City of Winnipeg
Water and Waste Department

Combined Sewer Overflow Management Study

PHASE 3 Technical Memorandum No. 1

CONTROL ALTERNATIVES

April 1998
0510-A-38-47

Internal Document by:

WARDROP Engineering Inc. and TetrES CONSULTANTS INC.

In Association With:

Gore & Storrie Limited and EMA Services Inc.

ACKNOWLEDGEMENTS

The Study Team acknowledges, with sincere appreciation, the contribution of many individuals and agencies consulted in the course of Phase 3 of the CSO Management Study. The Study Team especially acknowledges the assistance of the City of Winnipeg Project Management Committee and the Advisory Committee.

Contributors to the TMs included:

Consultants:

Wardrop Engineering Inc./Tetr*ES* Consultants Inc.
In Association with CH2M Gore & Storrie Limited and EMA Services Limited

G. Rempel, Project Manager
R.J. Gladding, Assistant Project Manager
D. Morgan
N. Szoke
G. Steiss
S. Quigley
R. Rempel
G. Mohr
D. Dagg
M. Sweet
M. Parente
M. Yue
R.G. Skrentner
D. Weatherbe
G. Zukovs
S. Black
L. Thompson
C. Rowney

City of Winnipeg:

E.J. Sharp, Project Manager
M.A. Shkolny
A. Permut
W.J. Borlase
P. Lagasse
D. Wardrop
D. Sacher
T.R. Pearson
D. McNeil
E. Zaleski
B. Station

PREAMBLE

This Technical Memorandum (TM) is one of a series of TMs intended for internal discussion. It is not intended as a report representing the policy or direction of the City of Winnipeg.

The two TMs produced in Phase 3 are:

TM #1	Control Alternatives
TM #2	Public Communication

Each of the Phase 3 TMs draws on information developed in the prior Phase 1 and Phase 2 TMs.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF FIGURES

LIST OF FIGURES (CONT'D)

LIST OF TABLES

LIST OF TABLES (CONT'D)

1. INTRODUCTION

The objective of Phase 3 of the Combined Sewer Overflow (CSO) Management Study is to identify and evaluate a range of potential combined sewer control plans for the City of Winnipeg. This is the third of four phases, each of which has a specific purpose, as shown on Figure 1-1.

The range of potential plans will include nominal additional works (related to optimizing the existing infrastructure) to installation of structurally intensive works such as regional storage or even separation of the combined sewer network. The assessment of potential plans will include consideration of the practicability of different technologies, the capital and operational costs, operational aspects, technical issues and environmental benefits, etc. Experience with CSO control measures elsewhere was reviewed to provide guidance on emerging technology and to identify/confirm proven technology and its reliability.

In Phase 2 of the study, region-wide application on different types of technologies or control methods, such as high-rate treatment, were assessed in terms of comparative costs and benefits. In Phase 3, different combinations of technologies, which responded to the specific characteristics of each combined sewer district, were identified and developed into potential overall plans for the entire combined sewer service area.

Potential plans were developed, building on the Phase 2 analyses, which would provide CSO control performance to match different selected performance benchmarks or degrees of CSO control. These benchmarks included the EPA "presumptive" control target of no more than four overflows per year or 85% capture of the combined sewage. The intent was to develop plans which would result in different increments of CSO control leading from the present situation to control performance equivalent to the separation of the entire combined sewer system.

Overall system hydrologic and hydraulic models, which included the addition of potential storage/transport/treatment elements, were used to assess the physical infrastructure requirements, such as size and location of storage elements in the system, which would meet the different performance targets. This resulted in a large number of distinct potential plans,

General Approach

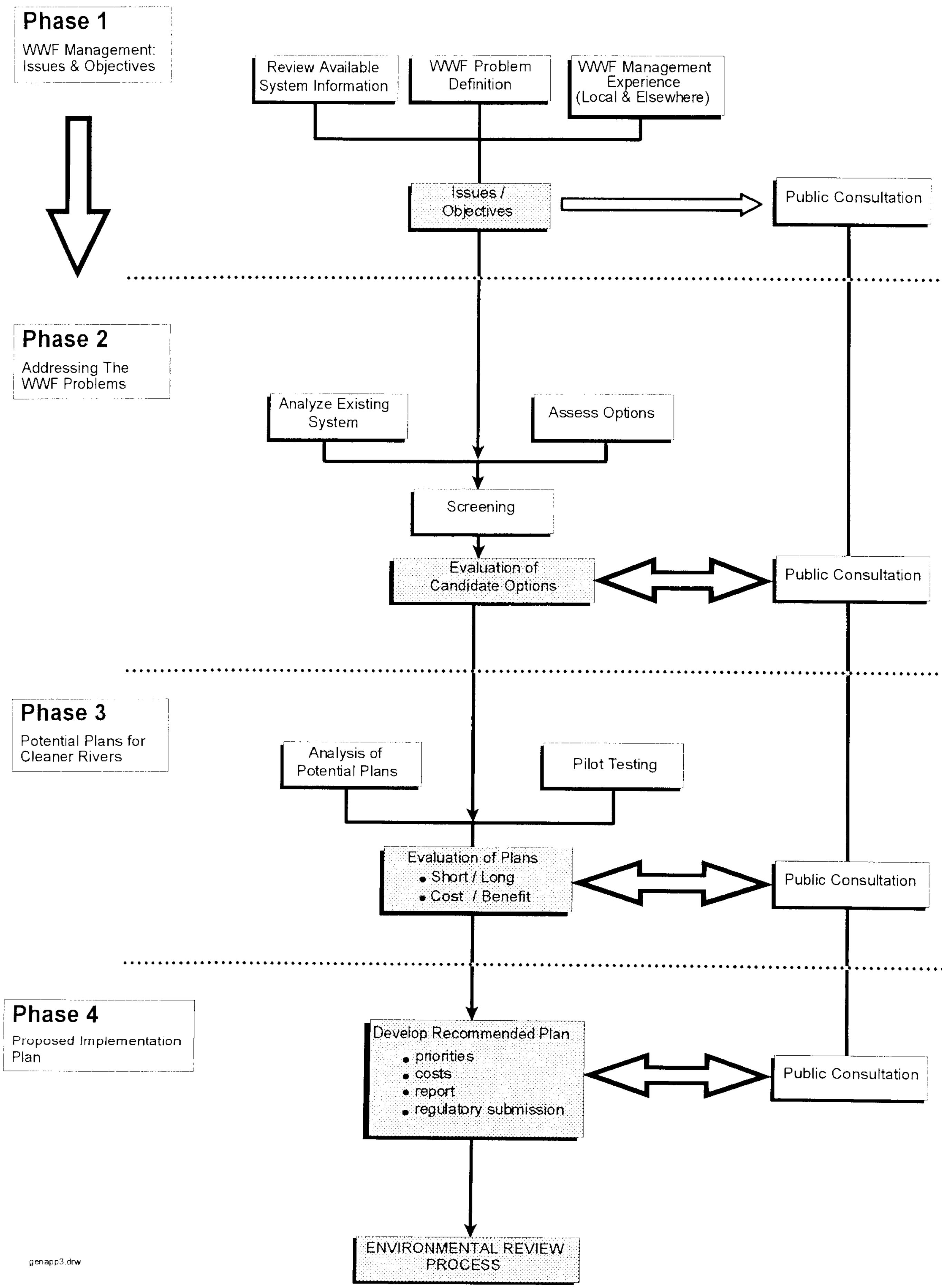


Figure 1-1

each offering different physical, socio-economic, and environmental performance characteristics.

This TM will present these potential control plans and display the different characteristics for the purpose of allowing technical and public assessment of the different "trade-offs" involved in the array of potential plans. It is expected that, after such evaluation, a relatively small number of potential plans (say five) will emerge as candidate plans for further assessment later in Phase 4.

2. PHASE 2 OVERVIEW

2.1 SUMMARY OF PHASE 2 ASSESSMENT

The Phase 2 Technical Memorandum No. 6, "Potential CSO Management Strategies" provided a detailed overview of the Phase 2 findings. The following are highlights from that document.

The three main components of the Phase 2 screening process were:

- definition of the water quality issues/objectives;
- definition of available technology; and
- evaluation of the technologies for the Winnipeg situation.

2.2 WATER QUALITY ISSUES

Water quality issues were reviewed in Phase 1 with due consideration for the Manitoba Surface Water Quality Objectives (MSWQO) and the manner in which the Clean Environment Commission (CEC) considered that CSO discharges should be studied. The review of water quality issues was repeated in Phase 2 and confirmed that the discharge of CSOs in Winnipeg are particularly relevant to surface water quality for:

- **aesthetics** - the river should be free from constituents attributable to sewage. The numerous outfalls in Winnipeg (CSO, Land Drainage Sewers (LDS), and sanitary sewage) represent a pollution control issue in this regard;

- **microbiological quality** - the current river quality, as measured by the indicator organism, fecal coliforms, often does not meet the MSWQO, chiefly because of the undisinfected, treated discharges from the City's Water Pollution Control Centres (WPCCs) during dry weather conditions, and because of CSOs, during wet weather conditions. The river use of most relevance to compliance with the fecal coliform density objective is water-based recreation.

The effect of CSO control measures on other ancillary water quality issues, such as ammonia loadings or other urban pollutants, such as heavy metals and nutrients, can be considered in terms of the degree of capture of combined sewage or minimization of CSOs, as discussed later.

For dry weather conditions, the CEC recommended that, in the Winnipeg area, the Red River be protected as a source for irrigation water and for primary and secondary recreation and the Assiniboine River as an irrigation source and for secondary recreation. The fecal coliform objectives associated with these classifications are 200 and 1,000 fc/100 mL for primary and secondary recreation, respectively. The irrigation objectives are similar, depending on the degree of contact with the workers. The current study resulted from the CEC recommendation that wet weather objectives be the subject of additional investigation.

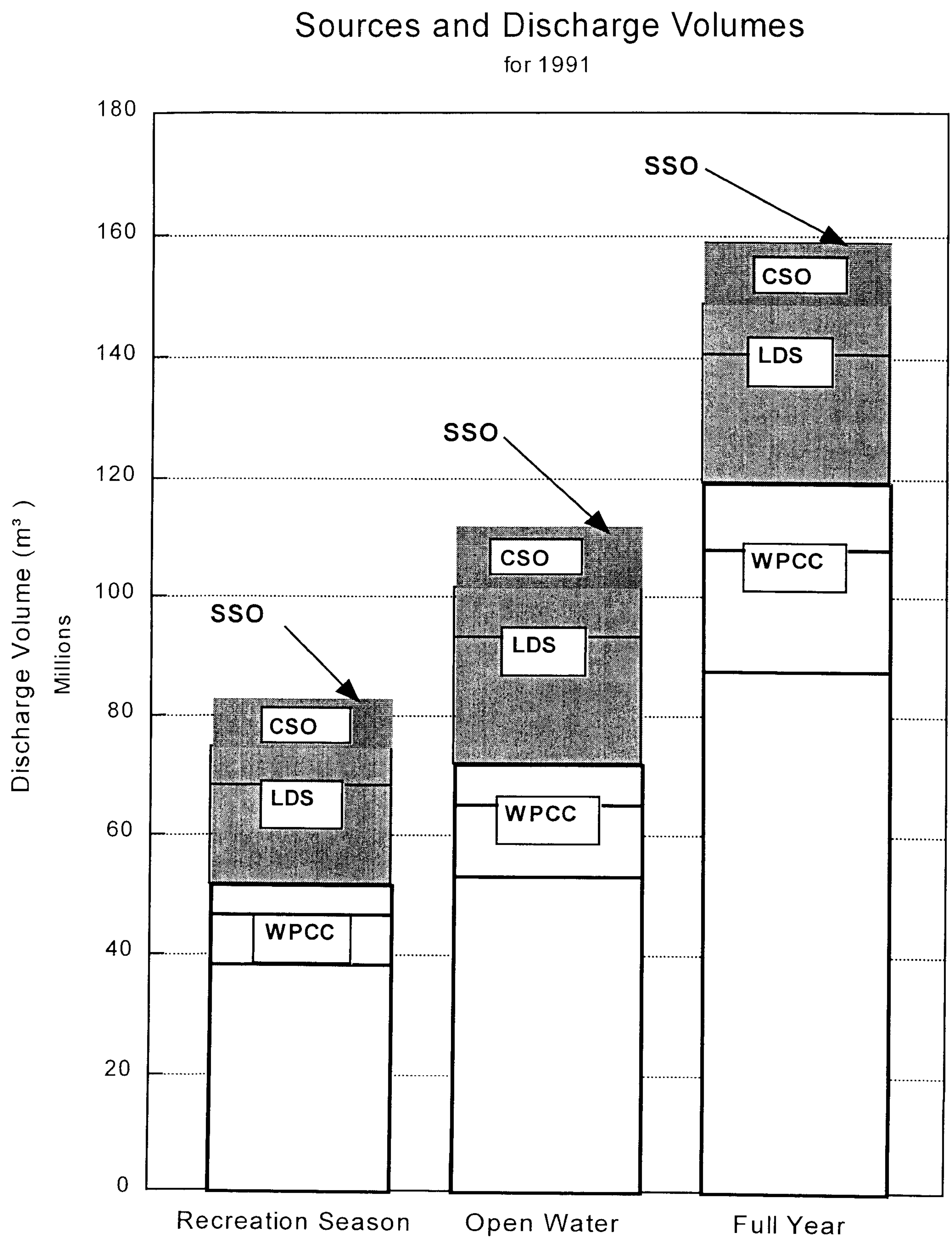
2.3 EXISTING CONDITIONS

Before considering potential means of improving existing conditions, it is essential to establish the baseline conditions, for measuring improvement.

- Number and Volume of CSO Discharges

  The number and volume of CSO discharges were analyzed. The indications were that approximately 60% of the 7,000,000 m^3 of runoff generated in the CSO areas discharges to the rivers, i.e., 40% of the combined sewage is captured for treatment. This volume of overflow represents an average of about 20 overflows per district over the recreation year (May 1 to September 30).

  The sources and discharge volumes from all urban wastewater sources, as determined for 1991, are indicated on Figure 2-1, taken from Phase 2 - TM No. 6. The results indicate that discharge volumes from the WPCC effluent and LDS discharges tend to dominate the recreational season, open water, and especially annual sewage volumes discharged to the



(Source Phase 2 T.M. no. 6, Figure 3-1)

Figure 2-1

rivers. CSOs are still significant volumes while SSOs and interceptor overflow volumes are insignificant in comparison.

- Relative Loadings to the River

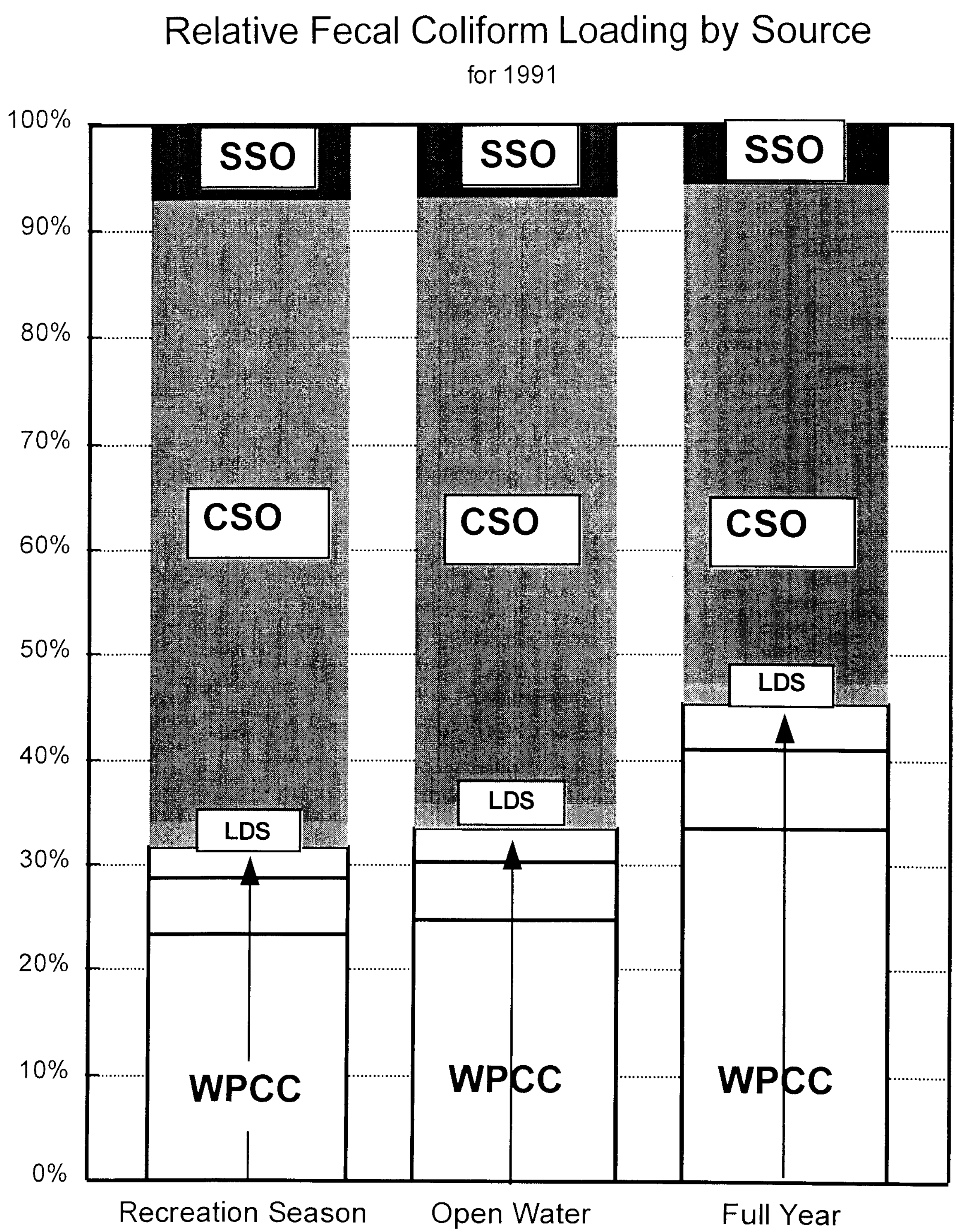
By applying event mean concentrations (EMC) for fecal coliforms for each type of discharge, the loading perspective changes, as indicated on Figure 2-2 (Phase 2 – TM No. 6). It is obvious that WPCC effluents (undisinfected) and CSO discharges dominate fecal coliform loadings to the rivers. LDS loadings are relatively small and SSO loadings, which occur only during intense rainfall, are relatively insignificant.

- Coliform Densities in the River

During dry weather flow (DWF), the undisinfected WPCC effluent discharges are the main reason for high coliform levels in the rivers. These continuous discharges often result in densities above the MSWQO for both primary and secondary recreation. During wet weather flows (WWF), the CSO loadings dominate. The CSO contributions greatly exceed those of the LDS and WPCC under WWF conditions. The resultant elevated fecal coliform levels in the rivers typically die-off to background levels in about 3 days. These events occur about 20 times over the recreation season, i.e., slightly less than once per week on average, and therefore represent a significant effect on typical river water quality. Figure 2-3 (as developed in Phase 3) shows a profile of fecal coliform concentrations for a 6-week period during the 1992 representative year. The spikes represent the impact on water quality resulting from rainfall events causing runoff and exceeding the existing interception capacity of the CS system.

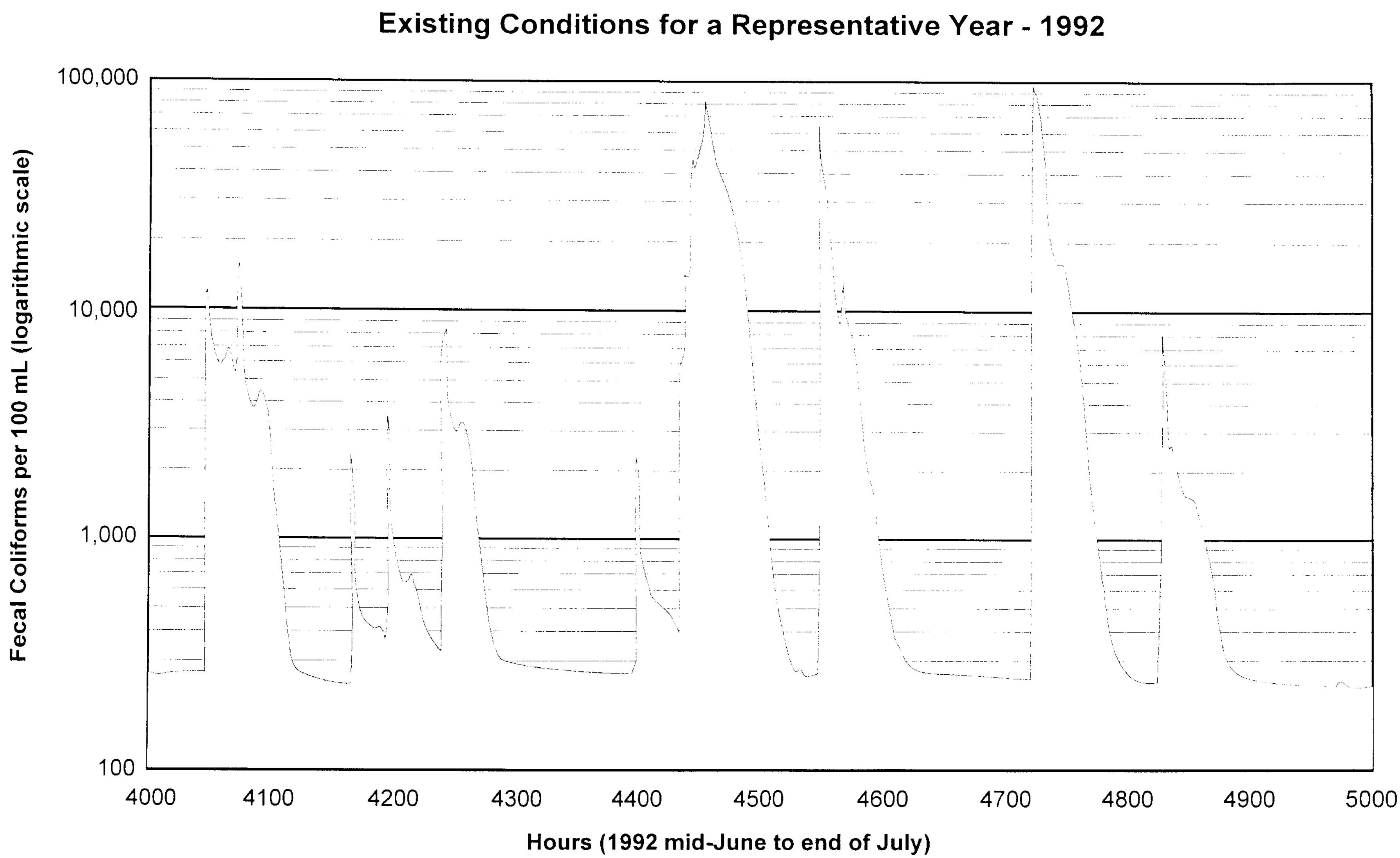
- Health Risk

The average coliform densities can be translated, using recognized epidemiological equations, to approximate the health risk associated with recreational use of the rivers. These computations were updated during Phase 3 of the CSO study. Using the Ferley (1989) equation and the 1992 representative year, the typical health risk rate downstream of



(Source Phase 2 T.M. no. 6, Figure 3-2)

Figure 2-2



spikepgm
s\01\0510

Figure 2-3

the NEWPCC was estimated at about 13 cases of gastrointestinal disease (GI) for 1,000 immersions and 10.5/1,000, downstream of the SEWPCC. This compares to about 10 cases of GI per 1,000 immersions at the provincial objective (200 fc/100 mL) for primary recreation. During dry weather conditions, this increment of health risk is due, mainly, to the currently undisinfected WPCC effluents. However, during and immediately after WWF, the incremental rise in health risk is due mainly to CSOs. The above estimates consider the overall average of wet and dry weather conditions.

- Aesthetics

  The MSWQO require that surface waters should be free of constituents attributable to sewage or other human-induced discharges. During DWF, the City of Winnipeg discharges are not major contributors of such constituents. During wet weather, however, land drainage and combined sewer overflows do discharge such materials. CSO are the main source of sewage-related constituents in wet weather discharges to the rivers.

2.4 CONTROL OPTIONS

2.4.1 Overview

The Phase 2 analysis of control options was broken down into three categories:

- addressing dry weather flow issues;
- optimizing the existing system for wet weather flow; and
- structurally-intense options.

In the case of the structurally intensive options, it was assumed that each of the technologies would be applied uniformly across all combined sewer districts. The purpose was to develop a first-cut of effectiveness of the options and, if possible, to reduce the number of options to be evaluated during the course of the Phase 3 analysis.

Each of the categories is discussed briefly below.

2.4.2 Addressing DWF Issues

- Dry Weather Overflows

  The overflow of sewage during dry weather is contrary to good environmental practice. The City of Winnipeg has made, and continues to make, substantial efforts to avoid dry weather overflows (DWOs). During the course of Phase 2 and 3 of the study, a few locations where intermittent DWOs occur were detected and corrective action was taken.

- WPCC Disinfection

  Disinfection of the treated effluents from the City's three WPCCs will reduce the levels of fecal coliforms in the rivers to well below the primary recreation objectives of 200 fc/100 mL during dry weather conditions. Currently, none of the plant effluents are disinfected. The City of Winnipeg has plans to implement disinfection at all three plants. Budget allocations for the SEWPCC are in place for 1998, with the other two WPCCs planned to follow.

2.4.3 Optimizing the Existing System for Wet Weather Flow (WWF)

The capture of combined sewage in an existing system can be optimized by maximizing the flow to the WPCC and/or using the storage available in the sewer network.

- Increased Flow in the Main Interceptor

  The existing system is intended to intercept a wet weather flow equivalent to nominal 2.75 times DWF from the combined sewer districts for conveyance to, and treatment in, the WPCCs. During Phase 2, it was determined that the main interceptor could be operated at a rate of flow of approximately 5 times the current DWF. This higher interception rate could be achieved by increasing the pumping rate from currently-pumped districts and upgrading flow regulators for the gravity districts. Such an increase in flow rate would transport more wet weather flow to the NEWPCC and could also make most effective use of the in-line storage and reduce the length of time required to dewater that storage.

- In-Line Storage

The use of in-line storage in the existing system is a cost-effective means of diverting significant volumes of WWF for treatment at the WPCCs. Because of the generally flat topography of the combined sewer districts, and the hydraulic restraint imposed on the outlet by the river levels, the main combined sewer trunks (and relief trunks) are generally very large and have very flat slopes. While sewers typically run full during large storms, during smaller storms large volumes are available for the temporary storage of combined sewage. The stored combined sewage could be held until such time as capacity is available in the main interceptor and the NEWPCC to convey and treat the stored flows. The potential for increasing the flow in the main interceptor enhances this possibility. The storage would be implemented in such a way as not to compromise basement flood protection.

2.4.4 Structurally-intensive Options

The potential structurally-intensive options investigated during Phase 2 comprised the following: a central CSO treatment facility; tunnel storage; off-line storage; high-rate treatment, and separation. All of these options have the potential to address the fecal coliform issue (MSWQO) and/or the number and volumes of overflow to the rivers.

The nature of these investigations follows:

- *Central CSO Treatment Facility*:

  Construction of a new interceptor system and a central wet weather treatment plant to receive virtually all wet weather flow, was considered impractical for the City of Winnipeg. Conveyance of flows up to the maximum delivery capacity of the main interceptor for treatment at the NEWPCC, is a far more practical alternative.

- *Tunnel storage*:

  The Phase 2 analysis considered relatively deep tunnel systems which would be used to store some or all of the combined sewage and to convey these flows to the NEWPCC (probably expanded) for treatment after the rainfall event.

- *Off-line storage*:

  The Phase 2 analysis considered near-surface storage tanks, near the outlet of the combined sewer trunks, using the capacity of the existing interceptor for conveyance to the NEWPCC after the rainfall event.

- *High-Rate Treatment Devices*:

  Two alternatives were reviewed in Phase 2. Vortex Solid Separators (VSS) comprise a high-rate solids-removal device designed to render wastewater suitable for disinfection. These units were also presumed to be located at the combined sewer outlet. In the Phase 2 analysis, they were sized on the basis of using UV disinfection. The second option comprised retention treatment basins (RTBs). These consisted of a combined storage/high-rate sedimentation facility. The stored flows would be conveyed to the NEWPCC on cessation of the storm. Flows in excess of the storage tank capacity (up to the capacity of the RTB as a sedimentation tank) would be disinfected. Flows in excess of the RTB settling capacity would be diverted directly to the river.

- *Separation*:

  The possibility of separating storm and sanitary collector systems in the combined sewer districts was reviewed. Separation would significantly reduce discharge into the rivers of fecal coliforms and solids attributable to sanitary wastes. The resultant land drainage sewers, however, would still discharge fecal coliforms into the rivers.

- *Floatables removal*:

  The possibility of addressing floatables discharge into the rivers was considered as a stand-alone option. The advantage would be significantly reduced structural needs and hence reduced capital costs to intercept floatables at each combined sewer outlet. The disadvantage would be that it would not address other CSO issues.

Table 2-1, from Phase 2 - TM No. 6, "Potential CSO Management Strategies", summarizes the options considered.

2.4.5 Phase 2 Performance Evaluation

The results of the Phase 2 analyses were illustrated graphically on Figures 2-4, 2-5 and 2-6 taken from the Phase 2 TM No. 6.

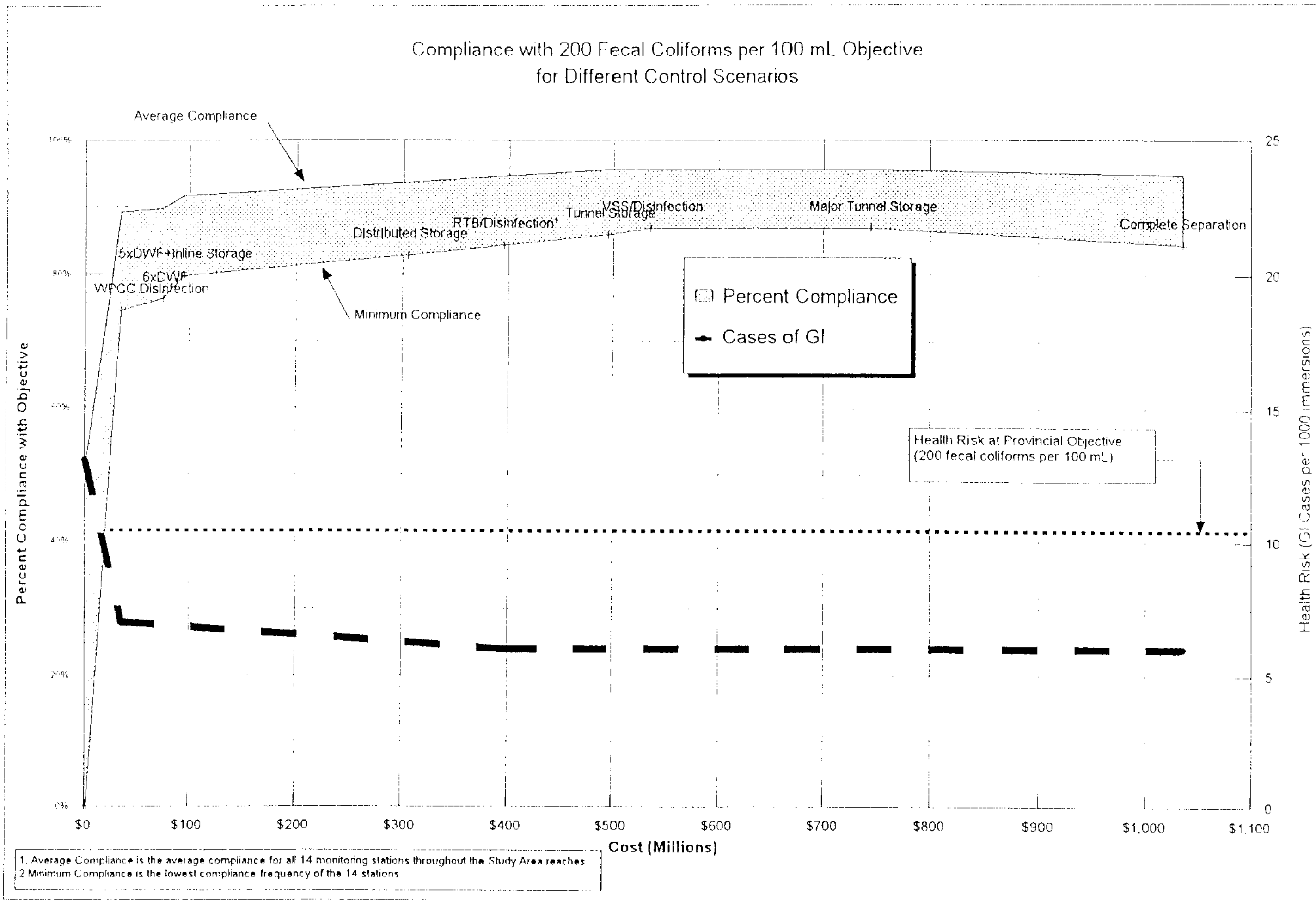
Figure 2-4 is a plot of compliance of the various technologies, as applied district-wide, with the MSWQO for fecal coliforms. As can be seen, implementation of disinfection of the WWPC DWF effluent derives the single-most significant improvement in compliance. Compliance with the primary recreation objective approaches 90% of the time. The next most cost-effective improvement is achieved through the implementation of potential in-line storage and main interceptor capacity improvement, although the increase in compliance is small (2%). A number of other options, entailing significant capital investments, result in modest additional improvements in frequency of compliance (3 to 5%). While costly, these result in better

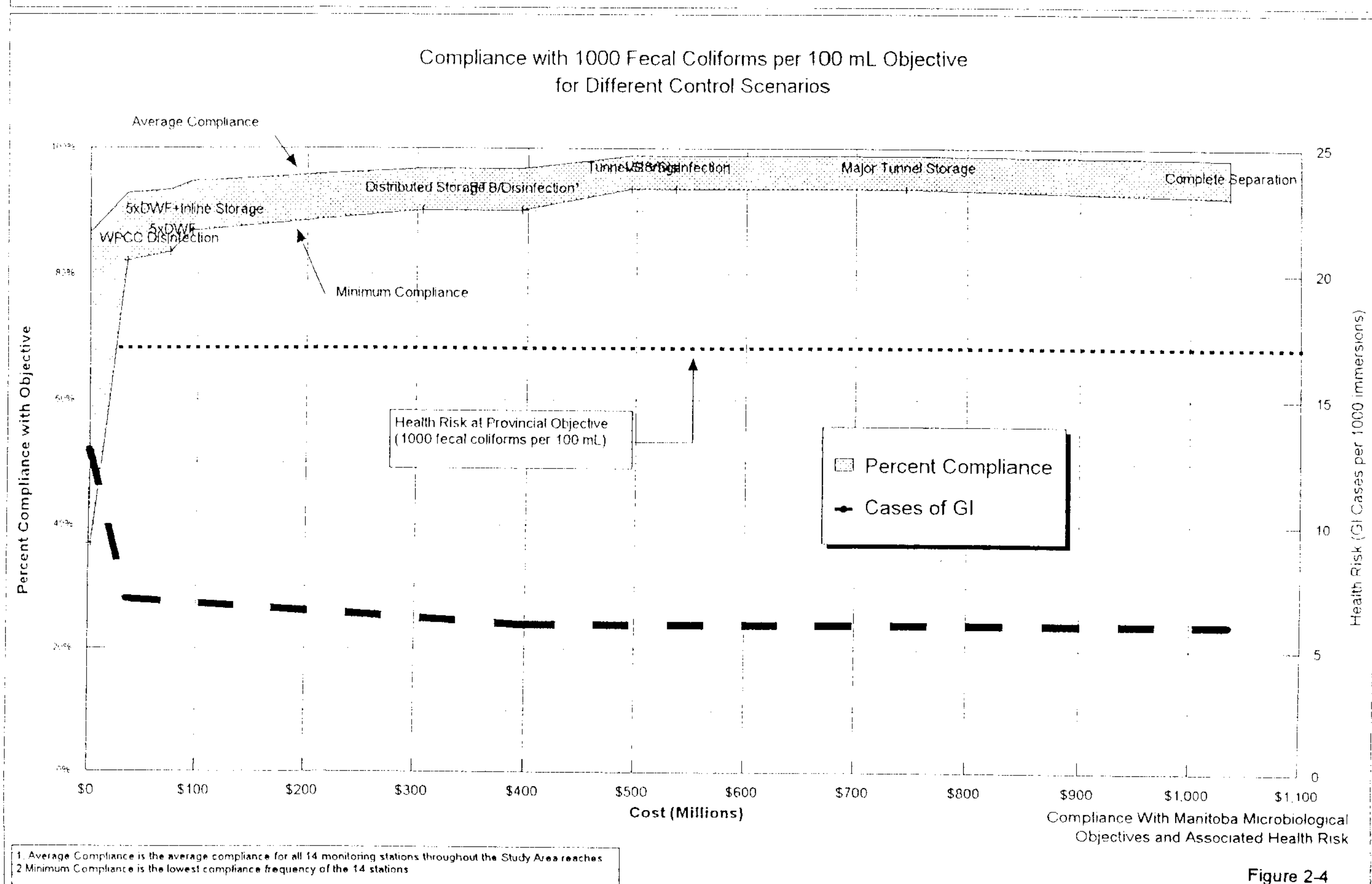
TABLE 2-1
(Source: T.M. No. 6)

POTENTIAL COMBINATIONS OF CSO TECHNOLOGIES

CONCEPTUAL OPTIONS		ROLE
1)	Disinfect WPCC effluent and DWO corrections	Common to all
2)	Intercept 5 X DWF	Supplemental to 1
3)	In-line storage and 5 x DWF	With 1 comprises first stage of WWF control
4a)	Distributed Storage (300,000 m^3)	Supplemental to 1 & 3
4b)	Tunnel Storage (300,000 m^3)	Supplemental to 1 & 3
4c)	Regional Tunnel Storage (1,000,000 m^3) - Eliminate CSO	Supplemental to 1
5)	Full CSO disinfection (this could be partial)	Supplemental to 1 & 3
6)	Full CSO separation	Supplemental to 1
7)	Floatables Removal	Supplemental to 1 & 3

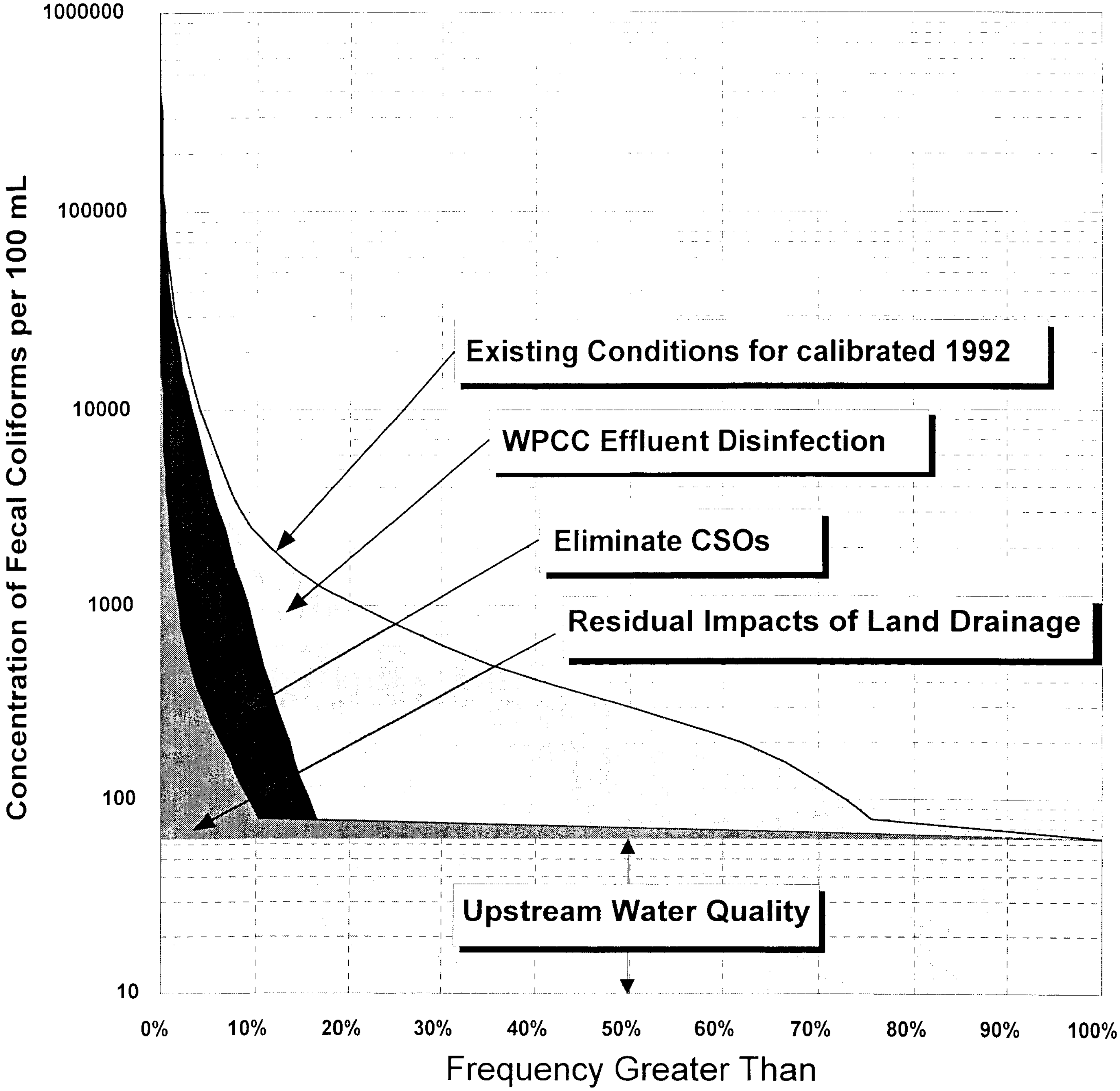
For all combinations, the correction of DWOs and the disinfection of WPCC effluents is common. For most logical combinations, the optimization of existing infrastructure is also a common component.





Compliance With Manitoba Microbiological Objectives and Associated Health Risk

Figure 2-4
Source TM 2-6



Note: All modelled segments, upstream of Winnipeg to Lockport, were used in this assessment

Accumulative Frequency of Modelled Fecal Coliforms Concentrations for Various Control Scenarios

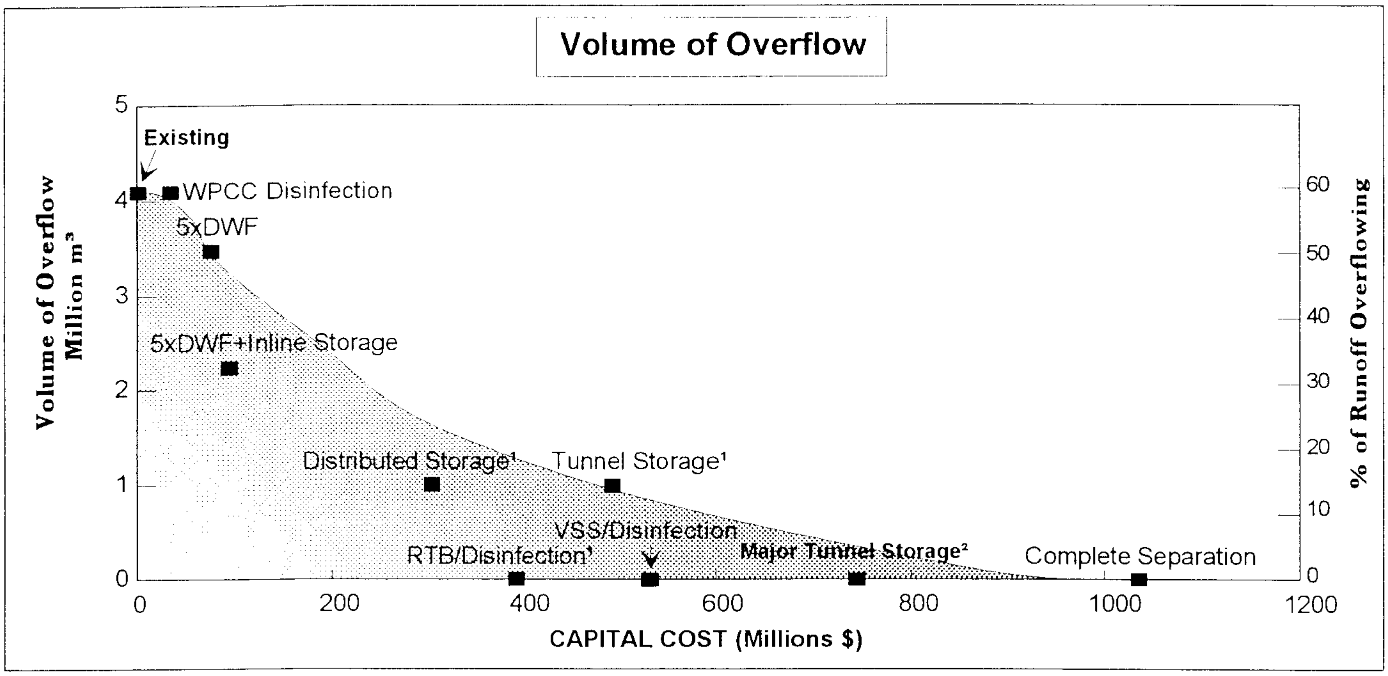
Figure 2-5

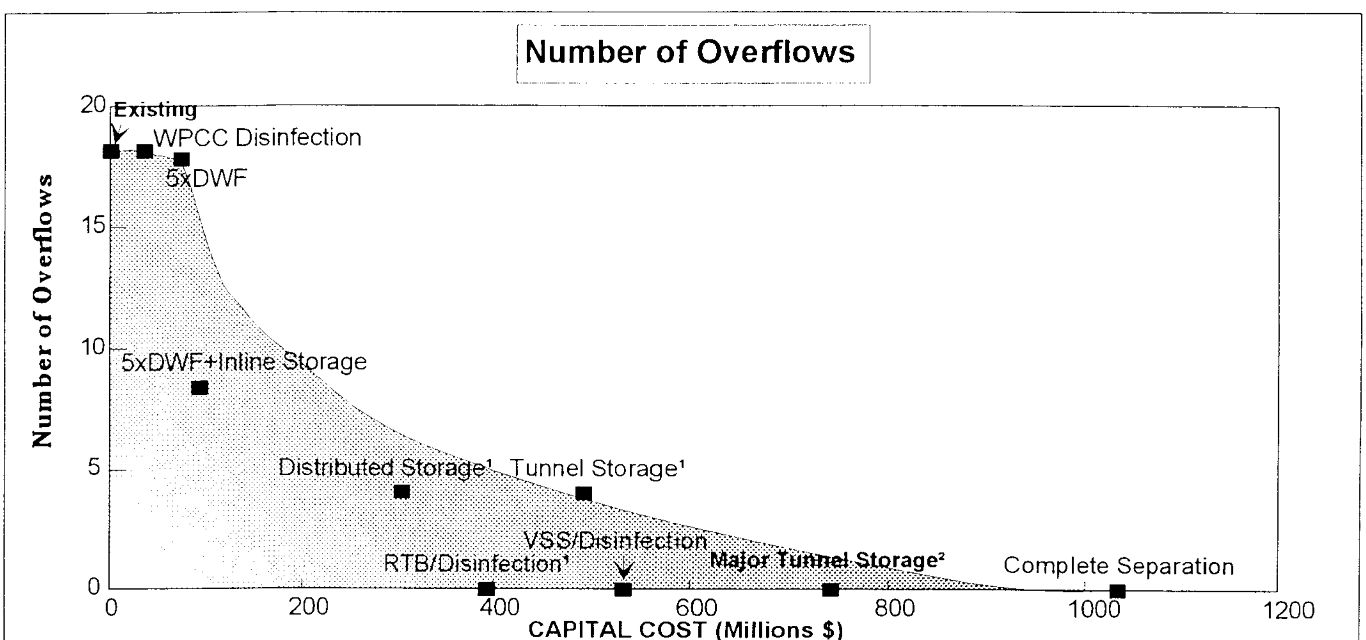
s:\01_files\0510_wrd\fcfreq.DRW

CSOs UNDER DIFFERENT CONTROL SCENARIOS

Option		Description	Volume of Overflow: Million Cu. M.	Volume of Overflow: % of Runoff	Number of Overflows: Average of Districts	Number of Overflows: % of Existing	CAPITAL COST: Option Millions	CAPITAL COST: Cumulative Millions
		Runoff	6.96	100%				
		Existing	4.09	59%	18.2	100%	$0	$0
1a	DWF	DWO Correction	4.09	59%	18.2	100%	$2	$2
1b		WPCC Disinfection	4.09	59%	18.2	100%	$33	$35
2a	System	5xDWF	3.47	50%	17.8	98%	$40	$75
2b	Optimization	5xDWF+Inline Storage	2.25	32%	8.4	46%	$20	$95
3a	Storage	Distributed Storage[1]	1	15%	4	22%	$210	$305
3b		Tunnel Storage[1]	1	15%	4	22%	$400	$495
3c		Major Tunnel Storage[2]	0	0	0	0	$650	$745
4a	Disinfection	RTB/Disinfection[1]	0	0	0	0	$300	$395
4b		VSS/Disinfection	0	0	0	0	$440	$535
5	Separation	Complete Separation	0	0	0	0	$1,000	$1,035
6a	Floatable	Trash Netting	2.25	32%	8.4	46%	$30	$125
6b	Control	Screening	2.25	32%	8.4	46%	$110	$205

Note: 1 Assumes 300,000m³ of Storage. Results are estimates Only
Assumes 1,000,000m³ of Storage. Results are estimates Only





numb-vol
01_files

Figure 2-6
Source: TM 2-6

performance than separation of the sewer systems. The reductions in health risk for any of the WWF control options is very slight.

The performance of the various options, from the perspective of control volume and number of overflows, is illustrated on Figure 2-6. For a relatively small investment, significant reductions of both volume and numbers are achieved through the implementation of in-line storage with enhanced interceptor flows. The % capture improves from the existing 40% to about 68% and the number of overflows is reduced about 50%. All further reductions in number and volume of overflow involve much larger capital investments.

Floatables capture have virtually no impact on fecal coliform concentrations in the river nor would it impact volume or number of overflows. It is possible that selective use of floatables capture might be appropriate for specific CS districts, but it does not appear to be an appropriate system-wide control technology in itself.

2.5 PHASE 2 OVERALL EVALUATION OF TECHNOLOGY

The Phase 2 analysis led to the following observations:

2.5.1 Dry Weather Flow

- DWO corrections are the first priority action and should be addressed as soon as possible and should be investigated on a continuing basis, as is now the case.
- Best management practices (BMPs), including public education programs, should form part of any CSO control program.
- WPCC effluent disinfection is a logical first-step in any CSO control program that involves microbiological control. The measure does not affect WWF discharges but it does provide a

modest reduction in public health risk. More importantly, however, it probably provides a significant benefit in improved public and regulatory perception.

2.5.2 Optimizing Existing Infrastructure

- Increased interception rates and/or developing in-line storage would provide a significant reduction in the number and volume of overflows at a relatively low cost. This would likely translate into improved public perception and environmental stewardship.

2.5.3 Structurally-Intensive Options

- During Phase 2, it was concluded that distributed storage, either in the form of near-surface tanks or localized tunnels, are practical methods of additional control.
- High-rate treatment at the different outfalls has the potential for additional control, if improved coliform control is the priority, although capital and operating costs are high.
- Separation of the existing combined sewer system and regional storage/conveyance tunnel systems appeared to be prohibitively expensive in light of the modest additional benefit which might result from their implementation. During the course of the Phase 2 Workshop, it was decided to carry the separation and regional tunnel options forward to the Phase 3 investigations. This would allow refinement of these options, particularly the regional tunnel, and also provide the public with the ability to assess the relative merits of the full spectrum of control options.

2.6 OVERVIEW

The Phase 2 screening process addressed the technologies in conceptual terms. The specific application of these technologies was studied further in Phase 3. The intent of the Phase 3 activities was to consider combinations of technologies and thus identify control plans which will respond to the full range of possible control objectives and to employ their different performance and technical characteristics. The Phase 3 focus is illustrated on **Figure 2-7**, taken from Phase 2 TM No. 6.

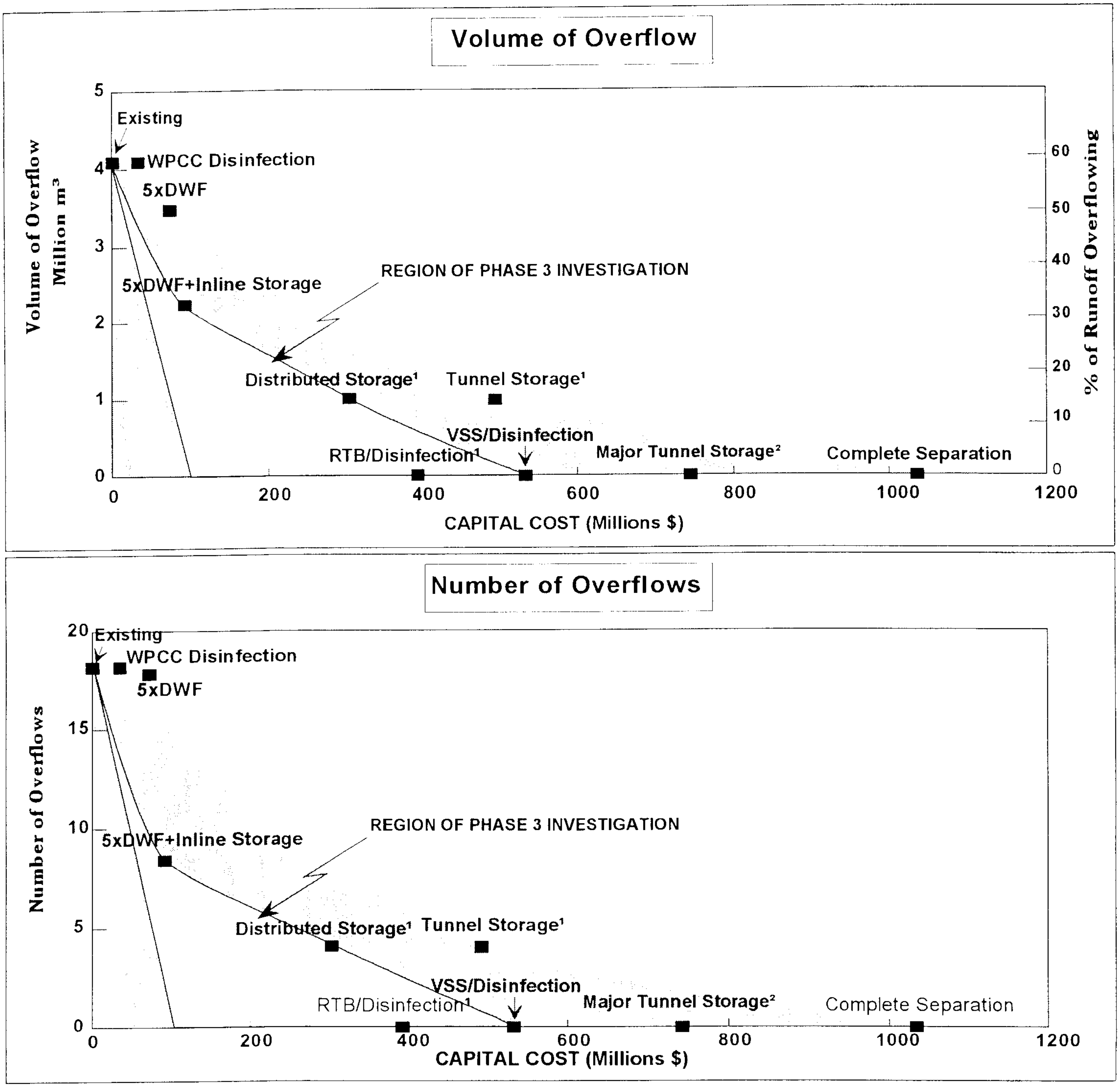


Figure 2-7
Source: TM 2-6

3. APPROACH TO PERFORMANCE ASSESSMENT

Phase 3 studies will identify a range of CSO control plans. In order to choose the most appropriate control plan, many factors must be considered. One of the most important considerations is the different performance or benefit achieved by the different control measures.

Assessment of performance requires consideration of the CSO control goals or objectives, the uses of the local rivers and the related water quality issues (discussed in Section 2.2), particularly as these matters pertain to wet weather conditions. This section proposes a set of performance measures to be used to characterize the ability of the different control plans to respond to different control targets, and discusses the approach to simulation of the performance of different control plans over an evaluation time period.

3.1 PERFORMANCE MEASURES

In considering methods to measure the different performances of alternative CSO control plans, it is useful, firstly, to consider the overall goals of the City of Winnipeg in terms of developing CSO control strategies.

The key product of the CSO Management Strategy for the City of Winnipeg is to establish a cost-effective prioritized implementation plan(s) for remedial work based on assessment of costs and benefits of practicable alternatives. The following goals of the study provide context to this objective:

1. provide protection for the beneficial uses of the Red and Assiniboine Rivers, including the aquatic ecosystems of the rivers;

2. respond to the reasonable expectations of the public and stakeholders;

3. recognize the provincial surface water quality objectives for the Red and Assiniboine Rivers;
4. consider the prevailing environmental practices and policies in terms of CSO control in Canada and the USA; and
5. ensure that any recommended remedial work be implemented in such a fashion as to not increase risk to basement flooding.

The study is intended to develop a range of alternative control plans which address the above goals and which consider the costs, benefits, practicability, affordability and cost-effectiveness of alternative control strategies. The study program is designed to communicate this information to policy-makers and interested publics and to facilitate informed judgments and decision-making.

Measures of performance are needed to allow informed evaluation of the benefits and drawbacks of different control plans.

Due to complexities involved in measuring the performance of different CSO control measures, such as the inherent variability of wet weather flows, the difficulty of measuring wet weather impacts on the stream, etc., there is little specific guidance from regulatory agencies or from experience elsewhere as to how the relative effectiveness of different control measures should be gauged. The prevailing practice is towards adapting general objectives to site-specific control indicators relevant to the local conditions.

A review of experience elsewhere with regard to control policy, performance measures or control indicators was undertaken. This experience was used to develop performance objectives for the purpose of evaluating performance of different control strategies for the City of Winnipeg.

A number of sources were explored in terms of evolving CSO control guidance or policy. These included the draft CSO Control Policy developed by the Environmental Protection Agency (EPA) of the USA, other Canadian provincial policies, and the Association of Metropolitan Sewerage Agencies (AMSA). Each source is discussed below.

3.1.1 EPA Control Policy

The USA has about 1,100 communities with combined sewers, with about 43 million people serviced by combined sewers (CSO, Guidance for Long-Term Control Plan, EPA 1995). The EPA has been involved in a lengthy process of developing and negotiating CSO control policy with state agencies, CSO communities, and environmental groups. While a National Combined Sewer Overflow Control Policy has been signed (EPA 1992), the negotiation of specific policies continues.

The EPA developed its CSO control policies with the expectation that interaction would occur between federal interests, such as the *Clean Water Act* (CWA) and the Natural Pollutant Discharge Elimination System (NPDES) permitting authority, and state water quality standards (WQS) authorities. In this regard, the EPA CSO Control Policy states that CSOs are point source discharges subject to NPDES permits and to CWA requirements (EPA 1995).

The EPA CSO Control Policy has three objectives:

- to ensure that if CSOs occur, they are only as a result of wet weather;
- to bring all wet weather CSO discharge points into compliance with the technology-based and water quality-based requirements of the CWA; and
- to minimize the impacts of CSOs on water quality, aquatic biota, and human health.

The EPA intended that there be a consistent national approach to controlling CSOs but also recognized the site-specific characteristics of combined sewer systems, the local receiving waters, water uses, and the significant cost implications of CSO control.

The CSO policy contains a number of key principles, which are:

- provide clear levels of control that would be presumed to meet appropriate health and environmental objectives;
- provide sufficient flexibility to municipalities, especially those that are financially disadvantaged, to consider the site-specific nature of CSOs and to determine the most cost-effective means of reducing pollutants and meeting CWA objectives and requirements;
- allow a phased approach for implementation of CSO controls considering a community's financial capability; and

- review and revise, as appropriate, WQS and their implementation procedures when developing long-term CSO control plans to reflect the site-specific wet weather impacts of CSOs.

The EPA CSO Control Policy also outlined a number of expectations, including the following:

- Permittees should implement the nine minimum controls (NMCs), which are technology-based actions or measures designed to reduce CSOs and their effects on receiving water quality, as soon as practicable but no later than January 1, 1997.

- Permittees should give priority to environmentally sensitive areas.

- Permittees should develop long-term control plans (LTCPs) for controlling CSOs. A permittee may use one of two approaches;
  1) demonstrate that its plan is adequate to meet the water quality-based requirements of the CWA ("demonstration approach"); or
  2) implement a minimum level of treatment that is presumed to meet the water quality-based requirements of the CWA, unless data indicate otherwise ("presumption approach").

- WQS authorities should review and revise, as appropriate, State WQS during the CSO long-term planning process.

- NPDES permitting authorities should consider the financial capability of permittees when reviewing CSO control plans.

The NMC referred to are outlined in Table 3-1.

The LTCP should be a comprehensive plan that recognizes the site-specific nature of CSOs and their impacts on receiving waters. It should provide for site-specific, cost-effective CSO controls that will meet state water quality standards while having flexibility to recognize affordability. The long term planning approach consists of four major elements: system characterization; development and evaluation of alternatives; selection and implementation of controls; and compliance monitoring. A primary objective of the LTCP is to develop and evaluate a reasonable range of CSO control alternatives sufficient to meet water quality

TABLE 3-1

SUMMARY OF THE NINE MINIMUM CONTROLS

1. Proper operation and regular maintenance programs for the sewer system and the CSOs. This control should consist of a program that clearly establishes operation, maintenance, and inspection procedures to ensure that a CSS and treatment facility will function in a way to maximize treatment of combined sewage and still comply with NPDES permit limitations.
2. Maximum use of the collection system for storage. This control consists of making relatively simple modifications to the CSS to enable the system to store wet weather flows until downstream sewers and treatment facilities can handle them.
3. Review and modification of pretreatment requirements to ensure that CSO impacts are minimized. The objective of this control is to minimize the impacts of discharges into CSSs from non-domestic sources during wet weather events, and to minimize CSO occurrences by modifying inspection, reporting, and oversight procedure within an approved pretreatment program.
4. Maximization of flow to the POTW for treatment. This control entails simple modifications to the CSS and treatment plant to enable as much wet weather flow as possible to reach the treatment plant.
5. Elimination of CSOs during dry weather. This control includes any measures taken to ensure that the CSS does not overflow during dry weather conditions.
6. Control of solid and floatable materials in CSOs. This control is intended to control, if not eliminate, visible floatables and solids using relatively simple measures including baffles, screens, racks, booms and skimmer vessels.
7. Pollution prevention programs to reduce contaminants in CSOs. This control is intended to keep contaminants from entering the CSS and prevent subsequent discharge to receiving waters through street cleaning, public education, solid waste collection and recycling.
8. Public notification to ensure that the public receives adequate notification of CSO occurrences and CSO impacts. The intent of this control is to inform the public of the location of outfalls, the actual occurrence of CSOs, the possible health and environmental effects of CSOs, and the recreational or commercial activities curtailed as a result of CSOs.
9. Monitoring to effectively characterize CSO impacts and the efficacy of CSO controls. This control involves visual inspections and other simple methods to determine the occurrence and apparent impacts of CSOs.

Source: CSOs: Guidance for Nine Minimum Controls. EPA 1995b.

standards, including attainment and protection of designated uses on CSO-impacted receiving waters. The CSO Policy is flexible in that it allows a CSO community to select controls that are cost-effective and tailored to meet local conditions.

With respect to attainment of WQS, the EPA Policy provides the municipalities with two approaches for showing that its selected CSO controls will achieve water quality standards:

- **"Demonstration Approach"** - in this approach, the municipality can provide information and data showing that the selected CSO controls meet water quality standards.

- **"Presumption Approach"** - in this approach, the municipality can provide a particular level of control that is presumed to meet water quality standards unless there is data to show otherwise. These specified levels of control are:

    - no more than an average of four overflow events per year with the provision that up to two additional overflow events may be allowed per year;

      or

    - the elimination or capture for treatment of no less than 85% by volume of the combined sewage collected in the combined sewer system on a system-wide annual average basis;

      or

    - the elimination or removal of no less than the mass of pollutants, identified as causing water quality impairment, for the volumes that would be eliminated or captured for treatment under the previous point.

The above two approaches, especially the presumption approach, have become performance targets for many US communities.

The EPA has clarified the presumption approach with the following guidance:

- For the above purposes, an overflow event is one or more overflows from a combined sewer system that does not receive minimum treatment (clarification, solids removal, disinfection, if necessary).

- In a CSS with three outfalls, therefore, if one, two, or three of the outfalls discharge untreated or inadequately treated combined sewage during a rain event, then a single overflow event has occurred.

- With respect to volume capture of combined sewage, the intent of EPA is to capture "85 percent by volume of the combined sewage". This refers to 85 percent of the total volume of flow collected in the CSS during precipitation events on a system-wide, annual average basis (not 85 percent of the volume being discharged). In other words, no more than 15 percent of the total flow collected in the CSS during storm events should be discharged without receiving the minimum specified treatment. The total volume of flow collected during wet weather on a system-wide annual average basis is most readily computed using a model of the CSS.

EPA believes the capture of 85% volume of combined sewage corresponds on average to the target of 4 to 6 overflows per year. It should be noted that this does not seem to be true for Winnipeg, probably due to the characteristic pattern of several large rainfalls contributing the major volume of the runoff each year.

The foregoing discussion illustrates the general principles in the EPA policy. EPA also encourages a coordinated effort between stakeholders to develop "measures of success". EPA supported the development of national measures of success (see discussion of AMSA below) but also noted that measures of success will vary from one location to another and will need to be determined on a site-specific basis.

3.1.2 Association of Metropolitan Sewerage Agencies (AMSA)

AMSA, in a study supported by EPA, developed a range of performance measures in 1996 (Performance Measures for the National CSO Control Program). The study built on four categories of potential measures of success proposed by EPA, namely:

- Administrative Measures
  - mainly for national tracking
- End-of-Pipe Measures
  - CSO frequency, volume, etc.

- Receiving Water Measures
  - in-stream concentrations; compliance with water quality criteria
- Ecological/Health/Resource Use Measures
  - restored habitat, reduced beach closures, etc.

A review of the potential performance measures developed by AMSA indicates that the "administrative measures" are meant for national tracking of CSO control activities. The "end-of-pipe" and "receiving water" measures are incorporated in the EPA "presumption" and "demonstration" approach and the "ecological/health/resource use" measures must be developed for the site-specific situation. Accordingly, the AMSA study provides little guidance beyond the EPA policy.

3.1.3 Province of Ontario

There are numerous communities in Ontario with combined, or partially-combined, sewer systems. Ontario advanced a CSO Control Procedure in 1997 (Procedure F-5-5, Determination of Treatment Requirements for Municipal and Private Combined and Partially Separated Sewer Systems).

The goals of the Policy 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*) at beaches impacted by CSOs for at least 95% of the four-month period (June 1 to September 30) for an average year.

Ontario provides the following clarification on their policy:

- 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.

- 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.

- An "average year" is:
  i) the long term average 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 "beach" is a strip of shoreline with the physiographic, climatic, access, and ownership attributes necessary to accommodate significant water contact and non-contact recreation.

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

- develop a Pollution Prevention and Control Plan (PPCP);
- meet specified minimum CSO controls; and
- provide additional controls:
  - for beaches impaired by CSOs where water quality is not meeting the PWQO for E. coli;
  - where required by other receiving water quality conditions.

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

The minimum CSO Controls are similar to the EPA except that Ontario proposes that the capture of wet weather flow should be 90%, i.e., the dry weather flow should be captured for treatment plus 90% of the average flow above this level resulting from wet weather. The minimum level of treatment should be primary treatment.

3.1.4 Province of Alberta

Although Edmonton is the only community in Alberta with combined sewers, the Alberta Environmental Protection Agency (AEP) has a policy on CSOs, as follows:

1. No new combined sewer systems or additional combined sewer overflows will be allowed in Alberta. The AEP's policy is to encourage the development of a comprehensive, cost-effective control strategy that will result in minimizing the environmental impacts of the Edmonton's combined sewer system. This may include immediate separation on a limited and opportunistic basis. Separate storm and sanitary sewers are to be used for new systems. AEP policy is to encourage ultimate, i.e., 50 to 100 years, elimination of CSOs or measures that would result in an equivalent or better level of environmental protection than would be achieved by complete separation.

2. Existing combined systems should be separated where possible, as old sewers are replaced or upgraded. It is recognized that a program extending over decades may be required. Alternative mitigative measures (e.g., storage, satellite treatment, relief sewers, etc.) should be used to control CSO impacts to acceptable levels.

3. Existing combined systems will be allowed to continue on an interim basis provided a CSO control strategy is developed to determine what interim treatment or reduction of CSOs is desirable in the near term, i.e., 5 to 25 years. A long term, i.e., 25 to 50 years, CSO mitigation strategy must also be developed and include public consultation and receiving stream environmental assessments. As a minimum, the City should:

   - determine methods to eliminate any dry weather overflows and implement immediately;
   - characterize the CSO quantity and quality;
   - determine the areal extent and the storm conditions beyond which Alberta Ambient Surface Water Quality Guidelines cannot be met;
   - evaluate mitigation methods to achieve water quality objectives;

- evaluate non-structural best management practices for CSOs and implement cost-effective controls;
- outline an implementation plan to cost-effectively mitigate CSO impacts in the long term, i.e., 25 to 50 years; and
- establish general timelines and schedules to achieve ultimate, i.e., 50 to 100 years, control objectives that either involve complete separation or have control measures that achieve an equivalent or better level of environmental protection than would be achieved through complete separation.

3.1.5 Province of Manitoba

Manitoba has no special permitting policies relating to CSOs at this time. Several communities have CSSs, with Winnipeg being by far the largest. The CEC has directed that wet weather water quality objectives for the Red and Assiniboine rivers within and downstream of Winnipeg be reviewed. The current study, with special focus on microbiological water quality, was recommended, with general public, scientific, and other stakeholder input. This Technical Memorandum addresses the results of Phase 3 of that study. At the end of the study, the results will be reviewed by the regulatory agencies in a public hearing process.

