iv) Most of the hydrometric stations within the NVCA have more than 14 years of historical records. In order to make use of this data, it is our recommendations that the NVCA use the available data for flood related studies. The QUALHYMO Model can be used to extend the record period at the various hydrometric stations for annual peak flows to 22 years and pro-rate these flows to upstream locations within the tributary catchment.

Based on the experience gained in simulating annual peak flows with the QUALHYMO model and the demonstrated transportability of model parameters, this simulation approach can be used with confidence on ungauged catchments tributary to the lower Nottawasaga River and watersheds draining directly to Nottawasaga Bay.

3.3 Design Flows

3.3.1 General

In the preceeding sections of the report, the QUALHYMO Model was calibrated and validated for both summer and spring events. The Model was subsequently used to generate 22 years (1963-1984) of flows using hourly precipitation and temperature data. From computer printouts, annual instantaneous peak flows were selected for the 22 years of simulated record at each of the existing hydrometric stations. Flood frequency analyses on the annual instantaneous peak flows were subsequently undertaken as discussed in Section 3.2.5.

The agreement between the observed and the simulated flood frequency curves with the same period of record is only marginal for most of the hydrometric stations. Possible explanation for differences between simulated and observed flow peaks are cited in Section 3.2.5.5; nevertheless, the number of spring flow peaks of small to intermediate magnitude which were underestimated by the QUALHYMO model is the underlying reason for the discrepancy between the frequency curves. The input data was also subjected to non-parametric testing and a number of low outliers were removed from the record; however, this did not alter the results of the frequency analysis.

Discussions were held with the Project Committee to resolve the discrepancy and develop a methodology to establish design flows. The procedure approved by the Committee is discussed in Section 3.3.2.

3.3.2 Present Watershed Flows

3.3.2.1 Methodology

The methodologies used to establish frequency based flows and to compute the Regional storm flows are discussed below.

3.3.2.1.1 Frequency Based Flows

From recommendations made in Section 3.2.5.5, Provincial Guidelines (Ref. 22) and direction given by the Project Committee, frequency based design flows (1 in 5, 1 in 10, 1 in 20, 1 in 50 and 1 in 100 year) computed from the existing flow record at each hydrometric station are to be used when the length of record is equal to or greater than 20 years (Table 3.15). For the length of record between 10 years and 20 years, single station analysis is substantiated through comparison with regional frequency analysis. This process leads to the selection of the regression and index flood regional analysis (Ref. 23, 24) for the Beeton Creek and Pine River gauge (Table 3.15)

After frequency based flows were estimated at the existing hydrometric stations, these design flows are transferred to other watershed locations using the discretized QUALHYMO Model. Initially, it was considered using the ratio of simulated peak flows from QUALHYMO based on two historical high flow events. However, it was felt that the ratio would reflect the areal distribution of the selected historical events. In order to avoid this anomaly, it was felt more appropriate to select the point rainfall from a larger spring event that had only minimal snowmelt and to apply this

TABLE 3.15

DESIGN FLOWS AT EXISTING HYDROMETRIC STATIONS

(All Flows in m^3/s)

Hydrometric Station	Drainage Area (km^2)	Flood Frequency Method Selected	Return Period (Years)				
			5	10	20	50	100
Beeton Creek near Tottenham	95	Multiple Regression Analysis	19.5	23.7	28.1	34.0	38.5
Bailey Creek near Beeton	207	Single Station Analysis (3 PLN)[1]	39	49	59	73	84
Boyne River at Earl Rowe Park	211	Single Station Analysis (3 PLN)	70	92	115	149	176
Pine River near Everett	195	Index Flood Method	41.7	50.7	59.1	70.3	79.0
Mad River near Glencairn	295	Single Station Analysis (3 PLN)	75.0	92.0	109.0	131.0	149.0
Nottawasaga River near Baxter	1180	Single Station Analysis (3 PLN)	179	221	264	323	370
Willow Creek above Little Lake	95	Single Station Analysis (Wakeby Distribution)	29.8	33.1	36.6	41.3	45.1

1. 3 PLN - Three Parameter Lognormal Distribution

temporal distribution over the entire watershed which is tributary to the gauge sites. The Shelburne precipitation data for the March 1974 event was used for this purpose.

The total amount and the temporal distribution of rainfall at Shelburne were applied to each sub-catchment in the QUALHYMO model which is tributary to the hydrometric gauges on Willow Creek, Boyne River, Bailey Creek, Beeton Creek, Pine River, Mad River and Nottawasaga River at Baxter. The peak flows at each sub-catchment outlet and at the hydrometric stations were computed with the March 1974 event recorded at Shelburne applied evenly over the Nottawasaga basin with the QUALHYMO model. The ratios of simulated peak flows (watershed location to gauge site) were used to transfer the 1 in 5 to 1 in 100 year flows from the gauges to the watershed locations.

For example:

1. Upper Nottawasaga River, location 60 (Figure 3.11)

   a) QUALHYMO March 1974 peak flow: Location 60: 27.26 m^3/s
      : Nottawasaga River at Baxter: 210.72 m^3/s

   b) Return period flows Nottawasaga River at Baxter from analysis of flow record: 1 in 5 Year: 179 m^3/s (Table 3.15)
      1 in 100 Year: 370 m^3/s

   c) Return period flows at Location 60 (Upper Nottawasaga River)
      i) 1 in 5 Year: 23.2 m^3/s
      ii) 1 in 100 Year: 47.9 m^3/s

2. Boyne River, Location 350 (Figure 3.11)

   a) QUALHYMO March 1974 peak flow: Location 350: 20.62 m^3/s
      : Boyne River at Earl Rowe: 34.93 m^3/s

b) Return period flows Boyne River at Earl Rowe Park from single station analysis of flow record:

i) 1 in 5 Year: 70 m^3/s (Table 3.15)

ii) 1 in 100 Year: 176 m^3/s

c) Return period flows at Location 350 on Boyne River

i) 1 in 5 Year: 41.3 m^3/s

ii) 1 in 100 Year: 103.9 m^3/s

The foregoing procedure was employed to compute frequency based flows of watershed locations within the Willow Creek, Boyne River, Bailey Creek, Beeton Creek, Pine River, Mad River and Nottawasaga River at Baxter Watersheds.

Within the NVCA, there are several watercouses which flow directly into Georgian Bay or the Nottawasaga River below the Minesing Swamp. These catchments have not been gauged and therefore, no streamflow records are available. In order to estimate the frequency based flows it was necessary to simulate historical flows over 22 years using the QUALHYMO Model and the meteorologic database.

In the absence of streamflow data to calibrate/validate the QUALHYMO Model for these catchments, the same snowmelt parameters calibrated for Mad River and Pine River basins were used. These parameters are:

SI	=	127 mm (5")	MFMAX	=	0.005
UADJ	=	0.0057	MFMIN	=	0.0018

As outlined in Section 3.2.5.3, some adjustments to MFMAX and MFMIN were required for the spring events of 1971, 1975, 1977, 1978 and 1982. To be consistent with the MFMAX and MFMIN used for Mad and Pine River catchments for the various years noted above, the same parameters were also used in simulating the 22 years of streamflow data for the Georgian Bay catchments.

After 22 years of streamflow data were run with QUALHYMO, the annual maximum instantaneous peaks were abstracted. Flood frequency analyses (Wakeby distribution) were then conducted for various points along the watercourses using the Environment Canada CFA88 program to obtain the 1 in 5 year to 1 in 100 year flows.

3.3.2.1.2 Regional Storm

The Timmins Storm is the applicable Regional Storm for all drainage areas under the jurisdiction of the NVCA. This is the summer storm which produced 193 mm of rain in a 12-hour period over Timmins on September 1, 1961.

The calibrated/validated QUALHYMO Model was used to generate the Timmins Storm flows for all catchments. Rainfall was input in 1-hour intervals and the equivalent circular area method was used to make areal adjustment to the point rainfall. The equivalent circular area method accounts for the elongation of a basin by using the longest length of the watershed as the diameter.

The Ministry of Natural Resources requires that for the Timmins Storm, AMC II (average conditions) be used to establish antecedent soil moisture conditions on a watershed. However, since the QUALHYMO Model accounts for soil moisture conditions by the use of the API, it was felt that a more accurate indication of the soil moisture conditions prior to the Timmins Storm could be established. The daily rainfall recorded at the Timmins Airport for the months of July and August 1961 (prior to the Timmins Storm on September 1, 1961) was obtained. The rainfall was transposed to the Nottawasaga River basin and the API computed using the QUALHYMO Model. The API value established prior to the Timmins Storm was 27.0 mm. Subsequently this value was used in QUALHYMO to simulate the Timmins Storm flows.

3.3.2.2 Documentation of Flows

The design flows for the locations along the various watercourses are presented in Appendix G. The location of each flow point is shown in Figure 3.11.

The QUALHYMO Model was used to compute the Regional Storm flows as outlined in Section 3.3.2.1.2. The peak flows at points of interest along the various watercourses are presented in Appendix G.

3.3.2.3 Comparison with Previous Studies

Several hydrologic studies have been undertaken for various watercourses (Fig. 1.1) within the NVCA. A brief description of these studies is presented in Appendix A. To ensure consistency in design flows and to obtain confidence in the results obtained, the design flows developed in this study were compared with those from previous studies. A comparison of the design flows for the 1 in 100 year event and Regional Storm is presented in Table 3.16.

At most locations the 1 in 100 year and Regional Storm peak flows are in close agreement; however, discrepancies are apparent on the Sheldon Creek, the Nottawasaga River at Hockley, Truax Creek, Beeton Creek at Beeton and Innisfil Creek east of Cookstown. For the above-noted flow points, the peak flows established in previous studies are higher than the MacLaren study, except for Truax Creek. The flows on Sheldon Creek at Sheldon and the Nottawasaga River at Hockley appear excessively large. However it is emphasized that the hydrologic models applied in the present MacLaren Plansearch study were calibrated while earlier investigations did not perform this exercise. Therefore the design flows developed in this study are more accurate than those developed in earlier studies.

