	3.66	4118	4118
CAM8	CAMMH4A	CAMMH3A	1591	1591	1125	1125	3.66	3.66	4118	4118
CAM7	CAMMH3A	CAMMH2B	1591	1591	1125	1125	3.66	3.66	4118	4118
CAM6	CAMMH2B	CAMMH1	1591	1591	1125	1125	3.66	3.66	4118	4118
CAM5	CAMMH1	CAMMH2A	1591	1591	1125	1125	3.66	3.66	4118	4118
CAM4	CAMMH2A	CAMMH3	1591	1591	1125	1125	3.66	3.66	4118	4118

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
CAM3	CAMMH3	CAMMH4	1591	1591	1125	1125	3.66	3.66	4118	4118
CAM2	CAMMH4	CAMMH2	1591	1591	1125	1125	3.66	3.66	4118	4118
CAM1	CAMMH2	KAMMH1	1591	1591	1125	1125	3.66	3.66	4118	4118
Neebing Interceptor										
N23	NMH23	MN22	4019	4019	2841	2841	3.33	3.33	9466	9466
N22	NMH22	NMH21	4019	4019	2841	2841	3.33	3.33	9466	9466
N21	NMH21	NMH20	4019	4019	2841	2841	3.33	3.33	9466	9466
N20	NMH20	NMH19	4019	4019	2841	2841	3.33	3.33	9466	9466
N19	NMH19	NMH18	4019	4019	2841	2841	3.33	3.33	9466	9466
N18	NMH18	NMH17	4191	4191	2963	2963	3.32	3.32	9823	9823
N17	NMH17	NMH16	4191	4191	2963	2963	3.32	3.32	9823	9823
N16	NMH16	NMH15	4191	4191	2963	2963	3.32	3.32	9823	9823
N15	NMH15	NMH14	4390	4390	3104	3104	3.30	3.30	10233	10233
N14	NMH14	NMH13	4881	4881	3451	3451	3.25	3.25	11231	11231
N13	NMH13	NMH12	4881	4881	3451	3451	3.25	3.25	11231	11231
N12	NMH12	NMH11	5239	5239	3704	3704	3.23	3.23	11949	11949
N11	NMH11	NMH10	5597	5597	3957	3957	3.20	3.20	12659	12659
N10	NMH10	NMH9	5597	5597	3957	3957	3.20	3.20	12659	12659
N9	NMH9	NMH8	5769	5769	4079	4079	3.19	3.19	12999	12999
N8	NMH8	NMH7	6101	6101	4313	4313	3.16	3.16	13646	13646
N7	NMH7	NMH6	6101	6101	4313	4313	3.16	3.16	13646	13646
N6	NMH6	NMH5	6101	6101	4313	4313	3.16	3.16	13646	13646
N5	NMH5	NMH4	6101	6101	4313	4313	3.16	3.16	13646	13646
N4	NMH4	NMH3	6199	6199	4383	4383	3.16	3.16	13837	13837
N3	NMH3	NMH2	6199	6199	4383	4383	3.16	3.16	13837	13837
ATH1	NMH2	www	6199	6199	4383	4383	3.16	3.16	13837	13837
Ford Drive										
F3	FMH3	FMH2	4773	4773	3375	3375	3.26	3.26	11014	11014
F2	FMH2	FMH1	4773	4773	3375	3375	3.26	3.26	11014	11014

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
F1	FMH1	WMMH6	4773	4773	3375	3375	3.26	3.26	11014	11014
Dease Trunk										
DT4	DTMH4	DTMH3	3713	3713	2625	2625	3.36	3.36	8825	8825
DT3	DTMH3	DTMH2	3713	3713	2625	2625	3.36	3.36	8825	8825
DT2	DTMH2	DTMH1	3713	3713	2625	2625	3.36	3.36	8825	8825
DT1	DTMH1	BCMH7	3713	3713	2625	2625	3.36	3.36	8825	8825
Brunswick Connector Sewer										
BC9	BCMH9	BCMH8	9945	9945	7031	7031	2.96	2.96	20791	20791
BC8	BCMH8	BCMH7	9945	9945	7031	7031	2.96	2.96	20791	20791
BC7	BCMH7	BCMH6	14241	14241	10068	10068	2.80	2.80	28200	28200
BC6	BCMH6	BCMH5	14241	14241	10068	10068	2.80	2.80	28200	28200
BC5	BCMH5	BCMH4	14241	14241	10068	10068	2.80	2.80	28200	28200
BC4	BCMH4	BCMH3	14241	14241	10068	10068	2.80	2.80	28200	28200
BC3	BCMH3	BCMH2	14241	14241	10068	10068	2.80	2.80	28200	28200
BC2	BCMH2	BCMH1	14241	14241	10068	10068	2.80	2.80	28200	28200
BC1	BCMH1	WMMH4	14241	14241	10068	10068	2.80	2.80	28200	28200
Legion Track Drive Trunk Sewer										
lt7	walsh	m1	9945	9945	7031	7031	2.96	2.96	20791	20791
562	mood	m1	9945	9945	7031	7031	2.96	2.96	20791	20791
564	mood	mood-ovf	9945	9945	7031	7031	2.96	2.96	20791	20791
lt6	m1	isa	9945	9945	7031	7031	2.96	2.96	20791	20791
	d-isa	isa	9945	9945	7031	7031	2.96	2.96	20791	20791
550	d-rid	ridge	9945	9945	7031	7031	2.96	2.96	20791	20791
548	ridge	ridovf	9945	9945	7031	7031	2.96	2.96	20791	20791
546	ridge	r1	9945	9945	7031	7031	2.96	2.96	20791	20791
lt5	isa	r1	9945	9945	7031	7031	2.96	2.96	20791	20791

SWMM ID No.	U/S NODE ID	D/S NODE ID	Existing Population	Future Population	Existing DWF (m3/d)	Future DWF (m3/d)	Existing Peak Factor	Future Peak Factor	Existing Peak DWF (m3/d)	Future Peak DWF (m3/d)
lt4	r1	a1	9945	9945	7031	7031	2.96	2.96	20791	20791
508	arthur	a1	9945	9945	7031	7031	2.96	2.96	20791	20791
lt3	a1	sills	9945	9945	7031	7031	2.96	2.96	20791	20791
lt2	sills	lt1	9945	9945	7031	7031	2.96	2.96	20791	20791
1	vict	lt1	9945	9945	7031	7031	2.96	2.96	20791	20791
527	lt1	BCMH9	9945	9945	7031	7031	2.96	2.96	20791	20791
Neebing McIntyre Interceptor										
WM7	WMMH7	WMMH6	0	32900	0	12100	4.50	2.44	0	29500
WM6	WMMH6	WMMH5	0	32900	0	12100	4.50	2.44	0	29500
WM5	WMMH5	WMMH4	0	32900	0	12100	4.50	2.44	0	29500
WM4	WMMH4	WMMH3	14241	47141	10068	22168	2.80	2.29	28200	50730
WM3	WMMH3	www	20439	53339	14451	26551	2.64	2.24	38193	59435
WM2	www	WMMH1	90735	136435	64150	81882	2.04	1.89	130550	154988

