TOWN OF QUALICUM BEACH

STORM DRAINAGE STUDY
YAMBURY BASIN

JANUARY 1993

KOERS
& ASSOCIATES
ENGINEERING LTD.
Consulting Engineers

PARKSVILLE, B.C.

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January, 29, 1993
File No.: M9133-04

Town of Qualicum Beach
140 West Second Avenue,
P.O. Box 130,
Qualicum Beach, B.C., V0R 2T0

ATTENTION: **Mr. M. Brown,**
Director of Finance and Administration

Dear Sirs:

Re: **Town of Qualicum Beach - Storm Drainage Study**
Yambury Basin

We are pleased to present 8 copies of the Yambury basin component of our overall drainage report for the Town of Qualicum Beach. The outlet for this basin has come to be within the Town's boundaries as a result of recent expansion, and threatens to be problematic in the future if improvements to this outlet are not made.

The Town requires these improvements to be constructed. This separate report is necessary to support the application to the Ministry of Municipal Affairs, Recreation and Culture to obtain the authorization to collect the required funds from the benefitting lands, most of which lie outside of the Town's boundaries.

Yours truly,

KOERS & ASSOCIATES ENGINEERING LTD.

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA D. A. KOERS

PROFESSIONAL ENGINEER PROVINCE OF BRITISH COLUMBIA R. K. WEIR

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DAK/RKW/rkw

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STORM DRAINAGE STUDY

YAMBURY BASIN

1.0 INTRODUCTION

This report forms part of an overall Storm Drainage Study undertaken for the Town of Qualicum Beach by Koers & Associates Engineering Ltd., as authorized in a letter of 18 November, 1991. The report for the Yambury Road basin is presented as a separate letter report owing to the combined interest of the Town and others, in the solution of the drainage problems of this basin.

2.0 SCOPE

The subject area is the drainage basin that makes its outlet at Yambury Road. The area encompasses the easternmost regions of the Town of Qualicum Beach, and a considerable area east of the Town presently unincorporated, and within the Regional District of Nanaimo (RDN). The area has a favourable location and topography and is currently experiencing extensive residential development. Figure 1 outlines the basin and the main or trunk drainage network.

The approval authority for this unincorporated area is the Ministry of Transportation and Highways (MOTH), and in fact most of the area in the basin has been developed under their authority. Only recently has the lower basin come to be in the Town of Qualicum Beach as a result of the 1991 boundary expansion. This expansion came as a result of the petitioning by the residents of the Eaglecrest residential development, and subsequent referendum, to have the area join the Town of Qualicum Beach.

3.0 PREDEVELOPMENT BEHAVIOUR OF THE BASIN.

The Yambury Basin is a drainage basin of 579 hectares (ha) in and to the east of the Town of Qualicum Beach. The basin can be subdivided for convenience into an upper basin of 458.6 ha (79.2%) lying south of the Island Highway, and a lower basin of 121.4 ha (20.8%) north of the Highway. The lower basin is now entirely within the Town of Qualicum Beach boundaries while a major portion, 370 ha, of the upper basin lies outside of the Town boundaries. In the upper basin, of the 88.6 ha within the Town boundaries, 77.8 ha lie within the Agricultural Land Reserve (ALR).

The Yambury basin rises from the Straight of Georgia in the north, sloping gently to the higher lands in the south where the topography crests over into the French Creek drainage basin. A narrow beach terrace exists adjacent to the Straight of Georgia behind which steep bluffs rise to an elevation of approximately 20 metres. From this point the rise is gradual to the uppermost basin at an elevation of 80 metres. The basin has a slight inclination to the east.

The basin is drained by a poorly evolved network of open ditching that converges at Yambury Road and flows north to the Straight of Georgia. This last section, known as the Yambury ditch, is a predominantly open channel of substantial dimensions located along the western edge of the Yambury Road right of way. Numerous 1200 mm diameter driveway culverts and ditch enclosures exist. A 1400 mm diameter culvert at a grade of over 60% transports the flow over a steep bank to the ocean terrace.

The Yambury ditch has not always been the outlet for the discharge from the upper basin. Following its slight inclination to the east, the upper basin historically drained to a small creek that crosses the island highway east of Yambury and flows to the ocean through what is now the Pebble Beach residential development east of Johnstone Road. From reviewing MOTH files for development in the area, it appears that problems in the lower reaches of this small creek began to become apparent at the time of development of the Sandpiper residential development to the south of the Island Highway, east of Yambury.

A diversion of the drainage from the upper basin west of Yambury was conceived to alleviate this problem in the lower reaches of the small creek, thus apportioning the available capacity in the creek to the increases that were to occur as a result of the Sandpiper development.

This diversion was accomplished by a culvert crossing of the Island Highway at Yambury Road and by the direction of the drainage from the upper basin to the roadside ditch on Yambury. The construction of the Yambury ditch to its present dimensions was made a condition of subdivision approval for the Eaglecrest residential development. The size of the conduits and channels was determined by a drainage study evidently conducted by the MOTH, but which could not be located in MOTH files.

The upper basin at the time was little developed. It contained large areas that were very flat, forested and with thick underbrush, particularly in DL 88 and the north eastern portion of DL 78. There did not exist well defined channels across these lands and they were punctuated by standing water and seasonal overland sheet flow on the forest floor during times of heavy precipitation.

Although the area was extensive, the natural storage and flow attenuation of the landform and cover, produced a peak runoff behaviour that was within the capacity of the Yambury ditch.

4.0 DEVELOPMENT IN THE UPPER BASIN

A portion of the upper basin is within the RDN Sewer Specified Area, and therefore has the potential for dense residential development. The area so specified is the portion of DL 88 north of the E & N Railway, and a fractional portion of northeastern DL 78 adjacent to the Town boundary. The balance of the area is either within the ALR or has a rural zoning allowing only large parcel subdivision.

At the time of application for development within DL 88, the MOTH recognized the limited capacity of the Yambury ditch and the requirement to make drainage improvements to accommodate the development. In its letter of 16 June, 1986, in response to a request for information on the drainage in the area they advised, in part, "As the area becomes more fully developed the run-off will increase beyond the capacity of the present ditch, and at such time it will be necessary to upgrade the drainage system." The Ministry went on to offer the following as a suggestion, "The upgrading could be achieved by several methods; the most favourable seems to be by diverting all the flow from the area south of the E & N Railway tracks directly into French Creek via a ditch parallel to the railway right-of-way."

The development of DL 88 was undertaken without any improvement to the downstream drainage system nor were any diversions of the upper basin pursued. At the time of the Town of Qualicum Beach boundary expansion, DL 88 was approximately 25% developed, and by inspection it could be determined that the Yambury ditch was flowing at capacity during rainfall events of less significance than 5 year recurrence interval statistics.

5.0 DEVELOPMENT IN THE LOWER BASIN

The lower basin is largely comprised of the Eaglecrest Golf Course and residential development. However, at the time of the boundary expansion, the

Oceanside development, to the east of Yambury, was nearing completion of its first phase. The significance of this development is that it was adding area and drainage previously not tributary to the basin, to the Yambury ditch. Although the drainage is introduced into the steep culvert to the beach terrace which does have substantial capacity, the open channel at the culvert outlet flowing across the terrace, does not.

It appears, and has been confirmed by long term residents in the area, that the lower basin historically had several small drainage channels over the bluffs. Threatened by the potential for erosion and associated failure of the bluffs from the increased runoff that would result from clearing and development, and avoiding the costs of constructing additional conduits to the beach, it is likely that the developers were motivated to intercept these channels and direct them east to Yambury. Thus it would seem that the existence of the culvert to the beach terrace has fostered the diversion of runoff to effectively increase the area of the drainage basin.

6.0 STUDY NEED

The Town of Qualicum Beach had identified the need to undertake a comprehensive review of the drainage system within the Town in order to accommodate and plan for future development and the upgrading of developed areas to acceptable standards. In particular, it was immediately apparent that the conditions within the Yambury basin were unacceptable. The problem in the Yambury basin was further complicated by the Town's lack of authority over the developing part of the upper basin. However, in order to gain the necessary understanding, the study of the entire basin was required since the burden of conveying the entire runoff to the sea was borne by the Yambury ditch, now within the Town boundaries.

7.0 PREVIOUS DRAINAGE STUDIES

Although references exist in the files of the MOTH to a previous study of the basin, conducted by the MOTH, the actual study was not located.

