EVALUATION OF INLET CONTROLS
IN DUAL DRAINAGE SYSTEMS

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A Report Prepared for
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by

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EXECUTIVE SUMMARY AND CONCLUSIONS

In order to reduce damages caused by basement floodings from combined and separated sewers, many municipalities have increased the return period of the design storm. The 1 or 2-year storm used in old projects, where the objective of drainage was traffic convenience was increased to a 5 or 10-year storm for property protection.

A previous report for CMHC by NOVATECH ENGINEERING on basement floodproofing methods (1984) has shown that this solution is not only expensive but inadequate since the risk of flooding over the life span of a dwelling is not eliminated. Damages tend to increase in modern homes with basements used for family or recreation rooms. Several solutions were examined and the most promising was found to be dual drainage with inlet controls.

The present study attempts to provide Canadian municipal engineers with all the tools required for an adequate design of inlet control devices (ICD). In a number of recent projects which already use this principle in new sub-divisions the level of control or maximum flow accepted in the storm sewer is about 42 lps/per catchbasin, and in some cases 28 lps. These limits were selected mainly to eliminate sewer surcharge for a 100-year storm. The sewer sizes were in general close to those obtained from an analysis for a free surface 5-year design storm. One of the main reasons for this approach was the lack of information on the performance of various available types of ICD's. The need for a computer model has also hindered a more widespread use of a more sophisticated ICD analysis. At the same time some relief sewer projects for older systems used lower limits such as 10 lps per CB or even less.

A review of the state of the art indicated that the only comparison of several commercial types of ICD's was conducted in the field in Skokie, Illinois for catchbasins with a different configuration than in Canadian subdivisions. Field observations may confirm if an operation is acceptable or not, but will not explain the hydraulic performance and clogging mechanism. Critical rainfall events are rare, and difficult to monitor. In fact, some field observations had to use an artificial catchbasin loading from fire hydrants. For this reason the present study compared the ICD's on a hydraulic model at the scale 1:1.

After a review in Chapter 2 of the main ICD and dual drainage concepts, the report describes in Chapter 3 the hydraulic operation of four types of ICD. The first three types are commercially available:

A. SCEPTER-an orifice type ICD with a self-cleaning notch.

B. CROMAC-an orifice type with a variable area slot ICD.

C. HYDROVEX-a vortex ICD, representing an improved German version of the HYDROBRAKE orifice.

The fourth type of the HANGING TRAP is a self made ICD proposed in the Skokie studies.

Head-Discharge curves were determined and found practically the same as indicated by the manufacturers. The discharge coefficients are relatively the same for the first two devices, but are much smaller for HYDROVEX.

Chapter 4 describes in detail the clogging experiments, which were in general conducted in a conservative mode as compared to natural occurrences. It was found that all devices will operate without permanent clogging if the control-limit is 20 lps and SCEPTER can be used for flows greater than 14 lps. HYDROVEX was tested and operated well down to 8.5 lps., although it may function adequately for even lower flows than in the tests. On the other hand for all ICD's it is possible to find a combination of leaves and branches loading which may temporarily plug the orifice and consequently visual inspection and cleaning is necessary. Clogging is easier for the simple orifices (SCEPTER and CROMAC). Another factor to be considered in the selection process is the effect of protruding ICD's on current pump cleaning. Based on these considerations the report does not recommend for typical Canadian catchbasins the HANGING TRAP device. The cost of HYDROVEX is at present higher than that of simpler orifices types ICD's.

The present practice in new subdivisions is to use a minimum control level of 28 lps. If this limit is reduced as low as 14 lps, orifice ICD's could still be used. On the other hand, the HYDROVEX is the only ICD recommended for lower levels of control, such as the ones used in some relief sewer studies.

Chapter 5 presents computational methods for the study of ICD's. A simple hand computation method, DUAL CHECKHYD, was developed and is compared in Chapter 6 with the sophisticated OTTSWM computer models. Results are relatively close, the main limitation of DUAL CHECKHYD being that it cannot analyze surcharge and has an approximative treatment of catchbasins without ICD. The method is mainly recommended for small subdivisions where the computational effort is limited.

Chapter 7 reviews several recent projects and shows that for ICD's with 28-42 lps per CB, it was possible to maintain a number of inlets without control devices. It is therefore recommended to avoid implementation of ICD's within this range at all inlets. A detailed comparison of five levels of control for a 42 ha subdivision is conducted and reveals that savings per ha, as compared to the present practice of 42 lps/CB can be very significant.

It was shown that by reducing the control level of ICD's the pipe flow corresponds to a traditional design for a more frequent storm. The previously used 2-year storm, corresponds for Metro-Toronto conditions to a 20 lps ICD. For a 20 lps ICD, park storages would operate about twice per year instead of once every five years as for the 42 lps ICD currently used. The depth of this frequent flooding would be however relatively small. A comparison of economics and operational characteristics for a typical area is given in the following table:

Level of Control (lps)	Approximative Savings in $1,000/ha 100-Year 7.3 ha	Approximative Savings in $1,000/ha 100-Year 42 ha	Maximum Gutter Depth (cm) 5-Year 7 ha	Maximum Gutter Depth (cm) 5-Year 42 ha	Maximum Gutter Depth (cm) 100-Year 7 ha	Maximum Gutter Depth (cm) 100-Year 42 ha	Area of Park Storage % of Total Area 100-year 42 ha	Average Depth in Park for a 5-Year Storm (cm)
42	0	0	7.4	14	13.7	27	1.1	18
28	2.9	3.2	8.4	16	15.0	30	1.4	17
20	4.6	5.5	9.6	18	15.7	31	1.8	18
14.0	4.7	6.6	10.2	20	16.2	33	2.2	20
8.5	6.4	8.6	10.9	22	16.5	34	2.7	23

CONCLUSIONS

1. By adequate selection of the type of ICD's using the information presented in this report, the level of control can be lowered as compared to present Canadian practice.

2. Savings as compared to the present practice could be significant, e.g., $3,200/ha in a typical residential area with inlet control at 28 lps and catchbasins.

3. The main limitation in achieving high savings with inlet controls is the concern related to higher street flow depth during major storms and increased frequency of park flooding. These effects vary with contributing areas, imperviousness, slopes, etc. and have to be assessed using detailed computations for the specific conditions of each project.

4. A hand computation method DUAL CHECKHYD developed and tested in this study gives municipal engineers a simple tool for analysis and verification of smaller projects. A computer model, OTTSWM, is available for more important studies and surcharge analysis.

5. An example of detailed studies such as the ones described in Chapter 7 can be used at least as a basis of reduction of ICD's from the present limits to 20 lps/CB.

6. Where street depths are acceptable in existing areas that have basement flooding the use of ICD's with 14 lps/CB or less selected on the basis of the data presented in this report will also be considered.

7. For relief sewer studies lower levels of inlet control may be used.

It is proposed to use this report as a basis for a dialogue with municipal engineers, consultants and developers on this new technology. Because of the significant savings and improved protection which can be achieved with the ICD's, it is suggested that workshops be organized on these issues in all major Canadian cities.

Chapter 1

INTRODUCTION AND STUDY OBJECTIVES

In 1983-84 NOVATECH ENGINEERING conducted a review for CMHC of the causes of basement flooding and solutions to avoid these occurrences in new developments (Wisner, Hawdur. 1983). The study included a review of damages, perceptions of homeowners and opinions of municipal engineers to large expenses for conventional relief sewer projects. It discussed the potential of many new techniques and the need for public education. The study emphasized that a first priority is to avoid basement flooding in new developments. It indicated that traditional drainage design has a built-in potential for basement flooding, no more acceptable by a public aware that this is not necessarily an "act of God" in legal terms. The study recommended a more widespread use in Canada of a new drainage approach in which the dual drainage concept is combined with inlet control devices and park storage. It identified research needs in connection with this solution.

With this new approach drainage design for new subdivisions is not solely a pipe sizing exercise, but one that considers runoff both on the surface and in pipes. In designing a new subdivision lot grading, street grading, backyard swales, park location and size as well as the pipe system are all important.

Sewer pipes are usually constructed to transport runoff from frequent storms, so as to prevent stormwater from being a traffic nuisance. Flows larger than the design flows will cause the system to surcharge. With inlet controls, protection of the minor system is provided against large events that exceed the design capacity. Inlet controls permit flow to enter the pipe system up to its design capacity, any extra flow will backup in the catchbasin and spill on to the street. Here, the major system design becomes critical, it must be able to transport the excess water to safe discharge points, whether a park storage site, a man-made channel or local stream. Proper design of street and lot grades will prevent surficial flooding.

First applications of this new concept were in Markham, where pipes were designed for a 5 year return period and inlet controls were introduced mainly to prevent surcharge for 25 or 100 year storms (Wisner et al 1979, Mukherjee et al 1983). In addition to this, the previous CMHC report indicated that inlet controls should also be used in new subdivisions to obtain a more economic design. By limiting the inflow to a level below that given by the minor system design return period, smaller pipes can be installed. If inlet control to a lower level is provided street inundation will occur more frequently. The major system outlets, whether parks or channels will be used more often and will require a greater capacity for larger events.

At present a range of inlet control levels and different devices are being used. In Metro Toronto, inlet controls limiting the flow to approximately 28 lps (1 cfs) is accepted as being a safe lower limit, but it has no basis in experimental observations. It is very conservative when compared with values used for relief of combined sewers, where flow restrictions were as low as 3 lps (Pisano, 1982). While inlet controls are already recommended by some Canadian municipal regulations, no standards for the level of control have been established. Minimum flow levels need to be determined so that units with unacceptable performance will not be applied.

Other practical concerns with inlet control devices are related to maintenance, economics and design procedures. Maintenance concerns are that inlet control devices will interfere with catchbasin cleaning, or that they may be easily clogged by debris. Previous work has shown that substantial savings are possible by using inlet controls in relief sewer studies. This has not been established for dual drainage systems. At present design of a dual drainage system requires the use of a sophisticated computer model. Simple design tools are required so that the principle of inlet control will receive widespread application in new developments.

The purpose of this study is to provide municipal engineers with the technical and economic information necessary to select inlet control devices and design systems using them. More specifically, the objectives of the present study are: to examine the performance and dependability of several inlet control devices applied in Canada and the U.S.A., to develop a simplified design methodology for practicing engineers and to examine economic savings that may result by using inlet controls. The report is divided into three parts, the first provides a general overview of inlet controls. In Chapter 2, the basic drainage concepts are reviewed for those who are not familiar with inlet control.

The second part describes an experimental program for studying inlet control devices. Chapter 3 describes several types of inlet control devices recently used in Canada and the U.S.A. and describes their hydraulic performance. In Chapter 4 a series of tests for the study of clogging aspects are described. Also the minimum flow for an acceptable performance is given for various inlet types.

In the third part, design and economic aspects of systems with inlet control devices are examined. Chapter 5 presents DUAL CHECKHYD, a simplified hand computation method for the analysis of inlet controls in small subdivisions. A design example is presented in Chapter 6 and is compared with the computer model OTTSWM. Chapter 7 compares for a small subdivision various levels of inlet control and presents a comparison in terms of economic and maintenance aspects. While the report and particularly part III is directed toward the application of inlet control devices for new subdivisions, the experimental data given in part II may be used in relief sewer studies.

The study benefited from the cooperation and assistance of many engineers and organizations. Dr. Townsend from the University of Ottawa provided advice on the testing methodology. The firms of Scepter, Cromac and John Meunier provided their inlet control devices. Mess'rs. Cromac, Meunier, Pouport and Dr. Townsend attended the tests and agreed to provide comments on their description in the report. Other comments were received from representatives of the cities of Ottawa and Nepean. The firm of Andrew Brodie Assoc. Inc. provided several examples of projects with inlet controls, performed many computations required for the comparison of computational methods and assisted in the economic analysis. Mr. C. Kochar from CMHC attended the tests and provided useful comments during the study. Their assistance and that of municipalities who provided answers to a questionnaire sent in connection with the 1984 Novatech study is gratefully acknowledged.

Chapter 2

REVIEW OF DRAINAGE CONCEPTS

In the last two decades stormwater management (SWM) has become an important concern to government agencies, municipal engineers, and the general public. One of the main SWM objectives is to minimize possible adverse consequences of stormwater runoff such as basement or surficial flooding. Before stormwater runoff can be controlled a detailed understanding of the system must be had.

One of the most important, but previously neglected, components in the runoff process for urban areas is the catchbasin inlet. The operation of the catchbasin inlet determines the proportion of runoff that enters the pipe system and that which remains on the street. The amount of street runoff captured is controlled by the magnitude of overland flow, the type of grating and the flow conditions in the storm sewer. INLETS formed by gratings and catchbasins are the major link between the surface and the pipe system. In some situations they may permit too much water to enter the storm sewer, and cause surcharging. On the other hand, if the inlet gratings are too small, insufficient flow enters the pipe system, and street flow depths may be excessive causing surface flooding.

To provide optimum operation of the drainage system the flows must be regulated so that neither surficial flooding or sewer backup occurs. The relatively new concept of regulating flow to the pipe system is also known as INLET CONTROL. The purpose of this relatively new technique is to limit the peak flow, entering the storm sewer so free flow or flow with limited surcharge is maintained. This is achieved by installing an INLET CONTROL DEVICE (ICD) or INLET RESTRICTOR (IR). As shown in Figure 2.1 all the flow up to the inlet control level enters the catchbasin. Higher flows than this remain on the street to be carried to a low point in the system.

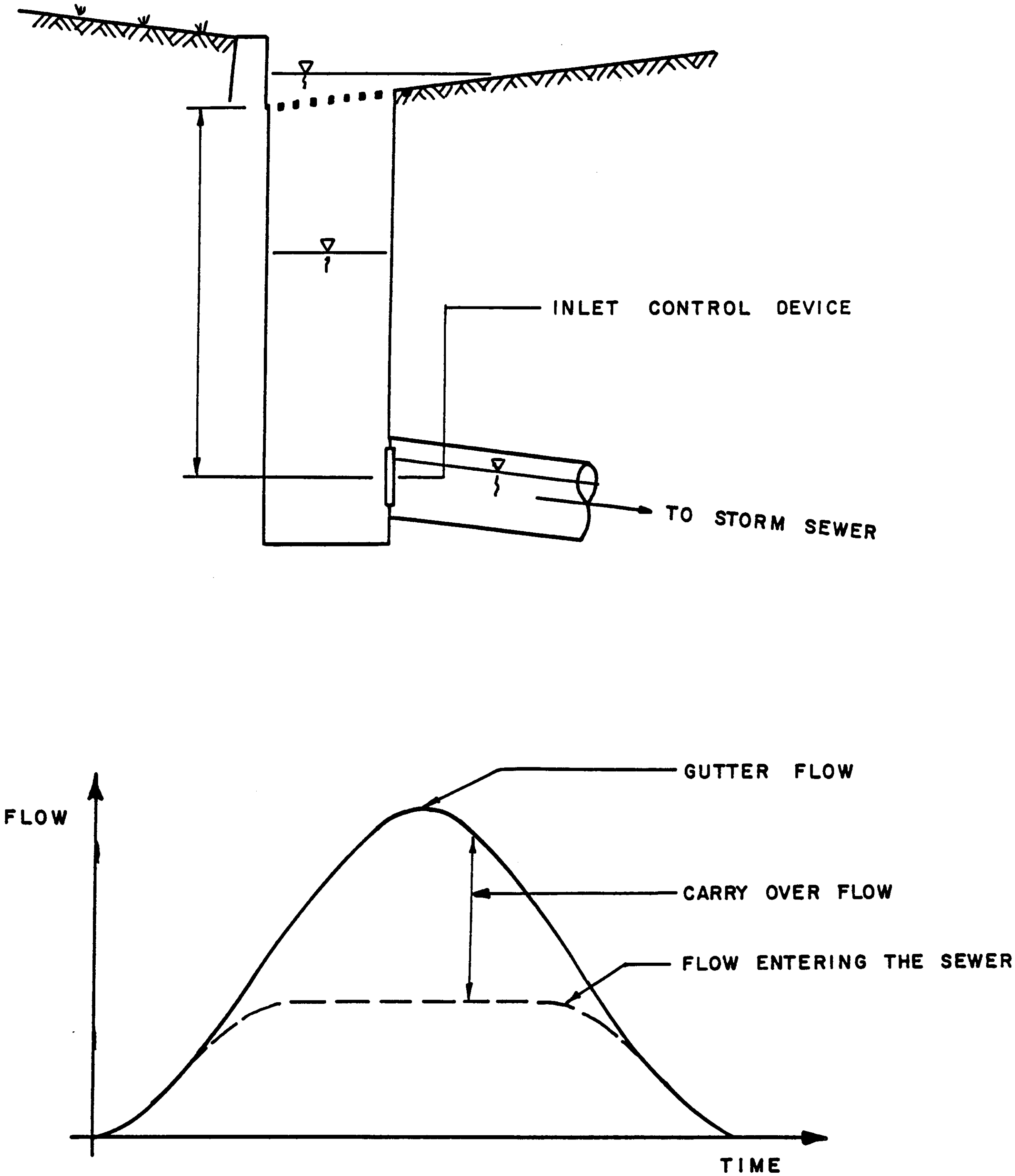


Figure 2.1

Principle of Inlet Control

2.1 Dual Drainage Concept

Municipal engineers that use inlet control recognize the dual drainage system, consisting of the surface flow network or major system and the sewer network or minor system. Properly designed and maintained, the minor system reduces the incidence of flooding inconvenience to both pedestrians and motorists. Flows are conducted away from intersections and pedestrian crossings where ponding would create a nuisance. The minor system is also called the convenience system and is designed for low return periods, usually between 2 and 10 years.

For events that would cause the convenience system capacity to be exceeded the major system becomes important. It is formed by streets and in some cases by channels and ponds which convey or store runoff from less frequent storms. By proper grading, streets can be designed to convey overland flow to designated discharge points. The major system always exists whether or not it was designed for. Water will seek the lowest levels despite buildings or other obstacles in its path. By designing the system, the lowest level can be designated as a park, stream valley or other depression and not be a building.

Not only is the recognition of the dual drainage system important, but also its integration with various runoff control methods, termed stormwater management techniques. This may involve disconnecting roof leaders, providing roof storage, or having in-system detention ponds, etc. While the dual drainage concept was first developed in the late sixties in Denver (Wright - McLaughlin, 1968), the key role of ICD's for its adequate operation has been first fully recognized in Metro Toronto (Wisner et al, 1979). Properly designed ICD's will permit low flows to enter the pipe system but will restrict large flows to a desirable level so pipe capacities are not exceeded. The operation of a dual drainage system using the inlet control principle is illustrated in Figure 2.2.

Figure 2.2a Operation of the Dual Drainage System During Frequent Storms

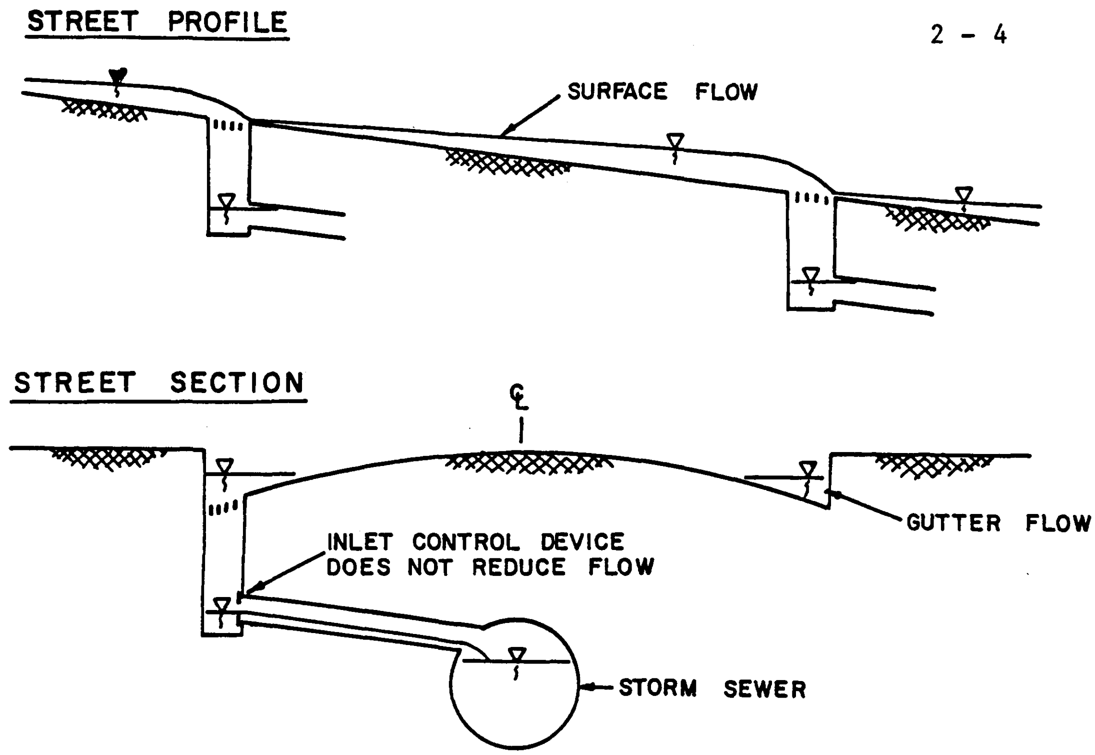
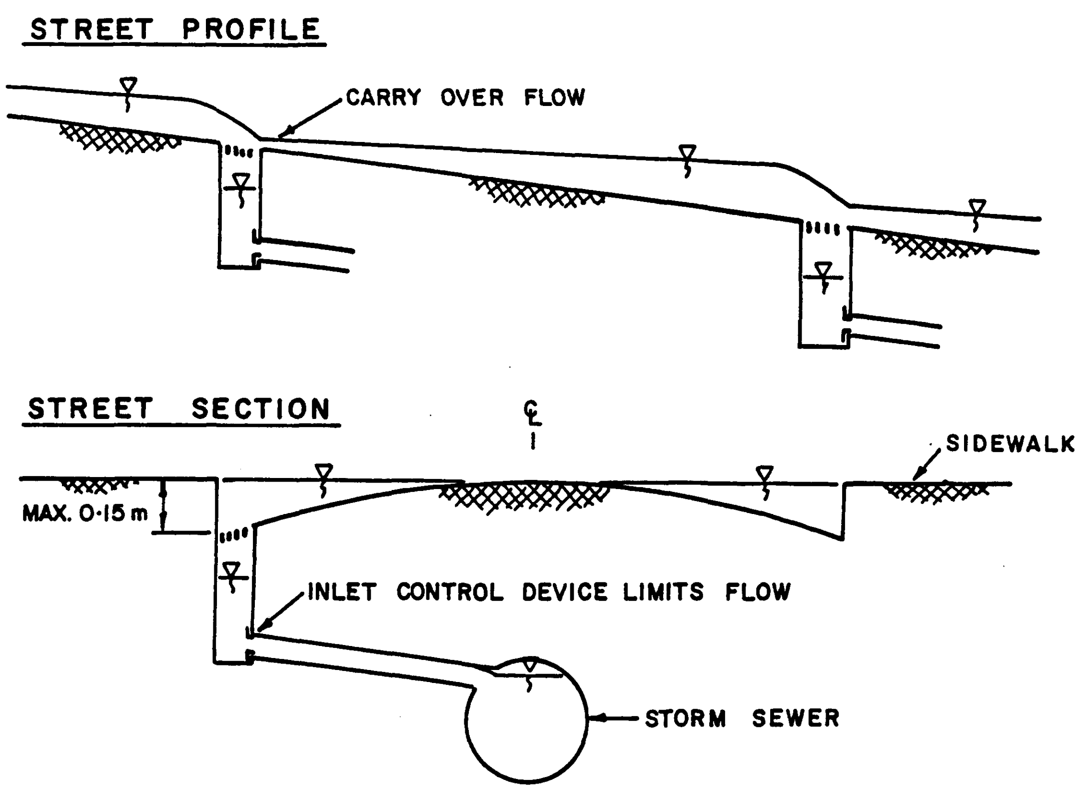


Figure 2.2b Operation of the Dual Drainage System During a Severe Storm



The design looks at the whole drainage system and controlling flow throughout it. For frequent storms, water on the streets should be removed as quickly as possible, but not at the expense of basement flooding. ICD's are inserted in catchbasins to provide an optimum controlling link. Street convenience is only affected during infrequent storms, when flows into the sewer are controlled and backup onto the streets may occur. The level of inlet control (maximum ICD flow) must be selected so that during very rare storms (e.g., 100 yr. return period) the minor system is without surcharge or with an acceptable surcharge. Overland flows are often directed towards offline park storage and released at a lower rate. By implementing the dual drainage concept a higher level of protection against flooding is provided, the probability of basement flooding is reduced, better planning of neighbourhoods is obtained and construction costs are often the same or lower.

2.2 Catchbasins

As indicated above, catchbasins form an important link in dual drainage systems they are examined here in more detail. Catchbasins are wells built at the street curb to allow surface water to enter the sewer system. The catchbasin frequently has a sediment sump for trapping coarse debris and in some cases, a water seal is provided to prevent sewer odours from leaving the catchbasin.

Solids are retained in the sump, heavy debris settles to the bottom while light solids float on top. Regular cleaning of the catchbasin is therefore necessary. A number of different methods are available, one frequently used in Canadian cities is the vacuum cleaning machine. A large suction tube is lowered into the catchbasin to remove the floating and settled debris as well as the water from the catchbasin sump. Inlet control devices placed in a catchbasin must either be flush with the catchbasin wall or be made of sufficiently strong material so that they will not break when hit with the suction tube. Also they should not protrude so as to prevent the suction tube from being lowered into the catchbasin.

Catchbasin inlets are primarily of three types: grate inlets on the street surface, curb inlets located on the curb face, and combination inlets which combine the characteristics of the previous two. Because the capture of gutter flow is important if the pipe system is to be used efficiently, many investigations into inlet gratings have been conducted (Larson 1944, Johns Hopkins Univ. 1956, U.S. Dept. of Transportation 1979, Marsalek 1980, Townsend and Moss 1980). The studies showed that for an inadequate inlet most carryover flow passes around it. A large portion of flow cannot be intercepted unless the inlet extends into the flow path. The inlet width will depend on the shape of the gutter and the allowable width of flow. The capture efficiency depends on the length of the openings in the direction of flow. Grating bars transverse to the flow diminish the inlet capacity. Increased capture is obtained by having the inlet grating covered with flow, but some additional bypass occurs. Flow capture by gratings corresponds in general to a capture efficiency of less than 100%. As shown in Figure 2.3 the effective capture can vary widely from one grating to another.

Catchbasins may vary in design from one municipality to another. One of the typical Canadian catchbasin is 0.6 x 0.6m square and 1.83m deep. The standards for this type of catchbasin are given in Table 2.1:

Table 2.1

Catchbasin Construction Standards

(typical for Ontario)

Equivalent Diameter (m)	0.61
Depth (m)	1.83
Outlet Location (m) above the bottom	0.6 - 0.9
Sump Capacity (M^3)	0.23
Storage Capacity(m^3)	0.45

	GRATING	BASIC SHAPE	PATTERN	HYDRAULIC AREA (cm²)	WIDTH (cm)
(A)	TORONTO	SQUARE	FISHBONE	1737	50
(B)	MONTREAL	ROUND	TRANSVERSE BARS	1868	65
(C)	OTTAWA	RECT.	GRID	779	31
(D)	EDMONTON	COMBINATION	LONGITUDINAL BARS	884	25

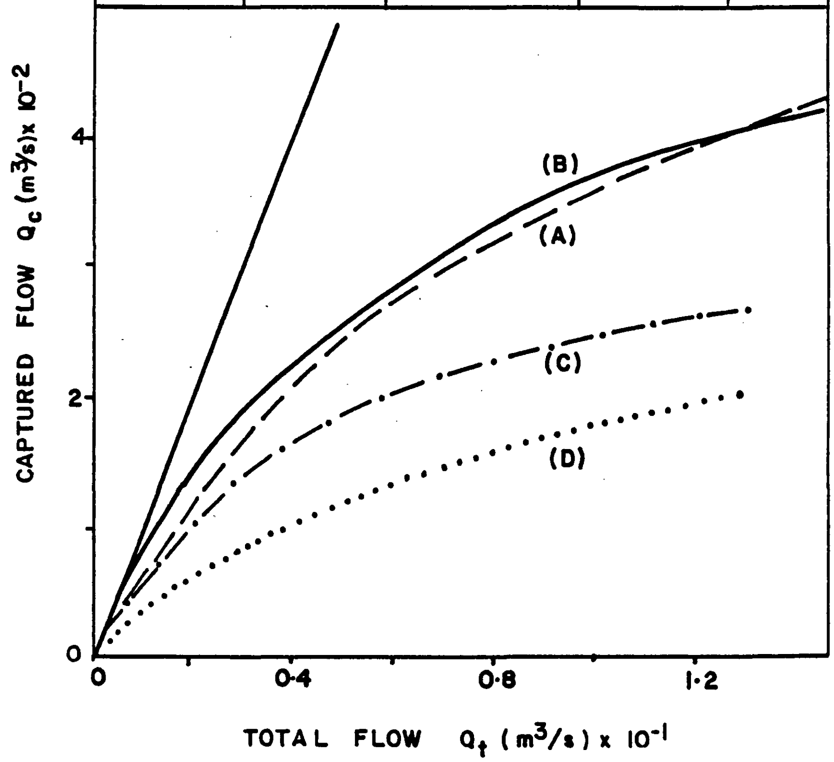


Figure 2.3 Performance Curves for Four Canadian Gratings
(Ref. Bouchard and Townsend, 1984)

There is some variation in catchbasin design with each municipality across the country. In Toronto and surrounding municipalities the square catchbasin is commonly used. The use of round catchbasins is more widespread in Quebec. Figure 2.4 shows two different designs found in North America.

In some European cities catchbasins do not have a sump. Rather a bucket sieve is placed in the inlet to trap the debris. The bucket can be easily removed, facilitating rapid cleaning by street crews. Figure 2.5 illustrates different catchbasin designs found in Europe. The high-intensity, short-duration, summer thunderstorms typical in North America rarely occur in Europe. In addition, many European homes do not have a finished basement, thus even if the sewer surcharges, serious flood damage will not likely occur. In areas with combined sewers homeowners have the responsibility for check valve maintenance and operation. Under these conditions dual drainage and inlet control devices are not used in European cities.

The location of catchbasins influences the inlet control level. In design, catchbasins are first placed at street intersections to intercept the pavement runoff, before it spreads across the street and at low points. The intermediate spacing should ideally be governed by the spread of water on the roadway and the maximum gutter flow. The ability of the catchbasin to capture the gutter flow should be considered. If there is excessive carryover, then the spacing should be reduced so that the flow between the first and second inlet does not exceed the allowable gutter flow. The spacing of catchbasins should be done when the minor system is being designed, for a low return period event. By restricting the number of catchbasins and using maximum spacing inflows to the pipe system can be partially limited. Control by adequate spacing is more frequently used for highway drainage (Marsalek, 1983). In most new developments catchbasin locations are simply determined on the basis

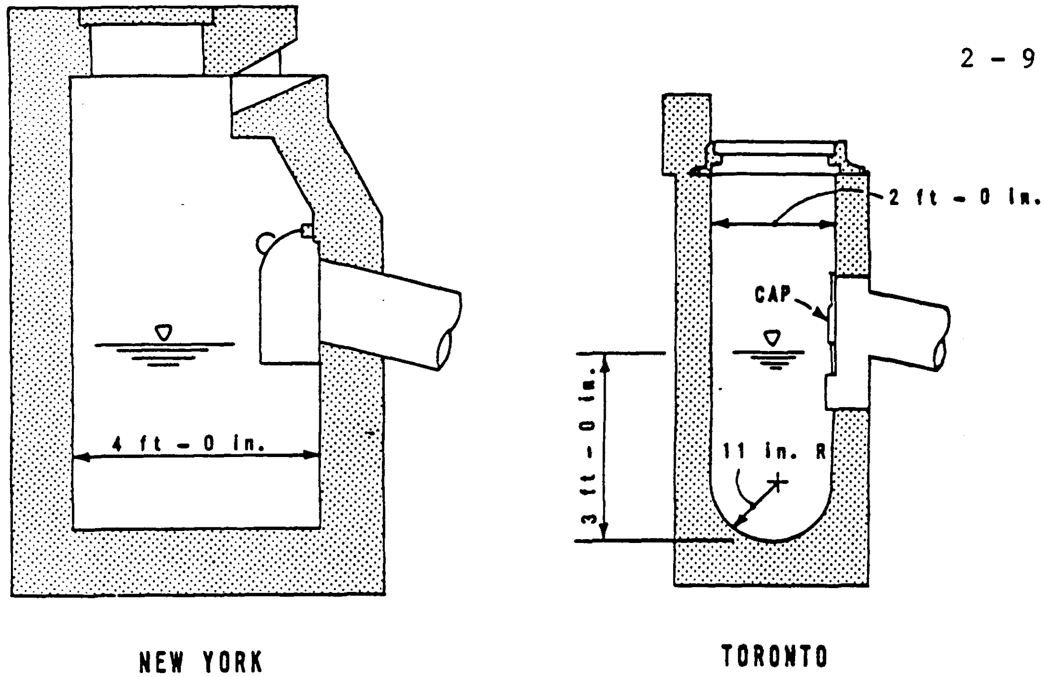


Figure 2.4 Representative catchbasin designs in United States and Canada.