TABLE 3.16

COMPARISON OF 1:100 YEAR AND REGIONAL STORM PEAK FLOWS WITH PREVIOUS STUDIES WITHIN THE BASIN

Location	MacLaren Study			Author	Previous Study		
	Drainage Area (km^2)	Timmins $Q(m^3/s)$	1:100 Year $Q (m^3/s)$		Drainage Area (km^2)	Timmins $Q(m^3/s)$	1:100 Year $Q (m^3/s)$
Willow Creek above Little Lake	95.1	233.5	45.1	Cumming-Cockburn	90.4	241.0	45.2
Black Ash Creek at Outlet	29.1	125.9	51.5	Ainley	30.3	129.6	~30% larger 66.9
Pine River at Outlet	335.9	220.0	101.4	MNR C.A.B	347.8	258.5	-
Lamont Creek at Stayner	26.9	124.5	56.8	Ainley	28.4	124.6	-
Spring Creek at Honda Plant	8.1	28.5	10.1	Giffels	12.2	43.0	20.18
Sheldon Creek at Sheldon	66.3	95.4	19.5	Burnside	59.6	261.0	185.4
Nottawasaga River at Hockley	175.9	164.2	44.7	Burnside	177.8	505.5	328.2
Sturgeon Creek	19.7	55.2	20.2	Cumming-Cockburn	21.02	60.7	14.4
Truax Creek	21.1	59.6	16.0	Cumming-Cockburn	17.0	35.2	-
Beeton Creek at Beeton	84.7	124.8	40.2	Triton	-([1])	180.9	-
Innisfil Creek East of Cookstown	65.4	121.0	26.1	Dillon	60.3	150.1	-

([1]) Same location as MacLaren flow point.

3.3.3 Future Watershed Flows

3.3.3.1 Methodology

Two separate discretized QUALHYMO Models were established for all the catchment areas within the NVCA: one model reflected present conditions while the other future conditions. The additional imperviousness due to future urbanization was simulated with the FRIMP parameter which is a fraction of impervious land. The methodology used to obtain estimates of future urbanization is outlined in Section 3.2.3.

The future Timmins Storm flows were simulated using the QUALHYMO Model set-up for future conditions. The methodology used to simulate these flows was the same as that outlined in Section 3.3.2.1.2 for Timmins Storm flows under present condition.

To establish future flows for the 1 in 5, 1 in 10, 1 in 20, 1 in 50 and 1 in 100 year events, it would be necessary to run the discretized QUALHYMO Model for 22 years. However, from the analysis of the Official Plans carried out in Section 3.2.3 for the urban areas, it was found that for most of the NVCA the increase in urbanization from present to future conditions is negligible. Approximately eleven (11) urban areas were identified where significant urbanization from a hydrologic point of view will take place. These are identified below:

- Boyne River at Shelburne and Alliston
- Spring Creek at Alliston
- Innisfil Creek at Cookstown
- Willow Creek near Barrie
- Bear Creek near Barrie
- Nottawasaga River at Glen Cross
- Lamont Creek at Stayner

- Black Ash Creek near Collingwood *
- Silver Creek near Collingwood
- Pretty River at Collingwood.

The discretized models for each of the above areas were separated from the main model. Downstream sub-catchment areas were included in each model to the point of a major confluence in order to assess the hydrologic impact of urbanization. Each sub-model was subsequently run for 22 years of historic events for both present and future conditions. Frequency analyses were then conducted at points of interest for both present and future conditions using the CFA88 program. At each point, the increase in flow from present to future conditions was determined for the various return periods (1 in 5 to 1 in 100 year). To obtain the future flows, the increase in flow was added to the corresponding return period flow established in Section 3.3.2.2 under existing conditions. An example of this procedure is provided in Appendix I.

A similar procedure was used for some of the other watercourses flowing into Georgian Bay and the local sub-catchments downstream of Minesing Swamp flowing directly into the Nottawasaga River.

3.3.3.2 Documentation of Flows

Using the methodology outlined in the previous section, the future flows for the Timmins Storm and 1 in 5 to 1 in 100 year events were determined. These flows are tabulated in Appendix G. The increase in future flows is small for both the Timmins Storm and the 1 in 5 to 1 in 100 year events.

3.3.4 Flow Estimation at Intermediate Locations

While the watercourses within the NVCA have been discretized into 191 sub-catchments for this study, it is recognized that the Conservation Authority may have future requirements to obtain design flows at other flow points. As indicated previously, design flows developed for this study are presented in Appendix G. The location of the flow points are cross-referenced to Figure 3.11. From a review of Figure 3.11, it can be seen that design flows may be required for (i) smaller headwater drainage basins than were modelled using QUALHYMO and (ii) along major waterways. This section of the report deals with the development of two methodologies to obtain design flows for headwater drainage areas and along major watercourses.

3.3.4.1 Headwater Drainage Areas

In consultation with the Project Committee, it was decided that the design flows for headwater drainage areas would be established by conducting linear regression analyses of peak flows versus drainage areas. The equation has the form of:

$$Q = CA^n$$

where:

Q is the peak discharge (m^3/s)

A is the drainage area (km^2)

and n and C are the slope of the line and intercept, respectively.

A computer program developed in-house was used to conduct linear regression analyses for the 1 in 5, 1 in 10, 1 in 20, 1 in 50, 1 in 100 year events and Timmins Storm. Regression analyses were conducted separately for the major watercourses to account for geographic and physiographic differences. Linear regression analyses were conducted for the following watercourses:

- Innisfil/Beeton/Bailey Creeks
- Upper Nottawasaga River (including Sheldon Creek)
- Boyne River
- Pine River
- Mad River
- Willow Creek
- Georgian Bay watercourses — BAC

The number of flows versus drainage area points used in the regression analyses ranged from seven to thirteen. The size of the drainage areas ranged from 1 km^2 to 134 km^2 and are documented in Table 3.17. Reference should be made to Appendix G for flow values. The result of the regression analyses for the above-noted watercourses are plotted in Figures 3.12(a) to 3.12(g) for the 1 in 20 and 1 in 100 year event and the Timmins Storm. From these figures, it can be seen that the slopes of the lines for each watercourse are similar for the 1 in 20, 1 in 100 year and Timmins Storm. This is also true for the 1 in 5, 1 in 10 and 1 in 50 year events.

A summary of the C and n parameters established from the linear regression analysis is presented in Table 3.18 for each watercourse. To compute the design flow at a point of interest for headwater drainage areas, the following procedure is recommended:

i) measure the drainage area to the point of interest using suitable mapping

ii) from Table 3.18, select the appropriate C and n values for the desired recurrence interval or Timmins Storm flows

iii) substitute the values obtained in (i) and (ii) above in $Q = CA^n$ to obtain design flow

TABLE 3.17

Headwater Basins Used in Regression Analysis for Watercourses Indicated

Watercourse	Ref. No.	Description	Drainage Area (km^2)
Innisfil Creek/ Beeton Creek/ Bailey Creek	302	Outlet of Basin 302 & 300	40.9
	240	Outlet of Basin 303	65.4
	311	Outlet of Basin 311	24.1
	316	Outlet of Basin 315 & 316	35
	290	Outlet of Basin 317	45.6
	314	Outlet of Basin 314	11.7
	1025	Confluence of Basins 314 & 317	57.3
	300	Outlet of Basin 318	60.4
	209	Outlet of Basin 209	27.7
	208	Outlet of Basin 208	24.7
	201	Outlet of Basin 201	23.4
	203	Outlet of Basin 203	11.3
	2100	Confluence of Basins 202 & 204	34.6
Mad River	800	Outlet of Basin 800	42.8
	670	Outlet of Basin 801	74.5
	804	Outlet of Basin 804	38.6
	710	Outlet of Basin 805	70.0
	810	Outlet of Basin 810	35.3
	720	Outlet of Basin 806	90.1
	814	Outlet of Basin 814	18.4
	790	Outlet of Basin 815	39.4
	680	Outlet of Basin 680	88.7

TABLE 3.17 (cont'd)

Headwater Basins Used in Regression Analysis for Watercourses Indicated

Watercourse	Ref. No.	Description	Drainage Area (km^2)
Upper Nottawasaga River	101	Outlet of Basin 101	27.2
	102	Outlet of Basin 102	10.4
	1001	Confluence of Basins 101 & 102	37.6
	114	Outlet of Basin 114	36.8
	1011	Confluence of Basins 114 & 116	51.0
	90	Outlet of Basin 116	66.3
	100	Outlet of Basin 117	100.7
Boyne River	400	Outlet of Basin 400	30.5
	401	Outlet of Basin 401	15.2
	1002	Confluence of Basins 400 & 401	45.7
	330	Outlet of Basin 402	57.4
	404	Outlet of Basin 404	21.1
	409	Outlet of Basin 409	26.1
	411	Outlet of Basin 411	5.21
Pine River	501	Outlet of Basin 501	74.9
	510	Outlet of Basin 502	104.2
	506	Outlet of Basin 506	21.8
	508	Outlet of Basin 508	29.6
	507	Outlet of Basin 507	35.2
	510	Outlet of Basin 502	8.5
	1101	Confluence of Basins 502 & 503	133.9

TABLE 3.17 (cont'd)

Headwater Basins Used in Regression Analysis for Watercourses Indicated

Watercourse	Ref. No.	Description	Drainage Area (km^2)
Georgian Bay Catchments	BAC 902	Outlet of Basin 902	11.1
	BAC 1510	Outlet of Basin 903	26.85
	Batteaux 909	Outlet of Basin 909	30.2
	Lamont 913	Outlet of Basin 913	26.9
	Warrington 915	Outlet of Basin 915	29.9
	Pretty 98	Outlet of Basin 906	43.4
	Silver 900	Outlet of Basin 900	20.3
	BAC 9012	Outlet of Basin 901B	1.65
	BAC 9013	Outlet of Basin 901C	0.85
Willow Creek	700	Outlet of Basin 700	16.1
	810	Outlet of Basin 702	25.2
	701	Outlet of Basin 701	27.3
	712	Outlet of Basin 712	23.6
	714	Outlet of Basin 714	39.8
	715	Outlet of Basin 715	20.5
	1400	Confluence of Basins 702 & 701	52.5

TABLE 3.18

SUMMARY OF LINEAR REGRESSION ANALYSIS PARAMETERS FOR HEADWATER DRAINAGE AREAS

WATERCOURSES	1 in 5 Year		1 in 10 Year		1 in 20 Year		1 in 50 Year		1 in 100 Year		Timmins	
	C	n	C	n	C	n	C	n	C	n	C	n
Innisfil/Beeton/Bailey Creeks	0.556	0.751	0.683	0.753	0.826	0.750	0.993	0.753	0.112	0.757	5.36	0.709
Upper Nottawasaga River	0.639	0.652	0.820	0.643	0.957	0.647	1.19	0.644	1.35	0.646	7.02	0.616
Boyne River	0.767	0.894	1.05	0.883	1.47	0.851	1.79	0.869	2.31	0.842	7.11	0.638
Pine River	0.228	1.00	0.289	0.999	0.332	0.999	0.396	0.998	0.443	0.999	0.928	1.00
Mad River	2.43	0.489	2.99	0.489	3.51	0.489	4.24	0.489	4.81	0.489	20.86	0.423
Willow Creek	0.553	0.880	0.624	0.880	0.698	0.878	0.789	0.878	0.879	0.872	3.21	0.959
Georgian Bay Inflows	1.58	0.817	2.02	0.808	2.51	0.790	3.37	0.749	3.99	0.729	9.33	0.754

