G&S

MOIRA RIVER CONSERVATION AUTHORITY

Upper No Name Creek Water Management Study

Final Report

LICENSED PROFESSIONAL ENGINEER
E. A. MARCETSON
PROVINCE OF ONTARIO

March 1995

MOIRA RIVER CONSERVATION AUTHORITY

Upper No Name Creek Water Management Study

Final Report

March 1995

G&S

Gore & Storrie Limited

Consulting Engineers

March 17, 1995

Moira River Conservation Authority
Box 698
Belleville, Ontario
K8N 5B3

Attention: Mr. Ernest Margetson, P.Eng.
Stormwater Management Co-ordinator

Sir:

Re: Upper No Name Creek Water Management Study

We take pleasure in submitting herewith our Final Report for the Upper No Name Creek Water Management Study. We trust that this report meets all the requirements of the Terms of Reference and that it will provide assistance to the Moira River Conservation Authority, the City of Belleville, and the Township of Thurlow in the development of the lands within the Upper No Name Creek watershed.

The draft report that was circulated to all Steering Committee members in December 1994 was reviewed, and detailed comments received. The most significant comments on the draft report were related to the following:

- A modification in watershed drainage area resulting from field observations during a significant rainfall/snowmelt event in January 1995.

- A requirement for a more detailed cost sharing discussion, including apportionment of costs amongst all proposed new development parcels.

The modification in drainage area required the re-modelling of watershed hydrology and hydraulics, for all proposed System Options. Although a significant increase in runoff volume resulted, the increase in peak flow was small, and resultant changes in required pond volumes, hydraulics, and conveyance channel dimensions were also small. The end result was that the recommended water management strategy outlined in the draft report (System Option 2) remains as the preferred strategy in the final analysis.

The cost sharing discussion was modified, and Draft Recommendations on Cost Sharing were submitted for review by the Conservation Authority, municipalities and developers on March 10, 1995. Following a discussion between Gore & Storrie and the Conservation Authority on March 15, 1995, agreement was reached on the final wording of the cost sharing section for inclusion in this Final Report.

Proj.No. 76.44

857 Norwest Road, Kingston, Ontario K7P 2N2 Telephone: (613) 384-0913 Fax: (613) 384-3048

In developing the recommended water management strategy for the Upper No Name Creek watershed, consultation with the Conservation Authority, both municipalities, and developers was maintained throughout the course of the study. We feel this has allowed for the development of a strategy which satisfactorily addresses the concerns of all parties, and which will provide valuable information for the completion of final design of the proposed stormwater management facilities and flow conveyance channels.

Thank you for giving us the opportunity to work on this study. We appreciate the support of Conservation Authority, City and Township staff, who have provided valuable input to the study.

Yours very truly,

Gillian K. Dagg-Foster, P.Eng.
Project Manager
Water Resources System Division

Harold P. Chard
Harold P. Chard, P.Eng.
Project Manager
Water Resources Systems Division

Proj.No. 76.44

Moira River Conservation Authority

UPPER NO NAME CREEK WATER MANAGEMENT STUDY

FINAL REPORT

TABLE OF CONTENTS

Moira River Conservation Authority

UPPER NO NAME CREEK WATER MANAGEMENT STUDY

FINAL REPORT

TABLE OF CONTENTS

Moira River Conservation Authority

UPPER NO NAME CREEK
WATER MANAGEMENT STUDY

FINAL REPORT

TABLE OF CONTENTS

Moira River Conservation Authority

UPPER NO NAME CREEK
WATER MANAGEMENT STUDY

FINAL REPORT

TABLE OF CONTENTS

SECTION 1

INTRODUCTION

INTRODUCTION

In November of 1993, the Moira River Conservation Authority (MRCA) retained Gore & Storrie Limited (G&S) to conduct a Water Management Study for the Upper No Name Creek subwatershed in the Bay of Quinte area. The MRCA, in cooperation with the City of Belleville and the Township of Thurlow, required a comprehensive stormwater (SWM) plan to ensure that future development may proceed in the subwatershed, while meeting relevant regulatory requirements.

The Upper No Name Creek subwatershed drains to the Tracey Street storm sewer within the City of Belleville, which outlets to the Moira River. The Moira River is, in turn, a major tributary of the Bay of Quinte. The catchment is traversed by Highway 401, which separates the Township of Thurlow (to the north) from the City of Belleville (to the south). The subwatershed boundary has been expanded in this study to include an area west of Sidney Street, within Sidney Township. The total catchment area is approximately 520 ha.

The objective of the study is to formulate a plan for stormwater management in the subwatershed. The plan must meet the following water management objectives:

- control of peak flows to reduce flooding risks and meet the existing hydraulic capacity constraints of the drainage system under future development conditions.

- control of stormwater quality under future development conditions to meet water quality standards of the Bay of Quinte Stormwater Implementation Strategy.

Previous studies have examined stormwater management for various developments in the No Name Creek watershed, but none to date have considered both stormwater quantity and quality control. This study builds on existing information, data and modelling from previous studies, considering updated topographic and land use information. A preferred SWM plan is recommended which considers all potential future development (the ultimate built-out condition) and that meets all relevant water management objectives. The preferred plan is set out in some detail, including plan and profile drawings showing the general arrangement and land requirements. The plan is accompanied by an implementation strategy which includes phasing and cost-sharing details.

The following report is organized into a main body and technical appendices. All technical details are reported in appendices, and summarized in the main report. Chapter 2 provides a detailed description of the study area. In Chapter 3, previous studies are reviewed and summarized. Chapter 4 presents a detailed discussion of water management objectives relevant to the Upper No Name Creek subwatershed, which are then used to develop a list of SWM alternatives (System Options) in Chapter 5. In Chapter 6, the five System Options are evaluated in detail, including environmental impacts and cost comparisons. Chapter 7 presents the preferred SWM plan, including an implementation strategy.

SECTION 2

STUDY AREA

STUDY AREA

2.1 WATERSHED AREA

The Upper No Name Creek subwatershed is located in the north end of the City of Belleville and the southwest corner of the Township of Thurlow. The subwatershed represents the northern portion of the No Name Creek watershed, which outlets to the Moira River near its mouth at the Bay of Quinte. The No Name Creek watershed has essentially been divided into two distinct systems with the construction of the Tracey Street storm sewer, which provides the outlet for the northern subwatershed. The Upper No Name Creek study area is traversed by Highway 401, which divides the Township of Thurlow (to the north) from the City of Belleville (to the south).

The study area consists of the areas which drain to the Upper No Name Creek either directly or indirectly via numerous storm sewer pipes and ditches. The watershed is roughly bounded by Highway 62 to the east, Tracey Street to the south, Vermilyea Road to the north, and Sidney Street to the west. In addition, a 250 ha area in Sidney Township, directly west of Sidney Street and lying roughly north of Cloverleaf Drive and ending just north of County Road 31 was included. It was determined from aerial photos taken on 1993 March 31 that some runoff from this agricultural subcatchment may enter Upper No Name Creek during spring runoff or during large rainfall events. This was also confirmed by field observations made during a significant rainfall/ snowmelt event in January, 1995.

Figure 2-1 illustrates the limits of the study area and outlines the subcatchment areas used in the study. The total watershed drainage area is 518.9 hectares.

2.2 EXISTING DRAINAGE SYSTEM

2.2.1 Description

The existing watercourse which traverses the study area is an unnamed tributary to the Moira River, referred to as Upper No Name Creek.

In general, the creek varies widely from section to section, and the various sections are separated by an assortment of culverts.

Above Maitland Drive, an ill-defined grassy depression collects flow from the north, (and from west of Sidney Street via four 0.76 m (2.5 ft.) culverts and one 0.60 m (2 ft.) culvert under the roadway for large events). Flow is then routed under Maitland Drive through a 1.22 m (4 ft.), 21 m long arch-type corrugated steel pipe (CSP) culvert. From there, water flows south through a small natural channel to Cloverleaf Drive. Drainage flows under Cloverleaf Drive through a 1.22 m (4 ft.), 11 m long CSP culvert. South of Cloverleaf Drive, the existing surface flowpath is poorly defined; there is no apparent channel visible in this area. In this section groundwater contribution to creek flow may be occurring, possibly through fractured sedimentary rock close to the surface. Downstream, the creek empties into a large system of beaver ponds which provide significant natural storage directly north of Highway 401. This wetland area is not classified due to its relatively small size.

Flows are then directed under Highway 401 through a 0.92 m (3 ft.) high by 1.22 m (4 ft.) wide by 55 m long concrete box culvert and into an existing marshy section adjacent to the Quinte Mall. South of this area, the natural channel has been improved for about 100 m down to Bell Boulevard. Underneath Bell Boulevard, flow is routed through a 1.52 m (5 ft.) reinforced concrete pipe 31 m long. From there, flow follows an improved earthen ditch through a 270 m long channel traversed by two small pathways. A 1.22 m (4 ft.), 6 m long CSP pipe underlies the most upstream pathway, and a 1.52 m (5 ft.), 11 m long CSP pipe routes water underneath the second pathway. All flow then enters the Tracey Street storm sewer inlet, a 1.52 m (5 ft.) concrete pipe that carries storm runoff, untreated, eastward to the Moira River.

2.2.2 *Meteorological Data*

Meteorological data for the study were obtained from the Atmospheric Environment Service (AES) of Environment Canada.

For design storm analysis, intensity-duration-frequency (IDF) data were obtained for the AES station in Belleville (station # 6150689). IDF curves were available for the period of 1960 through 1990. For continuous simulation, hourly rainfall data were obtained for the AES stations in Belleville (for 1975 through 1992), Trenton airport (for 1965 through 1975 - station # 6158875) and Picton (for 1965 through 1992 - station # 6156533).

The potential evapotranspiration (ET) values required for hydrologic simulations were obtained from Phillips (1976). The values for the AES station at the Lester B. Pearson International Airport (station # 6158733) were used.

2.2.3 *Estimate of Existing Floodline*

According to a previous study (G&S, 1986), there has been no major flooding in the Upper No Name Creek basin in recent times. The exception was local flooding at Bell Boulevard prior to the installation of the 1500 mm culvert at Bell Boulevard, and the diversion of drainage from the Highway 401-62 interchange directly to the Moira River.

The HEC-2 hydraulic model was applied to estimate the water surface profile resulting from a severe (100-year) storm event. The peak flows used in the HEC-2 model were estimated from the QUALHYMO hydrologic event modelling that was performed using a 100-year SCS Type II 6-hour storm for existing conditions. Details on the methods and results of this analysis are presented in Appendices D and E.

The 100-year water surface elevations computed by HEC-2 were used in conjunction with available topographic mapping to estimate the extent of the resulting floodway. The result is shown on Figure E-1.

With respect to the existing culverts, the HEC-2 modelling indicated that overtopping of the culvert crossing will occur at the following locations:

- both pathways crossing the improved ditch between Tracey Street and Bell Boulevard
- Cloverleaf Drive.

At the Cloverleaf Drive culvert, the HEC-2 model indicated that roadway overtopping occurs during the 100-year event. Springtime overtopping has also been noted at this location; this flooding may have been augmented by ice and debris buildup.

2.3 WATER QUALITY DATA

During the summer of 1994, a field program was conducted by the MRCA in an effort to obtain site-specific flow and water quality data for the Upper No Name Creek. The objective of the 1994 field program was to obtain calibration data for the modelling required in this study. Water quality sampling results are also available from 1993. All water quality data collected are tabulated in Appendix A.

Water quality sampling data collected in 1993 are tabulated in Table A-1. Samples were collected approximately monthly from spring through the fall. No information was available on whether samples were collected during wet or dry weather, although the elevated bacteria concentrations measured on June 7, July 5 and August 4 are indicative of wet weather.

Results of water quality sampling conducted during dry weather in 1994 are provided in Table A-2. Three dry weather periods were sampled for various parameters. Dry weather bacteria, suspended solids and nutrients were generally found to be low.

One wet weather event was sampled in June of 1994. The rainfall event that preceded the sampling was fairly minor, at 2.6 mm. No flow information was collected during the event. The water quality data collected are shown in Table A-3. As shown, the bacteria levels are much greater than the dry weather values in Tables A-1 and A-2. The measured *Escherichia coli* (EC) concentrations are within the range of literature values for stormwater. Suspended solids (SS) concentrations are also elevated, although are an order of magnitude less than typical wet weather stormwater SS concentrations.

The 1994 field program results show that the Upper No Name Creek watershed is subject to bacteriological contamination during wet weather, particularly in the urbanized areas south of Highway 401. The EC concentrations in the creek on the order of 5,000 to 7,000 no/dl are indicative of a problematic level of bacteriological contamination due to stormwater inputs to the creek. Bacteria levels as high as those observed in Upper No Name Creek will have a definite impact on bacteria loadings to the Moira River and may contribute to beach pollution problems downstream.

Late in the summer, it was hoped to measure flow data in the creek in conjunction with water quality during a significant rainfall event to aid in model calibration. A Marsh-McBirney velocity meter was obtained for this purpose, however no rainfall events suitable for monitoring occurred for the remainder of the field program. Therefore, no flow data were obtained for Upper No Name Creek.

2.4 *NATURAL FEATURES*

2.4.1 *Physiography*

The Upper No Name Creek watershed lies in the Napanee Plain physiographic region of Ontario. The Napanee Plain is a flat to undulating plain of limestone from which the glacier stripped most of the overburden (Chapman and Putnam, 1984). The land was early completely occupied by farms, but farming was relatively unproductive on the shallow soils (Chapman and Putnam, 1984) and much of the No Name Creek watershed is now urbanized or consists of abandoned farmland or pasture.

The original vegetation in the area was forest, with sugar maple the dominant tree. White elm, silver and red maple and cedar likely occupied the low ground. At present, white cedar occurs in fairly pure stands where it is invading old pastures (Chapman and Putnam, 1984).

2.4.2 *Vegetation Communities*

Field visits were made on November 9 and 10, 1994 to describe and map vegetation communities and make a list of plants (those identifiable at this time of year) within the watershed. Casual observations were made of wildlife seen during the field visits, as the time of year was not suitable for breeding bird surveys or for intensive surveys of reptile and amphibian distribution. Plants found are noted in Appendix B. Scientific names of all species mentioned can be found in this appendix.

The field visit included observations of the portion of the Upper No Name Creek watershed only as far north as the drainage ditch running west to east north of Maitland Drive, and a small portion of the area west of Sidney Street (sub-catchment 100C). Although additional areas, north of this ditch and west of Sidney Street, contribute to the watershed during large rainfall events, these areas were not included in the natural features inventory since no development is planned in the foreseeable future in these areas.

In general, the creek runs through fields in an advanced state of succession, at the edge of a highly urbanized area of Belleville. Two areas of the western part of the watershed, north of Highway 401, are dominated by mature lowland deciduous forest. South of Highway 401, where the watershed is relatively more urbanized, two small patches of mature mesic cedar forest comprise the only mature vegetation. Most of the creek channel was dry at the time of the field visit. Water was noted flowing in the channel south of Highway 401, but was seen only in sloughs and ponds north of the highway. All channels north of the beaver impoundment area were dry at the time of the field visit.

Vegetation communities discussed in this text are shown in Figure 2-2. This map categorizes the vegetation communities into Environmental Constraint Zones, from 1 to 5. Zone 1 represents a Water Dominant or Related area. According to the Bay of Quinte RAP, the objective is to achieve no net loss of Zone 1 areas within the study area. The other four Zones are: Special Wooded Areas (Zone 2), Wooded Areas (Zone 3), Old Field Regeneration (Zone 4) and Modified Areas (Zone 5).

2.4.2.1 Tracey Street to Highway 401

Agricultural Field

A hay field occupies the southwest side of the watershed in this section.

Buckthorn Dominated Old Field

These areas consist of agricultural fields in the process of succession by the invasive woody shrubs common buckthorn and grey dogwood. Other dominant species were herbs such as Canada goldenrod, wild carrot, orchard grass, and wild strawberry. There are also scattered pioneering trees such as poplar, cottonwood, Manitoba maple and red cedar.

Red Cedar Dominated Old Field

In some areas young to medium-aged red cedar grows in dense stands, with a sparse understorey of buckthorn saplings, raspberries and avens.

White Cedar Forest

Two areas, one immediately south of Highway 401 and one south of Bell Boulevard, are dominated by dense, and relatively large (between approximately 0.20 to 0.40 m diameter at breast height [dbh]) eastern white cedars.

The area south of Highway 401 contains a mixture of white spruce, balsam fir and white birch. Balsam fir and small buckthorn saplings are also very common in the understorey. This appears to be a mesic cedar forest community: there were few wetland species noted in the understorey. The ground was mainly covered with moss, with a few scattered herbaceous species such as helleborine orchid. There is a small log cabin in an advanced state of disrepair in this forest. This area was classed as Zone 2, a Special Wooded Area, due to the mature and diverse nature of the forest.

The cedar forest south of Bell Boulevard contains a high proportion of red cedar, which is invading old fields around the forest. The understorey is mainly of scattered old field shrubs and herbs, but is also very sparse. Again, this appears to be a mesic cedar forest community. This forest was classed as Zone 3 due to the higher degree of disturbance in this area.

Cattail Reed Canary-grass Marsh

A wide, wet slough apparently draining a culvert from beaver ponds immediately north of Highway 401 is dominated by common cattail and reed canary-grass, with other wetland species such as bugleweed, sedges, willows and balsam poplar. This slough drains into a small drainage ditch from the commercial area to the east, just before it enters No Name Creek. Water levels were high and the water was flowing in both the slough and the drainage ditch during the field visit, probably because the visit was conducted immediately after heavy rains in the area. The creek itself was approximately 0.5 m deep, but the water was stagnant.

Another small patch is found west of the creek channel north of Tracey Street.

These marshes fall into Zone 1, Water Dominated or Related areas.

Elm/Buckthorn Woods

A small patch of woods contains American elm, with buckthorn in the understorey. Other trees found here (generally young to intermediate-aged) include red ash, white oak, red maple and Manitoba maple. These woods may contain wet pockets at certain times of the year, but generally have an understorey of old field species.

Creek Banks

The banks of the creek are steep and mainly open, with reed canary-grass and other scattered wetland species only in wetter soil near the bottom of the channel.

2.4.2.2 Highway 401 to Maitland Drive

Red Cedar Dominated Old Field

Most of these sections are abandoned farmland which has been invaded by dense to scattered red cedar. The understorey consists mainly of old field species.

The old fields surrounding the beaver impoundment area also appear to have recently supported a dense cover of young red cedar, which was cut down a few years ago (as evidenced by the number of cut young red cedars lying on the ground in the area). However, the entire area around the wetland has now been cleared. The resulting open ground was subsequently invaded by extraordinarily dense common buckthorn. There were also some stands of more mature white spruce and white cedar here.

Beaver Impoundment

Most of this area is open water (the surrounding upland area has been bulldozed: refer to previous section). A narrow wetland around the ponds is vegetated mainly with purple loosestrife and reed canary-grass and other sedges, grasses and willows. Red-based spike-rush dominates the most saturated soil, with swamp milkweed and water horsetail. To the west of the ponds is a cattail marsh. There are narrow upland ridges between pockets of cattail and separate ponds, mainly vegetated with common buckthorn, red cedar and white cedar. The beaver impoundment area is Zone 1.

Silver Maple-Red Ash Swamp

A lowland forest dominated by silver maple, red ash, red maple and American elm occurs along the creek channel north of Cloverleaf Boulevard. Bur oak is also found, particularly in the western portions. Some of the maples are very large (approximately 1 m dbh), but most range from between 0.05 m and 0.20 m dbh. The herbaceous understorey is very sparse, consisting of scattered grasses and sedges. Buckthorn saplings occur wherever the soil is slightly drier, as at the edges. During the field visit, the soil was moist but not wet. An abandoned well within the woods contained water up to approximately 1 m below the top. Near the south end of the swamp, moss at the base of the trees indicates that there is probably standing water to a depth of approximately 0.15 m in the spring.

A small pond near the north edge of the woods contained water approximately 0.25 m deep. There was no vegetation on the bottom of the pond. Wet soil around the edge supported swamp milkweed, reed canary-grass and beggar's-ticks.

The silver maple/red ash swamp has been classified as Zone 1, Water Dominant or Related.

Creek Banks

The creek channel can be seen clearly in parts of the ash-maple swamp, but toward the south end it becomes indistinct. The channel becomes distinct as it leaves the woodlot, running south to Cloverleaf Boulevard. It was dry at the time of the field visit. The plant species found within and immediately along the creek channel are characteristic of disturbed soil, such as common sow-thistle and Manitoba maple.

The channel (with similar vegetation) continues south of Cloverleaf Boulevard, but has been bulldozed where it leaves the residential area.

2.4.2.3 Maitland Drive to Drainage Ditch

Agricultural Land

Most of the land north of Maitland Drive is currently used for pasture land or other agricultural purposes. No distinct watercourse leads south from the small pond just north of the road; the area is grazed. Hedgerows in this area appear to be the source of common buckthorn saplings moving into neighbouring fields, particularly along the drainage sloughs of the creek. However, the drainage paths are several and not well-marked. Hedgerows are mainly dominated by white ash trees and buckthorn.

Cattail/Reed Canary-grass Marsh

East of Sidney Street is a wetland (Zone 1) dominated by cattails and reed canary-grass. Edges support dense red-osier dogwood. The drainage channel from this marsh through the neighbouring fields is not distinct.

Pasture

A pasture surrounds the farm pond just north of Maitland Drive.

2.4.2.4 West of Sidney Street

Oak/Ash Lowland Forest

Agricultural land and hedgerows dominate most of the area west of Sidney Street. The one notable natural feature is a mesic to lowland deciduous forest, dominated by red ash, bur oak and white elm, with some red maple. This lowland forest is considered a Zone 1 area. Trees are mainly young to medium-aged (most with a dbh of 0.20 m or less) but there are a few very large oaks of approximately 1 m dbh. The woodland is drier in places, particularly along the west edge. A few shagbark hickories were found in this area. They were not seen in other areas of the watershed. However, shagbark hickory is considered common in this physiographic region (Cuddy, 1991).

2.4.3 Flora

Overall, 132 species were found in the Upper No Name Creek Watershed. Much of the terrain surrounding the creek has been disturbed recently, and many non-native species were abundant, such as orchard grass, awnless brome and common buckthorn. Non-native species are notoriously well able to colonize disturbed ground. Several uncommon species were found (noted in Appendix B), all from the area around the beaver ponds. "Uncommon" refers to a species which has been found at between 10 and 100 stations within its physiographic region.

One rare species was found: purple willow-herb. This species is fairly common in the region, and occurs in many highly disturbed habitats. Willow-herbs can be difficult to identify, and the species has likely been overlooked (personal communication, Wasyl Bakowsky, Natural Heritage Information Centre, Peterborough, Ontario).

2.4.4 Wildlife

All animals found were common to abundant winter residents of agricultural and successional land and small woodlands in eastern Ontario. Some, such as coyote, beaver and white-tailed deer, are highly adaptable and have demonstrated their ability to continue to live in relatively small natural areas at the outskirts of expanding cities. However, these species have also demonstrated many of the problems which can be caused by wildlife in cities.

Beaver appear to have been numerous in the area of the beaver impoundments in the past. No fresh signs of beaver were seen during the 1994 field visit, however, and it is not certain that the beaver still reside in the area. Beaver were observed during the G&S field survey in December 1993, in the marshy area south of Highway 401. Trapping was also in evidence at that time.

2.4.5 *Fish Habitat*

Fish habitat is defined as "spawning grounds and nursery, rearing, food supply and migration areas on which fish depend directly or indirectly for their life processes" (MNR, 1994). This definition is intentionally broad, and reflects the fact that fish need different types of habitat at different stages of life, at different seasons, and even different times of the day. Even intermittent streams can be considered fish habitat, depending on the nature of the vegetation and cover present, which may support fish at various times.

Most of Upper No Name Creek can be considered as fish habitat, despite much of it (particularly north of Highway 401) being intermittent. A small section north of Tracey Street contains submerged aquatic vegetation (watercress and water plantain), usually an indicator of consistent flow. The creek was only approximately 0.05 m deep at the time of the field visit, which was just after heavy rain. There is probably minimal flow here except after rain. Several culverts cross the creek between Tracey Street and Bell Boulevard, and the water is ponded north of the first culvert. No fish were seen in this part of the creek during the November 1994 field visit. However, fish (minnows) have been observed in the reach south of Bell Boulevard by MRCA staff (personal communication, E. Margetson, MRCA).

All the beaver ponds, as well as the woodland pond north of Cloverleaf Boulevard and the farm pond north of Maitland Avenue, contained small fish, and should be considered fish habitat.

The beaver ponds contained the largest number of fish: the habitat for many minnow species is good in these types of ponds in that there is an abundant aquatic vegetation to provide cover and forage for fish and their prey, and because beavers create ponds which are deep enough to prevent freezing to the bottom in winter.

2.5 *EXISTING AND FUTURE LAND USE*

The Township of Thurlow has completed a Secondary Plan for the Cannifton area, which dictates the form of urban development allowed within the Upper No Creek watershed north of Highway 401. Draft development plans have already been prepared for much of this area, for both residential and industrial/commercial development. Proposals have also been prepared for large commercial developments south of Highway 401 in the City of Belleville.

The watershed subcatchment numbers referred to in this section are illustrated on Figure 2-1.

Land use in Subcatchments 100B and 100C in Sidney Township, and most of 100A and 101B is predominantly agricultural. Presently, row crops are grown in this area, and this land use is likely to remain the same for at least the next decade.

Subcatchments 101A, 102, 105, parts of 100A and 101B, and the northern portions of 108 and 110 are comprised of rural residential subdivisions. These areas are expected to remain as such, and most of these areas have been designated low-density residential under the Cannifton Secondary Plan (Township of Thurlow, Ainley & Associates, May 1993). Refer to Figure 2-3. The proposed Cloverleaf Estates residential development will encompass the remainder of area 105 (Van Meer, 1994).

The majority of areas 108 and 110 is abandoned farmland or unimproved land at present. A salt storage dome is located on Ministry of Transportation (MTO) property in the centre of Subcatchment 110 that is expected to remain. A beaver pond is located in Subcatchment 110, directly north of Highway 401. A small portion of this area, (roughly where the beaver pond currently lies) has been set aside for "Community facilities, public uses, and utilities" under the Cannifton Secondary Plan (see Figure 2-3). The remaining area is to be developed for industrial uses as the Cloverleaf Industrial Park (Ecos Garatech, 1990).

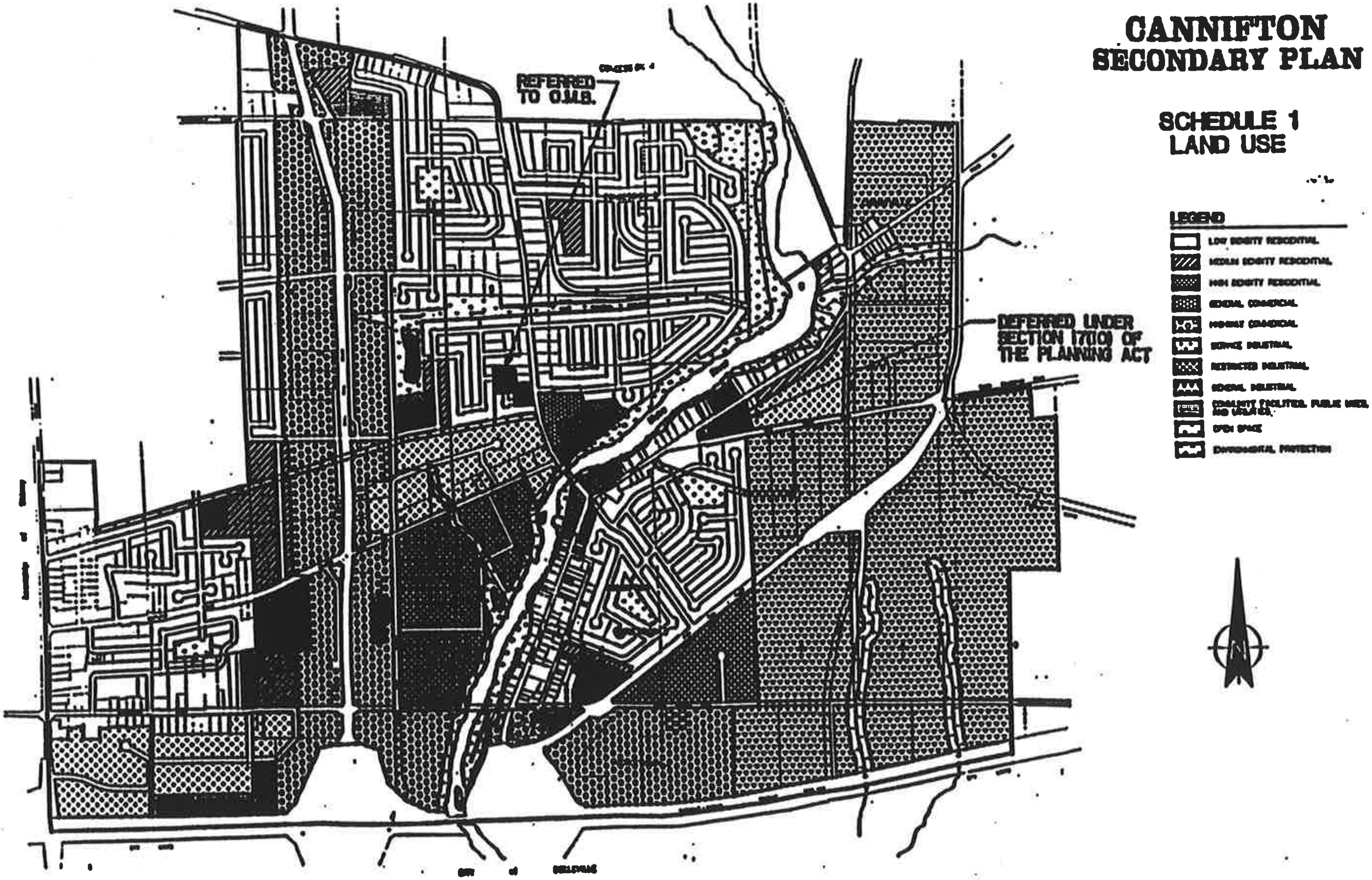
No existing urban development is present in Subcatchment 111. This area is predominantly abandoned farmland and bottomland. Commercial and industrial developments are proposed for this site in the future, the most immediate being a White Rose retail store to the west of the creek. To the east, in Subcatchment 112, lies the Quinte Mall shopping complex. No further development is expected on this site.

East of Highway 62, just south of the Highway 401 interchange lies area 119. South of Bell Boulevard, beside area 119, lies area 115. Major commercial development exists on these two sites, and no changes in land use are expected in these areas.

In area 114, there is presently open space with little development. A commercial development (Zellers retail store) is proposed for this area in the near future. Likewise, in Subcatchment 113, the current open space is slated to become the future site of the Belleville Home Centre.

Directly south of the proposed Belleville Home Centre, there is set aside an area (116) that is currently open space which has been designated for residential purposes under future land use.

Thus, it can be seen that there are various land uses in the Upper No Creek watershed, and numerous future land use designations are in place for land that is not presently developed. The majority of the watershed, south of the ditch running west to east north of Maitland Drive, will be completely developed in the future for either residential, industrial, or commercial use.



MOIRA RIVER CONSERVATION AUTHORITY	**CANNIFTON SECONDARY PLAN** Source: Twp. of Thurlow, Ainley Associates, 1993	**Gore & Storrie Limited G&S** Consulting Engineers FILE Nº \7644\DWGS\FIG2--3 — FIG Nº 2--3

SECTION 3

OVERVIEW OF PREVIOUS STUDIES

OVERVIEW OF PREVIOUS STUDIES

Several studies concerning the hydrology and hydraulics of the No Name Creek watershed have been conducted during the past 20 years. These are summarized below, with emphasis on information directly relevant to the Upper No Name Creek Water Management Study.

3.1 BELLEVILLE WATER MANAGEMENT STUDY

The Belleville Water Management Study (Crysler and Lathem, 1977) identified areas prone to flooding under the 100-year Regional storm and recommended measures to mitigate flooding. The study considered the entire No Name Creek watershed, under the land use conditions existing at that time.

The SCS model was used to generate peak flows, and the HEC-2 model to estimate flood prone areas. The study concluded that, for the upper basin (north of Tracey Street), detention basins were required to reduce Regional storm flows to a level that can be accommodated in the Tracey Street storm sewer (sized for approximately the 5-year storm). Storage basins were recommended immediately upstream of Highway 401, and immediately downstream of Bell Boulevard.

3.2 BELLEVILLE FLOODPLAIN MAPPING STUDY

A floodplain mapping study was conducted of the No Name Creek watershed in 1982 (MNR, 1982). The study included HYMO and HEC-II modelling of the No Name Creek to generate floodlines and to develop a remedial plan to reduce flooding potential. Again, only existing land use conditions were considered.

The peak flows generated in this study were significantly lower than the Crysler and Lathem (1977) study which did not consider the available storage north of Highway 401, but estimated floodlines were very similar to the original study. The recommended remedial works for the Upper No Name Creek basin was a variation on one of Crysler and Lathem's proposed solutions. The recommended works included a detention pond upstream of Highway 401, channelization of the creek downstream of Highway 401 to connect with the existing channel, and a swale to carry excess flow from the upper basin to Tracey Street, then into another swale to route flow back into the creek in the lower basin.

3.3 REPORT ON STORMWATER MANAGEMENT FOR THE NORTHWEST BELLEVILLE AREA

G&S completed a stormwater management study for the City of Belleville for the entire No Name Creek watershed in 1986. The City had postponed development in the watershed until the completion of this SWM report. Expansion of the Quinte Mall was approved on the condition that controls be enforced to control post-development peak flows to pre-development levels. This was achieved by surface detention on the mall parking lot and rooftop.

Required criteria to be met were flood controls for the 100-year Regional storm, control of runoff to pre-development levels, maintenance of the existing storage north of Highway 401, and control of flows north of Highway 401 to be the responsibility of developers to ensure no increase in flows from existing levels.

OTTHYMO was the hydrologic model used in the 1986 SWM study. The Chicago distribution was used to derive a 5-year synthetic storm of 3 hours in duration. For analysis of flood control, the Regional storm was applied, which is a 12 hour duration 100-year storm following the SCS Type II distribution. The model incorporated existing storage north of Highway 401, at the Quinte Mall, and upstream of the Tracey Street storm sewer. Storage volumes were estimated based on topographic information.

The capacity of the Tracey Street storm sewer was found to be 3.5 m^3/s at its inlet at Lemoine Street (outlet of Upper No Name Creek). At 1986 land use conditions, the modelling indicated that the storm sewer has sufficient capacity for the 5-year flows, and would have less than 10% surcharge at 100-year flows, when existing storage upstream of Tracey Street is taken into consideration. Peak flows generated in this study are provided in Appendix D. Flows were in reasonable agreement with the previous MNR (1982) study, except for outflows from the existing beaver pond north of Highway 401. The 1986 G&S study computed available storage based on recent 1:2,000 scale mapping, not available at the time of the MNR study.

The proposed development considered in this study included only the industrial/ commercial developments south of Highway 401. No development north of the Highway was considered. The analysis assumed that a drainage channel would be maintained through the proposed industrial development from Highway 401 to Bell Boulevard and that the existing channel south of Bell Boulevard to the storm sewer inlet would be maintained. A detention facility was recommended at Tracey Street to control runoff from severe storms, to allow the Tracey Street storm sewer to provide the outlet for all runoff events from the Upper No Name Creek basin. No major flows from Upper No Name Creek would therefore be routed to the southern portion of the No Name Creek watershed. The storage volume required in the detention facility, to eliminate surcharge in the storm sewer at future land use conditions for the 100-year storm, was estimated at 6,200 m^3.

3.4 UPDATE REPORT ON STORMWATER MANAGEMENT FOR THE NORTHWEST BELLEVILLE AREA

Since the 1986 G&S SWM study, parts of the No Name Creek watershed were developed, causing land use designations to change, leading to higher imperviousness. Therefore G&S conducted an update study in 1990. The 1986 OTTHYMO model was modified to reflect the changes, and storage requirements adjusted. The study also considered development north of Highway 401.

At the 'ultimate' land use conditions known at the time of the update study, it was estimated that the existing storage north of Highway 401 would have to be increased to 16,000 m^3 for the 100-year event, an increase of 6,000 m^3. South of Highway

401, the storage requirement at Tracey Street to maintain the storm sewer as an outlet for the 100-year event was increased to 10,700 m^3.

3.5 NO NAME CREEK STORMWATER MANAGEMENT STUDY

Falcone Smith (1990) conducted a hydrological study of the Upper No Name Creek watershed for the developers of the Belleville Home Centre and the Congress Centre. The study focused on SWM strategies which would affect these developments. The study area included all drainage to the Tracey Street storm sewer at Lemoine Street, as well as some analysis which was extended to include contributing areas along the length of Tracey Street to estimate the carrying capacity of the storm sewer all the way to the Moira River.

The MIDUSS model was used for the hydrologic assessment, with model discretization and input parameters the same as used by G&S (1986, 1990). A slightly different 5-year storm hyetograph was used, with 15 minute time steps (instead of 5). The same 100-year storm was used.

A hydraulic analysis was conducted of the entire Tracey Street storm sewer, which considered flows captured at Upper No Name Creek at Lemoine Street, as well as at numerous catchbasins located along Tracey Street from Lemoine Street to the Moira River. The flow from Upper No Name Creek is dominant, but the other inflows are significant, and limit the flow which can be accepted at Lemoine Street without producing surcharging. Considering the inflows all along the storm sewer as well as from Upper No Name Creek, the maximum elevation of hydraulic grade line at Lemoine Street was found to be 92.5 m, corresponding to an inflow rate of 3.5 m^3/s. This implies a modest degree of surcharge, not considered to be a problem. This estimate was almost identical to the one provided by G&S (1986).