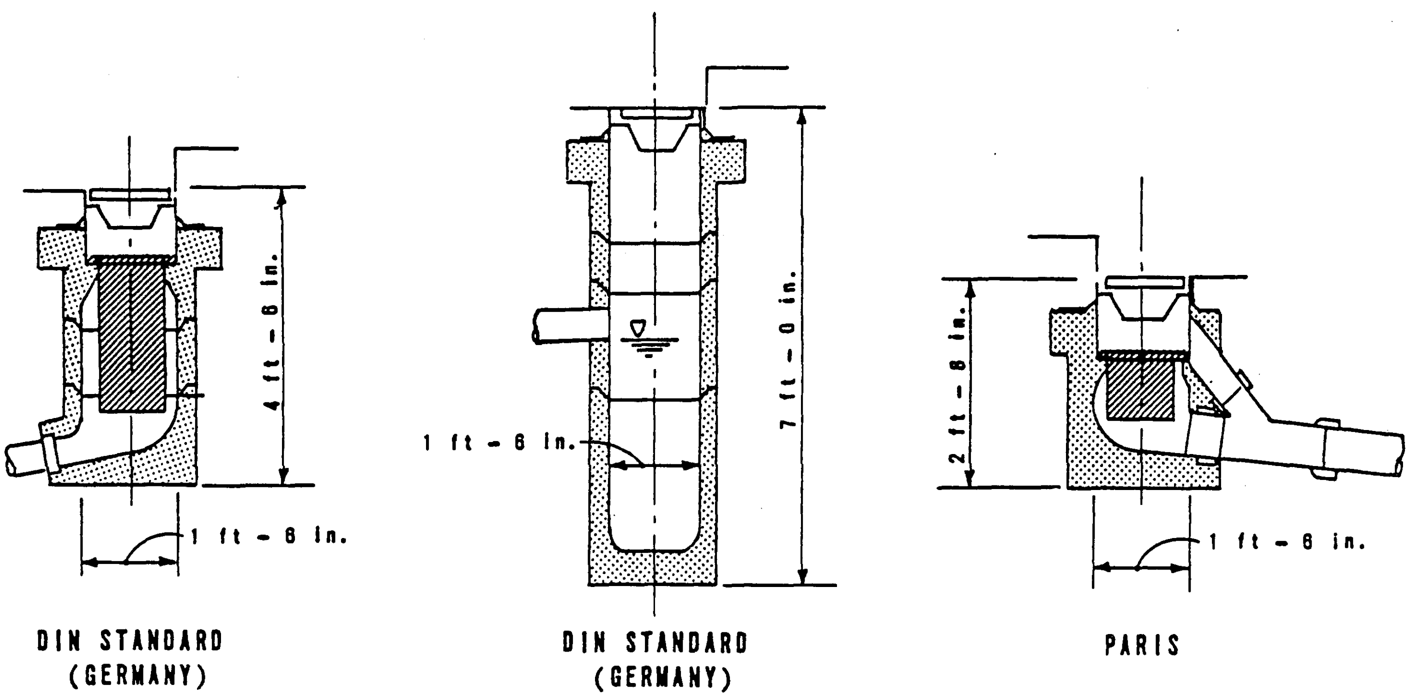


Figure 2.5 Representative catchbasin designs in Europe.

of regulations which require a minimum distance of, say, 100m between two catchbasins. These regulations are based on experience. They were established prior to dual drainage or ICD's and frequently may lead to overloading of storm sewers. With inlet controls inflow to the minor system may be altered in a flexible way.

2.3 Design Considerations

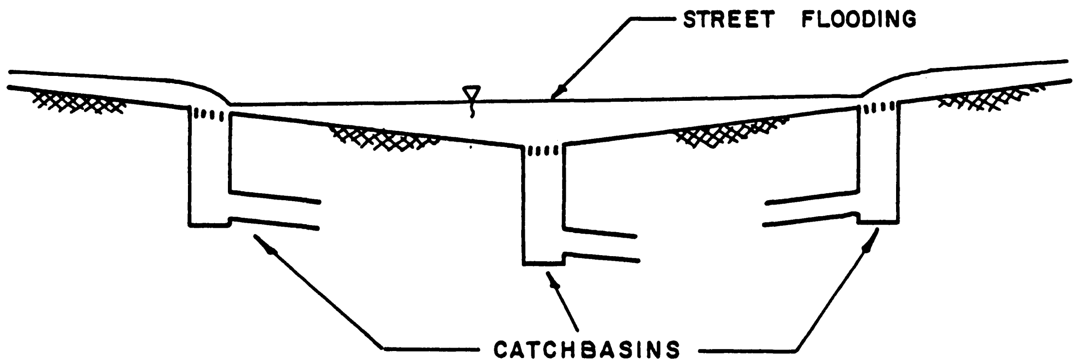
Existing Drainage Systems

Although the use of inlet controls for relief of surcharged pipes in older systems is not an objective of this report, readers are reminded that their use is important in existing drainage systems which may have either separated or combined sewers. Flooding problems will occur if the sewer pipes have insufficient capacity. This may be as a result of under design, re-zoning an area to a higher density, poor construction with consequent high infiltration rates, or old age and general deterioration of the pipe system. The overall result is the same, more water entering the system than its capacity (Figure 2.6). The traditional solution to these problems is reconstruction of the existing sewer, or provision of a relief sewer with sufficient capacity. This is an expensive solution as excavation has to be conducted around many installed services. A previous report for CMHC (Wisner, Hawdur, 1983) describes various innovative design methods that prevent complete sewer reconstruction and are increasingly considered by municipal engineers.

The report indicates that by combining a number of stormwater management techniques, including inlet controls, cost reductions up to 25% were possible on some relief sewer projects. The potential of inlet controls for savings in relief sewer studies is confirmed by projects in Evanston, Illinois (Pisano, 1982) and Laval (Dessau , 1985).

Design using inlet controls will be governed by the existing surface grades. The surface flow must not only be restricted from entering the pipe system at critical points but be redirected so

STREET PROFILE



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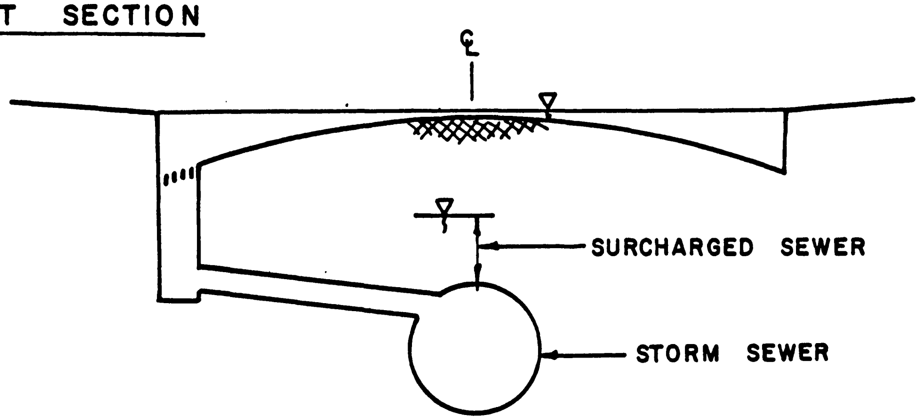


Figure 2.6 Overloaded System during a Large Storm Event

that it does not accumulate and cause surface flooding problems. In existing developments it is typically directed to detention storage locations from where it can be safely discharged into the sewer system when the danger of surcharging has passed. Ponding or detention storage may be provided in parking lots, on streets, in parks or in underground storage facilities.

Complete sewer construction was avoided by the innovative solution shown in Figure 2.7. Inlet controls were used so that upstream portions of the sewer pipe did not need to be reconstructed. The use of inlet controls permitted the pipe to be kept within its capacity. Flow prevented from entering the pipe system was conducted over the road surface and ponded at a low point further downstream. It was necessary to construct a large sewer pipe downstream from the ponding site so that the depth of water on the street did not exceed desired levels.

In areas of existing commercial developments, such as shopping centres or industrial complexes, inlet controls can be used in parking lots (Figure 2.8). The slower release rate provided by an inlet control device means that any excess flow is ponded on the parking lot, downstream surcharging or costly reconstruction are avoided. Infrequent parking lot flooding may not be damaging if the maximum water level is adequately controlled.

Figure 2.7 Inlet Control to reduce the Sewer Reconstruction Cost

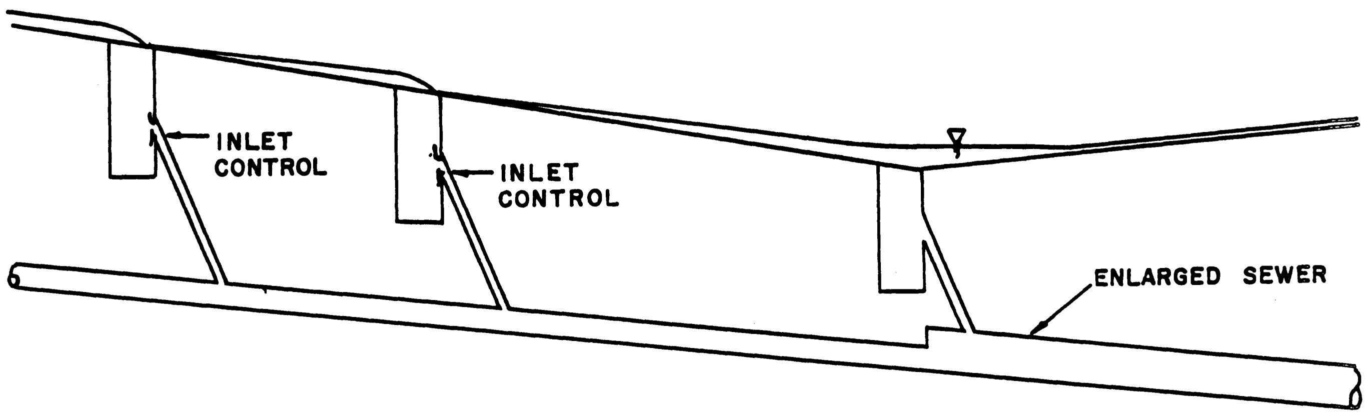
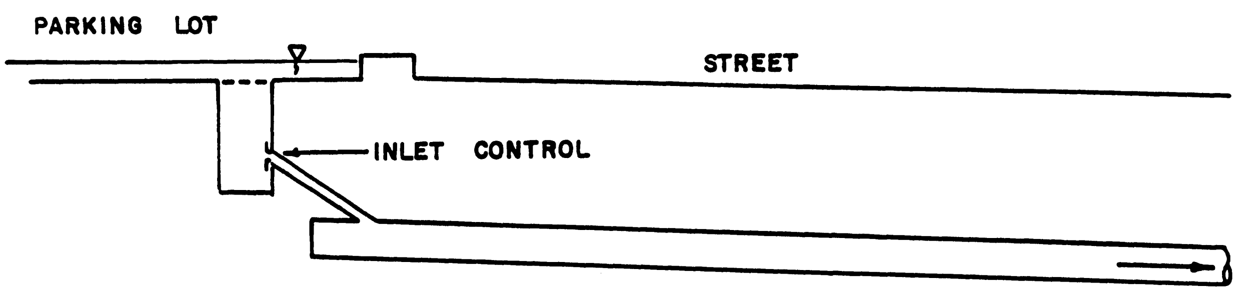


Figure 2.8 Parking Lot Inlet Control



Chapter 3

INLET CONTROL DEVICES (ICD)

3.1 Introduction

For a given number and spacing of catchbasins there are two different ways to regulate the inflow to the minor system (pipe system):

A) by changing the grating and outlet pipe without using a special flow control device,

B) with special inlet control devices (ICD's) of two kinds:

B1) by restricting the flow at or just below the grating,or,

B2) by placing a flow control device in the catchbasin outlet.

A) Gratings are not usually considered as being a flow controller and yet their capture efficiency drops with increased flows. Despite having an efficiency less than 100% during heavy storms, the total flow from the catchbasin inlets with standard grating control may exceed the minor system free flow capacity. Therefore additional methods of flow control must be considered. Two possibilities with a normal catchbasin configuration are to use a less efficient catchbasin gratings or to reduce the diameter of the outlet pipe. A catchbasin grating with a long opening in the direction of flow has a much greater capture capacity than one with openings transverse to the flow. Thus a grating with smaller area openings may effectively limit the inflow. The effect of control by changing the grating characteristics is shown in Figure 2.3, where captures for typical gratings in Ontario, Edmonton, and Laval (Montreal) are compared (Bouchard and Townsend, 1984). One can see that the Edmonton gratings are less efficient than the ones used in Ontario. It is unlikely that municipalities would want to modify in the near future entrenched standards although experiments with new gratings should be encouraged. In addition by using special restrictors, the capacity of only some catchbasins can be reduced, thus maintaining a greater design flexibility.

Typical municipal standards require a 200 mm - 250 mm diameter pipe for the catchbasin lead. The capacity for a 200mm pipe may range from 34.2 lps to 152.9 lps for slopes between 1% and 20% respectively. This is specified to minimize clogging and facilitate cleaning. Catchbasin leads of this diameter are large enough that they do not restrict the flow, however smaller pipes would act to diminish the outflow. Problems with this option are an increased possibility of clogging, cleaning is more difficult and after installation it is difficult to change. These considerations should dissuade municipalities from considering it as a valid alternative.

B1) The second type of inlet control may be obtained with flow restrictors installed at or below the catchbasin grating. The grating area may be reduced by mounting plates either on the top or bottom of the grating (Figure 3.1). This results in a lower capture efficiency and more flow left on the streets. A disadvantage with this method of flow restriction is that the grating capture efficiency is reduced over the complete range of flows. Thus even for small events during which the minor system has adequate capacity some flow reduction would be occurring.

Horizontal orifice plates are installed below the catchbasin grating (Figure 3.2). The size of the orifice in the plate restricts the flow to the desired level. A disadvantage of these units is that the horizontal plate restricts access to the catchbasin. The plate must be removed before the catchbasin can be cleaned. It was also found that the horizontal plates were susceptible to clogging, as discussed further in Chapter 4.

Figure 3.1 Catchbasin Grating Restriction

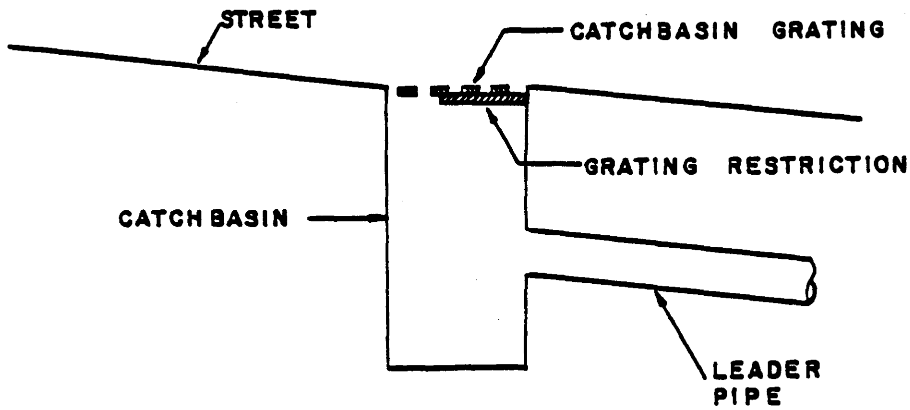
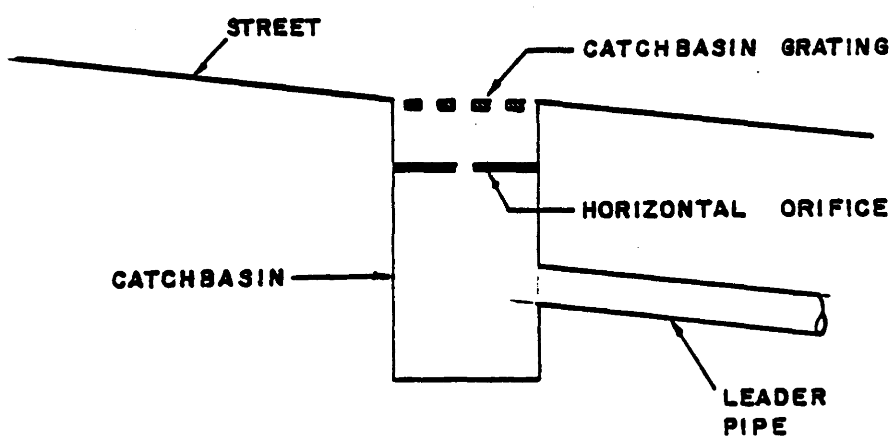


Figure 3.2 Horizontal Orifice Restrictor



B2) Inlet control devices placed in the catchbasin outlet are the third type of flow restrictor. The units reduce the outlet area, thus altering the catchbasins' rating curve. An advantage of these units is that for low flows they do not interfere with the grating capture. It is only during large flows, that exceed the capacity of the unit, that water backs up in the catchbasin and limits the amount of capture. Previous studies by Townsend, Wisner and Moss (1980) have indicated that the last type of control is superior to the previous ones for the following reasons:

1. The orifice can be easily installed.
2. A flush mounted orifice does not interfere with catchbasin cleaning.
3. The orifices performed well with debris in the catchbasin.

This study examines therefore only the third category. Four different types of these units are compared. The first three had orifices placed in the outlet pipe, one was designed to diminish the flow by causing the outflow to vortex, and in the last unit the orifice was submerged in the catchbasin. The first two outlet orifice devices are manufactured by the firms of Scepter and Cromac respectively. They are referred to hereafter as the Scepter and Cromac inlet control devices or flow restrictors. The unit that causes the flow to vortex is distributed by John Meunier Inc. on the basis of European patents. It has different options which have been marketed under the tradename Hydrobrake or Hydrovex. The final unit similar to the one tested in the field in Skokie (Donahue et al, 1984), the Hanging Trap, was put together in the lab to test the operation of an orifice restrictor in a submerged inlet. There are no independent manufacturers or patents for this device. A municipality may want to construct and install their own or modify those in existing installations.

3.2. Description of the Inlet Control Devices

Each of the units has its own unique design for controlling the flow and providing unobstructed operation. Manufacturers brochures describing their units' characteristics are given in Appendix A. A brief description of each of the four units is given below.

Scepter

The Scepter flow regulator is an orifice that can be placed in the catchbasin leader or mounted on the catchbasin wall so as to cover the leader. The orifice has a diamond shape with a keyhole at the bottom. The purpose of the keyhole is to lower the sump level and keep the upper part of the orifice free of floating debris as well as to induce sediment removal if the catchbasin sump becomes full. For any significant flow, the catchbasin fills up and passes flow through the diamond part of the orifice. A desired level of control can be attained by selecting the size of the diamond-shaped orifice. As shown in Figure 3.3a the size of the upper part of the orifice can be varied while the size of the keyhole remains constant. The head on the orifice is measured from above the line (x-x) Figure 3.3b.

Two versions of the Scepter Inlet Control Device are available, the plug type and the frame type. The 'plug'- type, Figure 3.4, is an injection molded PVC device, slightly tapered for insertion in the catchbasin outlet pipe. The orifice plate is flush with the end of the plug. The plug is held in place by friction and hydrostatic pressure. It can be fitted into leads with 200mm, 250mm or 300mm diameters. When properly inserted the orifice should be almost flush with the catchbasin wall.

The framed device consists of two parts: a catchbasin frame and a plate containing the orifice (Figure 3.5). The frame is installed over the outlet pipe by being bolted to the catchbasin wall.

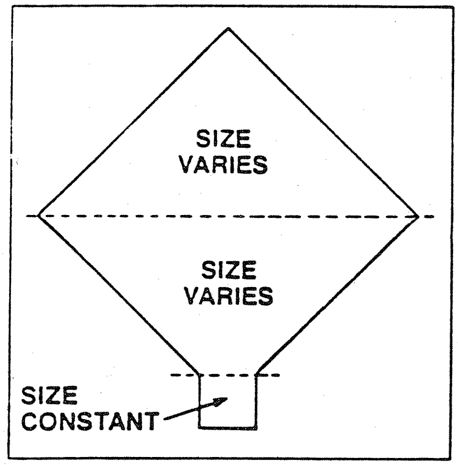


Figure 3.3a

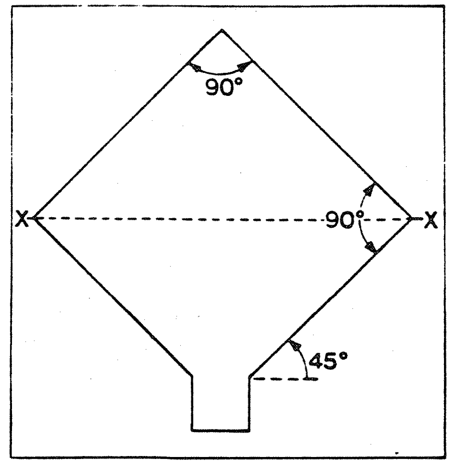


Figure 3.3b

Figure 3.3

Schematics of the Scepter Inlet Control Device

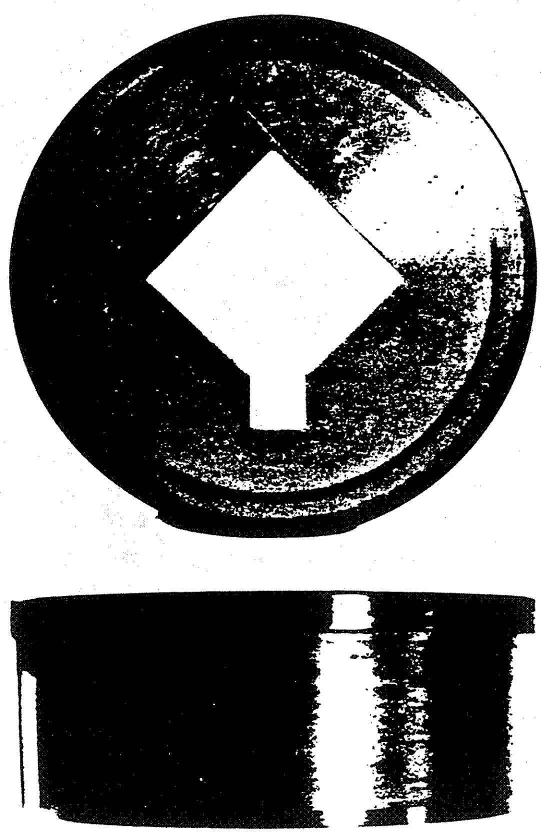


Figure 3.4

Plug Type Scepter ICD

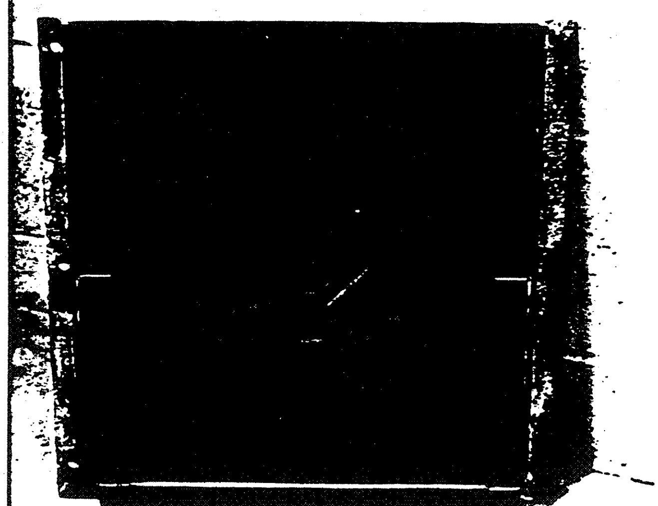


Figure 3.5

Frame Type Scepter ICD

The plate with the orifice is slid into the frame.

Cromac

This orifice device is mounted over the leader pipe inside the catchbasin. It consists of two portions, the first part, the frame, is bolted to the catchbasin wall and encircles the leader pipe. The second part is the orifice which slides into the frame (Figure 3.6 and 3.7). The orifice is made up of 2 PVC sheets. One contains the orifice plate which has a trapezoidal opening 250mm tall, 250mm wide at the bottom reducing to 90mm wide at the top. A weir plate is fastened, with nylon bolts, to the orifice plate in order to control the size of the orifice outlet. The weir plate can be fixed in various positions, allowing the desired control level to be easily obtained or modified. The orifice was made removable, so if the unit becomes clogged it could be lifted out of the catchbasin and cleaned. The plastic components ensure a long lasting rust free installation.

Hydrovex and Hydrobrake

The names Hydrovex and Hydrobrake refer to specially designed units that control the catchbasin outflow. (The term Hydrobrake familiar to many North American engineers is now included under the Hydrovex name.) These terms refer to units which are designed to control discharge by creating a vortex with the incoming flow. When operating in a vortex mode the energy losses are high and the catchbasin discharge can be significantly reduced. The discharge coefficient for the Hydrovex unit can be less than half that of an orifice outlet. In the past these units have been used mainly to control combined sewer overflows and the release rate from detention basins. Their usage in catchbasins to limit the flow to a sewer pipe is a more recent application.

Two different models are available for catchbasins, a German type and a Danish type. The unit is somewhat different than an or

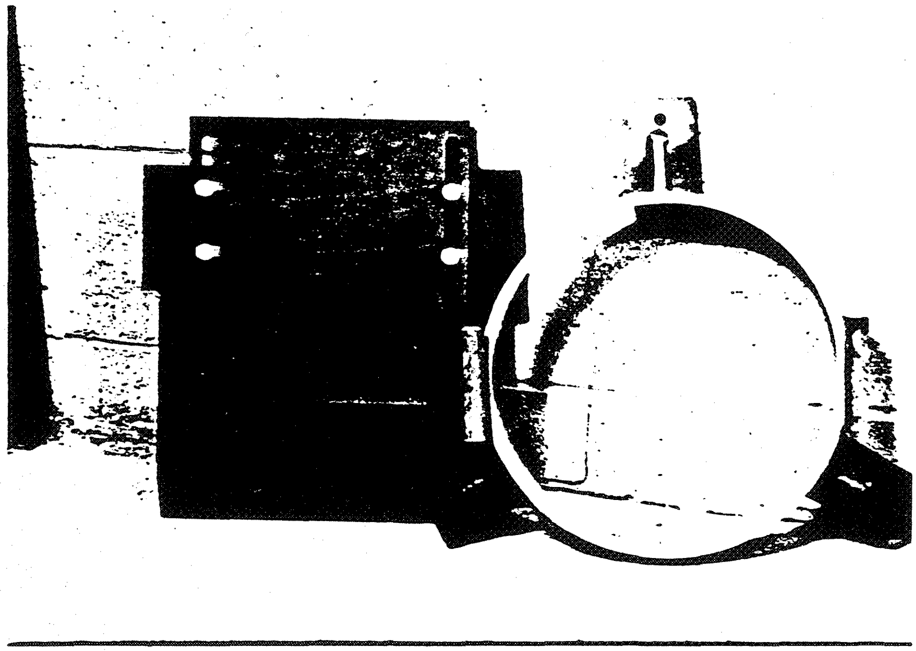


Figure 3.6 Cromac ICD

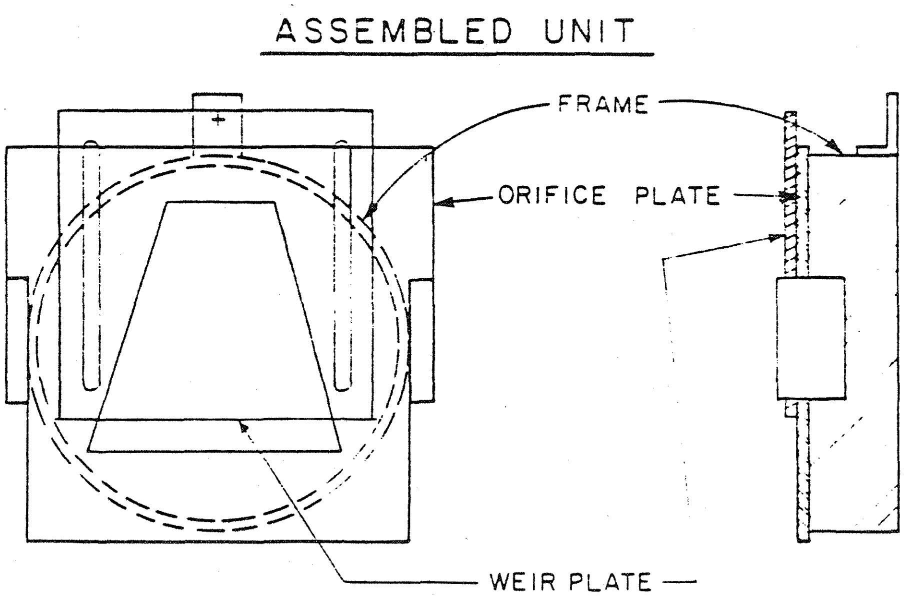


Figure 3.7 Schematic of the Cromac ICD

dinary orifice (Figure 3.8 and 3.9). When installed in a catchbasin it protrudes from the wall 75mm - 200mm and has an outer circumference about 3-4 times the diameter of the outlet. They are constructed of stainless steel to prevent corrosion. The unit's inlet is below the outlet and thus remains submerged at normal catchbasin sump levels. Flow enters the unit perpendicular to the outlet pipe. At low flows the water level in the catchbasin increases and is discharged through the circular outlet as under weir flow conditions. When the water level increases above the Hydrovex unit its operation changes from that of weir flow to a vortex motion (Figure 3.10). The design of the housing is such that when operating under a head, flow entering the unit starts to vortex, that is swirl around in the housing. The circular spinning action of the water in the housing reduces the outflow substantially.

Hanging Trap

Similar devices have been used in older catchbasin installations in many cities. The trap was often used when the catchbasin was connected to the combined sewer system, to prevent gas from escaping to the street. A hanging trap was built for the specific purpose of determining if it would perform well as an inlet control device. The unit consisted of a 150mm diameter ABS 90^{o} elbow which was glued to a support flange (Figure 3.11). A variety of circular orifices were made which could be placed in the inlet of the unit, so that the flow could be controlled to the desired level. Orifices to control the flow to 14.1 lps, 19.8 lps, and 36.8 lps under a 1.22m head were constructed.

3.3 Hydraulic Operation of Catchbasins

Hydraulically, a catchbasin outlet can operate in four different modes, they are: weir flow, orifice flow, short plug flow and pressure flow (Figure 3.12). When the catchbasin has been filled with water weir flow occurs first. The water spills through

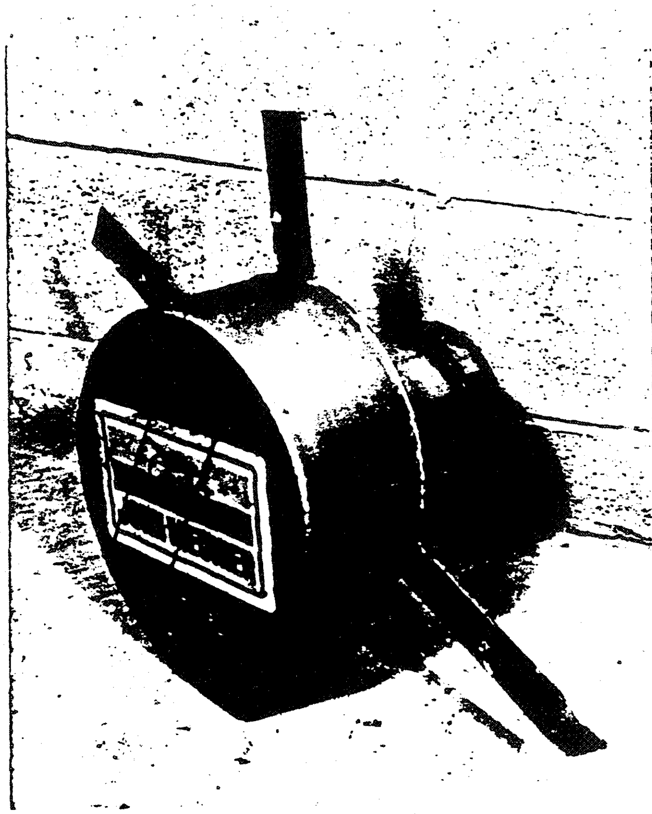


Figure 3.8

Danish Type Hydrovex ICD

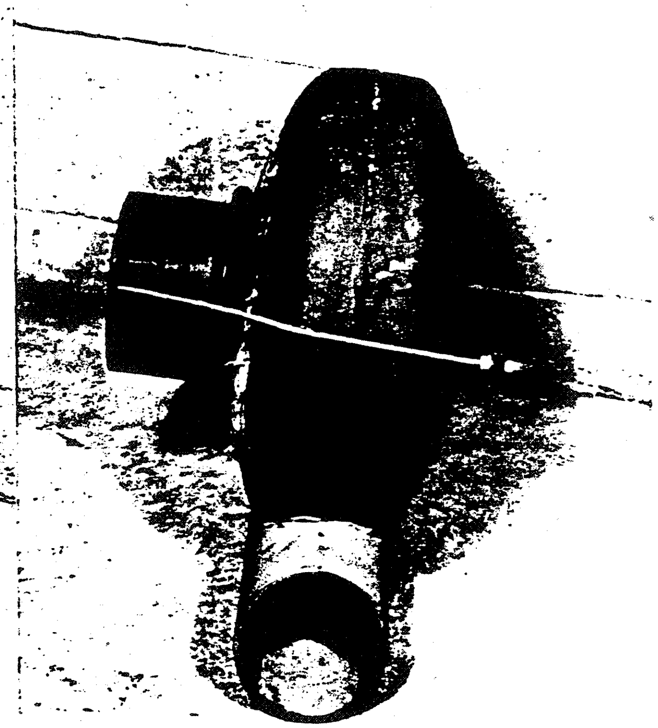


Figure 3.9

German Type Hydrovex ICD

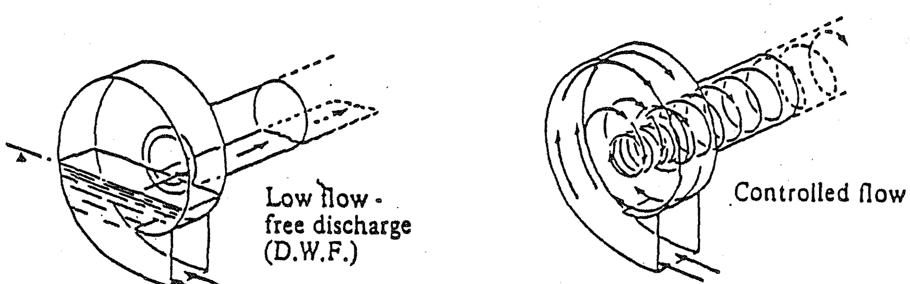


Figure 3.10

Schematic of the Hydrovex Unit in Operation

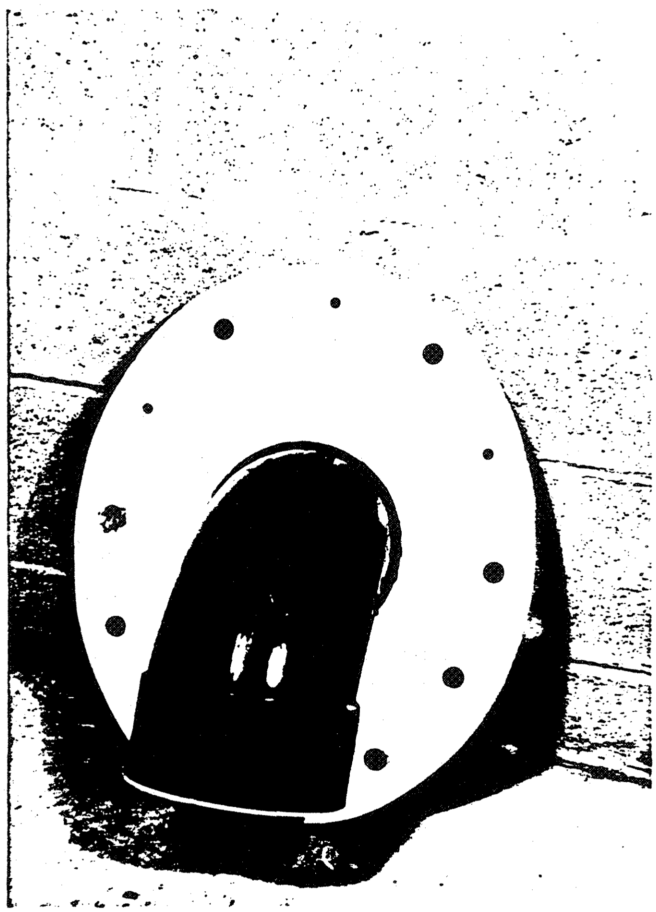


Figure 3.11 Hanging Trap ICD

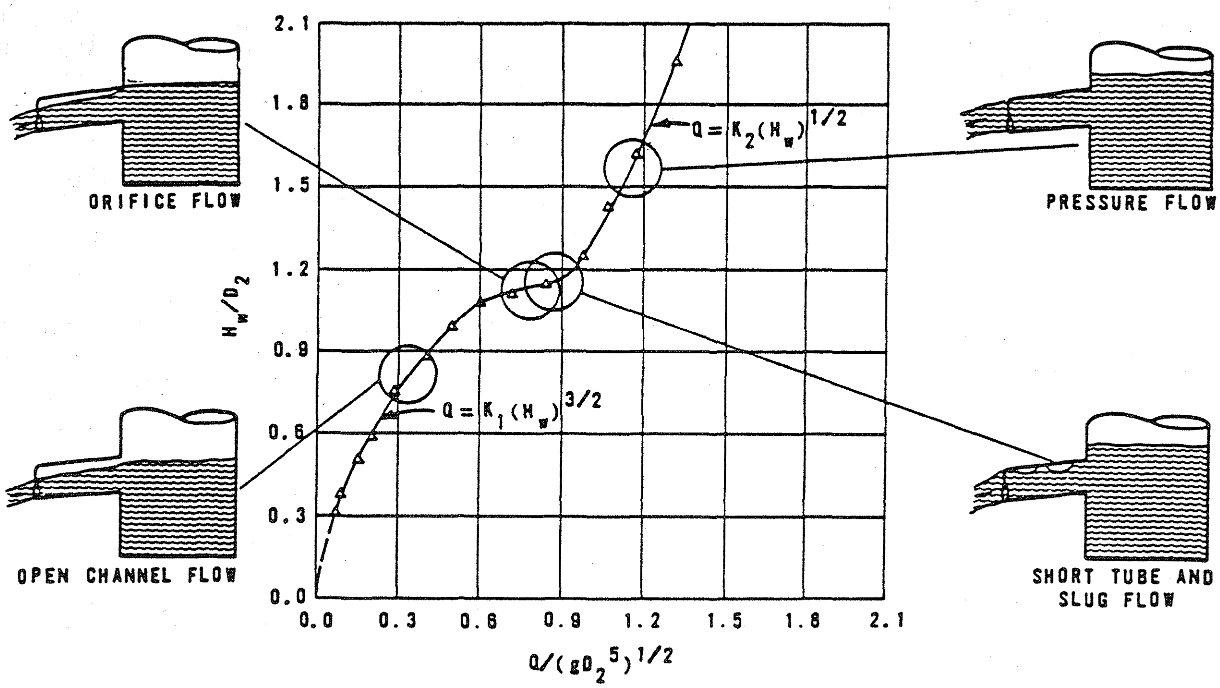


Figure 3.12

Typical Discharge Rating Curve for Four Flow Regimes the outlet, as over a weir, the flow rate is related to the critical depth at the outlet. When the outlet becomes submerged the discharge passes to the orifice regime. The discharge is controlled by the sharp edges of the catchbasin outlet, flow in the downstream channel occurs as completely open channel flow. The orifice discharge is unimpeded by the downstream flow. The third stage is the short tube control flow. This occurs in the transition phase from open channel flow to pressure flow. A drawdown of the water surface near the outlet, can cause pockets of air to be drawn into the outlet pipe. the flow is unstable as neither free surface flow or fully pressurized flow is occurring. The final phase is with the pipe fully pressurized and the outlet pipe flowing completely full. In the first three types of operation the discharge is controlled by the head in the catchbasin. In the last situation it is controlled by the difference of head between the catchbasin and the pipe.