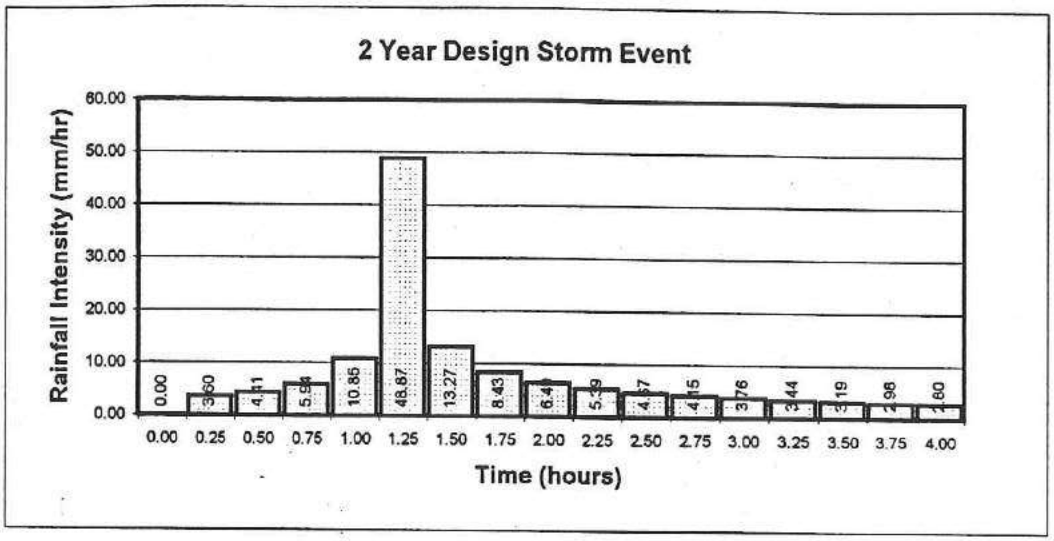
Appendix G
Basement Flooding Analysis Results

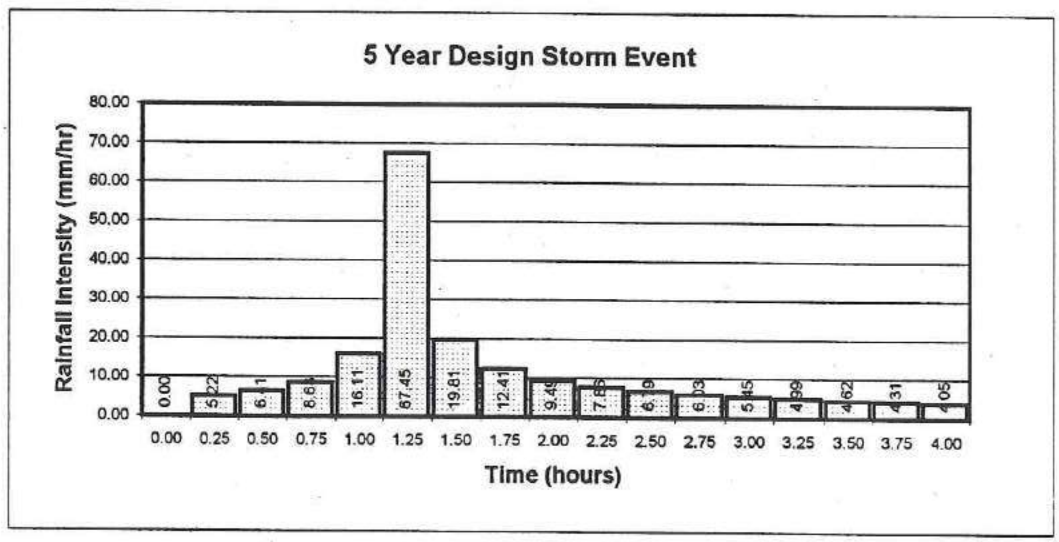
Table G-1 ***Model % Imperviousness Parameters***

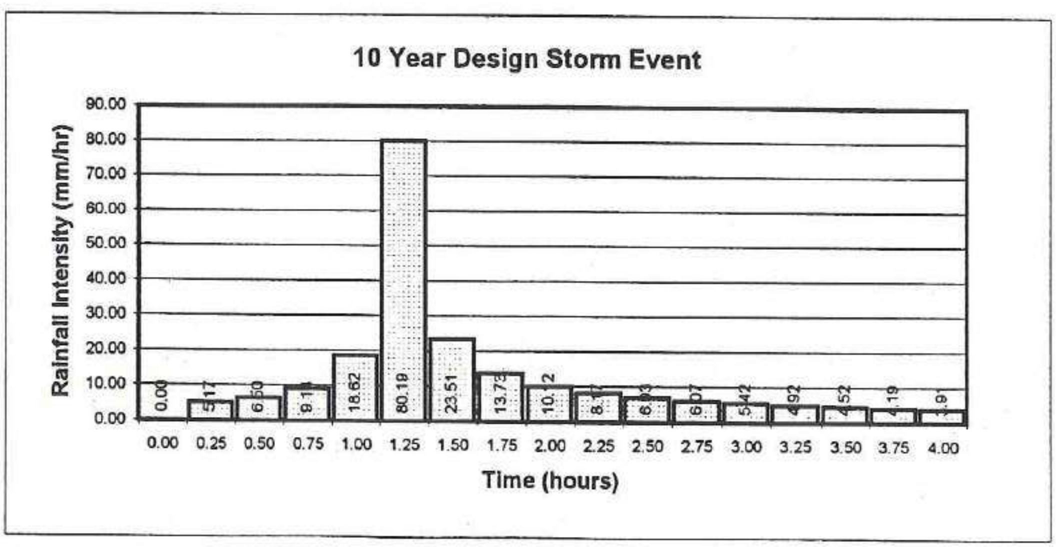
Thunder Bay - South Ward - Combined Areas

