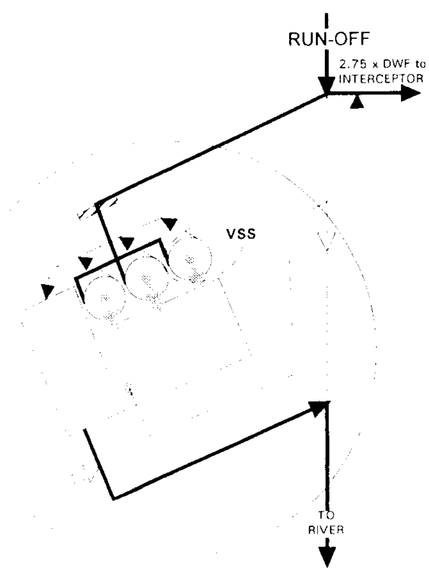
City of Winnipeg
Water and Waste Department

Combined Sewer Overflow Management Study

PHASE 2 Technical Memorandum No. 3

CONTROL ALTERNATIVES/ EXPERIENCE ELSEWHERE



Internal Document by:

WARDROP Engineering Inc. and TetrES CONSULTANTS INC.

In Association With:

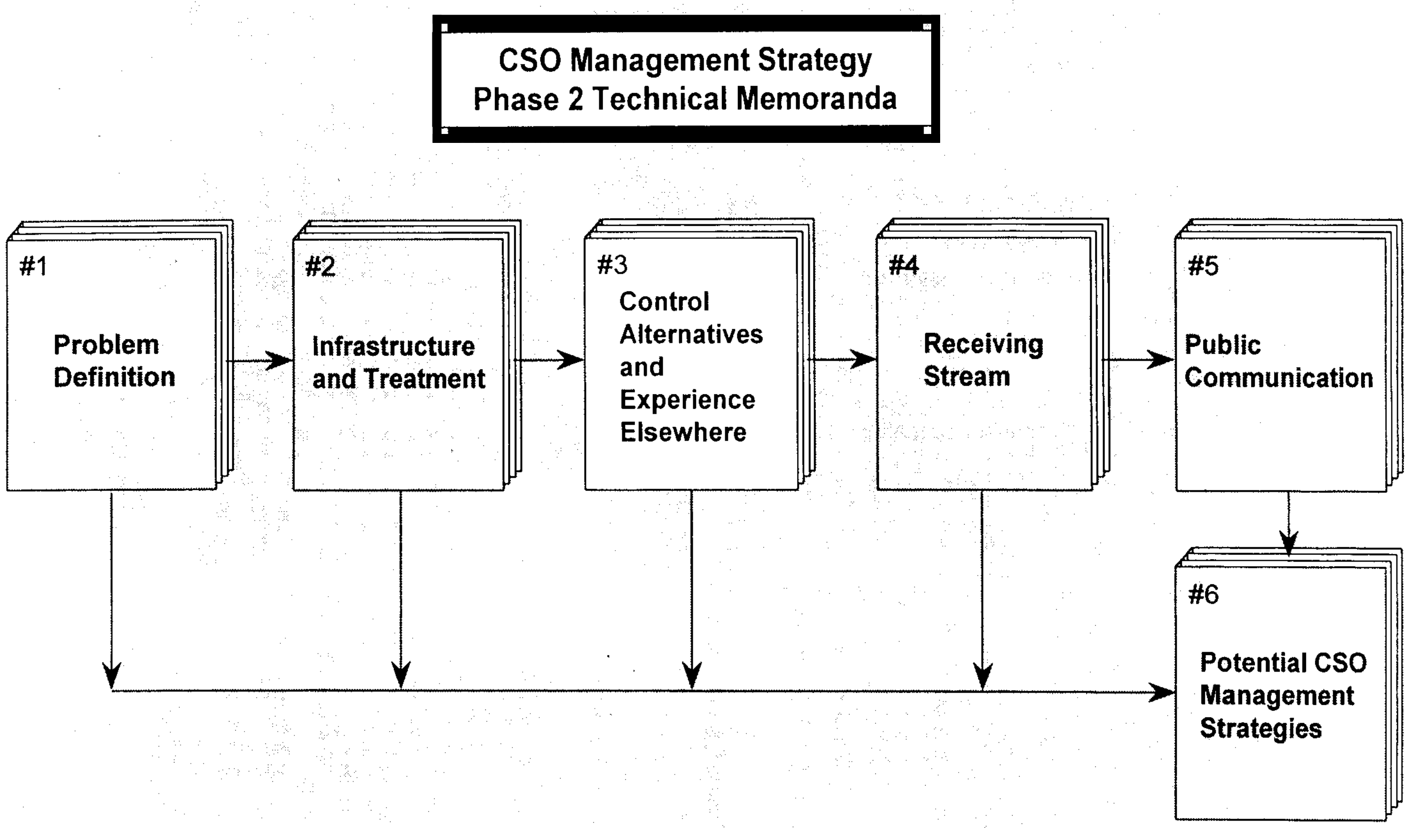
Gore & Storrie Limited and EMA Services Inc.

August 1995

PREAMBLE

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

This particular TM is part of a group of Phase 2 reports as shown in the schematic.



Each of the Phase 2 TMs draws on information developed in the prior Phase 1 TMs. In addition, the Phase 2 TMs document information and study analyses sequentially. Ideally, therefore, the TMs should be read in the sequence shown.

ACKNOWLEDGEMENTS

The Study Team acknowledges, with sincere appreciation, the contribution of many individuals and agencies consulted in the course of Phase 2 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 Gore & Storrie Limited/EMA Services Inc.

G. Rempel, Project Manger
R.J. Gladding, Assistant Project Manager
R.R. Foster
D. Morgan
N. Szoke
G. Steiss
S. Quigley
R. Rempel
G. Mohr
D. Dagg
J. Anderson
P. Nicol
M. Parente
W. Clarke
R.G. Skrentner
D. Weatherbe
G. Zukovs
S. Black
L. Thompson
C. Rowney

City of Winnipeg: Water & Waste Department Project Management Committee

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

TABLE OF CONTENTS
CONTROL ALTERNATIVES

LIST OF ILLUSTRATIONS

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

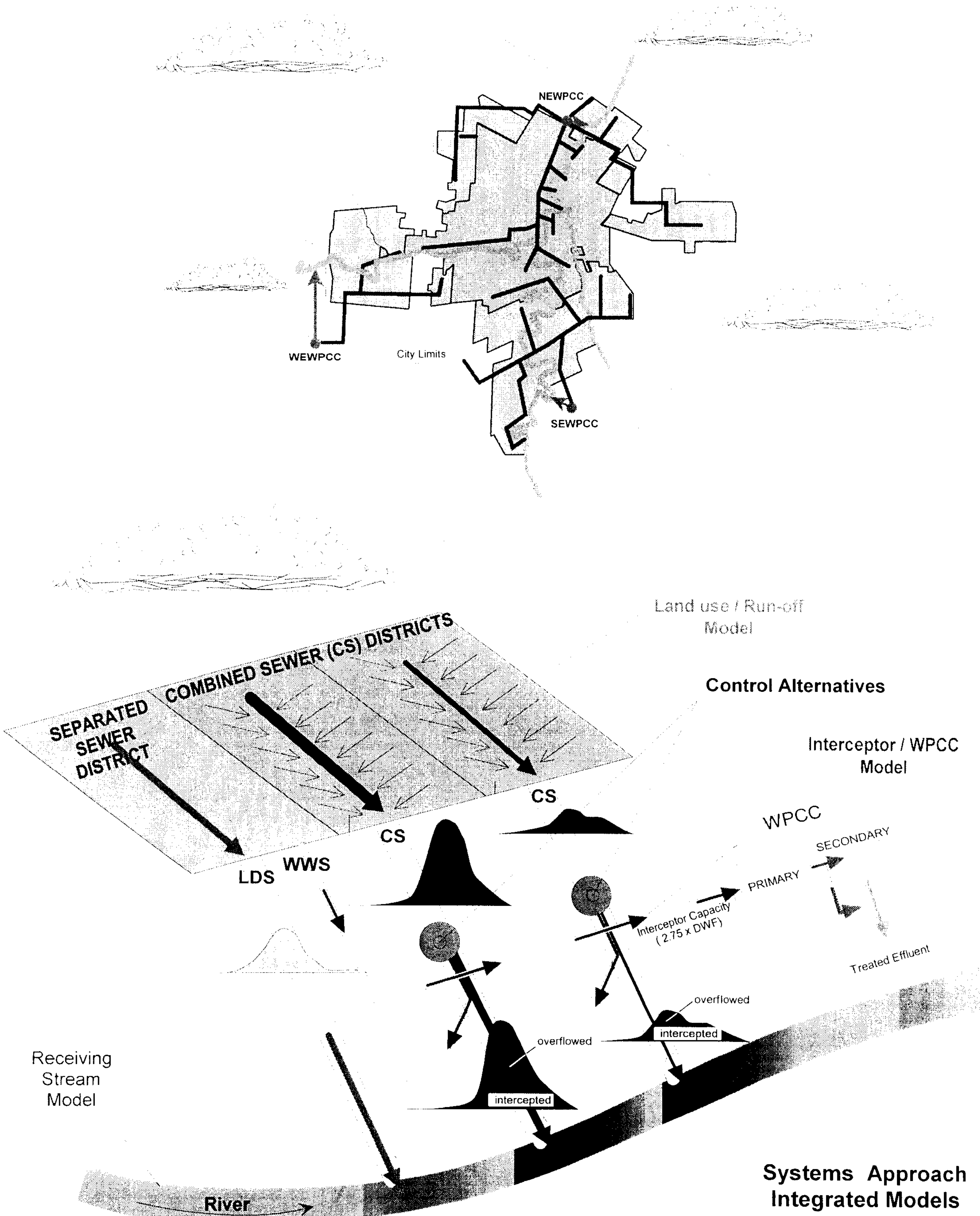
This technical memorandum discusses control alternatives assessment done in Phase 2. A review of regulations and technical experience elsewhere was conducted to determine the direction of other jurisdictions in controlling overflows. A screening of all potential technologies was done in order to develop the candidate options which are most appropriate to Winnipeg. The existing infrastructure was reviewed in order to determine the minimum structural controls necessary to optimize the system. The cost to increase the interceptor rate and utilize inline storage in the combined sewer districts was estimated. Various structurally intense storage and treatment systems were conceptually assessed for the Winnipeg system and costs of implementation were estimated. These options are:

- storage/central treatment;
  - regional tunnel (elimination of all CSO)
  - offline tanks or tunnel storage (capture of most frequent storms)
- high rate treatment (end-of-pipe);
- separation; and
- floatables control.

The potential for real time control to enhance these options is also discussed.

A control system model was developed to assess the impacts of the combined sewer systems as well as the land drainage, sanitary sewers, interceptor systems and water pollution control centres (WPCCs). The model tracks the volumes of wastewater (hourly from May to September) between the overall system and the receiving streams. The purpose is to produce pollutographs of waste loading to the receiving stream model as well as to estimate the number and volume of CSOs to the river for the various control alternatives (see Figure 1-1). The model results were compared to the City's FAST alarm data for overflows.

The results for the assessment of various control alternatives are compared and potential monitoring, modelling and pilot demonstrations discussed.



Systems Approach
Integrated Models
Figure 1-1

c:\01_files\510a38\x5_cntrl\CNTRLSYS.DRW

2.0 OVERVIEW OF TECHNOLOGIES

CSO control technologies range from relatively simple measures, such as improved system operation, to very complex stormwater management systems and/or capital-intensive remediation works, such as separation of the existing combined sewer systems. A very wide range of control technologies is potentially available with an associated wide spectrum of cost implications. There is, therefore, great difficulty in selecting the best control technology to address the relevant local water quality issues in the most effective manner. CSOs differ widely in the specific problems they represent, depending on the site-specific regulations, water use, and actual water quality impacts. It is an important fact that CSO control technology is also evolving. Treatment of wet weather flow represents different challenges than those associated with conventional wastewater treatment. CSO flow rates vary greatly in time and many of the high-rate technologies have simply not been proven under long-term service with combined wastewater. New technologies are being tested under many different circumstances and with different treatment objectives.

It is necessary to review the available potential CSO abatement technologies and to evaluate these for their particular effectiveness for the specific combined sewer area in Winnipeg, considering such factors as feasibility, complexity, proven experience, costs, land requirements, environmental and public acceptance considerations.

For this study, a preliminary analysis of potentially applicable technologies for the Winnipeg circumstances was provided in Phase 1 TM #5. For Phase 2, a process for "screening" control technologies was applied to identify a "short-list " of the most appropriate CSO control options. These were to be studied further in subsequent phases. In undertaking this evaluation, the study team considered the guidance for screening of CSO control options provided in the Water Environment Federation (WEF) manual practice FD-17 "Combined Sewer Overflow Pollution Abatement" (WEF 1989) and other WEF publications on CSOs. As well, the varied experience of the study team members in analyzing CSO control issues and technologies, in various North American and European jurisdictions, was applied.

Recognizing that the CSO control technology is evolving, members of the study team have attended all the WEF Specialty CSO Conferences, and many other relevant conferences, to keep current on the technologies that might be applicable to the Winnipeg situation. As well, members of the study team have visited a number of CSO control sites where various technologies were in place. This broad experience was used in the screening and evaluation of the control technologies described in the following sections.

In reviewing the potential CSO options, the study team was mindful of the evolving North American regulations pertaining to CSOs. In this regard, the US Environmental Protection Agency (EPA) provides important direction. In the late 1980s, CSOs began receiving attention from the EPA, which put forward a strategy that stated three objectives:

- to ensure that CSO discharges occur only as a result of wet weather, i.e., no dry weather overflows;
- to bring all wet weather CSO discharge points into compliance with the technology-based requirements of the national *Clean Water Act* and also applicable state water quality standards; and
- to minimize water quality, aquatic biota, and human health impacts from wet weather overflows.

In addition, the strategy called upon the states to develop state-wide CSO permitting strategies. The EPA recommended that permits include, as a minimum, the following six control measures:

1. Proper operation and regular maintenance.
2. Maximum use of the collection system for storage.
3. Review and modification of pre-treatment programs.
4. Maximum flow delivery to the treatment plant(s) for treatment.
5. Prohibition of dry weather overflows.
6. Control of solid and floatable materials in CSO discharges.

In 1991, the EPA, concerned that CSO implementation was not proceeding properly, added three additional points to be included in permits for CSOs. These, together with the original six points, became known as the nine minimum controls under EPA policy. The three additional points are:

7. Required inspection monitoring and reporting of CSOs.
8. Pollution prevention, including water conservation, to reduce CSO impacts.
9. Public notification for any areas affected by CSOs especially beach and recreational areas.

Recognizing the significant technical and economic issues associated with CSO control policy, the EPA also initiated a consultative process with key national stakeholders. Subsequently, the EPA published the final combined sewer overflow control policy in the federal registry in April 1994.

The CSO policy represents a national strategy to ensure that permitting authorities, municipalities, regulatory standards agencies, and the public engage in a comprehensive and coordinated planning effort to achieve cost-effective CSO controls that will ultimately meet the appropriate health and environmental objectives. The policy provides flexibility to municipalities to consider the site-specific nature of CSOs and the pollution reduction effectiveness along with the incremental cost of control options in developing long-term plans.

The policy allows for a phased approach to implementing CSO controls to consider a community's financial capability and, further, it provides for the review and revision, if required, of water quality standards and their implementation procedures to reflect the site-specific nature of CSOs and the local ecosystem.

The EPA CSO policy has the expectation that municipalities will:

- initiate the nine minimum controls;
- undertake the development of a long-term CSO control plan which will include the following components:
  - characterization of the combined sewer system and, CSOs and their impacts on the receiving waters;
  - special consideration for sensitive environmental areas;
  - evaluation of a range of alternatives to meet water quality standards;
  - coordination with permitting and water quality standards agencies when selecting control measures;

  - development of a public participation plan to ensure that the general public is involved in the development of the comprehensive CSO control program;
  - development of a schedule for implementation of selected control measures which considers the municipality's financial capability; and
- develop and implement a post-construction water quality monitoring program.

The policy further provides the municipalities with two approaches for showing that its selected CSO controls will achieve water quality standards. These have been called the "presumption approach" and the "demonstration approach". These are defined below:

- "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 four overflow events per year which do not receive minimum treatment (clarification, solids removal, disinfection if necessary); 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.

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

The EPA CSO control policy provides useful guidance for the potential evolution of Canadian and Manitoba policy. In fact, the Ontario policy seems to be patterned after the EPA policy. At present, Manitoba Environment does not have specific permitting policies relating to CSOs or other wet weather flow discharges to the receiving stream. The CEC has, in effect, declared that MSWQO for wet weather flow are under consideration. The CSO study is expected to contribute greatly to the definition of the policy to address these wet weather flow conditions.

2.1 SCREENING OF CSO CONTROL TECHNOLOGIES

The process of reviewing the wide array of CSO control technologies and screening these for the potential application to the particular Winnipeg water quality issues was done through the following activities:

- review of prior evaluation of CSO control options done on individual Winnipeg combined sewer districts;
- a working session of key study team and specialist consultants (who brought awareness of prior local and other external experience to the session), which resulted in a review of the full range of possible alternatives and the collective identification of the most promising options for the local situation;
- identification of options or specific topics requiring additional investigation, field monitoring data, or monitoring through site visits, literature review, etc.; and
- consideration of the potential need for pilot testing for demonstration projects for the identified promising technologies.

For the screening of control options, a list of the broad range of CSO control options was developed (see Table 2-1). In developing this list of potential options, the options were placed in a number of broad categories as described below:

- **Non-structural/best management practices** - these technologies involve relatively low cost operational or functional modifications to existing facilities. Any construction associated with these technologies is generally small.
- **Minimum structural alternatives** - these technologies focus on system optimization, such as elimination of dry weather overflows, and use of existing inline storage in the sewer pipes themselves, and maximum use of the existing treatment plant capacity during WWF.

TABLE 2-1

POTENTIAL CSO CONTROL OPTIONS

RANGE OF CONTROLS	EPA 9 MINIMUM CONTROLS	APPLIED IN WINNIPEG	REMARKS
NON-STRUCTURAL/BEST MANAGEMENT PRACTICES			
Sewer flushing	1		
Catchbasin cleaning	1	✓	City has annual program, not aimed at litter capture
Street sweeping	6	✓	City has limited program
Catchbasin inlet restriction	2	✓	City has applied in some areas for flood protection
Inflow and Infiltration reduction	2,4		
Overland flow attenuation	2		
Roof leader disconnection	2	✓	City had active public education program, 95% disconnected
Chemical addition	3		
Review/implementation of by-laws	8	✓	City has comprehensive industrial waste control by-laws
Industrial runoff control	8		
Water conservation	8	✓	City has implemented strong program
Receiving stream water quality monitoring	7,9	✓	City has long-term program in place
Public Education	9	✓	City has had campaigns for downspout disconnection, lot grading and litter control
Inspection, Monitoring, Reporting of CSOs	7		City has FAST alarm system, needs improving for this purpose

(Table 2-1 cont'd next page)

RANGE OF CONTROLS	EPA 9 MINIMUM CONTROLS	APPLIED IN WINNIPEG	REMARKS
MINIMAL STRUCTURAL ALTERNATIVES			
Flow balancing between districts	2		
Overland flow slippage	2		
Increase pervious area	2		
Elimination of dry-weather overflows	5	✓	City aims to avoid DWO, some occur
Hydraulic control devices	4	✓	City has devices in place but are not effective
Interception optimization	4		
Maximize WWF treatment at WPCCs	4	✓	Existing WPCCs are run at nominal full capacity during WWF
In-line storage	2		
STRUCTURALLY INTENSIVE ALTERNATIVES			
Elimination of dry-weather overflows	5	✓	Some additional effort needed
Surface storage			
Additional conveyance and storage			
Tunnel storage			
Off-line storage (near surface)			
Sub-district separation		✓	Implemented in a few areas as part of basement flood relief
Partial sewer separation			
Full sewer separation			
TREATMENT ALTERNATIVES			
Central treatment (primary)			
Retention treatment basins			
Vortex solid separators			
Screening devices			
Disinfection			
Trash netting			

- **Structural intensive alternatives** - these technologies involve substantial construction of new facilities and include options such as storage/treatment facilities, sewer separation, etc.

A more extensive description of the specific control options in each category was provided in TM #5 of Phase 1.

In considering these potential control options, the working session participants were aware of the need, as an essential starting point, to have a good understanding of the general performance of the existing combined sewer and treatment system with respect to such characteristics as interception rates, flow regulator performance, pumping capacities at the lift station and the NEWPCC, etc. Information of this nature was provided to the participants. A field trip to examine a number of lift stations and the interception points including the flood pumping stations, was conducted to assist in this understanding of the existing infrastructure. It also provided an opportunity to view the potential for these various sites to accommodate different types of new control measures.

With this background information and understanding, the study team considered the list shown in Table 2-1.

Table 2-1 provides a listing of potential CSO control measures, identifies those measures that are included in the EPA nine minimum controls, and then comments on those measures which have already been applied in Winnipeg.

As can be seen from Table 2-1 the City of Winnipeg has undertaken a significant degree of structural and operational modifications to the sewer system, i.e., it is well past the "no-action" status with respect to CSO control. The City has implemented many of the best management practices. The City has also implemented a number of measures in recent years to improve the operation of the combined sewer system and minimize CSOs. Some of these measures include:

- The City has implemented a FAST alarm system that alerts the Operations Department to incipient overflow at the interception point and/or trouble with pumping stations or other malfunctions at the lift station or interception point (discussed in Section 5 of this TM).

- The City has an aggressive program of educating the public on the merits of roof leader disconnection. It is believed that approximately 95% of the downspouts are disconnected in combined sewer districts. In separate sewer districts no downspouts are connected.

- The City has an annual program for cleaning of catchbasins. Approximately 9,000 to 10,000 cleans are accomplished on an annual basis.

- The City has a street cleaning program which concentrates mostly on a spring clean-up to capture much of the sand that has been applied for winter ice control. The City has made an effort to reduce the amount of sand spread on the streets. The quantity has reduced from approximately 140,000 tonnes in 1988/89 to about 75,000 tonnes in 1993/94. About 50% of this sand is recovered through mechanical sweeping. These reductions in applied sand mean less grit is delivered to the rivers through CSOs.

- The City encourages runoff control at source through parking lot storage, catchbasin inlets, etc.

- The City has also implemented structural modifications to improve CSO capture. The treatment plants are operated such that they provide at least primary treatment to the wet weather flows that are delivered to the WPCCs.

- The City monitors dry weather flows in the various districts and endeavours on this basis to assure that all dry weather flow is captured, i.e., no dry weather overflows. There are, however, several problem districts where DWOs appear to exist. Lift stations have been upgraded through the years, i.e., additional pumping capacity and other structural modifications have been made.

- The City has constructed sewer separation in selected districts during the course of basement flooding relief programs. This is done wherever it is economically advantageous. In some areas this has represented substantial proportions of the district being relieved for basement flooding protection.

Table 2-2 shows the screening evaluation of the potential for new controls to be applied in Winnipeg. It is considered that all applicable best management practices will be part of the ongoing program and in fact will be enhanced in the overall long-term CSO program. Table 2-2 shows the evaluation of the potential new controls that could also form part of the long-term CSO control program. Many of these technologies were also described in the Phase 1 TM #5.

The evaluation, as summarized in Table 2-2, was used to identify the candidate controls that should be considered in the screening analyses, which are discussed below.

2.2 CANDIDATE CONTROL OPTIONS

It is recognized that any new control options will build on the current best management practices and will also include the initiation of new best management practices. This Technical Memorandum is intended to focus on structural control alternatives for the long-term program.

Based on the evaluations summarized in Tables 2-1 and 2-2, the candidate control options for further screening analyses are shown in Table 2-3.

There is a logic to how these control options might be implemented in a coherent incremental manner to effect progressively greater control of CSOs. The potential role for these candidate control options in such an incremental control program is shown in Figure 2-1. It is recognized that there are variations of a number of these control options and that the likely long-term program would involve a mix of selected options, i.e., each district will not likely have the same optimum control options.

For the initial screening, these candidates were studied further with respect to their costs, general characteristics, and potential benefits with respect to reducing the number and volume

TABLE 2-2

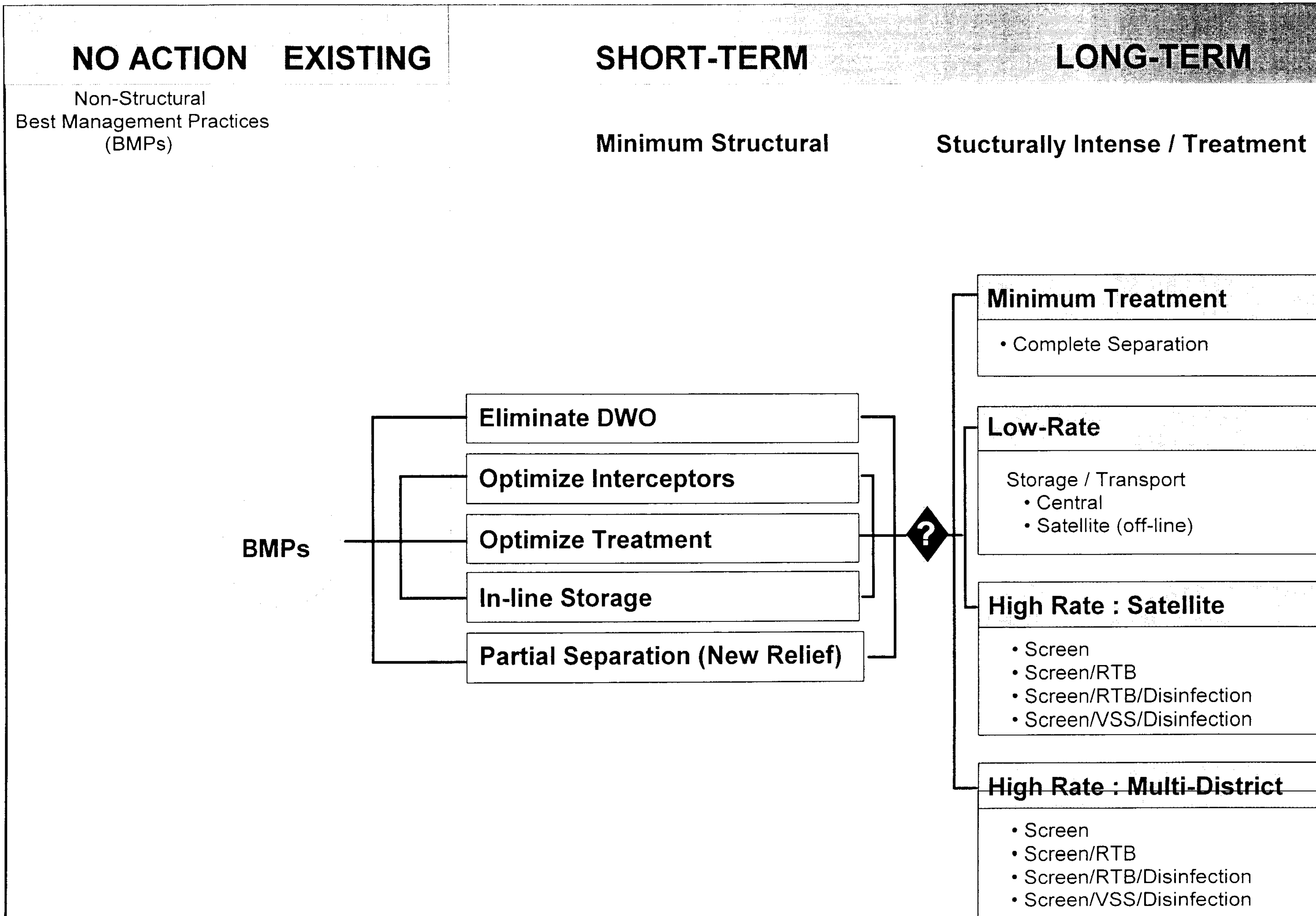
SCREENING OF POTENTIAL CSO CONTROL OPTIONS

RANGE OF CONTROLS	EPA 9 MINIMUM CONTROLS	POTENTIAL NEW CONTROLS	REMARKS
MINIMAL STRUCTURAL ALTERNATIVES			
Flow balancing between districts		√	Not likely to be effective
Overland flow slippage		√	Could be effective as part of relief program
Increase pervious area			Little potential
Elimination of dry-weather overflows	5	√	Essential - more effort needed
Hydraulic control devices	4	√	Devices will need upgrading (see TM #2), part of interceptor optimization
Interception optimization	4	√	Good potential (see TM #2)
Maximize WWF treatment at WPCCs	4	√	Good potential (see TM #2)
In-line storage	2	√	Good potential
STRUCTURALLY INTENSIVE ALTERNATIVES			
Elimination of dry-weather overflows	5	√	Will likely involve station upgrades
Surface storage			Not practical, environmental issues
Additional conveyance and storage		√	Modest potential, will be reviewed
Tunnel storage		√	Costly, has been used elsewhere (e.g., Chicago)
Off-line storage (near surface)		√	Good potential costly but effective, e.g., Hamilton
Sub-district separation		√	City has done selective separation as part of relief program
Partial sewer separation			Costly, takes street runoff only
Full sewer separation		√	Very costly, will be reviewed
TREATMENT ALTERNATIVES			
Central treatment (primary)	1	√	Not practical, will be reviewed conceptually
Retention treatment basins		√	Costly but effective, proven technology
Vortex solid separators		√	Costly but effective, evolving technology
Screening devices	6	√	Can assist in pretreatment for floatables/disinfection evolving technology
Disinfection		√	Chlorination proven, U-V technology evolving
Trash netting	6	√	Floatables capture, evolving technology

TABLE 2-3

CANDIDATE CSO CONTROL OPTIONS

CANDIDATE CONTROLS
MINIMAL STRUCTURAL ALTERNATIVES
Elimination of dry-weather overflows
Interception optimization
Maximize WWF treatment at WPCCs
In-line storage
STRUCTURALLY INTENSIVE ALTERNATIVES
Additional conveyance and storage
Tunnel storage
Off-line storage (near surface)
Sewer separation
TREATMENT ALTERNATIVES
Central treatment (primary)
Retention treatment basins
Vortex solid separators
Disinfection
Screening (including trash netting)



Control Options Available
Figure 2-1

bmpplan
cso1

of CSOs and improvements in water quality. The following sections review the control options identified through the screening process, with respect to their general application to the Winnipeg circumstances, the experience elsewhere and the estimated conceptual costs for application to Winnipeg. The technologies are discussed in the following general categories:

- Optimizing Infrastructure for WWF (Section 3); and
- Structural Intensive CSO Controls (Section 4).

The following discussions build on the Phase 1 Technical Memorandum No. 5 (TM #5) which discusses the complete range of minimum structural controls and structurally intensive controls. This TM provided descriptions of the available technologies and was supplemented, in depth, by TM #6, "Experience Elsewhere". The latter gave extensive details to technologies, preliminary costs, and experience with the technologies in North America and elsewhere. It is assumed that the reader has these reports available for reference and accordingly, descriptions of options given in Sections 3 and 4 are not elaborate.

Since Phase 1, three supplemental investigations on control technologies have been carried out, which are referred to in the following sections. The first of these documents is the Gore & Storrie report entitled, "Project Unit Costs Development", and dated May 1995. This document is included in Appendix A to this Technical Memorandum. The second document, prepared by W_2O, is entitled, "Review of Technologies for Removal of Disposables from Combined Sewer Overflows", and dated May 16, 1995. This document is included in Appendix B. The final document was prepared by EMA and is entitled, "Real Time Control". This document is dated July 1995 and is included in Appendix C.

The candidate options identified in this manner were studied further as described in Sections 3 and 4 and the impacts on water quality are discussed in TM #4 - Receiving Stream.

3.0 OPTIMIZING EXISTING INFRASTRUCTURE FOR WWF (MINIMUM STRUCTURAL CONTROLS)

A logical first step to achieving improved control of CSO is to optimize the use of existing facilities. This is also consistent with the EPA CSO control policy. Other more intensive options follow these upgrades, if and as required. Such optimization usually involves the maximization of conveying capacity of the interceptor sewer system and the associated treatment plants, followed by the efficient use of existing inline storage. Both of these measures are proven technology and were considered for their application to the Winnipeg situation, as discussed below.

3.1 INTERCEPTOR CAPACITY (5 x DWF)

As discussed in some length in Phase 2 TM #2, the Main Interceptor was designed to convey 2.75 times DWF from the combined sewer districts which it serves. This was based on full development of those combined sewer districts and was based on gravity, i.e. unsurcharged flow in the Interceptor. As a result of the model of the hydraulics of the NEWPCC/Interceptor/pumping system, it was determined that the Main Interceptor could convey about 5 times current DWF in a surcharged condition, but without overflow to the Red River through the designated overflow at St. John's combined sewer outfall.

Increasing the interception rate would result in a twofold benefit. Firstly, it would result in a modest reduction in volume of CSO to the rivers, as well as the number of CSO discharges into the rivers, i.e., a number of small storms would be intercepted. Secondly, an increase in the conveyance capacity of the interception system would permit the dewatering of inline or offline storage in a shorter period of time than would be the case for the lower diversion rate.

The interception rates in the existing districts vary widely. To regulate these rates to about 5 x DWF would require extensive upgrades of regulators, pumping station capacities, instrumentation, etc. A preliminary assessment was made of the costs of modifying the diversion systems to intercept 5 x DWF. Of the 42 combined sewer districts, about 31 are pumped. A review of the current diversion pumping capacities indicates that about 1/3 currently have sufficient capacity to pump 5 x DWF from their district. For estimating purposes, it is assumed that all pumping stations with insufficient existing capacity would be replaced, although in many cases, 5 x DWF could be achieved through upgrading of pumps. This assumption is conservative and is considered acceptable for first level screening purposes.

A review of pumping characteristics needed to divert 5 x DWF at a typical station indicate a flow of about 0.5 cms at 5 m of lift, or 40 kW/h of power consumption. Based on the Gore and Storrie Report (Appendix A), Table 2.5 shows that, for smaller stations, the cost is in the order of $10,000/kW or $400,000 per station. Allowing for a 20% estimating allowance and 20% for engineering, finance and administration, results in an estimated base cost of $600,000 per station. Accordingly, the 20 stations replaced could cost in the order of $12 million. This is considered to be a conservative estimate; however, since this allowance would also cover the costs of any changes made to the existing interception system (including vortex regulators on the gravity connections to the interceptor), the cost allowance is about $15 million for these system upgrades.