3.4 Hydraulic Operation of the Inlet Control Devices

By installing a flow control device in a catchbasin outlet, the hydraulic operation of the catchbasin is modified. For an adequate design of the inlet control device and an unsurcharged storm sewer system the discharge will always be controlled by the head in the catchbasin. If not the discharge is governed by the difference between the catchbasin head and the head in the pipe. The inlet control device usually limits the flow to less than the leader capacity so free surface flow is maintained in the leader. With the inlet control devices in place, outflow is governed by two regimes. For low flows water spills through the orifice as over a weir. When the orifice becomes submerged the water discharges as a jet. In this case and while free surface flow is maintained in the storm sewer and leader, the discharge may by computed using the orifice equation:

$$Q = C_d A \sqrt{2gh} \qquad (4.1)$$

Q	-	Orifice discharge
C_d	-	Discharge coefficient
A	-	Orifice Area
g	-	Acceleration due to gravity
h	-	Head on the orifice

The measured discharge coefficient can vary. With water depths in the outlet pipe, the shape of the discharging jet also varies with different catchbasin heads and affects the discharge coefficient. Each manufacturer has made hydraulic tests and some provide head vs. discharge relationships for the various types of restrictors and different orifice types.

Hydraulic tests were conducted using the four previously mentioned flow regulators. Each inlet control device was tested at four different flow levels. Throughout the tests each orifice opening is defined by its control flow rate at a head of 1.22m. Head-discharge curves were determined for each of units and compared to those issued by the manufacturer. Manufacturers curves were not available for the Cromac and Scepter devices. The tests were conducted by Novatech Engineering Consultants Inc. on a catchbasin model located in the hydraulic lab at the University of Ottawa. The first set of tests was conducted to establish the hydraulic behaviour and observe the discharging flow. This was followed by tests examining the clogging susceptibility of the various units, which is discussed in Chapter 4. The manufacturers of the different units provided Novatech with prototypes which could be mounted in the catchbasin model. Representatives from Cromac and John Meunier were able to participate in the testing of their units. In addition demonstrations were made to a group of municipal engineers and to officials from C.M.H.C.

Scepter

Four different orifice openings with the Scepter unit were tested. The orifices were designed to discharge at rates of 14.1, 19.8, 28.3 and 36.8 lps for a head of 1.22m. Each orifice was tested by placing it in the catchbasin outlet, then varying the flowrate through the catchbasin. The flowrates were adjusted so that the head would increase at fixed intervals of approximately 0.15m. All measurements were made with the catchbasin head above the centre line of the orifice. The jet discharging from the orifice was diamond shaped being modified by the discharge through the keyhole (Figure 3.13). The discharge traced a quasi-parabolic shape, until hitting the bottom of the outlet pipe.

The head-discharge curves measured in this experiment were compared with those obtained from the Scepter brochure. The curves given by the manufacturer were very close to the measured flows (Figure 3.14).

Cromac

Hydraulic testing of the Cromac units showed that they were able to control the discharge to the desired level. Orifice opening to give flows of 14.1, 19.8, 28.3 and 36.8lps at a 1.22m head were used. The Cromac orifice is wide (200mm), thus to control the flow at low levels (14.1 lps), the orifice opening is narrow (23mm). For small openings the discharging jet is deflected upward, then arcs back to the bottom of the leader pipe. At larger openings (40mm) the outlet jet traced a quasi-parabolic shape.

Head discharge curves were not provided in the manufacturers brochure for this device. The curves shown in Figure 3.15 were obtained from laboratory measurements. Discharge coefficients computed from the lab measurements were approximately the same as those computed for the Scepter unit.

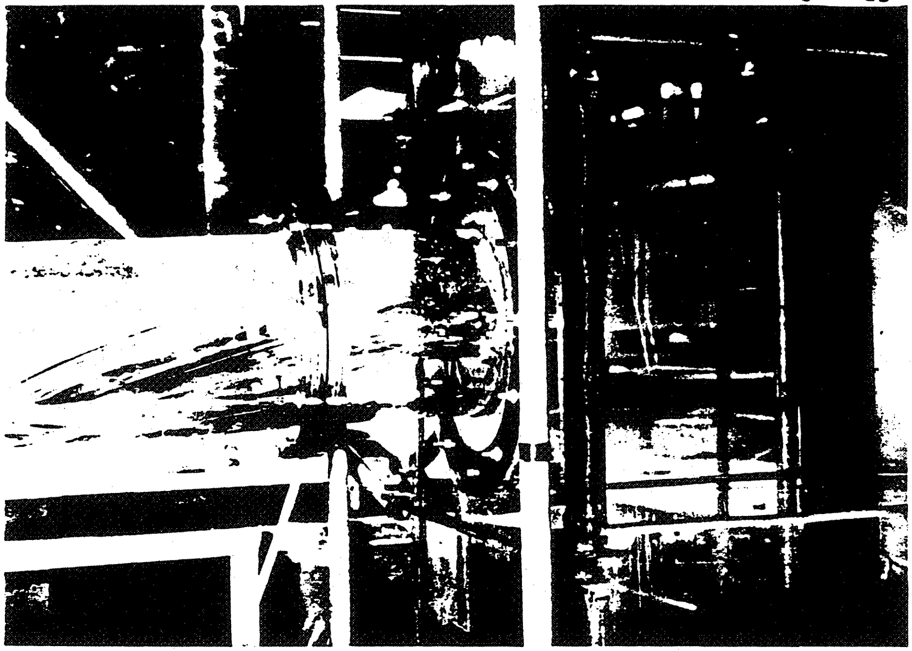


Figure 3.13 Discharge from the Scepter ICD

Figure 3.14 Head Discharge Curves - Scepter Orifice

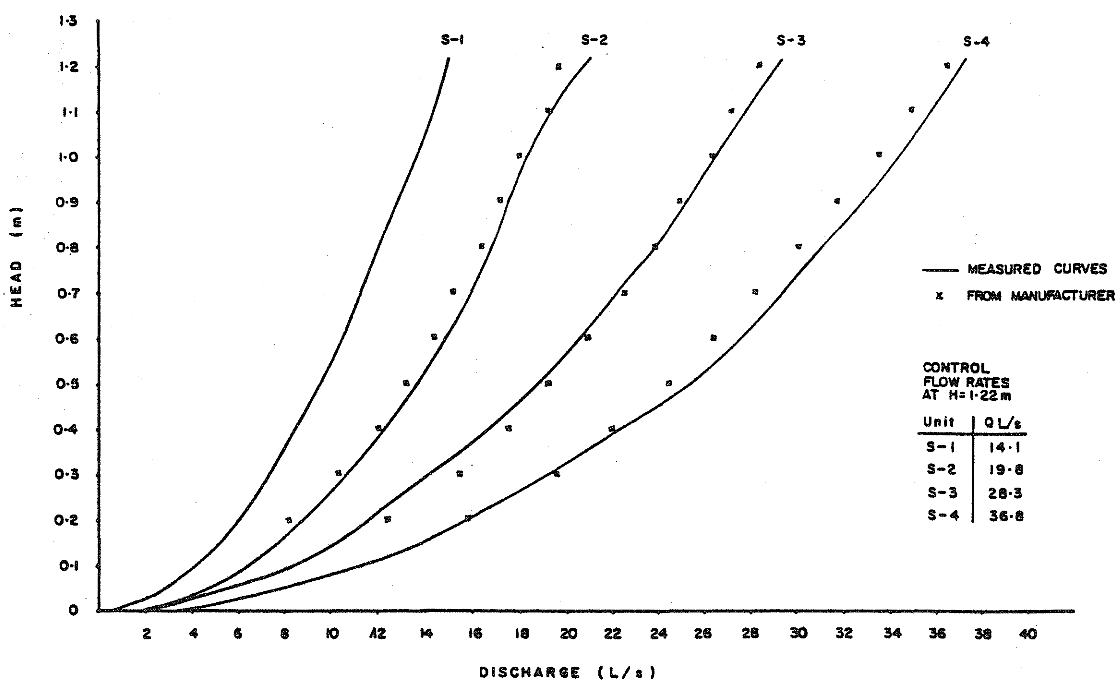
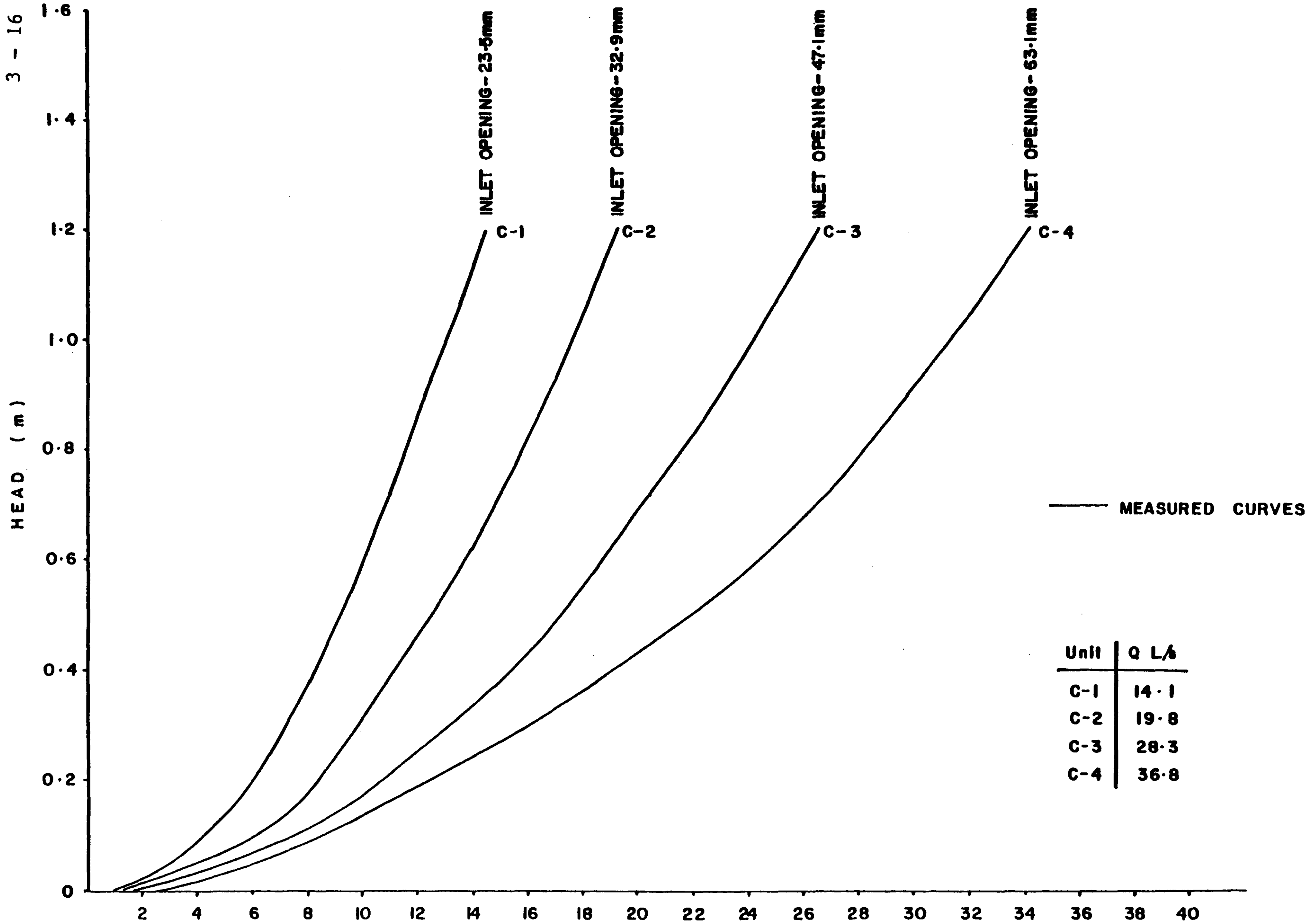


Figure 3.15 Head Discharge Curves — Cromac Orifice



<u>**Hydrovex**</u>

The manufacturer of the Hydrovex units design each one to customer specifications. Four units were loaned to Novatech Engineering for testing purposes. Two were of the German type, models G-1 and G-2 and two of the Danish type models D-1 and D-2. These units provided a much lower level of control than the orifice devices. The smallest unit tested provided control to a level of 3 lps, the largest to 19 lps under a head of 1.2m.

While the water level is below the top of the unit, water exits as weir flow. When above the top of the unit the vortex action starts and the water exits in swirling motion. The vortex motion of the flow resulted in a longer time being required for the units to reach a steady flow condition. During the transition period when the outflow changes from weir flow to vortex flow, the discharge is unsteady and the head in the catchbasin increases quite rapidly. When the unit starts to vortex there is a drop in outflow rate yet the inflow remains constant. Consequently to reach equilibrium, the inflow being equivalent to the outflow, the head in the catchbasin increases. For heads between 10 cm and 30cm the transition from weir to vortex outflow occurs. It is particularly evident with model D-2 (Figure 3.16) where the transition is quite abrupt, the discharge remains almost constant between heads of 10cm and 30cm.

A comparison of the measured curves and those provided by the manufacturer shows that for three types they are very similar (Figure 3.17). In the case of the Danish models the measured flows were less than those given by the manufacturer by a maximum of 13%. This difference is not of practical significance and may be caused by different set-up. The outlet areas with the Hydrovex unit were similar in size to the previously tested orifice areas. This shows that the vortex action causes a significant head loss. The discharge coefficient for the Hydrovex unit is about 1/3 that of the orifices. At low flow when the unit discharges as a weir, the coefficients were about the same as for other restrictors. With high

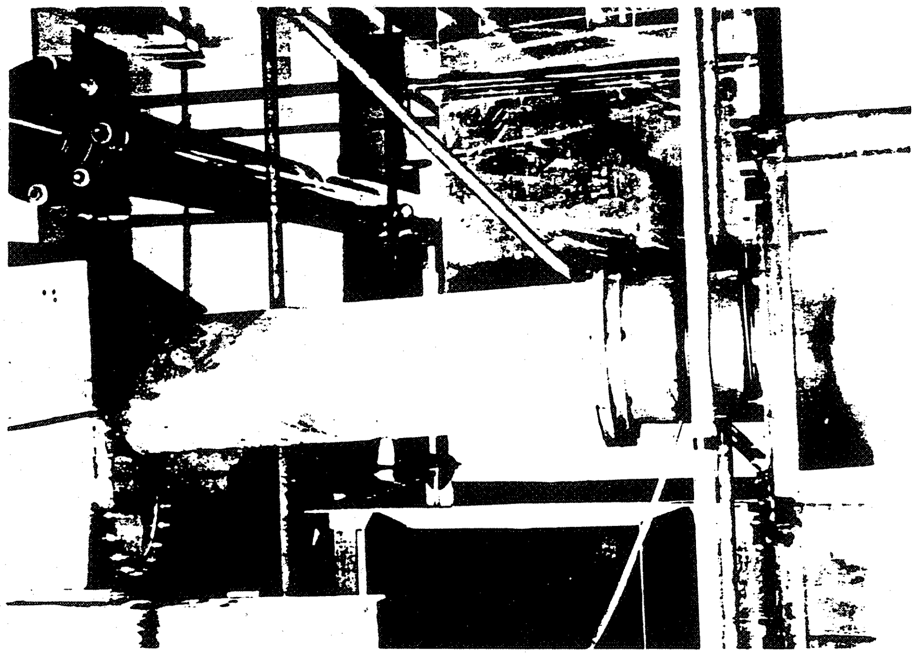
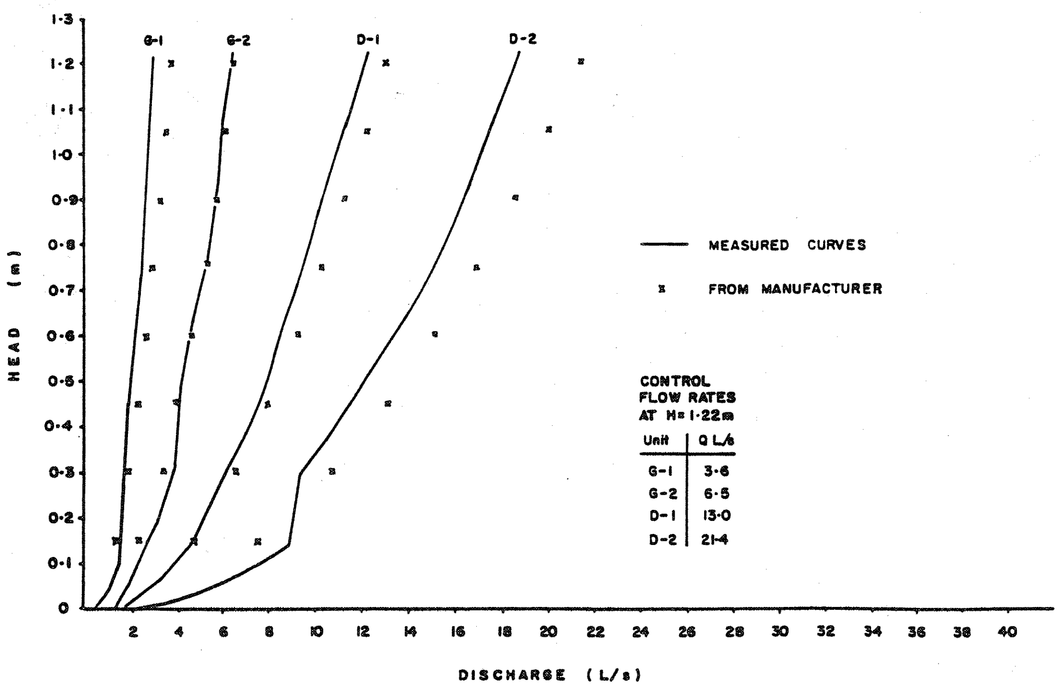


Figure 3.16 Discharge from the Hydrovex ICD

Figure 3.17 Head Discharge Curves — Hydrovex Flow Regulators



flows and the vortex motion, the discharge coefficients dropped dramatically.

Hanging Trap

The hanging trap device was tested with orifice openings that gave four different flowrates (14.1, 19.8, 28.3, 36.8 lps) at a head of 1.2m. The discharge from the hanging trap was different than that from a normal orifice. Instead of being an orifice jet, the water discharged as tube flow (Figure 2.18).

The measured head-discharge curves are given in Figure 3.19. The curve H-1, H-2 and H-3 are the same shape as the Cromac and Scepter head discharge curves. The H-4 curves is slightly modified between flowrates of 25 lps and 38 lps. This occurred because the flow in the leader pipe became surcharged. This reduced the difference in head between the catchbasin and the outlet, consequently the flow was reduced.

A measure of the variation of the discharge coefficients is provided in Figure 3.20. The Scepter, Cromac and Hanging Trap units have almost identical coefficients. The Scepter being slightly lower than the other two. However the discharge coefficient for the Hydrovex unit is very much lower. The shape of the curve also indicates the transition from weir flow to vortex flow.

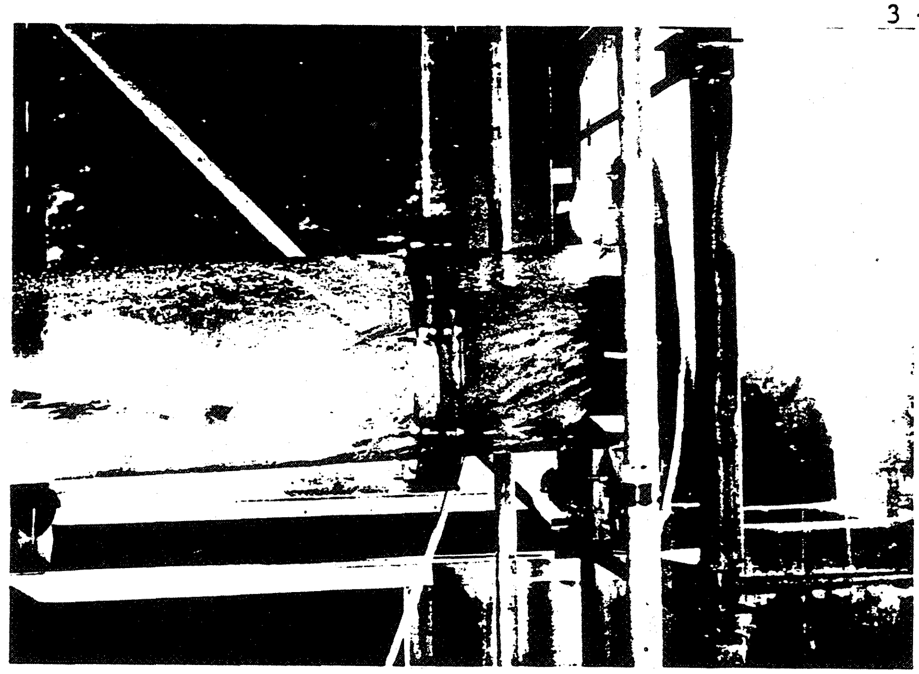


Figure 3.18 Discharge from the Hanging Trap ICD

Figure 3.19 Head Discharge Curves — Hanging Trap ICD

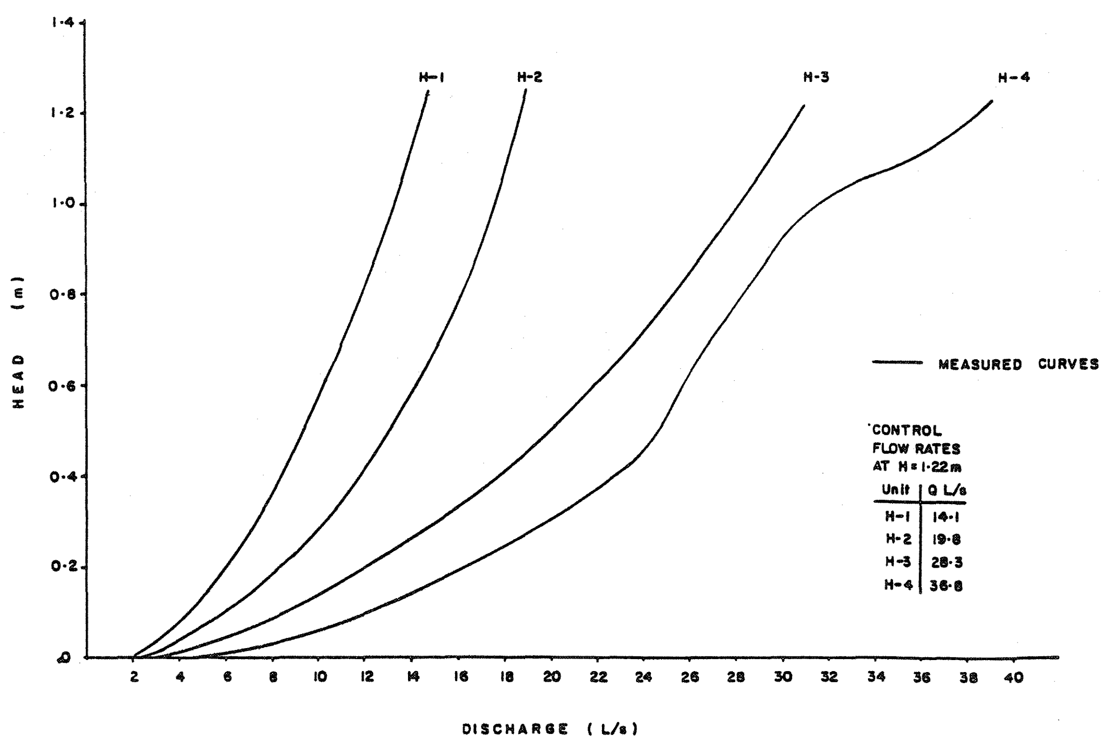
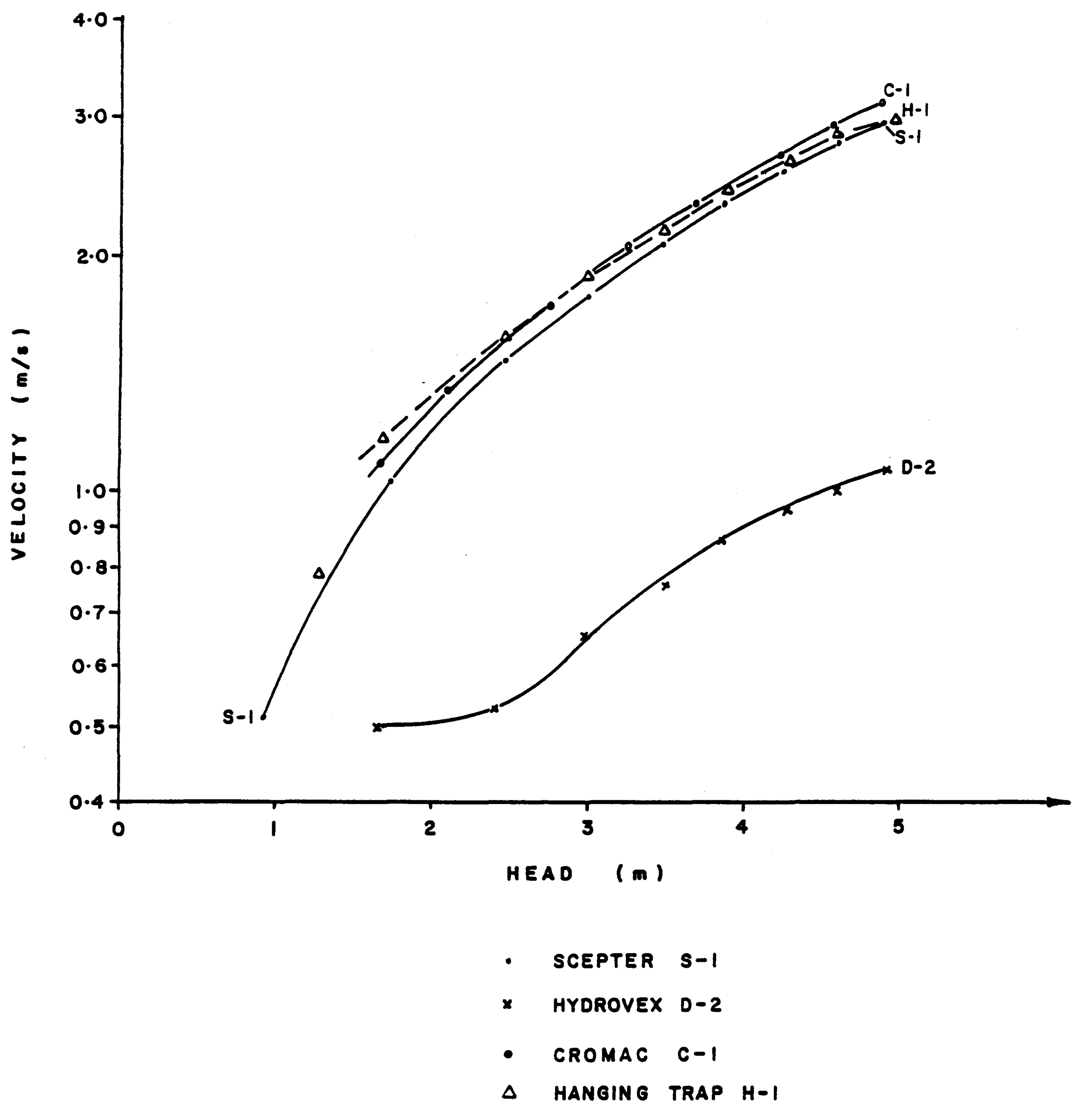


Figure 3.20 Comparison of Discharge Characteristics



Chapter 4
INVESTIGATION OF CLOGGING FOR VARIOUS INLET RESTRICTORS

4.1 Previous Studies

An examination of inlet clogging has been conducted by a number of researchers. Larson (1949) has examined clogging that may occur in a catchbasin grating. He used leaves and bits of paper in examining the ability of a grating to pass debris. It was found that a grating's self cleaning ability depends on its roughness and configuration. Gratings with longitudinal openings had evident self cleaning ability and were able to pass 90% of the debris. Gratings with transverse openings clogged more easily and were found to pass from 30% - 50% of the paper and leaves. The roughness of the grate bars and rounding of the bars also contributed to their self cleaning ability. Clogging of gratings in general is undesirable in that the frequency of street flooding may increase.

Testing at both inlet gratings and different flow restrictors has been conducted by Townsend and Moss (1980). Flow control was examined using catchbasin grating restrictions, an intermediate horizontal orifice, a hooded inlet and a flush mounted orifice plate. The catchbasin grating restriction was found to be more susceptible to clogging than the unrestricted grating. If the restriction was obtained by welding a plate to the bottom of the grating, then slots above it filled up with debris. Also debris collecting over the remaining inlet area caused the flow to bypass the catchbasin. Concerns that after installation of a plate it may be removed by citizens or maintenance crews not aware of its purpose as a flow regulator, has also prevented the recommendation of this solution.

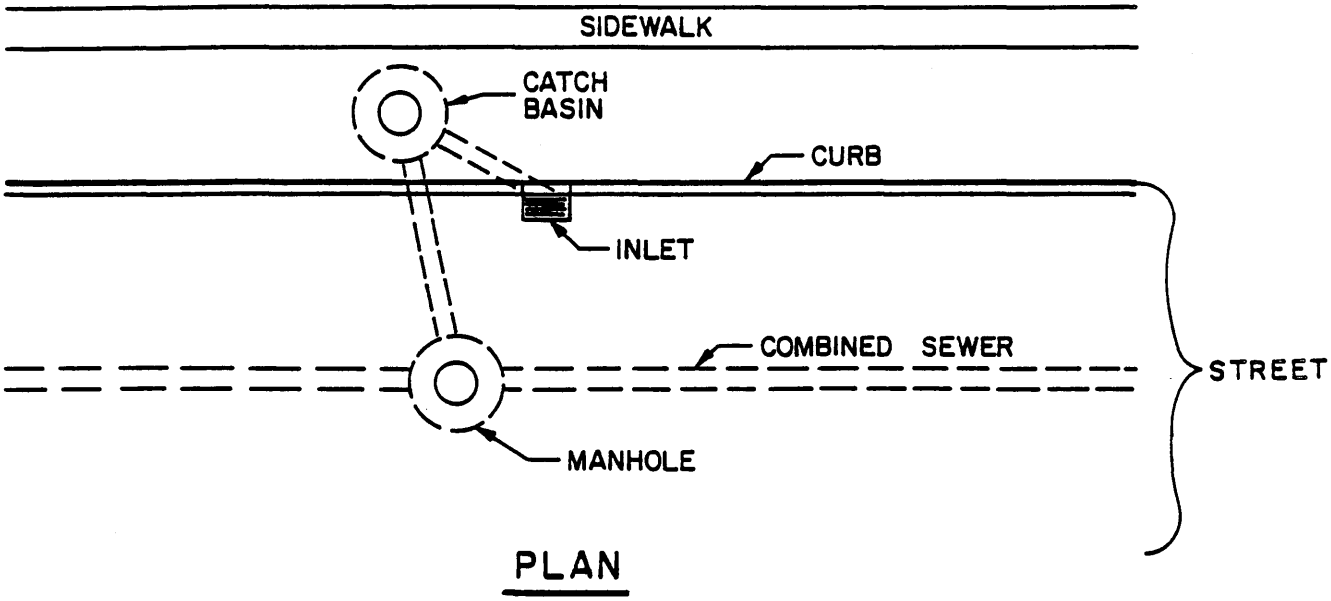
Flow control using an intermediate horizontal orifice has been examined, with some installations being made in parking lots in

Nepean, Ontario. Field inspections have shown that no clogging occurred from the gritty runoff characteristic of parking lots (Corey, 1985). Ice build-up problems were not experienced in winter and early spring operation. The city of Ottawa tested several of this type of restrictor in catchbasins located along tree-lined streets and found that significant clogging problems were caused by twigs and leaves. The units had to be removed due to inadequate operation.