Analyses showed that at existing land use conditions, the Tracey Street storm sewer has sufficient capacity to prevent flooding, however any development between Tracey Street and Highway 401 will result in increased flooding potential. Stormwater management measures will therefore be required in parallel with development.

Several land use scenarios were analyzed in the study. SWM measures considered included an on-line detention pond at Tracey Street, with and without on-site detention storage. Pond sizing assumed that attenuation of peak flows to pre-development levels from the future development north of Highway 401 would be implemented. The required pond volume for the fully developed condition was found to be 8,600 m^3 for the 100-year event.

The study considered the use of on-site detention storage as an alternative or supplement to the on-line pond. Both rooftop and parking lot storage on the commercial developments were modelled. Both of these types of storage are currently employed at the Quinte Mall, which reduces the 100-year peak flow by 80%. The impact of on-site storage at the Belleville Home Centre and Congress developments on the required volume in the on-line pond was assessed. The result was a significant reduction in volume of the pond to 5,400 m^3. A centralized on-line pond would still be required, however, even with on-site controls. The study recommended that this be designed as a dual purpose water quantity and quality control facility.

SECTION 4

WATER MANAGEMENT OBJECTIVES

WATER MANAGEMENT OBJECTIVES

The purpose of this section is to outline the urban runoff control requirements which have been formulated for the study area.

4.1 *BASIS FOR WATER MANAGEMENT TARGETS*

The water management goals and targets that apply to the study area are based on the following:

- Stormwater management requirements that have been developed for the Bay of Quinte Stormwater Management Implementation Area as part of the Bay of Quinte RAP.
- The province-wide policies of the Ontario Ministry of the Environment and Energy, with respect to protection of surface water quality
- Water management policies and requirements of the Moira River Conservation Authority, with respect to flood and erosion control
- Drainage service requirements and municipal drainage design standards of the City of Belleville and the Township of Thurlow.

4.2 *SUMMARY OF CURRENT POLICIES AND REGULATIONS*

The following summarizes the specific policies and regulatory requirements that affect water management planning for Upper No Name Creek. These are the requirements that this study addresses.

4.2.1 *Land Drainage Requirements: Level of Service*

The fundamental purpose of an urban drainage system is to provide an "acceptable level of service" which will translate into minimal risk to personal safety and minimal inconvenience from runoff. These level of service objectives can be summarized as:

- Protect buildings from surficial flood damage associated with the "Regional Storm" *i.e.* 100-year return period for the study area.
- Protect basements from flooding associated with the "Regional Storm" *i.e.* 100-year return period hydraulic grade line must be below foundation drains; alternatively provide a sump pump to the foundation drain.
- No unsafe ponding on roadway surfaces for 5-year return period runoff events and the extent and duration of ponding on residential or commercial lots must be kept to an acceptable minimum.

These land drainage objectives are best achieved through the application of the MAJOR/MINOR principle of urban drainage design. This principle is based on a drainage system which is composed of two interconnected sub-systems:

(1) Major system for flood protection:

> The major system is designed to provide a sufficiently low risk to life and property from more severe runoff events. The major system consists of swales, roadways, channels, and other overland flow routes, including natural streams and valleys.

(2) Minor or convenience system:

> The minor system is the convenience system designed to minimize inconveniences to pedestrians and motorists by accommodating the more frequent runoff events, usually runoff events up to a 5-year return period. It consists of roof gutters, rainwater leaders, swales, street gutters, catchbasins, and storm sewers.

The major and minor systems are not necessarily separate conveyance systems; they may share some elements. For instance, a grassed waterway or creek may be designed to handle both low flows and high flood flows.

In Upper No Name Creek watershed, the existing drainage system consists of a combination of natural and man-made ditch systems, along with storm-sewered areas such as the Quinte Mall. The creek itself acts as the primary drainage collector, with the final outlet being the inlet to the Tracey Street storm sewer. There is no continuous or engineered major overland flow system as such. Major flood flows would follow existing ditches and flow to the creek, and eventually to the Tracey Street sewer.

4.2.2 *Erosion Control Requirements*

For erosion control, the target is to not to aggravate or worsen any existing erosion problems within the study area and downstream.

This erosion-control requirement is enforced by the Conservation Authority and the MNR, and comes into play when seeking approval for works under the Lakes & Rivers Improvement Act (MNR) or the Fill and Alteration to Waterway Regulations (Conservation Authorities). Erosion control is needed to minimize risk to adjacent properties, and to help ensure the protection of downstream aquatic habitat.

4.2.3 *Water Quality Protection*

Requirements for controlling the impact of urban development on water resources stem from:

- The general Province-wide policies and objectives established by the Ontario Ministry of the Environment and Energy (MOEE) for the protection of surface water and groundwater quantity and quality.

- Policies and requirements of the Ontario Ministry of Natural Resources (MNR) related to protection of aquatic habitat, pursuant to the requirements of the Federal Fisheries Act.
- Local or site-specific interpretations of the Provincial policy that are developed from study and investigation of specific problems encountered in the area. The stormwater management guidelines developed for the Bay of Quinte and for the Rideau River in Ottawa-Carleton are examples.

Provincial policies

The Provincial policies and objectives are contained in the MOEE publication "Water Management: Goals, Policies, Objectives and Implementation Procedures" (MOEE, 1984). The so-called "Blue Book" sets out specific water-quality objectives and policies for attaining the overall goal: "To ensure that the surface waters of the Province are of a quality which is satisfactory for aquatic life and recreation".

The Province's objectives for protecting surface water quality for aquatic habitat are consistent with the objectives of the Federal Fisheries Act, which requires that discharges to water bodies not have any harmful effect on fish habitats. The Canadian Water Quality Guidelines (Canadian Council of Environmental and Resources Ministers, 1987) provides an additional scientific basis for establishing specific impact-control targets for protecting aquatic habitat.

The Bay of Quinte guidelines

Within the Bay of Quinte area, the Provincial objectives have been interpreted in the context of the Bay of Quinte Remedial Action Plan. Within the Bay of Quinte Stormwater Management Implementation Area, specific implementation guidelines for stormwater quality control have been developed by the Inter-Agency Storm Water Management Working Committee (MOEE *et al*, 1993). The water quality performance objectives can be summarized as follows:

- Stormwater discharges to existing watercourses are to meet the following requirements for protecting recreational water quality in the Bay of Quinte:

  Escherichia coli (EC) levels in stormwater discharges to be 100 no/Dl or less (from May 15 to September 15). The MOEE will currently allow up to four separate event exceedances of these bacteriological criteria to occur each swimming season.

  An additional guideline is that total suspended solids should be below 25 mg/L (from May 15 to September 15).

The rationale here is that

- Summertime coliform bacteria control is needed to protect the Bay for recreational use

- By keeping discharge SS levels to 25 mg/l, a high degree of solids control or solids removal will be assured. Since a wide range of pollutants are directly associated with suspended solids, it follows that a good degree of overall stormwater quality control will be assured.

The Bay of Quinte stormwater guidelines also provide specific limits or objectives for additional parameters related to protection of aquatic life. These generally conform with the MOEE Provincial Water Quality Objectives listed in the MOEE "Blue Book" (recently revised, MOEE, 1994b) and the Canadian Water Quality Guidelines (CCREM, 1987).

4.3 STORMWATER CONTROL TARGETS FOR UPPER NO NAME CREEK

The policies and guidelines summarized can be used directly or interpreted to provide a set of specific targets for stormwater management in Upper No Name Creek watershed. These targets are presented below.

4.3.1 Stormwater Quality Targets

In terms of water quality, the problem at hand becomes apparent when one considers the specific goals and regulatory targets versus the expected impact of future urban development within the watershed.

Over the last 20 or more years, research in North America has clearly shown that runoff from urban areas carries a number of contaminants of potential concern, including coliform bacteria (*e.g. E. coli*), heavy metals, hydrocarbons, chlorides, and suspended sediment. The reasonable expectation is that runoff from residential, commercial or industrial areas will contain contaminant concentrations and mass loadings that are can potentially have significant impact on receiving watercourses and waterbodies. In particular, it is reasonable to expect that without adequate source control and/or runoff treatment, *E. coli* levels will generally be significantly higher than 100 no/dl at the point of discharge to the receiver (*i.e.* at the storm sewer outfall). It is also reasonable to expect that suspended solids (SS) levels will often be above the 25 mg/l target, and that a range of contaminants (including bacteria) will be associated with those sediments. In the context of bacteria control, it should also be noted that recent Ontario research on die-off and persistence of coliform bacteria in surface water systems indicates that sediment resuspension may play a significant role in creating elevated coliform levels during runoff events. The implication is that coliform bacteria control is needed not only in wet weather but also during dry weather, to ensure that sediment-bound bacteria levels are not maintained by dry-weather inputs.

In terms of water quality, the target can be stated as follows:

- *Drainage systems for new urban development must be designed to adequately control pollutant loads delivered to Upper No Name Creek. In particular, E. coli and SS concentrations in discharges to the Tracey Street storm sewer* *must be controlled to comply with the Bay of Quinte guidelines: E. coli must be below 100 no/dl from May 15 to September 15 (with allowance of four event exceedances), and SS levels must be controlled to 25 mg/l.*

By meeting these specific targets at Tracey Street, the implication is that an adequate degree of stormwater quality control will be provided for compliance with all existing policies and regulations for surface water quality protection.

4.3.2 Targets for Control of Peak Flows

Previous studies have documented the hydraulic capacity of the existing drainage system. The most significant capacity limitation is the final system outlet (*i.e.* the inlet to the Tracey Street storm sewer) which has an estimated capacity of 3.5 m^3/s (Falcone Smith, 1990). The flow target at the Highway 401 culvert is based on maintaining the pre-development 100-year peak flow (1.4 m^3/s), since previous drainage planning for downstream areas has assumed that this control would be in place. The value of 1.4 m^3/s is based on hydrologic modelling carried out in this study. This value is the same as the estimated hydraulic capacity of the box culvert under Highway 401, therefore controlling flows to 1.4 m^3/s will also ensure that the hydraulic capacity of this culvert will not be exceeded.

For the purposes of this study, the targets for control of peak flows have therefore been set as follows:

Table 4-1
TARGETS FOR CONTROL OF PEAK FLOWS ALONG UPPER NO NAME CREEK

Location	Return Period	Target Flow
System outlet: Tracey street storm sewer inlet	100 years	3.5 m^3/s
Highway 401 culvert	100 years	1.4 m^3/s

4.3.3 Erosion Control Targets

The objective of no increase or aggravation in erosion can be achieved in two ways:

- Control the future flow regime such that there is no increase in the "erosive power" of the flow regime. This essentially means that the stormwater management system must be designed not to increase the duration of erosive flowrates and velocities.
- Design the conveyance system to increase resistivity to erosion wherever an increase in erosive power is expected.

In the case of Upper No Name Creek, there are no substantial erosion concerns along the creek at present. However, as the upper portion of the watershed becomes more heavily developed, there is definite potential for a significant increase in the erosive power of the flow regime in the lower reaches of the creek.

The flow attenuation provided by the stormwater management system will help to control this impact, by reducing peak flows during all runoff events. However, even if designed to control peak flows to present-day levels for return periods from 2 to 100 years, the stormwater control system will not likely provide full control of erosive power. This reflects the fact that the *duration* of erosive flowrates is likely to increase, due to the increased volume of runoff associated with increased watershed imperviousness.

The specific target that should be adopted for stormwater management planning is:

- *No increase in the duration with which flowrate or velocity exceeds the threshold at which erosion begins (i.e. control of excessive erosive impulse, as defined by Lorant, 1982).*

This can be achieved through additional stormwater flow-attenuation (sometimes referred to as "overcontrol"), or through increasing the erosive threshold by increasing the resistivity of the conveyance channel (*i.e.* through structural re-enforcement or natural/bioengineering techniques). Both approaches should be considered.

4.4 SUMMARY

The stormwater control targets listed above have been formulated during this study to guide the formulation and evaluation of specific options for the overall stormwater management system for Upper No Name Creek. The targets require:

- Summertime control of bacteria levels in discharges to the Tracey Street sewer to below 100 *E. coli* per 100 ml, with the allowance of up to four event exceedances per swimming season.
- Control of 100-year peak flowrates at the Tracey Street sewer inlet and at Highway 401.
- Consideration of flow-control or stream-channel protection measures to ensure adequate control of erosion along the Creek.

The following sections of this report deal with the various alternatives and drainage-system options that have been considered.

SECTION 5

FORMULATION OF ALTERNATIVES

FORMULATION OF ALTERNATIVES

The first step in the analysis was to review the general alternatives and possible measures that could be used to meet the various targets for stormwater control (quality control and hydraulic control) for Upper No Name Creek. This section of the report presents the review that has been carried out. It concludes by presenting a number of "SYSTEM OPTIONS" for consideration by the study's Steering Committee.

5.1 *OVERVIEW OF APPROACH*

At the initial screening level, there are numerous measures or so-called "Stormwater Management Practices" (SWMPs) that might be contemplated. The possible SWMPs can be put into three general categories, as shown in Table 5-1.

Table 5-1 OPTIONS FOR URBAN STORMWATER MANAGEMENT: GENERAL CATEGORIES
AT-SOURCE or ON-SITE MEASURES Measures that help to minimize surface runoff volumes, runoff rates and runoff contamination on a site-by-site basis (*e.g.* lot-by-lot within an urban development area). These measures can also assist with maintaining local water balance and water-table recharge.
CENTRALIZED FACILITIES FOR RUNOFF TREATMENT AND FLOW CONTROL Engineered runoff collection systems that direct runoff to engineered management/treatment facilities (*e.g.* detention/retention ponds, settling tanks, constructed wetlands) strategically located within the drainage system. The number of facilities should be minimized, in order to minimize complexity and costs of operation and maintenance.
FLOW CONVEYANCE CHANNELS Flow conveyance system that links the various SWM facilities and effectively contains flood flows within a designated conveyance route which does not unduly constrain proposed development.

For both at-source controls and centralized SWM facilities, there are a number of possible measures or technologies that can potentially be applied within an urban catchment to control downstream flowrates, control runoff contamination or to provide adequate treatment of polluted runoff. The various options are discussed below. The design concept of the flow conveyance channels is also discussed below.

5.2 SOURCE CONTROLS

Source controls are intended to do one or more of the following:

- Control the volume of direct surface runoff from the development site
- Control the rate of direct surface runoff from the development site
- Control the amount of pollutants transported by runoff from the development site.

In the case of pollutant source control, the ideal is to avoid any contamination of runoff in the first place. However, this can be difficult or impossible to achieve because of the variety and number of possible sources of contamination. As well, a substantial portion of urban pollutant washoff typically occurs from roadway surfaces, a source that is difficult to control.

5.2.1 General Source Control Options

Table 5-2 provides an overview of the various at-source or lot-level measures that can be considered for reducing runoff volume, rate and contamination.

Table 5-2
SOURCE CONTROLS FOR URBAN RUNOFF

Source control or on-site control of runoff rate and volume:

- Rooftop storage with gradual release to the drainage system
- Catchbasin inlet controls (*e.g.* flow restrictors such as orifices) that result in temporary ponding in parking lots, on grassed areas or on roadways
- Use of lot-by-lot infiltration measures such as
  - grassed swales
  - infiltration trenches, possibly including perforated pipe systems
  - diversion of rooftop runoff onto grassed areas
- Storage tanks or cisterns to provide temporary storage and gradual release to the drainage system
- Porous pavements to reduce direct runoff from paved areas

Source controls of runoff contamination:

- Intensive sweeping/vacuuming of roads and parking lots
- Housekeeping measures in industrial areas;
  - covering of chemical storage areas, garbage dumpsters, etc.
  - litter control
  - spill control and containment
- Pet control by-laws
- Control or elimination of the use of fertilizers, pesticides and herbicides
- Public education and awareness, including measures such as signage on all storm inlets (catchbasin grates, etc.)

5.2.2 *Evaluating Source Control Options*

Various factors will affect the feasibility, acceptability and practicality of any of the source-control options listed above, including:

- CATCHMENT PHYSIOGRAPHY AND LOCAL CLIMATE

  Soil types, water table elevations and surficial geology may mean that infiltration-type measures are not feasible or practical in the study area. As well, seasonal climate changes and winter conditions may render options such as runoff infiltration and porous pavements infeasible or at least of only limited seasonal effectiveness.

  Local climate will also play a fundamental role in determining runoff treatment requirements, since it will dictate the volumes and frequency with which runoff is generated. Winter conditions will also affect the level of runoff contamination that can be expected due to roadway grit, sediment and de-icing compounds resulting from winter road operations.

- TYPE OF EXISTING AND PROPOSED URBAN DEVELOPMENT

  The type and extent of urban development and its drainage system will significantly constrain the level of control that can realistically be achieved through at-source or lot-level measures. For instance, municipal drainage standards may mean that it is not be acceptable to use measures that cause frequent ponding within residential areas. If the roadways are mostly curbed with catchbasins, then source control of the various pollutants (sediments, trace metals, hydrocarbons) that wash off roadway surfaces will be difficult.

- CONTROL TARGETS

  The site-specific targets for control of urban runoff quality and quantity will have a direct impact on which control measures are suitable or applicable. As well, the specific targets will determine whether source controls alone are adequate. For instance, source controls alone cannot be expected to meet the specific target for control of summertime coliform bacteria concentration in runoff discharged to the Tracey Street sewer. Similarly, source controls may not be enough to meet site-specific control of impacts on flow velocities, flowrates and erosive impulses in a sensitive receiving watercourse.

All of the source-control options listed in Table 5-2 can be evaluated based on their known effectiveness, practicality and feasibility. Table 5-3 reviews the overall status and opportunities for source control measures directed at controlling runoff volume and rate. Table 5-4 evaluates at-source methods of runoff pollution control.

Table 5-3
EVALUATION OF SOURCE CONTROL OPTIONS
MEASURES TO REDUCE RUNOFF VOLUME & RATE

Control	Current Status in Upper No Name Creek	Consider when formulating System Options for Upper No Name Creek? Pros	Cons	Conclusion/Recommendation
Rooftop storage	In place within Quinte Mall	- Well accepted in building design - Significant flow control	- Long-term performance and maintenance may be problematic due to modification of roof inlets by owner	- Rooftop storage should be considered as a potential control component in new development, subject to further review by City and developers.
Catchbasin inlet control and surface storage in parking areas, etc.	In place at Quinte Mall	- Accepted technique - Inexpensive - Potential to reduce required sewer sizes	- Temporary ponding potential nuisance or safety hazard - Difficult to assure compliance due to modification by owner	- Can use where not expected to cause complaints - Use on new sites if ponding can be limited to low-use areas
Storage tanks or cisterns	Not in general use.	- Good potential for flow reduction	- Relatively expensive - Not well accepted building design feature - Maintenance (clean-out) needed	- Can be considered in new developments at builder's discretion and if municipality accepts.
Lot-by-lot infiltration measures - infilt. trenches - grassed swales - rooftop drainage onto grassed areas	Not in use.	- Good potential for runoff and flow reduction. - Helps maintain water table and groundwater recharge - Relatively inexpensive	- Not suitable for roadway runoff due to potential of groundwater contamination - Not feasible or acceptable on clay soils or where water table is high - Infiltration trenches will likely require routine maintenance and renewal	- Diversion of rooftop runoff onto grassed areas is recommended where lots have adequate grassed space and problems will not be caused on adjacent properties.
Porous pavement	Not in use.	- Theoretically good potential to reduce runoff	- Not proven feasible in local climate or in high traffic areas. Service life unknown.	- Not recommended, except at property owner's discretion for low-traffic areas (*e.g.* driveways, pathways)

Table 5-4
EVALUATION OF SOURCE CONTROL OPTIONS
MEASURES TO CONTROL RUNOFF CONTAMINATION

Control	Current Status in Upper No Name Creek	Consider when formulating System Options for Upper No Name Creek?		
		Pros	**Cons**	**Conclusion/Recommendation**
Sweeping/vacuuming of paved areas	Unknown (In use in Belleville urban area?).	Consistent with routine road maintenance programs.	Substantially increased effort & expenditure needed for marginal gain (Pitt, 1983)	No change to existing program is recommended.
Catchbasin cleaning	Unknown.	Consistent with routine sewer infrastructure maintenance programs.	Difficult to ensure that routine maintenance is undertaken by private property owners.	Routine catchbasin cleaning should be implemented if not already taking place.
Housekeeping: covering of storage and waste areas; litter and spill control/containment	Unknown.	Inexpensive. Represents direct control of sources of contamination.	Difficult to ensure consistent and continuing compliance. Success will rely on efforts by individual owners and tenants.	Survey of current practices is recommended, with follow-up action as needed.
Control fertilizers, pesticides & herbicides	Not formally in place.	Some potential for reduced contamination of residential/commercial runoff	Regulations not in place. Would rely on public education & willingness	Recommended component of public education
Public education and awareness; *e.g.* signage on storm inlets	Bay of Quinte RAP taking action.	Potentially substantial reduction in runoff contamination can be achieved.	Relies on public acceptability and adaptability. Effect will take time.	Recommend continued support of RAP education effort.

5.2.3 *Expected Adequacy of Source Control Efforts*

The preceding overview of source control options and possibilities is a necessary first step in any comprehensive drainage planning exercise, given that current Provincial policies and regulations emphasize the need to examine source control opportunities wherever they exist.

In reviewing the feasibility and applicability of source control methods, the following points should be recognized

- Sources of bacterial pollution are numerous and widespread. Pitt (1983) provides a thorough review of the various sources that can be present and active within an urban area. The net impact of many of these sources is difficult to control or predict, and reasons for coliform bacteria persistence in the natural water environment are not fully understood. *For these reasons, it is not reasonable to expect that source controls alone can meet the current Quinte RAP requirement that coliform levels in summertime runoff discharges to the Moira River be below 100 E. coli per 100 ml.*

- Source control of bacteria, nutrient and sediment washoff will also be difficult because of the nature of the existing and expected urban drainage systems in the watershed. Source control of pollutant wash-off from large commercial parking areas and from curbed roadways will be difficult. Roadway washoff is difficult to control without very frequent street sweeping/vacuuming. The effectiveness of street sweeping was examined in detail by Pitt (1983) who concluded that street cleaning may reduce fecal coliform discharges by as much as 20%, but that 10% is a more likely value. Pitt (1983) also notes that these marginal improvements would only be associated with major increases in street cleaning expenditures. On the basis of such research, street sweeping is generally considered to be of marginal benefit in reducing urban runoff pollution.

- Public education programs are now in place within the Quinte area to make the general public aware of the impact of various domestic sources and activities, including pet litter, landscape maintenance (use of fertilizers and herbicides, etc.), and car washing and maintenance on areas that drain onto the roadway. However, the net impact over the short-term and long-term is very difficult to predict. At this stage, little reliance can be placed on the effectiveness of public education.

- Source controls of runoff rate by rooftop and parking-lot storage is likely providing benefits in terms of reducing peak runoff flows from the Quinte Mall.

On the basis of these considerations, the following conclusions have been made:

(1) Source controls alone will not be enough to adequately control runoff pollution. Source control measures to reduce runoff pollution are likely to be of limited effectiveness in dealing with contaminants of primary concern: suspended sediments, coliform bacteria and phosphorus.

(2) Rooftop storage and catchbasin inlet controls can provide substantial benefits in terms of flow reduction (and some volume reduction) and should be continued and implemented wherever practical.

(3) To meet runoff quality targets, centralized runoff collection/treatment facilities will be needed to provide treatment prior to discharge to the Tracey Street storm sewer.

(4) Similarly, since on-site or source-control measures cannot be expected to provide the full amount of flow attenuation needed, the conclusion is that some form of centralized runoff detention will be needed within the drainage system to control peak flows upstream of Highway 401 and upstream of the Tracey Street outlet.

For this reason, the formulation of conceptual system options has focused on options for providing centralized runoff detention and treatment at strategic locations within the watershed. Opportunities to provide at-source flow control via rooftop and parking-lot storage and drainage of rooftop runoff onto grassed areas, have also been considered as part of overall system options, as discussed in Section 5.5.

5.3 CENTRALIZED STORMWATER MANAGEMENT

"Centralized" stormwater management facilities are engineered detention/retention facilities strategically located within the drainage system. For the Upper No Name Creek system, they will have to be designed to provide runoff treatment as well as flow detention (*i.e.* live runoff storage). The overall treatment process will have to be designed to meet the stringent *E. coli* control target at Tracey Street.

5.3.1 Centralized Treatment Options

Table 5-5 lists the various options that can be considered for centralized treatment of urban runoff.

This table includes an assessment of how easily each of these options can be designed to accommodate live runoff storage for flow control.

It is important to note that the stringent summertime coliform control target effectively implies the need to disinfect the stormwater flow prior to release to the Tracey Street sewer. Therefore, Table 5-5 includes an assessment of the level of bacteria load reduction that can be expected from the treatment process alone, as well as its adaptability to direct effluent disinfection via a physical or chemical wastewater disinfection process.

Table 5-5
GENERAL OPTIONS FOR CENTRALIZED RUNOFF TREATMENT

Option	Description and Considerations	Possible to incorporate live storage for flow control purposes?	Expected solids and coliform load reduction	Adaptable to direct effluent disinfection?
SETTLING POND (RETENTION POND)	Constructed ponds designed to provide adequate settling time. A sediment forebay should be situated at the facility inlet to facilitate routine removal of accumulated sediment.	YES.	HIGH, if adequate permanent pool volume provided to extend residence time	YES. Additional live runoff storage likely needed to control inflow rate to disinfection unit.
CONSTRUCTED WETLAND	Shallow pond/pool systems that support aquatic vegetation. Pollutant removal by settling and by transformations associated with aquatic vegetation. Inlet forebay needed to remove bulk of sediment upstream of main wetland area.	YES.	HIGH, if designed to provide enough residence time. Aquatic vegetation assist with bacteria removal.	YES. Additional live runoff storage likely needed to control inflow rate to disinfection unit.
UNDERGROUND SETTLING TANK	Tanks designed to provide adequate time for sediment removal by natural or chemically assisted settling. The system must be designed to allow routine removal of accumulated sludge.	Live storage potential is limited. Detention basin or pond also needed to meet flow control targets.	MODERATE bacteria removal expected if good solids removal effected.	Significant live storage must be provided to control flows.
VORTEX SEPARATORS	Devices that remove coarsest sediments through vortex action *i.e.* making use of centrifugal forces. Continuous removal of accumulated sediment is needed.	No live storage potential. Detention facility needed to effect flow control.	LOW bacteria removal expected since there is short residence time and low solids removal (*e.g.* 30%)	Significant live storage must be provided to control flows.
FILTRATION SYSTEMS	Engineered systems in which runoff filters through porous media (*e.g.* sand filter) to an underdrain system. The bulk of sediments should be removed prior to filtration to minimize filter clogging.	Limited live storage potential. Detention facility needed to effect flow control.	HIGH solids and bacteria removal possible.	Significant live storage must be provided to control flows.

5.3.2 *Evaluation*

Besides those listed in Table 5-5, there are a number of considerations that enter into the evaluation of these options:

- In general, centralized runoff treatment should occur at a minimum number of facilities, to minimize costs per hectare served, and to minimize the costs and complexity of operation and maintenance.

- Ponds and wetlands are the most readily designed to also include significant live storage for the purposes of downstream flow control. In a watershed such as Upper No Name Creek in which there are open-space opportunities along the existing drainage system, the preferred approach to providing flow control will be to use detention basins or ponds to temporarily hold runoff and allow for its gradual release. Therefore, combination retention/detention ponds or wetlands that provide both runoff treatment and flow control are a favourable option.

- Constructed wetlands will need adequate tributary area, in order to ensure an adequate supply of water to maintain the wetland. A general guideline is that a constructed wetland should comprise at least 1% to 2% of its drainage area and that a minimum tributary area of 10 hectares is needed (Schueler,1992) to ensure adequate baseflow into the wetland.

- Vortex separators are highly engineered wastewater treatment devices, the performance of which will be highly dependent on the physical characteristics of the stormwater sediment (particle sizes, etc.). They are best suited to smaller-scale end-of-pipe applications. Performance in a stormwater treatment situation is likely to be variable. Used alone, these devices are unlikely to provide adequate solids removal to meet treatment targets. Therefore, they can only be considered as possible system components that might be used in conjunction with final treatment in a settling facility such as a pond or wetland.

- Similarly, filtration systems can only be considered as a possible system component, since significant "pre-treatment" is needed to remove sediment prior to filtration. The initial sediment removal will require the use of a settling pond, tank or solids separation device. Filtration systems should be viewed as an option for providing improved treatment of effluent from an initial settling facility.

On this basis it has been concluded that pond and/or wetland facilities are the most appropriate form of centralized runoff detention and treatment for the Upper No Name Creek watershed.

5.3.3 *Disinfection Technology*

Due to the large variation in bacterial loadings along with the variability of natural die-off mechanisms, we cannot rely solely on natural die-off to achieve the high degree of bacteria removal required. This implies, as stated previously, that some form of direct stormwater disinfection must be provided in order to meet the target for control of coliform levels at the Tracey Street sewer.

As presented in Appendix H: *Review of Methods for Bacterial Control*, a number of options are available. These methods are basically those that have been successfully applied for effluent disinfection of municipal wastewater. They are:

- Chlorination / Dechlorination
- Chlorine dioxide
- Bromine Chloride
- Ozone
- Ultraviolet (UV) light

From the discussions found in Appendix H, it is evident that ultraviolet disinfection presents many advantages which are desirable for stormwater applications. Ultraviolet irradiation is a relatively simple process which requires a short contact time, has good bactericidal and virucidal properties, and does not create any known hazardous by-products. Comparing capital and operating costs of the various disinfection options also indicates that UV irradiation is cost effective.

For these reasons, the conclusion at this stage is that UV irradiation is the most suitable means of providing stormwater disinfection.

In formulating and sizing specific "System Options" for Upper No Name Creek (*i.e.* arrangement of stormwater management facilities), the system must take direct account of the flow-control and sediment-control requirements of the UV process. As well, the overall layout of the drainage system will determine the most logical location(s) for UV treatment.

In the case of the Upper No Name Creek watershed, the following factors come into play:

- The layout of the final drainage system will be such that all stormwater will drain to a central collector and be conveyed to the Tracey Street sewer inlet. Therefore, <u>the logical location for a disinfection facility within the Upper No Name Creek watershed is immediately upstream of the Tracey Street sewer</u>. Placement of UV facilities at intermediate points within the drainage system (*e.g.* upstream of Highway 401) should not be considered because of the fact that all stormwater will eventually converge in the central outlet collector.

- The overall SWM system must therefore include a <u>flow detention and settling facility just upstream of the Tracey Street sewer</u>, to provide final flow attenuation and clarification immediately prior to UV treatment.

Formulation of system options has proceeded on that basis.

5.4 FLOW CONVEYANCE CHANNELS

The suggested design concept is that the conveyance routes be designed using a "natural" channel design approach. That is, the channel/floodway system would be designed to have a natural character. This approach will help to minimize net impacts on the local environment, and will provide potential for local enhancement of aquatic or riparian habitats which are at present highly disturbed.

For example, the bed of the low-flow channel could be lined with cobbles and stones to minimize erosion potential while also improving physical diversity and aquatic habitat potential. The channel banks and sideslopes can be stabilized via vegetative plantings including grasses, sedges, shrubs, and aquatic species. To potentially allow use of the channel floodways as parkland, some areas of turf grass could be incorporated within the overall landscape plan.

Erosion protection and channel stability is a concern that must be addressed in the subsequent design of the conveyance routes. Along the two reaches from Maitland Drive to Cloverleaf Drive, and from Cloverleaf Drive down to the wetland area north of Highway 401, existing topography dictates that average channel slopes are likely to be more than 1% (depending on final alignment selected). This situation has the potential to cause flow velocities that could cause local erosion problems unless adequate erosion protection measures are put in place. As well, velocity-control measures (*i.e.* energy gradient control) could be used to assist with erosive power control. Within the context of a natural form of channel design this could possibly take the form of pool-riffle sequences along the creek to help reduce flow velocities, dissipate flow energy and also provide increased aquatic habitat potential.

5.5 PREFERRED APPROACH TO CENTRALIZED SWM

On the basis of the evaluations and discussion presented above, the following conclusions have been reached with regard to the preferred methods and approach to centralized stormwater management in the Upper No Name Creek watershed:

- Centralized treatment and flow control can best be achieved through the use of strategically located pond or wetland facilities. These facilities will provide stormwater treatment through the primary removal mechanisms of natural settling of particulates and natural die-off of bacteria. Secondary removal mechanisms which may also be present are biological uptake of nutrients by aquatic vegetation.

- Direct stormwater disinfection should be implemented at one central location at the downstream end of the system (*i.e.* at the Tracey Street storm sewer). Direct disinfection would best be achieved through use of ultra-violet irradiation. However, this selection should again be reviewed at final design to ensure that the best selection is made from the then-available technology.

- The overall system should strive to:

    - Minimize the number of facilities

- Facilitate phasing as upstream development proceeds
- Facilitate cost sharing and implementation recognizing that two municipalities are involved (Township of Thurlow and City of Belleville).

This general approach to centralized SWM should be combined with consideration of on-site implementation of rooftop storage and controlled-inlet catchbasins wherever considered practical and feasible, and with the use of flow conveyance channels, where required.

5.6 FORMULATION OF SYSTEM OPTIONS

System options for the Upper No Name Creek watershed have been formulated by applying this general approach and by considering the following:

- Location opportunities and space availability along the existing creek corridor for construction of stormwater ponds

- Type of proposed land use in the development areas and, therefore, the opportunities for consideration of rooftop and/or parking-lot storage for on-site flow attenuation

5.6.1 Facility Location Opportunities & Constraints

There are a number of opportunities along the creek corridor for siting of stormwater management pond or wetland facilities.

Constraints on siting of facilities are related primarily to the expected layout of the drainage system, topography and existing land use plans. These have been accounted for in defining opportunities. The following points should be noted:

- Within the proposed Cloverleaf Estates development, it has been assumed that an open corridor will be preserved along the existing creek channel as shown on the "Conceptual Development Plan" prepared by Van Meer Limited (1994).

- The existing wetland immediately north of the Highway 401 culvert is viewed as an opportunity to make use of an existing wetland to provide flow detention and wetland treatment. It has been assumed that modification of the outlet hydraulics and extent of this wetland (affected by beaver activity) is acceptable to the various review and approval agencies.

- It has been assumed that south of Highway 401, an open flow-conveyance and stormwater management corridor can be preserved during land development planning and design. In other words, it has been assumed that open-channel conveyance or installation of a stormwater management facility could be installed at any location along the creek south of Highway 401.

5.6.2 *Method of Preliminary Sizing of System Components*

Details of the hydrologic analyses are presented in Appendix D. Pond sizing details are provided in Appendix F. The key results are discussed in this section.

The QUALHYMO model was run on an event basis to produce 5 and 100-year return period peak flows for existing conditions. Since the SCS 6-hour storm distribution was found to produce the highest peak flows for these return periods, this distribution was used for all subsequent design storm analyses.

Existing condition peak flows at various locations in the watershed are shown in Appendix D (Table D-2). Results from previous studies are also shown for comparison. As shown in the table, the 100 year peak flow at existing conditions at Tracey Street exceeds the inlet capacity of the storm sewer of 3.5 m^3/s.

Table D-4 presents peak flows for the future developed conditions, with no SWM controls except the existing wetland at Highway 401. The table shows the need for additional quantity controls at both Highway 401 and at Tracey Street.

The hydrologic and hydraulic analyses have been used to provide preliminary estimates of the required size of the primary component of each of the system options that are described in Section 5.5. The primary components consist of:

- Stormwater detention\retention facilities (*i.e.* ponds or wetlands)
- Conveyance channels that link the SWM facilities

<u>Estimation of Live Storage requirements:</u>

As described in Appendix F, the QUALHYMO model has been used to determine the live storage requirements (*i.e.* detention volumes) needed at facility locations to control downstream peak flows to the target levels presented in Section 4.

<u>Estimation of Treatment Volume Requirements:</u>

At the SWM facilities, treatment will be effected primarily by the hydraulic residence time that results from a permanent pool volume held within the facility. Preliminary sizing of these pool volumes (*i.e.* retention volumes) has been based on the recent publication "Stormwater Management Practices Planning & Design Manual" (MOEE, 1994). These guidelines allow estimation of the permanent pool volume required per hectare of service area, to achieve any one of four levels of treatment (as indicated by solids removal). For the purposes of this study, preliminary pool sizing has been based on the most stringent level (Level 1) which is intended to achieve average annual solids removal on the order of 80%.

Once preliminary sizing had been accomplished, modelling of pollutant delivery, transport and removal was carried out using QUALHYMO, in order to examine the performance of each system option in terms of:

- SS removal (in %) by natural settling
- *E. coli* removal (in %) by natural die-off
- Number of separate exceedances of the target SS and *E. coli* levels of 25 mg/l and 100 no/dl respectively.

In this way it was possible to determine if the recent sizing guidelines would provide simulated system performance that meets the specific targets that apply to the study area.

Preliminary Sizing for Conveyance Elements:

For the flow conveyance system that links the various stormwater management facilities, the objective has been to determine the geometry of open channel and floodway needed to convey the 100-year flowrate while keeping the resulting 100-year water surface profile to a level that is judged to not unduly constrain proposed development.