																Existing					Post Separation			
Regulator ID	Tributary Area (ha)	No. of Houses	Total House Area (m^2)	10% Connected Roof Area (m^2)	20% Connected Roof Area (m^2)	50% Connected Roof Area (m^2)	80% Connected Roof Area (m^2)	Total D/W Area (m^2)	Road Area (m^2)	Lane Area (m^2)	Additional Imp. Area (m^2)	10% Imp. Area (ha)	20% Imp. Area (ha)	50% Imp. Area (ha)	80% Imp. Area (ha)	10% Existing % Imp.	20% Existing % Imp.	50% Existing % Imp.	80% Existing % Imp.	Separation Imp. Area (ha)	10% Post SS % Imp.	20% Post SS % Imp.	50% Post SS % Imp.	80% Post SS % Imp.
RK1	24.3	170	13,190	1,319	2,638	6,595	10,552	510	14,040	3,210	4,100	2.32	2.45	2.85	3.2	9.5%	10.1%	11.7%	13.3%	0.28	8.4%	8.9%	10.6%	12.2%
RK2	15.8	87	7,320	732	1,464	3,660	5,856	315	13,000	1,845	32,800	4.87	4.94	5.16	5.4	30.8%	31.3%	32.7%	34.1%	1.50	21.3%	21.8%	23.2%	24.6%
RK3	11.5	58	10,330	1,033	2,066	5,165	8,264	-	12,360	465	22,900	3.68	3.78	4.09	4.4	32.0%	32.9%	35.6%	38.3%	0.00	32.0%	32.9%	35.6%	38.3%
RK4	2.9	15	1,950	195	390	975	1,560	45	3,040	480	8,840	1.26	1.28	1.34	1.4	43.4%	44.1%	46.1%	48.2%	0.00	43.4%	44.1%	46.1%	48.2%
RK5	25.7	286	22,800	2,280	4,560	11,400	18,240	1,680	30,960	4,635	36,100	7.57	7.79	8.48	9.2	29.4%	30.3%	33.0%	35.6%	1.35	24.2%	25.1%	27.7%	30.4%
RK6	2.3	23	2,691	269	538	1,346	2,153	-	2,000	405	-	0.27	0.29	0.38	0.5	11.8%	12.8%	16.3%	19.8%	0.21	2.5%	3.7%	7.2%	10.7%
RK7	39.8	418	40,895	4,090	8,179	20,448	32,718	4,941	15,312	7,485	-	3.18	3.59	4.82	6.0	8.0%	9.1%	12.2%	15.3%	1.32	4.7%	5.7%	8.8%	11.9%
RK8	17.1	138	14,000	1,400	2,800	7,000	11,200	2,625	10,280	1,680	-	1.60	1.74	2.18	2.6	9.3%	10.2%	12.6%	15.1%	1.46	0.8%	1.6%	4.1%	6.5%
RK9	24.8	259	28,490	2,849	5,698	14,245	22,792	3,698	18,800	5,790	-	3.09	3.38	4.23	5.1	12.5%	13.6%	17.1%	20.5%	2.80	1.2%	2.3%	5.8%	9.2%
RK10	195.0	1,893	207,160	20,716	41,432	103,580	165,728	35,706	150,560	1,200	-	20.82	22.89	29.10	35.3	10.7%	11.7%	14.9%	18.1%	9.28	5.9%	7.0%	10.2%	13.4%
RK12	153.6	1,045	133,925	13,393	26,785	66,963	107,140	19,212	108,344	16,485	-	15.74	17.08	21.10	25.1	10.2%	11.1%	13.7%	16.4%	2.92	8.3%	9.2%	11.8%	14.5%
RN20	8.0	71	11,360	1,136	2,272	5,680	9,088	600	10,920	120	-	1.28	1.39	1.73	2.1	16.0%	17.4%	21.7%	25.9%	0.00	16.0%	17.4%	21.7%	25.9%
RN21	5.2	44	6,220	622	1,244	3,110	4,976	645	5,840	1,200	3,900	1.22	1.28	1.47	1.7	23.5%	24.7%	28.3%	31.8%	0.00	23.5%	24.7%	28.3%	31.8%
RN24	6.3	112	8,980	898	1,796	4,490	7,184	200	6,720	1,620	4,700	1.41	1.50	1.77	2.0	22.4%	23.9%	28.1%	32.4%	0.00	22.4%	23.9%	28.1%	32.4%
RN25	16.0	62	5,900	590	1,180	2,950	4,720	-	4,960	570	-	0.61	0.67	0.85	1.0	3.8%	4.2%	5.3%	6.4%	0.00	3.8%	4.2%	5.3%	6.4%
RN26	1.5	37	3,660	366	732	1,830	2,928	280	2,720	270	370	0.40	0.44	0.55	0.7	28.7%	29.1%	36.5%	43.8%	0.00	28.7%	29.1%	36.5%	43.8%
RN27	11.3	50	4,820	482	964	2,410	3,856	485	5,840	810	-	0.76	0.81	0.95	1.1	6.7%	7.2%	8.4%	9.7%	0.00	6.7%	7.2%	8.4%	9.7%
RN28	5.6	74	6,660	666	1,332	3,330	5,328	432	6,240	1,260	-	0.86	0.93	1.13	1.3	15.4%	16.5%	20.1%	23.7%	0.00	15.4%	16.5%	20.1%	23.7%
RN30	4.4	64	7,680	768	1,536	3,840	6,144	570	4,880	1,500	445	0.80	0.87	1.10	1.3	18.1%	19.8%	25.1%	30.3%	0.60	4.5%	6.3%	11.5%	16.7%
RN32	76.8	846	58,600	5,860	11,720	29,300	46,880	10,980	17,440	13,905	-	4.82	5.40	7.18	8.9	6.3%	7.0%	9.3%	11.6%	1.36	4.5%	5.3%	7.6%	9.9%
RN33	3.6	24	1,656	166	331	828	1,325	1,187	3,063	242	-	0.47	0.48	0.53	0.6	12.9%	13.4%	14.8%	16.2%	0.00	12.9%	13.4%	14.8%	16.2%
RN29	15.5	79	7,900	790	1,580	3,950	6,320	685	41,200	-	-	4.26	4.34	4.57	4.8	27.5%	28.0%	29.6%	31.0%	0.00	27.6%	28.0%	29.6%	31.0%
RN22	2.6	13	1,300	130	260	650	1,040	585	3,030	1,000	-	0.47	0.49	0.53	0.6	18.4%	18.9%	20.4%	21.8%	0.00	18.4%	18.9%	20.4%	21.9%