Another device examined by Townsend and Moss (1980) was a hooded inlet. A hood placed over the catchbasin outlet restricted the inflow area and acted as a flow regulator. This device was found to be susceptible to clogging. Leaves and other debris became trapped under the hood and were not cleared away.

Testing of the fourth device, the flush mounted orifice plate, proved to be the least susceptible to clogging. It consisted of an upper part diamond-shaped zone and a lower keyhole that exhibited good self-cleaning action when the flows contained a large detritus component. The lower "keyhole" feature permitted drawdown of the water level in the catchbasin, below the main orifice area, during dry periods. Moreover a recent study (Townsend, 1984) has demonstrated its effectiveness in keeping the orifice clear of possible grit buildup in the catchbasin sump area. During normal operation, the high velocities in the keyhole zone entrain grit that might otherwise accumulate next to the orifice in the sump area.

Donohue (1984) has evaluated five different devices for regulating stormwater flow. The emphasis in the study was on field performance, a number of units were purchased and installed in a catchment within the village of Skokie, Illinois. The five flow regulators were the Hydrovex, Scepter, Catchbasin grating restriction, Horizontal Orifice and the Hanging Trap device. Catchbasins in Skokie are somewhat different than those found in typical Canadian practice. Street flow enters an inlet at the curb, a pipe directs the flow to a catchbasin which is located between the sidewalk and street curb (Figure 4.1). It is possible that more



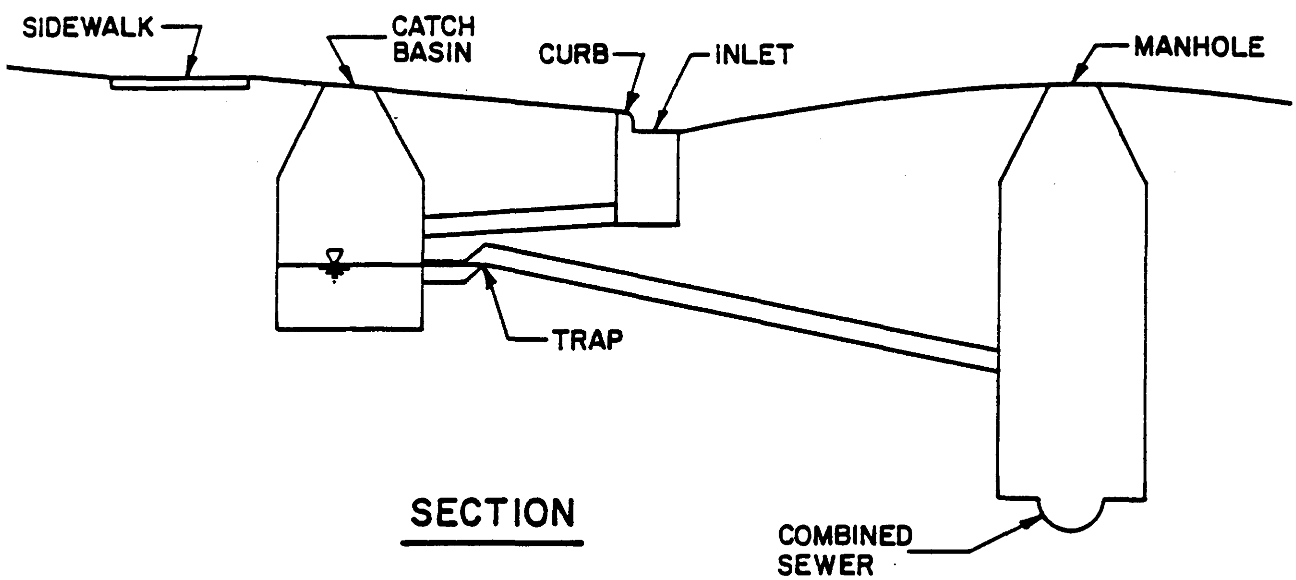


FIGURE 4.1
TYPICAL STREET DRAINAGE
SYSTEM CONFIGURATION
SKOKIE, ILLINOIS
(ref. Donohue, 1984)

than one inlet may be connected to a catchbasin. The catchbasin grating restriction and horizontal orifice regulators were applied to the inlet, the other regulators were placed in the catchbasin. The installed regulators, 29 in total, were observed during or after 15 rainfall evens, within a five month summer period in 1983. The study used flow regulators which limited the outflow between 3.4 lps/C.B. and 22 lps/C.B. most varied between 5.6 and 8.5 lps/C.B. This is a much lower level than commonly used for separated storm sewers and dual drainage in Canada where 28.3 lps/C.B. is typical. With a higher flow level the incidence of clogging may be expected to decrease.

The observations showed that clogging of the devices was mainly caused by leaves. Other debris that caused clogging was grass, mud, paper and twigs. The findings demonstrated the catchbasin grating restriction and horizontal orifice regulators did not perform acceptably. The Hydrovex, Scepter and Hanging Trap regulators had few cloggings and they found that these units performed equally well.

Previous results from studies by Donahue (1984) and Townsend and Moss (1980) are therefore in agreement regarding unsatisfactory performance of reduced catchbasin grating areas and the horizontal orifice plate regulators. The devices that performed well, namely the Hydrovex, Scepter and Hanging Trap regulators were therefore the only ones reexamined in detail in this study.

4.2 Laboratory Investigation

In contrast to the Donohue tests the present investigation was conducted in the laboratory. A field study does not permit visual observation of a unit's performance, or of the clogging mechanism. Also, the flow rates into the catchbasin and the loadings are unknown.

By conducting experiments in the lab with a plexiglass catchbasin the operation of the unit can be observed by researchers and visitors, the flowrates can be set at desired levels, the clogging behaviour can be observed and the debris loadings can be varied. By testing in the field where flow rates are an unknown one cannot determine if clogging occurs at high flows or low flows. In a lab study it is difficult to quantify loadings, in the field it is almost impossible. The amount of debris washed into a catchbasin is extremely variable, both in time and space. It depends on many things such as: the type, size, density, and location of trees; the frequency of grass cutting, neighbourhood cleanliness and the frequency of street cleaning. In the Donohue study (1984) no indication is given about the amount of debris found in the catchbasins. Discussions with municipalities such as Markham, Scarborough or Laval indicate only that in general restrictors did not create any maintenance problems. Municipalities seemed reluctant to conduct systematic tests or inspections on their own.

4.2.1 Procedure

The testing of four flow regulators as described in Chapter 3 was done using a full-scale hydraulic model of a street segment and catchbasin. The experimental setup is described in Appendix B. The flow regulators were mounted in the catchbasin outlet. In preliminary tests the flow was introduced through a pipe at the bottom of the catchbasin sump. Next flow was pumped up to the street surface, it was directed over the street, passing through the catchbasin grating into the catchbasin.

The tests used a severe debris loading which was manually introduced into the catchbasin. The floatable debris consisted of leaves, small sticks ranging in size from 100 mm, pieces of hockey sticks, styrofoam cups, bits of cloth and plastic. As Donohue (1984) found that leaves were the principle cause of regulators being plugged, initial tests only involved the introduction of leaves. As testing progressed other items of debris were added. It was observed that over a period of time the leaves became water logged and were suspended throughout the sump rather than floating on the surface.

After putting the debris in the catchbasin sump water was released into the catchbasin at a slow rate so the flow regulator would operate as an overflow weir. The leaf mass would rise and move toward the orifice outlet. With increased flow the orifice at times became obstructed, at other times free flow was maintained throughout the test period. If the orifice became obstructed then the head was increased to see if self cleaning could be induced. If clearing did not occur with a four foot head flow was stopped the water level returned to the normal sump level. A record was kept of the different tests which were repeated a number of times.

The Hydrovex and Hanging Trap units were somewhat different in that the inlets to the units were submerged. As the leaf mass rose it was observed that leaves were not drawn into the unit as no outflow was occurring. When flow through the unit did start the floating mass was usually above the inlet and debris did not enter the unit.

When the flow passed over the model street surface and through the catchbasin grating there was significant turbulence in the catchbasin sump. The increased turbulence did not help to keep the orifice clear, rather it contributed to clogging. Without the turbulence floating sticks and other debris had remained on the surface, the plunging jet increased the probability of them becoming trapped across or in the orifice. This occurred to some extent in all of the units. Consequently self-cleaning potential or operation under partial clogging is essential. To provide a more complete record of the operation of each unit some of the laboratory tests were recorded on video tape.

4.2.2 Scepter

The tests were conducted first using a loading of leaves and then with leaves, sticks and other debris in the sump. A typical test with leaves is shown in Figure 4.2. Initially the floating leaf mass partially reduces the flow. As the water level increased debris rose and left the orifice clear. A thick leaf mass was able to clog the smallest orifice (14.1 lps) tested. The clogging started with leaves being partially jammed in the orifice at low flow rates. As the head increased they would be held in place by hydrostatic pressure if the mass was sufficiently dense. <u>All of the leaf clogs eventually washed out at heads that varied from 0.3 to 1.2m (Figure 4.3)</u>. The larger orifice openings did not clog with a leaf mass alone. The tests showed that 14 lps is a satisfactory lower limit for this unit.

With sticks and other types of debris in the catchbasin, the units had a lower level of performance. Sticks that became trapped across the orifice or partway through the orifice were unable to free themselves. The shortest stick was 100mm with an equivalent diameter of 5mm, while the largest was 450mm having an equivalent diameter of 10mm. A couple of sticks with an equivalent diameter of 25mm and a length of 300mm were also tested. When a stick lodged across the orifice, leaves tended to collect around it partly clogging the opening. As the water level dropped two different scenarios were observed. In some cases the mass of debris would slump into the catchbasin sump leaving the orifice opening clear. At other times the leaves and sticks were observed to remain in the orifice leaving it clogged, even when the water level was below the keyhole. With this situation self-cleaning of the unit does not occur, manual cleaning was required to dislodge the debris.

Each of the two types of Scepter ICD's has its own advantages. The 'plug'-type unit is relatively easy to install and remove. The short pipe-stub section includes a mild taper to allow for minor anomalies in the catchbasin lead pipe dimension. It can be installed or removed by a single operator and is held in place by friction and hydrostatic pressure, no bolts are required. The 'frame'-type unit has the frame bolted to the catchbasin wall, the

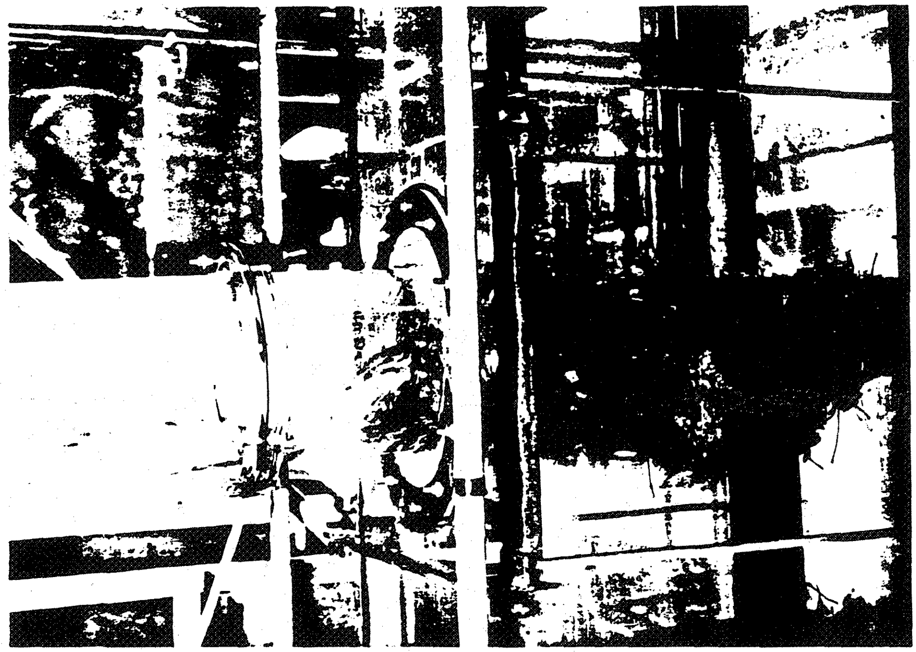


Figure 4.2 Flow Reduction caused by Floating Leaves

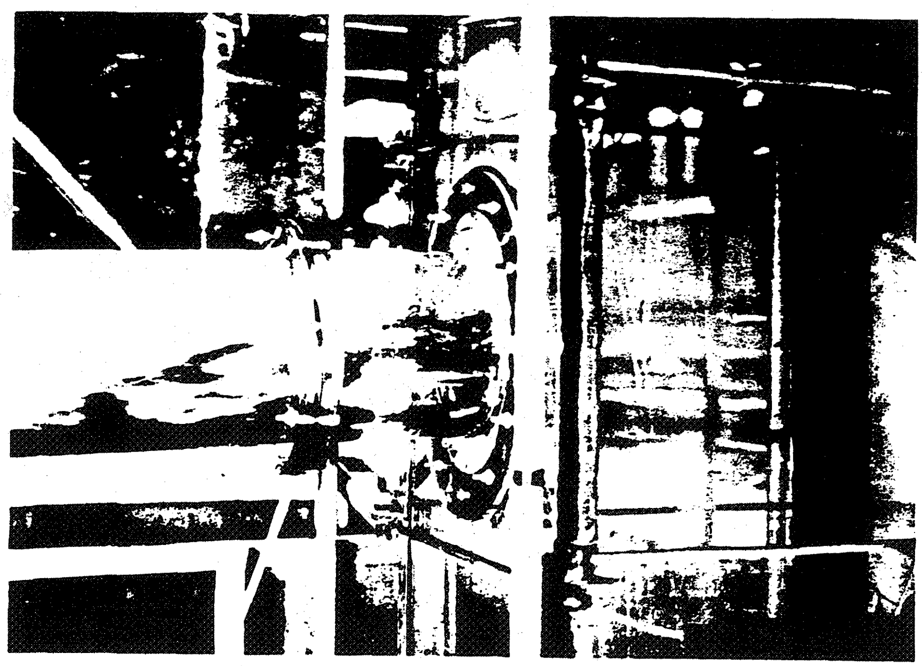


Figure 4.3 Cleaning of a Leaf Jam from the Scepter Unit

orifice plate being slid into the frame. The frame type unit was not tested in the model catchbasin. Minor leakage between the frame and the catchbasin wall is expected, but would not significantly alter the discharge in the leader pipe. If the orifice in the 'frame'-type unit is clogged, the orifice plate can be removed and cleaned without entering the catchbasin. With both of the units, the orifice opening is visible, inspection is therefore a relatively simple procedure.

In Scarborough it has been found (Cromas, 1985) that the Scepter plug type unit may not fit completely into the catchbasin lead if the lead enters the catchbasin at an angle. In this case a gap may be left at the bottom between the unit and the catchbasin wall. If the unit is not installed flush with the catchbasin wall then breakage may occur during catchbasin cleaning. The 'frame'-type unit should be used when the 'plug'-type cannot be properly installed. Discussions with other municipalities (Keliar, 1984) did not reveal similar problems although a very large number of units have been installed. Scepter indicated that their units have been widely used without complaints regarding the installation.

4.2.3 Cromac

The lab tests conducted with this device indicated that clogging will cause partial restriction of the flow. It was found that the unit plugged frequently at a flow rate of 14.1 lps. At this flow level the orifice opening is quite narrow, leaves and sticks blocked the opening quite easily and were not dislodged with an increased head. As the head on the orifice increased the velocity also increased, in most cases the hydrostatic pressure and larger velocity were not sufficient to wash away the blockage. Figure 4.4 shows the partially blocked orifice from the upstream side. Stopping the flow returned the water surface to the normal sump level, this did not dislodge any trapped leaves or sticks. For flows of 28.3 lps and 36.8 lps, the flow through depth is greater. The per

Figure 4.4 Partially Clogged Cromac ICD

Figure 4.5 Hydrovex ICD with Debris in the Inlet

formance is satisfactory as the plugging frequency is greatly reduced.

Throughout the testing this device was found to be somewhat more susceptible to clogging than the other units. This was primarily due to protrusions which provide a place for debris to become attached. Protrusions from the frame as well as the slightly protruding weir plate were the surfaces that helped to intercept floating debris. The narrow opening at low flows increases the risk of clogging.

Although this unit was more susceptible to clogging than the Scepter unit at low flows, both were comparable when using other criteria. From an installation and maintenance viewpoint the unit is good; it can be installed quickly and easily, the orifice opening is visible therefore clogging can be checked, the orifice plate can be easily lifted out of the frame if the unit needs cleaning or other maintenance. When installed the unit will protrude slightly more than the Scepter device from the catchbasin wall, being approximately 25mm, therefore interference with catchbasin cleaning should be minimal. Finally this device can be adjusted to obtain any desired outflow rate, all the other units require at least replacement of the orifice plate if not the whole unit.

4.2.4 Hydrovex

As indicated in Chapter 3 four different Hydrovex units were tested, two of the German design and two of the Danish. These units provided a much lower level of flow control than the Scepter or Cromac orifices. The units tested restricted the flow to between 3 lps and 20 lps for a head of 1.2m. The German type design has its inlet in a vertical position below the outlet pipe. The purpose of the submerged inlet is to prevent floating debris from getting trapped in the inlet. The tests showed that the water spilling from the street surface stirred up the debris, causing some of it to be washed through the unit. The location of the vertical inlet made it very difficult for sticks to become lodged across or in it. When the level in the sump was lowered below the inlet and then refilled clogging did not occur. As the sump level rose, debris floated by the inlet since no discharge was occurring to draw the debris into the unit. The unit demonstrated excellent self cleaning ability when sticks and debris were introduced into the inlet. In one instance some sticks inserted into the unit caused leaves entering it to collect. The debris in the unit caused the vortexing action to be delayed. At a head of 0.75 m the vortex action started and resulted in all of the debris being washed out of the unit.

The Danish type was found to be more susceptible to clogging under some conditions. The inlet of this unit is on a vertical angle at the bottom of the unit, as a result it is easier for debris to become trapped when the sump level rose after being below the unit. The debris on the water surface was carried into the inlet. In the second case, the plunging effect due to water falling from the street level caused sticks and leaves to be washed into the inlet. Partial clogging of the inlet occurred but plugging inside the unit was never observed. The flow pattern was able to wash out all debris that entered the unit.

Part of the reason for these units low susceptibility to clogging is that they have a low discharge rate and a relatively large inlet area. The inlet velocity is therefore lower than for any of the other units, thus tending to draw a smaller amount of material into the unit. Besides having a low susceptibility to clogging the Hydrovex system was able to give much lower levels of control than the other units. It was also found that they could be installed quite easily.

Some of the disadvantages of the unit are related to maintenance concerns. Since the unit inlets are submerged it is very difficult to observe if they are clogged. The catchbasin would have to be cleaned out before the unit could be checked. A concern raised by municipal engineers was freezing susceptibility. In

Canada's cold climate it is not uncommon for water in catchbasins to freeze. If this happened with a Hydrovex unit, the outlet is effectively blocked. A sudden January thaw or early spring meltwater could cause street flooding if the ice did not thaw quickly. Further research is required to determine where and when this phenomena occurs and if it would affect the operation of the Hydrovex inlet control device.

4.2.5 Hanging Trap

The performance of this unit in the lab was greatly influenced by the method of introducing water into the catchbasin. The first set of tests were run by introducing the water from the bottom of the catchbasin. In this way the water level rose with little turbulence on the surface. In these conditions the unit performed well as floating leaves and sticks remained on the surface and did not get trapped in the submerged orifice. The only way to cause clogging was to lower the sump level below the orifice or to manually place sticks across the inlet. This test is similar to the way the units operated in Skokie, Illinois, as described by Donohue (1984).

Most catchbasins in Canada also form the inlet, as such there is significant turbulence in the catchbasin. When water was released from the model street surface the plunging effect caused leaves and sticks to be forced under the surface. As a consequence they were drawn into the unit or lodged across the inlet orifice. The device clogged much more frequently when operated this way.

Leaves alone did not cause this unit to clog. Both sticks and leaves were required to block the orifice. Orifice controlling the flow to as low as 14.1 lps for a head of 1.22m may be installed.

Although this unit which can be constructed and installed by the municipality may be cheaper it is not recommended since if it becomes clogged with sticks and leaves, visual inspection is difficult. When installed in an inlet catchbasin it interferes with catchbasin cleaning by protruding about 200mm, if frozen it would block the catchbasin outlet. This conclusion is different from that found in the Donahue study, but the flow access conditions are not the same (see Figure 4.1).

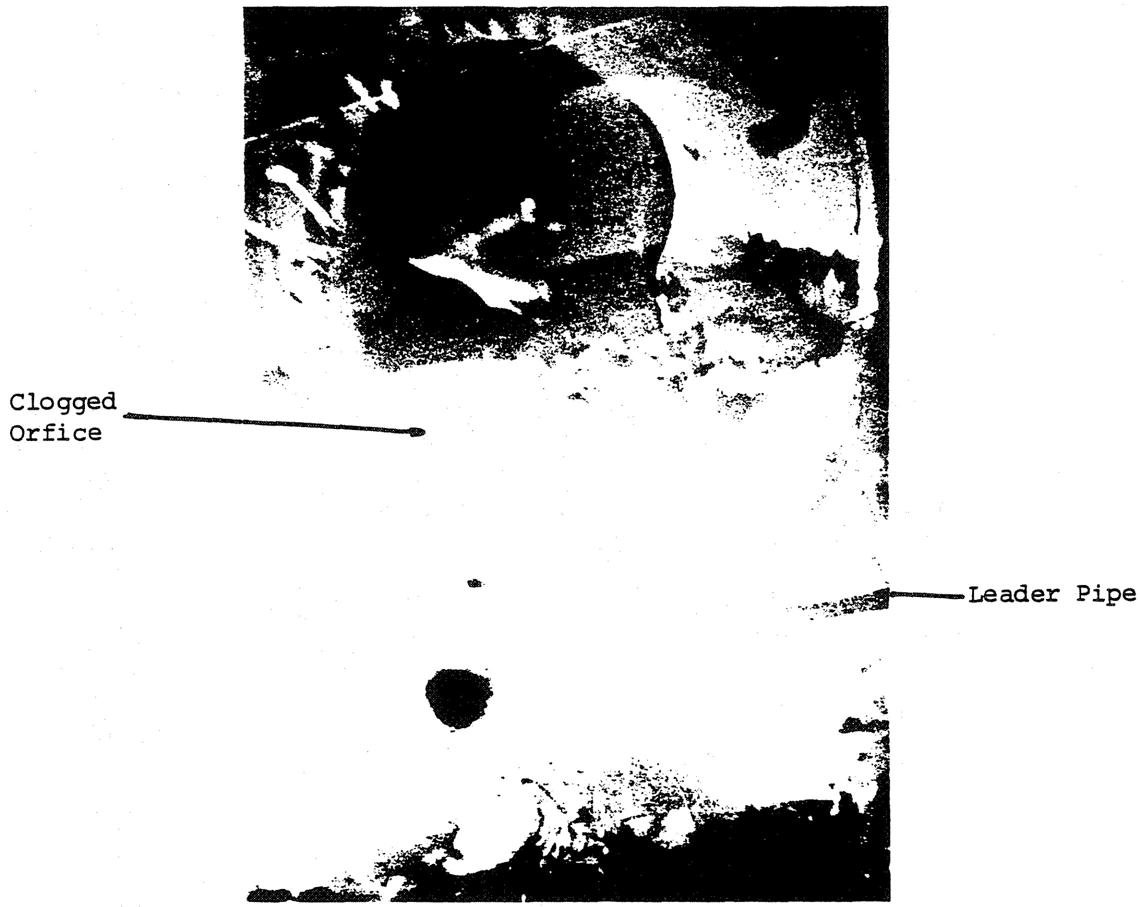


Figure 4.6 Partially Clogged Hanging Trap ICD

(View from the Bottom of the Catchbasin)

Chapter 5

SIMPLIFIED DUAL DRAINAGE COMPUTATION (Dual Checkhyd)

5.1 Introduction

The design of dual drainage systems is of primary importance if inlet controls are to be used properly. A wrong use of inlet controls may have adverse consequences. For example, if they were applied on an area experiencing surcharge problems without consideration for the surrounding system, the surcharging may only be moved from one site to another, or superficial flooding may occur rather an sewer surcharging. Two methods of designing dual drainage systems are available. The first is a simplified procedure developed in this study that does not require modelling expertise and can be used for determining the proportion of flow on the major and minor systems. The procedure is applicable if the size of the watershed is not too large. It can be used in pipes where free surface flow is maintained.

The simplified method is compared with a more advanced design method, a dual drainage computer model. The OTTSWM model, developed at the University of Ottawa, is the only nonproprietary model of this type. OTTSWM (Kassem 1980, Wisner and Kassem 1980) has the capability to analyze street flow, pipe flow, and the hydraulic characteristics of different inlets. Flow rates, depths and velocities can be found both on streets and in pipes. The layout and inlet control level is selected by the designer. The model can determine the pipe sizes, resize pipes in an existing system or do a surcharge analysis. A further description of the model with example input and output is given in Appendix C. In cases where pipe surcharging occurs the OTTSWM model can be interfaced with EXTRAN, a dynamic routing model.

Design using both of the methods must be done in two steps. First the minor system must be designed for a low return period. The major system is then designed using a large storm event. Initially a schematic of the street and pipe system is necessary so that the water flow pattern can be followed. In most designs the pipes are placed to follow the slope of the land. This is not always the case, at times the pipe system may be carrying storm water in one direction while the surface flow may be in the opposite. It is important that the schematics clearly show the system connectivity, and the direction of flow. The minor system schematic should have manhole numbers marked on it, each contributing area (determined by the catchbasin location and the surface flow pattern) and imperviousness. For the major system schematic all of the above information is necessary as well as the number of catchbasins contributing to each manhole.

When the system is being initially laid out, catchbasins should be located in pairs one on each side of the street. At intersections four or six catchbasins will typically be required. Catchbasins at an intersection may not all belong to the same subwatershed. It is therefore important to delineate the flow patterns and subwatershed boundaries on the base map, before translating them to the schematics.

A major difference between the two models is that the simplified approach uses a lumped procedure in which flows are determined at each point for the entire upstream area. It should be remembered that the Rational Method is also a lumped procedure. The procedure further developed for the ICD's is an extension of CHECK-HYD a method for finding the peak flow for a lumped area. It was developed on the basis or research conducted at the University of Ottawa by Cheung (1982) and further applied in Markham (Mukherjee et al 1983). The main advantage of this procedure is that it is compatible with master drainage plans and detailed design computer analysis. For the purpose of this study the method combines graphical and tabular computations, to design a drainage system. It can be used by any practicing engineer and will be further called DUAL CHECKHYD.

OTTSWM, like most computer models, determines the runoff for each subwatershed and routes it to the desired point in the system. No routing is performed with the simplified method. The computations for lumped areas only determines the peak flows, while the computer model provides flows throughout the duration of the event. In DUAL CHECKHYD, the inlet capture is assumed to be constant, this is closer to reality mainly for large surface flows during a high return period storm event. For low flows the capture flow is not constant, but varies with the street flow. The simplified method should therefore only be used with high return period events. In system storage is provided in many subdivisions using the dual drainage principle. The analysis of minor system storage in a subdivision is not possible with the simplified method. The simplified method also cannot be used for surcharge analysis. Its application for the simulation of many alternatives and large areas may become too difficult.

5.2 Dual Flow Principle in DUAL CHECKHYD

Design of the minor system follows preparation of the base map and schematics. The design can be conducted using the rational method, modified rational method or other simple pipe sizing procedure. When all of the pipes have been sized, the major system design can be conducted.

The design of the major system is based on the assumption that the total flow for any subarea is the sum of the street flow and pipe flow. Thus computing the total flow at any point in the system it can be divided into major and minor system components, for a selected level of inlet control. In design the total flows are determined for a 1/100 yr. frequency. Two methods are available to determine the required inlet control. The first selects inlet controls so that flow entering the pipes does not exceed their existing capacities. This is done by finding the excess downstream capacity at a control point and ensuring that flow control is used to limit the inflow so that capacity, is not exceeded.

The control points are those where water enters the minor system. An example is shown in Figure 5.1. The total peak flow, Q_{PT}, upstream of the entry point is computed as described in section 5.3. The maximum allowable inlet flow, Q_{PI}, is governed by the upstream pipe flow, Q_{PCI}, and the downstream pipe capacity Q_{PCD} equation (5.1)

$$Q_{PI} = Q_{PD} - Q_{PU} \qquad (5.1)$$

Q_{PI} = Total Inlet Flow
Q_{PD} = Downstream Pipe Capacity
Q_{PU} = Upstream Pipe Flow

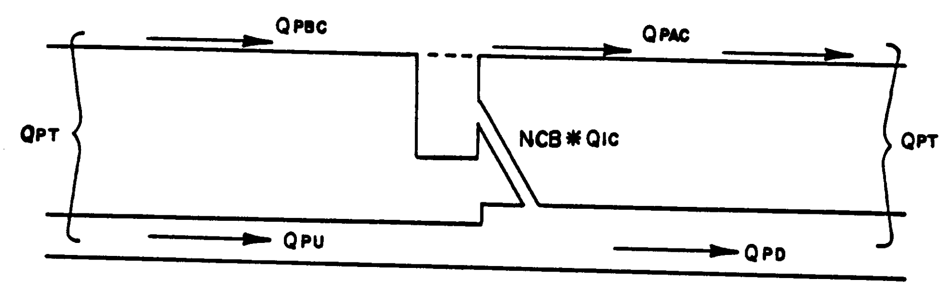
The inlet flow for each catchbasin can be found by dividing the allowable inlet flow by the number of catchbasins (equ. 5.2).

$$Q_{IC} = Q_{PI}/NCB \qquad (5.2)$$

Q_{IC} = Maximum Allowable Inflow per catchbasin
Q_{PI} = Total Inflow
NCB = Number of Catchbasins

Determining the inlet flow helps in selecting the inlet control level. Most inlet control devices are available in specific commercial sizes which give a flow control different from Q_{IC}. If the selected device controls to a value lower than Q_{IC}, then the

Figure 5.1 Major and Minor System Flow at an Outlet



QPBC - PEAK MAJOR SYSTEM FLOW BEFORE CAPTURE

QPAC - PEAK MAJOR SYSTEM FLOW AFTER CAPTURE

NCB - NUMBER OF CATCHBASINS

QIC - MAXIMUM ALLOWABLE INFLOW PER CATCHBASIN

QPU - PEAK UPSTREAM PIPE FLOW

QPD - PEAK DOWNSTREAM PIPE CAPACITY

downstream pipe should have excess capacity. A more severe restriction is being applied to inflow than is necessary, to maintain pipe flow within its capacity. On the other hand, if a selected control device is larger than Q_{IC}, and the total peak flow is larger than the downstream pipe capacity, the pipe will have to be resized.

$$Q_{PAC} = Q_{PBC} - NCB * Q_{ICD} \qquad (5.3)$$

Q_{PAC} - Surface Peak Flow after capture
Q_{PBC} - Surface Peak Flow before capture
NCB - Number of catchbasins
Q_{ICD} - Selected Inlet Flow Level

The street depths do not have to be computed at every entry point. If street slopes are very flat or when the major system outlet is being approached then the water depth on the streets should be computed. The street depths may be obtained from Figure 5.2.

The last step involves computing the downstream pipe flow so the pipe inflow at the next inlet will be known:

$$Q_{PD} = Q_{PU} + NCB * Q_{ICD} \qquad (5.4)$$

Q_{PD} - Downstream Pipe Flow
Q_{PU} - Upstream Pipe Flow
NCB - Number of catchbasins
Q_{ICD} - Selected Catchbasin Inlet Control Level

The amount of flow captured by the catchbasin is assumed to be constant and is at the level of the inlet control device. The approximation is satisfactory for large flows where the inlet control device capacity is limiting the flow. The inlet flow is not represented very accurately only if inlet controls are not used.

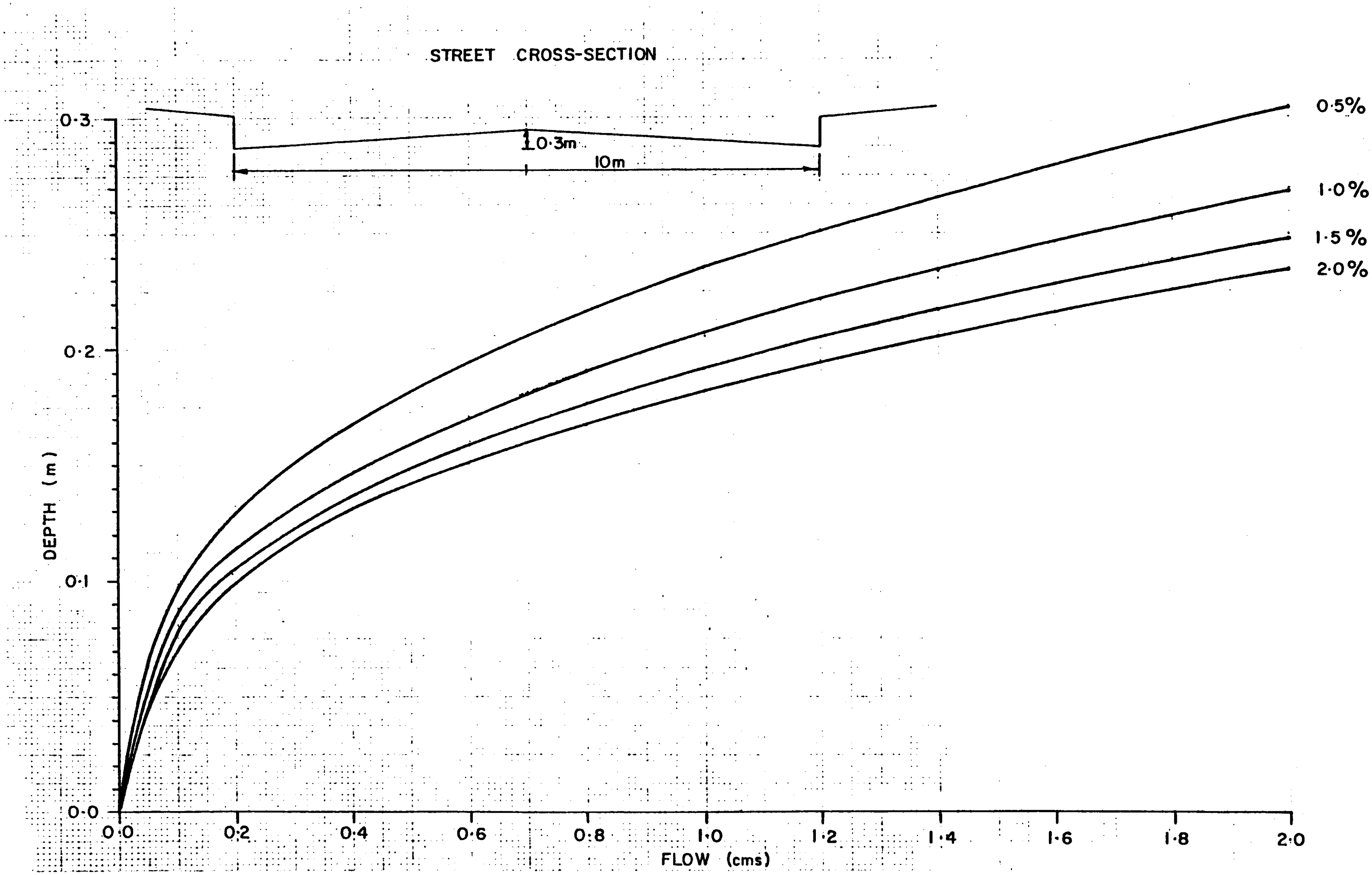


Figure 5.2 Street Depth vs. Flow for Various Slopes

As shown in Figure 5.3, the capture curve assumed with the simplified method is different from that used by OTTSWM. Without inlet controls the simplified method assumes that the capture is 100%. With inlet controls the capture is equivalent to the inlet control level. The OTTSWM model recognizes that even without inlet controls the capture efficiency is less than 100%. The grating capture curve is governed by the hydraulic capacity of the catchbasin grating. With inlet controls the catchbasin capture curve becomes horizontal as assumed with the simplified method.