As discussed in the Phase 2 TM #2, increasing the main interceptor capacity to 5 x DWF might necessitate significant capital expenditures for expansions to the NEWPCC primary plant. A preliminary estimate of the cost to expand the plant to treat the increased WWF is $25 million. This estimated cost includes a raw sewage pumping, screening and grit removal, 3 primary clarifiers and a new outfall but excludes additional sludge treatment, if needed. Accordingly, the total cost for increasing the interception rate to 5 x DWF could be about $40 million. This information will be developed further during the Phase 3 studies.

Implementation of all of the modifications involved in this option is practicable. There would be no need for additional land acquisitions (since the pumping stations and plant site already exist) and the environmental impacts would be limited to those caused by construction activities. Additional operating effort will be moderate since, at worst, the option represents an expansion of existing facilities.

3.2 INLINE STORAGE

In combined sewer districts, the major trunk sewers have a large capacity and run full during severe storms. Often, however, they are only partly full during less intense rainfall events. These trunks will generally convey flows between 50 and 100 times DWF and, therefore, during most storms, considerable unused capacity exists in these conduits. In-system or inline storage takes advantage of this unused, existing storage capacity by restricting flows at the overflow point, causing wastewater to backup in upstream lines. Winnipeg's generally flat terrain has dictated that combined sewer trunks are laid on as flat a grade as practicable and are generally large in diameter. Accordingly, the existing trunk and relief sewers (provided to limit basement flooding) lend themselves to adaptation for inline storage. Such storage could be effected by weirs, level control gates or by inflatable dams. It is fundamentally important that the use of inline storage must not compromise basement flood protection. Accordingly, any control device must be fail-safe, i.e., in case of system failure, the gates must open or the dams collapse. Phase 1, TM #6 notes that "in-pipe storage is the most economical storage option and should be considered first." This is consistent with EPA, CSO control policy.

A review was made of available inline storage in the Winnipeg districts, based on the results of analyses undertaken for basement flooding reports. The results of this review are provided on Table 4-1. As indicated, the average equivalent storage (without the Armstrong-Newton district), is about 1.2 mm. The Armstrong-Newton district was eliminated, firstly, because it was exceptionally high and, secondly, because the relief option proposed involved separation of the entire area, thus eliminating this availability.

The Phase 1, TM #6 indicated that costs for in-system storage systems in the US have ranged from $0.1 to $1.2 per US gallon (1994 US dollars). This amounts to a range of from $40 to $450 Canadian per cubic metre of storage. The total hectarage of the CS districts in Winnipeg is 10,500 ha. Applying 1.2 mm to 10,500 ha and using this cost range results in an estimated cost of developing inline storage from $5 million to $55 million.

As a cross check, EMA provided current cost experience for a recently tendered inline control device. These costs were adjusted for the Clifton district characteristics and extrapolated to the Winnipeg-wide situation:

TABLE 4-1

IN-SYSTEM STORAGE (EXISTING STUDIES)

District Studied	Available System Storage (cu.m)	Area (ha)	Equivalent Storage (mm)
Clifton	6 000	494	1.2
St.Johns/Polson	2 200	616	0.4
Ash		735	
Munroe/Roland/Hart	13 000	840	1.5
Tylehurst	2 500	216	1.2
Mager	7 000	782	0.9
Selkirk	15 000	326	4.6
Linden	600	160	0.4
Hawthorne	1 000	260	0.4
Armstrong	9 000		
Baltimore	1 800	247	0.7

WITHOUT ARMSTRONG/NEWTON: AVERAGE EQUIVALENT STORAGE = 1.2 mm

EMA Option	Clifton District
• 2 - 42" x 60" (1050 x 1500) gates (with cylinders and controls)	• 2 - 1800 x 2400 gates (with cylinders and controls)
• Area = 3.2 m^3	• Area = 8.6 m^3
• US $125,000 CAN $180,000	• $500,000 (pro rata)
• Allowance for structures	$ 200,000
• Add 20% Estimating Contingency + 20% Engineering, Finance and Administration	$ 300,000
Approx. costs for Clifton facility	$1,000,000

This cost assumes that a flow control device must be supplied for the Clifton District. In fact, the sluice gates already exist and could be adapted for flow control at much less than this estimated cost. For current estimating purposes, we propose to use the above gross cost since the actual nature of the control device is not yet decided.

Applying the estimated cost for the Clifton District to the total Winnipeg hectarage of 10,500 ha results in an estimated total cost of a system-wide, inline storage system of 10,500/500 x $1 million = $20 million. This estimate falls in the middle of the above range. It is considered to be a conservative (i.e., high) estimate of the cost of implementing inline storage for the City of Winnipeg.

There are still a number of combined sewer districts which require relief in order to reduce the frequency of basement flooding. An economical means of increasing inline storage in such systems would be to install oversized pipes for some or all of the downstream trunk relief system. The economics of this possibility will be investigated in Phase 3 on a preliminary basis, but should be a part of any study of basement flooding relief prior to any further construction of these relief facilities.

Since this option comprises, for the most part, an upgrade and automation of existing facilities, implementation will be practicable with virtually no land use implications, and environmental issues will likely be restricted to those associated with construction. Because of the additional labour associated with monitoring the control system, operating effort will increase but will still be low to moderate.

3.3 BASIC REAL TIME CONTROL (RTC)

The EMA memorandum on real time control (Appendix C) discusses inline storage as the first element of an area-wide RTC system. It references inline storage, combined with the use of available conveyance capacity of the existing collection system (i.e., the Main Interceptor) and the available treatment works capacity (i.e., the NEWPCC), as one of the more readily implemented and cost-effective approaches to achieving an immediate reduction in CSO volumes, The effective operation of these inline storage facilities require the first stage of real time control systems. The memorandum refers to this system as basic real time control.

The basic RTC system, as described in the memorandum, is included below (see also attached Figure 3-1, from Appendix C):

> "The first step is to install a basic RTC system to observe the system operation, collect data, and provide limited operator directed control. The system could act as a pilot system to train operators and gain knowledge of the CSO system operation under various conditions. A few control sites would provide initial experience with CSO abatement system operation.
>
> The RTC system would include limited monitoring and control capability as follows:
>
> 1. Sensors to measure rain, level, and CSO occurrence;
>
> 2. Control devices such as sluice gates or inflatable dams to restrict discharges and cause in-system storage;
>
> 3. Control devices such as mechanical regulators or lift stations to regulate the flow of stored wastewater back into the interceptor;

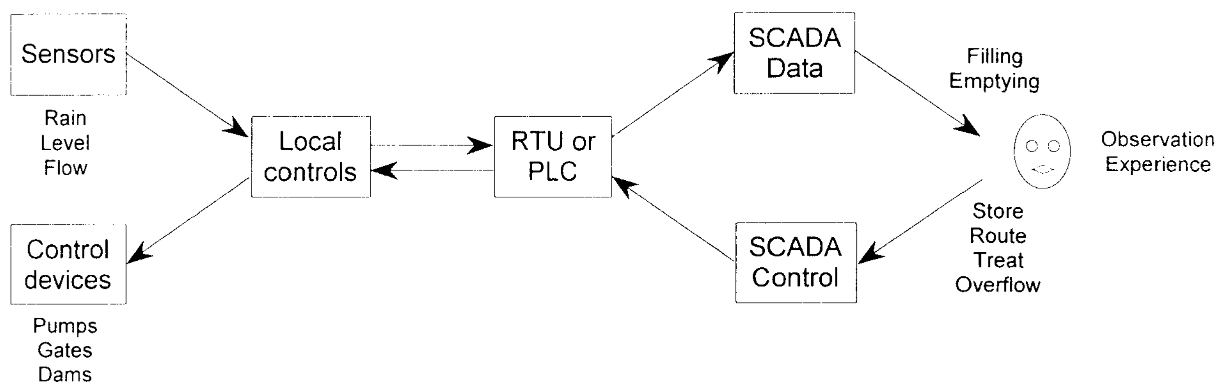


Figure 3-1 Basic Real Time Control (from Appendix C)

4. Local control systems to operate control devices in a fail-safe manner;

5. Remote terminal units (RTUs), SCADA computer(s) and communications systems to acquire and display operating information from sensors and control devices and to permit control commands to be sent from a central location.

As more facilities are added, a more proactive control approach may be needed."

The more proactive or area-wide control approach referred to in the above excerpt is discussed under the heading of "Real Time Control," in Appendix C. Such a system is much more complex and is intended to optimize use of system storage to reflect real distribution of rainfall.

With regard to inline storage, and recognizing that development of such available storage will likely be the most cost-effective means of reducing both volumes and frequency of CSOs, the EMA report goes on to recommend pilot testing such a facility in the near future. They note that early installation of the sensors and control devices that will be required to implement inline storage may be desirable. This installation will allow the City to acquire data for use in the planning studies; will allow them to learn more about their system operation during wet weather events; and will permit them to gain experience with sensor and control device operation and maintenance. More discussion of this pilot demonstration project follows later in this technical memorandum.

The costs for the provision of inline storage include the sensors needed to operate the control devices. They do not include the costs of a comprehensive central SCADA system or area-wide RTC. The latter will require more detailed investigation and possibly associated with the pilot test program.

4.0 STRUCTURALLY INTENSE CSO CONTROLS

The structurally intensive CSO controls options comprise the construction of substantial new facilities such as central treatment, storage tanks or sewers, "end-of-pipe" treatment or sewer separation. These are typically considered if optimization of the existing infrastructure is deemed to provide inadequate water quality protection.

In assessing the cost of all the "end-of-pipe" options, except inline storage, it has been assumed that the treatment or storage device would be located at the end of the basic combined sewer trunk or the relief outlet, but not both. The validity of this assumption will be assessed in Phase 3.

None of the costs given below include the cost of land or operation. Those aspects will be included in the Phase 3 analysis.

4.1 CENTRAL TREATMENT

As discussed in Phase 2, TM #2, it is possible to convey in the order of 5 x DWF to the treatment plants. Ninety percent of the flows from combined sewer districts are tributary to the North End plant. The Main Interceptor has the capacity to convey these flows to the NEWPCC. The plant may or may not be able to accommodate 5 x DWF over extended time periods. In the event that such periods do not permit treatment of all of the excess WWF flows delivered at this rate, then the NEWPCC primary plant capacity would have to be increased. As noted in Section 3.2, a preliminary estimate of the cost of such expansion is about $25 million. Further investigation will take place in Phase 3.

Intercepting the full CSO (upwards of 100 x DWF) and conveying these large peak flows to a central treatment facility is simply impractical. Other than expanding the existing facilities associated with conveying and treating flows at the NEWPCC (compatible with the maximum delivery capacity of the Main Interceptor), the provision of central treatment is not considered to be an economically viable option and will not be investigated further in this study.

4.2 STORAGE

As noted in Section 3, inline storage is the most economical means of reducing volumes of CSOs discharged to rivers and the frequency of such discharges. There are, however, more structurally intensive means of providing storage and thereby enhancing the reduction in volume and frequency of overflows to the river. The use of area-wide or distributed storage is proven technology for CSO control. The WEF Manual of Practice FD-17, (WEF 1989) notes that "storage should be considered in planning of the treatment and control system because it allows for maximum use of existing dry weather treatment facilities and results in the lowest cost system."

These options can comprise either deep underground tunnels or near-surface tanks, as discussed below.

4.2.1 Tunnel

Relatively deep underground tunnels can be used for storage or conveyance to the central treatment facility. These could be area-wide tunnel storage works or localized tunnels, where near-surface space is unavailable at or near the end of the combined sewer trunk. In-flow to the tunnels is usually effected by deep shafts (vortex chambers), and out-flow to the treatment facilities is by pumping.

A number of municipalities in North America have implemented CSO plans involving regional tunnel storage. Drawing from Phase 1 TM #6, Chicago has the most elaborate facility (TARP). Milwaukee has also virtually completed an extensive deep tunnel storage system, as has San Francisco. Oregon and Providence, Rhode Island have extensive plans for tunnel storage. A recent study completed in the City of Toronto has recommended extensive storage tunnels in conjunction with some tank storage. Ottawa also proposes major tunnel storage.

The results of the regional model analyses have indicated that up to 1,000,000 m^3 of storage could be required to store 100% of the runoff from the highest rainfall. An alternative approach to calculating the volume of capture in the regional tunnel is developed below

(Section 4.2.2) and requires 300,000 m^3 of storage. This would reduce, but not eliminate, the number of overflows and would likely approximate the EPA presumptive limit of 4 overflows/year.

Preliminary costs were developed for the tunnel storage option. Some 36 km of tunnel were considered potentially viable and would allow direct interception of most of the flows from the CS districts. The tunnels would be located on both sides of the Red from the Assiniboine River to the NEWPCC and on both sides of the Assiniboine from Sturgeon Creek to the Red River. Tunnel costs were developed using tunnel unit rates from Figure 3.2 of the Gore and Storrie Cost Memorandum (Appendix A), attached. These costs are (for 36 km):

Diameter (m)	Volume (m^3)	Costs ($M)
3.25	300,000	400
6	1,000,000	650

These costs were based on the Gore-Storrie cost estimating curve. For the sizes selected, the costs are $2.10 and $1.90/mm diameter/metre of tunnel (respectively), plus the 20% estimating contingency, plus 20% for engineering finance and administration and, finally, plus a 10% design contingency, allowing for drop connections and a major pumping station.

There are advantages and disadvantages to the major regional tunnel approach:

- Given good soil conditions, the tunnelling operation can be reasonable trouble-free. It is considered practicable to install a 6 m diameter tunnel in "normal" ground conditions in Winnipeg. However, it would be virtually impossible to avoid bad tunnelling conditions over some part of the routes in Winnipeg soils. This would increase costs and maybe practicability.

- A regional tunnel approach to CSO control is a major commitment, i.e., it must be all in place to be fully effective. The downstream end (i.e., location of pumps to the treatment plant(s)) would have to be installed initially to get any progressive benefit. This would

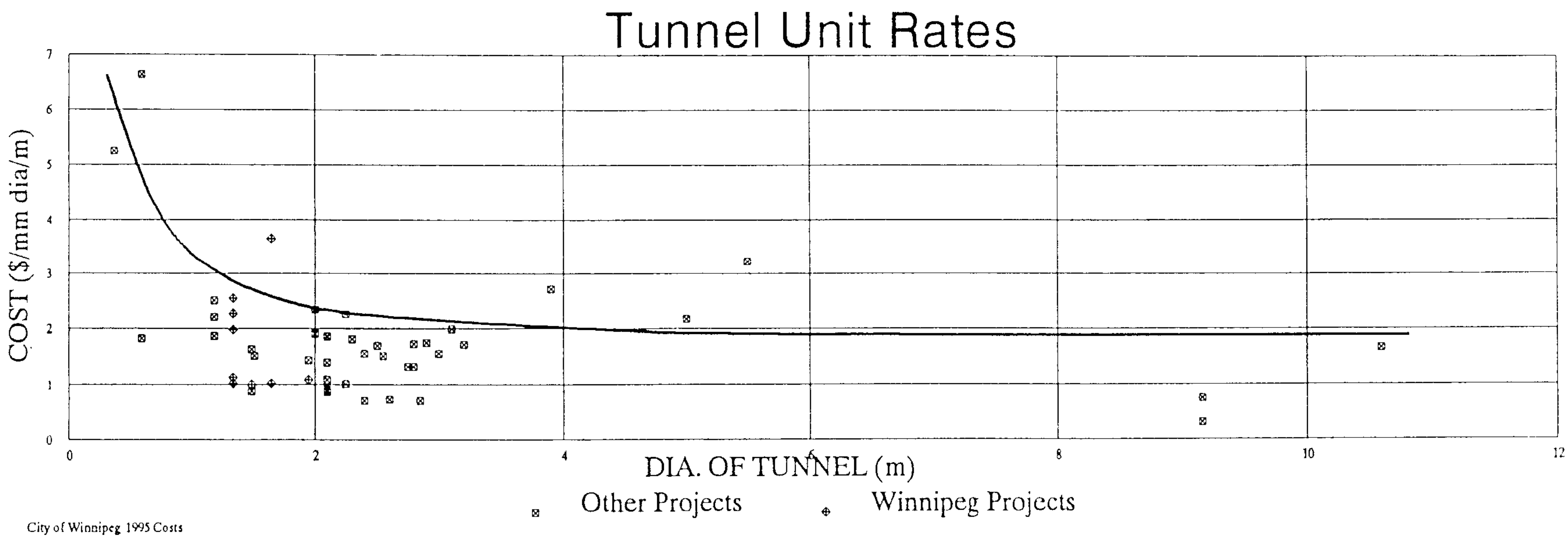


FIGURE 3.2

(from Appendix A)

mean that it would not be possible to be selective in the order of districts served (eg., it would not be possible to relieve districts upstream of the Forks first).

- Finding a continuous route for such a large tunnel (particularly through the downtown business section) could prove difficult.

- Underground utilities could interfere with tunnel routes.

- Shaft locations would have to be selected carefully, and constructed so as to limit disruption (this could present problems in built-up areas).

- The tunnel option has the advantage that it can be located in road allowances, i.e., it does not need large tracts of open space to implement.

- The system will be relatively easy to operate; the main activity is pumping. Cleaning should be addressed mainly in design.

It may be economically or practicably desirable to join up a number of selected, adjacent combined sewer districts via a single tunnel storage option. Possibilities such as this will be reviewed in the Phase 3 study.

4.2.2 Offline Storage

An alternative to multi-district or area-wide tunnel storage is offline, near-surface storage tanks. These tanks will likely be located near the outlet of the combined sewer. There are currently about 30 such installations in North America, with the greatest number being located in Michigan. The cities of Hamilton and Toronto have several offline storage tanks in operation.

This system of storage also has advantages and disadvantages:

- Offline storage tanks (distributed storage) have the advantage that they can be installed in districts on a prioritized basis (i.e., to obtain benefits downstream early in the program).

- Unlike the tunnel option, offline storage does require fairly large blocks of open space to install. The main land use issue will be to find large enough blocks of land near the CS trunk.

- Offline storage can generate odours, but these can be controlled with proper ventilation and odour control procedures.

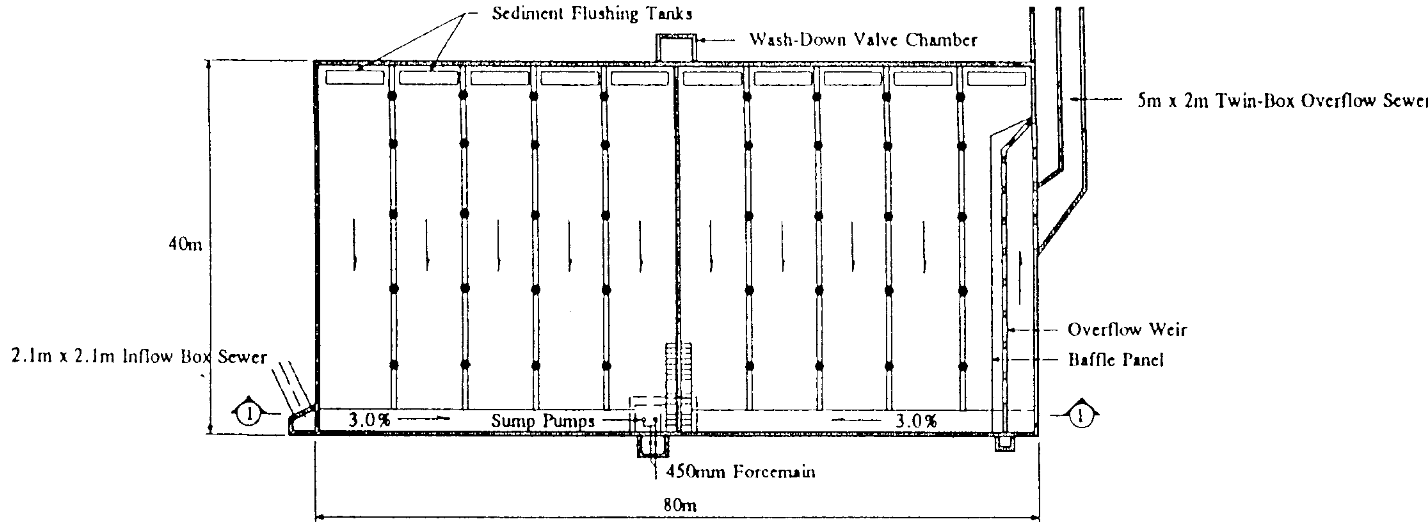
- Construction will be straight forward but disruptive.

- Most of the facilities will be located in residential areas and will be subject to the NIMBY syndrome.

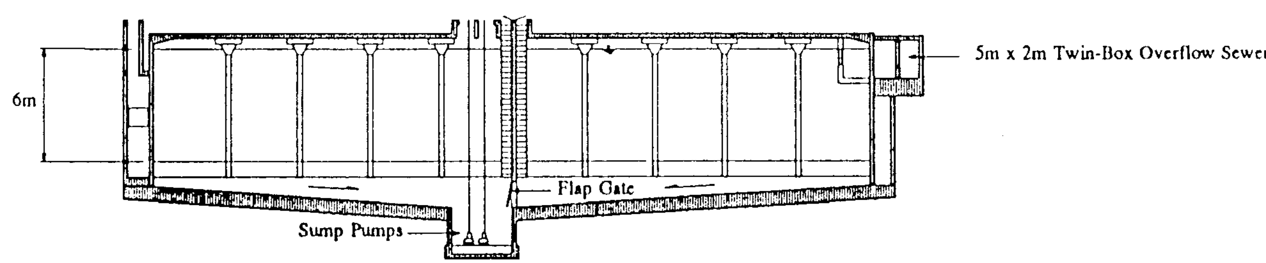
- The system will be relatively easy to operate. As with tunnels, the main operations will be pumping and cleaning. Each facility will require routine (likely weekly) inspections.

As discussed in Section 3, of Phase 2 TM #2, the costs of offline downstream storage could be significantly reduced through the use of the existing installed pumping capacity at the flood pumping stations. Flows to be stored would be raised by the flood pumps; would discharge into storage and, on completion of the storm and drainage of inline storage, would discharge, likely by gravity, into the adjacent interceptor. The pumping of the flows to be stored would effect substantial savings in costs as compared to deep excavation needed to allow discharge to the tanks by gravity.

During the study team's site visits to the storage facilities in the Toronto Beaches area (in-tank inspections) and Hamilton (office briefing and site inspections) in June 1995, the practicability of "end-of-pipe" tank storage of combined sewage was verified. In both cases, the facilities were very well adapted to their surroundings. They were successful in being virtually undetectable by passers-by and care had been taken to avoid odours (through ventilation and



Floor Plan



Section 1

FIGURE 3: WATERFRONT PARK CSO TANK; FLOOR PLAN AND SECTION

(from Paper entitled "Hamilton CSO Detection Tanks - Design Considerations"

TABLE 4-4
STORAGE TANK COSTS

Size m^3	**Cost/m^3 \$1,000**	**Cost \$million**
1,000	1.9	1.9
2,000	1.7	3.4
3,000	1.3	4
4,000	1.05	4.2
5,000	0.85	4.3
6,000	0.8	4.8
10,000	0.65	6.5
15,000	0.55	8.2
20,000	0.5	10
30,000	0.45	13.5

- Costs - G&S Curves:
  +20% estimating contingency
  +20% engineering, administration and finance
  +10% design contingencies
- **Storage** - costed on basis of volume alone (i.e., 30,000 m^3 would cost $13.5 M)

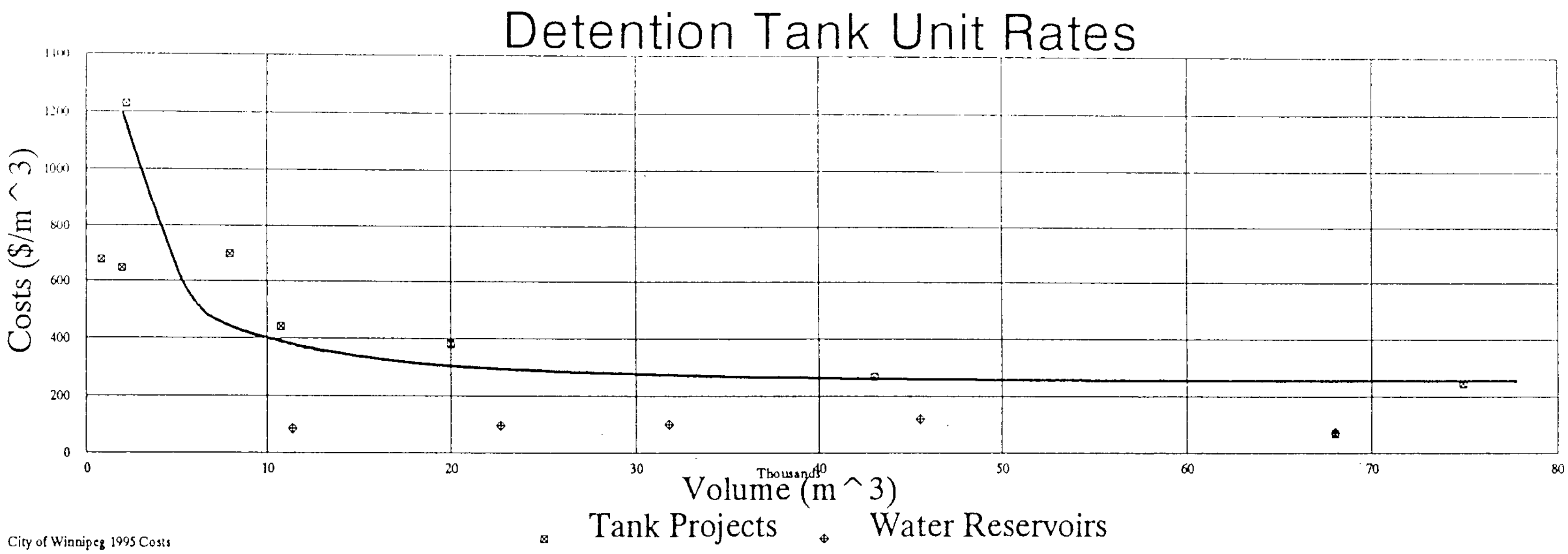


FIGURE 3.1

(Source: Appendix A)

filter systems) in the surrounding park-like sites. In the Hamilton case, the tanks were sized to hold the design event flows and ranged in size from 2,000 m^3 to 68,000 m^3. The attached Figure 3 is taken from a paper entitled "Hamilton CSO Detection Tanks. Design Considerations", and shows the layout of their "Waterfront Park CSO Detection Tank" of 20,000 m^3 capacity. In the Toronto Beaches facilities (2,000 to 8,000 m^3), allowance was made for flow through the tank to the effluent pipe in the event of the tank being full.

The Toronto Beaches and Hamilton facilities did not include upstream screens for floatables removal. In both cases, the tanks were reported to be successful in containing floatables without the screens. The Beaches facilities were designed with both pressure type flushing devices and gravity type (i.e., tilting bucket) gravity flush. The latter was very successful and generally less complicated than the pressure flush devices.

Storage for the Phase 2 analysis was sized on the basis of a hypothetical storm. The maximum treatment rate was derived for each district using the regional model for 1992. The results of this derivation are provided in Table 4-2. The flow rates shown are net of inline storage for each district. The storms were assumed to have a triangular shape with a one hour rise to peak and a two hour fall to zero (Figure 4-1).

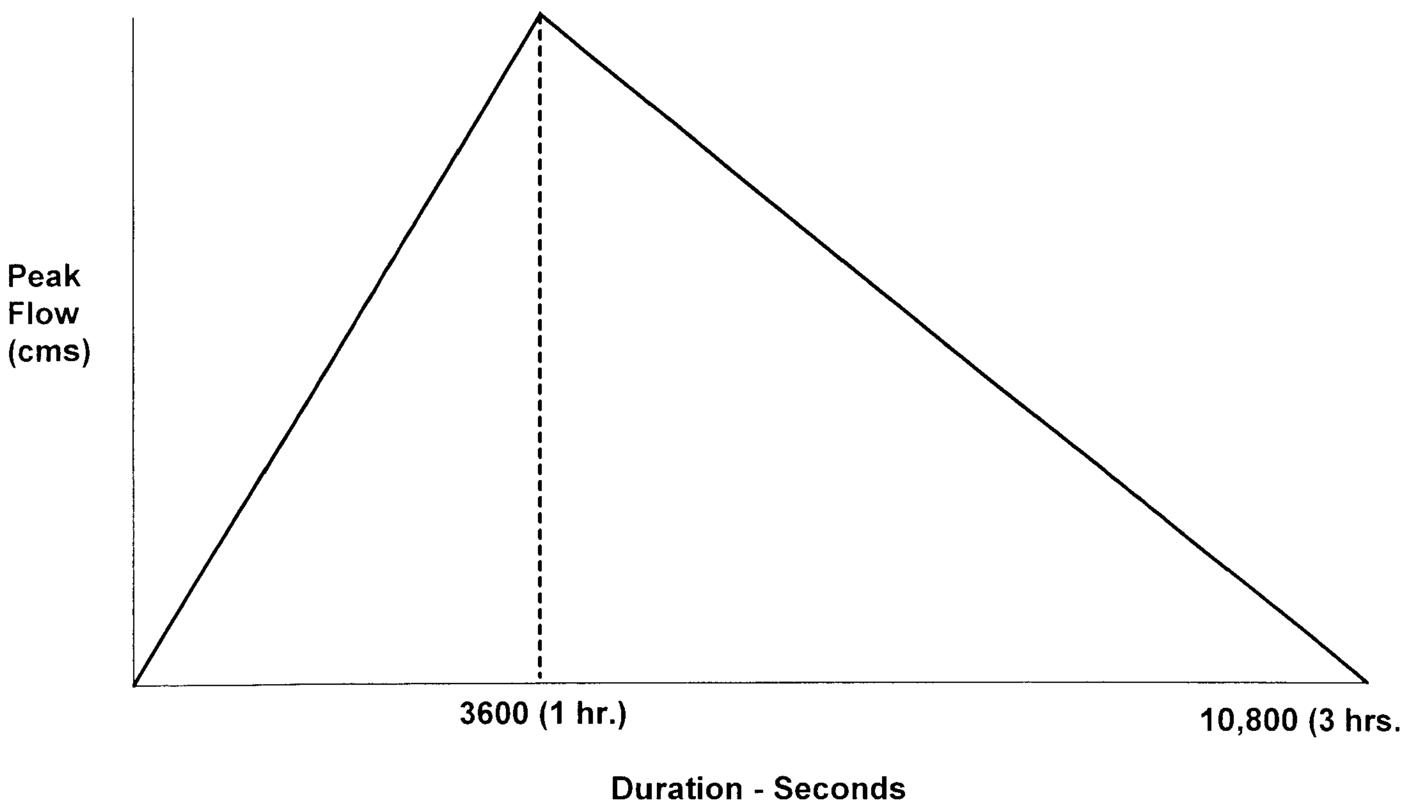
The results of this conversion of peak flow to volume are shown on Table 4-3. The total calculated volume of storage is 300,000 m^3. We believe that this approximate volume of storage would be sufficient to meet the requirements of the EPA "Presumptive Approach", i.e., four overflows or 85% capture of run-off. This has not been modelled in Phase 2. Such modelling will be done on a district-by-district basis early in Phase 3.

Costs were developed for storage tanks on a volume of storage basis. These are summarized on Table 4-4. The costs were derived from Figure 3.1 of the Gore and Storrie Cost Memorandum (Appendix A), attached to which were applied the allowances shown, that is, 20% estimating contingency, 20% engineering administration and finance allowance and 10% for design contingencies. Based on the Toronto Beaches and Hamilton experiences, no allowance is made for pre-screening the combined sewage before discharge to the tanks. Flows in excess of storage would be diverted to the rivers without disinfection.

TABLE 4-2

MAXIMUM CS PEAK FLOW RATES BY CSO DISTRICT - 1992

District	cu.m/hr	cms	District	cu.m/hr	cms
Alexander	3,500	1.0	Jefferson West	None	
Armstrong	2,600	0.7	Jessie	10,500	2.9
Ash	20,000	5.6	LeVerendrye	1,600	0.4
Assiniboine	7,500	2.1	Linden	3,400	1.0
Assiniboine Park	None		Mager Dr.	2,500	0.7
Aubrey	4,800	1.3	Marion	12,700	3.5
Baltimore	4,500	1.2	Metcalfe	1,500	0.4
Bannatyne	2,700	0.8	Mission	2,400	0.7
Boyle	2,000	0.6	Moorgate	1,500	0.4
Calrossie	800	0.2	Munroe	8,700	2.4
Clifton	9,000	2.5	Newton	1,800	0.5
Cockburn	3,000	0.8	Polson	9,600	2.7
Colony	5,200	1.4	River	2,800	0.8
Cornish	1,000	0.3	Riverbend	6,000	1.7
Despins	3,100	0.8	Roland	10,000	2.8
Doncaster	550	0.2	Selkirk	None	
Douglas Park	900	0.2	St. John's	15,000	4.2
Dumoulin	1,000	0.5	Strathmillan	500	0.1
Ferry Road	5,200	1.4	Syndicate	2,600	0.7
Hart	5,300	1.5	Tuxedo	1,600	0.4
Hawthorne	7,000	1.9	Tylehurst	8,200	2.3
Jefferson East	8,200	2.3	Woodhaven	1,200	0.3



eg. **Clifton District**
Peak Flow (Table 4-4) = 2.5 cms
Volume = ½ x (2.5 x 10,800) = 13,500 m³

Conversion of Peak District
Combined Sewer Flow to Volume
Figure 4-1

flow-vol
01_files

TABLE 4-3

STORAGE VOLUMES BY CSO DISTRICT - 1992

District	Volume (1000 m^3)	District	Volume (1000 m^3)
Alexander	5	Jefferson West	
Armstrong	4	Jessie	16
Ash	30	LeVerendrye	2
Assiniboine	11	Linden	5
		Mager Dr.	4
Aubrey	7	Marion	19
Baltimore	6	Metcalfe	2
Bannatyne	4	Mission	4
Boyle	3	Moorgate	2
Calrossie	1	Munroe	13
Clifton	13	Newton	3
Cockburn	4	Polson	15
Colony	8	River	4
Cornish	2	Riverbend	9
Despins	4	Roland	15
Doncaster	1	Selkirk	
Douglas Park	1	St. John's	23
Dumoulin	3	Strathmillan	1
Ferry Road	8	Syndicate	4
Hart	8	Tuxedo	2
Hawthorne	10	Tylehurst	12
Jefferson East	12	Woodhaven	2

Total Storage = 300,000 m^3

As noted on the table, costs for storage were based strictly on volume. The above estimate results in a capture of approximately 300,000 m^3 (i.e., a storage volume requirement of 300,000 m^3). The estimated cost of region-wide, offline storage for about 85% capture of CSO is $210 million.