In particular, the approach that has been taken north of Highway 401 has been to determine the required dimensions for a flow conveyance channel and floodway that would effectively contain the flood flows within a designated conveyance route that is narrower than the existing poorly-defined creek. (Refer to Figure E-1 in Appendix E for the estimated extent of the existing floodline along the creek).

All stormwater strategy options that have been formulated in this study include designed conveyance channels/floodways from north of Maitland Drive to the beginning of the existing ditch south of Highway 401. From a point about 100 metres north of Bell Boulevard to Tracey Street, no channel modifications or redesign are required to convey 100-year future flows, once the restrictions imposed by the two existing pathway culverts south of Bell Boulevard are removed.

For purposes of preliminary sizing of the flow conveyance routes, a typical channel/floodway cross-section was assumed (refer to Appendix E) that includes a small baseflow channel, a primary flow channel with capacity to the 5-year flow, and a wider terraced floodway.

5.6.3 Basis For System Options

At this stage in the study, these methods have been used to carry out preliminary sizing of five System Options. The system options consist of combinations and arrangement of centralized SWM facilities, with or without at-source flow controls included in proposed development areas.

While there are numerous possible combinations and arrangements for SWM facilities, a limited set of options has been formulated. The five options that are presented here are, in part, based on the following considerations:

- All options must include a treatment and detention facility at Tracey Street.

- A minimum number of facilities is desired.

- Cost sharing and implementation will be facilitated if flow control and treatment targets are met at the Belleville/Thurlow boundary. While it is not recommended that UV disinfection be placed at this municipal boundary, cost sharing and implementation should be facilitated if flow control targets are met and if suspended solids control targets are met. This would simplify system implementation and management. However, cost sharing arrangements would be required with regard to the end-of-system disinfection facility at Tracey Street.

Implementation issues are further discussed in Section 7.

The five System Options that have been formulated are described below.

5.6.4 System Option 1

This option consists of two stormwater management facilities and open channel conveyance links as shown schematically on Figure 5-1. SWM facility inflows, outflows and volumes are shown in Appendix F, Table F-1.

SWM FACILITY No. 1:

Location: Wetland just upstream of Highway 401

SWM FACILITY No. 2:

Location: Wet pond immediately upstream of Tracey Street

CONVEYANCE CHANNEL & FLOODWAY DIMENSIONS:

As shown in Appendix E.3

5.6.5 System Option 2

This option is the same as System Option 1 except that the following at-source flow controls have been included:

- Rooftop and parking-lot storage in all lands being considered for industrial/commercial development

- Diversion of runoff from residential rooftops onto grassed yard areas.

This scenario is also depicted in Figure 5-1. The result of the source controls is a significant reduction in size requirements for the central facilities as listed in Table F-1.

SWM FACILITY No. 1:

Location: Wetland just upstream of Highway 401

SWM FACILITY No. 2:

Location: Wet pond immediately upstream of Tracey Street

CONVEYANCE CHANNEL & FLOODWAY DIMENSIONS:

As shown in Appendix E.3.

5.6.6 *System Option 3*

This option incorporates three SWM facilities. An additional facility is added within the Cloverleaf Estates Development, for quantity control purposes. This scenario is illustrated in Figure 5-2. Pond volumes are included in Table F-1.

SWM FACILITY No. 1:

Location: Dry pond within Cloverleaf Estates development

SWM FACILITY No. 2:

Location: Wetland just upstream of Highway 401

SWM FACILITY No. 3:

Location: Wet pond immediately upstream of Tracey Street

CONVEYANCE CHANNEL & FLOODWAY DIMENSIONS:

As shown in Appendix E.3

5.6.7 *System Option 4*

This option is the same as System Option 3 except that the following at-source flow controls have been included:

- Rooftop and parking-lot storage in all lands being considered for industrial/commercial development

- Diversion of runoff from residential rooftops onto grassed yard areas.

This scenario is also shown in Figure 5-2. The result of the addition of the source controls is a significant reduction in size requirements for the central facilities as listed in Table F-1.

SWM FACILITY No. 1:

Location: Dry pond within Cloverleaf Estates development

SWM FACILITY No. 2:

Location: Wetland just upstream of Highway 401

SWM FACILITY No. 3:

Location: Wet pond immediately upstream of Tracey Street

CONVEYANCE CHANNEL & FLOODWAY DIMENSIONS:

As shown in Appendix E.3.

5.6.8 *System Option 5*

This option represents a significant departure from the previous four. The option is illustrated in Figure 5-3. The SWM facility upstream of Highway 401 is included for control of runoff from the Township of Thurlow. This flow is then diverted eastward directly to the Moira River via an underground conduit, just north of the Highway 401 right-of-way. A plan and profile drawing of the proposed diversion is provided in Figure 5-4. Further details are provided in Appendix G.

A SWM facility is also provided at Tracey Street, for treatment of runoff generated south of Highway 401 within the City of Belleville.

SWM FACILITY No. 1:

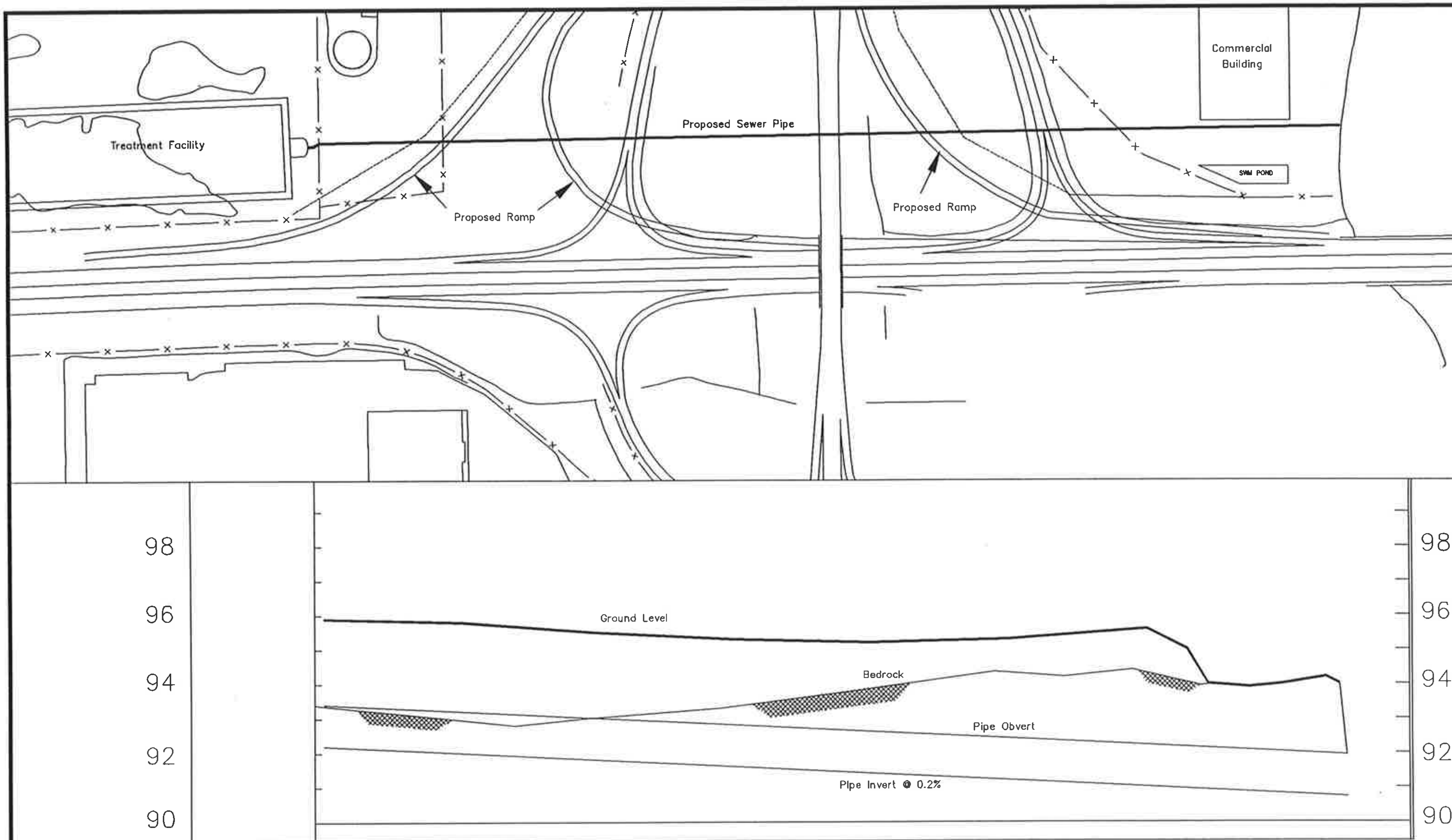
Location: Wetland just upstream of Highway 401

SWM FACILITY No. 2:

Location: Wet pond immediately upstream of Tracey Street

CONVEYANCE CHANNEL & FLOODWAY DIMENSIONS:

As shown in Appendix E.3.



DATE 12/05/94	SYSTEM OPTION #5 DIVERSION PROFILE AND PLAN	Gore & Storrie Limited G&S Consulting Engineers
SCALE		FILE Nº S5_divrt — DWG Nº 5-4

SECTION 6

EVALUATION OF SYSTEM OPTIONS

EVALUATION OF SYSTEM OPTIONS

Alternatives for stormwater management in the Upper No Name Creek watershed have been formulated from considering a wide range of possible measures, including source controls. This approach is consistent with fundamental principles of environmental assessment (EA), which require consideration of all feasible solutions to the problem at hand.

Evaluation and comparison of the five System Options has also been carried out in a manner that is consistent with EA principles. This recognizes that implementation of stormwater management facilities in the watershed will require compliance with the Province's EA Act.

6.1 PERFORMANCE EVALUATIONS IN TERMS OF POLLUTANT REMOVALS

The QUALHYMO model has been used to simulate pollutant loadings from the various subcatchments, and to simulate removal processes within the stormwater management facilities. Details of the modelling methodology are provided in Appendix D and F. Results obtained with respect to the predicted performance of each of the five system options described above are provided in Appendix F, Table F-3.

The table includes average annual performance of the various SWM facilities in terms of pollutant removal efficiency and numbers of exceedances of the regulatory thresholds for suspended solids (25 mg/l) and coliform bacteria (100 *E. coli* per 100 ml).

The overall conclusion from the performance evaluation was that there is little difference between System Options in terms of pollutant removal performance. This is due to the fact that each facility in each Option was sized for water quality control. Therefore, relative differences in pond performance in terms of water quality control between Options are not a factor in evaluating the Options.

6.2 APPROACH TO COMPARATIVE EVALUATION

The five System Options are therefore compared with respect to the following factors:

- Capital Costs (*i.e.* construction costs)
- Costs of Operation & Maintenance
- Impact on Natural Environmental Features within the watershed

As has been explained, all five Options are intended to meet specific stormwater management targets: stormwater quality control consistent with the Bay of Quinte RAP, and control of peak flows rates at critical points along the Creek. All five options therefore provide the same benefits by providing the same performance in terms of stormwater flow and quality control.

For this reason, it becomes possible to comparatively evaluate the five Options on the basis of their costs and impacts on the natural environment.

6.3 COST COMPARISON

6.3.1 Construction Costs

Table 6-1 presents the estimated costs for final design and construction for the drainage works and water management/treatment facilities associated with each of the five Options. Details on the costing estimates are provided in Appendix G.

Table 6-1
COSTS FOR SWM FACILITIES AND DRAINAGE WORKS

SYSTEM OPTION	Total Estimated Construction Cost	SUMMARY BREAKDOWN (See Appendix G)	
1	$4,348,000	Pond north of Hwy.401	$1,459,000
		Pond at Tracey St.	$1,458,950
		UV unit at Tracey St.	$705,000
		20% Contingency	$724,590
2	$3,750,000	Pond north of Hwy.401	$1,269,500
		Pond at Tracey St.	$1,275,500
		UV unit at Tracey St.	$580,000
		20% Contingency	$625,000
3	$4,659,000	Pond above Cloverleaf Dr.	$300,800
		Pond north of Hwy.401	$1,424,200
		Pond at Tracey St.	$1,452,550
		UV unit at Tracey St.	$705,000
		20% Contingency	$776,510
4	$4,092,000	Pond above Cloverleaf Dr.	$297,000
		Pond north of Hwy.401	$1,269,400
		Pond at Tracey St.	$1,263,550
		UV unit at Tracey St.	$580,000
		20% Contingency	$682,010
5	$6,299,000	Pond north of Hwy.401	$1,560,000
		UV unit at Hwy. 401 pond	$390,000
		Diversion to Moira River	$1,457,500
		Pond at Tracey St.	$1,261,350
		UV unit at Tracey St.	$580,000
		20% Contingency	$1,049,770

At this level of analysis, it is clear that Options 2 and 4 are least expensive in terms of up front capital costs.

With respect to the cost estimation, it should be noted that as explained in Appendix G, UV costs are directly dependent on the required UV treatment rate (*i.e.* the maximum flow that the UV facility must be able to treat to meet the regulatory requirement that there be no more than four untreated events per swimming season). For each System Option, the UV treatment rate has been estimated based on model simulations of the swimming-season outflows from the Tracey Street pond with the active storage volume needed for 100-year flow control (as presented in Appendix F, Table F-1).

It should be noted that for any System Option, it will be possible at final design to further optimize the active storage volume and operational hydraulics of the Tracey Street pond to minimize UV treatment rate while working within constraints dictated by 100-year flood hydraulics and the land area available for the facility. On this basis, this study's comparative analysis has estimated UV treatment rates from the model simulations by considering the pond outflow that is exceeded four times per swimming season. Some reductions were then applied to account for what is likely achievable through hydraulic optimization at the final detailed design stage. Resulting estimates of UV treatment rates are presented below in Table 6-2.

6.3.2 *Operation & Maintenance Costs*

Ongoing costs for operation & maintenance of the centralized drainage works and stormwater facilities will result from the following:

- Electrical power requirements for UV disinfection facilities
- Maintenance of UV systems, including routine cleaning and replacement of UV bulbs, maintenance of operating and monitoring equipment, building and grounds maintenance
- Routine inspection and clean-out of drainage channels and culverts
- Landscape maintenance.

Of these items, the O&M costs for UV facilities will be the largest component. Costs for routine inspection and maintenance of drainage channels, culverts and other conveyance structures will not be substantially different between System Options 1 to 4, but are likely to be higher for Option 5 due to maintenance requirements for the diversion conduit to the Moira River.

For these reasons, comparison of O&M costs has been based on examining only the costs for operation & maintenance of UV facilities.

G&S has acquired data on expected UV O&M costs relative to UV treatment rate from UV equipment suppliers such as Trojan Industries of London, Ontario. These data have been used to estimate annual costs for power supply, UV bulb maintenance and replacement, and routine equipment maintenance. Listed below in Table 6-2 are the resulting estimates for each of the five System Options.

Table 6-2
O&M COSTS FOR UV TREATMENT

System Option	UV treatment rate(s) (L/s)	Estimated annual O&M costs
1	300	$10,000
2	200	$7,000
3	300	$10,000
4	2 ~~3~~00	$7,000
5	80 and 200	$4,000 and $7,000

This analysis shows that annual UV O&M costs are not substantially different for Options 1 to 5.

6.4 *IMPACTS ON THE NATURAL ENVIRONMENT*

As described in Section 2.4 of this report, G&S field biologists carried out a survey of the natural features of the creek and watershed area in November, 1994. Appendix B contains details of the results of that investigation.

The results of the natural environment survey are depicted in Figure 2-2, and can be summarized as follows:

- In general, the creek runs through abandoned farmland that is highly disturbed and in an advanced state of vegetation succession. Many non-native plant species are present; an example is common buckthorn, a very invasive plant that is now ubiquitous in the region.

- North of Highway 401 there are two areas in the watershed that are dominated by mature lowland deciduous forest: one along the creek north of Cloverleaf Drive, and the other west of Sidney Street.

- South of Highway 401, two patches of mature cedar comprise the only mature vegetation.

- Most of the creek can be considered as fish habitat, even areas of the creek, particularly north of Highway 401, where flow is apparently intermittent. All beaver ponds, as well as the woodland pond north of Cloverleaf Drive and the farm pond north of Maitland Avenue, contained small fish and should be considered as fish habitat.

Given this perspective on existing natural features along the creek and within the proposed development areas, the five System Options can be compared in terms of their impacts by considering whether the following two objectives will be met:

(1) *The Bay of Quinte RAP specifies an objective of "no net loss" of any Zone 1 (Water Dominated or Related) areas.*

(2) *The MNR Fish Habitat Protection Guidelines (1994) require that the productive capacity of fish habitat be maintained through the application of the "no net loss" guiding principle.*

Therefore, assessment of the impacts of the System Options must consider the following questions:

- What level of disruption will be caused to the mature lowland woodlot that is situated along the creek channel between Cloverleaf and Maitland Drive (Zone 1, type 3 area shown on Figure 2-2)? Will water supply be maintained?

- Will flow be maintained to the small marsh area along the creek just south of Highway 401 (Zone 1 area shown on Figure 2-2)?

- Will flow be maintained in areas that are currently fish habitat, in particular the beaver pond north of Highway 401, as well as the small woodland pond between Cloverleaf and Maitland Drive?

Table 6-3 reviews each of the five Options with respect to these questions.

This comparison reveals that Options 1 and 2 are preferred in terms of impacts on the natural features along the creek. Options 3 and 4, which involve a stormwater pond north of Cloverleaf, will conflict with the "no net loss" objective of preserving the wet woodland in that area (Zone 1 area on Figure 2-2) by requiring tree removal and through alteration of water level fluctuations in that reach of the creek. The negative impacts of Option 5 result from the fact that it will not sustain water supply to the creek south of Highway 401. Therefore the "no net loss" objectives for both the Zone 1 marsh area and the fish habitat in the creek south of Highway 401 will not be met.

6.5 OVERALL EVALUATION

In summary, the comparison of the five System Options has shown that Options 2 and 4 are preferable in terms of costs, and Options 1 and 2 are preferable in terms of minimizing impacts on the natural environment.

In terms of cost considerations, Option 2 is somewhat less expensive than Option 4, with the Option 2 construction cost being about 10% lower than that for Option 4.

With respect to natural environment impacts, the discussion and comparison presented above shows that the Options 1 and 2 are generally the same in terms of satisfying the objectives for the natural environment in the watershed. Potential impacts on the Cloverleaf woodland and the beaver pond are minimized, and the water supply to the area south of Highway 401 is maintained. However, System Option 2 can be

Table 6-3
IMPACTS OF SYSTEM OPTIONS ON NATURAL FEATURES

System Option	Impact on Natural Features
1 and 2	• Will maintain water flow to Cloverleaf woodland area • Conveyance channel/floodway can be designed to preserve Cloverleaf woodland if sufficient land set aside along existing creek. • Will maintain wetland area north of Highway 401 • Will maintain supply of flow to creek south of 401, thereby maintaining supply to marsh south of 401 and also maintaining aquatic habitat potential in southern part of the watershed.
3 and 4	• Stormwater detention pond north of Cloverleaf will conflict with desire to preserve Cloverleaf woodland. • Will maintain wetland area north of Highway 401 • Will maintain supply of flow to creek south of 401, thereby maintaining supply to marsh south of 401 and also maintaining aquatic habitat potential in southern part of the watershed.
5	• Will maintain water flow to Cloverleaf woodland area • Conveyance channel/floodway can be designed to preserve Cloverleaf woodland if sufficient land set aside along existing creek. • Will maintain wetland area north of Highway 401 • Will not maintain supply of flow to creek south of 401, thereby reducing potential to protect marsh south of 401 or to maintain aquatic habitat potential in southern part of the watershed. • Diversion works to Moira River are likely to cause some environmental impacts along the diversion route.

considered to be marginally better environmentally than Option 1, since the use of source controls with Option 2 may provide better potential for maintenance of local water regimes affecting the Cloverleaf woodland, the wetland north of Highway 401, and water-related features south of Highway 401. For instance, discharge of residential roof drainage onto grassed areas will help maintain diffused water inputs to the creek via soil-water seepage and water table maintenance. As well, peak flows into the creek and the wetland north of Highway 401 will be somewhat attenuated, potentially helping to provide lower water level fluctuations than in Option 1. Similarly, south of Highway 401, use of rooftop storage and other source controls will help with flow attenuation and will help to prolong water inputs to the creek itself.

Option 4, although preferable in terms of costs, will potentially have a significant negative effect on the silver maple swamp north of Cloverleaf Drive. Therefore, it is ruled out based on the inability to meet the natural environment objective # 1 above.

From this review of cost and environmental considerations, it is clear that at this level of analysis, System Option 2 is the preferred approach to water management for the Upper No Name Creek Watershed.

6.6 PUBLIC REVIEW

An Open House on the Upper No Name Creek Water Management Study was held by the MRCA on December 7, 1994. This Open House was intended to provide the public an opportunity to review the Draft Report, and comment on the preferred System Option selected as a result of the analysis discussed in this report, and discussion with the Steering Committee. The Open House was well attended (an attendance list is provided in Appendix I). No objections to the selection of System Option 2 as the preferred water management strategy were brought forth at the Open House.

SECTION 7

IMPLEMENTATION

IMPLEMENTATION

7.1 THE PREFERRED OPTION

As concluded in the previous section, System Option 2 has been judged to be the preferred strategy for urban water management in the watershed. This conclusion has been based on considering costs as well as potential impacts on remaining natural features along the creek.

Figure 7-1 is a concept plan depicting the primary elements (flow conveyance channels and stormwater management/treatment facilities) of the preferred management strategy. Included on the plan are boundaries representing constraints to development with respect to existing natural features within the watershed. The constraints are based on protection of all Zone 1 areas in the watershed, as well as consideration of existing floodlines. The constraints include all areas labelled as 1 through 4 on the vegetation communities map in Figure 2-2. These include the two patches of marsh between the 401 and Bell Boulevard, the beaver pond area north of the 401, the lowland swamp between Cloverleaf and Maitland Drives, the marsh just east of Sidney Street, and the lowland forest west of Sidney Street.

The Zone 2 white cedar forest south of the 401 is an area also worth consideration for protection from future development, if feasible, due to its extent and maturity, although it is not included as a strict constraint in the Upper No Name Creek management plan.

The water management strategy incorporates two stormwater ponds to provide flow control and stormwater treatment by natural settling of pollutants and biological treatment:

Pond 1: Located along the creek immediately north of Ontario Hydro ROW and extending to Bell Boulevard. The facility is proposed to be a linear, multi-celled pond centred on the Lemoine Street road allowance. This facility will include an effluent UV disinfection unit.

Pond 2: Located at the existing wetland (beaver pond) located along the creek just north of Highway 401. This facility would be constructed as a wetland-type facility to make use of the existing wetland area.

The present study has provided planning-level estimates of the stormwater storage volumes that are required at both facilities to effect adequate stormwater treatment as well as control of peak flows up to the 100-year return period. These pond volumes are summarized in Table 7-1. Pond volumes are estimated based on full implementation of source controls in all new residential and industrial/commercial development. See Section 7.4 for details on source control implementation. Pond dimensions are also included in Table 7-1.

Table 7-1
SUMMARY OF STORAGE VOLUME, LAND AREA REQUIREMENTS AND DIMENSIONS FOR THE TWO STORMWATER PONDS

	POND 1 TRACEY STREET	POND 2 NORTH OF 401
Permanent pool volume required for adequate solids removal and clarification	15,000 m^3	6,700 m^3
Peak live storage volume at 100-year return period	5,300 m^3 below elevation 92.0 m	20,600 m^3 below elevation 95.1 m
Estimated land area requirement	1.2 ha	2.6 ha
Bottom width	32 m	50 m
Total length	210 m	360 m
Side slopes	3:1 (H:V)	4:1 (H:V)
Permanent pool depth	1.9 m	0.4 m
Total depth	2.5 m	1.4 m

Implementation of the stormwater strategy will require:

- final design analysis and detailed design of the recommended stormwater ponds, flow conveyance systems and culvert improvements.
- fulfilment of Environmental Assessment and regulatory approval requirements.
- determination of phasing requirements.
- definition of cost-sharing arrangements.

The various requirements are discussed in the following sections.

7.2 *FINAL DESIGN AND E.A. REQUIREMENTS*

Detailed design of the various components of the stormwater strategy will be needed to obtain regulatory approvals and thereby allow proposed land developments to proceed.

7.2.1 *Stormwater Treatment Facilities*

The design process for both Pond 1 and Pond 2 should follow the framework of the Class EA for Municipal Water and Wastewater Projects (June 1993), in order to meet the intent and requirements of the Province's EA Act. The present study will serve as a support document that will help with fulfilment of Phase 1 (Problem/Need

Identification) and Phase II (Selection of Preferred Alternative) of the Class EA process.

Design of the Tracey Street stormwater pond and UV facility should proceed according to the Municipal Engineers Association's Class EA for Municipal Water and Wastewater Projects (June 1993). If and when UV disinfection is implemented, it will be necessary to complete a "Schedule C" Environmental Study Report (ESR) that justifies the need for the facility and presents the rationale for the selected design.

Design of the two stormwater ponds will include:

1
2
3 [illegible] EA

- detailed site layout and grading plans, designed to provide the required stormwater storage volumes.
- detailed design of inlet and outlet control structures that regulate storage levels and peak water levels within each facility, while optimizing the requirements for satisfactory UV treatment.
- preparation of landscaping plans, and detailing of other design features such as access roads, fencing, etc.

It is important to note that there is an 18" sanitary sewer within the Lemoine Street road allowance. Therefore in the final design of Pond 1 at Tracey Street, consideration will have to be made of this sanitary sewer. Either sewer re-location or pond design refinements to allow for the construction of the pond above the sewer will have to be considered.

It must be emphasized that further hydrologic and hydraulic analyses are likely to be required at the final design stage, in order to optimize the size and operation of both of the facilities.

In particular, optimization analysis should be carried out during final design of the Tracey Street pond to ensure optimum sizing of the UV facility. Various operational design options could be considered, including possible use of automated outflow control (e.g. operated overshot gate; batch-type operation) to help minimize UV treatment rate and total treatment duration. Preliminary checks made during this study have indicated that it should be possible to further decrease the required UV treatment rate somewhat, by increasing active storage at the Tracey Street pond by, for instance, increasing pond surface area or operational water-level fluctuation. This optimization potential should be considered in detail during the final design process.

7.2.2 *Stormwater Conveyance Channels*

As described earlier, the preferred strategy incorporates conveyance channels along the existing creek alignment. These conveyance links will serve to collect runoff from new developments and provide safe conveyance to Ponds 1 and 2.

The technical analysis carried out in this study has resulted in planning-level estimates of the 100-year flowrates that must be accommodated by these flow conveyance channels.

Preliminary Design Concept:

The intent is that the conveyance channels will contain the 100-year flow within a designated floodway area. The benefit of this approach is that the area flooded during the 100-year Regulatory Storm can be limited to a floodway that is generally narrower than the estimated extent of the existing floodway (see Appendix E, Figure E-1).

It is suggested that design of the conveyance channel/floodway links consider the concepts of "natural" channel design, in which techniques such as placement of root wads and establishment of dense vegetation are used to maintain channel stability, as opposed to placing sole reliance on more traditional techniques such as gabions and imported rip-rap stone. Where velocity and erosion control is a concern, the natural design approach could include the creation of pool-riffle sequences that would help control velocities while enhancing local aquatic habitat.

Figure 7-2 depicts the general design concept envisaged for the conveyance channel/floodway links. As indicated on this sketch, the general concept is that the channel/floodway be designed as a naturalized flow route; that is, designed and landscaped to have a natural character. This approach will help to minimize net impacts on the local environment, and will provide potential for local enhancement of aquatic or riparian habitats which are at present highly disturbed.

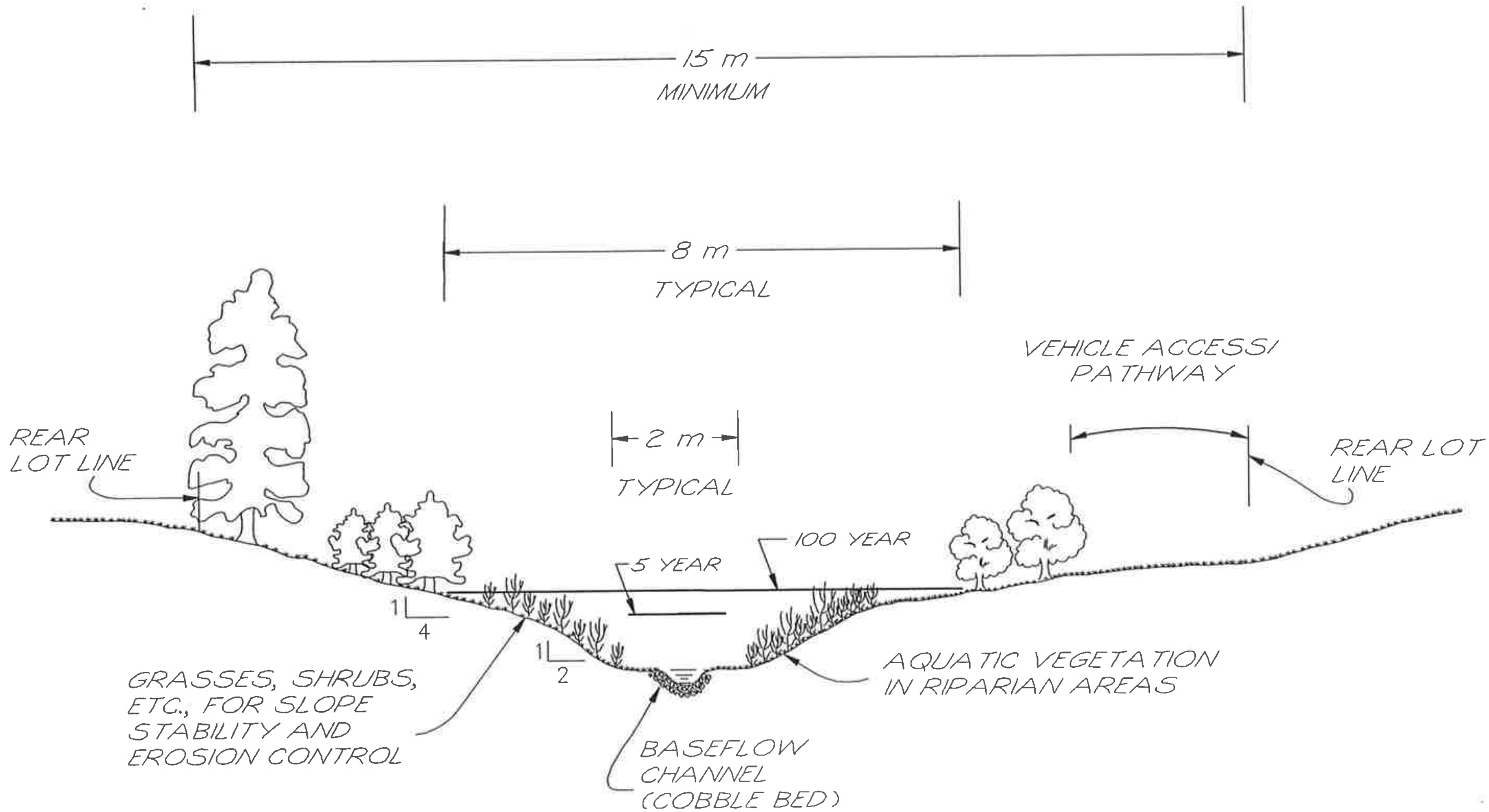
For example, the bed of the primary flow channel could be lined with cobbles and stones to minimize erosion potential while also improving physical diversity and aquatic habitat potential. The channel banks and sideslopes can be stabilized via vegetative plantings including grasses, sedges, shrubs, and aquatic species. Finally, to allow use of the channel floodways as parkland, some areas of turf grass could be incorporated within the overall landscape plan.

Estimation of Land Area Requirements:

Hydraulic calculations indicate that south of Maitland Drive to Pond 2 (just north of Highway 401), the floodway channel can be designed to provide a 100-year flow width of approximately 8 m; south of the 401 to Pond 1 at the Tracey Street sewer inlet, the floodway channel can also be designed to provide a 100-year flow width of about 8 m, with a 100-year flow depth of about 1 m along most reaches. These estimates can be used at the planning level to determine what land area should be set aside for the flow conveyance channels and associated floodplain area. However, the following factors must also be taken into account:

- Maintenance access along the channels will be needed to allow routine inspection, debris removal, repair of any erosion problems, etc. Vehicle access should be possible along at least one side of the flow conveyance channels.

- Energy-gradient control (i.e. velocity control) measures are likely to be needed along some sections of the proposed conveyance channels where the natural topography will cause energy gradients and flow velocities which could cause



NOTE: Typical 5-year flow depth is approx. 0.65 m
Design 100-year flow depth is 1.00 m

MOIRA RIVER CONSERVATION AUTHORITY	**TYPICAL CONVEYANCE CHANNEL CROSS SECTION**	**Gore & Storrie Limited G&S** Consulting Engineers
	FILE Nº \7644\DWGS\FIG7-2	FIG Nº 7-2

erosion problems. This will be an important design consideration in sections south of Maitland Avenue through Cloverleaf Drive to Pond 2 north of Highway 401. Final design must consider flow velocities and erosion potential. It may be necessary to incorporate drop structures to control velocities during the more frequent runoff events, along with channel protection measures such as rip-rap or bio-engineered forms of channel reinforcement (*e.g.* use of root wads, densely-rooted vegetation, etc.). As part of a "natural" design approach, there is potential to design velocity-control measures in the form of pool-riffle sequences that could provide enhancement to local aquatic habitat.

- A vegetated buffer should be maintained along each side of the conveyance channel to minimize the opportunity for direct pollutant washoff into the channel from adjacent development sites.

MAC5 lands

To allow final design of the flow conveyance channels to incorporate adequate conveyance capacity, maintenance access, opportunity for velocity-control and erosion-control measures (including opportunity for use of natural design approaches), a minimum conveyance channel/floodway allowance of 15 metres is recommended throughout the system, as presented in Table 7-2. Note that the width of the conveyance channels from downstream of Maitland Drive to Pond 2 at Highway 401 may need to be greater than 15 m due to the significant slope through these reaches, to permit adequate velocity-control via the addition of channel meanders. Final design is necessary to determine the required width.

Table 7-2
RECOMMENDED FLOODWAY WIDTH ALLOWANCE

CONVEYANCE LINK	Estimated peak 100-year flood flow (with System Option 2)	*Minimum* Recommended floodway width allowance
DITCH to MAITLAND DRIVE	2.6	15 m
MAITLAND DRIVE to CLOVERLEAF DRIVE	3.8	15 m
CLOVERLEAF DRIVE to POND 2 (at Hwy. 401)	4.6	15 m
POND 2 to HWY. 401	1.4	15 m
HWY. 401 to Existing Channel at 100 m north of Bell Boulevard	1.6	15 m

Figure 7-1 shows the proposed alignments for the conveyance routes; however, these alignments are subject to change as development concepts are formulated and designed. (Note that no conveyance channel has been proposed from 100 m north of Bell Boulevard to the Tracey Street storm sewer inlet. Preliminary calculations showed that the existing channel is capable of conveying the 100-year flows estimated for this reach). Between Cloverleaf Drive and Maitland Drive, and through the marsh area south of the 401, the constraint boundaries are wider than the 15 m conveyance channel width proposed for the remaining areas, to ensure protection of the identified Zone 1 areas. In these areas, a floodway approximately equivalent to the width of the existing floodline will need to be maintained.

7.3 REGULATORY APPROVALS

A number of specific regulatory approvals will be needed to allow implementation of the various components of the water management strategy. These include:

- <u>Lakes & Rivers Improvement Act</u> approval will be needed for:

  - approval of impoundment and outlet control works associated with the two stormwater ponds
  - culvert improvements at Maitland and Cloverleaf Drives.

  These approvals are administered by the Ontario Ministry of Natural Resources. Approval applications require detailed design information.

- <u>Alteration to Waterway Regulations (Conservation Authorities Act)</u> approval will be required in connection with the stormwater ponds and channel alterations needed for implementation of the flow conveyance channels. These approvals are administered by the Moira River Conservation Authority.

- <u>Ontario Water Resources Act</u> approval will be required for the two stormwater management ponds, and <u>Certificates of Approval</u> will be issued by the Ontario Ministry of Environment & Energy based on approval of the final facility designs. The Certificates will specify compliance requirements with respect to stormwater quality control (*e.g.* specific targets for bacteria and suspended solids concentrations in facility effluent) and will likely also specify monitoring required to demonstrate compliance.

In addition to the above, the Ontario Ministry of Natural Resources will likely require that the final design of the various components demonstrate that there will be <u>no net loss of existing fish habitat</u>, or that the proposed works will provide adequate replacement habitat. This requirement is likely to have implications for the final design of the stormwater ponds and the conveyance channels. More detailed surveys of existing aquatic habitat may be needed to define the existing habitat conditions.

7.4 IMPLEMENTING SOURCE CONTROLS

The recommended SWM strategy calls for the following at-source or on-site controls within new development areas:

- Residential Source Controls: ensuring that roof drainage is diverted onto grassed areas, as opposed to being drained onto paved areas or directly connected to the storm sewer.

- Industrial/Commercial On-site Controls: consisting of rooftop storage on flat-roofed buildings, along with catchbasin inlet controls in parking areas, to restrict peak outflow rates.

Modelling of System Option 2 has been based on explicit representation of these distributed control measures. They must therefore be incorporated in all new development sites for the overall system to provide the required performance in terms of peak flow control and stormwater treatment.

Modelling of the on-site measures has produced estimates of peak outflow rates that would result from the various types of urban development, as listed in Table 7-3.