5.3 Total Flow Computation

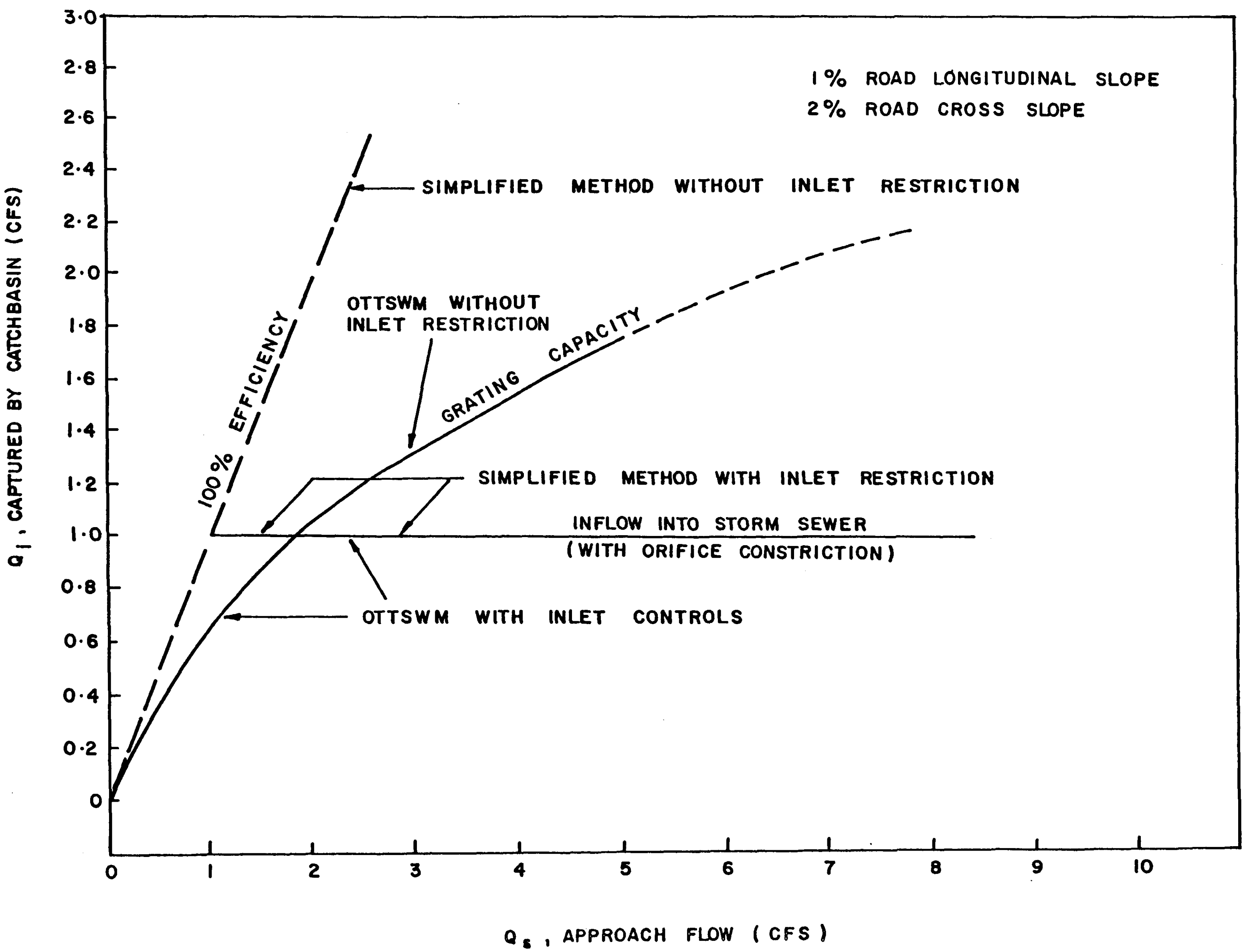
In general, computation of the total flow can be done with a user-friendly micro-computer model, which contains the URBHYD routine (Wisner and P'ng 1982). URBHYD gives results very close to the SWMM model and was tested on numerous watersheds (P'ng, 1982). The URBHYD routine is available in the OTTHYMO model (IMPSWM, 1982). User friendly hydrologic software developed for microcomputers are SIMHYD (Gesmec Inc., 1985), a non proprietory program developed for CMHC, and FASTHYMO (Andrew Brodie and Assoc., 1984) and can be used for quick peak flow computations. It was used instead of the Rational Method so that peak flows would be consistent with the URBHYD and SWMM models. For those who do not have continuous access to a computer model, the total flows can be found with DUAL CHECKHYD where only a first series of computer runs is required.

It can be demonstrated that for a given design storm, the peak flow is the product of the watershed area, the amount of rainfall/runoff and a peak factor (Cheung, 1983). The runoff volume to total rainfall volume ratio and the peak factor have to be established from many computer runs. The peak factor decreases as the area increases, thus incorporating peak flow reduction due to routing effects.

The graphs for Markham are available for a range of parameters

Figure 5.3 Hydraulic Performance of a Catchbasin Inlet (with and without a Grating Cover)

(Ref. Townsend, Wisner and Moss, 1980)



as shown in Table 5.1

Table 5.1

Range of Dual Checkhyd Parameters

<u>Areas</u> Urban 5, 20, 40, 80, ha.

<u>Storms</u> Toronto-Chicago Distribution 4 hr. storm.

Return Periods(yrs)	2,	5,	25,	100
Total Rainfall (mm)	39.2	48.4	66.1	83.7

<u>Urban Watershed Parameters</u>

Imperviousness 10%, 20%, 25%, 35%, 50%, 70%

Pervious Area Infiltration fo=76.2 mm/hr, fc=13.2 mm/hr, ~=4.14 hr^{-1}

Slopes 0.5%, 1.0%

The peak flow is given by equation (5.5)

$$Qp = F \cdot A \cdot Cv \cdot P \quad (5.5)$$

Qp - Peak Flow (cms)
A - Watershed Area (ha)
Cv - Runoff Coefficient
P - Precipitation (mm)
F - Peak Factor (cms/ha.mm)

Steps in Computing the Peak Flow:

1. Determine the cumulative contributing subarea.
2. Obtain the Peak Factor F from Figure 5.5.
3. Compute the runoff volumetric coefficient using the return period and impervious ratio Figure 5.4
4. Determine the precipitation volume P from Table 5.1
5. Compute the peak flow Qp = F . A. Cv . P

An explanation of each of the terms follows:

Runoff Coefficient

The runoff coefficient is necessary for finding the runoff volume. It is obvious that the runoff coefficient increases with higher impervious ratios. Also as the rainfall volume increases the proportion of runoff increases. These trends are shown in Figure 5.4 which was compiled from a number of runs. This figure can be used to obtain the runoff coefficient for a desired storm frequency and imperviousness.

Precipitation

To obtain the runoff volume requires that both the rainfall volume and runoff coefficient are known. The rainfall volume must be determined from local precipitation measurements. This information is available for many regions in Canada from Environment Canada's Atmospheric Environment Service. Precipitation volumes for Toronto are given in Table 5.1.

Peak Factor

The peak factor is required to convert the watershed area and runoff volume into a peak flow. Simulation runs were conducted using the URBHYD command in OTTHYMO and varying the watershed area and imperviousness. Peak flows obtained for each combination of area and imperviousness were divided by the watershed area and runoff volume to obtain the peak factors. They are shown in Figure 5.5 for various combinations of area and imperviousness. In the Markham applications CHECKHYD was only intended for Master Drainage Plans, the minimum area was 20 ha. For the development of DUAL CHECKHYD additional tests were made for smaller areas. It was found that for small areas a reasonable approximation of the peak factor

Figure 5.4 Runoff Volumetric Coefficient vs. Imperviousness

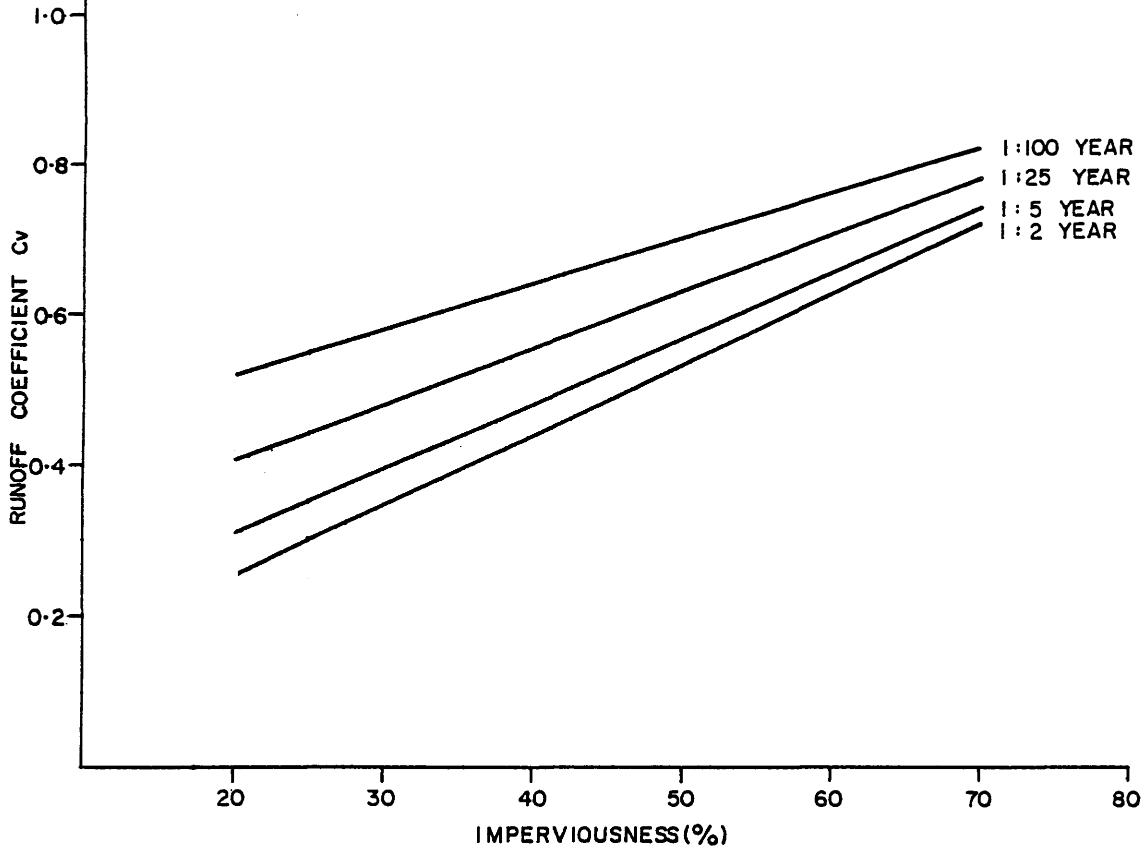
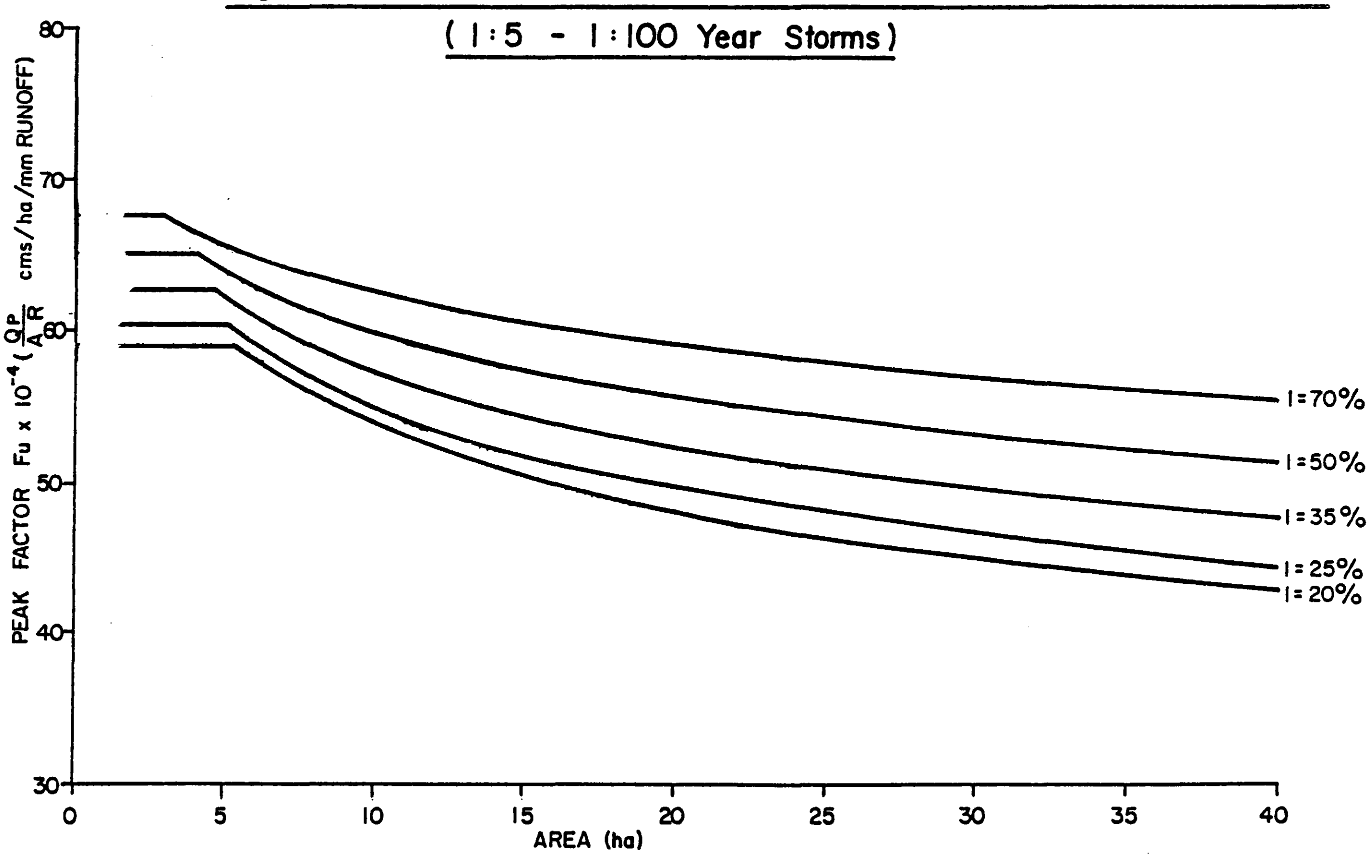


Figure 5.5 Peak Factor Fu vs. Area for Different Impervious Ratios

(1:5 - 1:100 Year Storms)



is obtained by assuming it is constant between approximately 0 and 5 ha.

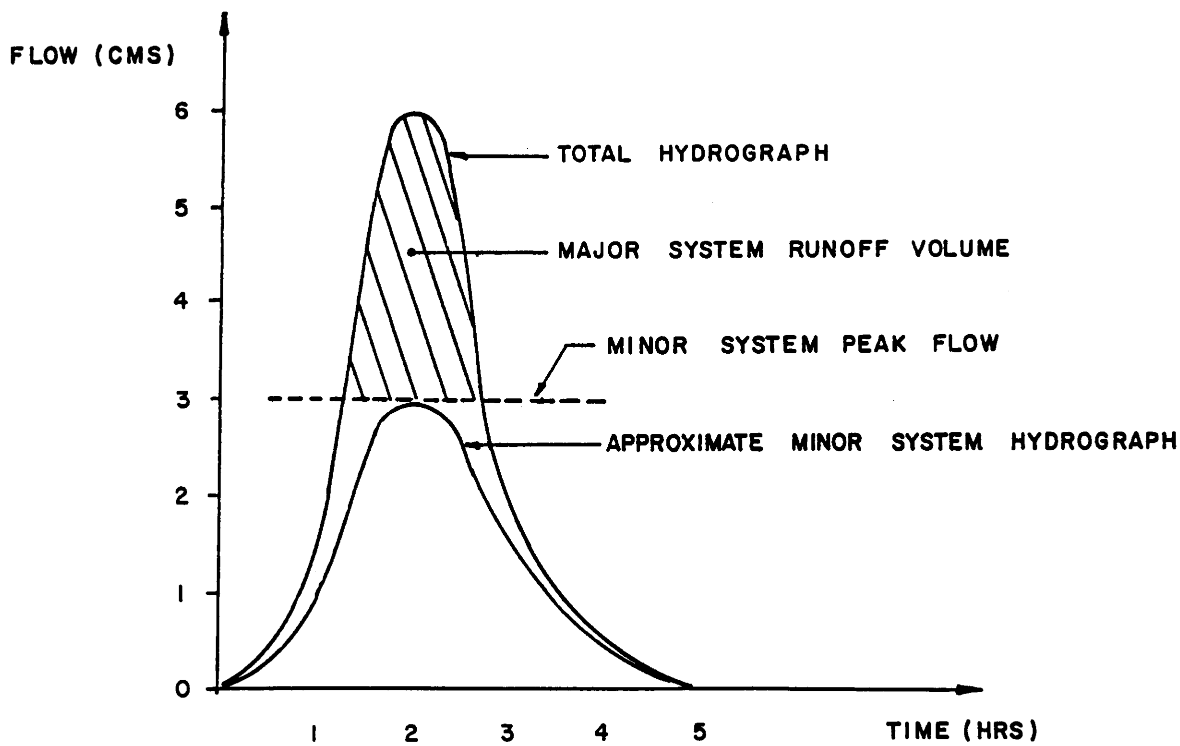
The peak factor for an urban watershed of a given area and imperviousness is found using Figure 5.5. These curves were developed for Markham, Ontario. The Markham Chicago design storms are close to those in other Metro Toronto municipalities and therefore results are valid for Metro Toronto. As the relationships are only appropriate in the jurisdiction for which they were developed, OTTHYMO, SIMHYD or FASTHYMO should be used to obtain curves for other municipalities where the method is to be applied. This computation should be done only once for each municipality.

5.4 Computation of the Major System Volume

For the major system outlet volume Chin (1983) found that in areas with ICD's a good approximation is possible by finding the total runoff hydrograph and then determining the cutoff flow entering the minor system. The total hydrograph should be found using the URBHYD routine available in OTTHYMO, SIMHYD or FASTMO.

The cutoff flow can be obtained by multiplying the total number of inlets by the average capture per inlet and is the same as the minor system peak flow. The area under the total hydrograph and above the cutoff is taken as the major system volume (Figure 5.6). It may be used in sizing a major system storage pond.

Figure 5.6 Hydrographs at Subdivision Outlet



Chapter 6

DUAL DRAINAGE DESIGN EXAMPLES

6.1 DUAL CHECKHYD Example

This chapter shows the computations required for using the simplified method to design a dual drainage system. A comparison between the simplified and the computer model is also made.

Sizing the Minor System

The pipe drainage network first needs to be established. From a base plan of the total watershed, smaller subbasins are delineated. The flow directions and street catchbasins should be located on it as shown in Figure 6.1. The example was taken from the ASCE, 1970. Design of the minor system can be conducted using the rational method (ASCE, 1970) or any other simple design procedure. The minor system pipe sizes form the basis from which the major system can be done.

Major System Computations

An important step in analyzing the major system is to first draw a schematic of the system. The major system flow directions can be different from those of the minor system. A schematic as shown in Figure 6.2, helps the designer keep the computations in the correct order. Designing the major system is done in two parts. First the total peak flows at the end of each major system segment is computed as shown in Table 6.1.

Once the major system flows have been computed the necessary inlet control level can be found. The procedure required for determining inlet control level and the street and pipe flows is shown in the flowchart Figure 6.3. The computations are conducted using a

TABLE 6.1 ILLUSTRATIVE FLOW COMPUTATION

Segment Number	Location		Length (m)	Slope %	Surface Elevation (m)	Area (ha)	Accum Area (ha)	Peak Flow Factor F (CMS/ha/mm)	IMP I%	RUNOFF COEFF. Cv.	PRECP. P (mm)	TOTAL FLOW $Q_{PT} = F.\Sigma A.Cv.P$ (CMS)
①	Upst	A-6	122	0.5	98.4	0.89	0.89	60.5×10^{-4}	30	0.58	83.6	0.26
②	A-6	A-5	122	0.875	94.9	1.42	2.31	60.5×10^{-4}	30	0.58	83.6	0.68
③	A-5	A-4	122	0.775	91.8	1.58	3.89	63.5×10^{-4}	42	0.625	83.6	1.29
⑧	A-4	A-3	122	0.775	89.9	–	3.89					
④	Upst	D-2	122	0.5	96.1	2.06	2.06	60.5×10^{-4}	30	0.58	83.6	0.60
⑤	D-2	D-1	122	0.75	92.2	1.54	3.60	60.5×10^{-4}	30	0.58	83.6	1.06
⑥	Upst	D-1	122	0.5	92.2	1.13	1.13	60.5×10^{-4}	30	0.58	83.6	0.331
⑦	D-1	A-3	122	0.75	89.9	1.58	6.31	60×10^{-4}	37.5	0.625	83.6	1.98
⑨	A-3	A-2	122			–	10.20	58×10^{-4}	39.3	0.63	83.6	3.11
	A-2	Exit										

TABLE 6.2 ILLUSTRATIVE MAJOR SYSTEM COMPUTATION

① Segment Number	② Location	③ Q_{PT} (CMS)	④ Downstream Pipe Capacity Q_{PCD} (CMS)	⑤ Inlet Control $Q_{PT} > Q_{PCD}$	⑥ Upstream Pipe Flow Q_{PU} (CMS)	⑦ Inlet Flow Q_{PI} = ④-⑥ (CMS)	⑧ No. of Catch basin NCB	⑨ Computed ICD Level $Q_{IC} \leq$ ⑦/⑧ (CMS)	⑩ Selected ICD Level $Q_{ICD} \leq Q_{IC}$ (CMS)	⑪ Pipe Flow Q_{PD} = ⑥+⑧×⑩ (CMS)	⑫ Size Increased (mm)	⑬ New Capacity (CMS)	⑭ Q_{PBC} = ③-⑥ (CMS)	⑮ Depth/Gutter (m)	⑯ Q_{PAC} = ⑭-⑧×⑩ (CMS)	⑰ Depth/Gutter (m)
①	Upst A-6	0.26	0.151	YES	0	0.151	3	0.050	0.042*	0.126			0.26	0.142	0.134	0.11
②	A-6 A-5	0.68	0.274	YES	0.126	0.148	3	0.049	0.042	0.252			0.55	0.163	0.43	0.15
③	A-5 A-4	1.29	0.553	YES	0.252	0.301	3	0.100	0.042	0.378			1.04	0.21	0.91	0.20
⑧	A-4 A-3	1.29	1.304	–	0.378	–	–	–	–	0.378			0.91	0.20	0.91	0.20
④	Upst D-2	0.60	0.288	YES	0	0.312	4	0.078	0.042	0.168			0.60	0.195	0.43	0.172
⑤	D-2 D-1	1.06	} 0.397	YES	0.168	} 0.229	3	} 0.038	0.042	0.294 } 0.42	Slightly Capacity	Exceeds OK.	0.89	0.198	0.76	0.188
⑥	Upst D-1	0.331		YES	0		3		0.042	0.126			0.33	0.136	0.204	0.127
⑦	D-1 A-3	1.98	1.304 − 0.378 = 0.926	YES	0.420	0.506	3	0.169	0.042	0.924			1.56	0.245	1.43	0.238
⑨	A-3 A-2	3.11			0.924								2.18	0.275	Exit →	

* Inlet Control of 0.042 CMS corresponds to 1.5 CFS

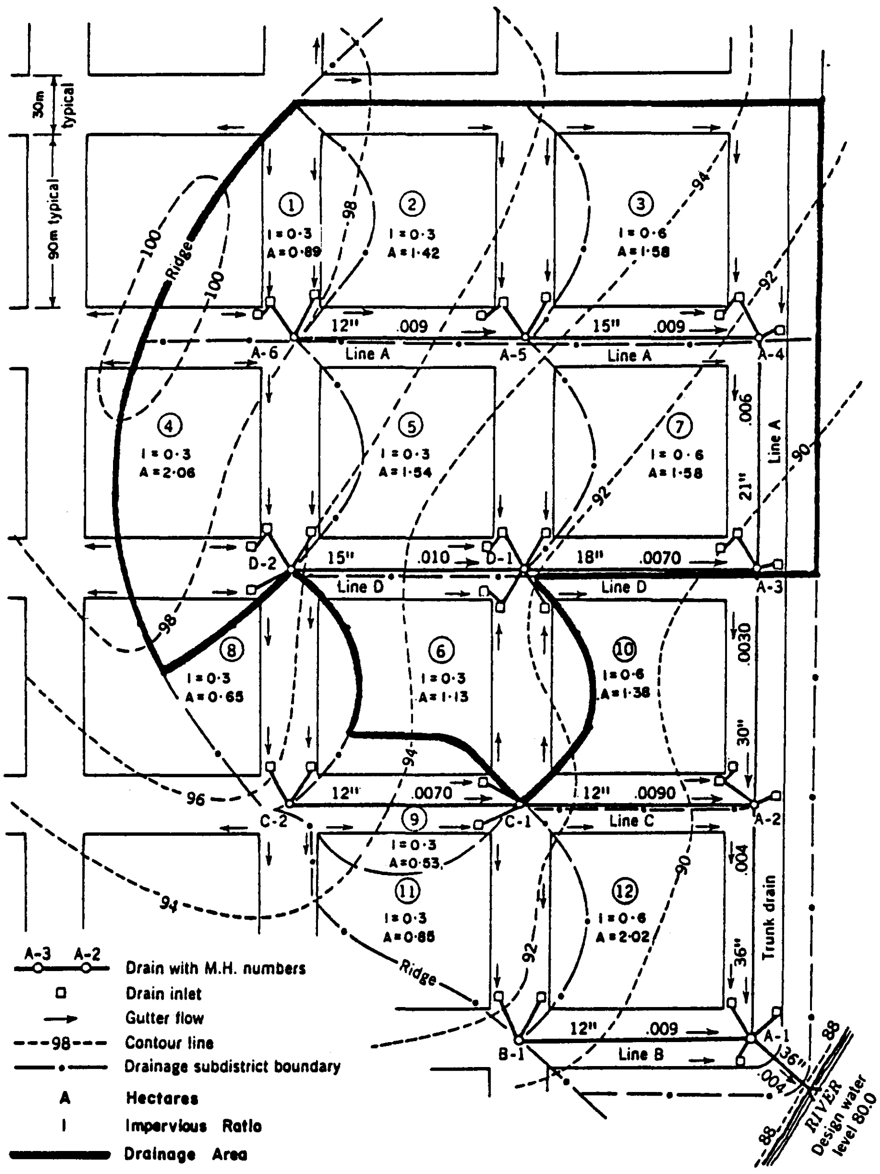
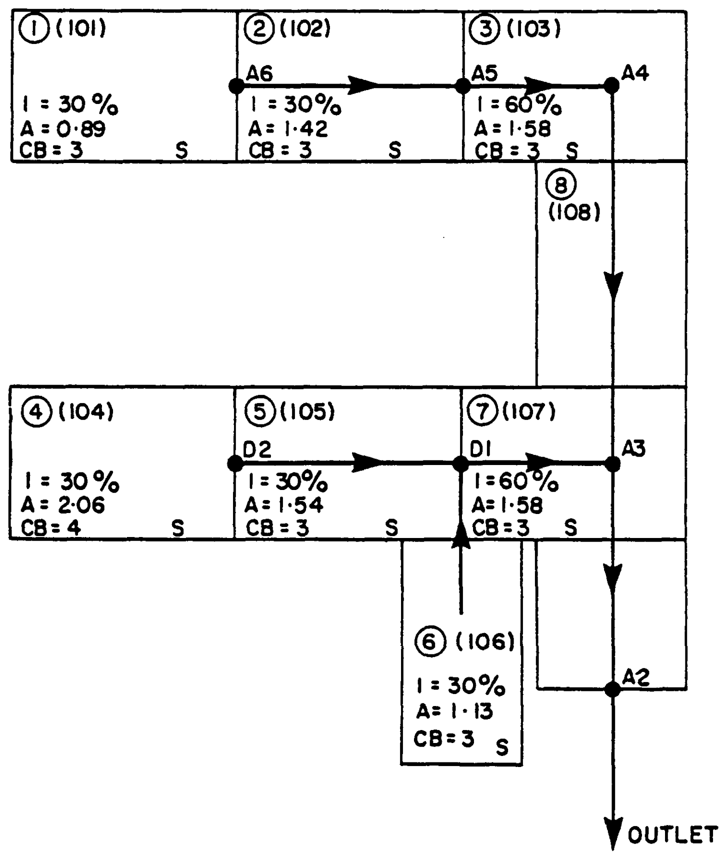


Figure 6.1 Illustrative Storm Drain Design Sketch (ASCE, 1970)



LEGEND

I = 30%	Impervious Ratio
A = 0·89	Area (ha)
CB = 3	Number of Catchbasins
S	Street Segment
A6	Manhole Number
①	Minor System Segment Number
(101)	Major System Segment Number

Figure 6.2 Major System Schematic

tabular format shown in Table 6.2. They commence with the upstream segments and proceed downstream one segment at a time.
For each major system segment the tabular method requires the following computational steps:

1. Record the Total Flow, the Downstream Pipe Capacity, the Upstream Pipe Flow and the Number of Catchbasins. cols. 2,3,5,7.

2. Determine the need for inlet controls $Q_{PT}>Q_{PCD}$ col.4

3. Determine the maximum allowable inlet flow. $Q_{PI}=Q_{PCD}-Q_{PU}$ col.6.

4. Compute the inlet control level to maintain downstream pipe within capacity $Q_{IC}=Q_{PI}/NCB$ col.8.

5. Select an inlet control device Q_{ICD}. To prevent pipe resizing then $Q_{ICD}<Q_{IC}$.

6. Compute the downstream pipe flow. $Q_{PD}=Q_{PU}+Q_{ICD}*NCB$ or $Q_{PD}=Q_{PT}$

7. Compute the pipe size and capacity if resizing is required. cols. 11 and 12.

8. Determine the street flow and depth before capture. cols. 13 and 14.

9. Verify that the street depth is below the maximum allowable depth.

10. Determine the street flow and depth after capture, cols. 15 and 16.

In the DUAL CHECKHYD method inlet controls are required if the peak flow is greater than the downstream pipe capacity. The desired level of inlet control, Q_{IC}, gives a guideline as to which commercial size of inlet control device should be selected. In most cases the inlet control device flow, Q_{ICD}, will be different from Q_{IC}. It is preferable that Q_{IC} be greater than Q_{ICD}, so that free surface flow is maintained in the downstream pipe. This may not always be

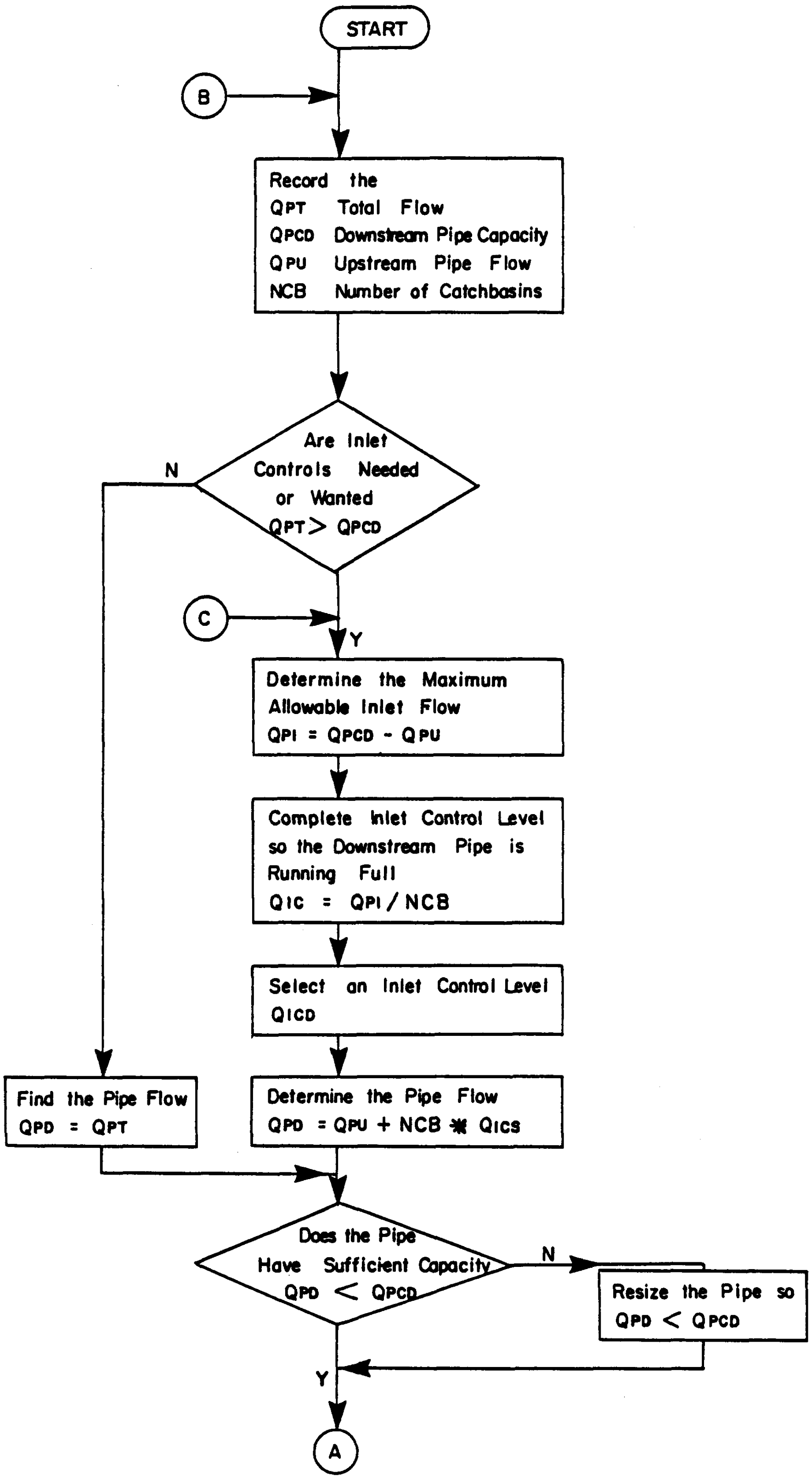


Figure 6.3 Flowchart of the Major System Computations with Dual CheckHyd.