In reviewing specific objectives for a watershed, particularly when the existing water quality is currently impaired thus affecting either a present or future water use, an evaluation is recommended. This evaluation is guided by the following general questions:

1. Which water uses are being impaired?
2. What are the water quality variables causing the impaired use?
3. To what extent do human activities contribute to the impairment?
4. What level of control is required to ameliorate the water quality exceedances?
5. Do control technologies actually exist in order to achieve the level of reclamation necessary?
6. Does the cost of achieving the water quality improvement bear a reasonable relationship to the benefits associated with attaining the water use?

Depending upon the result of this evaluation, surface water quality objectives could be recommended for the area under consideration such that the existing impaired water quality would be accepted. Alternatively, objectives could be recommended that would provide the basis for a plan that would improve water quality to the level necessary to protect the presently affected water use. For the current study, the answers to questions 1, 2 and 3 were, at least in part, arrived at in the 1991-1992 CEC Hearings. The current study is expanding on these answers plus developing the answer to questions 4 and 5. The CEC Hearings will develop the answer to question 6.

3.1.6 Proposed Measures of Performance for Winnipeg

Drawing from the review of the overall goals of the CSO study, water quality issues, guidelines for surface water quality, and experience elsewhere, potential CSO performance measures are proposed for the purpose of measuring the relative performance of different CSO control options for Winnipeg. These measures are intended to characterize the key strengths and weaknesses of the different alternatives. In so doing, the performance assessment will assist understanding of the options, their evaluation, and facilitate "trade-off" judgements. These potential performance measures are condensed into the list of proposed control indicators shown in Table 3-2.

It is expected that a range of control plans will be developed to display the full range of control options and the different performance characteristics, as measured by indicators of CSO control. The range of control plans will begin with assessment of the existing baseline situation, various levels of incremental control, such as optimizing use of the infrastructure, adding storage, etc., including the complete separation of the existing combined sewer system. In the identification of potential control plans, the "benchmarks" of 4 CSOs and 0 CSOs, 85% volumetric control and compliance with Manitoba objectives, will be used to define candidate control plans.

3.2 MODELLING APPROACH

3.2.1 Overview

In order to assess the performance of various control alternatives, and to compare them to the existing conditions and other alternatives it is necessary to develop and use modelling tools.

TABLE 3-2

PROPOSED MEASURES OF CSO CONTROL

	PEFORMANCE MEASURE	INTENT	REMARKS
1.0	"End-of-Pipe" Measures		
1.1	Number of CSOs	- to minimize # of overflows in the CS system to the Receiving Stream (individual and total)	- it is expected that the existing baseline performance, plans that will optimize the existing system, and incremental plans that reduce CSOs to benchmarks of about 4/year and about 0 CSOs will be developed
1.2	Volume of CSOs	- to minimize volume of CSO from the CS outfalls (individual and total)	- the volume capture of the existing baseline system, incremental improvements as above, will be assessed
1.3	Secondary Bypasses at NEWPCC	- to maximize flow to the NEWPCC for treatment	- # of times and volume of bypass of the secondary process at NEWPCC will be defined for different plans
2.0	Receiving Stream Measures		
2.1	Duration of Compliance with Primary Recreation Fecal Coliform Guidelines	- to achieve, to the extent practicable & cost-effective, compliance with Environment guidelines	- # of hours of compliance with 200 fc/100 mL guidelines at different locations during the recreation season will be defined for different plans
2.2	Duration of Compliance with Secondary Recreation Fecal Coliform Guidelines	- to achieve, to the extent practicable & cost-effective, compliance with Environment guidelines	- # of hours of compliance with 1000 fc/100 mL guidelines at different locations during the recreation season will be defined for different plans
2.3	Human Health Risk	- to estimate the incremental human health risk associated with CSOs arising from use of the Receiving Stream	- estimated disease, as predicted by dose-response models using fc densities and river use, will be estimated for different locations and control plans
2.4	Pollutant Loading		- the estimated mass-loading of nutrients, metals, TSS, from the CSOs will be estimated for different plans (volume of CSO will be used to provide this information)
2.5	Aesthetics	- to estimate changes in aesthetic impacts related to odours, floatables, visibility of outfalls, etc.	- # and volume of CSOs will be used as an indicator for aesthetic performance
2.6	Protection of Sensitive Reaches of Red and Assiniboine Rivers	- intent is to reduce or eliminate CSOs in especially-sensitive reaches	- the location of CS outfalls relative to sensitive river uses will be used
2.7	Protection of Aquatic Life in Red and Assiniboine Rivers	- intent is to minimize adverse effects of CSOs on aquatic life	- the DO resources in the Red and Assiniboine Rivers are ample; ammonia from CSOs are not expected to be toxic to fish. Volume of capture of CSOs will serve as a measure for ammonia control

These tools were used to define the infrastructure upgrades required to meet the different performance measures discussed in the previous section (number of overflows, volume of overflows and compliance with receiving stream water quality objectives). This section describes these modelling tools and the manner in which they were applied to define the storage/treatment elements of the various alternatives developed in Section 5. This information was then used to evaluate the candidate alternatives in Section 6.

This section will discuss the general approach to modelling used in this CSO management study and the specific approach used in Phase 3 to size control alternatives and evaluate their performance. The general approach assesses the impacts of all sources of wastewater on the Red and the Assiniboine rivers, from land drainage, sanitary sewer overflows, treatment plant effluents as well as combined sewer overflows. The more specific Phase 3 approach focusses on the combined sewer districts and the North End Water Pollution Control Centre (NEWPCC) and their impact (existing situation and with different upgrades) on CSO control. These two approaches are described in more detail in the following sub-sections.

3.2.2 General Modelling Approach

To evaluate the effects or benefits of the CSO control options, it is necessary to develop the loadings from CSOs, in terms of number of events, volumes, fecal coliform, and other parameters and place in perspective with the background river quality, other stream loadings such as LDS, plant effluents during DWF and WWF, etc. The temporal and spatial distribution of loadings is also necessary in order to assess the impacts of various control alternatives on the overflows to the river. A data management system was developed in order to achieve all of these objectives. This system model receives the intermittent hourly runoff data produced by the XP-SWMM model, combines these values with the continuous DWF information for each district, and produces an inventory of the WWF hydrographs and "pollutographs" for existing conditions and for various control alternatives.

A schematic of the various components in the Regional System is shown in **Figure 3-1**. The shaded area of **Figure 3-1** shows the development of stream loadings, which are input to the overall U.S. EPA WASP river quality model.

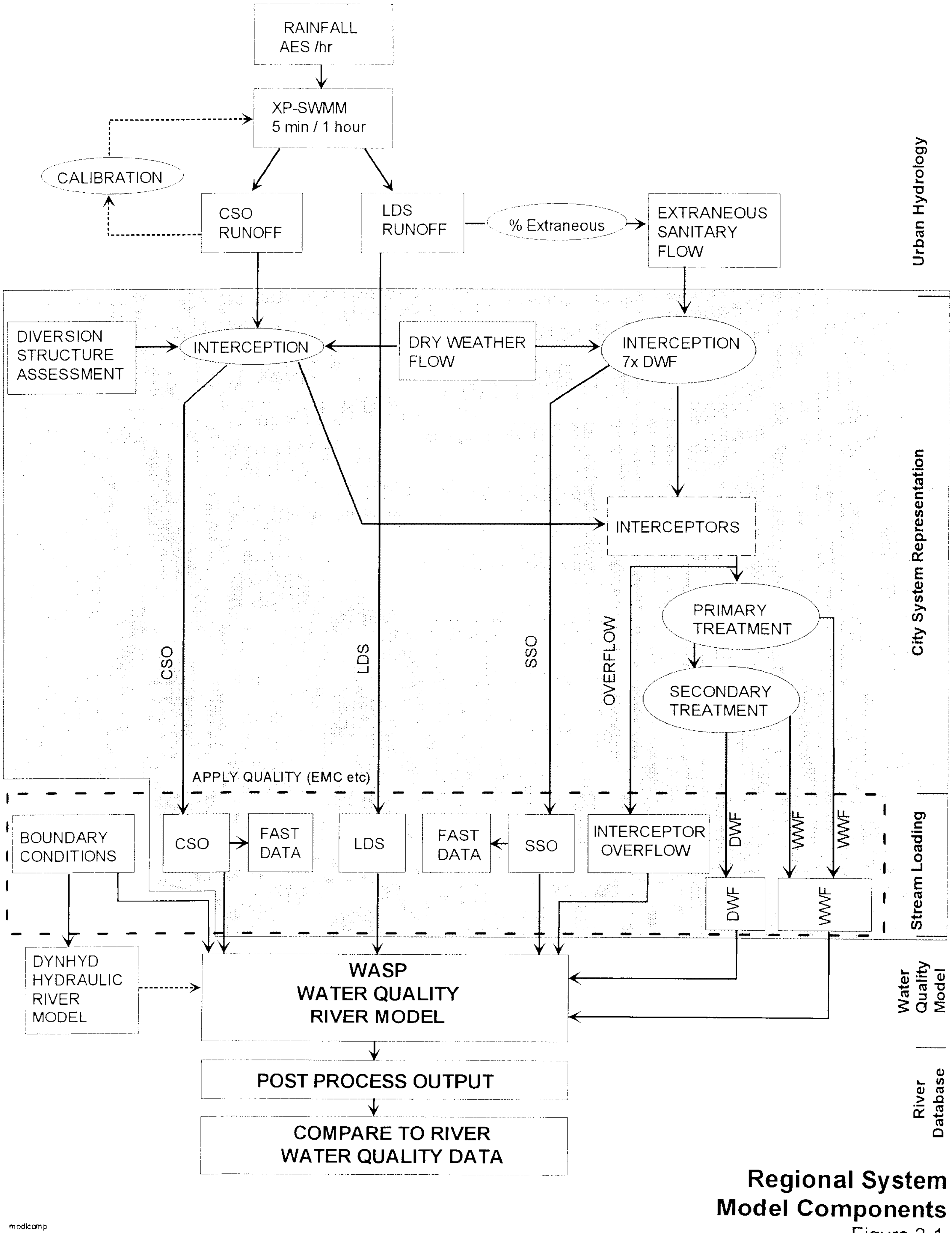
A comprehensive database management system was developed (using Paradox database software) which would "track" all dry weather and wet weather flows and their deposition, either to the interceptor, the WPCCs or to the rivers.

For each district, the data management system tracks the volume of combined sewage on an hourly basis to determine:

- volume intercepted by each interceptor sewer system (i.e., North End, South End or West End);
- the volume going into or out of in-line storage for each district;
- when the storage is filled (the excess is considered to overflow to the river).

This hydrograph (hourly overflow volume) is then processed into a "pollutograph" (by multiplying the volume by the appropriate EMC) and into the appropriate format to be a non-point source (#.NPS) input file for the US EPA WASP receiving stream model (see Phase 2 - TM #4 - Receiving Streams).

The interception or treatment of these flows was allowed for, including adjustment in EMCs. In this way, the mass loadings of the discharges to the rivers was accounted for under existing conditions and then for different control systems. Changing the characteristics of one component of the system will affect other components; in particular, various CSO control methods will impact the other systems. For example, separation of the combined sewer system will increase the amount of land drainage system (LDS) hydrographs into the rivers and separate sanitary sewer (SS) system flows to the WPCCs. This may result in a decreased wet weather flow (WWF) to the Water Pollution Control Centres (WPCCs) but the overall loading to the rivers may not be affected significantly. Increased interception of combined sewage and increased storage capacity will increase the rate, duration and total volume of wastewater sent to the interceptor systems and the



Regional System
Model Components
Figure 3-1

WPCCs and may result in reduced loadings to the rivers, even if treatment efficiencies at the WPCCs are impaired due to the WWF.

The data management system is essentially a mass balance model of the existing system and potential control systems using the results of the area-wide runoff model. This "Control System" model produces large data tables of 3600 records for each recreational year for each district (hourly from May 1 to September 30).

The regional system model *is not* a hydraulic model such as SWMM, TRANSPORT, or EXTRAN. It is a mass balance model which provides a dynamic inventory of hydrographs in hourly timesteps. While conceptually simple, the overall database, including all rainfall data, runoff, interception, etc. is actually massive. (The output file for all systems would be about 50 megabytes of data). This screening model offers strong advantages in that it can quickly assess an alternative for an entire recreation season (far more quickly than a more detailed and complex hydraulic model). The ability to assess alternatives quickly and provide an overall relative perspective, allows a larger number of alternatives to be assessed in a short time frame. The assessment of a wide variety of alternatives is an essential first step to selecting the best plan for the future. This screening process assumes that assessment of the system at a later stage will focus on the best alternatives.

Tributary small streams to the Red and Assiniboine Rivers (Seine River, La Salle River, Sturgeon Creek, Omands Creek, and Bunns Creek) were considered to be an intermittent loading source. The tributary streams around the City of Winnipeg generally serve the purpose of assisting in the conveyance of land drainage to the Red and Assiniboine Rivers. Accordingly, they were considered as part of the land drainage system to the Red and Assiniboine Rivers and were only considered to contribute loadings under wet weather or rainfall events.

In Phase 2, the various models such as the CSO model, the land drainage model, the SSO portion, and flows to the treatment plant, were all calibrated against City records of plant flows and overflows (FAST data). This fecal loading information, along with upstream boundary conditions, was used to model water quality throughout and downstream of the City of Winnipeg. The WASP model was used to produce hourly outputs for the entire recreation season from May 1 to

September 30. Using the data from the representative year (1992) this model was run and the instream data were compared to the 1992 water quality data.

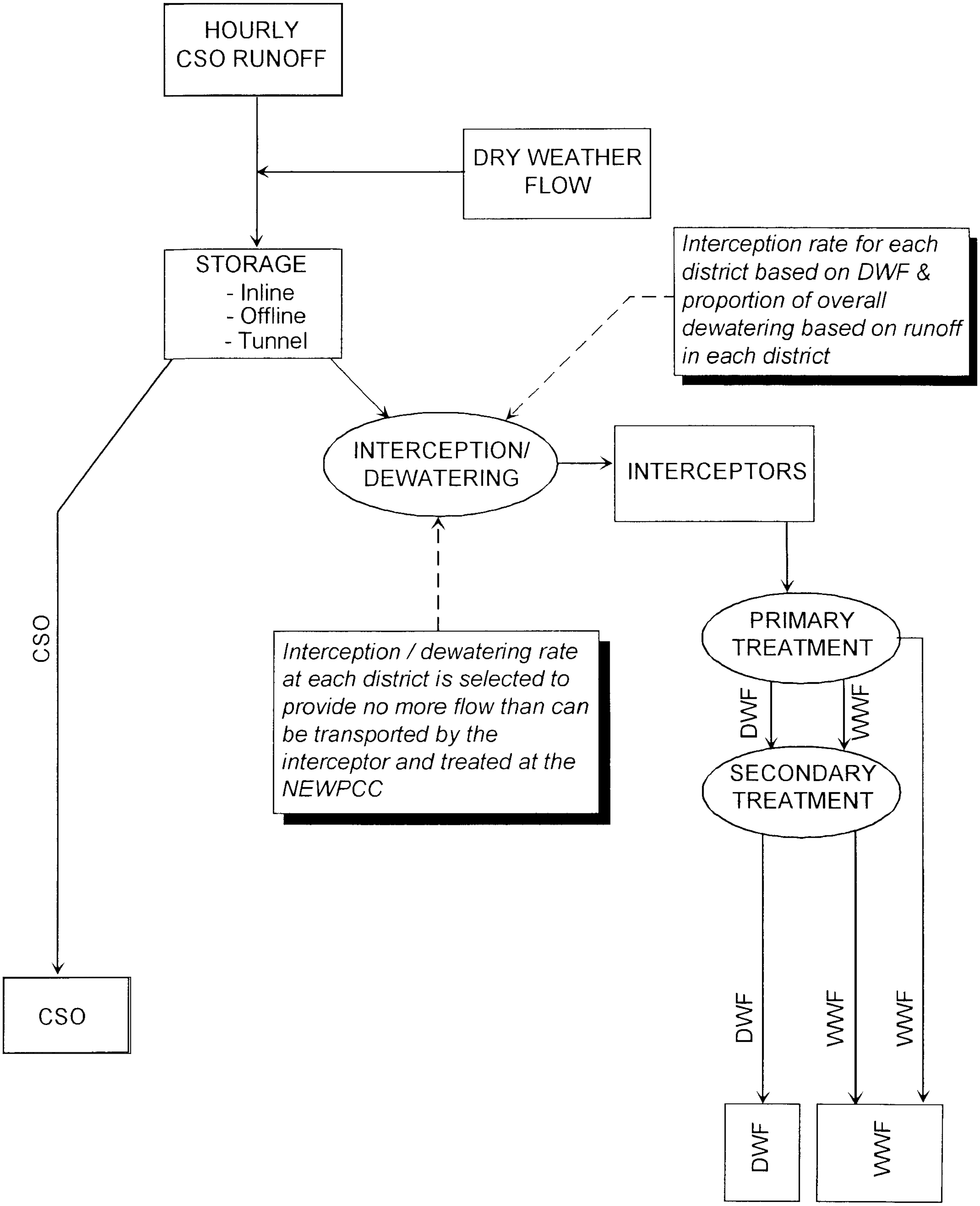
During the calibration/verification stage it was noticed that two significant overflows appeared to be missing. The AES rainfall data for the representative year were double-checked (hourly records were compared to daily records) and it was noted that AES's hourly data were missing two significant rainfalls. When this information was added, it explained the discrepancies in water quality modelling output and data. The representative year was corrected, and this is described later in Section 3.3.

The output from the WASP model can be post-processed to determine compliance to Manitoba's Surface Water Quality Objectives. The two objectives which were considered are the primary recreation objective of 200 fc/100 mL, and the secondary recreation objective of 1,000 fc/100 mL. This information was developed in Phase 3 for a number of options, including separation. This information was still applicable to the Phase 3 evaluation.

This method of assessing the receiving water quality benefits from various control alternatives, considering all wastewater sources to the river, will be used in Phase 4 to estimate frequency and direction of compliance with objectives for a select number of the candidate plans. In Phase 3, however, a specific modelling approach was used to develop and assess dozens of candidate control plans in order to ensure that the entire range of alternatives was considered. This modeling focussed on the number and volume of CSOs (as discussed below). The degree of compliance was estimated from Phase 3 analyses.

3.2.3 Phase 3 Modelling Approach

The modelling efforts in Phase 3 focussed on the key components of the combined sewer system in the regional model which would be directly affected by CSO controls. Figure 3-2 illustrates these components.



Regional System Model Components used in Candidate Option Assessment
Figure 3-2

regmodl
s\01\0510

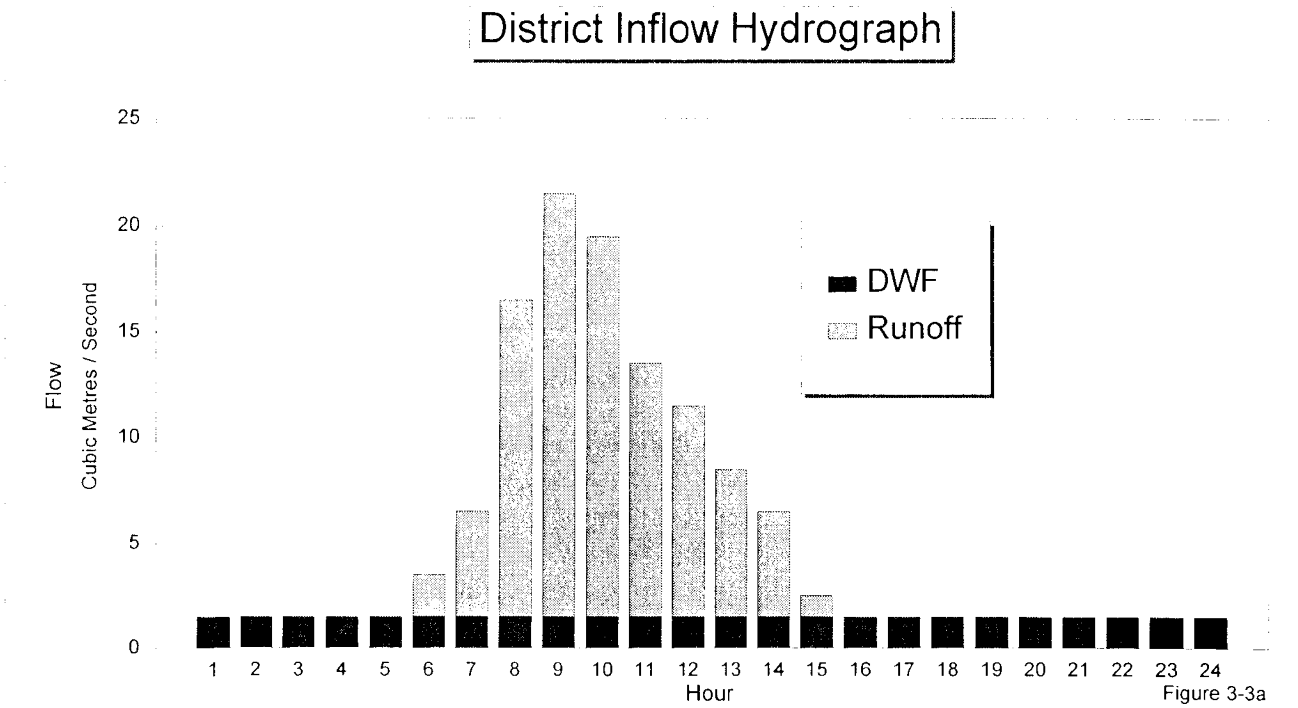
Each Combined Sewer District was modelled to determine the volume of CSO intercepted and the volume of overflow for each hour in the recreation season (May 1 to September 30). The district inflow hydrograph is developed by adding the dry weather flow to the seasonal runoff hydrograph (see Figure 3-3a) developed by the calibrated XP-SWMM model (see Phase 2 - TM #1 - Problem Definition).

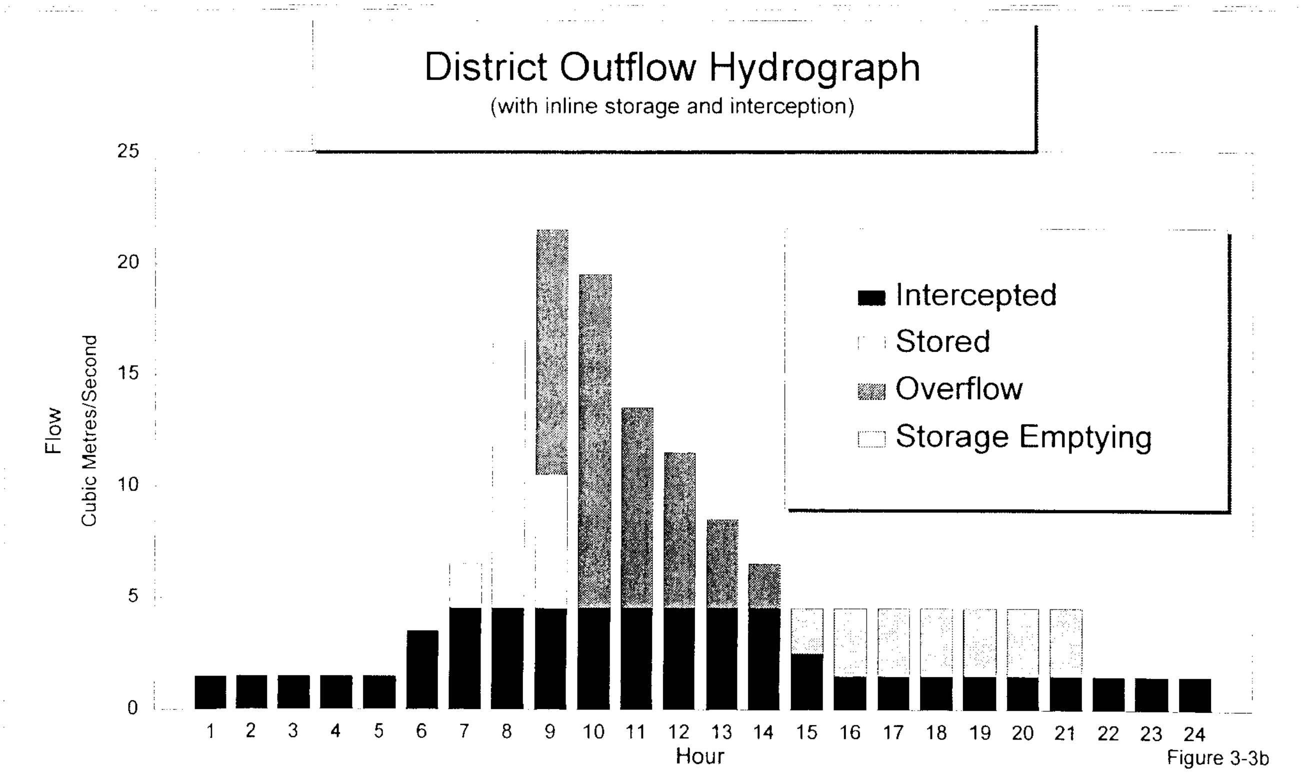
In Phase 2, the dry weather flow was estimated for each district by calculating the January 1993 water consumption from water usage records and multiplying by 1.35 to account for infiltration. It is recognized that the City has been monitoring DWF in a number of districts. At this stage, the impact of (reportedly) high summer DWF in certain districts (i.e., Tylehurst and Cockburn) has not been accounted for in the model. Therefore, dry weather overflows are not explicitly modelled in the system model. The diurnal variations of DWF have not been considered in the model as this is not considered a necessary refinement at this stage. (For further detail on DWF, see Phase 2 - TM #2 - Infrastructure). The assumed DWF for each district in shown in Table 3-3.

The next stage of the control system model determined the impact of various controls on the amount of CSO intercepted and overflowing to the river. The inflow hydrograph was allocated (see Figure 3-3b) using the following values:

- All DWF plus runoff intercepted up to the district interception rate given in Table 3-4. (See Phase 2 - TM #2 - Infrastructure for details on calculations of existing interception rates). New interception rates are discussed later.
- Once the hourly flow rate is greater than the interception rate, the volume is considered to be in storage (in-line or off-line), if storage is available. If no storage is available, the excess flow is considered to overflow to the river.
- Once the storage is full, the excess is considered to overflow.
- If the inflow drops below the interception rate, the storage will begin dewatering at the defined interception rate.

The hourly CSO runoff for each district was calculated using the XP-SWMM model in Phase 2. Dry weather flow estimates were developed using monitoring and water consumption records.





MODELLING OF CSO CONTROL
(INLINE STORAGE AND INTERCEPTION)
Figure 3-3

Table 3-3
Dry Weather Flow
In Combined Sewage Districts

District	District Name	Combined With	DWF (CMS)	Remarks
1	Alexander		0.035	
2	Armstrong		0.02	
3	Ash		0.082	
4	Assiniboine		0.084	
5	Aubrey		0.071	
6	Baltimore		0.028	
7	Bannatyne		0.153	
8	Boyle	Syndicate	0.014	
9	Clifton		0.077	
10	Cockburn		0.033	
10a	Calrossie		0.001	
11	Colony		0.134	
12	Cornish		0.035	
13	Despins	Marion	0.032	
14	Doncaster		0.025	
15	Douglas Park	Ferry Road	0.001	
16	Dumoulin		0.013	
17	Ferry Road		0.059	
18	Hart		0.039	
19	Hawthorne		0.036	
20a	Jefferson E		0.143	
20b	Jefferson W	Jefferson E	0	
21	Jessie		0.066	
22	La Verendrye	Dumoulin	0.009	
23	Linden		0.017	
24	Mager Drive		0.091	
25	Marion		0.032	
26	Metcalfe		0.005	
27	Mission		0.144	
28	Moorgate		0.023	
29	Munroe		0.077	
30	Newton		0.01	
32	Polson		0.032	
33	River		0.07	
34	Riverbend/Parkside Dr.		0.053	
35	Roland		0.026	
36	Selkirk		0.067	
37	St. Johns		0.084	
38	Strathmillan		0.003	
39	Syndicate		0.01	
40	Tuxedo		0.004	
41	Tylehurst		0.05	
42	Woodhaven		0.00227	

EXISTIN.WK4

Table 3-4
Runoff Based Interception Rates
825 ml/d at NEWPCC

District	District Name	Combined With	DWF (CMS)	Existing Rate (CMS)	Runoff Based (CMS)	2x RO Based Installed (CMS)	3x RO Based Installed (CMS)	Remarks
1	Alexander		0.035	0.155	0.230	0.459	0.689	
2	Armstrong		0.02	0.524	0.138	0.275	0.413	
3	Ash		0.082	0.301	0.892	1.784	2.676	
4	Assiniboine		0.084	0.425	0.228	0.455	0.683	
5	Aubrey		0.071	0.214	0.333	0.667	1.000	
6	Baltimore		0.028	0.201	0.153	0.306	0.458	
7	Bannatyne		0.153	0.613	0.342	0.685	1.027	
8	Boyle	Syndicate	0.014	0.03	100.000	100.000	100.000	Assume all goes to Sydicate
9	Clifton		0.077	0.236	0.403	0.807	1.210	
10	Cockburn		0.033	0.075	0.084	0.168	0.252	
10a	Calrossie		0.001	0.028	100.000	100.000	100.000	Assume all goes to Cockburn
11	Colony		0.134	0.425	0.358	0.715	1.073	
12	Cornish		0.035	0.107	0.106	0.211	0.317	
13	Despins	Marion	0.032	0.132	100.000	100.000	100.000	Assume all goes to Marion
14	Doncaster		0.025	0.075	0.144	0.287	0.431	
15	Douglas Park	Ferry Road	0.001	0.095	100.000	100.000	100.000	Assume all goes to Ferry Road
16	Dumoulin		0.013	0.136	0.156	0.312	0.468	
17	Ferry Road		0.059	0.126	0.305	0.610	0.915	
18	Hart		0.039	0.101	0.212	0.423	0.635	
19	Hawthorne		0.036	0.113	0.236	0.472	0.708	
20a	Jefferson E		0.143	0.569	0.652	1.305	1.957	
20b	Jefferson W	Jefferson E	0	100	100.000	100.000	100.000	Assume all goes to Jefferson E.
21	Jessie		0.066	0.176	0.420	0.839	1.259	
22	La Verendrye	Dumoulin	0.009	0.015	100.000	100.000	100.000	Assume all goes to Dumoulin
23	Linden		0.017	0.06	0.046	0.091	0.137	
24	Mager Drive		0.091	0.309	0.309	0.618	0.927	
25	Marion		0.032	0.22	0.340	0.680	1.019	
26	Metcalfe		0.005	0.044	0.015	0.031	0.046	
27	Mission		0.144	0.518	0.435	0.870	1.305	
28	Moorgate		0.023	0.085	0.104	0.208	0.312	
29	Munroe		0.077	0.237	0.470	0.941	1.411	
30	Newton		0.01	0.166	0.073	0.146	0.219	
32	Polson		0.032	0.356	0.279	0.558	0.836	
33	River		0.07	0.094	0.188	0.376	0.565	
34	Riverbend/Parkside Dr		0.053	0.107	0.254	0.507	0.761	
35	Roland		0.026	0.324	0.265	0.530	0.794	
36	Selkirk		0.067	0.453	0.254	0.507	0.761	
37	St. Johns		0.084	0.173	0.459	0.917	1.376	
38	Strathmillan		0.003	0.062	0.031	0.063	0.094	
39	Syndicate		0.01	0.069	0.144	0.287	0.431	
40	Tuxedo		0.004	0.036	0.057	0.114	0.171	
41	Tylehurst		0.05	0.176	0.277	0.553	0.830	
42	Woodhaven		0.00227	0.027	0.039	0.077	0.116	

100 cms asssumed when districts interconnected

The key elements in each district are the amount of storage available and the rate of interception or dewatering available.

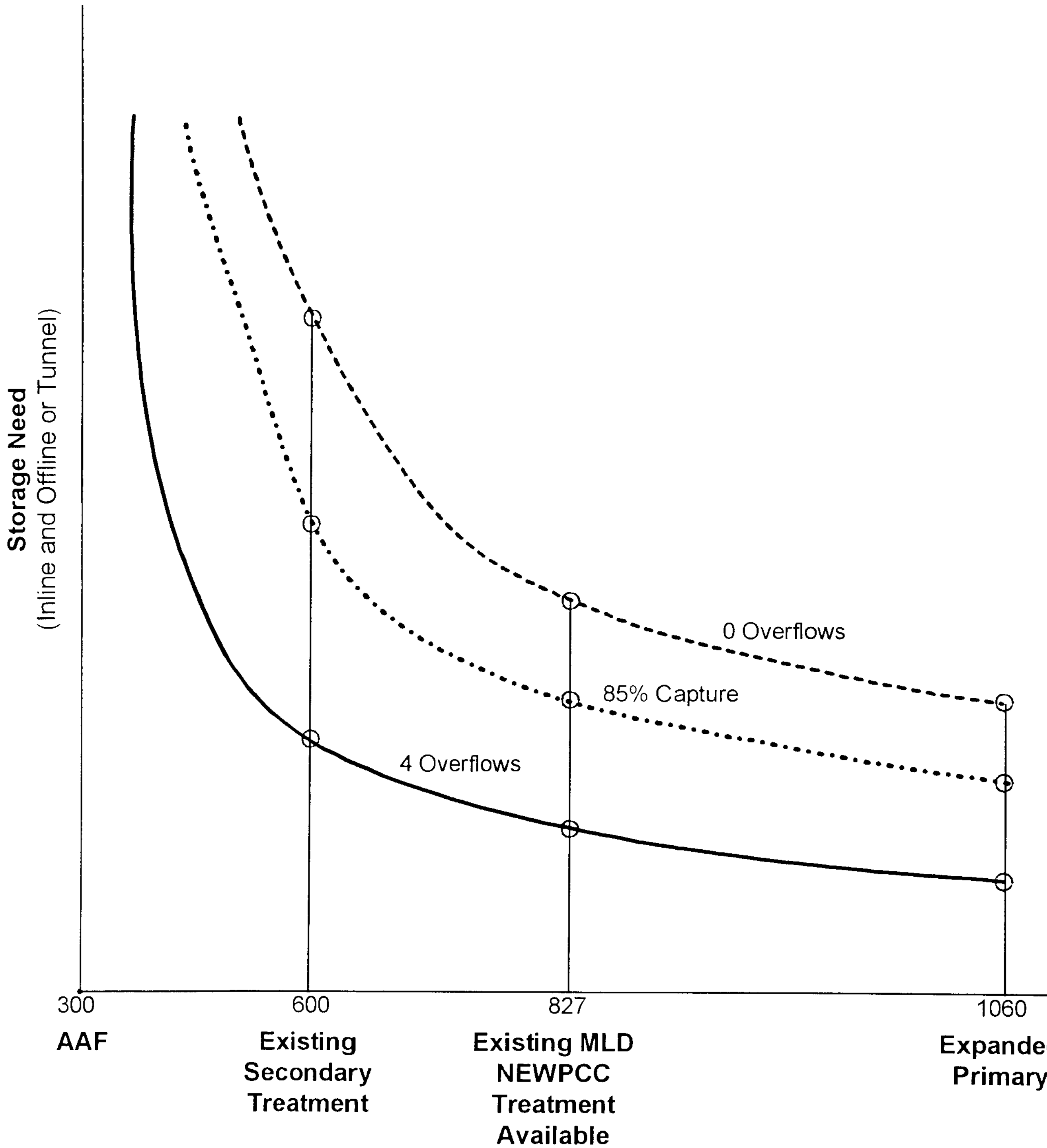
Storage for each local district can come in the form of in-line or off-line storage, and on a regional basis there is potential for tunnel storage. For the tunnel storage option, the hourly CSO runoff from each district and the total dry weather flow in all districts were used to size and analyze the regional tunnel (i.e., it is assumed that the entire CSO district acts like one large district in which the interceptor capacity is equivalent to the treatment rate available at the NEWPCC).

The number and volume of overflows occurring in any year or any district is dependent upon the storage and interception rate available in these districts. Figure 3-4 shows this relationship between storage/treatment and number or volume of overflows. In designing to meet a performance target, say a given number of overflows, either 4 of 0, or a given percent capture (i.e., 85% capture) the greater the dewatering rate available, the less storage volume is required. The rate of flow available for dewatering is equivalent to the treatment capacity minus the average summer dry weather flow.

3.2.4 <u>Interceptor/Dewatering Rate</u>

The first step in developing a potential control scenario was to develop a method for allocating interception/dewatering rates to each district. The existing system has been designed to intercept about 2.75 x DWF from each district. Currently, the interception rates are usually close to 2.75 x DWF for districts in which pumping is required to the interceptor. However, in many areas, the interception is by gravity connections. The WWF interception is uncontrolled and can vary significantly, depending on the depth of water in the trunk sewer. The rate is often much higher than 2.75 x DSF.

For potential CSO control systems, the interception rate can be more systematically allocated to each district. The method used in Phase 3 assumes two components to the interception rate, i.e., the dry weather interception rate and the runoff interception or dewatering rate. The dry



TetrES
CONSULTANTS INC.

stgeneed
s\01\0510

STORAGE / TREATMENT relationship to meet various performance benchmarks

Figure 3-4

weather portion should be equal to the average dry weather flow in each district. The WWF interception or dewatering portion was calculated (system-wide) as being limiting to the available treatment rate at the NEWPCC. (This treatment rate at the NEWPCC could vary, depending on the dewatering strategy, as discussed in a later section). For the districts being dewatered to the WEWPCC or SEWPCC, the strategy was similar, i.e., limit the WWF from all districts to the existing total peak WWF flow to each of these plants. The calculations for these total available dewatering capacities (or excess capacity) is shown on Table 3-5.

The next step was to assign the dewatering rate to each district based on runoff. This is done by assigning a proportion of the available total system-wide dewatering rate to each district based on the proportion of runoff of each district relative to all runoff in the combined sewer area. Based on the 825 ML/d scenario at NEWPCC, the runoff-based interception rates for each district are shown in Table 3-4.

3.2.5 Using a Screening Model to Size Storage

The amount of storage required at each district will depend on various factors such as the amount of runoff in each district, the interception rate available in each district, and the performance benchmark required for that district. Three system-wide interception rates were considered based on various treatment capacities available at the NEWPCC. The three capacities assessed were:

- 600 ML/d - equivalent to the existing secondary treatment capacity;
- 827 ML/d - equivalent to the existing primary treatment capacity;
- 1,060 ML/d - equivalent to the existing pumping capacity, therefore we are assuming that the primary treatment and the interceptor capacity could be expanded to this capacity.

The two performance benchmarks which were assessed were either 4 overflows or 0 overflows during therecreation year. Interception rates were estimated based on the existing primary capacity at the NEWPCC (825 ML/d) being used for dewatering the DWF and runoff at each district. The representative year (1992) was used for this screening model. (A screening model

Table 3-5

Current Capacity

CSO to NE

• NE Secondary	825	ML/d	
• NE Secondary	9.549	m^3/s	
• Assumed Peak from Separate districts	1.000	m^3/s	
Assumed Capacity	8.549	m^3/s	
DWF	1.804	m^3/s	
Excess Capacity	6.745	m^3/s	
• Current Sum of Interception	7.285	m^3/s	based on incipient overflow at uncontrolled

CSO to SE (cascades through Mager)

from Mager	0.309	m^3/s
	26.7	ML/d
Assumed Capacity	0.309	m^3/s
DWF	0.158	m^3/s
Excess Capacity	0.151	m^3/s
• Current Sum of Interception	0.309	m^3/s

CSO to WE

	15.0	ML/d
• Current Sum of Interception	0.174	m^3/s
Assumed Capacity	0.174	m^3/s
DWF	0.02827	m^3/s
Excess Capacity	0.14573	m^3/s

using the same logic as the regional model was developed using a spreadsheet.) Storage for each district was then calculated in an iterative approach until either 4 or 0 overflows occurred at each district.

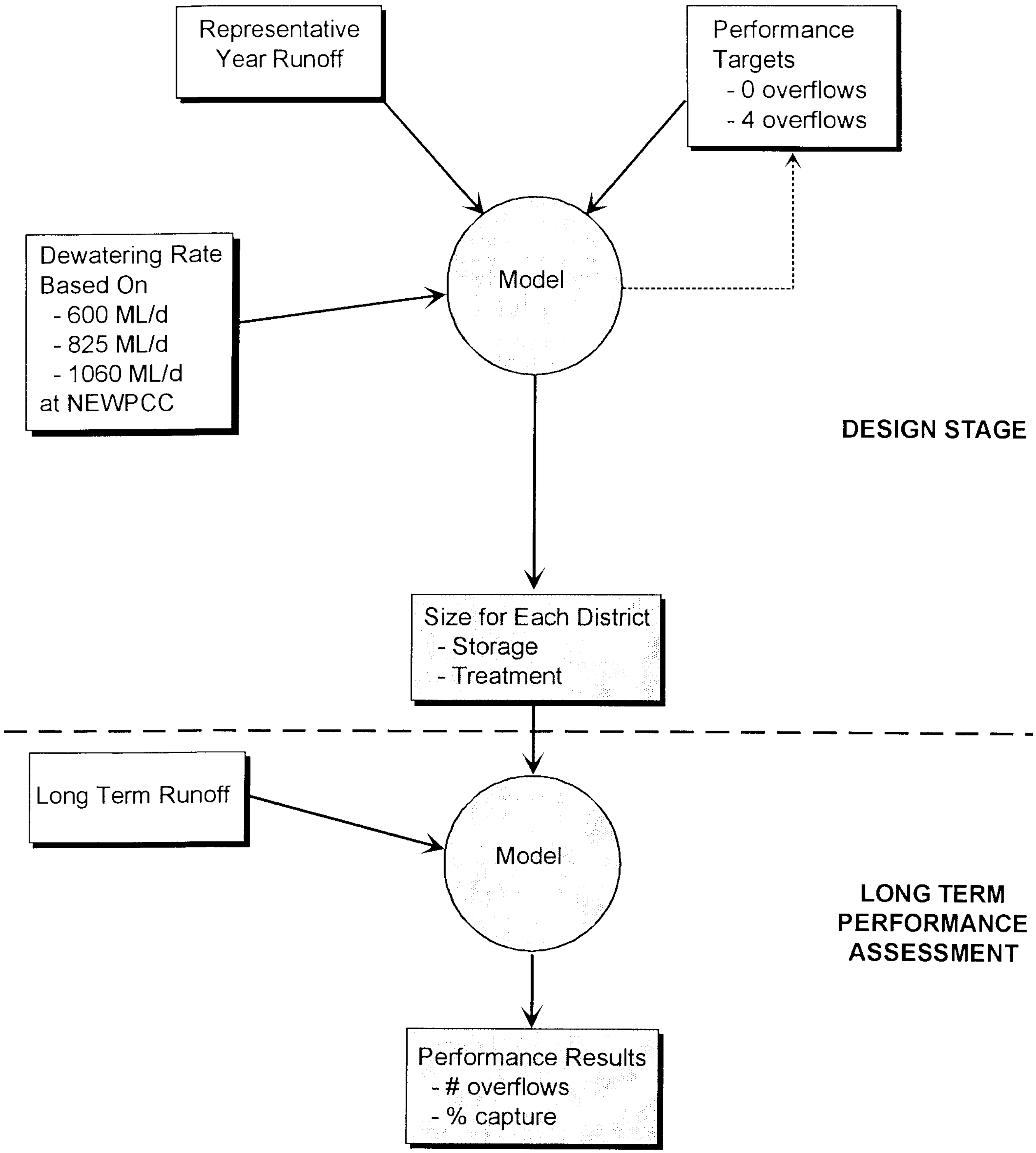
Once this base-case interception/storage/treatment was determined (825 ML/d), the amount of storage at each district was varied inversely proportional to the interception rate for the other two dewatering scenarios (i.e., 600 ML/d or 1,060 ML/d). These results are also shown in Section 5. Once the storage requirements were estimated, the regional planning model was used for runoff during the representative year to test the results. Most districts showed 4 overflows per district with a few districts showing either 3 or 5. Therefore the screening model accurately represented the more detailed planning model.

The regional planning model was then used to evaluate the performance of each one of these storage control alternatives and the in-line storage control alternative (see Section 5.2) using the long term period of record from 1960 through to 1994. The results of this performance evaluation are discussed in Section 6 of this TM. Figure 3-5 illustrates this methodology of system sizing followed by performance assessment.

3.3 EVALUATION PERIOD

3.3.1 <u>General</u>

It was recognized at the beginning of the study that the entire integrated model of rainfall/runoff loadings and water quality involved an enormous data-handling exercise. For the assessment of existing conditions and for first level screening of alternatives, it was considered that using a representative year approach was appropriate, i.e., the use of a single year, using actual rainfall and river flow for that year, would be representative of the results from a simulation of a longer period of rainfall and river flow record. The longer period of record would be used eventually to evaluate a shorter list of selected scenarios. This approach has been used in other CSO planning studies (Hamilton 1991; Toronto, Gore & Storrie 1989). A representative year would describe the river behaviour and runoff response for average conditions of rainfall. The joint consideration of



Method of Sizing and Evaluating Storage / Treament
Figure 3-5

sizemeth
s\01\0510

rainfall and river flows in both the Red and Assiniboine Rivers was used to select a representative year(s) (Phase 2 - TM #4). Having selected the representative year, these data would be used to predict the expected average performance of conceptual control alternatives so as to identify the most promising options for additional study under continuous multi-year modelling.

Another consideration in choosing evaluation periods is that the improved ability to assess a long-term record of 30 years with the system model was made possible, due to increased speed of personal computers and optimization of the regional model codes.

3.3.2 Representative Year Rainfall

To select a representative rainfall year, AES rainfall records (for the Winnipeg International Airport) since 1960 were reviewed and partitioned into specific rainfall ranges. The rain size was determined by considering the millimetres of rain per storm, where a storm is defined by 6 hours of no rain between storms. The size of the individual ranges was increased for larger storms (i.e., from a 1 mm range for storm sizes 1 and 2, to a 25 mm range for storm sizes 9 and 10) to account for the fact that large storms are less frequent. A similar method was used in Toronto (Gore & Storrie 1989). Rainfall records for each year were then sorted into the predefined rainfall ranges for further analysis (see Table 3-6). The analysis comprised the estimation of the long-term average of each of rainfall ranges and a comparison of specific years to this average. It was then possible to identify years that exhibited a nearly similar rainfall distribution to the long-term averages for each of the ranges. A listing of the rainfalls since 1960 sorted into the selected ranges and the long-term averages are shown in Table 3-6. A least square fit relative to the long-term partition range average was also used to rank the years and select those which were most representative. Only storms greater than 2 mm were used since smaller storms would have little or no runoff.

The ten most representative years in terms of rainfall were selected, as shown in **bold** and shaded on Table 3-6. These years were then assessed in terms of river flow. The joint assessment was described in Phase 2 - TM #3 - Control Alternatives.

TABLE 3-6
FREQUENCY OF STORMS OF VARIOUS SIZES FROM 1960 TO 1992

Year[1]	Number of Rains (6 Hours Between)	Total Rainfall mm	Size 1 0-1mm	Size 2 1-2mm	Size 3 2-5mm	Size 4 5-10mm	Size 5 10-20mm	Size 6 20-30mm	Size 7 30-40mm	Size 8 40-50mm	Size 9 50-75mm	Size 10 75-100mm
1960	**42**	**208.5**	12	9	**9**	**7**	**3**	**1**	**0**	**1**	**0**	**0**
1961	31	148.3	7	6	11	4	1	1	1	0	0	0
1962	**48**	**512.5**	8	6	**11**	**10**	**7**	**1**	**3**	**0**	**1**	**1**
1963	**47**	**263.6**	13	8	**9**	**9**	**6**	**1**	**0**	**1**	**0**	**0**
1964	37	254.5	18	3	6	3	3	1	1	1	1	0
1965	56	332	14	7	13	9	10	3	0	0	0	0
1966	**42**	**281.5**	12	4	**11**	**7**	**5**	**1**	**1**	**1**	**0**	**0**
1967	34	247.5	8	7	5	5	5	3	0	0	1	0
1968	53	519.5	14	9	9	9	3	1	6	1	0	1
1969	60	405	14	13	13	9	7	1	0	1	2	0
1970	46	361.1	8	6	13	12	4	0	0	1	2	0
1971	56	278	19	5	14	10	7	1	0	0	0	0
1972	28	155.5	9	9	1	4	3	0	2	0	0	0
1973	**43**	**388.2**	12	2	**10**	**7**	**8**	**2**	**2**	**0**	**0**	**0**
1974	51	356.1	10	8	17	7	5	1	2	0	0	1
1975	54	378.2	9	13	13	10	5	2	0	0	2	0
1976	39	294.4	7	10	6	7	4	4	1	0	0	0
1977	77	592.3	15	18	14	15	4	7	1	2	1	0
1978	**49**	**317.4**	18	4	**11**	**6**	**6**	**2**	**1**	**0**	**1**	**0**
1979	46	235.4	10	14	8	9	3	0	0	2	0	0
1980	49	260.5	20	7	6	6	9	0	1	0	0	0
1981	50	351.9	12	9	7	9	10	1	1	0	1	0
1982	**50**	**296.3**	12	10	**13**	**8**	**4**	**1**	**2**	**0**	**0**	**0**
1983	50	335.5	15	6	8	12	5	1	1	1	1	0
1984	41	368	8	7	11	5	4	1	3	0	2	0
1985	59	379.8	16	9	16	10	5	1	0	1	0	1
1986	61	266.4	23	9	15	5	8	0	0	1	0	0
1987	55	333.9	20	5	16	4	6	2	1	0	1	0
1988	38	264.9	7	12	5	4	7	0	1	2	0	0
1989	**40**	**275**	12	4	**11**	**6**	**4**	**1**	**0**	**1**	**1**	**0**
1990	**46**	**196.5**	18	7	**8**	**6**	**6**	**1**	**0**	**0**	**0**	**0**
1991	40	318.2	9	9	7	4	5	4	2	0	0	0
1992	**60**	**320**	21	9	12	9	6	2	1	0	0	0
1993	**69**	**491.7**	27	7	16	7	6	3	0	0	2	1
1994	**52**	**353.4**	16	5	11	10	6	1	2	0	1	0
1995	**40**	**253**	10	5	11	7	4	2	0	0	1	0
Average	48.3	322.1	13.4	7.8	**10.5**	**7.5**	**5.4**	**1.5**	**1.0**	**0.5**	**0.6**	**0.1**

Notes
1 Bold Indicates one of the 10 most representative years on Record

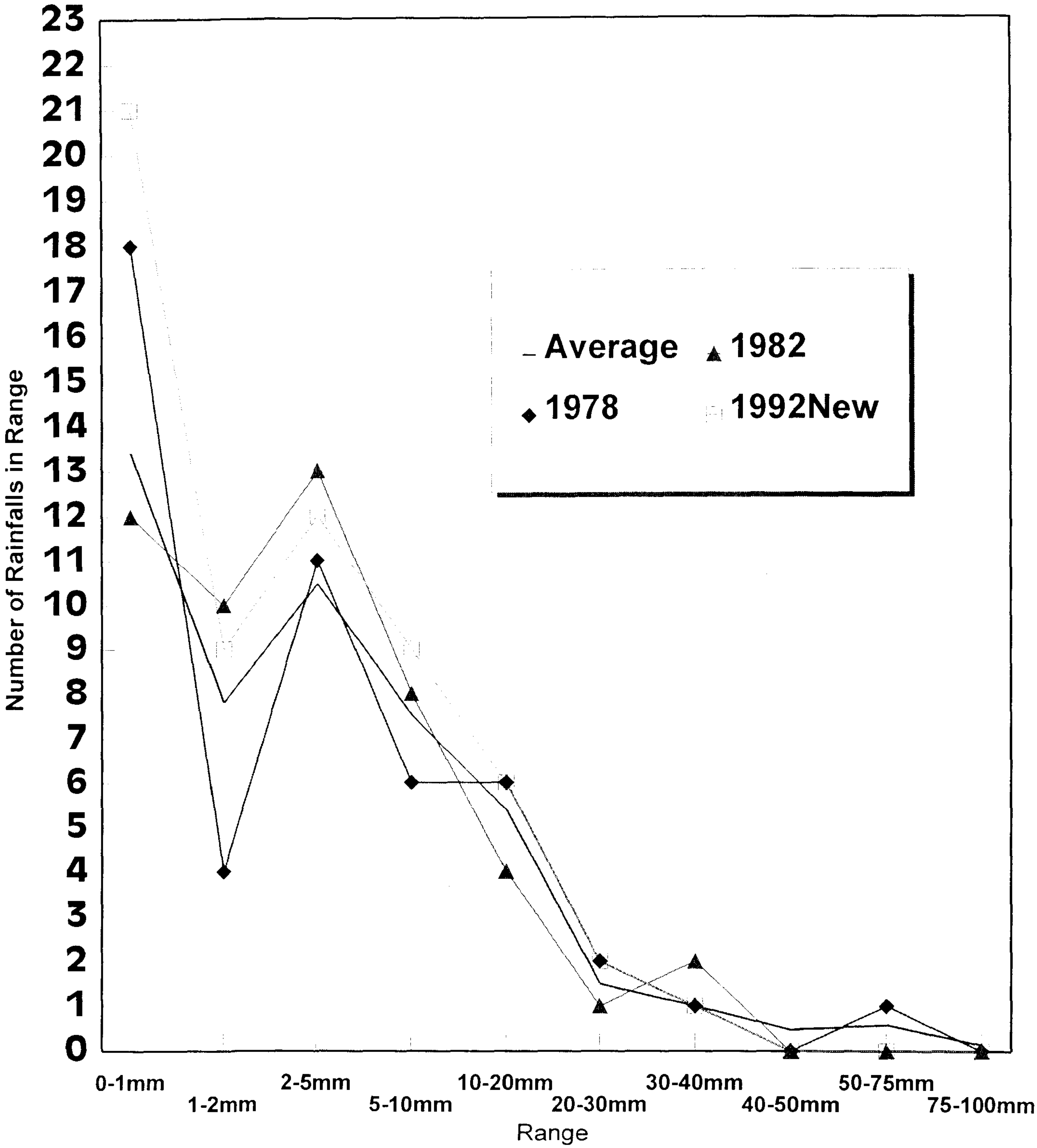
1992New	60	320	21	9	12	9	6	2	1	0	0	0
1992old	51	279	18	6	10	9	6	1	1	0	0	0
			+3	**+3**	**+2**			**+1**				

Since the Phase 2 Workshop, there has been some additional work done on assessing the representative year. As discussed earlier, the calibration/verification exercise identified the fact that the two significant rainstorms which were monitored in the river, were not recorded in the AES hourly dataset. A check against the AES daily dataset determined that there were several days in which there were missing records which were not reported in the hourly dataset. To rectify the problem of this missing data, the City of Winnipeg's Airport rain gauge was used to infill missing hourly data. When the corrected hourly record was compared to the daily data, it appears to explain the missing data adequately. **Table 3-6** shows the differences between the 1992 new dataset and the old dataset used in Phase 2. One very significant storm of between 20 and 30 mm was missed in the original dataset and two moderate storms were also missed (2 - 5 mm). Six rather small storms of less than 2 mm were also included in the new dataset. These storms likely caused minimal runoff and, therefore, would likely not cause a CSO. In addition, three more years of records were added to the dataset, 1993 to 1995.

In conclusion, when the new 1992 data were compared to the long-term average (see **Figure 3-6**), 1992 was confirmed as the selection of the most representative year.

3.3.3 Long-Term Analysis

Although 1992 is the most representative year of the past 35 years, it should be recognized that the rainfall occurring over each year is complex in terms of the variety of different parameters which could be used to compare rainfall. Some of the typical parameters used are the number of rainfall events, number of rainfall events over 2 mm, total rainfall over the year, distribution of various sizes of rainfalls, and average duration of rainfall. It should be recognized that no individual year, or even a synthetically-derived rainfall, can represent the full range of rainfall events over a third of a century. It would therefore always be most beneficial to compare CSO control scenarios using the long-term record, since the long-term record could also give the City an understanding of the variation in CSO volumes and numbers which could occur from year to year.



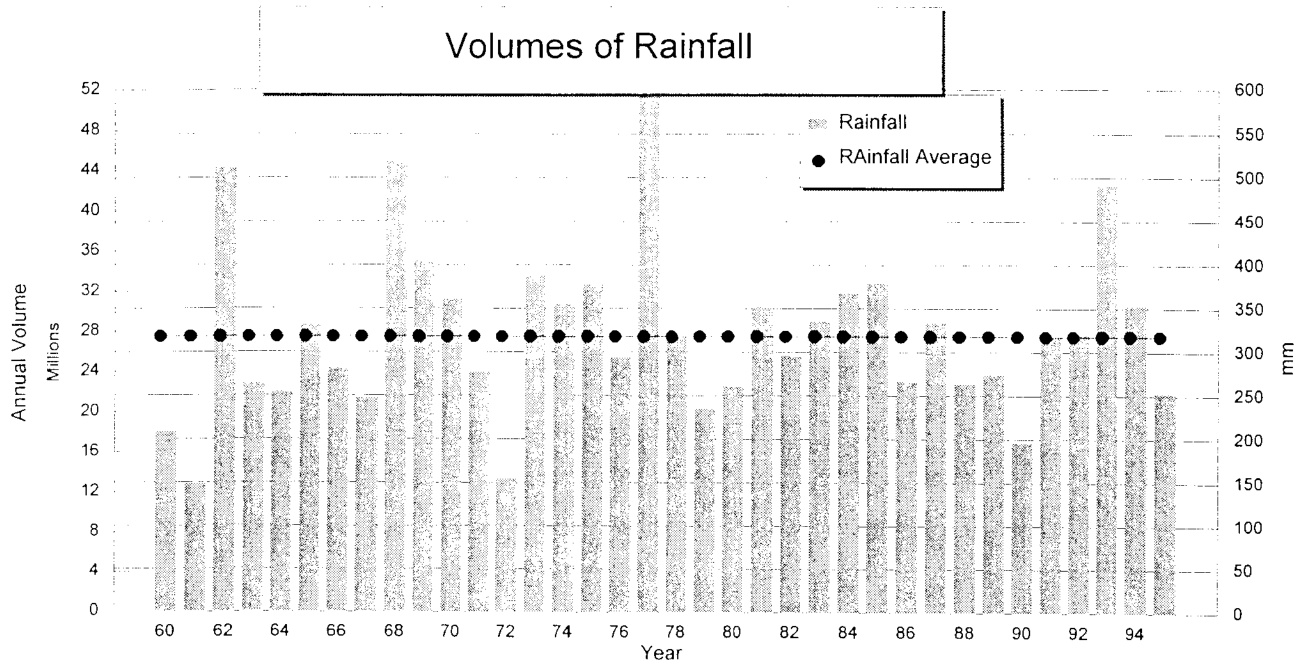
Rainfall Years with Best Fits

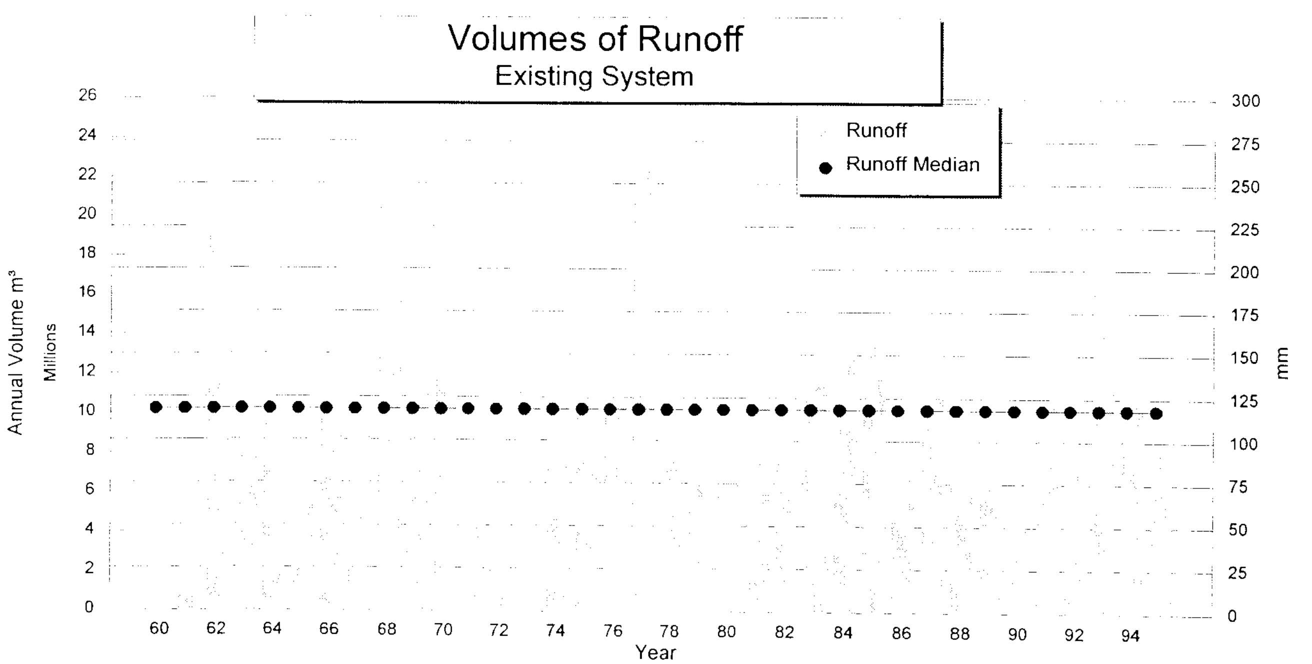
Figure 3-6

Looking closely at 1992 (see Table 3-6), it can be seen that the "average year" has half a storm between 40 - 50 mm, and half a storm between 50 - 75 mm. 1992 has no storms greater than 40 mm. This would indicate that, although 1992 may be very appropriate for assessing control alternatives which have greater than 1 overflow, it may not have a storm large enough to allow for the design of a true 0 overflow option. Figure 3-7 illustrates the annual volume of rainfall in the period of record from 1960 to 1995 in millimetres and millions of m^3 falling on the CSO districts. When compared to the long-term rainfall average, 1992 appears to be exactly equivalent to this long-term average. However, when the volumes of runoff from the districts (as modelled by XP-SWMM runoff block), are compared in the lower figure, 1992 is less than the long-term average. 1992 has about 7.7 million m^3 of runoff in the CSO districts, while the long-term average is closer to 10 million m^3. The comparison of the rainfall and runoff illustrates that it is more than just the **volume of rainfall** which determines the volume of runoff. The "shape" of the rainfall is very important, a long drawn-out rainfall will have more infiltration and therefore less runoff than a "peaky" rainfall.

All of the above discussion illustrates the importance in using a long-term record in determining the runoff, overflow volumes, percent runoff capture, and number of overflows for all the candidate options. Although this extensive amount of work was not considered in the original workplan, improvements in computer technology and modelling techniques allowed this assessment to be done in Phase 3.

The modelling approach, using the performance targets outlined earlier, was used to define different control plan requirements which were evaluated for both the representative year and the long term period of record.





Rainfall and Runoff from 1960 to 1994
Figure 3-7

SENEXOF.WK4

4. OPTIMIZING THE USE OF EXISTING COMBINED SEWER SYSTEM INFRASTRUCTURE

4.1 OVERVIEW

The existing combined sewer system consists of the following main elements:

- wet weather flow interception controls at each district;
- main interceptor conveyance;
- central treatment (NEWPCC).

All of the control plans involve consideration of the capacity of these existing elements, their upgrades and/or the addition of further control options, as shown below:

- wet weather flow interception controls;
- interceptor conveyance;
- storage (in-line, distributed storage, or regional storage);
- high rate treatment at one or more districts;
- central treatment (NEWPCC).

See Figure 4-1.

The different types of control methods have different implications for the overall interacting system requirements. To illustrate, the amount of storage to achieve a given performance level, either in-line, distributed or regional, will depend on the interception flow rate which is also the flow rate which would be used to dewater the storage after the rainfall event (see Figure 4-2). As shown in the figure, the amount of storage will depend on the interception/ dewatering rate, i.e., the greater the rate the lesser storage is required. This figure is illustrative only, as the amount of storage at the various individual districts will need to be determined as discussed further in Section 3.2.5. To define individual district characteristics, system modelling analysis was done as discussed in Section 3.2.4, which identified the specific storage characteristics

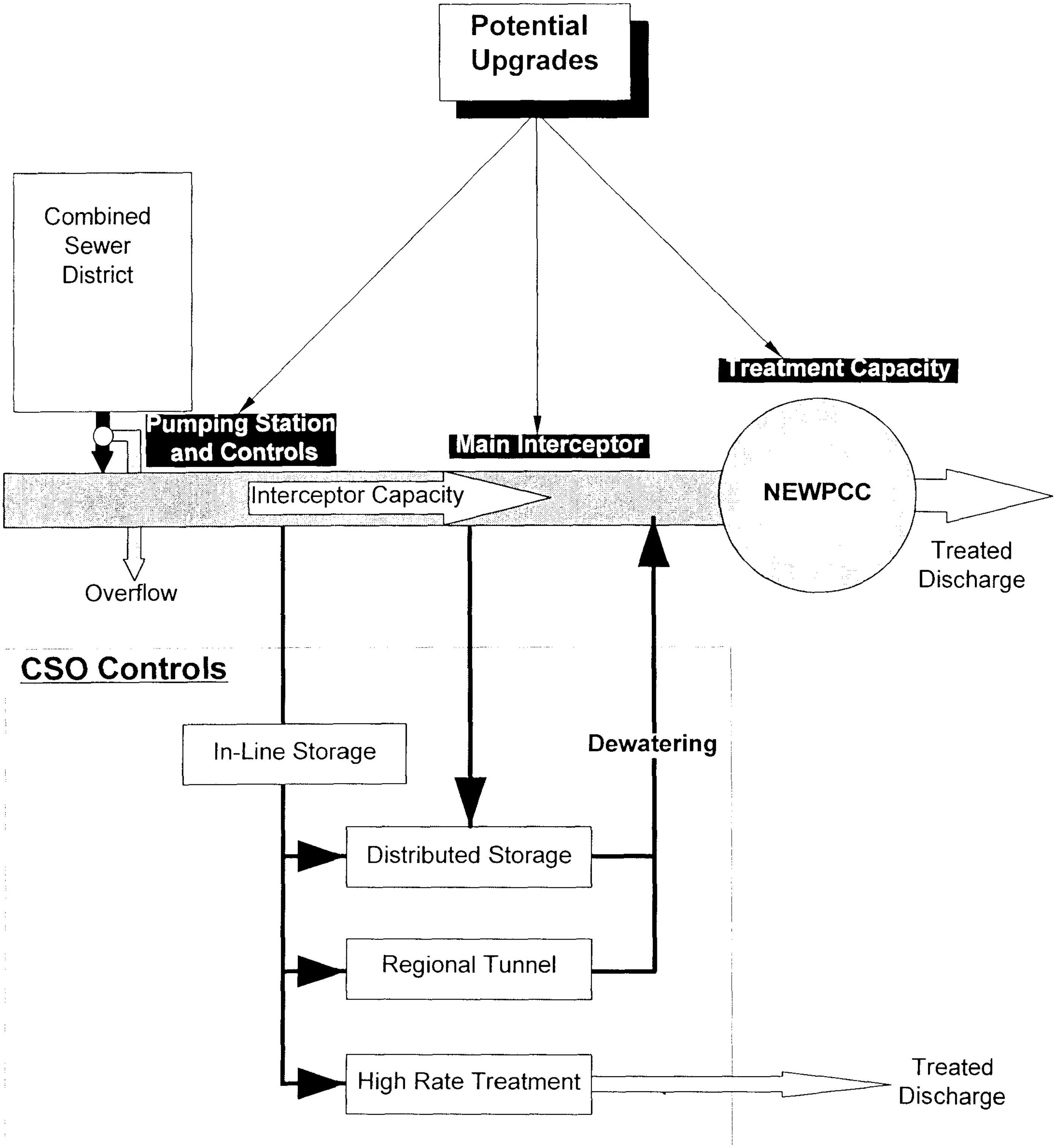


Figure 4-1

Potential Options for CSO Control

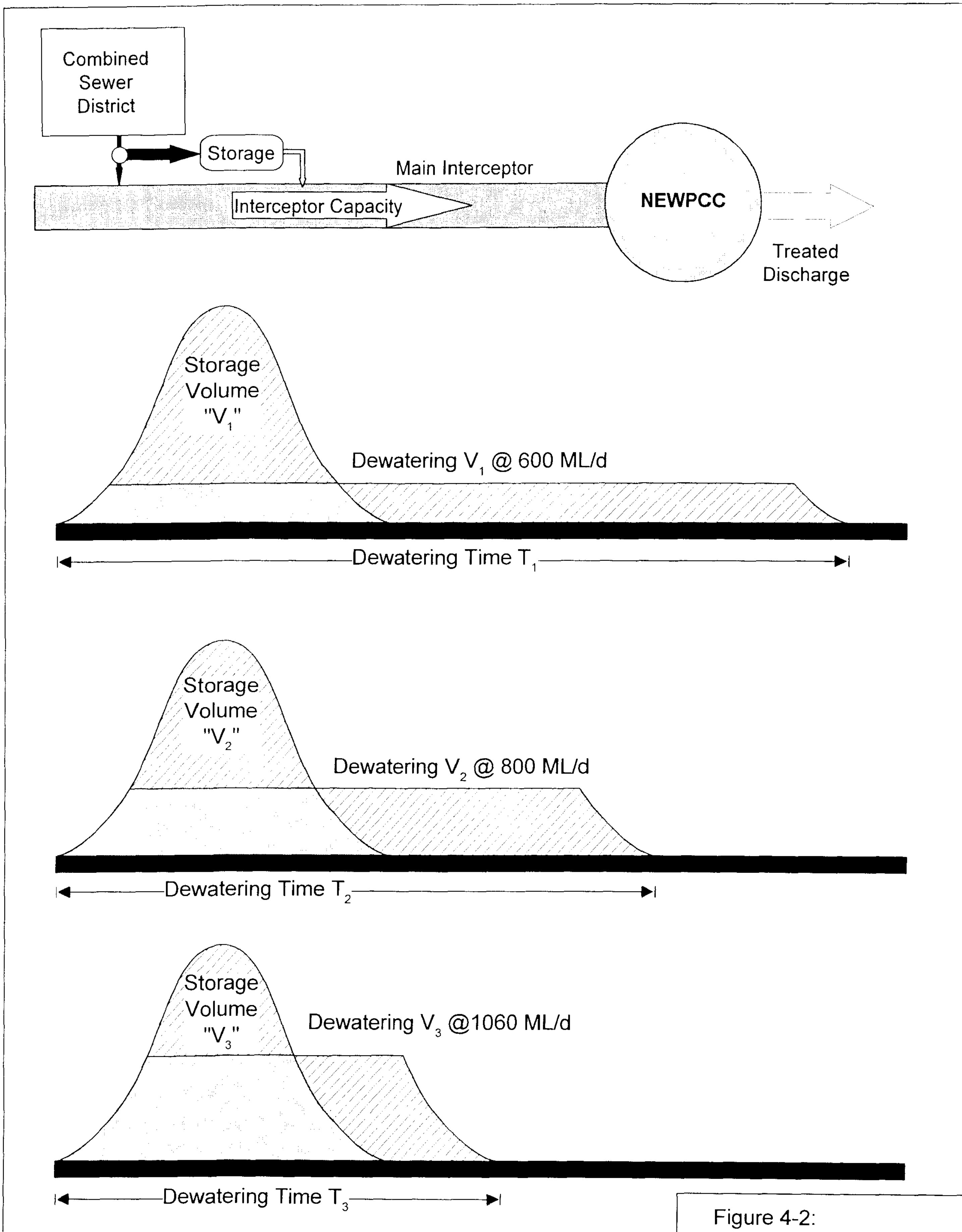


Figure 4-2:

Potential Interception at Various Dewatering Rates required for different dewatering rates for the various districts considering all the system characteristics.