Table 7-3
TARGET PEAK FLOWS WITH SOURCE CONTROLS

Return Period	Unit-area Outflow Rates (litres/sec per ha of catchment area)	
	Industrial/Commercial sites	Residential areas
2 years	15	15
5 years	20	20
10 years	25	25
25 years	30	30
100 years	40	40

The unit-area values are the same for both residential and industrial/commercial development. This results from the fact that although industrial/commercial development will typically produce more runoff per unit area (due to higher imperviousness), more flow attenuation is possible on the industrial/commercial sites by way of rooftop and parking-lot storage.

7.5 PHASING

Phasing is an important aspect of implementation. The recommended facilities should be constructed concurrently with land development, but should be implemented only when it is clear that development will proceed. Phasing and timing of the facilities must also recognize that development within the watershed is likely to extend over a number of years, as individual land developers proceed on their respective properties.

7.5.1 *Phasing of the Two Ponds*

Construction of the Tracey Street pond and the Highway 401 wetland pond will be needed as development begins in each respective drainage area. At this juncture it is difficult to predict the timing for proposed developments on specific properties in each drainage area. However, it is expected that the most immediate development possibilities are as follows:

- City of Belleville:
  - Zellers retail store and White Rose retail store: Commercial developments adjacent to the creek north and south of Bell Boulevard.
- Township of Thurlow:
  - Cloverleaf Industrial Park: Industrial/commercial development on lands on both sides of the creek, between Highway 401 and Cloverleaf Drive
  - Cloverleaf Estates: Residential development adjacent to both sides of the creek between Cloverleaf Drive and Maitland Avenue.

In other words, there appears to be immediate potential for significant development within both the Belleville and Thurlow portions of the watershed. The Zellers and White Rose proposals are the most immediate.

Design and construction of both stormwater ponds would ideally occur concurrently, so that both facilities are in place to allow for concurrent development in Belleville and Thurlow. However, the most recent information indicates that the Zellers and White Rose proposals in Belleville are scheduled to proceed the most quickly.

Therefore, it appears that proceeding with the Tracey Street pond (Pond 1) should be considered as the top priority. This facility can be constructed ahead of the Thurlow pond (Pond 2), provided that no substantial development takes place within the Thurlow portion of the watershed. Pond 2 should be constructed concurrently with any significant development in the Thurlow portion.

As discussed in Appendix F, the Tracey Street pond might ideally be designed as a multi-cell facility. This can help to optimize stormwater treatment. As well, a multi-cell design could potentially allow the Tracey Street pond to itself be phased as development occurs in Belleville. This is a consideration that should be brought into the final facility design process. Based on the expected timing of specific developments at the time of final pond design, it may be possible to design and construct only a portion of a multi-cell design, in order to provide adequate treatment and flow control for some initial amount of development. However, it must be emphasized that the design and approval process should be based on final detailed design of the ultimate facility.

Further discussions are recommended between the two municipalities, land developers and the Moira River Conservation Authority, to establish a realistic schedule for design and phasing of the two stormwater ponds.

7.5.2 *Implementing Active Disinfection*

The recommended SWM strategy calls for the installation of active disinfection of the effluent from Pond 1 (at the Tracey Street sewer inlet). Review of available physical and chemical disinfection technologies has concluded that UV irradiation is the preferred technique for effluent disinfection. The UV process is being more commonly applied in municipal wastewater disinfection, due to the fact that UV technology has become cost-competitive, as well as the fact that UV produces no known hazardous by-products or residuals.

The conclusion that active effluent disinfection is needed in the Upper No Name Creek watershed results from the requirements for stringent control of coliform bacteria levels in summertime discharges to the Moira River. In keeping with the Bay of Quinte RAP, mean event discharge concentration to the Moira must be 100 *E. coli* per 100 ml or lower during the swimming season (with an allowance that there can be up to 4 events per season in which this target is not met).

While natural pollutant settling and bacteria die-off in stormwater ponds can provide a high degree of bacteria removal, research in Ontario (*e.g.* Rideau River Stormwater Management Study, Regional Municipality of Ottawa-Carleton, 1983 and 1992) has shown that stormwater ponds cannot reliably provide effluent coliform levels as low as 100 *E. coli* per 100 ml. In effect, research to date indicates that natural settling and bacteria die-off in stormwater ponds cannot be relied upon to meet the *E. coli* target and therefore cannot be used as a basis for design to achieve regulatory compliance.

Nonetheless, there are a number of factors that should be considered:

- It is known that natural settling and natural bacteria die-off can provide a high degree of bacteria removal (*i.e.* a high level of bacteria load reduction), even though resulting effluent *E. coli* levels will remain consistently above 100 per 100 ml. Monitoring in Ottawa-Carleton (RMOC, 1992) and elsewhere has shown that coliform load reductions of 90% or greater are consistently achievable in stormwater ponds that provide adequate retention time.

- Research has also shown that constructed wetland ponds can also provide a high level of bacteria removal that is assisted by microbial activity associated with aquatic vegetation (Pullin and Hammer, 1991; Taylor, 1992).

- Effluent disinfection at the Tracey Street sewer inlet may not be the optimum location for active disinfection in the Tracey Street sewershed. Disinfected flows from the Upper No Name Creek would likely become re-contaminated along the Tracey Street sewer, which currently receives untreated urban stormwater from an additional area of about 50 hectares with an estimated imperviousness of 40%. The Upper No Name Creek watershed consists of approximately 511 hectares (not including Subcatchment 119, which contributes only major flows) with an average impervious level under fully developed conditions of 22%; that is, an estimated 15% of the Tracey Street sewershed enters the sewer downstream of the Upper No Name Creek input. This means that there is definite potential for recontamination along the Tracey Street sewer. In the context of overall pollution control planning, it may be more reasonable and practical to install active disinfection at the bottom end of the Tracey Street system, immediately before the outfall to the Moira, to provide control for the entire sewershed. The ultimate location of the UV disinfection facility will have to be determined during the final design stage.

- Further urban development in the Upper No Name Creek watershed is likely to occur over a number of years, and it may be 10 or more years before ultimate "build out" conditions. Therefore, it will likely be a number of years before the full UV treatment capacity is required.

Based on these considerations, and given that installation of a UV disinfection facility at Pond 1 will involve considerable capital costs and ongoing operating & maintenance costs, it is recommended that implementation of active disinfection proceed as outlined in Table 7-4.

It is recommended that discussions be held with the Ontario Ministry of Environment and Energy (MOEE) to determine if interim Certificates of Approval for the stormwater treatment ponds can be acquired by following the above approach in which installation of active disinfection would be delayed until monitoring of pond performance has shown whether or not it is needed to meet regulatory and policy objectives.

Table 7-4
SUGGESTED APPROACH TO IMPLEMENTATION OF ACTIVE DISINFECTION AT TRACEY STREET POND

Step 1: CONSTRUCT THE PONDS	Construct the recommended two-pond stormwater management & treatment system without incorporating the UV unit (but designing the Tracey Street pond and setting aside land area as needed to allow UV unit to be installed in future)
Step 2: MONITOR	Monitor system performance in terms of effluent coliform levels and in terms of coliform load reduction. This will require monitoring of coliform influent & effluent loads at both ponds.
Step 3: REVIEW	Review the performance data vis-a-vis regulatory requirements and policies to determine if bacteria control is adequate. Also review pollution control planning strategies to determine the optimum location for active disinfection in the Tracey Street system.
Step 4: IMPLEMENT IF NEEDED	Proceed with implementation of active disinfection at the Tracey Street pond if: • Performance data indicates that natural removal is not adequate, and • Stormwater disinfection at the Tracey Street pond fits with overall pollution control plans for the Tracey Street system and the larger Belleville area.

7.6 *COST SHARING*

An equitable cost-sharing arrangement for the major components of the recommended stormwater management system is necessary.

7.6.1 *Cost to be Shared*

The estimated capital costs, including 20% contingency, for the major components of the recommended SWM system are included in Table 7-5.

The need for these proposed facilities results from the fact that

- If flows are not controlled, proposed development will cause peak flows along Upper No Name Creek that will exceed the hydraulic capacity of the existing system at Highway 401 (1.4 m^3/s) and will result in a 100-year flow of 8.1 m^3/s at the Tracey Street sewer inlet, where inlet capacity is only 3.5 m^3/s.

Table 7-5
CAPITAL COSTS OF PROPOSED WORKS

FACILITY DESCRIPTION	Estimated Capital Cost (to nearest $10,000)
POND 1: Tracey Street detention/treatment pond	$1,530,000
UV FACILITY at the Tracey Street pond	$700,000
POND 2: Detention/treatment wetland pond north of Hwy. 401	$1,520,000
TOTAL	$3,750,000

- Stormwater treatment is needed for approval of all new urban development, to meet the requirements of policies and regulations put in place by the Province and the Bay of Quinte RAP. Control of suspended solids and bacteria levels in stormwater discharges to the Moira River and Bay of Quinte is the particular requirement in the Quinte RAP area.

7.6.2 *Rationale for Cost Sharing*

The fundamental principle on which cost sharing should be based is as follows:

> ***Costs for the proposed facilities should be shared equitably amongst those who benefit from them.***

In applying this principle, there are some specific considerations that come into play:

- The recommended stormwater system represents a "centralized" approach, in which flow control and treatment occur at two central pond facilities located along the existing drainage channel, Upper No Name Creek. This is in contrast to a piecemeal approach in which each individual development site would incorporate its own flow-control/treatment facility.

- This centralized approach is based on minimizing overall costs (initial capital costs and annual O&M costs) by minimizing the number of separate stormwater facilities within the watershed.

- Because they are situated along the watershed's natural channel, the size and cost of these central facilities will be partly due to the fact that they are receiving runoff from the entire upstream area, not just the proposed urban development that has brought about the need for the facilities.

Essentially, the proposed centralized system must accommodate runoff from lands that presently drain to the creek. This must be done in a way that does not cause any negative impacts (e.g. flood risk or erosion) to those lands.

Based on these considerations and the principle that those who benefit should share the costs, the following recommendation is made:

> ***The cost of the proposed works should be shared amongst only new land development within the watershed.***

The primary reason is that the proposed facilities are needed only to allow proposed land developments to be approved. Therefore, only those properties receive direct benefits.

While a portion of the size (and, therefore, costs) of the facilities is due to the fact that the recommended system will also handle runoff from existing agricultural, rural residential and urban development, it is not reasonable or equitable to require that this existing development share the costs. Outlet for existing development must be maintained. The fact that doing so affects the total cost is a circumstance that cannot be considered as a reasonable basis for asking that existing development cover a portion of the cost.

7.6.3 Inter-Municipal Considerations

The proposed facilities are based on the stormwater management requirements for proposed development in the City of Belleville and in the Township of Thurlow.

It is expected that these two municipalities will ultimately be owners and operators of the two stormwater facilities; that is, that City of Belleville will be owner/operator of the Tracey Street facility (Pond 1 and UV unit) and that Township of Thurlow will be owner/operator of the wetland facility north of the 401 (Pond 2).

In assessing inter-municipal cost sharing, the following considerations come into play:

- By way of Pond 2, peak-flow control to existing levels will be provided for the entire watershed north of Highway 401.
- For the area north of the 401, Pond 2 will also provide stormwater quality control in the form of suspended solids removal by natural settling. The level of treatment is consistent with recent Provincial guidelines (as given in the MOEE's "Stormwater Management Practices Planning & Design Manual, June 1994).

On this basis, it is apparent that Pond 2 will meet the stormwater management and treatment requirements for proposed development in the Thurlow portion of the watershed, except that the bacteria control requirement for Thurlow stormwater will not have been met. Therefore,

> ***Thurlow Township does not have an obligation to defray any portion of the costs of stormwater management south of Highway 401 that are associated with peak-flow control or removal of suspended solids.***

Thurlow Township should be responsible for a portion of the costs for the end-of-system UV disinfection unit.

Essentially, the cost-sharing requirement is that the two municipalities should share the cost of the UV disinfection unit.

7.6.4 Cost Sharing Calculations

To apportion costs between the two affected municipalities or between individual development properties within either municipality, we need a means of estimating the net contribution of specific areas or properties to the cost of each of the proposed facilities.

This could theoretically be done using the system model created in this study. Simulations could be carried out to examine the cost contribution of individual properties, by eliminating individual developments from the model, and then re-sizing and re-costing the facilities. This would obviously involve numerous modelling and re-costing scenarios. The procedure would also be complicated by the hydraulics of the system; for instance, there would not necessarily be a clear relationship between active storage required at the Tracey Street pond and tributary development area, because of the relative timing of runoff hydrographs and the flow attenuation that happens along the system. Essentially, a model-based multi-scenario approach to isolating the cost attributable to specific development areas would be complex and would place heavy reliance on modelling and costing assumptions. Furthermore, the results could be difficult to interpret and convert to a cost-sharing agreement that is considered reasonable and equitable by all affected parties.

Consequently, it is recommended that a more direct method be used to estimate the cost share for specific areas or properties. To do so, the following points should be recognized:

- Costs for Pond 1 and Pond 2 are dictated primarily by the recommended permanent pool and active storage volumes needed for peak-flow control and treatment. These design volumes are, in turn, dictated largely by the volume of runoff produced by the tributary areas. To a lesser extent, the facility sizes will also be dictated by the peak rates of runoff and the "peakiness" of inflows into those facilities. However, total inflow volume will be the dominant factor.

- Costs for the UV facility is determined primarily by the design treatment rate (estimated to be 200 litres/sec for the recommended system). The treatment rate is dictated by the rates, peakiness and volumes of runoff entering the system, as well as the active storage and operational hydraulics of the system. However, because of the amount of active storage in the system (as required for flow control) the design UV treatment rate will be largely dictated by event runoff volumes.

For these reasons it is recommended that:

> ***Facility costs should be allocated in proportion to the volume of storm-event runoff generated by the proposed developments that drain to the facility.***

To implement cost-sharing on this basis, some measure or index of runoff volume is needed. It is suggested here that the most reasonable method is to use a "runoff coefficient" to characterize the runoff volume generated by a unit of development area. The runoff coefficient would therefore be based on the imperviousness of the development area. By multiplying the runoff coefficient with the land area, we arrive at an indicator of the relative runoff volume contribution for each area or property.

This approach has been applied by:

- Identifying the individual development areas or properties within the Thurlow and Belleville portions of the watershed. These are indicated on Figure 7-3.
- Estimating the overall imperviousness of each parcel, by considering the expected type of development and any areas alongside the creek that are expected to remain undeveloped due to environmental or floodplain constraints; these areas are delineated by the green line shown on Figure 7-3.
- Estimating a runoff coefficient for each parcel.
- Tabulating runoff coefficient-times-area values for each parcel, and computing totals for development areas within Thurlow and Belleville.

The parcel areas and impervious estimates are shown on Figure 7-3.

The resulting tabulation is presented in Table 7-6.

The results can be summarized in terms of the percentage shares that are attributable to each of the two affected municipalities, as shown in Table 7-7.

It is expected that each municipality, as ultimate owner/operator of each facility, will collect or recoup costs from individual developments as they occur or as they are approved.

Therefore, the cost-sharing percentages presented in Table 7-6 also need to be expressed in terms of the costs attributable to each of the 13 development parcels shown on Figure 7-3. See Table 7-8.

7.6.5 Conclusion

The above recommendations and tabulations on cost sharing have been reviewed by the City of Belleville and the Township of Thurlow, as well as representatives of the affected land developers. Agreement has been reached that the above rationale and method of calculation are reasonable and equitable. On this basis, the

TABLE 7-6
DATA FOR COST-SHARING CALCULATIONS

Belleville portion:

DEVELOPMENT PARCELS SOUTH OF HWY 401

Property number (Fig 7-3)	Reference	Type of development	Area (ha)	Impervious Percent	Est. Runoff Coeff.	Area times R.C.	PERCENTAGES within Belleville	of total:
1	Citation Group	COMMER (Zellers)	7.1	80%	0.77	5.432	27.5%	12.2%
2	Hawley/Ming Dev.	COMMER/IND.	7.1	80%	0.77	5.432	27.5%	12.2%
3	Hawley/Ming P.II	RESID	2.3	25%	0.33	0.748	3.8%	1.7%
4	R. & E. Robinson	RESID	0.6	25%	0.33	0.195	1.0%	0.4%
5	Bradlaw Ent.	COMMER/IND.	4.8	80%	0.77	3.672	18.6%	8.2%
6	CharComp Dev. Inc.	COMMER (White Rose)	2.0	60%	0.61	1.210	6.1%	2.7%
7	Cream of the Crop Dev.	COMMER/IND.	2.3	70%	0.69	1.575	8.0%	3.5%
8	Cambridge Group	COMMER/IND.	2.8	50%	0.53	1.470	7.4%	3.3%
		TOTALS	29.0			19.733	100.0%	44.3%

Thurlow portion:

DEVELOPMENT PARCELS NORTH OF HWY 401

Property number (Fig 7-3)	Reference	Type of development	Area	Impervious Percent	Est. Runoff Coeff.	Area times R.C.	PERCENTAGES within Thurlow	of total:
10	Immediately N. of 401	COMMER/IND.	20.6	75%	0.73	14.935	60.2%	33.5%
11	Between 401 and Cloverleaf	COMMER/IND.	3.0	80%	0.77	2.295	9.3%	5.2%
12	Between Cloverleaf and Maitland	RESID.	14.6	22%	0.30	4.395	17.7%	9.9%
13	N. of Maitland	RESID.	8.7	30%	0.37	3.175	12.8%	7.1%
		TOTALS	46.9			24.800	100.0%	55.7%

Table 7-7
INTER-MUNICIPAL APPORTIONMENT

Municipality	SHARE OF COSTS based on runoff coefficient-times-area for proposed development areas		
	Tracey Street pond	**Pond north of 401**	**UV facility at Tracey pond**
Belleville	100%	0%	44.3%
Thurlow	0%	100%	55.7%

recommendation is that Tables 7-6, 7-7 and 7-8 should be the basis for formal cost-sharing agreements and arrangements amongst the affected parties. It is recommended that both initial capital costs and annual costs for operation, maintenance and monitoring of the facilities should be shared according to the percentages in the tables.

Each municipality will need to consider the mechanisms by which cost-sharing and cost recovery can be achieved. For instance, site-plan agreements could be one option for recouping cost shares from specific developments. In finalizing cost-sharing agreements and mechanisms, both municipalities should explicitly account for the possible eventual need for the UV facility. In other words, collection of cost shares should be based on including the costs associated with the UV facility so that if and when the need for the UV facility arises, the funds are available.

In future, it needs to be recognized that the stormwater strategy and cost sharing recommended by this study are based on the extent of land development indicated on Figure 7-3. If additional urban development unforeseen in this study is contemplated at some time in the future, then that development will be responsible for its own stormwater management/treatment measures to meet regulatory targets. For instance, if any development is eventually proposed within the Sidney Township portion of the watershed, then that development would be responsible for meeting stormwater management/treatment requirements prior to discharge to the downstream system.

TABLE 7-8
COST-SHARING TABULATIONS FOR DEVELOPMENT PARCELS

Belleville portion:

DEVELOPMENT PARCELS SOUTH OF HWY 401

Property number (Fig 7-3)	Reference	Type of development	Area (ha)	PERCENTAGES within Belleville	PERCENTAGES of total:	COST SHARES Pond 1	UV facility	Pond 2	TOTAL
1	Citation Group	COMMER (Zellers)	7.1	27.5%	12.2%	$421,000	$85,000	$0	$506,000
2	Hawley/Ming Dev.	COMMER/IND.	7.1	27.5%	12.2%	$421,000	$85,000	$0	$506,000
3	Hawley/Ming P.II	RESID	2.3	3.8%	1.7%	$58,000	$12,000	$0	$70,000
4	R. & E. Robinson	RESID	0.6	1.0%	0.4%	$15,000	$3,000	$0	$18,000
5	Bradlaw Ent.	COMMER/IND.	4.8	18.6%	8.2%	$285,000	$58,000	$0	$343,000
6	CharComp Dev. Inc.	COMMER (White Rose)	2.0	6.1%	2.7%	$94,000	$19,000	$0	$113,000
7	Cream of the Crop Dev.	COMMER/IND.	2.3	8.0%	3.5%	$122,000	$25,000	$0	$147,000
8	Cambridge Group	COMMER/IND.	2.8	7.4%	3.3%	$114,000	$23,000	$0	$137,000
		TOTALS	29.0	100.0%	44.3%	$1,530,000	$310,000	$0	$1,840,000

Thurlow portion:

DEVELOPMENT PARCELS NORTH OF HWY 401

Property number (Fig 7-3)	Reference	Type of development	Area	PERCENTAGES within Thurlow	PERCENTAGES of total:	COST SHARES Pond 1	UV facility	Pond 2	TOTAL
10	Immediately N. of 401	COMMER/IND.	20.6	60.2%	33.5%	$0	$235,000	$915,000	$1,150,000
11	Between 401 and Cloverleaf	COMMER/IND.	3.0	9.3%	5.2%	$0	$36,000	$141,000	$177,000
12	Between Cloverleaf and Maitland	RESID.	14.6	17.7%	9.9%	$0	$69,000	$269,000	$338,000
13	N. of Maitland	RESID.	8.7	12.8%	7.1%	$0	$50,000	$195,000	$245,000
		TOTALS	46.9	100.0%	55.7%	$0	$390,000	$1,520,000	$1,910,000

7.7 *OVERALL IMPLEMENTATION STRATEGY*

Based on the foregoing discussion of requirements regarding final design, environmental assessment, phasing and cost sharing, an overall implementation strategy can be summarized as outlined in Table 7-9.

This overall strategy fits within the framework of the Class EA process for Water and Wastewater projects. Throughout design and implementation, the Class EA process should be regarded as the guiding framework. This will ensure sound decision making based on consideration of all cost and environmental factors.

Table 7-9
OVERVIEW OF SUGGESTED IMPLEMENTATION STRATEGY

Step 1: PRELIMINARY ENGINEERING DESIGN	• Initiate design process for the two stormwater ponds by starting Class EA process (including public notification) • Review UV phasing strategy with MOEE • Determine necessary timing for construction of each pond via discussions between municipalities, developers, MRCA • Examine design options, submit for public & governmental review (as per Class EA process) • Submit EA documents (*e.g.* ESR) once preferred designs (preliminary engineering designs) are ready
Step 2: FINAL DESIGN & APPROVAL	• Proceed with final detailed design of the facilities • Apply for various regulatory approvals including Certificates of Approval from MOEE
Step 3: CONSTRUCTION	• Proceed with tendering and construction of the facilities, without UV disinfection (subject to MOEE interim approval)
Step 4: MONITOR	• Establish monitoring program to measure bacteria removal efficiency of the two pond system • Compile and document data annually
Step 5: REVIEW NEED FOR DISINFECTION	• Review performance data with MOEE and MRCA to decide upon need for effluent disinfection • Proceed with installation of disinfection facility if needed

REFERENCES

REFERENCES

Canadian Council of Resource and Environment Ministers, 1987. Canadian Water Quality Guidelines. Prepared by Task Force on Water Quality Guidelines.

Chapman, L.J., and Putnam, D.F., 1984. *Physiography of Southern Ontario*. Third Edition. Ministry of Natural Resources, Ontario.

Crysler and Lathem Ltd., 1977. Belleville Water Management Study. Prepared for Quinte Planning Board.

Cuddy, D.G., 1991. Vascular Plants of Eastern Ontario. Draft. Eastern Region, Ontario Ministry of Natural Resources, 20 pp.

Ecos Garatech Associates Ltd., 1990. Storm Water Management Study - Cloverleaf Industrial Park, Rainbell Investments Limited, Township of Thurlow. Prepared for Van Meer Limited.

Ecos Garatech Associates Ltd., 1993. 1:16,000 scale map - "Potter Creek Subwatershed Plan - Surface Water Strategy 1", drawing number 1137-16-18.

Energy Mines and Resources, 1971. 1:25,000 Scale Map of Belleville West, Ontario.

Environmental Protection Agency, 1986. Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality. EPA/440/5-87/001.

Environmental Protection Agency, 1986b. Design Manual - Municipal Wastewater Disinfection. Center for Environmental Research Information, Cincinatti, OH.

Falcone Smith Associates, 1990. City of Belleville No Name Creek Stormwater Management Study. Prepared for JRH Holdings Inc.

Gietz, R.J., 1983. Urban Runoff Treatment in the Kennedy-Burnett Settling Pond. Rideau River Stormwater Management Study, Regional Municipality of Ottawa Carleton.

Gore & Storrie Limited, 1986. Report on Stormwater Management for the North West Belleville Area. Prepared for City of Belleville.

Gore & Storrie Limited, 1990. Update Report on Stormwater Management for Northwest the Belleville Area - North No Name Creek. Prepared for City of Belleville.

Gore & Storrie Limited, 1991. Rideau River Stormwater Management Study. Prepared for Regional Municipality of Ottawa-Carleton.

Gore & Storrie Limited, 1992. South Urban Community Master Drainage Plan Study. Prepared for City of Gloucester.

Gore & Storrie Limited, 1994. Pollution Control Scoping Study for Riverside Park Beaches. Prepared for Moira River Conservation Authority.

Hydrologic Engineering Center, 1991. HEC-2 Water Surface Profiles - User's Manual (September 1990, Revised September 1991). U.S. Army Corps of Engineers, Davis, CA.

Lorant, F.I., 1982. Vulnerability of Natural Watercourses to Erosion by Different Flowrates. Prepared for Ontario Ministry of Natural Resources by M.M. Dillon Limited.

Ministry of Environment and Energy, 1994. Stormwater Management Practices Planning and Design Manual. Prepared by Marshall Macklin Monaghan Limited.

Ministry of Environment and Energy, 1994b. Water Management: Policies, Guidelines, Provincial Water Quality Objectives of the Ministry of Environment and Energy.

Ministry of Environment and Energy, Ministry of Natural Resources, Quinte Conservation Authorities, 1993. Storm Water Management Design Criteria/ Guidelines - Bay of Quinte Storm Water Management implemenation Area. Prepared by Inter Agency Storm Water Management Working Committee (January 1993, Amended May 1993).

Ministry of Natural Resources, 1982. Belleville Floodplain Mapping Study. Prepared by W. Herold, J. MacDonald, N. Chin.

Ministry of Natural Resources, 1994. Fish Habitat Protection Guidelines for Developing Areas.

Ministry of Transportation and Communications, 1982. MTC Drainage Manual. Highway Design Office, MTC, Downsview, Ontario.

Olson, R.M. and S.J. Wright, 1990. *Essentials of Engineering Fluid Mechanics*. Harper & Row, New York, NY.

Phillips, D.W., 1976. Monthly Water Balance Tabulations for Climatological Stations in Canada. Atmospheric Environment Service, DS#4-76.

Pitt, R., 1983. Urban Bacteria Sources and Control by Street Cleaning in the Lower Rideau River Watershed, Ottawa, Ontario. Rideau River Stormwater Management Study.

Ponce, V.M., 1989. *Engineering Hydrology Principles and Practices*. Prentice Hall, Englewood Cliffs, N.J.

Pullin, B.P., and Hammer, D.A., 1991. Aquatic Plants Improve Wastewater Treatment, from Water Pollution Control Federation, Water Environment & Technology, Vol. 3, No. 3.

Regional Municipality of Ottawa-Carleton, 1983. Report on the Rideau River Stormwater Management Study, Phase II/III, prepared by Gore & Storrie Limited/Proctor & Redfern Limited.

Roads and Transportation Association of Canada, 1982. Drainage Manual, Volume 1.

Rowney, A.C. and C.R. MacRae, 1991a. QUALHYMO User Manual - Release 2.1. Royal Military College of Canada, Kingston, Ontario.

Rowney, A.C. and C.R. MacRae, 1991b. QUALHYMO Technical Reference Manual - Release 2.1. Royal Military College of Canada, Kingston, Ontario.

Rowney, A.C. and P.E. Wisner, 1983. An Approach to Analysis of Stormwater Detention Pond Systems for Water Quality Control. Report to the Ontario Ministry of the Environment. Department of Civil Engineering, University of Ottawa, Ottawa, Ontario.

Schueler, T.R., 1987. Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs. Department of Environmental Programs, Metropolitan Washington Council of Governments.

Taylor, M.E., 1992. Constructed Wetlands for Stormwater Treatment. Prepared for the Ont. Ministry of the Environment and the Metro Toronto & Region Conservation Authority.

Township of Thurlow, Ainley and Associates Limited, 1993. Section 6 to the Township of Thurlow Official Plan - Cannifton Secondary Plan.

Van Meer Limited, 1994. Conceptual Development Plan - Part of Lots 1 & 2, Concession 3, Township of Thurlow, County of Hastings.

Van Meer Limited, 1995. 1:5,000 scale map - "Pre-Development Drainage Area Plan - Nor-Belle Developments Inc. Subwatershed Area", drawing number 485DPRE.

Watt, W.E., K.W. Lathem, C.R. Neill, T.L. Richards and J. Rousselle (eds.), 1989. *Hydrology of Floods in Canada: A Guide to Planning and Design*. National Research Council of Canada.

APPENDIX A

WATER QUALITY DATA

Table A-1 – 1993 Water Quality Data - Upper No Name Creek

Date	Location	Parameter												
		NH_3 (as N)	NO_2 (as N)	NO_3 (as N)	TKN (as N)	TP	Turb. (FTU)	Cl	BOD_5	SS	DRP	FC (#/dl)	FS (#/dl)	EC (#/dl)
April 4, 1993	Tracey Street 401	.02 .02	.008 .004	.25 .10	.20 .21	.012 .014	1.7 2	115.4 35.6	< 1 1	6 3	< .002 < .002	< 10 < 10	10 < 10	< 10 < 10
May 3, 1993	Tracey Street 401	< .01 .01	.004 .003	.02 < .02	.21 .38	.011 .024	5.1 2	211.1 57.1	1 < 1	< 1 < 1	.002 .010	< 10 < 10	10 270	< 10 30
June 7, 1993	Tracey Street 401	< .01 .01	.002 .003	.02 < .02	.40 .74	.016 .077	0.79 1.4	182.1 66.5	< 1 < 1	< 1 28	.002 .010	100 60	170 10	120 260
July 5, 1993	Tracey Street 401	< .01 < .01	.006 .003	.04 < .02	.47 .83	.016 .072	1.2 2.2	246.7 80.4	1 5	4 12	.002 .012	610 1160	110 190	580 1390
August 4, 1993	Tracey Street 401	.20 No	.360 Flow	.74 --	3.7 --	.58 --	1.2 --	48 --	5 --	2 --	--	13,000 --	Est. 17,000 --	14,000 --
September 2, 1993	Tracey Street 401	No No	Flow Flow	-- --	-- --	-- --	-- --	-- --	-- --	-- --	-- --	-- --	-- --	-- --
October 5, 1993	Tracey Street 401	.04 .01	.01 .008	.05 .03	.29 .40	.013 .010	.90 2.6	144.1 105.7	< 1 2	4 4		70 60	290 < 10	90 10

Note: All units in mg/l except as noted.

Table A-2 – 1994 Water Quality Data - Dry Weather

Date	Location	Lab Analysis												
		NH_3 (as N)	NO_2 (as N)	NO_3 (as N)	TKN (as N)	TP	DRP (as P)	Turb. (FTU)	Cond. (umhos/cm)	TOC	Cl	EC (#/dl)	FS (#/dl)	PA (#/dl)
June 6, 1994	Bell Blvd.	.01	.008	.27	.35	.010	.002	0.60	1265	4.8	233.4	50	70	< 10
	Tracey Street	< .01	.002	< .02	.29	.012	.002	1.2	1287	6.0	238.8	120	320	< 10
	401	< .01	.004	< .02	.45	.034	.010	1.4	635	6.2	82.6	10	240	< 10

Date	Location	Field Observations				
		D.O.	Temp. (°C)	TDS (g/l)	Cond. (ms/cm)	pH
June 6, 1994	Bell Blvd.	5	15.2	0.64	1.26	7.34
	Tracey Street	9.2	14.1	0.65	1.28	7.50
	401	4.6	17.3	0.31	0.63	7.47

Date	Location	Field Observations			
		D.O.	Temp. (°C)	Cond. (ms/cm)	pH
July 19, 1994	Bell Blvd.	8.2	23.0	1.58	7.06
	401	12.6	27.6	0.77	6.9

Date	Location	Lab Analysis				
		SS	TP	TOC	EC	FS
August 22, 1994	Bell Blvd.	< 1	.013	6.0	40	40
	401	40	.64		320	80

Note: All units in mg/l unless otherwise noted.