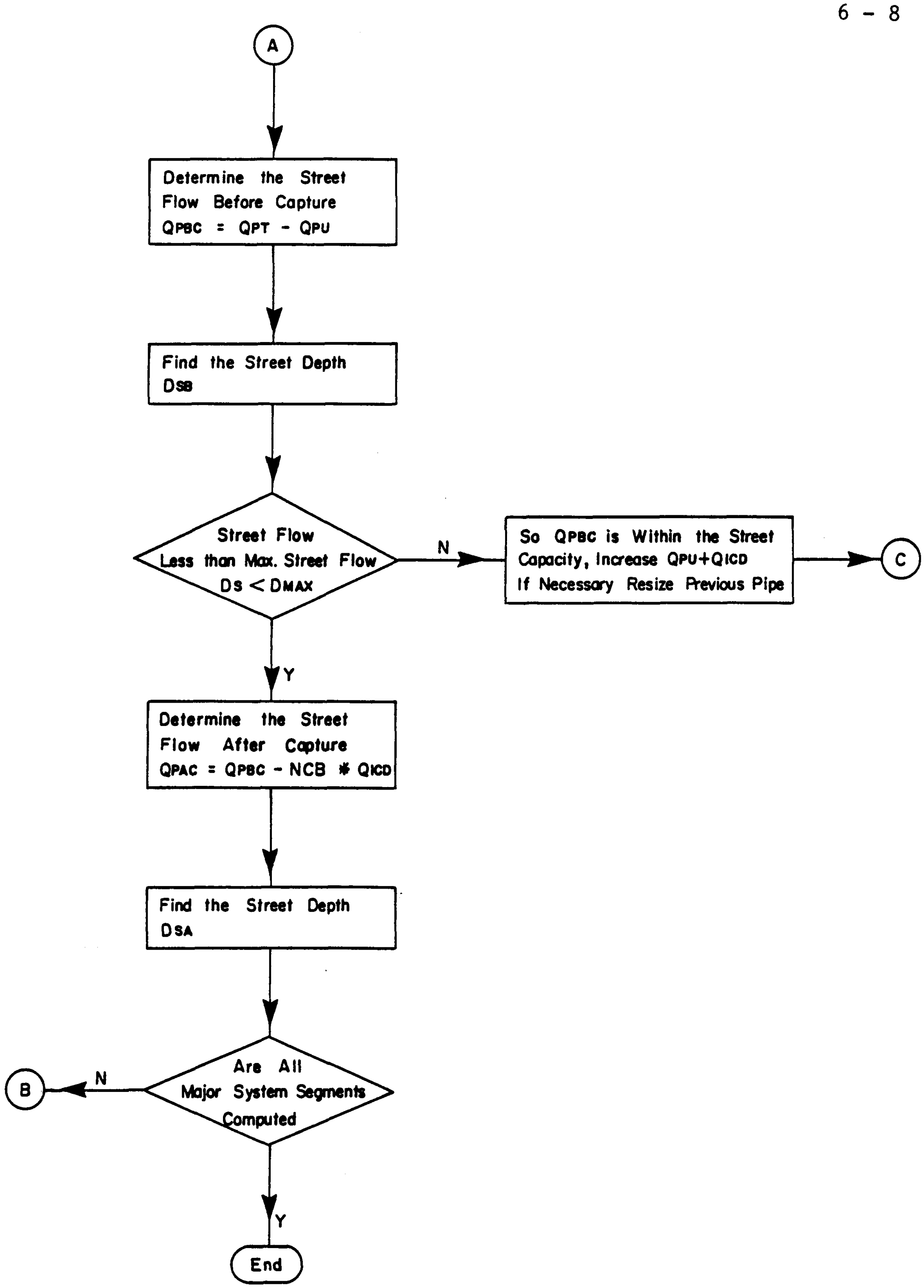


Figure 6.3 Continued

possible, other restrictions, such as an acceptable lower limit for Q_{ICD}, or insufficient downstream capacity may result in Q_{IC} being less than Q_{ICD}. This means that the downstream pipe will have insufficient capacity. To continue using the simplified method pipe resizing will be required. (If not resized the pipe becomes surcharged, a computer model using dynamic routing is required for proper surcharge analysis.) With the inlet control level determined the pipe flow and then subsequently the street flows and depths can be found. If street flow depths are exceeded, then the inlet control level will have to be increased. This may require resizing pipes, both upstream and downstream of the particular segment. The flows on the surface and in the pipes must be within the flow capacity of these segments.

At a junction inlet controls for the incoming segments may be computed simultaneously or the flows may be determined for each individual segment while ensuring that the downstream pipe capacity is not exceeded. In the illustrative example, the inlet control was found simultaneously for segments (5) and (6), Table 6.2. The second method was used at the junction of segments (8) and (7) where the flow was first found for segment (8). This controls the pipe flow from segment (7) which should not exceed the difference between the downstream pipe capacity and the flow from segments (8). In the example, the downstream pipe had sufficient capacity with a control level of 0.042 cms. If a uniform level of inlet control is being used throughout a subdivision, the second approach can usually be used without having to resize the downstream pipe or provide excessive control in one of the segments

In a previous study by Novatech 1984, the performance of a conventional system was analyzed with a 100 yr. storm. Then inlet controls were used to reduce the extent of surcharge. As a conventional system, the dual drainage principle had not been applied, consequently the analysis showed that 40% of the pipes had surcharges between 0.09 m and 4.57m. The extent of basement flooding was estimated by assuming that surcharges of more than 0.9m would be critical. The results showed that 43% of the homes in the subdivision would have been flooded. Thus even without inlet controls all of the homes in the subdivision are not flooded.

To limit the surcharging an inlet restriction of 28.3 lps was used in about half of the subdivisions catchbasins. The inlet restriction reduced the flow in the pipes and increased the street flows. The street flow depth increases ranged from 0 to 20 mm. The greatest street depth was 186mm, which is less than the maximum usually allowed by municipalities of 250-300mm.

This indicates the importance of designing the system and locating restrictors only where needed. Placing inlet control devices in every catchbasin in a subdivision, may result in excess expenditure.

6.2 Comparison of the Simplified and Advanced Methods

A comparison of the advanced and simplified methods was conducted for a small residential subdivision of 11.95 ha (29.5 ac) (Figure 6.4). The watershed was broken into 31 subareas based on different watershed characteristics and drainage patterns. Impervious ratios of 20% for backlots and 45% for streets and front lots were used in determining the runoff. Figure 6.4 shows the subareas, the street and pipe layout, inlet locations and manholes. The roofs are not directly connected to the sewer pipes but drain into the backlots. A total of 51 catchbasins are located throughout the systems giving a catchbasin density of 4.26 CB/HA or (1.7 CB/AC).

The system was designed using the dual drainage principle so that there are no low points in the system. On this watershed the major and minor system conduct the flow in the same direction, as shown in the schematic Figure 6.5.

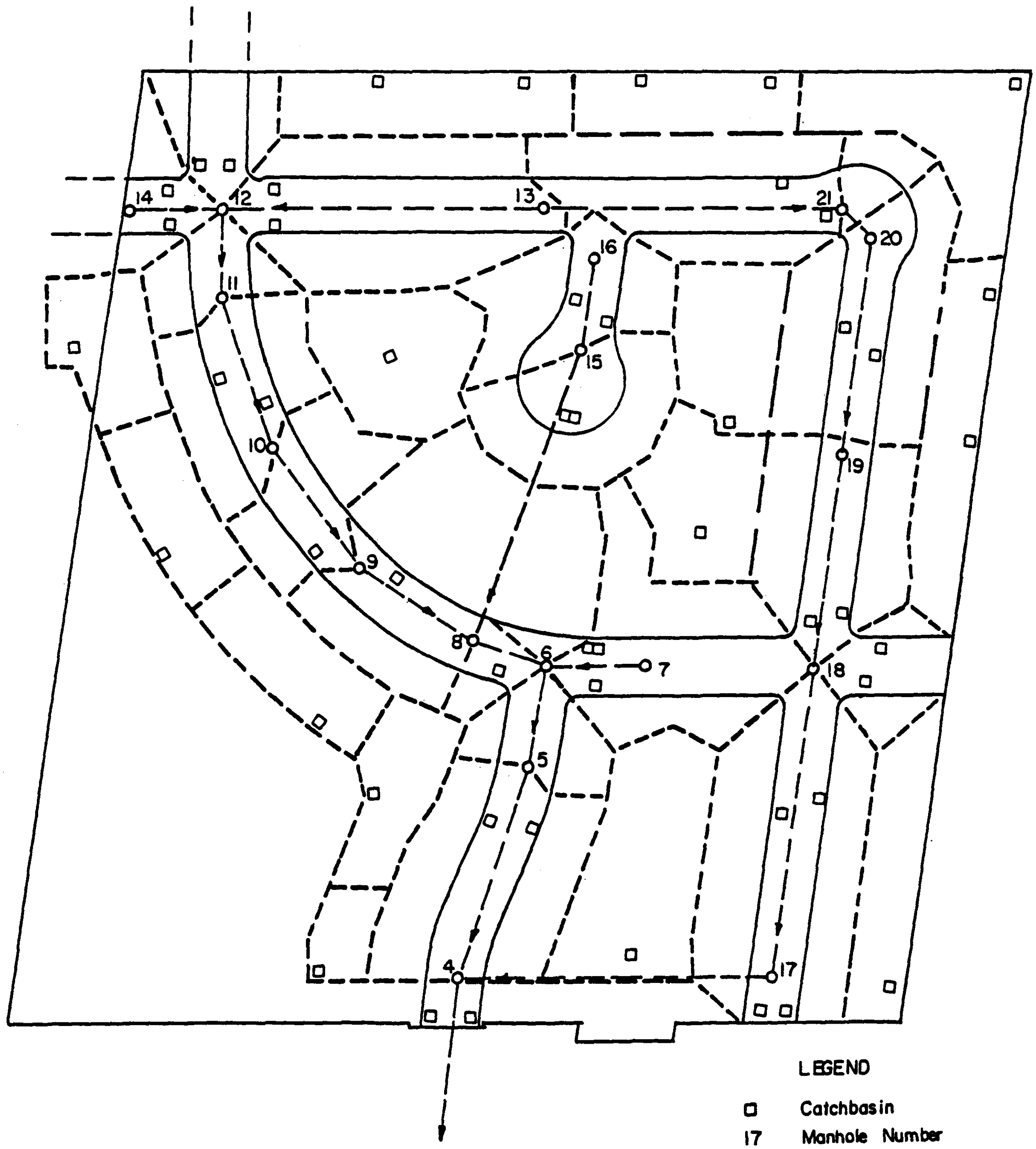


Figure 6.4 Residential Subdivision with a Major & Minor System Drainage Plan

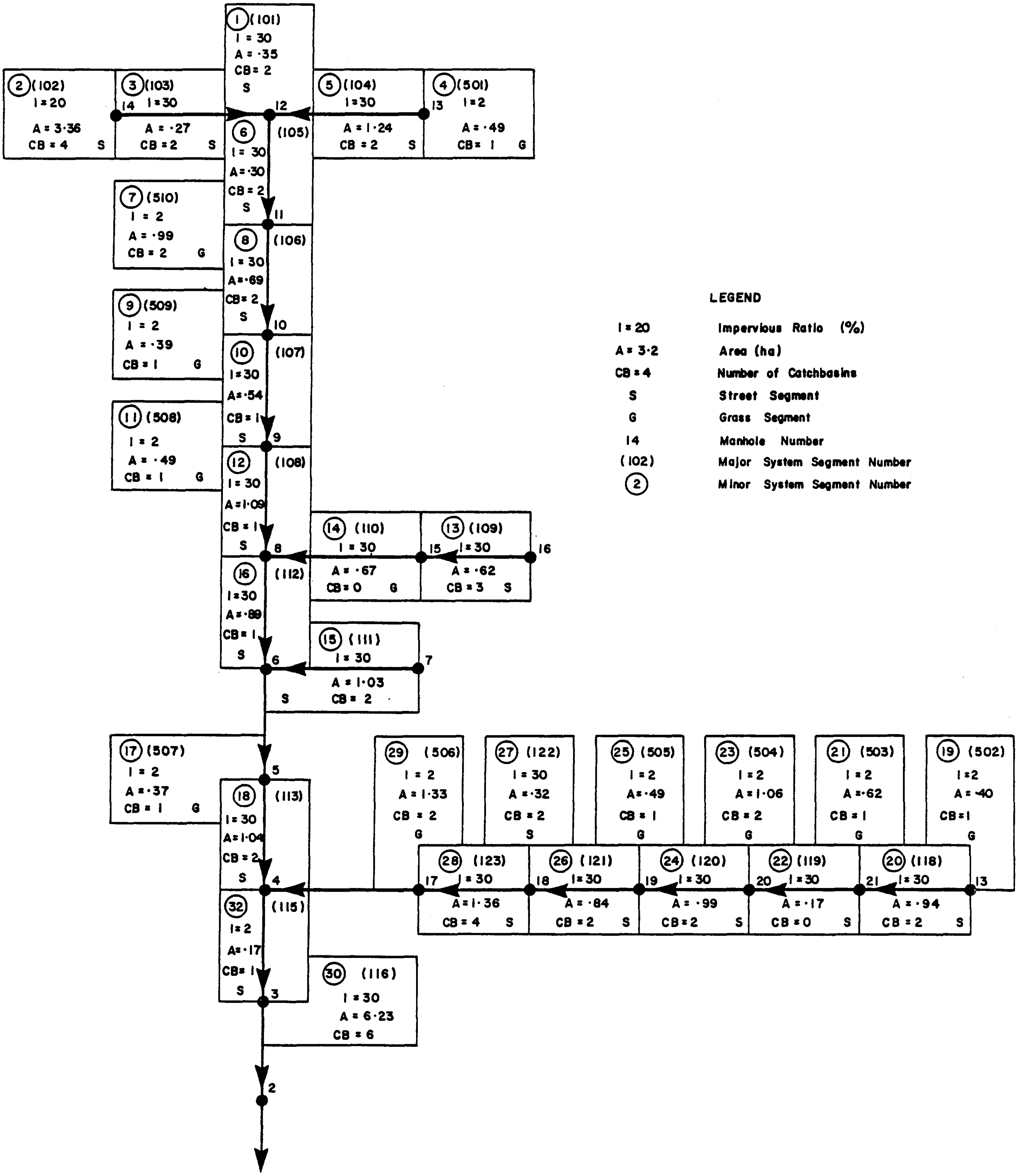


Figure 6·5 Schematic of the Residential Subdivision (Major and Minor System)

Design of the system was first conducted with the rational method using a 2 yr. return period. Pipes were sized according to the 2 yr. peak flows. The pipe sizes and flow direction form the basis from which the major system analysis was conducted.

The major system was analyzed using the simplified method and the computer method. The simplified method requires two steps. First the computation of the cumulative total flow at the outlet of each sub area. This is followed by the computation of the street and pipe flows at each segment. It was found that the total flows computed with the simplified procedure were slightly larger than those estimated with the OTTSWM model.

The computation of the major system flows were done using level of control of 28.3 lps. In some cases, the upstream pipes had sufficient capacity, no inlet control was required. As the design progressed downstream, inlet controls were required, at the outlet of the subdivision, the pipes did not have sufficient capacity, resizing was necessary. To prevent resizing a lower level of inlet control could have been used. This was not attempted in this example.

The OTTSWM model analyzes the surface runoff by first computing the flow captured by each inlet and then computing the major system flow. The routing and surcharge analysis was done next using EXTRAN. The analysis of the major system flows showed that the flow depths in the gutter ranged between 1 and 6 inches. the flows in the minor system exceeded the pipe capacity, consequently a number of pipes were surcharged. A comparison of pipe flows showed that those which were surcharged in OTTSWM were about the same as those that required resizing in the simplified method (Table 6.3).

Both models showed that inlet controls were not needed in all of the street segments. In particular it was found that most upstream segments with small areas had sufficient capacity. The higher flows given by the simplified method, resulted in the gutter being 48 mm. deeper at the outlet than estimated by OTTSWM. For most of the street segments the flow depth was between 0 and 38 mm deeper.

The comparison of the tabular method with the computer model, showed that both methods gave similar results under certain conditions. The peak flows computed with the simplified method were slightly larger than those obtained with the OTTSWM model. The tabular method showed that resizing was required for most of the same pipes that surcharged with the OTTSWM model. In addition inlet controls were used in most of the same catchbasins with both design methods. The simplified method can be used in designing inlet controls for small subdivisions, as long as free surface flow within pipe and street capacities is maintained. It is mainly intended for municipalities without computer expertise who want to review studies or intend to explore the inlet control design. The advantages for consultants of the computer model OTTSWMM and its micro computer verson P.C. OTTSWMM are not only that several options can be easily compared, but also that larger areas can be analysed and the output with and without surcharge can be incorporated in reports. System schematization and a good understanding of operation are essential for both hand and computer method.

Table 6.3
Comparison of OTTSWM and DUAL CHECKHYD

STREET FLOWS

ID		SEGMENT		OTTSWM				DUAL CHECKHYD			
MH	MH	No.	Type	Flow (lps)	Depth (mm)	Inlet Rest.	Sur.	Flow (lps)	Depth (mm)	Inlet Rest.	Pipe Resizing Required
UPST	12	101	S	46.	38.	N		42.		N	
UPST	13	501	G	36.	35.	Y		40.	25.	N	
13	12	104	S	110.	58.	Y		131.	61.	Y	
UPST	14	102	S	242.	79.	Y		360.	89.	Y	
14	12	103	S	164.	66.	Y		277.	81.	Y	
12	11	105	S	213.	74.	-		385.	93.	-	
UPST	11	510	G	45.	38.	N		82.	51.	N	
11	10	106	S	292.	84.	Y	YES	463.	100.	Y	
UPST	10	509	G	32.	36.	Y		33.	31.	N	
10	9	107	S	300.	86.	Y	YES	472.	101.	Y	YES
UPST	9	508	G	36.	36.	Y		41.	37.	N	
9	8	108	S	380.	94.	Y	YES	590.	111.	Y	YES
16	15	109	S	70.	46.	N		75.	48.	Y	
15	8	110	G	103.	51.	-		99.	55.	Y	
7	6	111	S	136.	63.	Y		96.	55.	Y	
8	6	112	S	632.	114.	Y	YES	700.	121.	Y	YES
UPST	4	507	G	32.	36.	Y		30.	32.	Y	
6	4	113	S	720.	122.	Y	YES	830.	124.	Y	YES
out4	3	115	S	682.	119.	Y	YES	1468.	167.	Y	
UPST	3	116	-	507.	104.	Y		-	-		
UPST	13	502	G	33.	36.	Y		33.	31.	N	
13	21	118	S	116.	61.	Y		103.	52.	Y	
UPST	21	503	G	39.	38.	Y		51.	39.	N	YES
21	20	119	S	81.	46.	-	YES	57.	40.	-	YES
UPST	20	504	G	45.	38.	N		88.	51.	Y	YES
20	19	120	S	193.	71.	Y	YES	196.	71.	Y	YES
UPST	19	505	G	36.	36.	Y		40.	36.	N	
19	18	121	S	236.	76.	Y	YES	236.	76.	Y	YES
UPST	18	122	S	41.	38.	N		39.	-		
18	17	123	S	353.	91.	Y	YES	362.	91.	Y	YES
UPST	17	506	G	49.	41.	N		110.	57.	Y	
17	4	124					YES			-	YES

G - Grass
S - Street

Chapter 7

SELECTION OF TYPE AND LEVEL OF CONTROL DEVICES

7.1 Present Practice

Several projects were reviewed in order to analyze the experience with inlet control devices (ICD). The projects fall into two categories:

(i) Master Drainage Plans (MDP) where inlet control devices were considered on a preliminary level.

(ii) Detailed analysis based on design drawings.

Table 7.1 gives the main features of these ICD applications. The MDP's give only an indication of maximum street flow depths.

From detailed designs, it is seen that the density of inlets varies from 2.7 to 4.2 catchbasins per hectare.

In most case studies which have been analyzed in detail with OTTSWMM, some of the inlets are not restricted. With one exception, the number of inlets with restrictors (ICD's) varies widely from 25% to 75%. The projects with ICD's of 28 lps are more recent and were verified for surcharge. The street depth is in general less than 30 cm during the 100 year storm (Case F which is an exception, has a large external area and had the pipe system initially designed for the 5 year storm without verification for the 100 year storm). It was found that for all case studies, the sewer system without inlet restrictors will be surcharged at most locations during the 100 year storm. In some cases, inlet controls of 42 lps were found adequate to reduce or eliminate surcharges if the pipe size corresponds to the 5 year Rational Method design.

TABLE 7.1 EXAMPLES OF CASE STUDIES USING INLET CONTROL DEVICES

CASE STUDY	AREA ha.	LAND USE	AV.IMP. (%)	TOTAL NO. OF INLETS	INLET DENSITY (CB/ha)	%INLETS WITH RESTRICTORS	LEVEL OF INLET CONTROL (LPS)	SURCHARGE ELIMINATED	MAX.STREET FLOW DEPTH (cm)	PARK STORAGE USED	EXTERNAL AREAS (ha)	COMMENTS
A	465	RESID	31	MDP STUDY			5 YEAR CONTROL	AVOIDED	28	YES	-	A detailed inlet control analysis was not conducted. ICD's used mainly to avoid surcharging the sewers during rare storms.
B	117	RESID	30	320	2.73	25	42	Limited & Acceptable		YES	-	
C	10	RESID	25	32	3.2	75	42	Negligible	20	NO	15.4	
D	32	RESID	40	100	3.1	50	34 @ 28 lps 16 @ 42 lps	Limited & Acceptable	24	NO	-	
E	44	RESID	35	166	3.7	51	28	Limited & Acceptable	26	YES	-	
F	118	RESID	35			100	28	ELIMINATED	37	YES	90.8	Severe inlet controls were required because of the status of the project at time of analysis.
G	12	RESID	25	50	42	68	28	Limited & Acceptable	12	NO	-	

For case E, the analysis indicated that in order to avoid surcharge, there is a need to reduce the level of control to 28 lps. The results of this example also indicated that the pipe size reduction on several segments as compared to the 5 year Rational Method design is possible if small surcharges are accepted.

Table 7.1 shows that the practice used in some municipalities where all inlets are restricted without conducting a detailed analysis should be changed. The need for an ad hoc analysis is also demonstrated by the fact that the level of inlet control in order to avoid surcharges during the 100 year storm can vary from one project to another (even if conduits are sized for the 5 year storm in all cases). This is a consequence of different system configurations and pipe slopes.

It was also found that in most new developments, no attempts were made to reduce the level of inlet control mainly in order to achieve savings. The explanation is concern on performance and therefore the objectives for the use of inlet controls devices was mainly to avoid surcharges and not to achieve savings as compared to a traditional design.

In general, it was possible to limit the maximum depth of flow on streets to 37 cm or less for the 100 year storm, and park storage was limited to 1-1.5% of the total area.

Based on these and other examples, it was very difficult to define simple empirical rules for storm sewer analysis using inlet control devices and case-by-case analysis is required. In general, the inlet control devices applied over systems which are designed on a traditional basis.

Section 4, however, indicates that lower controls are possible provided that proper inlets are selected as discussed further.

7.2 Selection of Inlet Control Devices

It should be reminded that the debris loadings used in the tests described in Chapter 4 were heavier than those occurring in real catchbasins. Tests were run to see if debris placed on the street readily washed into the catchbasins. The tendency was for the leaves and sticks to accumulate in small piles in the vicinity of the street grating with the water washing around them (Wash off of debris into the catchbasin did not occur). Observations indicate that small amount of debris in the catchbasin sump would wash through the orifice. Larger accumulations of debris as used in the tests have a greater tendency to clog them. The worst conditions occurred when sticks became lodged in or across an orifice, followed by an accumulation of leaves. In this situation, the orifice units operated partially clogged.

Although, the Cromac units are somewhat more susceptible to clogging because of the narrow opening of the orifice, it was found however, that an acceptable operation is possible with this device for controls of more than 20 lps, at a 1.2m head.

The Scepter unit was found to exhibit good self-cleaning capabilities when handling leaf-laden flows in excess of 14 lps at a 1.2m head. For smaller flow rates, the unit should only be employed when the flow is clean, or when debris fractions are relatively small.*

The Hanging Trap device becomes clogged under turbulent catchbasin conditions, when both leaves and sticks were present in the catchbasin. Once this unit is clogged, only manual cleaning is able to clear it.

The Hydrovex devices were the least susceptible to clogging, and showed good self-cleaning characteristics

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* The unit has also demonstrated a good grit entrapment capability, so that a sediment buildup in the sump will not diminish the unit's performance (Townsend, 1984). Tests with grit were not conducted in the present studies.

All of the commercial flow regulators performed well above a critical control level, in that complete blockage never occurred. Clogging tests show that from an installation and maintenance viewpoint, each unit has a particular range of applicability which is governed by a range of flows. For low flows between 3 lps and 14 lps, which are typical for application in some relief sewers of existing systems, the Hydrovex device is the only one that should be used. The other orifice devices were not developed to operate at these low controls and are recommended mainly for situations where very severe reductions are not necessary such as new developments.

It may be cheaper for a municipality to install a Hanging Trap device but when compared to the other options, it has several drawbacks: it protrudes from the catchbasin wall; catchbasin turbulence induces clogging; and it is difficult to determine if the unit is clogged as the inlet orifice is submerged. In non-turbulent conditions such as when flow was introduced from the bottom of the catchbasin, or in the installation (Figure 4.1), the device performed well. Protrusion of some Hydrovex systems and the effect of cleaning should also be considered.

A regular street cleaning and catchbasin sump cleaning program is necessary to ensure good operation at low levels of control for the ICD's. It is recommended that municipalities maintain their current catchbasin cleaning practice, but now include a visual inspection of inlet control device. However, if low levels of inlet control are being used the inspection frequency should be increased.

7.3 Selection of Level of Control of ICD's for New Developments

The previous sections show that since the level of control is not limited by the operation of the ICD's, its rational selection should be based on such criteria as economics, maintenance and operation of the entire system. The OTTSWMM model gives the possibility of relatively fast comparisons of many options. Since conditions may very from one sub-division to the other, the conclusions from the following specific example have to be applied with some caution in different situations. The discussion illustrates, however, the various factors to be accounted for in most projects.

The study was conducted for the system shown in Figure 7.1 which present a schematization of a real system (with some modifications). The total area of 42 ha is developed for residential land use with I=30%. The study was conducted in the following steps:

(i) Determination of pipe sizes for the system for the 5 year storm with controls exerted only by the effect of the gratings.

(ii) OTTSWMM computation of pipe reduction for the 5 year storm with inlet control devices for 42 lps, 28 lps, 19.8 lps, 14 lps and 8.5 lps.

(iii) Same as (ii) for the 100 year storm.

In all of the above simulations, surcharges were not accepted. Additional reduction of pipe sizes is possible if some surcharge is accepted. Results in Table 7.2 to 7.9 lead to the following observations:

(i) Variation of depth of flow on streets with respect to level of inlet control at typical locations. One can see that for the upper areas, the differences are small or practically negligible. If the control is reduced from 42 lps to 20 lps at the downstream part of the system and for the 100 year storm, the depth increased from 25 to 34 cm. For more frequent occurrences a feeling for the inconvenience is given by the 5 year maximum depth which increased from 14 to 18 cm, if the control

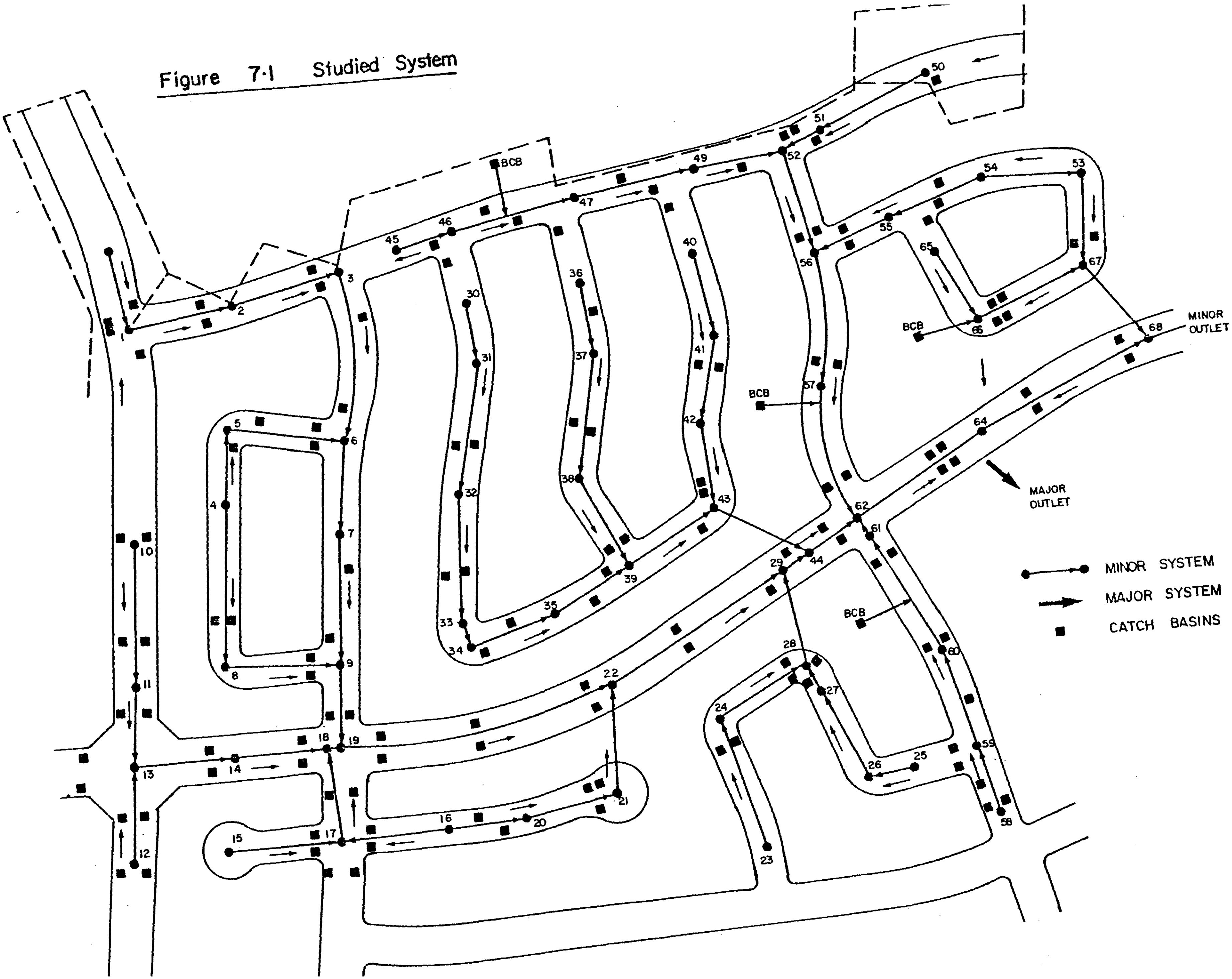


Figure 7·1 Studied System

is reduced from 42 lps to 20 lps. An additional increase of 4 cm. occurs for further reduction to 8.5 lps.

TABLE 7.2 VARIATION OF DEPTH OF FLOW ON STREETS WITH LEVEL OF INLET CONTROLS

STORM	LOCATION*	AREA (HA)	DEPTH OF FLOW ON STREETS (cm) NIC	42LPS	28LPS	19.8LPS	14LPS	8.5LPS
100 YEAR	MH19-MH2	15.4	17	18	19	20	20	21
	MH43-MH44	7.3	13	13.7	15	15.7	16.2	16.5
	MH62-MH64 OUTLET	42.0	25	27	30	31	33	34
5 YEAR	MH19-MH22	15.4	9	10	11	12	13	14
	MH43-MH44	7.3	6.2	7.4	8.4	9.6	10.2	10.9
	MH62-MH64 OUTLET	42.0	13	14	16	18	20	22

* SEE FIGURE 7.1

NIC - NO INLET CONTROLS

(ii) Comparison of pipe flow and number of required restrictors in terms of level of control. One can see that for a level of control of less than 28 lps, the number of ICD's for the 100 year condition is close to the total number of catchbasins. If the level of control is 20 lps or less, the outlet pipe flow for the 5 year and 100 year storms are very close.

TABLE 7.3 VARIATION OF PIPE FLOW WITH LEVEL OF INLET CONTROLS

STORM	INLET CONTROL (LPS)	PIPE FLOW (cms) MH43-MH44 (7.3 ha)	MH19-MH22 (15.4 ha)	MH64-MH68 (42.0 ha)
100 YEAR	NIC	1.0	2.8	5.6
	42	0.8	2.4	4.9
	28	0.5	1.7	3.6
	19.8	0.4	1.3	2.7
	14	0.2	0.9	2.0
	8.5	0.2	0.6	1.2
5 YEAR	NIC	0.6	1.6	3.3
	42	0.6	1.6	3.2
	28	0.5	1.4	2.9
	19.8	0.4	1.1	2.3
	14	0.2	0.8	1.7
	8.5	0.2	0.5	1.1

NIC - NO INLET CONTROLS

TABLE 7.4 VARIATION OF PERCENTAGE NUMBER OF CONTROLLED INLETS WITH LEVEL OF CONTROLS

LEVEL OF CONTROL (lps)	PERCENTAGE NO. OF CONTROLLED INLETS* (%) 5 YEAR	100 YEAR
42	13	56
28	45	80
19.8	75	94

TABLE 7.4 CONT'D

14	85	99
8.5	93	100

* TOTAL NUMBER OF INLETS = 143

(iii) Reduction of pipe diameter and variation of pipe cost with level of inlet controls. Reductions in total pipe cost are not significant for ICD's of 42 lps as compared to traditional designs. They are however, 17% for 28 lps and 23% for 20 lps. A reduction of 33% is possible with 8.5 lps controls.