File: TB-%imperviousness.xls


2 Year Design Storm Event
Rainfall Intensity (mm/hr)
Time (hours)


5 Year Design Storm Event
Rainfall Intensity (mm/hr)
Time (hours)


10 Year Design Storm Event
Rainfall Intensity (mm/hr)
Time (hours)

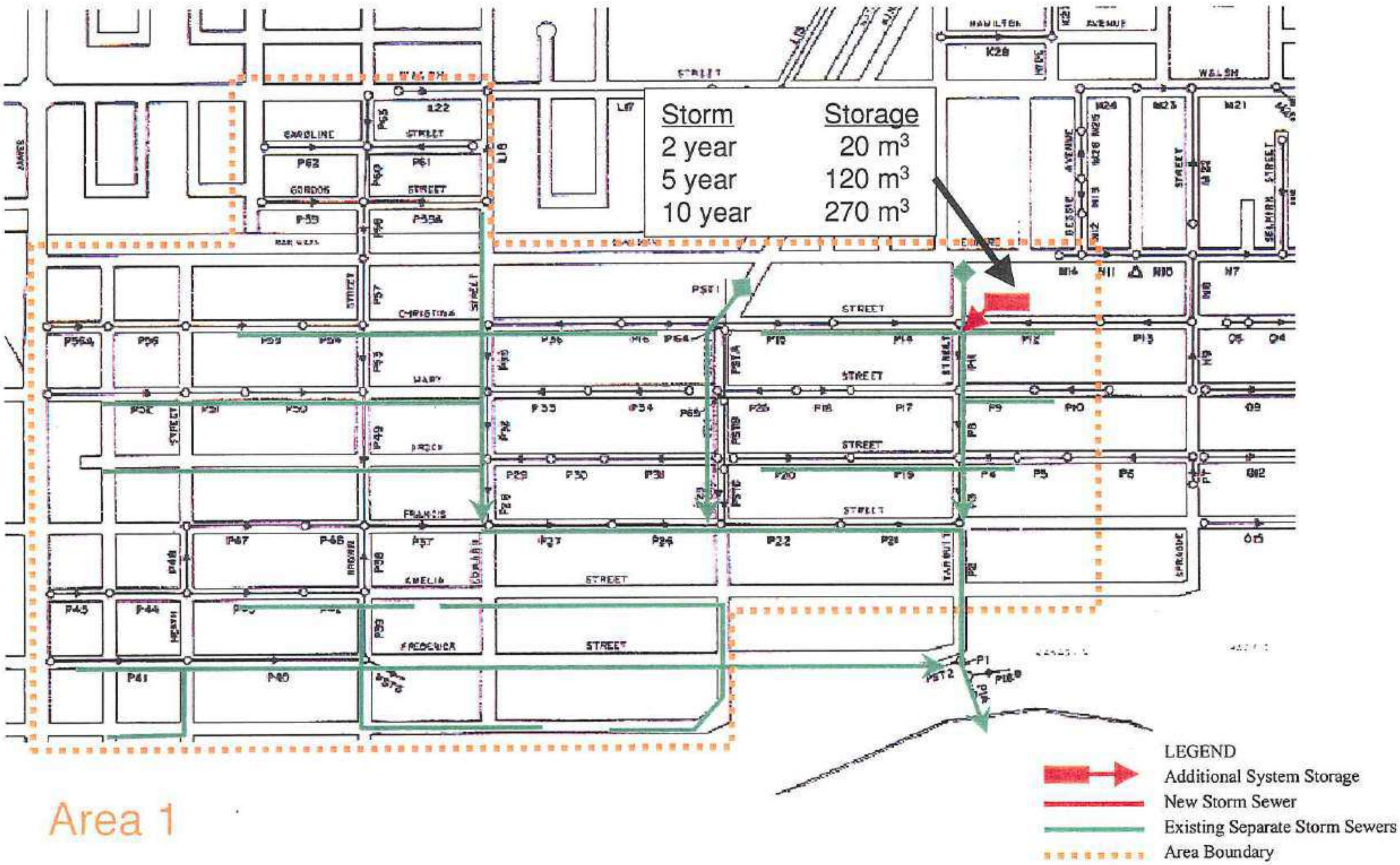


Figure G-1 : Area 1 - 2, 5 and 10 Year Storm Event Protection

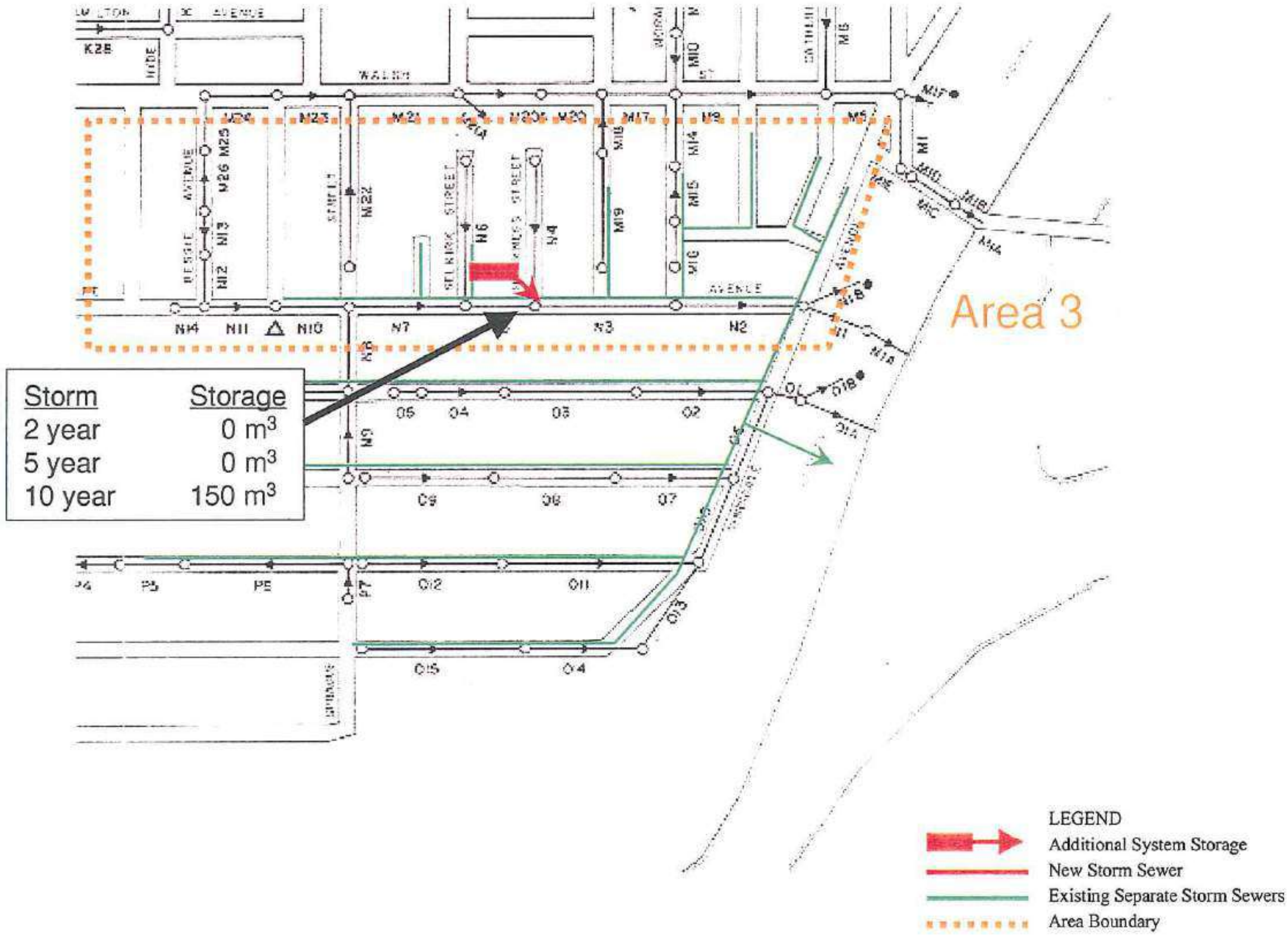


Figure G-2 : Area 3 - 2, 5 and 10 Year Storm Event Protection