The projected volume for total capture, i.e., the deep tunnel option, was 1,000,000 m^3 at a capital cost of $650 million. In Phase 3, the impact on river quality of a number of capture scenarios will be evaluated using the Regional model.

4.3 HIGH RATE TREATMENT

All of the above options are based on the conveyance of the stored combined sewage to NEWPCC for treatment. The high-rate treatment alternatives, on the other hand, include for some form of treatment in situ and then discharging directly to the rivers. They also include storage of some portion of the combined sewage for subsequent treatment at the NEWPCC. Whether or not conveyance of the stored portion of the flows would necessitate expansion of the NEWPCC will be the subject of later investigations (Phase 3). An allowance has been made, for such expansion, in the costs for increasing CS interception capacity to 5 x DWF to facilitate the dewatering of the storage components after the storm.

It has been established that there are two major concerns with regard to the impacts on river quality of combined sewer overflow discharges to rivers, namely, fecal coliform concentrations and aesthetics. The objective of the high-rate treatment options is to remove solids to the point where disinfection is practicable, either chlorination/ dechlorination or UV. The costs of chlorination/de-chlorination have been included in the following estimates as a surrogate for disinfection. The two types of high-rate treatment considered involve Vortex Solids Separators (VSS) and Retention Treatment Basins (RTB).

4.3.1 Vortex Solid Separators (VSS)

Vortex technology was originally developed for grit-removal facilities and has been used in a number of wastewater treatment plants. It comprises a circular basin with the in-flow introduced tangentially, imparting a moderate centrifugal force on the solids. Experience elsewhere indicates that the device does enhance solids removal, as compared to normal sedimentation processes, at overflow rates higher than can be used in the latter.

VSS technology has been developed and installed in North America and Europe. These installations range in size from 40 L/sec to 4 m^3/sec. There are currently about 45 full size VSS units operating in North America, with 4 planned (P. Moffa *pers. comm.*; HIL Technology *pers. comm.*).

The sizing of VSS is dependent on the design overflow rate. The latter, in turn, is dependent on the desired results, i.e., if disinfection is the objective, the overflow rate must be sufficiently low as to remove solids to the point where disinfection is practicable. In order to arrive at a preliminary design basis for these devices, overflow rates for sedimentation technology were reviewed, along with experience elsewhere.

Phase 1, TM #6 indicates that overflow rates in conventional sedimentation basins in wastewater treatment plants are in the order of 2 to 3 metres per hour. The primary sedimentation basins at the SEWPCC are designed to operate at 3.5 m/hr during PWWF. Claims for the three VSS technologies (EPA swirl concentrator; Storm King, UK technology; and Fluidsep, German technology) are that the devices can be effective at peak design flows of anywhere from 17 to 150 m/hr. There is a large discrepancy between these extreme rates for sedimentation technology, i.e., 2 m/hr to 150 m/hr.

The study team visited the VSS pilot facility at Scarborough in June 1995. These tests were being run to test the performance of the VSS on solids removal and, more particularly, to determine its use as pretreatment for UV disinfection. The preliminary results of the pilot tests indicate that for their particular waste, and for UV technology disinfection, 10 m/hr is about as high an overflow rate which will permit effective disinfection. 5 m/hr results in much greater effectiveness of the UV disinfection.

In accordance with the foregoing, the current cost analysis was based on using an overflow rate of 10 m/hr., which might permit the use of UV disinfection. It may be that higher rates could be used for chlorination (say 20 to 40 m/hr.), but these would probably not be as high as the upper range quoted by suppliers.

In addition to the swirl basin, the VSS operation requires storage for the underflow. This underflow represents the concentrated solids collected through the settling process. It is returned to the combined sewer when the flows return to normal. The general allowance for this storage volume is 10% of the throughput flow.

The cost for different sizes of VSS options are provided in Table 4-5. In developing these costs, it was necessary to develop the conceptual unit operations involved in such a facility. Reference to literature and operating experience elsewhere implies that screens are generally used at the head end of a VSS facility, probably 20-25 mm openings such as those installed at the NEWPCC and the SEWPCC. The screen facilities would include channels, control gates, screens, screening conveyors and an allowance for ancillaries. Given that these facilities will, for the most part, be located in residential districts, the screening devices will have to be housed in closed structures. These factors were included in the estimated costs of the screening facilities. The cost of the basic VSS unit was derived from the Gore & Storrie report, Figure 3.6 (attached). In addition to the VSS proper, an allowance has been made for sludge storage. Because of the ambiguity with regard to costs for the various VSS units (as given in Appendix A), it may be that sludge storage has already been included in the costs. It may be, therefore, that the costing for the units has a redundant additional 15% contingency. For purposes of this screening stage of the study, this is not considered to be a major factor in the comparative evaluation. An estimating allowance of 20% and an allowance of 20% for engineering, administration and finance has been added to the costs.

The maximum rates for CSOs for the 1992 representative year were given earlier in Table 4-2. If the VSS facilities were able to treat, and therefore subsequently disinfect these flows, there would be no undisinfected overflow during such a representative year. The costs given on Table 4-5 were applied to the rates of flow given on Table 4-2 to arrive at an estimated total capital cost of the VSS installations for the CSO districts in the City of $440 million, excluding

TABLE 4-5

VORTEX SOLIDS SEPARATORS COSTS ($ MILLION)

	Size of VSS			
	1 m^3/s	2 m^3/s	4 m^3/s	6 m^3/s
Screen and Housing	$1.7 M	$2.0 M	$2.7 M	$4.4 M
Chlorination System	1.1	2.2	4.5	5.7
VSS (w/o Multiplier)	2.1	2.8	5.1	7.3
Sludge Storage	1.0	1.5	2.2	2.8
SUBTOTAL	5.9	8.5	14.5	20.2
+ 20% estimating contingency & 20% E,A & F*	2.6	3.7	6.4	8.9
TOTAL	$8.5 M	$12 M	$25 M	$29 M
VSS Configuration	3 @ 12 m	4 @ 15 m	6 @ 18 m	9 @ 18 m

Note: Size based on overflow rate of 10 m/hr (as opposed to original H.I.L. figure of 120 m/hr used in earlier analyses).

*Engineering, Administration and Finance

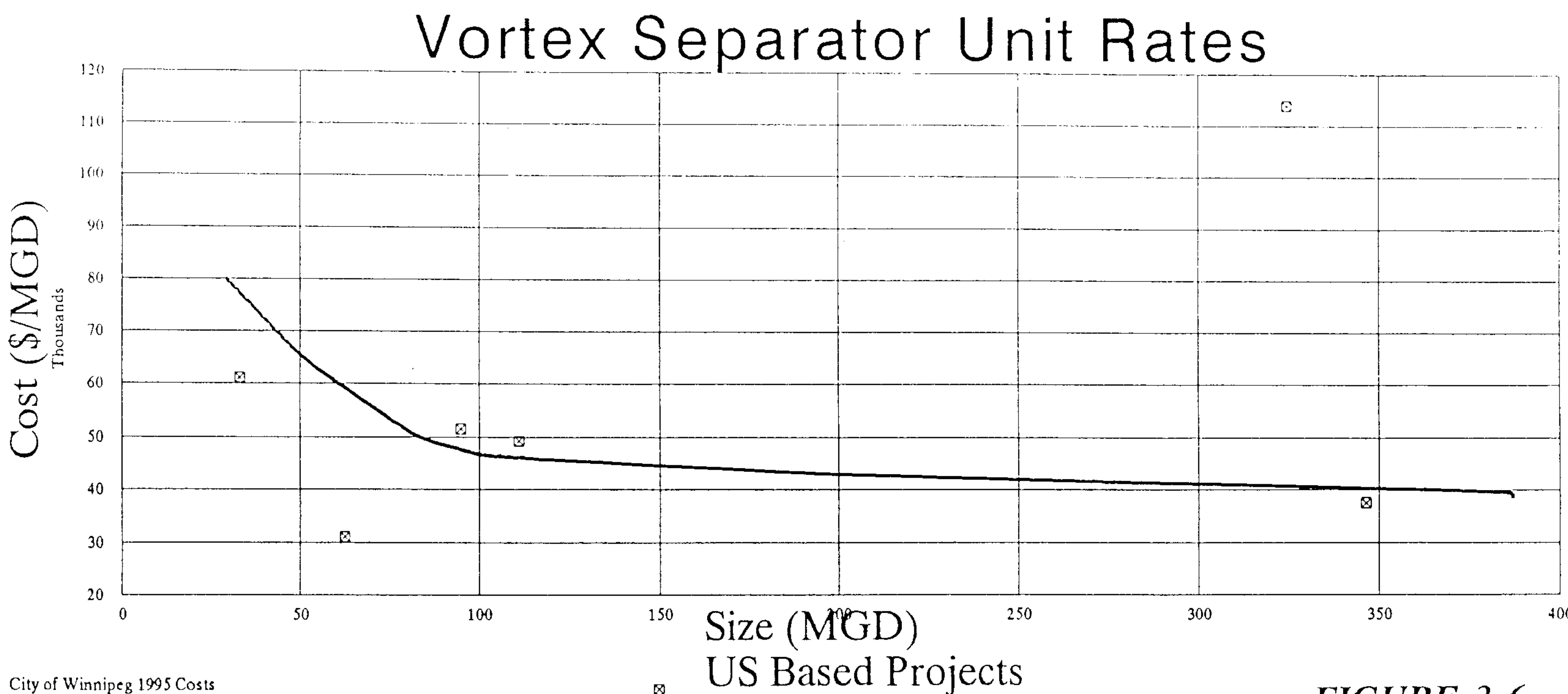


FIGURE 3.6

(Source: Appendix A)

land costs. Figure 4-2 is an example of the size and configuration of the VSS facilities designed on the above basis (for Clifton CS district).

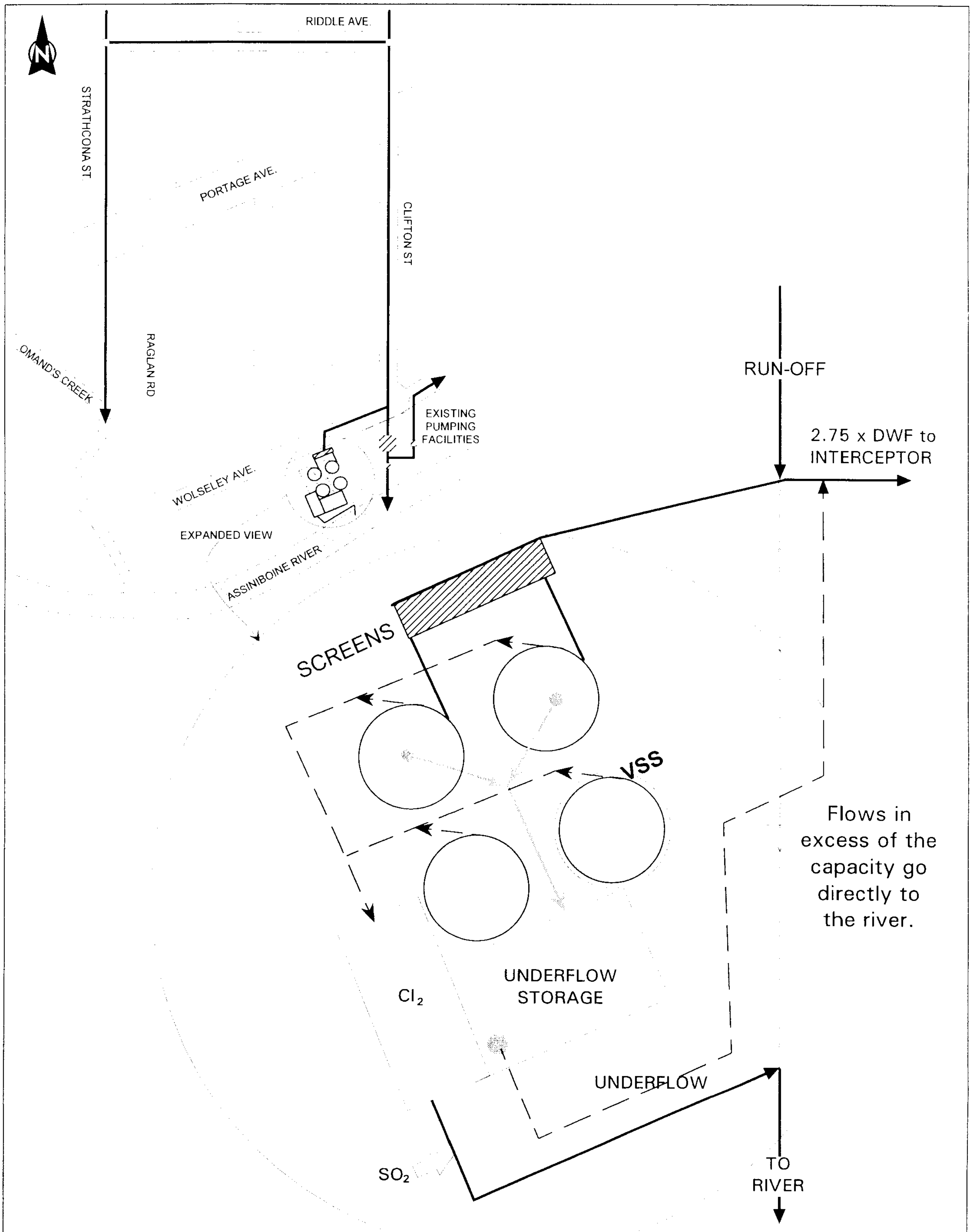
It should be noted that the design rate of 10 m/hr used in the above analyses compares to a rate of 120 m/hr, as provided by HIL in the River and Linden/Hawthorne studies (Wardrop/Tetr*ES* 1991 and Wardrop/Tetr*ES* 1994). These current estimated $440 million for City-wide VSS compares to the prior $300 million carried in the 1990 River study.

The VSS installations would have the same advantages and disadvantages as offline storage, that is, prioritized siting; requiring fairly large blocks of open space for siting; generating odours; construction will be disruptive; NIMBY. In addition, these facilities would include the operation of screening facilities (equipment maintenance and screenings handling and removal), plus the complexities of operating disinfection facilities, i.e., either transporting, storage and handling of hypochlorite and metabisulphite or operating and maintaining U-V disinfection facilities. Environmental issues will be medium to high.

4.3.2 Retention Treatment Basin (RTB)

Retention Treatment Basins (RTBs) are essentially storage tanks designed for a flow-through treatment performance, similar to high-rate primary clarifiers. Such use of a storage tank results in smaller tank units which have the geometry of clarifier basins. That is, for large volumes, the RTB would comprise a number of units (of specific size), whereas the storage would be sized to store the volume in one large tank.

The retention basin is a variation of the offline storage basin. During the initial stages of the storm, all flows are diverted to the RTB. Chlorine (which would be the usual disinfectant for this technology) is added during the filling period. When the basins are filled, the tanks overflow. The flow will continue to pass through the tanks, and continue to be chlorinated, until it reaches the design overflow rate for wet weather sedimentation. The flow discharging from the tank would be chlorinated. For purposes of the current analysis, the overflow rate was selected as being 4 m/hr.



Vortex Solids Separation (VSS)
(Illustrative Only)
Figure 4-2

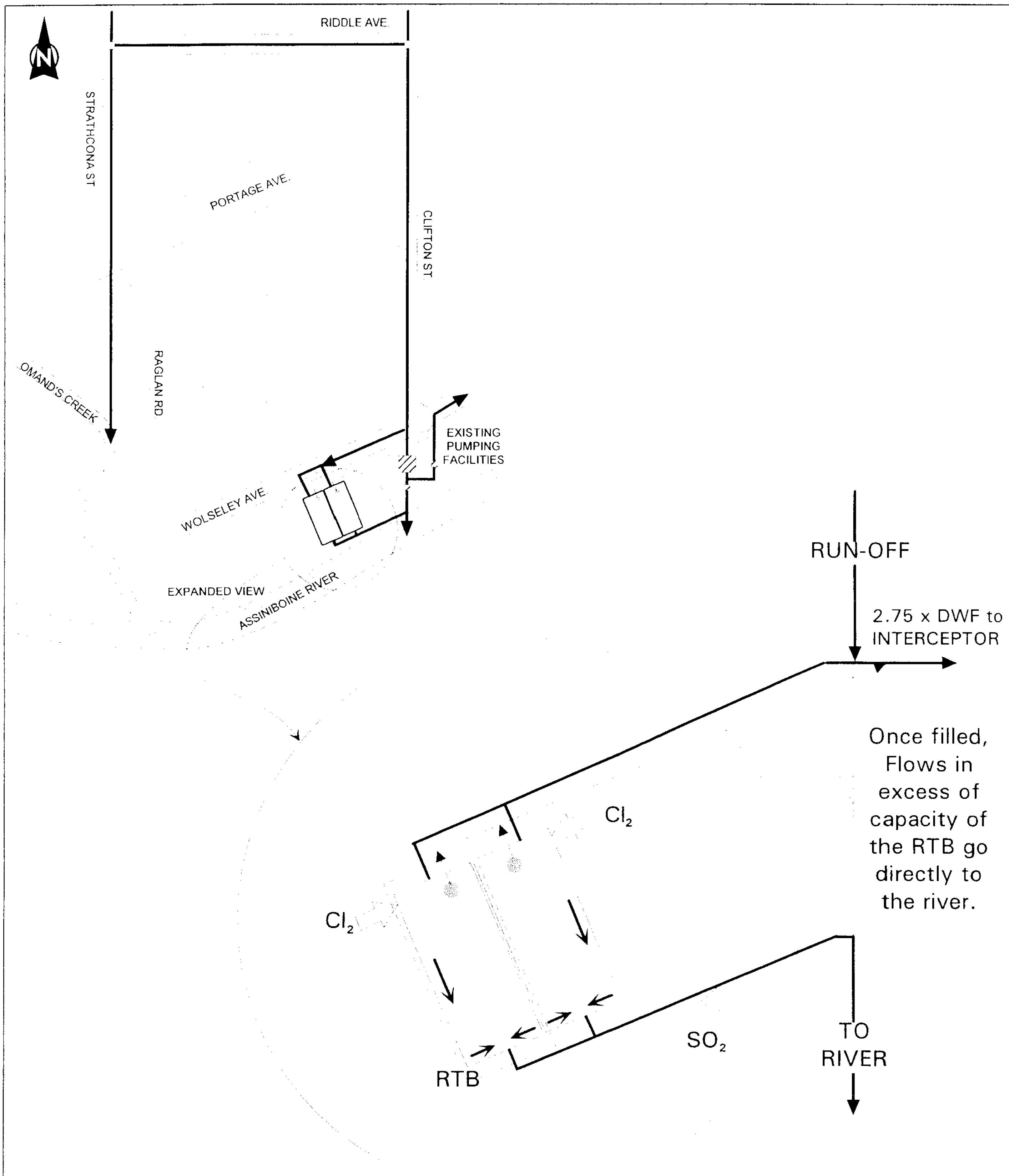
vss-rtb2
01_files

Because the RTB units are intended to act as primary clarifiers, the dimensions of the tanks selected were 50 m long x 20 m wide x 5 m deep - the approximate dimensions of the recently installed clarifiers at the SEWPCC. This represents a rational basis for sizing the units as clarifiers, as well as storage basins. Accordingly, each tank will store up to 5000 m^3. Once full, the tank could perform as a clarifier for an additional 1.1 m^3/sec (4 m^3/m^2/hr x 50 m x 20 m ÷ 3600 sec. = 1.1 m^3/sec). Given the operating basis, this would mean that the additional 1.1 m^3/sec would be disinfected and dechlorinated. Flows in excess of this rate would be diverted directly to the river as untreated CSO. On completion of the storm, the RTB contents would be emptied into the interceptor when capacity was available and would be conveyed to the WPCC. Based on the experience in the Toronto Beaches and Hamilton facilities, screening has not been included in the estimated costs of the RTB facilities, as was the case in the offline storage basins. The RTB configuration has been used in a number of installations in Michigan. Figure 4-3 shows an example of the application of this technology applied to the Clifton combined sewer district.

The effect of the RTB is to enhance the operation of simple offline storage. Flows up to the design flow-through rate all receive chlorination/dechlorination, whereas in the case of the offline storage facility of the same size, the only impact is to reduce the volume and frequency of CSOs. Both systems capture floatables, the storage facilities up to the capacity of the tank and the RTB up to the capacity of the tank plus the flow-through volumes.

The advantages and disadvantages of the RTB technology are generally the same as for offline storage, i.e., prioritized siting; requires large blocks of open space for siting; may generate odours (will need control); construction will be disruptive; NIMBY with medium to high environmental concerns. As with the VSS, the RTB has the added disadvantage involved in the disinfection process: either transporting, storage and handling of hypochlorite and metabisulphite or operating and maintaining UV disinfection facilities.

The costs of the RTBs were based on the costs given in Table 4-4. The implication of the 5,000 m^3 units is that the RTBs would be more expensive than the storage units, eg., 30,000 m^3 of storage would be constructed as one large reservoir; 30,000 m^3 of RTBs comprise six units at 5,000 m^3. The result would be a cost for 30,000 m^3 of storage of $13.5 million (Table 4-4) and the cost of the RTBs would be 6 x $4.3 million, that is, $26 million.



Retention/Treatment Basins (RTB)

(Illustrative Only)

Figure 4-3

vss-rtb2
01_files

The storage aspect of the RTBs was sized on the same basis as the offline storage tanks. The number of 5000 m^3 units per district ranges from 1 to 6, with the average falling between 2 and 3. The difference is that the RTB then has additional capacity to settle and disinfect a flow-through component of the CSO. Thus, for the 1992 situation, there will be no untreated overflows. We believe that the RTB's capacity for treatment has been underestimated by this approach, i.e., it is likely that smaller total volumes would suffice because of the treatment of the flow-through portion. This will be tested by the regional model on a district-by-district basis in Phase 3.

The estimated cost of the region-wide RTB system for the 1992 season is $300 million.

4.4 SEPARATION

The wet weather discharge of combined sewage can be eliminated by construction a separate collector system for the sanitary sewage. The benefits of such a separation would be significant for reduction of fecal coliforms. Aesthetically, separation would eliminate the floatables associated with domestic sewage being discharged to the rivers. Storm sewage by itself, however, still carries pollution loads (particularly BOD and TSS) and will continue to discharge floatables associated with surface debris to the rivers. Partial separation has been used in the City of Winnipeg wherever it is economically viable as part of the basement flood relief program. Probably the most significant arguments against total separation are its very large cost and the significant and wide-spread disruption associated with the retrofit separation of sewer systems in built-up areas. In addition, although it does mitigate to a large degree the fecal coliform issue, aesthetics is still not totally addressed.

The costs of sewer separation are summarized in Table 4-6. The estimates, for the most part, are based on studies carried out for other cities and extrapolated to City circumstances. The first estimate originates from the Red/Assiniboine Study (Wardrop/Tetr*ES* 1990) drawing on experience across Canada. The second grouping comes from various basement flooding studies for the City of Winnipeg: the Munroe Annex (Wardrop 1985); Baltimore and Selkirk (IDE, November 1993 and July 1993, respectively). The recent US studies were referenced in the Association of Metropolitan Sewerage Agencies, "Report on Approaches to CSO

TABLE 4-6

SEWER SEPARATION
COST FOR WINNIPEG - CITY WIDE

Estimates for 10,500 ha - based on cost/ha in other specific studies

1. River study (Red and Assiniboine)

    - from $700 million (Weatherbe, Ontario) to $1,000 million (Vancouver GRD)

2. Recent Winnipeg studies (Basement Flooding)

- Monroe Annex (Wardrop)	$1,100 million
- Baltimore (IDE)	950 million
- Selkirk (IDE)	550 million*

3. Recent U.S. studies

- Sacramento, Cal (1992 Study)	1,800 million
- Hartford, Conn (1978-1992)	1,700 million

---

4. Range $550 M to $1,800 M

Range $550 M to $1,800 M, cost estimate of about $1,000 M consider valid.

* Not considered representative

Program Development", (AMSA, 1994). Given the much higher figures reported on most of the remaining estimates, the $500 million estimate using the Selkirk study would seem to be suspect. It probably reflects especially favourable conditions in this particular combined sewer district. The remaining estimates support the previous indication that $1,000 million is still a reasonable estimate of the cost of separating the combined sewer area of the City of Winnipeg.

As has been found in other cities (WEF 1989), complete separation incurs very high capital costs, is time consuming, disruptive to the community and generally deemed impractical. Also, it does not eliminate pollution, but simply converts a CSO to a land drainage overflow.

4.5 FLOATABLES CONTROL

The prior technologies address both the fecal coliform issue and floatables control. This is accomplished either by a reduction in the number and volume of overflows, (i.e., increased interception capacity through inline or offline storage) or by high rate treatment and separation, which reduce the levels of both contaminants either by treatment or by separation of sanitary storm overflows at the source. There are also devices available that would address the floatables issue in isolation from fecal coliforms. These are the subject of this section.

As part of the Phase 2 investigation, the study team compiled a review of technologies for removal of floatables from combined sewer overflows. This document is included in Appendix B (W2O, May 16, 1995). The study examined control technologies for the removal of floatable materials, excluding oil and grease, from CSOs. The categories of treatment technologies reviewed in that report were:

- 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 report characterizes floatables and screenings, provides a description of each of the technologies reviewed, operating experiences, and costs. The latter comprise equipment costs only. A significant portion of the cost associated with screening can be associated with peripherals such as control gates, conveyors, housing and ancillary equipment.

The types of screening and their use comprise the following:

- Mechanically cleaned bar screens, which can be used to protect downstream equipment, e.g., the screens included with the VSSs, or can be free standing to remove the larger floatables.

- Fine screen technologies, typically used in lieu of sedimentation for primary treatment or to upgrade existing primary sedimentation facilities. These can be expected to remove between 15 - 50% of suspended solids.

- Weir mounted CSO screens, comprise an alternative to bar screens and fine screens, particularly where headroom and/or floor space can present installation difficulties.

- Netting trap systems comprise disposable nylon mesh bags installed at a CSO outfall, or within a channel overflow structure. Floatables and coarse solids are strained from the CSO and captured in the bags. Once full, the bag is removed and hauled to a landfill for disposal.

It is too early in the study to decide whether or not solids removal only would be a suitable option for some or all of the combined sewer districts. Accordingly, these options will be carried forward to Phase 3 and their use will form a part of the total evaluation of modified CSO discharges on river quality.

For the purposes of preliminary evaluation, costs have been developed for the installation of trash racks, their housing and ancillaries at the end of each combined sewer outfall. Table 4-7 provides the costs for free standing screens from 1 m^3/s to 6 m^3/s capacity. These were applied to the maximum treatment rates provided in Table 4-2 and resulted in an estimated total cost, if this technology was applied to each combined sewer trunk, of $110 million.

TABLE 4-7

FREE STANDING SCREEN FACILITIES

	Size of Screen			
	1 m^3/sec	2 m^3/sec	4 m^3/sec	6 M^3/sec
Gates	$100,000 (2)	200,000 (4)	300,000 (6)	400,000 (8)
Screens	450,000 (2)	450,000 (2)	1,000,000 (3)	1,700,000 (4)
Conveyors	200,000 (2)	200,000 (2)	300,000 (3)	500,000 (5)
Ancillaries (30%)	200,000	300,000	500,000	800,000
Housing	100,000	100,000	200,000	300,000
	1,100,000	1,300,000	2,300,000	3,700,000
Contingencies (20% Estimating) x (20% E, A & F*)	500,000	600,000	1,000,000	1,600,000
	$1,600,000	$1,900,000	$3,300,000	$5,300,000
TOTAL PROJECTED COST (Using Flow Rates on Table 4-2) = $110 Million				

*Engineering, Administration and Finance

These installations will require: land; a fair amount of operating; odour control. They will be subject to the NIMBY syndrome.

An approach similar to the estimate for a total screening system was applied to the TrashTrap system. In the W2O document (Appendix B), costs were developed for a system capable of treating a peak CSO flow of 500 L/s. The report noted that this was preliminary since the real costs are very site specific. The costs developed, including equipment, engineering support and installation costs, were $153,000. Applying the 20% estimating contingency and 20% allowance for engineering, administration and finance brought this number up to $225,000. In order to approximate the cost of applying the trash trap technology to all the outfalls, the costs were pro-rated (i.e., $225,000 was allowed for every increment of 500 L/s) and applied to the peak flows given for each district on Table 4-2. The results are given in Table 4-8. The total estimated costs for a system-wide installation would be in the order of $30 million. The operating costs are in the order of $1,000 per CSO event, to a maximum of about 30 events per outfall per year. With 40 outfalls, the total operating cost could be in the order of $1.2 million per year.

There is limited experience with these devices, although extensive pilot tests have been run (New Jersey). The distinct advantage is the fact that the infrastructure is all in the river. That is, aboveground structures are unnecessary. This avoids the possible odour problems associated with screenings handling. In addition, there should be no land requirements since the devices will be accessed over lands already used as the CS trunk right-of-way. Environmentally, there will be visual impact, but this does not appear to be too objectionable. It will be possible to locate these devices on a prioritized basis. The bags will have to be removed and replaced on a routine basis.

4.6 AREA-WIDE REAL-TIME CONTROL

The EMA document on real time control is included in Appendix C (EMA, July 1995). The building blocks for system-wide RTC are the control systems installed for the fail-safe control of in-system storage, combined with fail-safe operation of subsequently installed control devices. The individual systems can be designed so that filling and emptying of in-system

TABLE 4-8

COSTING FOR TRASH TRAP USING W_2O COSTS

Flow Range	# of Outfalls ($ per outfall	Cost Extension ($1,000)
0 to 500	12 (@ 225K)	2,700
500 to 1,000	11 (@ 450K)	4,950
1,000 to 1,500	5 (@ 675K)	3,375
1,500 to 2,000	2 (@ 900K)	1,800
2,000 to 2500	5 (@ 1125K)	5,625
2,500 to 3,000	3 (@ 1350K)	4,050
3,000 to 3,500	1 (@ 1575K)	1,575
4,000 to 4,500	1 (@ 2025K)	2,025
5,500 to 6,000	1 (@ 2700K)	2,700
ESTIMATED TOTAL FOR REGION-WIDE SYSTEM		$28,800

<u>BASIS</u>

- 2 Bag System = 500 Lps = 150,000 x 1.44 = $225,000 storage and the control devices, and the subsequent conveyance of those stored flows to the WPCC, can operate safely by setting the discharge rate at a fixed, workable maximum. Extension of this approach to the RTC (referred to as Smart Real Time Control in the EMA document) would comprise the combination of the number of the inline storage and end-of-pipe treatment devices into a single SCADA system which would optimize the release of the stored water so as to maximize the conveyance capacity of the interceptor and the WPCC facilities and minimize flows to the river (see Figure 3-2, from Appendix C, attached).

Typically, area-wide RTC is applicable if there is long, large-area sewers (Seattle) or many near surface storage tanks (Hamilton) for optimum use of storage. The likely level of practicability for RTC systems would be the establishment of operating rules for the emptying of in-system/offline storage or the variation of the diversion rates to the interceptor system. Offline computer models could simulate various operating scenarios to refine and enhance the operating rules. This system would require additional sensors to measure flow in the sewers and in the overflows and possibly even pollution concentration in the overflow and in the receiving water. The development of such a system would only be considered after numerous treatment facilities were in place. In other words, it would become an optimization technology to provide a increment of enhancement to a CSO control system.

Anything beyond "Smart RTC" (ie. rule driven with human interaction) is unlikely to be implemented in the foreseeable future.

It is too early to estimate the applicability and cost of an area-wide RTC system. This will be done as more information as to the nature of the system is gathered. In any case, it is further optimization of the use of storage, either inline or constructed storage.

4.7 COST AND EVALUATION SUMMARIES

A summary of the costs of the various alternatives discussed in Sections 3 and 4 , is provided in Table 4-9. Table 4-10 represents a summary of subjective evaluations of cost and non-cost items as discussed in the sections on each option.