Table A-3 -- 1994 Water Quality Data - Wet Weather

Location	Time	Lab Analysis						Field Observations			
		TOC	SS	TP	EC (#/dl)	FS (#/dl)	PA (#/dl)	pH	DO	Temp. (°C)	Cond. (ms/cm)
Bell Blvd.	10:00		70	.240	3,800	> 15,000	400	7.8	13.0	19.9	0.32
	10:30		60	.240	5,700	> 15,000	100	7.04	7.0	20	0.27
	11:00		72	.440	6,600	> 15,000	700	7.37	6.7	19.7	0.38
	11:30		24	.160	5,700	> 15,000	300	6.73	6.5	19.2	0.39
Tracey Street	9:45		41	.200	3,500	> 15,000	1,900	7.2	--	19.4	0.72
	10:15		33	.200	7,400	> 15,000	< 10	7.2	--	19.6	0.52
	10:45	49.0	30	.190	6,800	> 15,000	5,300	6.9	6.4	19.5	0.43
	11:15	44.6	34	.160	6,300	> 15,000	2,200	7.1	5.8	19.5	0.42
	11:45	38.4	30	.200	6,600	> 15,000	1,200	7.01	5.6	19.4	0.37
401	9:35	7.2	7	.050	1,400	1,230	< 10			17	
	9:50	7.4	7	.050	1,600	1,900	< 10			17	
	10:05		6	.050	2,100	1,260	< 10			17	
	10:20		6	.045	2,500	3,000	< 10			16.8	
	10:35	7.2	5	.036	4,000	4,400	< 10			16.8	
	10:50	7.6	4	.032	4,100	3,700	< 10			16.8	
	11:05	7.4	5	.035	4,000	4,300	< 10			17	
	11:20	7.4	7	.030	3,200	2,700	< 10			17.5	
	11:35	7.8	5	.030	3,100	2,900	< 10			17.5	
	11:50	6.8	6	.035	2,400	2,000	< 10			17.5	

Note:
- -- Wet Weather even sampled June 24/94
- -- 2.6 mm rainfall recorded prior to sampling
- -- no flow measurements available

APPENDIX B

VEGETATION AND WILDLIFE LISTS

PLANTS AND WILDLIFE OBSERVED

TABLE B-1: PLANTS OBSERVED

	Scientific Name	Common Name
	EQUISETACEAE	**HORSETAIL FAMILY**
U	Equisetum fluviatile	Water Horsetail
U	Equisetum variegatum	Variegated Horsetail
	POLYPODIACEAE	**FERN FAMILY**
	Onoclea sensibilis	Sensitive Fern
	PINACEAE	**PINE FAMILY**
	Abies balsamea	Balsam Fir
	Juniperus communis	Common Juniper
	Juniperus virginiana	Red Cedar
	Picea glauca	White Spruce
	Pinus strobus	White Pine
	Thuja occidentalis	White Cedar
	TYPHACEAE	**CATTAIL FAMILY**
	Typha latifolia	Common Cattail
	ALISMATACEAE	**WATER-PLANTAIN FAMILY**
	Alisma plantago-aquatica	Water Plantain
	POACEAE	**GRASS FAMILY**
+	Agropyron repens	Witch Grass
+	Agrostis gigantea	Redtop
+	Agrostis stolonifera	Creeping Bent-Grass
+	Bromus inermis	Awnless Brome
+	Dactylis glomerata	Orchard Grass
	Elymus virginicus	Virginia Wild-Rye
+	Glyceria maxima	English Water Grass
	Glyceria striata	Fowl Meadow-Grass
	Leersia oryzoides	Rice Cut-Grass
	Muhlenbergia mexicana	Muhly Grass
	Panicum sp.	Panic Grass
	Panicum capillare	Old Witch Grass
	Phalaris arundinacea	Reed Canary-Grass

TABLE B-1: PLANTS OBSERVED		
+	Phleum pratense	Timothy
	Poa compressa	Canada Bluegrass
	Poa palustris	Fowl Meadow-Grass
+	Poa pratensis	Kentucky Bluegrass
	CYPERACEAE	**SEDGE FAMILY**
	Carex bebbii	Bebb's Sedge
	Carex retrorsa	Retrorse Sedge
	Carex vulpinoidea	Fox Sedge
	Eleocharis erythropoda	Red-based Spike-Rush
	Scirpus acutus	Hard-stemmed Bulrush
	Scirpus atrovirens	Dark Green Bulrush
	LEMNACEAE	**DUCKWEED FAMILY**
	Lemna minor	Lesser Duckweed
	JUNCACEAE	**RUSH FAMILY**
	Juncus dudleyi	Dudley's Rush
	ORCHIDACEAE	**ORCHIS FAMILY**
+	Epipactis helleborine	Helleborine
	SALICACEAE	**WILLOW FAMILY**
	Populus deltoides	Cottonwood
	Populus grandidentata	Large-tooth Aspen
	Populus tremuloides	Trembling Aspen
	Salix discolor	Pussy Willow
	Salix exigua	Sandbar Willow
	JUGLANDACEAE	**WALNUT FAMILY**
	Carya ovata	Shagbark Hickory
	BETULACEAE	**BIRCH FAMILY**
	Betula papyrifera	White Birch
	Carpinus caroliniana	Blue Beech
	FAGACEAE	**BEECH FAMILY**
	Quercus alba	White Oak
	Quercus macrocarpa	Bur Oak
	Quercus rubra	Red Oak

TABLE B-1: PLANTS OBSERVED

	ULMACEAE	**ELM FAMILY**
	Ulmus americana	White Elm
+	Ulmus pumila	Siberian Elm
	URTICACEAE	**NETTLE FAMILY**
	Urtica dioica var. procera	Stinging Nettle
	POLYGONACEAE	**BUCKWHEAT FAMILY**
+	Rumex crispus	Curled Dock
	RANUNCULACEAE	**CROWFOOT FAMILY**
	Anemone virginiana	Thimbleweed
	Clematis virginiana	Virgin's-Bower
	Ranunculus hispidus	Swamp Buttercup
	BRASSICACEAE	**MUSTARD FAMILY**
+	Alliaria petiolata	Garlic Mustard
+	Barbarea vulgaris	Yellow Rocket
+	Hesperis matronalis	Dame's Rocket
+	Lepidium campestre	Cow Cress
+	Nasturtium officinale	Watercress
+	Rorippa sylvestris	Creeping Yellow Cress
	SAXIFRAGACEAE	**SAXIFRAGE FAMILY**
U	Penthorum sedoides	Ditch Stonecrop
	Ribes triste	Swamp Red Currant
	ROSACEAE	**ROSE FAMILY**
	Agrimonia gryposepala	Tall Hairy Agrimony
	Fragaria virginiana	Common Strawberry
	Geum canadense	White Avens
+	Potentilla recta	Rough-fruited Cinquefoil
	Rosa blanda	Smooth Rose
	Rubus idaeus	Red Raspberry
	Rubus occidentalis	Black Raspberry

TABLE B-1: PLANTS OBSERVED

	FABACEAE	**PEA FAMILY**
+	Medicago lupulina	Black Medick
+	Melilotus alba	White Sweet Clover
+	Trifolium pratense	Red Clover
+	Trifolium repens	White Clover
+	Vicia cracca	Cow Vetch
	RUTACEAE	**RUE FAMILY**
	Zanthoxylum americanum	Northern Prickly-Ash
	ANACARDIACEAE	**CASHEW FAMILY**
	Rhus typhina	Staghorn Sumac
	ACERACEAE	**MAPLE FAMILY**
	Acer negundo	Manitoba Maple
	Acer rubrum	Red Maple
	Acer saccharinum	Silver Maple
	RHAMNACEAE	**BUCKTHORN FAMILY**
+	Rhamnus cathartica	Common Buckthorn
+	Rhamnus frangula	Glossy Buckthorn
	VITACEAE	**VINE FAMILY**
	Vitis riparia	Riverbank Grape
	CLUSIACEAE	**ST. JOHN'S-WORT FAMILY**
+	Hypericum perforatum	Common St. John's-Wort
	LYTHRACEAE	**LOOSESTRIFE FAMILY**
+	Lythrum salicaria	Purple Loosestrife
	ONAGRACEAE	**EVENING-PRIMROSE FAMILY**
	Epilobium coloratum	Purple Willow-Herb
	APIACEAE	**PARSLEY FAMILY**
+	Daucus carota	Wild Carrot
	CORNACEAE	**DOGWOOD FAMILY**
	Cornus foemina ssp. racemosa	Gray Dogwood
	Cornus stolonifera	Red-osier Dogwood
	OLEACEAE	**OLIVE FAMILY**
	Fraxinus americana	White Ash

TABLE B-1: PLANTS OBSERVED		
	Fraxinus pennsylvanica	Red Ash
	APOCYNACEAE	**DOGBANE FAMILY**
	Apocynum androsaemifolium	Spreading Dogbane
	ASCLEPIADACEAE	**MILKWEED FAMILY**
	Asclepias incarnata	Swamp Milkweed
	Asclepias syriaca	Common Milkweed
	BORAGINACEAE	**BORAGE FAMILY**
+	Echium vulgare	Viper's-Bugloss
+	Lithospermum officinale	Common Gromwell
	LAMIACEAE	**MINT FAMILY**
+	Leonurus cardiaca	Motherwort
+	Lycopus europaeus	European Water-Horehound
	Lycopus uniflorus	Bugleweed
	Mentha arvensis	Field Mint
+	Nepeta cataria	Catnip
+-	Prunella vulgaris	Heal-All
	SOLANACEAE	**NIGHTSHADE FAMILY**
+	Solanum dulcamara	Nightshade
	SCROPHULARIACEAE	**FIGWORT FAMILY**
+	Verbascum thapsus	Common Mullein
	PLANTAGINACEAE	**PLANTAIN FAMILY**
+	Plantago major	Common Plantain
	RUBIACEAE	**MADDER FAMILY**
	Galium asprellum	Rough Bedstraw
	CAPRIFOLIACEAE	**HONEYSUCKLE FAMILY**
+	Lonicera tatarica	Tartarian Honeysuckle
	Viburnum trilobum	Highbush Cranberry
	DIPSACACEAE	**TEASEL FAMILY**
+	Dipsacus fullonum	Teasel
	ASTERACEAE	**ASTER FAMILY**
+-	Achillea millefolium	Yarrow
+	Arctium minus	Common Burdock

TABLE B-1: PLANTS OBSERVED

+	Artemisia absinthium	Absinthe
	Aster cordifolius	Heart-leaf Aster
U	Aster ericoides	Heath Aster
	Aster lanceolatus	Panicled Aster
	Aster macrophyllus	Large-leaved Aster
	Aster novae-angliae	New England Aster
U	Aster pilosus	Pringle's Aster
	Aster puniceus	Purple-stemmed Aster
	Bidens frondosa	Beggar's-Ticks
+	Carduus nutans	Nodding Thistle
+	Chrysanthemum leucanthemum	Ox-eye Daisy
+	Cichorium intybus	Chicory
+	Cirsium arvense	Canada Thistle
	Cirsium muticum	Swamp Thistle
+	Cirsium vulgare	Bull Thistle
+	Inula helenium	Elecampane
	Solidago canadensis	Canada Goldenrod
	Solidago graminifolia	Lance-leaved Goldenrod
	Solidago nemoralis	Gray Goldenrod
+	Sonchus oleraceus	Annual Sow-Thistle
	Xanthium strumarium	Cocklebur
	Total Number of Species	132
	Number of Native Species	85
	Percent Native Species	64
	Number of Introduced Species	47
	Percent Introduced Species	36

Legend: U = Uncommon
\+ = Introduced

TABLE B-2: WILDLIFE OBSERVED

	Common Name	Scientific Name
	AMPHIBIANS	
	Northern Leopard Frog	Rana pipiens
	BIRDS	
	Red-tailed Hawk	Buteo jamaicensis
	Mourning Dove	Zenaida macroura
	Blue Jay	Cyanocitta cristata
	American Crow	Corvus brachyrhynchos
	Black-capped Chickadee	Parus atricapillus
	Golden-crowned Kinglet	Regulus satrapa
	Dark-eyed Junco	Junco hyemalis
	American Goldfinch	Carduelis tristis
	MAMMALS	
	Snowshoe Hare	Lepus americanus
	Woodchuck	Marmota monax
	Grey Squirrel	Sciurus carolinensis
	Beaver	Castor canadensis
	Muskrat	Ondatra zibethicus
	Meadow Vole	Microtus pennsylvanicus
	Coyote	Canis latrans
	White-tailed Deer	Odocoileus virginianus

APPENDIX C

RAINFALL ANALYSIS

RAINFALL ANALYSIS

Two forms of precipitation data have been used in the analysis. To estimate peak flows and establish the floodlines for existing land use conditions, a design event was used. For stormwater treatment simulations, a continuous rainfall record was employed.

C.1 DESIGN STORMS

A design event is a single rainfall event which is generally statistically related with a return period. Both 5-year and 100-year design storms were considered in the analysis.

Three well-accepted design storm distributions were modelled, with several different storm durations being considered:

- the Environment Canada - Atmospheric Environmental Service (AES) Type II 30 percentile distribution
  - 1 and 12 hour durations
- the HYDROTEK distribution
  - 1 hour duration, and
- the Soil Conservation Service (SCS) Type II distribution
  - 6, 12, and 24 hour durations

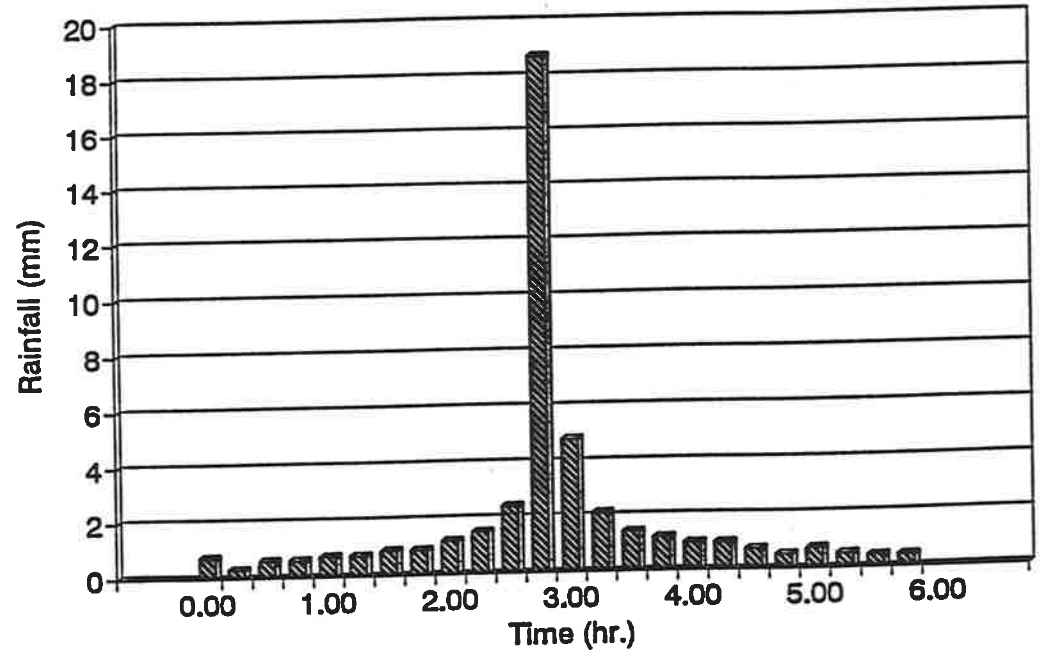
The AES Type II 30 percentile distribution is a synthetic distribution that is based on regional average values of time-to-peak intensity, peak/total depth, and rainfall amount prior to the peak, and is available for most regions in Canada, including Southern Ontario (Watt et al, 1989). The 30 percentile probability distribution is recommended for Ontario. The two component synthetic HYDROTEK distribution incorporates exponential rise and linear decay for late peaking storms such as those that occur in the Quinte region. One-hour urban design events have been developed for most of Canada, with regional parameters having been derived for most of the country, making it a convenient distribution for use in this study. The synthetic SCS Type II distribution was considered since it is applicable to the dominant thunderstorm events that develop in the Quinte region (Ponce, 1989).

The watershed's response to each design storm was modelled using QUALHYMO (details provided in Appendix D) to determine which one produced the highest flows both upstream of the existing beaver ponds north of Highway 401 and directly upstream of the Tracey Street storm sewer inlet. A summary of these preliminary peak flows is provided in Table C-1. Since the SCS Type II 6-hour distribution produced the highest peak flows (in bold) in three out of four possible cases, it was chosen as the critical distribution from which peak flows and corresponding floodlines would be determined.

Hyetographs of the 5-year and 100-year SCS Type II 6-hour distribution are provided in Figure C-1. Total rainfall for the 5-year and 100-year storms were 43.7 mm and

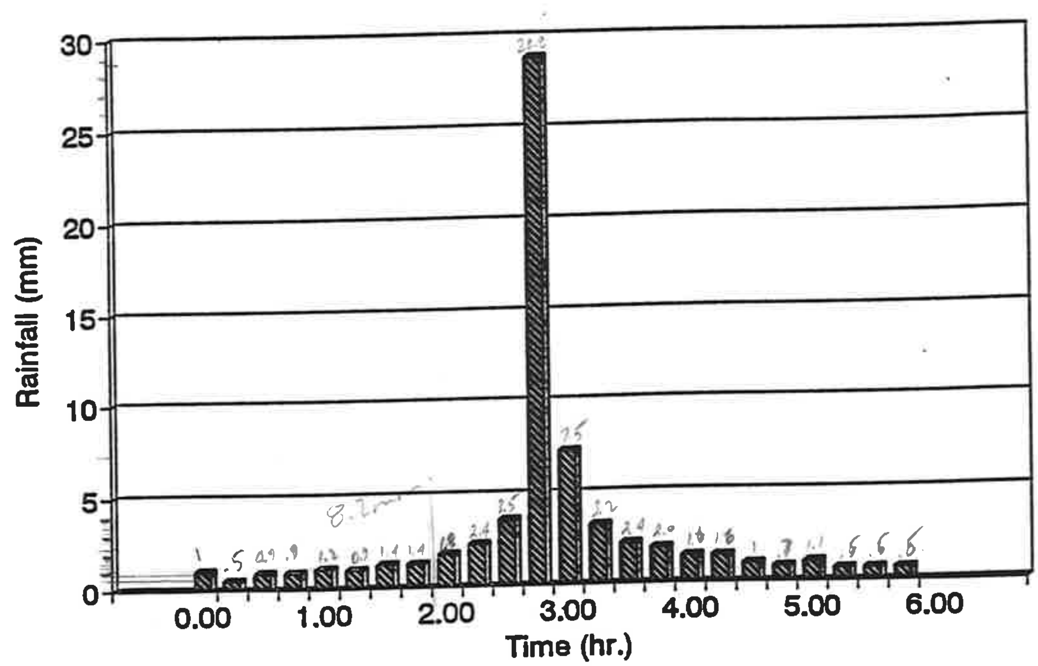
SCS Type 2 HYETOGRAPH

5-year 6-hour distribution



SCS Type 2 HYETOGRAPH

100-year 6-hour distribution



MOIRA RIVER CONSERVATION AUTHORITY	**HYETOGRAPHS FOR CRITICAL DESIGN STORMS**	**Gore & Storrie Limited** G&S Consulting Engineers FILE Nº \7644\DWGS\FIGC-1 — FIG Nº C-1

67.3 mm, respectively. These values were provided in the 1990 AES rainfall intensity-duration frequency curves for Atmospheric Environment Service station 6150689 (Belleville Airport).

Table C-1
SUMMARY OF PEAK FLOWS GENERATED FROM DESIGN STORMS

DESIGN STORM	PEAK FLOW U/S OF BEAVER POND @ 401 (m^3/s)		PEAK FLOW U/S OF TRACEY ST. SS INLET (m^3/s)	
	5-YEAR	100-YEAR	5-YEAR	100-YEAR
AES 12 hr.	0.969	2.135	1.095	2.082
AES 1 hr.	0.711	2.031	1.972	3.540
HYDROTEK 1 hr.	0.766	2.155	2.205	3.896
SCS 6 hr.*	1.485	**3.573**	**2.681**	**4.672**
SCS 12 hr.	1.514	**3.573**	2.365	4.097
SCS 24 hr.	**1.535**	3.446	1.924	3.283

* distribution chosen for event modelling

C.2 CONTINUOUS RAINFALL RECORD

A continuous hourly precipitation record is required for estimating annual or seasonal pollutant loads and examining the performance of stormwater treatment options, using the QUALHYMO simulation package.

Hourly rainfall data for the period from April 1 to October 31 is available from the Atmospheric Environmental Service (AES) branch of Environment Canada. Precipitation for the remainder of the year is also available from AES but in a six hour duration or 24 hour duration format depending on the climatological station.

For this study, the rainfall data from the Belleville, Trenton, and Picton stations were obtained. This provided a continuous rainfall record for the Quinte region covering from 1965 to 1992. Where possible, rainfall data from the Belleville station were used. In cases of missing data at the Belleville station, the rainfall file was filled in with data from Trenton (or Picton if both Belleville and Trenton data were missing). In this way, a complete data set for the Quinte region was obtained.

Since water quality during the recreational season (from about May 15 to September 15) is the main concern, input rainfall files were prepared only for May 1 to October 31 only. However, modelling of pollutants was performed using rainfall data over an "effective" recreational season from June 1 to September 30, for simplicity. This minor discrepancy in seasonal limits (about two weeks) can be expected to have negligible affect on the results of the analysis.

To reduce the computer simulation running time and memory requirements, it is preferable to base the analysis on a number of representative years instead of simulating the entire rainfall record. An effective method prescribed for determining six representative years was developed by G&S for the Rideau River Stormwater Management Study (RRSMS - G&S, 1991). This method is described in the following paragraphs.

A computer program (AESSCAN) was developed by G&S that would scan the AES hourly rainfall data file and calculate the following statistics for each event:

- Start date and time
- End date and time
- Duration
- Hours during which rain occurred
- Total rainfall (mm)
- Maximum intensity (mm/h)
- Average intensity (mm/h)

An event was defined by specifying the number of hours with no rainfall between events. Event statistics were generated for the 6, 12, 24, 48, and 72 hour inter-event dry periods.

A second computer program (YEARSTAT) was written by G&S that would calculate the rainfall event magnitude distribution for each year between June 1 to September 30, and compute the following data for the recreational season:

- Total number of events in each of the following ranges:
    - 0-5 mm
    - 5-10 mm
    - 10-20 mm
    - 20-30 mm
    - 30-40 mm
    - 40-50 mm
    - 50-75 mm
    - > 75 mm

Results from YEARSTAT for the Quinte dataset for the 6 hour inter-event period are included in Table C-2. YEARSTAT was used to generate the rain event magnitude distributions for all five inter-event periods above, from 1965 to 1992 inclusive.

Selection of representative years is based on first selecting the "average" year. Therefore, it was necessary to determine the total summer rainfall for each year. These are shown in Table C-3 for all years.

Table C-2: SUMMER EVENT DISTRIBUTION FOR 6-HR. INTER-EVENT TIME

		RAINFALL EVENT DISTRIBUTION by depth in mm							
YEAR	# EVENT	0-5	5-10	10-20	20-30	30-40	40-50	50-75	>75
1965	47	29	9	6	2	1	0	0	0
1966	33	18	7	4	3	0	1	0	0
1967	36	19	8	6	2	0	0	1	0
1968	34	19	6	6	2	0	1	0	0
1969	37	21	5	6	4	1	0	0	0
1970	41	29	7	2	3	0	0	0	0
1971	32	22	4	5	0	1	0	0	0
1972	43	23	11	4	2	2	1	0	0
1973	31	19	3	6	2	0	0	1	0
1974	41	30	5	4	2	0	0	0	0
1975	37	19	7	8	2	1	0	0	0
1976	43	29	4	8	2	0	0	0	0
1977	41	23	7	7	3	0	0	1	0
1978	39	23	9	5	2	0	0	0	0
1979	37	30	4	2	0	0	0	1	0
1980	45	26	10	3	1	2	3	0	0
1981	46	31	2	7	3	0	2	1	0
1982	35	16	9	6	3	0	1	0	0
1983	31	19	4	6	1	1	0	0	0
1984	38	23	8	3	2	1	0	1	0
1985	35	21	6	5	1	2	0	0	0
1986	47	27	7	6	3	3	0	0	1
1987	41	23	8	8	0	2	0	0	0
1988	42	28	11	2	1	0	0	0	0
1989	24	10	4	6	4	0	0	0	0
1990	38	24	6	6	0	2	0	0	0
1991	42	31	5	5	0	1	0	0	0
1992	46	29	7	5	4	0	0	1	0
MEAN	39	24	7	5	2	1	0	0	0

Table C-3
SUMMER RAINFALL TOTALS
(June 1 - September 30)

YEAR	RAINFALL (mm)
1965	272.9
1966	255.6
1967	300.1
1968	260.5
1969	275.7
1970	221.6
1971	190.9
1972	361.2
1973	248.8
1974	197.0
1975	275.6
1976	236.9
1977	349.1
1978	217.0
1979	157.6
1980	383.2
1981	377.3
1982	285.7
1983	214.4
1984	283.6
1985	236.5
1986	442.3
1987	284.8
1988	167.0
1989	231.9
1990	248.3
1991	209.1
1992	315.6
AVERAGE	**267.8**

The average rainfall summer was then selected by finding the year which most closely met the following three criteria:

- Total rainfall closest (within 10 mm) to average rainfall (267.8 mm)
- Event magnitude distribution close to average for all five inter-event distributions
- Monthly totals as close to monthly averages as possible

From analysis of the years 1965 to 1992, the arithmetic mean (μ) and standard deviation (σ) of summer total rainfall were found to be:

$\mu = 267.8$ mm $\qquad \sigma = 67.72$ mm

Based on the arithmetic mean, six years of various magnitudes (within about 10 mm) were selected as follows:

Table C-4
POSSIBLE AVERAGE YEARS

YEAR	RAINFALL (mm)
1965	272.9
1966	255.6
1968	260.5
1969	275.7
1975	275.6
AVERAGE	**267.8**

To select the average years (two are recommended) from this group, the last two criteria listed above were applied. Statistical calculations were applied to the event magnitude distributions to define the best years. A score was calculated for each year for each inter-event time as the sum of the squares of the residuals between the mean and actual number of events in each total precipitation range. A lower score indicated that a year is closer to having an average distribution.

A total score for each year was calculated by summing the scores for each inter-event period, with each period being equally weighted in the analysis. The results indicated that the average year was 1968, with 1969 coming in a very close second.

Next, intermediate to extreme wet and dry years were determined. This selection was based on the standard deviation of summer total precipitation from the long-term mean value using the following definitions:

Extreme wet year = $(\mu + 2.0\sigma)$ mm (= 403.2 mm)
Intermediate wet year = $(\mu + 1.5\sigma)$ mm (= 369.4 mm)
Intermediate dry year = $(\mu - 1.5\sigma)$ mm (= 166.2 mm)
Extreme dry year = $(\mu - 2.0\sigma)$ mm (= 132.4 mm)

Once representative years were determined, their monthly precipitation values were checked to ensure that rainfall was not unusually distributed during the summer months.

Based on all criteria, the key years were then determined. Table A-5 summarizes the key years used.

From the RRSMS (G&S, 1991), it was shown that use of these six years would provide results representative of the complete period of record. Differences from results obtained with the complete rainfall database (all 28 years) would likely be less than 6% for removal and exceedances parameters. Thus, this six key year technique has been demonstrated as an excellent method of decreasing model running and analysis time as well as computer memory requirements, while providing accurate results.

All QUALHYMO continuous simulations performed in the present study used the following six years on a representative continuous rainfall series basis.

Table C-5
SUMMARY OF KEY YEARS

YEAR DESCRIPTION	YEAR SYMBOL	YEAR	TOTAL SUMMER RAINFALL (mm)
Extreme Wet	W	1980	383.2
Intermediate Wet	WI	1972	361.2
Average #1	A1	1968	260.5
Average #2	A2	1969	275.7
Intermediate Dry	DI	1988	167.0
Extreme Dry	D	1979	157.6

APPENDIX D

HYDROLOGIC ANALYSIS

HYDROLOGIC ANALYSIS

D.1 QUALHYMO MODEL SETUP - EXISTING CONDITIONS

The QUALHYMO hydrologic model (G&S Version 2.11-D) was selected for use in the Upper No Name Creek SWM study because of its wide range of capabilities. QUALHYMO is a continuous water quantity/quality simulation model designed for lumped application in urbanizing Ontario basins. The model was originally developed by Rowney and Wisner (1983), and has been modified and expanded significantly since that time. Several major modifications have been made by G&S in recent years, including adding the capability to run the model on either an event or continuous basis. The model is described in detail in Rowney and MacRae (1991a, 1991b).

QUALHYMO is distinct from its HYMO and OTTHYMO/INTERHYMO precursors in its ability to simulate the generation and routing of pollutants, soil freeze-thaw and in-stream erosion potential, and in its orientation towards continuous simulation. The pollutant simulation ability of QUALHYMO is one of the reasons for its use in the present study, as well as its continuous hydrologic simulation capability. The model is also capable of routing flows and pollutants through stormwater detention ponds of various configurations.

The watershed was discretized into subcatchments as shown in Figure 2-1. Discretization was based on recent 1:2,000 scale topographic mapping, the previous model setup by G&S (1986), as well as recent information received from Van Meer Limited regarding Subcatchments 100 and 101. Drainage areas of each subcatchment are shown on the figure. Additional detail in topography north of Highway 401 resulted in a significantly larger contributing drainage area than the earlier studies (G&S 1986, 1990, Falcone Smith, 1990). Drainage areas south of Highway 401 are very similar.

Additional areas west of Sidney Street (sub-basins 100B and 100C), and east of Sidney Street in the vicinity of Sunningdale Drive (sub-basin 100A), were added to the model, based on aerial photographs taken on 1993 March 31, and field observations in January 1995, during significant rainfall/snowmelt events. Runoff from sub-basin 100A flows westerly and is conveyed via a culvert under Sidney Street into sub-basin 100B. It was determined that runoff from the two subcatchments 100B and 100C is then conveyed easterly through five culverts under Sidney Street into the ditch running west to east located approximately 500 m north of Maitland Drive. This runoff may overflow the ditch and enter Upper No Name Creek during spring runoff or during large rainfall events. The total additional drainage area was estimated to be 328.2 ha, based on 1:25,000 EMR mapping (1971), and the January 1995 information from Van Meer Limited. Additionally, it was determined that runoff from Subcatchment 101B (17.0 ha) also may overflow into Upper No Name Creek during similar events.

Preferably, calibration of the model to flows measured during rainfall events in the study area can be conducted to refine model input parameters. Since no flow data were available for the Upper No Name Creek, model calibration could not be conducted. Input parameters were derived based on knowledge of the study area, and experience gained on calibrated watersheds throughout Ontario.

Times to peak were computed based on two formulae; the SCS Curve Number formula (RTAC, 1982) and the Watt and Chow formula (Watt et al, 1989). The Nash unit hydrograph approach was used for both impervious and pervious areas for all sub-basins. For impervious areas, 4 linear reservoirs were assumed, and for pervious, 2 linear reservoirs. For initial abstractions, 2.5 mm was used for impervious areas, 5 for pervious areas in urban areas, and 10-15 mm for pervious areas in rural areas. Since the watershed consists largely of clay and clay loam soils over pervious limestone bedrock, interspersed with pockets of low lying organic bottom soils (Crysler and Lathem, 1977), an SCS soil classification of 'C' was selected. This information, along with land use breakdowns was used to determine soil storage parameters for each subcatchment. Fractions of imperviousness for existing conditions are shown in Table D-1. These were determined from examination of recent aerial photographs.

Table D-1
FRACTIONS OF IMPERVIOUSNESS - EXISTING

Sub-basin	Percent Impervious
100A & 100B	3
100C	2
101A	6
101B	6
102	6
105	4
108	5
110	7
111	9
112	93
113	2
114	2
115	91
119	90

During an October 1994 field visit, the dimensions of the Sidney Street culvert leading directly to the drainage ditch were measured, and the ditch itself examined.

The culvert is a 0.60 m diameter corrugated steel pipe (CSP). To model flows conveyed through this single culvert in preliminary modelling runs, reservoir routing was simulated, using a 0.60 m pipe as an outlet structure. It was assumed that Sidney Street would not be overtopped during a 100-year storm. The reservoir simulated had a maximum depth of 1 m.

For the west-east drainage ditch, several cross-sections were obtained from survey information. The average flow capacity of the ditch was estimated using Manning's equation to be 0.14 m^3/s. Any flows less than this value were assumed to remain in the drainage ditch and not enter the Upper No Name Creek. Flows exceeding this value were added to the creek.

However, updated information provided to the MRCA by Van Meer Limited in late January 1995 demonstrated that the actual drainage area potentially emptying into this drainage ditch was actually about 357.3 ha. As discussed above, five culverts exist which carry the flow from west of Sidney Street under the roadway and into the ditch (four 0.76 m CSPs as well as the 0.60 m culvert previously modelled). Thus, for the final modelling runs, flows conveyed through these five culverts were modelled using two reservoir routing routines (one to simulate the 10 ha wetland in sub-basin 100C drained by one 0.76 m CSP and one 0.60 m CSP, the other to simulate the 20 ha wetland in sub-basin 100B drained by three 0.76 m CSPs). Again, it was assumed that Sidney Street would not be overtopped during a 100-year storm event, and that the average flow capacity of the ditch was 0.14 m_3/s. These modifications significantly increased the volume of runoff that was estimated to enter Upper No Name Creek, though peak flows simulated throughout the watershed only moderately increased due to the presence of the two large reservoirs west of Sidney Street in the updated models.

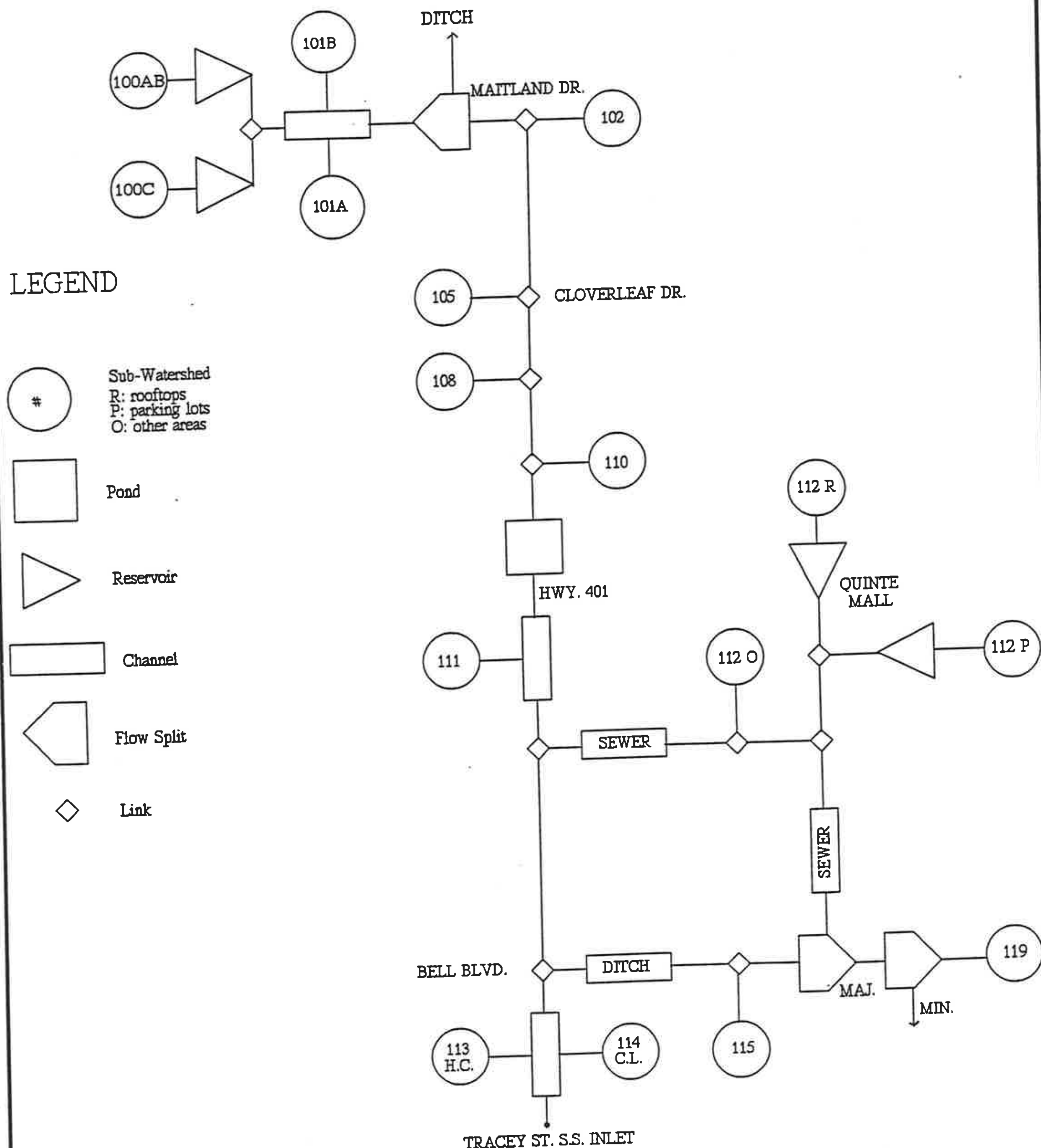
All reach information in the models was obtained from channel dimensions measured during the G&S field survey.

The outflow/stage/storage information for the existing storage pond at Highway 401 was obtained from the previous modelling by Falcone Smith (1990). In addition, the rooftop and parking lot storage information for the Quinte Mall was taken from Falcone Smith (1990). Flow splits into major/minor components from sub-basin 119 in the City of Belleville were calculated based on number of catchbasins and catchbasin inlet capacity. Minor flows from this sub-basin are removed from the Upper No Name Creek watershed.

The model schematic for existing land use conditions is shown in Figure D-1.

D.1.1 Event Modelling

The QUALHYMO model was run in a design event mode to generate peak flows at



MOIRA RIVER
CONSERVATION
AUTHORITY

QUALHYMO SCHEMATIC
EXISTING CONDITIONS

Gore & Storrie Limited GSS
Consulting Engineers

FILE Nº \7644\DWGS\FIGD-1	FIG Nº D-1

existing conditions for the 5 and 100-year return period storms. The synthetic storm used for the simulation was the SCS Type II, 6-hour duration storm, with 15 minute time steps. This was selected as the critical storm for the Upper No Name Creek watershed in terms of peak flows, as described in Appendix C.1. A 15 minute time step was also used in the model.

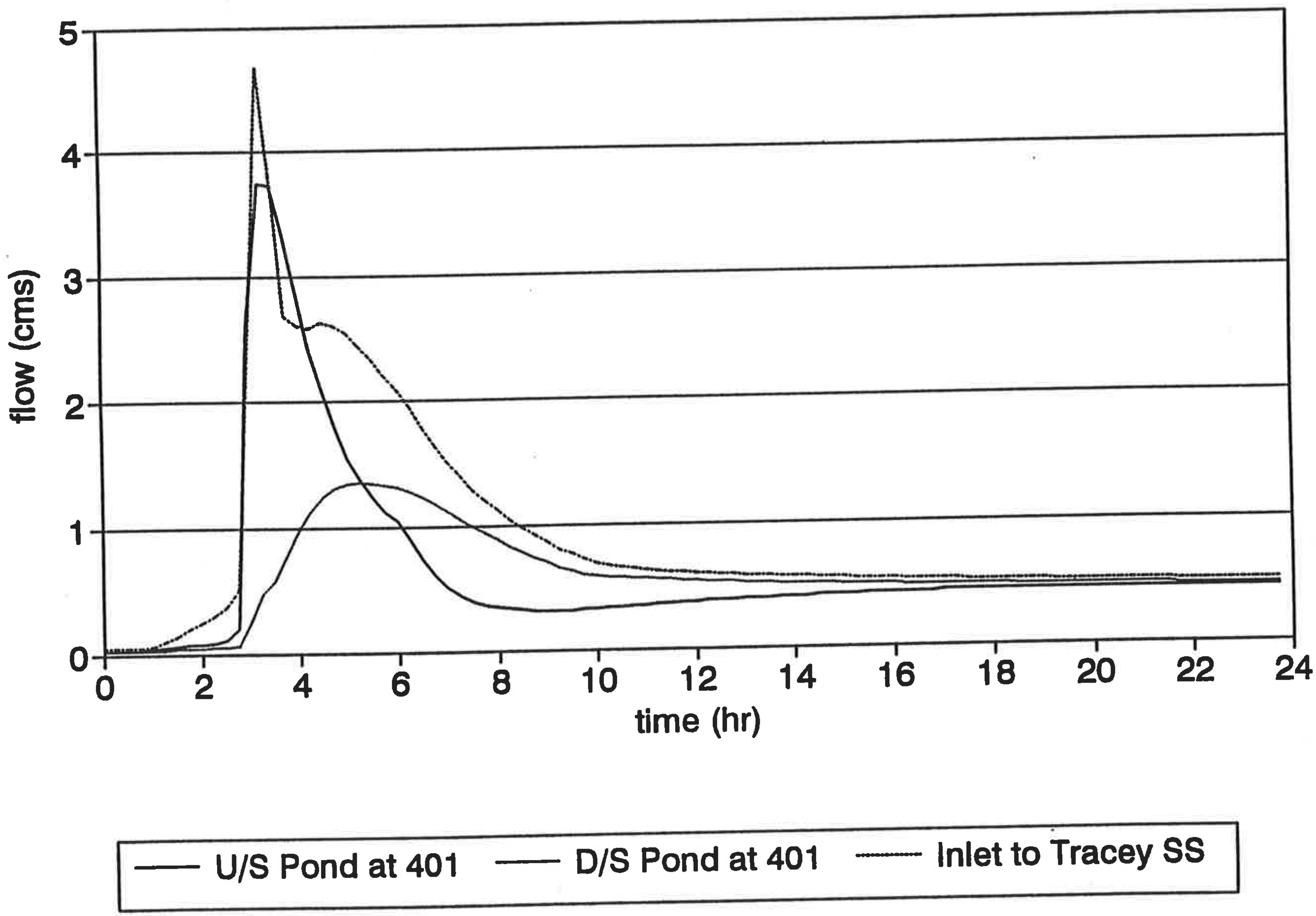
The peak flows generated by the model are shown in Table D-2. Results from previous studies are also shown for comparison. The flows simulated in the present study are somewhat higher than in previous studies. This can be attributed to the larger drainage area modelled in this study, as a result of updated topographic mapping, recent aerial photographs, and Van Meer Limited mapping (1995).