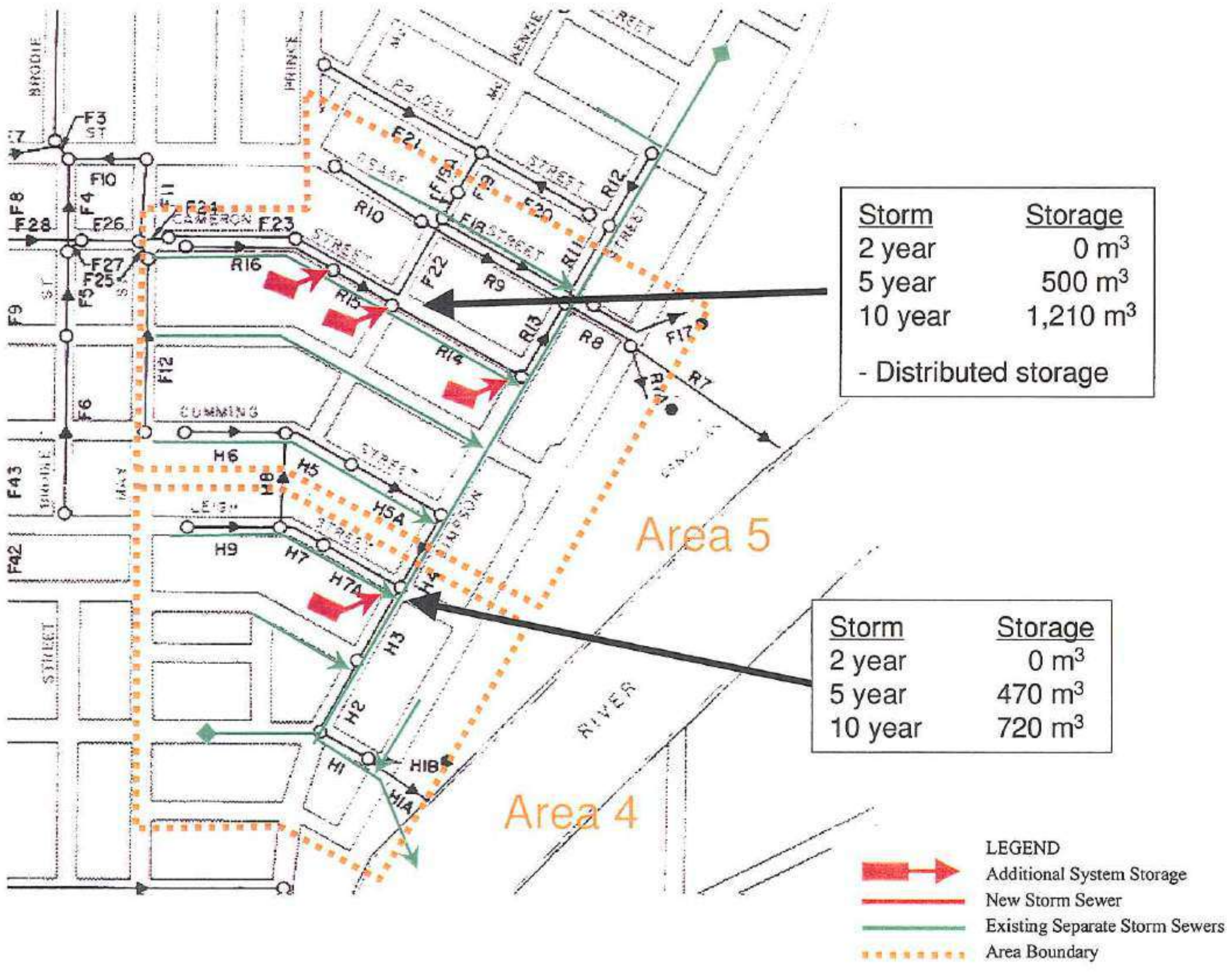


Figure G-3 : Area 4 and 5 - 2, 5 and 10 Year Storm Event Protection

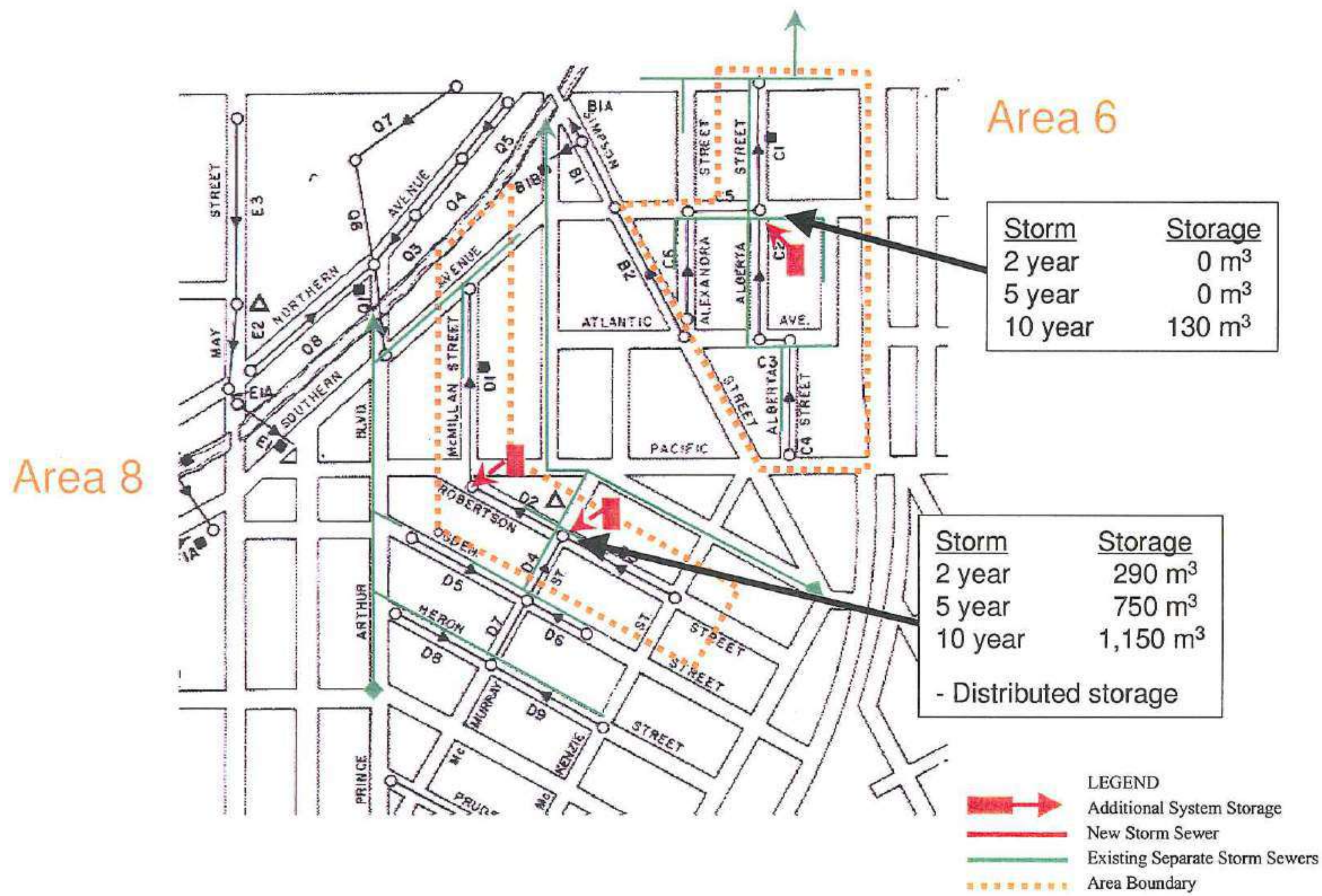


Figure G-4 : Area 6 and 8 - 2, 5 and 10 Year Storm Event Protection