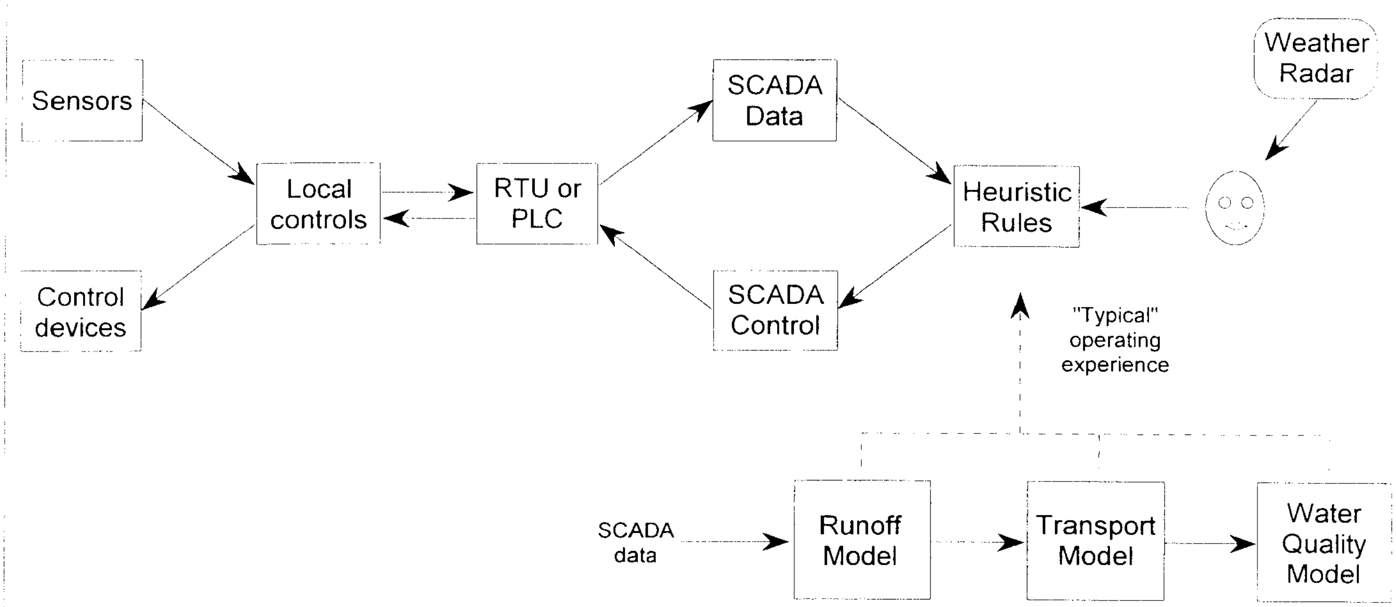


Figure 3-2 Smart Real Time Control (from Appendix C)

TABLE 4-9
SUMMARY OF REGION-WIDE CONTROL OPTIONS
COSTS AND BENEFITS

	Technology	Cost ($ Million)	Benefit
1.	Minimum Structural Controls		
	• Increased Interception Rate (5 x DWF)	40	Reduction in volume and frequency of CSOs (includes cost of possible NEWPCC upgrade)
	• In-line Storage	20	Reduction of CSOs
2.	Structurally Intensive Controls		
	• Storage		
	○ Tunnel (Storage = 300,000 m^3)	400	Dramatic reduction in CSOs
	○ Major tunnel (Storage = 1 million m^3)	650	Virtual elimination of CSOs
	○ Off-line (Storage = 300,000 m^3)	210	Dramatic reduction of CSOs
	• High Rate Treatment		
	○ VSS (with disinfection)	440	Dramatic reduction of FC and floatables
	○ RTB (300,000 m^3 storage + disinfection of additional 70 m^3/sec.)	300	Reduction of volume and frequency of CSOs and Disinfection of additional overflows
3.	Separation	1,000	Eliminates raw sewage discharges
4.	Floatables Control		
	• Screening	110	No reduction of fecal coliforms
	• Trash Netting	30	No reduction of fecal coliforms

Notes:

1. Reduction of volume and frequency of CSOs equates to reductions of fecal coliform concentrations and quantity of floatables in the river.

TABLE 4-10

SUMMARY OF REGION-WIDE CONTROL OPTIONS

OPTION	Relative Cost	Construction Practicality	Operating Effort	Land Use Issues	Environmental Issues
DWF Issues WPCC disinfection/ DWO correction	Low	High	Moderate	N/A	Low
Optimizing Existing Infrastructure BMP	Low	High	Low	Low	Low
5 x DWF	Low	High	Moderate	Low	Low
5 X DWF/in-line storage	Low	High	Moderate	Low	Low
Structurally intensive complete separation	Very High	Low	Low	Very High	High
Storage - Tunnel (some overflows) - Tanks (some overflows) - Tunnel (eliminate CSO)	 High High Very High	 High Medium High	 Moderate Moderate Moderate	 Short Term Medium Short Term	 Construction Only Medium Construction Only
High Rate Treatment - RTB (with disinfection) - VSS	 High High	 Medium Medium	 High High	 High High	 High High
Central Treatment		Impractical			

The options will likely be used in groups. Logical combinations of the various individual options are provided in Table 4-11. The effects of these various options and combinations are discussed in Section 5 and the costs and effects are compared in Section 6. Item 5 (Full CSO Disinfection) could in fact be partial disinfection of larger storms because of the bypassing of flows in excess of the design capacity of the VSS or RTB.

5.0 CSOs FOR VARIOUS CONCEPTUAL CONTROLS

5.1 GENERAL APPROACH

In order 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. These loadings must be placed in perspective with the other stream loadings such as LDS, plant effluents, etc. This 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. To complete this, a data management system was developed. 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.

In order to put into perspective the impact of various CSO control alternatives on the receiving stream, the impacts of other Land Drainage and Sewerage System discharges must be considered. These other discharges include the plant effluents during DWF and WWF. A schematic of the various components in the Regional System is shown in Figure 5-1. The shaded area of the Figure shows the development of stream loadings, which are input to the WASP river quality model.

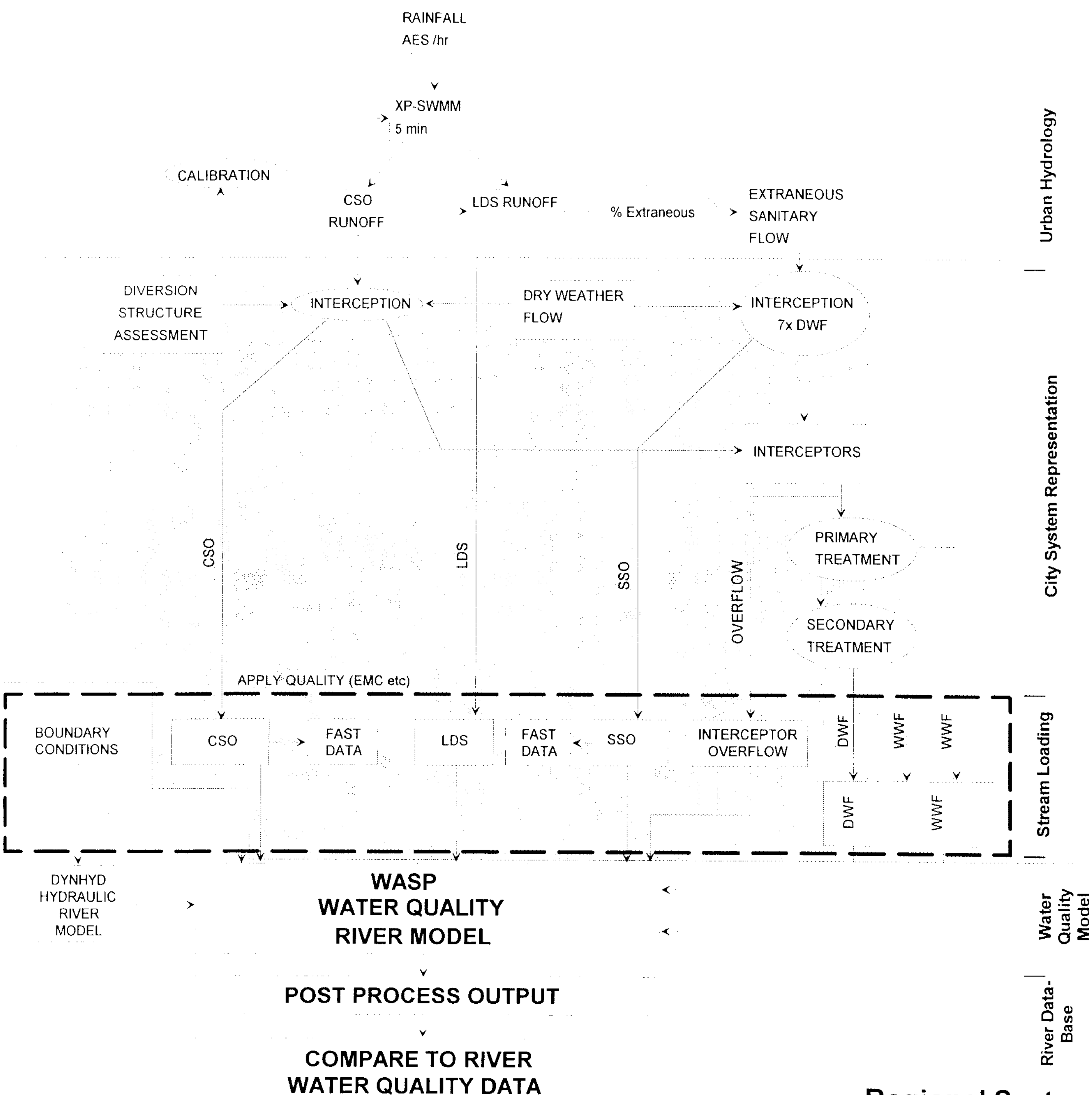
A comprehensive database management system was developed (using Paradox database software) which would "track" all dry weather and wet weather flows and their deposition, either to the interceptor, the WPCCs or to the rivers.

TABLE 4-11

POTENTIAL COMBINATIONS OF CSO TECHNOLOGIES

CONCEPTUAL OPTIONS	
1)	Disinfect WPCC effluent and DWO corrections
2)	Intercept 5 X DWF
3)	In-line storage and 5 x DWF
4a)	Distributed Storage (300,000 m^3)
4b)	Tunnel Storage (300,000 m^3)
4c)	Regional Tunnel Storage (1,000,000 m^3) - Eliminate CSO
5)	Full CSO disinfection (this could be partial)
6)	Full CSO separation
7)	Floatables Removal
LOGICAL COMBINATIONS	
A = 1	
B = 1 + 2	
C = 1 + 3	
D = 1 + 3 + (4a or 4b)	
E = 1 + 4c	
F = 1 + 3 + 5	
G = 1 + 6	
H = 1 + 3 + 7	

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. Other factors, such as cost, enter into this evaluation, as discussed in Section 5.



Regional System Model Components

Figure 5-1

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 inline 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 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 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 hydraulics 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.

The dry weather coliform loading and the wet weather loading from upstream of the city is not considered to be significant and is therefore not modelled as a stream load for this first level screening analysis. (The upstream boundary conditions are considered in the water quality model).

Using this methodology, an annual summary of the overflow data on a city-wide basis and for each district was produced for existing conditions to provide:

- the number of overflows;
- the volume of overflows;
- the volume intercepted; and
- the percent of runoff intercepted.

A conceptual assessment of the structurally intensive options was done in Phase 2 using the existing system overflow hydrographs (or the hydrographs for the optimized infrastructure) and modifying the "pollutographs" by applying different EMCs. To illustrate, the effect of separating the existing combined sewer system can be simulated by converting the CSOs to land drainage, i.e., the overflow hydrograph volume is converted to the pollutograph associated with an LDS. This is described in more detail in Section 5.4.3.

Similar performance data was developed for a range of conceptual CSO control alternatives.

These data were used, in addition to the receiving stream assessment, to compare the performance of various control alternatives. These results are shown in Sections 5.4.1 and 5.4.2.

5.2 THE CONTROL SYSTEM MODEL (REGIONAL MODEL)

5.2.1 Combined Sewer District

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 5-2a) developed by the calibrated XP-SWMM model (see 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. In Phase 3, the DWF information will use sewer gauging reports when information is available for the specific district. At this stage, the impact of (reportedly) extreme summer DWF on 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

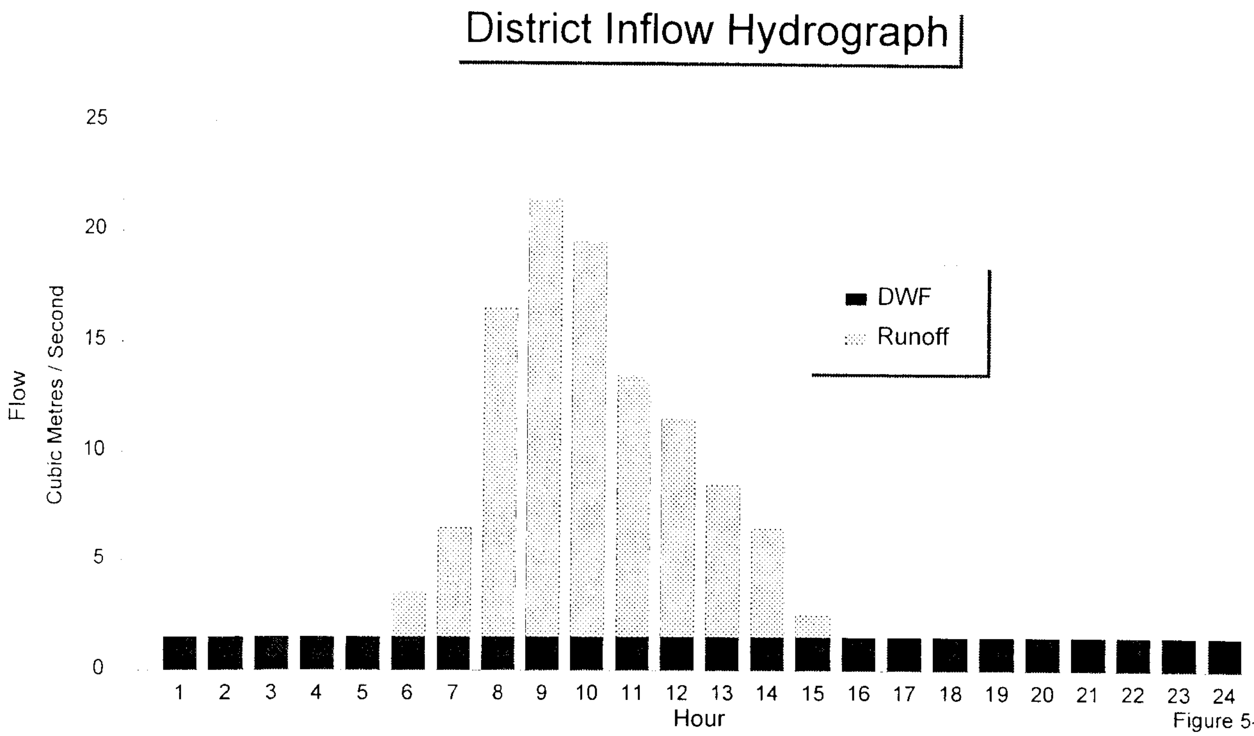


Figure 5-2a

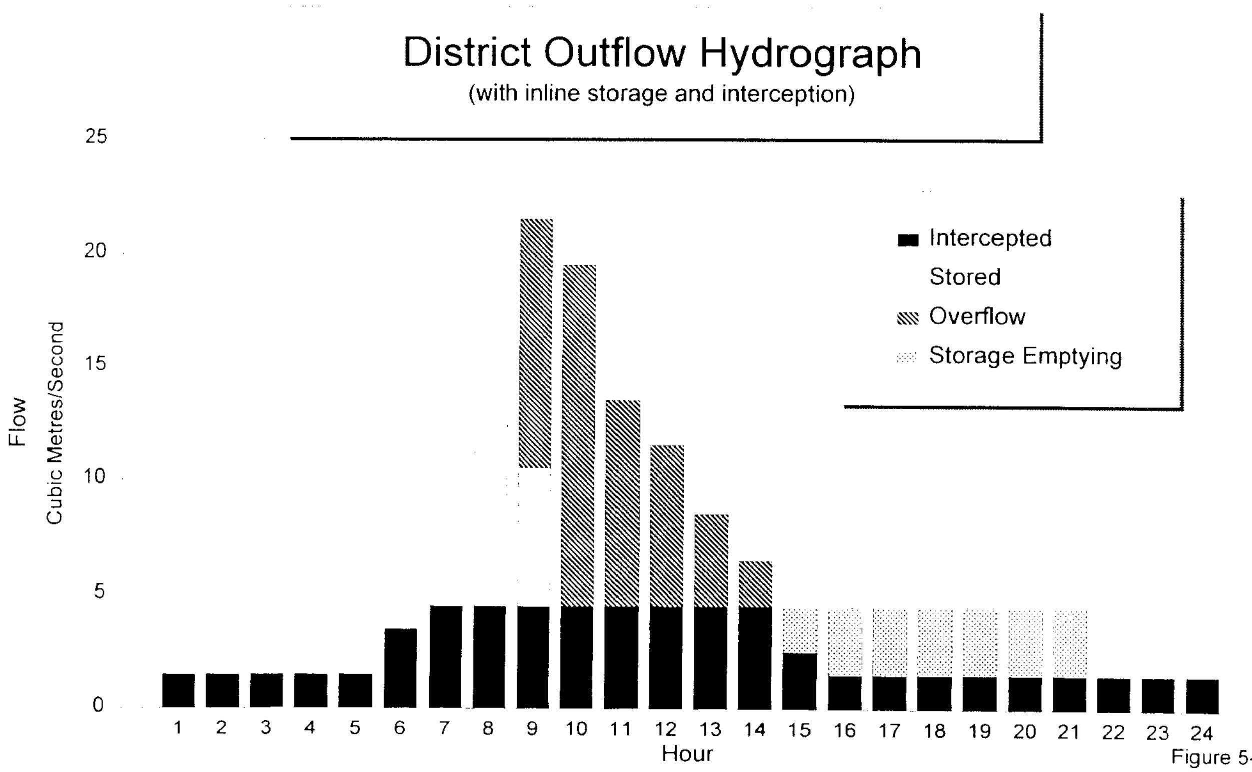


Figure 5-2b

MODELLING OF CSO CONTROL
(INLINE STORAGE AND INTERCEPTION)
Figure 5-2

refinement at this stage. For further detail on DWF, see TM #2 - Infrastructure). The assumed DWF for each district in shown in Table 5-1.

The next stage of the control system model is to determine the impact of various controls on the amount of CSO intercepted and overflowing to the river. The inflow hydrograph is allocated (see Figure 5-2b) using the following values:

- All DWF plus runoff is intercepted up to the district interception rate given in Table 5-1. (See TM #2 - Infrastructure for details on calculations of interception rates).

- Once the hourly flow rate is greater than the interception rate, the volume is considered to be in storage (inline or offline), 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 empty.

The outflow hydrograph is shown on Figure 5-2b. In the database model, the overflow hydrograph is stored in a separate table and the appropriate EMC is multiplied by the volume of CSO for each hour to create a "pollutograph". This pollutograph is further processed to provide an intermittent " non-point source" (*.NPS) input file for US EPA's WASP dynamic water quality simulation model. The intercepted hydrograph, (interception plus storage emptying) which will be conveyed to treatment, is stored in a separate data field (Figure 5-2b). The intercepted hydrographs can be added together (along with sanitary sewer interception) to estimate flow at the WPCCs. This is discussed in Section 5.2.4 Interception System.

It should be noted that increased storage in the combined sewer district, i.e., inline storage, will increase the duration during which the peak discharge to the interceptor will occur since the captured flows will be released as quickly as possible, after the rainfall event, to the WPCC for treatment.

TABLE 5-1
Summary:Existing Control

District Number	District Name	DWF m³/s	Interception m³/s	X DWF	Comments
1	Alexander	0.035	0.155	4.4	
2	Armstrong	0.02	0.524	26.2	
3	Ash	0.082	0.301	3.7	
4	Assiniboine	0.084	0.425	5.1	
5	Assiniboine Park	0.0001	0.0003	3.0	Assumed 3xDWF (not CSO)
6	Aubrey	0.071	0.214	3.0	
7	Baltimore	0.028	0.201	7.2	to Mager Drive
8	Bannatyne	0.153	0.613	4.0	
9	Boyle	0.014	0.03	2.1	
10	Calrossie	0.001	0.028	28.0	
11	Clifton	0.077	0.236	3.1	
12	Cockburn	0.033	0.075	2.3	to Baltimore
13	Colony	0.134	0.425	3.2	
14	Cornish	0.035	0.107	3.1	
15	Despins	0.032	0.132	4.1	
16	Doncaster	0.025	0.075	3.0	Assumed 3xDWF to Ash
17	Douglas Park	0.001	0.095	95.0	
18	Dumoulin	0.013	0.136	10.5	
19	Ferry Road	0.059	0.126	2.1	
20	Hart	0.039	0.101	2.6	
21	Hawthorne	0.036	0.113	3.1	
22	Jefferson E	0.143	0.569	4.0	
23	Jefferson W				Send ALL to Jeff East
24	Jessie	0.066	0.176	2.7	
25	La Verendrye	0.009	0.015	1.7	
26	Linden	0.017	0.06	3.5	
27	Mager Drive	0.091	0.309	3.4	PS (flows from Balt Metcalf)
28	Marion	0.032	0.22	6.9	
29	Metcalfe	0.005	0.044	8.8	to Mager
30	Mission	0.144	0.518	3.6	
31	Moorgate	0.023	0.085	3.7	
32	Munroe	0.077	0.237	3.1	
33	Newton	0.01	0.166	16.6	
34	Polson	0.032	0.356	11.1	
35	River	0.07	0.094	1.3	
36	Riverbend	0.053	0.107	2.0	
37	Roland	0.026	0.324	12.5	
38	Selkirk	0.067	0.453	6.8	
39	St. Johns	0.084	0.173	2.1	
40	Strathmillan	0.003	0.062	20.7	
41	Syndicate	0.01	0.069	6.9	
42	Tuxedo	0.004	0.036	9.0	to Doncastor
43	Tylehurst	0.05	0.176	3.5	
44	Woodhaven	0.00227	0.027	11.9	Pumped
	Minimum	**0.0001**	**0.0003**	**1.342857**	
	Maximum	**0.153**	**0.613**	**95**	
	Sum	**1.99037**	**8.3883**		
	Average			**8.5**	
	Weighted Average			**4.2**	

Discharging to Another Combined Sewer District

In some cases the existing infrastructure is such that the intercepted hydrograph from an individual district is not discharged directly into an interceptor, but is conveyed to another combined sewer district. The districts which have been modelled in this manner are:

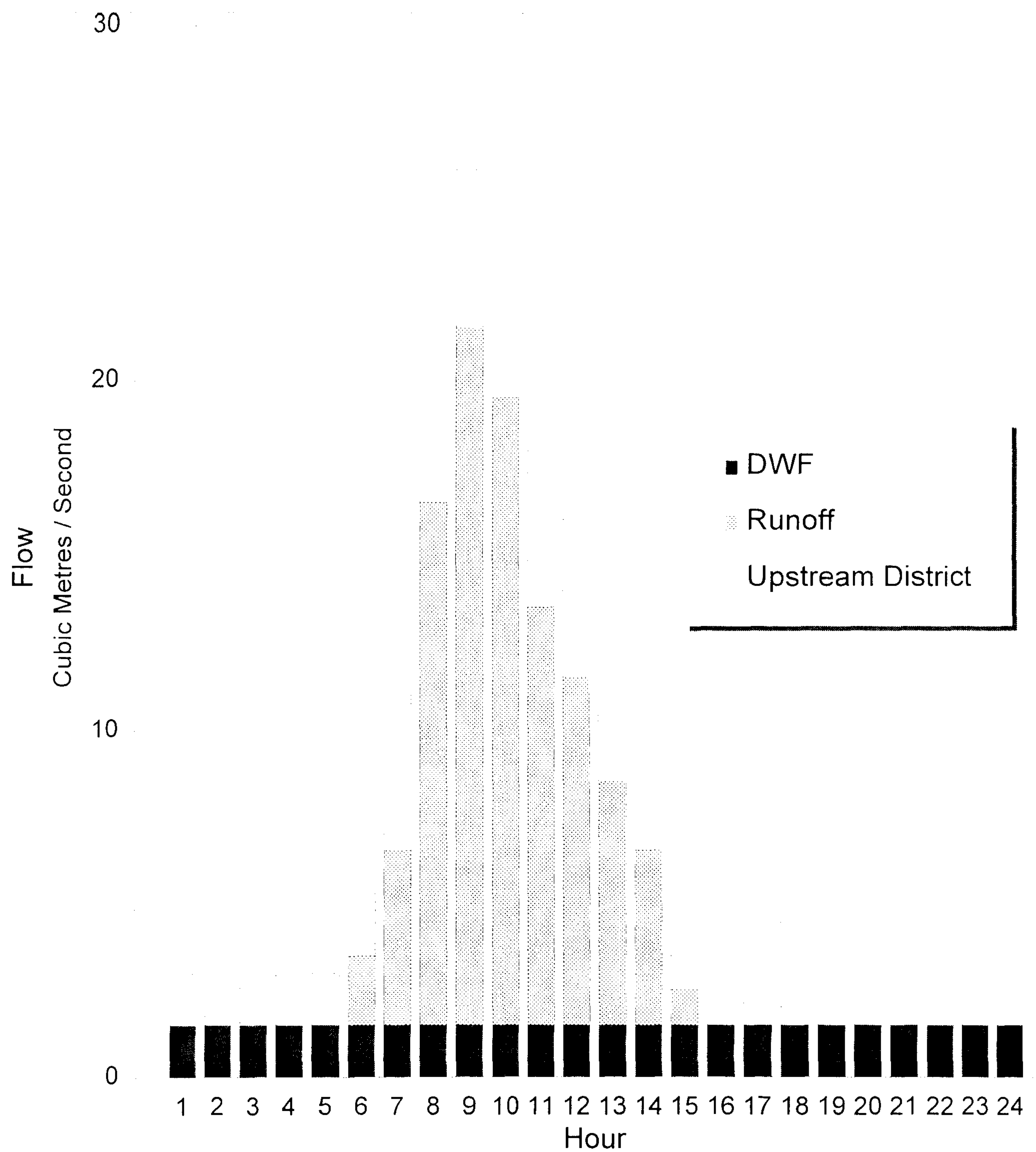
- in the South End System;
  - Calrossie to Cockburn - Cockburn to Baltimore - Baltimore to Mager Drive and
  - Metcalf to Mager Drive.
- in the North End System:
  - Tuxedo to Doncaster - Doncaster to Ash.

In addition, all flows from Jefferson West district are considered to be inflow to Jefferson East district.

In order to account for the cumulative effect of these wastewater transfers, the model adds the intercepted hydrograph from the upstream district to the inflow hydrograph of the downstream district (see Figure 5-3).

5.2.2 Land Drainage System

The runoff from the land drainage system in the separate sewered area is calculated by XP-SWMM model (see TM #1 - Problem Definition). Since there are no controls, the "system model" is very straight-forward. The data is imported into the database (Paradox) tables where the model calculates summary statistics (i.e., volume of runoff, mm of runoff etc.). These data are then multiplied by the appropriate EMC and processed into an intermittent "non-point" source file (LDS.NPS) to be input into the US EPA WASP model. These LDS runoff data are stored in hourly format for each district (see Figure 5-4 districts 45 to 98) and are subsequently used in estimating the extraneous flow in the sanitary system as discussed below.

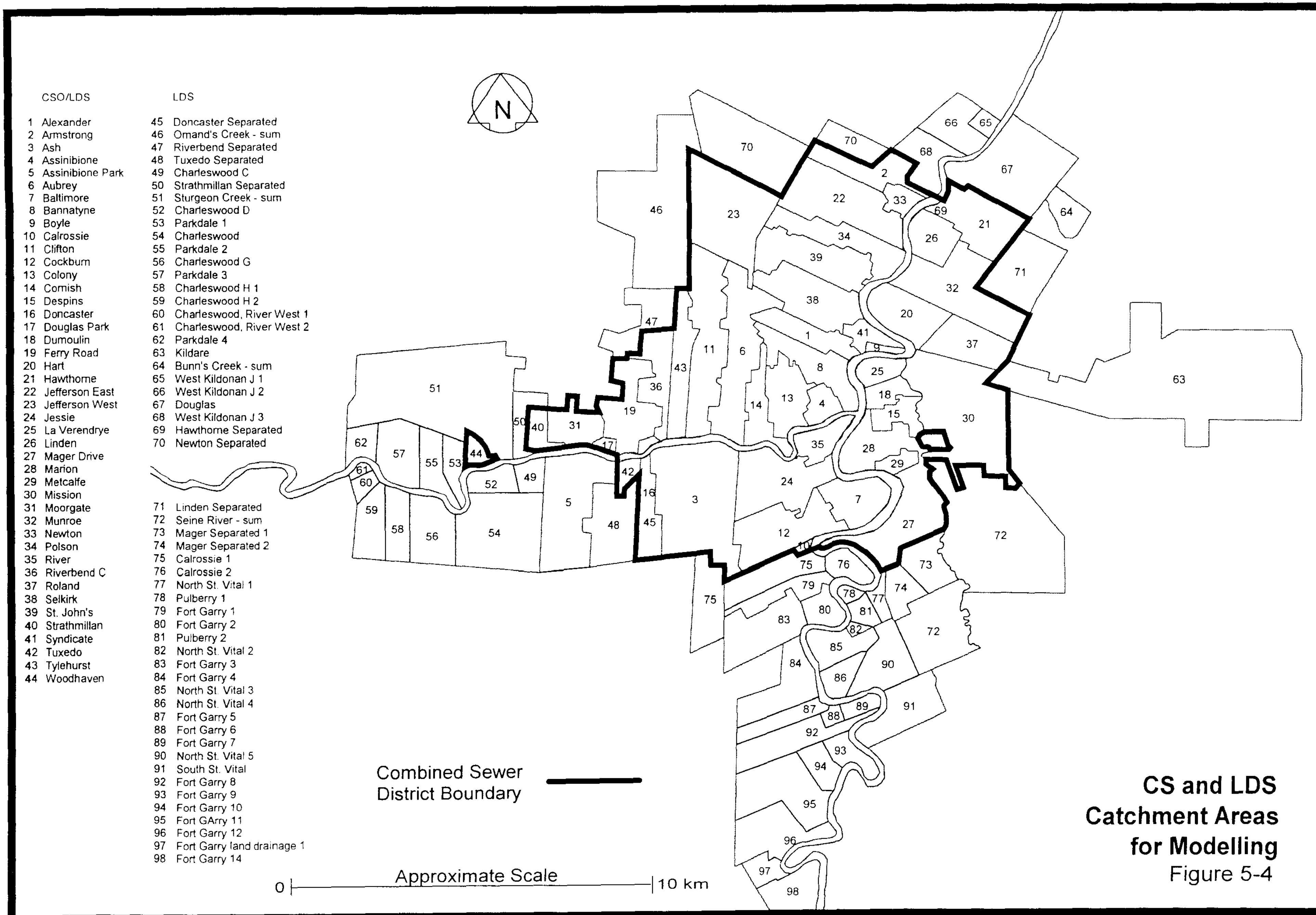


District Inflow Hydrograph

(when an uptream district is discharged to the district)

Figure 5-3

PH2REP.WK4


CSO/LDS
1 Alexander
2 Armstrong
3 Ash
4 Assinibione
5 Assinibione Park
6 Aubrey
7 Baltimore
8 Bannatyne
9 Boyle
10 Calrossie
11 Clifton
12 Cockburn
13 Colony
14 Cornish
15 Despins
16 Doncaster
17 Douglas Park
18 Dumoulin
19 Ferry Road
20 Hart
21 Hawthorne
22 Jefferson East
23 Jefferson West
24 Jessie
25 La Verendrye
26 Linden
27 Mager Drive
28 Marion
29 Metcalfe
30 Mission
31 Moorgate
32 Munroe
33 Newton
34 Polson
35 River
36 Riverbend C
37 Roland
38 Selkirk
39 St. John's
40 Strathmillan
41 Syndicate
42 Tuxedo
43 Tylehurst
44 Woodhaven
LDS
45 Doncaster Separated
46 Omand's Creek - sum
47 Riverbend Separated
48 Tuxedo Separated
49 Charleswood C
50 Strathmillan Separated
51 Sturgeon Creek - sum
52 Charleswood D
53 Parkdale 1
54 Charleswood
55 Parkdale 2
56 Charleswood G
57 Parkdale 3
58 Charleswood H 1
59 Charleswood H 2
60 Charleswood, River West 1
61 Charleswood, River West 2
62 Parkdale 4
63 Kildare
64 Bunn's Creek - sum
65 West Kildonan J 1
66 West Kildonan J 2
67 Douglas
68 West Kildonan J 3
69 Hawthorne Separated
70 Newton Separated
71 Linden Separated
72 Seine River - sum
73 Mager Separated 1
74 Mager Separated 2
75 Calrossie 1
76 Calrossie 2
77 North St. Vital 1
78 Pulberry 1
79 Fort Garry 1
80 Fort Garry 2
81 Pulberry 2
82 North St. Vital 2
83 Fort Garry 3
84 Fort Garry 4
85 North St. Vital 3
86 North St. Vital 4
87 Fort Garry 5
88 Fort Garry 6
89 Fort Garry 7
90 North St. Vital 5
91 South St. Vital
92 Fort Garry 8
93 Fort Garry 9
94 Fort Garry 10
95 Fort GArry 11
96 Fort Garry 12
97 Fort Garry land drainage 1
98 Fort Garry 14
N
Combined Sewer
District Boundary
Approximate Scale
0
10 km
CS and LDS
Catchment Areas
for Modelling
Figure 5-4

sewrmap2

5.2.3 Sanitary Sewer System

It is important to understand potential wet weather effects on the sanitary system for two reasons:

- to estimate the volume and frequency of sanitary sewer overflows (SSOs) to the river. While sanitary sewers are not intended to carry significant wet weather flows, it is a recognized fact that extraneous wet weather flows can easily overload sanitary sewers and cause overflows. The impact of these overflows will help to place into perspective the benefits of various CSO control strategies, i.e., there is a point at which increased CSO control may not improve river water quality, due to the frequency and volume of SSOs.

- to estimate wet weather inflow hydrographs from separated sanitary systems to the interceptor systems and WPCCs. This is important since some CSO control options such as increased storage or interception rates will increase the wet weather flow to the interceptor system and WPCCs. Reductions in extraneous flow in the sanitary district may complement any CSO control strategy by making more capacity available in the interceptor system and WPCCs during wet weather events.

Earlier river studies (Wardrop/Tetr*ES* 1991) assumed 10% of LDS runoff was discharged to the river as SSOs. This was recognized as a likely over-estimate of the actual volumes. SSOs were considered to occur only infrequently; maybe once in ten years. Recently, experience in the City during the relatively wet summers of 1993 and 1994 indicates that sanitary overflows may occur more frequently than previously assumed although the volume of overflow may still be less. In order to obtain a better perspective of SSOs it was decided to expand the scope of this study. As with the CSO control model, a simple mass balance model was determined to be the most appropriate at this stage of the study. The concept is similar to the combined sewer system. An inflow hydrograph is created by adding extraneous flow to the dry weather flow (DWF) for the district. The outflow hydrographs (interception hydrograph and overflow hydrograph) are determined by assuming a given interception rate (multiple of DWF) for all districts. Rather than model each sanitary district, a conceptual district was assumed for each LDS area modelled.