Table D-2
PEAK FLOWS – EXISTING

Location	5-Year		100-Year	
	Previous Study (G&S 1986, 1990)	Present Study	Previous Study	Present Study
U/S Maitland Dr.	N/A	0.71	N/A	1.78
U/S Cloverleaf Dr.	N/A	1.11	N/A	2.76
U/S Pond N. of 401	1.31	1.56	3.54	3.74
D/S Pond N. of 401	0.45	0.54	0.79	1.36
D/S Bell Blvd.	1.99	2.82	3.65	4.42
U/S Storage @ Tracey St.	1.92	N/A (no storage simulated)	4.07	N/A (no storage simulated)
Inlet to Tracey St. S.S.	1.92	2.68	3.65	4.68

The 100-year peak flow downstream of the existing pond at Highway 401 is estimated at 1.4 m^3/s, compared to 0.8 m^3/s from previous modelling. This is due to the higher flow volume entering the pond. The 1.4 m^3/s becomes the target peak flow for future conditions from north of Highway 401. The 100-year peak flow at Tracey Street is also increased, and exceeds the inlet capacity of the storm sewer of 3.5 m^3/s. Even at existing conditions, detention storage is required at Tracey Street to prevent sewer surcharging during the 100-year storm.

Hydrographs for the 100-year storm are shown in Figure D-2, at locations upstream and downstream of the pond at Highway 401, and at the inlet to the Tracey Street storm sewer. The hydrograph at the storm sewer inlet demonstrates the peakiness of the runoff from the portion of the watershed south of Highway 401.



MOIRA RIVER CONSERVATION AUTHORITY	**1:100 YEAR HYDROGRAPHS FOR EXISTING CONDITIONS**	**Gore & Storrie Limited G&S** Consulting Engineers
		FILE Nº \7644\DWGS\FIGD-2
		FIG Nº D-2

D.1.2 Continuous Simulations

As described in Appendix C.2, a six-year hourly rainfall record was produced for the Quinte region for the summer recreational period (June 1 to September 30). This rainfall record was selected to be representative of the long-term period of record. This rainfall file formed the basis of the continuous simulation with QUALHYMO. A one-hour time step was used for continuous simulations, to produce a six-year time series of hourly flows and pollutants. The primary purpose of the long-term simulation of hourly runoff was for the subsequent analyses of long-term performance of detention ponds, described in Appendix F.

D.2 QUALHYMO MODEL SETUP - FUTURE CONDITIONS

Future land use conditions are outlined in Section 2.5. Low density residential development is proposed for the northern portions of the watershed, in sub-basins 101, 102 and 105. Sub-basin 105 will contain the Cloverleaf Estates Development (Van Meer, 1994). The remainder of the northern section of the watershed, sub-basins 108 and 110, will be developed into industrial and commercial uses (Cloverleaf Industrial Park, Ecos Garatech, 1990). South of Highway 401, open areas will be replaced by industrial and commercial developments, including the Belleville Home Centre, the White Rose retail store and the Zellers retail store. Further residential development has been assumed for a small area in the south west corner of the basin. An additional sub-basin was added (116) to take this area into account.

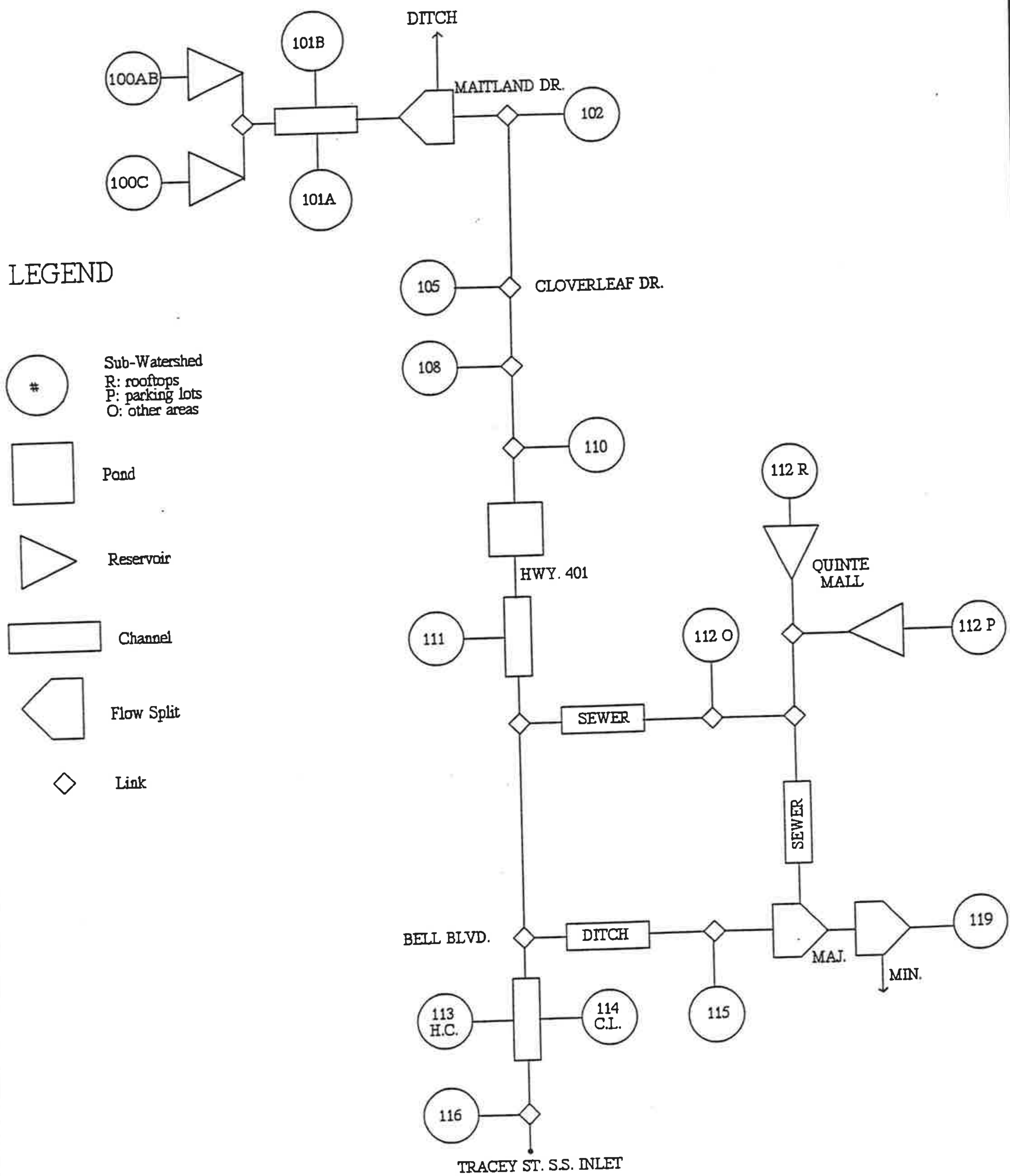
To model future land use, fractions of imperviousness were modified from existing conditions. These are provided in Table D-3. The model schematic for the future condition with no SWM is shown in Figure D-3.

Table D-4 presents the modelled peak flows for the 5 and 100-year storms for the future developed conditions, with no SWM controls except the existing wetland at Highway 401. The table also includes predicted future flows from previous studies, where available.

The 100-year peak outflow from the existing pond at Highway 401 is increased from 1.4 m^3/s to 2.3 m^3/s. Therefore additional detention volume will be required at this location to maintain pre-development peak flow. The 100-year peak flow at the inlet to the Tracey Street storm sewer has greatly increased, to 8.0 m^3/s, thus demonstrating the need for quantity control upstream of the storm sewer inlet.

Five System Options for control of quantity and quality for future conditions have been developed. These are described in Section 5.5. In summary, the primary components of each option are as follows:

System Option 1
- detention pond at 401
- detention pond at Tracey Street



MORA RIVER CONSERVATION AUTHORITY

QUALHYMO SCHEMATIC
FUTURE NO CONTROLS

Gore & Storrie Limited G&S
Consulting Engineers

FILE Nº \7644\DWGS\FIGD-3

DWG Nº D-3

Table D-3
FRACTIONS OF IMPERVIOUSNESS - FUTURE

Sub-basin	Percent Impervious
100A & 100B	4
100C	2
101A	28
101B	9
102	28
105	28
108	70
110	70
111	80
112	93
113	80
114	80
115	91
116	25
119	90

System Option 2
- detention pond at 401
- detention pond at Tracey Street
- source controls in new residential and industrial/commercial areas

System Option 3
- detention pond at Cloverleaf Drive
- detention pond at 401
- detention pond at Tracey Street

System Option 4
- detention pond at Cloverleaf Drive
- detention pond at 401
- detention pond at Tracey Street
- source controls in new residential and industrial/commercial areas

System Option 5
- detention pond at 401
- diversion of flows from north of 401 to Moira River
- detention pond at Tracey Street
- source controls in new residential and industrial/commercial areas

Table D-4
PEAK FLOWS - FUTURE (NO NEW CONTROLS)

Location	5-Year		100-Year	
	Previous Study	Present Study	Previous Study	Present Study
U/S Maitland Dr.	N/A	1.42	N/A	2.63
U/S Cloverleaf Dr.	N/A	1.99	N/A	3.80
U/S Pond N. of 401	N/A	4.25	N/A	7.35
D/S Pond N. of 401	N/A	1.24	N/A	2.33
D/S Bell Blvd.	3.25	3.85	5.13	5.91
U/S Storage @ Tracey St.	3.75	N/A (no storage simulated at Tracey St.)	6.70	N/A (no storage simulated at Tracey St.)
Inlet to Tracey St. S.S.	2.53	5.04	3.33	8.05

Model schematics for each of the five System Options are included as Figures D-4 through D-8.

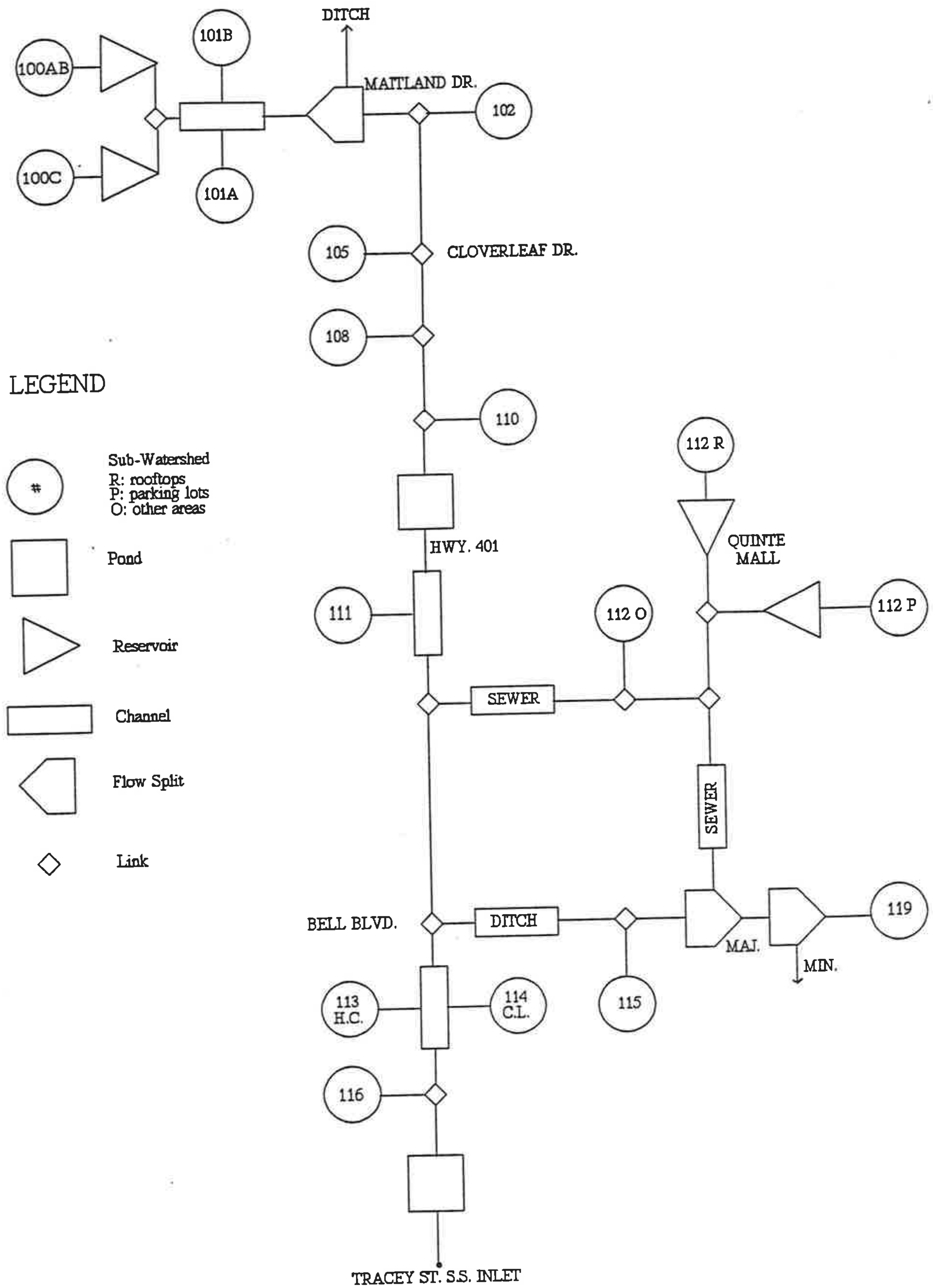
D.2.1 Modelling of Source Controls

The MOEE's Stormwater Management Practices Planning and Design Manual (MOEE, 1994) describes stormwater source controls as measures to control flows before reaching the subdivision/development conveyance system (*i.e.* Upper No Name Creek). As outlined in Section 5.2, the following source controls have been recommended and are applicable within the study area:

- Disconnection of roof leaders to grassed areas (in residential areas)
- Use of rooftop storage (in industrial/commercial areas)
- Use of parking lot storage (in industrial/commercial areas)

All of these measures are designed to detain stormwater for a limited time to reduce peak runoff flows. They are generally not meant to reduce the volume of runoff or treat the quality of the storm runoff (due to the short detention times involved). Roof leader disconnection, however, will reduce runoff volumes and improve water quality to some extent.

Modelling of source controls was undertaken using the guidelines published in MOEE (1994). Source controls were included as part of System Options 2, 4, and 5. Assumptions and methodologies are discussed in the following paragraphs.



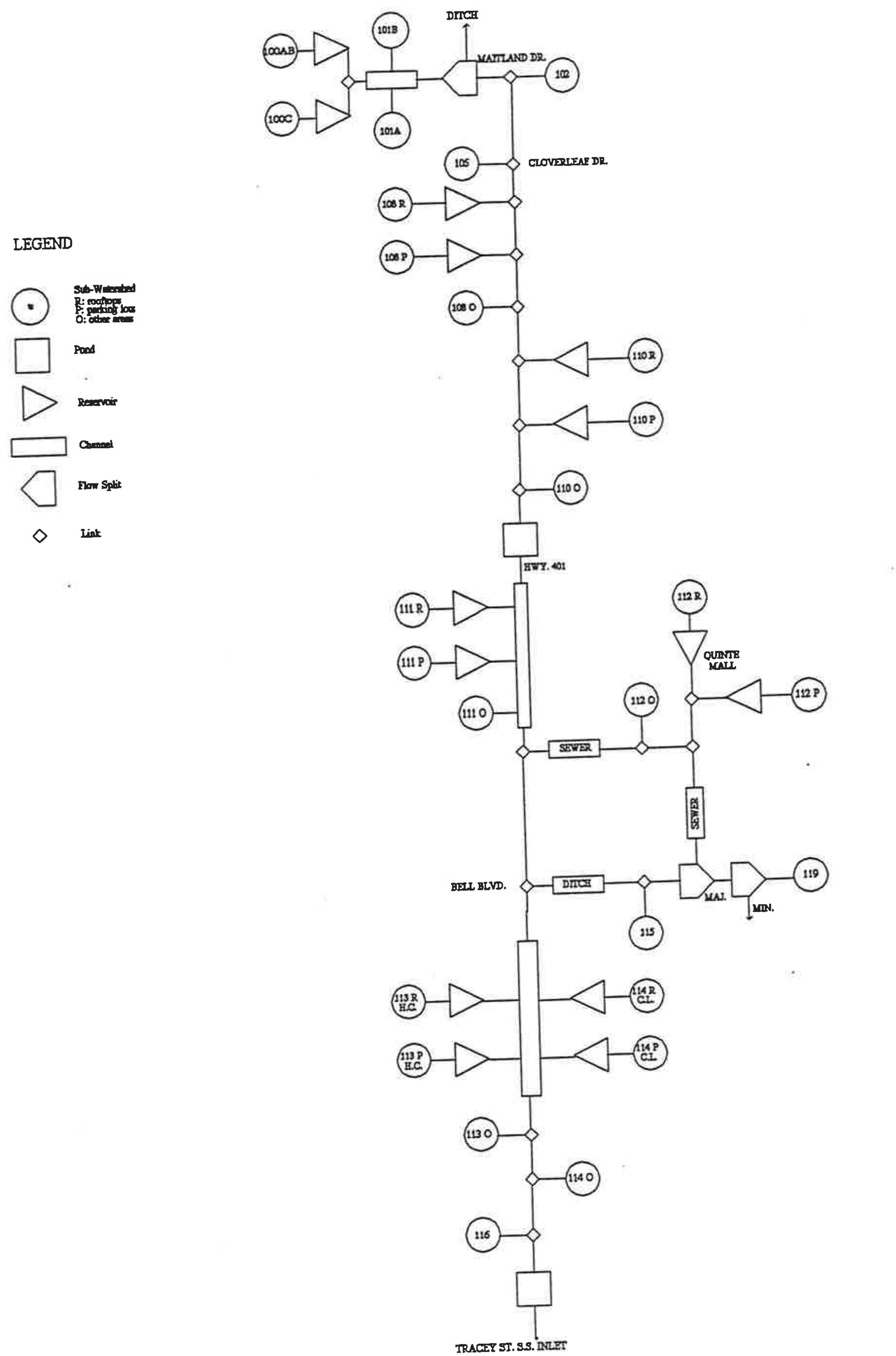
MORA RIVER CONSERVATION AUTHORITY

QUALHYMO SCHEMATIC SYSTEM OPTION 1

Gore & Storrie Limited GSS
Consulting Engineers

FILE Nº \7644\DWGS\FIGD-4

FIG Nº D-4



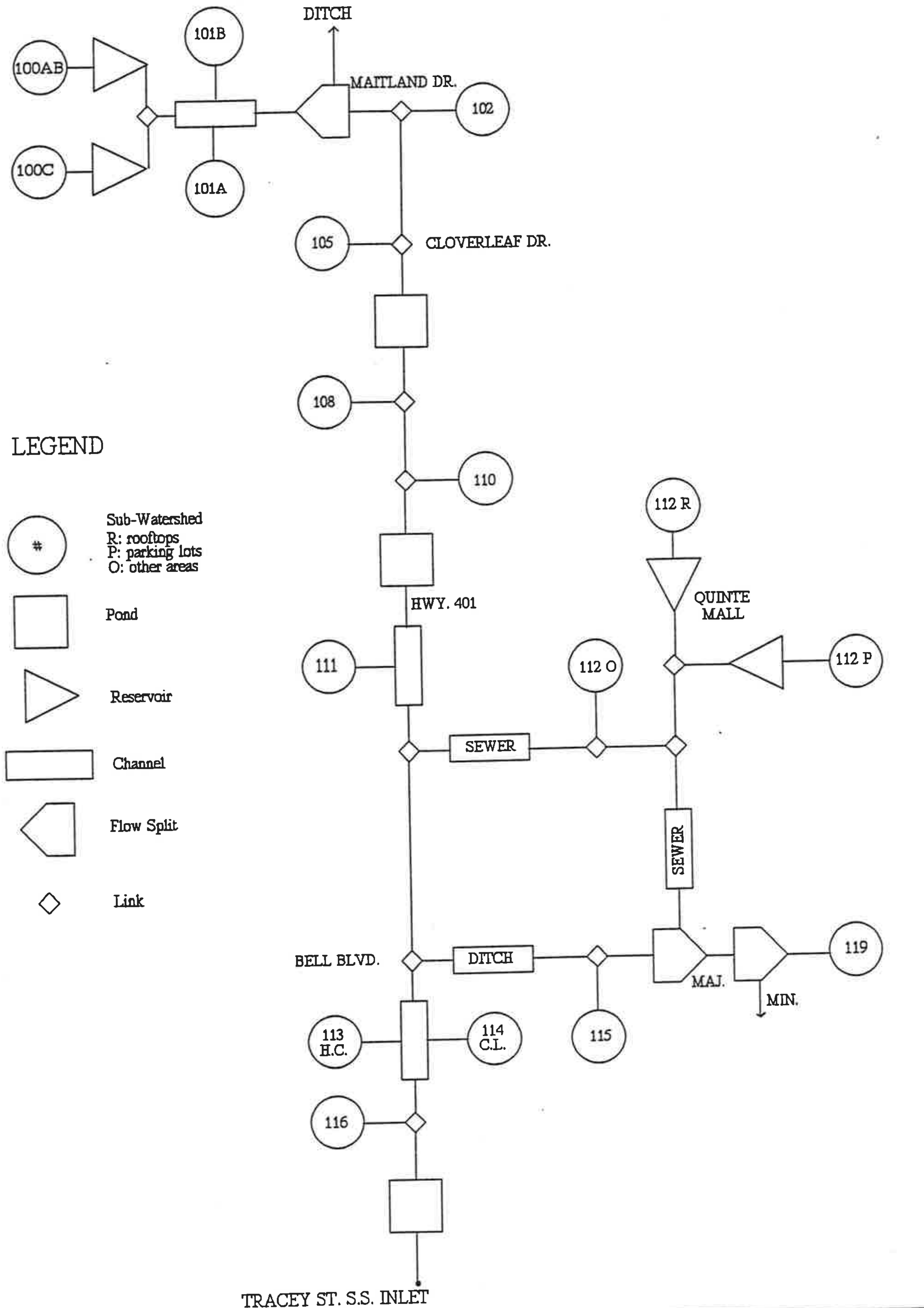
MOIRA RIVER CONSERVATION AUTHORITY

QUALHYMO SCHEMATIC SYSTEM OPTION 2

Gore & Storrie Limited G&S
Consulting Engineers

FILE Nº \7644\DWGS\FIGD-5

FIG Nº D-5



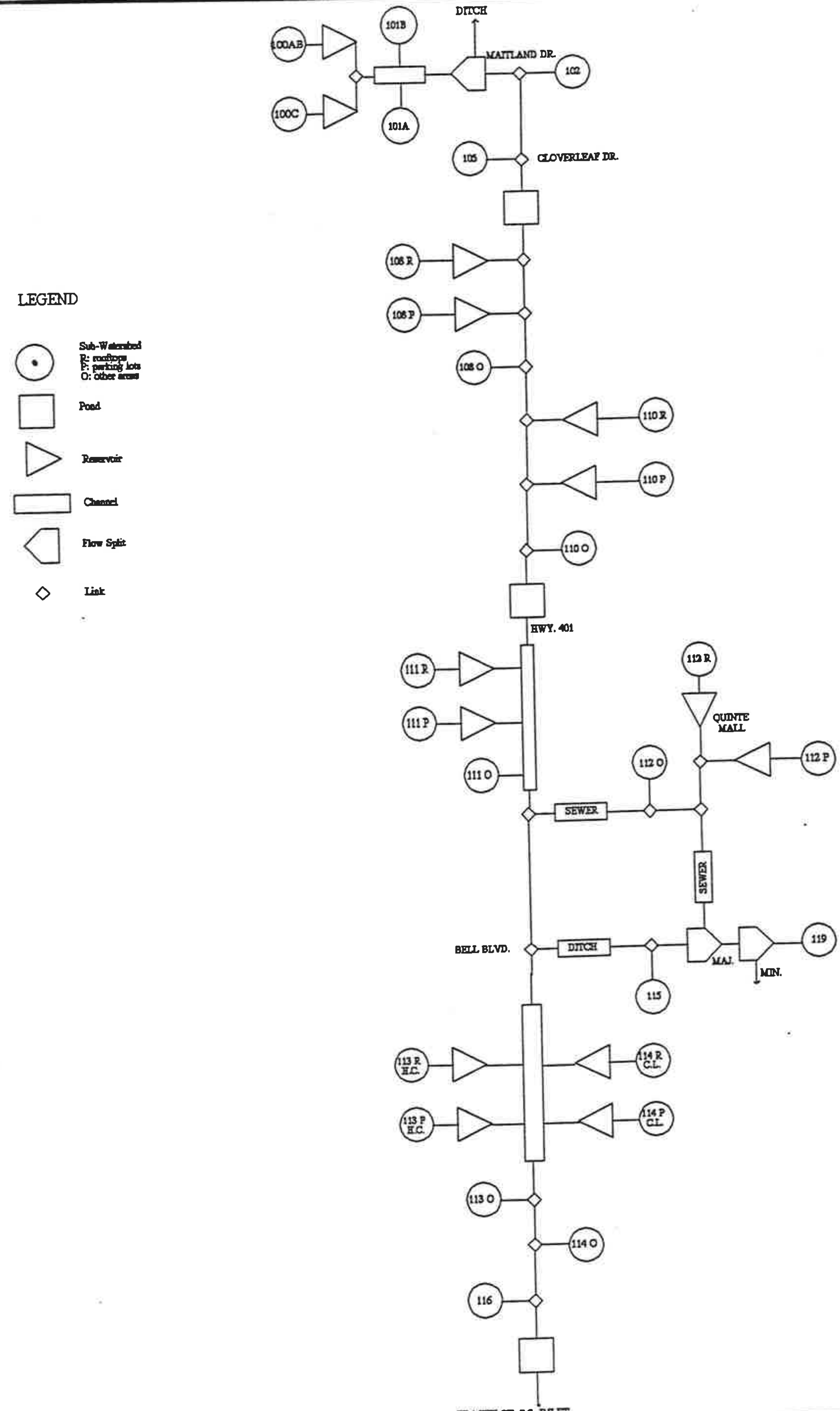
MORA RIVER CONSERVATION AUTHORITY

QUALHYMO SCHEMATIC
SYSTEM OPTION 3

Gore & Storrie Limited
Consulting Engineers

FILE Nº \7644\DWGS\FIGD-6

FIG Nº D-6



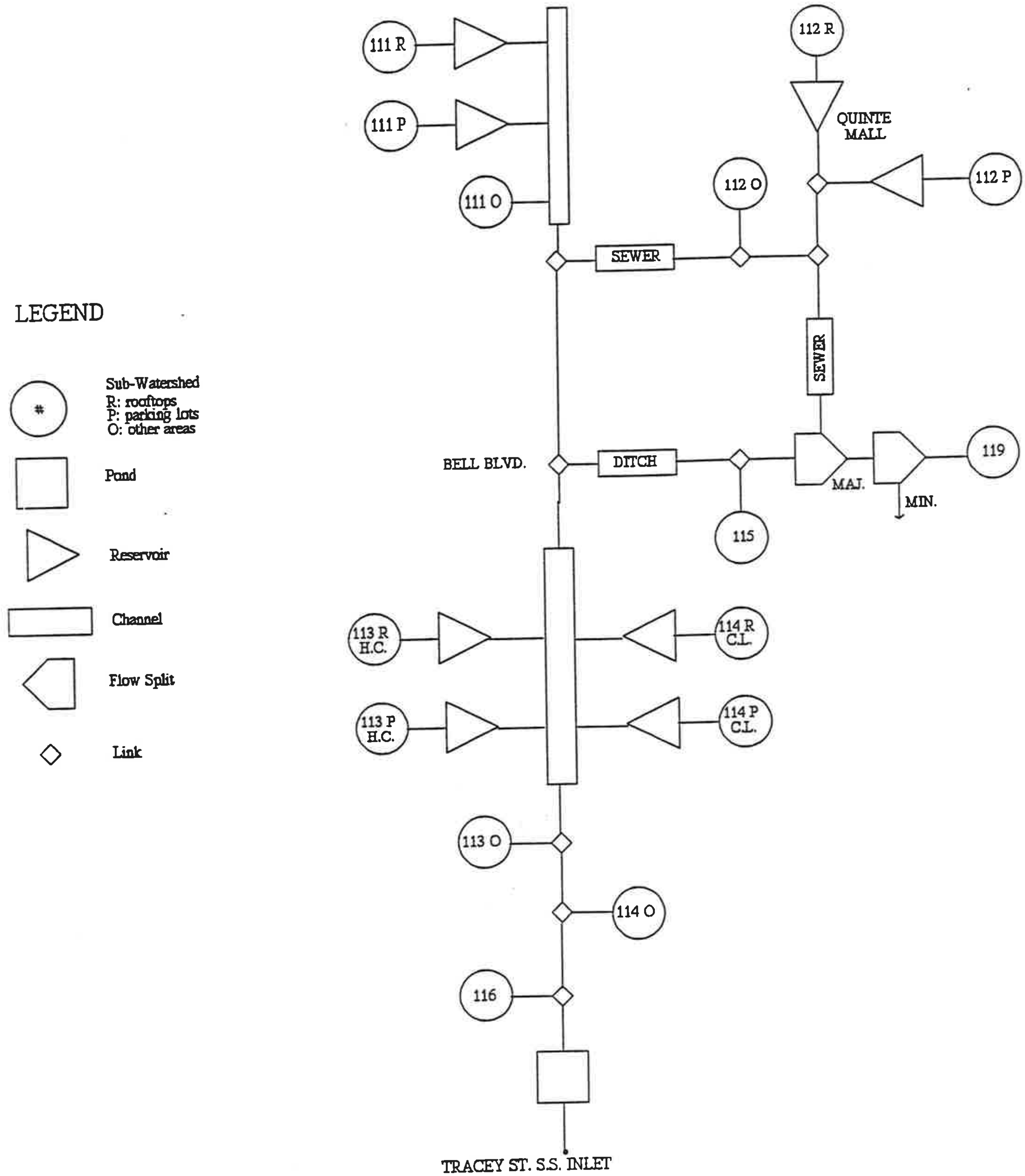
MOIRA RIVER CONSERVATION AUTHORITY

QUALHYMO SCHEMATIC
SYSTEM OPTION 4

Gore & Storrie Limited G&S
Consulting Engineers

FILE Nº \7644\DWGS\FIGD-7

FIG Nº D-7



MORA RIVER CONSERVATION AUTHORITY	QUALHYMO SCHEMATIC SYSTEM OPTION 5	Gore & Storrie Limited G&S Consulting Engineers	
		FILE Nº \7644\DWGS\FIGD-8	FIG Nº D-8

Disconnection of roof leaders to grassed areas (Residential):

To model roof leader disconnection in QUALHYMO for System Options 2, 4, and 5, the initial abstraction for impervious areas was set to 8 mm in residential subcatchments. Initial abstractions result in direct reduction of runoff production. The value of 8 mm was obtained using a weighted average of rainfall abstraction over the impervious parts of a subcatchment. Since the initial impervious abstraction was assumed to be 2.5 mm in the remaining 70% of impervious areas and 20 mm of rainfall would be abstracted (from MOEE, 1994) over rooftops (which were assumed to comprise 30% of impervious areas) then:

$$\begin{aligned} DSI &= 2.5 \times 0.7 + 20 \times 0.3 \\ &= 8 \text{ mm} \end{aligned}$$

where DSI = impervious area depression storage (mm).

Rooftop Storage (Industrial/Commercial)

Rooftop storage can be used where the majority of a building has flat roofs capable of storing up to 75 mm (3") of rainfall. Runoff is controlled by flow limiters in the rooftop leaders to restrict flows to about 1 L/s per 400 m^2 of rooftop.

The following assumptions were used to model this source control:

- rooftops comprise 33% of industrial/commercial areas and are 100% impervious
- roof structures limit the available detention storage to 75% of the roof area
- rainfall from the entire roof is detained in storage
- no pollutant removal occurs
- the stage/discharge/storage relationship given in Table D-5 is applicable for control devices such as the "Raintrol" drain or equivalent:

To model rooftop storage in QUALHYMO for System Options 2, 4, and 5, rooftops were considered separately, and each industrial/commercial (I/C) subcatchment was divided into rooftops (33%), parking lots (26.7%, or 40% of the remaining 67%), and other areas (various percentages dependent on the assumed imperviousness used for System Options 1 and 3). Reservoir routing for rooftops was modelled using the storage characteristics in Table D-5.

Parking Lot Storage (Industrial/Commercial)

Parking lot storage is useful in shopping plazas with large sections of continuous impervious pavement encircling the buildings. The maximum depth of water on the parking lots should be limited to about 160 mm to avoid damage to vehicles and to permit emergency access during flooding events. It is preferable to locate storage areas (where maximum depth can be as much as 300 mm) as far from main buildings as possible.

Table D-5
ROOFTOP STORAGE CHARACTERISTICS
(per hectare of rooftop)

STAGE (m)	DISCHARGE (m^3/s)	STORAGE (m^3)
0.000	0.000	0
0.025	0.008	250
0.050	0.017	500
0.075	0.025	750
0.100	0.035	1000

NOTE: Figures have been based on assumed density of 1 control device per 450 m^2
SOURCE: Falcone Smith Associates. "City of Belleville No Name Creek SWM Study", Dec. 1990.

Outlet control may include limiting the number of catchbasins, using inlet control devices (ICDs) in catchbasins, or by employing orifice control devices in the conveyance system. With any of these, it is important to ensure robust design is complemented by informed maintenance technicians to avoid tampering.

The following assumptions were used to model this source control:

- parking lots comprise 26.7% of industrial/commercial areas (40% of the remaining 67% that is not covered by rooftops)
- the fraction of impervious area for parking lots is 0.90 to account for landscaped dividers and other pervious areas
- no pollutant removal occurs
- a maximum equivalent runoff depth of 160 mm across the entire parking lot is applicable
- actual maximum runoff depth would be much less
- the drainage system hydraulics are such that the 100-year design flood would produce the 160 mm maximum depth assumed above

To model parking lot storage in QUALHYMO for System Options 2, 4, and 5, parking lots were considered separately, and each industrial/commercial (I/C) subcatchment was divided into rooftops (33%), parking lots (26.7%, or 40% of the remaining 67%), and other areas (various percentages dependent on the assumed imperviousness used for System Options 1 and 3).

A storage/discharge relationship was developed by assuming that discharge occurs immediately upon arrival of any runoff. The magnitude of the output was entered such that, when 160 mm of runoff was in storage, the outflow was the same as the 100-year peak flow generated for existing land uses using QUALHYMO. Considering Manning's equation for sheet flow, it can be shown that when the flow depth is reduced by half, the flow is reduced by a factor of 0.315. This relationship is illustrated in Table D-6 for an example 100-year existing peak flow of 1.000 m^3/s.

Table D-6
EXAMPLE PARKING LOT STAGE-STORAGE-OUTFLOW RELATIONSHIP

RUNOFF STAGE (m)	STORAGE (m^3/ha)	FLOW (m^3/s)
0.00	0	0.000
0.08	800	0.315
0.16	1600	1.000

D.3 *QUALITY MODELLING*

QUALHYMO is capable of generating and routing pollutants which either follow first order decay or discrete particle settling mechanisms. Bacteria (*Escherichia coli*, EC) and suspended solids (SS) were the pollutants of interest which were modelled in the Upper No Name Creek study.

QUALHYMO incorporates several algorithms for calculating runoff quality. This ensures that the pollutant generation routines used are conceptually related to a number of other models commonly used in Ontario, such as STORM, SWMM and HSPF. A common element to these quality models is the assumption that pollutants tend to build up at a certain rate during dry periods, through such time-dependent processes as atmospheric deposition and waste from vehicular traffic, and are subsequently entrained in runoff as washoff during wet periods.

Pollutant buildup is simulated in QUALHYMO during dry periods according to one of two possible methods: power linear and exponential. It is assumed that no buildup occurs during wet periods. For pollutant washoff, exponential or rating curve functions may be used. The exponential decay method is suitable for constituents which appear to have a pronounced first flush effect. The rating curve method is effective for those parameters which are strongly correlated with discharge rate.

For SS simulation, particle settling velocities are required. There is limited information available in the literature on settling velocities of sediments in urban stormwater runoff. For the present study, sediment data collected during the Nationwide Urban Runoff Program (NURP) (EPA, 1986) were used.

The rating curve approach is generally accepted as a reasonable approach for the estimation of SS loadings from both urban and non-urban systems for planning level studies. Sediments generally demonstrate a direct correlation with discharge rate. The major drawback of this approach is that it does not account for the temporal variability between rainfall events typically associated with variations in antecedent conditions. The buildup/washoff method may be better able to handle these temporal variations, but this procedure also has its drawbacks, in that more parameters need to be calibrated. Since no long-term pollutograph data are available for the Upper No Name Creek, the rating curve approach was used to estimate SS loading for this study.

For SS washoff rate, a unit flow approach was derived by G&S (1992) which relates washoff rate to both flow and drainage area. A regression analysis was performed on measured sediment concentrations and flows from five urban watersheds in the Ottawa area. For an average year, this relationship resulted in predicted average and peak SS concentrations within acceptable limits of literature values. Therefore the same unit flow approach for SS washoff was used for the present study.