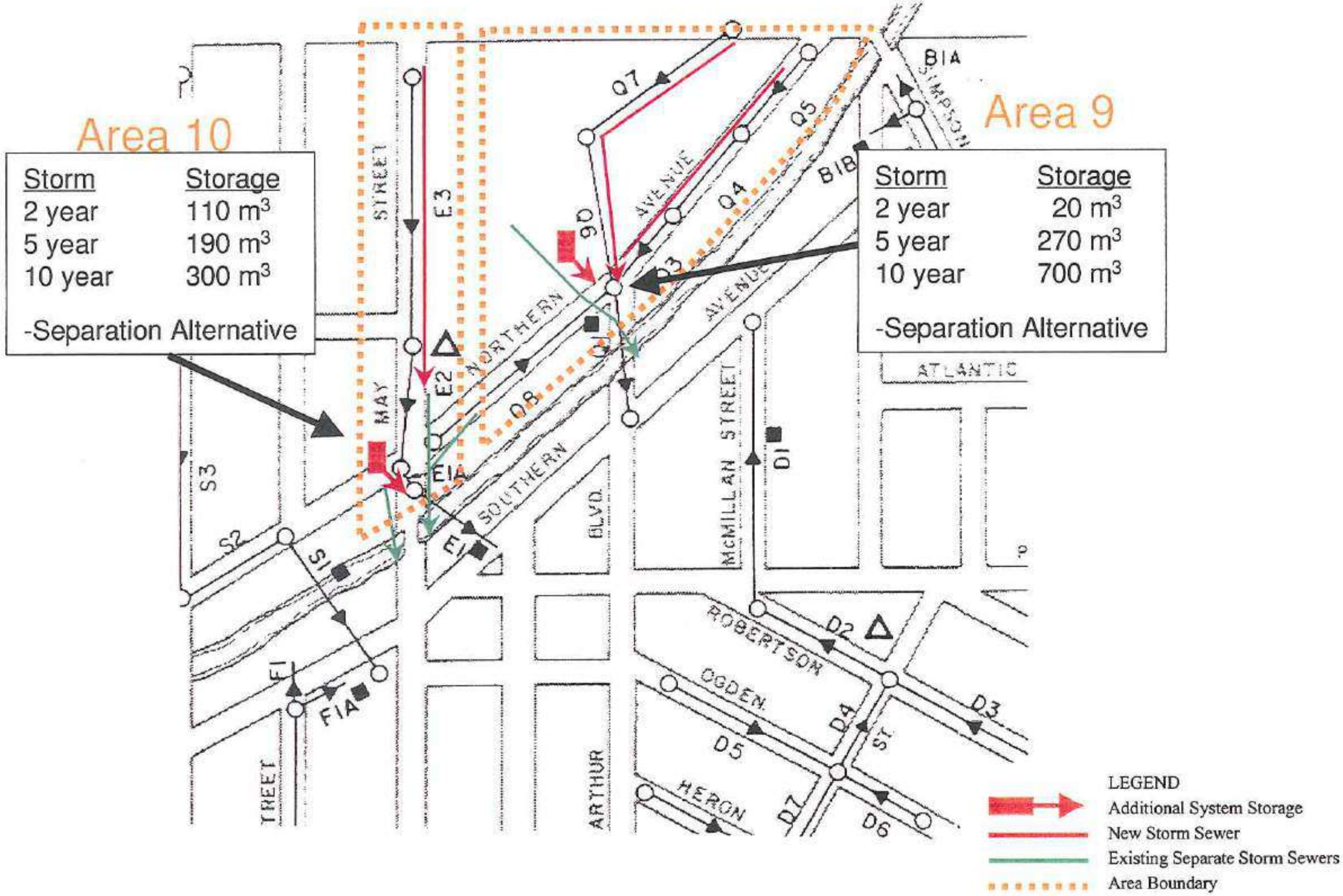


Figure G-5 : Area 9 and 10 - 2, 5 and 10 Year Storm Event Protection

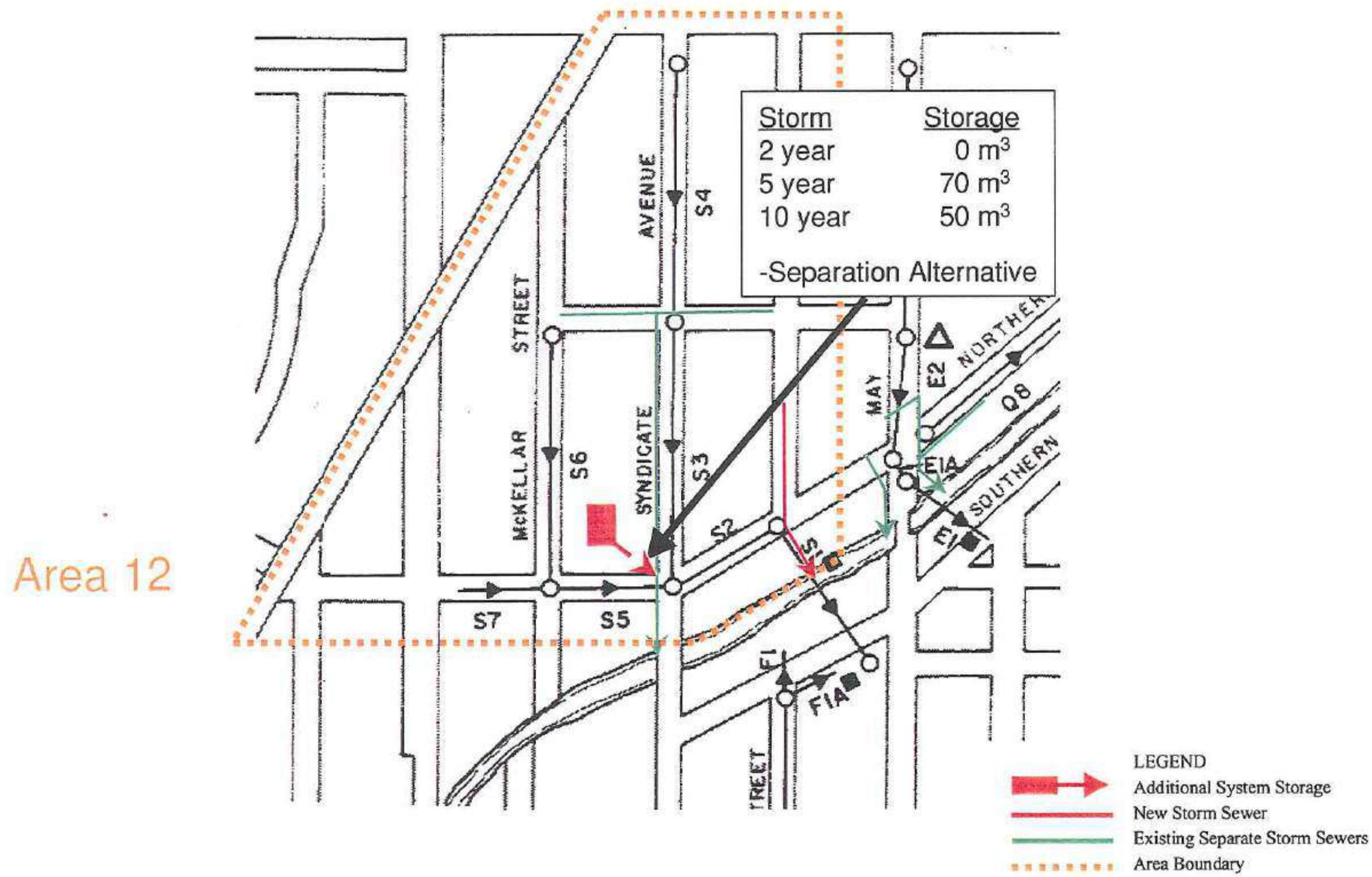


Figure G-6 : Area 12 - 2, 5 and 10 Year Storm Event Protection