Equation : $Q = CA^n$

where Q = peak discharge (m^3/s)

A = drainage area (Km^2)

c, n as given above

FIGURE 3.12 (a)

INNISFIL / BEETON/BAILEY CREEKS

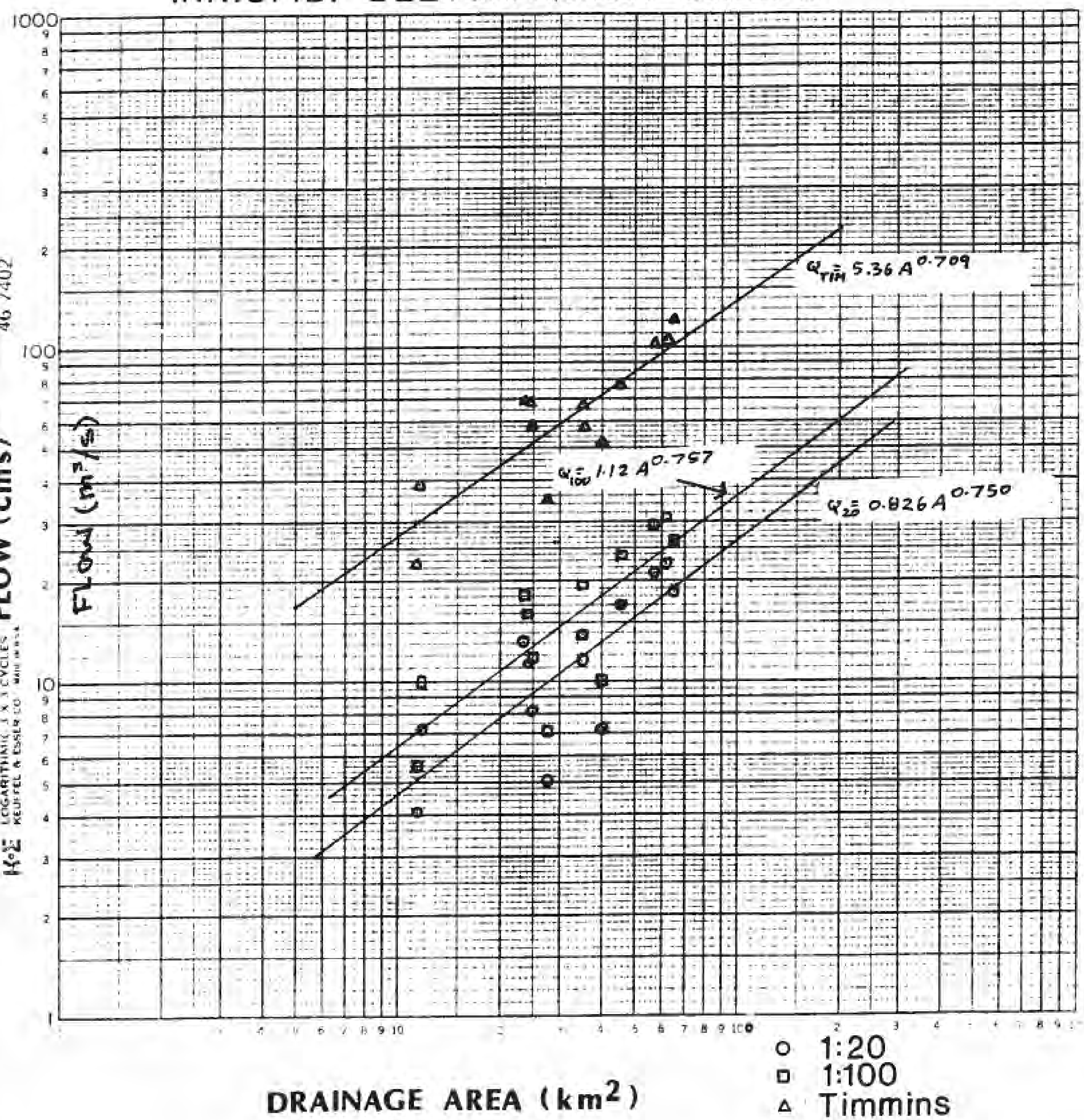


FIGURE 3.12(b)

UPPER NOTTAWASAGA

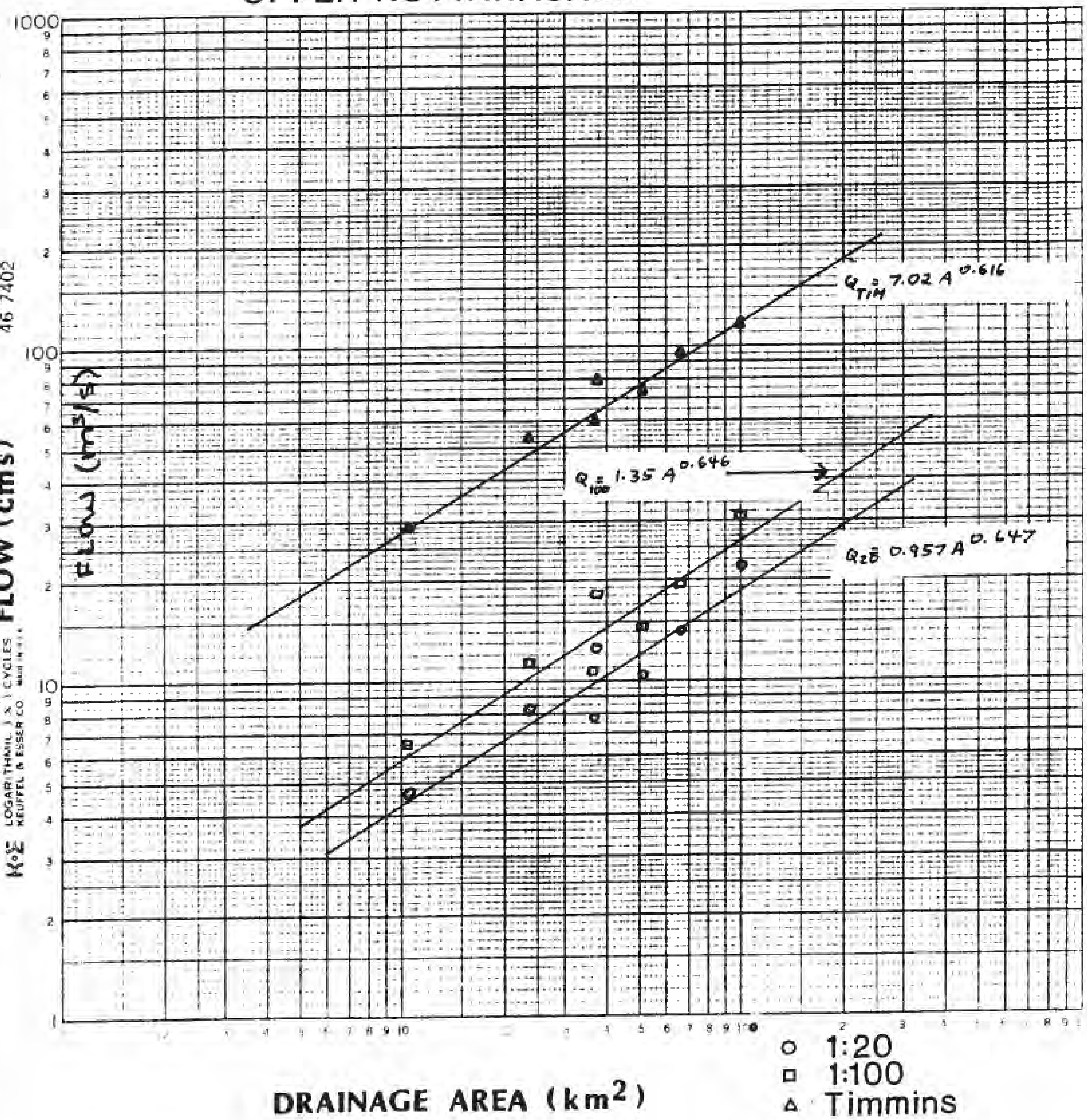


FIGURE 3.12 (c)

BOYNE RIVER

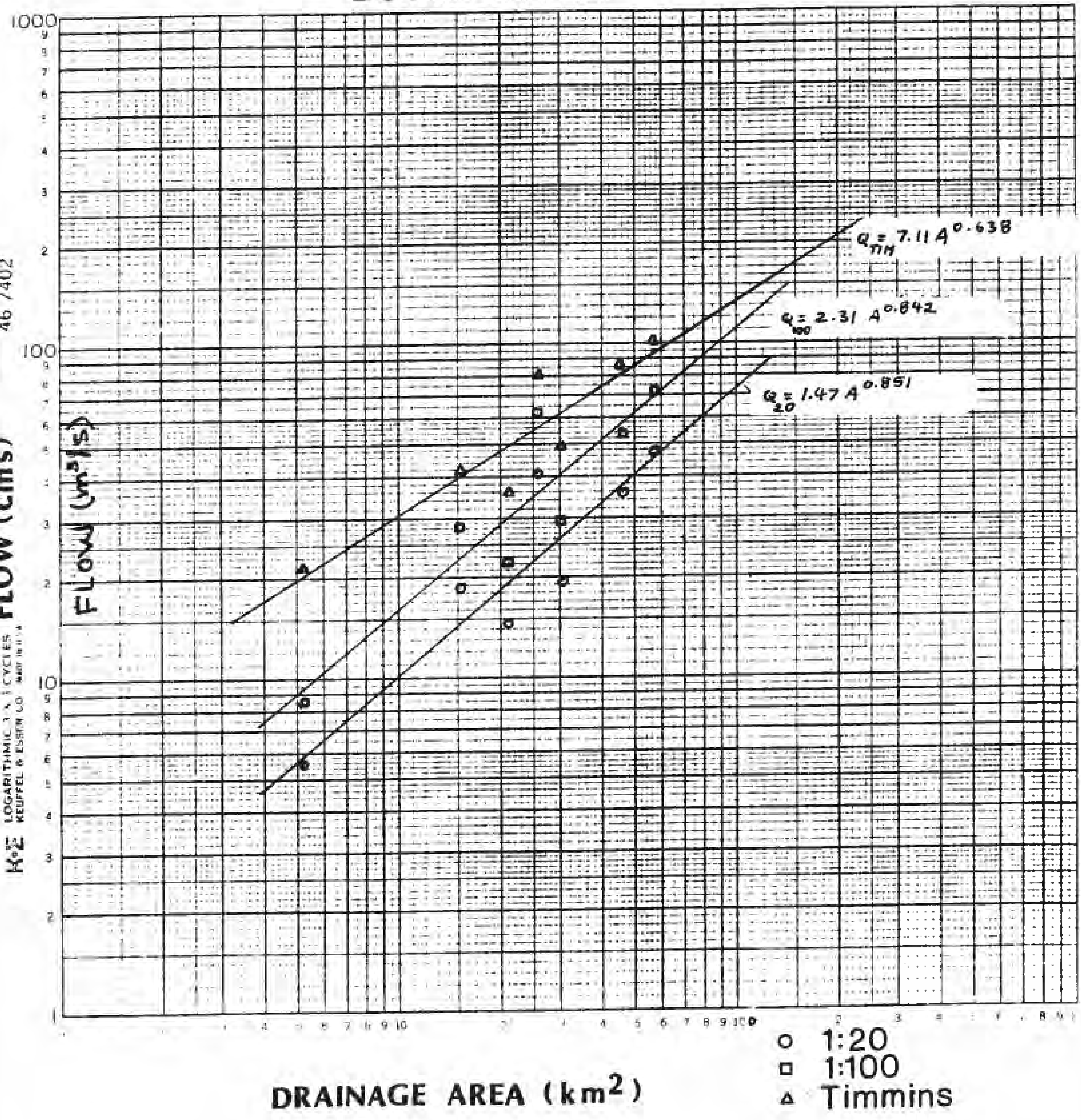


FIGURE 3.12(d)