Several partial studies of the upper basin have been conducted by Engineers on behalf of developers. These studies typically assume the diversion, along the E & N Railway right of way to French Creek, of large portions of the upper basin and in so doing, allocate the available capacity of the Yambury Road ditch to the increased runoff resulting from the development in question. In one study all upstream areas except DL 88 are ignored from the analysis and assumed to be diverted elsewhere by others.

It would seem that it was the intention of the MOTH, from the earliest correspondence, to indicate the requirement to upgrade structures or otherwise provide for increased runoff resulting from development. However, the suggestion made by MOTH seems to have been interpreted by the developers and their Engineers to mean that the diversion will be done by MOTH or others.

8.0 ANALYSIS OF THE BASIN

An analysis of the basin was conducted to estimate the runoff from precipitation events, in order to determine the structures required to transport the flow. For the purposes of this study, the analysis of the basin has been conducted with a microcomputer simulation program called MIDUSS (Microcomputer Interactive Design of Urban Stormwater Systems). The program was developed at McMaster University and combines a graphical user interface with accepted hydrologic analysis routines. A technical specification of the software is included in the Appendices. MIDUSS has gained widespread use in the Municipalities of the lower mainland and is one of two computer programs recommended by the District of Surrey for use in the planning and design of drainage systems.

Like other computer simulation programs, the computational abilities of the computer are exploited to conduct hydrologic and hydraulic calculations sufficient to model the actual response of the basin over time to simulated rainfall events. Hydrologic models are built into the program. Rainfall options include Chicago, Huff, Canadian 1 hr., customized mass rainfall distribution, and customized historic storm. Infiltration can be estimated by SCS Curve No. & Runoff Coefficient, Green-Ampt, or Horton. Overland flow can be calculated as Triangular IUH, Rectangular IUH, SWMM Runoff or Single Linear Reservoir.

With the advent of the microcomputer there has been an increasing movement away from the empirical and limited analysis by the Rational Method. Although it has the positive feature of being apparently simple and is one of the most frequently used urban hydrology methods, the Rational Method is often applied incorrectly, particularly in large basins.

9.0 RAINFALL DATA

Following the recommendations of W.D. Hogg of the Atmospheric Environment Service (AES) in the publication "Distribution of Rainfall With Time: Design Considerations", a Huff 2nd Quartile distribution was selected as

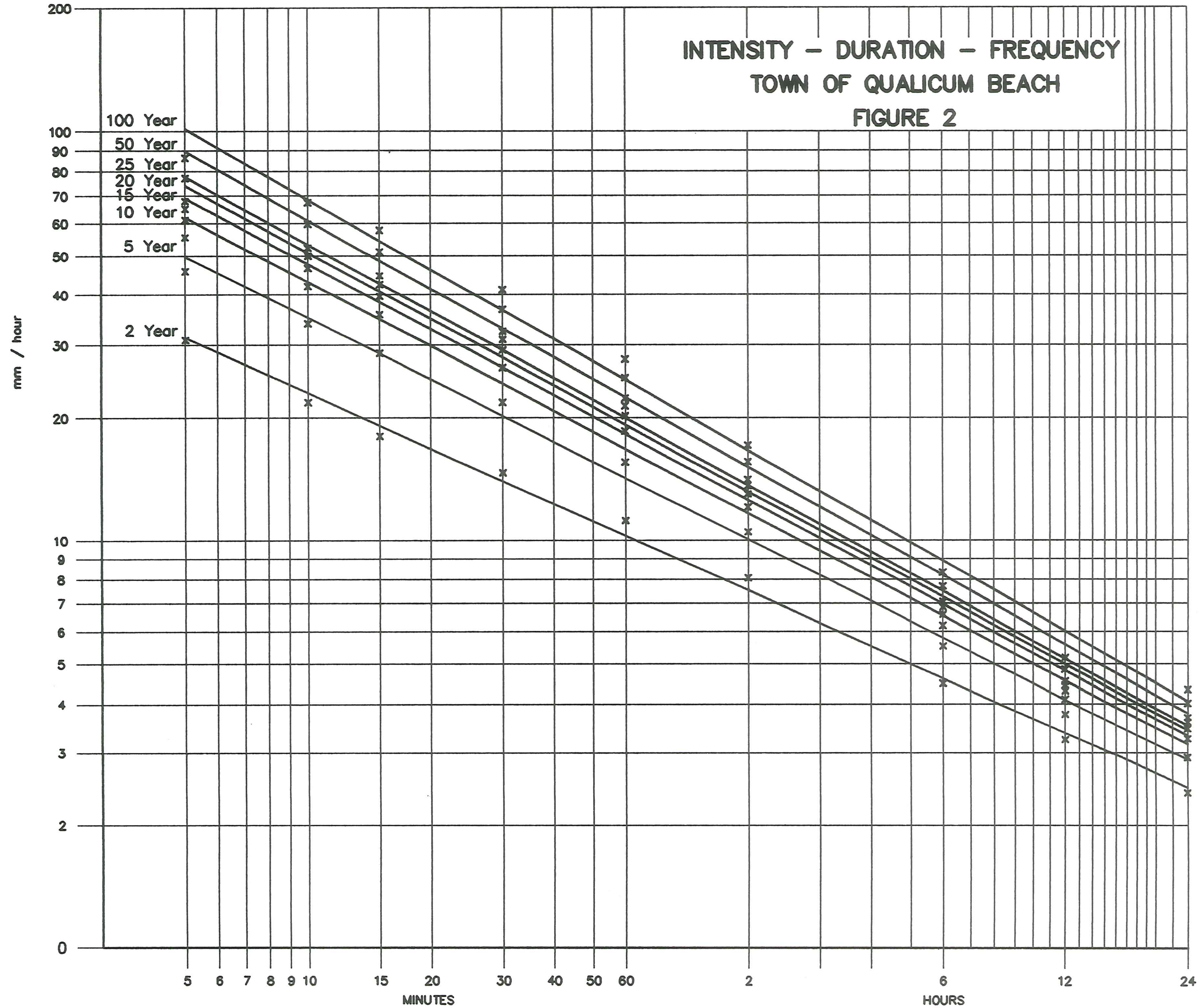
TABLE 1 : TOWN OF QUALICUM BEACH – IDF – TABULATED

AS TOTAL AMOUNTS DURING PERIOD

DURATION	X	S	2YR	5YR	10YR	15YR	20YR	25YR	50YR	100YR
24 HR	60.000	14.000	57.704	70.066	78.270	82.890	86.124	88.616	96.288	103.918
12 HR	40.000	7.000	38.852	45.033	49.135	51.445	53.062	54.308	58.144	61.959
6 HR	28.000	7.000	26.852	33.033	37.135	39.445	41.062	42.308	46.144	49.959
2 HR	17.000	5.500	16.098	20.955	24.178	25.993	27.263	28.242	31.256	34.254
1 HR	12.000	5.000	11.180	15.595	18.525	20.175	21.330	22.220	24.960	27.685
30 MIN	8.000	4.000	7.344	10.876	13.220	14.540	15.464	16.176	18.368	20.548
15 MIN	5.000	3.000	4.508	7.157	8.915	9.905	10.598	11.132	12.776	14.411
10 MIN	4.000	2.300	3.623	5.654	7.002	7.760	8.292	8.701	9.962	11.215
5 MIN	2.800	1.400	2.570	3.807	4.627	5.089	5.412	5.662	6.429	7.192

AS INTENSITIES

AS INTENSITIES	2YR	5YR	10YR	15YR	20YR	25YR	50YR	100YR
24 HR	2.404	2.919	3.261	3.454	3.589	3.692	4.012	4.330
12 HR	3.238	3.753	4.095	4.287	4.422	4.526	4.845	5.163
6 HR	4.475	5.506	6.189	6.574	6.844	7.051	7.691	8.327
2 HR	8.049	10.477	12.089	12.996	13.632	14.121	15.628	17.127
1 HR	11.180	15.595	18.525	20.175	21.330	22.220	24.960	27.685
30 MIN	14.688	21.752	26.440	29.080	30.928	32.352	36.736	41.096
15 MIN	18.032	28.628	35.660	39.620	42.392	44.528	51.104	57.644
10 MIN	21.737	33.922	42.009	46.563	49.751	52.207	59.770	67.291
5 MIN	30.845	45.679	55.524	61.068	64.949	67.939	77.146	86.302


INTENSITY – DURATION – FREQUENCY
TOWN OF QUALICUM BEACH
FIGURE 2
100 Year
50 Year
25 Year
20 Year
15 Year
10 Year
5 Year
2 Year
mm / hour
200
100
90
80
70
60
50
40
30
20
10
9
8
7
6
5
4
3
2
0
5
6
7
8
9
10
15
20
30
40
50
60
2
6
12
24
MINUTES
HOURS

being appropriate for the study area.