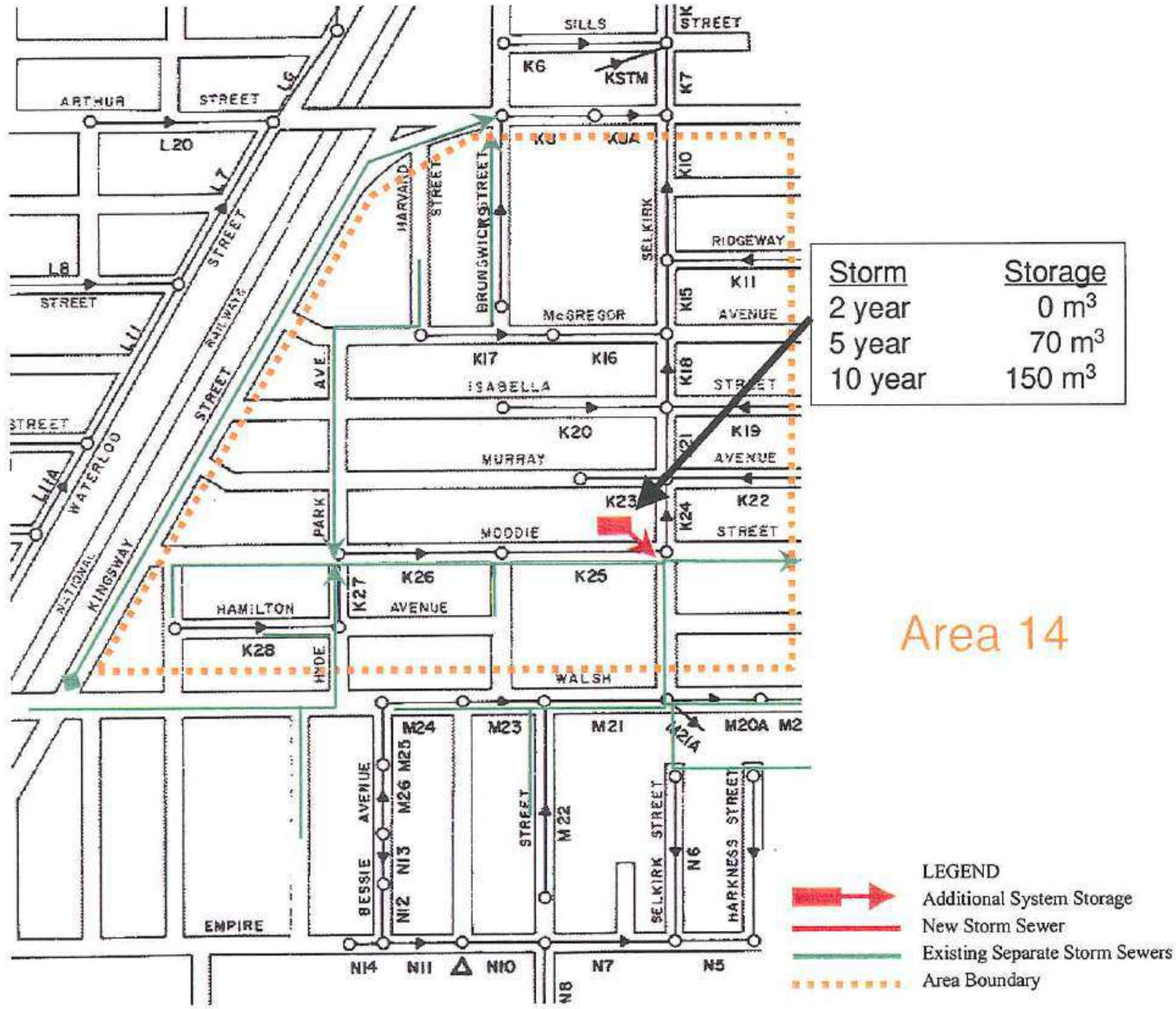


Figure G-7 : Area 14 - 2, 5 and 10 Year Storm Event Protection

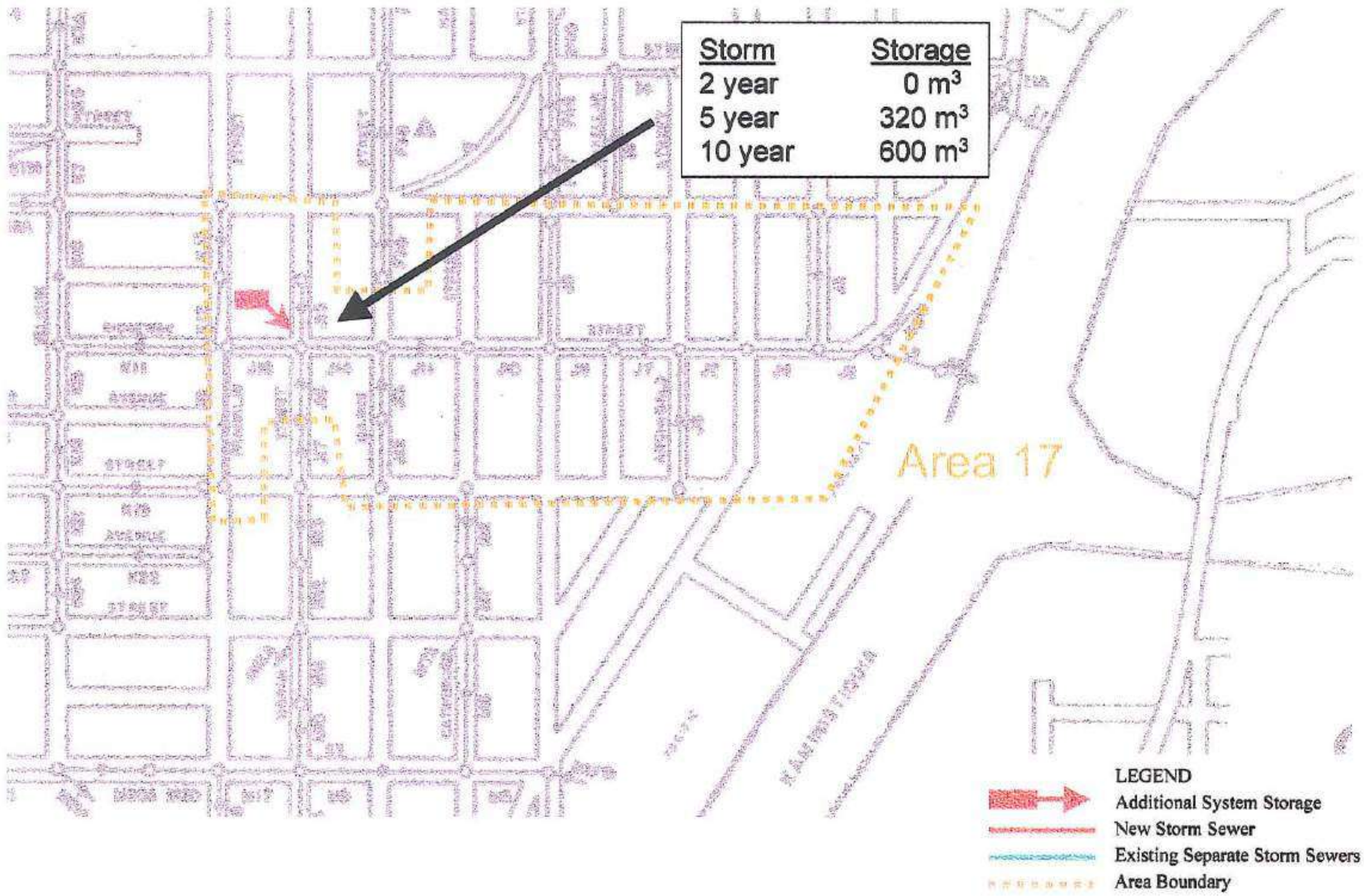


Figure G-8 : Area 17 - 2, 5 and 10 Year Storm Event Protection