The following will discuss the capabilities of the existing infrastructure and the different levels of upgrades that would be necessary to allow the system to operate effectively to meet various control performance targets. This is relevant to optimizing the use of existing infrastructure (through Best Management Practices or structural controls), and also for the addition of structurally intensive controls such as off-line storage and high rate treatment, etc. Infrastructure will be discussed in terms of the main interceptor and the NEWPCC.

4.2 MAIN INTERCEPTOR

4.2.1 Interceptor/Treatment Capacity

As previously noted, 90% of the combined sewer districts in the City of Winnipeg are tributary to the North End Water Pollution Control Centre (NEWPCC). Accordingly, the districts tributary to the NEWPCC dominate any solution for reduction of the CSO impacts on the rivers passing through the City.

The main interceptor, which conveys the flow from the CS districts to the NEWPCC, was originally designed to convey 2.75 x design "ultimate" DWF from the fully-developed combined sewer districts. The interceptor capacity was based on gravity, unsurcharged flow in the interceptor. The Phase 2 XP-SWMM model of the hydraulics of the NEWPCC/interceptor/ pumping system indicated that the main interceptor could convey about 5 x current DWF (780 ML/d) in a surcharged condition, without overflow to the Red River through the first-duty overflow at the St. John's District combined sewer outfall and through potential overflow points upstream (e.g., Assiniboine District and Omand's Creek). The unsurcharged capacity was calculated to be 7.15 cm (615 ML/d).

In order to increase the rates of diversion to the interceptor, most of the pumps which currently discharge from the trunk sewers to the interceptors will require upgrades in capacity. Further, the eleven districts which currently discharge by gravity will require some mechanical means of controlling the flow diverted to the interceptor. In developing the costs for the various CSO control technologies, an allowance was made for modifications to the flow control system. This allowance comprised $200,000 per device for 40 sites. Including a 20% allowance for estimating contingency and a 20% allowance for engineering, administration and finance, the total amount for 40 sites was taken as $12 million.

The development of costs for the various capital options is dependent, in part, on the dewatering rate. The lower the dewatering rate, the more storage (in-line and/or supplementary off-line) is needed to limit or prevent overflows. Briefly, the three dewatering rates for the North End Plant comprised 600 ML/d which coincides with the design capacity of the secondary treatment facility; 825 ML/d which coincides with the current primary plant capacity and provides primary treatment for the 225 ML/d which bypasses the secondary plant; and finally 1,060 ML/d, which approximates the current total pumping capacity of the NEWPCC. The latter at least would require an expansion of the primary treatment facilities at the NEWPCC.

As noted, the main interceptor can currently convey about 5 times current dry weather flow (DWF) uniformly from all of the combined sewer districts tributary to the NEWPCC (9.0 m^3s or 780 ML/d). During the course of the analysis of other flows tributary to the NEWPCC, i.e., separate sanitary sewage flows from the northeast and northwest interceptors (NE and NW), it was estimated that upwards of 8 x DWF could result from inflow infiltration into these systems during severe rainfall events. The latter wet weather flow (WWF) amounted to some 3.3 m^3/s or 280 ML/day. The NEWPCC has an installed pumping capacity of some 1,060 ML/d. Accordingly it would be able, on an installed pump basis, to lift the combined flows from the main interceptor, at 5 x DWF, plus the NE/NW interceptors, at 8 x DWF.

In developing the potential dewatering rates for the current plant capacity (Section 3.2.4, the "Interceptor/Dewatering Rate"; Table 3-6), it was assumed that the peak flow from the separate districts would be 1 m^3 or less. The results of the 1996 monitoring of the NW interceptor indicate that a 20-mm rainfall would result in a flow of approximately 0.5 m^3/s in the interceptor. Given that the NW interceptor contributes approximately half the flow to the NEWPCC from these sanitary sewer areas, the combined peak of NW and NW interceptors was taken as about

1 m^3. On average, there are only 3.5 storms larger than 20 mm a year, therefore the assumption should hold for an option meeting 4 overflows or more a year.

In developing the scenarios for zero overflows for the period of record, it was assumed that the City of Winnipeg would ensure that new developments in the sanitary districts would be able to limit the peak flows in the NE and NW separated system to the current WWF contribution (the 1 m^3 is approximately 2½ x current DWF). It was thus assumed that the City of Winnipeg would initiate programs in order to reduce the WWF contribution from the existing developed sanitary sewage districts to maintain a flow less than 1 m^3/s. In the absence of such programs, the WWF for the separate systems would limit the capacity of the NEWPCC for CSO control.

In accordance with the foregoing (and based on the discussion in Section 3.2.4), the peak flow which would be conveyed to the NEWPCC through the interceptor would be 6.9 m^3s for the 600 ML/d option and 11.5 m^3s for the 1,060 ML/d option. The impacts of these flows, and the associated changes in loading, on the NEWPCC are discussed in Section 4.3.

The impact on the interceptor, i.e., modifications required to increase the capacity of the interceptor to convey these flows, is discussed below.

There are two aspects of the proposed dewatering rates which impact on the main interceptor. Firstly, the flows from some of the branches to the main interceptor exceed the generalized capacity of 5 x DWF (825 ML/d). Secondly, a possible dewatering rate of 1,060 ML/d (the total pumping capacity at the NEWPCC) results in a piping system which is overloaded for most of the length of the main interceptor. These two conditions can be seen on Table 4-1. The impacts are discussed below.

The capacities on Table 4-1 which are in **bold** and *italicized*, represent those districts where flows, based on the runoff based interception rates, exceed the generalized 5 times DWF, which the overall system was found able to accommodate. The district branches affected are:

600 ML/d Scenario

- the Tuxedo-Doncaster-Ash branch contributions to the main interceptor

Table 4-1: Main Interceptor
Impacts of Dewatering Rates

DISTRICT	COMB W/	CURRENT 5*DWF BRANCH	POTENTIAL 5*DWF MAIN	600ML/d BRANCH	600ML/d MAIN	825ML/d BRANCH	825ML/d MAIN	1060ML/d BRANCH	1060ML/d MAIN
armstrong			8.87		6.036		8.717		11.456
linden		0.265		0.194		***0.283***		0.374	
hawthorne		0.18		0.159		***0.237***		0.317	
newton			8.605		5.701		8.223		10.897
munroe		0.385		0.318		***0.472***		0.629	
polson			8.22		5.383		7.751		10.17
jefferson			8.06		5.199		7.471		9.792
st. john's			7.345		4.743		6.817		8.934
hart		0.195		0.145		***0.212***		0.281	
selkirk			6.73		4.284		6.145		8.043
roland		0.85		0.496		0.702		0.913	
mission		0.72		0.323		0.436		0.552	
boyle	syndicate	0							
syndicate			5.545		3.606		5.189		6.801
alexander			5.425		3.5		5.045		6.609
bannatyne			5.25		3.346		4.815		6.301
assiniboine			4.485		3.077		4.472		5.882
despins	marion	0	0						
dumoulin	laverendrye	0	0						
laverendrye		0.11		0.11		***0.157***		0.21	
jessie		0.68		0.426		***0.61***		0.798	
river		0.35		0.143		***0.209***		0.236	
marion		0.32		0.221		***0.341***		0.451	
colony			2.955		2.148		***3.136***		4.138
cornish			2.285		1.877		***2.778***		3.69
aubrey			2.11		1.799		***2.672***		3.556
clifton			1.2		0.853		***1.242***		1.641
tylehurst			0.815		0.576		***0.837***		1.106
riverbend			0.565		0.387		0.56		0.738
ferry road			0.3		0.211		0.306		0.404
douglas pk	ferry road	0	0						
tuxedo		0.555		***0.714***		***1.096***		***1.476***	
doncaster		0.535		***0.677***		***1.039***		***1.398***	
ash		0.41		***0.579***		***0.895***		***1.208***	
all districts		5.555							

Note: Arrows indicate transfer of flows from branch interceptor to main interceptor

INTRCPQ5.WK4

825 ML/d Scenario

- all branch contributions except the Roland/Mission; and

1,060 ML/d Scenario

- all main interceptor and branch contributions except the Mission portion of the Roland-Mission branch.

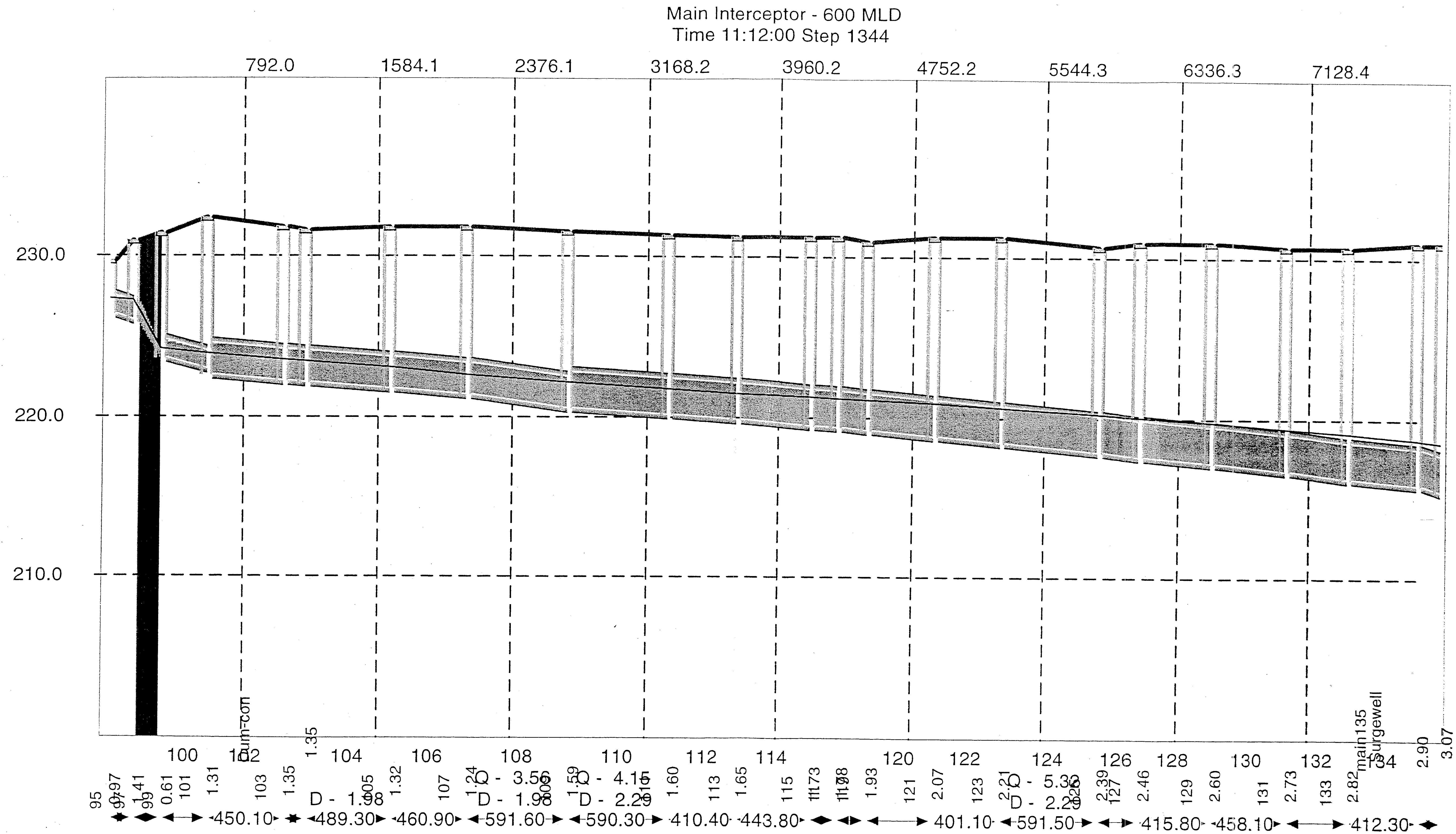
The implications of these exceedances, i.e., potential surcharges to the main interceptor, and the need to provide additional piping, were determined through an analyses, using the revised district dewatering rates and the XP-SWMM model of the interceptor. The results of this analysis along with the costs associated with these changes are provided below.

It was determined, through the XP-SWMM analysis, that the main interceptor is unaffected by the revised interception rates associated with the 600 ML/d scenario. Only a part of the western branch of the main interceptor, from Cornish upstream to Tylehurst, is affected in the 825 ML/d scenario. This is partly due to some of the local districts and interception rates, but mainly results from the Tuxedo-Doncaster-Ash branch increases. The capacity of the main interceptor for the 1,060 ML/d option is exceeded throughout its length. As with the branch analysis, the cumulative rates shown on the table were applied to the XP-SWMM model and the necessary additions to the system were determined.

A sample of the XP-SWMM analysis of the 600 ML/d option confirms the above conclusion that the system is well able to convey these flows (Figures 4-3 and 4-4).

Figure 4-5 shows the modifications needed to the main interceptor to accommodate the 825 ML/d options. These comprise: the addition of a 750 mm connection between the River and Assiniboine districts; the addition of a 1350 mm pipe paralleling the interceptor between Clifton and the main interceptor; and the addition of a 600 mm pipe under Omand's Creek.

Figure 4-6 shows the modifications needed to the main interceptor to accommodate the 1,060 ML/d option. These comprise the paralleling of the main interceptor from the NEWPCC to the



Main Interceptor - 600 MLD
Time 11:12:00 Step 1344
792.0
1584.1
2376.1
3168.2
3960.2
4752.2
5544.3
6336.3
7128.4
230.0
220.0
210.0
main135
surgewell
Q - 3.56
D - 1.98
Q - 4.15
D - 2.29
Q - 5.32
D - 2.29
450.10
489.30
460.90
591.60
590.30
410.40
443.80
401.10
591.50
415.80
458.10
412.30

Figure 4–3

Main Interceptor - West Leg - 600 MLD
Time 10:55:30 Step 1311

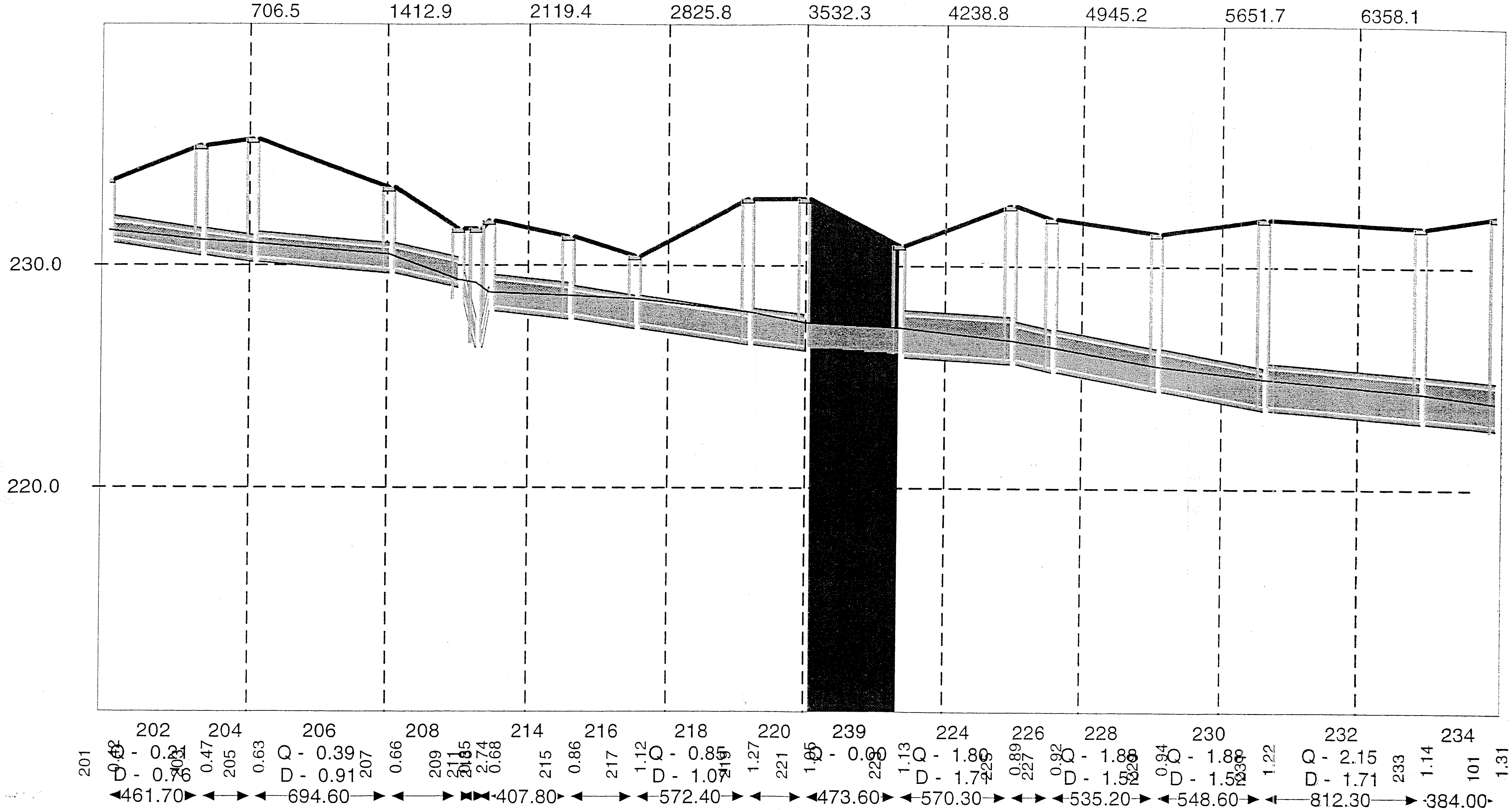
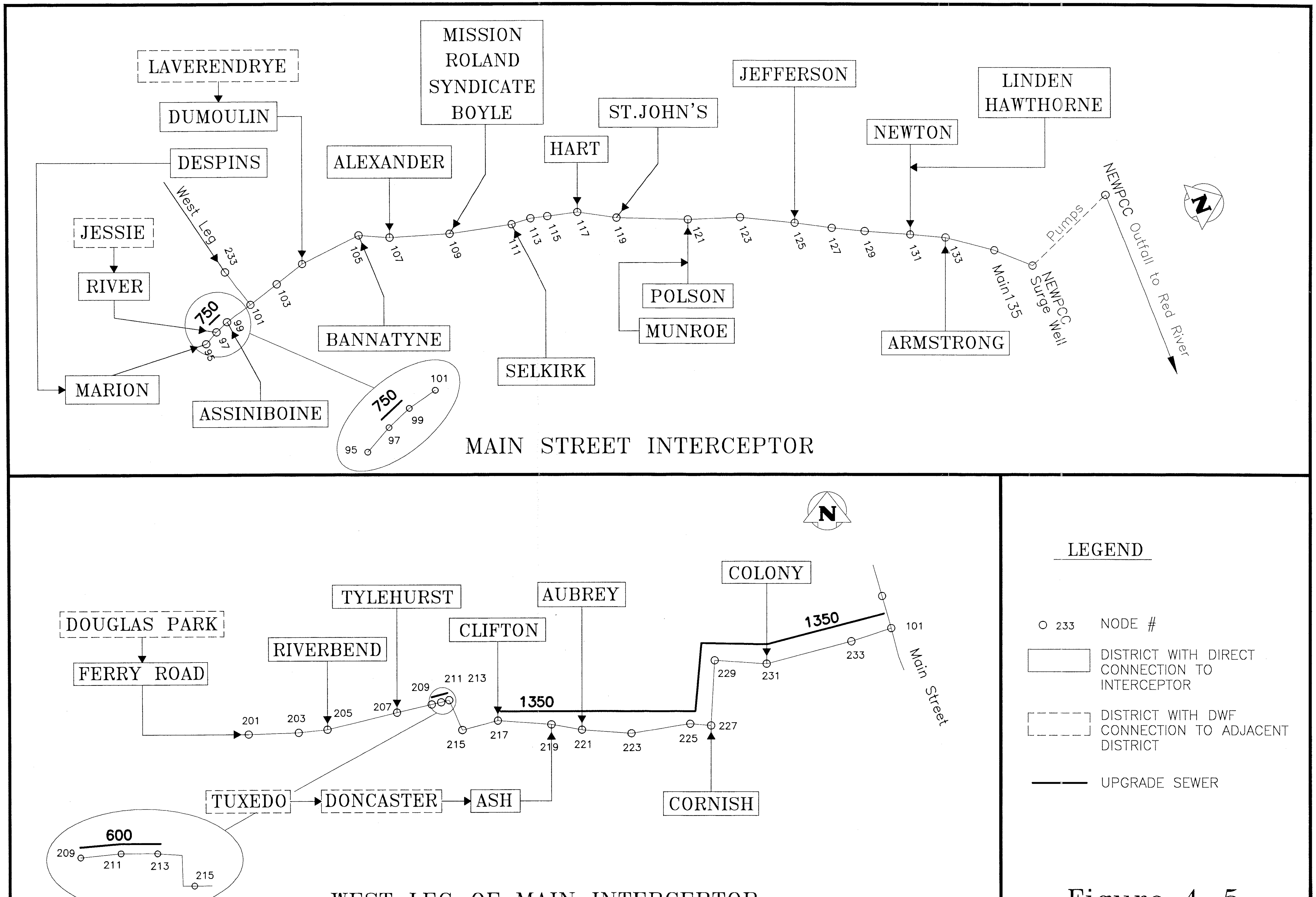


Figure 4. 4

MAIN INTERCEPTOR SEWER
UPGRADING FOR 825 MLD (9.55 CMS)

CSO-275A.dwg



MAIN INTERCEPTOR SEWER
UPGRADING FOR 1060 MLD (12.27 CMS)

CSO-275B.dwg

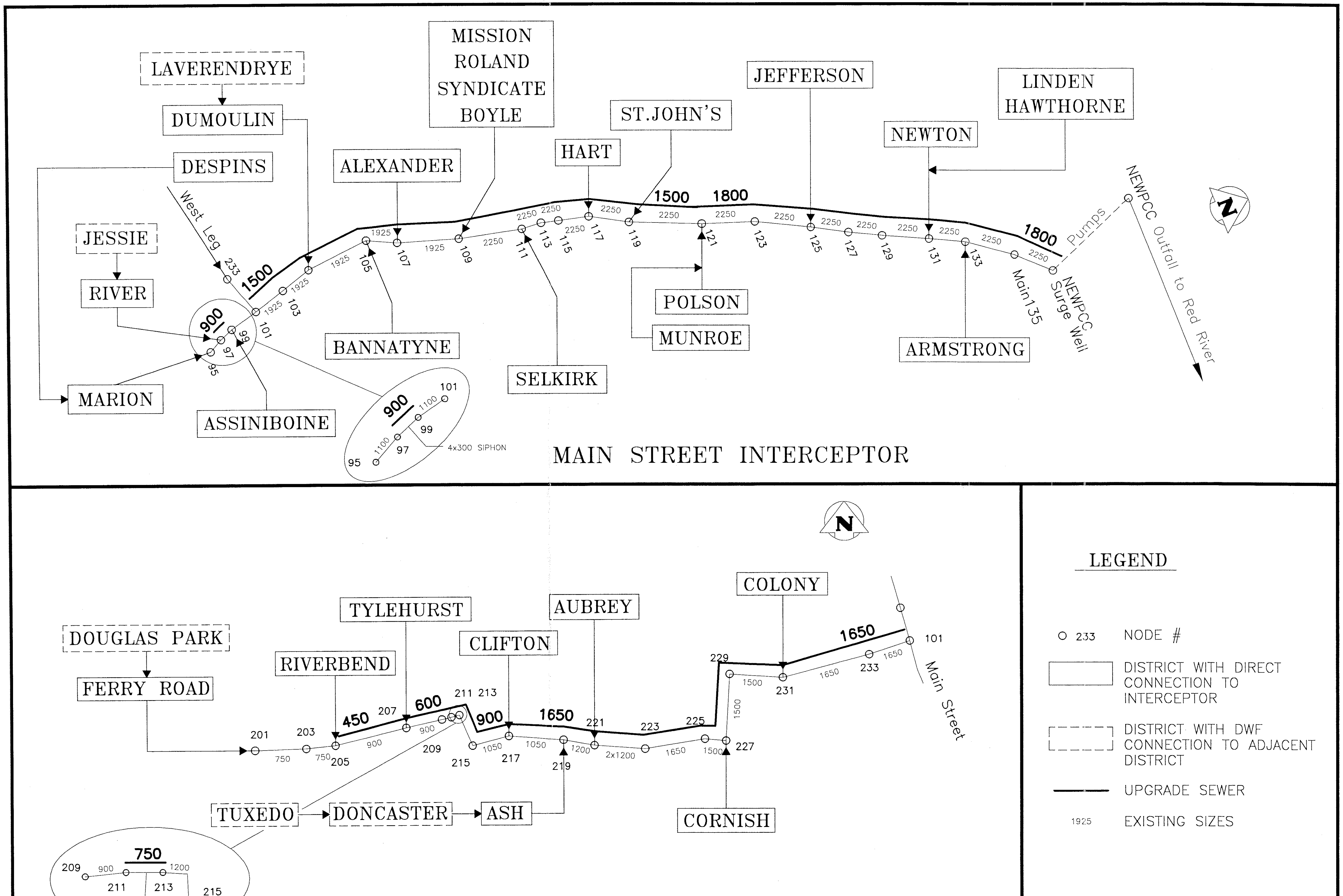


Figure 4-6

Riverbend district, plus a 900 mm addition to the connection between River and Assiniboine districts.

The cost of these additions to the main interceptor system were based on a revision to the cost curve prepared by CG&S for Phase 2. It was believed that the cost curve was higher than those expected for Winnipeg in the range of the smaller diameter tunnels. This was confirmed through discussions with a local contractor. The revised curve is shown on Figure 4-7. As can be seen, this revision falls within the range of costs used to develop the original CG&S curve over the range of tunnels considered.

The costs of installing additional pipe capacity, prepared on the above basis (including a 20% allowance for estimating contingencies and 20% for engineering, finance and administration), were:

825 ML/d option:	$15 million
1,060 ML/d option:	$46 million

These cost were applied to the appropriate scenarios in Section 5.

4.3 NEWPCC REQUIREMENTS

Since the use of storage of combined sewage during the rainfall event is the possible long-term solution to the reduction of CSO impacts on the City's rivers, it was necessary to investigate the impacts of operating the NEWPCCs at full capacity for some two to three days after the rainfall event in order to dewater the stored combined sewage.

The three flow options as identified elsewhere were as follows:

- *600 ML/d.* This represents the design hydraulic capacity of the secondary facility at the NEWPCC. The entire flow would be given secondary treatment and, in the long term, disinfected. The plant effluent would be the least detrimental to the river in this scenario.

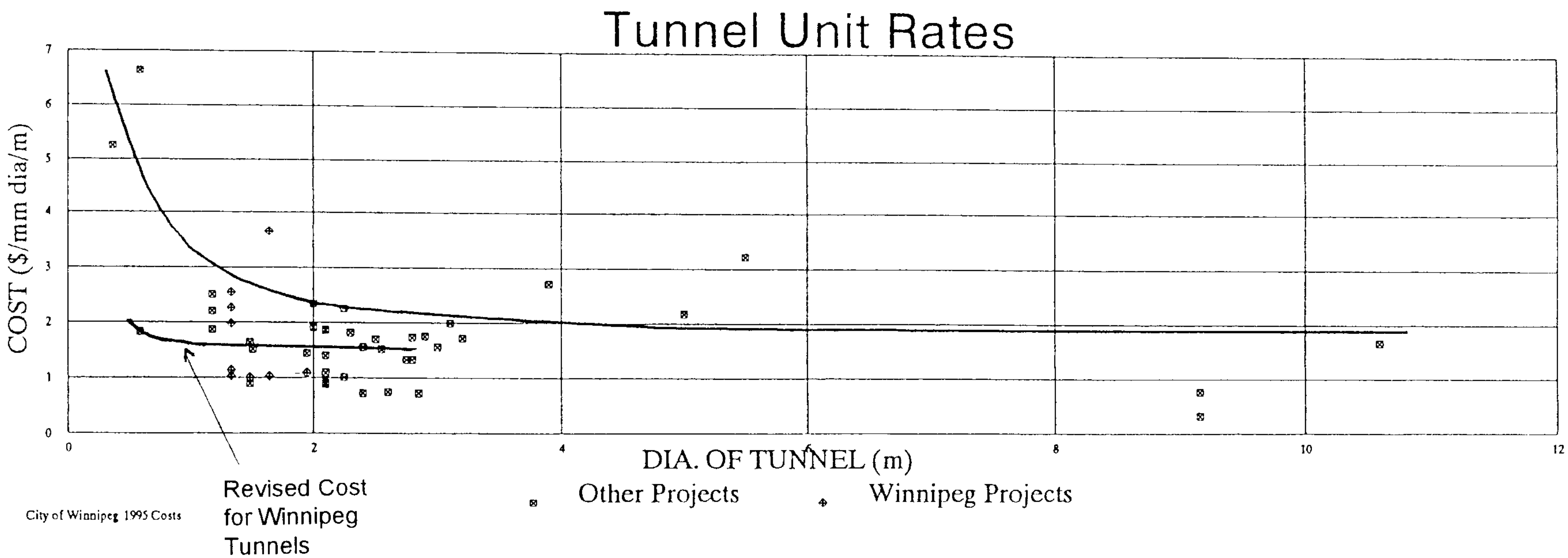

Tunnel Unit Rates
COST ($/mm dia/m)
DIA. OF TUNNEL (m)
Revised Cost for Winnipeg Tunnels
Other Projects
Winnipeg Projects
City of Winnipeg 1995 Costs

Figure 4-7

- *830 ML/d.* This represents the design hydraulic capacity of the primary facility of the plant. The portion of the flow exceeding the hydraulic capacity of the secondary treatment facility would receive primary treatment only (with disinfection), as is the case with the current plant operation.

- *1,060 ML/d.* This option represents the current installed hydraulic capacity of the raw wastewater pumping station. An additional facility would be required to increase the firm pumping capacity, as well as the capacities of the headworks, the primary clarifiers and an auxiliary effluent conduit. This option would have the greatest impact on the configuration of the plant. The secondary bypass would also be disinfected.

Once storage of CSO is in place, the entire NEWPCC will operate at a peak hydraulic load for longer periods of time, i.e., during and after the rainfall event, than is the case under present conditions. These extended periods of peak hydraulic flows will affect the operation and performance of the plant. Increased sludge quantities, due to treatment of the CSO, will also affect the digester operation.

An analysis of the NEWPCC under these conditions was undertaken by CH2M Gore & Storrie Limited. Details of the investigations are contained in Appendix _ entitled "Combined Sewer Overflow Control Study - Impacts on North End WPCC". The following is a summary of the background of the investigations for information, followed by the recommendations for the three dewatering scenarios and the associated costs.

<u>Combined Sewage (CS) Quality</u>

The available plant quality data was insufficient to develop the CS quality for the study. Accordingly, a combination of literature review and the 1996 plant operating data for the NEWPCC were used to develop the estimated WWCS quality for the model analyses.

ASSUMED CS QUALITY

PARAMETER	CONCENTRATION
BOD	212 mg/L
COD	530 mg/L
TSS	250 mg/L
NH_3/NH_4	18 mg/L
TKN	27 mg/L
Total P	3.5 mg/L
Alkalinity	150 mg/L
PH	7.2 mg/L

These differ significantly from the EMCs for CSOs developed in Phase 2 (TSS = 846 ± 588 mg/L; BOD 141 ± 29 mg/L). Continued sewage monitoring, and more importantly monitoring of the impacts of storage on CSO quality, will be required to establish the proper values for WPCC design.

Plant Effluent Criteria

The criteria used for determining the nature and extent of the treatment plant modifications were as follows:

Secondary Treatment Effluent Criteria

- TSS $\leq$ 30 mg/L
- $BOD_5 \leq$ 25 mg/L

Primary Treatment Effluent Criteria

- TSS $\geq$ 50% removal
- $BOD_5 \geq$ 40% removal

Some modelling considerations were:

- Oxygen Activated Sludge Process Requirement
  - the design point of the existing oxygen-activated sludge system was checked with process design criteria published in the book entitled "The Use of High Purity Oxygen in the Activated Sludge Process", 1978, CRC Press Inc. A curve of the organic loading rate and organic removal rate of the oxygen-activated sludge system had been developed by Union Carbide Corporation, by using a collection of pilot plant and full-scale plant data. The design conditions of the oxygen-activated sludge system at the NEWPCC matched with the results obtained from the curve. On this basis, it would appear that the plant can meet its design objectives.

- Determination of Sludge Digestion Requirement
  - the sludge dry solids production for treatment of the WWCS flows at the NEWPCC was estimated from the four sources currently digested at that plant including: primary sludge; secondary sludge; chemical sludge (where appropriate); and sludge from the SEWPCC/ WEWPCC. These projected sludge dry solids were determined. All sludge solids are stabilized by an anaerobic sludge digestion process. A solids detention time of 10 days was used for determination of sludge digester requirement. A volatile solids reduction of 45% was assumed in the digestion process.

- Digested Sludge Dewatering Requirement
- the current operation of this facility was used to develop the capacity of the centrifuges based on volume applied and solids load.

- Disinfection Requirement
  - it was assumed that the WWCS flow exceeding the secondary treatment capacity would receive primary treatment and disinfection. Due to the primary treatment effluent quality and the intermittent nature of the flows (they occur during wet weather situations), sodium hypochlorite was considered for effluent disinfection and sodium bisulphite, for dechlorination.

4.3.2 Impact of CS Flows

The existing plant was designed for a peak flow of 830 ML/d for the primary treatment section and a peak flow of 590 ML/d for the secondary treatment section. The latter was also designed for an effluent BOD of 25 mg/L/d at a maximum BOD_5 load of 89,600 kg/d. The corresponding BOD_5 load to the secondary process at the design peak flow was 152 mg/L.

The impact of the three flow rate options, 600 ML/d, 830 ML/d and 1,060 ML/d, of the collected and stored CS flows at the NEWPCC were assessed. It is certain that the plant cannot handle the CS flow of 1,060 ML/d without upgrading, because this flow rate is higher than the design flows for both primary and secondary. The impacts of the sustained dewatering CS flow of 600 ML/d and 830 ML/d were calculated and the results are as follows:

IMPACT OF CS FLOWS

CS flow, ML/d	600	830
Primary Clarifier area, m^2	6503.7	6503.7
Surface overflow rate, $m^3/m^2.d$	92.3	127.6
Estimated primary removal efficiency* BOD_5%	23	17
TSS %	42	32
Estimated primary effluent BOD_5, mg/L	163	176
Estimated primary effluent TSS, mg/L	145	170
BOD_5 load to secondary**, kg/d	97,900	105,600
Max. design BOD_5 load for secondary, kg/d	89,600	89,600

Note: * BOD_5 is assumed at 53% of TSS
** BOD_5 load is based on maximum flow of 600 ML/d

Under these two CS flow conditions, the secondary treatment section is overloaded and the prime effluent quality would be detrimentally affected.

4.3.3 Plant Upgrading Requirements

Several alternatives for upgrading the NEWPCC were investigated. The process design calculations for plant upgrading were carried out using a computer model supplemented with manual calculations for sludge treatment and dewatering requirements.

A brief description of each alternative for treatment of the CS flows is provided below.

CS Flow of 600 ML/day

Three alternatives for plant upgrading were considered:

Alternative 1: Expansion of the Final Clarifiers

Under the 600 ML/day scenario, the existing primary tanks would operate as at an overflow rate of 92.3 m^3/m^2.day. For extended periods at this overflow rate, it is expected that the prime rate removal efficiency would reduce to approximately 21% for BOD_5 and 40% for TSS. Under this condition, a BOD_5 load of 100,200 kg/day would discharge to the secondary facilities. The computer model indicated that the existing oxygenation reactors could handle the BOD but that the final clarifiers would need expansion due to the high solids loading. The resultant increased sludge solids would require additional primary digester capacity to stabilize it. No change would be required to the sludge dewatering equipment.

Alternative 2: Chemically-Enhanced Primary Treatment for the Entire Flow

The objective is to increase primary removal facility and reduce organic load to the secondary system. The increased primary clarifiers needed to operate at the lower overflow rate necessary to separate the chemically-precipitated particles would require expansion. The resultant BOD_5 load would preclude the need to expand the secondary system. Additional primary digester capacity would be required. An expansion of the sludge dewatering facility is not required.

Alternative 3: Expansion of Primary Clarifiers

This alternative is similar to Alternative 1 only the increased load to the secondary system is reduced through an expansion of the primary clarifiers. The resultant BOD_5 load would be treated in the secondary without expansion. As with the other options, the increase in sludge solids would require additional primary digester volume, but without an expansion of the sludge dewatering facility.

CS Flow of 830 ML/day

Alternative 4: Expansion of Secondary Treatment to Produce a Nitrified Effluent

This alternative involves the expansion of the secondary treatment system using a single-sludge nitrification process to produce a nitrified effluent. Expansion of both the oxygenation reactors and final clarifiers would be required.

The flow pattern of the single-sludge nitrification system is identical to that of carbonaceous oxygen-activated sludge design, but the system is required to remove carbonaceous BOD as well as ammonia. This is done by providing in the system the proper conditions to cultivate nitrified bacteria among the more prevalent carbonaceous bacteria in the biomass. Since nitrifiers grow much more slowly than the carbonaceous micro-organisms, maintenance of the proper conditions consists primarily of assuring that the time which the biomass spends in the reactor, the sludge retention time (SRT), is at least long enough to provide time for the nitrifiers in the biomass to grow. Single-sludge oxygen nitrification system will typically have 2½ to 6 hour retention times compared to the standard design oxygen-activated sludge system for carbonaceous removal only, which had a 1.5 to 2.5 hour retention time. Accordingly, the design of the oxygenation reactors for single-sludge nitrification system is provided with a retention time of 3.6 hours and an additional reactor volume is required.

The settling rate of the nitrifying biomass is slower than the carbonaceous biomass. Final clarifier design requires a surface overflow rate lower than that of the carbonaceous biomass for effective solid separation. Accordingly, additional clarifier volume is required and was based on a surface overflow rate of 29.5 m^3/m^2.day. As with the other options, additional anaerobic digester capacity is required but the sludge dewatering facility remains adequate.

Alternative 5: Expansion of Primary Clarifiers for Treatment

The expanded clarifier facility was designed so as to be able to continue to use the existing secondary system. It was designed for a removal efficiency of 30% BOD_5 and 50% TSS.

Six hundred ML/day of the primary effluent will be treated in the existing secondary treatment facility. The remaining 230 ML will be disinfected and will bypass the secondary facility. As with the other options, the additional primary digester volume will be needed but the dewatering facility is adequate.

CS Flow of 1,060 ML/day

Alternative 6: Expansion of Primary Clarifiers for Treatment

This alternative is similar to Alternative 4, with the primary treatment facility designed for removal efficiencies of 30% BOD_5 and 50% TSS. Six hundred ML/d of the primary effluent will be further treated in the existing secondary facility, and the remaining 460 ML/day will be disinfected and will bypass the secondary facility. As with all the other options, additional primary sludge digester volume will be required but expansion of the sludge dewatering facility is not required.

Plant Upgrade Requirement Summary

The sizes of the additional facilities needed to address the above requirements, are provided on Table 4-2. These sizes were used to develop the costs in the next sub-section.

Table 4.2 - Plant Upgrading Requirements

Additional Facility Requirement	600 ML/d Alternative 1	600 ML/d Alternative 2	600 ML/d Alternative 3	600 ML/d Alternative 4	830 ML/d Alternative 5	1060 ML/d Alternative 6
Primary Clarifier						
Area, m^2	-	8,500	6,262	-	11,156	16,049
Volume, m^3	-	30,600	22,543	-	40,162	69,011
Oxygen Reactor						
Area, m^2	-	-	-	13,974	-	-
Volume, m3	-	-	-	60,088	-	-
Final Clarifier						
Area, m^2	3,923	-	-	6,074	-	-
Volume, m3	14,123	-	-	21,866	-	-
Digester						
Volume, m3	7,600	12,860	6,070	3,186	14,890	26,578
Dewatering						
Digested Sludge, kg/d	-	-	-	-	-	-
Meeting Effluent Requirement	Yes	Yes	Yes	Yes	Yes	Yes

Estimated Capital Cost for the Plant Upgrading Requirements

Conceptual capital costs for the plant upgrading were based on the most recent contracts completed by CH2M Gore & Storrie Limited. In order to facilitate cost calculations for this study, unit costs for various unit processes were established using the construction contract price and were then brought up to January 1998 costs using ENR construction cost index and included an allowance of 11% for engineering and construction supervision. The costs originally developed for Appendix did not include land costs, taxes or piling foundation costs. In order to bring all of the costs in line with the basis of estimate used for all other options in the CSO study, 8% was added to the costs for foundations; 9% for finance AND administration charges; plus a 20% estimating contingency. Land costs were not included.

The costs provided for the 1,060 ML/d option, based on the CG&S treatment study, must have added to them the costs of pumping, an expanded screen and grit facility, and a new outfall. Allowances for these are as follows:

- pumping (based on CG&S curves developed for Phase 2) $4.5 M x 1.44 = $6.5 M;
- screens (based on NEWPCC experience) $1.7 M x 1.44 = $2.5 M;
- grit tanks (based on CG&S curves) $1.4 M; and
- outfall (based on CG&S curves) $1.2 M x 1.44 = $1.7 M.

The costs as developed for the report, and as modified above, are given in Table 4-3.

These costs were carried forward to the appropriate scenarios developed in Section 5.

4.4 ADDITIONAL CSO CONTROL OPTIONS

With the above understanding of the existing combined sewer system and treatment capabilities, various storage treatment and/or interactions involved in the addition of CSO control plans could be assessed. These plans as well as other plans that have lesser

Table 4-3

Estimated Capital Costs for Plant Upgrades - $M

Additional Facilities	600 ML/d				830 ML/d	1060 ML/d
	Alternative 1	Alternative 2	Alternative 3	Alternative 4	Alternative 5	Alternative 6
Primary Clarifiers	-	$15.8	$11.7	-	$20.9	$29.9
Oxygen Reactors	-	-	-	$26.1	-	-
Final Clarifiers	$7.3	-	-	$11.3	-	-
Flash Mix & Floc Tanks	-	$5.2	-	-	-	-
Chemical System	-	$5.7	-	-	-	-
Disinfection*	-	-	-	-	$1.3	$2.6
Sludge Digestion	$7.3	$12.4	$5.9	$3.1	$14.4	$25.6
Sludge Dewatering	-	-	-	-	-	-
Raw Sewage Pumps	-	-	-	-	-	$6.5
Screens & Grit	-	-	-	-	-	$3.9
Outfall	-	-	-	-	-	$1.7
Total Estimated Cost, $M	$14.7	$39.1	$17.5	$40.5	$36.5	$70.3

* Disinfection includes chlorination/dechlorination of secondary bypass only

Costs include 20% Estimating Contingency and 20% for E, F & A

phtcost
s\0110510

implications on the existing system such as sewer system separation and the addition of floatables are presented and discussed in the following **Section 5**.

5. CANDIDATE PLANS

5.1 OVERVIEW

This section will present alternative CSO control plans and their system-wide requirements to meet the range of selected performance targets discussed in Section 3.

The range of control plans will include:

- in-line storage;
- off-line storage;
- regional tunnels;
- high-rate treatment;
- separation of combined sewers; and
- floatables capture.

Some plans involve combinations of technology, such as in-line plus off-line storage, etc. Each category of plan will be discussed with respect to the concept, the siting issues, determination of requirements for the related performance target, operational considerations, and costs.

5.2 IN-LINE STORAGE

5.2.1 Conceptual Considerations

In combined sewer districts, the major trunk sewers have a large capacity and run full during severe storm events. Often, however, they are only partly full during the more frequent, less intensive rainfall events. The capacity of these trunk sewers, and where applicable, their associated storm relief sewers, are between 50 to 100 fold of the base DWF and, therefore, during most storms, considerable unused capacity exists in the conduits. In-system or in-line storage takes advantage of this unused existing storage capacity by restricting flows at the overflow point (through a control device) causing wastewater to be stored temporarily in the upstream pipes. During and after the rainfall event, this captured WWF is dewatered and taken to treatment via the existing interceptor system.

In-line storage is thus the volume contained within the existing sewer pipe network that can be safely accessed through the use of a control device. Figure 5-1 illustrates storage of wet weather flows in a combined sewer trunk system.

While the use of this storage is very cost-effective, it can only be used if basement flooding protection is not jeopardized. This was a key consideration in utilizing in-line storage. A minimum depth of 3 m (approx. 10 ft) below minimum ground level was used as the upper boundary to protect against basement flooding under in-line storage conditions, i.e., depths 3 m or greater below minimum ground level were considered adequate to protect against basement flooding for the current level of service. Typically, 2.5 m (approx. 8 ft) below existing ground level is used in detailed SWMM modelling exercises to estimate the maximum allowable surcharge permitted in the sewer system for a specific design storm to protect against basement flooding.

Figure 5-2 illustrates the key dimensions and aspects used to derive a maximum allowable surcharge depth of 2.5 m below ground surface. The maximum allowable surcharge depth of 2.5 m below ground level, as measured at the centerline of the roadway, includes a buffer of 0.5 m below the actual basement floor elevation to provide for model prediction inaccuracies and variation in actual basement elevations. It was deemed necessary to include an additional 0.5 m of depth below basement elevations to account for possible backwater effects in the laterals during a rain due to an artificially-elevated water level in the combined sewer trunk. Accordingly, a 1.0 m depth below basement elevations, or an equivalent of 3.0 m below centerline of the roadway, was considered adequate to maintain the existing level of basement flood protection.

Analyses performed during the course of Phase 3 to estimate storage potential in the existing combined sewer trunks and relief pipes, considered the following important factors:

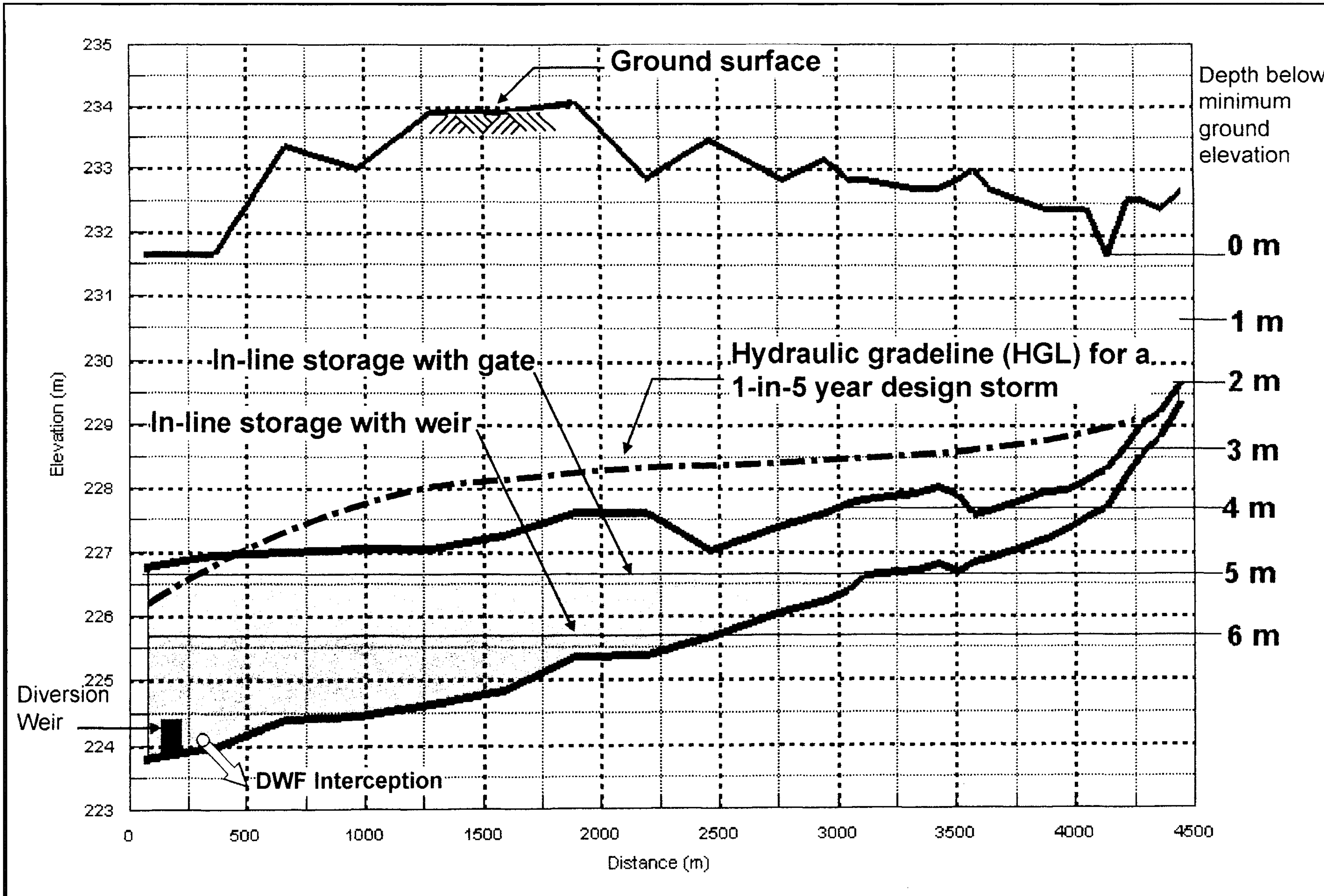


Figure 5-1: Illustration of Potential In-line Storage Available in a Combined Sewer Trunk

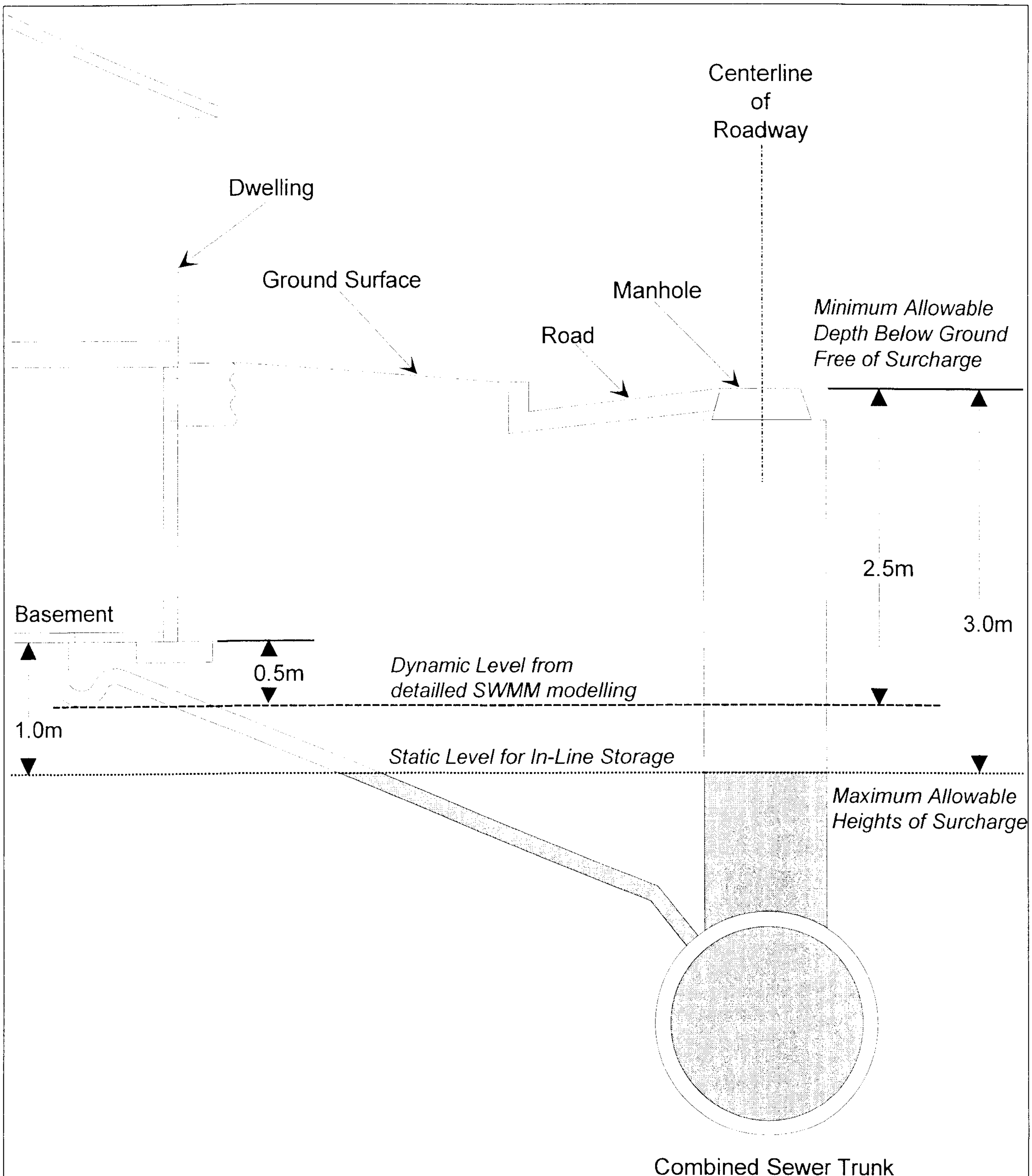


Figure 5-2:

Illustration of Key Dimension and Aspects Relevant to Surcharge Levels

- the local situation
  - topography/sewer design
  - installed basement flooding relief sewers

- available storage
  - method of calculation
  - pipe system details
  - control options constraints

- operational considerations
  - hydraulics
  - number of outfalls
  - fail-safeness
  - concerns
- pilot testing program
- cost estimates

The following sections will discuss the key aspects of the factors noted above, relevant to in-line storage analyses and estimates.

5.2.2 Winnipeg Situation

Due to Winnipeg's relatively flat topography, intensive rainstorms, and the highly impervious clay soils, designers of the original combined sewers found that the most cost-effective and practical sewer design was to place sewers at minimum grade (minimum slope to achieve scouring velocity under full-flow conditions). This resulted in very large egg-shaped or circular sewers with typical heights/diameters ranging between 2 to 3 m. **Figure 5-3** is a representative example of cross-sectional size of a trunk and relief sewer found in Winnipeg. These large pipes extend a significant distance into combined sewer districts (3 km), as shown in **Figure 5-1** and, potentially, contain storage volume that could be used to minimize the volume and number of CSO events.

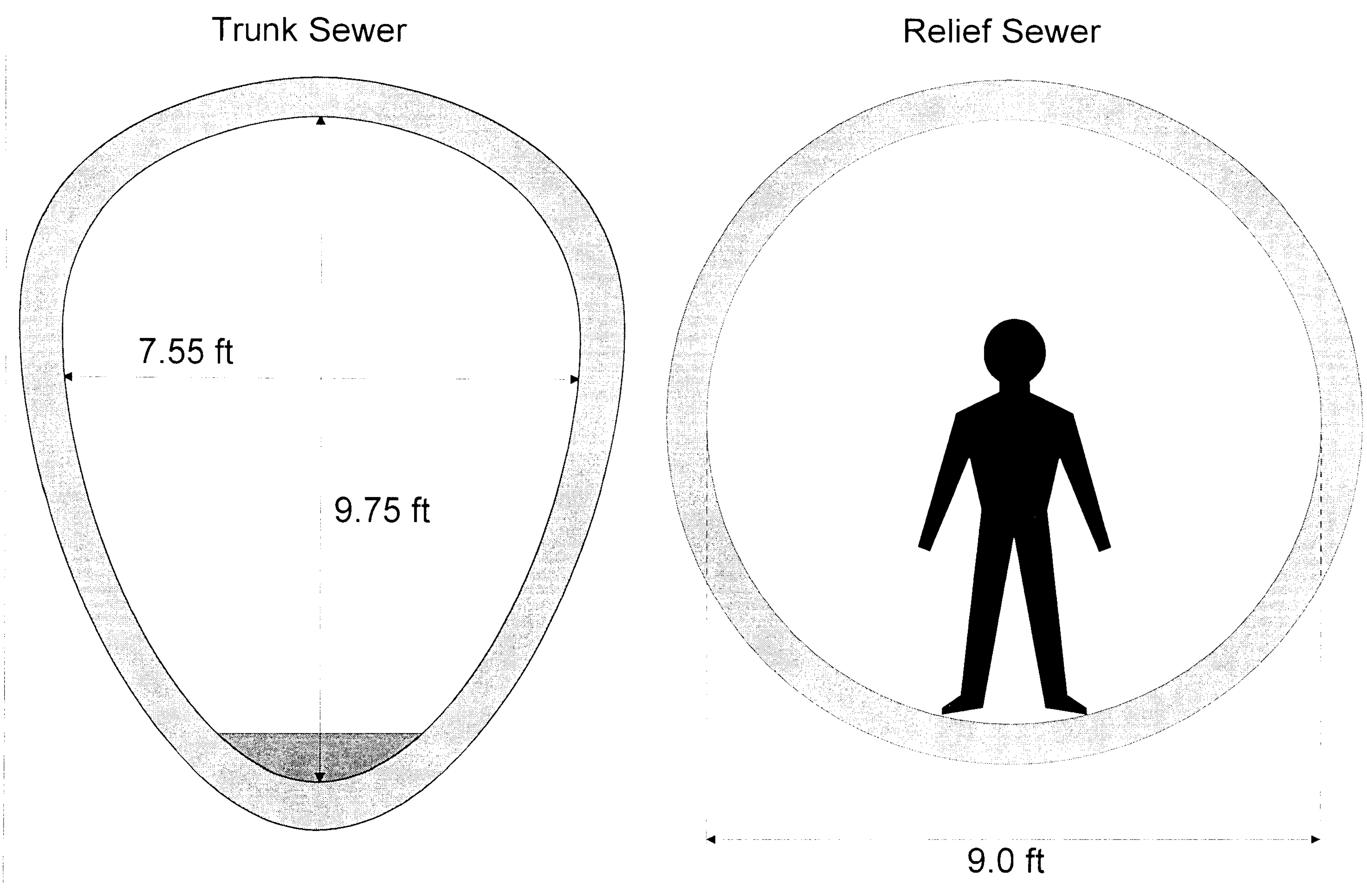


Figure 5-3:

Representative Illustration of Cross-sectional Size of a Combined Sewer Trunk and Sanitary Relief Sewer (SRS)

As Winnipeg continued to grow and develop, areas within the CS districts underwent changes in runoff characteristics. Green spaces decreased and were replaced with new and more impervious surfaces. This change, along with increased development "in-filling", has caused an increase in wet weather flows and resulted in higher incidents of basement flooding. The City recognized this growing problem and began taking action to improve the hydraulic conveyance of these systems through various relief programs. The primary objective of these basement flood relief programs was to improve the hydraulic capacity of the combined sewer system and improve the level of service (i.e., basement flood protection) to safely convey, at a minimum, a 1-in-5 year design storm. This was typically accomplished through the addition of relief piping, at strategic locations, to reduce the hydraulic gradeline (HGL) in the sewer network to a level (2.5 m below ground surface) that did not threaten basement flooding for a specific design storm.

Dry weather flows and their conveyance were typically unaffected by these programs. The relief sewers add to the potential available in-line storage but an important consideration relates to the level at which wet weather flows split into the relief systems. Figure 5-4 schematically illustrates a typical overflow manhole used to hydraulically relieve the main combined sewer trunk at a set elevation. The potential in-line storage in each district is a function of its original combined sewer system, and where applicable, the additional storage associated with the relief piping network.

The combined sewer system was originally designed to convey about 2.75 DWF to the wastewater treatment plants (WWTPs). Weir heights in the combined sewer trunk, just downstream of the off-take point to the interceptor, were set to satisfy this condition. Over the course of time, the interception capacity from the various CS districts has changed, typically being increased to higher than 2.75 x DWF.

The assessment of in-line storage potential involves the consideration of the storage volume contained within each CS district (empty pipe volume minus dry weather flow volume, and its specific dewatering rate [i.e., existing interception capacity or augmented capacity] minus dry weather flow), to empty the combined sewer system within a reasonable period of time (e.g., 24 to 48 hours) thus preparing for the next rainfall and preventing septicity-related nuisances.

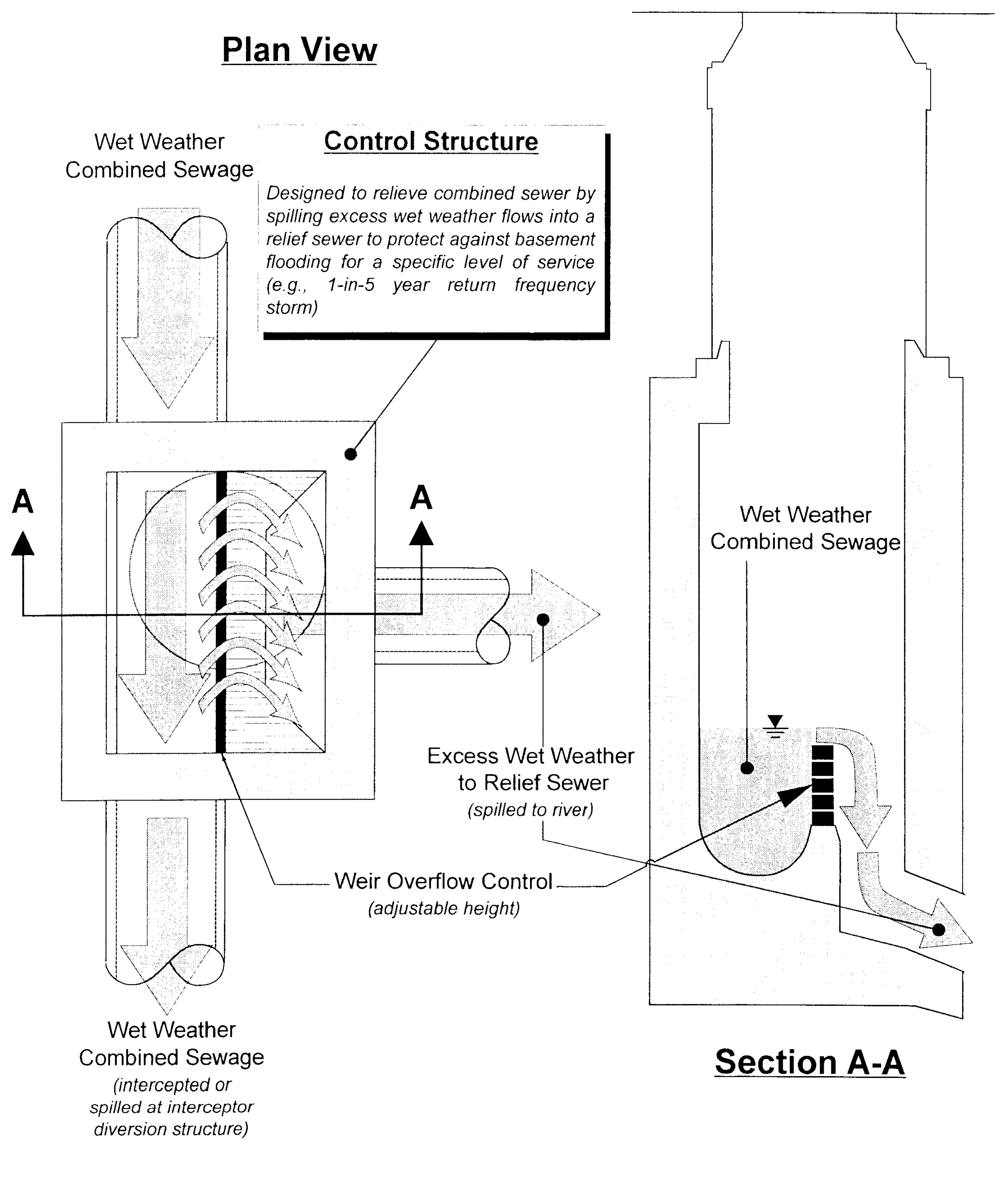


Figure 5-4:

Typical Overflow Weir Structure Used in Winnipeg to Hydraulically Relieve CS System for Basement Flooding Protection Purposes

Section 3.2.4 and 4.1 have discussed the dewatering aspects and associated performance in more detail. The following sections deal with the in-line storage potential associated with fully dewatered pipe volumes.

5.2.3 Estimating Potential In-line Storage

The following sequential steps were performed to evaluate in-line storage potential in the existing combined sewer systems.

- A preliminary evaluation based on existing basement flooding relief reports was used to determine a "first-cut" estimate of storage potential existing in the combined sewer system:
  - Phase 2 modelling results indicated sufficient storage may be available area-wide for in-line storage to be a valid control method.

- SWMM files from the City of Winnipeg 1986 Basement Flooding Relief (BFR) Study were used to better quantify pipe volumes and reduce uncertainty in volume estimates and area-wide extrapolation:
  - only 34 of the 43 CS districts had information, 9 were outstanding;
  - results indicated that a greater amount of in-line storage may be available and has significant potential by itself to reduce the number and volume of overflows;
  - remaining 9 districts could contain between 20% to 50% additional in-line storage volume and warranted further investigation and analysis.

- Detailed sewer information from City of Winnipeg Land-Based Information System (LBIS) or recent reports from other consultants were used to assemble sewer information (e.g., diameter, length, and invert elevations) on the remaining 9 outstanding CS districts.

- Detailed XP-SWMM modelling was applied on Clifton CS district as a test case to evaluate in-line storage potential;
  - Continuous modulation of an automated control gate was simulated to maximize available in-line storage and determine the maximum surcharge level that could be achieved without decreasing the existing level of basement flood protection;
  - 1-in-5 year synthetic design storm was applied;
  - inlet restriction was assumed in place;
  - results indicate for above conditions that the system could be controlled dynamically to utilize in-line storage while maintaining the existing level of service with respect to basement flood protection, even under design storm conditions.

- Concerns associated with the use of an automated gate required reassessment of permissible surcharge level and system operation. These concerns included:
  - fail-safeness of control mechanism, i.e., a failure could cause basement flooding;
  - possibility of water hammer with gate opening and closing;
  - development of air surges in response to a rapid fill of the sewers with the gate closed;
  - development of sink holes and reduction in sewer structural integrity resulting from increased frequency of surcharging;
  - increased sediment buildup in the sewers;
  - development of odour nuisance problems, water quality changes, corrosion from extended storage of sewage in the sewer pipes (dewatering rate considerations).

- A pilot project was proposed to test the operational considerations, and to identify other concerns that might need to be addressed with the use of in-line storage. The two locations considered for testing were:
  - Clifton CS district (see Appendix for details);
  - Hart CS district (see Appendix for details).

- At Working Session 3-4 held on January 14, 1997, it was identified, based on pipe configuration and geometry of combined sewer systems in Winnipeg, that many of the hydraulic concerns associated with in-line storage could be addressed by limiting the maximum storage level to the obvert of the trunk sewer at the control point. Advantages of this constraint include:

- does not permit the conditions for water hammer to set up;
- does not permit the condition for air surges to develop;
- does not increase the frequency of sewer surcharging (avoids sink hole and structural integrity concerns);
- utilizes significant portion of accessible in-line storage volume in major trunks of sewers;
- pilot testing project can be designed to address concerns associated with sediment buildup, odour potential, and water quality changes.

- During initiation of the Hart in-line pilot project (October 1997), direction was received from the City of Winnipeg that the basement flooding consequences associated with failure of the automated gate in the closed position, even though the risk is very low, could not be accepted by the City of Winnipeg at this time:
  - alternative concepts were explored to provide a fail-safe control option while maximizing in-line storage.

- A fixed weir design was found to be a viable solution (see **Section 5.2.4.4** for description):
  - inherently fail-safe;
  - weir elevations and location must be carefully selected to ensure the hydraulic gradient line during a design storm is not increased at any location which would cause or exacerbate basement flooding;
  - long weir lengths are required to minimize depth of flow over the weir (achieved through installation of "finger weirs" in chambers within existing right-of-ways);
  - opportunity to influence future relief programs to accommodate such structures and associated sewer pipe sizing;
  - must assess each district individually to determine storage potential and factors influencing control design.

As a result of the latter investigation, it was decided that any in-line storage program must include catchbasin inlet restrictor control to ensure that basement flood protection would not be compromised. This recommendation, combined with the decision to limit in-line storage to the obvert of the trunk sewer at the control point, demonstrated the potential for safe operation of in-line storage.

Operational implications associated with the use of in-line storage will require actual field data to answer the potential concerns associated with temporary storage of combined sewage, such as:

- additional sediment accumulation in sewer trunks;
- odour generation;
- changes in water quality of stored combined sewage;
- existing structural integrity;
- flushing considerations;
- dewatering limitations/constraints; and
- related maintenance issues.

5.2.4 Deriving Available Storage Volumes

The key factors required to estimate in-line storage are:

- adequate geometric description of the pipe network (i.e., diameter, lengths, invert, ground elevation, node network diagram) in each combined sewer district;
  - required for pipe volume calculations and depth below ground surface to estimate safe storage levels.

- maximum allowable storage levels in the combined sewer network to protect against basement flooding and hydraulic-related impacts due to in-line storage:
  - minimum of 3 m below ground level to protect against basement flooding;
  - maximum level at the control point not to exceed the obvert of the sewer pipe;
  - control device must not reduce existing level of service with respect to basement flood protection (i.e., must not exceed hydraulic grade line for design level of service).