Intensity Duration Frequency (IDF) curves and tabulated information are not yet available for the weather station nearest the study area due to the insufficient period of record. The Town of Qualicum Beach provides a set of IDF curves in its Engineering Standards however, not knowing their origin, a set of IDF curves was produced as a check using the method suggested, and maps contained in the "Rainfall Frequency Atlas for Canada", also published by the Atmospheric Environment Service. The curves are reproduced in Fig.2. The tabulated information appears in Table 1. The family of curves produced from the AES material showed reasonable agreement to the curves presently contained in the Town of Qualicum Beach Standards

10.0 CATCHMENT DATA

The physical parameters of the basin are required as input to the computer program to facilitate the modelling of the basin. The basin was divided into logical catchments by considering their land use and natural physical delineation. The proportion of the catchment covered by impervious surfaces was determined by inspection of aerial photography, mapping, and by considering the zoning and future development of the area. The slope of the catchment was measured from 1 metre contour interval mapping at a scale of 1:2000. Typical overland flow distances were also estimated from this mapping. Catchment areas were determined by digitizing the areas into the computer and calculated by AutoCAD.

The Soil Conservation Service (SCS) Curve Number Method was chosen to estimate infiltration and runoff, due to the wide acceptance of the method. Impervious areas were assigned a Curve Number of 98. The assignment for pervious areas varied depending on their individual nature but typically ranged from 65 to 75 with 75 being most commonly assigned . Manning's "n" for overland flow on impervious areas was typically assigned as 0.015 to 0.017, and for pervious areas 0.2 to 0.4 with 0.25 being most commonly assigned.

The dimensions, materials and condition of the ditches, channels and pipes were determined by field reconnaissance and by inspection of record drawings. The grades of the ditches and channels were estimated from the contour mapping if better information was not available.

11.0 DISCUSSION OF RESULTS

The ultimate development of the basin as anticipated by current zoning, was assumed. The response of the basin to synthetic storm events having recurrence intervals of 10 and 100 years was then modelled by the computer simulation. These intervals correspond to the events for which the minor and major systems are commonly designed. Several storm durations were examined to determine the event that would produce the peak stormwater runoff. A storm duration of one hour was chosen as the design storm for the basin considered in its entirety.

It should be noted that rainfall events are a naturally occurring and highly variable phenomenon. Modelling rainfall and runoff is at best a statistical exercise undertaken to hopefully establish the efficient yet prudent application of resources for the management of stormwater runoff. Historic and future rainfall events are unlikely to match those assumed for analysis. The occurrence of an event that would cause greater runoff cannot be discounted. Events of shorter duration and higher intensity will produce greater localized runoff. In this report it is the peak runoff in the trunk system, and the Yambury ditch in particular, that we are attempting to estimate, given the summary statistics compiled to date.

In attempting to analyze the overall system, the level of discretization, or the degree to which the system is divided into distinct parts for modelling, must be limited. Thus the analysis of all minor channels and conduits in the system is impractical and beyond the scope of this investigation. However, it should be noted that much of the basin, and especially Eaglecrest, is served by open ditches with many and varied conduits and channels. Landscaping, ditch infilling and other local improvements have not always given due consideration to maintaining the capacity and integrity of the drainage system. The potential for localized flooding should be recognized throughout the basin.

Summaries of the computer analyses can be found in the Appendices.

11.1 10 YEAR RECURRENCE INTERVAL STORM

The results of the computer simulation indicate that under exposure to rainfall having a recurrence interval of 10 years, the runoff from the developed basin will exceed the present capacity of the Yambury ditch in its lower reaches, above and below the steep culvert to the beach. The required capacity in the Yambury ditch just north of the Island Highway is estimated to be 2.5 cubic metres/second (m^3/s). The required capacity at Pintail is 3.5 m^3/s and from the outfall of the steep culvert, 3.9 m^3/s.

The design capacity of the Yambury ditch as originally specified by MOTH was 2.12 m^3/s (75 cfs). The present capacity as determined by inspection of the installed driveway and ditch infill culverts is estimated to be approximately 1.55 m^3/s, beyond which overtopping onto private property would occur.

The drainage channel flowing through the Eaglecrest development that receives runoff from the upper basin through the culvert under the Highway between Bennett Rd. and Chartwell Blvd. is also overloaded under these conditions.

To alleviate the problems in the channel through Eaglecrest, subsequent simulation assumes the diversion of this flow east along the Island Highway to an improved Yambury Road conduit. This diversion on the highway is modelled as upgraded open channel with an enclosed circular conduit laid under Chartwell Blvd. MOTH have indicated that although the direction of major flows adjacent to the Island Highway is undesirable, this diversion would be acceptable. The possibility of enclosing some or all of this section has been discussed and would be at the discretion of MOTH.

The model assumes that the flows from the upland catchments west of and tributary to Bennett Rd., and historically draining across DL 88, are transported across DL 88 via piped storm sewer. Provision to accommodate upland runoff does not appear to have been accounted for in new storm sewer structures installed to date by the developers. The requirement to continue to accommodate upland runoff is accepted drainage practice in engineering design, and although it has not been enforced to date by MOTH, the analysis assumes that this will be addressed by the time of full development.

The ability to redirect upland flow at Bennett Rd. north to the Island Highway and thus around DL 88, is possible, but this would be a further diversion and concentration of flows to the highway and along to Yambury. As previously noted, MOTH have already indicated that the transport of major flows along the highway is undesirable. To eliminate the hazard perceived by the MOTH, enclosure of this ditch would be indicated. If a diversion around DL 88 of upland flows is pursued, the capacity of the affected structures would also require upgrading.

Since the design of improvements to the Yambury Road system will be based on a more severe condition, the improvements for the 10 year recurrence interval were not addressed.

11.2 100 YEAR RECURRENCE INTERVAL

In the 100 year condition, system capacity is exceeded in many locations throughout the basin. This is to be expected since the structures of the minor system are designed to less severe conditions.

In the modelling of a 100 year storm, the overland diversion of excess stormwater from the minor or underground system is generally considered as acceptable practice. Such diversion has been allowed in all situations except lower Yambury Road. Considering the likelihood and potential for damage to property, structures, and utilities in the event of a major overtopping of the Yambury ditch in its lower reaches, the required recurrence interval for the design of improvements should be at least 50 years and preferably 100 years. To design for a lesser condition would require the provision of a major stormwater flow path over the bluff and to the ocean. The future existence and integrity of such a flow path is difficult to ensure.

In some areas, the ability to accumulate excess flows as local ponding and storage exists, and in such cases the flows rejected by the minor system are not rerouted. This is typical of the upper, rural areas of the basin. In parts of the Eaglecrest development, temporary ponding on the golf course has been assumed.

Where the flows can be anticipated to rejoin the trunk system after overland flow, the rerouting has been modelled. This is typical of DL 88 and some areas of Eaglecrest.

In DL 88, the excess flow is assumed to be rejected by the storm sewers and is routed as flow within the roadways across DL 88. In the Oceanside development the excess or major stormwater runoff will follow the natural lay of the land and flow to the northeast. It is not known if the development's engineers have accounted for this major flow path. The Town should ensure future phases account for and manage this flow.

The required capacity of the Yambury system under ultimate development conditions and a 100 year storm, ranges from approximately 1.75 m^3/s in the most upstream portion south of the highway, to 6.4 m^3/s at the outlet. The required capacity ranges from 4.1 to 4.7 m^3/s for the portion between the Island Highway and a point south of Pintail where flows are introduced from Eaglecrest. These capacities are clearly beyond that which should be conveyed in an open roadside ditch through a residential area.

The solution recommended for the Yambury Ditch within the Town boundaries is for enclosure with a conduit. The installation of such a large utility is complicated by the existing servicing and the exact configuration would be subject to detailed design, but a combination of circular and box concrete culvert is indicated.

A conceptual design has been prepared for preliminary analysis and cost estimating. The extent of work anticipated is from the Island Highway in the south to the Straight of Georgia in the north, corresponding to the areas falling within Town of Qualicum Beach boundaries. The design assumes the incorporation (not replacement) of the steep culvert to the beach terrace. The cost of the project is estimated at $950,000 including an allowance of 20% for contingency and engineering services.