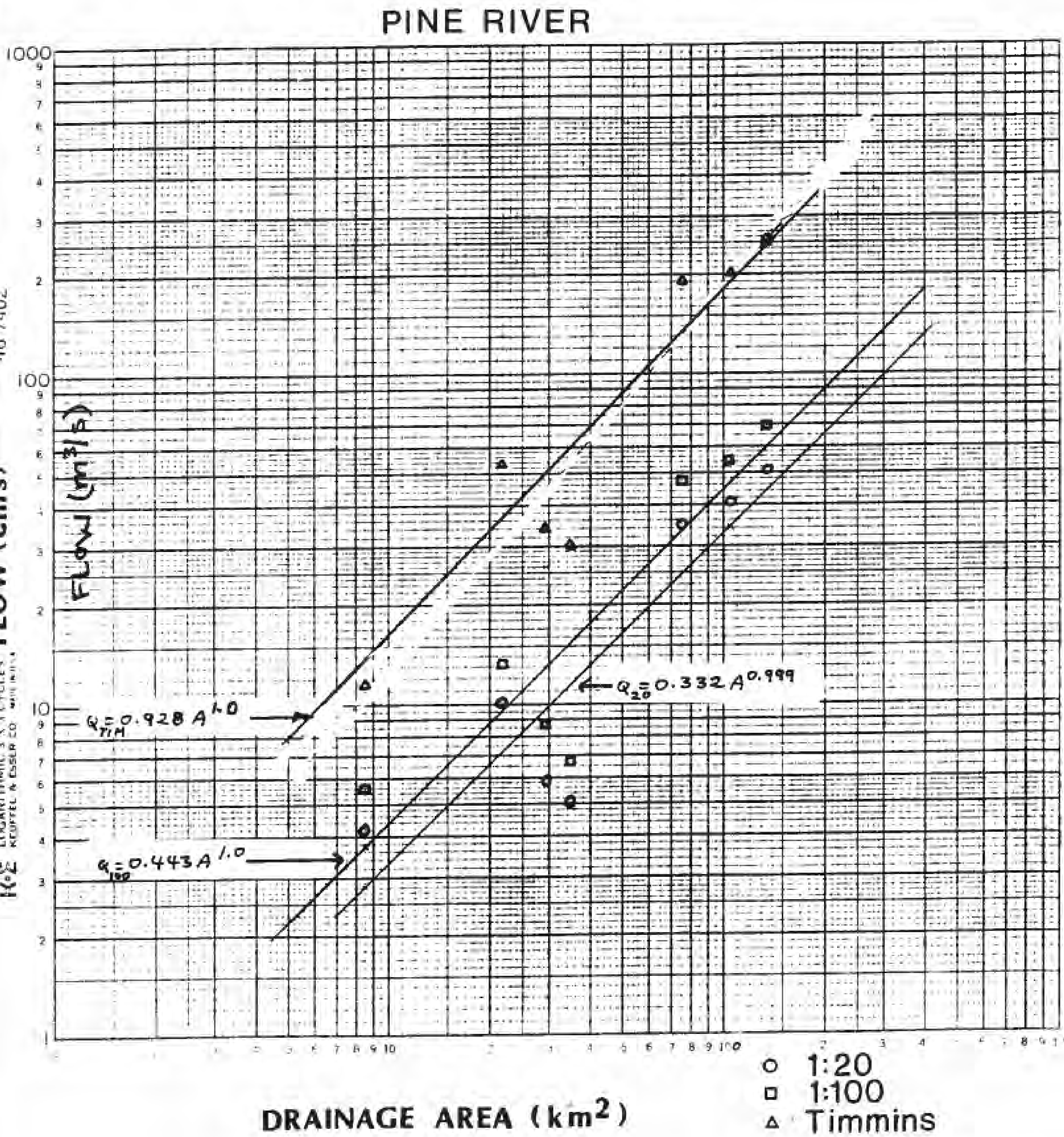

PINE RIVER
FLOW (cfs),
FLOW (m^3/s)
DRAINAGE AREA (km^2)
$Q_{TIM} = 0.928 A^{1.0}$
$Q_{100} = 0.443 A^{1.0}$
$Q_{20} = 0.332 A^{0.999}$
1000
100
10
1
1:20
1:100
Timmins
K&E LOGARITHMIC 3 X 3 CYCLES
KEUFFEL & ESSER CO.
46 7402

FIGURE 3.12 (e)

WILLOW CREEK

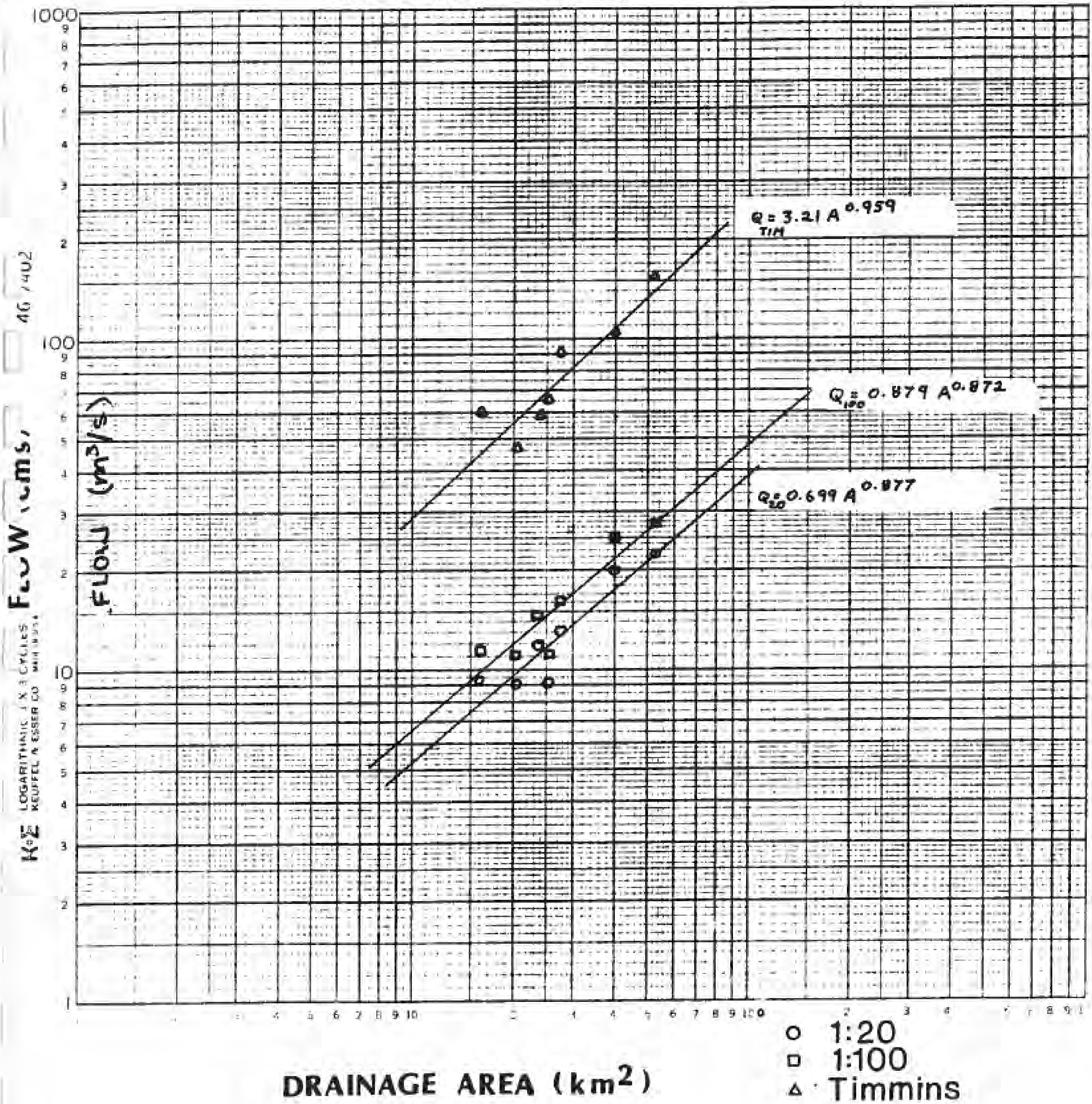


FIGURE 3.12(f)