A percentage of the LDS runoff is considered to be extraneous flow which is added to the DWF for that area to produce a conceptual inflow hydrograph. In other words, a wet weather sanitary sewer hydrograph is pro-rated from the LDS hydrograph. The DWF for each area was estimated by using the mm/hour wastewater generation rate for each WPCC service area. In this case the service area was defined as the total area of the LDS in the area serviced by each WPCC. The unit DWF rates for areas serviced by each of the plants were:

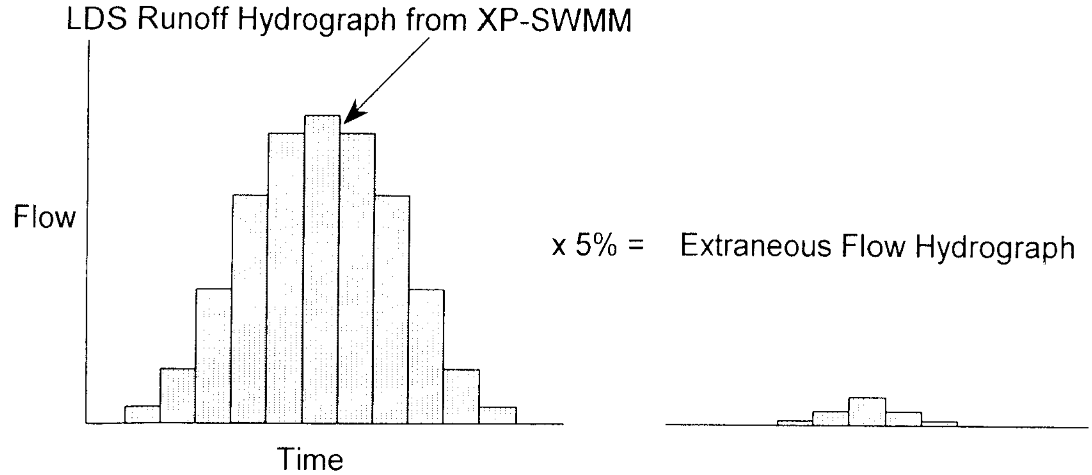
North End Service Area - 0.043 mm/hour
South End Service Area - 0.038 mm/hour
West End Service Area - 0.036 mm/hour

These unit rates can then be multiplied by the area in each LDS district to give an estimate of DWF.

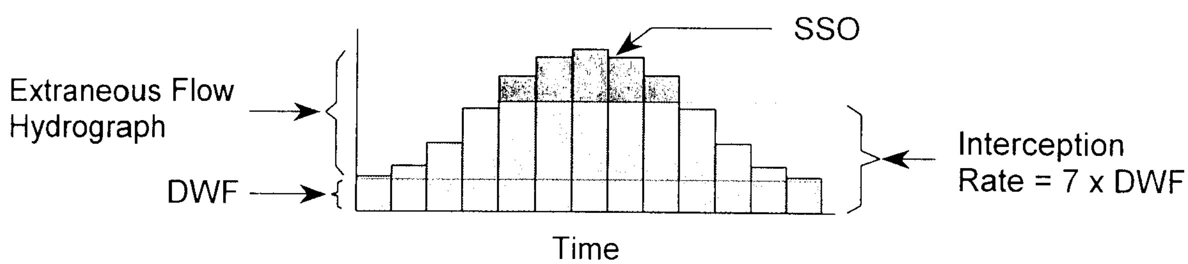
The conceptual sanitary sewer district model is summarized in Figure 5-5. Extraneous flow was assumed to be 5% of the runoff from the LDS hydrograph produced by XP-SWMM runoff block. Studies on extraneous flow in the Pulberry district (Rempel 1972) indicated about 10% of the runoff in a separate sewer district could enter the sanitary sewer. Subsequent studies on various Winnipeg separate areas also found that the volume of extraneous flow represents about 10% of the normal storm runoff (Tottle 1972). These studies were done before the City: updated lot grading by-laws for new homes; required sump pumps for new homes; and provided public information about the importance of lot drainage to existing homeowners. These efforts have probably reduced the area-wide volumes of extraneous flows. Accordingly, the lower estimate of 5% of the storm runoff extraneous flow was chosen. This was added to the estimated DWF and the interception rate was assumed to be 7 x DWF. Actual peak values of extraneous flow have been found to vary greatly. In 1972, the typical peak flows were found to be about 7 times the average dry weather flow. This also corresponds approximately to the City design criteria, i.e., total wet weather design allowance/dry weather criteria = about 7. Therefore, the model assumed a typical sanitary system had capacity to convey 7 x DWF to the interceptor. The portion of the wet weather hydrograph in excess of these rates was considered to be an SSO.

For each LDS District

(a) Extraneous Flow



(b) Interception and Sanitary Overflow



DWF = 0.043 mm/hr - NE area
0.038 mm/hr - SE area
0.036 mm/hr - WE area

Conceptual Sanitary Sewer Model

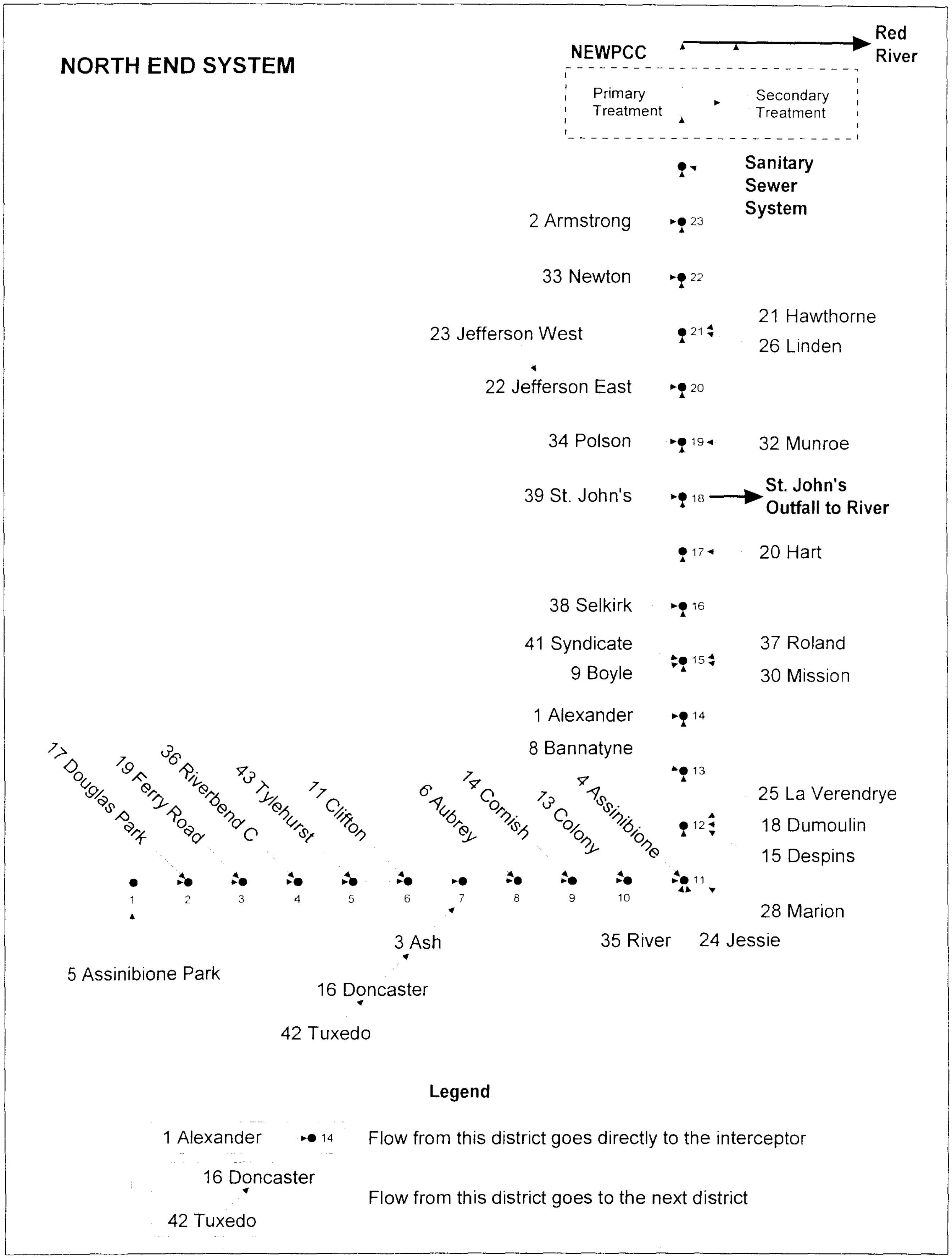
Figure 5-5

5.2.4 Interceptor System

The collection and transport of the intercepted hydrographs is the function of the interceptor system. A complete description of the interceptor system and the development of a detailed XP-SWMM-EXTRAN model is given in TM# 2 - Infrastructure. For the purpose of assessing the flow to the WPCC over the recreation season on an hourly basis, a simple hydrograph addition was performed. Figures 5-6 and 5-7 shows a schematic of the system as represented in this screening collection/treatment model.

In general, most districts discharge directly to an interceptor, either by a diversion weir and gravity connection or by being pumped through a lift station. In the model, intercepted flow is considered to be equal to the inflow until a constant interception rate is achieved during a runoff event. This would be fairly accurate for a pumped system, however, intercepted flow is under-estimated by this method for gravity connections. In a gravity connection, the level in the trunk sewer will increase with flow, thus increasing the head driving the flow to the interceptor. In reality, the flow to the interceptor will increase as a function of the flow in the trunk sewer when the level in the interceptor does not cause a backwater effect. These complex hydraulics are best modelled in a model such as XP-SWMM EXTRAN block. More information on the XP-SWMM model and its application to the interception system is given in TM #2 - Infrastructure.

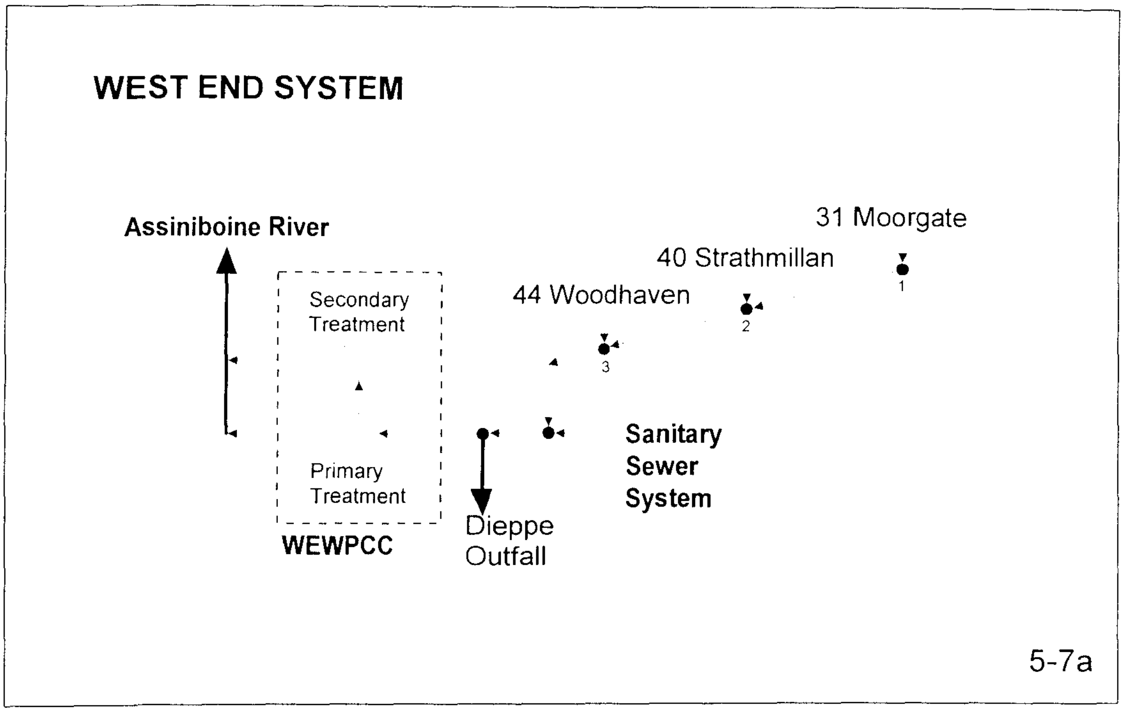
The screening model for the interceptor system assumes the interception rate is constant at the value calculated at incipient overflow (see TM #2- Infrastructure), i.e., the interception at the water level coincident with the top of the interception weir. If necessary in Phase 3, it would be possible to use an interception rate which is a function of the flow in the trunk sewer. In the current model, the intercepted hydrographs for every combined and sanitary sewer district within each WPCC service area, are added each hour. No time lag between hydrographs is assumed (i.e., transport time is not modelled). In this phase, it is deemed unnecessary to account for these time lags since other planning level modelling assumptions have been made (e.g., the airport rainfall is assumed to be uniformly distributed across the city). In the context of a screening activity, these generalizations are appropriate.

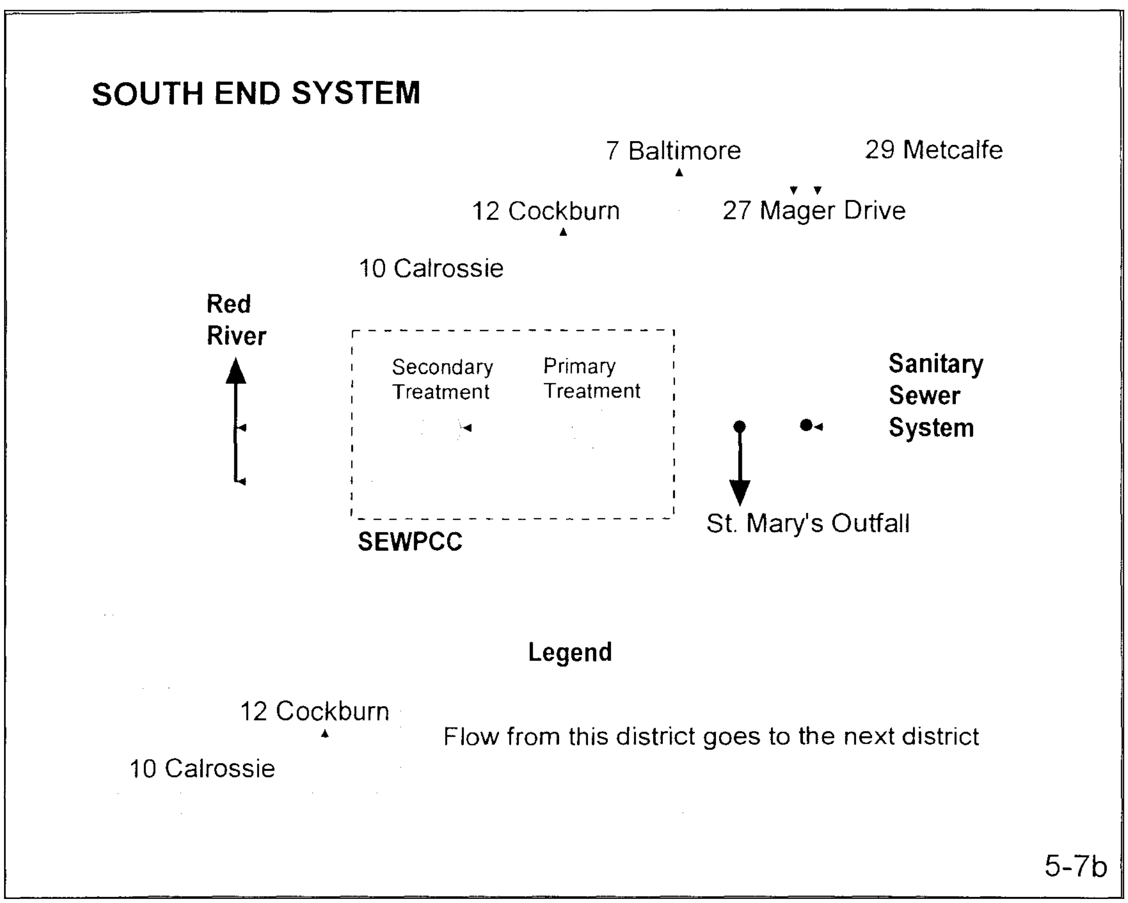


SCHEMATIC OF INTERCEPTION SYSTEM FOR NORTH END SYSTEM

Figure 5-6

intcep
01_pwong





SCHEMATIC OF INTERCEPTION SYSTEM FOR WEST & SOUTH END SYSTEMS

Figure 5-7

intcep
01_pwong

The dynamic hydraulic constraints within the interceptor system are not accounted for and all flow is considered to reach the WPCCs. If the accumulated flow rate in the interceptor exceeds the plant pumping capacity, the excess is assumed to discharge to the river via an interceptor emergency outfall. The location of the assumed discharge for the three service areas was as follows:

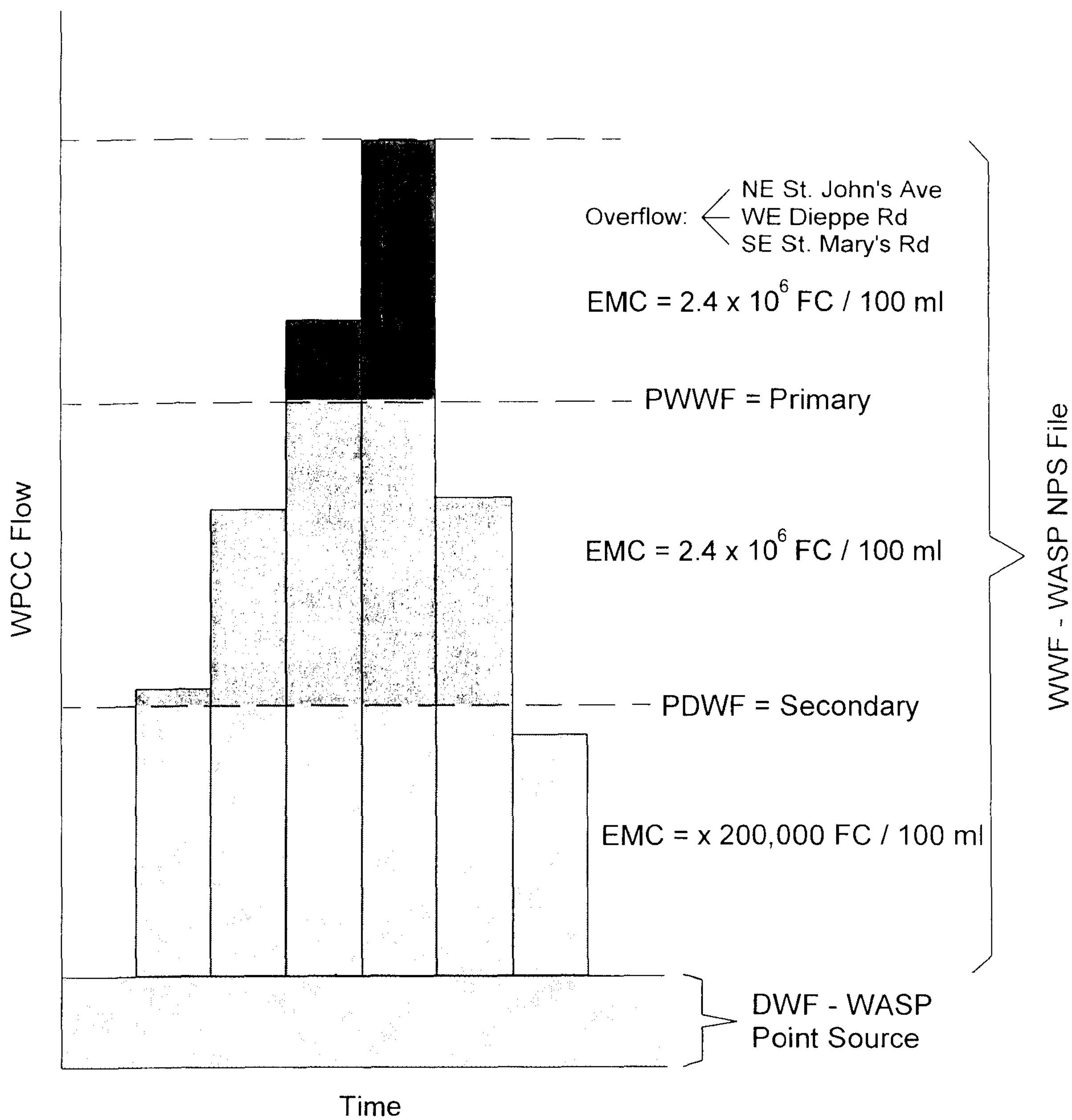
- North End Interceptor - St. John's outfall;
- South End Interceptor - St. Mary's outfall; and
- West End Interceptor - Dieppe outfall.

These interceptor overflow hydrographs are processed into pollutographs by applying the EMC estimated for CSO discharges (2.4 x 10^6 fecal coliform/100 mL). These results are then processed into intermittent 'non-point source' input files (*.NPS) for the US EPA WASP water quality model (see TM #4 - Receiving Stream).

In summary, the screening model for the interceptor system is not a complex hydraulic model but is a comprehensive mass balance inventory system which serves the purpose of estimating hourly volumes of WWF going to each WPCC or to the river via an emergency outfall. The model is able to process hourly volumes (dry weather and wet weather) for an entire season (3600 hours) quickly, thereby allowing numerous control alternatives to be assessed in a short time.

5.2.5 Water Pollution Control Centres (WPCC)

The inventory of WPCC effluents involved an hourly timestep database for tracking effluent volumes. The purpose is to assess, for each hour and for each plant, the volume of inflow to the WPCC receiving primary and secondary treatment or only primary treatment. The inflow hydrograph compiled for each WPCC is described in the previous section. The dry weather flow, which is assumed constant, is abstracted from the wet weather cumulative hydrograph as this is considered a continuous flow point source in the US EPA WASP model (see Figure 5-8). The remaining incremental WWF hydrograph is then apportioned as follows:



Modelling Interceptor/WPCC Loadings

Figure 5-8

load
sewer

- the flow up to secondary capacity (about peak dry weather flow) minus DWF is assumed to have complete secondary treatment and is multiplied by in EMC of 2 x 10^5 fecal coliform/100 mL to produce a pollutograph (similar to the routine dry weather flow).

- the excess WWF up to primary capacity minus secondary capacity is assumed to have only primary treatment and an EMC of 2.4 x 10^6 fecal coliform/100 mL is applied to produce a pollutograph, i.e., all inflow to the plant is assumed to receive primary treatment.

- flows in excess of primary treatment capacity are overflowed upstream in the interceptor system, as discussed earlier.

The WPCC pollutographs are combined and processed to produce an intermittent or non-point source file for US EPA'S WASP water quality model (WPCCWWF.NPS). As stated in the previous section, flows in excess of the WPCC's treatment capacity are assumed to be shed from the appropriate emergency outfall.

To simulate the effect of disinfection at the plant, the primary and secondary effluent were assumed to receive treatment which could produce a three log reduction in fecal coliform concentrations. Therefore, the EMC for primary treatment and disinfection, i.e., bypass of secondary treatment, was assumed to be 2400 fecal coliform/100 mL (wet weather conditions) while effluent with secondary treatment and disinfection was assumed to have an EMC of 200 fecal coliform/100 mL (dry weather conditions). The new EMCs can then be applied to the appropriate hydrograph to produce a pollutograph for WPCCs under this assumed plant disinfection scenario.

5.3 COMPARISON TO FAST ALARM DATA

For some time, the City of Winnipeg has been collecting and storing alarm information from the collection system using a FAST alarm system. The information collected is from a series of alarms such as pumps overheating, flooding, break-ins, as well as overflow alarms. Data collected by the City for 1990 and 1991 pump station alarms contained about 15,000 records each year for all types of alarms. These data were imported into a Paradox database and screened for overflow alarms. Each record stated the time of day of the alarm (to the minute) and the duration of the alarms. Discussion with the City Operations Division indicated that the alarms are generally set to respond just prior to anticipated overflow. The alarms are not located at the overflow weir but at a location and elevation in the pump station which attempts to correspond to an overflow at the weir. Due to the complexity of some of the interception/overflow connections some alarms may not be as accurate as others. This FAST alarm data was used, not as a calibration tool, but as a cross-check that the runoff and control model representation is realistic when compared to the available system data. The comparison was done on two sets of data:

1. the combined sewer system alarm data was compared for each district over one season to the predictions of the existing system control model; and

2. a summary of the sanitary sewer alarms from all districts was compared to the predictions of the sanitary sewer system model.

Each of these data sets are discussed below.

5.3.1 Combined Sewer System

The FAST alarm data were compiled and processed in order to estimate the number of days of overflow and average duration of overflow as indicated by the alarm data for each district monitored in 1991. The 1991 rainfall data was used in the XP-SWMM model to estimate runoff for each combined sewer district. These hydrographs were input into the control system model, along with DWF, to produce seasonal output hydrographs for each district. This output was then processed to produce data comparable to the summary of the FAST alarm data, namely, the predicted number of days with overflows and the average duration of overflow. Table 5-2 shows the model predictions for each district and compares them to the FAST alarm summary for the 1991 recreation season (May to September).

TABLE 5-2
Comparison of Modelled Overflows
and FAST Alarm Data for each CS District
(1991 data)

		Model Predictions		FAST Alarm Data				
District	District Name	Modelled Number of Days with Overflows	Average Duration Hours	Station Name	Number of Days with Alarm	Average Alarm Duration Hours	Ratio[1] (Alarm/Model)	Calibration District
1	Alexander	23	5.6	ALEXANDER	10	2.7	**0.43**	✓
2	Armstrong	6	3.2	ARMSTRONG	5	1.2	0.83	
3	Ash	35	7.6	ASH	28	7.9	0.80	
4	Assiniboine	16	2.4					
5	Assiniboine Park	50	8.5					
6	Aubrey	29	7.2	AUBREY	29	6.6	1.00	
7	Baltimore	24	8.0	BALTIMORE	29	3.7	1.21	✓
8	Bannatyne	15	5.3	BANNATYNE	15	2.6	1.00	
9	Boyle	21	4.0	BOYLE	26	3.7	1.24	✓
10	Calrossie	12	4.3					
11	Clifton	23	5.7	CLIFTON	29	6.7	1.26	✓
12	Cockburn	37	7.7	COCKBURN	76	25.2	2.05	
13	Colony	18	4.8	COLONY	22	4.4	1.22	✓
14	Cornish	22	6.3	CORNISH	25	4.1	1.14	
15	Despins	31	6.4	DESPINS	20	7.7	**0.65**	
16	Doncaster	32	8.2					
17	Douglas Park	9	2.6					
18	Dumoulin	14	4.1	DUMOULIN	19	3.9	1.36	
19	Ferry Road	29	7.3	FERRY	22	2.1	0.76	
20	Hart	24	6.5	HART	30	7.5	1.25	
21	Hawthorne	35	7.7	HAWTHORNE	27	5.7	0.77	
22	Jefferson E	44	9.6	JEFFERSON_MAIN	14	4.3	**0.32**	
24	Jessie	29	6.6	JESSIE	27	7.4	0.93	
25	La Verendrye	26	6.7	THIBAULT	3	3.8	**0.12**	
26	Linden	29	6.9	LINDEN	19	2.4	**0.66**	
27	Mager Drive	26	6.6	MAGER_DRIVE	24	20.9	0.92	✓
28	Marion	19	3.6	MARION	24	5.1	1.26	✓
29	Metcalfe	19	4.7	METCALFE	14	2.8	**0.74**	
30	Mission	19	5.5	MONTCALM	1	0.9	**0.05**	
31	Moorgate	28	7.0	CONWAY	26	5.2	0.93	
32	Munroe	20	4.7	MUNROE	15	9.1	0.75	
33	Newton	11	3.5	NEWTON	1	11.0	**0.09**	
34	Polson	16	5.3					
35	River	31	6.7	RIVER	10	3.9	**0.32**	
36	Riverbend	29	7.2	RIVERBEND	9	3.8	**0.31**	
37	Roland	18	3.8	MONTCALM	1	0.9	**0.05**	
38	Selkirk	12	4.8	SELKIRK	10	3.8	0.83	
39	St. Johns	29	7.2	ST_JOHN'S	5	4.7	**0.17**	
40	Strathmillan	9	2.9					✓
41	Syndicate	22	6.4	SYNDICATE	24	3.7	1.09	✓
42	Tuxedo	22	5.6	CHATTAWAY	11	4.7	**0.50**	✓
43	Tylehurst	24	6.2	TYLEHURST	37	3.6	1.54	
44	Woodhaven	23	5.6					

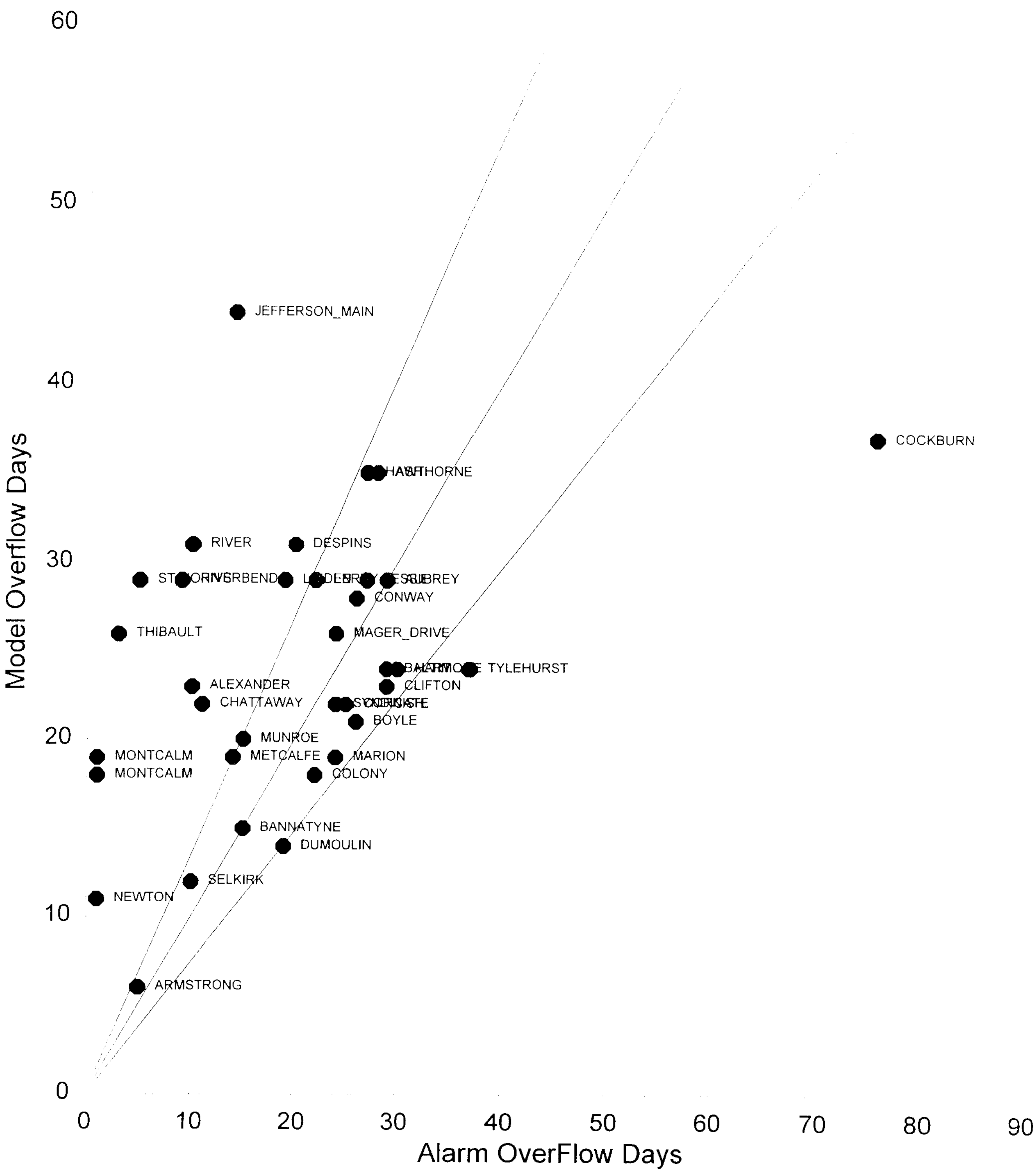
1. Ratio = Overflow Alarm Days/Overflow Model Days (Bold Indicates outside a 35% + or - Range)

On seven districts there were no data available at the corresponding pump station or diversion:

- Assiniboine;
- Assiniboine Park (not actually a CS district);
- Calrossie;
- Doncaster;
- Douglas Park;
- Polson;
- Strathmillan; and
- Woodhaven.

Roland and Marion districts share the same station as Montcalm (only this station has alarm data).

For comparison purposes, the two sets of data were grouped in pairs and number of days of predicted overflow for the model was plotted against the alarm data (see Figure 5-9). The perfect-fit line is plotted on the graph to indicate which districts have a good correlation between modelled and measured data. An example of a district which shows a perfect fit is Bannatyne, in which model and alarm data both indicate 15 days with overflow. A ±35% range was plotted to help place the plotted data into perspective. The results show a reasonable correlation between modelled overflows and monitored FAST alarm data for most stations. Nineteen of the 35 stations with paired data produced very good modelled results, including 8 of the 10 districts in which the runoff for that district was specifically calibrated to available flow data. In general, for the calibrated districts, the model predicts about 20% less overflow days than indicated from the FAST alarm data. This understatement could be expected since the alarms are set to respond when overflow is imminent, not when it is occurring.



Alarm Days vs Model Overflow Days
(1991 data)

Figure 5-9

MODVSALM.WK4

Three districts had greater than 35% more days with alarms than the model indicated. These districts were:

- Cockburn - 105% more days with alarms;
- Tylehurst - 54% more days with alarms; and
- Dumoulin - 36% more days with alarms.

The results for Cockburn and Tylehurst indicated that dry weather overflows might be occurring at these stations. The City has indicated that these have been recognized and investigations have occurred into events in these districts. Excessive dry weather flow from unknown sources is occurring in the summer. The City is investigating various options to control this problem. Dumoulin is only slightly higher than the arbitrary "margin of error" and is likely not a significant problem.

Thirteen of the districts appear to predict significantly more overflows than are indicated by the alarm data. These districts and the corresponding alarm days and model days of overflow are:

DISTRICT(S)/PUMPSTATION	PREDICTED OVERFLOW DAYS	MEASURED ALARM DAYS
Mission-Roland/Montcalm	18-19	1
St. Johns	29	5
Newton	11	1
LaVerendrye/Thibault	26	3
Riverbend	29	9
River	31	10
Jefferson	44	14
Alexander	23	10
Tuxedo/Chattaway	22	11
Despins	31	20
Linden	29	19
Metcalfe	19	14

These districts listed should be investigated further to determine the phenomena causing the discrepancy in the results. The differences could be due to two reasons:

1. the alarms are not located in a position which records an overflow and are therefore underpredicting overflows (this is likely the case for Mission/Roland, for example).