12.0 COST RECOVERY OPTIONS

Although the Yambury ditch lies within the Town of Qualicum Beach, the requirement for upgrading comes as a result of development outside the Town. It is thus not considered appropriate that the Town bear the cost of the entire Yambury Road drainage improvements. It is further assumed that the Town does not have the funds to construct the improvements outright, seeking recovery of costs at a later time.

As a result of the cooperation of MOTH as approval authority for the developing lands, approvals for development have been held subject to a solution being found for the drainage problems in the area.

Meetings have been conducted between the Town, MOTH, RDN and interested developers. Several methods of cost allocation and recovery have been discussed. Out of these meetings has come a preliminary proposal to have the developers of lands tributary to the Yambury system contribute to the cost of the construction.

The general terms of the proposal would see the Town collecting a drainage levy as a condition of development approval from every single family lot (or appropriate equivalent) created within the directly benefitting area of the drainage basin as shown in Figure 3.

These areas of Figure 3 are based on the current potential for development as dictated by the RDN Sewer Specified Area (SSA). If in the future, the SSA is expanded, or zoning or other land rating changes occur such that the development character of other areas in the basin are significantly different with regard to their stormwater runoff, then the subject area would require re-evaluation. If any spare capacity in the system is available to accommodate other areas, they should be required to make the contribution to the Town.

Without the upgrading of the Yambury ditch, no development in the basin should occur. The benefits to the developing areas are twofold. Each development is required to make only a proportional contribution to the improvements rather than undertake the overall project in some fashion to accommodate their own development. Development may proceed as soon as the construction is completed, which is anticipated to be late summer 1993, as opposed to having to wait until it is viable to construct the improvements alone or until other willing participants can be found.

The exact number of potential lots still to be developed within the basin is indeterminate, but the estimated probable number is around 450 with an absolute maximum of perhaps 500. A likely number to develop in the short term would be approximately 330.

Given the cost of the works and the number of potential lots still to develop in the basin, a contribution of $2000 per lot is presently proposed. This is a user pay philosophy that follows the practice of apportioning costs on an equal per unit basis, as is done with Development Cost Charges (DCC's) in other Municipalities, to avoid both the unfair allocation of cost and the inevitable petitioning for adjustment on the part of the Developers.

No areas within the basin are considered to be adversely affected by the proposed Yambury improvements.

In order to avoid the need to borrow, the collection of a portion of this money is required ahead of the construction. If a substantial amount can be committed, the Town could either elect to fund the balance, or, a phased concept for the construction could be investigated. If a substantial amount cannot be raised ahead of construction, and the Town borrows to finance the construction, then the additional costs would have to be reflected in the exact amount of the drainage levy.

A proposal has been received by the Town from the Solicitors representing the owners of lots 1,7 & 9 within DL 88, whereby a contribution of $3,000 per lot for 204 lots would be collected to ensure the project is undertaken. The contribution would be understood to be comprised of a non-refundable portion of $2,000 per lot with a further $1,000 per lot to assist the financing of the project. This later portion would then hopefully be recovered by the developers through a late comers agreement.

The willingness on the part of all developers to contribute to these offsite works has not been assured. Developers in the lower basin consider the obligation to fall entirely to the developers in the upper basin. Considerable animosity exists between the developers within the basin. This is understandable given the contribution they are now being asked to make, weighed against the degree to which one development in particular has been allowed to advance with no requirement thus far for offsite drainage improvements.

This apparent imbalance has arisen from the drainage submissions made on behalf of this development being based on the assumption that the development need not consider or provide for, a substantial portion of the historic upland runoff. The situation tempts for review by MOTH.

13.0 RECOMMENDATIONS

Based on the findings of this report we provide the following recommendations:

1. That Council adopt this report for the purposes of seeking authorization to collect funds for and construct Yambury Road ditch improvements pursuant to Section 590 of the Municipal Act, as recommended and directed by the Town's Solicitor, Grant Anderson, in his letter to the Town of 23 November, 1992.

2. The owners of land within the benefitting area (as shown in Figure 3 to this report) shall contribute to the cost of drainage improvements as follows:

    (a) Upon approval of a plan of subdivision (including subdivision under the Land Title Act and a bare land strata subdivision), the owner shall pay to the Town of Qualicum Beach a drainage levy of $2,000.00 for each parcel created in addition to the number of parcels existing before the subdivision.

    (b) The drainage levy shall be paid to the Treasurer at the time of approval of the subdivision, before endorsement of the plan of subdivision by the approving officer.

    (c) The amount of the drainage levy shall be adjusted on April 30 of each calendar year beginning in 1994, by a percentage equal to the change in Statistics Canada's Consumer Price Index for Victoria, B.C. for the preceding calendar year.

(d) The drainage levy collected by the Treasurer shall be held in a separate fund and shall be used (together with any interest thereon) solely towards the capital costs (including planning, engineering and legal costs directly related to the work) of the drainage work described in this report.

3. That copies of this report be forwarded to MOTH with the request that the diversion of stormwater currently flowing to Eaglecrest between Bennett Rd. and Chartwell Blvd. be undertaken by MOTH.

   That an obligation on the part of MOTH be obtained to carry out this diversion, and that in the event that the diversion will not be undertaken by MOTH, then the costs of the diversion work be added to the project cost of Yambury Road for the purposes of establishing the cost allocations of recommendation 2 above.

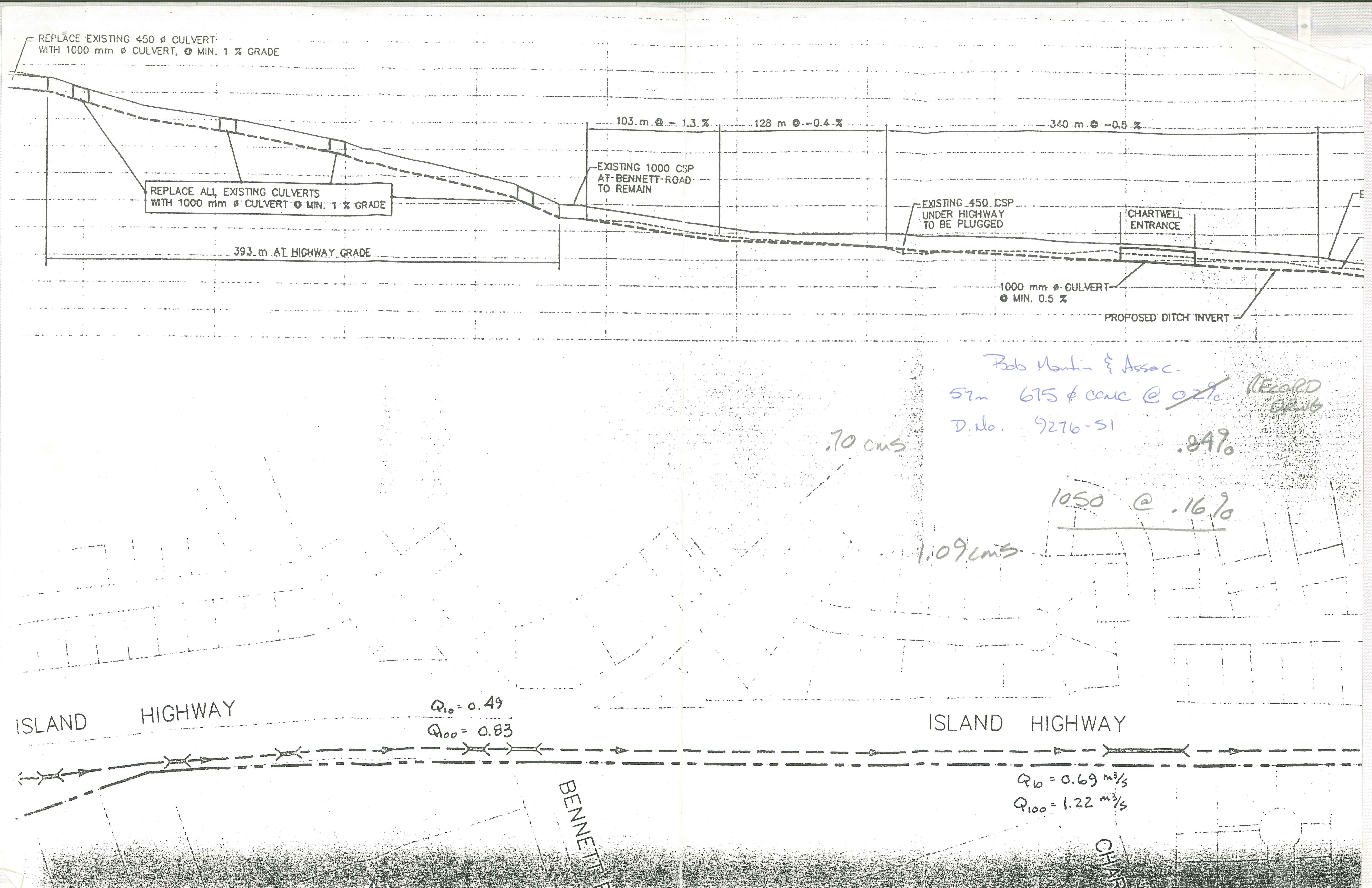
4. That in the event that a significant portion of the monies are not committed and/or collected ahead of construction, a phased concept for the construction be considered. The financial impact of phasing would require review and possible adjustment of the drainage levy.