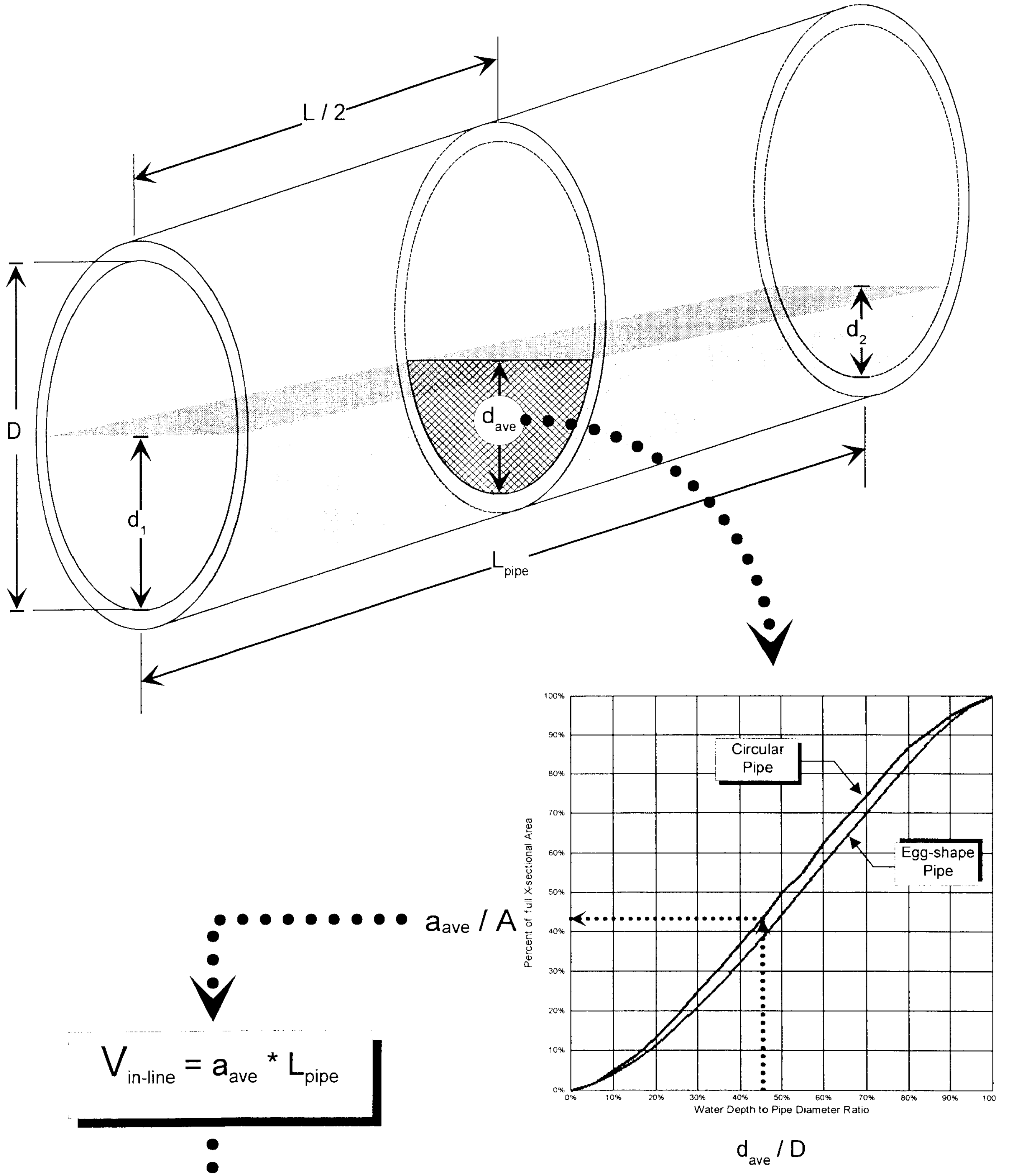
5.2.4.1 Methodology

The estimate of in-line storage volume accessible within the existing combined sewer systems was based on a dry pipe condition. Specifically, existing volumes associated with dry weather flows were not considered, at this stage, in the volumetric calculations. The pipe geometry (i.e., diameter, length and inverts) was used to calculate available in-system volume for a specific control evaluation. It was assumed that a horizontal plane would represent the maximum water surface within the pipe network for specific geodetic control elevation. The elevation of this horizontal plane is dependent upon the control technology selected (e.g., automated gate, fixed "finger" weir) and a specific minimum depth below ground level to protect against basement flooding. The lowest elevation was assumed to govern the achieveable in-line storage level and used to calculate the available in-system volumes. The volume contained below this plane in the pipe network represents the in-line storage available and potentially accessible for CSO control.

The volume for each pipe in the combined sewer system was based on the average depth in each pipe at its mid-point (i.e., halfway between nodes) for a specific control elevation (refer to Figure 5-5 for details). The individual pipe volumes were summed to calculate the available storage within each combined sewer district. The procedure was repeated for each of the 43 combined sewer districts found in Winnipeg. The estimated volumes represent the potential storage available in each individual CS district and on a system-wide basis for selected control elevations.

As well as this detailed review of the combined sewer systems on pipe geometry, critical elevations (invert and ground) were used to calculate in-line storage volume available in each of the 43 CS districts. Sewer pipe information and critical elevations originated from sources discussed in Section 5.2.3.

A standardized spreadsheet model was developed and applied to each CS district. The spreadsheet model was formulated to reference each pipe and node in the sewer network for information pertaining to:



$$V_{in\text{-}line} = a_{ave} * L_{pipe}$$

$$V_{district} = \sum_{i=1}^{n} (a_{ave} * L_{pipe})_i$$

Figure 5-5:

In-Line Storage Calculation
Volume Contained within Sewer Pipes for a Specific Control Elevation (horizontal plane)

- upstream and downstream nodes;
- invert and ground elevation at each node;
- pipe type (circular or egg-shaped) and associated cross-sectional area; and
- pipe lengths.

Using this information, it was possible to estimate the volume of potential storage available in each pipe below a specified elevation (i.e., horizontal plane), as previously shown in Figure 5-5. The information used represented the actual sewers in the ground and not proposed relief piping.

5.2.4.2 Outflow Control Technology

As expected, in-line storage varies significantly between districts and is unevenly distributed across the city. As well, CS districts that have been relieved to improve basement flood protection contain the greatest volumes of in-line storage potentially available for CSO control.

In order to access in-line storage, it is necessary to install a means of temporarily constricting overflows from a combined sewer (and the relief system, where applicable) to capture all or a portion of a rainfall event. Since the sewers were designed to reduce the frequency of basement flooding, the control device used to contain in-line storage must be designed to permit these design storms (either a 1-in-5 or 1-in-10-year return frequency storm) to be discharged to the river while not affecting the existing level of basement flood protection.

The review found that the available effective storage is influenced by the control technology selected. Specifically, the definition of the safe operating level in the sewer is a key parameter with respect to estimating in-line storage volume. The different control technologies studied were:

- automated gate controls;
- fixed weirs; and
- the use of existing latent storage.

Figure 5-7 compares the levels in the sewer for the various systems discussed below.

5.2.4.3 Automated Gate

The first control technology considered a dynamically-controlled motorized sluice gate (with suitable redundancy both in mechanical and in control systems) to utilize available in-line storage. Some of the issues that arose are discussed below. The following discussion addresses some of the operational concerns which were identified with regard to the hydraulics involved in restricting combined sewer overflows to utilize available in-line storage.

A main issue associated with the use of in-line storage is maintaining existing basement flooding protection or the current fail-safeness of the system. This issue was specifically addressed in the computer modelling and design of two proposed pilot projects discussed below.

Clifton Pilot

During the development of the conceptual design for the Clifton Pilot Project, the technology proposed comprised the modification of the existing gate on the Clifton trunk combined sewer and the insertion of an inflatable dam ("Fabridam") in the storm relief sewer. Because the Clifton trunk and the SRS operate in an integrated fashion, it was necessary to have two control devices in the system. The study team considered that this provided the opportunity to test these two devices, which had been successfully used in Cleveland, Ohio.

The proposal for the Clifton facility was to replace the existing screw shaft, for raising and lowering the gate, with a hydraulic piston. To ensure fail-safeness, the hydraulic pump would have a back-up air accumulator in case of power interruption or failure, the gate would be raised by the hydraulic system being driven by an air accumulator. For the inflatable dam, in the event of a power failure, the air would be released from the dam and therefore the sewer would be left virtually unobstructed. Insofar as level of storage was concerned, the control system (illustrated in Figure 5-7) would have been set such that, when the water level reached

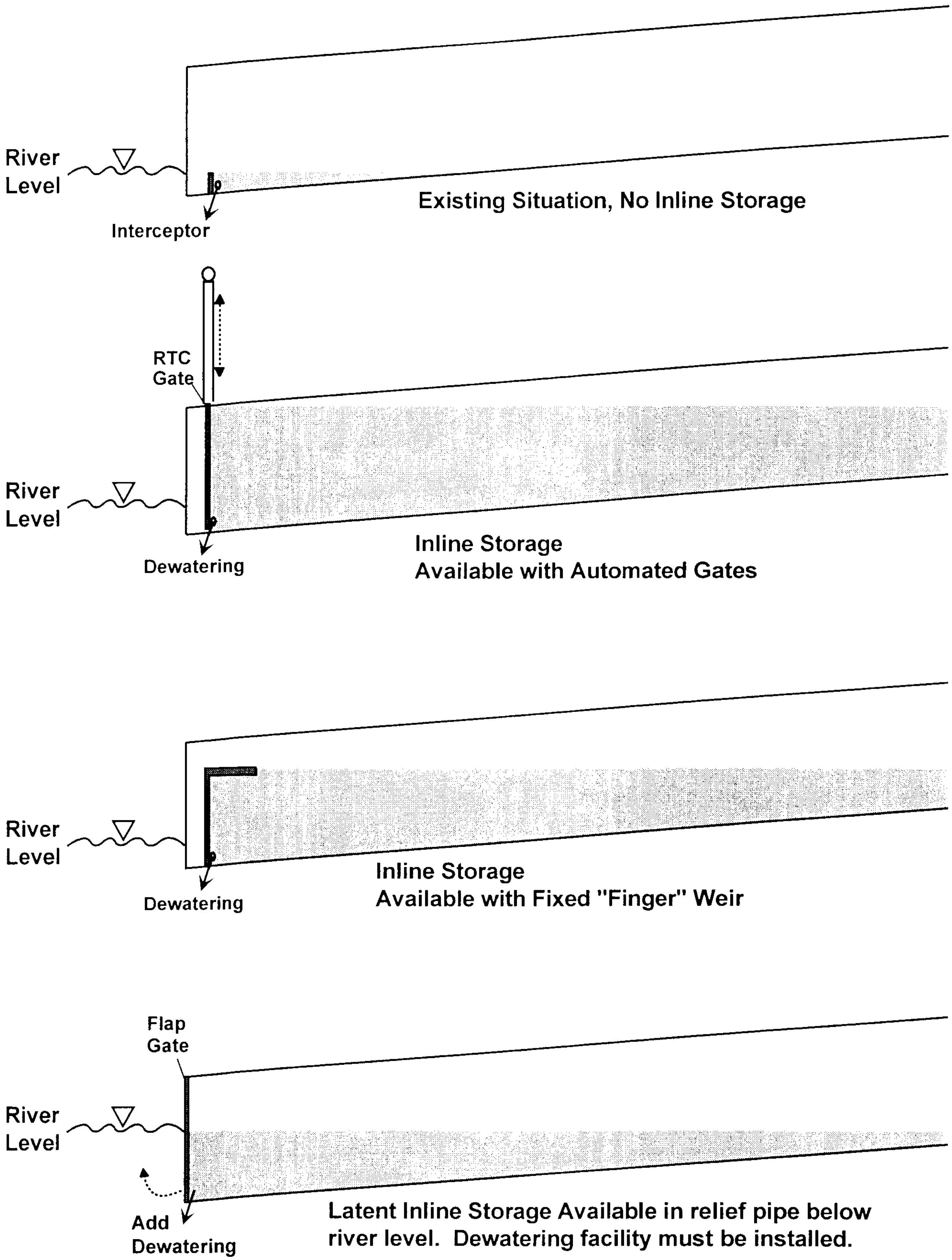
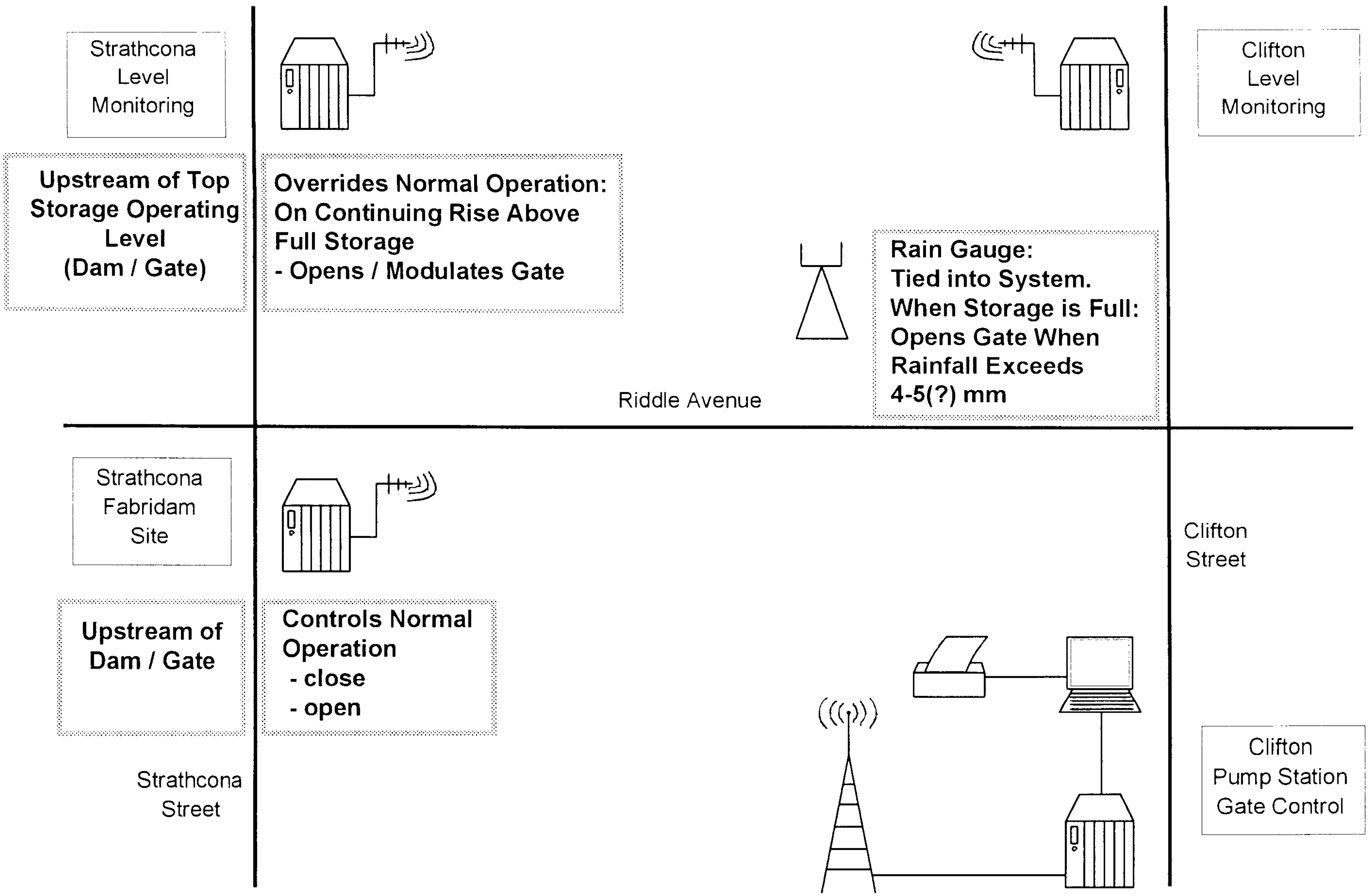


Figure 5-6:

Illustration of In-Line Storage Levels for Specific Control Systems



CSO Monitoring and Control System
Figure 5-7

csosyst
s:\01\0510

the desired level for in-line storage, any further increase in level would open the gate (or collapse the dam). Both of these level controls would have been overridden by a control further upstream. In the event that in-line storage was full and a second storm occurred which increased the flows and corresponding levels at the upstream control point, the device would override the level control at the outlet and open the devices. A control system would have been designed such that any failure in the system, either electrical or control related, would result in the opening of the gate, i.e., the system was designed to be virtually fail-safe.

As a part of the Clifton Pilot investigations, the system was modelled using the XP-SWMM software. The purpose was to test the impacts of the control devices, and their use for in-line storage, on basement flood protection. The intent was to ensure that the operation of the system would not reduce the existing level of basement flood protection provided by the trunk sewer and its relief. The detailed SWMM modelling is discussed in the Clifton Appendix.

The first modelling test was to simulate the response of the system for a 5-year design storm with the control devices in place and with the system starting empty. The results indicated that in-line storage would not interfere with the capability of the system to prevent basement flooding with this scenario. It was determined that the main reason was the fact that the peak in the trunk collectors was passed before the peaks were reached in the lateral sewers. Accordingly, there was more than adequate capacity to convey the flow in the laterals.

The second modelling test comprised the application of the 5-year storm to sewers which had already reached the in-line storage levels. In this case, there was back-up in the laterals and hence basement flooding would have resulted. In order to rectify this, catchbasin inlet restrictor controls were applied to the system (in the model). Assuming a blanket application of controls over the whole district (this would be difficult to achieve in fact), it was determined that the levels reached in the system would be less than the first modelling scenario. It was presumed that a detailed investigation and appropriate application of inlet controls would, at least, result in a condition in which the in-line storage operation would not interfere with basement flood relief.

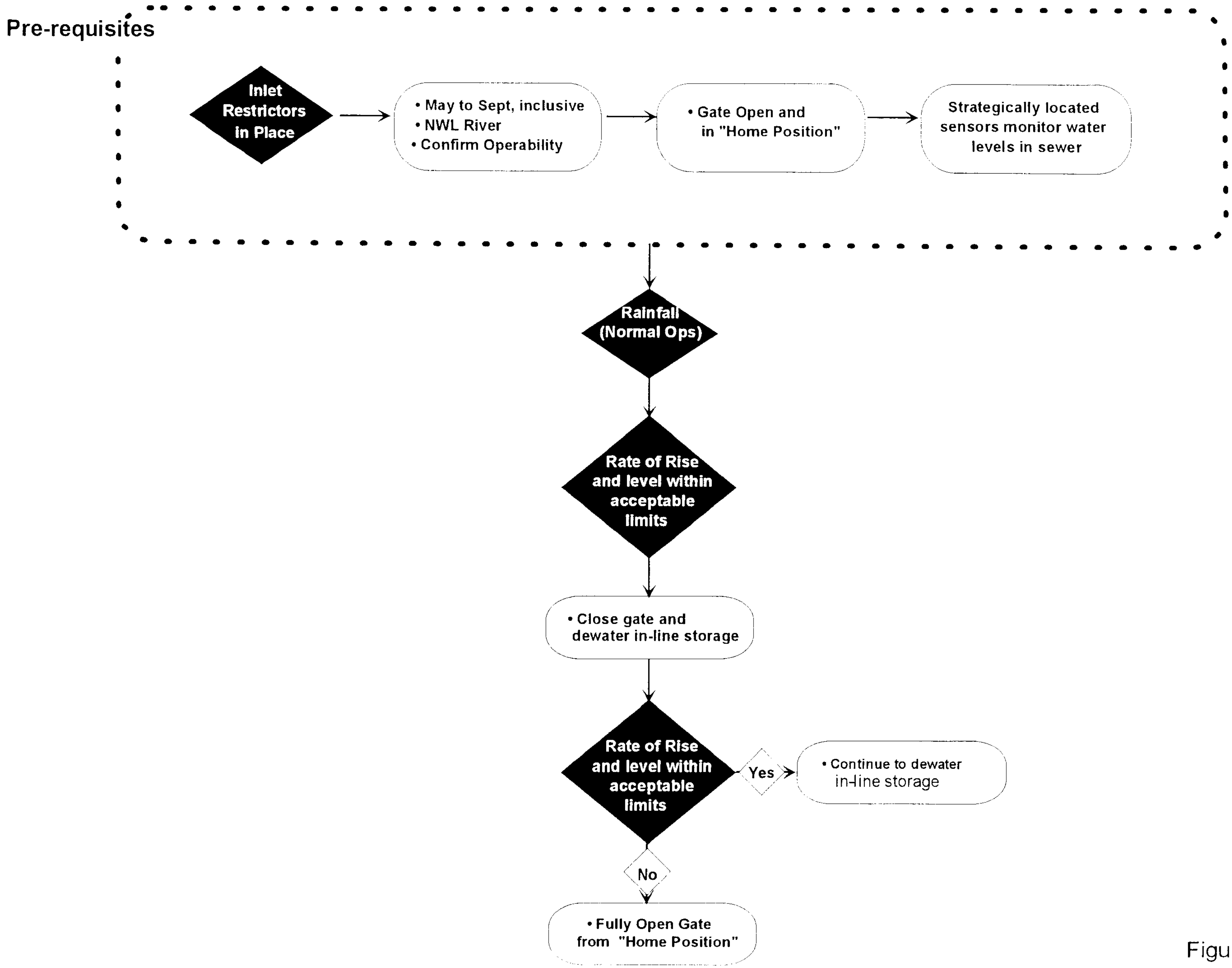
During the functional design of the Clifton facility, a difficulty arose resulting from the fact that the storm relief sewer was on an easement crossing a private Church property. The easement agreement was such that the Church had control about what could be constructed on the easements. The "Fabridam" would have meant the construction of an aboveground facility for control and housing the air-compressors. Rather than becoming involved in an extended negotiation period, it was decided to look elsewhere for the pilot project. It was determined that the in-line storage concept could be proven by a single gate on a system which was relieved but had not required a separate relief sewer outfall. This would reduce the cost of the pilot project and meant that any alternative technology, such as the "Fabridam", could be tested at some later date. The Hart district met these requirements and it was decided to consider this district as the in-line storage pilot.

Hart Pilot

It was initially intended to adapt the existing Hart sluice gate to be the control device for in-line storage. It was planned to motorize the gate and to use a stand-by air-driven motor as a back-up to the electric drive normally used to open the gate. The back-up control systems would have been similar to those proposed for the Clifton system. However, during the course of the functional design, the senior City of Winnipeg Water and Waste Management Team (October 8, 1997) expressed concern with the risk associated with the use of a motorized gate as a means of controlling in-line storage in the trunk systems. The added risk, however small, of impinging on basement flood protection could not be accepted by the City. The study team was asked to develop alternative strategies for the pilot.

This direction led to two undertakings, the first was to develop a response plan in case of gate failure and to determine the associated level of basement flooding risk. Figure 5-8 comprises the operating procedures which would have been put in place for the pilot, and presumably, subsequently, for real-time installations modified through the results of the pilot. Figure 5-9 outlines the contingency events and the back-up systems to mitigate anticipated failure of the automated gate control system. Several malfunctions would have to occur in a specific order before there would be any chance of increased basement flooding risk. The final scenario envisages failure of the gate shaft so the gate cannot open. The concern in this situation is the limited amount of storage available before the flood pumps could be initiated to respond to gate failure in the closed position. If the storm peak arrives after available in-system storage is

Normal Operations

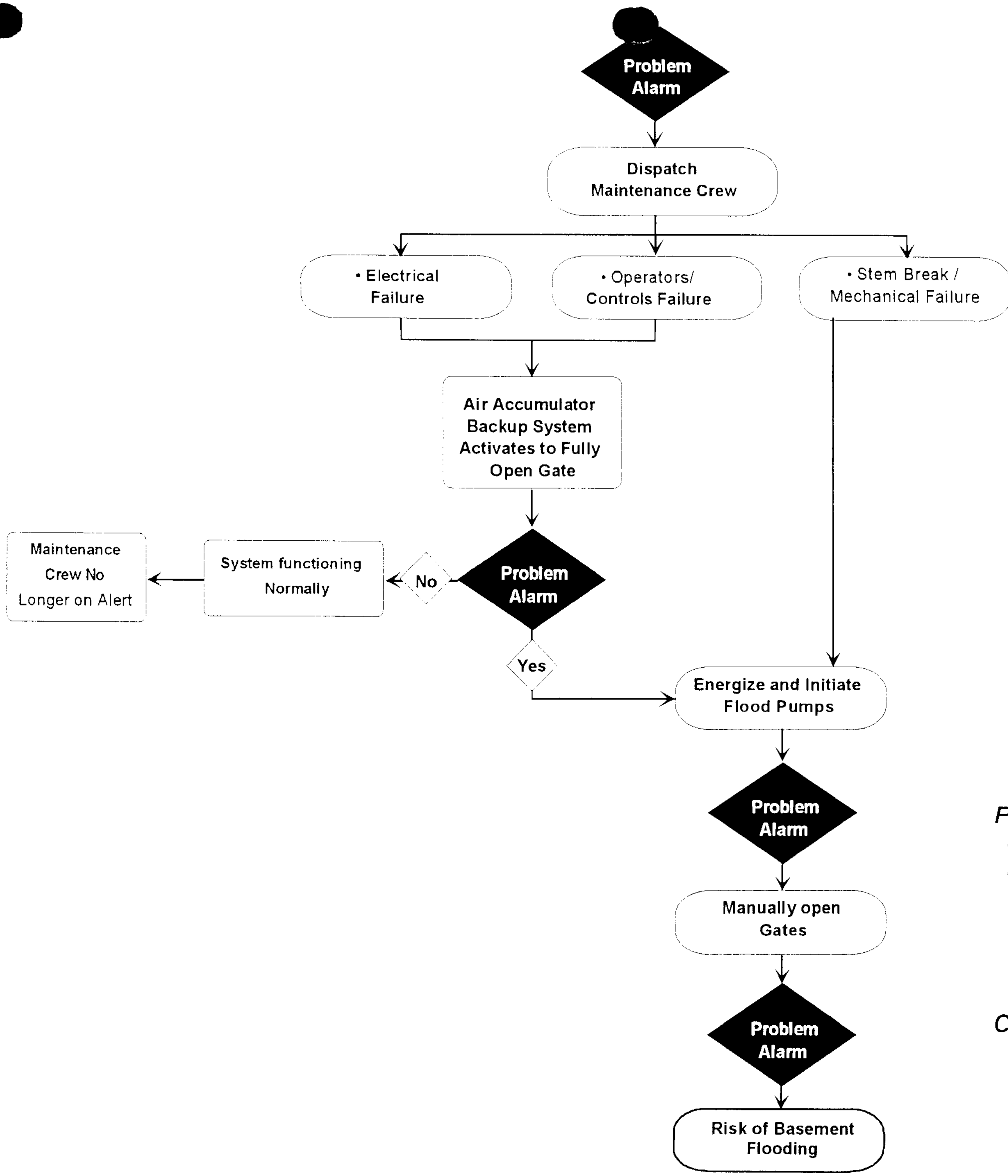


hartflow

Figure 5-8

Contingency Events

Main System Failure



Backup System Failure

Flood pumps can't keep up with R/O
- pumps did not activate
- intense rainfall

Crew fails to Open Gate fast enough

hartflow

Figure 5-9

exhausted and before the flood pumps are started, there would likely be insufficient time to open the gates manually and avert the possibility of basement flooding. However, the circumstance, as noted, requires a number of concurrent failures in order to become real. It is recognized that the risk is low but the consequences are large and accordingly warrant careful review of an automated gate system and associated backup.

The Management Committee suggested consideration of a number of activities which could be incorporated into the automated gate option to improve its reliability and make it "virtually fail-safe". These considerations comprised the installation of backwater valves throughout the combined sewer district; the option of the City purchasing flood insurance (or self-insuring) for all dwellings in the combined sewer districts that could be potentially affected; and the installation of catchbasin inlet restriction. Inlet restriction is already considered as part of the basement flood relief programs and is typically employed to raise the basement flood protection from a 1-in-5 year to a 1-in-10 year level of service. Its use, in conjunction with the in-line storage, would result in a re-prioritization of the installation of the inlet restrictors. Preliminary estimates were performed on the first two options (Table 5-1) and tended to indicate that these options would likely cost more than the fail-safe option of a fixed "finger" weir system discussed in Section 5.2.8.1.

The Management Committee also suggested that other alternatives be considered which were inherently fail-safe, which lead to a re-examination of fixed weir (Section 5.2.4.4).

Waterhammer

During the initial conceptual design of a dynamic control gate (using the Clifton District as an example), it had been proposed (and evaluated by computer model) to develop storage up to a safe surcharge level that could maximize available storage without compromising the existing level of basement flood protection. Further, it was proposed to modulate the gate position to maintain this safe level of surcharge and reduce combined sewer overflows even during larger storms. Detailed hydraulic modelling for the existing level of services (i.e., 1-in-5 year storm) found that a modulating gate could maintain storage levels up to 2.5 m above the centerline of

TABLE 5-1

IN-LINE STORAGE – INDIRECT OPTIONS
ADDITIONS TO VIRTUALLY FAIL-SAFE OPTION (GATE)

•	**Backwater valves (including sump pump for footing tiles)**
-	1986 report described this as "flood-proofing" and included for sump pumps. City of Winnipeg estimate direct costs @ \$1,200 to \$1,600 + 25% for add-ons (say \$1,500 per installation) @ 120,000^{+} buildings (CS districts only) = \$180 million
-	eliminates concerns with gate failure
-	potential to provide significant BFR benefits (spring and summer)
-	depends on reliable operation by building tenant
-	not without its concerns (responsibility?)
-	sewer relief would still be needed (overland flooding)
•	**City Flood Insurance**
-	say \$10/building/year
-	specifically addressing gate failure @ 120,000^{+} buildings and 4% discount rate, P.V. = \$160 million
-	there is a need to determine if such a limited coverage is obtainable or practicable
•	**Catchbasin Inlet Restriction**
-	already part of BFR program – to protect against 1-in-10 year return frequency storm
-	impact of in-line storage gate installation would be to reprioritize installation

the outlet (about 4.5 m below minimum ground level) of the CS trunk and SRS without compromising the existing level of basement flood protection.

Concern was expressed that changes in flow rate in the trunk due to operations of the gate could set up hydraulic transients and create a potential for waterhammer to develop. Theoretical calculations were performed for a worst-case scenario and confirmed that the potential for such a situation to develop exists. This potential could be avoided by not modulating the gate during WWF. Once the gate opened during a WWF event, the gate would remain open until the situation returned to "normal". At this time, the gate would return to its "home" position. This method of operation would have eliminated any waterhammer potential. It was also found that this potential problem can be avoided by limiting storage level not to exceed the obvert of the CS trunk or SRS at the location where the control device is to be installed.

Air Surges

During the course of the Working Session 3-5, it was identified that there was a secondary concern associated with the particular configuration of the City of Winnipeg's combined sewer trunks. As can be seen from Figure 5-1, there is a potential, under near full pipe flow, for entrapment of air at the crown of the pipe. This could result in air surges in the trunk sewer and cause pressure surges in service connections along this reach of trunk sewer.

This potential problem can be avoided by limiting storage level not to exceed the obvert of the CS trunk or SRS at the location where the control device is to be installed.

Sink Holes

At Working Session 3-5, the City operating personnel expressed concern that the continual surcharging of pipes in response to in-line storage might cause and accelerate more sinkholes to form. It was believed that "pumping" of the soil above and around the pipe and removing it via cracks or discontinuities in the sewers in response to increased frequency of sewer surcharging. This could result in the undesirable condition of sink holes forming under the road surface. The City has already experienced this problem with the large trunk sewers. The decision to operate in-line storage up to a maximum of the obvert at the proposed point of control would eliminate this concern, insofar as in-line storage is concerned. It does not address the concern of periodic operation of the trunks at hydraulic gradients above the crown of the pipe during major storms and without catchbasin inlet restrictor controls.

<u>Summary</u>

For the gate control option, the obvert of the pipe is the point of control on the combined sewer trunk or the minimum ground elevation less 3.05 m (10 ft) whichever results a lower elevation, was used to establish the set point to calculate accessible in-line storage. Table 5-2 summarizes the accessible in-line storage volume that could be achieved under these criteria for each combined sewer district as they now exist.

Automated gate control on both combined and relief sewer systems can achieve approximately 370,000 m^3 of storage. This in-line storage system was assumed when modelling the performance of in-line storage with respect to number and volume of overflows (in June 1997).

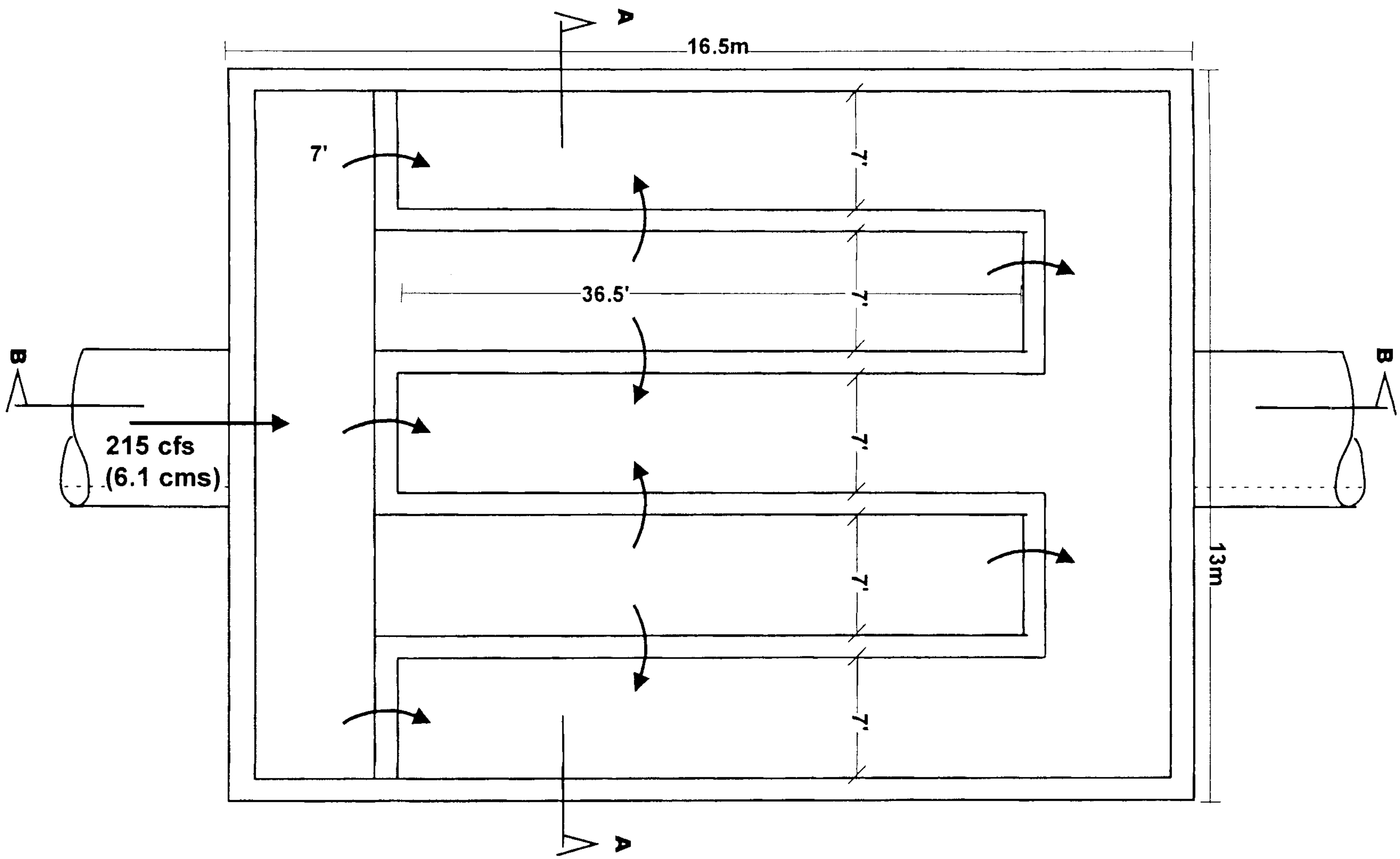
5.2.4.4 Fixed Weir Option

In response to the concerns raised by Management Committee over the potential for gate control failure and basement flooding, a "fail-safe" option was developed comprising the use of a fixed weir as a control device. The fixed weir utilized long weir length "fingers" to minimize flow depth over the weir to safely convey the design storm while maximizing available in-line storage. The use of a fixed "finger" weir system is considered inherently fail-safe and practicable. Plans and sections of such a device as might be installed in the Hart CS District trunk sewer near its outlet are shown on Figures 5-10 and 5-11.

The height of the weir was established by selecting a design criteria of 150 mm (6 in) depth of flow over the weir and integrating this with the maximum allowable water surface profile for the design storm for the existing pipe system. For the Hart District, it was estimated that

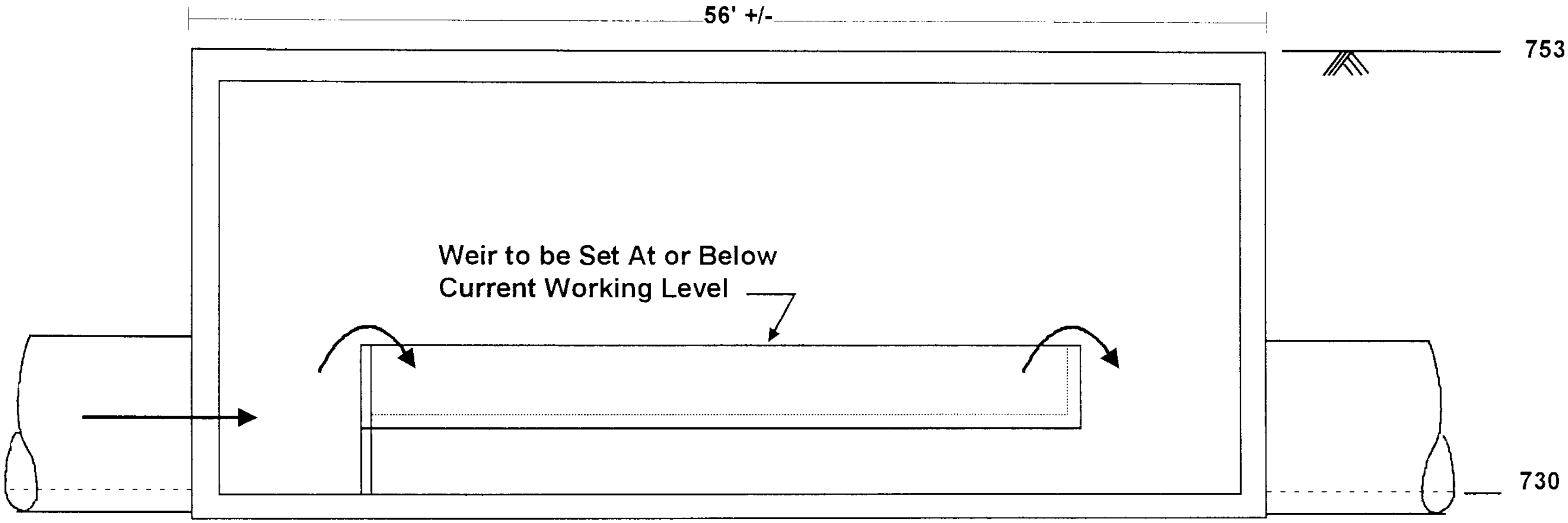
Table 5-2: Existing Conditions
Potential Storage Available Utilizing an Automated Gate

			1	2	1+2 =3	4	3	19	20	21	22
District Name	Relief Status	Tributary Area (Ha)	Height of Sewer	Invert	Obvert	Min. Ground	Obvert	10 ft below Min Ground	Control Elevation	Depth below min ground	Volume (m³)
Alexander	no	146	5.50	736.990	742.490	758.500	742.49	748.500	742.490	16.0	3,690
Armstrong	no	148	9.00	730.770	739.770	748.500	739.77	738.500	738.500	10.0	5,313
Ash	yes	823	10.00	732.500	742.500	762.139	742.50	752.139	742.500	19.6	33,978
Assiniboine	yes	75	4.00	740.720	744.720	752.570	744.72	742.570	742.570	10.0	6,495
Aubrey	yes	390	9.33	732.890	742.220	760.000	742.22	750.000	742.220	17.8	50,316
Baltimore	yes	211	6.00	732.159	738.159	753.000	738.16	743.000	738.159	14.8	1,026
Bannatyne	yes	206	5.00	736.970	741.970	759.000	741.97	749.000	741.970	17.0	14,015
Boyle	no	25	3.00	734.040	737.040	754.500	737.04	744.500	737.040	17.5	39
Clifton	yes	415	9.75	734.180	743.930	762.000	743.93	752.000	743.930	18.1	6,782
Cockburn/Calrossie	no	243	8.83	732.540	741.370	759.750	741.37	749.750	741.370	18.4	5,576
Colony	yes	227	6.00	738.280	744.280	759.500	744.28	749.500	744.280	15.2	12,464
Cornish	yes	127	5.00	734.850	739.850	756.000	739.85	746.000	739.850	16.2	5,439
Despins/Marion	no	317	4.43	730.774	735.203	746.063	735.20	736.063	735.203	10.9	4,443
Doncaster	no	133	7.50	743.100	750.600	763.770	750.60	753.770	750.600	13.2	5,823
Douglas Pk/Ferry Rd	no	251	10.00	738.840	748.840	758.000	748.84	748.000	748.000	10.0	6,204
Dumoulin/La Verendrye	no	136	11.48	737.690	749.173	754.500	749.17	744.500	744.500	10.0	1,148
Hart	yes	142	9.33	731.370	740.700	749.250	740.70	739.250	739.250	10.0	5,465
Hawthorne	no	219	5.50	734.010	739.510	752.800	739.51	742.800	739.510	13.3	8,397
Jefferson E & W	yes	977	11.83	731.480	743.310	751.500	743.31	741.500	741.500	10.0	21,046
Jessie	yes	338	7.87	731.791	739.665	759.514	739.67	749.514	739.665	19.8	5,372
Linden	yes	149	4.50	733.180	737.680	748.000	737.68	738.000	737.680	10.3	1,455
Mager Drive	yes	260	11.25	734.900	746.150	755.500	746.15	745.500	745.500	10.0	9,427
Metcalfe	no	34	5.33	731.810	737.140	757.500	737.14	747.500	737.140	20.4	967
Mission	no	421	9.75	730.910	740.660	756.000	740.66	746.000	740.660	15.3	8,007
Moorgate	no	157	8.25	744.430	752.680	765.000	752.68	755.000	752.680	12.3	3,592
Munroe	yes	375	10.50	733.600	744.099	752.850	744.10	742.850	742.850	10.0	42,482
Newton	no	56	6.00	734.380	740.380	750.620	740.38	740.620	740.380	10.2	1,847
Polson	yes	238	7.14	730.217	737.352	751.706	737.35	741.706	737.352	14.4	21,854
River	yes	108	5.00	730.970	735.970	753.800	735.97	743.800	735.970	17.8	2,855
Riverbend/Parkside Dr.	no	189	7.50	740.670	748.170	760.000	748.17	750.000	748.170	11.8	6,872
Roland	yes	178	9.50	733.410	742.910	756.500	742.91	746.500	742.910	13.6	26,462
Selkirk	yes	259	6.67	733.900	740.570	750.000	740.57	740.000	740.000	10.0	7,912
St. Johns	yes	335	6.33	732.841	739.173	755.249	739.17	745.249	739.173	16.1	24,975
Strathmillan	no	69	3.00	749.930	752.930	764.500	752.93	754.500	752.930	11.6	96
Syndicate	no	79	3.50	739.940	743.440	751.000	743.44	741.000	741.000	10.0	35
Tuxedo	no	50	3.00	740.000	743.000	759.000	743.00	749.000	743.000	16.0	241
Tylehurst	no	185	8.83	738.180	747.010	762.500	747.01	752.500	747.010	15.5	4,829
Woodhaven	no	42	4.00	755.110	759.110	766.800	759.11	756.800	756.800	10.0	75
		8,733									367,012



Fail Safe Option
Figure 5-10

failsaf1
s\01\0510



Section B - B'

$Q_{5\ YR}$ = 215 cfs
$V_{5\ YR}$ (not full) = 5.6 fps
$hv_{5\ YR}$ = 0.5'
V_{AOF} = 1 fps
hv = negligible

failsaf2
s\01\0510

Figure 5-11

approximately 55 m (180 ft) of effective weir length would be required for a 150 mm depth of flow to achieve the flow capacity needed to convey a 1-in-5 year design storm. Establishing a weir height based on this basis would not significantly interfere with the existing hydraulic gradeline (HGL) and would assure that the existing level of basement flood protection for the 5-year storm is preserved. The improved flood protection is achieved at the sacrifice of reduction in the volume of in-line storage available before overflow.

Basement Flooding Protection

The utilization of a fixed weir system in the main combined sewer trunk (and where applicable in the relief piping) can access in-line storage without jeopardizing the existing level of basement flood protection. Using a design criteria of 150 mm (6 inches) of flow over the weir, the height of the weir plus 150 mm was set to equal the modelled HGL at the point of control. The HGL in both the combined sewer trunk and relief pipes at the select point of controls was compared (**Figure 5-12**). It was possible in some cases to move the selected point of control up the system to synchronize HGL elevations and maximize in-line storage. On review of the relief systems, it was found that many of the outlet pipes are partially submerged under normal river water levels (NWL) and gated to prevent river flow into the system under high river water levels. As well, it was found that most of these relief pipes run at near constant grade to the river, including the outfall section of pipe. For practical reasons, the hydraulic water surface was selected one or two pipes back from the outfall. Where applicable, the elevation of the hydraulic gradeline in the combined sewer and relief pipe were compared and the lower of the two used for estimating the top elevation of the weirs. Where it was not possible to do so, the lowest HGL elevation was used in establishing the top of the weir elevations at all control points. The top elevation of the weir or the minimum ground elevation less 3.2 m (10.5 ft.), whichever is lower, was then used as the set point to calculate insystem storage that could be attended using a fixed weir option.

Available conceptual and design hydraulic reports for combined sewer systems were reviewed to compile the computed hydraulic gradelines. The computed water surface elevation (for the specific design rainfall event) at the diversion structure in the main CS trunk and at a logical break point in the relief point was selected as the control elevation for the fixed weir control

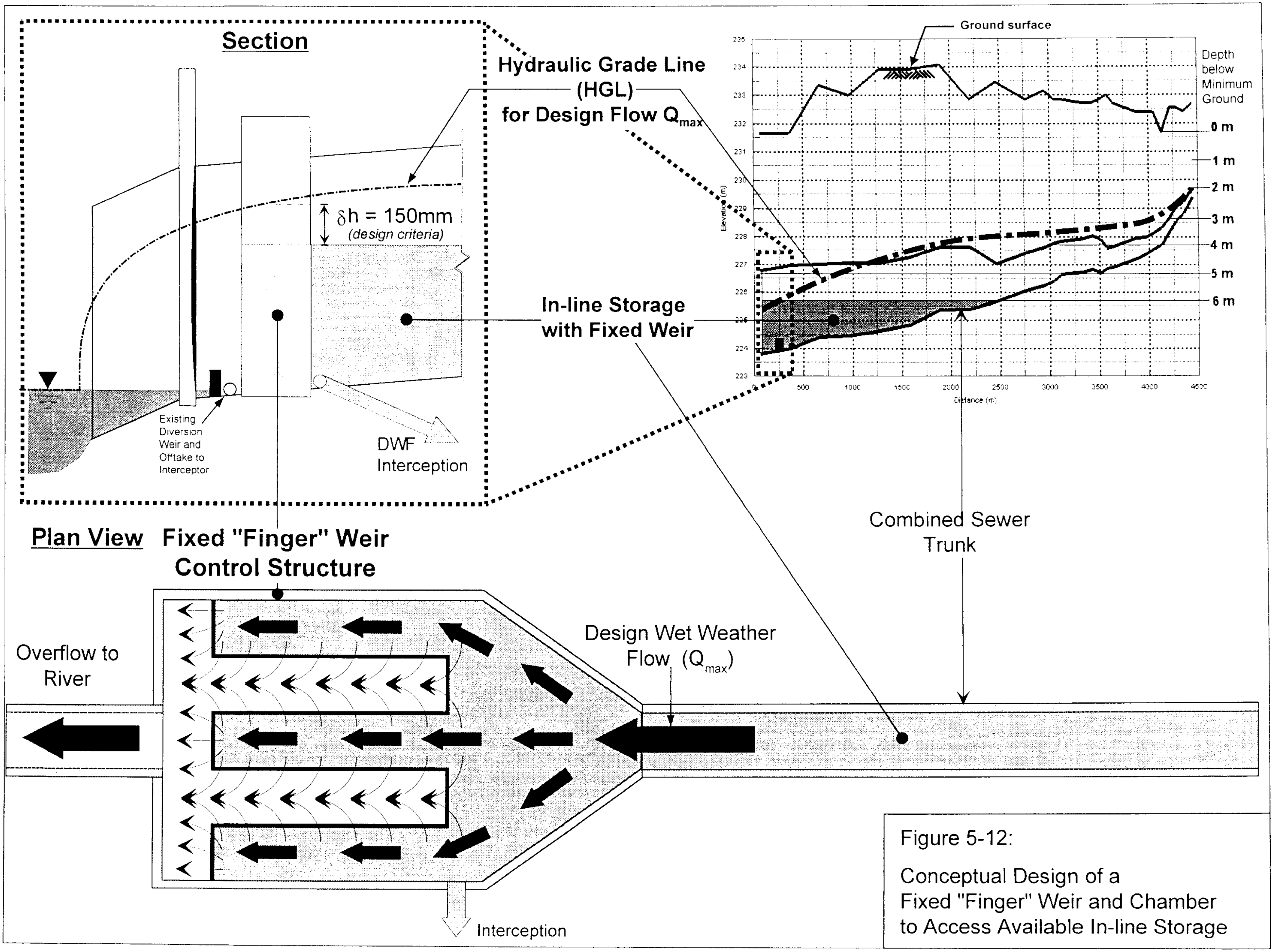


Figure 5-12:

Conceptual Design of a
Fixed "Finger" Weir and Chamber
to Access Available In-line Storage

option. Many of the CS trunks are at a near uniform grade except in the near vicinity of the river, as shown in Figure 5-13. The hydraulic gradient, for the selected design storm, tends to have a steep gradient in the outfall pipes themselves. From this configuration, it can be determined that the most logical location and greatest benefit of a fixed weir, in terms of in-line storage, would be gained from its placement at or upstream of the diversion structure in the main CS trunk.

Figures 5-14 and 5-15 graph the key pipe characteristics of combined sewer and relief pipes along the Red and Assiniboine Rivers. As discussed, the fixed weir design was based on the criteria to safely pass the design storm (e.g., 1-in-5 year return frequency) utilizing a 150 mm (6 inch) maximum depth of flow over the weir. Accordingly, the maximum weir height was set 150 mm (6 inches) below the modelled water surface at the selected control locations and represents the volume of storage that is potentially available with a fixed weir.

Since information on HGL (for the specific design storm used to assess basement flood protection) was limited to available studies, it was necessary to extrapolate the results area-wide to estimate in-line storage available for each district without this information. As can be seen from Figures 5-14 and 5-15, the height of the HGL above the invert was found to range between 0.6 to 1.5 and average 0.85 of the height of the pipe. The actual HGL water depth was used for the specific sewer districts where information was available. A value of 0.85 of the pipe height was used for those remaining combined sewer districts without modelling information. It should be noted that unrelieved districts have the opportunity to integrate this condition (or better) in future relief designs as well as oversizing relief pipes to maximize future potential storage.

Weir Chamber Hydraulics

The insertion of a fixed "finger" weir in a combined sewer trunk, and where applicable in the relief sewer, was recognized as a significant hydraulic control structure that could affect design flow and associated water levels. Additional analyses may be required to determine the effective height of the weir and the corresponding in-line storage volume that could be achieved.

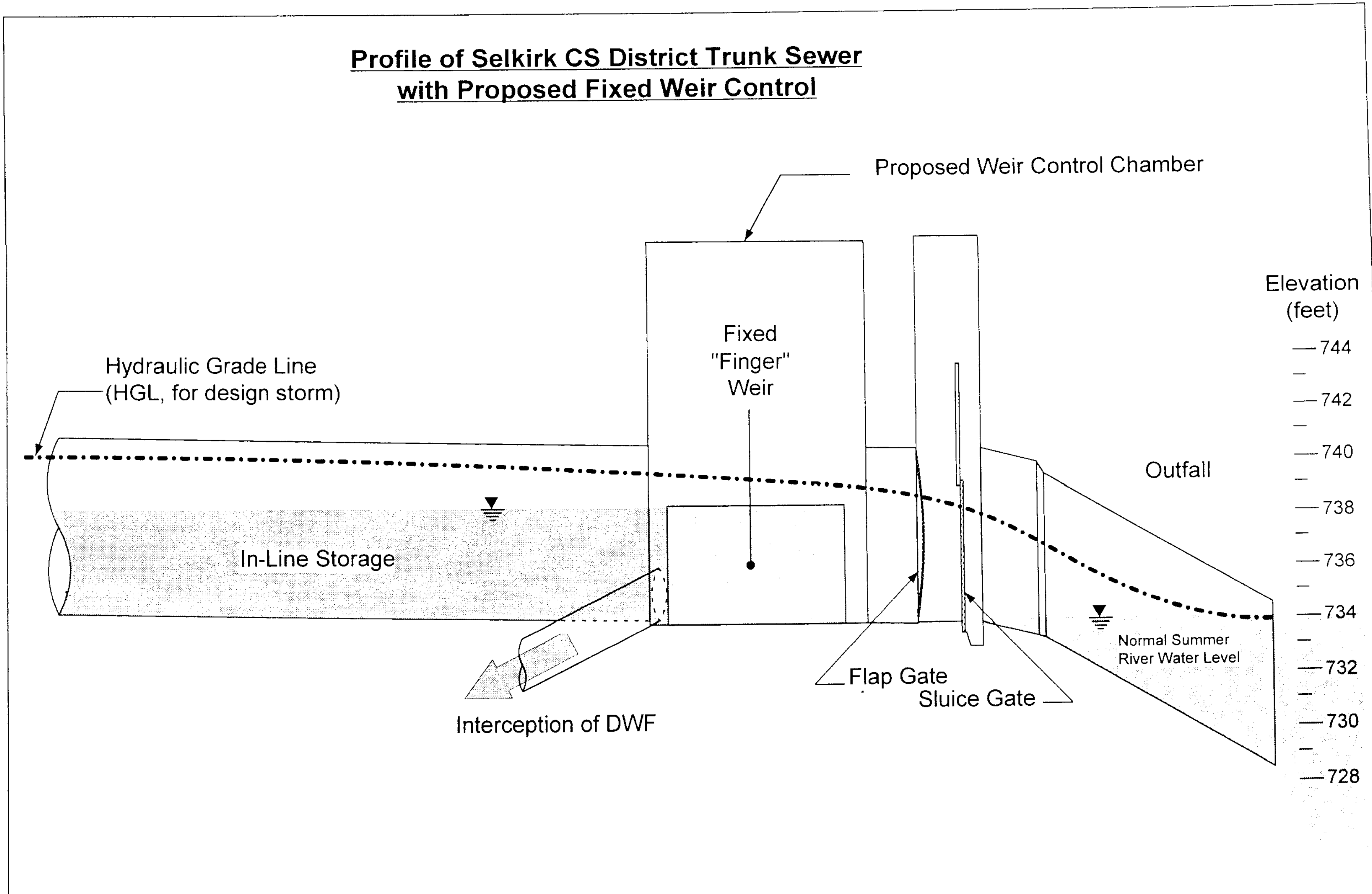


Figure 5-13:

Example of a Typical CS Outfall
With a Proposed Fixed Weir Control

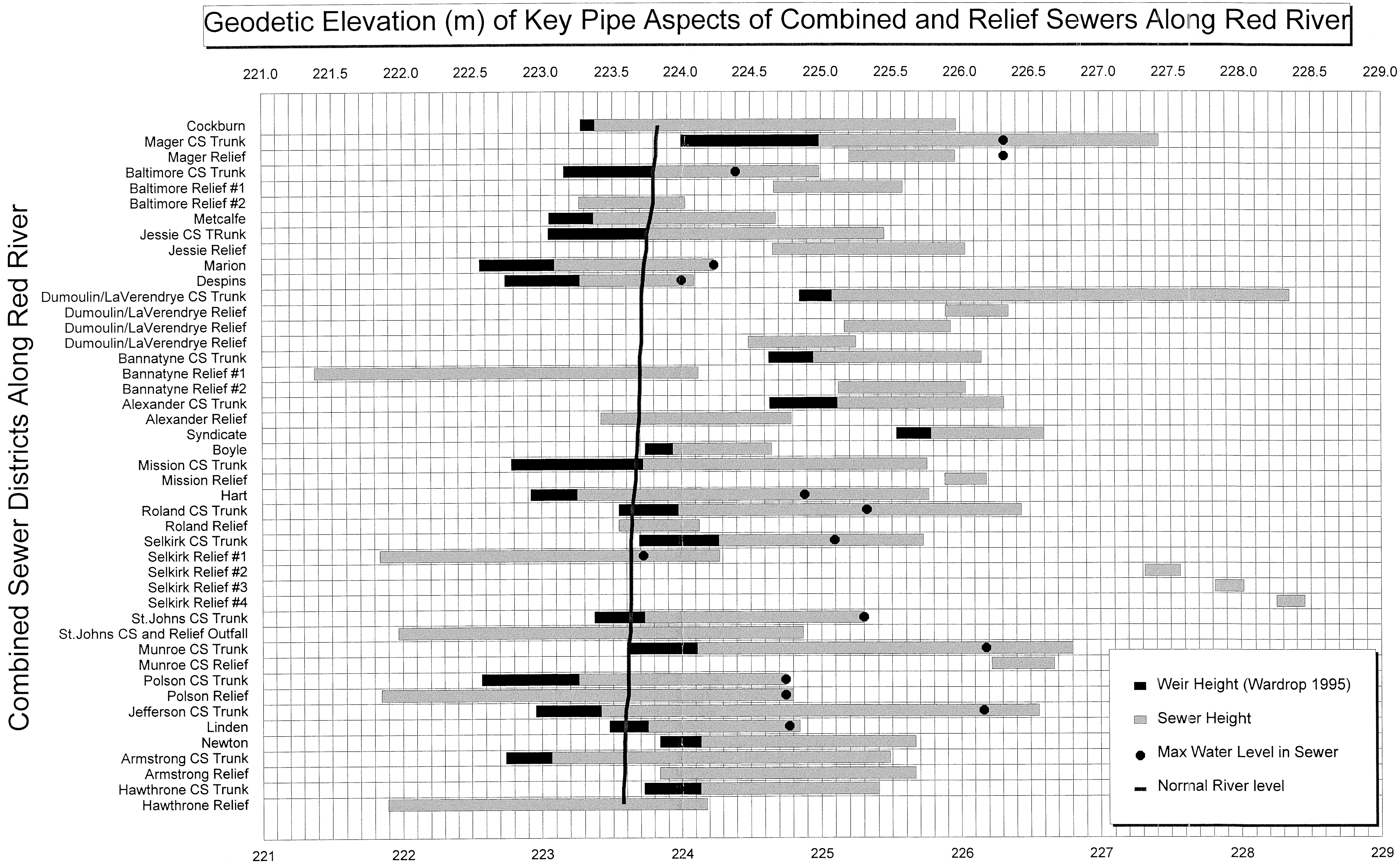
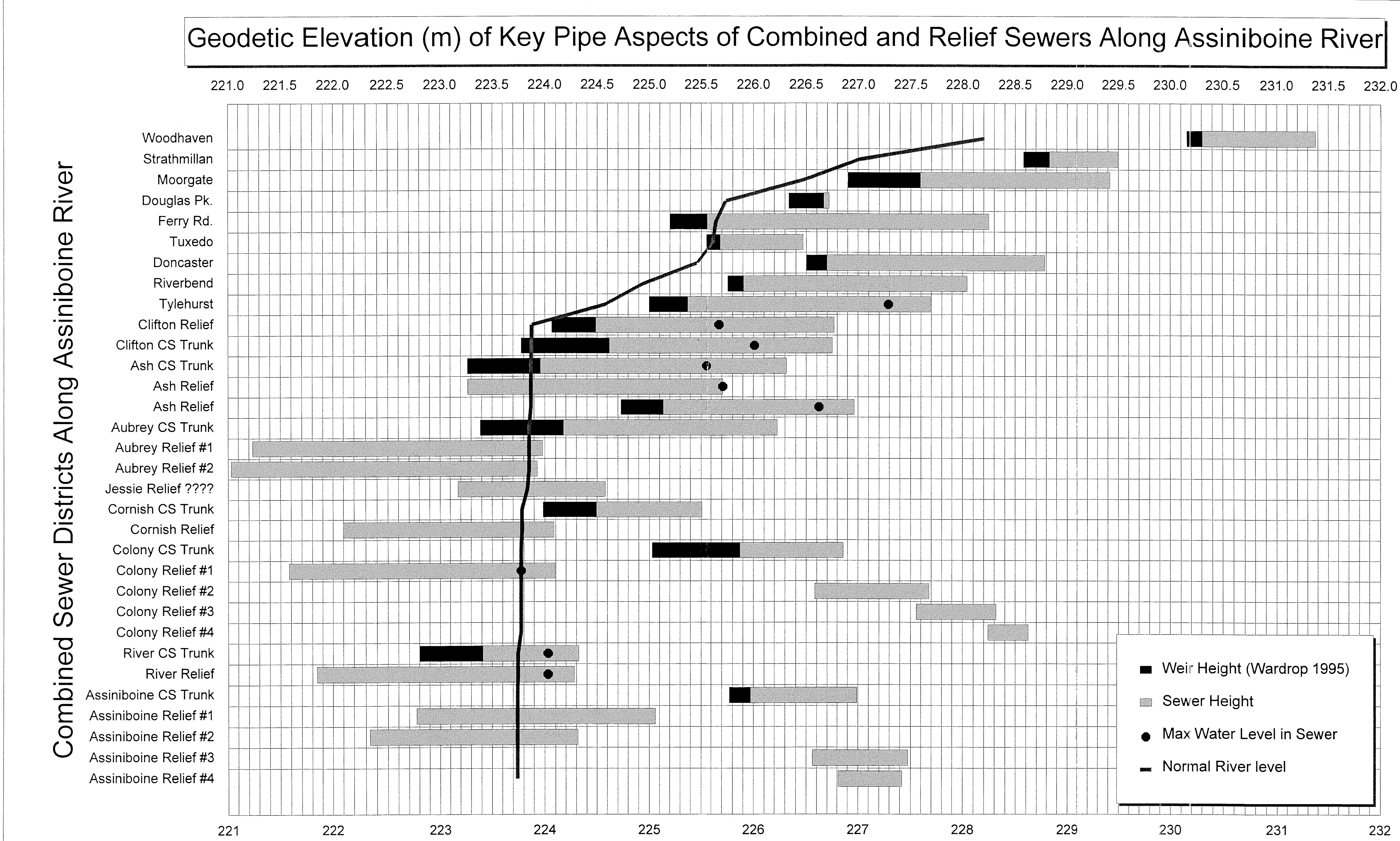
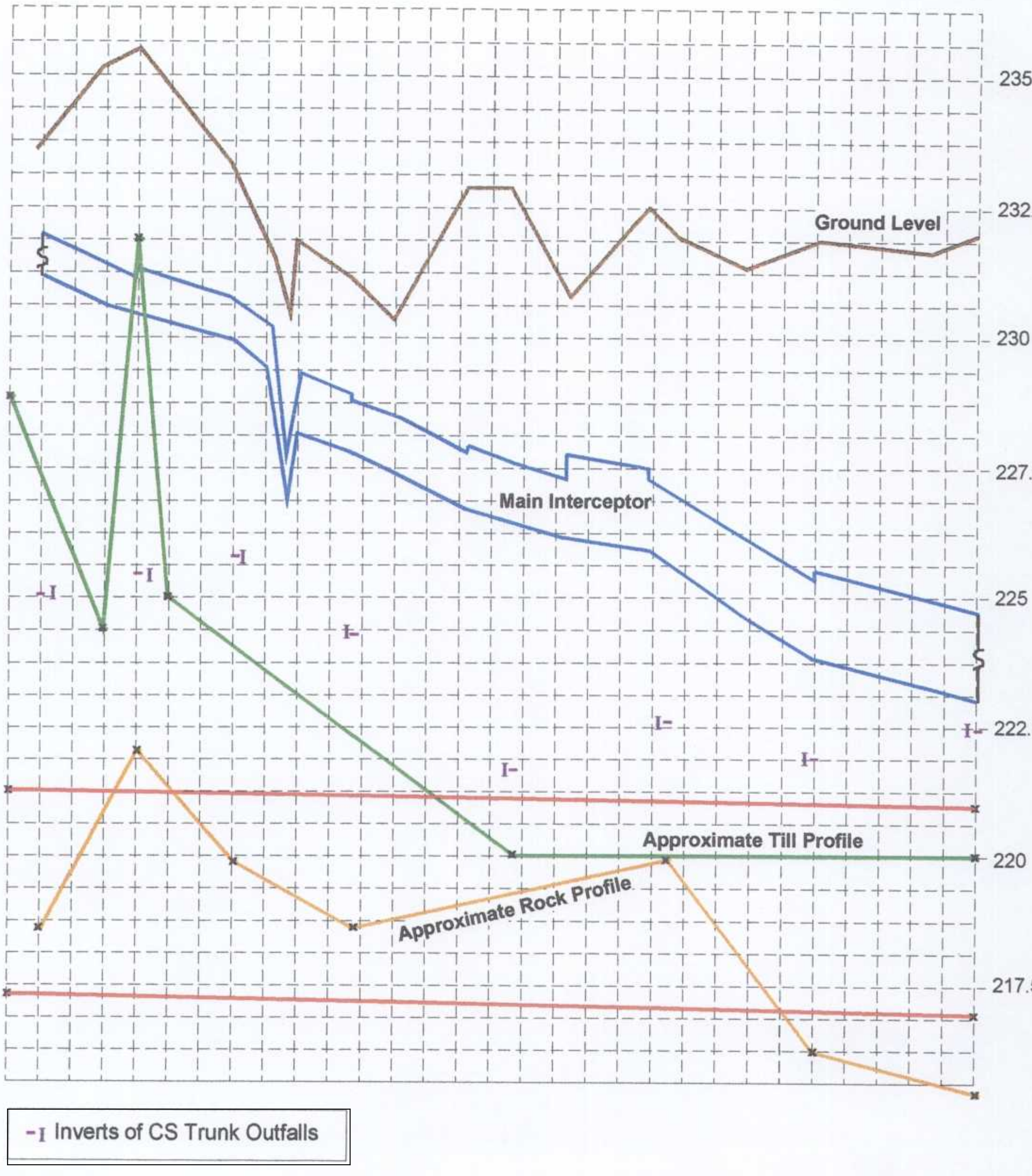

Geodetic Elevation (m) of Key Pipe Aspects of Combined and Relief Sewers Along Red River
221.0
221.5
222.0
222.5
223.0
223.5
224.0
224.5
225.0
225.5
226.0
226.5
227.0
227.5
228.0
228.5
229.0
Combined Sewer Districts Along Red River
Cockburn
Mager CS Trunk
Mager Relief
Baltimore CS Trunk
Baltimore Relief #1
Baltimore Relief #2
Metcalfe
Jessie CS TRunk
Jessie Relief
Marion
Despins
Dumoulin/LaVerendrye CS Trunk
Dumoulin/LaVerendrye Relief
Dumoulin/LaVerendrye Relief
Dumoulin/LaVerendrye Relief
Bannatyne CS Trunk
Bannatyne Relief #1
Bannatyne Relief #2
Alexander CS Trunk
Alexander Relief
Syndicate
Boyle
Mission CS Trunk
Mission Relief
Hart
Roland CS Trunk
Roland Relief
Selkirk CS Trunk
Selkirk Relief #1
Selkirk Relief #2
Selkirk Relief #3
Selkirk Relief #4
St.Johns CS Trunk
St.Johns CS and Relief Outfall
Munroe CS Trunk
Munroe CS Relief
Polson CS Trunk
Polson Relief
Jefferson CS Trunk
Linden
Newton
Armstrong CS Trunk
Armstrong Relief
Hawthrone CS Trunk
Hawthrone Relief
Weir Height (Wardrop 1995)
Sewer Height
Max Water Level in Sewer
Normal River level
221
222
223
224
225
226
227
228
229

Figure 5-15

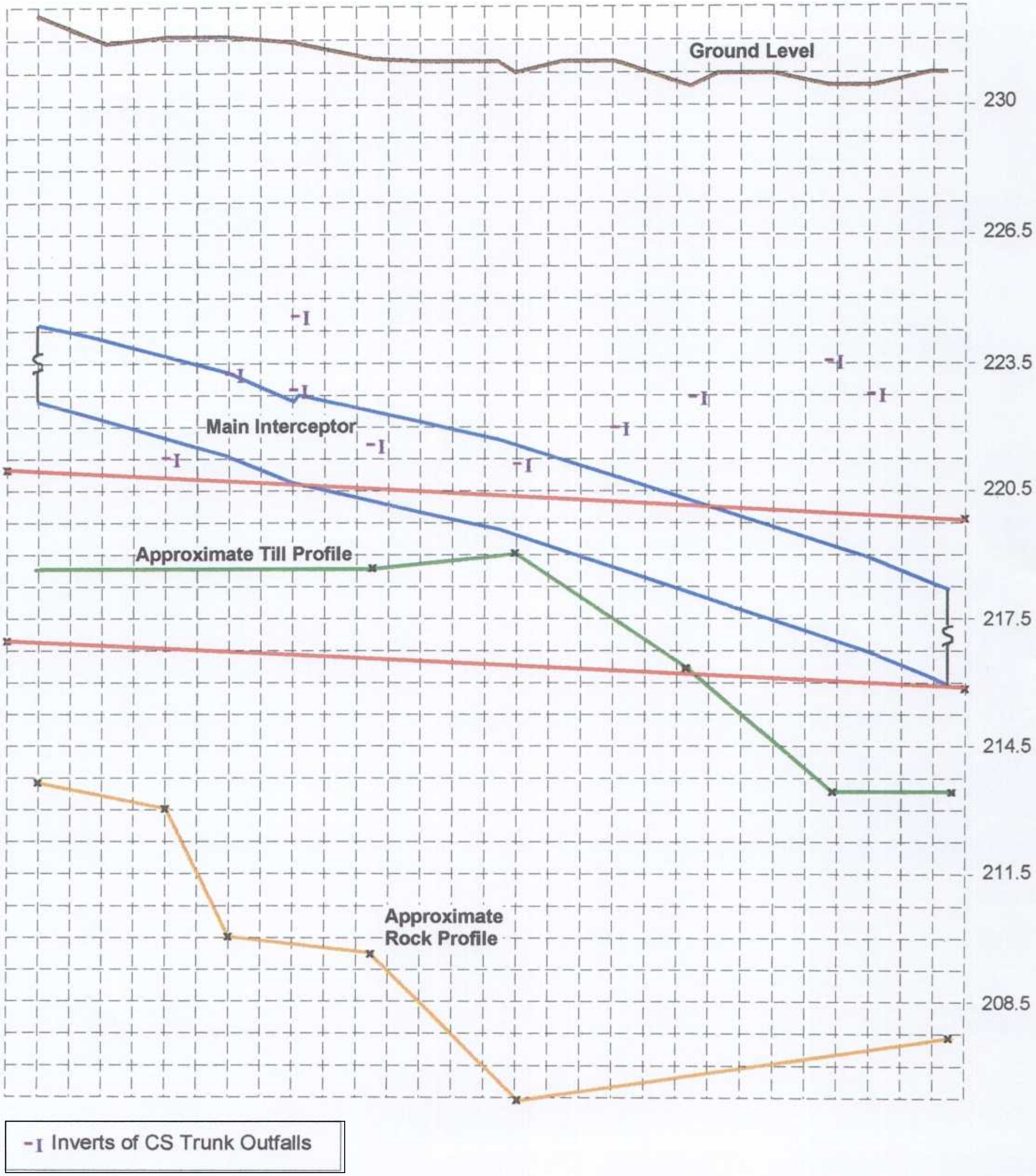


Geodetic Elevation (m) of Key Pipe Aspects of Combined and Relief Sewers Along Assiniboine River
221.0
221.5
222.0
222.5
223.0
223.5
224.0
224.5
225.0
225.5
226.0
226.5
227.0
227.5
228.0
228.5
229.0
229.5
230.0
230.5
231.0
231.5
232.0
Combined Sewer Districts Along Assiniboine River
Woodhaven
Strathmillan
Moorgate
Douglas Pk.
Ferry Rd.
Tuxedo
Doncaster
Riverbend
Tylehurst
Clifton Relief
Clifton CS Trunk
Ash CS Trunk
Ash Relief
Ash Relief
Aubrey CS Trunk
Aubrey Relief #1
Aubrey Relief #2
Jessie Relief ????
Cornish CS Trunk
Cornish Relief
Colony CS Trunk
Colony Relief #1
Colony Relief #2
Colony Relief #3
Colony Relief #4
River CS Trunk
River Relief
Assiniboine CS Trunk
Assiniboine Relief #1
Assiniboine Relief #2
Assiniboine Relief #3
Assiniboine Relief #4
Weir Height (Wardrop 1995)
Sewer Height
Max Water Level in Sewer
Normal River level
221
222
223
224
225
226
227
228
229
230
231
232



Assiniboine River
Approximate Rock / Till Profile
Figure 5-23

sewrprof
s\01\0510



Main Street (Red River)
Approx. Rock \ Till Profile
Figure 5-24

sewrprof2
s\01\0510

The previous analysis assumed that a fixed weir could be integrated such that it does not significantly alter the existing HGL associated with the design storm used for basement flood protection. Specifically, the modelled HGLs at the diversion structures and in the relief pipes (where applicable) were used to calculate the height and length of a weir for a specific depth of flow (i.e., 150 mm) to achieve a given design flow, as shown in Figure 5-12. The conceptual design of the weir is based on bending the length (in the shape of fingers) to achieve the length of weir required to spill the design flow without exceeding the existing HGL. This assumes that the weir will be operating under free-fall conditions, that is, no backwater influence. Under design flow conditions, however, hydraulics may set-up in the outfall pipe such that a backwater is formed and submerges the fixed "finger" weir. For this condition, the hydraulic behaviour of the fixed "finger" weir changes and could result in a number of possible HGLs to form. Since the chamber for this weir control system is significantly larger than the entrance and exit pipes, and employs a specialized weir shape, the resulting hydraulics are complex and may not be accurately described with standard weir equations and correction factors.