MAD RIVER

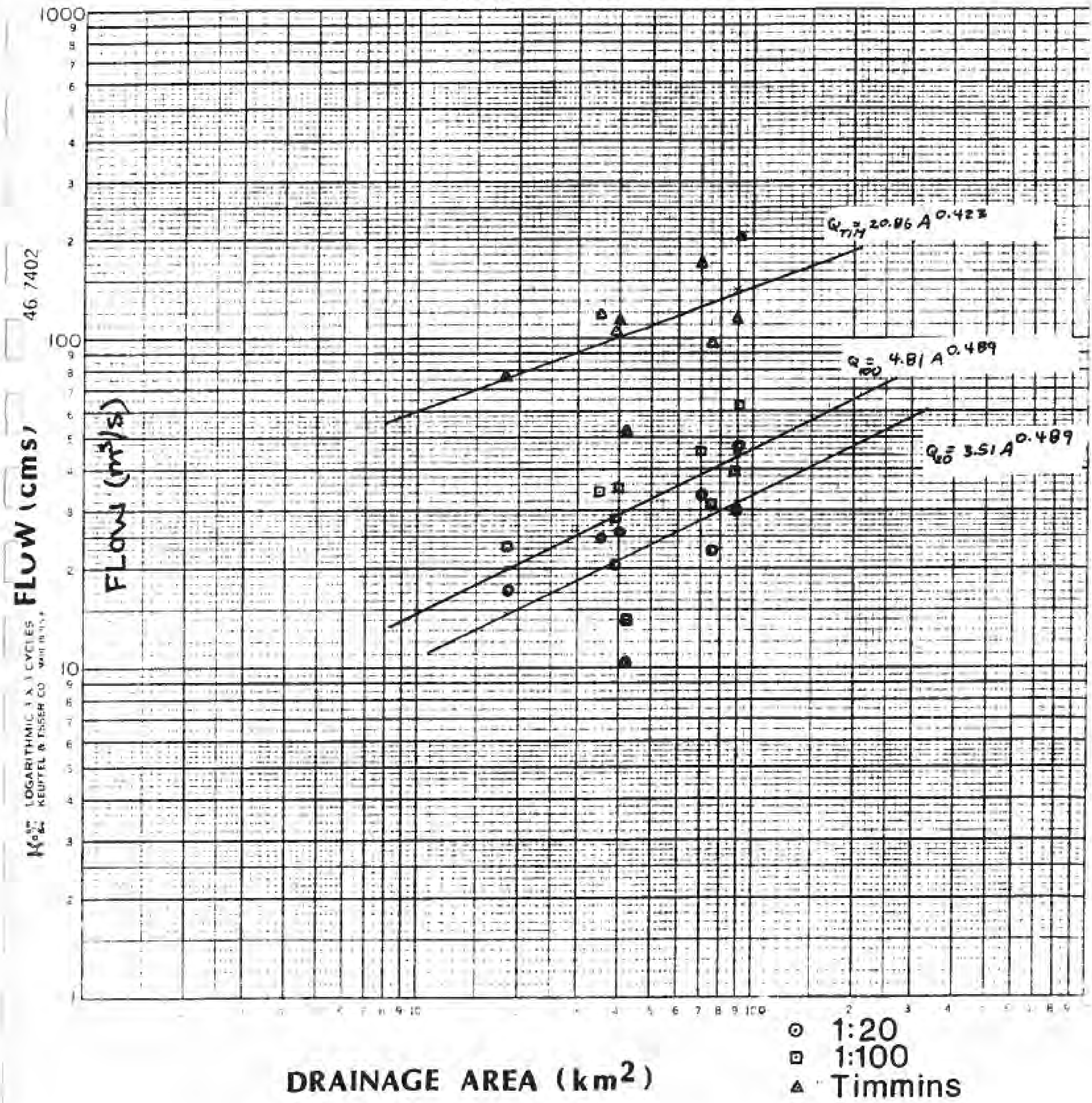
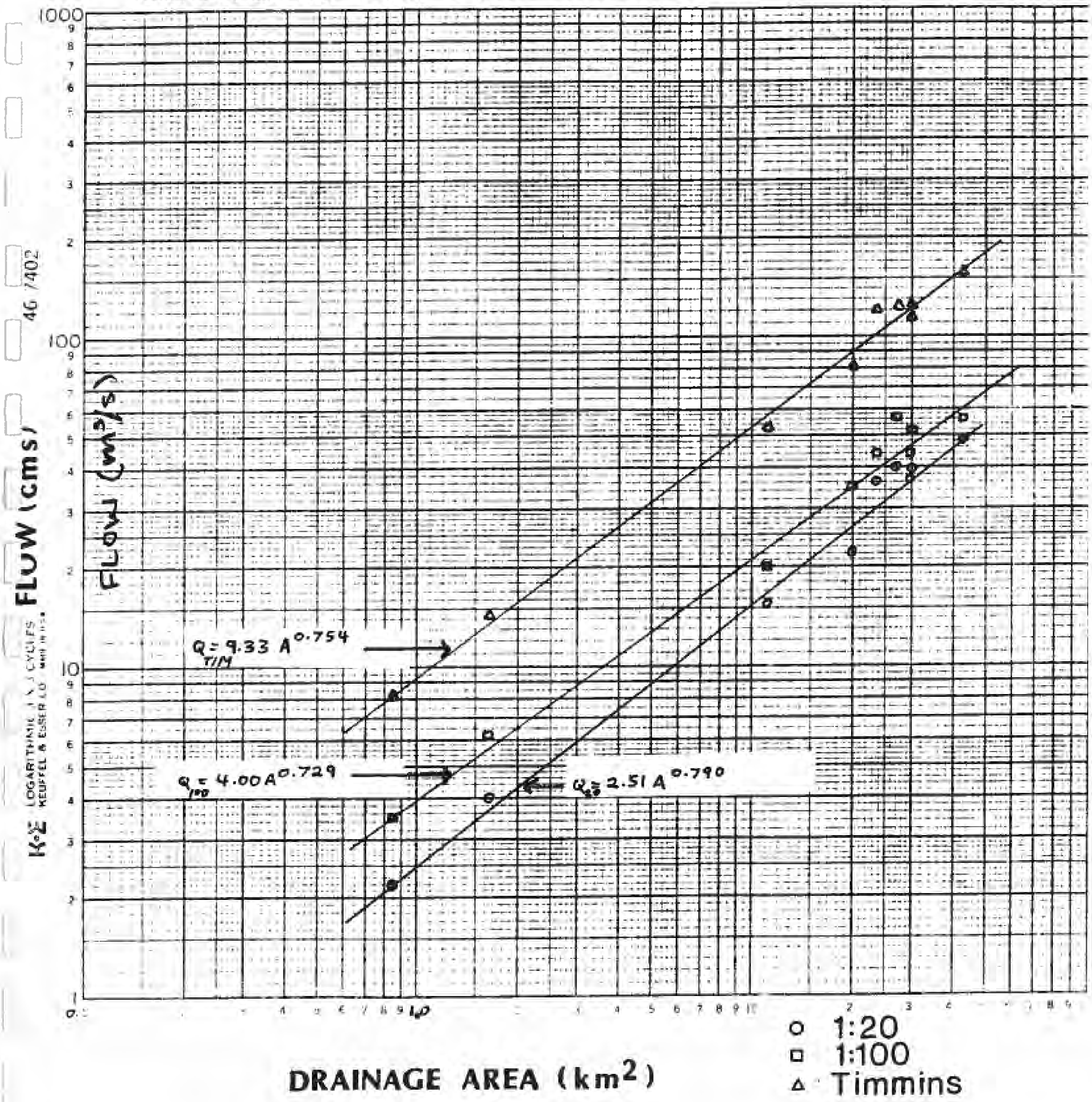


FIGURE 3.12(g)

GEORGIAN BAY CATCHMENTS



iv) repeat steps (ii) and (iii) for other recurrence interval flows or Timmins Storm flow

The above procedure is presented in flow chart format in Figure 3.12(h).

3.3.4.2 Major Waterways

Flow points along the major watercourses have been established at fairly close intervals as indicated in Figure 3.11. The change in flows from one flow point to the next point downstream is usually not very great. Therefore, to obtain design flows for intermediate points along the major watercourses, it is recommended that the flows be pro-rated linearly based on the incremental drainage area for those occasions where peak flows increase in a downstream direction. Careful attention must be taken when evaluating the tributary drainage at intermediate locations. Topographic maps are to be used to determine the portion of the incremental drainage area between flow points used in this study (Figure 3.11) which is tributary to the waterway upstream of the location of interest. Linear interpolation of flows based on stream length between flow points is not recommended as a computational procedure since the incremental drainage area may not be proportional to stream length. In those locations where design flows calculated at flow points decrease in the downstream direction, flow estimation at intermediate locations should be based on a linear interpolation of stream length between the flow points. Flow routing effects are considered of primary importance in attenuating peak flows in a downstream direction and this is most readily reflected in the stream length.

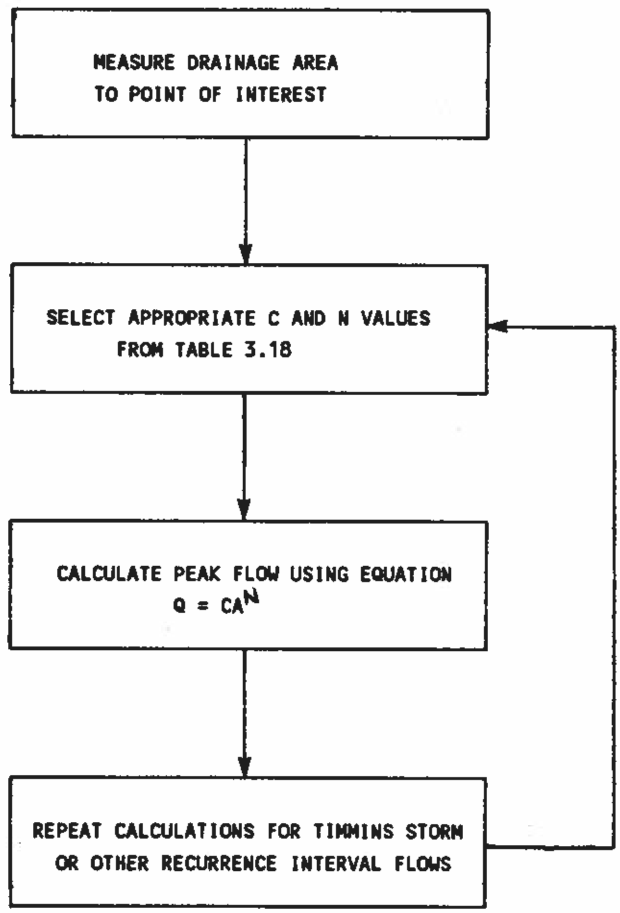
The above procedure is presented in flow chart format in Figure 3.12(i).