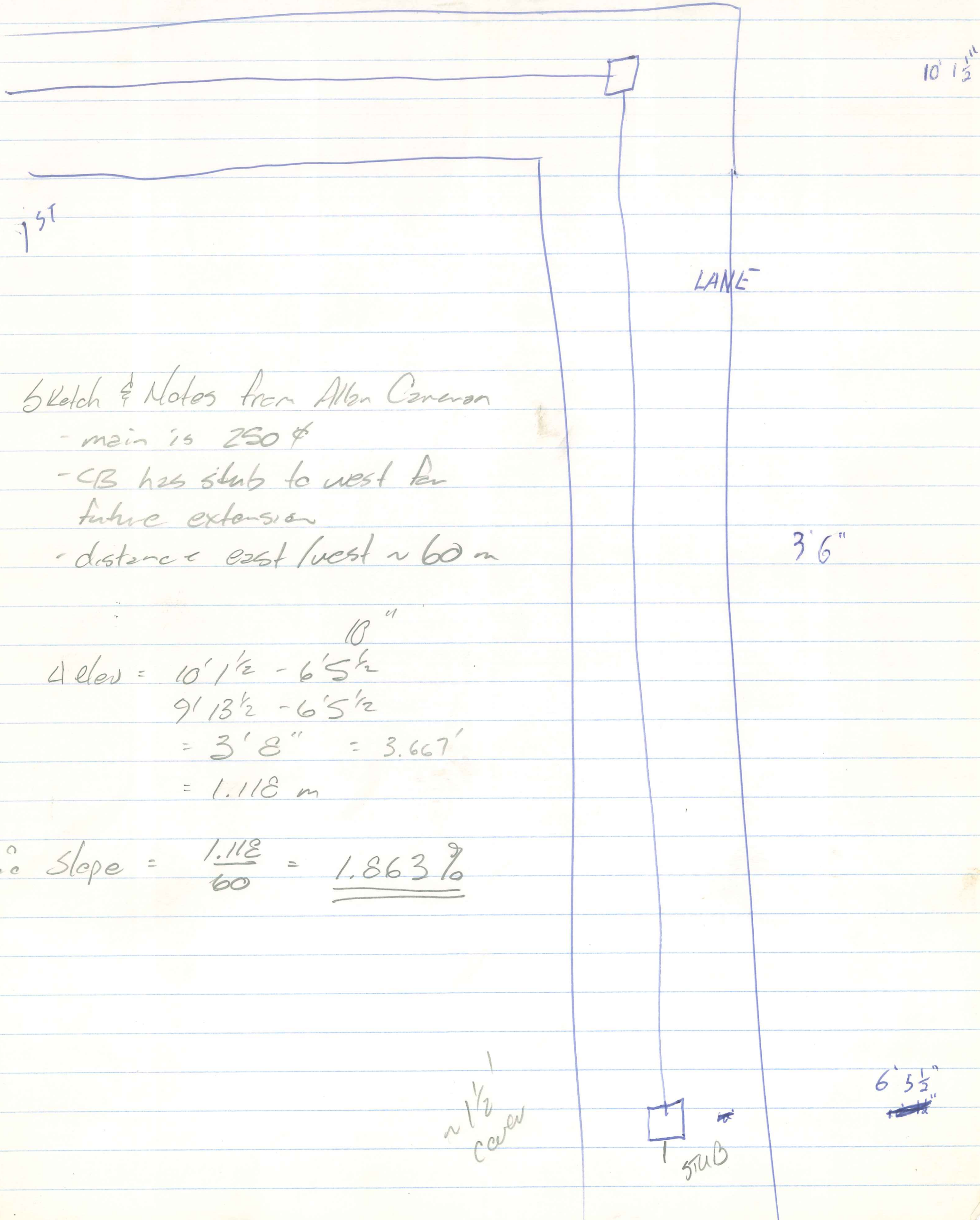
5. That copies of this report be forwarded to the RDN with the request that any changes to zoning and/or specified areas that could potentially affect the ultimate stormwater runoff to Yambury Rd., be forwarded for review by the Town.

6. That no development within the Town boundaries, tributary to Yambury Rd. be approved without contribution to the Yambury Rd. improvements.

7. That future developments provide plans for the management of major stormwater flows to the satisfaction of the Town.

8. That the minor drainage structures of areas newly acquired by the Town be scrutinized for impediments to the intended capacity. That the Town strive to have any such impediments eliminated.


REPLACE EXISTING 450 ø CULVERT WITH 1000 mm ø CULVERT, @ MIN. 1 % GRADE
REPLACE ALL EXISTING CULVERTS WITH 1000 mm ø CULVERT @ MIN. 1 % GRADE
393 m AT HIGHWAY GRADE
103 m @ -1.3 %
128 m @ -0.4 %
340 m @ -0.5 %
EXISTING 1000 CSP AT BENNETT ROAD TO REMAIN
EXISTING 450 CSP UNDER HIGHWAY TO BE PLUGGED
CHARTWELL ENTRANCE
1000 mm ø CULVERT @ MIN. 0.5 %
PROPOSED DITCH INVERT
Bob Martin & Assoc.
57m 675 ø conc @ ~~0.2%~~
RECORD DRWG
D. No. 9276-51
.70 cms
.84%
1050 @ .16%
1.09 cms
ISLAND HIGHWAY
Q10 = 0.49
Q100 = 0.83
ISLAND HIGHWAY
Q10 = 0.69 m3/s
Q100 = 1.22 m3/s
BENNETT



Sketch & Notes from Allan Cameron

- main is 250 ⌀
- CB has stub to west for future extension
- distance east/west ~ 60 m

Δ elev = 10' 1½ - 6' 5½

9' 13½ - 6' 5½

= 3' 8" = 3.667'

= 1.118 m

$\therefore$ Slope = $\frac{1.118}{60}$ = **1.863 %**

TOWN OF QUALICUM BEACH
POPULATION STATISTICS

AGE	1976	1981	1986	1991
0-19	435 (25%)	700 (25%)	750 (22%)	855 (19%)
20-34	280 (16%)	510 (18%)	515 (15%)	505 (12%)
35-54	360 (22%)	560 (20%)	715 (21%)	1,045 (24%)
55-64	330 (19%)	515 (18%)	570 (17%)	675 (15%)
65+	320 (18%)	550 (19%)	860 (25%)	1,340 (30%)
TOTALS:	1,725	2,835	3,410	4,420*
# OF HOUSEHOLDS	685	1,125	1,415	1,910*
AVG. # OF PERSONS PER HOUSEHOLD	2.5	2.5	2.4	2.3

*does not include Eaglecrest

MDB\Stats.mdb

306 359 0929

M E M O R A N D U M

TO: Mayor & Councillors, Town of Qualicum Beach

FROM: M.D. (Mark) Brown, Director of Finance & Administration

RE: **TOWN OF QUALICUM BEACH POPULATION PROJECTIONS TO THE YEAR 2,000 (VARYING GROWTH RATES)**

NOTE: **The average growth rate during the past 30 years is 6% per year and excluding Eaglecrest boundary extension it was 5.3% per year for the last 5 year census period 1986-1991.**

DATE: July 7, 1993

YEAR	5%	6%	7%
1991 (census)	5,137	5,137	5,137
1992	5,394	5,445	5,497
1993	5,664	5,772	5,881
1994	5,947	6,118	6,293
1995	6,244	6,485	6,734
1996	6,556	6,874	7,205
1997	6,884	7,287	7,709
1998	7,228	7,724	8,249
1999	7,590	8,188	8,826
2000	7,969	8,679	9,444

MDB\jh
mdb\popproj.mdb

TOWN OF QUALICUM BEACH

INCORPORATED 1942

141 West Second Avenue
P.O. Box 130
Qualicum Beach, B.C.
V9K 1S7

Telephone: (604) 752-6921
Fax: (604) 752-1243

POPULATION STATISTICS

YEAR	CENSUS	POPULATION INCREASES NO.	% - 5 YEAR	%PER YEAR
1961	759			
		) 247	= 32.5%	= 6.58%
1966	1,006			
		) 239	= 23.75%	= 4.37%
1971	1,245			
		) 479	= 38.47%	= 6.7%
1976	1,724			
		) 1,120	= 64.97%	= 10.5%
1981	2,844			
		) 566	= 19.9%	= 3.7%
1986	3,410			
		) 1,008	= 29.6%	= 5.3%
*1991	4,418			

Average growth during the past 30 years = 6% per year.