2. the model is under-estimating the interception rate for the district or overpredicting runoff and dry weather flow.

It is important to determine the real reason for these discrepancies, especially in some key districts such as Jefferson. If in reality there are 44 overflows per year at this district, Jefferson would be the first priority district to improve interception. However, if only 14 overflows occur (as indicated by the alarm), this would make it one of the lower priority districts. Another station which would be important to investigate is St. Johns. St. Johns is the most likely location for overflow relief of the Main Street interceptor's excess flows. It is important to determine whether or not the alarm system records these overflows accurately.

The follow-up on these questions will enhance the effectiveness of both the model and the FAST alarm system.

5.3.2 Sanitary Sewer Districts

The sanitary sewer system was not modelled district by district in the same manner as the combined sewer system. Each LDS district was assumed to have a counterpart conceptual sanitary district. Therefore, the pumping station alarm for actual districts did not correspond directly to the modelled districts. The comparison between model prediction and alarm data was done on an overall basis to determine whether frequency and duration generally correspond to each other.

The modelled results are compared to the alarm data in Table 5-3. There were 27 sanitary pumping stations which had overflow information reported to the FAST alarm database in 1991. The data were screened to summarize the results when more than two stations were reporting overflows of longer than 6 minutes. This was done to screen out very localized rainfall or some cause other than excess extraneous flow causing the overflow (i.e., pump failure, excessive DWF). Table 5-3 shows the number of stations reporting overflows for each day, the percent of stations (of the total of 27) indicating overflows and the average duration of the alarm.

Fifteen days in the 1991 recreation season (May 1 to September 30) reported sanitary sewer overflow (SSO) alarms compared to six days in which SSOs were predicted by the model during the same period. An initial observation could be that results are not compatible, however, closer examination indicated that seven of the overflows not predicted by the model were of a duration of less than one hour on average. Since the modelling timestep is one hour this is not considered significant. The two SSOs of longer duration (5.6 and 4.4 hours) that was not predicted were circumstances where only 3 (11%) of the stations reported alarms. This may have been isolated intense rainstorms. The two large storms which caused overflows (June 13, 1991 and June 25-27, 1991) were both predicted by the sanitary sewer model.

There were some differences in duration and extent of overflows between the model predictions and the alarms which should be noted. The model tends to predict overflows to be of greater areal extent throughout the City (70-85% of districts) than indicated by the alarms (38-45% of districts), and the model predicts shorter duration of overflows (1-2 hours) than indicated by the alarm data (1-10 hours). There are some likely explanations for these discrepancies:

- all districts are assumed to have 5% of runoff as extraneous flow. In reality, many districts likely have less extraneous flow since new areas have greatly improved lot grades, sump pumps, etc. These districts would not likely have overflows. Some older separate districts may have up to 10% of runoff as extraneous flow which would cause longer duration of overflows.

Table 5-3
COMPARISON OF SSO FAST ALARM DATA AND MODEL PREDICTIONS

	MODEL PREDICTIONS				FAST ALARM Information		
Date	Number of Modeled LDS with Overflows	% of Modeled LD Districts With Overflow	Average Duration	Sum of Hour	Sanitary Pumping Stations Indicating Overflow	Number of % of Sanitary Pumping Stations Indicating Overflow	Average Duration
08-May-91					6	22%	0.3
09-May-91					4	15%	0.6
29-May-91					3	11%	0.2
31-May-91	29.0	54%	0.5	14.0	3	11%	3.1
07-Jun-91	1.5	3%	1.3	2.0	3	11%	1.0
13-Jun-91	38.0	70%	0.2	8.0	12	44%	3.9
25-Jun-91	45.4	84%	1.9	85.0	13	48%	10.7
26-Jun-91	1.0	2%	2.0	2.0	7	26%	1.3
27-Jun-91					7	26%	0.2
01-Jul-91					3	11%	5.6
23-Jul-91					7	26%	0.2
17-Sep-91					3	11%	4.4
22-Sep-91	42.3	78%	0.3	12.0	5	19%	6.4
26-Sep-91					5	19%	0.7
27-Sep-91					5	19%	0.5
	6 Days				15 days		

27 Sanitary Pumping Stations Reporting Alarms
54 LDS districts Modelled

- all the districts were assumed to have 7 x DWF interception rates. In reality, some have less interception capacity, which would cause longer durations of overflows.

In summary, the SSO model will give a reasonable representation of the frequency of SSOs for a first level perspective on their significance in river loadings. The extent and duration of overflow could be more accurately modelled by using a range of extraneous flow (2 to 10%) percentages and interception rate (3 to 8 x DWF) depending upon the specific characteristics of the area being modelled. This is not considered necessary at this stage to determine relative impacts of SSOs on the rivers. A more refined sanitary sewer system model may assist in determining priority districts to target for reducing extraneous flows, for collection of SSOs and discharging them into the interceptor system or for reducing WWF to the WPCCs.

Comparison of SSOs to Rainfall

To put into perspective how much rainfall results in a sanitary sewer overflow, the duration of overflow for each day (as given by the alarm data) was plotted against the millimetres of rainfall on that day as shown in Figure 5-10. In Figure 5-10a, the labels for the plot points indicate the date of the overflow, while Figure 5-10b labels indicate the number of stations in which overflows were recorded. As a general observation, these figures indicate that SSOs can be expected when there is more than 13 mm of rain in one day. In addition, overflow can occur with less intense rainfall, however, this is likely due to adverse antecedent conditions or heavy localized rainfall. This provides an important perspective in that a CSO control alternative which would provide for control of about 13 mm of rain before overflow would essentially be providing wet weather capture performance equivalent to the existing separate system in many areas.

5.4 REPRESENTATIVE YEAR

It was recognized that the entire integrated model of rainfall, 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

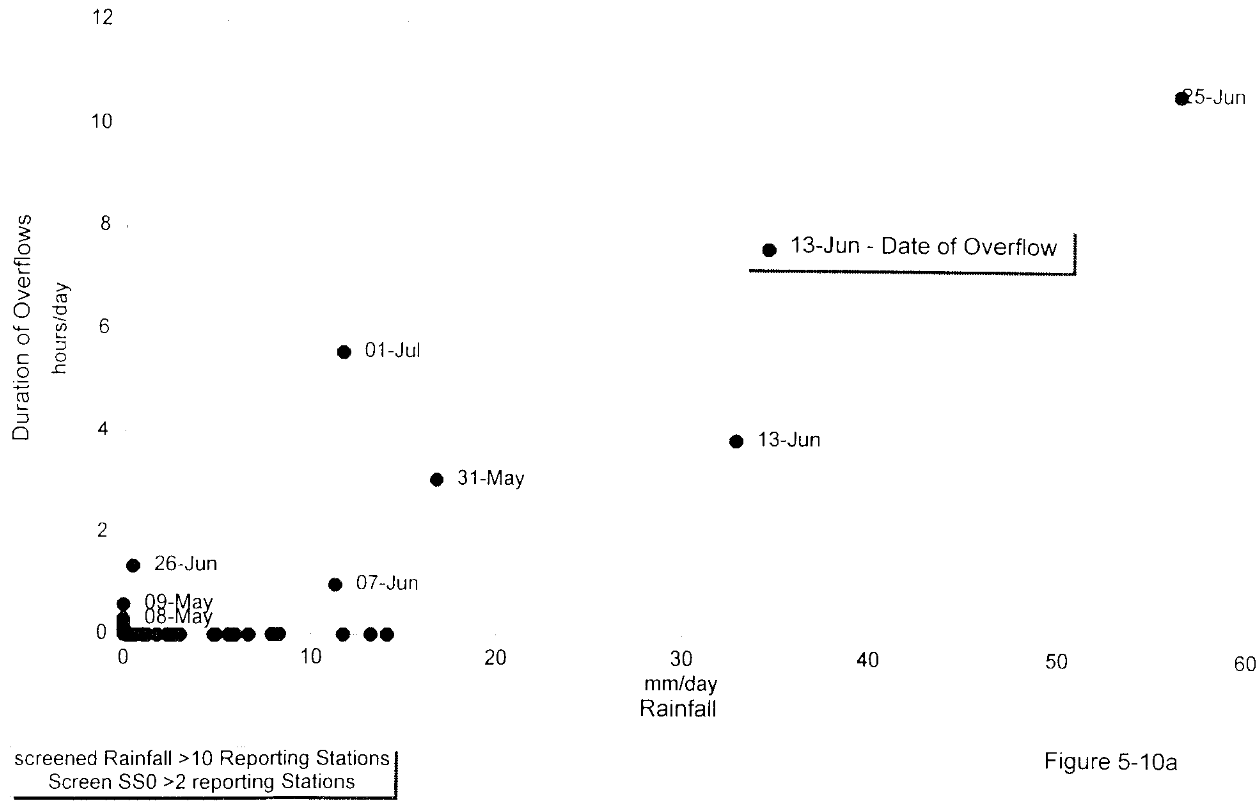


Figure 5-10a

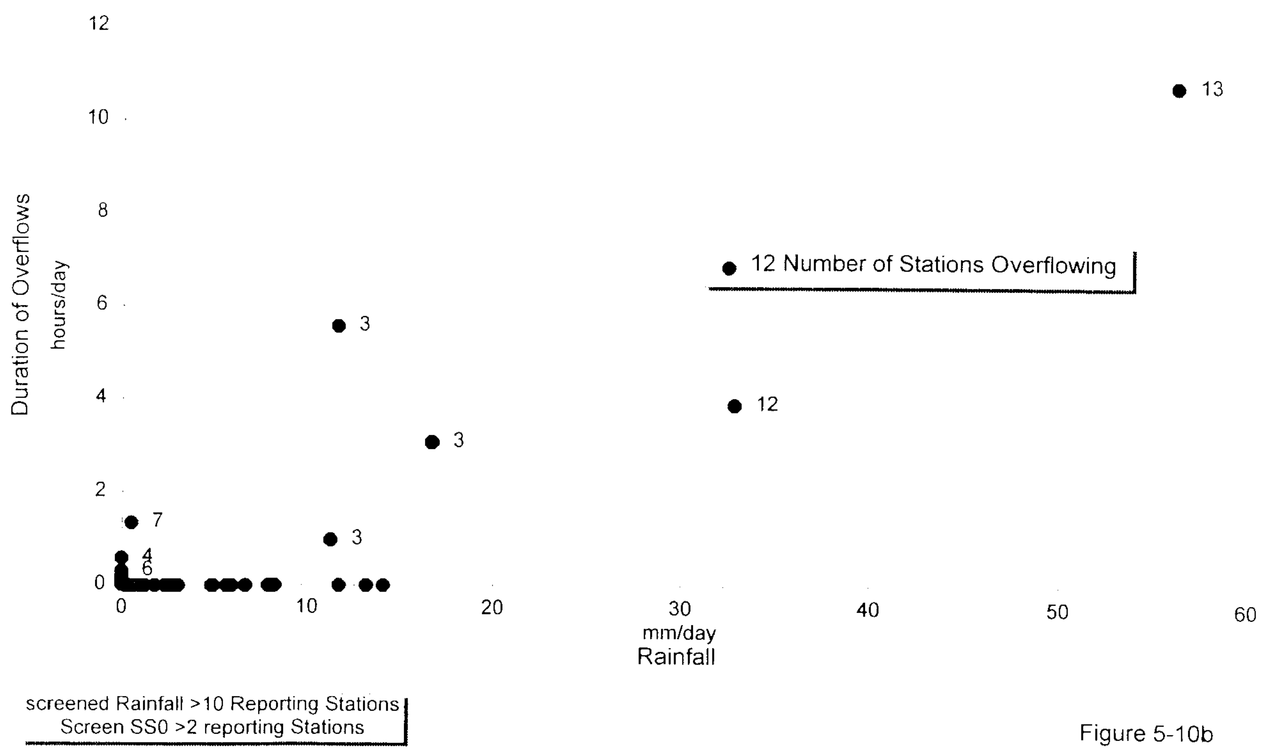


Figure 5-10b

SSO Duration vs. Daily Rainfall

Figure 5-10

OFVSRNSS.WK4

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

To establish a representative year, both rainfall and river flows for a given year were jointly assessed. This involved the review of river flows from 1962 to 1992, in conjunction with the rainfall records specific to the recreation season (May to September, inclusive) for each year.

5.4.1 Representative 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 measuring 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 sizes 1 and 2, to a 25 mm range for 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. 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 5-4. 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.

TABLE 5-4
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	**51**	**279**	18	6	**10**	**9**	**6**	**1**	**1**	**0**	**0**	**0**
Average	**47.5**	**316.8**	12.9	7.9	**10.2**	**7.5**	**5.4**	**1.4**	**1.0**	**0.5**	**0.5**	**0.1**

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

RAINNUM.WK4

The ten most representative years in terms of rainfall were selected, as shown in **bold** and shaded on Table 5-4. These years were then assessed in terms of river flow. The joint assessment is described later in this section.

5.4.2 Representative River Flow

River flows at Headingley, St. Agathe and Lockport for the period of record from 1962 to 1992 were ranked relative to long-term river flow averages, as shown in Table 5-5, using median flow for the recreation season. The rank of the average river flow during May through September is given as a percent exceedence in Table 5-5, i.e., the number of years (in %) that exceed the flow for the given year. For example, a ranking of 90% would indicate a very low flow since 90% of the years would exceed the flow for that given year, while a 10% ranking refers to a relatively high flow. A value close to a median value of 50% indicates a more representative year in terms of average river flow.

5.4.3 Joint Assessment

The ten representative years selected by the rainfall assessment were then reviewed in terms of river flows. The joint assessment is as follows:

- 1966 - had too high river flow - discarded;
- 1992 - had average river flow - good;
- 1978 - had average river flow - good;
- 1963 - average to low river flow, little water quality data available - discarded;
- 1989 - Headingley flow was extremely low - discarded;
- 1973 - very low river flow - discarded;
- 1960 - river flow not used - little quality data - discarded;
- 1982 - average river flow - good;
- 1990 - low river flow - discarded; and
- 1962 - very high river flow - discarded.

TABLE 5-5
RANKING OF SEASONAL RIVER FLOWS

Year	River Flow			Ranking % years Excedeed		
	Average of Headingley m³/s	Average of St.Agathe m³/s	Average of Lockport m³/s	Rank by Headingley	Rank By St.Agathe	Rank by Lockport
1962	32	412	457	63%	9%	16%
1963	32	116	161	66%	59%	66%
1964	32	139	183	69%	50%	56%
1965	62	266	381	34%	25%	25%
1966	63	288	386	31%	22%	22%
1967	52	217	287	41%	34%	34%
1968	20	172	207	81%	38%	50%
1969	127	291	446	19%	19%	19%
1970	129	371	537	13%	13%	13%
1971	75	93	173	28%	66%	59%
1972	85	157	251	22%	41%	41%
1973	17	41	60	88%	84%	88%
1974	187	362	626	3%	16%	6%
1975	165	444	620	9%	6%	9%
1976	184	66	262	6%	78%	38%
1977	18	21	45	84%	97%	97%
1978	27	153	185	72%	44%	53%
1979	129	489	680	16%	3%	3%
1980	16	38	58	91%	91%	91%
1981	14	57	78	94%	81%	84%
1982	47	140	215	44%	47%	47%
1983	75	131	223	25%	53%	44%
1984	25	130	167	75%	56%	63%
1985	45	249	302	47%	31%	31%
1986	61	260	346	38%	28%	28%
1987	33	92	129	56%	69%	72%
1988	34	22	57	53%	94%	94%
1989	10	88	111	97%	75%	78%
1990	35	39	80	50%	88%	81%
1991	24	88	121	78%	72%	75%
1992	32	104	147	59%	63%	69%

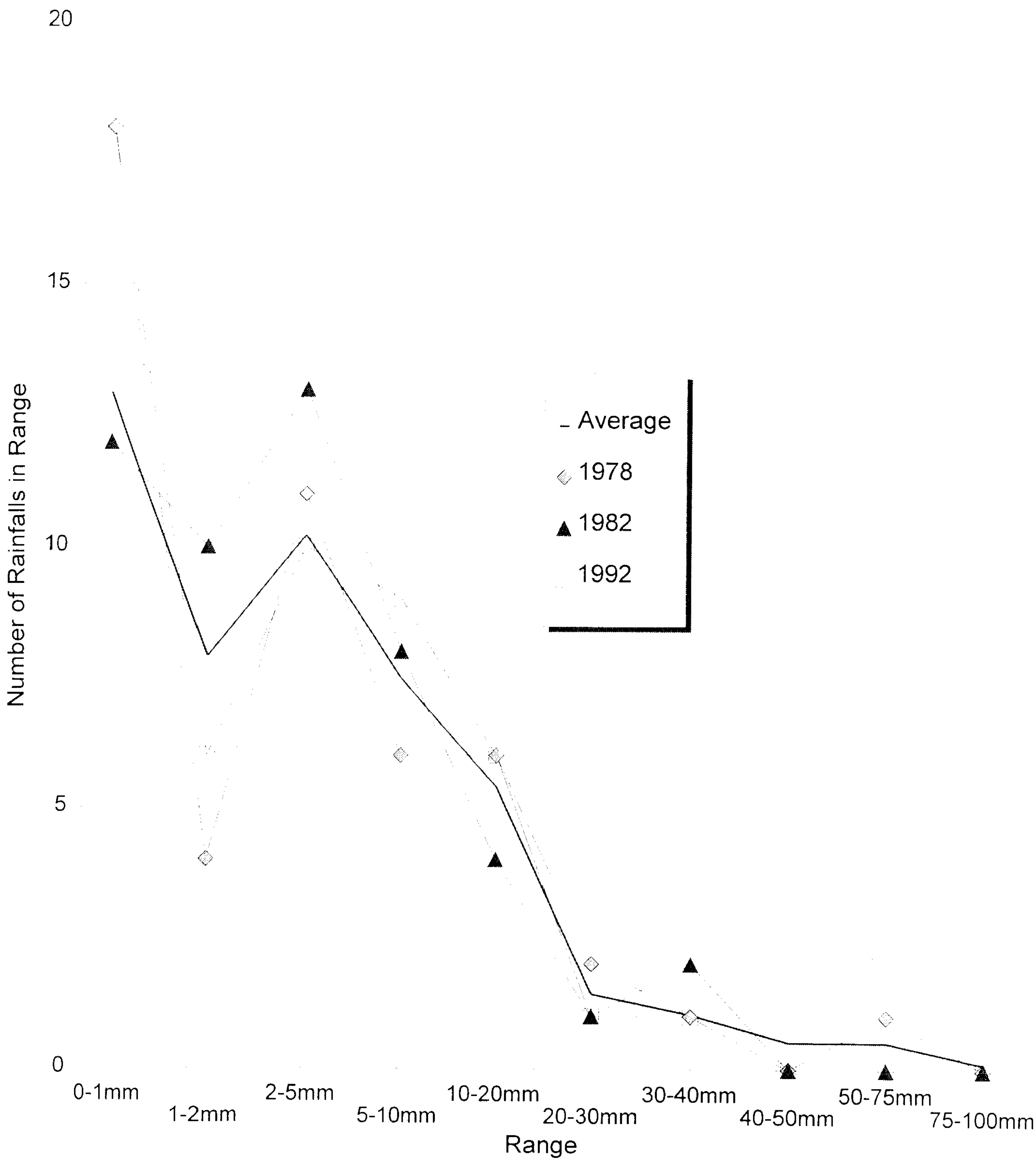
Eg. for 1962- Rank of Headingley: the Actual flow of 32 m³/s is exceeded in 63% of years

Table 5-4 indicates that the years 1978, 1982, and 1992 were representative of the long-term median river flow conditions. The rainfall distribution for the three years, selected on the basis of the joint assessment, is shown on Figure 5-11. On closer inspection, the year 1978 was found to have an excellent representative rainfall and only a fair representation of river flows because the Assiniboine River flows tended to be on the low side. The year 1982 was found to have a good representative rainfall and excellent river flows when compared to the long-term average. The most recent year of 1992 had excellent rainfall and river flows that were slightly lower than the long-term average. 1992 was selected for receiving stream modelling, because of its currency and overall representative character in terms of rainfall and river flows. This year, 1992, was therefore used as the representative year for Phase 2 screening purposes.

5.5 RESULTS

The regional model discussed in Section 5.2 can accept seasonal runoff (from the XP-SWMM runoff model) for all CSO and LDS districts to produce outflow hydrographs for combined sewer districts, the sanitary sewer system and the interceptor system, as well as hydrographs for the Land Drainage System (LDS). The inflow hydrograph for each WPCC can be developed by adding DWF and intercepted WWF from their respective service areas. Appropriate event mean concentrations (EMCs) for fecal coliforms developed in other technical memoranda and summarized in Table 5-6 can be applied to these hydrographs to produce mass loadings of fecal coliforms (pollutographs) at specific times and locations. The pollutographs can be used to develop loadings to the river water quality model as discussed in TM #4 - Receiving Stream. This allows various control alternatives to be assessed, such as:

- the existing system;
- optimization of the existing system;
    - 5 x DWF interception of CSO
    - inline storage plus 5 x DWF interception of CSO;



Rainfall Years with Best Fits

Figure 5-11

RAINNUM.WK4

Table 5-6

Summary of Event Mean Concentrations of Fecal Coliforms

Source	Organisms/100mL
WPCCs	
- ADWF	200,000
- PDWF	200,000
- PWWF	2,400,000
Land Drainage	
- Direct	40,000
- Ponds	20,000
CSO	2,400,000
SSO	10,000,000
Interceptors	
- CSO	2,400,000
- SSO	10,000,000

- structurally intensive options
  - separation of the combined sewer;
  - high rate treatment plus CSO disinfection;
  - storage of CSOs

The assessment of the impact of these various options on river water quality is described in TM #4 - Receiving Stream. The comparison of the seasonal mass loadings (volume of CSO, fecal coliform) from the various discharges to the stream is discussed in the next subsection, Wet Weather Perspective, of TM #4. This section discusses the results of the assessment of various alternatives in terms of:

- the number of overflows;
- the volume of overflows;
- the volume of runoff intercepted; and
- the percent runoff intercepted.

5.5.1 Wet Weather Perspective

To put the volume of CSOs, being considered for control, into perspective, a comparison was made with other sources of discharge from the City. The other sources considered were:

- the three treatment plants;
  - NEWPCC
  - SEWPCC
  - WEWPCC
- Land Drainage;
  - with and without storm retention basins (SRBs)
- Sanitary Sewer Overflows;
- Overflows from the interceptor systems;
  - CSO - Main Interceptor
  - SSO - from South end and West end Interceptors.

The volumes were compared over three time periods:

- Recreational Season - the time during which boating, waterskiing, etc. occurs, May to September;
- Open Water Season - the ice free period (April to October inclusive); and
- Year Round.

The wet weather volumes for the recreational season were calculated using results from the model for 1991 data. This year was used earlier in the project, prior to selection of 1992 as a representative year, to test the model inputs and outputs. The relative volumes of each source is generally very similar for different years. The volumes for the Open Water Season were pro-rated based on the rainfall which occurred in April and October. The results are shown in Table 5-7 and Figure 5-12. These results indicate that volumes for WPCC and LDS discharges tend to dominate recreational, open water, and especially annual total volumes, although CSOs are significant volumes. The SSOs and interceptor overflow volumes are insignificant in comparison. By applying the EMC for fecal coliforms compiled in Table 5-6, the perspective changes. Relative loadings of fecal coliforms are shown in Figure 5-13. In this Figure, it is obvious that CSOs along with plant discharges tend to dominate loadings to the rivers. LDS loadings are relatively small although SSO fecal coliform loadings are significant at 4-8% of the total. It must be noted that fecal coliforms are not conservative and tend to die-off relatively quickly, therefore the location and time of the overflow influences the relative impact of each discharge. The true impact of fecal loading is assessed in TM #4 - Receiving Stream by a dynamic water quality model (US EPAs WASP).

5.5.2 Existing Combined Sewer Control System

The existing control system for the combined sewer system consists of weirs, and usually pump stations, which divert the DWF from the combined sewer district to an interceptor system. The system was designed to intercept a nominal 2.75 x DWF, however, the interception rate varies considerably from district to district. A summary of dry weather flow and interception rates in the districts modelled is shown in Table 5-8. As can be seen, the

Table 5 -7
Volumes of Discharge to Rivers
(1991)

a) as m^3

	Recreation Season m^3	Open Water m^3	Full Year m^3
NEWPCC	38,260,000	53,160,000	87,890,000
SEWPCC	8,560,000	11,940,000	20,090,000
WEWPCC	4,880,000	6,790,000	11,320,000
LDS	16,630,000	21,340,000	21,340,000
LDS w/SRB	6,440,000	8,270,000	8,270,000
CSO	7,960,000	10,220,000	10,220,000
SSO	118,000	153,000	153,000
Interceptor CSO	20,000	26,000	26,000
Interceptor SSO	107,000	139,000	139,000
Total	83,000,000	112,000,000	159,000,000

b) as percentage

	Recreation Season (May to Sept, inclusive)	Open Water (April to Oct, inclusive)	Full Year
NEWPCC	46.1%	47.5%	55.3%
SEWPCC	10.3%	10.7%	12.6%
WEWPCC	5.9%	6.1%	7.1%
LDS	20.0%	19.1%	13.4%
LDS w/SRB	7.8%	7.4%	5.2%
CSO	9.6%	9.1%	6.4%
SSO	0.1%	0.1%	0.1%
Interceptor CSO	0.0%	0.0%	0.0%
Interceptor SSO	0.1%	0.1%	0.1%

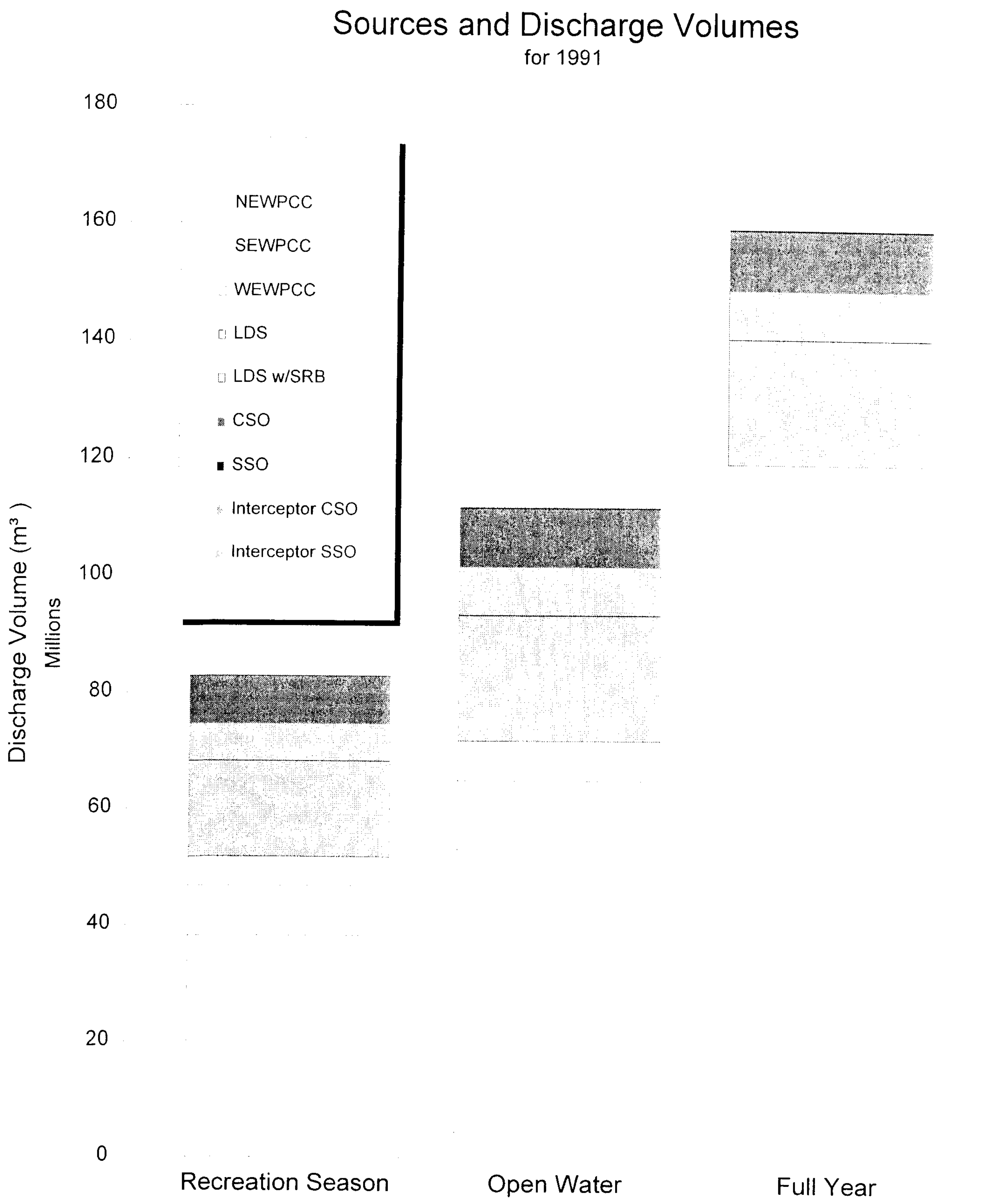


Figure 5-12

PERSPEC.WK4

Relative Fecal Coliform Loading by Source
for 1991

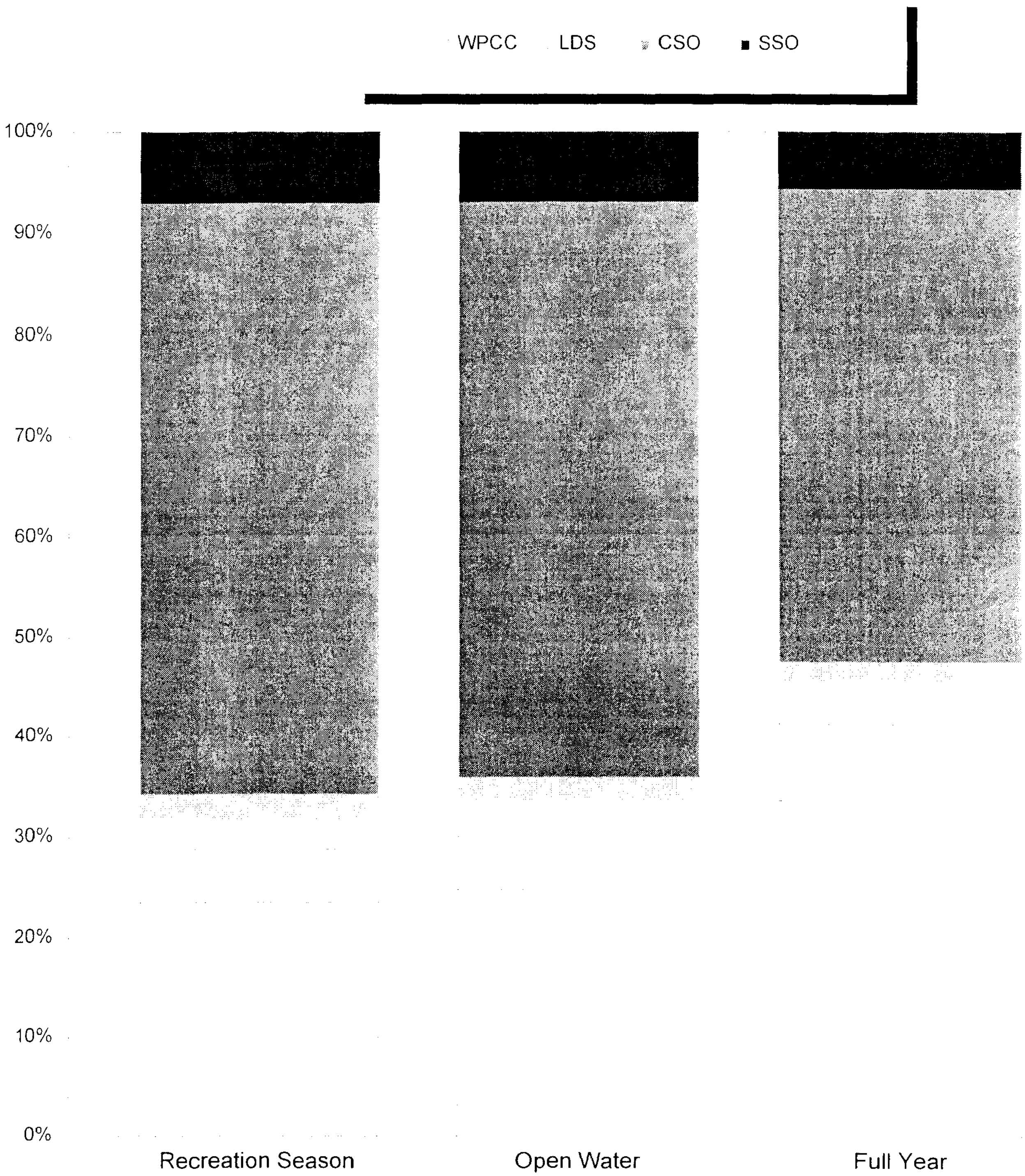


Figure 5-13

PERSPEC.WK4

TABLE 5-8

Summary:Existing Control

(Representative Year)

District Number	District Name	DWF m³/s	Interception m³/s	X DWF	OverFlows	Comments
1	Alexander	0.035	0.155	4.4	21	
2	Armstrong	0.02	0.524	26.2	7	
3	Ash	0.082	0.301	3.7	22	
4	Assiniboine	0.084	0.425	5.1	16	
5	Assiniboine Park	0.0001	0.0003	3.0	37	Assumed 3xDWF (not CSO)
6	Aubrey	0.071	0.214	3.0	17	
7	Baltimore	0.028	0.201	7.2	17	to Mager Drive
8	Bannatyne	0.153	0.613	4.0	9	
9	Boyle	0.014	0.03	2.1	22	
10	Calrossie	0.001	0.028	28.0	12	
11	Clifton	0.077	0.236	3.1	19	
12	Cockburn	0.033	0.075	2.3	22	to Baltimore
13	Colony	0.134	0.425	3.2	16	
14	Cornish	0.035	0.107	3.1	15	
15	Despins	0.032	0.132	4.1	16	
16	Doncaster	0.025	0.075	3.0	18	Assumed 3xDWF to Ash
17	Douglas Park	0.001	0.095	95.0	9	
18	Dumoulin	0.013	0.136	10.5	13	
19	Ferry Road	0.059	0.126	2.1	22	
20	Hart	0.039	0.101	2.6	23	
21	Hawthorne	0.036	0.113	3.1	21	
22	Jefferson E	0.143	0.569	4.0	16	
23	Jefferson W					Send ALL to Jeff East
24	Jessie	0.066	0.176	2.7	22	
25	La Verendrye	0.009	0.015	1.7	30	
26	Linden	0.017	0.06	3.5	22	
27	Mager Drive	0.091	0.309	3.4	22	PS (flows from Balt Metcalf)
28	Marion	0.032	0.22	6.9	17	
29	Metcalfe	0.005	0.044	8.8	16	to Mager
30	Mission	0.144	0.518	3.6	13	
31	Moorgate	0.023	0.085	3.7	17	
32	Munroe	0.077	0.237	3.1	21	
33	Newton	0.01	0.166	16.6	10	
34	Polson	0.032	0.356	11.1	16	
35	River	0.07	0.094	1.3	28	
36	Riverbend	0.053	0.107	2.0	24	
37	Roland	0.026	0.324	12.5	16	
38	Selkirk	0.067	0.453	6.8	9	
39	St. Johns	0.084	0.173	2.1	24	
40	Strathmillan	0.003	0.062	20.7	9	
41	Syndicate	0.01	0.069	6.9	19	
42	Tuxedo	0.004	0.036	9.0	19	to Doncastor
43	Tylehurst	0.05	0.176	3.5	20	
44	Woodhaven	0.00227	0.027	11.9	20	Pumped
	Minimum	**0.0001**	**0.0003**	**1.343**	**7**	
	Maximum	**0.153**	**0.613**	**95**	**37**	
	Sum	**1.99037**	**8.3883**			
	Average			**8.5**	**18.2**	
	Weighted Average			**4.2**		

interception rate varies from a low of 1.3 x DWF to up to 95 x DWF. The weighted average interception rate is about 4.2 x DWF.