3.3.5 Flow Hydrographs

In order to establish design flows along the lower Nottawasaga River downstream of Minesing Swamp, it is necessary to carry out dynamic flow modell-

FIGURE 3.12 (h)

PROCEDURE TO CALCULATE FLOWS FOR HEADWATER DRAINAGE AREAS



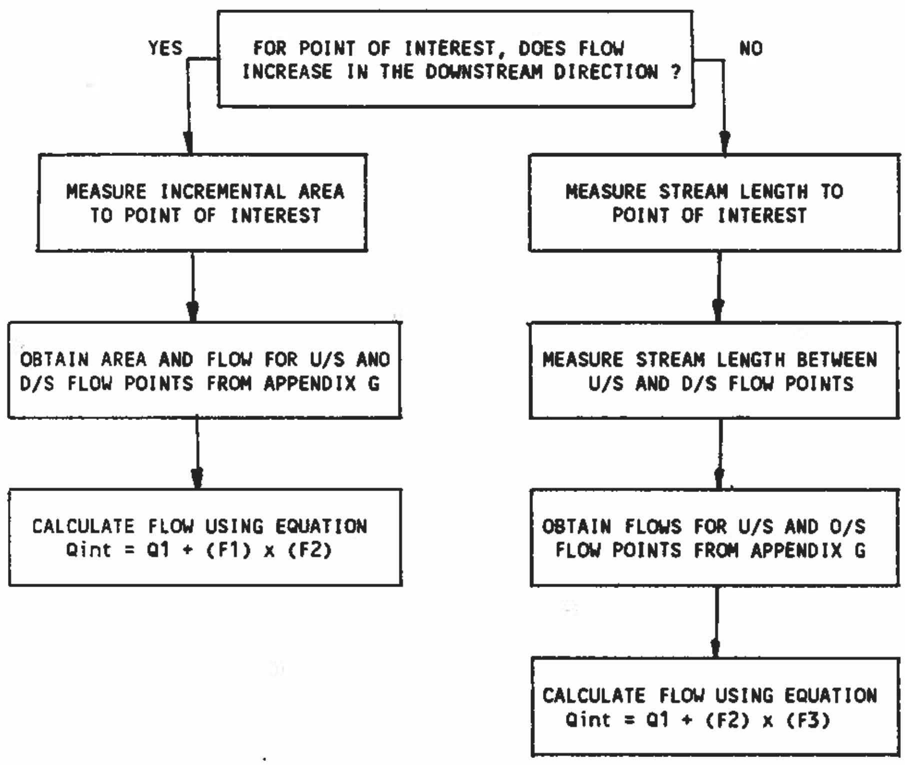
WHERE : Q = PEAK DISCHARGE (M^3/S)

A = DRAINAGE AREA (KM^2)

N and C VALUES ARE AS PRESENTED IN TABLE 3.18

FIGURE 3.12 (i)

PROCEDURE TO CALCULATE INTERMEDIATE FLOWS FOR MAJOR WATERWAYS



WHERE : Qint = DESIRED FLOW AT POINT OF INTEREST (M^3/S)

Q1 = FLOW AT UPSTREAM FLOW POINT (M^3/S)

F1 = $\left(\frac{Aint - A1}{A2 - A1}\right)$ (AREA FACTOR)

Aint = TOTAL DRAINAGE AREA AT POINT OF INTEREST (KM^2) (TOTAL AREA AT UPSTREAM FLOW POINT PLUS INCREMENTAL AREA)

A1,A2 = TOTAL DRAINAGE AREAS (KM^2) AT UPSTREAM AND DOWNSTREAM FLOW POINTS, RESPECTIVELY

F2 = (Q2-Q1) (FLOW FACTOR)

Q1,Q2 = FLOW (M^3/S) AT UPSTREAM AND DOWNSTREAM FLOW POINTS RESPECTIVELY

F3 = (L1/L2) (LENGTH FACTOR)

L1 = STREAM LENGTH FROM UPSTREAM FLOW POINT TO POINT OF INTEREST (KM)

L2 = STREAM LENGTH FROM UPSTREAM FLOW POINT TO DOWNSTREAM FLOW POINT (KM) ing using the DWOPER Model. The model requires inflow hydrographs to Minesing Swamp and local inflow for the lower reaches of Nottawasaga River. Due to the storage attenuation effect of the Minesing Swamp on design flows within the lower Nottawasaga River, careful evaluation of inflow volumes to the Swamp was required. The methodology that will be used to develop flow hydrographs is discussed below.

3.3.5.1 Inflow to Minesing Swamp

The major source of flow contribution to Minesing Swamp is from the Nottawasaga River, Pine River, Mad River and Willow Creek. To ensure volumes contained within the design inflow hydrographs were in close agreement with historical observations, Environment Canada conducted volume frequency analyses based on historical flow records. The 7-day and 10-day maximum annual flows for the period of record were selected for Nottawasaga River near Baxter, Mad River near Glencairn, Pine River near Everett and Willow Creek above Little Lake. For each hydrometric station, the maximum annual 7-day and 10-day volumes were calculated. Subsequently frequency analyses were carried out for the 7-day and 10-day volumes for each hydrometric station.

The volumes calculated by Environment Canada for each recurrence interval (1 in 5 to 1 in 100 year) were pro-rated on a drainage area basis from each gauging station to the inflow to Minesing Swamp. The 7-day volumes were selected for flood analysis since this duration was felt to be representative of high flow events on the Nottawasaga River system.

Since the above-noted watercourses are gauged, it is possible to develop dimensionless hydrographs using flow records. For each of the hydrometric stations, the hourly flow data from 1976 to 1984 was screened. High flow spring events were selected and plotted on the same graph paper for each hydrometric station. The observed hydrographs were subsequently reduced to dimensionless hydrographs. From the plotted dimensionless hydrographs, a typical dimensionless hydrograph was drawn representing the "average" shape of the observed hydrographs for each gauging station. The design hydrographs for each watercourse at the inflow to Minesing Swamp were obtained by multiplying the ordinate of each dimensionless hydrograph by the 1 in 5, 1 in 10, 1 in 20, 1 in 50 and 1 in 100 year peak flows presented in Appendix G. Where discrepancies in volumes occurred, the ordinates of the design hydrographs were adjusted to agree with the volumes determined by Environment Canada.

Design inflow hydrographs to Minesing Swamp for Nottawasaga River, Pine River, Mad River and Willow Creek are shown in Appendix J.

The Timmins Storm flows under present and future conditions were generated using the QUALHYMO Model. The flow hydrographs are available at inflow points to Minesing Swamp for the four watercourses identified above. These inflow hydrographs were used as input to the DWOPER Model to establish the Timmins Storm flows downstream of the Swamp for both present and future conditions.

3.3.5.2 Lower Reaches of Nottawasaga River

The sub-catchments downstream of Minesing Swamp draining directly to the Nottawasaga River are not gauged. Consequently to develop local inflow hydrographs for these sub-catchments, it was necesesary to simulate the hydrographs based on historical precipitation input. This approach was similar to that used for the Georgian Bay sub-catchments described in Section 3.3.2.1.1.

Five spring high flow events were selected to develop a dimensionless hydrograph for each tributary or local sub-catchment. The same procedure outlined in Section 3.3.5.1 was used to obtain local inflow hydrographs for each sub-catchment/tributary shown in Figure 3.11. Design flow hydrographs at the outlet of some of the sub-catchments are shown in Appendix J.

The Timmins Storm flow hydrographs simulated using the QUALHYMO Model were used as input to the DWOPER Model.

3.4 Innisfil Creek Investigation

3.4.1 Agricultural Drainage Improvements

During the Nottawasaga River Hydrology Study, the effect of agricultural drainage improvements within the Innisfil Creek basin on downstream peak flows was investigated. Concern has been expressed in recent years that construction and upgrading of municipal drains in the Innisfil watershed has resulted in more frequent and more severe flood flows within the Nottawasaga River at Beeton Flats due to increased flow velocity in the waterway and reduced response time of the basin to rainfall. These flood prone lands adjacent to the confluence of the Nottawasaga River and Bailey Creek within Tecumseth Township are under active cultivation.

A proposed major municipal drain on Innisfil Creek and the Nottawasaga River reaching from 140 metres upstream of the Bailey Creek confluence to a point 2400 metres downstream of the Innisfil Creek and Nottawasaga River confluence was the subject of a second hydrological investigation.

An inventory of existing municipal drains in the Innisfil Creek basin was obtained from township maps (Tecumseth, Innisfil, and West Gwillimbury) prepared by the Ontario Ministry of Agriculture and Food and reviewed with the Project Committee. This information is documented in Figure 3.13. An examination of applications for agricultural drainage improvements under the

Drainage Act from the late 1970's to the present indicates that most projects are comprised of upgrading or cleaning of existing municipal drains.

The Contract drawings for the proposed Innisfil Creek and Nottawasaga River drainage works provided the channel dimensions for the investigation of this municipal drain.

Hydrologic investigations of agricultural drainage impacts during the study focussed on municipal drain improvements and was restricted to those drains on the major waterways consisting of Innisfil Creek, the lower reaches of Penville Creek, Bailey Creek, Cookstown Creek, and the Nottawasaga River. These stream reaches were represented in the watershed hydrologic model (QUALHYMO) and could be studied with respect to flow travel times and attenuation of flows due to routing and storage effects. The remaining municipal drains within the watersheds that are tributary to Beeton Flats were simulated indirectly in the hydrologic model by empirical runoff response relationships which are related to physiographic characteristics of the subwatersheds. For this reason, it was not possible to study the effect of improvements to the drains that are located on local watercourses; nevertheless, their effect on the flow magnitudes within the Beeton Flats area was considered small in relation to the major waterways.

For purposes of the investigation of improved drains tributary to Beeton Flats, field measurements of cross-sections on Innisfil Creek, Cookstown Creek, the lower reaches of Penville Creek and the Nottawasaga River (Figure 2.2) were left unaltered but the observed conveyance roughness was changed to reflect a system of well-maintained drains which have been cleaned and straightened. A Manning's "n" value of 0.02 was adopted for flow purposes.

During the examination of the flow impacts caused by the proposed Innisfil Creek and Nottawasaga River drainage works, the existing natural waterway was replaced in the flow routing computations by a representative channel cross-section. A Manning's "n" value of 0.02 was employed for the channelized reach.

3.4.2 Impacts on Flood Hydrology

In order to assess the effects of the foregoing municipal drain improvements on the peak flows within the Innisfil Creek at Beeton Flats, rainfall for two summer events (July 28, 1980 and August 15, 1986) were selected and used as input to the rainfall-runoff model of the watershed. These two events contain the largest summer rainfalls in recent years.

Hydrographs of these two events at Innisfil Creek, south of Cookstown and the confluence with Beeton Creek, are shown on Figures 3.14 and 3.15 for the 1986 and 1980 events respectively. A detailed summary of flows under the existing drain conditions and after cleaning is presented in Tables 3.19 and 3.20.

The analysis of the existing municipal drains on Innisfil Creek under existing conditions and after cleaning has indicated that resultant peak flows during two summer rainfall events increase only marginally with the increment never exceeding one or two percent, within the Beeton Flats waterway.