*Note: 1991 figure does not include the Eaglecrest area that was incorporated into the municipality effective March 1, 1991.

1991 estimate including Eaglecrest = **5,137**.
(Source: Ministry of Municipal Affairs).

m\mdb\popstats.mdb

BOB MARTIN & ASSOCIATES (1995) LTD.
MUNICIPAL & CIVIL ENGINEERS
LAND DEVELOPMENT CONSULTANTS

File No.

Date

Project ____________________

1022

R = 0.6. Area = 5.43 Ha = 3.24 Ha

$Q_{10} = 0.6 \times 5.43 \times 44 \times 2.78 = 398$ lps.

$Q_{100} = \pm 615$ lps

1023

R = 0.4 Area = 5.4 Ha = 2.16

Total area = 5.4 Ha

Tc = 13.3 i = 37 Q_{10} = 555 lps.

$Q_{100} = 858$ lps

1024

R = 0.3 Area = 18.43 = 5.53

Total Area = 10.93 Ha

Tc = 36.3

$Q_{10} = 10.93 \times 23 \times 2.78 = 699$ lps

$Q_{100} = 1080$ lps

1025

R = 0.3 A = 13.44 = 4.03

Cum Area = 14.96 Ha

Tc = 52.6.

i = 17

Q_{10} = 707 lps

Q = 1092 lps.

Kogrs & Assoc
675Ø @ Worthwell Blvd 832 lps

KOERS
& ASSOCIATES
ENGINEERING LTD.
Consulting Engineers

CLIENT: TOWN OF QUALICUM BEACH

PROJECT: STORM DRAINAGE STUDY
YAMBURY BASIN

TITLE: BENEFITING AREAS

APPROVED

DATE: JAN. 1993

JOB No. M9133

SCALE: 1;10,000

DWG No. FIGURE 3

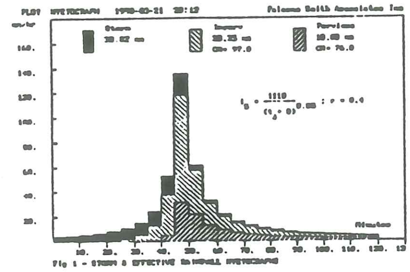
APPENDIX A

Software Technical Specification

TECHNICAL SPECIFICATION

MODEL TYPE

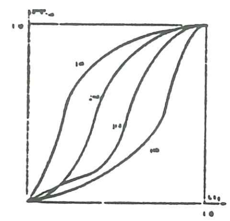
- single event rainfall-runoff simulation for discretized urban or rural catchments
- design of pipes, channels, detention ponds and diversion structures
- hydrographs are routed and combined in a tree network of any size and complexity
- snowmelt and water quality are not included



HYDROLOGY

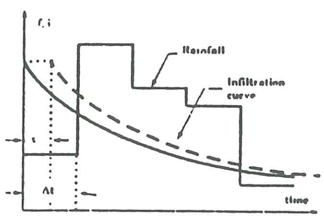
Storm Options:–

(1) Chicago hyetograph design storm $i = a/(t_d+b)^c$
(2) Any one of Huff's four quartile rainfall distributions.
(3) A user defined mass rainfall distribution.
(4) The Canadian AES 1-hour design storm (after Watt *et al*, Can.J.C.E., **13**, June 1986)
(5) Historic storm entered from the keyboard or read from a file.



Infiltration Options:–

(1) Modified SCS Curve Number method using either CN ($1 < CN \leq 100$) or a runoff coefficient C ($0.0 < C \leq 1$). If C is defined the equivalent CN value (*i.e.* for the current storm) is displayed. Initial abstraction I_a can be defined directly (*i.e.* as a depth) or indirectly by the ratio I_a/S where S is the potential storage depth in the soil.



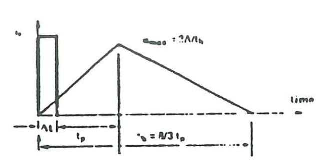
(2) Horton moving curve method. The user defines f_o, f_c and the decay parameter K (in hours not 1/h) plus the surface depression storage depth.

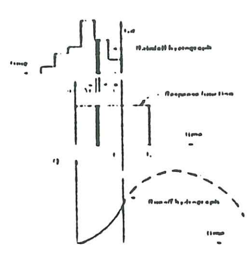
(3) Green-Ampt method. Uses soil moisture deficit, suction head, soil conductivity and surface depression storage depth.

Type of Watershed Routing:–

A runoff hydrograph is computed separately for the impervious and pervious fractions of the sub-catchment and added together. The user has the option of four alternative overland flow routing procedures.

(1) Convolution of the effective rainfall with a nonlinear SCS-type triangular IUH in which time to peak $t_p = dT/2 + t_c$. The time of concentration t_c is defined by a kinematic wave equation and thus varies with the effective rainfall intensity.

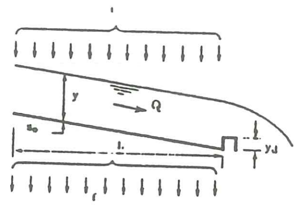


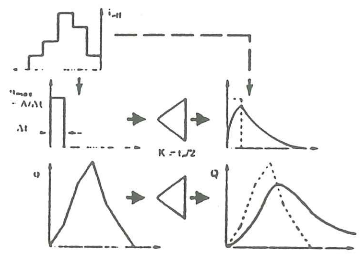


(2) Convolution of the effective rainfall with a nonlinear rectangular response function of duration $t_b = t_c$ where t_c is the time of concentration given by the kinematic wave equation.

Type of Watershed Routing (Continued):-

(3) The SWMM/RUNOFF algorithm based on a surface water budget. The Horton moving curve loss model must be used with this option.





(4) Convolution of the effective rainfall with an IUH obtained as the response of a linear reservoir of lag K = $t_c/2$ (where t_c is the time of concentration based on the peak rainfall intensity). This is similar to Pederson's method as used in the OTTHYMO/URBHYD model.

REQUIRED WATERSHED DATA.

Drainage area (ha or acres); flow length (m or ft); gradient (%); percent impervious; Manning roughness for impervious and pervious fractions; rainfall abstraction parameters (for SCS, Horton or Green-Ampt method) for impervious and pervious fraction.

Calibration Parameters

Curve Number CN (or C), Horton or Green-Ampt parameters
Surface depression storage depth
Subcatchment length L
Subcatchment roughness as Manning's 'n'
Subcatchment gradient s (%)

It should be noted that runoff is sensitive to the time of concentration t_c which in turn is dependent on the parameter ($L.n/s^{0.5}$) so that only one of L, n or s need be varied. The runoff from the pervious and impervious fractions is thus dependent on Manning's 'n' and i_{eff} for each component of the rainfall hyetograph.

Runoff volumes are sensitive to CN (or C) or f_o, f_c and K and the percent impervious. Hydrograph shape and flow peaks are sensitive to the parameter ($L.n/s^{0.5}$).

OUTPUT

Displays effective rainfall hyetograph on impervious and pervious surfaces (tabular or graphical).
A summary table is generated during the session which can be copied to a printer.
Runoff hydrographs are displayed in tabular or graphical form.
Plots of hydrographs and hyetographs may be combined.
Runoff volume, peak flow and percentage continuity error are computed.