The simulation of bacteria (EC) requires a first order decay rate and buildup/washoff coefficients. Since EC was found to be very closely related to fecal coliform (FC) concentrations in data from the Moira River (G&S, 1994), coefficients available in the literature for FC were used as representative of EC behaviour. The decay rate selected for preliminary model runs was 1.8 per day, or a T_{90} (time required for 90% bacteria decay) of 30 hours. This was the average coliform decay rate found in a study by Gietz (1983). The power linear buildup method was used. For washoff, the exponential method was used, as it is generally assumed that bacteria exhibit a first flush effect, which is best described by this method. The buildup/washoff parameters given in Rowney and MacRae (1991b) were used for initial model runs, which were obtained from calibration to several urban watersheds in the Ottawa area.

D.3.1 Comparison of Results to Literature Values

As discussed in Section 2.3, no simultaneous water quality and flow data were obtained during wet weather in the 1994 field program suitable for model calibration. Therefore event mean concentrations simulated by the model were simply compared to literature values.

Using the preliminary input parameters described above, event mean concentrations of SS at existing land use conditions ranged from 120 to 140 mg/l, and at future conditions, from 120 to 900 mg/l. These are within the expected range of SS concentrations in urban stormwater. No further adjustments were made to SS input parameters.

Event mean concentrations for EC ranged from 2,000 to 2,500 organisms/dl during initial runs. This is within (but at the low end) of the range of concentrations in the literature, as well as lower than the measured concentrations in Upper No Name Creek during the rainfall event in June, 1994. Adjustments in the EC washoff coefficients were therefore made during final simulation runs of all System Options, as well as for the existing and future (no SWM) cases. EC washoff rate parameters were adjusted until predicted simulated event mean concentrations for existing land use conditions upstream of the ponds were approximately 7,500 no/dL. This is closer to the mid-range of literature values, and was in good agreement with values measured during the wet weather event of June, 1994 (Appendix A).

APPENDIX E

HYDRAULIC ANALYSIS

HYDRAULIC ANALYSIS

E.1 ESTIMATION OF EXISTING 100-YEAR FLOODLINES

Hydraulic analysis was carried out in order to estimate flood levels and resulting floodway areas for the Regional Storm event (*i.e.* the 100-year rainfall event) for present-day conditions in the watershed. The HEC-2 hydraulic modelling package was used for this analysis.

The HEC-2 model (Hydrologic Engineering Center, 1991) was developed to calculate water surface elevations for steady, gradually varied flow in channels. The one-dimensional hydraulic energy equation is the basis of the computation. Input requirements for HEC-2 include starting water elevation at the downstream end of the system (at the Tracey Street storm sewer in this study), discharges at various locations along the channel, energy loss coefficients (for culverts), and floodway cross-sectional geometry (from survey data).

HEC-2 is used to compute water surface elevations (WSELs) caused by particular runoff events. In this study, 100-year WSELs were determined for existing land uses in an effort to determine the potential extent of flooding across the present study area.

The HEC-2 model that has been created to represent Upper No Name Creek consists of 32 sections. These were derived from survey data acquired from a limited survey of the flow channel, conducted by G&S in December, 1993. These survey data were augmented by topographic information available from 1:2,000 scale topographic mapping that had been provided by the Moira River Conservation Authority.

The model extends from the Tracey Street storm sewer inlet (@ 0+000 m) to 30 m north of Maitland Drive (@ 1+820 m). Six culverts between these two sections were also entered into the model using data from surveys and previous reports. Culverts were modelled using the U.S. Federal Highway Administration culvert routines recently added to the HEC-2 model.

To obtain 100-year floodlines for existing conditions, it was necessary to enter 100-year peak flows obtained from QUALHYMO hydrologic modelling into the HEC-2 input file. These peak flows are summarized in Table E-1. Corresponding floodline water surface elevations are summarized in Table E-2.

Figure E-1 presents the resulting estimate of the areal extent of the 100-year flood under existing conditions. It should be noted that the certainty and accuracy of this floodline declination is somewhat limited by the available topographic mapping and survey information. Nonetheless, this estimate is considered to be a reasonable representation of the existing situation. It should also be noted that the addition of 300 ha of drainage area in Subcatchments 100 and 101 in the final modelling runs did not significantly alter the estimated floodlines.

Note that the HEC-2 modelling indicates that the Cloverleaf Drive culvert cannot convey the existing 100-year flow resulting in flow overtopping the roadway.

Table E-1
SUMMARY OF PEAK FLOWS INPUT TO HEC-2 MODEL

SECTION NUMBER	100-YEAR PEAK FLOWS (m^3/s)
0+000 Tracey St. SS Inlet	4.675
0+011 Lower conveyance ditch	4.665
0+094 Lower conveyance ditch	4.587
0+104 Culvert #1	4.577
0+205 Culvert #2	4.482
0+270 D/S of Bell Blvd. (Culvert #3)	4.422
0+311 U/S of Bell Blvd. (Culvert #3)	1.536
0+400 Upper conveyance ditch	1.509
0+540 Natural channel	1.467
0+660 Natural channel (wetland)	1.431
0+886 U/S of Hwy. 401 (Culvert #4)	1.364
0+900 Natural channel D/S of pond	1.360
1+100 Natural channel U/S of pond	3.435
1+200 Natural channel	3.135
1+300 Natural channel	2.835
1+327 U/S of Cloverleaf Dr. (Culvert #5)	2.764
1+500 Natural channel	2.418
1+700 Natural channel	2.018
1+790 Natural channel	1.838
1+817 U/S of Maitland Dr. (Culvert #6)	1.777

Table E-2
SUMMARY OF 100-YEAR WATER SURFACE ELEVATIONS FOR EXISTING LAND USES

SECTION NUMBER	100-YEAR WSELs (m)
0+000 Tracey St. SS Inlet	91.06
0+011 Lower conveyance ditch	91.40
0+094 Lower conveyance ditch	91.90
0+104 Culvert #1	92.46
0+199 2nd. conveyance ditch	92.46
0+205 Culvert #2	92.47
0+220 3rd. conveyance ditch	92.47
0+270 D/S of Bell Blvd. (Culvert #3)	92.50
0+311 U/S of Bell Blvd. (Culvert #3)	93.17
0+400 Upper conveyance ditch	93.21
0+540 Natural channel	93.30
0+660 Natural channel (wetland)	93.38
0+836 D/S of Hwy. 401 (Culvert #4)	94.13
0+886 U/S of Hwy. 401 (Culvert #4)	94.66
0+900 Natural channel D/S of pond	94.87
1+100 Natural channel U/S of pond	95.87
1+200 Natural channel	97.98
1+300 Natural channel	101.39
1+316 D/S of Cloverleaf Dr. (Culvert #5)	102.25
1+327 U/S of Cloverleaf Dr. (Culvert #5)	102.71
1+500 Natural channel	104.82
1+700 Natural channel	105.92
1+790 Natural channel	106.94
1+796 D/S of Maitland Dr. (Culvert #6)	107.32
1+817 U/S of Maitland Dr. (Culvert #6)	107.77

E.2 ANALYSIS OF CULVERT CAPACITIES

Hydraulic computations have also been carried out to examine the capacity of existing culverts at Maitland Drive, Cloverleaf Drive, Highway 401, and Bell Boulevard. In particular, it was required to determine whether culvert capacities would be adequate to convey 100-year peak flows under future land use conditions, for the recommended SWM strategy.

It is important to note that this analysis was meant to determine only if 100-year flood flows could be conveyed by the culverts. The analysis is *not* meant to be used as a basis for redesign of the culverts for future conditions.

For each culvert, capacity was first checked for "inlet control" conditions in which the culvert inlet is the limiting factor. The checks were performed using the Ontario Ministry of Transportation's "Drainage Manual" culvert design charts (MTO, 1985). This set of nomographs is considered accurate to within 10%. It is important to note that they were developed for a 2% (0.02 m/m) typical culvert grade. For different culvert grades (such as those encountered at culverts in the study area), the nomographs are less accurate since inlet control headwater will vary with culvert grade. Nonetheless, they still serve as a good method to provide approximate capacity values.

Outlet control conditions (in which the culvert barrel or the tailwater conditions limits culvert flow) were also reviewed using these nomographs. The outlet-control head loss estimated for the future flow condition would ideally not be substantially different than that computed by HEC-2 modelling of 100-year existing condition flows.

A summary of the assumed and calculated parameter values used in the analyses is included in Table E-3. Manning's "n" values were based on the MTO Drainage Manual and the HEC-2 User's Manual (Hydrologic Engineering Center, 1991). Culvert grades were obtained from survey data and previous reports. For all culverts except Cloverleaf Drive, inlet-control capacity was based on a maximum allowable headwater-to-diameter ratio corresponding to a 100-year freeboard allowance of at least 1.0 m (*i.e.* capacity is when upstream water level is at 1.0 m below the crest of the roadway). At Cloverleaf Drive, a ratio of 1.21 was used in an effort to determine the maximum possible flow that could be conveyed through this culvert under inlet control while keeping the water level directly below the roadway (0 m freeboard).

A summary of flows for each culvert is provided in Table E-4. Future 100-year flows were obtained from QUALHYMO modelling of System Option #2. From this table, it can be seen that two of the culverts can adequately convey 100-year flows under existing and future land uses. The culvert underlying Cloverleaf Drive would need to be upgraded to convey the existing or future 100-year flow without allowing roadway overtopping. However, a lower level of service may be permissible at Cloverleaf Drive. It is recommended that further HEC-2 modelling be performed in the vicinity of Cloverleaf Drive to design adequate culvert improvements at this location. More detailed surveying is required to properly design an adequate replacement culvert. Also, it is recommended that inlet/outlet improvements be

Table E-3
SUMMARY OF PARAMETER VALUES USED FOR CULVERTS

LOCATION	DESCRIPTION	n	S (%)	(HW/D)
Maitland Drive	1.22 m CSP*, 21 m long	0.021	0.0	1.04
Cloverleaf Drive	1.22 m CSP*, 11 m long	0.021	1.3	1.21
Highway 401	0.92x1.22 m^2 box culvert, 50 m long	0.012	0.1	1.00 assumed
Bell Boulevard	1.52 m RCP*, 31 m long	0.014	0.5	1.00 assumed

*CSP: corrugated steel pipe, RCP: reinforced concrete pipe,
S: culvert grade, HW/D: headwater-to-diameter ratio used in inlet-control capacity estimation

Table E-4
SUMMARY OF CULVERT FLOWS

	PEAK FLOW FROM HYDROLOGIC MODELLING		CULVERT CAPACITY ASSESSMENT		
LOCATION	Q100 (m^3/s) EXIST.	Q100 (m^3/s) FUTURE	Q_{in} (m^3/s)	Q_{out} (m^3/s)	REMARK
Maitland Dr.	1.8	2.6	2.4	2.2	UPGRADE
Cloverleaf Dr	2.8	3.8	2.4	2.3	UPGRADE
Highway 401	1.4	1.4	1.5	2.2	OK
Bell Blvd	2.2	2.1	3.9	5.2	OK

Note: Q_{in} is the estimated inlet control capacity

Q_{out} is the estimated outlet control flow capacity with head loss across the culvert the same as indicated in the HEC-2 analysis of existing 100-year conditions, except at Cloverleaf where head loss reduced to amount that will eliminate flow over the roadway.

considered at the Maitland Drive culvert to enable it to convey flows due to future development above this location. It is recommended that further modelling be performed to optimize headwall/wingwall remedial works.

E.3 PRELIMINARY SIZING OF PROPOSED FLOW CONVEYANCE CHANNELS

To minimize the extent of flooding during severe events up to the 100-year return period, stormwater management options are based on the proposal that the existing natural channels comprising the various sections of Upper No Name Creek be modified to be designed flow conveyance channels. These channels would be designed to convey event flows within a well-defined and designated floodway. This approach would facilitate land development and help to ensure a high level of flood protection. Moreover, it would provide the opportunity to design the flow channels as linear greenspace forming a greenway system linking the various neighbourhoods and developments.

E.3.1 General Concept for the Conveyance Channels

Preliminary sizing has been carried out via hydraulic calculations to estimate the dimensions needed to convey flows up to the estimated 100-year peak flows reported earlier. The sizing has been based on the following geometry:

- A primary flow channel with a depth of 0.65 m, with average bank slopes of 2:1 (H:V). This primary flow channel would be capable of conveying flows up to about the 5-year flow.

- A terraced grassed floodway capable of conveying the 100-year peak flow while maintaining a maximum total flow depth of 1.0 m. It was assumed that the sideslopes of the floodway area could have an average value of about 4:1 (H:V)

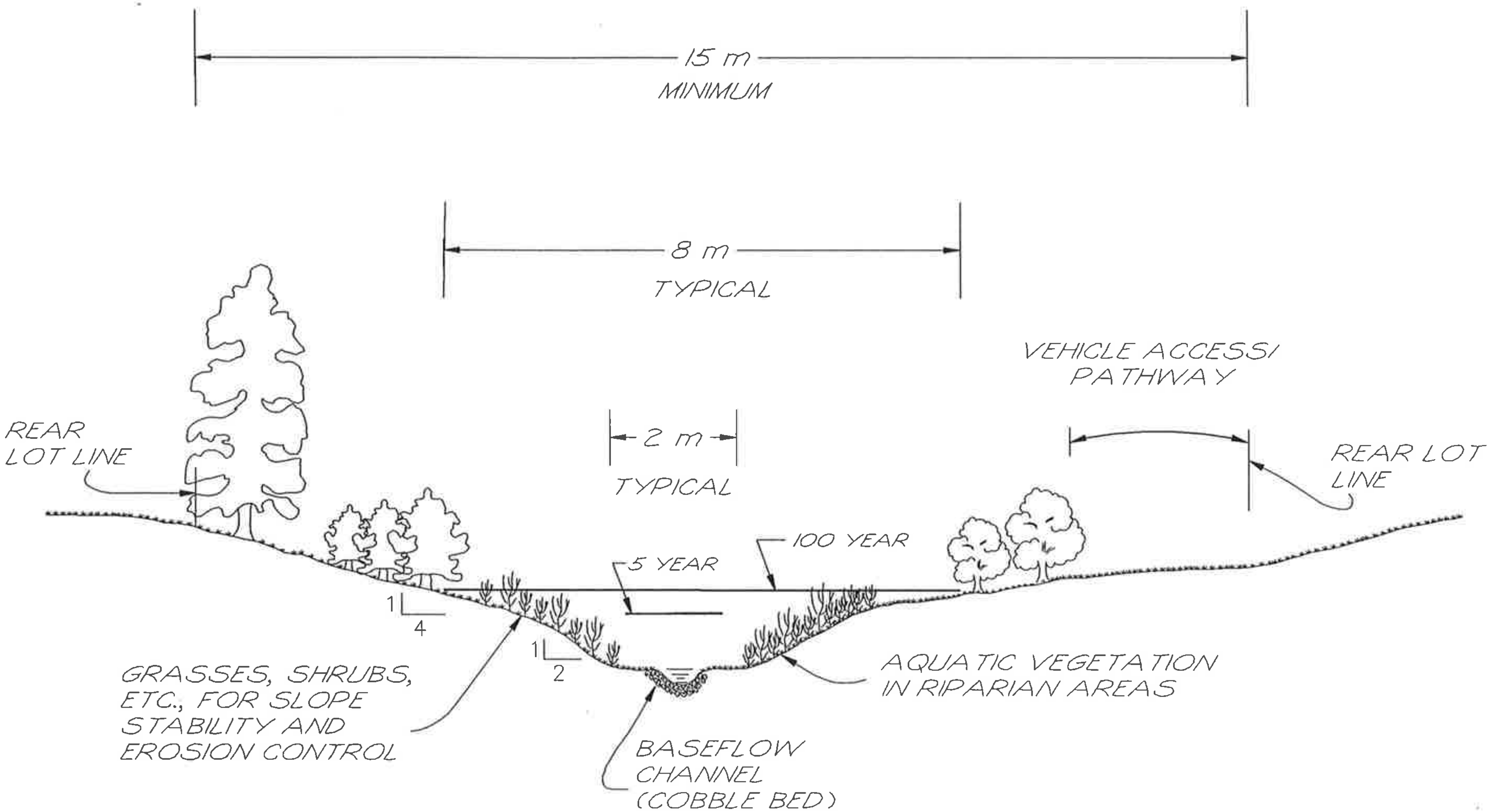
The resulting general floodway/channel configuration is depicted in Figure E-2.

As indicated on this sketch, the general concept is that the channel/floodway be designed as a naturalized flow route; that is, designed and landscaped to have a natural character. This approach will help to minimize net impacts on the local environment, and will provide potential for local enhancement of aquatic or riparian habitats which are at present highly disturbed.

For example, the bed of the primary flow channel could be lined with cobbles and stones to minimize erosion potential while also improving physical diversity and aquatic habitat potential. The channel banks and sideslopes can be stabilized via vegetative plantings including grasses, sedges, shrubs, and aquatic species. Finally, to allow use of the channel floodways as parkland, some areas of turf grass could be incorporated within the overall landscape plan.

E.3.2 Method of Preliminary Sizing

Preliminary sizing has been carried out for five separate lengths of the creek. Analysis has been based on Manning's equation for steady, uniform flow. The width of the main channel, W, was determined using the assumed channel dimensions as



NOTE: Typical 5-year flow depth is approx. 0.65 m
Design 100-year flow depth is 1.00 m

MORA RIVER CONSERVATION AUTHORITY	**TYPICAL CONVEYANCE CHANNEL CROSS SECTION**	**Gore & Storrie Limited G&S** Consulting Engineers
		FILE Nº \7644\DWGS\FIGE-2
		FIG Nº E-2

well as the estimated peak 100-year peak flow that would need to be conveyed through each section. This analysis has been conducted for each of the five System Options under consideration in this study.

In carrying out the computations, average channel slope was assumed to be the slope between the upstream and downstream channel inverts for each section, yielding constant energy gradients for each reach.

E.3.3 Erosion and Velocity Considerations

For the two reaches from Maitland Drive to Cloverleaf Drive, and from Cloverleaf Drive down to the stormwater management facility north of Highway 401, the resulting channel slopes are more than 1% (1.1% and 2.4%, respectively). Thus, 5-year flow velocities were estimated at about 1.4 m/s and 2.0 m/s, respectively. It is expected that flow velocities on this order could cause local erosion problems unless adequate erosion protection measures are put in place. As well, velocity-control measures (*i.e.* energy gradient control) could be used to assist with erosive power control. Within the context of a natural form of channel design this could possibly take the form of pool-riffle sequences along the creek to help reduce flow velocities, dissipate flow energy and also provide increased aquatic habitat potential.

E.3.4 Preliminary Sizing and Land Area Requirements

A summary of results is provided in Table E-5. Note that typical floodline breadth (B) for the 100-year storm is 6 to 9 metres. This width should be regarded as the estimated minimum width needed for flow conveyance only. Total land width needed should also be based on the following considerations:

- Maintenance access along the flow conveyance channels will be required. A width of at least 3 metres should be provided for this purpose to allow vehicle access along the channel system.

- As well, vegetated buffer areas should be provided alongside the channels to allow potential for pedestrian or recreational pathways, and to protect the creek from direct washoff from adjacent properties.

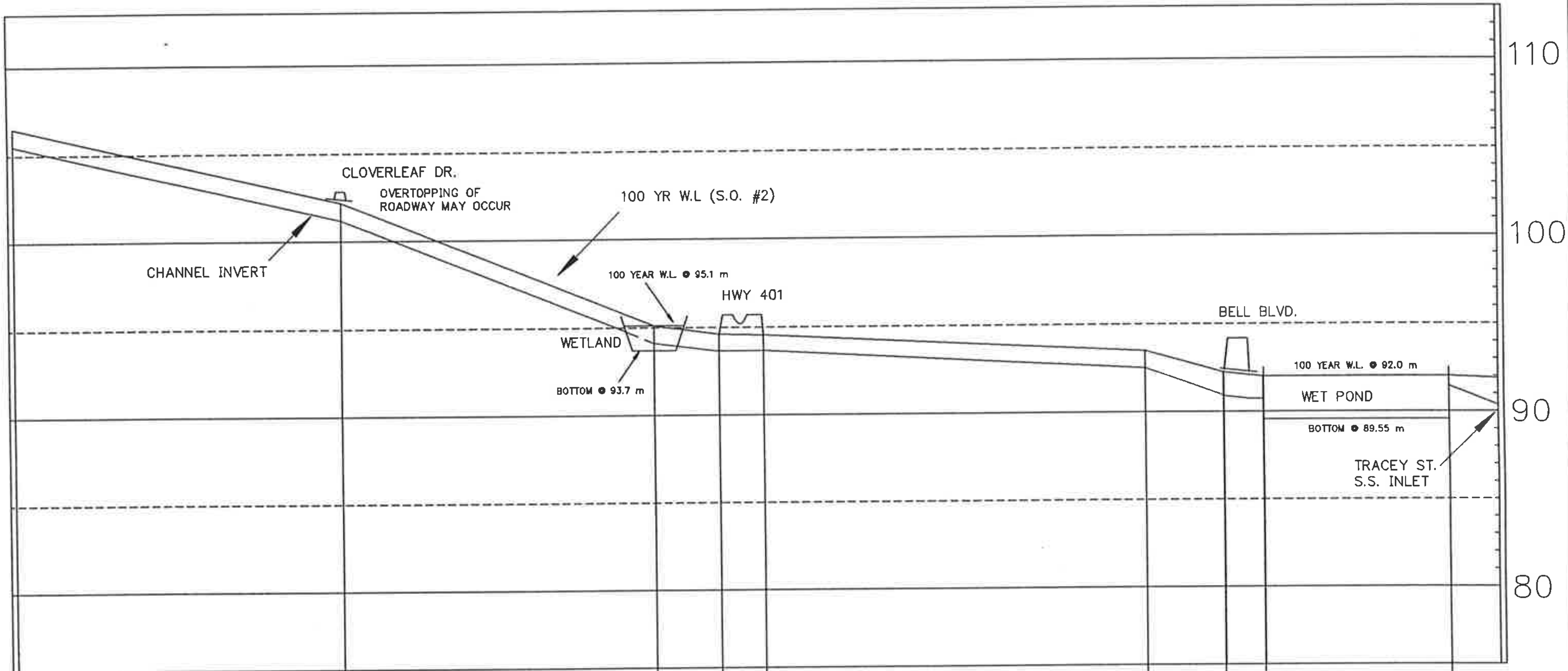
For these reasons, it is recommended at this stage that an allowance of a 15 metre minimum width be reserved for the flow conveyance channels for most of the system.

E.3.5 Resulting Hydraulic Conditions For System Option 2

Approximate 100-year water surface profiles that would result under System Option 2 have been estimated. These are illustrated in Figure E-3.

Table E-5
PROPOSED CONVEYANCE CHANNEL WIDTHS AND BREADTHS

LOCATION	OPTION 1		OPTION 2		OPTION 3		OPTION 4	
	W (m)	B (m)	W (m)	B (m)	W (m)	B (m)	W (m)	B (m)
N. ditch-Maitland	2.6	8.8	2.6	8.8	2.6	8.8	2.6	8.8
Maitland-Cloverleaf	1.9	8.1	1.8	8.0	1.9	8.1	1.8	8.0
Cloverleaf-Pond 2	2.6	8.8	1.5	7.7	2.0	8.2	1.1	7.3
Pond 2-401	0.6	6.8	0.6	6.8	0.7	6.9	0.6	6.8
401-S. ditch	1.8	8.0	1.5	7.7	1.7	7.9	1.4	7.6



100 YEAR WATERLEVEL	106.5	102.2	95.1	94.6	94.55	93.5	92.25	92.0	92.0
INVERT ELEVATION	105.5	101.2	94.1	93.65	93.65	92.5	90.9	90.75	91.45
CHAINAGE	1700	1320	960	885	835	400	310	264	54

DATE 95-03-07	PROFILE: 100 YEAR WATER LEVEL SYSTEM OPTION #2	Gore & Storrie Limited G&S Consulting Engineers
SCALE V ~ 1:400 H ~ 1:7000		FILE Nº FLD-PRFL — DWG Nº E-3

APPENDIX F

POND PERFORMANCE ASSESSMENT

POND PERFORMANCE ASSESSMENT

F.1 BASIS FOR SIZING OF SWM FACILITIES

As discussed in Section 5.6, five System Options for stormwater management have been formulated. Detention ponds are primary components of each option. Options 1, 2 and 5 include a wetland immediately upstream of Highway 401, in the location of the existing beaver ponds, and a wet pond located immediately upstream of the Tracey Street storm sewer inlet. Options 3 and 4 include the same two detention basins, but also incorporate an additional dry pond, located between Cloverleaf and Maitland Drives.

For each System Option, pond sizing was required for subsequent comparative analyses of pond performance in terms of pollutant removals, and of comparative cost analyses.

With the exception of the pond north of Cloverleaf Drive, the ponds in all System Options were assumed to be dual-purpose facilities, for both quantity and quality control.

F.1.1 Pond Sizing for Quantity Control

Volumes of active storage and pond rating curves were determined for each detention pond in order to meet the peak flow targets discussed in Section 4. Assumed pond configurations were trapezoidal with either 3:1 or 4:1 side slopes.

Tracey Street Treatment Pond

A detention pond for stormwater treatment at Tracey Street was incorporated in all five System Options. The peak flow constraint for future conditions for the 100-year storm is 3.5 m^3/s, which is the maximum inlet capacity of the Tracey Street storm sewer. Through an iterative procedure using the QUALHYMO event model with the 100-year storm, the required active storage volume to meet the outflow target was computed.

The Tracey Street pond was configured for final modelling runs and costing purposes as a long, linear facility. This decision was based on discussion with the study Steering Committee on October 20, 1994. This pond configuration was preferred to the configuration originally proposed, which was designed to approximately fit into the two parcels of land presently deeded to the City of Belleville from landowners for possible use for a detention pond. The linear facility is preferable in terms of land requirements and ease of implementation with the proposed developments in the vicinity (Belleville Home Centre and Zellers). The facility was therefore designed to fit in a long, narrow corridor extending from the Ontario Hydro ROW to Bell Boulevard. The facility was also modelled to simulated a three-celled system, in order to increase pollutant removals.

A simple weir was assumed as the outlet to this pond, with a rating curve given by (Olson and Wright, 1990):

$$Q=K\frac{2}{3}b\sqrt{2g}H^{3/2}$$

where:

- Q = flow (m^3/s)
- K = flow coefficient, given by:

$$K=0.611+0.075\frac{H}{z}$$

- z = permanent pool depth (m)
- b = breadth of weir (m)
- g = acceleration due to gravity
- H = height of water in pond (m).

Weir length was determined by setting Q = 3.5 m^3/s with H determined by trial-and-error, then a rating curve was developed.

Active storage volumes computed for the Tracey Street pond for the five System Options, as well as peak inflow and outflow rates, are provided in Table F-1. The large decrease in required volume from Option 1 to 2, and from Options 3 to 4 are the result of the incorporation of source controls, in the form of parking lot and rooftop storage for industrial and commercial areas, and roof leader disconnection to pervious areas for residential areas, as described in Appendix D.

It is important to emphasize that optimization of active storage volume is necessary at the final design stage to minimize the size of the UV treatment facility. Preliminary checks using multiple orifices and a spill weir outlet structure have demonstrated that it may be possible to reduce the magnitude of the flow exceeded four times per swimming season, though an increase in active storage volume would be necessary.

Highway 401 Treatment Pond

The modification of the existing beaver pond upstream of Highway 401 was also incorporated in all five System Options. The peak flow constraint at this location for future conditions for the 100-year storm is 1.4 m^3/s, which is the equivalent to the pre-development peak flow, as well as the culvert capacity underneath the 401. Again through QUALHYMO event modelling, the required active storage volume to meet the outflow target was computed.

The Highway 401 wetland was configured in order to minimize the overall depth of the facility and to maximize the surface area (to enhance pollutant removals). The entire available area in the space designated in the Cannifton Secondary Plan (see Figure 2-3) was used.

Table F-1
SWM FACILITY SIZING -- FUTURE CONDITIONS

SWM Scenario #	Facility Location	Facility Type	Inflow (100-Year) (m^3/s)	Active Storage (m^3)	Permanent Pool (m^3)	Outflow (100-Year) (m^3/s)
1	N. of 401	Wetland	7.35	34,300	7,400	1.33
	U/S Tracey St. S.S.	Wet Pond	7.41	14,900	15,000	3.41
2	N. of 401	Wetland	4.60	20,600	7,400	1.37
	U/S Tracey St. S.S.	Wet Pond	4.79	5,300	15,000	3.47
3	Cloverleaf Dr.	Dry Pond	3.80	5,500	0	2.75
	N. of 401	Wetland	5.74	32,000	7,400	1.40
	U/S Tracey St. S.S.	Wet Pond	7.42	14,200	15,000	3.52
4	Cloverleaf Dr.	Dry Pond	3.78	5,200	0	2.76
	N. of 401	Wetland	3.62	20,600	7,400	1.33
	U/S Tracey St. S.S.	Wet Pond	4.74	5,000	15,000	3.50
5	N. of 401	Wetland	4.60	20,600	7,400	1.37
	U/S Tracey St. S.S.	Wet Pond	4.68	4,800	15,000	3.45

A simple weir structure was assumed as the outlet to this pond. The rating curve for the weir is given by the same governing equations as those presented above, and was developed in the same manner.

Active storage volumes, inflows and outflows computed for the 401 pond for the five System Options are also provided in Table F-1. Again, significant decreases in required volume from Option 1 to 2, and from Options 3 to 4 are observed as a result of source controls.

Cloverleaf Drive Detention Basin

A dry detention pond located between Cloverleaf Drive and Maitland Drive was incorporated in System Options 3 and 4. The active storage of this pond was sized tomatch the pre-development peak flow rate at this location of 2.6 m^3/s. A v-notched weir was assumed as the outlet device.

Table F-1 provides the required active storage volume for the Cloverleaf pond for System Options 3 and 4, along with peak inflow and outflow rates. The source controls incorporated with System Option 4 result in a small decrease in active storage requirement. The difference is smaller than for the other two ponds because the on-site storage potential for residential areas is not as great as for industrial/commercial areas. Only residential source controls are incorporated upstream of the Cloverleaf pond.

F.1.2 Pond Sizing for Quality Control

In the proposed SWM facilities, treatment is primarily effected by the hydraulic residence time that results from a permanent pool volume held within the facility. Preliminary sizing of these pool volumes (or retention volumes) was based on the recently published Stormwater Management Practices Planning and Design Manual (MOEE, 1994).

The guidelines allow estimation of the required permanent pool volume per hectare of serviced area for varying levels of imperviousness. Four levels of treatment are included, with Level 1 the most stringent. For the purposes of the Upper No Name Creek study, due to the sensitivity of the downstream receiver (Moira River and Bay of Quinte), the pool sizing was based on a Protection Level of 1. The storage requirements from the manual are reproduced as Table F-2. The values shown in Table F-2 include an additional volume of 40 m^3/ha of extended detention which must be subtracted from the total to get permanent pool volume alone.

Tracey Street Wet Pond

The proposed Tracey Street detention pond is intended as a dual purpose quantity/quality facility, and as such is a wet pond incorporating a permanent pool. The weighted average percent imperviousness for the sub-basins contributing to the Tracey Street pond (all sub-basins south of the 401 - sub-basins north of the 401 were not included, as water quality treatment for these areas is provided in the pond at

Table F-2: WATER QUALITY STORAGE REQUIREMENTS BASED ON RECEIVING WATERS (from MOEE, 1994)

Protection Level	SWMP Type	Storage Volume (m^3/ha) for Impervious Level			
		35 %	55 %	70 %	85 %
Level 1	Infiltration	25	30	35	40
	Wetlands	80	105	120	140
	Wet Pond	140	190	225	250
	Dry Pond (Batch)	140	190	210	235
Level 2	Infiltration	20	20	25	30
	Wetlands	60	70	80	90
	Wet Pond	90	110	130	150
	Dry Pond (Batch)	60	80	95	110
Level 3	Infiltration	20	20	20	20
	Wetlands	60	60	60	60
	Wet Pond	60	75	85	95
	Dry Pond (Batch)	40	50	55	60
	Dry Pond	90	150	200	240
Level 4	Infiltration	15	15	15	15
	Wetlands	60	60	60	60
	Wet Pond	60	60	60	65
	Dry Pond (Batch)	25	30	35	40
	Dry Pond	35	50	60	70

the 401) was 83.5%, with a total contributing area of 71.3 ha. For a wet pond, the required permanent pool volume interpolated from Table F-2 was found to be 247 m^3/ha, less the 40 m^3/ha of extended detention, for a total of 207 m^3/ha. Therefore the total permanent pool volume is estimated at 15,000 m^3, as shown in Table F-1.

Highway 401 Wetland

Since the existing pond at the 401 is natural wetland, this facility is to be designed as a wetland for stormwater quality and quantity control. For a wetland, required permanent pool volumes are less than for wet ponds.The future weighted average percent imperviousness for the sub-basins contributing to the wetland is approximately 13.6%, and the total contributing area is 447.6 ha. From Table F-2, the required permanent pool volume was extrapolated to be 55 m^3/ha, less the 40 m^3/ha of extended detention, for a total of 15 m^3/ha. Therefore the minimum required permanent pool volume for the 401 wetland is at 6,700 m^3. With a pool depth of 0.4 m assumed, the total permanent pool volume becomes 7,400 m^3 (see Table F-1).

F.2 COMPARATIVE PERFORMANCE EVALUATIONS

Once sizing was accomplished, as described above, modelling of pollutant delivery, transport and removal was carried out with the continuous QUALHYMO model. Pollutant generation input parameters used are described in Appendix D.3.

The modelling was conducted to examine the performance of each System Option in terms of:

- percent removal of SS by particle settling
- percent removal of EC by natural die-off
- number of individual exceedances of the target SS and EC levels of 25 mg/l and 100 no/dl respectively.

To streamline the performance analysis, a six-year set of hourly rainfall was used covering the summer recreational season (June 1 to September 30). The selection of the representative six-year dataset is described in Appendix C. Using the six years of simulation results, average results on a seasonal basis were obtained for percent removals from detention ponds and numbers of exceedances of the above SS and EC targets for each scenario.

The results are summarized in Table F-3. Results for the existing and future with no controls cases are included with the System Option results for comparison. The results clearly show the significant decrease from the existing and future cases with no controls in exceedances of both SS and EC with the implementation of stormwater management facilities in the five System Options. Pollutant removals, particularly of EC, are significantly improved in the pond at Highway 401 when it is modified from its existing form to a wetland sized for water quality control.

The differences in pond performance between System Options are not significant, however, since all ponds were sized on the same basis for water quality control. The

Table F-3
POND PERFORMANCE -- POLLUTANT REMOVAL

Scenario	Facility	% Removal of SS Load	# of Exceedances of SS > 25 mg/L D/S of Pond	% Removal of EC Load	# of Exceedances of EC > 100/Dl D/S of Pond
Existing No Controls	N. of 401	70	23	14	21
	U/S Tracey St. S.S.[1]	N/A	33	N/A	20
Future No Controls	N. of 401	72	26	14	19
	U/S Tracey St. S.S.[1]	N/A	25	N/A	19
System Option 1	N. of 401	88	5	88	6
	U/S Tracey St. S.S.	55	6	76	6
System Option 2	N. of 401	85	4	79	6
	U/S Tracey St. S.S.	54	7	75	6
System Option 3	Cloverleaf Dr.	56	17	20	18
	N. of 401	87	5	87	6
	U/S Tracey St. S.S.	55	7	76	9
System Option 4	Cloverleaf Dr.	53	17	19	18
	N. of 401	83	4	80	6
	U/S Tracey St. S.S.	55	7	66	6
System Option 5	N. of 401	85	4	79	6
	U/S Tracey St. S.S.	79	14	91	4

[1] Number of exceedances shown are upstream of inlet to storm sewer - no pond for these scenarios

percent removals of SS in the pond at Highway 401 range from 83 to 88%. Percent removals of SS in the pond at Tracey Street are lower, ranging from 54 to 55%. The dry pond at Cloverleaf Drive achieves 53 to 56% reduction in SS loadings. Removals of EC from the water quality ponds at the 401 and Tracey Street are generally high. Reductions in loadings of 79 to 88% are observed in the 401 pond, and reductions of 66 to 91% in the Tracey Street pond. Removals from the Cloverleaf pond, on the order of 20%, are significantly lower due to the absence of a permanent pool.

Numbers of exceedances of both SS and EC targets range from four to fourteen per season for the various System Options. Although low, the exceedances are still generally greater in number than the stringent Bay of Quinte RAP criteria of four exceedances per season. In the case of bacteria control, physical effluent disinfection will be required at the outlet of the SWM facility at Tracey Street, as discussed in Section 5.

The pond at Cloverleaf shows much more frequent exceedances (between 17 and 18) of both SS and EC than the 401 and Tracey Street ponds, again due to the absence of a permanent pool for the enhancement of pollutant removal.