The areal definition of the August 15, 1986 rainfall event over the upper Nottawasaga River basin was not sufficient for hydrologic modelling purposes especially in view of the marked variation in local rainfall intensities that were experienced. The hydrologic investigation of the Innisfil Creek and Nottawasaga River drainage works therefore focussed on the July 28, 1980 event. The construction of the proposed municipal drain was found to increase the peak discharge for this event by less than one percent within the channelized reach (Table 3.21) and to marginally decrease the maximum downstream discharge due to slight changes in flow travel times.

Table 3.19

Depth of Flow

Innisfill Creek

Existing And Improved Municipal Drain Condition

Flow Point	X-Sect #	Existing Condition				Improved Conditions			
		1980		1986		1980		1986	
		Q (m^3/s)	depth (m)	Q (m^3/s)	depth (m)	Q (m^3/s)	depth (m)	Q (m^3/s)	depth (m)
302	11	3.16	0.80	1.09	0.53	3.16	0.72	1.09	0.49
250	11	10.44	1.25	5.73	0.84	10.55	1.14	5.93	0.81
260	11	10.90	1.28	5.81	0.97	11.16	1.16	6.07	0.90
270	11	6.78	0.68	2.95	0.52	6.78	0.63	2.95	0.46
280	11	19.42	1.55	9.43	1.12	19.84	1.38	9.72	1.02
300	6	10.23	1.24	4.83	0.97	10.23	1.16	4.83	0.89
310	11	32.78	2.16	15.48	1.57	32.91	1.98	15.71	1.47
140	5	6.47	0.72	1.18	0.33	6.53	0.65	1.22	0.31
200	0	13.25	0.92	2.95	0.47	13.32	0.92	3.00	0.47
1041	7	45.62	2.01	18.22	1.30	45.91	1.86	18.48	1.20
320	7	46.55	2.04	18.33	1.30	46.64	1.87	18.71	1.21

TABLE 3.20

PEAK FLOW: INNISFIL, BEETON, BAILEY CREEKS

EXISTING AND IMPROVED MUNICIPAL DRAIN CONDITION

DESIGN FLOWS:

REF. NO.	DESCRIPTION	TRIBUTARY AREA (km^2)	DISCHARGE (m^3/s) 1980 JULY EVENT EXISITING	PROPOSED	1986 AUGUST EVENT EXISTING	PROPOSED
INNISFIL						
302	Outlet of catchment 302 and 300	40.9	3.16	3.16	1.09	1.09
1020	Outlet of catchment 303	58.2	5.08	5.05	2.10	2.10
304	Outlet of catchment 304	7.2	0.99	0.99	0.61	0.61
1021	Confluence of Innisfil Creek at catchment 304	65.4	5.91	5.89	2.57	2.58
305	Outlet of catchment 305	14.6	2.15	2.15	1.35	1.35
240	Confluence of Innisfil Creek at catchment 305	80.0	7.81	7.81	3.79	3.81
250	Outlet of catchment 306	105.4	10.44	10.55	5.73	5.93
260	Outlet of catchment 310	112.7	10.90	11.16	5.81	6.07
311	Outlet of catchment 311	24.1	4.93	4.93	2.20	2.20
270	Outlet of catchment 312	36.8	6.78	6.78	2.95	2.95
1044	Confluence of catchment 312 and 310	149.5	17.68	17.87	8.64	8.87
280	Outlet of catchment 313	158.2	19.42	19.84	9.43	9.72

DESIGN FLOWS:

REF. NO.	DESCRIPTION	TRIBUTARY AREA (km^2)	DISCHARGE (m^3/s) 1980 JULY EVENT EXISITING	PROPOSED	1986 AUGUST EVENT EXISTING	PROPOSED
316	Confluence of catchment 315 and 316	35.0	6.27	6.27	2.68	2.68
290	Outlet of catchment 317	45.6	7.73	7.73	3.18	3.18
314	Outlet of catchment 314	11.7	3.01	3.01	2.77	2.77
1025	Confluence of catchment 314 and 317	57.3	9.67	9.67	4.56	4.56
300	Outlet of catchment 318	60.4	10.23	10.23	4.83	4.83
1045	Confluence of catchment 313 and 318	218.6	29.53	29.87	14.26	14.43
310	Outlet of catchment 321	249.9	32.78	32.91	15.48	15.71

DESIGN FLOWS:

REF. NO.	DESCRIPTION	TRIBUTARY AREA (km^2)	DISCHARGE (m^3/s) 1980 JULY EVENT EXISITING	PROPOSED	1986 AUGUST EVENT EXISTING	PROPOSED
BEETON/BAILEY CREEKS						
209	Outlet of catchment 209	27.7	1.14	1.14	0.28	0.28
208	Outlet of catchment 208	24.7	1.78	1.78	0.39	0.39
1028	Confluence of catchment 208 and 209	52.4	2.92	2.92	0.67	0.67
120	Outlet of catchment 210	54.9	3.17	3.17	0.71	0.71
130	Outlet of catchment 211	80.5	5.26	5.26	1.09	1.09
140	Outlet of catchment 212	99.2	6.47	6.53	1.19	1.22
213	Outlet of catchment 213	11.5	0.99	0.99	0.20	0.20
1032	Confluence of catchment 212 and 213	110.7	7.46	7.52	1.37	1.41
150	Outlet of catchment 214	113.9	7.56	7.60	1.36	1.40
201	Outlet of catchment 201	23.4	2.23	2.23	0.75	0.75
170	Outlet of catchment 202	33.6	2.85	2.85	0.89	0.89
203	Outlet of catchment 203	11.3	0.71	0.71	0.20	0.20
2100	Outlet of catchment 204	34.6	2.69	2.69	0.37	0.37
1035	Confluence of catchment 204 and 202	68.2	5.07	5.07	4.03	4.03
180	Outlet of catchment 205	84.7	5.32	5.32	2.69	2.69

DESIGN FLOWS:

REF. NO.	DESCRIPTION	TRIBUTARY AREA (km^2)	DISCHARGE (m^3/s) 1980 JULY EVENT EXISITING	1980 JULY EVENT PROPOSED	1986 AUGUST EVENT EXISTING	1986 AUGUST EVENT PROPOSED
BEETON/BAILEY/INNISFIL CREEKS						
190	Outlet of catchment 206	87.4	5.51	5.51	2.46	2.46
1039	Confluence of catchment 206 ANO 214	201.3	13.05	13.09	2.92	2.95
200	Outlet of Beeton and Bailey Creeks	204.8	13.25	13.32	2.95	2.10
1041	Confluence of Beeton and Innisfil Creeks	454.8	45.62	45.91	18.22	18.48
320	Outlet of Innisfil Creek	472.2	46.55	46.64	18.34	18.71

TABLE 3.21

PEAK FLOW IMPACT OF PROPOSED INNISFIL CREEK AND NOTTAWASAGA RIVER MUNICIPAL DRAIN

Flow Point	Description	Peak Flow Existing Waterway (m^3/s)	28 July 1980 event Proposed Municipal Drain (m^3/s)
320	Outlet of Innisfil Creek	46.7	46.8
1050	Downstream of confluence of Innisfil Creek and Nottawasaga River	68.4	68.5
220	Nottawasaga River upstream of Boyne River	69.8	69.6
1078	Nottawasaga River at Baxter	96.9	96.8
1252	Nottawasaga River at Angus (Highway 90)	125.1	124.8

Note: Channel dimensions

1. Upstream limit to confluence Innisfil Creek and Bailey Creek
   :Bottom width 6 metres
   Side slopes 2H:1V

2. Between confluences of Innisfil Creek with Bailey Creek and Nottawasaga River
   :Bottom width 12 metres
   Side slopes 2H:1V