The design of the fixed "finger" weir in-line control system can be adapted through the use of a bendable weir to increase the depth of flow over the crest of the weir. Since the length of the weir is strongly influenced by depth of flow over the weir crest, the use of a bendable weir in the weir control chamber could effectively reduce the length of weir needed to achieve the flow capacity required to safely convey the design storm. The length of the weir and size of the control facility could, accordingly, be reduced and result in a lower cost solution.

It is recommended that if a weir-based in-line storage control system is pursued, that a numerical model be developed, and a physical model be constructed, to establish specific design information for this control option.

<u>Summary</u>

Table 5-3 summarizes the storage available for the weir option under existing conditions for each combined sewer district. It is possible to access approximately 290,000 m^3 of in-line storage volume under existing sewer infrastructure utilizing fixed weirs in the combined sewer

Table 5-3: Existing Conditions
Potential Storage Available Utilizing a Fixed Weir

			1	2	1+2 =3	4	12	13	14	15	16=Min(14,15)	17	18
District Name	Relief Status	Tributary Area (Ha)	Height of Sewer	Invert	Obvert	Min. Ground	Max Water Level from Computer Modelling	Estimated Max WL	Estimated Max WL -0.5 ft	10.5 ft below Min Ground	Control Elevation	Depth below min ground	Volume (m^3)
Alexander	no	146	5.50	736.990	742.490	758.500		741.665	741.165	748.000	741.165	17.3	2,576
Armstrong	no	148	9.00	730.770	739.770	748.500		738.420	737.920	738.000	737.920	10.6	4,380
Ash	yes	823	10.00	732.500	742.500	762.139	740.00	740.000	739.500	751.639	739.500	22.6	26,085
Assiniboine	yes	75	4.00	740.720	744.720	752.570		744.120	743.620	742.070	742.070	10.5	6,123
Aubrey	yes	390	9.33	732.890	742.220	760.000		740.821	740.321	749.500	740.321	19.7	44,533
Baltimore	yes	211	6.00	732.159	738.159	753.000	736.22	736.220	735.720	742.500	735.720	17.3	301
Bannatyne	yes	206	5.00	736.970	741.970	759.000		741.220	740.720	748.500	740.720	18.3	11,776
Boyle	no	25	3.00	734.040	737.040	754.500		736.590	736.090	744.000	736.090	18.4	14
Clifton	yes	415	9.75	734.180	743.930	762.000	741.50	741.500	741.000	751.500	741.000	21.0	3,870
Cockburn/Calrossie	no	243	8.83	732.540	741.370	759.750		740.046	739.546	749.250	739.546	20.2	3,953
Colony	yes	227	6.00	738.280	744.280	759.500		743.380	742.880	749.000	742.880	16.6	11,012
Cornish	yes	127	5.00	734.850	739.850	756.000		739.100	738.600	745.500	738.600	17.4	4,478
Despins/Marion	no	317	4.43	730.774	735.203	746.063	734.91	734.908	734.408	735.563	734.408	11.7	2,932
Doncaster	no	133	7.50	743.100	750.600	763.770		749.475	748.975	753.270	748.975	14.8	3,541
Douglas Pk/Ferry Rd	no	251	10.00	738.840	748.840	758.000		747.340	746.840	747.500	746.840	11.2	4,935
Dumoulin/La Verendrye	no	136	11.48	737.690	749.173	754.500		747.450	746.950	744.000	744.000	10.5	1,046
Hart	yes	142	9.33	731.370	740.700	749.250	737.80	737.800	737.300	738.750	737.300	12.0	3,203
Hawthorne	no	219	5.50	734.010	739.510	752.800		738.685	738.185	742.300	738.185	14.6	6,553
Jefferson E & W	yes	977	11.83	731.480	743.310	751.500	742.00	742.000	741.500	741.000	741.000	10.5	19,395
Jessie	yes	338	7.87	731.791	739.665	759.514		738.484	737.984	749.014	737.984	21.5	4,496
Linden	yes	149	4.50	733.180	737.680	748.000	737.43	737.434	736.934	737.500	736.934	11.1	1,020
Mager Drive	yes	260	11.25	734.900	746.150	755.500	742.50	742.500	742.000	745.000	742.000	13.5	4,905
Metcalfe	no	34	5.33	731.810	737.140	757.500		736.341	735.841	747.000	735.841	21.7	512
Mission	no	421	9.75	730.910	740.660	756.000		739.198	738.698	745.500	738.698	17.3	4,623
Moorgate	no	157	8.25	744.430	752.680	765.000		751.443	750.943	754.500	750.943	14.1	2,324
Munroe	yes	375	10.50	733.600	744.099	752.850	742.06	742.060	741.560	742.350	741.560	11.3	37,556
Newton	no	56	6.00	734.380	740.380	750.620		739.480	738.980	740.120	738.980	11.6	1,349
Polson	yes	238	7.14	730.217	737.352	751.706	737.35	737.352	736.852	741.206	736.852	14.9	21,397
River	yes	108	5.00	730.970	735.970	753.800	735.00	735.000	734.500	743.300	734.500	19.3	1,404
Riverbend/Parkside Dr.	no	189	7.50	740.670	748.170	760.000		747.045	746.545	749.500	746.545	13.5	4,487
Roland	yes	178	9.50	733.410	742.910	756.500	739.24	739.240	738.740	746.000	738.740	17.8	18,261
Selkirk	yes	259	6.67	733.900	740.570	750.000	738.50	738.500	738.000	739.500	738.000	12.0	4,833
St. Johns	yes	335	6.33	732.841	739.173	755.249	739.17	739.173	738.673	744.749	738.673	16.6	24,716
Strathmillan	no	69	3.00	749.930	752.930	764.500		752.480	751.980	754.000	751.980	12.5	28
Syndicate	no	79	3.50	739.940	743.440	751.000		742.915	742.415	740.500	740.500	10.5	2
Tuxedo	no	50	3.00	740.000	743.000	759.000		742.550	742.050	748.500	742.050	17.0	168
Tylehurst	no	185	8.83	738.180	747.010	762.500	745.70	745.700	745.200	752.000	745.200	17.3	2,419
Woodhaven	no	42	4.00	755.110	759.110	766.800		758.510	758.010	756.300	756.300	10.5	25
		8,733											295,233

trunks, and were applicable in relief systems. This is about 80,000 m^3 or 20% less than potentially available with a dynamic control gate.

5.2.4.5 Latent Storage

The existing sewer system, by virtue of the weirs and the flap gates in place, already incurs a finite amount of in-line storage before overflowing to the rivers. Most of this storage originates from relief sewers that are partially submerged under normal summer river water levels. This storage is considered "latent" because it exists now without having to install any additional controls. If the storage is not dewatered after WWF, it has not provided any improvement in overflow control. The relief systems were designed to increase flow conveyance to the rivers to protect against basement flooding. As such, the need for dewatering was not necessary.

It was found that many of the relief outfall pipes were set partially below normal river level and designed with a flap-gate to prevent high river levels from entering the combined sewer system. The activation level of the relief system was set, typically through the use of adjustable weirs, to provide hydraulic relief to the main combined sewer trunk. The sizing of relief was based on providing a minimum level of basement flood protection (e.g., 1 in 5 year return frequency storm). An important hydraulic design consideration was the establishment of relief activation levels so that all dry weather flows were conveyed to the wastewater treatment plant. For the purposes of this study, it is assumed that the activation level for the relief sewers coincides with the activation level of the combined sewer trunk overflow weir at the offtake point to the interceptor. These factors were important conditions pertinent to the estimate of in-line storage that was potentially available now.

Overflows will be trapped in these relief pipes and represents latent storage until such time that the contained volume can be actively dewatered. Table 5-4 summarizes the in-line storage available now for existing conditions. Approximately 130,000 m^3 of storage can be accessed now with minor changes to system operation and the addition of dewatering pumps and piping.

Table 5-4: Existing Conditions
Latent Storage Potentially Available Now

			1	2	1+2 =3	4	5	6	7=Max(5,6)	8	9=Min(7,8)	10	11
District Name	Relief Status	Tributary Area (Ha)	Height of Sewer	Invert	Obvert	Min. Ground	Normal River level	Weir Elevation	Existing Control Level	10 ft below Min Ground	Control Elevation	Depth below min ground	Volume (m^3)
Alexander	no	146	5.50	736.990	742.490	758.500	733.906	738.599	738.599	748.500	738.599	19.9	995
Armstrong	no	148	9.00	730.770	739.770	748.500	733.537	731.853	733.537	738.500	733.537	15.0	184
Ash	yes	823	10.00	732.500	742.500	762.139	734.474	734.764	734.764	752.139	734.764	27.4	10,143
Assiniboine	yes	75	4.00	740.720	744.720	752.570	734.058	741.289	741.289	742.570	741.289	11.3	5,522
Aubrey	yes	390	9.33	732.890	742.220	760.000	734.426	735.482	735.482	750.000	735.482	24.5	24,236
Baltimore	yes	211	6.00	732.159	738.159	753.000	734.247	734.259	734.259	743.000	734.259	18.7	81
Bannatyne	yes	206	5.00	736.970	741.970	759.000	733.925	738.009	738.009	749.000	738.009	21.0	17,810
Boyle	no	25	3.00	734.040	737.040	754.500	733.857	734.696	734.696	744.500	734.696	19.8	3
Clifton	yes	415	9.75	734.180	743.930	762.000	734.500	736.936	736.936	752.000	736.936	25.1	677
Cockburn/Calrossie	no	243	8.83	732.540	741.370	759.750	734.365	732.867	734.365	749.750	734.365	25.4	166
Colony	yes	227	6.00	738.280	744.280	759.500	734.173	741.036	741.036	749.500	741.036	18.5	9,484
Cornish	yes	127	5.00	734.850	739.850	756.000	734.204	736.523	736.523	746.000	736.523	19.5	2,833
Despins/Marion	no	317	4.43	730.774	735.203	746.063	734.000	732.513	734.000	736.063	734.000	12.1	2,216
Doncaster	no	133	7.50	743.100	750.600	763.770	739.700	743.756	743.756	753.770	743.756	20.0	8
Douglas Pk/Ferry Rd	no	251	10.00	738.840	748.840	758.000	740.300	740.021	740.300	748.000	740.300	17.7	522
Dumoulin/La Verendrye	no	136	11.48	737.690	749.173	754.500	733.970	738.445	738.445	744.500	738.445	16.1	57
Hart	yes	142	9.33	731.370	740.700	749.250	733.780	732.569	733.780	739.250	733.780	15.5	277
Hawthorne	no	219	5.50	734.010	739.510	752.800	733.500	735.355	735.355	742.800	735.355	17.4	3,193
Jefferson E & W	yes	977	11.83	731.480	743.310	751.500	733.570	732.989	733.570	741.500	733.570	17.9	349
Jessie	yes	338	7.87	731.791	739.665	759.514	734.089	734.121	734.121	749.514	734.121	25.4	1,200
Linden	yes	149	4.50	733.180	737.680	748.000	733.558	734.099	734.099	738.000	734.099	13.9	4
Mager Drive	yes	260	11.25	734.900	746.150	755.500	734.313	738.148	738.148	745.500	738.148	17.4	663
Metcalfe	no	34	5.33	731.810	737.140	757.500	734.180	732.818	734.180	747.500	734.180	23.3	126
Mission	no	421	9.75	730.910	740.660	756.000	733.828	733.994	733.994	746.000	733.994	22.0	301
Moorgate	no	157	8.25	744.430	752.680	765.000	743.000	746.841	746.841	755.000	746.841	18.2	177
Munroe	yes	375	10.50	733.600	744.099	752.850	733.640	735.273	735.273	742.850	735.273	17.6	8,606
Newton	no	56	6.00	734.380	740.380	750.620	733.537	735.364	735.364	740.620	735.364	15.3	24
Polson	yes	238	7.14	730.217	737.352	751.706	733.640	732.480	733.640	741.706	733.640	18.1	16,070
River	yes	108	5.00	730.970	735.970	753.800	734.074	732.971	734.074	743.800	734.074	19.7	1,144
Riverbend/Parkside Dr.	no	189	7.50	740.670	748.170	760.000	738.000	741.162	741.162	750.000	741.162	18.8	93
Roland	yes	178	9.50	733.410	742.910	756.500	733.726	734.800	734.800	746.500	734.800	21.7	6,582
Selkirk	yes	259	6.67	733.900	740.570	750.000	733.703	735.803	735.803	740.000	735.803	14.2	2,579
St. Johns	yes	335	6.33	732.841	739.173	755.249	733.688	734.022	734.022	745.249	734.022	21.2	17,949
Strathmillan	no	69	3.00	749.930	752.930	764.500	744.800	750.889	750.889	754.500	750.889	13.6	0
Syndicate	no	79	3.50	739.940	743.440	751.000	733.865	740.760	740.760	741.000	740.760	10.2	14
Tuxedo	no	50	3.00	740.000	743.000	759.000	740.200	740.200	740.200	749.000	740.200	18.8	61
Tylehurst	no	185	8.83	738.180	747.010	762.500	736.800	739.410	739.410	752.500	739.410	23.1	20
Woodhaven	no	42	4.00	755.110	759.110	766.800	748.700	755.558	755.558	756.800	755.558	11.2	2
		8,733											134,371

5.2.5 Operational Considerations

The utilization of available in-line storage was identified as having the potential to impact the existing operation of the sewer system. The Clifton and Hart Pilot projects sere established to address several unknowns:

- confirm predicted levels (XP-SWMM modelling), response times, and hydraulic parameters;
- confirm the reliability of control systems;
- determine optimum level for in-line storage without affecting existing level of basement flood protection;
- test sediment build-up, quality, and flushing requirements;
- monitor fecal coliform concentration in stored combined sewage to assess die-off;
- monitor ammonia concentrations in stored combined sewage to determine nitrogen dynamics; and
- establish operator comfort with automated gate operations.

The City of Winnipeg Management Committee raised concerns related to basement flooding due to failure of the automated gate in the closed position. They requested that other alternatives, which are inherently fail-safe, be developed to safely access available in-line storage. As previously discussed, this could involve a weir control system. Unfortunately, the estimated cost to install a pilot fixed "finger" weir system was significantly greater than the proposed automated gate system and would require a new and permanent structure. Upon review of the costs and implementation requirements, the City decided to discontinue the proposed pilot project in Hart CS district and document the knowledge gained from the process (refer to Clifton CS XP-SWMM modelling and Hart CS District XP-SWMM modelling, and functional design technical memorandum appendices).

The intent of the pilot was to gain operating experience with an automated gate and the maximization of in-line storage without affecting the existing level of basement flood protection. It is not possible to gain this information and experience without field testing a proposed control system such as an automated gate, inflatable dam, bending weir, or fixed "finger" weir.

5.2.5.1 Odours/Sedimentation/Water Quality

Since the pilot project did not fully proceed, monitoring of the existing system is being considered, where conditions may already be simulating in-line storage, to gain valuable local data and knowledge on potential problems or issues associated with:

- sedimentation build-up and flushing/cleaning requirements;
- water quality changes; and
- odour/H_2S generation.

Many of the relieved systems have separate conveyance systems and outfalls that run parallel to the main CS trunk to the rivers. Most of these separate relief pipes were constructed at constant grade and are fully or partially submerged at its outlet to the rivers. The outlets on both the CS trunks and relief pipes contain flap gates and a positive gate to prevent river water from entering the system. This condition can effectively cause the river to hold back some overflow volume in the relief systems, as previously shown in Figure 5-6 (latent in-line storage). Since a specific volume of combined sewage will remain in the relief pipe, until it is flushed by the next rainfall event, it already sets up the conditions needed to assess the above noted potential problems/issues (i.e., sediment, WQ changes, odour/H_2S).

Efforts are currently underway to identify suitable locations and initiate a monitoring program to gather specific data on the possible impacts of in-line storage for the noted potential problems/issues. As well, consideration should be given to include dewatering of the relief system selected for this monitoring program. Dewatering is required to simulate activation of in-line storage in relief pipes (latent in-line storage), allow access for sediment sampling, and evaluate the water-tight seal of the flap gate.

5.2.6 <u>Inlet Restrictions</u>

Typically, a 1-in-5 year synthetic design storm is used to access the improvements needed to the existing sewer system to convey the wet weather flows while reducing the resulting hydraulic gradeline to a level below basement flood threatening levels. Additionally, previous hydraulic modelling has found the inlet restriction has the ability to improve the level of basement flood protection from a 1-in-5 year to about a 1-in-10 year return frequency storm. In essence, inlet restriction constricts the rate of runoff into the sewer system and temporarily utilizes street storage to limit flows in the sewer system to achieve the equivalent system flows as a 1-in-5 year rainfall (without inlet restriction). Inlet restriction provides several important advantages to sewer relief projects and should be considered as a fundamental aspect of all future relief projects because of its cost effectiveness and benefit too in-line storage control options.

5.2.7 Integration with Basement Flooding Relief Projects

The previous in-line storage analysis found that most of the available storage volume exists in combined sewer districts that have been relieved for purposes of improving basement flood protection. Clearly, the addition of new relief pipes can provide improved basement flood protection and reduce the number and volume of CSOs.

To fully account for the possible benefit of in-line storage, it is necessary to estimate the amount of storage that could result from future relief projects. To accomplish this, combined sewer districts were partitioned into districts with and without relief and the receiving river. The volume of storage estimated for each district was divided by its tributary area to generate an equivalent storage depth per unit area. This was done for each condition (i.e., Latent Storage; Fixed Weir Option; Automated Gate) to define:

- the low range is equivalent to existing conditions;
- expected case which is the average of all systems with relief in place for the river system it overflows to; and
- high range is based on enlarging the expected case by 20% and is equivalent to a reasonable oversizing of relief pipes.

Table 5-5 summarizes the range of equivalent depths of storage (mm) per unit of tributary area for CS districts grouped by river system and relief status for the three cases outlined above. Applying these equivalent depths to each of the unrelieved CS districts, it is possible to extrapolate the possible increase in available in-line storage for CSO control. Table 5-6 summarizes the possible increase in in-line storage that could result from future BFR projects. Figure 5-16 graphically depicts the results to help illustrate the range of in-line storage volume for each case considered. The results indicate that:

- for the expected case, the increase in in-line storage volume could range between 68,000 and 84,000 m^3; and
- for the optimistic case, the increase in in-line storage volume could range between 93,000 m^3 and 125,000 m^3.

New relief projects represent a very important future opportunity with respect to supplemental in-line storage volumes that could significantly reduce the need for more expensive and complicated control technologies. The oversizing of the relief pipes and inclusion of dewatering capabilities warrants further study. Recognizing the need for CSO control provides the opportunity to expand the relief design criteria to maximize in-line storage as a design goal of the BFR projects. The cost associated with implementing outflow control or automated gate must be considered when determining if there is an advantage to oversizing relief pipes to achieve the same volume of in-line storage. Dewatering will be a common aspect. Operational considerations are an important consideration and will need to be reviewed carefully to evaluate the benefits and drawbacks of relief oversizing.

5.2.8 <u>Cost</u>

5.2.8.1 Fixed Weir Cost

The fail-safe (finger-weir) option was costed for the Hart Pilot Project. The results are given in Table 5-7 and are based on a preliminary take-off of quantities for such a structure. As indicated on the plan shown on Figure 5-13, the structure could be 13 m (±) wide and could be

Table 5-5
Equivalent Depth of Storage (mm)
per Unit of Tributary Area

• ***Relieved CS Distrists Along Assiniboine River***

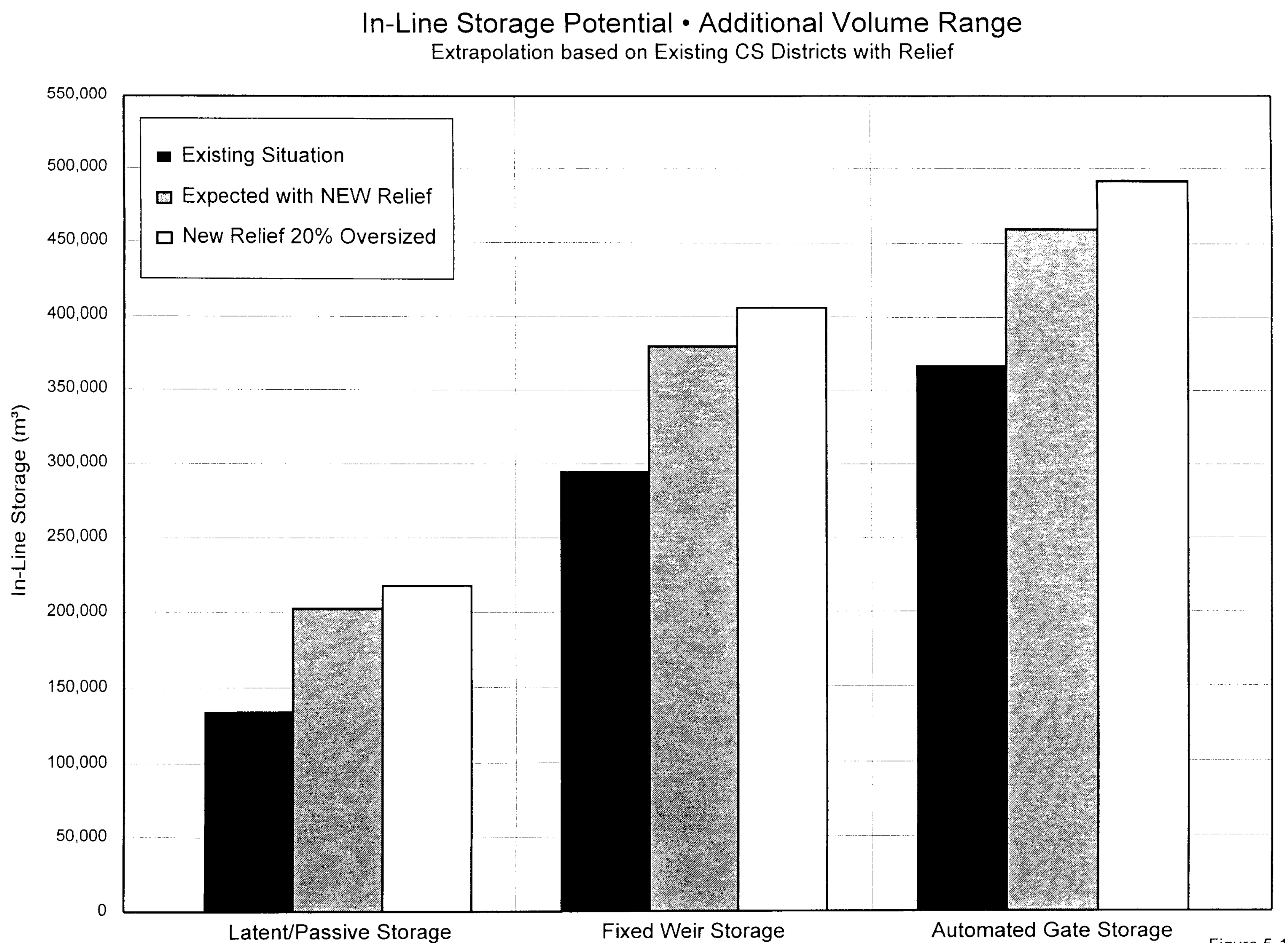
	Existing Conditions (low range)	**Expected case (average)**	Expected +20% (high range)
Latent / Passive	0.2	**3.2**	3.8
Fixed Weir	0.9	**4.8**	5.7
Automated Gate	1.6	**5.7**	6.8

• ***Relieved CS Distrists Along Red River***

	Existing Conditions (low range)	**Expected case (average)**	Expected +20% (high range)
Latent / Passive	0.0	**2.4**	2.9
Fixed Weir	0.1	**4.4**	5.3
Automated Gate	0.5	**5.4**	6.5

Table 5-6
Possible Area-Wide In-Line Storage Volumes (m^3)
Resulting from Future BFR Projects

	Existing Conditions (low range)	**Expected case (average)**	Expected +20% (high range)
Latent / Passive	134,371	**202,665**	217,956
Fixed Weir	295,233	**379,725**	405,797
Automated Gate	367,012	**459,837**	491,841


In-Line Storage Potential • Additional Volume Range
Extrapolation based on Existing CS Districts with Relief
Existing Situation
Expected with NEW Relief
New Relief 20% Oversized
In-Line Storage (m³)
550,000
500,000
450,000
400,000
350,000
300,000
250,000
200,000
150,000
100,000
50,000
0
Latent/Passive Storage
Fixed Weir Storage
Automated Gate Storage
Figure 5-16

installed in any road allowance. Since most of the combined sewer trunk outfalls and their associated relief pipes are installed in road allowances, this would not entail disruption of private property.

It is conceptualized that the wall of the distribution channel upstream of the finger-weirs would be provided with two automated gates (likely motor-driven) with their opening being initiated by the overtopping of the weir, i.e., a level sensor would open the gates when combined sewage overtopped the weir. Automatic opening in response to an overflow would permit flushing of the sewers. The installation would not require controls with the level of sophistication or redundancy needed for the virtually fail-safe option since failure would not affect basement flood protection and simply result in no flushing for a specific event.

The estimated cost of the weir option at Hart, including a 20% estimating allowance and a 20% allowance for engineering, administration and finance is in the order of $1.4 million.

Table 5-7 shows estimates of the cost for units larger than the Hart district. **Table 5-8** comprises a listing of all of the outlets to the Red and Assiniboine rivers that currently exist. The listing shows the diameter of these outlets and also indicates whether or not in-line storage would be appropriate. In assessing the outlets, the assumption was made that anything 900 mm or less would not justify the expense of the weir structure (too small to have significant volumes) and that such overflows would be modified or directed to off-line storage in order to correct the CSO. The result is that the whole combined sewer area (including those districts tributary to the SEWPCC and WEWPCC) would require 45 weir boxes to develop all of the current potential in-line storage. This would leave 27 outlets which would need to be modified. In order to arrive at a preliminary sizing of the weir boxes, each of the districts which had been designated for in-line storage were compared to the Hart district on the basis of the area drained. The premise was that the runoff would be closely proportional to that area. The results of this analysis are shown on **Table 5-9**. As noted above, the costs were broken down into single units, 1.5 x single units, 2 x single units, and 3 x single units, with the projected costs having been given. For the outlets without in-line storage, an allowance was made for modifying each of these outlets, in some as yet undesignated manner, at $350,000 base cost each or $530,000 with allowances.

Table 5-7:

INLINE STORAGE
BASE COST ESTIMATE
HART SURROGATE

(single unit costs)

EXCAVATION/BACKFILL	$ 45K
SHORING	$265
REINFORCED CONCRETE	$360
Sub Total	$670
MISC. (including gates)- 25%	$170
	$840
OH & PROFIT - 15%	$130
TOTAL CONCEPTUAL DESIGN COST	$970K
+ ALLOWANCES (20% est'g + 20%EAF)	$1.4M

OVERSIZED UNITS

(six tenths rule)

1.5*SINGLE UNIT	$1.8M
2*SINGLE UNIT	$2.1M
3*SINGLE UNIT	$2.7M

ALLOWANCE - OUTLETS W/O INLINE STORAGE

$300K*1.44 = $530K

Table 5-8:

DISTRICT	#	LOCATION ASSINIBOINE RIVER	DIAM. mm	IN-LINE YES	IN-LINE NO	WEIR STRUC	MODIFY
WOODHAVEN	29	WOODHAVEN BLVD	450		✓		✓
	33	ASSINIBOINE CRES. E	760		✓		✓
STRATHMILLAN	37	STRATHMILLAN EXT.	900		✓		✓
	37A	STRATHMILLAN COMM CHAMBER					
	37B	STRATHMILLAN CFB	300		✓		✓
MOORGATE	42	CONWAY STREET	2500*1900	✓		✓	
	53	DEER LODGE PL.	300		✓		✓
	57	DOUGLAS PARK	300		✓		✓
FERRY ROAD	59	FERRY ROAD	1800	✓		✓	
TUXEDO	60	CHATTAWAY	900		✓		✓
DONCASTER	61	DONCASTER ST.	2250	✓		✓	
	62	PARKSIDE @ ASSINIBOINE	685		✓		✓
RIVERBEND	63	S. OF RIVERBEND CRES.	2266		✓		✓
TYLEHURST	69	TYLEHURST @ WOLESLEY	2250	✓		✓	
CLIFTON	74	CLIFTON	2300*2900	✓		✓	
		STRATHCONA & OMAND'S CRK.	2700	✓		✓	
	75	CLIFTON					FPS*
ASH	76	ASH					FPS*
ASH	77	ASH	3048*2900	2✓		✓	
	78	WELLINGTON CRES @ MONTROSE	762		✓		✓
AUBREY	79	SHERBURN @ PALMERSTON	2900	✓		✓	
	80	AUBREY @ PALMERSTON	2100*2850	✓		✓	
	82	RUBY @ PALMERSTON	2700	✓		✓	
	81	AUBREY					FPS*
	83	ARLINGTON @ PALMERSTON	380		✓		✓
	84	ARLINGTON @ PALMERSTON	350		✓		✓
CORNISH	85	CANUBA	1980	✓		✓	
	86A	CORNISH @ MARYLAND					FPS*
	86B	CORNISH BTN MARYLAND BRIDGES	600		✓		✓
JESSIE	87	WELLINGTON CRES.@ GROSVENOR	1400	✓		✓	

Table 5-8:

DISTRICT	#	LOCATION ASSINIBOINE RIVER	DIAM. mm	IN-LINE		WEIR STRUC	MODIFY
				YES	NO		
CORNISH	88	88 - CORNISH E OF LANGSIDE	1500	✓		✓	
	89	SPENCE S. OF BALMORAL	2500	✓		✓	
COLONY	90	COLONY S. OF MOSTYN	1800	✓		✓	
COLONY	91	KENNEDY S. OF ASSINIBOINE	760		✓		✓
RIVER	92	FORT ROUGE PARK	2400	✓		✓	
	93	WEST OF MIDTOWN	700		✓		✓
	94	EAST OF MIDTOWN	1900	✓		✓	
ASSINIBOINE	95	WEST MAIN ST. BRANCH	1200	✓		✓	
	96	@ MAIN ST. BRIDGE					FPS*
	98	E. OF CN BRIDGE	1500*1016	✓		✓	
CALROSSIE	37	CALROSSIE	450		✓		✓
COCKBURN	38	COCKBURN					FPS*
	39	COCKBURN @ CHURCHILL	2200	✓		✓	
		ST. VITAL BRIDGE	750		✓		✓
MAGER	44	MAGER	2250*3400	✓		✓	
BALTIMORE	45	BALTIMORE	1500	✓		✓	
METCALFE	46	METCALFE PL.	2200	✓		✓	
BALTIMORE	47	CHURCHILL @ ECCLES	760		✓		✓
JESSIE	49	JESSIE E. OF OSBORNE	1879*2489	✓		✓	
	50	JESSIE E. OF OSBORNE					FPS*
MARION	51	MARION					FPS*
	52	MARION	1830	✓		✓	
DESPINS	54	DESPINS	1800	✓		✓	
	55	DESPINS					FPS*
	56	WATER AVE.	457		✓		✓
DUMOULIN	57	DUMOULIN	1200	✓		✓	
	58	DUMOULIN	600		✓		✓
LAVERENDRYE	59	LAVERENDRYE	1200	✓		✓	
	60	LAVERENDRYE	600		✓		✓
	61	LOMBARD	900		✓		✓
BANNATYNE	62	McDERMOT	2700	✓		✓	

Table 5-8:

DISTRICT	#	LOCATION ASSINIBOINE RIVER	DIAM. mm	IN-LINE		WEIR STRUC	MODIFY
				YES	NO		
	63	BANNATYNE					FPS*
ALEXANDER	64	GALT	1500	✓		✓	
ROLAND	70	WATT	3700	✓		✓	
SYNDICATE	65	BOYLE	1060				W/ SYND.
	66	BOYLE	900		✓		✓
	71	SYNDICATE	1060	✓		✓	
	72	SYNDICATE					FPS*
SELKIRK	74	SELKIRK	1800	✓		✓	
	75	PRITCHARD	250		✓		✓
	76	BURROWS	2400	✓		✓	
	77	ALFRED	200		✓		✓
SELKIRK	78	ABERDEEN	200		✓		✓
MISSION	67	SEINE R.	2400	✓		✓	
HART	79	HART	2850*2160	✓		✓	
ST. JOHN'S	80	ST.JOHN'S	2900	✓		✓	
POLSON	83	POLSON	2200*1778	✓		✓	
MUNROE	84	MUNROE	2500	✓		✓	
POLSON	85	INKSTER	2900	✓		✓	
MUNROE	86	KILDONAN	2275	✓		✓	
JEFFERSON	88	JEFFERSON	3300	✓		✓	
LINDEN	91	LINDEN	1676				FPS*
	93	ROSSMERE	2900		✓		✓
NEWTON	94	NEWTON	1800	✓		✓	
ARMSTRONG	95	ARMSTRONG	2700	✓		✓	
HAWTHORNE	98	HAWTHORNE	1375	✓		✓	
TOTAL UNITS					27	45	

FPS denotes: Flood Pumping Station discharge pipe

Table 5-9:

DISTRICT	DIST #	IN-LINE YES	IN-LINE NO	AREA ha	SIZE RELATIVE TO HART	DISTRIBUTION 1X	1.5X	2X	2.5X	3X
NEWPCC										
FERRY ROAD	17	✓		226	1.5X		1			
DONCASTER	14	✓		133	1X	1				
TUXEDO	40		✓							
RIVERBEND	34		✓							
TYLEHURST	41	✓		185	1X	1				
CLIFTON	9	✓		415	2(1.5X)		2			
ASH	3	✓		823	2(3X)					2
AUBREY	5	✓		390	3(1X)	3				
CORNISH	12	✓		127	3(1X)	3				
COLONY	11	✓		227	1.5X		1			
RIVER	33	✓		108	2(1X)	2				
ASSINIBOINE	4	✓		75	1X	1				
JESSIE	21	✓		338	2(1.5X)		2			
MARION/DESPINS	25/13	✓		317	2(1X)	2				
LAVEREN./DUMOULIN	22/16	✓		136	2(1X)	2				
BANNATYNE	7	✓		206	1.5X		1			
ALEXANDER	1	✓		146	1X	1				
MISSION/ROLAND	27/35	✓		599	2(2X)			2		
SYNDICATE/BOYLE	39/8	✓		104	1X	1				
SELKIRK	36	✓		259	2(1X)	2				
HART	18	✓		142	1X	1				

Table 5-9:

DISTRICT	DIST	IN-LINE		AREA	SIZE	DISTRIBUTION				
	#	YES	NO	ha	RELATIVE TO HART	1X	1.5X	2X	2.5X	3X
ST. JOHN'S	38	✓		335	2.5X				1	
POLSON	32	✓		238	2(1X)	2				
MUNROE	29	✓		375	2(1.5X)		2			
JEFFERSON/NEWTON	20/30	✓		410	2(1.5X)		2			
LINDEN	23		✓							
ARMSTRONG	2	✓		148	1X	1				
HAWTHORNE	19	✓		219	1.5X		1			
WEWPCC										
WOODHAVEN	42		✓							
STRATHMILLAN	37		✓							
MOORGATE	28	✓		158	1X	1				
SEWPCC										
CALROSSIE	10A		✓							
COCKBURN	10	✓		347	2.5X				1	
MAGER/METCALFE	24/26	✓		294	2(1X)	2				
BALTIMORE	6	✓		211	1.5X		1			
TOTALS						**26**	**13**	**2**	**2**	**2**

The final estimated cost of the in-line storage option with weir boxes being installed for all the currently developable in-line storage is shown on Table 5-10. This amounts to a total estimated cost of $100 million, including the dry weather flow allowance discussed elsewhere.

5.2.8.2 Automatic Gate Costs

The costs developed for take in-line storage with automatic gates, for both the Clifton and Hart Pilots, was $350,000/unit. As noted above, this was the same cost carried for modifying outlets without in-line storage. Accordingly, the total cost for this option was:

75 units at 350K x 1.44	=	$38 million
+ modification of DWF diversions		
40 units at 200K x 1.44	=	$12 million
TOTAL	=	$50 million

5.2.8.3 Latent Storage Cost

There are some 15 combined sewer trunks with potential to develop "latent storage". At an allowance similar to the modifications of DWF diversions, the estimated cost would be:

15 x 200K x 1.44	=	$5 million

5.2.9 Review In-line Storage Assessment

Three different control methods for in-line storage were assessed. The costs are:

- $100 million for weir option;
- $50 million for the gate option; and
- $5 million for the latent storage option.

Table 5-10:

INLINE STORAGE OPTION - WITH WEIRS
TOTAL ESTIMATED COSTS

26 UNITS "= HART"	=	26*$1.4M	= $36.2M
13 UNITS "1.5*HART"	=	13*$1.8M	= $23.0M
2 UNITS "2*HART"	=	2*$2.1M	= $4.2M
2 UNITS "2.5*HART"	=	2*$2.4M	= $4.8M
2 UNITS "3*HART"	=	2*$2.7M	= $5.4M
27 UNITS "NO INLINE"	=	27*0.53M	= $14.3M
	+ DWF ALLOWANCE		= $12.0M
TOTAL ESTIMATED COST			**= $100M**

The automated gate control appears to be the most effective in terms of volume of in-line storage which can be accessed. About 370,000 m^3 is probably available using this method. Early on in the assessment there were concerns about the practicality of using real-time controls in operating gates for in-line storage. In January of 1997, a Working Session involving the consultant study team and outside experts from the U.S. and Europe, was conducted, along with the City of Winnipeg Operations and Engineering Departments. Many concerns on the operating levels in the sewers, the fail-safeness of gate control, and other operating conditions such as sedimentation and odours were addressed at this Working Session. The direction from this workshop indicated that experience elsewhere has indicated that this method can be developed, with appropriate engineering design and operating procedures, to alleviate any potential problems.

Subsequently, extensive modelling of the in-line storage system in all districts was done assuming a modulating gate control. These results were used to develop the number and volume of overflows at each district, which in turn, could be used to estimate the percent compliance for the in-line storage option. Three different dewatering rates were used to reflect various potential treatment strategies at the NEWPCC. Other variations on this gate control strategy, such as releasing the in-line storage completely after an overflow rather than modulating the gate, were considered. These would result in an increased volume of overflow, however, would not change the number of overflows at each district.

In addition, work began on developing pilot projects to demonstrate in-line storage control on a district in the City. Several districts were screened, a district was selected, and conceptual design began on piloting an in-line storage project. The plan involved adding an operator to an existing gate in the Hart district. A presentation of the conceptual plan was made to the Senior Management Committee at the Water and Waste Department, direction was given that a gate control system could not be considered completely fail-safe, and other alternatives should be investigated.

One of the alternatives which was investigated was a fixed weir design, discussed earlier. This fixed weir design provides about 20% less volume of storage, 290,000 m^3, than the automated gate control method. This estimate was made early in 1998, and has not undergone the extensive modelling assessment provided for the automated gate control. It is the best professional judgement of the modellers that the number of overflows would increase by one overflow on average, across all districts (i.e., from 5 to 6). The cost of this option is considerably higher at $100 million. This cost has been carried into the evaluation of plans in the next section.

A much more conservative approach of dewatering the existing storage in the relief sewer pipes (latent storage) was also considered. The method proposes virtually no risk and would have a cost of only $5 million. This method was not modelled, although a similar total volume was used in the Phase 2 analysis, and resulted in an average of 7 overflows per district.

The Phase 3 Workshop will give direction as to which alternatives are most promising. Modelling in Phase 4 may therefore entail regional modelling of the most promising alternative or more specific detailed modelling at a single district level.

5.3 OFF-LINE STORAGE

Storage of combined sewage can be provided in off-line facilities, either instead of or supplementary to in-line storage. This section will review off-line storage in terms of near-surface tanks or local tunnels.

5.3.1 Conceptual Off-line Storage Systems

The volumes of storage needed for the representative 1992 scenario, to conform to the performance targets of 0 and 4 overflows per year, are listed on **Tables 5-11, 5-12 and 5-13**. These are all entitled "Summary of Storage Required at Each District" and sub-titled "NE System 600 ML/d at NEWPCC", NE System 825 ML/d at NEWPCC" and "NE System 1,060 ML/d at NEWPCC", respectively. Each of these Tables shows the storage required for the 4 and 0 overflow scenarios for the options both with and without in-line storage.

Table 5-11: Summary of Storage Required at Each District
NE System 600 ML/d at NEWPCC

District Number	District	Combine With	DWF (m³/s)	Existing Rate (m³/s)	Runoff Based m³/s)	Dewatering Rate (m³/s)	Storage for 4 Overflows	Storage for 0 Overflows	Inline Storage	Storage for 4 Overflows with Inline	Storage for 0 Overflows with Inline
1	Alexander		0.035	0.155	0.154	0.119	10,465	23,375	3,803	6,662	19,572
2	Armstrong		0.020	0.524	0.141	0.111	7,395	16,500	10,060	0	16,065
3	Ash		0.082	0.301	0.579	0.497	30,698	89,375	40,418	0	48,957
4	Assiniboine		0.084	0.425	0.172	0.088	9,767	16,500	8,421	1,346	8,079
5	Aubrey		0.071	0.214	0.232	0.161	9,488	30,250	50,708	0	0
6	Baltimore		0.028	0.201	0.153	0.041	10,000	30,000	1,553	8,447	28,447
7	Bannatyne		0.153	0.613	0.269	0.116	7,674	23,375	2,378	5,297	20,997
8	Boyle	Syndicate	0.014	0.030			0	0			
9	Clifton		0.077	0.236	0.277	0.200	19,535	35,750	27,059	0	8,691
10	Cockburn	Cockburn	0.033	0.075	0.084	0.050	11,000	31,000	516	10,484	30,484
10a	Calrossie		0.001	0.028	0.000						
11	Colony		0.134	0.425	0.271	0.137	9,488	28,875	12,638	0	16,237
12	Cornish		0.035	0.107	0.078	0.043	2,512	7,700	5,596	0	2,104
13	Despins	Marion	0.032	0.132	0.032	0.000	0	0			
14	Doncaster		0.025	0.075	0.098	0.073	1,953	6,875	5,616	0	1,259
15	Douglas Park	Ferry Road	0.001	0.095			0	0			
16	Dumoulin		0.013	0.136	0.110	0.088	5,233	17,875	630	4,603	17,245
17	Ferry Road		0.059	0.126	0.211	0.151	9,070	24,750	4,676	4,394	20,074
18	Hart		0.039	0.101	0.145	0.106	8,651	22,000	13,393	0	8,607
19	Hawthorne		0.036	0.113	0.159	0.123	8,372	25,438	3,875	4,497	21,562
20	Jefferson E		0.143	0.569	0.456	0.313	16,744	57,750	15,484	1,260	42,266
20a	Jefferson W	Jefferson E	0.000	0.000			0	0		0	0
21	Jessie		0.066	0.176	0.283	0.217	16,047	42,625	6,662	9,385	35,963
22	La Verendrye	Dumoulin	0.009	0.015			0	0		0	0
23	Linden		0.017	0.060	0.035	0.018	1,074	3,850	777	298	3,073
24	Mager Drive		0.091	0.309	0.309	0.050	11,500	34,000	7,531	3,969	26,469
25	Marion		0.032	0.220	0.221	0.189	15,349	41,250	4,080	11,269	37,170
26	Metcalfe		0.005	0.044	0.015	0.010	3,000	7,000	1,007	1,993	5,993
27	Mission		0.144	0.518	0.323	0.179	10,744	33,000	7,621	3,123	25,379
28	Moorgate		0.023	0.085	0.104	0.081	2,900	11,000	3,771	0	7,229
29	Munroe		0.077	0.237	0.318	0.241	18,140	61,875	38,360	0	23,515
30	Newton	Armstrong	0.010	0.166							
32	Polson		0.032	0.356	0.184	0.152	11,163	28,875	23,401	0	5,474
33	River		0.070	0.094	0.143	0.073	5,581	16,500	4,620	961	11,880
34	Riverbend	Parkside Dr.	0.053	0.107	0.176	0.123	9,070	24,750	293	8,777	24,457
35	Roland		0.026	0.324	0.173	0.147	12,837	27,500	22,455	0	5,045
36	Selkirk		0.067	0.453	0.182	0.115	6,977	22,000	10,254	0	11,746
37	St. Johns		0.084	0.173	0.314	0.230	17,442	44,000	24,895	0	19,105
38	Strathmillan		0.003	0.062	0.031	0.028	875	4,000	165	710	3,835
39	Syndicate		0.010	0.069	0.106	0.082	5,696	15,125	449	5,247	14,676
40	Tuxedo		0.004	0.036	0.037	0.033	2,791	8,250	405	2,386	7,845
41	Tylehurst		0.050	0.176	0.189	0.139	11,512	27,500	6,394	5,118	21,106
42	Woodhaven		0.00227	0.027	0.039	0.036	1,900	5,800	96	1,804	5,704

Table 5-12: Summary of Storage Required at Each District NE System 825 ML/d at NEWPCC

District Number	District	Combine With	DWF (m³/s)	Existing Rate (m³/s)	Runoff Based m³/s)	Dewatering Rate (m³/s)	Storage for 4 Overflows	Storage for 0 Overflows	Inline Storage	Storage for 4 Overflows with Inline	Storage for 0 Overflows with Inline
1	Alexander		0.035	0.155	0.230	0.195	7,500	17,000	3,803	3,697	13,197
2	Armstrong		0.020	0.524	0.211	0.181	5,300	19,000	10,060	0	8,940
3	Ash		0.082	0.301	0.895	0.813	22,000	65,000	40,418	0	24,582
4	Assiniboine		0.084	0.425	0.228	0.144	7,000	12,000	8,421	0	3,579
5	Aubrey		0.071	0.214	0.334	0.263	6,800	22,000	50,708	0	0
6	Baltimore		0.028	0.201	0.153	0.041	10,000	30,000	1,553	8,447	28,447
7	Bannatyne		0.153	0.613	0.343	0.190	5,500	17,000	2,378	3,122	14,622
8	Boyle	Syndicate	0.014	0.030							
9	Clifton		0.077	0.236	0.405	0.328	14,000	26,000	27,059	0	0
10	Cockburn		0.033	0.075	0.084	0.050	11,000	31,000	516	10,484	30,484
10a	Calrossie		0.001	0.028	0.000						
11	Colony		0.134	0.425	0.358	0.224	6,800	21,000	12,638	0	8,362
12	Cornish		0.035	0.107	0.106	0.071	1,800	5,600	5,596	0	4
13	Despins	Marion	0.032	0.132							
14	Doncaster		0.025	0.075	0.144	0.119	1,400	5,000	5,616	0	0
15	Douglas Pa	Ferry Road	0.001	0.095							
16	Dumoulin		0.013	0.136	0.157	0.144	3,750	13,000	630	3,120	12,370
17	Ferry Road		0.059	0.126	0.306	0.247	6,500	18,000	4,676	1,824	13,324
18	Hart		0.039	0.101	0.212	0.173	6,200	16,000	13,393	0	2,607
19	Hawthorne		0.036	0.113	0.237	0.201	6,000	18,500	3,875	2,125	14,625
20	Jefferson E		0.143	0.569	0.654	0.511	12,000	42,000	15,484	0	26,516
20a	Jefferson	Jefferson E	0.000	0.000	0.000	0.000				0	0
21	Jessie		0.066	0.176	0.421	0.355	11,500	31,000	6,662	4,838	24,338
22	La Verendr	Dumoulin	0.009	0.015						0	0
23	Linden		0.017	0.060	0.046	0.029	770	2,800	777	0	2,023
24	Mager Drive		0.091	0.309	0.309	0.050	11,500	34,000	7,531	3,969	26,469
25	Marion		0.032	0.220	0.341	0.309	11,000	30,000	4,080	6,920	25,920
26	Metcalfe		0.005	0.044	0.015	0.010	3,000	7,000	1,007	1,993	5,993
27	Mission		0.144	0.518	0.436	0.292	7,700	24,000	7,621	79	16,379
28	Moorgate		0.023	0.085	0.104	0.081	2,900	11,000	3,771	0	7,229
29	Munroe		0.077	0.237	0.472	0.395	13,000	45,000	38,360	0	6,640
30	Newton	Armstrong	0.010	0.166							
32	Polson		0.032	0.356	0.280	0.248	8,000	21,000	23,401	0	0
33	River		0.070	0.094	0.189	0.119	4,000	12,000	4,620	0	7,380
34	Riverbend/Parkside Dr.		0.053	0.107	0.254	0.201	6,500	18,000	293	6,207	17,707
35	Roland		0.026	0.324	0.266	0.240	9,200	20,000	22,455	0	0
36	Selkirk		0.067	0.453	0.254	0.187	5,000	16,000	10,254	0	5,746
37	St. Johns		0.084	0.173	0.460	0.376	12,500	32,000	24,895	0	7,105
38	Strathmillan		0.003	0.062	0.031	0.028	875	4,000	165	710	3,835
39	Syndicate		0.010	0.069	0.144	0.134	4,082	11,000	449	3,633	10,551
40	Tuxedo		0.004	0.036	0.057	0.053	2,000	6,000	405	1,595	5,595
41	Tylehurst		0.050	0.176	0.277	0.227	8,250	20,000	6,394	1,856	13,606
42	Woodhaven		0.00227	0.027	0.039	0.036	1,900	5,800	96	1,804	5,704

Table 5-13: Summary of Storage Required at Each District
NE System 1060ML/d at NEWPCC

District Number	District	Combine With	DWF (m^3/s)	Existing Rate (m^3/s)	Runoff Based m^3/s)	Dewatering Rate (m^3/s)	Storage for 4 Overflows	Storage for 0 Overflows	Inline Storage	Storage for 4 Overflows with Inline	Storage for 0 Overflows with Inline
1	Alexander		0.035	0.155	0.308	0.273	5,400	12,200	3,803	1,597	8,397
2	Armstrong		0.020	0.524	0.185	0.165	5,800	20,900	10,060	0	10,840
3	Ash		0.082	0.301	1.208	1.126	15,900	46,900	40,418	0	6,482
4	Assiniboine		0.084	0.425	0.285	0.201	5,000	8,600	8,421	0	179
5	Aubrey		0.071	0.214	0.439	0.368	4,900	15,700	50,708	0	0
6	Baltimore		0.028	0.201	0.125	0.097	4,300	12,800	1,553	2,747	11,247
7	Bannatyne		0.153	0.613	0.419	0.266	3,900	12,200	2,378	1,522	9,822
8	Boyle	Syndicate	0.014	0.030							
9	Clifton		0.077	0.236	0.535	0.458	10,000	18,600	27,059	0	0
10	Cockburn		0.033	0.075	0.084	0.050	11,000	31,000	516	10,484	30,484
10a	Calrossie		0.001	0.028							
11	Colony		0.134	0.425	0.448	0.314	4,900	15,000	12,638	0	2,362
12	Cornish		0.035	0.107	0.134	0.099	1,300	4,000	5,596	0	0
13	Despins	Marion	0.032	0.132							
14	Doncaster		0.025	0.075	0.190	0.165	1,000	3,600	5,616	0	0
15	Douglas Park	Ferry Road	0.001	0.095							
16	Dumoulin		0.013	0.136	0.210	0.188	2,900	9,900	630	2,270	9,270
17	Ferry Road		0.059	0.126	0.404	0.344	4,700	12,900	4,676	24	8,224
18	Hart		0.039	0.101	0.281	0.242	4,400	11,400	13,393	0	0
19	Hawthorne		0.036	0.113	0.317	0.281	4,300	13,200	3,875	425	9,325
20	Jefferson E		0.143	0.569	0.858	0.715	8,600	30,000	15,484	0	14,516
20a	Jefferson W	Jefferson E	0.000	0.000							
21	Jessie		0.066	0.176	0.562	0.496	8,200	22,200	6,662	1,538	15,538
22	La Verendrye	Dumoulin	0.009	0.015							
23	Linden		0.017	0.060	0.057	0.040	600	2,000	777	0	1,223
24	Mager Drive		0.091	0.309	0.281	0.190	3,000	8,900	7,531	0	1,369
25	Marion		0.032	0.220	0.451	0.387	8,800	23,900	4,080	4,720	19,820
26	Metcalfe		0.005	0.044	0.015	0.010	3,000	7,000	1,007	1,993	5,993
27	Mission		0.144	0.518	0.552	0.408	5,500	17,200	7,621	0	9,579
28	Moorgate		0.023	0.085	0.104	0.081	2,900	11,000	3,771	0	7,229
29	Munroe		0.077	0.237	0.629	0.552	9,300	32,200	38,360	0	0
30	Newton		0.010	0.166	0.098	0.088				0	0
32	Polson		0.032	0.356	0.378	0.346	5,700	15,000	23,401	0	0
33	River		0.070	0.094	0.236	0.166	2,900	8,600	4,620	0	3,980
34	Riverbend/Parkside Dr.		0.053	0.107	0.334	0.281	4,600	12,900	293	4,307	12,607
35	Roland		0.026	0.324	0.361	0.335	6,600	14,300	22,455	0	0
36	Selkirk		0.067	0.453	0.329	0.262	3,600	11,400	10,254	0	1,146
37	St. Johns		0.084	0.173	0.610	0.526	8,900	22,900	24,895	0	0
38	Strathmillan		0.003	0.062	0.031	0.028	900	4,000	165	735	3,835
39	Syndicate		0.010	0.069	0.192	0.168	3,300	8,800	449	2,851	8,351
40	Tuxedo		0.004	0.036	0.078	0.074	1,400	4,300	405	995	3,895
41	Tylehurst		0.050	0.176	0.368	0.318	5,900	14,300	6,394	0	7,906
42	Woodhaven		0.00227	0.027	0.039	0.036	1,900	5,800	96	1,804	5,704

Conceptually, there are two means of providing off-line storage at any given district. These comprise near-surface storage tanks, where space is available, and local tunnels. The latter could comprise individual tunnels for each district or continuous tunnels providing storage for contiguous districts. Near surface storage tanks were the preferred means of providing off-line storage, since, in general, these proved to be more economical than the use of tunnels.

Scenarios were developed for each of the dewatering rates, combined with 0 and 4 overflows and finally with or without in-line storage. All of the scenarios included an allowance for flow control, that is, either modifications to the existing pumped interception rate or some device to limit the gravity flow connections. Details of this device or the nature of the pumping station revisions were not investigated for this study. Rather, as discussed later, an allowance was made for modification or installation of the device. Each of the storage options with in-line storage had to include the cost of the in-line storage facilities. This was developed in Section 4.2.6. The allowance for in-line storage included for flow control in the systems.

The rationale behind the three dewatering rates has already been discussed. The implications of each are discussed briefly below.

The 600 ML/d dewatering rate results in the smallest impact on the NEWPCC as well as providing the most complete treatment of the stored combined sewage. The results of this lower dewatering rate, however, on the CSO facilities, are to require larger volumes of storage. In developing the 600 ML/d option, it has been assumed that disinfection will be implemented at the NEWPCC and, therefore, the stored combined sewage will be disinfected.

For the 825 ML/d dewatering rate, in order to provide an equivalent level of treatment to the 600 ML/d dewatering rate (i.e., disinfection of the total NEWPCC plant discharge), an allowance was made for the disinfection of the 225 ML/d which would only receive primary treatment. The cost was based on chlorination/dechlorination.

For the dewatering rate of 1,060 ML/d, an allowance had to be included for the necessary upgrade of the NEWPCC in order to be able to provide primary treatment to the 225 ML/d which currently cannot be accommodated at the NEWPCC. The necessary upgrade included for additional pumping capacity as well as new headworks, new primary sedimentation facilities and disinfection for the flow bypassing the secondary plant.

In addition to in-line and off-line storage, two scenarios were analyzed to determine the benefits which might accrue from transfers from one district without in-line storage to a district which had in-line storage, e.g., Riverbend to Clifton; or transfers from districts without space for near-surface tanks to a district which had such space. The limiting factor in these analyses proved to be the high cost of pumping and forcemain facilities. The latter costs dramatically limited the number of transfers which were economically viable. In general, it appeared to be more economical to install a tunnel storage unit than it would be to transfer over longer distances.

In the Phase 2 analysis of options, it was assumed each method of control, e.g., near-surface storage tanks, would be applied to each of the districts throughout the City. During the course of the Phase 3 analysis, it was determined that there were not sites available in every district for the near-surface tanks. Accordingly, it was considered that the most feasible alternative where such surface sites were not available, would be local tunnels serving the district requiring storage. The characteristics of these two types of storage are discussed below.

5.3.2 Near-Surface Tanks

Near-surface storage tanks would take the form of concrete storage basins located just beneath the surface of the sites. After installation, the site would be restored as closely as practicable to its original condition.

The adequacy of this technology has been proven elsewhere. For example, the study team made visits to the "Beaches" facilities designed by CG&S for the City of Toronto. These facilities are flushed by combined sewage stored in a head-tank for that purpose, after the stored combined sewage is removed. The operation is straightforward and satisfactory and is considered suitable technology for this supplementary storage option.

During the course of the Phase 3 investigations, the availability and location of public lands was investigated in order to determine potential locations for near-surface storage basins. Wherever in-line storage was either non-existent or insufficient to meet the regulatory benchmarks (e.g., number of overflows), the additional storage needed and, where space was available, the additional CSOs were to be pumped to these basins. The identification of suitable sites is discussed in the next section.

5.3.2.1 Siting Considerations

In order to determine the availability of sites for near-surface facilities, a survey was undertaken to determine which districts had suitable publicly-owned lands which might be used as sites for such devices. The site-selection process was staged.

The first stage comprised a review of aerial photographs of the City, in combination with the Sherlock City Map and Guide, to locate areas of open space near the combined sewer outlets which would be suitable for development. The nature of the land that was considered suitable compromised parks, school-yards, community centres and the like. The blocks of land were chosen on the basis that they would be sufficiently large to be able to construct at least one 20 m by 50 m basin within their boundaries. The units proposed would be 5-m deep reinforced concrete basins. This is the size of the primary sedimentation tanks installed in the recent upgrading of the SEWPCC, and would accordingly lend themselves to self-contained flushing mechanisms. By selecting this shape, it was recognized that units could serve as sedimentation basins if necessary (i.e., could act as retention treatment basins [RTBs]), and could likely be suitably flushed if only used as storage basins.

The photographs showed the relationships between roads and buildings, primarily, and vegetation to some degree, and were useful in determining which sites should be evaluated further. Having selected these sites, these were visited and evaluated by representatives of the study team. The report on this survey is entitled “CSO Storage/High-rate Treatment Option: Site Inventory/Neighbourhood Evaluation” and dated April 1996 (see attached Appendix). It was assumed that the City would only purchase private lands, i.e., a series of adjacent residential lots and buildings, as a last resort. This was not considered to be an option at this stage of analysis, i.e., only publicly-owned land was considered as available for siting.

The site evaluation matrix used for the sites selected by the initial survey comprised:

- ownership (public or private);
- land availability for near-surface structures;
- physical attributes of site
  - vegetation cover
  - surface morphology variance
  - presence of structures; and
- neighbourhood considerations
  - on-site construction disruption/displacement
  - off-site construction disturbance.

The nature of these considerations and their evaluation is described in the report, along with the evaluation results.

Thirty-five locations were selected as potential sites for near-surface facilities, as listed on Table 5-14. These sites had the potential to provide off-line storage for 37 of the combined sewer districts in the City.

The evaluation report itself recommended against some of the sites because of site constraints, e.g., limited space, sloping embankments, major vegetative cover, etc. A few of these were reassessed (desktop) during the course of final evaluation and some (limited) modifications were made to the report conclusions.

The final outcome of the site-selection procedure is summarized in Table 5-15. This Table shows the combined sewer areas served, the site number (if any) which was considered the best prospect for provision of space for that area; the storage potential for each of the sites (if any) and finally, the name of the site and/or if unusable, the main factor in precluding its use.

Table 5-14

#	Site Name	Site Address	Catchment Area	Current Zoning
1.	Elmwood Park	380 Henderson Hwy.	Hart (18); Munroe (29)	PR1
2.	Luxton School	111 Polson Ave.	Polson (32)	PR1
3.	Seven Oaks Historic Site	Rupertsland Ave. E.	Jefferson East (20a)	PR1, R1-4
4.	Aubrey Park	139 Aubrey St.	Aubrey (5)	R2
5.	Higgins @ McFarlane	52 Higgins Ave.	Boyle (8)	M2
6.	Bonnycastle Park	Assiniboine Ave.	Assiniboine (4)	S1, BR8, Ra, NRf, P/L2, RB
7.	Mayfair Park	River @ Donald	River (33)	RM4
8.	Fort Rouge Park	River Ave.	River (33)	RM4
9.	La Verendrye Park	Tache Ave.	Durmoulin (16); Despins (13)	PR1
10.	Fraser's Grove Park	Kildonan Drive	Hawthorne (19); Linden (23)	PR1
11.	Montcalm Playground	Nairn Ave.	Mission (27)	PR1, C2
12.	Chalmers South Playground	Elmwood Drive	Roland (35)	PR1
13.	N. Promenade	733 Tache Ave.	La Verendrye (22)	PR1
14.	Norwood Community Centre	Lawndale Ave.	Marion (25)	PR1
15.	Coronation Park	41 St.Mary's Road	Metcalfe (26)	PR1
16.	Nelson McIntyre Collegiate	188 St.Mary's Road	Metcalfe (26)	R1-5
17.	Glenwood School	51 Blenheim Ave.	Mager Drive (24)	PR1
18.	Churchill Drive Park E.	Churchill Drive	Baltimore (6)	PR1
19.	Riverview Community Club	Ashland @ Eccles	Baltimore (6)	PR1
20.	N. of Transit Garage	421 Osborne St.	Jessie (21)	C2
21.	Churchill Drive Park W.	Churchill Drive	Calrossie/Cockburn (10)	PR1
22.	McKittrick Park	Beresford Ave.	Calrossie/Cockburn (10)	PR1
23.	Berwick Athletic Field	Argue St.	Calrossie/Cockburn (10)	M2
24.	CNR Fort Rouge Yards (south)	Argue St.	Calrossie/Cockburn (10)	M2
25.	Hebrew College Constr'n Site	Wellington Cres.	Tuxedo (40); Doncaster (14)	A5
26.	Bourkevale Community Centre	100 Ferry Road	Douglas Park (15); Ferry Road (17); Riverbend C (34)	PR1
27.	Omand Park	Portage @ Empress	Clifton (9); Tylehurst (41)	R2
28.	Great West Life parking lot	Balmoral @ Mostyn	Colony (11); Cornish (12)	Rc, NRb, BR1, P/L1, S2, LB
29.	George @ Argyle St.	George @ Argyle	Alexander (1); Bannatyne (7)	M1
30.	Juba Park	James Ave. E.	Alexander (1); Bannatyne (7)	S1, BR8, Ra, NRf, P/L2, RB
31.	Barber Park	Barber @ Rover Ave.	Syndicate (39)	PR1
32.	Norquay Community Centre	65 Granville St.	Selkirk (36)	PR1/R2
33.	St. John's Park	1199 Main St.	St. John's (37)	PR1
34.	Marymound School	442 Scotia Ave.	Newton (30)	R1-4
35.	Kildonan Park SW	2021 Main St.	Armstrong (2)	PR2

Table 5-15:
Potential Sites for Near Surface CS Facilities

CS AREA	SITE NO.	STORAGE POTENTIAL	REMARKS
1	30	3 UNITS	JUBA PARK
2	35	3 UNITS	KILDONAN PARK SW
3			***NO SITE AVAILABLE***
4	6		SITE CONSTRAINED (BONNYCASTLE PARK)
5	4		SITE CONSTRAINED (AUBREY PK)
6	19	3 UNITS	RIVERVIEW CC
7	30	3 UNITS	JUBA PARK
8	5		DIFF. SOLUTION FOR BOYLE
9	27	6 UNITS	OMAND PARK
10	23	5 UNITS	BERWICK ATH. FIELD
10	22		McKITTRICK PARK ALSO POSS.
11	28		GREAT WEST PARKING LOT
12	28		GREAT WEST PARKING LOT
13/16/22	9	1 UNITS	LA VERENDRYE PARK
14/40	25		HEBREW SCHOOL
15/17/34	8	8 UNITS	FORT ROUGE PARK
18/29	1	4 UNITS	ELMWOOD PARK
19/23	10		CONSTRAINED BY TREES (FRASER'S GROVE PK)
20/30	3		HISTORIC SITE (SEVEN OAKS)
21	20	5 UNITS	NORTH OF TRANSIT GARAGE
24/26	17	2 UNITS	GLENWOOD SCHOOL
25	14	7 UNITS	NORWOOD CC
27/35	11/12	7 UNITS	MONTCALM/CHALMERS PLAYGROUNDS
28			***NO SITE AVAILABLE***

Table 5-15:
Potential Sites for Near Surface CS Facilities

CS AREA	SITE NO.	STORAGE POTENTIAL	REMARKS
31			***NO SITE AVAILABLE***
32	2	3 UNITS	LUXTON SCHOOL
33	7	1 UNIT	MAYFAIR PARK
34			***NO SITE AVAILABLE***
36	32	4 UNITS	NORQUAY CC
37	33	6 UNITS	ST. JOHN'S PARK
38			***NO SITE AVAILABLE***
39	31	1 UNIT	BARBER PARK
41			***NO SITE AVAILABLE***
42			***NO SITE AVAILABLE***

The potential sites listed in Table 5-15 have a total capability of installing 72 of the rectangular basins chosen as being representative of storage tanks (that is 50 by 20 m in plan). Accordingly, these 72 potential units represent some 360,000 m^3 of potential off-line storage, based on the 5,000 m^3 of each rectangular basin.

Part of the site evaluation comprised on-site disruption or displacement during construction and off-site construction disturbance. Both of these aspects would be dealt with as reasonably as possible, with every effort being made during construction to limit such impacts. On completion of construction, the storage units would be located below grade and these surfaces would be restored as closely as possible to the conditions prior to construction.

5.3.2.2 Flow Conveyance

It was assumed that the size of the pumps needed to convey the flow to near-surface storage would be equal to the predicted peak flow of the largest storm to be captured in order to prevent any overflow from that storm (Table 5-16). Although conservative, this assumption provided a ready means of developing a cost for the pumping station and the associated forcemain. This assumption was made whether or not in-line storage was in place.

5.3.2.3 Operational Considerations

Once installed, and the surface restored, the major potential impact of the near-surface tanks on the adjacent areas could be odour. The experience at the Scarborough Beaches facilities installed in Toronto has been good. Odour-scrubbing facilities were installed and have had very limited, if any, use. Neither has there been complaints of odours from the tanks.

Insofar as operations are concerned, the tanks at the Beaches have been designed for two different cleaning operations. One design was a high-pressure flushing system with nozzles distributed along the length of the tank. This has been the more expensive of the two, and relatively operator intensive. The second system involved a hydraulic head tank installed at the

Table 5-16

District Name	Transfer Rate Runoff Based m^3/s
Alexander	1.27
Armstrong	0.77
Ash	4.72
Assiniboine	0.94
Aubrey	1.71
Baltimore	1.02
Bannatyne	1.24
Clifton	2.13
Cockburn & Calrossie	1.25
Colony	1.46
Cornish	0.46
Doncaster	0.40
Ferry Road & Douglas Park	1.60
Hart	1.13
Hawthorne	1.31
Jefferson E	3.32
Jessie	2.31
La Verendrye & Dumoulin	0.87
Linden	0.19
Mager Drive	1.25
Marion & Despins	1.80
Metcalfe	0.26
Mission	1.90
Moorgate	0.66
Munroe	2.57
Newton	0.41
Polson	1.61
River	0.77
Riverbend & Parkside Dr.	1.31
Roland	1.56
Selkirk	1.22
Strathmillan	0.23
St. Johns	2.44
Syndicate & Boyle	0.78
Tuxedo	0.35
Tylehurst	1.48
Woodhaven	0.29

upstream end of the basin. On completion of basin dewatering, the stored water is released and has satisfactorily flushed the floor with virtually no manpower requirements. The latter system or its equivalent would be incorporated in any such tanks installed in Winnipeg.