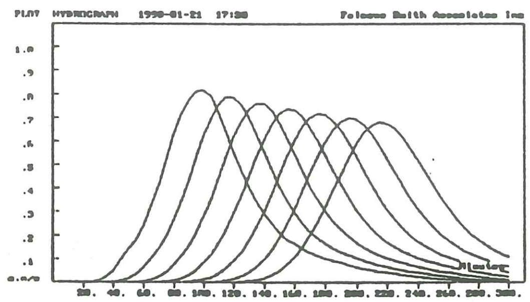
```
Input file: FUT_100.IN     Run time: 1989-11-20   9:50                                     Falcone Smith Associates Inc

<--------Catchment---------><---- Runoff in c.m/s  ---->              <----------Conduit Design----------><--Conduit params-->
ID#    Hect    L    s%  %Imp Local  Total  Minor  Major               Capac  Lnth Dia/Ht  S%  Depth  HGL   n  Base Left Right
-----------------------------------------------------------------------------------------------------------------------------
                                                       100-year Storm defined
                                                        Return      'a'       'b'      'c'       'r'      Td (min)
                                                        100       1264.57    7.72    0.7810     0.45     360

                                                       Impervious    SCS  n/CN/Ia=             .02  98.  1.5
                                                       Pervious      SCS  n/CN/Ia=              .25 75.   5.
 1.03  51.    50.  1.   33.  7.088                     Lag & Route   0.    7.5
                             5.3352 5.3352             Pipe    s/ch 3.1613      1.5    .2          .57  .013
                                           3.32  2.0152 Diversion
                                    3.32   3.32
 1.01  69.    45.  1.   31.  9.28   3.32
                             6.2966 9.5938
```

CHANNEL ROUTING COMPONENT

Routing Time Step:

The routing time step is automatically computed as a submultiple of the runoff time step to ensure stability of the kinematic routing method.

Type of routing:



The multiple reach Muskingham-Cunge method is used. For short reaches the time step is computed as a submultiple of the runoff time step; for long reaches the total reach length is subdivided to ensure stability.

The Muskingham 'X' is computed by the Cunge method. The lag 'K' is based on the reach length and the average of the flow velocity and the wave celerity calculated for peak discharge.

Required data:

The cross-section of the last designed conduit or channel is automatically 'remembered' by MIDUSS. The user specifies the total reach length. The user may over-ride the computed values for 'X' and/or 'K'. The final display indicates whether or not multiple reaches have been used.

Lateral inflows & backwater effects are not included

Accuracy:

Very close agreement (*e.g.* better that 5%) is obtained in comparison to results obtained by a 4-point implicit solution of the dynamic wave equations.

Output:

In addition to the peak outflow, the outflow hydrograph can be displayed in either tabular or graphical form by means of appropriate commands.

RESERVOIR ROUTING COMPONENT

Time Step: Equal to the runoff timestep dT except when dT < 2.(dS/dQ) when it is set equal to the largest submultiple of dT which satisfies the stability criterion. (Note: dS/dQ is the gradient of the storage volume/discharge curve).

Algorithm: Standard storage-indication reservoir routing procedure.

RESERVOIR ROUTING COMPONENT (Continued)

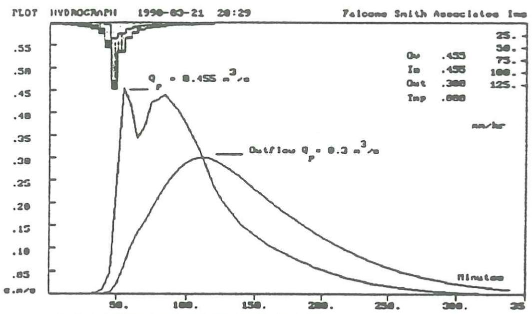


Fig 2 - INFLOW & OUTFLOW AT DETENTION POND

Data requd: For a user-specified peak outflow, MIDUSS computes the required storage volume. The user may then adjust a table of Depth-Discharge-Volume relations to describe the characteristics of the reservoir and the outflow structure. The maximum and penultimate stages are computed by MIDUSS; if only 3 stages are used no further data is required from the user. The user may optionally reduce the routing time-step to a submultiple of the runoff time increment dT.

Output: Maximum values of storage volume, outflow and depth are displayed. The user may then accept or revise the design by editing the Depth-Discharge-Volume table. The outflow hydrograph can be displayed in both tabular and graphical form using appropriate commands. The user may interactively extend the routing process beyond the end of the inflow hydrograph to determine reservoir drainage time.

Accuracy: Truncation errors may occur if the user employs coarse time steps to describe very 'peaky' inflow hydrographs.

AUTOMATIC OR MANUAL USE

An output file (if defined) stores all commands, input data and some results. This may be used subsequently as an input file for Automatic re-run, thus allowing:

- Designs to be completed in two or more stages
- Designs to be subjected to different storms
- Design decisions to be modified

GRAPHICS

High resolution graphics displays of hydrographs and hyetographs can be generated and superimposed without limit. All plots are automatically scaled to preferred numbers but hydrographs can be scaled manually if desired.

Hardcopy of graphics plots can be produced on a wide variety of dot matrix, ink jet and laser printers. Hardcopy on plotters is not supported.

EXPERTISE REQUIRED

The package is well documented and exceptionally easy to use. Context sensitive help is always available. Typically, new users can produce designs within one hour.

The program is intended for use by engineers who have an adequate knowledge of the principles involved in the modelling and design of stormwater drainage systems.

UNITS Metric or Imperial units can be used.

APPENDIX B

Computer Summaries

Input file: YAMC1HR.2ND Run time: 1993-01-29 9:57 Anthony Koers, Ph.D., P.Eng.
Output File: YAMC1HR.3RD

<--------Catchment	---------	---->			<---- Runoff	in c.m/s		---->			<----------Conduit		Design	----------	-------->	<--Conduit		params-->		
ID#	Hect	L	s%	%Imp	Local	Total	Minor	Major			Capac	Lnth	Dia/Ht	S%	Depth	HGL	n	Base	Left	Right
Huff's mass curve 27.685 mm in 60 min									Parameters		2									
									Impervious		SCS	n/CN/Ia=			.017	98	.52			
									Pervious		SCS	n/CN/Ia=			.25	70	10.9			
1000	16.19	180	3.5	2	.05742	.05742			Pipe		.3155		.5	2	.144		.022			
							.05739		Route			12								
						.05739			Channel		1.138		.5	1.75	.111		.04	.6	2	2
							.05742		Route			400								
						.05742														
									Pervious		SCS	n/CN/Ia=			.25	70	10.9			
1001	33.35	450	2.5	5	.2183	.2695			Pipe		.3155		.5	2	.355		.022			
							.2693		Route			12								
						.2693			Channel		2.407		.6	3.5	.21		.04	.6	2	2
							.2611		Route			320								
						.2611			Channel		1.616		.5	2	.194		.04	1	2	2
							.2567		Route			150								
							.2567------>		Node	101	.2567									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1002	7.15	250	5	5	.05716	.05716														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1003	4.77	200	5	5	.03746	.09443			Channel		.8771		.3	.5	.0864		.04	3	5	5
							.09251		Route			250								
							.09251------>		Node	101	.3476									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1004	10.06	500	.6	2	.0231	.0231														
							<-------------		Node	101										
						.3707			Channel		17.744		2	.5	.338		.04	1	2	2
							.3587		Route			800								
							.3587------>		Node	103	.3587									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1005	5.88	150	1.33	10	.08767	.08767			Channel		1.8619		1	.2	.246		.04	.6	2	2
							.08279		Route			150								
							.08279------>		Node	100	.08279									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1006	6.51	150	1.8	10	.1027	.1027			Channel		1.3543		.8	.3	.24		.04	.6	2	2
							.09811		Route			100								
							.09811------>		Node	100	.1754									
							<-------------		Node	100										
						.1754			Channel		1.8619		1	.2	.346		.04	.6	2	2
							.1711		Route			100								
							.1711------>		Node	102	.1711									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			

1007	14.27	150	1.8	10	.225	.225			Channel		1.1058		.8	.2	.39		.04	.6	2	2
							.2015		Route			250								
							.2015------>		Node	102	.3726									
							<-------------		Node	102										
						.3726														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1008	30.85	200	1.3	5	.2307	.5765			Channel		2.2597		.5	2.7	.276		.04	.6	4	4
							.5924		Route			550								
							.5924------>		Node	103	.7593									
							<-------------		Node	103										
						.7593			Pipe		.9303		.75	2	.515		.022			
							.759		Route			15								
						.759			Channel		2.9439		1	.5	.553		.04	.6	2	2
							.7486		Route			140								
						.7486			Pipe	s/ch	.5131		.6	2		4.257	.022			
							.525	.2236	Diversion											
						.525	.525		Route			12								
						.525														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1014	8	60	.8	40	.4914	.831			Pipe	s/ch	.5492		.6	.8		1.832	.013			
							.6	.231	Diversion											
						.6	.6		Route			450								
						.6														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1015	4.875	60	.8	40	.2995	.8636			Pipe	s/ch	.7518		.675	.8		1.055	.013			
							.765	.09856	Diversion											
						.765	.765		Route			400								
							.765	------>	Node	104	.765									
									Pervious		SCS	n/CN/Ia=			.25	70	10.9			
1017	63.65	500	2.3	1	.114	.114			Pipe		.2945		.4	2	.173		.013			
							.114		Route			12								
						.114														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1018	6.82	200	1.8	10	.1013	.1817			Channel		1.6274		.6	1.6	.21		.04	.6	2	2
							.1775		Route			270								
									Pipe		.534		.5	2	.201		.013			
							.1814		Route			12								
						.1814														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1019	29.25	300	2.2	10	.4165	.5979			Channel		17.423		1.5	1.2	.363		.04	.6	3	3
							.5919		Route			800								
						.5919														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			