TABLE 7.5 COMPARISON OF PIPE SIZES WITH DIFFERENT LEVEL OF INLET CONTROLS

STORM	LOCATION* MH-MH	AREA (HA)	PIPE SIZE (mm) NIC	42LPS	28LPS	19.8LPS	14LPS	8.5LPS
100 YEAR	19-22	15.4	1500	1350	1350	1050	975	825
	43-44	7.3	825	825	675	600	600	500
	64-68 OUTLET	42.0	1950	1950	1650	1500	1350	1200
5 YEAR	19-22	15.4	1200	1200	1200	1050	900	750
	43-44	7.3	750	750	675	600	600	500
	64-68 OUTLET	42.0	1650	1650	1650	1500	1350	1050

* SEE FIGURE 7.1 FOR LOCATION

NIC - NO INLET CONTROLS

TABLE 7.6 VARIATION OF PIPE COST WITH LEVEL OF INLET CONTROLS

LEVEL OF CONTROL (LPS)	PIPE COST (x $1000)
NIC	1295
42	1259
28	1122
19.8	1029
14	940
8.5	858

(iv) Variation of pipe cost plus inlet control cost (total cost) with Level of controls. Although the ICD's for lower level control (Hydrovex) are more expensive, the total cost has the same trend as pipe cost. Table 7.8 gives the savings per hectare if the level of control is reduced compared to 42 lps, which corresponds to the present practice. Saving per hectare for the smaller and the larger areas are of the same order of magnitude.

Reduction of ICD level from 42 to 28 lps as accepted in several projects in Table 7.1 represents only a small part of potential savings*. As an example, if the ICD's are designed for 20 lps which represents a safe operation for all commercial inlet types, saving would represent $5500 per hectare (100 year storm), that is, two times the saving for the 28 lps ICD.

* In this analysis, it was assumed that the cost of pipe is $0.39 millimeter diameter per meter length ($3.00/inch diameter/foot length). The cost of inlet controls was considered to be $110 for restrictors greater than 14 lps and $350 for restrictors less than and equal to 14 lps. The trends should be the same even if the costs in specific projects are somewhat different.

TABLE 7.7 VARIATION OF PIPE COST PLUS INLET COST (TOTAL COST) WITH LEVEL OF INLET CONTROLS

LEVEL OF CONTROL (LPS)	TOTAL COST (PIPE + INLET) (x $1000)
NIC	1296
42	1268
28	1135
19.8	1044
14	989
8.5	908

NIC - NO INLET CONTROLS

TABLE 7.8 SAVINGS IN TOTAL COST PER HECTARE AS COMPARED TO 42 LPS INLET CONTROL DESIGNS

INLET CONTROL	SAVINGS PER HECTARE (x $1000) A = 7.3 ha	A = 42.0 ha
42	0	0
28	2.9	3.2
19.8	4.6	5.5
14	4.7	6.6
8.5	6.4	8.6

Although indirect costs due to items such as increased street flow depths and park storage as well as the increased frequency of per operation were not quantified, the previous observations show that from an economic perspective, inlet controls can provide a significant advantage.

The main limitations in adopting lower level ICD's are increases in park storage volumes with respect to level of inlet controls. The variation of depth of flow on streets for the 5 year storm with level of inlet control is given in Table 7.2 as discussed previously.

TABLE 7.9 CHANGES IN PARK STORAGE VOLUMES WITH LEVEL OF INLET CONTROLS

LEVEL OF CONTROL (LPS)	PARK STORAGE (1000 X m^3)		PARK AREA* / TOTAL AREA (42 ha) %	
	5 YEAR	100 YEAR	5 YEAR	100 YEAR
NIC	0.8	3.9	0.20	0.9
42	0.8	4.5	0.20	1.1
28	1.0	5.9	0.24	1.4
19.8	1.4	7.5	0.32	1.8
14	1.8	9.2	0.44	2.2
8.5	2.6	11.5	0.62	2.7

* AVERAGE DEPTH IN PARK = 1 m

The area required for park storage as a percentage of the total area is based on an average depth of one metre for the depressed park, which corresponds to the present practice. It was found that even of the lowest level for 8.5 lps inlet controls, the park area is still lower than the 5% which represents the percentage of dedicated land for open space in many Canadian municipalities. For 20 lps, the area represents 1.8% and this percentage could be reduced if the average depth is further increased to 1.2m. The maximum depth of water in the gutters for a 5 year storm in Table 7.2 shows a more substantial increase mainly for the larger area and ICD of less than 20 lps. The maximum area contributing to park outlet is therefore critical. For the larger area, the ICD level should be increased. Examination of the project in Table 7.1 shows that contributing areas to a park storage or major system outlet vary significantly.

An other interpretation for the ICD detailed analysis in the system (Figure 7.1) is to compare flows for various level on controls with flows in a traditional system with different return periods. Design storms with various return periods were used as an input in URBHYD and resulting flows were compared with the total pipe flow for two areas with various ICD's. Results indicate that the 2 year storm corresponds to an ICD level of 20 lps. There is significant experience with sewers designed for two year return period and in general traffic inconvenience was not a problem. In the proposed approach basement flooding is avoided since sewers don't surcharge. Consequently the reduction to 20 lps seems possible.

GLOSSARY

CATCHBASIN - A chamber or well, usually built at the curbline of a street to intercept the gutter flow. It has sump (located below the outlet pipe), designed to retain sediment and debris.

COMBINED SEWER - A sewer receiving both surface runoff and sewage.

CURB OPENING INLET - Vertical opening in the curb face designed to intercept surface flow.

INLET - A structure that provides an entrance for surface water into a drain located below ground.

INLET GRATE (CATCHBASIN COVER) - Framework of bars over an inlet or catchbasin for admitting surface water.

INLET CONTROL DEVICE, FLOW REGULATOR, FLOW CONSTRICTOR - A device to limit the flow into the sewer system.

LATERAL - A sewer which discharges into another sewer or branch without a tributary flowing into it.

SANITARY SEWER - A sewer which normally carries domestic sewage.

SEWER - A pipe or conduit generally closed, for carrying sewage and other waste liquids.

STORM SEWER - A sewer which carries stormwater and surface water but excludes sewage and other industrial wastes.

CROMAC ICD - Brand name for a trapezoidal shaped orifice flow regulator placed in a catchbasin's outlet pipe.

CATCHBASIN LEADER, LEADER PIPE - The pipe connecting the catchbasin with the sewer.

DUAL DRAINAGE SYSTEM - The surface and subsurface drainage system in an urban area. The major and minor systems.

HYDROVEX - Product name for a flow controller placed in the catchbasin outlet. It causes the discharge to vortex before discharging.

HANGING TRAP - A device where the orificeis submerged below the catchbasin outlet.

MINOR SYSTEM - Sewer pipe system that has been designed to have a conveyance capacity for a return period between 2 years and 10 years.

MAJOR SYSTEM - The street system used to convey the flow when the minor system capacity is exceeded. The major system is designed for return periods up to 100 years.

SCEPTER ICD - Product name for a diamond shaped orifice, flow regulator.

STORMWATER MANAGEMENT - The utilization of runoff control techniques to reduce peak flows caused by urbanization.

References

1. Andrew Brodie and Assoc., "PlanHymo and FastHymo", Thornhill, Ontario, 1984.

2. Bouchard M. and Townsend R.D., "Laboratory Tests on the Influence of Grating Pattern on Street Inlet Hydraulics", CSCE Annual Hydraulics Conference, June 1984.

3. Cheung P., "A Standard Hydrograph Method for the Preliminary Analysis of Stormwater Management Projects", M.A. Sc. Thesis, Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, 1982.

4. Chin N., "Comparison of OTTHYMO and OTTSWM", Department of Civil Engineering, University of Ottawa, Unpublished Report, May 1983.

5. Corey L., City of Nepean, Engineering Dept, Personnel Communication, 1985.

6. Crosmas R., City of Scarborough, Engineering Dept., Personnel Communication, 1985.

7. Dessau, 'Etude Micro Saveriol', Ville de Laval, Les Consultants Dessau Inc., December 1984.

8. Donahue and Assoc. Inc., "Flow Regulator Pilot Study Runoff Control Program Howard Street Sewer District", Village of Skokie, Illinois, March 1984.

9. Geomec Inc., "Application des micro-ordinateurs dans les plans de drainage SIMHYD: Un modele pour l'hydrologie urbaine destinee aux ingenieurs municipaux", Societe canadienne d'hypotheques et de logement, Ottawa, juin 1985.

10. John Hopkins University, "The Design of Storm-Water Inlets", Department of Sanitary Engineering, June 1956.

11. Joint Committee of the American Society of Civil Engineers and the Water Pollution Control Federation, "Design and Construction of Sanitary and Storm Sewers", American Society of Civil Engineers, New York, 1960.

12. Kassem, A., 'Development and Application of Simultaneous Routing Model for Dual Drainage", Ph.D. thesis, Department of Civil Engineering, University of Ottawa, 1982.

13. Keliar D., Town of Markham, Engineering Department, Personnel Communication, 1984.

14. Lager J., Smith W.G. and Tchobanoglous G.,"Catchbasin Technology Overview and Assessment", Environmental Protection Agency, Report No. EPA - 600/2-77-051, May 1977.

15. Larson C.L., "Grate Inlets for Surface Drainage of Streets and Highways", St. Anthony Falls Hydraulic Laboratory, Minneapolis, 1944.

16. Marsalak J., "Sewer Inlets Study - Laboratory Study of Selected Inlets", Environmental Hydraulics Section, National Water Research Institute, Burlington, Ontario, July 1980.

17. Marsalak J, "?Highway" Drainage, 1983.

18. Moss D., "Inlet Controls for Catchbasins", M.Eng. Thesis, University of Ottawa, 1980.

19. Mukherjee D. et al, "Stormwater Management in Markham", Markham Department of Engineering, 1983.

20. Ontario Ministry of the Environment, "Ontario Drainage Design Guidelines", Draft 1983.

21. Pisano W.C., 'An Overview of Four Inlet Control Studies for Mitigating Basement Street Flooding in Cleveland and Chicago Areas', Presented to ASCE Luncheon Meeting, Cleveland, November 1982.

22. Pisano W.C., 'Study of Small Scale Technology Alternatives for Reducing Localized Flooding in Evanston, Illinois', Hydro International Engineering, Boston, November 1982.

23. P'ng, C.E., 'Conceptual Hydrologic Modelling for Master Plans in Urban Drainage', M.A. Sc. thesis, Department of Civil Engineering, University of Ottawa, 1982.

24. Richards R., "Catchbasin Maintenance Cuts Flood Complaints", Public Works, April 1976.

25. Townsend R.D., Wisner P., and Moss D., "Inlet Control Devices for Stormsewer Catchbasins: A Laboratory Study", Canadian Hydrology Symposium, Toronto, May 1980.

26. Townsend R.D., "A Novel Inlet Control Device for Storm Sewer Systems", Proceedings, International Symposium on Urban Hydrology, Hydraulics and Sediment Control, Lexington, Kentucky, July 1984.

27. U.S. Department of Transportation, "Design of Urban Highway Drainage - The State of the ARt", Federal Highway Administration, Office of Research and Development, (FHWS-TS-79-226), August 1979.

28. Wisner, P.E. and Kassem, A.M., "Street Overland Flow and Inlet Control", Canadian Society for Civil Engineering, 1980 Annual Conference, Winnipeg, Manitoba, Canada, May 1980.

29. Wisner P., "IMPSWM - Implementation of Stormwater Management Procedures for Urban Drainage Modelling - 2nd Edition", University of Ottawa, February 1983.

30. Wisner P. and Hawdur M., "Evaluation of Urban Drainage Methods for Basement Flood Proofing". A report prepared for Canada Mortgage and Housing Corporation, October 1983.

31. Wisner P.E. and P'ng, C.E., 'OTTHYMO - A Planning Model for Master Drainage Plans in Urban Areas', Proceedings First International Seminar on Urban Drainage Systems, Southampton, U.K., Pitman Advanced Publishing, 1982.

32. Wisner et al, "New Approaches in Drainage Design".

33. Wright-McLaughlin Engineers, "Urban Storm Drainage Criteria Manual", Denver Regional Council of Governments, Denver, Colorado, 1968.

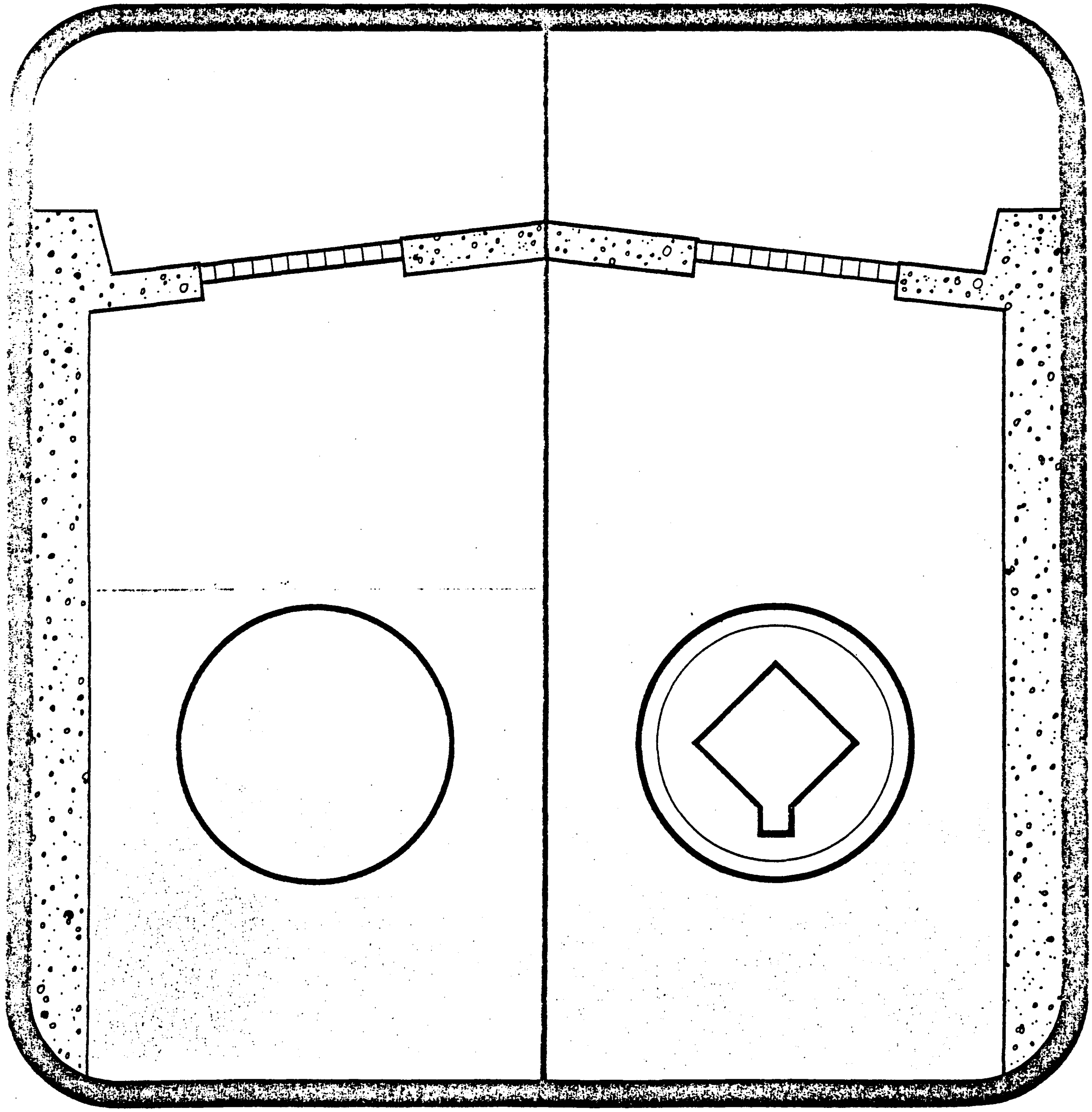
Appendix A

EXAMPLES OF MANUFACTURERS BROCHURES

Scepter

ICD - Inlet Control Device

A SIMPLE ADDITION TO NEW OR EXISTING STORM SEWER SYSTEMS TO ELIMINATE FLOODED BASEMENTS BY TEMPORARILY DIVERTING EXCESS RAINFALL TO THE SURFACE



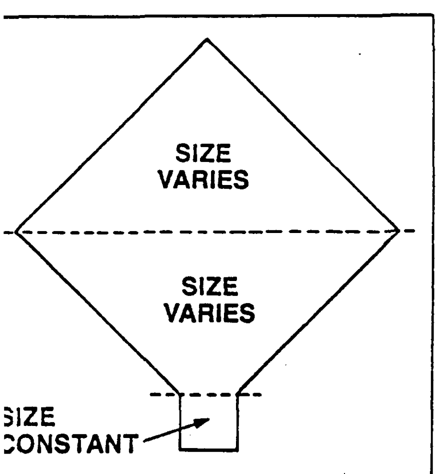
SCEPTER ICD Controls storm water

:ESCRIPTION

he Scepter Inlet Control Device CD) is a fabricated or injection nolded PVC flow orifice. eveloped in the Department of ivil Engineering, University of)ttawa, the ICD is available in vo standard versions to restrict ne flow of storm water to 0.7, 1.0, nd 1.3 cu. ft./sec. (19.8, 28.3, nd 36.8 litre/sec.) at a pre-etermined vertical head of 4 feet .22 m). Other flow-against-head equirements can be met on pecial order.

:ESIGN FACTORS

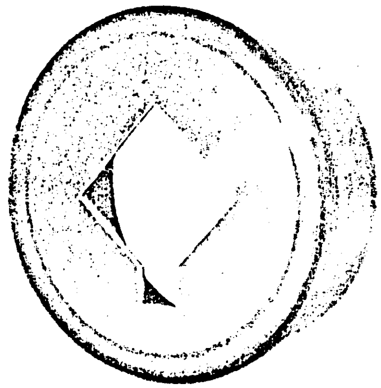
'he unique compound orifice hape promotes self-cleaning ction in debris-laden flows, specially important in the critical arly stages of flow capture. Vhen submerged, the sharp orners of the orifice contract the ow, which helps to "centre" the ebris as it traverses the plane of ne orifice.



he dimensions of the orifice are djusted in accordance with a esign discharge formula to roduce the required flow at the ated vertical head.

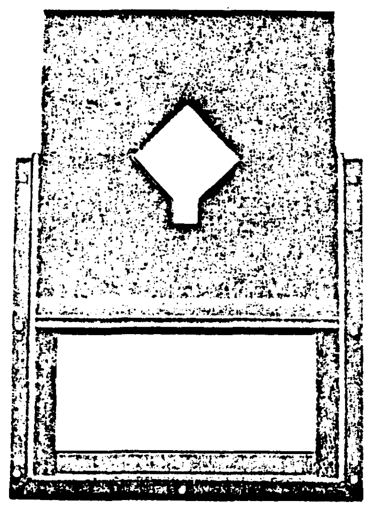
ADAPTABILITY

The Scepter ICD is available in two versions:



PLUG ICD

A short, slightly-tapered plug for insertion in the outlet pipe from the catchbasin (the catchbasin lead). It is held in place by friction and hydrostatic pressure. Made to fit 8", 10" or 12" pipe in any material (clay, A-C, concrete, PVC, etc.). The orifice plate is flush with the end of the plug.



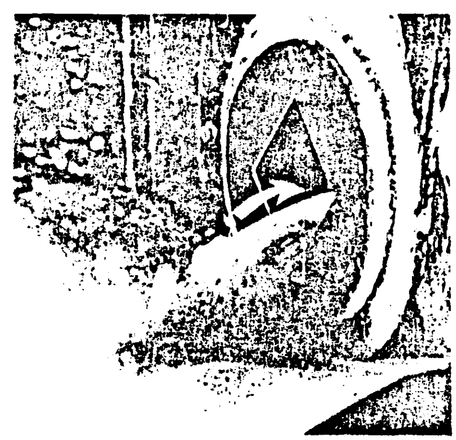
FRAMED ICD

A plate containing the orifice is held in channels in a metal frame. The framed ICD is installed over the outlet pipe from the catchbasin. Both versions of the ICD can be removed for inspection. As installed, they do not limit access to the catchbasin.

ELIMINATES BASEMENT FLOODING AND FOUNDATION SEEPAGE

The patented* function of the ICD is to control surcharging of storm sewers by restricting the flow into the sewer pipe. In the normal course of events drainage system surcharging is unavoidable. During major storms a surcharged sewer may back up into foundation drains (or basement drains in combined systems) and the result is a public outcry against "inadequate" sewer systems. Designing for "100-year storms" or even "five-year storms" can be a costly alternative to simply diverting excessive rainfall to temporary surface storage, and away from the community's basements.

SUMP SCOURING ACTION



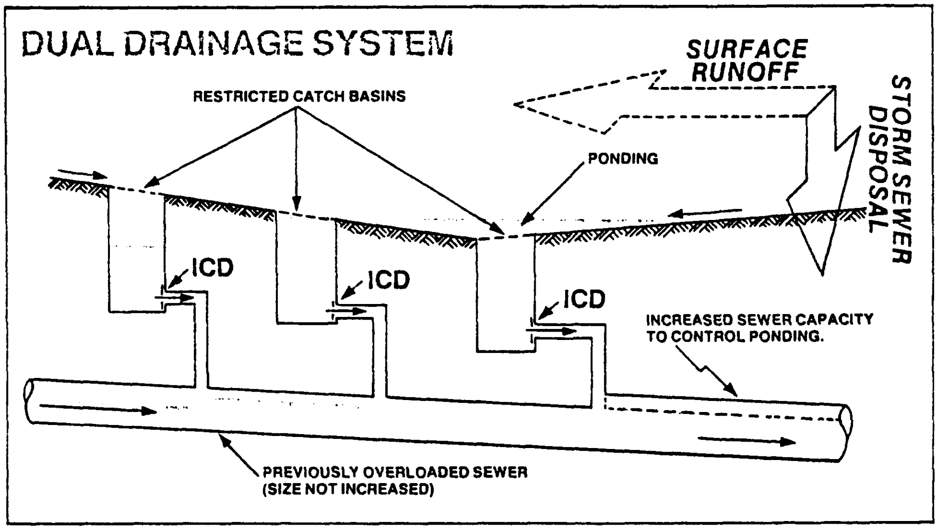
The rectangular slot at the bottom of the orifice works effectively in two ways. First, during dry periods it draws the water level below the main orifice area keeping it clear of floating debris. Second, it generates strong vortex action in the approach flow during heavy rainfalls, which vigorously scours sediment from the sump of the catchbasin and away from the orifice. Field trials and laboratory testing prove this action.

*CANADIAN PATENT 1165207 [redacted]
MEXICO PATENT PENDING

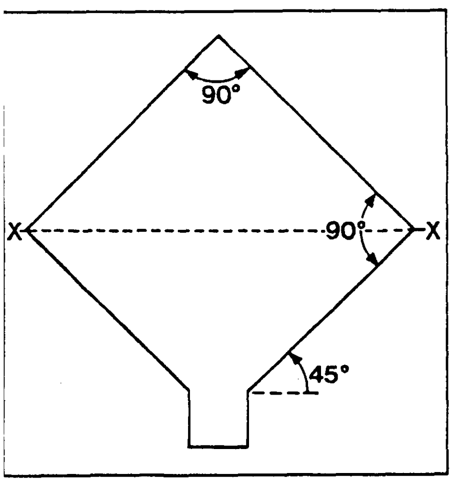
SCEPTER ICD
Preserves storm sewer capacity

THE SCEPTER ICD ROLE IN THE "DUAL DRAINAGE SYSTEM"

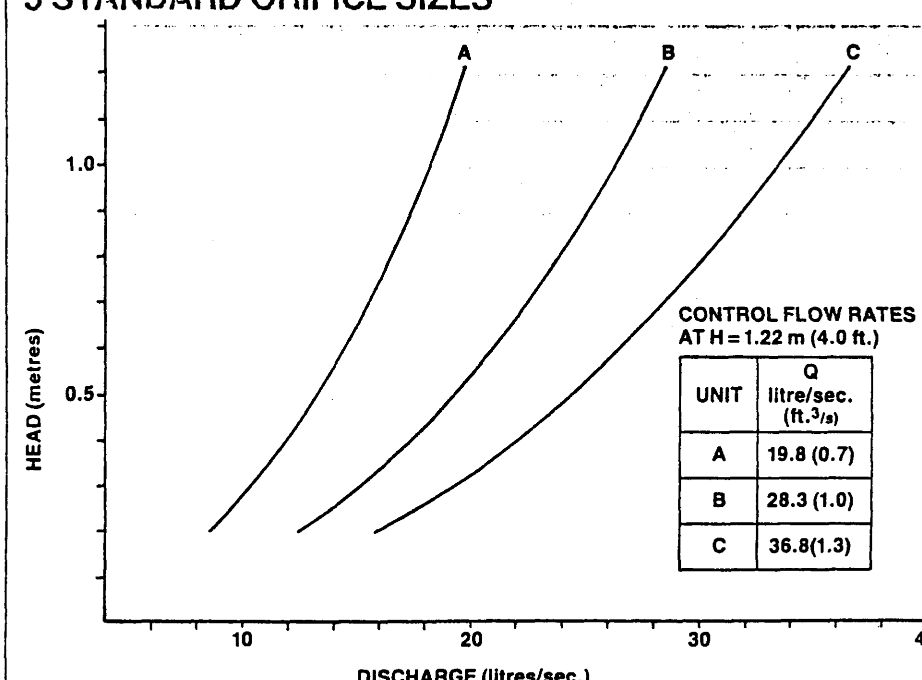
Typically the disposal of storm water is via overland flow and an underground pipe system. A rational design of storm sewer systems will consider not just the capacity of the storm sewer itself, but also the hydraulic characteristics of the sewer inlets and the potential for overland flow via streets and other surface drainage features. A community's storm water disposal system can be modelled on computer. (Your Scepter Representative can suggest a contact for this service.) From such an analysis will emerge an optimum design of pipe size, location of Scepter ICD's, depth of ponding, and the duration and spread of ponding. Together with the strategic location of parkland and proper street grading, the use of Scepter ICD's plays a key role in the elimination of flooding by controlling sewer inlets. Studies in a number of communities show that systems designed with the "dual drainage" principle using Scepter ICD's can avoid surcharging during flows having return periods up to 100 years. Existing systems can make use of this principle if suitable temporary storage facilities are introduced. Relief sewer projects for existing systems can also use the ICD's. In the above figure only the lower conduit has an increased capacity. Without ICD's, the entire system would have required changes.



HYDRAULICS



The head is measured from the centreline of the triangle (X-X) to the catchbasin inlet (flood level). Calibration curves for the three standard orifice sizes under various heads are shown on the right.



UNIT	Q litre/sec. (ft.³/s)
A	19.8 (0.7)
B	28.3 (1.0)
C	36.8(1.3)

WARRANTY

All Scepter products are guaranteed against defects resulting from faulty workmanship or materials. If any such product is found to be defective by reason of faulty workmanship or materials, upon written notice and return of the product, the defective product will be replaced by Scepter free of charge, including shipping charges for the replacement product. Claims for labour costs and other expenses required to replace such defective product or to repair any damage resulting from the use thereof will not be allowed by Scepter.

Our liability is limited to the price paid for the defective product. Scepter Mfg. Co. Ltd. shall not be bound by any warranty other than above set forth unless such warranty shall be in writing. This literature is published in good faith and is believed to be reliable. However, Scepter Manufacturing Company Limited, does not represent and/or warrant in any manner the information and suggestions contained in this brochure. Data presented is the result of laboratory tests and field experience.

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SCEPTER IS A MEMBER OF
THE UNI-BELL PVC PIPE ASSOCIATION

ICD 9/84

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Annacis Island Estates,
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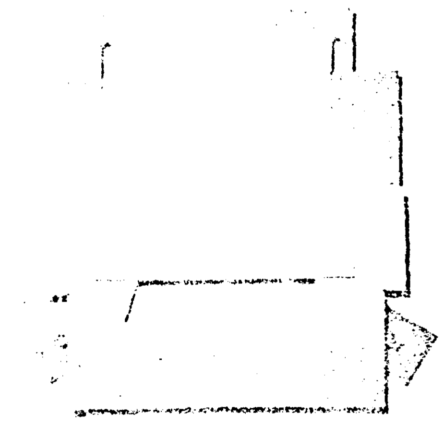


CROMAC ENTERPRISES

VARIABLE FLOW CONTROL DEVICE OR V.C.D.

The Cromac V.C.D. is capable of discharging a range of flows. (see table)

Rating Curve	Discharge (cfs)	(m^3/s)	Gate Opening (mm)
	0.5	.0142	18
	1.0	.0284	44
	1.5	.0426	72
	2.0	.0567	91
	2.5	.0709	108
	3.0	.0851	140

For What Head?

Environment Canada Hydraulics Division Test Results.

WHERE IT CAN BE USED

The Cromac V.C.D. is ideal for use in new areas of development to segregate the major and minor flows.

It is also very applicable in a retrofit situation where surcharging of the existing storm sewer is presently occurring.

The proper arrangement of these devices can limit the sewer capture to it's carrying capacity.

One of the advantages of the Cromac V.C.D. is its capability of being monitored and adjusted to optimise the efficiency of the entire sewer network.

WHERE IT IS INSTALLED IN CATCHBASIN

The Cromac V.C.D. is mounted on the inside wall of the catchbasin over the outgoing discharging pipe.

MATERIAL

The Cromac V.C.D. is completely constructed of 10 mm PVC. Nylon bolts and washers are used ensuring a long lasting, rust free, non-corrosive installation.

The Cromac V.C.D. weir plate and orifice plate were constructed with a sharp edged orifice in order to impede the hydraulics and thereby increase the opening size for the minimum desirable flow rate reducing the susceptability to clogging.

ANCHORING SYSTEM

Oaks Precast Industries (Guelph, Ontario) cast catchbasins with arranged anchoring nuts to receive the Cromac V.C.D. ensuring a quick, secure installation.

TEST RESULTS

The Cromac V.C.D. underwent intensive testing at the Government of Canada Environmental Branch Hydraulics Division.

Clog tests were conducted for (3) three orifice openings h=18mm, h=72mm and h=140mm. Two types of debris materials were used in the test, plastic squares and straw

The plastic squares were carried through the opening for all (3) three settings with no flow retardation.

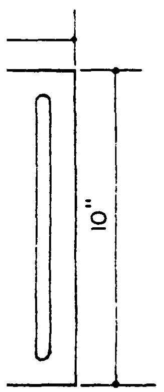
The straw debris, tested at the smallest opening stayed inside the catchbasin floating on the surface. At the intermediate opening most of the straw was carried through the opening. The tests on the largest opening were observed to be the same as the intermediate setting.

For all (3) three openings subjected to the straw clog test the hydraulics were not adversely affected.

REMOVABLE WEIR & ORIFICE PLATE

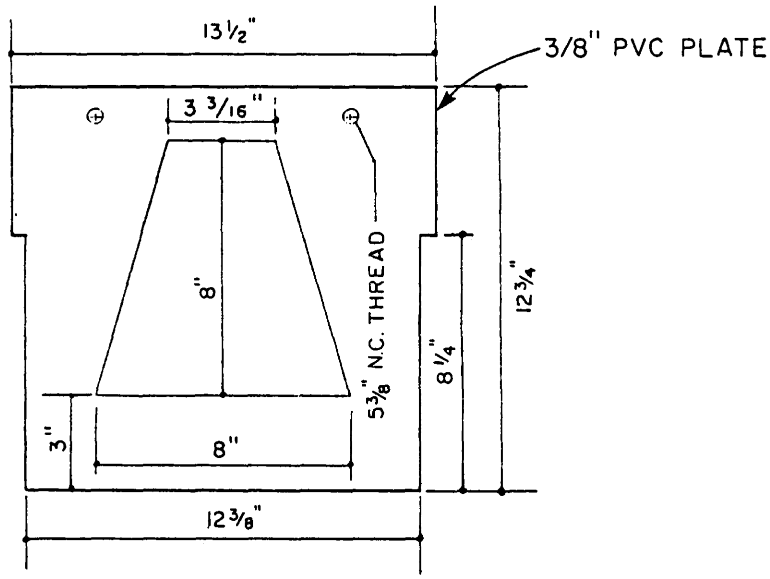
If for some reason a plugging of a unit occurs in the field the maintenance crews can simply lift the catchbasin top and with a hook, remove the weir and orifice plate, allowing the water to flow free.

WEIR



PVC PLATE

ORIFICE PLATE DETAIL



NOTES

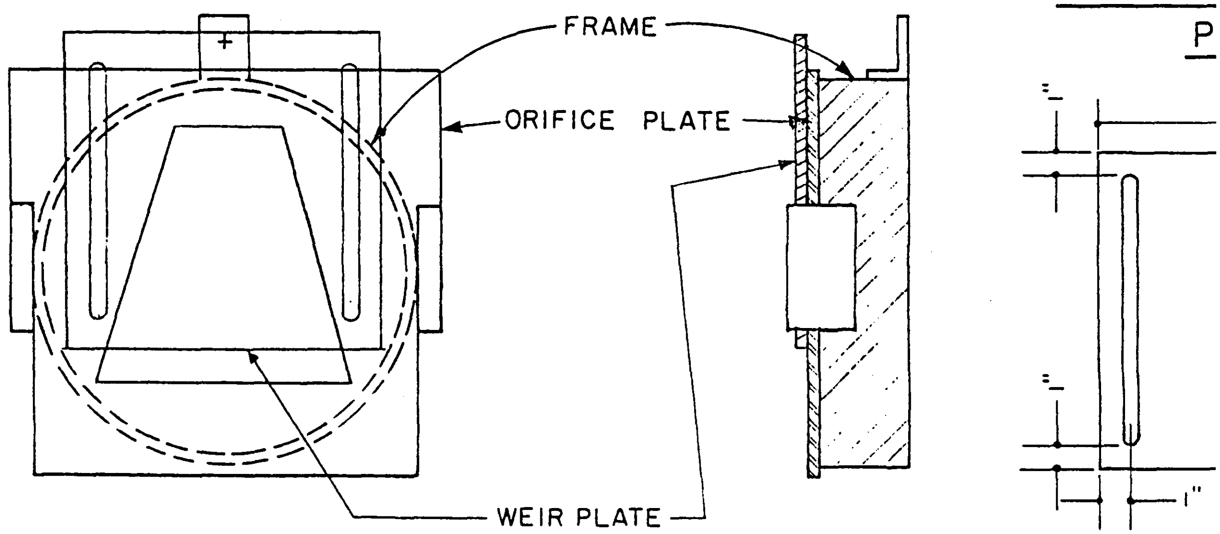
1. WEIR PLATE FASTENED TO ORIFICE PLATE WITH TWO (2) 3/8"ø x 3/4" NYLON BOLTS.
2. THREE (3) 2" x 1-1/4" x 1-3/4" ANGLES & RETAINER TO BE SOLVENT WELDED, TO BE TREATED WITH PVC PRIMER PRIOR TO PVC SOLVENT WELDING CEMENT AND HEAT WELDED.
3. LOCATE IN CATCHBASIN WITH CENTRE OF ORIFICE AT CENTRE OF OUTLET PIPE.

cromac enterprises

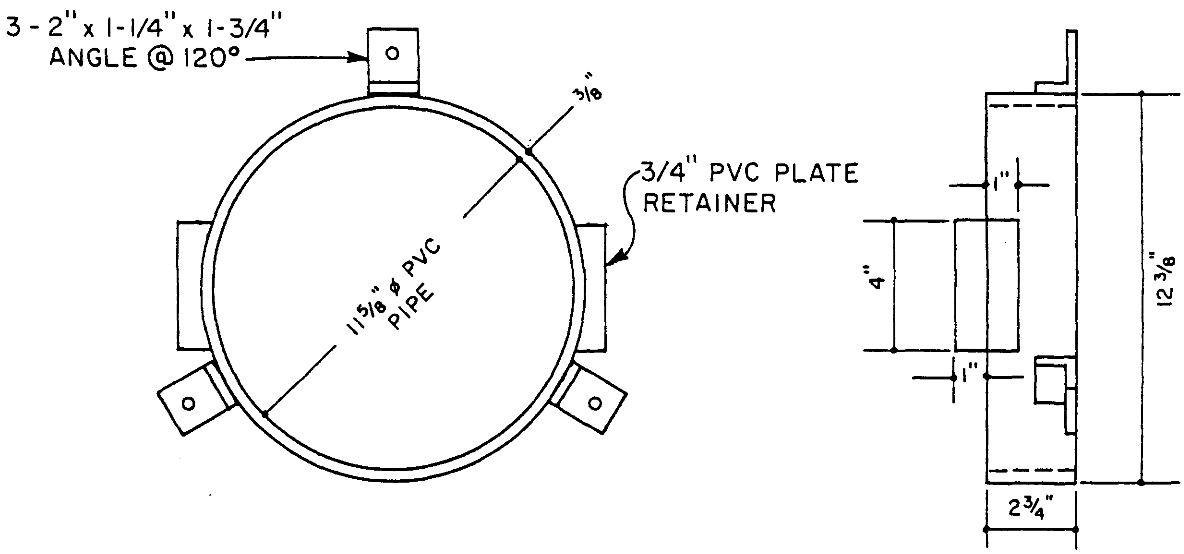
VARIABLE FLOW CONTROL DEVICE	
Drawn By; B Crossmul	Date Jan /83
Chk'd By; D McQ	Scale

ASSEMBLED UNIT

ADJUST
P



FRAME DETAIL



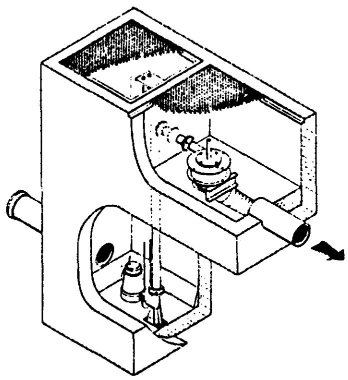
HYDROVEX

FLOW RESTRICTOR – FLOW REGULATOR
FLOW CONTROLLER IN CLOSED LOOP SYSTEMS

COMBINED SEWER REGULATION
SANITARY FLOW INTERCEPTION
STORMWATER MANAGEMENT
FLOOD CONTROL

JOHN MEUNIER INC.