This system was evaluated for 1992 (the representative year, as discussed earlier, see Section 5.4). The results of the predicted number of overflows during the recreation season (May 1 to September 30) are shown in Table 5-8. The number of overflows varies from 7 per season at Armstrong to 37 per season at Assiniboine Park. Assiniboine Park was mistakenly identified as a combined district. In reality, the vast majority of drainage is overland to the Assiniboine River. Of the true combined districts, the largest number of overflows is 30 per season at LaVerendrye. The average number of CSOs per season is 18.

The storm runoff and the overflow volume per district for current conditions are shown in Table 5-9. The percent of runoff captured for each district was also estimated. These values range form only 9% capture at Mager Drive to 94% capture at Armstrong. The apparent excess interception capacity at Mager Drive is used to accept the discharge from the tributary Baltimore and Metcalfe pump stations which then only leaves limited capacity available to intercept runoff.

In general, the system could be considered relatively effective in capturing runoff considering it is designed as a DWF interception system. The average interception rate is about 4 x DWF and 40% of runoff is captured and conveyed to treatment.

5.5.3 Optimize Interceptor Capacity (5 x DWF)

The interception rates from district to district vary considerably as discussed in the previous section. The main interceptor system was assessed (see TM #2 - Infrastructure) to determine its maximum carrying capacity. It was determined that 5 x DWF could potentially be delivered to the NEWPCC. This interception rate was then assumed to apply to all districts, i.e., it is assumed that the appropriate regulators and pumps can be upgraded to provide this interception rate. The control system model was then applied to all districts to determine number of overflows and volume of overflows for each district under this scenario. The results are shown on Table 5-10 and Table 5-11.

TABLE 5-9
Runoff and Overflow Volumes: Existing Conditions

District	District Name	Hectares	Sum of Runoff m³	Sum of Overflow m³	Runoff mm	Overflow mm	Per Cent Captured
1	Alexander	160	173,989	111,449	108.8	69.7	36%
2	Armstrong	146	105,657	6,589	72.5	4.5	94%
3	Ash	735	650,000	504,193	88.5	68.6	22%
4	Assiniboine	88	127,803	47,740	144.9	54.1	63%
5	Assiniboine Park	142	113,520	113,240	79.7	79.5	0%
6	Aubrey	442	236,047	126,976	53.5	28.8	46%
7	Baltimore	247	141,103	73,702	57.2	29.9	48%
8	Bannatyne	263	170,162	31,179	64.7	11.9	82%
9	Boyle	27	31,579	24,343	118.2	91.1	23%
10	Calrossie	10	11,774	5,050	121.2	52.0	57%
11	Clifton	494	291,735	204,213	59.0	41.3	30%
12	Cockburn	347	160,097	126,228	46.1	36.4	21%
13	Colony	230	200,574	71,569	87.3	31.1	64%
14	Cornish	143	63,705	22,314	44.5	15.6	65%
15	Despins	118	91,982	47,133	78.1	40.0	49%
16	Doncaster	155	55,463	36,121	35.7	23.2	35%
17	Douglas Park	25	23,968	3,144	95.5	12.5	87%
18	Dumoulin	83	61,708	19,241	74.0	23.1	69%
19	Ferry Road	292	195,975	141,420	67.2	48.5	28%
20	Hart	227	154,498	113,882	68.2	50.3	26%
21	Hawthorne	260	179,626	120,271	69.0	46.2	33%
22	Jefferson E	1003	459,248	184,100	45.8	18.4	60%
23	Jefferson W						
24	Jessie	399	317,024	230,236	79.4	57.6	27%
25	La Verendrye	72	58,443	51,949	81.6	72.5	11%
26	Linden	159	128,530	93,944	80.6	58.9	27%
27	Mager Drive	781	172,171	156,249	22.0	20.0	9%
28	Marion	231	158,602	93,856	68.6	40.6	41%
29	Metcalfe	35	35,494	19,170	100.8	54.4	46%
30	Mission	753	261,911	77,522	34.8	10.3	70%
31	Moorgate	158	91,006	46,352	57.7	29.4	49%
32	Munroe	590	352,170	247,058	59.7	41.9	30%
33	Newton	82	56,364	13,895	68.9	17.0	75%
34	Polson	262	221,409	78,927	84.4	30.1	64%
35	River	126	105,849	86,741	84.1	68.9	18%
36	Riverbend	207	179,765	135,683	86.9	65.6	25%
37	Roland	208	213,494	96,250	102.4	46.2	55%
38	Selkirk	326	167,955	31,249	51.6	9.6	81%
39	St. Johns	355	335,744	261,080	94.7	73.6	22%
40	Strathmillan	85	31,786	6,419	37.4	7.6	80%
41	Syndicate	76	75,499	44,039	99.2	57.9	42%
42	Tuxedo	53	47,491	30,016	90.3	57.1	37%
43	Tylehurst	216	202,931	128,989	93.9	59.7	36%
44	Woodhaven	66	40,498	26,817	61.8	40.9	34%
		10,900	**6,954,000**	**4,091,000**	**63.8**	**37.5**	**41%**

TABLE 5-10

Summary: Optimized Interceptor (5 x DWF)

District Number	District Name	DWF m³/s	Interception m³/s	X DWF	OverFlows	Comments
1	Alexander	0.035	0.175	5	20	
2	Armstrong	0.02	0.1	5	17	
3	Ash	0.082	0.41	5	19	
4	Assiniboine	0.084	0.42	5	16	
5	Assiniboine Park	0.0001	0.0005	5	36	Assumed 3xDWF
6	Aubrey	0.071	0.355	5	13	
7	Baltimore	0.028	0.14	5	22	to Mager Drive
8	Bannatyne	0.153	0.765	5	9	
9	Boyle	0.014	0.07	5	15	
10	Calrossie	0.001	0.005	5	24	
11	Clifton	0.077	0.385	5	16	
12	Cockburn	0.033	0.165	5	17	to Baltimore
13	Colony	0.134	0.67	5	9	
14	Cornish	0.035	0.175	5	9	
15	Despins	0.032	0.16	5	16	
16	Doncaster	0.025	0.125	5	11	Assumed 3xDWF to Ash
17	Douglas Park	0.001	0.005	5	28	
18	Dumoulin	0.013	0.065	5	19	
19	Ferry Road	0.059	0.295	5	16	
20	Hart	0.039	0.195	5	18	
21	Hawthorne	0.036	0.18	5	17	
22	Jefferson E	0.143	0.715	5	14	
23	Jefferson W					Send ALL to Jeff East
24	Jessie	0.066	0.33	5	17	
25	La Verendrye	0.009	0.045	5	19	
26	Linden	0.017	0.085	5	21	
27	Mager Drive	0.091	0.455	5	16	PS (flows from Balt Metcalf)
28	Marion	0.032	0.16	5	21	
29	Metcalfe	0.005	0.025	5	21	to Mager
30	Mission	0.144	0.72	5	9	
31	Moorgate	0.023	0.115	5	16	
32	Munroe	0.077	0.385	5	18	
33	Newton	0.01	0.05	5	19	
34	Polson	0.032	0.16	5	19	
35	River	0.07	0.35	5	9	
36	Riverbend	0.053	0.265	5	16	
37	Roland	0.026	0.13	5	22	
38	Selkirk	0.067	0.335	5	14	
39	St. Johns	0.084	0.42	5	17	
40	Strathmillan	0.003	0.015	5	19	
41	Syndicate	0.01	0.05	5	21	
42	Tuxedo	0.004	0.02	5	23	to Doncastor
43	Tylehurst	0.05	0.25	5	17	
44	Woodhaven	0.00227	0.01135	5	28	Pumped
	Minimum	**0.0001**	**0.0005**	**5**	**9**	
	Maximum	**0.153**	**0.765**	**5**	**36**	
	Sum	**1.99037**	**9.95185**			
	Average			**5.0**	**17.7**	
	Weighted Average			**5.0**		

TABLE 5-11

Runoff and Overflow Volumes: Optimized Interceptor (5 x DWF)

District	District Name	Hectares	Sum of Runoff m^3	Sum of Overflow m^3	Runoff mm	Overflow mm	Per Cent Captured
1	Alexander	160	173,989	104,842	108.8	65.6	40%
2	Armstrong	146	105,657	53,719	72.5	36.9	49%
3	Ash	735	650,000	455,683	88.5	62.0	30%
4	Assiniboine	88	127,803	48,294	144.9	54.7	62%
5	Assiniboine Par	142	113,520	112,965	79.7	79.3	0%
6	Aubrey	442	236,047	79,935	53.5	18.1	66%
7	Baltimore	247	141,103	148,993	57.2	60.4	-6%
8	Bannatyne	263	170,162	20,909	64.7	7.9	88%
9	Boyle	27	31,579	15,203	118.2	56.9	52%
10	Calrossie	10	11,774	9,817	121.2	101.1	17%
11	Clifton	494	291,735	154,184	59.0	31.2	47%
12	Cockburn	347	160,097	77,502	46.1	22.3	52%
13	Colony	230	200,574	40,150	87.3	17.5	80%
14	Cornish	143	63,705	10,778	44.5	7.5	83%
15	Despins	118	91,982	40,546	78.1	34.4	56%
16	Doncaster	155	55,463	14,455	35.7	9.3	74%
17	Douglas Park	25	23,968	20,268	95.5	80.8	15%
18	Dumoulin	83	61,708	34,738	74.0	41.7	44%
19	Ferry Road	292	195,975	79,434	67.2	27.2	59%
20	Hart	227	154,498	80,645	68.2	35.6	48%
21	Hawthorne	260	179,626	92,330	69.0	35.5	49%
22	Jefferson E	1003	459,248	143,695	45.8	14.3	69%
23	Jefferson W						
24	Jessie	399	317,024	166,633	79.4	41.7	47%
25	La Verendrye	72	58,443	34,514	81.6	48.2	41%
26	Linden	159	128,530	81,288	80.6	51.0	37%
27	Mager Drive	781	172,171	63,608	22.0	8.1	63%
28	Marion	231	158,602	107,477	68.6	46.5	32%
29	Metcalfe	35	35,494	25,022	100.8	71.1	30%
30	Mission	753	261,911	46,210	34.8	6.1	82%
31	Moorgate	158	91,006	35,879	57.7	22.7	61%
32	Munroe	590	352,170	190,247	59.7	32.2	46%
33	Newton	82	56,364	35,420	68.9	43.3	37%
34	Polson	262	221,409	136,371	84.4	52.0	38%
35	River	126	105,849	24,595	84.1	19.5	77%
36	Riverbend	207	179,765	77,437	86.9	37.4	57%
37	Roland	208	213,494	152,925	102.4	73.4	28%
38	Selkirk	326	167,955	46,089	51.6	14.1	73%
39	St. Johns	355	335,744	163,406	94.7	46.1	51%
40	Strathmillan	85	31,786	20,535	37.4	24.2	35%
41	Syndicate	76	75,499	51,220	99.2	67.3	32%
42	Tuxedo	53	47,491	36,747	90.3	69.8	23%
43	Tylehurst	216	202,931	104,823	93.9	48.5	48%
44	Woodhaven	66	40,498	33,935	61.8	51.8	16%
		10,900	**6,954,000**	**3,473,000**	**63.8**	**31.9**	**50%**

The number of overflows per year changed very little from current conditions, a range of 9 to 28 compared to 7 to 30 (not considering Assiniboine Park). The average number of overflows based on 5 x DWF remains at about 18 per recreational season. The percent of runoff captured varies from 15% to 88%, which is slightly less variation than the existing conditions. This large variation indicates that using an interception rate based on DWF in order to design a WWF capture program may not be appropriate. The nominal interception rate of 2.75 x DWF was developed more to account for diurnal variation and uncertainties of production of DWF. The selection of an interception rate based on WWF is discussed later in Section 8.0 - Phase 3 Considerations. The amount of runoff captured increases to about 50% from about 40% with the existing system.

There are some anomalies to be noted; the Baltimore district shows more overflow than runoff for the season. This is because of the impact of upstream districts such as Cockburn, whose intercepted discharge is diverted into the Baltimore system. The Baltimore system was designed for 5 x DWF to the Baltimore District only and it should be designed for 5 x DWF for all upstream districts (i.e., Cockburn and Calrossie). Such anomalies will be addressed in Phase 3.

5.5.4 Optimize System Storage (5 x DWF With Inline Storage)

Each combined sewer district has the potential to store combined sewage in the larger diameter pipes (inline storage) and therefore prevent some smaller storms from overflowing. This would increase the percentage of runoff captured. In order to assess this potential, the volume of inline storage for each district, for which information was available, was compiled (see Section 4) and a region-wide factor for inline storage was developed (for those areas for which information was not available) at 1.2 mm/ha. Table 5-12 shows the estimated storage available for each district in terms of cubic metres and equivalent millimetres of storage per district. The results of the system model evaluation are shown for each district in Table 5-12 and Table 5-13. The Assiniboine Park result should be ignored since in reality the runoff does not enter the system. The range of overflows per season is from 0 at Selkirk to 16 at Polson. Inline storage has a dramatic effect in the Selkirk district since it apparently has 4.8 mm equivalent storage versus only 0.4 mm in the Polson system.

TABLE 5-12

Summary: Inline Storage(with 5 x DWF Interception)

District Number	District Name	DWF m³/s	Interception m³/s	X DWF	Storage m³	Storage mm	OverFlows
1	Alexander	0.035	0.175	5	9,000	5.63	3
2	Armstrong	0.02	0.1	5	1,748	1.20	7
3	Ash	0.082	0.41	5	8,814	1.20	12
4	Assiniboine	0.084	0.42	5	1,059	1.20	10
5	Assiniboine Park	0.0001	0.0005	5	1,709	1.20	19
6	Aubrey	0.071	0.355	5	5,298	1.20	5
7	Baltimore	0.028	0.14	5	1,800	0.73	15
8	Bannatyne	0.153	0.765	5	3,157	1.20	2
9	Boyle	0.014	0.07	5	321	1.20	10
10	Calrossie	0.001	0.005	5	117	1.20	13
11	Clifton	0.077	0.385	5	6,000	1.21	9
12	Cockburn	0.033	0.165	5	4,167	1.20	7
13	Colony	0.134	0.67	5	2,758	1.20	7
14	Cornish	0.035	0.175	5	1,719	1.20	2
15	Despins	0.032	0.16	5	1,413	1.20	7
16	Doncaster	0.025	0.125	5	1,865	1.20	3
17	Douglas Park	0.001	0.005	5	301	1.20	15
18	Dumoulin	0.013	0.065	5	1,000	1.20	8
19	Ferry Road	0.059	0.295	5	3,501	1.20	7
20	Hart	0.039	0.195	5	2,720	1.20	8
21	Hawthorne	0.036	0.18	5	1,000	0.38	12
22	Jefferson E	0.143	0.715	5	12,024	1.20	3
23	Jefferson W						
24	Jessie	0.066	0.33	5	4,793	1.20	9
25	La Verendrye	0.009	0.045	5	860	1.20	9
26	Linden	0.017	0.085	5	600	0.38	16
27	Mager Drive	0.091	0.455	5	7,000	0.90	3
28	Marion	0.032	0.16	5	2,773	1.20	9
29	Metcalfe	0.005	0.025	5	423	1.20	10
30	Mission	0.144	0.72	5	9,033	1.20	2
31	Moorgate	0.023	0.115	5	1,894	1.20	7
32	Munroe	0.077	0.385	5	7,080	1.20	7
33	Newton	0.01	0.05	5	981	1.20	8
34	Polson	0.032	0.16	5	1,048	0.40	16
35	River	0.07	0.35	5	1,510	1.20	7
36	Riverbend	0.053	0.265	5	2,482	1.20	7
37	Roland	0.026	0.13	5	2,501	1.20	13
38	Selkirk	0.067	0.335	5	15,000	4.60	0
39	St. Johns	0.084	0.42	5	1,416	0.40	15
40	Strathmillan	0.003	0.015	5	1,020	1.20	6
41	Syndicate	0.01	0.05	5	913	1.20	11
42	Tuxedo	0.004	0.02	5	631	1.20	11
43	Tylehurst	0.05	0.25	5	2,500	1.16	9
44	Woodhaven	0.00227	0.01135	5	787	1.20	11
	Minimum	**0.0001**	**0.0005**	**5**	**136,735**	**1.25**	**0**
	Maximum	**0.153**	**0.765**	**5**			**19**
	Sum	**1.99037**	**9.95185**				
	Average			**5.0**			**8.6**
	Weighted Average			**5.0**			

TABLE 5-13

Inline Storage Result
Runoff and Overflow Volumes

District	District Name	Hectares	Sum of Runoff m^3	Sum of Overflow m^3	Runoff mm	Overflow mm	Per Cent Captured
1	Alexander	160	173,989	26,904	108.8	16.8	85%
2	Armstrong	146	105,657	33,439	72.5	23.0	68%
3	Ash	735	650,000	333,435	88.5	45.4	49%
4	Assiniboine	88	127,803	32,174	144.9	36.5	75%
5	Assiniboine Park	142	113,520	107,047	79.7	75.1	6%
6	Aubrey	442	236,047	38,131	53.5	8.6	84%
7	Baltimore	247	141,103	153,256	57.2	62.1	-9%
8	Bannatyne	263	170,162	4,667	64.7	1.8	97%
9	Boyle	27	31,579	10,885	118.2	40.8	66%
10	Calrossie	10	11,774	7,923	121.2	81.6	33%
11	Clifton	494	291,735	83,619	59.0	16.9	71%
12	Cockburn	347	160,097	36,766	46.1	10.6	77%
13	Colony	230	200,574	17,413	87.3	7.6	91%
14	Cornish	143	63,705	2,114	44.5	1.5	97%
15	Despins	118	91,982	25,654	78.1	21.8	72%
16	Doncaster	155	55,463	4,413	35.7	2.8	92%
17	Douglas Park	25	23,968	15,543	95.5	61.9	35%
18	Dumoulin	83	61,708	23,158	74.0	27.8	62%
19	Ferry Road	292	195,975	44,914	67.2	15.4	77%
20	Hart	227	154,498	48,208	68.2	21.3	69%
21	Hawthorne	260	179,626	76,080	69.0	29.2	58%
22	Jefferson E	1003	146,845	57,239	14.6	5.7	61%
23	Jefferson W		312,404	0			
24	Jessie	399	317,024	106,427	79.4	26.6	66%
25	La Verendrye	72	58,443	23,163	81.6	32.3	60%
26	Linden	159	128,530	70,595	80.6	44.3	45%
27	Mager Drive	781	172,171	17,453	22.0	2.2	90%
28	Marion	231	158,602	70,481	68.6	30.5	56%
29	Metcalfe	35	35,494	18,972	100.8	53.9	47%
30	Mission	753	261,911	6,276	34.8	0.8	98%
31	Moorgate	158	91,006	18,356	57.7	11.6	80%
32	Munroe	590	352,170	114,045	59.7	19.3	68%
33	Newton	82	56,364	23,482	68.9	28.7	58%
34	Polson	262	221,409	118,685	84.4	45.3	46%
35	River	126	105,849	11,534	84.1	9.2	89%
36	Riverbend	207	179,765	49,359	86.9	23.9	73%
37	Roland	208	213,494	111,114	102.4	53.3	48%
38	Selkirk	326	167,955	0	51.6	0.0	100%
39	St. Johns	355	335,744	139,550	94.7	39.4	58%
40	Strathmillan	85	31,786	10,435	37.4	12.3	67%
41	Syndicate	76	75,499	37,826	99.2	49.7	50%
42	Tuxedo	53	47,491	27,560	90.3	52.4	42%
43	Tylehurst	216	202,931	71,212	93.9	33.0	65%
44	Woodhaven	66	40,498	23,294	61.8	35.5	42%
		10,900	**6,954,000**	**2,253,000**	**63.8**	**20.7**	**68%**

PH2REP.WK4

The average number of overflows has reduced considerably to about 8.6 overflows per season from about 18 without inline storage.

The percent of runoff captured shows similar improvement with inline storage. The overall capture of the total runoff in the combined sewer district increases to about 68%.

5.5.5 Structurally Intensive Controls

In Phase 2 it was decided to assess the costs and impacts of options that would provide maximum combined sewer overflow control. These structurally intensive controls present the best possible conditions which could be achieved by controls. By assessing these structurally intensive controls, along with the existing system optimization, both ends of the spectrum of control alternatives can be considered. Three alternatives were considered as potential long-term solutions:

- separation of the combined sewer districts into land drainage and sanitary sewer systems;
- disinfection of all combined sewer overflows at each district prior to discharge to the receiving stream (this would require high rate treatment with either RTBs or VSSs); and
- storage of combined sewage and withdrawal to the WPCC for primary treatment and disinfection after the rain event (this could be with tunnel storage or distributed offline storage).

The cost of each of these alternatives was estimated in Section 4. The control system model was not used to perform this "first cut" assessment of the benefits, but rather implicit knowledge of the system was used to develop pollutographs for use by the receiving stream model (through modification of the stream loadings to reflect the potential controls). The results of this approach are discussed below.

Separation of Combined Sewer Districts

Separation would essentially convert the CSO volumes into LDS loadings, i.e., the EMC would be changed. The overflow hydrographs for the existing combined sewer system were thus considered to be LDS hydrographs. The EMC for an LDS system (40,000 FC/100 mL) was applied to create pollutographs for a separate system. These updated pollutographs were then used in the receiving stream model to estimate compliance.

With complete separation, the number and volume of CSOs would be eliminated.

Disinfection of All CSOs

In this scenario, all CSOs would be given treatment, probably at satellite "end-of-pipe" facilities, sufficient to allow effective disinfection. This would involve RTB's or VSS and disinfection. The hydrograph from the system model results, for inline storage with 5 x DWF interception, were used for volume of CSOs and a new EMC of 200 FC/100 mL was applied to create pollutographs for this scenario. The system model was used to estimate the peak hourly rate of overflow for the representative year (1992). As with the other alternatives these pollutographs were used in the receiving stream model to assess impact on compliance.

Under these conditions, there would still be substantial CSOs but these overflows would be treated.

Storage of CSOs

The storage option considered the provision of regional storage either as a regional storage tunnel or as distributed storage tanks, such that all CSOs are captured. Storage of all CSOs assumes no overflow will be discharged to the receiving stream. In Phase 2, the system model for total storage was not used to estimate storage volume required for the entire City. The volume (1,000,000 m^3) was estimated by extrapolating the results form earlier work (Linden/Hawthorne Study, Wardrop/Tetr*ES* 1994). A spreadsheet model was used assuming the entire City was a single district of 10,000 hectares. The withdrawal rate, i.e., dewatering after the rainfall, was assumed to be equal to the NEWPCC primary treatment capacity. The rainfall data from 1989 was used since it was available in the spreadsheet model, therefore the estimate for the tunnel storage is relatively coarse.

In Phase 3 the representative year (1992) will be used in conjunction with the Control System Model to assess the storage requirement at each district.

With 1,000,000 m^3 of storage, there would be no CSOs.

Floatables Control

Floatables control, such as screens or net bags, was not modelled since this option would address the aesthetics issue only. The number and volume of overflows would not change but the visible character of the CSOs would be improved. The operating costs of a floatable control option would be impacted by the optimization option such as inline storage with increased interceptions. The frequency of overflow will be reduced from 18 to 8, thereby decreasing the frequency of use of net bags or screens. This option is assessed in TM #6 - Implementation Plan in this context.

6.0 COMPARISON OF CSO CONTROLS

A summary of the results of the assessment of various controls and the costs of these alternatives is shown on Table 6-1. The number of runoff events per year and the volume of runoff per year is also shown in the Table.

A comparison of the results indicates that an increased interception rate does increase the percentage of runoff captured (reducing overflow from 59% to 50% of runoff) but it has little impact on number of overflows. However, the addition of inline storage, thus optimizing the use of existing infrastructure, does reduce the number of average overflows from 18 to 8 in addition to reducing the percent of runoff overflowing to 36% from 55%. When compared to the US EPA objectives, described in their "Presumptive Approach" - of allowing four overflows per year, or 85% of runoff captured, the inline storage alternative is remarkably effective as a first step to improved control. This assumes that the storage (and the 5 x DWF rate) can be handled effectively at the NEWPCC.

Table 6-1
CSOs UNDER DIFFERENT CONTROL SCENARIOS

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

Notes
1. Assumes 300,000 m^3 of Storage, Results are estimates Only
2. Assumes 1,000,000 m^3 of Storage, Results are estimates Only
3. Disinfection of WPCCs is $23 million (Wardrop 1992) plus 20% Engineering, Administration and Finance and 20% Estimating Contingency

The volume required for distributed and tunnel storage of 300,000 m^3 was estimated using the synthetic storm analysis in Section 4. This was modelled in Phase 2, however, we estimate this should effect 85% runoff capture and result in 4 overflows per year. This option will be studied district by district early in Phase 3. The Retention Treatment Basins were sized on the same basis as distributed storage, however the overflow will be disinfected. Accordingly, we would expect no untreated overflows during a representative year.

The structurally intense alternatives studied are considerably more expensive ($290 - $1,000 million) to obtain nearly complete removal (or treatment) of CSOs. The assessment of number and volume of CSOs indicates that there is great potential in storage alternatives at a cost in the area of $300 million. The effectiveness of inline storage indicates the potential for selective offline storage to reduce number and volume of overflows to an acceptable level. In one district, Selkirk, the inline storage available (4.6 mm) could capture all the storms in the representative year. This indicates great potential in oversized, in-system relief pipes to be used for storage. This information is used, together with receiving water quality assessments, to provide an overall assessment in TM #6 - Implementation Plan.

7.0 MONITORING

It is recognized that some of the technology for CSO control is evolving and will need to be monitored in terms of experience in other applications. Screening devices and their effectiveness for floatables capture and pre-treatment to facilitate disinfection with UV are two examples.

If floatables control was to become the main issue, the Fresh Creek "TrashTrap" might be a candidate technology. So far, installations are limited to the New York/New Jersey area. A visit could be combined with a visit to high rate devices in the area.

Virtually all of the high-rate treatment devices and disinfection technologies are evolving. There are several pilot tests or demonstration programs underway. For example, in Columbus, Georgia, Vortex Solid Separators are being tested in combination with medium-intensity UV technology to attempt to demonstrate that this combination can provide cost-effective disinfection of CSOs. This particular project will be operational in 1996 and the results will be monitored. Vortex Solid Separators and UV technology are also being tested in the pilot project in Scarborough, Ontario. Again, this experience is being monitored. Operational experience with storage/treatment facilities will be reviewed as well. As noted previously, many examples of this technology are located in Michigan.

It is recognized that operating attention and operating costs are a very important factor in the overall selection of the appropriate technology for Winnipeg. Any visits would pay particular attention to this aspect of the technology.

8.0 PHASE 3 CONSIDERATIONS

8.1 DEFINITION OF CANDIDATE OPTIONS

A number of aspects of the control alternatives will need to be refined in order to proceed with the Phase 3 analysis. Some points which will be reviewed will be:

Inline Storage: the best means of effecting inline storage in the Winnipeg system will have to be established. Four Manning dippers have been installed in the Clifton district in order to assess the time-to-rise for "typical" rainfalls. This information will be used in the design of the inline storage pilot study proposed for Phase 3 (Section 8.5).

5 x DWF: the main interceptor appears able to carry 5 x current DWF to the NEWPCC. As discussed in TM #2, the ability of the plant to treat these flows must be assessed. Further study is also required of the practicability of the emptying of all inline and offline storage between storms. This will be reviewed.

End-of-pipe facility: at some stage during Phase 3, it will be necessary to assess the availability of sufficient land to accommodate the various end-of-pipe facilities. The shape of the properties which are available could dictate the facility's configuration or indeed could dictate the nature of the facility (e.g., distributed storage vs. local tunnels).

The practicability of the concept of only one end-of-pipe location per district will have to be assessed. This would involve a check as to whether there was sufficient infrastructure (i.e., pipe capacity) in place to transfer large enough flows between the CS trunk and the relief sewer, or would construction of such infrastructure be required.

R.T.B.: the sizing (and hence sorting) of the RTB concept requires refining. There are two aspects of capacity: storage and flow-through. The size of the storage (surface area) dictates the rule of flow-through (based on overflow rate). The combination dictates the capacity for treatment.

V.S.S.: the design overflow rate for the VSS, required to meet the needs of disinfection, will need to be assessed. This will likely be determined through the experience of others.

UV Disinfection: progress in the area application of UV to CSO treatment will need continuous monitoring. This is closely related to the assessment of VSSs.

Storage: the effect of various rates of storage of combined sewage on river quality will have to be modelled.

Miscellaneous Costs: land costs and operating costs will be significant for some of the options. This will require investigation in Phase 3.

8.2 CONTROL SYSTEM MODELLING

Updating the Control System

The system model is a representation of the modeller's understanding of the system for a given stage of the study and to address certain objectives. In this case, the system model was used for first level screening purposes. During the course of this study, as more knowledge of the system is gained, adjustments will be made to the system model, as necessary to analyze the selected options better. Some of the proposed adjustments to be made early in Phase 3 are:

- convert Assiniboine Park district to an LDS. Although the DWF from the park building does discharge to the Main Interceptor, the runoff goes directly to the Assiniboine River;

- sub-divide Riverbend Combined district to Riverbend and Parkside Drive districts as in the 1986 flood relief study (Girling and Sharp 1986);

- partially separate Jefferson West. The St. John's/Polson/Jefferson study (ID 1980) recommends that part of Jefferson West be separated and the runoff diverted to land drainage sewers in the adjoining area to the west. The portion of the area separated is being investigated at this time;

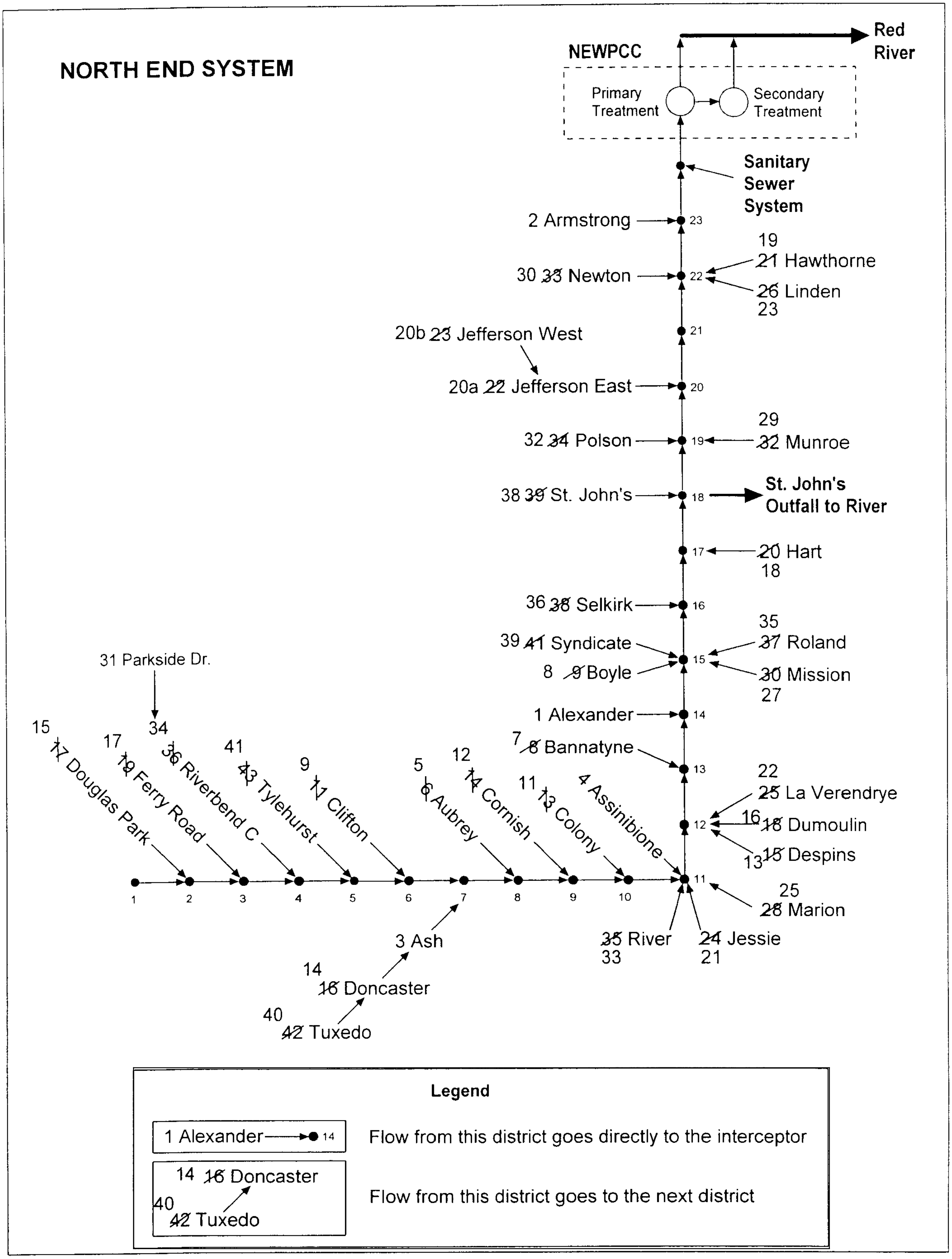
- renumbering of districts. The districts can be renumbered (see Figure 8-1 and Figure 8-2) to provide consistency to the 1986 Relief Study (Girling and Sharp 1986) report and the interceptor system model (TM #2 - Infrastructure).