5.3.3 Local Storage Tunnels

5.3.3.1 Siting Considerations

In those districts in which sites were not available for near-surface storage tanks, the cost of the off-line storage system was based on the use of local tunnels. The original intent had been to use relatively shallow storage tunnels, located under road allowances, probably at right angles to the river. It became evident, through subsequent analysis, that this would have entailed a relatively large number of independent dewatering systems which, depending upon the approach taken, could have resulted in a very expensive system. Accordingly, the entire NEWPCC combined sewer area was divided into groupings of contiguous areas (wherever possible), the tunnel storage for which would be provided by deeper continuous tunnels parallel to the Assiniboine and Red Rivers. These tunnel groupings, designated as A to G inclusive, have been shown on Figure 5-17. The groupings are as follows:

- Group A
  - Ferry Road
  - Riverbend
  - Tylehurst
  - Clifton
  - Aubrey
  - Cornish
  - Colony
  - Assiniboine

- Group B
  - Tuxedo
  - Doncaster
  - Ash

- Group C
  - Jessie
  - River

- Group D
  - Mission/Roland
  - LaVerendrye/Dumoulin
  - Despins
  - Marion

- Group E
  - Bannatyne
  - Alexander
  - Syndicate/Boyle
  - Selkirk
  - St. John's
  - Polson

- Group F
- Cockburn
- Baltimore

- Group G
  - Moorgate
  - Strathmillan

These conceptual tunnels were sized, generally, so as to provide the required volume of storage in the length available. Where length was not constrained, generally, 4 metres was selected as being a reasonable (practicable) diameter.

Flows to the storage tunnels would be by gravity from the trunk or SRS and, as with the off-line storage tanks, the tunnels would be emptied after the in-line storage had been removed and transferred to the interceptor. Flows from the tunnels would be pumped to the interceptor.

5.3.3.2 Flow Conveyance

The storage tunnels would be installed with the crown located below the invert of the trunk. Indeed, because of potential construction difficulties and cost factors, these group tunnels would likely be constructed at some depth, within the rock, below the clay and till, so as to avoid mixed face tunnelling. Accordingly, flow from the combined sewers would be by gravity directly into the tunnels. Flows stored in the tunnels would be pumped to the interceptor after runoff had ceased and after in-line storage had been conveyed to the interceptors. The pumps would be sized to equal the dewatering rate for the districts involved. An estimating allowance of $200,000 net was made for each combined sewer district to cover the costs of the pumping stations associated with these tunnel groupings.

5.3.3.3 Operational Considerations

As noted, the normal operation of the tunnel storage groupings is straightforward. Flows from the combined sewer trunks and SRSs are controlled by gravity and flows from the storage tunnels, on cessation of runoff and dewatering of in-line storage, is directed to the interceptor by pumping. Since the dewatering rate is established on the basis of the pre-determined rate at which each combined sewer district contributes to the interceptor, the normal velocity resulting from this dewatering rate will not be sufficient, firstly, to prevent solids from settling out in the tunnels and, secondly, to scour such settled particles. Accordingly, it was necessary to devise a conceptual system which could be used to flush these tunnels.

CG&S developed such a flushing system, in concept, for the City of Toronto combined sewer storage tunnel. That particular tunnel was 5 metres in diameter and the concept was to generate a wave flowing down the tunnel at a velocity of 1 metre/second (3 fps+) which would scour such settled solids. The resultant system comprised a discharge of 3 m^3/s at the upstream end of the tunnel, which generated a velocity at the upstream end of 3 metres/second and resulted in a minimum velocity in 2 km of tunnel of the desired 1 metre/second. The withdrawal rate at the end of the 2 km reach was modest (0.3 m^3/s), and hence did not impose an undue load on the interceptor. The sump for the dewatering sewer was some 400 m^3 and between 2 and 3 metres deep.

For purposes of estimating the cost of this dewatering strategy for each of the tunnel groupings, and the individual tunnel reaches, the geometry involved in the above concept was applied to the Winnipeg tunnels.

5.3.4 Costing

Table 5-17 comprises the results of the analysis on the various off-line storage scenarios. As can be seen, these results are based on alternatives with and without in-line storage; with 0 and 4 overflows for the 1992 representative year; and for the 3 dewatering scenarios. The basis of these calculations are provided in tabular form in the costing appendices. A sample of one of these tables is provided for the least cost option, i.e., 4 overflows, with in-line storage at a dewatering rate of 825 ML/d. This is Table 5-18.

As can be seen on Table 5-18, each of the combined sewer districts is listed, along with the storage required to limit the CSO to 4 overflows. The various tunnel groupings are indicated and relate to the groups as described above. The off-line storage units available in each district are listed and, if required, the number actually used are shown. In general, near-surface tanks were the preferred devise. Where tunnels are used, the tunnel diameter length and volumes are outlined and a decision was made as to whether this storage should be provided via near-surface structures or tunnel. For major pumps and forcemains (mainly for near-surface

Table 5-17: OFF-LINE STORAGE - COST SUMMARY

1992 REPRESENTATIVE YEAR

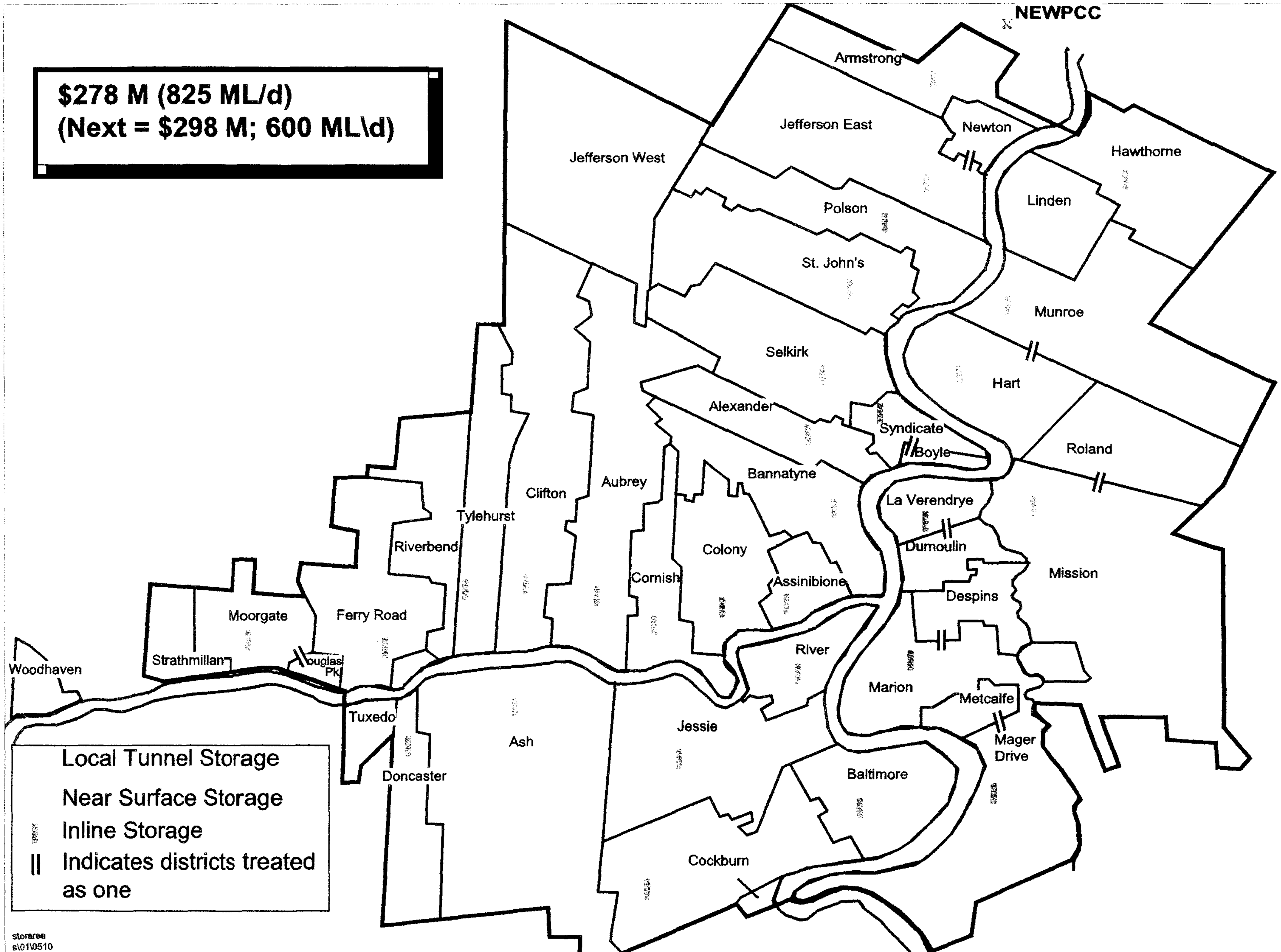
	WITH IN-LINE STORAGE			WITHOUT IN-LINE STORAGE		
DEWTR RATE	600 ML/d	825 ML/d	1060 ML/d	600 ML/d	825 ML/d	1060 ML/d
0 OVRFLOWS	$M	$M	$M	$M	$M	$M
STRGE VOL.	820,000 m^3	610,000 m^3	530,000 m^3	820,000 m^3	610,000 m^3	530,000 m^3
BASE COST*	570	425	415	781	647	520
FLO CNTROL				12	12	12
IN-LN STRGE	100	100	100			
FLUSHING	43	31	28	64	50	44
INTERCEPTOR		15	46		15	46
NEWPCC	15	36	70	15	36	70
TOTAL 0 O/F	$728M	$608M	$659M	$872M	$760M	$691M
4 OVRFLOWS	$M	$M	$M	$M	$M	$M
STRGE VOL.	300,000 m^3	220,000 m^3	185,000 m^3	300,000 m^3	220,000 m^3	185,000 m^3
BASE COST*	168	119	90	358	313	280
FLO CNTROL				12	12	12
IN-LN STRGE	100	100	100			
FLUSHING	16	8	8	26	22	23
INTERCEPTOR		15	46		15	46
NEWPCC	15	36	70	15	36	70
TOTAL 4 O/F	$298M	$278M	$315M	$411M	$398M	$430M

* BASE COSTS INCLUDE MULTIPLIERS

Table 5-18: OFFLINE STORAGE - 220,000 m³ (4 Overflows; Dewater @ 825mL/d) With In-line Storage

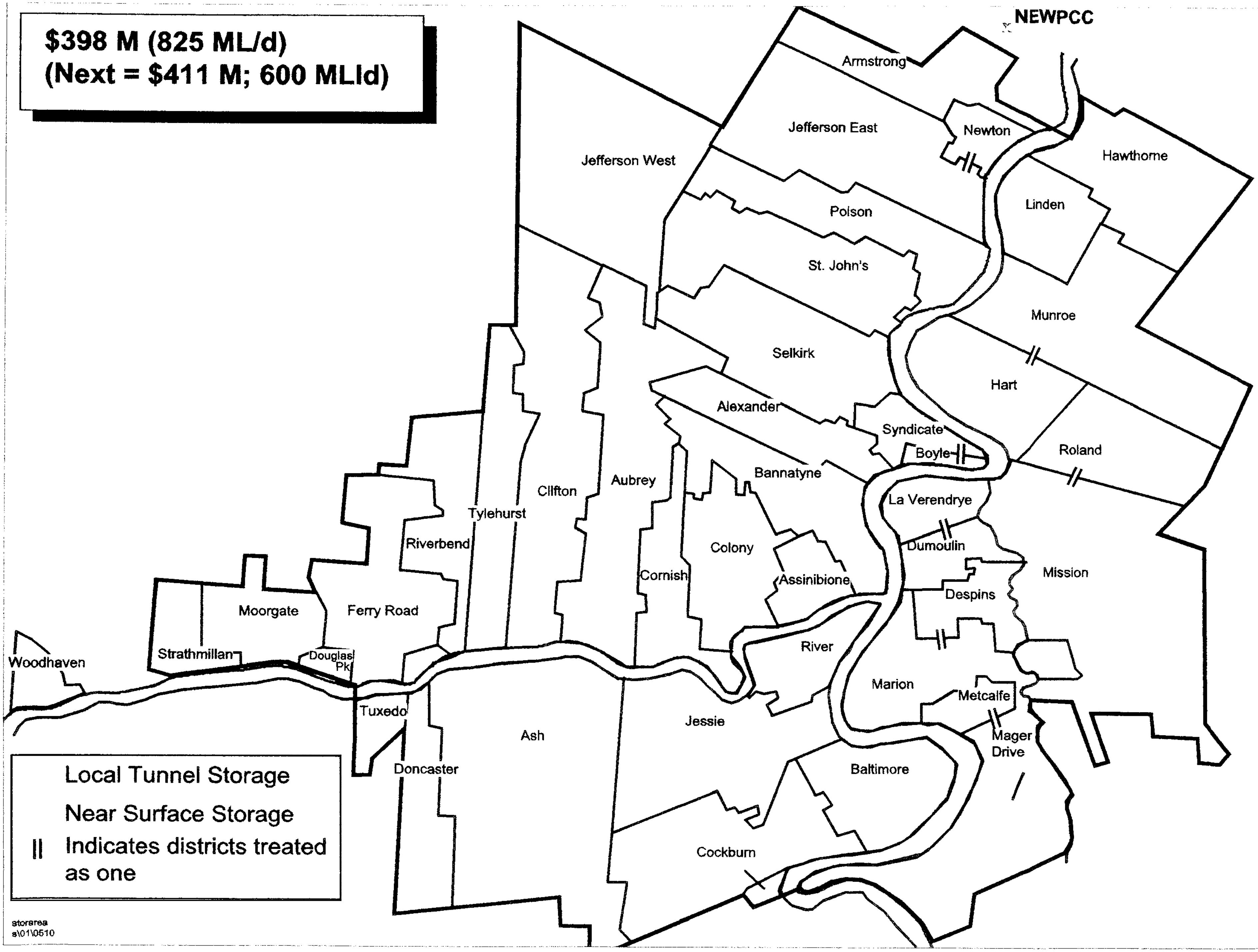
DISTRICT	STORAGE NEEDED m³ 4 Overflows	GROUP	OFFLINE UNITS AVAILABLE (5000 m³)	FFLINE UNITS USED 5000 m³	TUNNEL DIAMETER USED m	TUNNEL LENGTH NEEDED m	TUNNEL VOLUME m³	PUMP CAPACITY NEEDED (tanks)	PIPE DIAMETER (2m/s) m	PIPE LENGTH m	POWER kW	COST TANKS $M	COST TUNNELS $M	COST PUMP $M	COST PUMP STNS+F.M $M
DIAMETER					3										
FERRY ROAD	1820	A	8	0.0	2.60	1863	9891	0.0	0.0	400	0	0.0	11	0.0	0.2
RIVERBEND	6210	A	0	0.0			0		0.0				0	0.0	0.2
TYLEHURST	1860	A	0	0.0					0.0				0	0.0	0.2
CLIFTON	0	A	6	0.0				0.0	0.0	0	0	0	0	0.0	0.2
AUBREY	0	A	0	0.0		0			0.0				0	0.0	0.2
									0.0				0	0.0	
TUXEDO	1600	B	0	0.0	2.26	400	1605		0.0				2	0.0	0.2
DONCASTER	0	B	0	0.0		0			0.0				0	0.0	0.2
ASH	0	B	0	0.0		0			0.0				0	0.0	0.2
									0.0				0	0.0	
CORNISH	0	A	0	0.0		0			0.0				0	0.0	0.2
COLONY	0	A	0	0.0		0			0.0				0	0.0	0.2
									0.0				0	0.0	
JESSIE	4840	C	5	1.0		0		2.3	1.2	400	203	3.0	0	1.9	2.5
RIVER	0	C	1	0.0		0			0.0				0	0.0	0.2
									0.0				0	0.0	
ASSINIBOINE	0	A	0	0.0		0			0.0				0	0.0	0.2
									0.0				0	0.0	
MISSION/ROLAND	80	D	7	0.0		0		3.5	1.5	650	309	0.0	0	2.6	3.5
LaVERENDRYE/DUMOULI	3120	D	1	0.6		0		0.9	0.8	150	79	3.2	0	1.2	1.4
DESPINS		D							0.0				0	0.0	0.2
MARION	6920	D	7	1.4		0		1.8	1.1	250	159	3.1	0	1.6	2.0
									0.0				0	0.0	
BANNATINE	3120	E	3	0.6		0		1.2	0.9	300	106	3.1	0	1.3	1.7
ALEXANDER	3700	E	3	0.7		0		1.3	0.9	60	115	3.7	0	1.3	1.5
SYNDICATE/BOYLE	3630	E	1	0.7		0		0.8	0.7	200	71	3.6	0	1.0	1.3
SELKIRK	0	E	4	0.0				0.0	0.0	0	0	0.0	0	0.0	0.2
ST.JOHN'S	0	E	9	0.0				0.0	0.0	0	0	0.0	0	0.0	0.2
									0.0				0	0.0	
HART/MUNROE	0		4	0.0		0		0.0	0.0	0	0	0.0	0	0.0	0.2
									0.0				0	0.0	
POLSON	0	E	3	0.0				0.0	0.0	0	0	0.0	0	0.0	0.2
JEFFERSON/NEWTON	0		0	0.0		0			0.0				0	0.0	0.2
									0.0				0	0.0	
HAWTHORNE	2120		0	0.0	3.00	300	2120		0.0				2	0.0	0.2
													0	0.0	
ARMSTRONG/CONNECTO	0		3	0.0				0.0	0.0	0	0	0.0		0.0	0.2
	39020		65	5.075			13616								
TUNNEL VOL. SUPP.															
TANKS VOL. SUPP.				25375											
MAIN TUNNEL															
Diameter Required					3										
COCKBURN	10480	F	5	2.1		0		1.3	0.9	750	197	4.2	0	1.8	2.5
BALTIMORE	8450	F	3	1.7		0		1	0.8	500	152	3.6	0	1.6	2.1
													0	0.0	
METCALFE/MAGER	5960		2	1.2		0		1.5	1.0	900	147	3.6	0	1.5	2.3
						0							0	0.0	
						0							0	0.0	
MOORGATE/DOUG.PARK	0	G			1.5	400	707						2	0.0	0.2
						0							0	0.0	
STRATHMILLAN	710	G											0	0.0	0.2
						0							0	0.0	
WOODHAVEN	1800					255							2	0.0	0.2
												$31	$19		$25

TOTAL P.S. COST	**$25 M**
TOTAL TUNNEL COST	**$19 M**
TOTAL TANK COST	**$31 M**
TOTAL NET COST	**$75 M**
TOTAL BUDGET COST (1.58*NET)	**$119 M**
+ FLOW CONTROL	
+ INLINE STORAGE	**$100 M**
+ INTERCEPTOR	**$15 M**
+ NEWPCC	**$36 M**
+ FLUSHING	**$8 M**
TOTAL ESTIMATED COST	**$278 M**



In-Line / Off-Line Storage: Least Cost Configuration - 4 Overflows

Figure 5-18



Off-Line Storage (No In-Line): Least Cost Configuration - 4 Overflows
Figure 5-19

facilities) the pump sizes and forcemain data are listed. Finally, the cost for the tanks, the tunnels, and the pumps and forcemains are calculated.

The summary at the bottom of the table provides the total budget cost, including mark-ups. In this case, the mark-ups are 10% for ancillaries, 20% for estimating contingency, and 20% for engineering, administration and finance. Because the example in **Table 5-18** has in-line storage, there is no allowance for flow control, because it is included in the allowance for in-line storage. As discussed earlier, a segment of the interceptor must be supplemented in order to be able to convey the revised new flow rates from the combined sewers to the NEWPCC. The costs for this addition are added. Each of the 3 dewatering scenarios required expansion of the NEWPCC. Their costs were included. The flushing cost at the bottom of the table allows for costs, where needed, to provide the flushing of the group tunnels and the individual segments of tunnels which are listed on the table.

The costs for near-surface storage tanks, tunnels, pumping facilities, forcemains, etc., were all based on the curves developed for the Phase 2 analyses. The curves selected at that time were considered to be sufficiently conservative to still be applicable to the Phase 3 estimates.

Figure 5-18 indicates the nature of the storage used for the least cost configuration (the basis for **Table 5-18**). The system is based on the 825 ML/d scenario and its estimated cost is $278 million. The designations N and T indicate whether or not the district is provided with near-surface or tunnel storage. The designation or prefix I represents districts with in-line storage. As noted on the figure, the next least cost for this configuration is $298 million for the 600 ML/d scenario.

Figure 5-19 shows the distribution of near-surface and tunnel storage for the off-line storage system without in-line storage. As with the previous example, this is for 4 overflows. The configuration shown has an estimated cost of $398 million and is for the 825 ML/d scenario. The next least cost option is that dewatered at 600 ML/d and at $411 million.

As discussed in **Section 5.3.1**, two scenarios were developed to determine the nature of the savings which might be incurred by means of transfers. The results are provided on **Table 5-**

19, for the 600 ML/d dewatering scenario, with 4 overflows and on Table 5-20, for the 825 ML/d dewatering rate and 4 overflows. The resultant costs are estimated to be $252 million and $264 million, respectively. Figure 5-20 indicates graphically the nature of the storage in the various districts and the transfers, both for the lesser cost option. The transfer from Riverbend to Clifton would use the available in-line storage in Clifton. Likewise the transfers from Mission and LaVerendrye would use available in-line storage in Roland and from Syndicate Boyle would use available in-line storage in Selkirk.

As can be seen from a comparison between the options with and without transfers, the apparent saving could be in the order of 10 to 15%. This difference might be reduced by a further optimization of the off-line storage without transfers. In any case, it is not likely to increase. Refinement of the lower cost options might be a worthwhile investigation during more detailed assessments of each of the districts.

5.4 REGIONAL TUNNEL

5.4.1 Concept

The potential for a regional storage/conveyance tunnel as a means of CSO control was raised in Phase 2. While it appeared to be one of the most expensive options, it was decided that this should be carried forward into the Phase 3 evaluation.

As outlined in Section 3.3 Modelling Approach and Section 4.2, Interceptor/Treatment Capacity, the storage required for each CS district, was prepared for each of the three dewatering rates (600 ML/d, 825 ML/d, and 1,060 ML/d at the NEWPCC). Similar dewatering rates were developed for the SEWPCC and WEWPCC districts. As noted earlier, the rates for these latter districts were not varied. The storage requirements were further refined by being calculated for 4 and 0 overflows to the river, for the 1992 "Representative Year", both with and without in-line storage. Accordingly, a broad spectrum of systems was reviewed. The storage requirements developed for this combination of conditions are included in Tables 5-11, 5-12 and 5-13.

Table 5-19: OFFLINE STORAGE - 300,000 m³ (4 Overflows; Dewater @ 600mL/d)
With In-line Storage,Transfers and Extra Tanks

DISTRICT	STORAGE NEEDED m³ 4 Overflows	OFFLINE UNITS AVAILABLE (5000 m³)	OFFLINE UNITS USED (5000 m³)	TUNNEL DIAMETER USED m	TUNNEL LENGTH NEEDED m	PUMP CAPACITY NEEDED (tanks)	PIPE DIAMETER (2m/s) m	PIPE SLOPE	PIPE LENGTH m	POWER kW	COST TANKS $M	COST TUNNELS $M	COST PUMPS $M	COST PUMP STNS+F.M. $M	PUMP TO	EXTRA TANKS AT
DIAMETER				3												
FERRY ROAD	4390	8	0.9		0	1.6	1.0		400	141	2.9		1.5	2.0		
RIVERBEND	8780	0	0.0		0	1.3	0.9	0.0	2310	258		0	3.2	5.0	Clifton	
TYLEHURST	5120	0	0.0		724		0.0					4	0.0	0.2		
CLIFTON	0	6	0.0			0.0	0.0		0	0	0.0		0.0	0.2		
AUBREY	0	0			0		0.0					0	0.0	0.2		
							0.0						0.0			
TUXEDO	2300	0			325		0.0					2	0.0	0.2		
DONCASTER	0	0			0		0.0					0	0.0	0.2		
ASH	0	0			0		0.0					0	0.0	0.2		
							0.0						0.0			
CORNISH	0	0			0		0.0					0	0.0	0.2		
COLONY	0	0			0		0.0					0	0.0	0.2		
							0.0						0.0			
JESSIE	9390	5	1.9		0	2.3	1.2		400	203	3.8		1.9	2.5		
RIVER	960	1	0.0		136	0.0	0.0		0	0	0.0	1	0.0	0.2		
							0.0						0.0			
ASSINIBOINE	1350	0			191		0.0					1	0.0	0.2		
							0.0						0.0			
MISSION/ROLAND	3120	7	0.0			3.5	1.5	0.0021	100	282	0.0		2.4	2.7	Roland	
LaVERENDRYE/DUMOULI	4600	1	0.0			0.9	0.8	0.0048	2130	161	0.0		1.6	3.0	Roland	
DESPINS							0.0						0.0	0.2		
MARION	11270	7	2.3		0	1.8	1.1		250	159	4.3		1.6	2.0		
							0.0						0.0			
BANNATINE	5300	3	1.1		0	1.2	0.9		300	106	3.2		1.3	1.7		
ALEXANDER	6660	3	1.3		0	1.3	0.9		60	115	3.2		1.3	1.5		
SYNDICATE/BOYLE	5250	1	0.0			0.7	0.7	0.0045	1050	87	0.0		1.2	1.9	Selkirk	
SELKIRK	0	4	0.0			0.0	0.0		0	0	0.0		0.0	0.2		
ST.JOHN'S	0	6	0.0			0.0	0.0		0	0	0.0		0.0	0.2		
							0.0						0.0			
HART/MUNROE	0	4	0.0		0	0.0	0.0		0	0	0.0	0	0.0	0.2		
							0.0						0.0			
POLSON	0	3	0.0			0.0	0.0		0	0	0.0		0.0	0.2		
JEFFERSON/NEWTON	1260	0			178		0.0					1	0.0	0.2		
							0.0						0.0			
HAWTHORNE	4500	0			637		0.0					4	0.0	0.2		
													0.0			
ARMSTRONG/CONNECTO	0	3	0.0			0.0	0.0		0	0	0.0		0.0	0.2		
													0.0			
	74250	62	7		2191								0.0			
TRANSFERRED				21750									0.0			
TUNNEL VOL. SUPP.					15490								0.0			
TANKS VOL. SUPP.			37010										0.0			
MAIN TUNNEL													0.0			
Diameter Required				3									0.0			
COCKBURN	10480	5	2.1		0	1.3	0.9		750	197	4	0	1.8	2.5		
BALTIMORE	8450	3	1.7		0	1	0.8		500	152	4	0	1.6	2.1		
													0.0			
METCALFE/MAGER	5960	2	1.2		0	1.5	1.0		900	147	3	0	1.5	2.3		
													0.0			
													0.0			
MOORGATE/DOUG.PARK	0				0							0	0.0	0.2		
													0.0			
STRATHMILLAN	710				100							1	0.0	0.2		
													0.0			
WOODHAVEN	1800				255							2	0.0	0.2		
											$28	$16		$33		

TOTAL P.S. COST	$33 M
TOTAL TUNNEL COST	$16 M
TOTAL TANK COST	$28 M
TOTAL NET COST	$77 M
TOTAL BUDGET COST (1.58*NET)	$122 M
+ FLOW CONTROL	
+ INLINE STORAGE	$100 M
+ INTERCEPTOR	$0
+NEWPCC	$15 M
+ FLUSHING	$15 M
TOTAL ESTIMATED COST	$252 M

Table 5-20: OFFLINE STORAGE - 220,000 m³ (4 Overflows; Dewater @825mL/d) With In-line Storage,Transfers and Extra Tanks

DISTRICT	STORAGE NEEDED m³ 4 Overflows	OFFLINE UNITS AVAILABLE (5000 m³)	OFFLINE UNITS USED (5000 m³)	TUNNEL DIAMETER USED m	TUNNEL LENGTH NEEDED m	PUMP CAPACITY NEEDED (tanks)	PIPE DIAMETER (2m/s) m	PIPE SLOPE	PIPE LENGTH m	POWER kW	COST TANKS $M	COST TUNNELS $M	COST PUMPS $M	COST PUMP STNS+F.M. $M	PUMP TO	EXTRA ANKS AT
DIAMETER				3												
FERRY ROAD	1820	8	0.4		0	1.6	1.0		400	141	2.5	0	1.5	2.0		
RIVERBEND	6210	0	0.0		0	1.3	0.9	0.0	2310	258		0	3.2	5.0	Clifton	
TYLEHURST	1860	0	0.0		263		0.0					2	0.0	0.2		
CLIFTON	0	6	0.0			0.0	0.0		0	0	0.0		0.0	0.2		
AUBREY	0	0	0.0		0		0.0					0	0.0	0.2		
							0.0						0.0			
TUXEDO	1600	0	0.0		226		0.0					1	0.0	0.2		
DONCASTER	0	0	0.0		0		0.0					0	0.0	0.2		
ASH	0	0	0.0		0		0.0					0	0.0	0.2		
							0.0						0.0			
CORNISH	0	0	0.0		0		0.0					0	0.0	0.2		
COLONY	0	0	0.0		0		0.0					0	0.0	0.2		
							0.0						0.0			
JESSIE	4840	5	1.0		0	2.3	1.2		400	203	2.9	0	1.9	2.5		
RIVER	0	1	0.0		0	0.0	0.0		0	0	0.0	1	0.0	0.2		
							0.0						0.0			
ASSINIBOINE	0	0	0.0		0		0.0					0	0.0	0.2		
							0.0						0.0			
MISSION/ROLAND	80	7	0.0		0	0.0	0.0	0.0021	100	0	0.0	0	0.0	0.2		
LaVERENDRYE/DUMOULI	3120	1	0.0			0.9	0.8	0.0048	2130	161	0.0		1.6	3.0	Roland	
DESPINS							0.0						0.0	0.2		
MARION	6920	7	1.4		0	1.8	1.1		250	159	4.3		1.6	2.0		
							0.0						0.0			
BANNATINE	3120	3	0.6		0	1.2	0.9		300	106	3.1	0	1.3	1.7		
ALEXANDER	3700	3	0.7		0	1.3	0.9		60	115	3.0	0	1.3	1.5		
SYNDICATE/BOYLE	3630	1	0.0		0	0.7	0.7	0.0045	1050	87	0.0	0	1.2	1.9	Selkirk	
SELKIRK	0	4	0.0			0.0	0.0		0	0	0.0	0	0.0	0.2		
ST.JOHN'S	0	6	0.0			0.0	0.0		0	0	0.0	0	0.0	0.2		
							0.0						0.0			
HART/MUNROE	0	4	0.0		0	0.0	0.0		0	0	0.0	0	0.0	0.2		
							0.0						0.0			
POLSON	0	3	0.0			0.0	0.0		0	0	0.0		0.0	0.2		
JEFFERSON/NEWTON	0	0	0.0		0		0.0					0	0.0	0.2		
							0.0						0.0			
HAWTHORNE	2120	0	0.0		300		0.0					2	0.0	0.2		
													0.0			
ARMSTRONG/CONNECTO	0	3	0.0			0.0	0.0		0	0	0.0		0.0	0.2		
													0.0			
	39020	62	4		789								0.0			
TRANSFERRED				12960									0.0			
TUNNEL VOL. SUPP.					5580								0.0			
TANKS VOL. SUPP.			20400										0.0			
MAIN TUNNEL													0.0			
Diameter Required				3									0.0			
COCKBURN	10480	5	2.1		0	1.3	0.9		750	197	4	0	1.8	2.5		
BALTIMORE	8450	3	1.7		0	1	0.8		500	152	4	0	1.6	2.1		
													0.0			
METCALFE/MAGER	5960	2	1.2		0	1.5	1.0		900	147	3	0	1.5	2.3		
													0.0			
													0.0			
MOORGATE/DOUG.PARK	0				0							0	0.0	0.2		
													0.0			
STRATHMILLAN	710				100							1	0.0	0.2		
													0.0			
WOODHAVEN	1800				255							2	0.0	0.2		
											$26	$8		$31		

TOTAL P.S. COST	$31 M
TOTAL TUNNEL COST	$8 M
TOTAL TANK COST	$26 M
TOTAL NET COST	$65 M
TOTAL BUDGET COST (1.58*NET)	$103 M
+ FLOW CONTROL	
+ INLINE STORAGE	$100 M
+ INTERCEPTOR	$15 M
+NEWPCC	$36 M
+ FLUSHING	$10 M
TOTAL ESTIMATED COST	$264 M

OFL77TAA.WK4

Off-Line Tunnels Grouping
Figure 5-17

5.4.2 Study Considerations

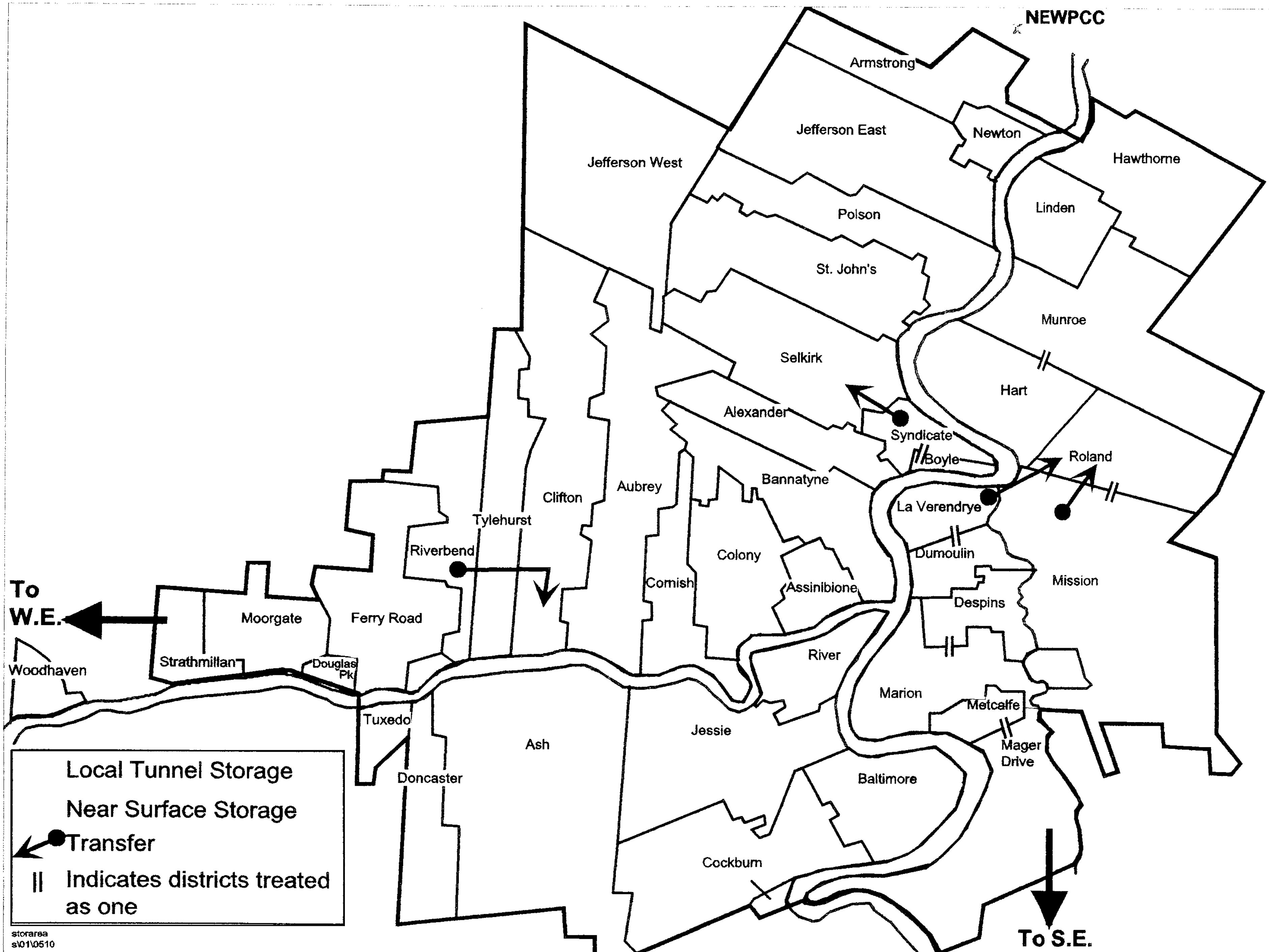
The Phase 2 analysis was simplistic in that it assumed that there would be large diameter collector tunnels on both sides of the Assiniboine River and the Red River which would collect CSOs and direct them to the NEWPCC. This approach was refined somewhat in Phase 3. It was found that it was more cost-effective to have a main tunnel on the north side of the Assiniboine and the west side of the Red and collector tunnels south of the Assiniboine and east of the Red. The flows from the latter would be transferred to the main tunnel and thence conveyed to the NEWPCC. The plan of the potential scheme is shown on Figure 5-21.

The collector tunnels for the SEWPCC and WEWPCC districts were also sized and the conceptual locations are shown on the Figure. In general, the routes of the tunnels were located along road rights-of-way so as to avoid property acquisition complications.

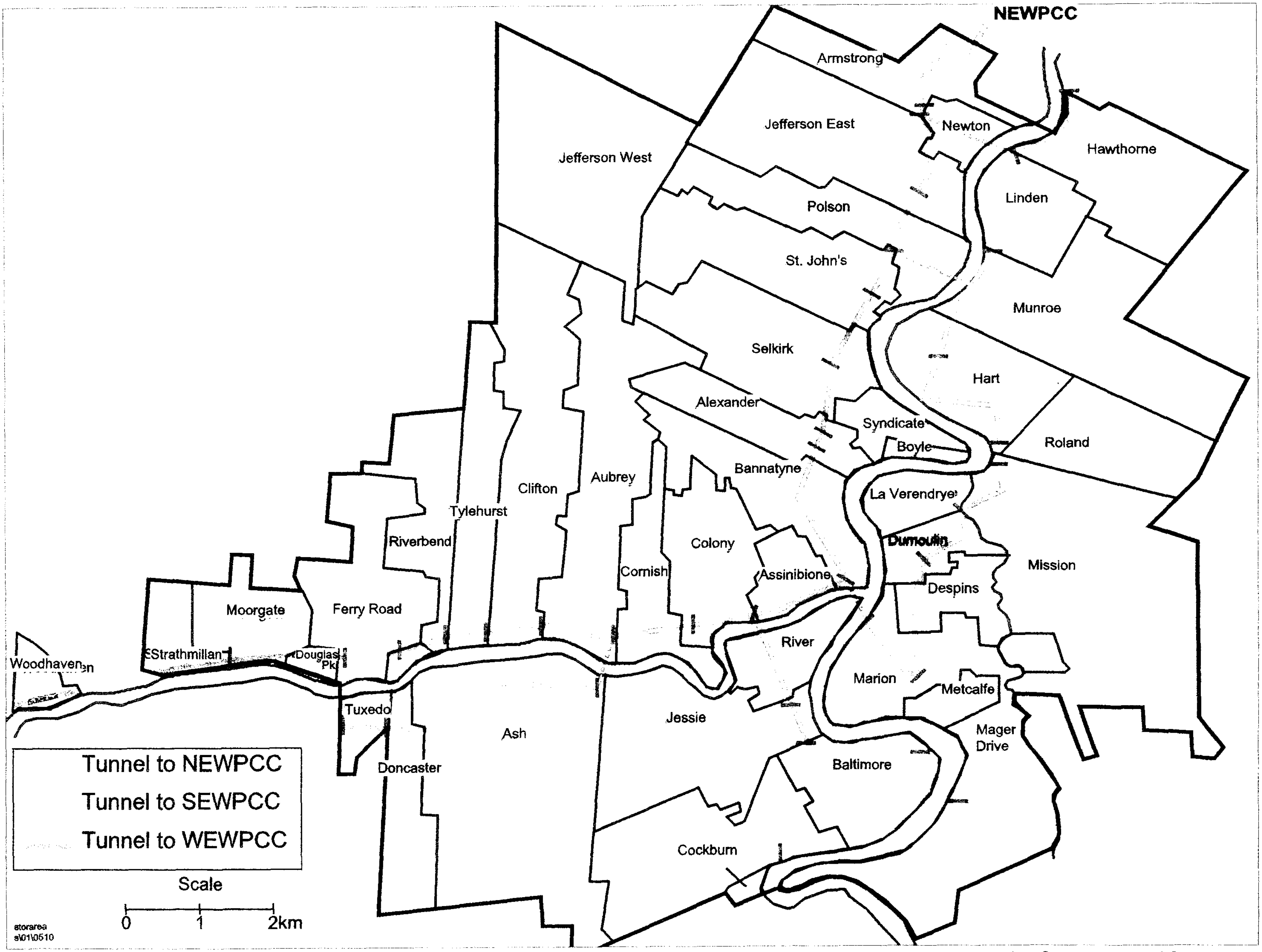
5.4.3 Sizing

The volumes of storage required in order to meet the 0 and 4 overflows per year for the 1992 representative year, were developed in Section 3. For the 0 overflows per year, these amounted to total volumes of 820,000 m^3 for the 600 ML/d option; 610,000 m^3 per year for the 825 ML/d option; and 530,000 m^3 per year for the 1,060 ML/d option. For the 4 overflows per year, the volumes required were 300,000 m^3, 240,000 m^3, and 185,000 m^3 respectively. The total volumes required to be stored, with in-line storage, remain the same; the volumes to be stored in the tunnels was reduced by the volumes which could be stored in-line.

The tunnels were laid out so that runoff could be stored and conveyed, through adjacent districts and finally via the main storage/transport tunnel, to the NEWPCC. Having selected the conceptual lengths for each district and the required storage volume needed, the diameter for each reach was determined. The overall results of are provided in the Appendices. An example (Table 5-21) illustrates the manner in which the calculations were made.



In-Line / Off-Line / Transfer:
Practicable Configuration - 4 Overflows
Figure 5-20



Regional Tunnels Conceptual Layout
Figure 5-21

Table 5-21: REGIONAL TUNNEL - 300,000 m^3
(4 Overflows; Dewater @ 600mL/d)

DISTRICT	LENGTH m	GROUP LENGTH m	CUMULATIVE LENGTH m	DEWATERING RATE m^3/s	STORAGE NEEDED m^3 4 Overflows	GROUP STORAGE m^3 4 Overflows	DIAMETER NEEDED m	DIAMETER USED m	POWER kW	REACH VOLUMES m^3	CUM. VOLUMES m^3	PUMP STATION + FM $M
DIAMETER							3.5	3.6				
FERRY ROAD	780				9070					7939		
RIVERBEND	650				9070					6616		
TYLEHURST	1040				11510					10586		
CLIFTON/AUBREY	2280				29030					23208		
		4750	4750			58680	3.97	3.60		0	46341	
TUXEDO	830			0.033	2790					8448		
DONCASTER/ASH	2900			0.57	32650					29518		
		3730				35440	3.48	3.60		0		
			8480									
LINK			770	0.603			0.62	0.62	79	0		1.7
										0		
CORNISH	970				2510					9873		
COLONY	790				9490					8041		
		1760				12000	2.95	3.60		0		
			10240							0	64256	
JESSIE/RIVER	1730			0.29	21630					17609		
		1730				21630	3.99	4.00		0		
			11970									
LINK			400	0.29			0.43	0.43	31	0		1.1
										0		
ASSINIBOINE	1180				9770					12011		
		1180				9770	3.25	3.60		0		
			13150							0	76267	
MISSION	1280			0.179	10740					13029		
LaVERENDRYE/DUMOULI	1460			0.088	5230					14861		
DESPINS	1240									12622		
MARION	890			0.189	15350					9059		
		4870				31320	2.86	2.90		0		
			18020							0		
LINK			720	0.456			0.54	0.54	58	0		1.5
										0		
BANNATINE	480				7670					4886		
ALEXANDER	1170				10460					11909		
SYNDICATE/BOYLE					5700							
SELKIRK	1160				6980					11807		
ST.JOHN'S	860				17440					8754		
		3670				48250	4.09	3.60		0		
			21690							0	113623	
ROLAND/HART	2130			0.253	21490					21681		
MUNROE	1700			0.241	18140					17304		
		3830				39630	3.63	3.60		0		
			25520							0		
LINK			1150	0.494			0.56	0.56	78	0		1.9
										0		
POLSON	1050				11160					10688		
JEFFERSON/NEWTON	1220				16740					12418		
		2270				27900	3.96	3.60		0		
			27790							0	136729	
HAWTHORNE	1220			0.123	8370					12418		
		1220				8370	2.96	3.00		0		
			29010							0		
LINK			1150	0.123			0.28	0.28	19	0		1.1
										0		
ARMSTRONG/CONNECTO	1780				7400					18118		
		1780				7400	2.30	3.60		0		
										0		
	30790	30790	30790			300390	3.52			300390		
MAIN TUNNEL										313405	154847	
Diameter Required							3.68	3.60				
COCKBURN	1910				11000					11345		
BALTIMORE	1690				10000					10038		
		3600				21000	2.73	2.75		0	21383	
LINK			380	0.091			0.24		10			0.8
METCALFE/MAGER	1840	1840			14500	14500	3.17	3.20		14798		
		5440	5440							349585		
MOORGATE/DOUG. PK	1620				2900							
STRATHMILLAN	1200	2820	2820		875	3775	1.31	1.3		3743		
LINK			1000	0.109			0.26		16			1.0
WOODHAVEN	1000	1000			1900	1900	1.56	1.6		2011		
										355339		
										313405		

TOTAL P.S. COST	$9 M
TOTAL TUNNEL COST	$316 M
TOTAL NET COST	$325 M
TOTAL BUDGET COST (1.44*NET)	$468 M
FLOW CONTOL COST	$12 M
FLUSHING	$17 M
NEWPCC	$15 M
TOTAL	$511 M

The tunnels for the lateral collectors, that is on the south side of the Assiniboine and the east side of the Red River, were sized to provide the volumes needed for each sub-grouping as provided on the Tables. The size of the main storage/transport tunnel on the north side of the Assiniboine and west side of the Red was kept uniform throughout its length. It would thus act as a massive reservoir whose capacity would only be totally consumed if the design storm happened uniformly across the whole CS district, i.e., the system would have significant cushion for a non-uniform distribution of rainfall.

The system was laid out such that the storage for the satellite tunnel collectors would be discharged through the main tunnel collector at the combined dewatering rate for those districts. The dewatering rates for each of the three scenarios are also provided on Table 5-11 through 5-13.

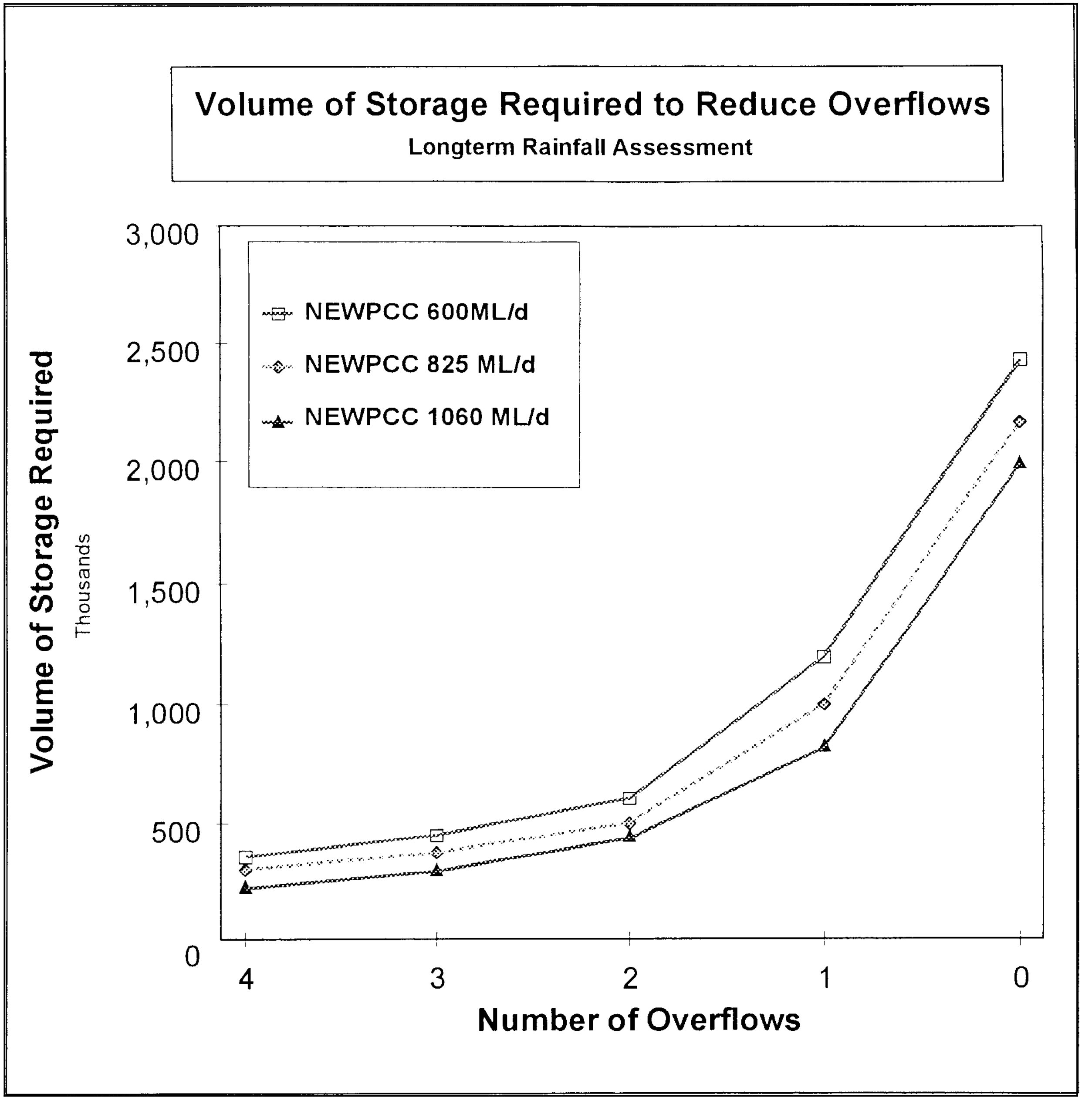
During the course of the evaluation of options comparing the relative performances of control facilities sized for the 1992 representative year vis a vis the long-term average, it was found that there were some discrepancies. For 4 overflows per year, on the basis of the 1992 year, the results closely related to long-term average. However, for the performance target of 0 overflows per year, on the 1992 basis, the result more closely represented the required infrastructure for 2 overflows per year on the long-term basis. Figure 5-22 shows the results of the analysis for the required storage in order to address 0 to 4 for overflows per year on the long-term average.

Also included on the Figure is the degree to which the number of overflows must be reduced on the long-term basis in order to approach 85% capture. Under all conditions of dewatering this amounts to 1 overflow per year. The volumes required to meet the target of 0 overflows per year on the long-term average (2 million to 2.4 million m^3) were used to make a crude assessment of the size of regional tunnel which would be needed in order to achieve this result. As discussed below, this sizing was used to obtain an approximation of the cost of such a regional facility.

The main tunnel diameters, as developed for the regional tunnels for all scenarios, have been provide on Table 5-22.

Number of Overflows	Volume of Storage Required		
	600 ML/d	825 ML/d	1060 ML/d
4	362,000	312,000	238,000
3	450,000	375,000	312,000
2	600,000	500,000	450,000
1	**1,200,000**	**1,000,000**	**825,000**
0	**2,438,000**	**2,175,000**	**2,000,000**

Number of Overflows	Median % Capture		
	600 ML/d	825 ML/d	1060 ML/d
4			
3			
2			
1	**84.0%**	**84.6%**	**84.7%**
0	**100%**	**100%**	**100%**



storreqd
s\01\0510

Figure 5-22

5.4.4 Operational Considerations

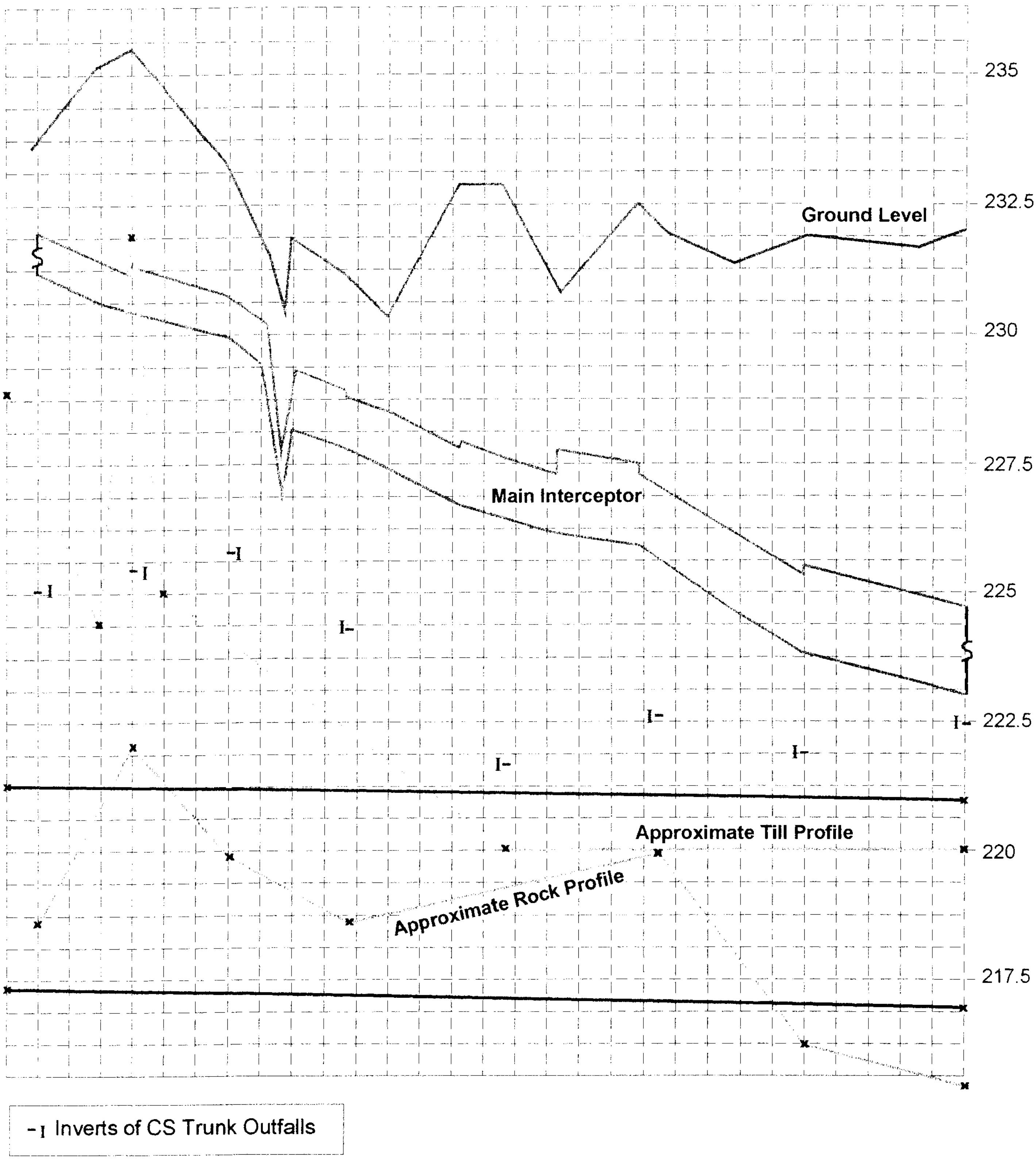
In developing the profiles for the regional tunnels, it had been the intent that the tunnels would generate self-cleaning velocities (1 m /s) at the full dewatering rate, and that the tunnel profile would be so located that the existing NEWPCC pumps could dewater the tunnel. This concept is illustrated on Figure 5-23 and 5-24. Since the collector tunnels would normally be dewatered at less than a flushing rate, flushing systems, similar to those described in Section 5.3.3, would also be installed. Accordingly, operation and maintenance of the regional tunnel storage system should be minimal.

5.4.5 Costing

The basic tunnel costs were obtained from the Gore & Storrie cost-estimating curve as prepared for Phase 2 (included here as Figure 5-25). CG&S indicate that the construction costs have not changed significantly in the interim and that the cost curves are still valid.

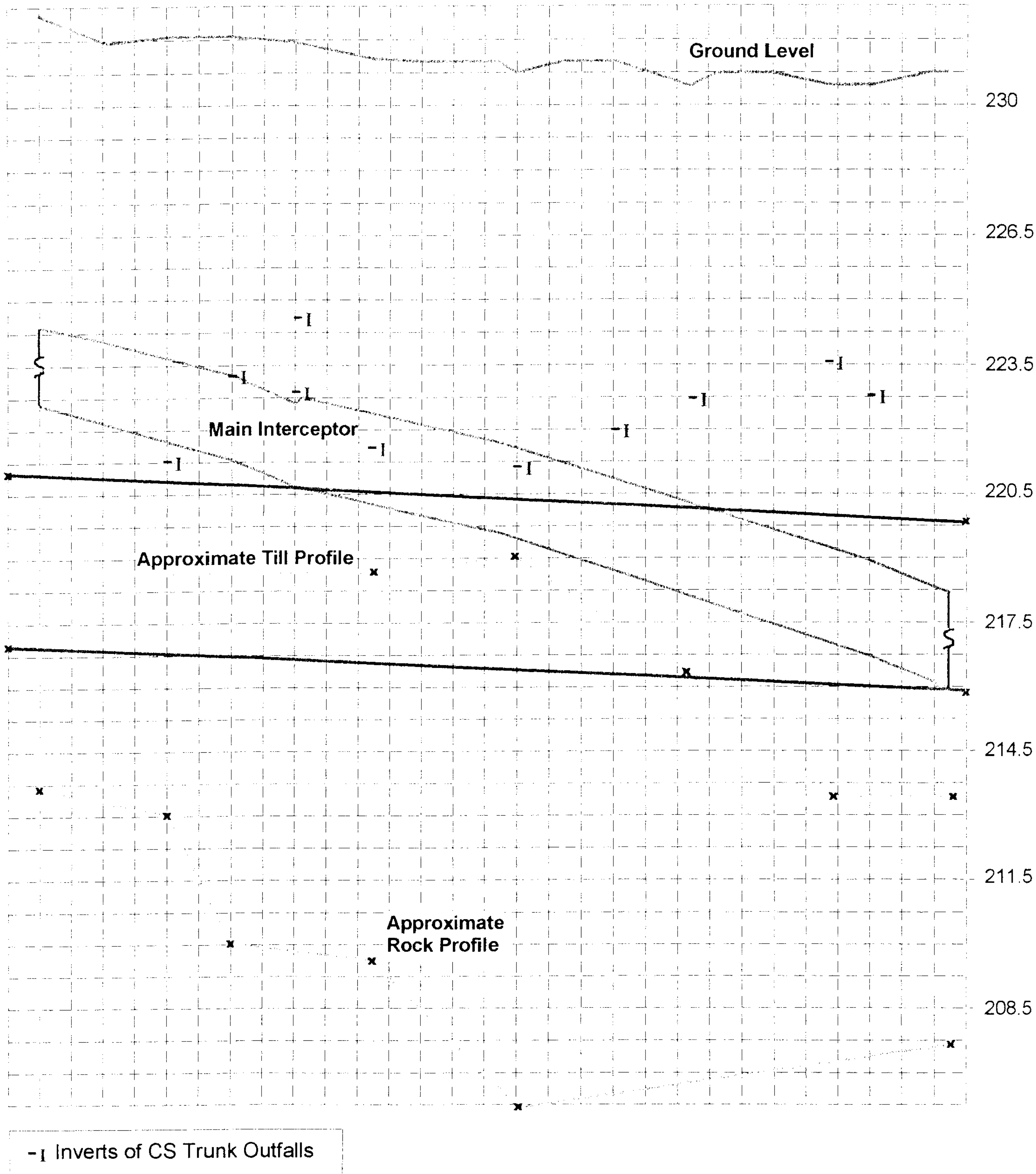
A preliminary assessment of bedrock profile and till profile was taken from available regional plans and profiles ("Geological Engineering Maps and Report for Urban Development of Winnipeg" 1983). This information is plotted on Figures 5-23 and 5-24, along with the corresponding invert for a 4-metre diameter trunk tunnel. As can be seen, the tunnel could be dewatered by the existing NEWPCC pumping station and could be installed so as not to interfere with the inverts of the existing CS trunks (also plotted on the diagram).

The original cost estimate for the regional tunnels was made prior to Working Session 3-6, held on October 7, 1997. The estimates were based on the assumption that mixed face tunnels (then defined as being tunnels in the interface between rock and till profiles) would be more expensive. An allowance of 50% over the rates as shown on Figure 5-25 was used to allow for the increased difficulties. At the Working Session it was noted that tunnels in the mixed face between till and clay would be just as difficult to construct as those between rock and till. Accordingly, it was decided that the regional tunnels would have to be lowered so as to be entirely within the rock.



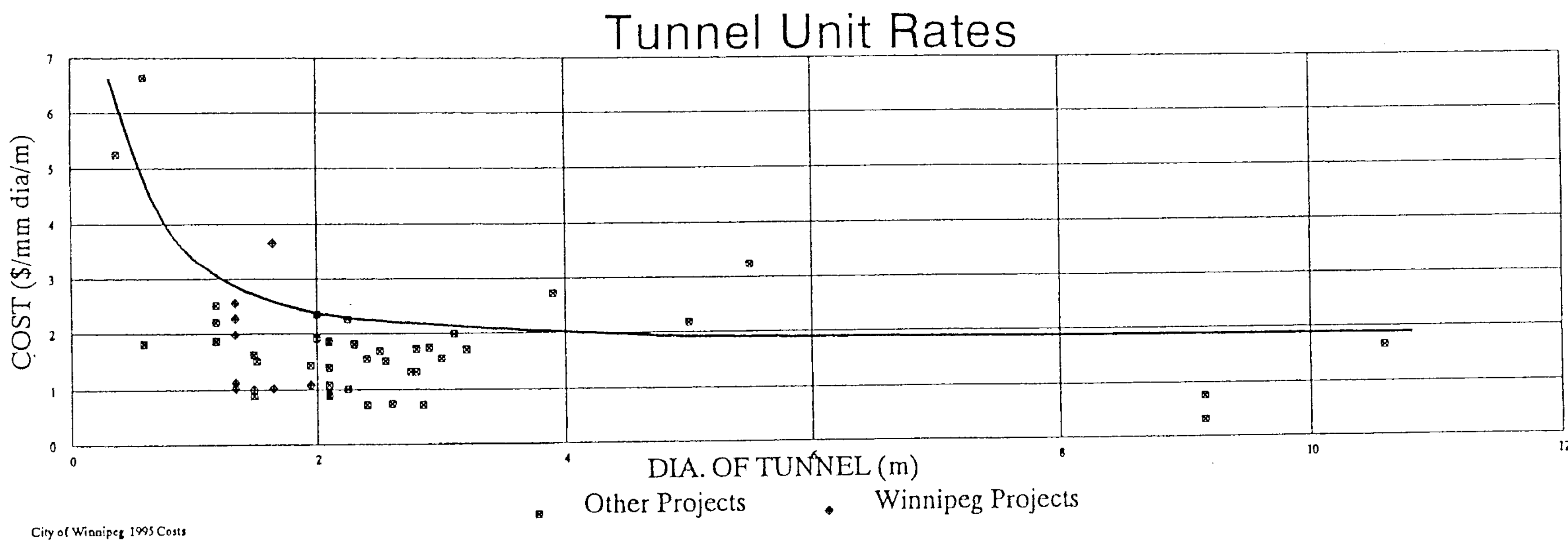
Assiniboine River
Approximate Rock / Till Profile
Figure 5-23

sewrprof
s\01\0510



Main Street (Red River)
Approx. Rock \ Till Profile
Figure 5-24

sewrprof2
s\01\0510



Tunnel Unit Rates
COST ($/mm dia/m)
DIA. OF TUNNEL (m)
Other Projects
Winnipeg Projects
0
1
2
3
4
5
6
7
0
2
4
6
8
10
12
City of Winnipeg 1995 Costs

Figure 5-25

CG&S, who prepared Figure 5-25, indicated that the costs of tunnel in bedrock would not differ significantly from the unit rates shown in that figure, for the larger tunnels, that is, 3 metres and above. Accordingly, it was decided not to re-estimate the regional tunnels on the basis that the additional costs included for the earlier definition of mixed face would probably more than cover the additional costs associated with the deeper tunnel. The latter would likely be more elaborate drop shafts as well as a major pumping station at the NEWPCC.

In accordance with the foregoing, the costs of the regional tunnels are summarized in Table 5-22a. The base costs in this case include an allowance of 20% for estimating contingencies and 20% for engineering, administration and finance. The only other limitation in the development of the costs is that the minimum size of tunnel used was 1.5 metres.

As can be seen from Table 5-22a, there is a significant reduction in cost between all the options for 0 overflows for the 1992 representative year and all the options for 4 overflows for the 1992 representative year. The difference is not so marked between most of the options without in-line storage and most of the options with in-line storage, although there is still a savings to be made.

As noted above, the performance of 0 overflows per year, on the 1992 basis, is not a suitable surrogate for the long-term average. Accordingly, the volumes of storage required for 0 overflows on the long-term basis were projected. These were listed on Figure 5-22 for 0 and 1 overflow. Using the same length of tunnel as for the other regional options (40,000 metres) the diameters needed to provide these volumes of storage were calculated. The results have been listed in Table 5-23. The projected costs (excluding any allowances for pumping or NEWPCC expansion) are listed on the table.

Table 5-22a: REGIONAL TUNNEL - COST SUMMARY

1992 REPRESENTATIVE YEAR

	WITH IN-LINE STORAGE			WITHOUT IN-LINE STORAGE		
DEWTR RATE	600 ML/d	825 ML/d	1060 ML/d	600 ML/d	825 ML/d	1060 ML/d
0 OVRFLOWS	$M	$M	$M	$M	$M	$M
STRGE VOL.	820,000 m^3	610,000 m^3	530,000 m^3	820,000 m^3	610,000 m^3	530,000 m^3
BASE COST*	565	479	406	676	612	600
FLO CNTROL				12	12	12
IN-LN STRGE	100	100	100			
FLUSHING	23	20	16	27	24	22
NEWPCC	15	36	70	15	36	70
TOTAL 0 O/F	$703M	$635M	$592	$729M	$684M	$704M
4 OVRFLOWS						
STRGE VOL.	300,000 m^3	220,000 m^3	185,000 m^3	300,000 m^3	220,000 m^3	185,000 m^3
BASE COST*	288	264	263	468	416	377
FLO CNTROL				12	12	12
IN-LN STRGE	100	100	100			
FLUSHING	11	10	11	17	16	14
NEWPCC	15	36	70	15	36	70
TOTAL 4 O/F	$414M	$410M	$444M	$512M	$479M	$473M

* BASE COSTS INCLUDE MULTIPLIERS

regcost.wpd

TABLE 5-22

REGIONAL TUNNELS
MAIN TUNNEL DIAMETERS

OPTIONS	DIAMETER (m)
"1992 REPRESENTATIVE YEAR"	
0 OVERFLOW	
w/o in-line	
• 600 ML/d	5.8
• 825 ML/d	5.0
• 1,060 ML/d	4.4
w/in-line	
• 600 ML/d	4.5
• 825 ML/d	3.4
• 1,060 ML/d	2.6
4 OVERFLOWS	
w/o in-line	
• 600 ML/d	3.6
• 825 ML/d	3.1
• 1,060 ML/d	2.7
w/in-line	
• 600 ML/d	1.8
• 825 ML/d	1.5
• 1,060 ML/d	1.5
LONG TERM	
1 OVERFLOW	
• 600 Ml/d	6.1
• 825 ML/d	5.6
• 1,060 ML/d	5.1
0 OVERFLOW	
• 600 Ml/d	8.8
• 825 ML/d	8.4
• 1,060 ML/d	8

TABLE 5-23

TUNNELS REQUIRED FOR 0-1 OVERFLOWS, LONG TERM*

NUMBER OF OVERFLOWS	VOLUME (m^3)	DIAMETER (m)	ESTIMATED BASE COST $M
1 OVERFLOW			
• 600 ML/d	1,200,000	1 @ 6.1	685
• 825 ML/d	1,000,000	1 @ 5.6	630
• 1,060 ML/d	825,000	1 @ 5.1	575
0 OVERFLOW			
• 600 ML/d	2,438,000	1 @ 8.8	990
• 825 ML/d	2,175,000	1 @ 8.35	950
• 1,060 ML/d	2,000,000	1 @ 8	900

*Tunnel Length = 40,000 m

5.5 HIGH-RATE TREATMENT

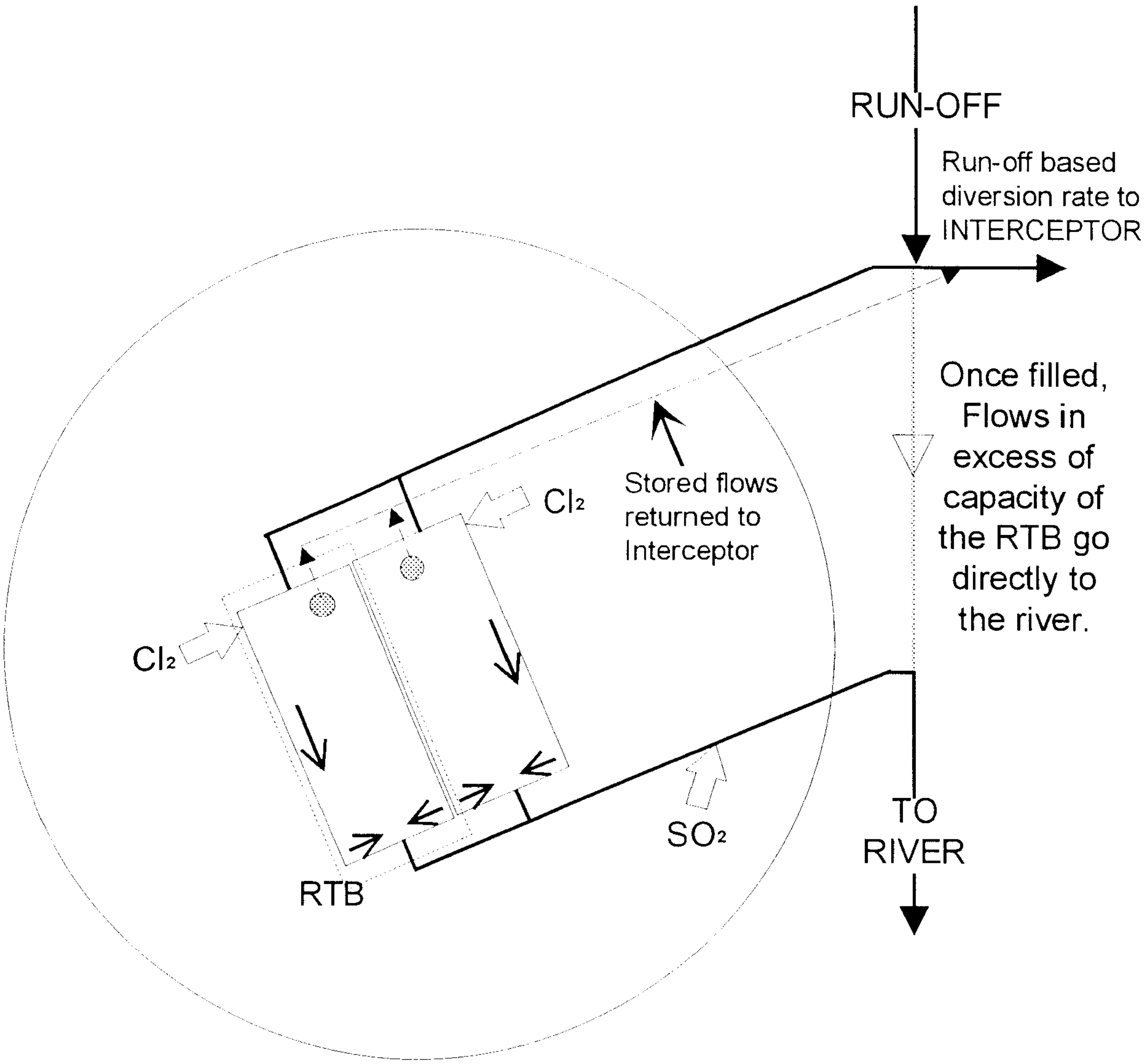
5.5.1 Conceptual Considerations

The high-rate treatment options comprise either Vortex Solid Separators (VSS), or retention treatment basins (RTBs) (a combination storage and high-rate sedimentation tank). The VSS are high-rate sedimentation devices whose prime purpose is to remove solids to the point where disinfection, likely chlorination, can be effected in order to reduce fecal coliform bacteria. In the case of the RTB, as with the storage basins, the volume of combined sewage up to the storage capacity of the RTB would be returned to the interceptor and thence the treatment plant for further treatment. The remainder of the flow, up to the capacity of the RTB to act as a sedimentation tank to allow effective disinfection, would be disinfected and then discharged directly to the river. The combined sewage flows in excess of the RTB capacity would discharge directly to the river undisinfected. As with the storage tanks, flows stored in the interceptor, at a sufficient depth to permit pumping, would have to be pumped from the combined sewer trunks to the treatment facility at the rated capacity of the device. The operation of the RTB is shown schematically on Figure 5-26.