HYDROVEX PUMP THROTTLE MODEL PV and PK



Description:
Flow throttling with light super quadratic characteristics.

Principle:
Vortex flow, with little turbulence and high flow resistance, is caused by the housing shape.

Features:
Self cleaning effect.

Installation:
Dry, in separate regulating chamber.

Materials:
Mild steel, hot dip galvanized and epoxy coated, or stainless steel.

Moving parts:
None.

External energy:
None.

Throughput adjustability:
Type PV not adjustable, type PK continuously adjustable with ratio 1 to 2 nominal capacity.

Remote control:
No.

Options:
None.

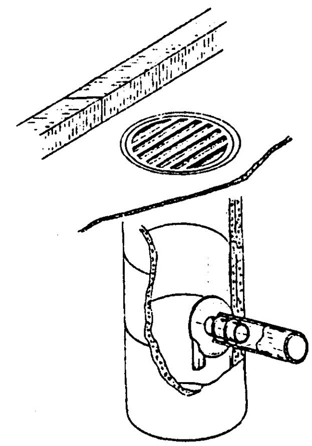
Maintenance:
Low.

Application:
Flow variation attenuation on small sewage pumping stations.

Discharge range
(for comparison):
Nominal diameter 200 mm with 2 m head.
Q = 34 - 68 L/S.

Nominal diameter 8" with 8'-2" head.
Q = 1.20 - 2.40 CFS.

HYDROVEX VORTEX VALVE MODEL CB



Description:
Discharge throttling with S-shaped discharge curve.

Principle:
Flow through effect resulting in changes in the flow resistance.

Features:
Large passagė, reducing risk of cloggage.

Installation:
Wet, in standard catch basin or manhole.

Materials:
Stainless steel or plastic.

Moving parts:
None.

External energy:
None.

Throughput accuracy:
±3%.

Discharge adjustability:
Continuous with ratio 1 to 2 nominal capacity.

Remote control:
No.

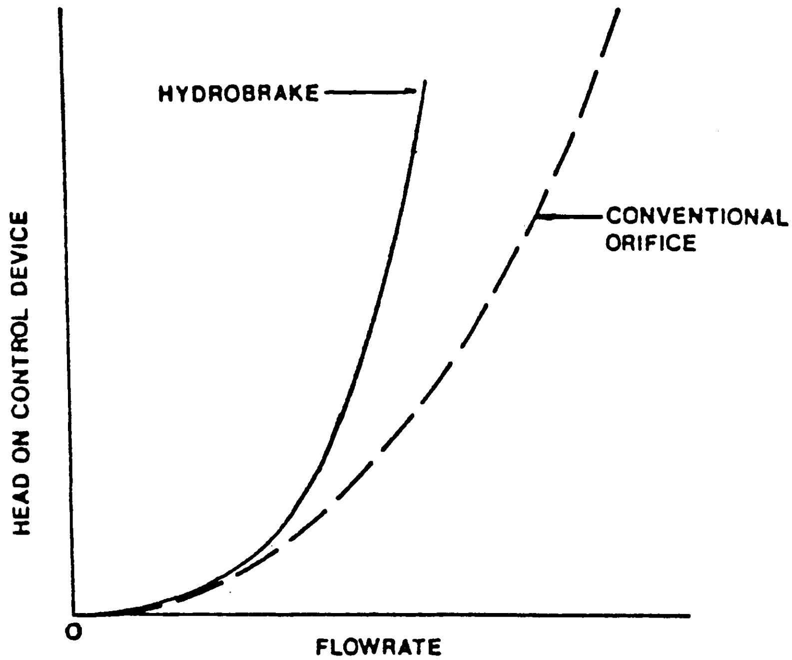
Options:
None.

Maintenance:
Low.

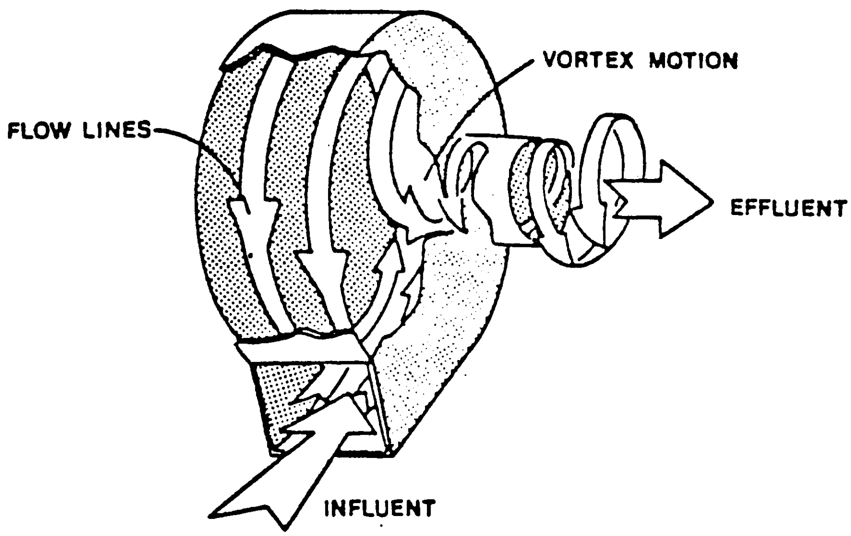
Application:
Stormwater management — inlet control. Surface or in system retention.

Discharge range
(for comparison):
Passage 65 mm diam. with 1.8 in head. Q = 2.8 L/s.

Passage 2.5" diam. with 6' head. Q = .1 CFS.



A - CONCEPTUAL PERFORMANCE COMPARISON OF THE HYDROBRAKE AND A CONVENTIONAL ORIFICE



B - SCHEMATIC OF HYDROBRAKE OPERATION SHOWING PRINCIPAL OF VORTEX FLUID MOTION

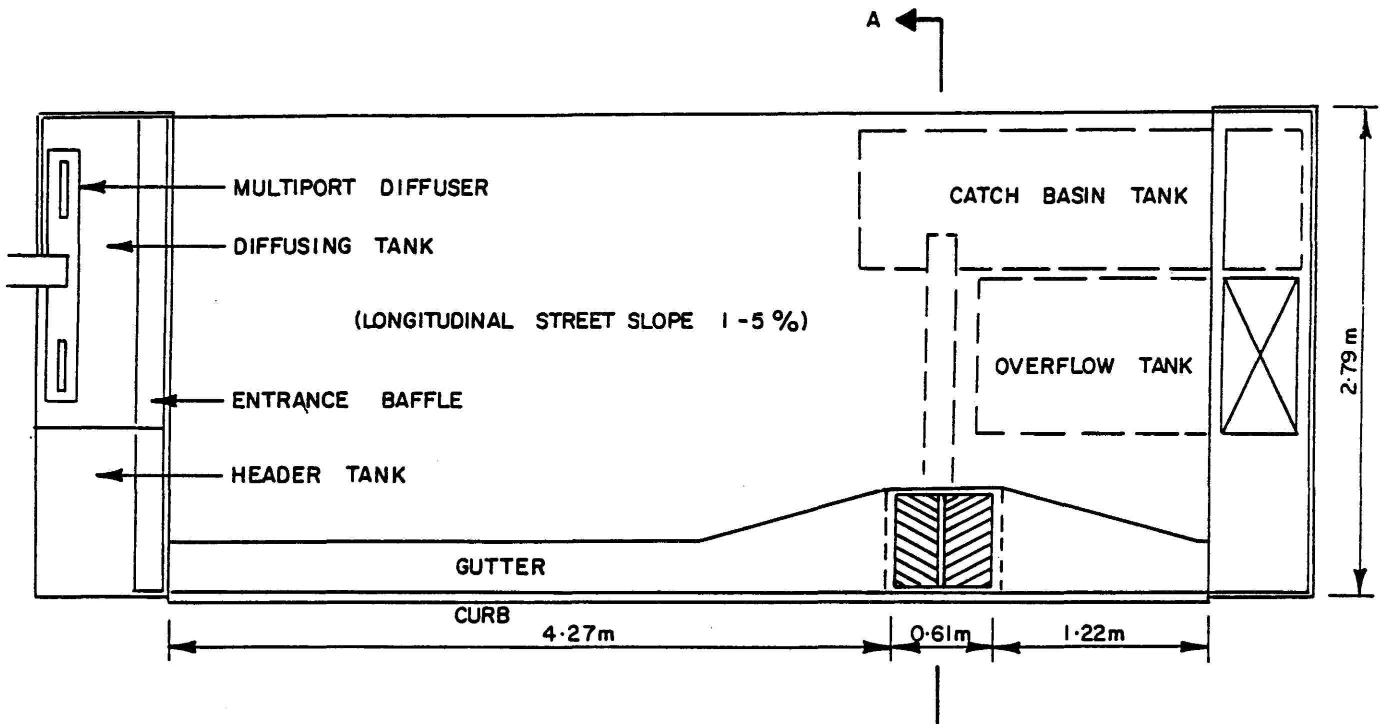
HYDROBRAKE APPLICATION

Appendix B

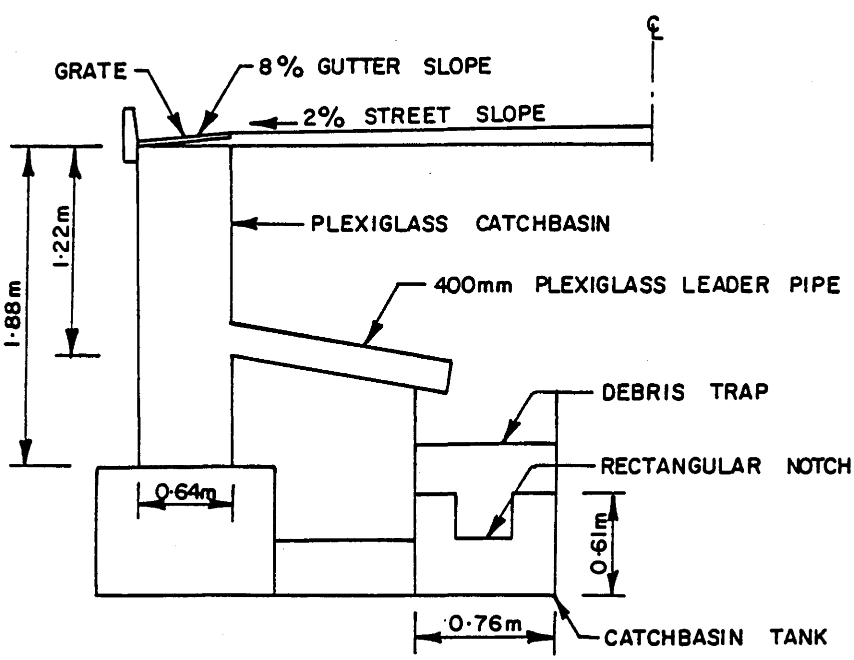
EXPERIMENTAL SETUP

The experimental setup consists of a full scale street and catchbasin model, Figure B.1. A pump and flow measuring weir were used to convey the flow to the street level and measure the catchbasin discharge.

The pump draws water from a large sump located below the laboratory floor. The discharge flow is directed to the model, the flow rate being regulated by a valve on the discharge line. The model consists of a portion of a street surface 2.7m wide and 6.1m long that directs surface flow to a plexiglass catchbasin with dimensions 0.6m x 0.6m x 1.8m. The catchbasin has a sump, 0.6m in depth for trapping debris. The end of the street terminates in an overflow tank. Adjustments can be made to the street slopes both in the longitudinal and transverse directions. The slopes were not varied throughout the testing of the inlet control devices. The flow is pumped to a header tank at the upstream end of the model. When flow leaves the tank it flows over the street surface, down the gutter to the catchbasin. A second testing method was to introduce a discharge supply pipe directly into the catchbasin, thus avoiding flow over the street. Flow entering the catchbasin resulted in much lower turbulence. After entering the catchbasin water discharged through the outlet pipe into a collecting tank. A screen was installed at the entrance end of the tank to capture exiting debris. Discharge is measured by a rectangular weir located in the collecting tank. Manometers were installed in the catchbasin to measure head on the flow regulator and also in the collecting tank to measure the flow rate. The depth of water above the orifice center line was taken as the head on the orifice. Each unit was tested by mounting it in the catchbasin outlet, then introducing flow and debris into the catchbasin.



PLAN



SECTION A·A

Figure B - 1 The Physical Model

Appendix C

Dual Drainage Computer Model - OTTSWM

Most urban storm drainage models assume that all the catchment runoff is transferred directly into the minor system. they have been developed for designing or analyzing systems with low return period rainfall events. However, if the dual drainage concept is to be employed, drainage systems must be designed for events with a high return period. In this case, all the storm water runoff is not captured by catch basins and transferred into the pipe system. A portion of the street flow is captured and transferred to the pipe system while the remaining carry over flow is transported by the streets.

The OTTSWM model was designed specifically for analyzing dual drainage systems. The program has the capability of determining the surface flow, the hydraulic capture by catch basin inlets and the pipe flow. It can be used in four modes:

i) to determine pipe sizes for free surface flow

ii) to analyze on existing system or proposed design and resize pipes to maintain free surface flow,

iii) to determine the level of inlet restriction to maintain free surface flow in pipes,

iv) to conduct a pipe surcharge analysis.

Whatever mode of operation is being used, the basic assumptions of the model remain the same. The model conducts an analysis of two interconnected systems, the surface or major system and the pipe or minor system. Since the computations are done on two levels, the surface and sewer network flows do not necessarily have to be in the same direction.

The major system is formed by the street network and must fol-

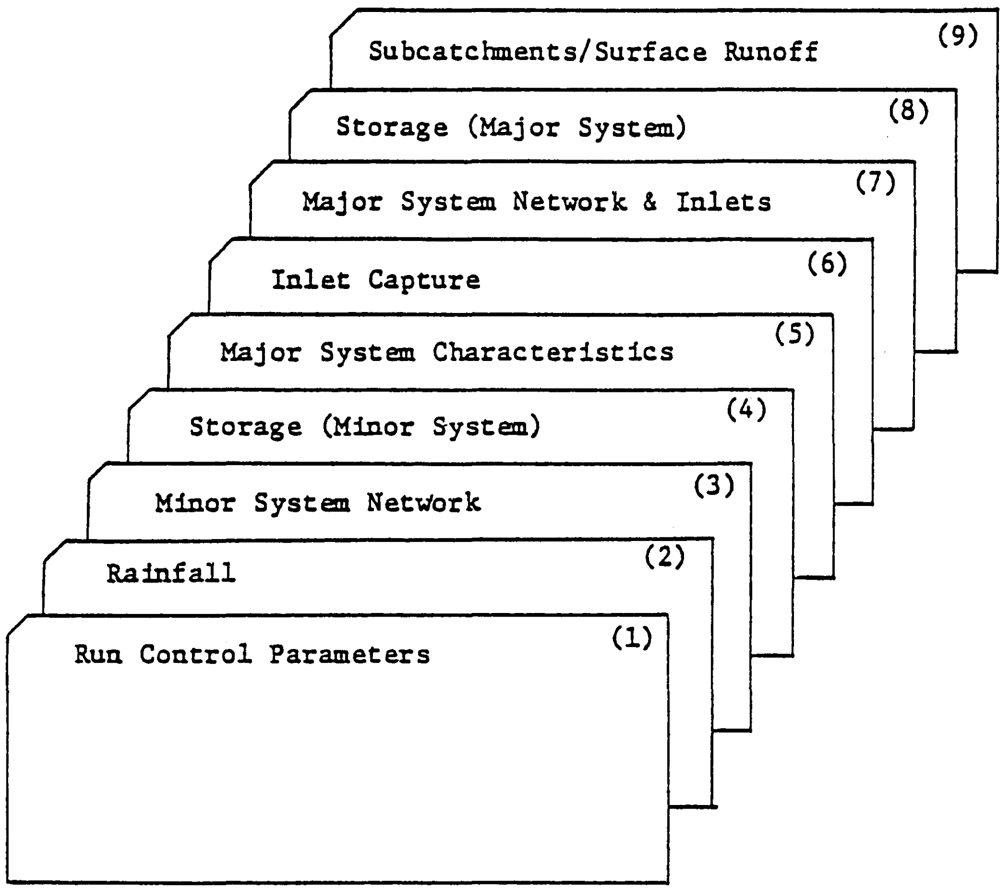


Fig. C1 Main Card Groups For Input Data to OTTSWMM

OTTSWMM EXAMPLE INPUT

```
)EVELOPMENT
EAR STORM 1.0 CFS INLET CONTROLS
    5   48    3    0    0  1.0
   48
  .18  .20  .20  .23  .23  .27  .27  .33  .33  .44  .44  .66  .66 1.62  1.6
 8.11 2.15 2.15 1.11 1.11  .76  .76  .58  .58  .48  .48  .41  .41  .36   .3
  .32  .28  .28  .26  .26  .24  .24  .22  .22  .21  .21  .20  .20  .18   .1

 13.9  .02   .5 .013 .005  .05 .025  1.5
 13.9  .02   .5 .013.0075  .05 .025  1.5

   11
    0    1  .67    2 1.04    3  1.3    4 1.55    5 1.75    6  1.9    7   2.
  2.1  120  2.1  300  2.1
   13
    0    1    1    2    2    3    3    4    4    5    5    6    6    7   7.
   20  120  120  300  300  500  500 1500 1500
  101  105  148    1    2    1        12             0
  501  104  164    1    2    2   .5   13             0
  104  105  356    1    2    1        13             0
  102  103  420    1    4    1        14             0
  103  105  105    1    2    1        14             0
  105  106   97    1    0    1        12             0
  510  106  164    1    2    2        11             0
  106  107  176    1    2    1        11             0
  509  107  164    1    2    2   .5   10             0
  107  108  164    1    1    1        10             0
  508  108  164    1    2    2   .5    9             0
  108  112  149    1    1    1         9             0
  109  110  148    1    3    1        16             0
  110  112  197    2    0    1        15             0
  111  112  459    1    2    1         6             0
  112  113  194    1    1    1         6             0
  507  113  164    1    2    2   .5    5             0
  113  115  243    1    2    1         5             0
  115  126   49    1    2    1         4             0
  126  129  328    1    2    1         2             0
  116  129  328    1    6    1         3             0
  502  118  164    1    2    2   .5   22             0
  118  119  315    1    2    1        22             0
  503  119  164    1    2    2   .5   21             0
  119  120   66    2    0    1        21             0
  504  120  164    1    2    2        20             0
  120  121  240    1    2    1        20             0
  505  121  164    1    2    2   .5   19             0
  121  123  240    1    2    1        19             0
  122  123  120    1    2    1        18             0
  123  127  380    1    4    1        18             0
  506  127  164    1    2    2        17             0
  127  129  328    1    0    1         1             0
  129  130  328    1    0    1         1             0
  130  131  328    1    0    1         1             1

  3     .52  .00115

    1  101 0.35   30 .013  .25 .003  296 .062 .184    0
    2  501  .49    2 .013  .25 .003  328 .062 .184    0
    3  104 1.23   30 .013  .25 .003  210 .062 .184    0
    5  102 3.36   20 .013  .25 .003  840 .062 .184    0
    6  103  .27   30 .013  .25 .003  210 .062 .184    0
    7  510  .99    2 .013  .25 .003  328 .062 .184    0
    8  105  .30   30 .013  .25 .003  194 .062 .184    0
```

1	101	0.35	30	.013	.25	.003	296	.062	.184	0
2	501	.49	2	.013	.25	.003	328	.062	.184	0
3	104	1.23	30	.013	.25	.003	210	.062	.184	0
5	102	3.36	20	.013	.25	.003	840	.062	.184	0
6	103	.27	30	.013	.25	.003	210	.062	.184	0
7	510	.99	2	.013	.25	.003	328	.062	.184	0
8	105	.30	30	.013	.25	.003	194	.062	.184	0
9	106	.69	30	.013	.25	.003	352	.062	.184	0
10	509	.39	2	.013	.25	.003	328	.062	.184	0
11	107	.54	30	.013	.25	.003	328	.062	.184	0
12	508	.49	2	.013	.25	.003	328	.062	.184	0
13	108	1.09	30	.013	.25	.003	298	.062	.184	0
14	109	.62	30	.013	.25	.003	296	.062	.184	0
15	110	.67	30	.013	.25	.003	394	.062	.184	0
16	111	1.01	30	.013	.25	.003	918	.062	.184	0
17	112	.89	30	.013	.25	.003	388	.062	.184	0
18	507	.37	2	.013	.25	.003	328	.062	.184	0
19	113	1.04	30	.013	.25	.003	486	.062	.184	0
21	115	.17	2	.013	.25	.003	328	.062	.184	0
22	116	6.23	30	.013	.25	.003	656	.062	.184	0
23	502	.39	2	.013	.25	.003	328	.062	.184	0
24	118	.94	30	.013	.25	.003	630	.062	.184	0
25	503	.62	2	.013	.25	.003	328	.062	.184	0
26	119	.17	30	.013	.25	.003	132	.062	.184	0
27	504	1.06	2	.013	.25	.003	328	.062	.184	0
28	120	.99	30	.013	.25	.003	480	.062	.184	0
29	505	.49	2	.013	.25	.003	328	.062	.184	0
30	121	.84	30	.013	.25	.003	480	.062	.184	0
31	122	.32	30	.013	.25	.003	240	.062	.184	0
32	123	1.36	30	.013	.25	.003	760	.062	.184	0
33	506	1.33	2	.013	.25	.003	328	.062	.184	0

The major system is formed by the street network and must follow a dendritic pattern converging to a downstream outlet. The major system should be continuous, no water ponding is allowed except at storage locations. In new subdivisions, streets can be designed so that low points are avoided. Existing developments often have low points. In recognizing this the model permits two types of inlets. Normal inlets are those where flow partly enters the minor system and is partly passed down the major system. Storage inlets are located at low points, all of the water enters the minor system at these inlets.

The sewer network or minor system can follow either a dendritic or looped pattern. When a new system is being designed, the pipes should follow a dendritic pattern. More sophisticated analysis of looped or surcharged pipes is possible with the EXTRAN (Extended Transport) submodel. The pipe slopes control the flow direction in the minor system. Water enters the minor system through storm inlets which are connected to manholes at sewer junctions. At each junction the flow from upstream pipes is added to the street inlet flow giving the total flow to be routed to the next segment. Pipes are sized for the peak flow at each junction.

MODEL OPERATION

The model is composed of four main submodels, a surface runoff submodel, inlet submodel, minor system submodel and the major system submodel.

The input data consisting of subareas, street segments, pipe segments, and storages are required and connectivity machines set up. The input data order is shown in Figure C.1.

The computations are down in a number of steps. First the low a dendritic pattern converging to a downstream outlet. The major system should be continuous, no water ponding is allowed except at storage locations. In new subdivisions, streets can be designed so that low points are avoided. Existing developments often have low points. In recognizing this the model permits two types of inlets. Normal inlets are those where flow partly enters the minor system and is partly passed down the major system. Storage inlets are located at low points, all of the water enters the minor system at these inlets.

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The <u>input data</u> consisting of subareas, street segments, pipe segments, and storages are required and connectivity machines set up. The input data order is shown in Figure C.1.

The computations are down in a number of steps. First the runoff for each subarea is computed using a routine borrowed from

EPA-SWM Model. This is followed by the major system routing. Starting at upstream major system segment subarea runoff is routed down the street segment with any upstream carryover flow. In conjunction with the major system routing, inlet flows are determined. Any flow captured by the inlets are stored for minor system analysis. Excess flow not captured by the inlets forms carryover flow to be routed down the following major system segment. The street segment routing and inlet capture are continued until all of the street segments have been considered.

The computations for the minor system components are performed next. The user selected input determines how the pipe system is analyzed. Design of a new system starts with the most upstream sewer segment and proceeds downstream. The pipe flow is the total of inlet capture flow and the upstream pipe flow. Design or resizing of individual pipes is performed with the MINOR submodel. When the pipes are sized, free surface and a dendritic sewer system is assumed. Pipes are selected based on the input slopes, pipe roughness and computed peak flow, using the Manning equation. The model selects the smallest commercial sewer size that will maintain free surface flow.

If the pipe surcharge analysis option is selected the inlet flows are saved in a separate file, to be used by EXTRAN subroutine. Analysis with EXTRAN can be conducted in looped pipe systems and in systems where pipes have insufficient capacity for free surface flow. Some small surcharging may be desirable during major storm events, to prevent resizing of surcharged pipes. Surcharge levels must be kept below foundations, to prevent basement flooding damage. The EXTRAN model must be used to determine if the surcharge levels are acceptable.

Storage can be provided to control either minor or major system runoff. The minor system storage may be provided by a super pipe, or other in system detention facility. For the major system, street flow is typically directed to a surface storage facility such as a depressed park or detention pond. Minor system storage is provided to obtain economies in pipe sizing or to ensure that peak flows do not exceed predevelopments levels. Major system storage is necessary to intercept street flow, and release it at a controlled rate into the minor system or receiving stream. The designer inputs the stage-discharge curve to provide the desired control, the computer output gives the required storage volumes. If a storage volume was input and it is exceeded the overflow volume is given in the output.

The basic computer output of OTTSWM provides the following information:

1. a print-out of the input data as well as a summary statistics of the watershed: number of subareas, sewers, storage units, total drainage area, etc., density of inlets (number of inlets per unit area), average distance between inlets, etc.;

2. required sizes of sewers for free surface flow conditions;

3. inlet control requirements, that is locations of inlets which may need flow constricting devices, and limiting capacities if the latter is not specified;

4. detailed simulation results for specified elements, in printed and plotted forms:
   a) time history of surface runoff,
   b) time history of major system flows and depths,
   c) time history of sewer flows;

5. a summary of simulation results including maximum flows

MAJOR SYSTEM

SUMMARY OF SIMULATION RESULTS

OTTSWMM – OUTPUT SUMMARY TABLE

	SEGMENT NO	MAX. FLOW (CFS)	MAX. DEPTH (IN)	MAX. CAPTURE (CFS)	INLET RESTRICTION	MAX. STORAGE (CF)
1	201	3.68	2.0	2.26	NO	.00
2	202	4.05	2.2	2.40	NO	.00
3	204	3.18	2.0	1.78	NO	.00
4	248	1.23	1.4	.41	NO	.00
5	206	4.32	2.3	2.16	NO	.00
6	207	1.69	1.4	.57	NO	.00
7	208	2.46	1.5	1.64	NO	.00
8	209	4.35	2.3	.00	-	.00
9	210	6.45	2.7	2.71	NO	.00
10	211	1.88	1.4	1.26	NO	.00
11	212	2.06	1.3	1.36	NO	.00
12	213	5.16	2.4	2.38	NO	.00
13	214	2.85	1.7	1.91	NO	.00
14	215	2.22	1.4	1.49	NO	.00
15	216	2.45	1.6	1.64	NO	.00
16	217	2.47	1.6	1.52	NO	.00
17	218	1.89	1.4	1.27	NO	.00
18	219	2.64	1.6	1.58	NO	.00
19	220	1.40	1.3	.94	NO	.00
20	221	2.25	1.4	1.43	NO	.00
21	222	7.21	2.8	4.47	NO	.00
22	223	1.37	1.3	.92	NO	.00
23	224	3.12	1.7	.88	NO	.00
24	225	2.24	1.5	.71	NO	.00
25	226	6.01	2.6	3.12	NO	.00
26	227	2.25	1.6	1.43	NO	.00
27	228	2.23	1.5	.71	NO	.00
28	229	.78	1.2	.00	-	.00
29	230	3.29	1.8	.91	NO	.00
30	232	3.90	2.2	1.02	NO	.00
31	233	6.25	2.6	1.33	NO	.00
32	234	1.00	1.3	.00	-	.00
33	235	2.94	1.9	1.69	NO	.00
34	236	4.24	2.3	2.14	NO	.00
35	237	3.22	2.0	1.79	NO	.00
36	238	2.65	1.8	1.58	NO	.00
37	239	1.70	1.5	1.14	NO	.00
38	240	2.74	1.8	1.61	NO	.00
39	241	2.03	1.6	1.35	NO	.00
40	242	3.11	1.8	.87	NO	.00
41	243	1.93	1.5	.00	-	.00
42	244	3.58	2.0	1.92	NO	.00
43	245	2.93	1.8	.84	NO	.00
44	246	4.32	2.2	1.08	NO	.00
45	247	8.63	3.0	3.23	NO	.00
46	249	2.06	1.6	1.38	NO	.00
47	412	.07	.1	.07	NO	.00
48	250	2.02	1.5	1.35	NO	.00
49	251	1.39	1.4	.46	NO	.00
50	428	6.50	2.2	4.35	NO	.00
51	252	3.64	1.8	1.95	NO	.00
52	254	5.77	2.5	2.54	NO	.00
53	264	1.43	1.4	.00	-	.00
54	255	2.73	1.7	1.83	NO	.00
55	256	1.62	1.4	1.09	NO	.00
56	257	5.85	2.5	2.56	NO	.00
57	[illegible]	[illegible]	[illegible]	[illegible]	NO	[illegible]

EXTRAN — SUMMARY TABLE

JUNCTION NUMBER	GROUND ELEVATION (FT)	UPPERMOST PIPE CROWN ELEVATION (FT)	MAXIMUM COMPUTED DEPTH (FT)	TIME OF OCCURENCE HR.	MIN.	FEET OF SURCHARGE AT MAX. DEPTH	FEET MAX. DEPTH IS BELOW GROUND ELEVATION	LENGTH OF SURCHARGE (MIN)
10	350.90	347.10	5.09	1	0	3.59	.21	13.2
9	353.00	346.32	4.74	1	0	3.02	3.66	14.3
8	350.70	345.51	4.14	1	0	1.93	3.26	11.8
11	351.00	345.03	3.72	1	0	2.49	3.48	21.5
7	351.80	345.30	4.03	1	0	1.63	4.87	11.5
15	351.70	346.00	7.20	0	60	5.70	.00	2.8
6	349.90	345.02	4.04	1	0	1.72	3.16	16.7
5	353.30	344.76	4.18	1	0	1.72	6.82	20.2
1	347.80	343.57	4.18	0	2	1.81	2.42	118.2
90	348.80	343.21	2.90	0	0	.69	4.90	120.0

ENVIRONMENTAL PROTECTION AGENCY **** EXTENDED TRANSPORT PROGRAM **** WATER RESOURCES DIVISION
WASHINGTON, D.C. **** **** CAMP DRESSER & MCKEE INC.
**** ANALYSIS MODULE **** ANNANDALE, VIRGINIA

- IMPERIAL UNITS

25-YEAR STORM

''''''''''''''''''' SUMMARY STATISTICS FOR CONDUITS ''''''''''''''''''

CONDUIT NUMBER	DESIGN FLOW (CFS)	DESIGN VELOCITY (FPS)	CONDUIT VERTICAL DEPTH (IN)	MAXIMUM COMPUTED FLOW (CFS)	TIME OF OCCURENCE HR.	MIN.	MAXIMUM COMPUTED VELOCITY (FPS)	TIME OF OCCURENCE HR.	MIN.	RATIO OF MAX. TO DESIGN FLOW	MAXIMUM DEPTH ABOVE INVERT AT CONDUIT ENDS UPSTREAM (FT)	DOWNSTREAM (FT)
110	4.6	2.6	18.0	5.9	0	55	3.3	0	56	1.3	5.09	4.58
109	6.8	2.9	20.6	10.4	1	0	4.5	0	55	1.5	4.74	3.65
111	2.7	2.3	14.8	1.3	1	5	1.1	1	5	.5	3.72	3.97
108	8.7	2.3	26.5	12.5	1	0	3.3	1	1	1.4	4.14	3.87
115	3.2	1.8	18.0	1.3	1	8	1.2	1	28	.4	7.20	3.13
107	9.8	2.5	26.5	13.3	1	0	3.5	1	0	1.4	4.03	3.93
106	10.7	2.8	26.5	13.3	1	0	3.5	1	0	1.2	4.04	3.93
105	9.3	2.4	26.5	13.3	1	0	3.5	1	1	1.4	4.18	4.02
101	10.5	2.7	26.5	13.3	1	0	3.5	1	1	1.3	4.18	2.90

JUNCTION NUMBER 10
JUNCTION NUMBER 9
CONDUIT NUMBER 101
CONDUIT NUMBER 109

***** EXTENDED TRANSPORT MODEL SIMULATION ENDED *****