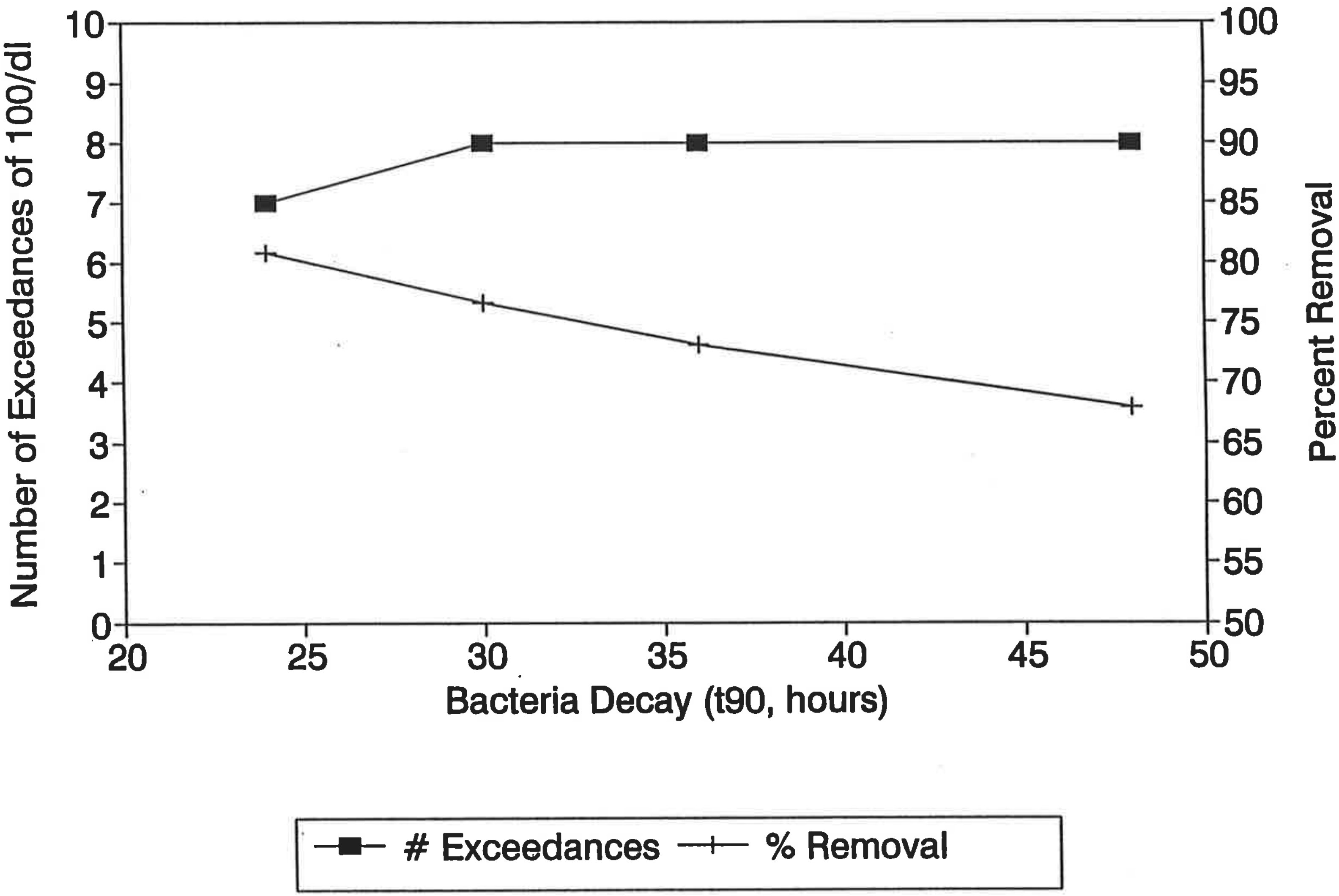
F.3 SENSITIVITY OF BACTERIA REMOVAL TO DECAY RATE

The simulated bacteria (EC) removals and numbers of exceedance may be fairly sensitive to the first order decay rate used. Since the value used in modelling runs was based on the literature and not on site-specific data, a sensitivity analysis was conducted to test the effect of varying decay rates on the results.

The T_{90} (time required for 90% of bacteria to decay) used in preliminary runs was 30 hours, for a decay rate of 1.8 per hour (as discussed in Appendix D.3). Three other decay rates were tested in the analysis: 24 hour T_{90} (decay rate 2.3 per day), 36 hour T_{90} (1.5 per day) and 48 hour T_{90} (1.15 per day). The higher the T_{90} the higher the simulated EC numbers will be.

The results for the two ponds proposed for System Option 2 are provided in graphical form in Figures F-1 and F-2. Figure F-1 presents the results for the wetland at Highway 401. As shown, percentage removal of EC loadings is somewhat sensitive to decay rate, with removals as high as 81% with a 24 hour T_{90}, and as low as 68% at 48 hours. Numbers of exceedances are not very sensitive to decay rate, dropping only as much as one per season at the highest decay rate. As shown in Figure F-2, percent removal of EC at the Tracey Street pond ranges from a high of 79% down to 69% at a T_{90} of 48 hours. Numbers of exceedances drop from eight per season down to as low as six.

Even at the highest decay rate of 24 hours, it is shown that it is unlikely that the four exceedances per season criterion can be met for bacteria, with the use of wet retention ponds alone. Therefore, as discussed in Sections 5 and 7, the use of effluent disinfection will have to be considered.



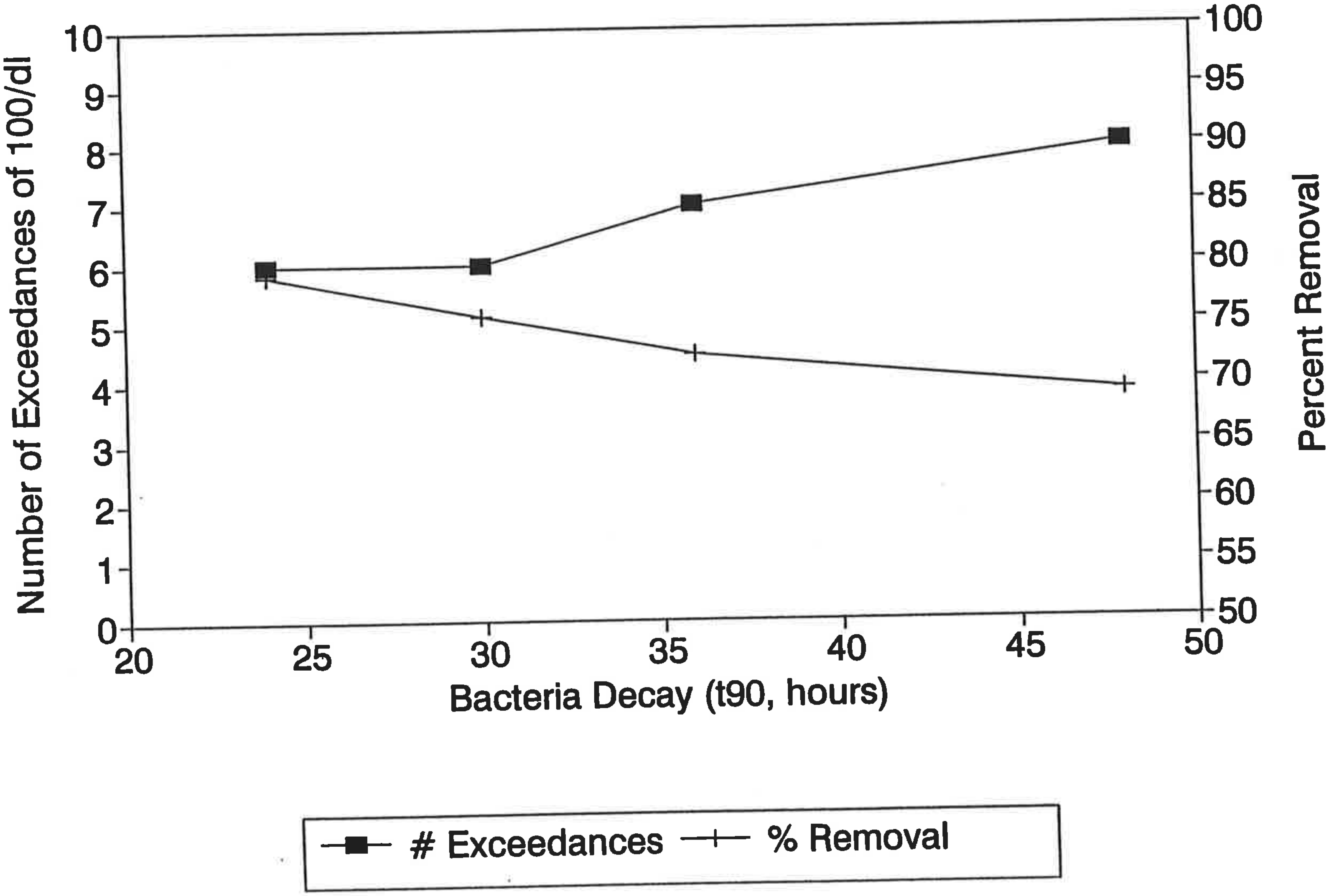
MOIRA RIVER CONSERVATION AUTHORITY

SENSITIVITY OF POND PERFORMANCE TO T_{90} WETLAND AT 401

Gore & Storrie Limited G&S
Consulting Engineers

FILE Nº \7644\DWGS\FIGF-1

FIG Nº F-1



MOIRA RIVER CONSERVATION AUTHORITY

SENSITIVITY OF POND PERFORMANCE TO T_{90} POND AT TRACEY STREET

Gore & Storrie Limited G&S
Consulting Engineers

FILE Nº \7644\DWGS\FIGF-2

FIG Nº F-2

APPENDIX G

COMPARATIVE COST ANALYSIS

COMPARATIVE COST ANALYSIS

G.1 *ESTIMATION OF SYSTEM OPTION COSTS*

Costs were estimated for all of the five System Options under review. For each System Option, preliminary cost estimates have included construction costs of end-of-pipe stormwater management facilities and disinfection facilities. Costing of the engineered conveyance channels was not performed since the cost difference for these channels between options would have a negligible impact on the comparisons.

G.1.1 *Costing for Stormwater Ponds*

Pond costs were estimated with assistance from the "Stormwater Management Practices Planning and Design Manual" (MOEE, 1994). Cost estimates that were included in the total facility costs were as follows:

- Land Acquisition
  - estimated at $350,000/ha in Thurlow Township (based on the cost of land acquired by Wal-Mart Canada in 1993, and other recent acquisitions)
  - estimated at $875,000/ha in the City of Belleville (based on the cost of land acquired by Canadian Tire Corporation in 1993, and other recent acquisitions)
  - land area required assumed to be the facility itself, including a 3 m buffer on either side, 13 m of erosion protection on either end, and a 15 m by 15 m area for each disinfection facility
  - resulting estimates of required land areas are summarized in Table G-1
- Excavation
  - estimated at $8/m^3 using in-house data and recent experience
  - volume estimates are summarized in Table G-1
- Landscaping
  - assumed to be $6/m^2 using in-house data and recent experience
  - estimated based on areas above approximately the 2-year flood level
- Outlet Structure
  - estimated using in-house data
  - estimated at $10,000 for the facilities upstream of Hwy. 401 and Tracey Street
  - estimated at $8,000 for the small dry pond at Cloverleaf (for System Options 1 and 3)
- Erosion Protection
  - assumed that 100 m^2 of rip-rap would be necessary at both the inlet and the outlet of each facility
  - estimated at $50/m^2 assuming rip-rap and geotextile treatment required
- Contingency
  - assumed to include engineering, design, approvals, tendering, mobilization, and demobilization costs
  - estimated to be 20% from engineering experience

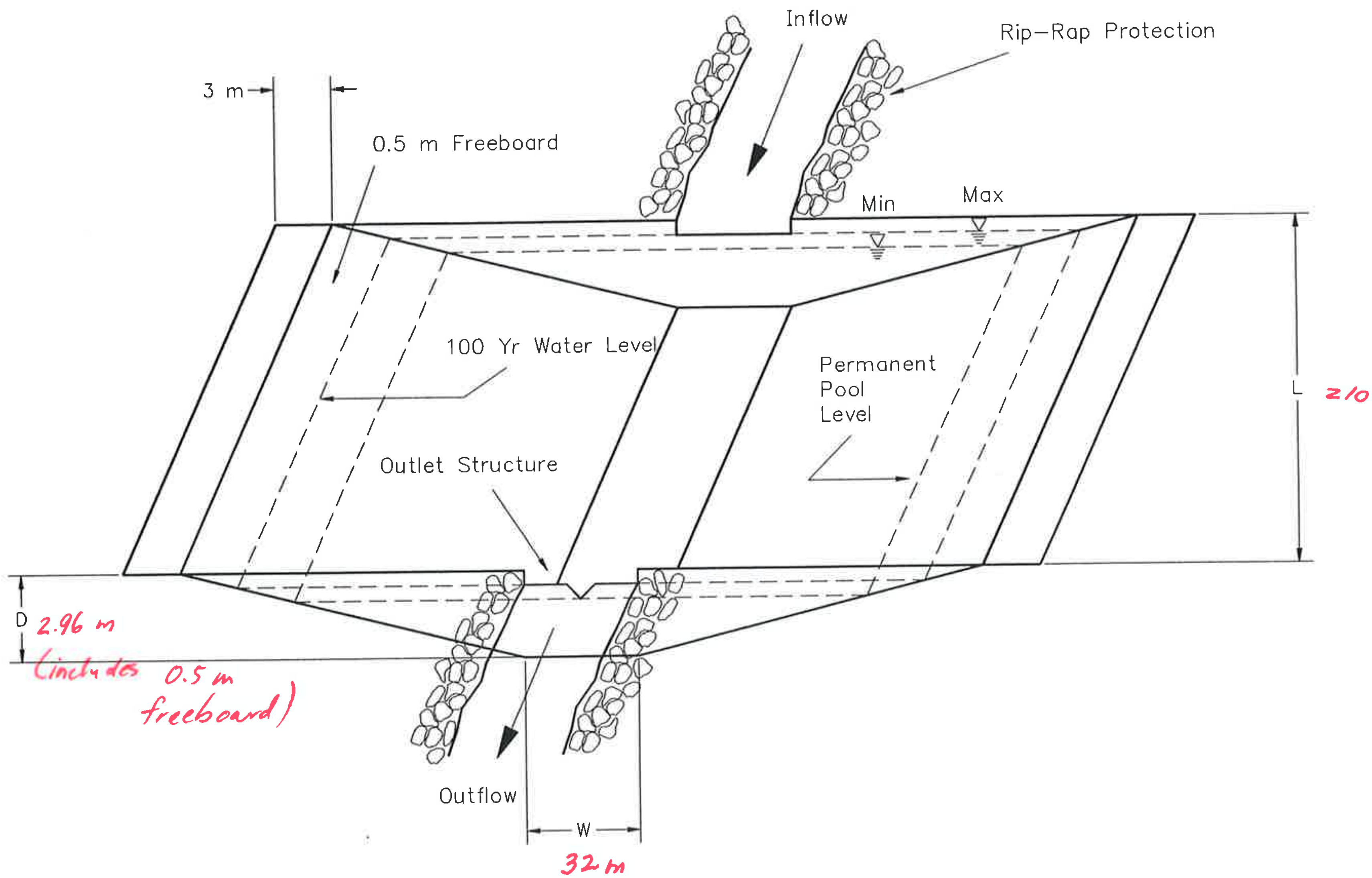
Table G-1
EXCAVATION VOLUME & LAND AREA REQUIREMENTS

		SYSTEM OPTION				
POND U/S OF		**1**	**2**	**3**	**4**	**5**
Cloverleaf Drive	L* (m)	0	0	150	142	0
	D* (m)	0	0	2.00	2.00	0
	W* (m)	0	0	20	20	0
	V (m^3)	0	0	7800	7384	0
	LA (ha)	0	0	0.6	0.6	0
Hwy. 401	L (m)	360	360	360	360	360
	D (m)	2.50	1.90	2.40	1.90	1.90
	W (m)	50	50	50	50	50
	V (m^3)	54000	39400	51500	39400	39400
	LA (ha)	2.7	2.6	2.7	2.6	2.6
Tracey Street	L (m)	210	210	210	210	210
	D (m)	3.88	2.96	3.82	2.93	2.91
	W (m)	32	32	32	32	32
	V (m^3)	35900	25700	35200	25300	25100
	LA (ha)	1.3	1.2	1.3	1.2	1.2

* L: total length of facility basin (m)
D: total depth of facility basin (including 0.5 m freeboard) (m)
W: bottom width of facility basin (m)
V: volume (m^3) of excavation
LA: land area (ha) required
see Figure G-1 for a general facility schematic

G.1.2 *Costing of UV Facilities*

Capital cost estimates for construction of ultraviolet disinfection facilities proposed for the Tracey Street stormwater management facility (and for the facility upstream of Highway 401 for System Option 5) are included in the costing tables in section G-2 of this appendix. Annual operation and maintenance costs are not included in these estimates, but have been estimated and are included in Table 6-2.



MOIRA RIVER CONSERVATION AUTHORITY

General Facility Schematic

Gore & Storrie Limited G&S
Consulting Engineers

FILE Nº	\7644\DWGS\FIGG-1	FIG Nº	G–1

The estimated costs for the UV units have been based on considering the costs of:

- The service building needed for housing the UV flow channels, UV disinfection equipment, electrical power supplies and system control equipment
- The UV equipment, including lamp units, water level controls, etc.
- The electrical and control equipment required by the UV system.

These costs are directly affected by the required UV treatment rate (*i.e.* the peak flowrate that the UV facility must be designed for). This design treatment rate has been estimated for each System Option using the following approach:

- The QUALHYMO system model was used to continuously simulate outflow from the Tracey Street pond, for the swimming season (June-September) of each of the six selected years identified in Appendix C. These six years represent a range of wet, dry and average rainfall years.

- For each System Option, pond outflow simulation was based on the active storage volume presented in Appendix F (Table F-1), this being the active storage needed to meet the 100-year outflow target of 3.5 m^3/s.

- The resulting six summers of pond outflow data (at 15-minute time steps) were then scanned to extract various statistics, including outflow duration statistics (*i.e.* how long various flow thresholds are exceeded) and outflow event statistics (*i.e.* on how many occasions various flow thresholds are exceeded).

- Outflow time series were also plotted and reviewed with respect to the various duration/exceedance statistics.

- Required UV treatment rates were then estimated by considering the flowrate exceeded 4 times per swimming season, and then applying adjustments to account for the fact that final design optimization is very likely to result in some reduction in the flowrate exceeded four times per season, based on optimizing the active storage volume and its operational hydraulics.

It is important to note that the active storage volumes at the Tracey Street pond, as tabulated in Table F-1, are those needed for 100-year flow control, and are not necessarily optimum in terms of minimizing UV treatment rate. The final design of the Tracey Street facility should be based on a hydraulic optimization analysis, in which optimum active storage volume and operational hydraulics (as defined by the detailed design of the pond's outlet structure) are selected to meet both the 100-year outflow constraint *and* minimize the UV design flow. This optimization analysis should include consideration of increasing active storage (by increasing active head within the pond and/or increasing pond area), along with various design options for the pond outlet control (possibly including automated overshot gates and/or batch-type operation). Any increases in pond cost must be weighed against savings in UV costs to arrive at the optimum design.

This type of design-level optimization analysis was beyond the scope of this study, particularly given that, to be meaningful, it would have to be carried out for all five System Options. Nonetheless, to account for the optimization potential and also provide a reasonable basis for cost comparisons, the simulated outflow duration/ exceedance statistics for the Tracey Street pond were interpreted and adjusted to account for the fact that optimization analysis will very likely result in some reduction in the required UV treatment. The adjustments were based on G&S design experience, such as a recent optimization analysis carried out for a large stormwater management pond being designed for the City of Gloucester (Region of Ottawa-Carleton).

From this procedure, it was concluded that the following UV treatment rates would be needed in order to achieve regulatory compliance:

Table G-2
ESTIMATED UV TREATMENT CAPACITY REQUIREMENTS

System Option 1	300 L/s
System Option 2	200 L/s
System Option 3	300 L/s
System Option 4	200 L/s
System Option 5	
• At Pond north of 401	80 L/s
• At Tracey Street pond	200 L/s

Once these UV design treatment rates had been established, it was possible to estimate capital costs and annual operation costs for the UV facilities. In recent years, G&S has obtained equipment supply costs for a range of treatment rates, and has estimated costs for the building structures needed to house UV systems.

The cost estimates for UV disinfection facilities were computed as follows:

- UV Equipment
  - estimated from data acquired by G&S
- Structure
  - estimated to be $200,000 for 80-100 L/s facility capacities, and $300,000 for 200-300 L/s facility capacities
  - estimates from recent experience
- Electrical and Control Equipment
  - estimated from data acquired by G&S

G.1.3 Costing for System Option 5 Diversion

For System Option 5, the cost of creating a diversion in Thurlow Township, from the facility upstream of Highway 401 east to the Moira River was also estimated so that it could be included in the costing tables. It is important to note that this option would require more land acquisition (some of it through the Ministry of Transportation land at the Highways 62/401 interchange as well as some through the Wal-Mart property adjacent to the interchange) and that an additional UV disinfection facility would be required. Based on a meeting with MTO Regional Office staff on 1994 November 22, it was determined that the MTO would require that the conduit be buried for almost the entire 750 m length due to liability concerns and future expansion of the interchange. In addition, the average depth to bedrock here is 1.7 metres, adding significantly to excavation costs. These additional costs contribute to the high total estimate for this option.

The additional costs of providing the diversion for System Option 5 were estimated using in-house data and engineering experience. An additional land acquisition of 0.8 ha (about 10 m by 800 m assumed) was added to the land area of the Highway 401 pond. A 1220 mm pipe was estimated to be required from the east end of the stormwater facility east to the Moira River. Lump sum costs were estimated for the inlet works ($10,000), ramp crossings ($60,000), and tunnelling under Highway 62 ($300,000). To supply and install the pipe a cost of $1,450/m was used, which takes into account the need for rock excavation.

G.2 COSTING TABLES

From the estimates and assumptions discussed in Section G-1 of this appendix, the following costing tables were prepared to permit a critical cost comparison between the five proposed system options. A summary table of total estimated costs is included in Table G-6.

Table G-3: COST COMPARISON BETWEEN SYSTEM OPTIONS 1 & 2

			System Option #1		System Option #2	
Item	Unit	Price/unit	Quantity	Total cost	Quantity	Total cost
POND ABOVE HWY. 401						
Land Acquisition	ha	$350,000	2.7	$959,000	2.6	$899,500
Excavation	m3	$8	54000	$432,000	39400	$315,200
Landscaping	m2	$6	8000	$48,000	5800	$34,800
Outlet structure	each	$10,000	1	$10,000	1	$10,000
Erosion protection	m2	$50	200	$10,000	200	$10,000
SUBTOTAL				$1,459,000		$1,269,500
POND ABOVE TRACEY ST.						
Land Acquisition	ha	$875,000	1.3	$1,128,750	1.2	$1,032,500
Excavation	m3	$8	36000	$288,000	25700	$205,600
Landscaping	m2	$6	3700	$22,200	2900	$17,400
Outlet structure	each	$10,000	1	$10,000	1	$10,000
Erosion protection	m2	$50	200	$10,000	200	$10,000
SUBTOTAL				$1,458,950		$1,275,500
U.V. DISINFECTION FACILITY	(300 L/s for S. O. #1) (200 L/s for S. O. #2)					
UV equipment				$230,000		$150,000
Structure				$300,000		$300,000
Electrical & control				$175,000		$130,000
SUBTOTAL				$705,000		$580,000
GRAND SUBTOTAL				$3,622,950		$3,125,000
Contingency, mobilization, demobilization (20 %)				$724,590		$625,000
TOTAL ESTIMATE (rounded)				**$4,348,000**		**$3,750,000**

580,000
390,000

40%
60%

40

Table G-4: COST COMPARISON BETWEEN SYSTEM OPTIONS 3 & 4

Item	Unit	Price/unit	System Option #3		System Option #4	
			Quantity	Total cost	Quantity	Total cost
POND ABOVE CLOVERLEAF DR.						
Land Acquisition	ha	$350,000	0.6	$203,000	0.6	$203,000
Excavation	m3	$8	7800	$62,400	7400	$59,200
Landscaping	m2	$6	2900	$17,400	2800	$16,800
Outlet structure	each	$8,000	1	$8,000	1	$8,000
Erosion protection	m2	$50	200	$10,000	200	$10,000
SUBTOTAL				$300,800		$297,000
POND ABOVE HWY. 401						
Land Acquisition	ha	$350,000	2.7	$952,000	2.6	$899,500
Excavation	m3	$8	51500	$412,000	39400	$315,200
Landscaping	m2	$6	6700	$40,200	5800	$34,800
Outlet structure	each	$10,000	1	$10,000	1	$10,000
Erosion protection	m2	$50	200	$10,000	200	$10,000
SUBTOTAL				$1,424,200		$1,269,500
POND ABOVE TRACEY ST.						
Land Acquisition	ha	$875,000	1.3	$1,128,750	1.2	$1,023,750
Excavation	m3	$8	35200	$281,600	25300	$202,400
Landscaping	m2	$6	3700	$22,200	2900	$17,400
Outlet structure	each	$10,000	1	$10,000	1	$10,000
Erosion protection	m2	$50	200	$10,000	200	$10,000
SUBTOTAL				$1,452,550		$1,263,550
U.V. DISINFECTION FACILITY	(300 L/s for S. O. #3) (200 L/s for S. O. #4)					
UV equipment				$230,000		$150,000
Structure				$300,000		$300,000
Electrical & control				$175,000		$130,000
SUBTOTAL				$705,000		$580,000
GRAND SUBTOTAL				$3,882,550		$3,410,050
Contingency, mobilization, demobilization (20 %)				$776,510		$682,010
TOTAL ESTIMATE (rounded)				**$4,659,000**		**$4,092,000**

Table G-5: COST ESTIMATES FOR SYSTEM OPTION 5

Item	Unit	Price/unit	System Option #5	
			Quantity	Total cost
POND ABOVE HWY. 401				
Land Acquisition	ha	$350,000	3.4	$1,190,000
Excavation	m3	$8	39400	$315,200
Landscaping	m2	$6	5800	$34,800
Outlet structure	each	$10,000	1	$10,000
Erosion protection	m2	$50	200	$10,000
SUBTOTAL				$1,560,000
1220 mm PIPE TO MOIRA RIVER				
Inlet works	l.s.	$10,000		$10,000
Ramp crossings	l.s.	$60,000		$60,000
Tunnelling under Hwy. 62	l.s.	$300,000		$300,000
Supply & install pipe	m	$1,450	750	$1,087,500
SUBTOTAL				$1,457,500
U.V. DISINFECTION FACILITY #1	(80 L/s @ Hwy. 401)			
UV equipment				$100,000
Structure				$200,000
Electrical & control				$90,000
SUBTOTAL				$390,000
POND ABOVE TRACEY ST.				
Land Acquisition	ha	$875,000	1.2	$1,023,750
Excavation	m3	$8	25100	$200,800
Landscaping	m2	$6	2800	$16,800
Outlet structure	each	$10,000	1	$10,000
Erosion protection	m2	$50	200	$10,000
SUBTOTAL				$1,261,350
U.V. DISINFECTION FACILITY #2	(200 L/s @ Tracey St.)			
UV equipment				$150,000
Structure				$300,000
Electrical & control				$130,000
SUBTOTAL				$580,000
GRAND SUBTOTAL				$5,248,850
Contingency, mobilization, demobilization (20 %)				$1,049,770
TOTAL ESTIMATE (rounded)				$6,299,000

Table G-6
SUMMARY OF CONSTRUCTION COST ESTIMATES FOR THE FIVE SYSTEM OPTIONS

SYSTEM OPTION	ESTIMATED CONSTRUCTION COST
1 Two ponds, no site-level controls, and UV at Tracey Street	$ 4,348,000
2 Two ponds, site-level controls and UV at Tracey Street	$ 3,750,000
3 Three ponds, no site-level controls, and UV at Tracey Street	$ 4,659,000
4 Three ponds, site-level controls and UV at Tracey Street	$ 4,092,000
5 Two ponds, site-level controls, UV units at Tracey Street and north of 401, and diversion to Moira River	$ 6,299,000

APPENDIX H

REVIEW OF METHODS FOR BACTERIA CONTROL

REVIEW OF METHODS FOR BACTERIA CONTROL

The following sections will briefly discuss six potential disinfection options for stormwater management applications. The applicability and costs of these alternative disinfection techniques will be compared and discussed in the last section.

H.1 DISINFECTION TECHNIQUES

H.1.1 Chlorination

Chlorination is the most used disinfectant at water and wastewater treatment plants. Chlorine reacts very rapidly when mixed with water, and both hydrolysis and ionization occur. The efficiency of chlorine is influenced by a number of environmental factors such as pH, temperature, alkalinity, suspended solids, COD and nitrogen.

Chlorine is known to react rapidly with ammonia and certain organic compounds to form chloramines and chlorinated organic compounds. Chlorine induced toxicity, via toxic levels of chlorine residuals and formation of potentially toxic halogenated organic compounds, is difficult to control. Chlorine residuals have been found to be acutely toxic to some species of fish at very low levels. Other toxic or carcinogenic chlorinated compounds can bioaccumulate in aquatic life.

Chlorine is an extremely volatile, corrosive and hazardous chemical and proper safety precautions must be exercised during shipment, storage and use.

The primary advantage of hypochlorination over chlorination is the increased safety in transportation, storage and handling of the chemicals. However, chemical costs per unit of free chlorine are generally much higher.

Dechlorination after a chlorination process is usually achieved with sulphur dioxide or sulphite reducing compounds or activated carbon. This process reduces the environmental impacts associated with chlorination or hypochlorination by decreasing chlorine residuals below toxic levels. The potential for formation of halogenated organics may be reduced, however, it appears that many halogenated organics are formed rapidly and that dechlorination probably does not affect these compounds.

The total cost of chlorination systems will be increased by 30 to 50% with the addition of a dechlorination process.

H.1.2 Chlorine Dioxide

Chlorine dioxide has been used to disinfect potable waters. It is a proven bactericide which has a higher oxidation potential than chlorine. It has also been indicated to be a much more effective virucide than chlorine. Chlorine dioxide does not react with ammonia and has a lower potential of forming halogenated organic compounds.

Chlorine dioxide is a very unstable and explosive gas and any means of transport is extremely hazardous. Therefore, it needs to be generated on site using sodium chlorite and chlorine. There are very few known applications due to its on-site generation requirements and high chemical costs.

H.1.3 Bromine Chloride

Bromine chloride hydrolyses rapidly to form bromamines. Since they are more unstable than chloramines, bromamines are very effective as a disinfectant and have shorter lived residuals compared to chloramines.

Environmental impacts associated with bromine chlorides disinfection are less than those associated with chlorine. However, brominated organics such as bromoform and mixtures of chlorinated and brominated organics can be expected to be formed.

Bromine chloride disinfection facilities are similar to chlorination facilities. Shorter contact times are possible resulting in potential construction cost savings due to a smaller contactor. Bromine chloride is a hazardous and corrosive chemical which essentially requires the same safety and handling procedures as chlorine.

H.1.4 Ozone

Ozone is an extremely reactive oxidant and a very effective bactericide and virucide. Unlike other disinfectant agents previously discussed, ozone can exert beneficial impacts on the environment since it dissociates rapidly to form oxygen. This oxygen can increase dissolved oxygen levels often to the point of saturation.

Ozone residuals can be acutely toxic to aquatic life; however, due to its instability, residuals are short lived and normally not found in the effluent. Ozone is believed to present fewer potential environmental and health hazards than chlorine.

Since ozone is extremely unstable, it must be generated on site. Ozonation systems are considered to be relatively expensive and complex to operate compared to chlorine processes. Operating costs may also be restrictive depending on power cost as these systems are power intensive.

H.1.5 Ultraviolet Light

The effectiveness of ultraviolet light as a bactericide and virucide has been well established. Ultraviolet disinfection is a simple physical process which has a negligible likelihood of producing harmful chemical residuals. The disinfection process by UV radiation relies on the transference of electromagnetic energy from a source to an organism's cellular material. The lethal effects of this energy results primarily from the cell's inability to replicate. The major advantages of ultraviolet disinfection are its simplicity, lack of environmental impacts as well as minimal space requirements.

Some environmental factors such as high suspended sediment concentrations, colour and turbidity in the wastewater may absorb the ultraviolet light and reduce the disinfection performance.

Total cost appear to be competitive with chlorination. The major operating costs are the power requirements and the replacement/maintenance of the ultraviolet lamps.

H.1.6 Gamma Radiation

Gamma ray disinfection is achieved by inactivating microorganisms in much the same way as UV by causing alterations in the genetic material. Gamma rays are emitted from radioactive isotopes such as ^{60}CO. Gamma radiation generally does not appeal to the public since it involves the use of radioactive isotopes. One key advantage is that the disinfection process does not require a power source unlike UV and ozonation. There appears to be little experience of water disinfection using gamma radiation although procedures were tested and proposed for wide scale applications in the food processing industry before public reaction caused cancellation of the program.

H.2 COMPARATIVE EVALUATION OF STORMWATER DISINFECTION OPTIONS

In conducting a review of various disinfection options, one must keep in mind a number of factors which greatly influences the decision making process. These factors relate to the disinfection process itself and to the potential adverse impacts it may have on the receiving water and the environment. They may be summarized as follows:

Effectiveness - The effectiveness may be described as the ability to achieve target objectives gauged by predetermined levels of indicator organisms. For efficiency, an option must be reasonably reliable while possessing a broad spectrum of disinfecting ability.

Use-Cost - This factor basically reflects various costs such as the initial capital cost, amortization cost, operation and maintenance cost and cost for stormwater pre-treatment and/or post-treatment.

Practicality - Need and ease of transport and storage or on-site generation of disinfection products. This also entails the relative ease/complexity of application and control as well as special safety considerations. Will there be a need for pre- and/or post-treatments? The flexibility of the disinfection alternatives and the ability to predict results may also be used as evaluation factors.

Potential adverse effects - Will there be adverse effects on aquatic life? Is there formation and transmission of undesirable bio-accumulating substances or maybe toxic, mutagenic or carcinogenic substances.

Pilot studies required - Is there a need to conduct pilot studies to determine dose requirements and/or to refine design details.

Table H-1
COMPARISON OF CAPITAL AND OPERATING COSTS OF DISINFECTION OPTIONS (EPA, 1986)

DISINFECTION MODE	Plant size (10^3 m^3/d)			
	3.78	18.9	37.8	378
CAPITAL COST RATIOS				
Chlorination	2.01	1.76	1.45	0.86
Dechlorination	0.68	0.48	0.36	0.14
Chlorination-dechlorination	2.69	2.24	1.81	1.00
Ozonation by air	9.98	7.46	7.27	6.96
Ultraviolet (Trojan UV2000)	1	1	1	1
OPERATION COST RATIOS				
Chlorination	2.77	2.46	2.11	1.47
Dechlorination	1.19	0.90	0.72	0.41
Chlorination-dechlorination	3.96	3.36	2.84	1.88
Ozonation by air	8.67	6.87	6.73	7.05
Ultraviolet (Trojan UV2000)	1	1	1	1

Table H-2: Applicability of Alternative Disinfection Techniques (EPA, 1986b)

Consideration	Cl_2	Cl/deCl_2	BrCl	ClO_2	O_3	UV
Size of plant	all sizes	all sizes	all sizes	small/ medium	medium/ large	small/ medium
Applicable level of treatment prior to disinfection	all levels	all levels	secondary	secondary	secondary	secondary
Equipment reliability	good	fair to good	?	?	fair/good	fair/good
Process control	well developed	fairly well developed	problematic	no experience	developing	developing
Relative complexity of technology	simple/ moderate	moderate	moderate	moderate	complex	simple/ moderate
Safety concerns Transportation on site	yes substantial	yes substantial	yes substantial	yes substantial	no moderate	no minimal
Bactericidal	good	good	good	good	good	good
Virucidal	poor	poor	fair/good	good	good	good
Fish toxicity	toxic	non-toxic	slight/ moderate	toxic	none expected	non-toxic
Hazardous by-products	yes	yes	yes	yes	none expected	no
Persistent residual	long	none	short	moderate	none	none
Contact time	long	long	moderate	moderate/ long	moderate	short
Contributes dissolved oxygen	no	no	no	no	yes	no
Reacts with ammonia	yes	yes	yes	no	yes (high pH)	no
Colour removal	moderate	moderate	?	yes	yes	no
Increased dissolved solids	yes	yes	yes	yes	no	no
pH dependent	yes	yes	yes	no	slight (high pH)	no
O&M sensitive	minimal	moderate	moderate	?	high	moderate
Corrosive	yes	yes	yes	yes	yes	no

APPENDIX I

OPEN HOUSE ATTENDEES

MOIRA RIVER CONSERVATION AUTHORITY

UPPER NO-NAME CREEK
WATER MANAGEMENT PLAN
OPEN HOUSE
December 7, 1994

ATTENDANCE SHEET

Name	Address	Tel.#
P. John Halloran	R.R. 5, Belleville	969-0000 966-5212
Michael Guerrera	R.R. 5, Belleville	962-1272
C.L. Donald	R.R. 5, Belleville, 31 Cloverleaf Dr.	968-3101
A.H. Simpson	R.R. 5, 33 Cloverleaf Dr.	968-8826
Gary Dyke	Van Meer Ltd.	969-0171
Carl Cannon	Sidney Twp.	966-8330
Karen Poste	Thurlow Twp.	968-5553
Gerry Masterson	Thurlow Twp.	968-5553
Rick Tait	Greer Galloway Group Inc.	966-3068
Ralph Swan	R.R. 5, Belleville	962-5721
Stewart Murray	Planning Dept., City of Belleville	
Carolyn Ross	Planning Dept., City of Belleville	968-6481
Susan Deans	Loyalist College	969-1913 ext.295
Gillian Dagg-Foster	Gore & Storrie, 857 Norwest Rd. Kingston	384-0913
David Clark	2255B Queen St. E., Suite 342, Toronto	(416)691-2735
Glenn Walker	City of Belleville, Economic Development	967-3273
Bill Usborne	Box 21, Foxboro, Ont. K0K 2B0	962-8056
Walter Watzon	218 Church St. Belleville	962-9521
Harold Chard	Gore & Storrie Limited, 857 Norwest Rd. Kingston	384-0913