5.5.2 Siting Considerations

Both high-rate treatment devices considered could be either near-surface or above-surface facilities, the only difference being a matter of cost or aesthetics. From a cost perspective, the difference relates to whether the tankage would be reinforced in order to support exterior ground pressure or whether the devices would be housed in aesthetically-pleasing facilities in keeping with the land use. In either event, the devices are similar to the off-line storage tanks in that they require sufficient land to accommodate them.

The high-rate devices would have similar difficulties with odour as the near-surface storage facilities. These could be addressed in a similar fashion through the use of air scrubbers, as discussed above. In addition to this concern, both high-rate devices would entail the storage and handling of chemicals: probably sodium-hypochlorite (liquid) for disinfection, and sodium-



Schematic of
Retention/Treatment Basins (RTB)

Figure 5-26

rtbonly
s\01\0510

metabisulphite (powder or liquid) for dechlorination before discharge to the rivers. In themselves, neither the storage nor the handling of these chemicals should present any serious hazards. Notwithstanding their stability, however, they will be located generally in or near residential areas. The neighbours may perceive the use of these chemicals in their area as being undesirable.

The odour control and chemical storage and handling facilities would likely be aboveground.

5.5.3 Operation Concerns

Because of the chemical addition, the operation of these high-rate facilities will be somewhat more complex and demanding than would be the operation of the near-surface or local tunnel storage. This difference in complexity would likely be reflected in more frequent, routine inspection visits and would likewise be reflected in O&M costs.

5.5.4 Treatment Effectiveness

During the course of Phase 3, a CSO treatability evaluation was undertaken on the Aubrey District during WWF to determine the effectiveness of the high-rate treatment options. The following is a summary of the results of that evaluation. The full report is contained in Appendix __, under the title "The City of Winnipeg, CSO Treatability Evaluation" and dated December 23, 1996.

- CSO Characterization Results

  The study found that the CSO quality observed at Aubrey generally compared with the results from other studies. The concentrations of BOD, nitrogen and phosphorus were found to be near the lower end of the range in other studies, while TSS and bacteriological results were comparable.

- CSO Treatability

The tests run on the samples simulated treatment effectiveness of various schemes by varying the settling time for the sample. The 5 minute settling was used to simulate a Vortex separator under design hydraulic loading while the 50 minute settling time was used to simulate conventional sedimentation basin performance. Tests were also run using a 50-minute settling aided with chemical addition in order to evaluate enhanced sedimentation.

- Particulate Solids Characterization

The high-rate treatment evaluation was carried out through column testing as described in the report. The results are shown on **Figure** 9 from the report entitled "Aubrey CSO - Settleable Solids Curves". The three Aubrey CSO settling curves are quite similar, with a median velocity of about 0.3 cm/s. The report notes that a large fraction of relatively light material, with settling velocities between 0.1 cm/s and 0.6 cm/s, is indicated by the steep slope of the curve. A second curve based upon 50 experiments carried out throughout the United States is shown on Figure 9 for reference. The report notes that approximately 90% of the material had settling velocities of less than 0.5 cm/s. This represents a majority of 'very fine' solids fraction. The report notes that this material would likely be too light for effective solids removal with a Vortex solids separator, and concludes that conventional sedimentation technology, at the lower hydraulic loading rates coupled with screening for floatables, would be more effective for solids/floatables control at Aubrey.

- Disinfection Studies

The purpose of the disinfection studies was to establish the dose response relationships (bacteria reduction versus disinfectant dose and contact time) for both sodium hypochlorite and ultra-violet light irradiation. In both cases, it was assumed that a two-stage process would be followed:

- first stage
    - treatment by either physical or physical-chemical means;
- second stage

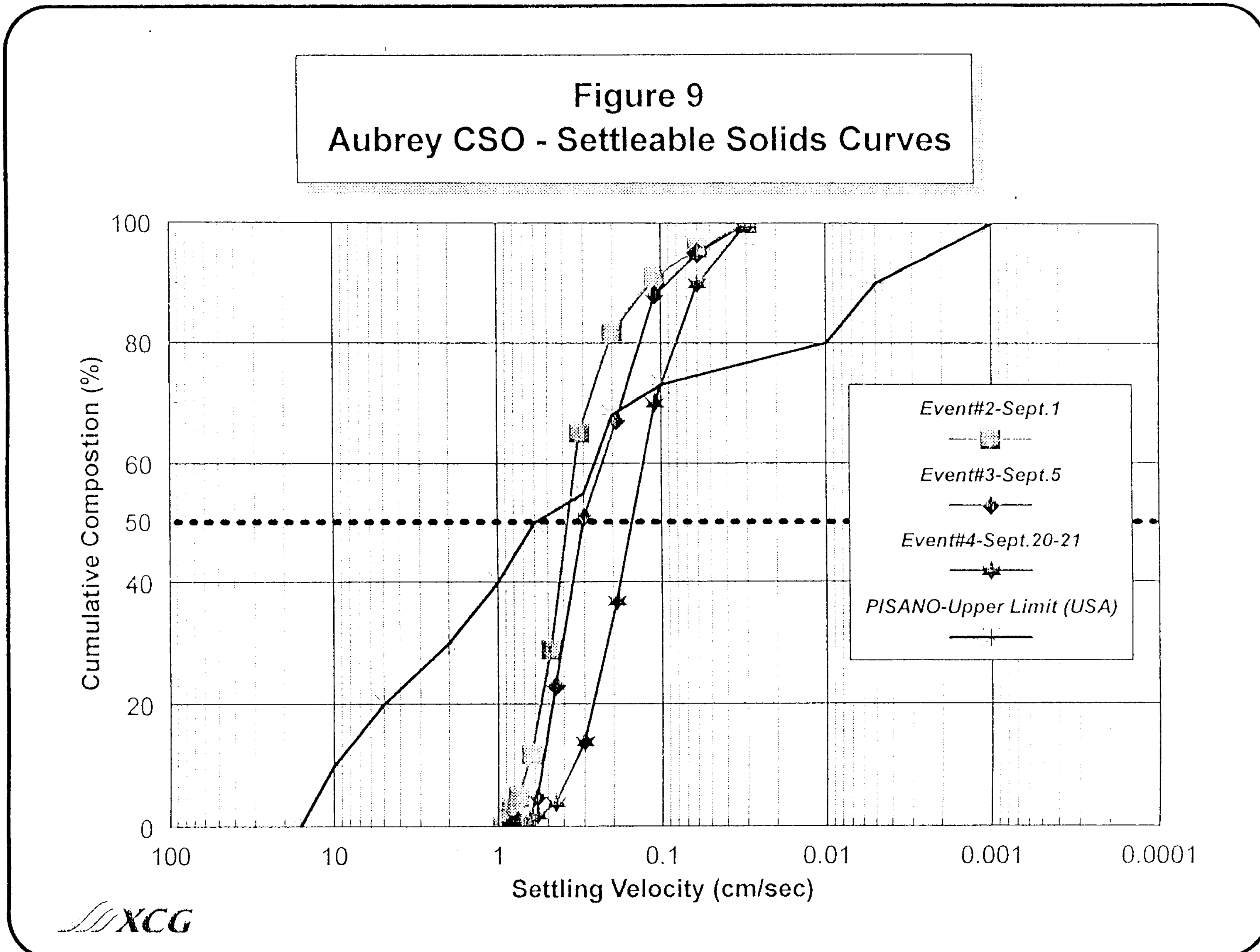


Figure 9
Aubrey CSO - Settleable Solids Curves
Cumulative Compostion (%)
100
80
60
50
40
20
0
100
10
1
0.1
0.01
0.001
0.0001
Settling Velocity (cm/sec)
Event#2-Sept.1
Event#3-Sept.5
Event#4-Sept.20-21
PISANO-Upper Limit (USA)
XCG

- disinfection in a second vessel with hypochlorite or ultra-violet light irradiation.

The results of these disinfection studies are summarized as follows:

- UV dosage requirements to achieve target reduction are within the same range (20 to 40 mWs/cm^2) used in secondary effluent applications and may be an effective technology to treat Aubrey CSO;
- dose-response experiments for chlorination of CSO indicated the following TRCs (total residual chlorine) and Cl_2 dosages at the contact times indicated.

EMULATED PROCESS	**WASTEWATER**	**CONTACT TIME**	**TRC (mg/L as Cl_2)**	**Cl_2 Dosage (mg/L)**
Vortex Separator	5 minute settled	2	27	28
		20	12	15
Conventional Sedimentation Basin	50 minute settled	2	>40	-
		7	17	20
		20	4.0	5.0
	Chemically Enhanced	2	4.0	5.0
	50 minute settled	20	2.0	3.0

- Treatability Conclusions

The report concludes that the Vortex separator technology would not be appropriate for the Aubrey CSO application. It was noted that the absence of heavier grit in the Aubrey CSO could have been associated with the particular site configuration found at Aubrey. Because of site constraints, the samples had to be taken downstream of the diversion weir and could not be confirmed by taking a manual sample collection upstream of weir was not possible. It was believed that the addition of this sample for analysis might have provided a more complete indication of the general CSO solids characteristic if an alternative diversion configuration was present. There was a concern that the diversion weir might have inhibited heavier grit from overflowing.

In addition to the above concern, there is also a question as to whether or not the Aubrey CSO could be inferred as being typical of the quality of Winnipeg CSOs in general. The only way to determine whether or not this was the case would be to carry out tests on other combined sewer outfalls. If high rate treatment devices are considered to be a viable option for CSO control in the City of Winnipeg, further testing, on at least one other CS outfall, will be required.

5.5.5 Sizing and Costing

The cost of these two devices, as carried out for the Phase 2 analysis, was based on an overflow rate of 10 m/hr for the Vortex solids separator and 4 m/hr for retention treatment basin. The indications of the treatability evaluation are that, if the Aubrey CSO is representative of Winnipeg-wide CSO characteristics, the Vortex solid separator technology would be unsuitable for the Winnipeg situation. Rather than developing costs for the high-rate technology for both these alternatives, and in the absence of additional information on the suitability of the VSS for Winnipeg, it was decided to develop conceptual costs for the RTB.

A spreadsheet model was developed and run to simulate the RTBs which would be required to effectively accommodate the volumes of storage/treatment for the 825 ML/d options for 0 and 4 overflows per recreation season for the 1992 representative year. The sizing was based on a 10-metre/hr overflow rate and the general configuration of the tank units was 20 m x 50 m x 5 m deep. As with the near-surface storage, the availability of public lands provided constraints as to which districts could accommodate RTBs. Also, as with the near-surface storage, the ancillary tunnel storage required was based on the tunnel groupings developed for the off-line storage scenarios. The results of the analyses are provided in Tables 5-24 and 5-25.

The RTB systems were designed using the runoff generated for each district for the representative year (1992). This was combined with the dry weather flow to obtain an hourly record of the flow rates expected at the end of the pipe for each district. It was assumed that the RTB storage would be full when the peak for the design storm arrived. By screening through the data for each district, the largest hourly peak rate for any storm was determined along with the maximum peak rate for the fifth largest storm. Using these two criteria would allow the systems to be designed for either no untreated overflows or four untreated overflows. These relate to the targets described earlier in Section 3.

Table 5-24: RTB - 220,000 m³ (4 Overflows; Dewater @ 825mL/d)

DISTRICT	STORAGE NEEDED m³ 4 Overflows	ROUP	OFFLINE UNITS AVAILABLE (5000 m³)	OFFLINE UNITS USED (5000 m³)	TUNNEL DIAMETER USED m	TUNNEL LENGTH NEEDED m	TUNNEL VOLUME m³	PUMP CAPACITY NEEDED (tanks)	PIPE DIAMETER (2m/s) m	PIPE LENGTH m	POWER kW	COST TANKS $M	COST TUNNELS $M	COST PUMP $M	COST PUMP STNS+F.M. $M	TOTAL FLOW THROUGH m³	MAXIMUM TREATMENT RATE m^3/hr	MAXIMUM TREATMENT RATE ML/d	CAPITAL COST Cl2/deCl2 $M	CHEMICAL COST Cl2/deCl2 $M
DIAMETER					3.6															
FERRY ROAD	5000		8	1.00		0		1.6	1.0	400	141	3.3		1.5	2.0	75,743	10,000	240	1.0	
RIVERBEND	6500	A	0		2.62	6910	37254		0.0				39	0.0	0.2					
TYLEHURST	8250	A	0						0.0				0	0.0	0.2					
CLIFTON	8125	A	6	1.63		0		2.1	1.2	450	185	3.7	0	1.8	2.4	148,656	16,250	390	1.6	
AUBREY	6800	A	0						0.0				0	0.0	0.2					
									0.0				0	0.0						
TUXEDO	2000	B	0		2.94	3730	25322		0.0				23	0.0	0.2					
DONCASTER	1400	B	0						0.0				0	0.0	0.2					
ASH	22000	B	0						0.0				0	0.0	0.2					
									0.0				0	0.0						
CORNISH	1800	A	0						0.0				0	0.0	0.2					
COLONY	6800	A	0						0.0				0	0.0	0.2					
									0.0				0	0.0						
JESSIE	8500	C	5	1.70		0		2.3	1.2	400	203	3.7	0	1.9	2.5	139,572	17,000	408	1.7	
RIVER	0	C	1	0.00		0		0.0	0.0				0	0.0	0.2					
									0.0				0	0.0						
ASSINIBOINE	7000	A	0						0.0				0	0.0	0.2					
									0.0				0	0.0						
MISSION/ROLAND	11250	D	7	2.25		0		3.5	1.5	650	309	4.2	0	2.6	3.5	205,400	22,500	540	2.3	
LaVERENDRYE/DUMOULI	1250	D	1	0.25		0		0.9	0.8	150	79	1.6	0	1.2	1.4	15,133	2,500	60	0.3	
DESPINS		D							0.0				0	0.0	0.2			0		
MARION	6500	D	7	1.30		0		1.8	1.1	250	159	3.3	0	1.6	2.0	62,634	13,000	312	1.3	
									0.0				0	0.0				0		
BANNATINE	4000	E	3	0.80		0		1.2	0.9	300	106	3.2	0	1.3	1.7	69,987	8,000	192	0.8	
ALEXANDER	5875	E	3	1.18		0		1.3	0.9	60	115	3.2	0	1.3	1.5	89,736	11,750	282	1.2	
SYNDICATE/BOYLE	3125	E	1	0.63		0		0.8	0.7	200	71	3.0	0	1.0	1.3	27,060	6,250	150	0.6	
SELKIRK	3375	E	4	0.68		0		1.2	0.9	200	106	3.2	0	1.3	1.6	62,719	6,750	162	0.7	
ST.JOHN'S	9375	E	6	1.88		0		2.4	1.2	80	212	3.9	0	1.9	2.2	152,304	18,750	450	1.9	
									0.0				0	0.0					0.0	
HART/MUNROE	13500		4	2.70		0		3.7	1.5	700	326	5.0	0	2.7	3.7	243,989	27,000	648	2.7	
						0			0.0				0	0.0						
POLSON	5625	E	3	1.13		0		1.6	1.0	500	141	3.3	0	1.5	2.0	96,188	11,250	270	1.1	
JEFFERSON/NEWTON	12000		0		3.60	1179	12000		0.0				9	0.0	0.2					
									0.0				0	0.0						
HAWTHORNE	6000		0		3.60	589	6000		0.0				4	0.0	0.2					
													0	0.0						
ARMSTRONG/CONNECTO	2125		3	0.43		0		0.8	0.7	150	71	2.6	0	1.0	1.3	26,743	4,250	102	0.4	
													0							
							80576													
	168175		54	18																
TUNNEL VOL. SUPP.						0														
TANKS VOL. SUPP.				87625																
MAIN TUNNEL								25												
Diameter Required					3															
COCKBURN	3875	F	5	0.775		0		1.3	0.9	750	197	3.3	0	1.8	2.5	120,759	7,750	186	0.8	
BALTIMORE	3125	F	3	0.625		0		1	0.8	500	152	3.0	0	1.6	2.1	64,098	6,250	150	0.6	
														0.0						
METCALFE/MAGER	4625		2	0.925		0		1.5	1.0	900	147	3.4	0	1.5	2.3	88,056	9,250	222	0.9	
														0.0						
														0.0						
MOORGATE/DOUG.PARK	2900	G			1.55	2000	3774						8	0.0	0.2					
														0.0						
STRATHMILLAN	875	G											0	0.0	0.2					
														0.0						
WOODHAVEN	1900					269							2	0.0	0.2					
												$57	$85		$39	1,688,777	198,500	4,764	$20	$2
								25												

TOTAL P.S. COST	$39 M
TOTAL TUNNEL COST	$85 M
TOTAL TANK COST	$57 M
TOTAL NET COST	$181 M
TOTAL BUDGET COST (1.58*NET)	$287 M
+ FLOW CONTROL	$12 M
+ IN-LINE STORAGE	
+ INTERCEPTOR	$15 M
+ NEWPCC	$36 M
+ FLUSHING	$18 M
+ DISINFECTION (CAP. + O&M)	$22 M
TOTAL ESTIMATED COST	$389 M

Table 5-25: RTB - 610,000 m³ (0 Overflows; Dewater @ 825mL/d)

DISTRICT	STORAGE NEEDED m³ 0 Overflows	ROU	OFFLINE UNITS AVAILABLE (5000 m³)	OFFLINE UNITS USED (5000 m³)	TUNNEL DIAMETER USED m	TUNNEL LENGTH NEEDED m	TUNNEL VOLUME m³	PUMP CAPACITY NEEDED (tanks)	PIPE DIAMETER (2m/s) m	PIPE LENGTH m	POWER kW	COST TANKS $M	COST TUNNELS $M	COST PUMP $M	COST PUMP STNS+F.M. $M	TOTAL FLOW THROUGH m^3	MAXIMUM REATMENT RATE m^3/hr	MAXIMUM REATMENT RATE ML/d	CAPITAL COST Cl2/deCl2 $M	CHEMICAL COST Cl2/deCl2 $M
DIAMETER					4															
FERRY ROAD	11250		6	2.3		0		1.6	1.0	400	141	4.3		1.5	2.0	83,057	22,500	540	2.3	
RIVERBEND	18000	A	0	0.0	4.30	6910	100347		0.0				58	0.0	0.2					
TYLEHURST	20000	A	0	0.0					0.0				0	0.0	0.2					
CLIFTON	20000	A	6	4.0				2.1	1.2	450	185	6.2		1.8	2.4	162,906	40,000	960	4.0	
AUBREY	22000	A	0						0.0				0	0.0	0.2					
									0.0					0.0						
TUXEDO	6000	B	0		5.00	3880	76184		0.0				38	0.0	0.2					
DONCASTER	5000	B	0						0.0				0	0.0	0.2					
ASH	65000	B	0						0.0				0	0.0	0.2					
									0.0					0.0						
CORNISH	5600	A	0						0.0				0	0.0	0.2					
COLONY	21000	A	0						0.0				0	0.0	0.2					
									0.0					0.0						
JESSIE	18750	C	5	3.8		0		2.3	1.2	400	203	6.0	0	1.9	2.5	152,475	37,500	900	3.8	
RIVER	4000	C	1	0.0	4.00	318	4000	0.0	0.0	0	0	0.0	3	0.0	0.2					
									0.0					0.0						
ASSINIBOINE	12000	A	0						0.0				0	0.0	0.2					
									0.0					0.0						
MISSION/ROLAND	26250	D	7	5.3		0		3.5	1.5	650	309	7.4	0	2.6	3.5	226,120	52,500	1260	5.3	
LaVERENDRYE/DUMOULI	3125	D	1	0.6		0		0.9	0.8	150	79	3.0	0	1.2	1.4	17,370	6,250	150	0.6	
DESPINS		D				0			0.0				0	0.0	0.2			0		
MARION	18750	D	7	3.8		0		1.8	1.1	250	159	6.0	0	1.6	2.0	78,402	37,500	900	3.8	
						0			0.0				0	0.0				0		
BANNATYNE	9375	E	3	1.9		0		1.2	0.9	300	106	3.9	0	1.3	1.7	76,904	18,750	450	1.9	
ALEXANDER	12500	E	3	2.5		0		1.3	0.9	60	115	4.5	0	1.3	1.5	97,778	25,000	600	2.5	
SYNDICATE/BOYLE	7500	E	1	1.5		0		0.8	0.7	200	71	3.5	0	1.0	1.3	27,762	15,000	360	1.5	
SELKIRK	8125	E	4	1.6		0		1.2	0.9	200	106	3.6	0	1.3	1.6	68,747	16,250	390	1.6	
ST.JOHN'S	21250	E	6	4.3		0		2.4	1.2	80	212	6.4	0	1.9	2.2	166,374	42,500	1020	4.3	
									0.0					0.0					0.0	
HART/MUNROE	31250		4	4.0	4.00	895	11250	3.7	1.5	700	326	8.4	7	2.7	3.7	263,908	40,000	960	4.0	
									0.0					0.0						
POLSON	13750	E	3	2.8		0		1.6	1.0	500	141	4.8	0	1.5	2.0	105,460	27,500	660	2.8	
JEFFERSON/NEWTON	42000		0		4.00	3342	42000		0.0				27	0.0	0.2					
									0.0					0.0						
HAWTHORNE	18500		0		4.00	1472	18500		0.0				12	0.0	0.2					
														0.0						
ARMSTRONG/CONNECTO	6250		3	1.3		0		0.8	0.7	150	71	3.3	0	1.0	1.3	30,381	12,500	300	1.3	
	447225		48	39.375		16818														
TUNNEL VOL. SUPP.							252281													
TANKS VOL. SUPP.				196875																
MAIN TUNNEL								25												
Diameter Required					4															
COCKBURN	12500	F	5	2.5		0		1.3	0.9	750	197	4.5	0	1.8	2.5	126,770	25,000	600	2.5	
BALTIMORE	7500	F	3	1.5		0		1	0.8	500	152	3.5	0	1.6	2.1	69,277	15,000	360	1.5	
														0.0						
METCALFE/MAGER	11250		2	2	4	99	1250	1.5	1.0	900	147	4.3	1	1.5	2.3	96,050	20,000	480	2.0	
														0.0						
														0.0						
MOOGATE/DOUG.PARK	11000	G			3.1	2000							13	0.0	0.2					
														0.0						
STRATHMILLAN	4000	G											0	0.0	0.2					
														0.0						
WOODHAVEN	5800					462							4	0.0	0.2					
												$84	$162		$39	1,849,741	453,750	10,890	$46	$2

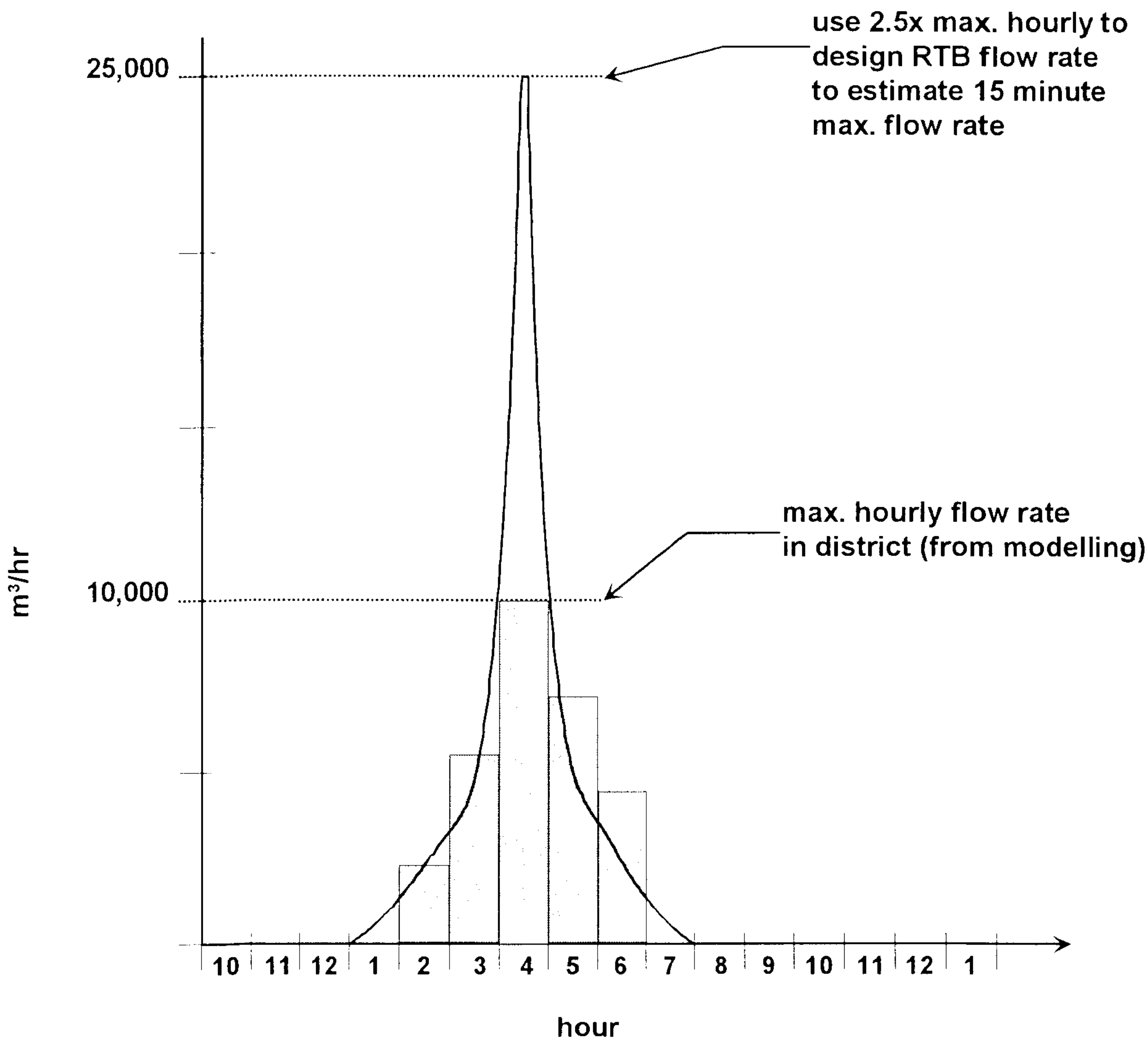
TOTAL P.S. COST	$39 M
TOTAL TUNNEL COST	$162 M
TOTAL TANK COST	$84 M
TOTAL NET COST	$284 M
TOTAL BUDGET COST (1.58*NET)	$449 M
+ FLOW CONTROL	$12 M
+ IN-LINE STORAGE	
+ INTERCEPTOR	$15 M
+ NEWPCC	$36 M
+ FLUSHING	$35 M
+ DISINFECTION (CAP. + O&M)	$48 M
TOTAL ESTIMATED COST	$595 M

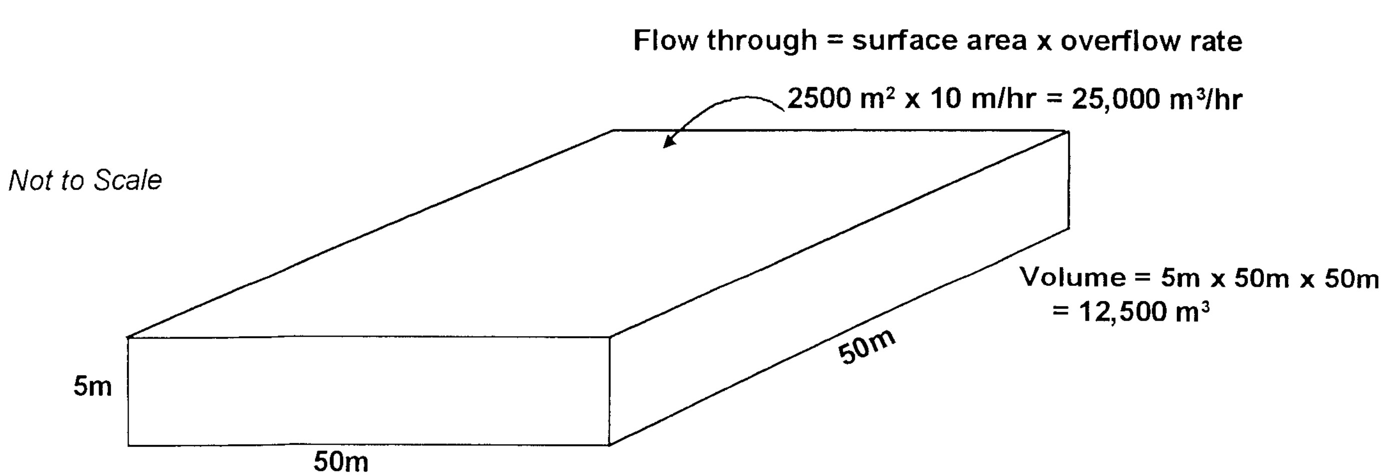
Once the maximum hourly flow rate in a district for the selected design storm was determined (see Figure 5-27), an estimate could be made of the design treatment rate. Preliminary investigations indicated that using the hourly peak rate for unit design was not sufficiently conservative to design a treatment system on the basis of overflow rate. Accordingly, the 15-min maximum peak for each storm, was estimated. The relative intensities, for design storms for the 1-hr duration peak and the 15-min duration peak, were compared using intensity-duration-frequency curves for Winnipeg (Acres 1978). For all the design storms, the 15-min intensity was approximately 2.5 times the maximum 1-hr intensity. Accordingly, the flow rate used for the RTB design was taken as 2.5 times the maximum hourly flow rate for the selected design storm for that district (see Figure 5-27). Once this flow rate was determined, the RTB was sized on the basis of the 10 m/hr design overflow rate. The required surface area was calculated and, assuming a 5-m depth tank, the volume, determined. The volumes as determined on this basis, and as required for each district to obtain either 0 untreated overflows or 4 untreated overflows, are shown in Tables 5-24 and 5-25.

In addition to the sizes of the off-line units needed for 4 overflows and 0 overflows, for the 825 ML/d option, the Tables 5-24 and 5-25 also provide the diameter and lengths of tunnel needed to supplement these devices. One interesting feature of the RTBs is that given a district like Metcalfe-Mager, which has a limited space available (i.e., for 2 off-line units), the use of the RTB means that there is sufficient space available to treat the total CSO for 4 overflows from that district. For the equivalent off-line storage option, there was insufficient area available for surface devices so that there was a need for supplemental tunnel storage. The obvious impact of the RTB, therefore, is to decrease the investment in tunnels as well as the investment in near-surface tankage.

Part of the function of the retention treatment basin, in addition to the portion stored and returned to the NEWPCC, is to disinfect the remaining portion of its "treatment capacity" so that at least part of the discharge up to the design capacity of the units will have significantly reduced numbers of fecal coliforms. Accordingly, the tables include for the capital cost of chlorination and dechlorination and the chemical cost for the recreation period.

Design of RTB





Hydrograph used in RTB Design Method
Figure 5-27

rtbdesgn
s\01\0510

Because the chemical usage is relatively small, and because of the hazards associated with the gaseous chemical usage, disinfection would likely be undertaken with sodium hypochlorite and dechlorination with sodium bisulphite. The capital costs were based on those developed for the report entitled "City of Winnipeg Report for the Combined Sewer Overflow Control Study – Impacts on the North End WPCC", that is, \$4.20/m^3 of flow per day. The system used in the RTBs will be simpler, since the hypochlorite would be added directly to the RTB and would not require a chlorine contact chamber. However, the isolated nature of the RTB facilities will likely require somewhat more elaborate facilities than those needed for a system incorporated in the NEWPCC. Therefore, the savings in one area would be counterbalanced by the extra cost in others. The present value of the chemical costs shown on the tables was based on disinfecting the total design flow during the recreation season. The disinfectant selected was sodium hypochlorite (12% solution) with a chlorine dosage of 12 mg/L. The assumed chlorine residual was 1 mg/L and therefore the sodium bisulphite required was 2 mg/L. The estimated chemical costs on these bases was \$50,000/year for hypochlorite (\$0.25/L) and \$34,000/year for sodium bisulphite (\$0.75/kg). The \$2 million total chemical cost is based on present value and was doubled to allow for labour associated with chemical handling.

5.6 SEPARATION OF COMBINED SEWERS

An obvious alternative to the obsolete combined sewers is to consider their conversion to the current standard, i.e., separate combined sewers. To a certain extent, such evaluation has already taken place in Winnipeg, on a district-specific basis. In the process of evaluating options for reducing basement flooding, a chronic problem in the existing combined sewer districts, the CoW has given consideration to installing new storm sewers (to provide localized separation of sewers) as an alternative to installing relief combined sewers. In some cases, (usually in areas close to the Red or Seine Rivers) it has proven to be more economical to separate certain areas rather than installing relief sewers. As a result, opportunistic separation of combined sewer areas has taken place, driven entirely by the best economics of providing improved basement flooding protection. A possible 600 ha of the 9,000 ha of the CoW combined sewer area have been separated in this manner, i.e., land drainage have been separated from the combined sewer and been diverted through land drainage sewers to the

Red or Seine Rivers. These actions have been opportunistic, in the sense that the localized separation was undertaken only if this option was less costly than other means of providing basement flooding relief.

Complete separation of the existing combined sewer system would involve a decision to separate land drainage and sanitary sewers on a regional basis, i.e., a conversion of the remaining existing one-pipe combined sewer system to a two-pipe system. This section will discuss the method of accomplishing such retrofit separation, the costs, and associated implications.

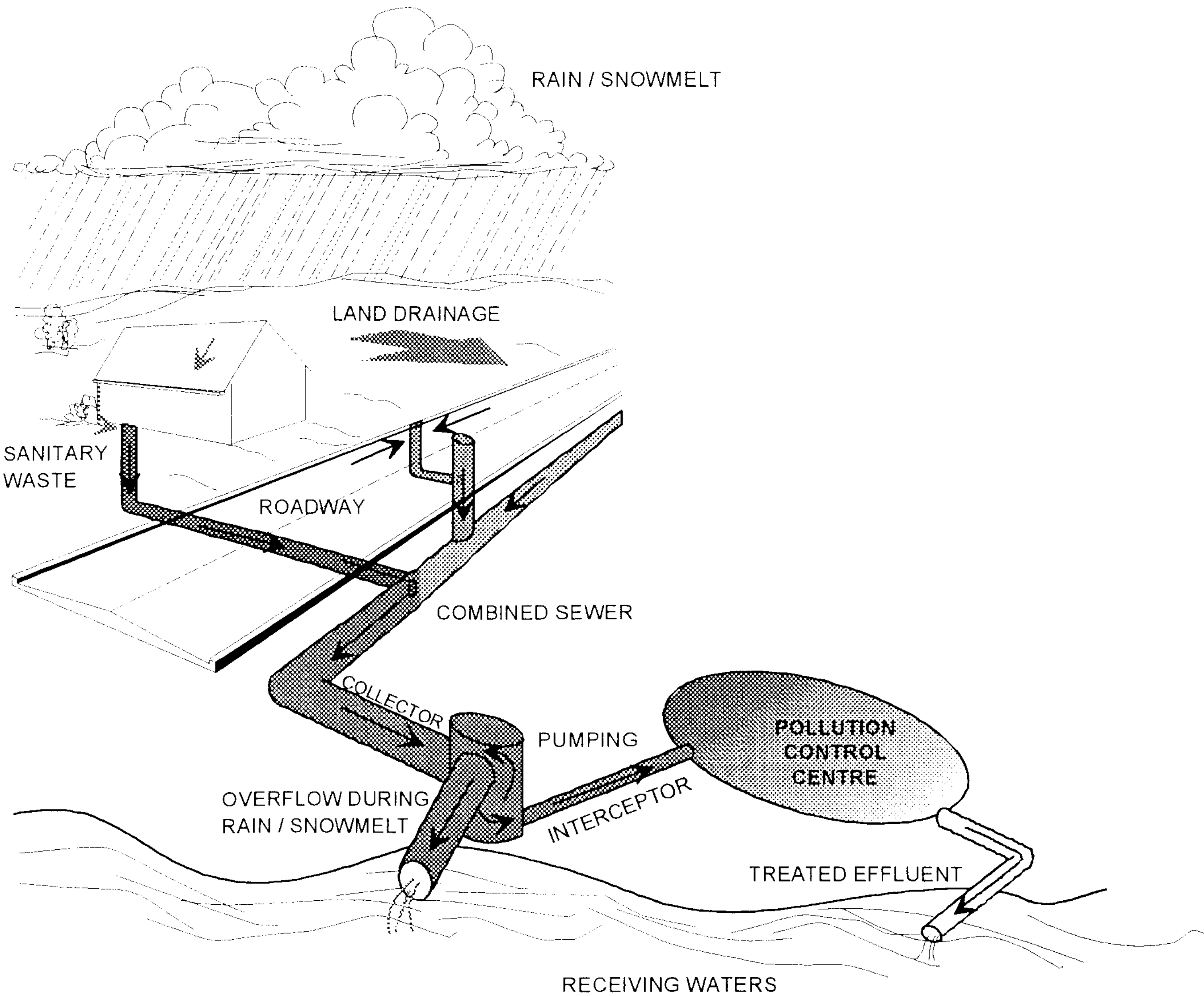
5.6.1 Separation of Existing Winnipeg Combined Sewer Systems

The existing combined sewer system in Winnipeg, as in other cities, is a one-pipe combined sanitary and land drainage pipe system (see Figure 5-28). With a separation option, the two wastewater streams must be separated.

Because of the many existing house connections to the existing combined sewers, it is most practicable to designate the existing combined sewer to act as the separate sanitary sewer and to install new land drainage sewers to carry the stormwater runoff. This means a new network of storm sewers will need to be constructed, in general, following the pattern of the combined sewers to capture overland flow and street runoff. Street catchbasins would be disconnected from the existing combined sewer and re-connected to the new land drainage sewers. These separate land drainage sewers would drain directly to the rivers or creeks, similar to new separate sewered areas, as these would carry only land drainage. The construction of such regional retrofit separation would involve significant costs, since these actions would all take place in an existing built-up area, and would involve significant community disruption.

A number of studies have been done, in Winnipeg and in other areas, on the cost of such retrofit separation. These data have been used to estimate the approximate costs and physical effects of this CSO control option.

COMBINED SEWER AREA



Combined Sewer System

- **Both street runoff and domestic sewage carried in one pipe.**
- **During dry weather combined sewers carry all wastewater to Control Centres.**
- **During heavy rain and snowmelt the volume of water exceeds sewerage system capacity and diluted sewage flows directly into the rivers.**

Figure 5-28

combonly
s\01\0510

5.6.2 Siting Considerations

The retrofit of the LDS system would likely affect 70 to 80% of the 9,000 ha currently served by combined sewers. This will entail significant community disruption over a major portion of the City of Winnipeg.

5.6.3 Operational Considerations

Retrofit separation would achieve the separation of street runoff from domestic/commercial wastewater, i.e., sanitary sewage. During rainfall, foundation drainage would still enter the wastewater sewer (former combined sewer) through the weeping tiles/foundation drainage system but this would be little different from the situation with separate systems. The land drainage system would now carry the street runoff directly to the rivers, along with the pollutants associated with this runoff.

The major difference in stream loadings would relate to microbiological loadings, as measured by fecal coliforms. Although the levels of fecal coliforms in the LDS would be significantly reduced, there would still be "spikes" in the river during and after rainfalls. As discussed in Section 6, there are other options which would provide equal, or greater, improvement to river quality at lower cost.

The retrofit separation would provide a benefit with respect to basement flooding protection for those districts that have not been provided with relief sewers (possibly 600 ha). The separation of the existing combined sewers would provide basement flooding protection to these areas, probably superior to installing relief sewers. For those districts with relief sewers, the added protection would be nominal as the existing system provides basement flood protection to acceptable standards.

The retrofit separation of the combined sewer system would not change the need for rehabilitation of the existing combined sewer system in that the existing combined sewer system would be expected to continue to perform as one part of the two-pipe system. If the existing combined sewer system is in need of rehabilitation, this need is common to the situation where these pipes perform as combined sewer or if these pipes perform as "separate sanitary sewers" in a retrofit separation system.

5.6.4 Cost of Retrofit Separation

To separate the existing combined sewers in Winnipeg would involve installing land drainage sewers, generally of similar capacity to the existing combined sewers and relief sewers; since the required capacity of the combined sewer system is governed entirely by the storm drainage component of the combined wastewater stream. The land drainage trunk sewers would therefore essentially replace the role of the combined sewers. The lateral sewer network would not be as extensive as the existing combined sewer network since the drainage capacity of the street itself can be used to carry runoff to catchbasins, thus avoiding the need for an intervening land drainage sewer. The total length of new storm drainage sewers is thus less than the existing combined sewer (where every house has to have access to the sewer for domestic waste).

Estimates of the cost of retrofit separation of this land have been made in Winnipeg and elsewhere. These estimates were reviewed to establish an approximate cost estimate of retrofit separation of the existing combined sewer system in Winnipeg.

5.6.4.1 General Experience

The costs of retrofit separation as reported in various project studies were reviewed in a prior study (Red/Assiniboine Water Treatment). These indicated unit costs of such separation, at that time, as $60,000 to $95,000/ha. For Winnipeg, based on 9,000 ha of residual combined sewer area, this would correlate to about $700 million to $1,000 million. (Costs were adjusted ion a 2%/yr inflation basis).

Other more recent studies, in the U.S., have developed figures for separation which, for Winnipeg in Canadian dollars, would support a separation cost of $1,700 million (Sacramento, CA) and $1,600 million (Hartford, Conn.).

5.6.4.2 Edmonton Experience

The City of Edmonton has recently developed an estimate of the cost of separation of their existing 5,000 ha of combined sewer district. This estimate is based on the same concept of utilizing the existing combined sewers as the separate domestic wastewater sewer and installing new land drainage sewers. Their estimate of the cost of this retrofit separation was $1,900,000 (approx. 1997 $).

The Edmonton context is not entirely applicable to Winnipeg as the depth and terrain is somewhat different. For Winnipeg, some adjustments to allow for reduced lengths of lateral sewers is considered reasonable and the adjusted comparable cost is about $1,100 million for 5,000 ha. Pro-rating this for the Winnipeg area of 9,000 ha results in an approximate cost of retrofit separation of $1,500 million for the Winnipeg combined sewer areas.

5.6.4.3 Comparative Cost Estimate

The costs for separation of the Winnipeg combined sewers are very high, regardless of which base is used for estimation. Given all the more recent supporting evidence, and in the absence of a specific study of costs for significant areas of Winnipeg, we have carried a cost for sewer separation of $1,500 million for comparisons with the other control options. This cost estimate is considered adequate for planning level comparisons, however, if complete separation was deemed worthy of further consideration, a Winnipeg-specific regional estimate would be required to firm up the cost estimates. If this option was deemed to be a preferred option, specific studies would be required to obtain better estimates of the localized construction costs in the various areas, as these are all in developed neighbourhoods.

5.7 FLOATABLES

The technologies discussed above address, in addition to reducing the frequency, volume and/or quality of CSO, both the fecal coliform issue and floatables control. The latter are addressed either by a reduction in the number and volume of overflows (i.e., increased interception capacity through in-line or off-line storage) or by high-rate treatment and sewage separation, which either reduce the levels of both contaminants by treatment or by eliminating the combination of wastewater and stormwater at the source, respectively. If the benchmark to be addressed consists only of floatables, i.e., the concern is neither fecal coliforms nor the number and volume of overflows, then, there are devices available that could address the floatables issue in isolation. In Phase 2, the devices available comprised the following categories:

- coarse screen technologies (screen openings of 6 mm or greater);
- fine-screen technologies (screen openings less than 2 to 6 mm);
- weir-mounted screens; and
- trap systems.

The mechanically least complicated and most cost-effective means of addressing floatables capture alone, appeared to be the "TrashTrap" system developed in the United States. The technology is basic, comprising large net bags and supporting infrastructure at the end of each of the combined sewer outlets. Experience with this system is limited, although extensive pilot tests have been run (New Jersey). The distinct advantage of this system is that all of the hardware is in the river, i.e., aboveground structures are unnecessary. Further, there are no land requirements, since the devices will be accessed over lands already used as the CS trunk right-of-way. There is a visual impact, but this does not appear to be too objectionable. This technology could be used in specific areas where downstream floatables are a particular concern. So long as fecal coliforms are the overall objective, it is unlikely that this technology would be applied over the whole region.

5.7.1 Floatables Evaluation

CSOs are known to contain sewage, spent hygiene products, and residential/commercial/ industrial waste that are aesthetically unappealing. For other venues, the visible traces of sewage, i.e., debris that is buoyant, "floatables", has been identified as being the most offensive aspect of CSOs to the public.

Although aesthetic impacts associated with Combined Sewer Overflows (CSO) were identified as a potential water quality issue in Winnipeg, little quantitative information was available for urban discharges to the local rivers. Experience elsewhere indicated that site-specific information needs to be gathered to evaluate the impact and determine the most appropriate course of action. Accordingly, an in-river-netting program using a floating boom and curtain was conducted in 1996 and 1997 to capture and quantify wet weather discharges from both CSOs and land drainage.

5.7.2 Results of Pilot Testing

During the two summer seasons that the field program was conducted (1996 and 1997), a total of five outfall locations were fitted with a boom system which was left in place at each location until each location experienced a number of rainfall events. Results thus far suggest that for the City of Winnipeg system of outfalls, the loading of floatable debris is highly variable from outfall to outfall. Results to date are shown in Tables 5-26 and 5-27 and summarized on Figure 5-29. As can be seen from the tables and figure, the greatest percent of floatables captured was natural debris, followed by oils/greases/films. Very low quantities of hygiene waste were observed in all cases.

While some outfalls were found to episodically introduce significant floatable debris loadings to the river subsequent to rainfall events, others were found to discharge very low quantities of debris. In some cases, debris was specific to individual industry sectors in the sewer district (such as animal processing plants). Based upon the observations obtained from the five boom installation locations thus far, it appears likely that floatable debris management and control on

TABLE 5-26

1996 SUMMER FLOATABLES - RECOVERY PROGRAM

Outfall Location	Boom Service Date	Previous Event Accumulated Rainfall (mm)	Captured Floatable Mass kg (lbs)	Spread-flat Area m2 (ft2)	Fraction of Total Captured (Percent of Area)						
					Paper Products	Plastics		Natural Debris (branches, leaves, grass)	Surface Films (oil, grease, scrum)	Health & Hygiene	Other Material
						Hard	Soft s				
Alexander	July 21	9.0	5.0 (11)	0.8 (9)	8	2	15	25	40	1	<1% (syringes recovered)
Alexander	July 30	0.6	2.0 (4.5)	0.4 (4)	8	-	5	20	65	2	-
Alexander	Aug 6	42	14.5 (32)	6.0 (65)	5	7	20	30	33	5	<1% (syringes recovered)
Alexander	Aug 19	10	34.7 (76.5)	19.5 (210)	13	10	20	14	35	7	<1% (syringes recovered)
Alexander	Aug 21	37.4	15.2 (33.5)	6.7 (72)	14	-	15	20	45	5	<1% (syringes recovered)
Alexander	Aug 23	15	3.2 (7.0)	0.5 (5)	15	5	10	35	30	5	-
Juba Park	Sept 2	12	0.9 (2)	0.4 (4)	5	5	20	70	-	-	-
Juba Park	Sept 10	3	1.8 (4)	0.6 (6)	2.5	2.5	20	60	10	5	-
Juba Park	Sept 30	12	1.4 (3)	0.4 (4)	5	5	20	50	10	1-	-

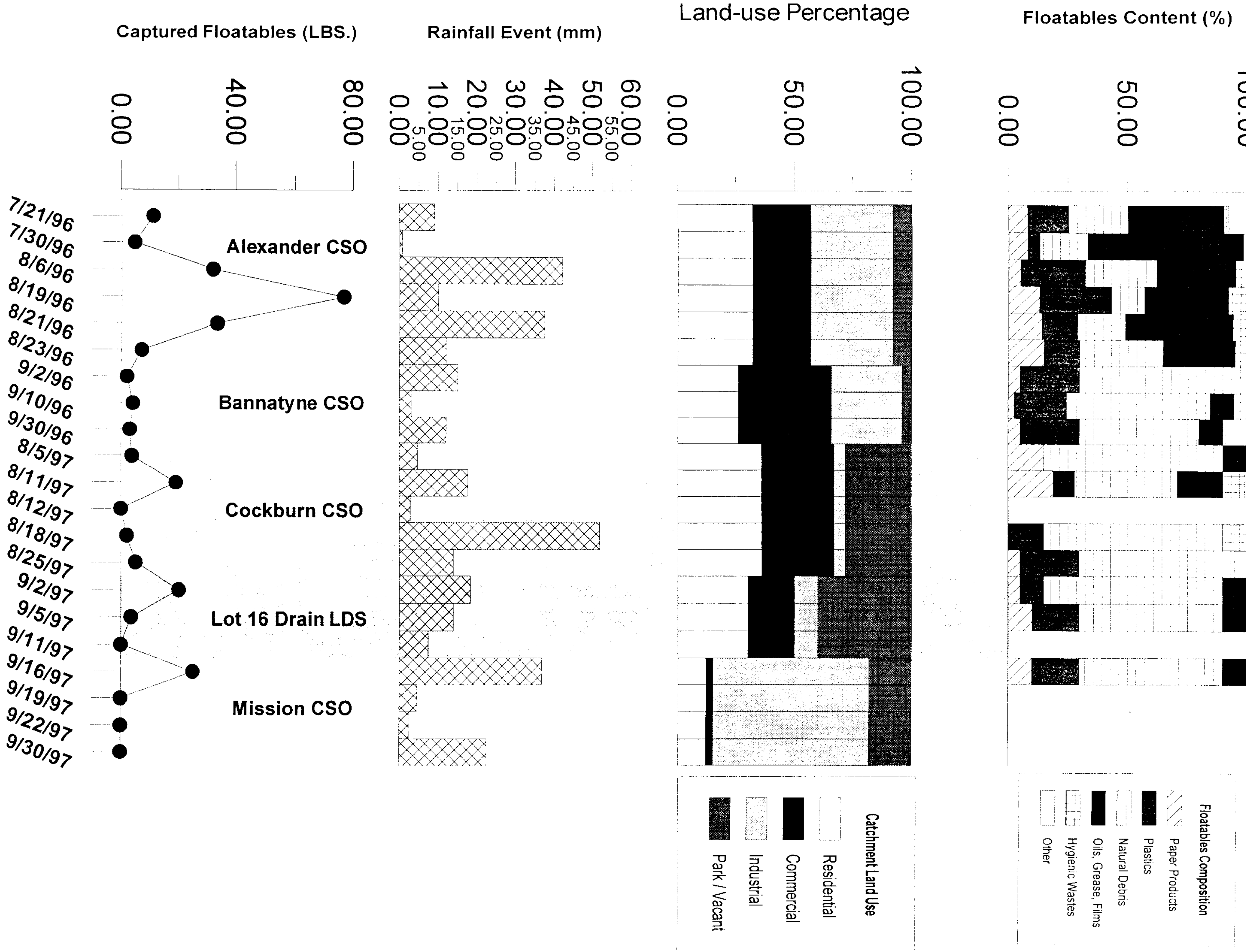
TetrES
CONSULTANTS

TABLE 5-27

1997 SUMMER FLOATABLES - RECOVERY PROGRAM

					Fraction of Total Captured (Percent of Area)						
						Plastics					
Outfall Location	Boom Service Date	Previous Event Accumulated Rainfall (mm)	Captured Floatable Mass kg (lbs)	Spread-flat Area m2 (ft2)	Paper Products	Hard	Soft	Natural Debris (branches, leaves, grass)	Surface Films (oil, grease, scrum)	Health & Hygiene	Other Material
Cockburn	Aug 5	4.6	1.7 (3.7)	0.4 (4.0)	15			75	10		
Cockburn	Aug 11	16.2	8.6 (19)	1.9 (21)	19		9	43	19	9	hockey ball, fishing gear
Cockburn	Aug 12	2.8	Negligible	-	-	-	-	-	-	-	-
Cockburn	Aug 18	51.8	0.9 (2)	0.2 (2)		10	5	75		10	
Cockburn	Aug 25	13.8	2.3 (5)	0.44 (5)	5	10	15	65			5
Lot 16 Drain	Sept 2	14.6	9.1 (20)	0.53 (6)	5		10	75	10		1 dead muskrat
Lot 16 Drain	Sept 5	13.8	1.6 (3.5)	0.40 (4)	10		20	60	10	-	
Lot 16 Drain	Sept 11	6.0	Negligible	-		-	-	-	-	-	Plastic pop bottles
Mission	Sept 16	36.6	11.3 (25)	1.42 (16)	10	10	10	60	10		Dark oil slick, animal tissues, strong diesel odour, large wood chunks
Mission	Sept 19	4.4	no debris								
Mission	Sept 22	2.4	no debris								
Mission	Sept 30	12.8 9.6	no debris								
Boom Removal for Winter Storage	Oct 1	-	-	-	-	-	-	-	-	-	-

TetrES
CONSULTANTS



Captured Floatables (LBS.)
Rainfall Event (mm)
Land-use Percentage
Floatables Content (%)
Alexander CSO
Bannatyne CSO
Cockburn CSO
Lot 16 Drain LDS
Mission CSO
7/21/96
7/30/96
8/6/96
8/19/96
8/21/96
8/23/96
9/2/96
9/10/96
9/30/96
8/5/97
8/11/97
8/12/97
8/18/97
8/25/97
9/2/97
9/5/97
9/11/97
9/16/97
9/19/97
9/22/97
9/30/97
Catchment Land Use
Residential
Commercial
Industrial
Park / Vacant
Floatables Composition
Paper Products
Plastics
Natural Debris
Oils, Grease, Films
Hygienic Wastes
Other

Figure 5-29

Winnipeg's rivers, in the absence of other CSO control options, could be achieved through selective targeting of only the outfalls which have demonstrated problematic floatable debris loadings or an aggressive source control program.

An important finding at only the Alexander CS outfall was the presence of hypodermic needles in the floatables captured. Although the quantities were low, it does present a potential health risk to river users. The origin of these hypodermic needles are unknown and presents a concern as to their previous use, specifically the possible infection of river users through inadvertent contact with these "sharps". It is recommended that a field investigation be conducted to attempt to locate the possible origin, i.e., determine if these "sharps" are discarded on the street, or in catchbasins, or from dwellings. As well, it is recommended that a floatables capture program be repeated at this location during this field investigation program.

5.7.3 Costs

The estimates prepared in Phase 2 are still considered valid for Phase 3 since the conceptual methods of capture and disposal would be similar. A combination of in-system screening and end-of-pipe netting, depending on district-specific conditions, would cost about $30 to $110 million, with the higher cost involving more screening and automation of debris removal. Refer to Phase 2 Technical Memorandum No. 3 "Control Alternatives/Experience Elsewhere", Section 4.5 (p29) for details.

5.8 OPERATION AND MAINTENANCE COSTS

For purposes of completeness, the costs of Operation and Maintenance (O&M) of the candidate options developed for achieving overflow control, were estimated. The results are provided in Table 5-28.

TABLE 5-28: O&M REQUIREMENTS

OPTION **OFF-LINE STORAGE**	DEWATER RATE	1 PUMPS BASINS #	2 TOTAL 1000kW-hr (300hrs)	3 SMALL PUMPS (TUNNELS) #	4 DWTR PUMP (TUNNELS) # 2/TUNNEL	5 TANKS #	6 TANK INSTAL'NS	7 MUs $M	8 TANKS $M ($.0004/UNIT)	9 PUMP kW .06/kW-hr	10 NEWPCC $M	11 TOTAL O&M/yr $M	12 PV 20yrs@4% $M
4 O/F W/ IN-LINE													
	600ML/d	11	500	9	14	15		$0.4	$0.0	$0.0	$0.7	$1.1	$15
	825ML/d	6	300	10	10	6		$0.3	$0.0	$0.0	$1.0	$1.3	$18
	1060ML/d	4	150	10	10	5		$0.3	$0.0	$0.0	$1.8	$2.1	$28
4 O/F W/O IN-LINE													
	600ML/d	17	750	17	18	58		$0.6	$0.0	$0.0	$0.7	$1.4	$19
	825ML/d	17	800	16	16	62		$0.6	$0.0	$0.0	$1.0	$1.6	$22
	1060ML/d	17	750	16	18	23		$0.6	$0.0	$0.0	$1.8	$2.5	$33
0 O/F W/ IN-LINE													
	600ML/d	17	800	31	28	56		$0.9	$0.0	$0.0	$0.7	$1.6	$22
	825ML/d	16	800	17	22	40		$0.6	$0.0	$0.0	$1.0	$1.7	$23
	1060ML/d	17	800	18	22	28		$0.7	$0.0	$0.0	$1.8	$2.5	$34
0 O/F W/O IN-LINE													
	600ML/d	17	800	31	28	68		$0.9	$0.0	$0.0	$0.7	$1.6	$22
	825ML/d	17	800	28	26	64		$0.8	$0.0	$0.0	$1.0	$1.9	$26
	1060ML/d	17	800	22	26	56		$0.8	$0.0	$0.0	$1.8	$2.6	$36
OFF-LINE W/ TRANSFERS													
W/IN-LINE	600ML/d	8	400	8	16	12		$0.4	$0.0	$0.0	$0.7	$1.1	$15
W/IN-LINE	825ML/d	8	400	8	10	9		$0.3	$0.0	$0.0	$1.0	$1.3	$18
RTB W/O IN-LINE													
4 O/F	825ML/d	17	800	16	12	20	17	$1.1	$0.0	$0.0	$1.0	$2.1	$29
0 O/F	825ML/d	17	800	28	18	45	17	$1.3	$0.0	$0.0	$1.0	$2.4	$32
REGIONAL TUNNEL													
4 O/F W/ IN-LINE													
	600ML/d	7	100		14			$0.2			$0.7	$0.9	$13
	825ML/d	7	100		14			$0.2			$1.0	$1.2	$17
	1060ML/d	7	100		14			$0.2			$1.8	$2.0	$28
4 O/F W/O IN-LINE													
	600ML/d	7	100		14			$0.2			$0.7	$0.9	$13
	825ML/d	7	100		14			$0.2			$1.0	$1.2	$17
	1060ML/d	7	100		14			$0.2			$1.8	$2.0	$28
0 O/F W/ IN-LINE													
	600ML/d	7	100		14			$0.2			$0.7	$0.9	$13
	825ML/d	7	100		14			$0.2			$1.0	$1.2	$17
	1060ML/d	7	100		14			$0.2			$1.8	$2.0	$28
0 O/F W/O IN-LINE													
	600ML/d	7	100		14			$0.2			$0.7	$0.9	$13
	825ML/d	7	100		14			$0.2			$1.0	$1.2	$17
	1060ML/d	7	100		14			$0.2			$1.8	$2.0	$28

O&M costs for the existing system were taken as being applicable to all options. It was assumed that the O&M costs for the in-line storage weir option could be achieved within the current O&M budget. No allowance was made for additional O&M costs of sewer separations.

The bases for the development of the O&M costs are provided in Table 5-29. As noted Maintenance Unit Costs (MUs) were developed on the basis of the City of Winnipeg's current costs associated with O&M of 70 small pumping stations. These costs were used to develop a unit cost per small pumping station (MU) of $11,000/yr. The current operating budget for the NEWPCC is provided on the table. The application of these O&M costs is shown on the table and discussed below.

The application of MUs is as follows:

- Small pumping stations: same as at present
- Large pumping stations: labour and material = 1.5 x small stations
  power separate @ $.06/kW/hr
- Tanks $100/event/tank (as per CG&S study for Sarnia facility) x 2 (safety factor)
- RTBs: same event cost + 1-2 hr visit/inspection team/week
- NEWPCC: The estimates of the proportion of the annual budget which could be attributed to the expansion are based on conjecture.

Referring to Table 5-28, the numbered cost columns were calculated as follows:

- Column 7 = 0.11 (1.2 x Column 1 + Column 3 + Column 4)
  for all but RTBs
  which = 0.11 (1.2 x Column 1 + Column 3 + Column 4 + 3 x Column 6)
- Column 8 = .0004 x Column 5
- Column 9 = .06 x Column 2/1,000
- Column 10 = as noted

TABLE 5-29: BASES FOR O&M REQUIREMENT COSTS

MAINTENANCE UNITS (MU) The City of Winnipeg currently has some 70 small pumping stations to maintain.
The level of effort involves 6 - 3-man crews full time (3 crews inspection; 3, maintenance.
Costs: Labour - 2*3*3*$42,000*1.3 = $980,000/50 weeks (payroll burden = 30%)
power = $350,000/50 weeks
materials = $170,000/50 weeks
= 1.5M/50 weeks
Assume 22 weeks of operation for CSO control systems.
Cost per CSO maintenance unit =1/70*(22/50*(980+350)$K+170$K) (assume materials costs to be 100%)
= $11,000/CSO maintenance unit /year.
= 1MU

NEWPCC

Current operating budget =

Labour: $1.7M/year
Services: $2M/year
Supplies: $3M/year
Total: $7M/year

MU Allocations

Small pumping stations = 1
Large pumping stations = 1.5*MU(labour + material) = 1.5*.8 = 1.2MU
+ power separate
power was costed @ $ .06/kW-hr.
Tanks = $100/event/tank*2 (safety factor). Say $4,000/year /tank
RTBs = $4,000/year/tank + 2-hour team visit/installation/week (2/35*$580K)/year. $580K = seasonal labour + power cost.
= $4,000/yr/tank+33,000/yr/installation(3MU/yr/tank)

NEWPCC 600ML/day involves expansion of secondaries & digesters, say 10% budget increase = $.7M/year
825ML/day involves expansion of primaries& digesters, say 15% increase = $1M/year
1060ML/day involves expansion of headworks, primaries & digesters, say 25% increase = $1.8M/year

5.9 SUMMARY

Section 3 provided the proposed performance measures by which the relative performance of the various control options could be determined. The surrogate for all of these measures, used as a basis for determining the cost of options, was the number of overflows. The target performance level, for most of the technologies, was 0 and 4 overflows for the 1992 Representative Year. The costs of the Regional Tunnel Storage/Transport options for 1 and 0 overflows, long term, were also determined. Finally, optimizing Existing Infrastructure and Separation were costed and their performance evaluated in Section 6.

The cost for the candidate options reviewed on the above bases is summarized in Table 5-30. These results were used in the evaluation of options undertaken in Section 6.

Table 5-30
Summary of Candidate Options Costs

	Plan Number	Dewatering Rate at NEWPCC ML/d	Treatment Cost Millions	Interceptor Cost Millions	Inline Storage Cost or Regulator for Offline[1]	Required Offline Storage Volume m³	New Structural Cost Millions	O & M Cost PV	Total Cost Millions
Existing Situation									
Existing	0	825							$0
Optimizing Existing Infrastructure									
Inline Storage	1	600	$15		$100				$115
	2	825	$36	$15	$100				$151
	3	1060	$70	$46	$100				$216
Target of 4 Overflows									
Distributed Offline Storage	4	600	$15		$12	300,000	$384	$19	$430
	5	825	$36	$15	$12	215,000	$335	$22	$420
	6	1060	$70	$46	$12	185,000	$303	$34	$465
Distributed Inline/Offline Storage	7	600	$15		$100	102,000	$184	$15	$314
	8	825	$36	$15	$100	66,000	$127	$18	$296
	9	1060	$70	$46	$100	38,000	$98	$28	$342
~~Distributed~~ Inline/Offline Storage with Transfers	10	600	$15		$100	80,000	$137	$15	$267
	11	825	$36	$15	$100	54,000	$113	$18	$282
	12	1060	$70	$46	$100				
Tunnel Transport/Storage	13	600	$15		$12	300,000	$485	$13	$525
	14	825	$36		$12	215,000	$432	$17	$497
	15	1060	$70		$12	185,000	$391	$28	$501
Inline with Tunnel Transport/Storage	16	600	$15		$100	102,000	$299	$13	$427
	17	825	$36		$100	66,000	$274	$17	$427
	18	1060	$70		$100	38,000	$274	$28	$472
Hirate Treatment RTB	19	825	$36	$15	$12	160,000	$327	$29	$390
Target of 0 Overflows - Representative Year									
Distributed Storage	20	600	$15		$12	825,000	$845	$22	$894
	21	825	$36	$15	$12	600,000	$697	$25	$785
	22	1060	$70	$46	$12	530,000	$564	$33	$725
Distributed Inline/Offline Storage	23	600	$15		$100	606,000	$613	$22	$750
	24	825	$36	$15	$100	393,000	$456	$23	$630
	25	1060	$70	$46	$100	230,000	$443	$28	$687
Tunnel Transport/Storage	26	600	$15		$12	825,000	$703	$13	$743
	27	825	$36		$12	600,000	$636	$17	$701
	28	1060	$70		$12	530,000	$622	$28	$732
Inline Plus Tunnel Transport/Storage	29	600	$15		$100	606,000	$588	$13	$716
	30	825	$36		$100	393,000	$519	$17	$672
	31	1060	$70		$100	230,000	$422	$28	$592
Hirate Treatment RTB	32	825	$36	$15	$12	385,000	$532	$32	$595
Target of 1 Overflows - Long Term									
Tunnel Transport/Storage	33	600	$15		$12	1,200,000	$685	$13	$725
	34	825	$36		$12	1,000,000	$630	$17	$695
	35	1060	$70		$12	825,000	$575	$28	$685
Target of 0 Overflows - Long Term									
Tunnel Transport/Storage	36	600	$15		$12	2,438,000	$990	$13	$1,030
	37	825	$36		$12	2,175,000	$950	$17	$1,015
	38	1060	$70		$12	2,000,000	$900	$28	$1,010
Separation									
	39								$1,500

6. PERFORMANCE EVALUATION OF ALTERNATIVE PLANS

A total of 39 potential control plans were identified in Section 5 along with their respective storage/conveyance/treatment requirements and costs. These plans span the spectrum of technologies from maximizing the use of the existing infrastructure to plans with structurally-intensive additions to the existing system and include the conversion of the system to a separate system.

This section will discuss the comparative performance of these plans in terms of meeting a range of performance measures (discussed in Section 3) and a number of evaluation criteria. As explained in Section 3, a number of possible performance measures were selected as "targets" for the definition of comparative control plans, which include:

- optimizing existing infrastructure;
- limiting CSOs to about 4 per year;
- 85% capture of combined sewage;
- limiting CSOs to 0 per year;
- compliance with MSWQO objectives for fecal coliform.

This was done to present a broad range of controls which would have different performance levels, physical characteristics and costs, and thus allow stakeholders to assess these "trade-off's" and offer opinions on the preferred action.

The data presented in Table 6-1 provides the basis for assessing relative benefits and costs.

6.1 EXISTING PERFORMANCE

The performance of the existing system provides a context for assessing the relative performance of potential CSO controls.