In addition, the 1992 AES rainfall data appears to be missing two important storms, therefore the City airport gauge will be used to provide 1992 data for Phase 3 modelling. The runoff (TM #1 - Problem Definition) and control system model will need to be used to update the representative year results. The receiving stream model impacts will also be updated so that a new compliance perspective can be assessed. The direction provided at the end of the Phase will not likely change with the more complete data set.

Wet Weather Flow Interception Rates

As discussed earlier in Section 5.4.3 the interception rate is generally selected as a multiple of DWF. For the objective of controlling overflows, a more appropriate selection should be based on the district-specific wet weather flow or runoff.

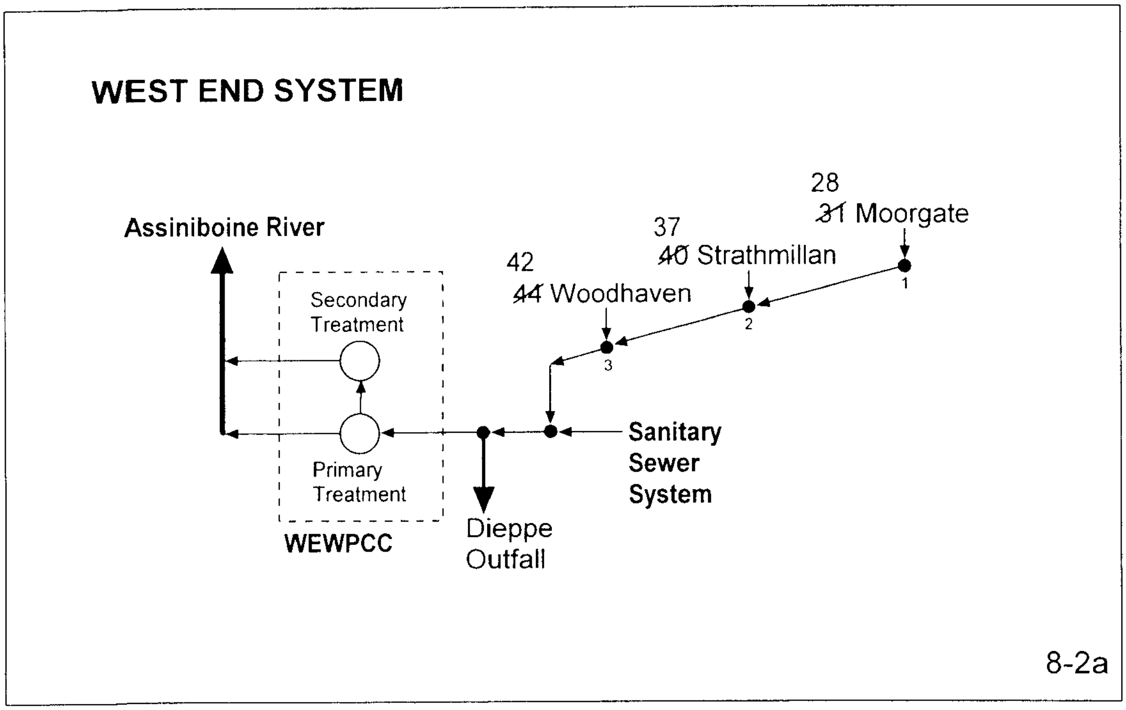
The amount of runoff occurring in a district may not be proportional to the DWF for that same district. The DWF for each district (in mm/day) was plotted against runoff (see Figure 8-3) (in mm/year). The units were chosen to cancel out the influence of the size of district. In general, there is not a strong correlation between DWF and runoff in a district. In some cases, both the DWF and runoff is high, as in Assiniboine district, however in general, the DWF rate

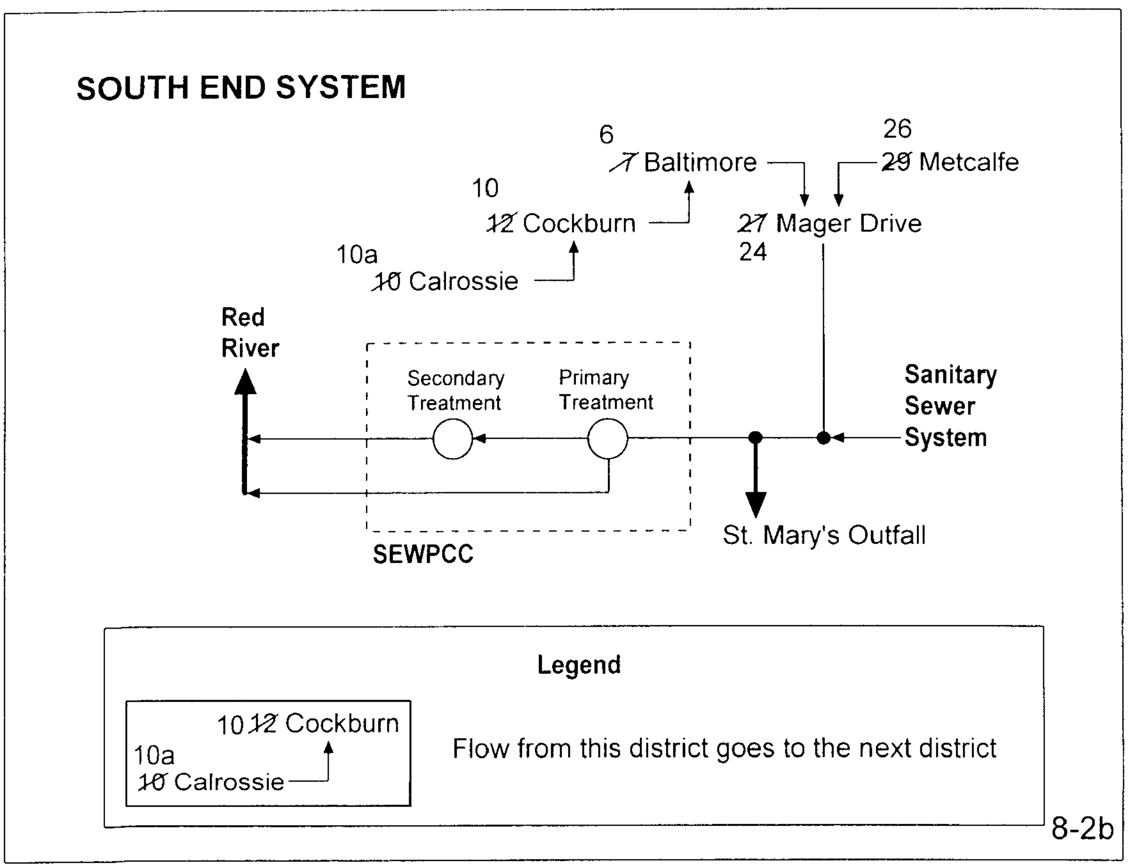


REVISED SCHEMATIC OF INTERCEPTION SYSTEM FOR NORTH END SYSTEM

Figure 8-1

intcep2
01_pwong





REVISED SCHEMATIC OF INTERCEPTION SYSTEM FOR WEST & SOUTH END SYSTEMS

Figure 8-2

intcep2
01_pwong

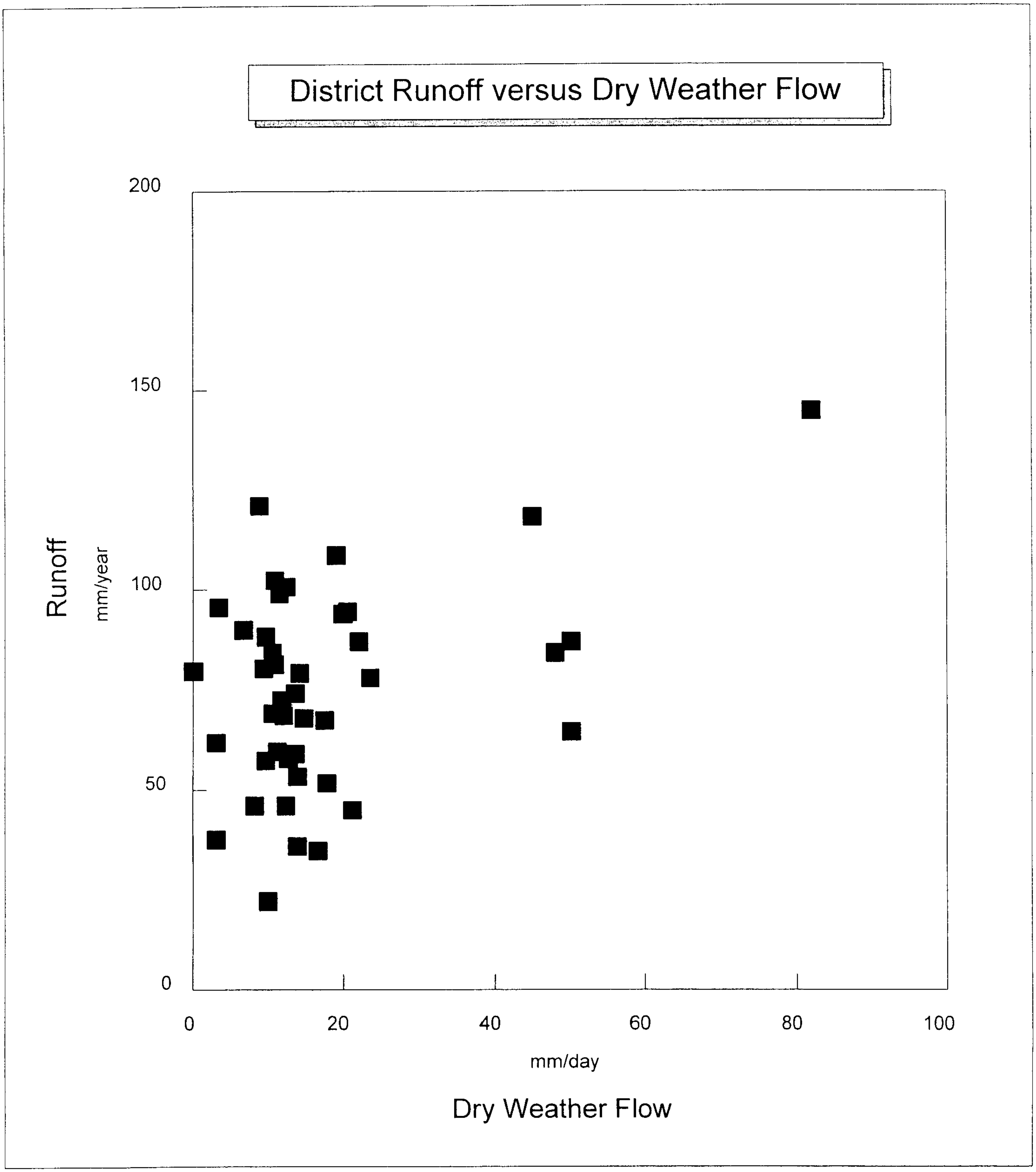


Figure 8-3

rundwf
sewer

is about 10 to 20 mm/day (a factor of 2) while the runoff varies from 20 to 120 mm/year (a factor of 6).

In principle, the interception rate should be designed as a factor of both DWF and WWF. The "Excess Interception Rate" can be defined as the rate of interception greater than a district's DWF, i.e., a measure of the capture of the runoff or combined sewage. This excess is used to capture the runoff directly, or empty inline storage (or offline storage). The excess interception rate should be some function of the runoff for that district. In order to test whether an interception rate for WWF flow could be developed based on runoff in each district, the excess interception rate for each district based on the use of "design" interception rates of 5 x DWF was calculated as follows:

$$\text{Excess Interception Rate} = (\text{mm/hr}) \quad \frac{(5-1) \times \text{DWF} \times 3600 \text{ seconds/hr} \times 1000 \text{ mm/m}}{\text{Area} \times 10{,}000 \text{ m}^2\text{/ha}}$$

where Excess Interception Rate - is in mm/hr

DWF - District DWF in m^3/s

Area - District Area in hectares

The average hourly runoff rate during the recreation season is

$$\text{Runoff Rate} = \frac{\text{Runoff (in mm)}}{\text{3600 hours from May 1 to September 30}}$$

Where Runoff = Runoff (in mm/hr) for district from Table 5-9

The wet weather flow interception rate was calculated as:

$$\text{Wet Weather Flow Interception Rate} = \frac{\text{Excess Interception Rate}}{\text{Runoff Rate}}$$

This value was calculated for each district and plotted against number of overflows per season in each district and percent of runoff captured. The results are shown on Figure 8-4 and Figure 8-5. Both Figures show a trend:

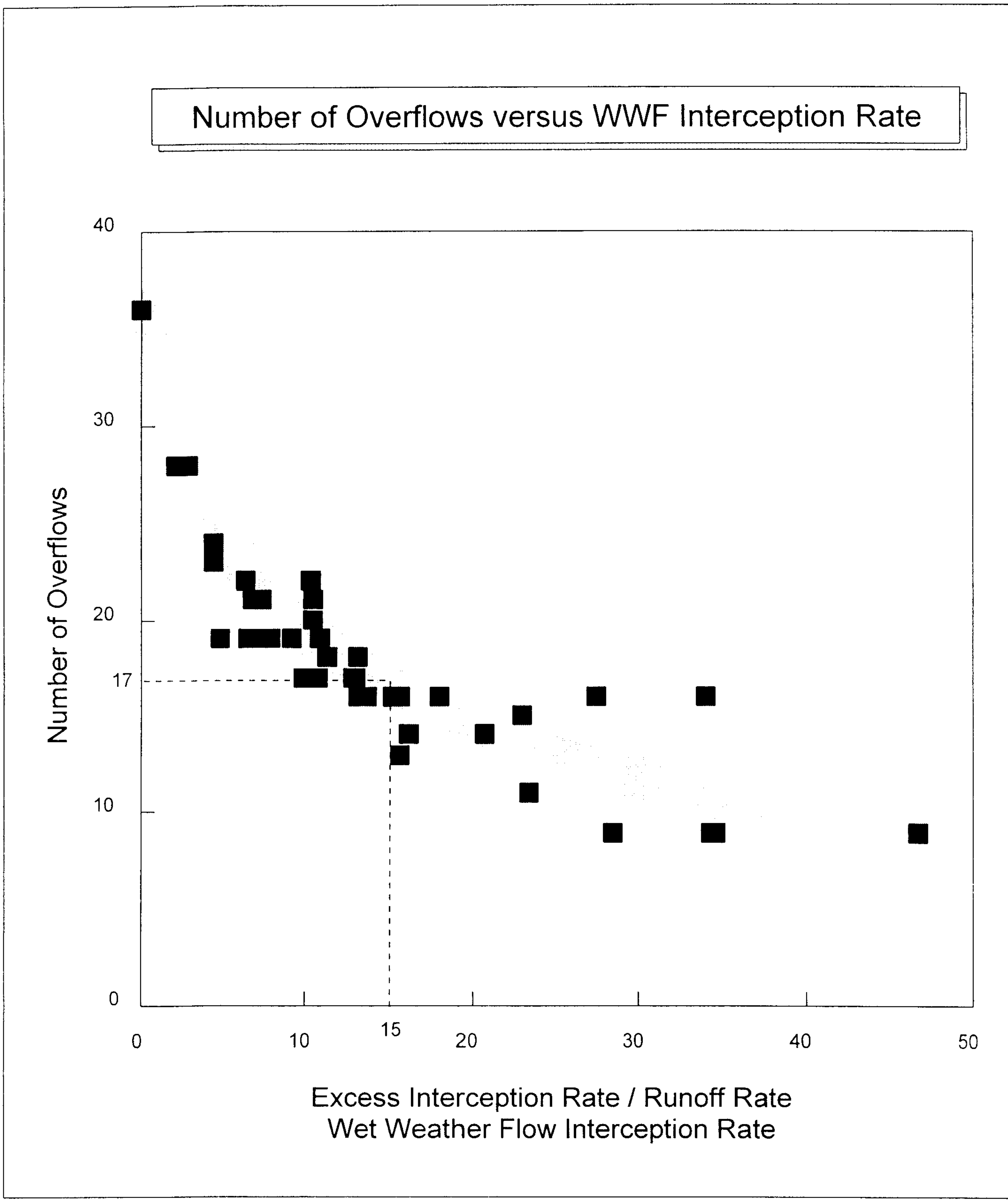


Figure 8-4

ovflo
sewer

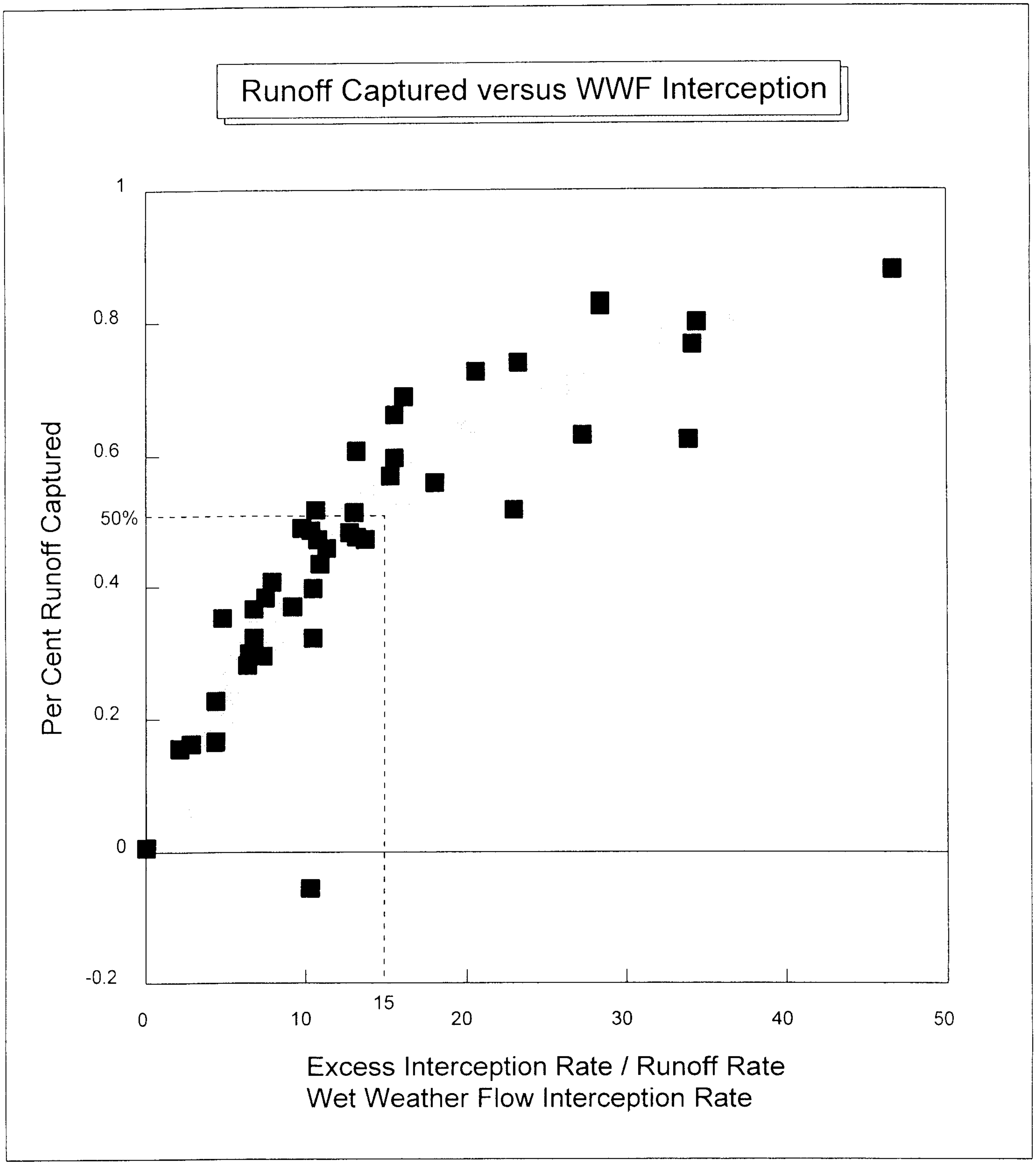


Figure 8-5

capint
pw/sewer

- as the WWF interception rate increases, the number of overflows decreases (Figure 8-4). After the value increases past 20, the reduction in overflows tends to remain about the same. This probably reflects that it is not practical to intercept the runoff from large storms.

- the capture of about 85% of the runoff volume (EPA Policy) would require a wet weather interception rate of about 40, which is not practical without storage (Figure 8-5). The graphs also indicate that to capture about 50% of the runoff for each district and to have about 17 overflows for each district, a wet weather flow interception rate of about 14-15 would be required in each district.

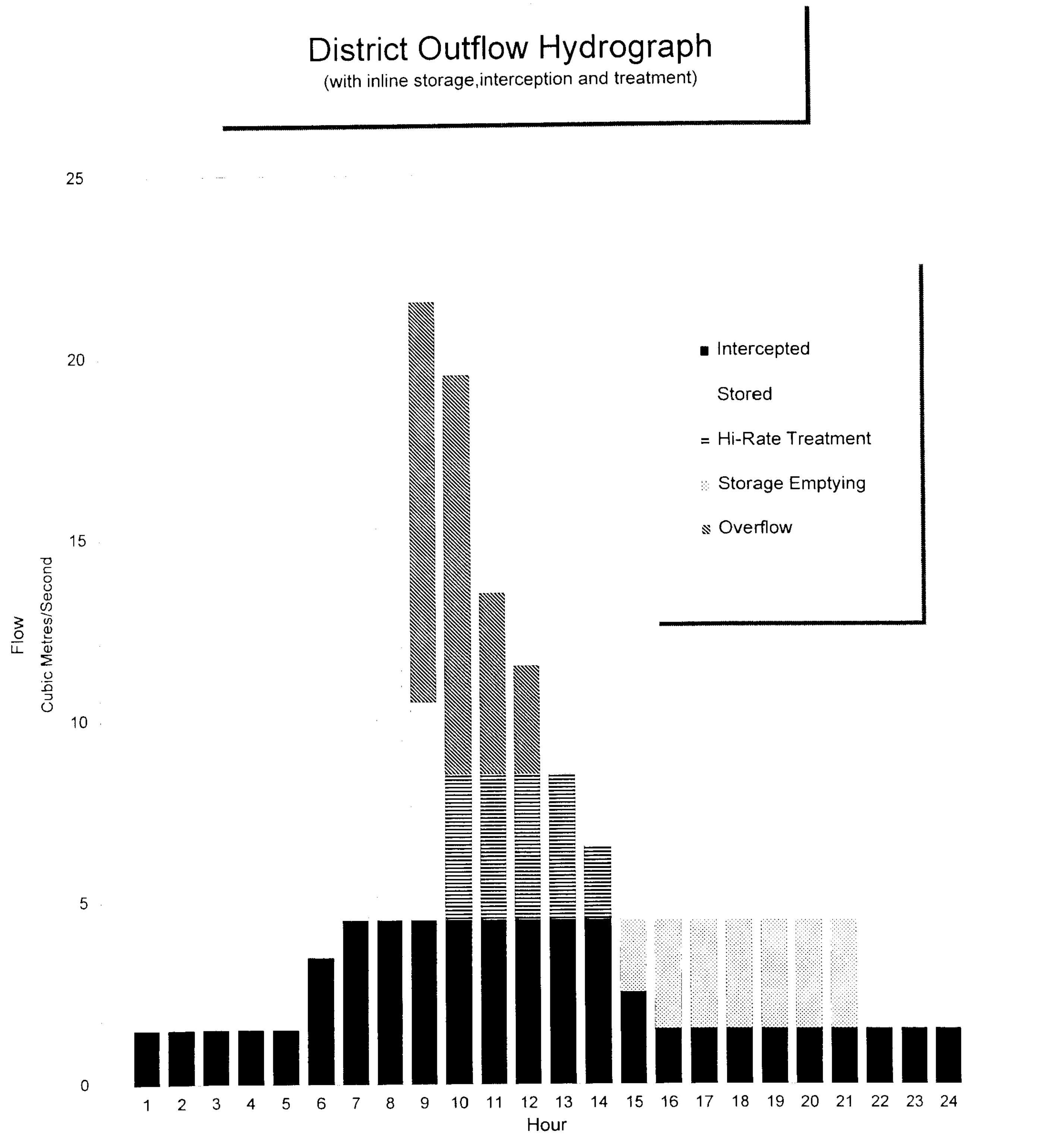
The interception rate for each district can be calculated and used in Phase 3 in order to reduce the variability of overflows from district to district and to assign interceptor capacity most effectively.

Modelling High Rate "End-of-Pipe" Treatment

In Phase 3, High Rate End-of-Pipe Treatment (either VSS or RTB) will be assessed with the control system model. Such options would be sized for particular "design events". Figure 8-6 shows how the output hydrograph would be partitioned. This hydrograph shows the impact when the storage capacity is utilized first and, once the storage is full, the overflow is treated. In this example, the treatment capacity is limited to the design event. Once the capacity is reached, the excess by-passes the disinfection process. An alternative method would be to apply a variable disinfection efficiency (i.e., 2 log, 3 log or 4 log) depending upon the flow rate and consider that the entire overflow would receive treatment but at varying performances.

8.3 HYDRAULIC MODELLING

Inline storage appears to be a cost-effective control option. In Phase 3 a more detailed hydraulic model of a district will be used to assess its validity and effectiveness. The district selected will have a relief system installed (in order to maximize storage) and will therefore have been modelled. It should also have a mix of residential, commercial and industrial land



High Rate Treatment

Figure 8-6

use. Until now, the Clifton district has been identified as an appropriate candidate and level monitors have been installed to determine time-to-rise after the commencement of rainfall. This aspect is of vital importance since the level control systems must operate in a fail-safe manner, i.e., it must be designed and operated so that no equipment failure will cause basement flooding. This aspect will be discussed at the Phase 2 Workshop.

8.4 MAXIMIZING EXISTING INFRASTRUCTURE (FLOOD PUMPING STATION)

As noted in TM #2, the availability of significant pumping capacities at most of the Flood Pumping Stations will permit the construction of end-of-pipe facilities, close to grade. This will realize reductions in the cost of construction. As alternatives are investigated in more detail (and on a more site-specific basis), the value of this feature will be able to be assessed.

8.5 PILOT DEMONSTRATION PROJECT

The candidate technologies were reviewed as to the need for pilot testing or demonstration in Winnipeg. It was considered that there were sufficient numbers of tests being done, either in pilot or demonstration stages, with respect to Vortex Solid Separators (New York, Richmond, Scarborough, Columbus) that Winnipeg-specific tests were not necessary. It may be that some tests of CSO characteristics would be useful to assess the applicability of VSS for the Winnipeg sewage but, at this stage, pilot testing is not considered necessary.

Trash netting has merit for consideration for pilot testing.

Inline storage, while it is a proven technology, has substantial merit for a demonstration project in Winnipeg. Discussions have taken place throughout Phase 2, with regard to the implementation of an inline storage pilot test (or demonstration). This has been referenced in this TM as well as in Appendix C. Inline storage will likely be among the first control alternatives to be implemented as well as the most cost-effective. It is well worth any effort required to obtain a better understanding of the methodology and control techniques involved and in the manner in which it would operate on a real time basis. As noted in Section 8.3, Clifton District has been discussed as being an appropriate location for such a demonstration.

Monitoring is in place and results should soon be obtained. An outline of such a pilot facility in Clifton is provided in Appendix C. The arrangement is shown on Figure 4-1, (attached) of the Appendix. It would comprise an inflatable dam at the Strathcona Street Relief Sewer outfall and a motorized control gate (the gate is in place) at the Clifton Street Sewer outfall. Several level sensors upstream of the facilities would control the system so that storage could be optimized and, in the case of severe storms, would open the gate (or lower the dam), so as not to contribute to basement flooding. This system could be in place by 1996, if a decision is made in the near future.

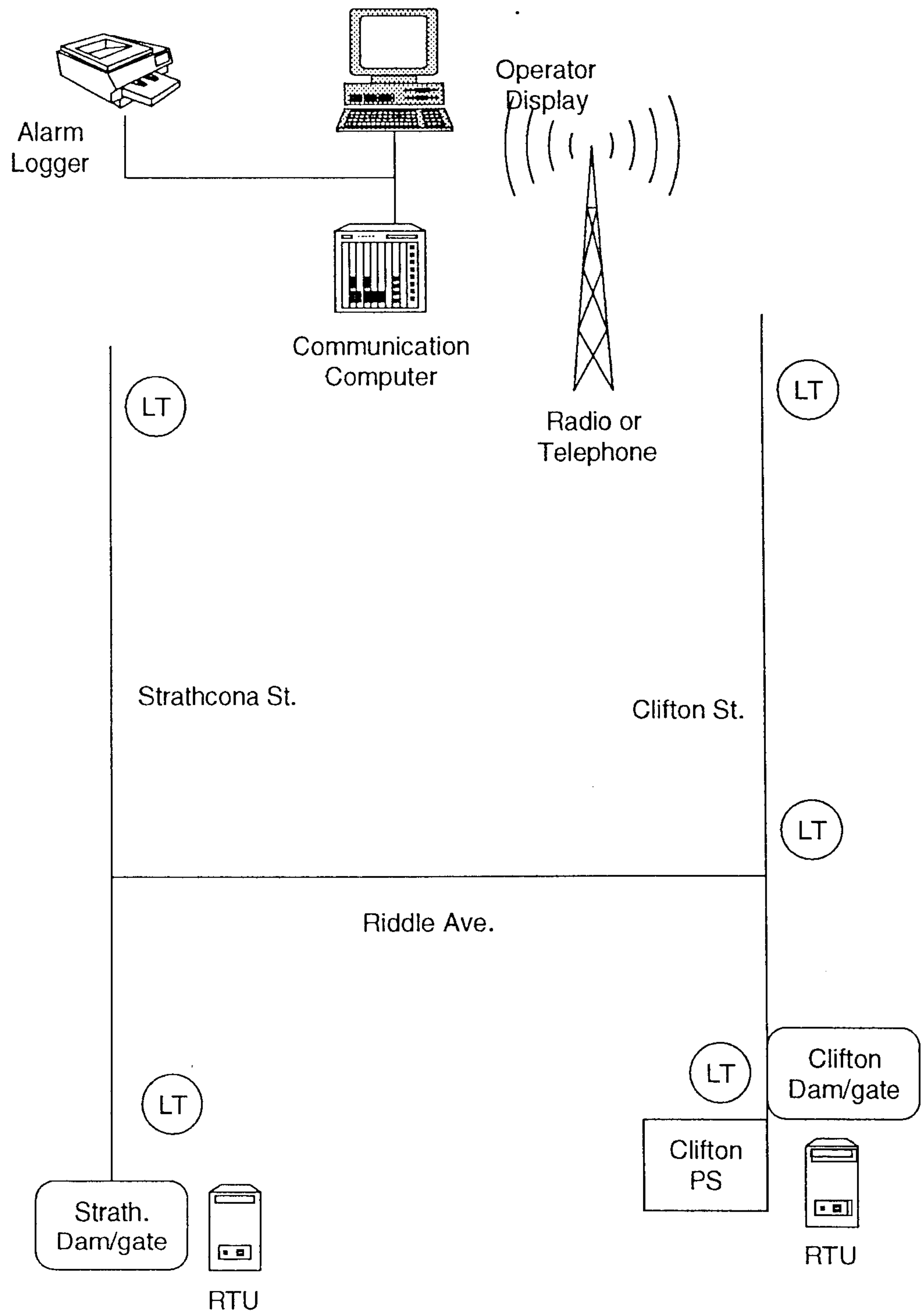


Figure 4-1 Pilot Real Time Control System

(from Appendix C)

REFERENCES

Association of Metropolitan Sewerage Agencies. 1994. Report on Approaches to CSO Program Development. November 1994.

Girling and Sharp. 1986. D. Girling and E. Sharp. Basement Flooding Relief Program Review. R. Girling and E. Sharp. City of Winnipeg. November 1986.

Gore & Storrie. 1989. Town of Richmond Hill. Master Drainage Servicing Plan for the Lake Wilcox/Oakridge District Plan.

Hamilton. 1991. Pollution Control Plan. Paul Thiel Associates Limited/Beak Consultants Limited, for the Regional Municipality of Hamilton-Wentworth. December 1991.

I.D. Engineering. 1993. Baltimore Combined Sewer District, Sewer Relief Study. November 1993.

I.D. Engineering. 1993. Selkirk Combined Sewer District, Sewer Relief Study. July 1993.

Rempel. 1972. "Extraneous Flows in a City of Winnipeg Separate Sanitary Sewer System. G. Rempel, Master's Thesis, 1972.

Tottle. 1972. "The Extraneous Flow Report". C.H. Tottle, P. Eng. May 1972.

Wardrop Engineering Ltd. 1985. Munro, Roland, Hart Combined Sewer Relief Study. June 1985.

Wardrop Engineering Ltd. and Tetr*ES* Consultants Inc. 1991. Red and Assiniboine Rivers. Surface Water Quality Objectives, Technical Report. September 1991.

Wardrop Engineering Ltd. and Tetr*ES* Consultants Inc. 1994. Linden and Hawthorne Districts. Combined Sewer Relief Study. Conceptual Design Report. WEI. May 1994.

WEF. 1989. Water Environment Federation, MOP, FD-17: "Combined Sewer Overflow Pollution Abatement". 1989.