1021	45.29	250	2.5	5	.3439	.8126			Channel		13.606		1.4	1.7	.383		.04	.9	2	2
							.8169		Route			750								
						.8169														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1020	25	500	2	5	.1613	.9302			Pipe		2.5602		.9	2	.375		.013			
							.9298		Route			15								
									Channel		1.9646		.75	.6	.528		.04	.9	2	2
							.9179		Route			100								
							.9179------>		Node	104	1.5651									
						.2236<--------------			DIV00200.100											
									Channel		.9935		.15	.5	.0845		.015	.3	50	50
							.226		Route			300								
						.226														
					.231 <---------------------				DIV00201.100											
						.2872			Channel		.9935		.15	.5	.0931		.015	.3	50	50
							.3004		Route			400								
						.3004														
					.09856<---------------------				DIV00202.100											
						.3004			Channel		.9935		.15	.5	.0947		.015	.3	50	50
							.2845		Route			200								
							.2845------>		Node	104	1.7509									
							<--------------		Node	104										
						1.7509			Channel		1.9646		.75	.6	.711		.04	.9	2	2
							1.7318		Route			300								
							1.7318------>		Node	105	1.7318									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1009	7.62	150	.7	5	.05678	.05678														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1010	14.83	160	1.4	10	.2211	.2779			Channel		2.1316		.75	1	.292		.04	.6	2	2
							.2769		Route			400								
						.2769														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1011	43.39	250	2.7	7	.4559	.6976			Channel		2.1316		.75	1	.454		.04	.6	2	2
							.6938		Route			400								
						.6938			Pipe	s/ch	.3155		.5	2		9.669	.022			
							.32	.3738	Diversion											
						.32	.32		Route			12								
						.32														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1012	8.19	60	.8	40	.5031	.8212			Pipe	s/ch	.5884		.75	.8		1.558	.022			
							.6	.2212	Diversion											
						.6	.6		Route			450								
						.6														

										Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1013		4	60	.8	40	.2457	.8457			Pipe		.9957		.75	.8	.531		.013			
								.8214		Route			400								
								.8214------>		Node	105	2.2512									
								<-------------		Node	105										
							2.2512														
										Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1016		8	60	.8	40	.4914	2.4331			Channel		3.7803		.75	2	.609		.04	1	2	2
								2.4236		Route			220								
								2.4236------>		Node	106	2.4236									
										Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1022	5.43		70	3.3	35	.2813	.2813			Channel		.502		.5	.3	.394		.04	.3	3	3
								.2719		Route			150								
							.2719														
										Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1023	5.4		100	4.7	30	.2426	.5145			Channel	s/ch	.502		.5	.3		.769	.04	.3	3	3
								.514	.000466	Diversion											
							.514			Channel		.7864		.6	.3	.505		.04	.3	3	3
								.4882		Route			350								
							.4882														
										Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1024	18.43		200	2.5	15	.4072	.853			Channel		6.7131		1	2.6	.399		.04	.6	2	2
								.8318		Route			400								
							.8318			Pipe		3.3907		1	2	.337		.013			
								.8307		Route			18								
							.8307			Channel		5.4282		1	1.7	.435		.04	.6	2	2
								.8015		Route			170								
							.8015														
										Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1025	13.44		65	5	35	.6859	1.2187			Pipe		1.3654		1.05	.25	.773		.013			
								1.2005		Route			400								
							1.2005														
										Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1026	11.2		60	2	40	.6664	1.7103			Channel		4.5606		1	1.2	.654		.04	.6	2	2
								1.6718		Route			250								
								1.6718------>		Node	106	4.0556									
								<-------------		Node	106										
							4.0556			Pipe		4.6076		1.2	4	.874		.022			
								4.0541		Route			25								
							4.0541			Pipe		7.7975		1.2	4	.614		.013			
								4.0487		Route			125								
							4.0487														
										Pervious		SCS	n/CN/Ia=			.25	75	8.47			

1027	3.49	60	5	25	.1276	4.1264			Pipe		4.7739		1.35	.8	.969		.013			
							4.1192		Route			100								
						4.1192			Pipe		11.196		1.35	4.4	.567		.013			
							4.0877		Route			135								
						4.0877														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1028	7.38	70	5	35	.3757	4.3094			Pipe		4233.4		13.5	2.92	.327		.013			
							4.3039		Route			70								
						4.3039			Pipe		5.0635		1.35	.9	.956		.013			
							4.2809		Route			220								
							4.2809	------>	Node	107	4.2809									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1029	6.48	70	3	35	.3369	.3369			Pipe		.5131		.6	2	.355		.022			
							.3362		Route			40								
									Channel		.5119		.45	1	.376		.04	.3	2	2
							.3357		Route			350								
						.3357														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1032	7.04	70	3	25	.2629	.5813			Pipe	s/ch	.3628		.6	1		2.567	.022			
							.37	.2113	Diversion											
						.37	.37		Route			80								
							.37	------>	Node	108	.37									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1030	31.05	80	6.7	25	1.1311	1.1311			Channel		2.6107		.75	1.5	.517		.04	.6	2	2
							1.1294		Route			300								
						1.1294	.7025		Pond	vol=	712.2				.662					
						.7025														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1031	16.48	200	2.7	5	.1271	.7929			Pipe	s/ch	.7564		.9	.5		.549	.022			
							.77	.02294	Diversion											
						.77	.77		Route			180								
						.77			Channel		2.9439		1	.5	.557		.04	.6	2	2
							.7684		Route			150								
						.7684			Pipe	s/ch	.7564		.9	.5		.516	.022			
							.7684		Route			120								
							.7684	------>	Node	108	1.1161									
							<-------	-------	Node	108										
						1.1161			Channel		17.744		2	.5	.585		.04	1	2	2
							1.1058		Route			250								
						1.1058														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1033	25.58	100	3	20	.7868	1.5575			Channel		17.744		2	.5	.685		.04	1	2	2
							1.5554		Route			200								

							1.5554------>		Node	107	5.8363									
							<-------------		Node	107										
						5.8363			Channel		10.77		1.5	.72	1.15		.04	1	2	2
									Pipe		5.9981		1.5	.72	1.194		.013			
							5.8268		Route			50								
									Pipe		9.0477		1.75	.72	1.023		.013			
							5.8337		Route			14								
						5.8337			Pipe		5.9981		1.5	.72	1.194		.013			
							5.8241		Route			50								
							5.8241		Route			50								
							5.8241------>		Node	109	5.8241									
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1039	15.46	100	2	22	.5248	.5248			Pipe	s/ch	.4119		.6	.45		.731	.013			
							.43	.09482	Diversion											
						.43	.43		Route			300								
						.43														
									Pervious		SCS	n/CN/Ia=			.25	75	8.47			
1035	10.39	100	2	22	.3527	.7827			Pipe	s/ch	.7468		.75	.45		.494	.013			
							.76	.02271	Diversion											
						.76	.76		Route			360								
						.76			Pipe	s/ch	.7468		.75	.45		.466	.013			
							.76	0.	Diversion											
						.76	.76		Route			190								
						.76			Pipe		1.1133		.75	1	.455		.013			
							.756		Route			90								
							.756 ------>		Node	109	6.4305									
							<-------------		Node	109										
						6.4305			Pipe		26.918		1.4	60	.466		.022			
							6.4287		Route			51								
						6.4287			Pipe	s/ch	2.6918		1.4	.6		3.422	.022			
							6.4287		Route			8.5								
						6.4287			Pipe	s/ch	6.361		2	.5		.511	.022			
							6.4287		Route			130								
						6.4287	6.4287		Route			130								