Table 6-1
Evaluation of Candidate Options

08:33 AM 27-Apr-98

	Plan Number	Dewatering Rate at NEWPCC ML/d	Treatment Cost Millions	Interceptor Cost Millions	Inline Storage Cost or Regulator for Offline[1]	Required Offline Storage Volume m³	New Structural Cost Millions	O & M Cost PV	Total Cost Millions	1992 Number of OF	1992 % Capture	Longterm Median Number of OF	Longterm Median % Capture	Longterm MAX OF	Remarks
Existing Situation															
Existing	0	825							$0	20.8	40%	17.3	32%	29	
Optimizing Existing Infrastructure															
Inline Storage	1	600	$15		$100				$115	7.2	77%	6.8	52%	14.7	
	2	825	$36	$15	$100				$151	6.2	84%	6.2	59%	12.7	
	3	1060	$70	$46	$100				$216	5.3	87%	5.3	62%		
Target of 4 Overflows															
Distributed Offline Storage	4	600	$15		$12	300,000	$384	$19	$430	3.9	83%	5.1	54%	12.2	
	5	825	$36	$15	$12	215,000	$335	$22	$420	3.8	87%	5.3	62%	11	
	6	1060	$70	$46	$12	185,000	$303	$34	$465	4.8	85%	6	58%		
Distributed Inline/Offline Storage	7	600	$15		$100	102,000	$184	$15	$314	3.2	87%	4.4	59%	11.1	
	8	825	$36	$15	$100	66,000	$127	$18	$296	2.9	91%	4.3	64%	9.8	
	9	1060	$70	$46	$100	38,000	$98	$28	$342	3.8	88%	4.3	65%		
Distributed Inline/Offline Storage with Transfers	10	600	$15		$100	80,000	$137	$15	$267	2.6	87%	3.5	59%		
	11	825	$36	$15	$100	54,000	$113	$18	$282	2.3	91%	3.6	64%		
	12	1060	$70	$46	$100										
Tunnel Transport/Storage	13	600	$15		$12	300,000	$485	$13	$525	4	84%	5	54%		
	14	825	$36		$12	215,000	$432	$17	$497	4	86%	5	62%		
	15	1060	$70		$12	185,000	$391	$28	$501	4	86%	5	64%		Estimated not Modelled
Inline withTunnel Transport/Storage	16	600	$15		$100	102,000	$299	$13	$427	4	84%	5	59%		
	17	825	$36		$100	66,000	$274	$17	$427	4	86%	5	64%		
	18	1060	$70		$100	38,000	$274	$28	$472	4	86%	5	64%		Estimated not Modelled
Hirate Treatment RTB	19	825	$36	$15	$12	160,000	$327	$29	$390	4	86%	5	64%		Estimated not Modelled
Target of 0 Overflows - Representative Year															
Distributed Storage	20	600	$15		$12	825,000	$845	$22	$894	0.4	100%	2.4	74%	6.4	
	21	825	$36	$15	$12	600,000	$697	$25	$785	0.1	100%	2.5	74%	6.3	
	22	1060	$70	$46	$12	530,000	$564	$33	$725	0	100%	2.4	74%		Estimated not Modelled
Distributed Inline/Offline Storage	23	600	$15		$100	606,000	$613	$22	$750	0.3	100%	2.4	74%	6.3	
	24	825	$36	$15	$100	393,000	$456	$23	$630	0.1	100%	2.3	74%	6.2	
	25	1060	$70	$46	$100	230,000	$443	$28	$687	0	100%	2.3	74%		Estimated not Modelled
Tunnel Transport/Storage	26	600	$15		$12	825,000	$703	$13	$743	0	100%	2.3	74%		
	27	825	$36		$12	600,000	$636	$17	$701	0	100%	2.3	74%		
	28	1060	$70		$12	530,000	$622	$28	$732	0	100%	2.3	74%		Estimated not Modelled
Inline PlusTunnel Transport/Storage	29	600	$15		$100	606,000	$588	$13	$716	0	100%	2.3	74%		
	30	825	$36		$100	393,000	$519	$17	$672	0	100%	2.3	74%		
	31	1060	$70		$100	230,000	$422	$28	$592	0	100%	2.3	74%		Estimated not Modelled
Hirate Treatment RTB	32	825	$36	$15	$12	385,000	$532	$32	$595	0	100%	2.3	74%		Estimated not Modelled
Target of 1 Overflows - Long Term															
Tunnel Transport/Storage	33	600	$15		$12	1,200,000	$685	$13	$725	0	100%	1	84%		
	34	825	$36		$12	1,000,000	$630	$17	$695	0	100%	1	85%		
	35	1060	$70		$12	825,000	$575	$28	$685	0	100%	1	85%		
Target of 0 Overflows - Long Term															
Tunnel Transport/Storage	36	600	$15		$12	2,438,000	$990	$13	$1,030	0	100%	0	100%		
	37	825	$36		$12	2,175,000	$950	$17	$1,015	0	100%	0	100%		
	38	1060	$70		$12	2,000,000	$900	$28	$1,010	0	100%	0	100%		
Separation															
	39								$1,500	0	100%	0	100%		

The in-line storage option, which includes disinfection of the dry weather flows at the treatment plants, showed an average compliance of about 90% at the 14 stations selected along both rivers. The in-line storage considered in Phase 3 represented a volume of about 2 to 3 times greater than assumed in Phase 2, therefore, the compliance in the receiving stream should be slightly better than earlier estimated. As discussed in Phase 2, the in-line storage option results in only a slight increment in improved compliance compared to the planned implementing disinfection of the WPCCs and correcting dry weather overflows. This is because all non-compliance resulting from CSOs are of relatively short duration.

In-line storage improves the overall frequency of compliance somewhat, however, excursions would still occur during those WWF events when a system overflow would occur. This would occur about 5 to 8 times a year on an overall system basis, but some districts could overflow more frequently and would need other control measures to conform to the average system performance.

6.2.4 Evaluation of In-line Storage

In-line storage is a very cost-effective way of reducing the volume of overflows to the rivers and reducing the numbers of overflows at most districts. The percentage of capture can increase from 32% for the existing situation to a range of 50 to 60% for various in-line control options ranging in cost from $115 million to $215 million. The number of overflows decrease on average from 17 to between 7 to 5 overflows (see Figure 6-3).

This benefit of reducing the number of overflows with in-line storage is not distributed evenly across the City. Figure 6-4 is a schematic showing where the in-line storage is distributed throughout the system and the performance (in terms of number of overflows) expected from in-line storage for one of the options (825 ML/d at NEWPCC). This is shown for the representative year of rainfall and would be typical of other years. Many of the districts have less than 4 overflows, and a few, such as Clifton and Aubrey, could be reduced to 0 overflows if their full potential of in-line storage can be realized. However, five other districts, Cockburn, Baltimore, Dumoulin, Riverbend and Syndicate have overflows ranging from 11 to more than

Table 6-4

08:33 AM 27-Apr-98

Evaluation of Candidate Options Targeting 0 Overflows

	Plan Number	Dewatering Rate at NEWPCC ML/d	Treatment Cost Millions	Interceptor Cost Millions	Inline Storage Cost or Regulator for Offline[1]	Required Offline Storage Volume m^3	New Structural Cost Millions	O & M Cost PV	Total Cost Millions	1992 Number of OF	1992 % Capture	Longterm Median Number of OF	Longterm Median % Capture	Longterm MAX OF	Remarks
Target of 0 Overflows - Representative Year															
Distributed Storage	20	600	$15		$12	825,000	$845	$22	$894	0.4	100%	2.4	74%	6.4	
	21	825	$36	$15	$12	600,000	$697	$25	$785	0.1	100%	2.5	74%	6.3	
	22	1060	$70	$46	$12	530,000	$564	$33	$725	0	100%	2.4	74%		Estimated not Modelled
Distributed Inline/Offline Storage	23	600	$15		$100	606,000	$613	$22	$750	0.3	100%	2.4	74%	6.3	
	24	825	$36	$15	$100	393,000	$456	$23	$630	0.1	100%	2.3	74%	6.2	
	25	1060	$70	$46	$100	230,000	$443	$28	$687	0	100%	2.3	74%		Estimated not Modelled
Tunnel Transport/Storage	26	600	$15		$12	825,000	$703	$13	$743	0	100%	2.3	74%		
	27	825	$36		$12	600,000	$636	$17	$701	0	100%	2.3	74%		
	28	1060	$70		$12	530,000	$622	$28	$732	0	100%	2.3	74%		Estimated not Modelled
Inline PlusTunnel Transport/Storage	29	600	$15		$100	606,000	$588	$13	$716	0	100%	2.3	74%		
	30	825	$36		$100	393,000	$519	$17	$672	0	100%	2.3	74%		
	31	1060	$70		$100	230,000	$422	$28	$592	0	100%	2.3	74%		Estimated not Modelled
Hirate Treatment RTB	32	825	$36	$15	$12	385,000	$532	$32	$595	0	100%	2.3	74%		Estimated not Modelled
Target of 1 Overflows - Long Term															
Tunnel Transport/Storage	33	600	$15		$12	1,200,000	$685	$13	$725	0	100%	1	84%		
	34	825	$36		$12	1,000,000	$630	$17	$695	0	100%	1	85%		
	35	1060	$70		$12	825,000	$575	$28	$685	0	100%	1	85%		
Target of 0 Overflows - Long Term															
Tunnel Transport/Storage	36	600	$15		$12	2,438,000	$990	$13	$1,030	0	100%	0	100%		
	37	825	$36		$12	2,175,000	$950	$17	$1,015	0	100%	0	100%		
	38	1060	$70		$12	2,000,000	$900	$28	$1,010	0	100%	0	100%		
Separation															
	39								*********	0	100%	0	100%		

Table 6-3
Evaluation of Candidate Options Targeting 4 Overflows

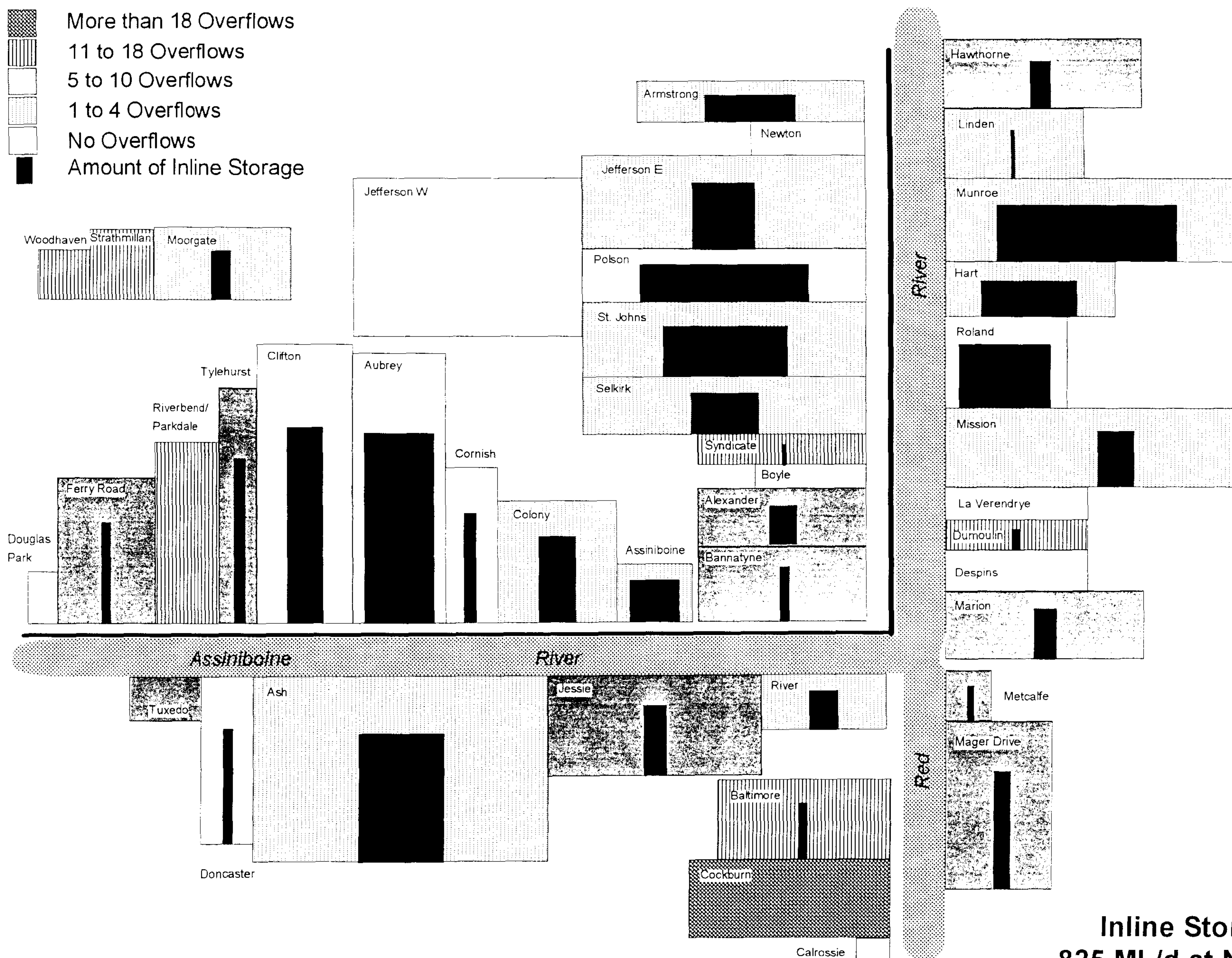
08:33 AM 27-Apr-98

	Plan Number	Dewatering Rate at NEWPCC ML/d	Treatment Cost Millions	Interceptor Cost Millions	Inline Storage Cost or Regulator for Offline[1]	Required Offline Storage Volume m^3	New Structural Cost Millions	O & M Cost PV	Total Cost Millions	1992 Number of OF	1992 % Capture	Longterm Median Number of OF	Longterm Median % Capture	Longterm MAX OF	Remarks
Target of 4 Overflows															
Distributed Offline Storage	4	600	$15		$12	300,000	$384	$19	$430	3.9	83%	5.1	54%	12.2	
	5	825	$36	$15	$12	215,000	$335	$22	$420	3.8	87%	5.3	62%	11	
	6	1060	$70	$46	$12	185,000	$303	$34	$465	4.8	85%	6	58%		
Distributed Inline/Offline Storage	7	600	$15		$100	102,000	$184	$15	$314	3.2	87%	4.4	59%	11.1	
	8	825	$36	$15	$100	66,000	$127	$18	$296	2.9	91%	4.3	64%	9.8	
	9	1060	$70	$46	$100	38,000	$98	$28	$342	3.8	88%	4.3	65%		
Distributed Inline/Offline Storage with Transfers	10	600	$15		$100	80,000	$137	$15	$267	2.6	87%	3.5	59%		
	11	825	$36	$15	$100	54,000	$113	$18	$282	2.3	91%	3.6	64%		
	12	1060	$70	$46	$100										
Tunnel Transport/Storage	13	600	$15		$12	300,000	$485	$13	$525	4	84%	5	54%		
	14	825	$36		$12	215,000	$432	$17	$497	4	86%	5	62%		
	15	1060	$70		$12	185,000	$391	$28	$501	4	86%	5	64%		Estimated not Modelled
Inline withTunnel Transport/Storage	16	600	$15		$100	102,000	$299	$13	$427	4	84%	5	59%		
	17	825	$36		$100	66,000	$274	$17	$427	4	86%	5	64%		
	18	1060	$70		$100	38,000	$274	$28	$472	4	86%	5	64%		Estimated not Modelled
Hirate Treatment RTB	19	825	$36	$15	$12	160,000	$327	$29	$390	4	86%	5	64%		Estimated not Modelled

Table 6-3
Evaluation of Candidate Options Targeting 4 Overflows

08:33 AM   27-Apr-98

	Plan Number	Dewatering Rate at NEWPCC ML/d	Treatment Cost Millions	Interceptor Cost Millions	Inline Storage Cost or Regulator for Offline[1]	Required Offline Storage Volume m^3	New Structura l Cost Millions	O & M Cost PV	Total Cost Millions	1992 Number of OF	1992 % Capture	Longterm Median Number of OF	Longterm Median % Capture	Longterm MAX OF	Remarks
Target of 4 Overflows															
Distributed Offline Storage	4	600	$15		$12	300,000	$384	$19	$430	3.9	83%	5.1	54%	12.2	
	5	825	$36	$15	$12	215,000	$335	$22	$420	3.8	87%	5.3	62%	11	
	6	1060	$70	$46	$12	185,000	$303	$34	$465	4.8	85%	6	58%		
Distributed Inline/Offline Storage	7	600	$15		$100	102,000	$184	$15	$314	3.2	87%	4.4	59%	11.1	
	8	825	$36	$15	$100	66,000	$127	$18	$296	2.9	91%	4.3	64%	9.8	
	9	1060	$70	$46	$100	38,000	$98	$28	$342	3.8	88%	4.3	65%		
Distributed Inline/Offline Storage with Transfers	10	600	$15		$100	80,000	$137	$15	$267	2.6	87%	3.5	59%		
	11	825	$36	$15	$100	54,000	$113	$18	$282	2.3	91%	3.6	64%		
	12	1060	$70	$46	$100										
Tunnel Transport/Storage	13	600	$15		$12	300,000	$485	$13	$525	4	84%	5	54%		
	14	825	$36		$12	215,000	$432	$17	$497	4	86%	5	62%		
	15	1060	$70		$12	185,000	$391	$28	$501	4	86%	5	64%		Estimated not Modelled
Inline withTunnel Transport/Storage	16	600	$15		$100	102,000	$299	$13	$427	4	84%	5	59%		
	17	825	$36		$100	66,000	$274	$17	$427	4	86%	5	64%		
	18	1060	$70		$100	38,000	$274	$28	$472	4	86%	5	64%		Estimated not Modelled
Hirate Treatment RTB	19	825	$36	$15	$12	160,000	$327	$29	$390	4	86%	5	64%		Estimated not Modelled



Inline Storage and 825 ML/d at NEWPCC

Figure 6-4

pdoxmodl

18. Baltimore district information may not be accurate as it was relieved recently and Dumoulin and Syndicate districts are very small and Syndicate district is planned for separation for basement flood relief.

This option by itself, while very cost-effective, may not be considered a comprehensive CSO management plan. It may be necessary to add additional storage or treatment in those districts without significant in-line storage in order to reduce the number of overflows system-wide. This could be done by a number of methods such as adding off-line storage, either by tunnel or tank, or high-rate treatment (with retention treatment basins). The performance of these other options will be explored in the following sections.

An example of the manner in which in-line storage could be significantly improved could involve additions to the Cockburn, Dumoulin and Riverbend districts. Such improvement can be projected from the analysis of Plan No. 11 (distributed off-line storage with transfers and 4 overflow).

The following assumes that Baltimore and Syndicate have already been resolved:

- Cockburn – add off-line storage @ $6.5 million
- Dumoulin – transfer to Roland @ $3 million
- Riverbend – transfer to Clifton @ $5 million

Total cost of upgrade = $14.5 million x 1.2(estimating contingency) x 1.2(EA&F) = $23 million for budget purposes. This would bring the total cost of the improved in-line storage scenario to $174 million (from the $151 million for Plan No. 2).

A second alternative, which was discussed in Section 5, would be to increase the in-line storage available during the installation of basement flooding relief programs being undertaken at these districts. If relief pipes are added to the district, which provide an increase in in-line storage, the number of overflows may be reduced to less than 4. Another alternative would be to oversize the relief pipes during the design in order to add an extra CSO control benefit. These would have to be investigated on a case-by-case district while planning for basement flooding relief.

Figure 6-3 illustrates the trade-off between cost and performance in terms of number of overflows and percent capture, for the three dewatering methods. Each upgrade in the rate at which dewatering can be performed shows an improvement in the average number of overflows.

6.3 MEETING 4 OVERFLOWS/YEAR

Sixteen alternative control plans (Nos. 4 to 19 inclusive) were identified that approximated the 4 overflows/year target for either representative year or the long-term record. It should be noted that the 4 overflows/year 'target' is based on the EPA "presumption" that such control will meet water quality standards so it is not a rigid target in itself. It should also be noted that meeting 4 overflows/year in the long-term record is a more stringent target than a representative year. The data shown on Table 6-3 illustrates this.

These control plans involve a very significant increase in cost, relative to in-line storage, as these are structurally intensive, all involving the addition of major system components and operational complexities.

6.3.1 Reduction in CSOs

The three different interception/dewatering rates (600, 825, 1,060 ML/d) are again possible with these alternatives. Plans were developed with and without the use of in-line storage.

Plans using distributed off-line storage (Plans 4-6) were sized using a screening model to obtain a target of 4 overflows per year in each of the districts for the representative year. Sizing was done considering 3 different dewatering rates based on the 3 plant capacities considered in Section 4. For the representative year, the planning model showed that the target could be closely met for the 600 ML/d and 825 ML/d options (there was slightly less than 4 overflows per year when averaged across all districts), however, the performance evaluation showed about 5 overflows per year with the higher dewatering rate. For the long term evaluation, the estimated performance ranged between 5 and 6 overflows on average, and the percent capture was between 54 and 62%. The costs range from $420 million to $465 million with the 825 ML/d option providing the best performance at the lowest cost.

For the distributed storage supplementing the in-line storage (Plans 7-9), the screening model was used to size storage in all districts in which in-line storage was not available and to provide additional storage in districts in which in-line storage was not enough to reduce the number of overflows to 4 per year during the representative year. In the districts in which in-line storage reduces overflows to less than 4, no additional storage is required. The average number of overflows in all districts ranged between 3 and 4, depending on the dewatering rate. When the long-term record was assessed, the average number of overflows in all districts was very close to 4, while the percent capture ranged between 59 and 65%. The cost of these plans ranged from $296 to $342 million. For all dewatering rates (600, 825 and 1,060) the use of in-line storage reduced the cost by about $120 million and reduced the average number of overflows by at least one. In-line storage thus represents a very valuable opportunity.

Plans 10-12 were developed to test the sensitivity of replacing storage at some districts with a transfer of the combined sewage to a nearby district where in-line or off-line storage could be available. Two of the dewatering strategies (600 and 825 ML/d) were assessed. Several transfers were considered to be cost-effective, as discussed in Section 5. For the long-term analysis, the average number of overflows was less than 4 (a reduction of 0.5) and the percent capture was 59 to 64%. The overall cost of these alternatives ($270 to $280 million) is $15 to $35 million less costly than alternatives with no transfer. The 1,060 ML/d dewatering strategy can be estimated at a later date, however, it is unlikely to produce an overall savings. The multiple-transfer system may be more difficult to operate than the no-transfer option and, therefore, may not provide the full cost savings demonstrated in this early analysis.

Plans 13, 14 and 15 are plans using tunnel transport/storage, again employing the three dewatering strategies. In the long term, 5 overflows would be expected and the percent capture would range between 54 and 62%. The cost of these options is higher, ranging from $500 to $525 million.

Using in-line storage in conjunction with a regional tunnel, provides a cost saving (Plans 16-18). The performance evaluation would be very similar to the above alternative, however, costs could be reduced to $430 to $470 million by utilizing in-line storage.

Using a high-rate treatment system of a retention treatment basin (RTB), the required volume of storage can be greatly reduced (Plan 19). A district-wide conceptual design of RTBs was done using the data from the representative year. It can be expected that this design, over the long term, would have 5 untreated bypasses of the RTB a year. With disinfection of the RTB effluent, it should provide similar benefits to the river in terms of fecal coliforms. In terms of water quality parameters other than fecal coliforms, however, the treatment provided at the NEWPCC for the storage option should be considered more effective.

6.3.2 Evaluation of "4 Overflows per Year Options"

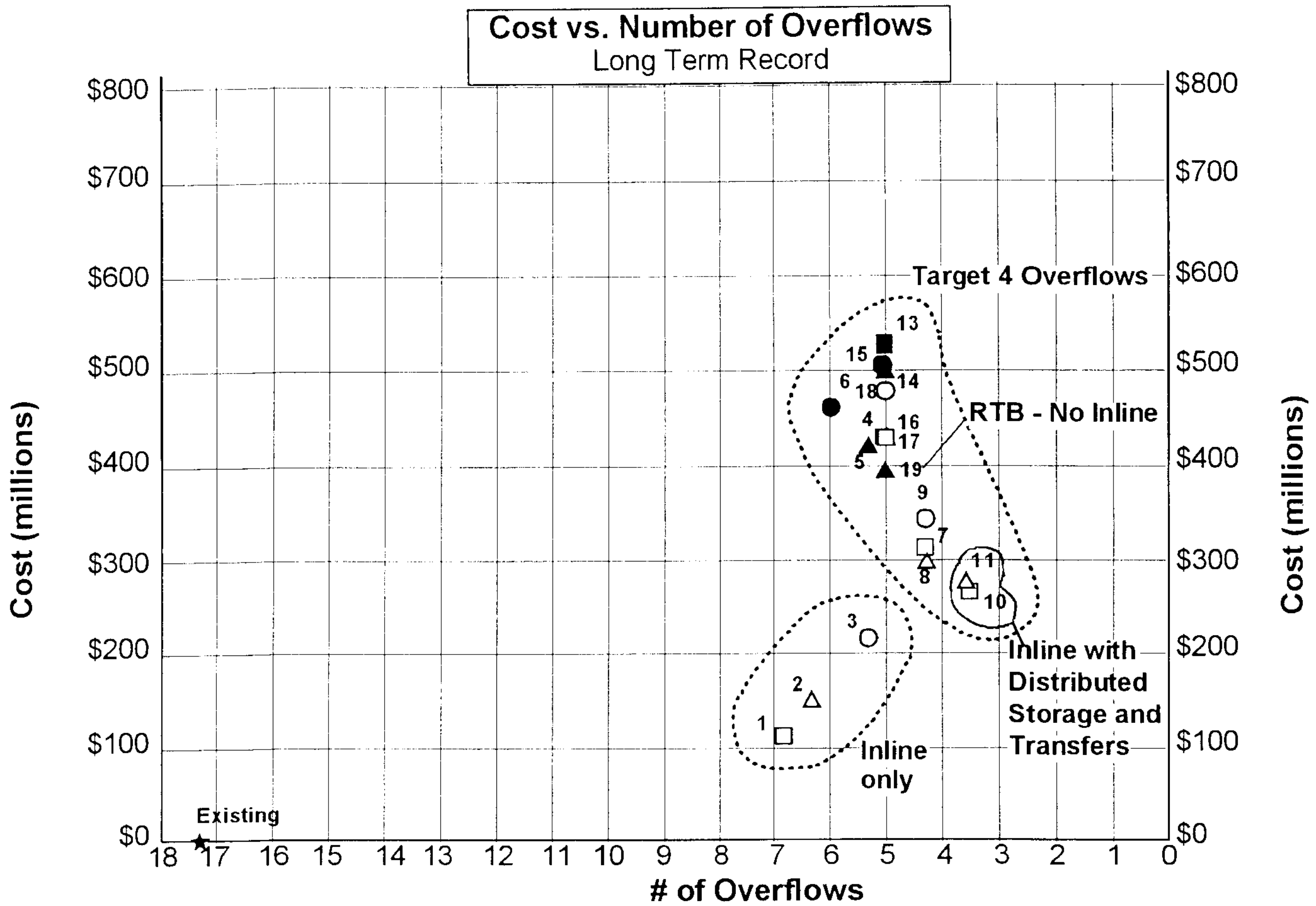
Figure 6-5 illustrates the trade-off between performance and cost of the 16 options which were design to meet a target of 4 overflows per year. The options can be divided into two groups; those without in-line storage (Plans 4-6 and 13-15) and those utilizing in-line storage (Plans 7-12 and 16-18).

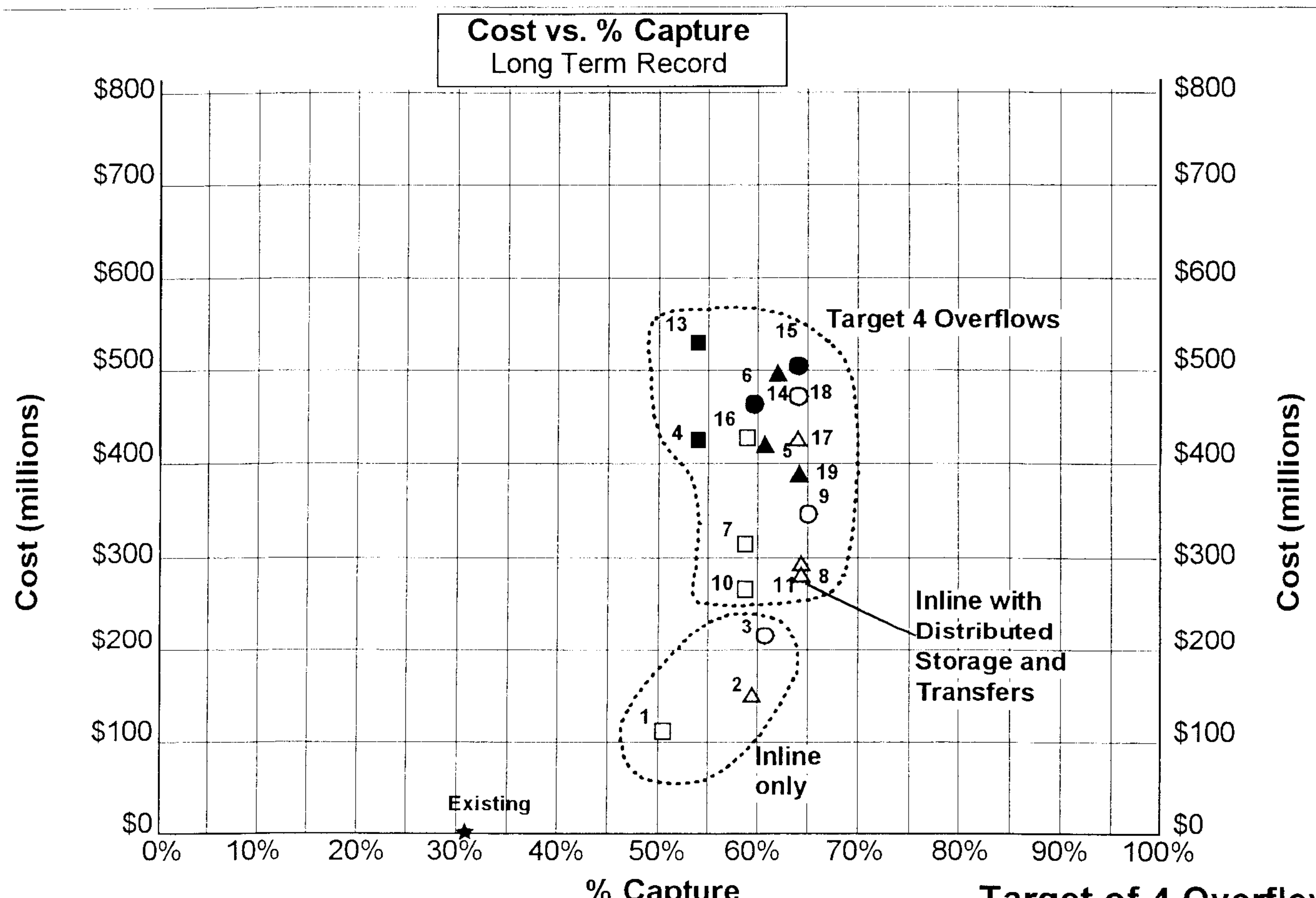
Of those plans not using in-line storage, the dominant option appears to be the use of high-rate treatment (RTBs). The regional tunnel options appear to be the least cost-effective.

The use of in-line storage, however, provides two major benefits:

- the cost is reduced by approximately $120 million; and
- there is an increase in the performance in terms of reduction of the average number of overflows by 1 to 1½ overflows per year.

The most promising of these in-line storage options appears to be the ones which blend in-line and distributed off-line storage with selected transfers ($270 to $280 million). As noted earlier, however, the selected transfers may be more operationally difficult that the





Target of 4 Overflows

(Plan 4 - 19)

Figure 6-5

#-%capt
s\01\0510

simpler no-transfer options. In terms of percent capture, all the options appear similar with those plans using 825 ML/d as a dewatering rate having a slightly higher percent capture.

In terms of compliance with the fecal coliform objectives for the river, none of these options will be significantly different from those with in-line storage only. Therefore, compliance to river quality objectives is likely not a governing factor in selecting one option over another.

In general, it will cost between $270 and $300 million to develop an option which will meet the target of 4 overflows for the representative year using in-line storage and a mix of off-line storage and transfers. Integration of basement flooding relief to provide greater in-line storage could reduce this cost substantially. If in-line storage is not used, then the cost may increase by $100 million to develop a high-rate treatment base system or up to $120 million if only distributed off-line storage is used.

6.4 MEETING 0 OVERFLOWS/YEAR

One of the goals of developing candidate options was to develop and cost an option which would have 0 overflows in an average year (see Table 6-4 for these options). The representative year (1992) was used to design various distributed storage. When the long-term performance modelling was done, it was realized that these options only realized a performance of 2 to 2½ overflows per year and only 74% capture. It was therefore determined to consider two more targets, i.e., one overflow a year for the long-term record (which is equivalent to about 85% capture and 0 overflows per year for the long-term record).

6.4.1 Representative Year

Three alternative dewatering rates (600 ML/d, 825 ML/d, 1,060 ML/d) were considered and a screening model was used to estimate the size of storage required in each district. The planning model performance evaluation for the representative year showed a slight difference in some districts, however, the performance evaluation showed that the number of overflows and percent capture are very close to the targets of 0 and 100%. The long-term record analysis, however, shows a very different picture and is discussed below.

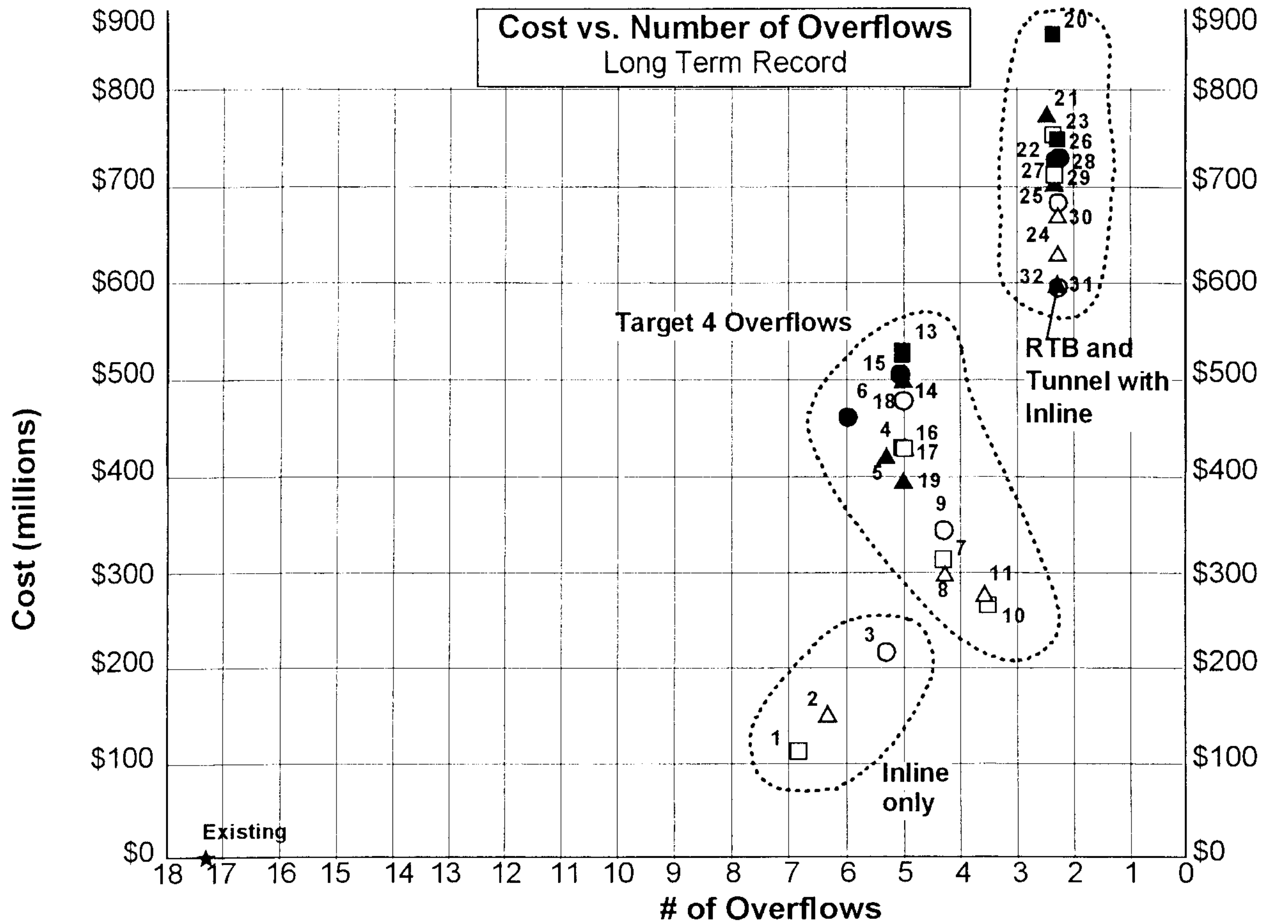
Plans 20-22, using off-line storage show that even building considerable storage at the cost of $725 to $900 million would not result in 0 overflows or 100% capture for 50% of the years in the future. Figure 6-6 shows these trade-offs between cost and performance.

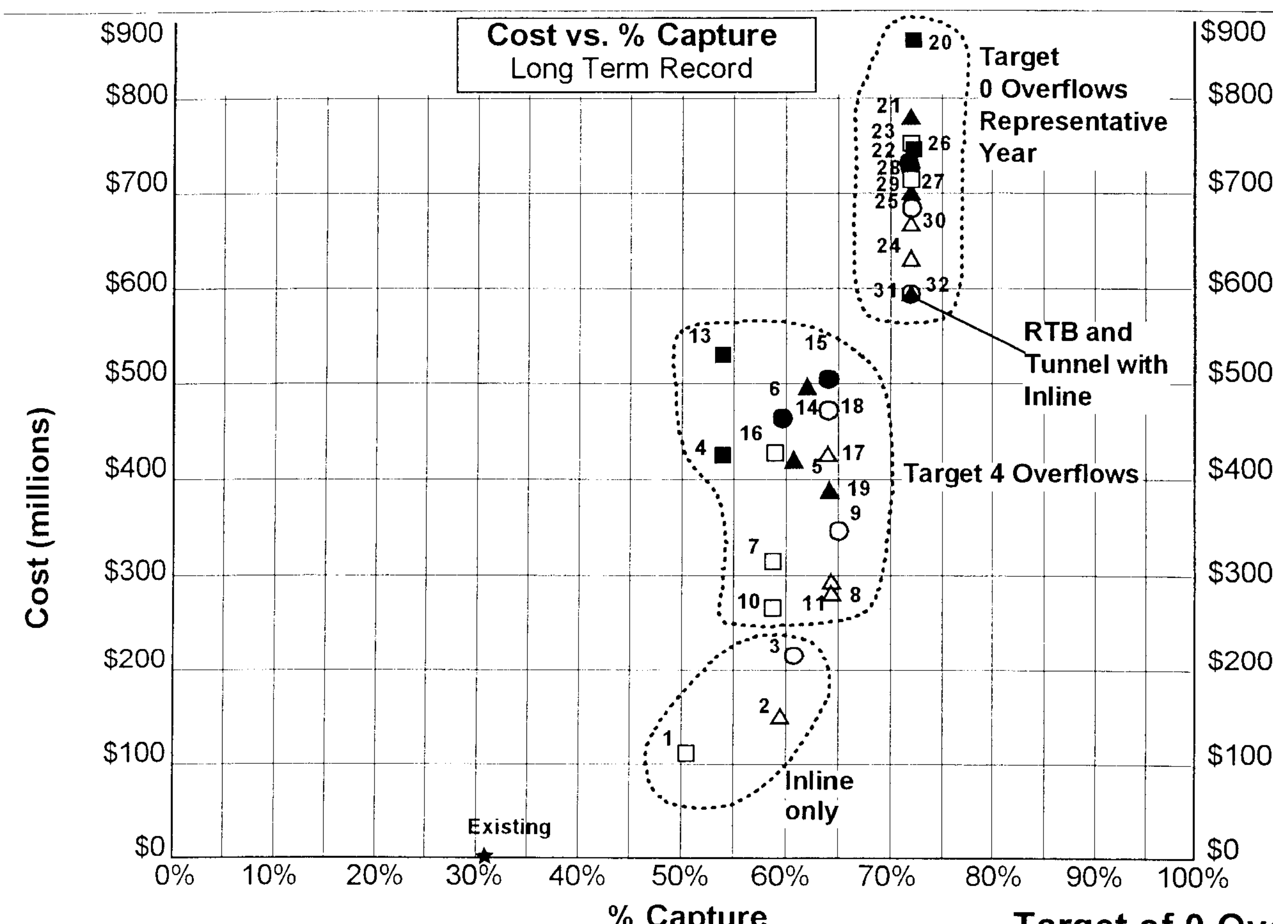
When in-line storage is used to reduce the required storage volume for off-line storage (Plans 23-25), the cost can be reduced to a range of between $630 and $750 million. Again, the same three dewatering strategies as discussed above were considered and the results are very similar.

Alternatives 26 to 28 illustrate a tunnel/transport storage option which used 0 overflows in 1992 as a target performance factor. For the long-term record, however, the performance was very similar to the above distributed storage option in which 2.3 overflows per year could be expected and a 74% capture rate. The costs are still considerable, between $620 and $700 million. When in-line storage is used to reduce the size of the tunnel (options #29 to #31), the cost can be reduced to a range of $592 to $716 million.

A high-rate treatment design was done for each district to test a plan where no overflows in the representative year would go untreated (Plan 32). Only the 825 ML/d dewatering strategy at NEWPCC was considered for this option. The cost of this option to obtain a similar performance to those others described above is about $595 million.

To reduce the number of overflows from roughly 4 to 2 per year involves a considerable incremental expense. The additional cost for this reduction of 2 overflows is roughly $300 million. The best options in order to meet this 2 overflow performance result are either the high-rate treatment (RTB) or a regional tunnel. These options both cost roughly $600 million. Another promising option for future considerations would be to combine RTBs with in-line storage. At this time, this has not been analyzed. This could be considered in Phase 4, if the Phase 3 Workshop provides such direction.





#-%capt
s\01\0510

Target of 0 Overflows Representative Year (Plan 20-32)

Figure 6-6

6.4.2 Long-Term Record

1 overflow/year (Meeting 85% Capture)

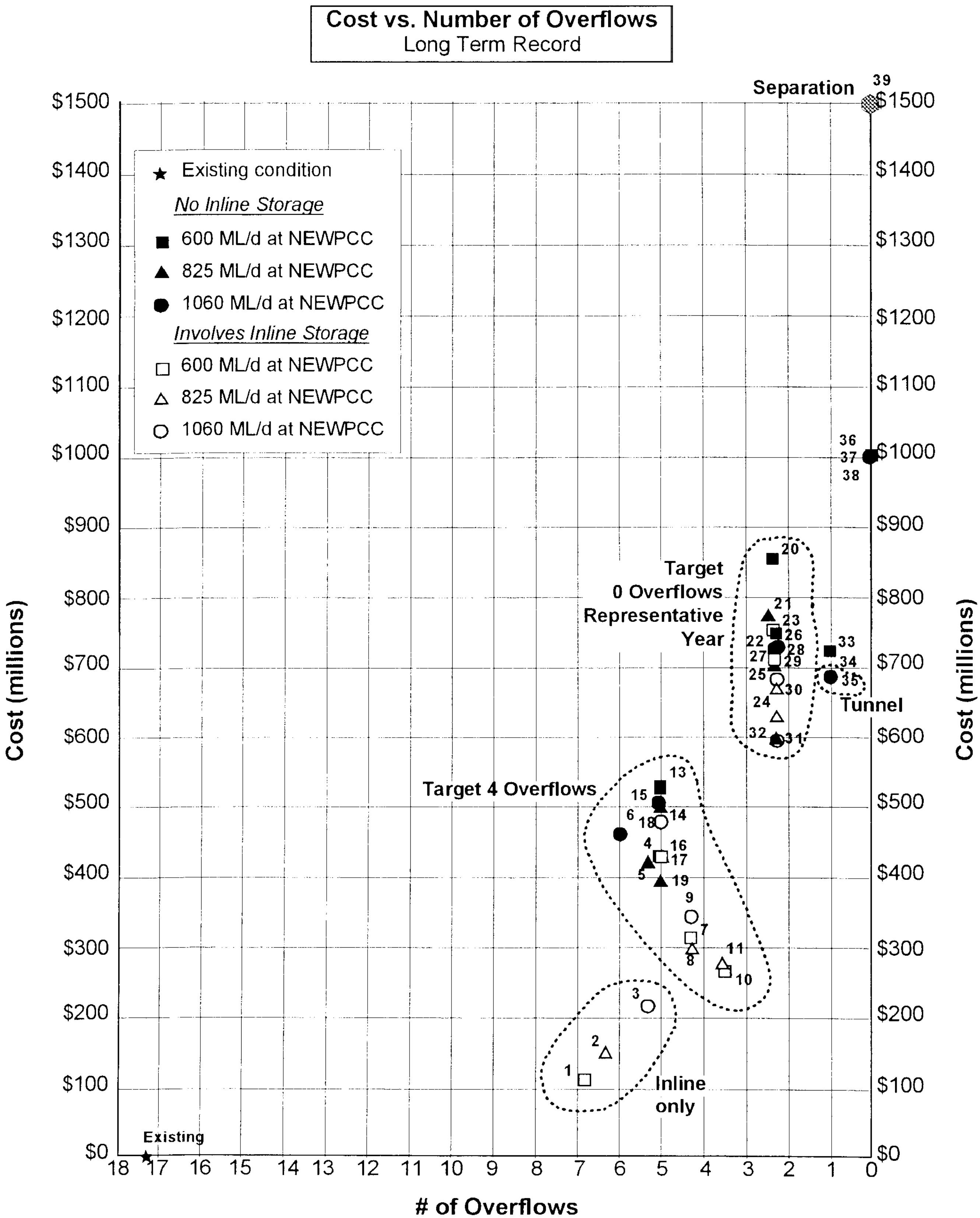
One overflow per year for the long-term record produces treatment to 85% of the combined sewage. Three different dewatering strategies were reviewed for these alternatives (options #33 to #35). The range of volume of the tunnel is between 825,000 m^3 and 1.2 million m^3, and the cost ranges from $685 to $725 million. Meeting this long-term performance target of 1 overflow results in a percent capture of about 84 to 85%. There may be potential to reduce the size of the tunnel by combining with in-line storage, however, the cost saving would likely not offset the $100 million (cost of in-line storage). Tank storage as an alternative to tunnel storage was considered costly at these massive storage volumes.

0 overflow/year

Three transport/storage tunnels without in-line storage alternatives were considered which would meet the long-term performance of 0 overflows and 100% capture (Plans 36 to 38). The size of these tunnels would be massive, ranging from 2 to 2.5 million m^3 and the cost would range between $1,000 and $1,030 million. For tunnels of this size, reducing the volume by 200,000 m^3 to 300,000 m^3 by combining with in-line storage would not be cost-effective. Distributed tank storage and even RTB would also require large amounts of land, which would not all be available on public property.

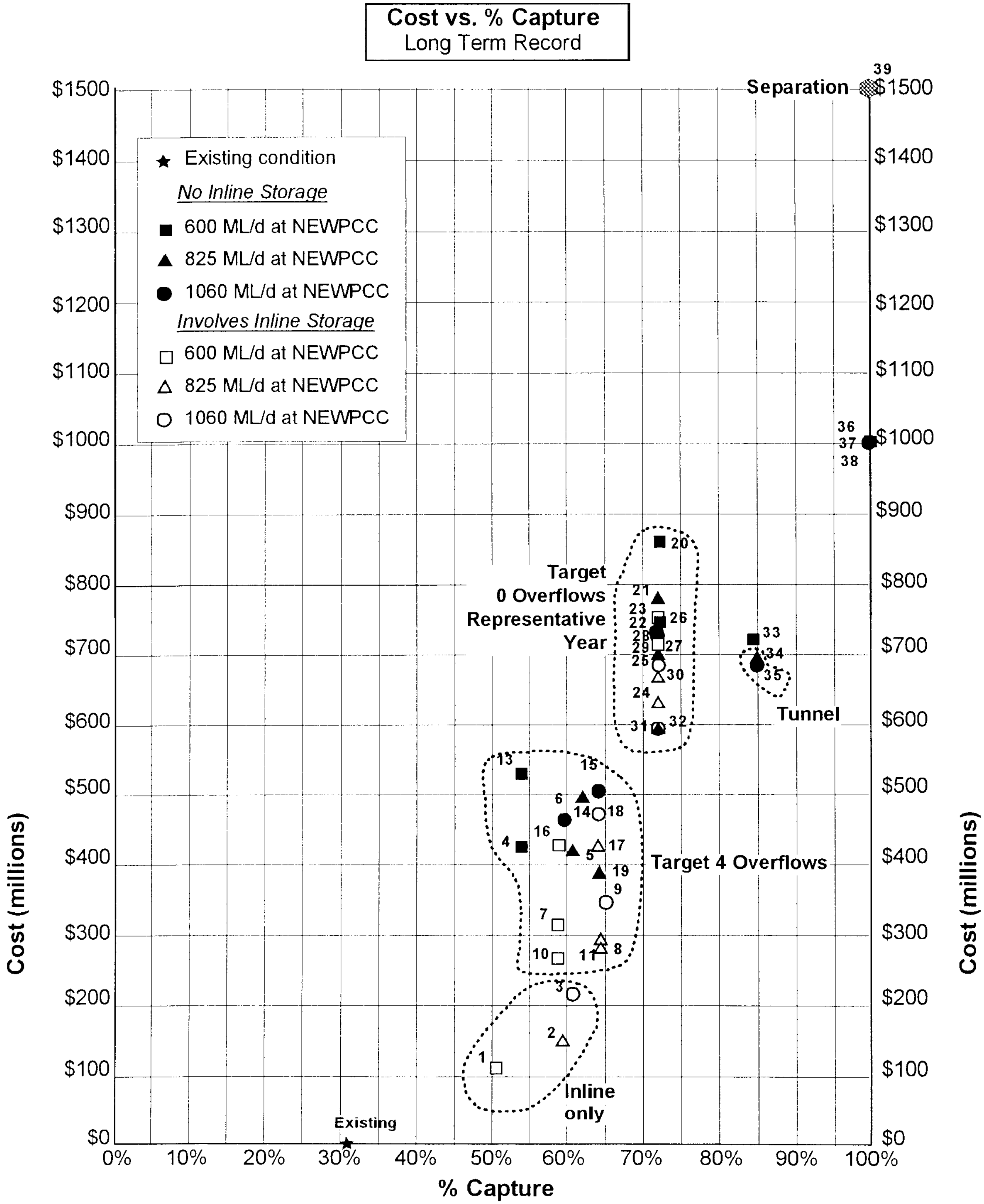
6.4.3 Evaluation of "0" Overflows Per Year Plans

Figures 6-7 and 6-8 show the cost/performance trade-offs for the representative year and the long term. **The cost for these plans range from $595 to $1,500 million. For all conditions, the cost to gain a reduction of about 2 to 4 overflows/year (4 to 0 or 1) is very high, i.e., about $150 to $400 million per unit overflow reduction**. The cost/benefit relationship of these marginal improvements is questionable.



Target of 0 Overflows - Long Term
Number of Overflows (Plan 33 - 39)
Figure 6-7

#-%capt
s\01\0510



Target of 0 Overflows - Long Term
% Capture (Plan 33 - 39)
Figure 6-8

#-%capt
s\01\0510

For these options to virtually eliminate CSOs, the plans involving high rate RTB(s) and tunnel transport/storage appear to be most economical. For 0 overflows, the tunnel/transport storage plan is most cost-effective, due to the massive storage involved.

6.5 SEPARATION

The separation plan would achieve virtually 100% capture of combined sewage (sanitary sewage portion, not stormwater), and would eliminate CSOs, but at a cost of about $1.5 billion. In terms of compliance with objectives, other plans achieve equivalent or better results at less cost. Except for special cases, the prevailing practice in CSO control is to consider area-wide separation as cost-prohibitive.

6.6 FLOATABLES

A plan involving the capture of floatables would only be relevant if floatables (for aesthetic issue) were the primary control issue with respect to CSOs. There is no indication that this applies on an area-wide basis. If this was the case, control of floatables from storm sewers would also need attention. **The cost of such a plan for the overall CSO system would be in the range of $30 to $110 million**. As discussed in Section 5.7, there are likely a few districts where special attention may be warranted, e.g., to deal with capture of "sharps". These districts could be addressed on a site-specific basis.

6.7 OTHER EVALUATION CRITERIA

As well as considering the relative performance or capability of the different plans to control CSOs, it is necessary to consider other important characteristics in the evaluation of the alternative plans. These other evaluation criteria include the following:

- Cost
  - The total capital costs and operation and maintenance costs are important considerations as is cost effectiveness. Any one of the control plans is a massive public works program in terms of cost implications. As such, it would be a major factor in overall public works programming for the City of Winnipeg.

- Cost Effectiveness
  - Cost-effectiveness could be considered in different ways. CSO control could be considered in terms of its effectiveness in delivering benefits relative to other potential public works programs or the alternative plans could be considered in terms of their relative cost-effectiveness. In terms of the latter, the optimization of existing infrastructure through the use of in-line storage is relatively very cost-effective, compared to the other structurally-intensive options. In terms of effectiveness, overflows can be reduced to an average of 5 to 7 over the system by using in-line storage. The cost per overflow to achieve increments of better control, say to 4 or even 0, increases sharply.

- Environmental Benefits
  - The effectiveness of the different plans in providing benefit should be a key factor. The measurement of benefit from CSO control is very difficult. The frequency of compliance with objectives can be considered as a surrogate measure of environmental benefit, assuming that the objectives are based on protection of a beneficial use. However, the microbial objectives, which are most relevant to CSO control, are only approximately linked (through epidemiologic science) to the protection of human health. The degree of compliance is only marginally improved with any of the control plans, on average, although some are more effective on improving compliance during and shortly after runoff. The benefits in terms of reduced health risk are immeasurable for all plans.

    The plans offer different degrees of improvement in reducing the number and volume of overflow. This represents less pollutant loadings to the river and less aesthetic insult.

- Operations
  - CSO control plans will add significantly to the operational complexity of the wastewater collection and treatment system. The technologies involved are reliable and have proven to be practicable and considerable automation can be built into the operation. Wet weather controls are operated intermittently, however, and will add to the complexity and cost of operations. Some technologies, such as in-line storage, are relatively simply while high-rate treatment would be expected to require more attention.

- Constructability
  - Some plans will be easier to construct than others. A large regional tunnel may involve difficult mixed-face soil conditions and incur higher costs. Fixed weirs for in-line storage will involve a major portion of the street rights-of-way and could involve conflicts with existing utilities. All plans should, however, be practical for construction.

- Staging/flexibility
  - The ability to stage the work will be an important advantage. District-specific controls, such as in-line storage or off-line storage tanks, or high-rate treatment can be implemented in stages and will deliver improved controls as these are installed. Separation could be staged on a district-specific basis. Regional tunnels are less suited to deliver progressive benefits as large sections of the tunnel would need to be constructed to achieve benefits.

  - Flexibility is provided by the in-line storage plan. Such a plan could be upgraded, when and if required, by the addition of further controls such as off-line storage or high-rate treatment. In contrast, selection of a separation plan would involve a long-term commitment and would offer little flexibility.

- Potential to Affect Basement Flooding Protection
  - All of the plans should protect the existing level of basement flooding protection. In-line storage, with inlet restriction controls, would offer some improved protection.

Separation would have the advantage of a significant improvement in basement flooding protection, in that street runoff would be diverted from the existing combined sewer to a new storm sewer. It would also avoid the need to install additional relief piping in those districts without storm relief sewers at present (nineteen districts and an estimated cost of about $125 million).

- Public Acceptance
  - Public acceptance of the control plans will involve considerations such as compatibility with land use, safety (chemicals), aesthetics of the control structures, odour potential, and community disruption.

    In-line storage should not interfere with existing land use except in the immediate vicinity of the fixed weir installation. No land acquisition should be required and no chemicals are involved. A regional tunnel is similar. Off-line storage will require access to public property, such as parks, schoolyards, etc. and probably some property acquisition. This would be similar for high-rate treatment. Separation would involve wide-spread disruption over a very long period of time since new storm sewers would be installed in built-up areas over the entire combined sewer area. Storage facilities will likely raise odour concerns, however, experience elsewhere indicates this is not a serious problem.

- Affordability
  - The question of affordability of CSO control raises a number of questions:
    - How would a plan be financed (debt-finance, pay-as-you-go, etc.)?
    - What would be the effect on wastewater bills (residential, commercial, industry)?
    - How long an implementation time period would be involved?
    - How much is the public willing to pay for the improved CSO control?

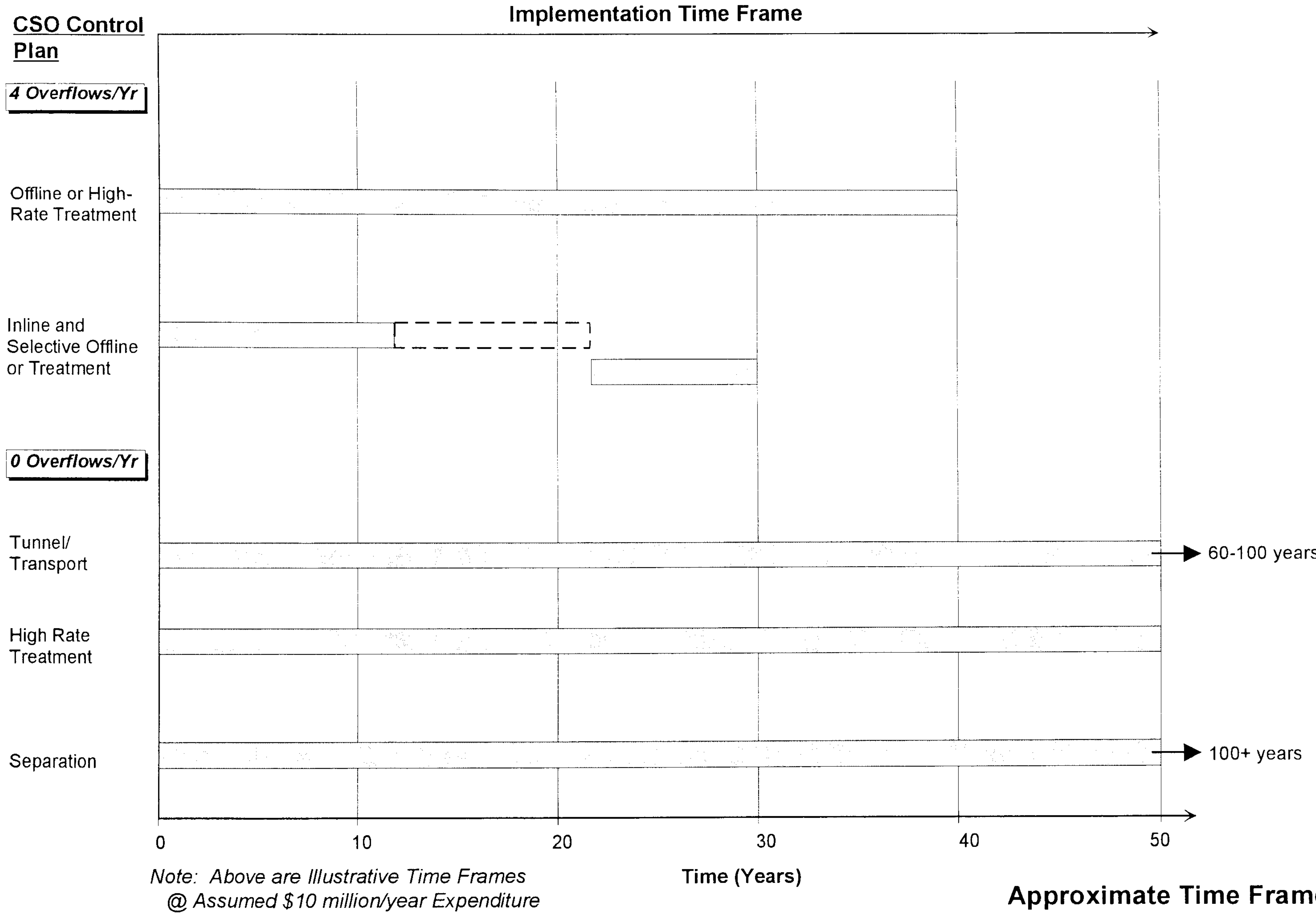
    The City of Winnipeg appears to be moving to a "pay-as-you-go" system. To date, the costs of upgrades to the wastewater system have been recovered largely from the sewer bill, with some frontage-based levies. Most cities implementing CSO control programs have used long time periods, i.e., 25 years or more.

Any of the control plans would involve a major commitment. **Figure 6-9** illustrates approximate timeframes for alternative plans, using an assumed annual rate of expenditure of $10 million. Timeframes extend from about 20 years to in excess of 100 years.

A survey will shortly be done asking the public for their willingness to pay for CSO control. A limited response from questionnaires picked up by attendees at several public events, to date indicates most people are willing to pay more on their annual sewer bill to control CSOs. The largest group of respondents stated a willingness to pay an additional $26-50/yr. This could translate to a significant CSO control program, perhaps $200 million or more, depending on the method of financing, if this is an accurate reflection of the overall willingness to pay.

- Political Acceptability
  - The acceptability of control plans to City decision-makers will need to be tested and will likely depend on expenditure priorities, available public funding, provincial cost-sharing benefits, and public attitude.

- Regulatory Acceptability
  - Manitoba Environment is responsible for advancing surface water quality objectives. CSOs are an issue with respect to compliance with microbiological objectives (fecal coliform) and are also a policy issue in that these involve discharges of raw sewage to the rivers. Manitoba Environment will need to consider questions such as the following:
    - Should the dry weather objectives apply for wet weather conditions?
    - After WPCC disinfection, compliance during dry weather will be achieved and, overall compliance will be high (over 90% of the time), WWF controls will add slight improvements to the overall duration of compliance.
    - Should there be wet weather waivers of coliform objectives?



Approximate Time Frames for Different Plans

Figure 6-9

optsced
s\01\0510

- What degree of compliance with the coliform objective is deemed adequate (90%, 95%, 100%?).

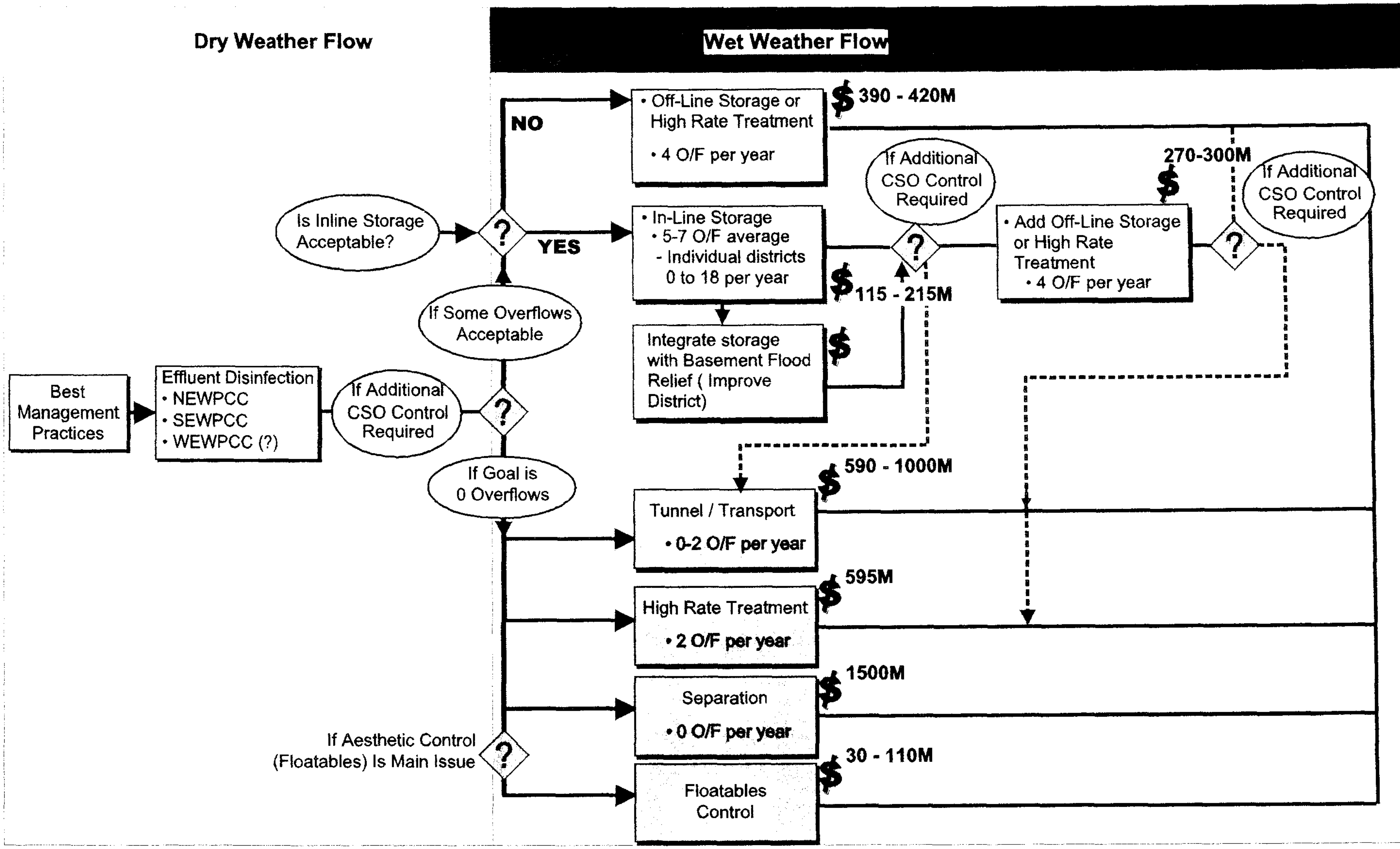
6.8 OVERVIEW OF CONTROL PLANS

Figure 6-10 provides an overview of the control plans. Costs and performance characteristics are shown for those plans that appear to be the best in their group.

In evaluating the plans, a number of questions arise which influence the potential identification of a preferred plan. These questions include:

- Is additional CSO control, i.e., beyond the existing system, required?
  - The existing system, once effluent disinfection at three WWPCs is in place, can meet the provincial water quality objectives (microbiological) for primary and secondary recreation for a high percentage of the time, i.e., in excess of 50%, however, during and shortly after runoff events, the CSOs do contribute significantly to the exceedance of these objectives. CSO's occur during virtually all the rainfalls in excess of 4 mm. The Plan Winnipeg states that "*The City shall maintain the highest practical and cost-effective level of river water quality consistent with the natural characteristics of our rivers and in accordance with water quality objectives established for the Red and Assiniboine Rivers.*"

- Are some overflows per year acceptable?
  - It makes a huge difference in the overall cost of CSO control if some overflows are acceptable, as compared to the virtual elimination of overflows. The U.S. EPA had indicated that controlled overflows to 4 on average (with up to 6 allowed) is a "presumption" of adequate compliance with water quality standards. Manitoba does not have such guidelines. Figure 6-10 shows that if some overflows are acceptable, control plans are possible in the cost range of $115 million to $420 million. These plans also offer significant flexibility in staging. To illustrate, in-line storage could be implemented, with CSO control integrated with ongoing basement flooding relief



Note: Costs are in millions (M) and are cumulative

Potential CSO Management Strategies: Phase 3
Figure 6-10

s:\01_files\0510_wrd\38_cso\ptctrpln5.Dsf programs (at an approximately cost of $150 million) as a first-step control program. After evaluation, additional off-line storage or high-rate treatment could be added subsequently in the next phase, to reduce the overflows even more, to an average of 4 or even 2 per year, if this was deemed justified. The added costs would be in the range of $150 to $500 million.

- Is the goal to eliminate overflows?
  - If it were to be agreed that the goal is to virtually eliminate CSOs, it must also be recognized that the costs will be in the range of $685 to $1,500 million. Such massive costs raise questions of affordability, willingness to pay, etc. Some options, such as tunnel storage/transport would require this direction at the outset so the design would be based on providing such storage. High-rate treatment design and safety requirements, including land, would thus be based on the ultimate requirement. The staging of a separation plan would probably begin with the unrelieved districts but would require careful consideration beyond this. A regional tunnel plan would probably begin near the NEWPCC. It would be a very long time before improvements in CSO control would be tangible.

- Is in-line storage an acceptable and practicable technology?
  - In-line storage is a major opportunity to gain significant improvement in CSO control at relatively modest costs. Even allowing for fail-safe operations, the use of in-line storage is very cost-effective compared to constructing equivalent new storage or treatment. In-line storage represents a value of about $120 million. There may be some operating concerns (odour, sedimentation, etc.) with in-line storage but these can be tested to determine how best to address the issue. They do not appear to be significant, based on experience elsewhere. The use of in-line storage gives every indication of being a practical cost-effective technology for the City of Winnipeg.

- Is control of floatables a key issue?
  - If the aesthetic impacts of CSOs were a central issue, controls could be put in place for a relatively low cost, i.e., $30 to $110 million. It does not appear from inspections of the river or from a limited testing program, that floatables from CSOs are a major

visible impact in most districts. Street runoff to the rivers occurs in separate districts as well and would still occur if the combined sewers were separated. In other cities, floatables control typically is used where there is significant beach activity.

The answers to these questions are policy-related and involve public value-judgements. Figure 6-10 shows the applicability of potential plans, depending on the answers to these questions.

In considering their position, Manitoba Environment will also consider whether the benefits of controls justify the costs (Williams 1988).

Many of these criteria are qualitative, i.e., value-judgements and require input from sources external to the study team, i.e., the public, politicians, or the regulatory agencies.

It is expected that the Phase 3 Workshop will include a review of these criteria for subsequent use in evaluation of those plans considered appropriate by the Phase 3 Workshop participants.

7. THE NEXT STEPS

The Phase 3 Workshop participants will review the range of potential control plans (39 in total) and review the technical basis for their definitions, their performance and their technical practicability. In the process, revisions to the plans may be made or new plans may be defined. The Workshop will also provide guidance on additional technical analyses required. River quality modelling will be done on the short list of plans in Phase 4. As well, there may be further work required, for example on RTBs, hydraulics of fixed weirs, and other issues to be discussed at the Workshop.

The Workshop will result in the identification of a range of alternative control plans that are technically practicable. It is expected that a number of plans will be considered to be of little further interest on the basis that there are physically similar plans that offer better benefits at lower costs. In such cases, the best plan(s) representative of that category of technology and/or performance will likely be identified for further review. A much shorter list of candidate plans (that merit further consideration) should emerge from the workshop, if the Workshop participants endorse this action.

Plans covering the full range of different performance levels or degrees of CSO control will be carried forward into Phase 4. This may involve 5 or 6 different types of plans. The intent is to outline an available range of potential plans and to present their various characteristics for review by the public, City policy-makers and the regulatory agencies. These plans will be presented for further evaluation in Phase 4, providing opportunity for public, political, and regulatory judgements (to the extent that the study team is able to determine these).

Because there is a substantial amount of time available between the conclusion of Phase 3 and the anticipated CEC hearings, there is an opportunity to re-assess Phase 4 activities with respect to how best to obtain these important external inputs and opinions. A Phase 4 work plan will be developed following the Workshop.

Table 6-2
Evaluation of Candidate Options Maximizing Existing Infrastructure

	Plan Number	Dewatering Rate at NEWPCC ML/d	Treatment Cost Millions	Interceptor Cost Millions	Inline Storage Cost or Regulator for Offline[1]	Required Offline Storage Volume m³	New Structural Cost Millions	O & M Cost PV	Total Cost Millions	1992 Number of OF	1992 % Capture	Longterm Median Number of OF	Longterm Median % Capture	Longterm MAX OF	Remarks
Existing Situation															
Existing	0	825							$0	20.8	40%	17.3	32%	29	
Optimizing Existing Infrastructure															
Inline Storage	1	600	$15		$100				$115	7.2	77%	6.8	52%	14.7	
	2	825	$36	$15	$100				$151	6.2	84%	6.2	59%	12.7	
	3	1060	$70	$46	$100				$216	5.3	87%	5.3	62%		