3. Downstream of Innisfil Creek and Nottawasaga River confluence
   :Bottom width 15 metres
   Side slopes 2H:1V

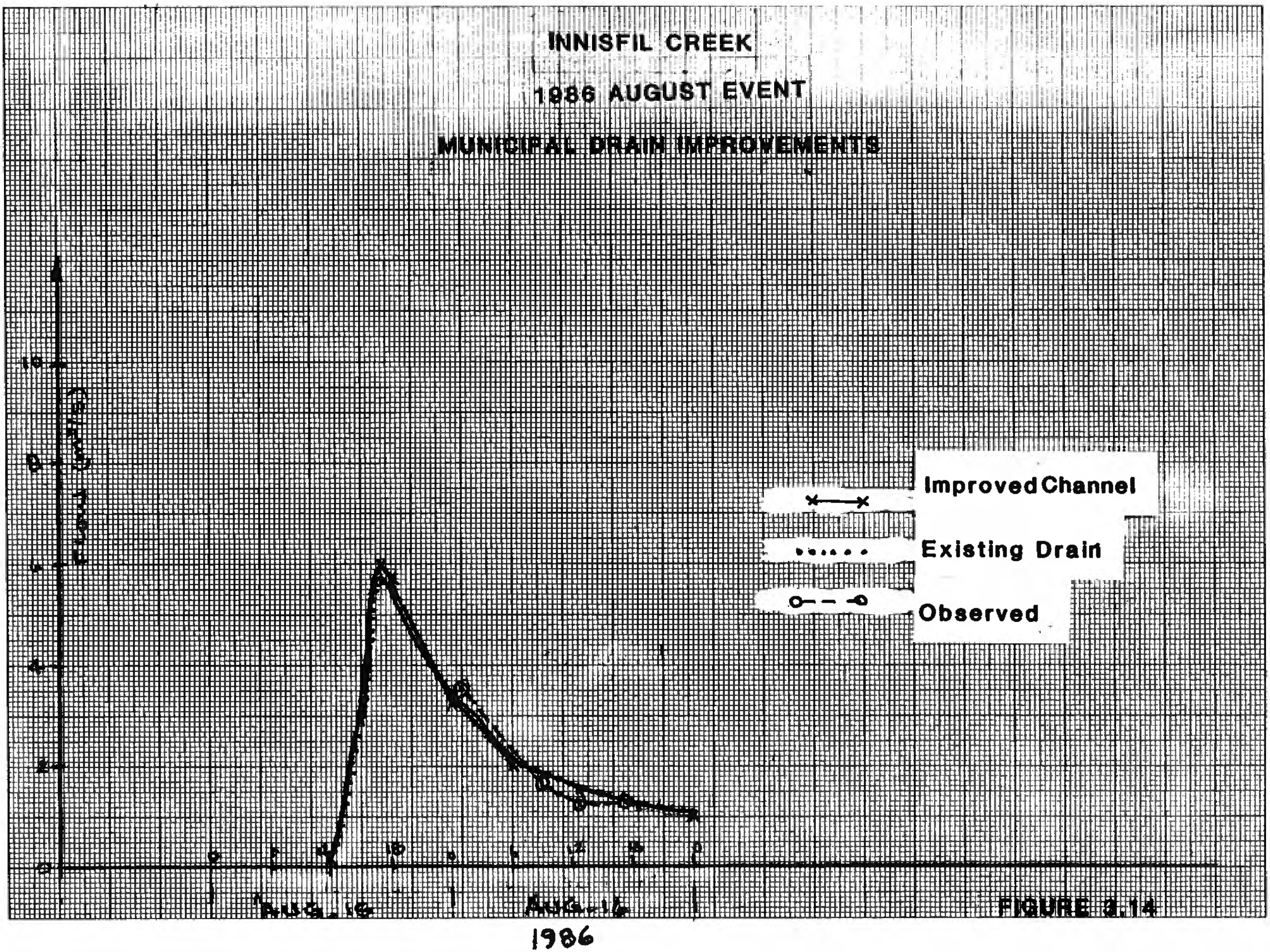
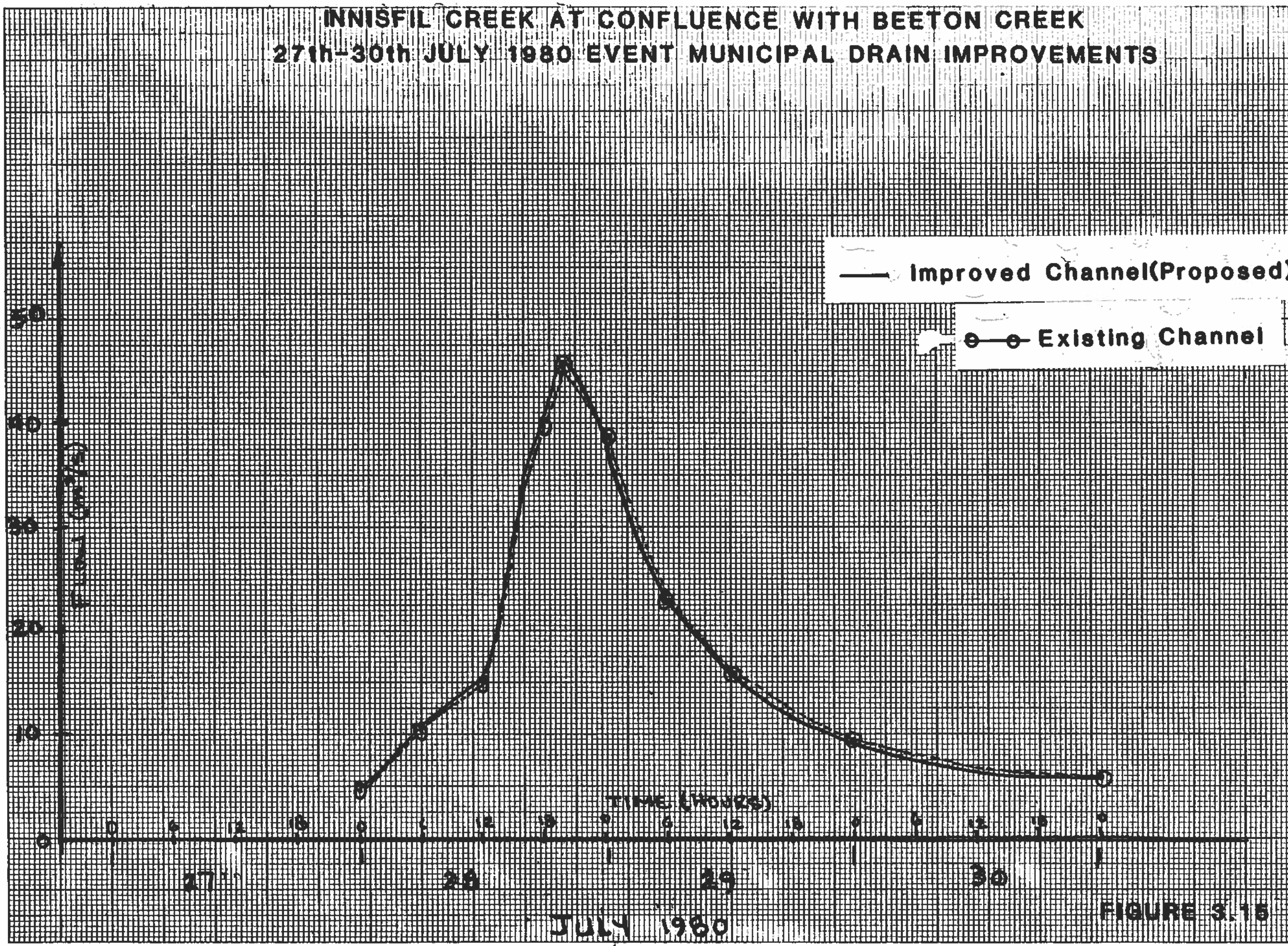

INNISFIL CREEK
1986 AUGUST EVENT
MUNICIPAL DRAIN IMPROVEMENTS
FLOW (m³/s)
10
8
6
4
2
0
Improved Channel
Existing Drain
Observed
AUG. 15
AUG. 16
1986

FIGURE 3.14



INNISFIL CREEK AT CONFLUENCE WITH BEETON CREEK
27th–30th JULY 1980 EVENT MUNICIPAL DRAIN IMPROVEMENTS
Improved Channel(Proposed)
Existing Channel
FLOW (m³/s)
0
10
20
30
40
50
TIME (HOURS)
0
6
12
18
27
28
29
30
JULY 1980

FIGURE 3.15

4.0 DYNAMIC FLOW MODELLING

4.1 Introduction

The streamcourse of the Nottawasaga River between Angus and Edenvale is characterized by a large off-channel flood storage area called the Minesing Swamp. Downstream of Edenvale the stream channel is quite flat and contains a couple of small flood storage areas called Jack's Lake and Doran Lake. Due to the significant off-channel storage areas and the flat channel slopes it is not possible to determine flow routing effects using standard hydrologic routing techniques such as the lag and route and storage indication methods which do not account for flow continuity (i.e., off-channel storage effects)and the influence of backwater. Flow routing in the Lower Nottawasaga River system can only be reliably determined using unsteady, non-uniform flow modelling techniques which account for flow continuity and conservation of momentum.

The Dynamic Wave Operational Model (DWOPER) was selected for this purpose.

4.2 Description of Model

DWOPER is a dynamic wave routing model based on an implicit finite difference solution of the complete one-dimensional St. Venant equations for unsteady flow. The model was developed by the United States National Weather Service primarily for flood and day-to-day river forecasts.

The model is generalized for wide applicability to rivers of varying physical features such as irregular channel geometry, variable channel roughness, lateral inflows, flow diversions, off-channel storage, local head losses such as bridge contraction-expansion, lock and dam operations, and wind effects.

The model possesses a highly efficient automatic calibration feature for determining channel roughness factors based on observed hydrographs along the streamcourse.

Boundary conditions at the upstream and downstream limits of the model can be specified as either stage or discharge hydrographs. The downstream boundary conditions can also be specified by a known relationship between stage and discharge such as a rating curve.

DWOPER has the capability to model a dendritic river system consisting of the main channel and its tributaries. If only the main channel is of interest then tributary inflow can be specified as lateral inflows.

4.3 Geometric Properties of the Streamcourse

4.3.1 Cross-sections

A field survey of the streamcourse was conducted to obtain channel cross-sections at representative locations. The river channel between Highway No. 90 near Angus and the bridge at Edenvale was surveyed from July 15 to 17, 1987. The remainder of the channel downstream of Edenvale was surveyed from October 15 to 18, 1987. A total of fifty-two (52) cross-sections were surveyed.

Several of the surveyed cross-sections are very closely spaced and the inclusion of all of the measured sections in the DWOPER model would require extremely short simulation time steps with the associated high computer costs. Hence, only thirty-two (32) of the surveyed cross-sections were used in the DWOPER simulations. These were sufficient to represent the geometric properties of the streamcourses and their locations are shown in Figure 4.1.

The spacing of the cross-sections is variable at an average distance of 1,500 m. The minimum and maximum cross-sectional spacings are 340 m and 5,843 m respectively. The thirty-two selected cross-sections were considered sufficient to adequately represent the stream geometry and avoid the need for extremely short time steps with the associated high computer costs.

TABLE 4.3 - SUMMARY OF PEAK FLOW AND PEAK STAGE AT SELECTED LOCATIONS

LOCATION	CROSS SECTION NUMBER	PEAK FLOW AND ASSOCIATED STAGE						PEAK STAGE AND ASSOCIATED FLOW						
		5-YR	10 YR	20 YR	50 YR	100 YR	REGIONAL	5-YR	10 YR	20 YR	50 YR	100 YR	REGIONAL	
Edenvale	012	161 183.46	180 183.73	201 184.00	224 184.28	247 184.49	243 185.05	159 183.46	178 183.73	199 184.00	223 184.28	244 184.49	237 185.07	flow (m^3/sec) stage (m)
Bridge D/S Doran Lake	D17	160 182.75	180 183.00	200 183.26	224 183.52	247 183.72	256 184.11	158 182.76	178 183.01	199 183.26	223 183.52	243 183.72	254 184.55	flow (m^3/sec) stage (m)
Bridge D/S Jack Lake	D23	158 182.13	178 182.36	199 182.58	223 182.81	242 182.98	340 183.98	158 182.13	178 182.36	199 182.58	223 182.81	242 182.98	330 184.01	flow (m^3/sec) stage (m)
Schooner-Town Bridge	D27	213 178.03	242 178.18	289 178.39	317 178.52	336 178.59	469 179.06	213 178.03	242 178.18	289 178.39	317 178.52	336 178.59	469 179.06	flow (m^3/sec) stage (m)
Wasaga Beach	D30	211 177.38	240 177.46	288 177.59	318 177.69	338 177.75	469 178.08	211 177.38	240 177.46	288 177.59	318 177.69	338 177.75	464 178.08	flow (m^3/sec) stage (m)

FIGURE 4.1

LOCATION OF CROSS SECTIONS

Based on the DWOPER cross-section numbering scheme, the cross-sections are numbered consecutively starting at the upstream end. A tabulation of cross-section numbers and the corresponding distances from the river mouth is given in Table 4.1. Cross-sectional plots are shown in Appendix M.

During the field survey (when the river was not in flood) the average top width of the channel was 35 m. The narrowest channel sections were at the upper and through the Minesing Swamp where the channel was approximately 20 m wide. The river was widest along the reach through Schoonertown and Wasaga Beach where it was approximately 100 m wide.

The average flow depth (at the thalaweg) was 3 m. The shallowest reach occurred through the Minesing Swamp where it was 1 to 2 m in depth and the deepest sections occurred along the reach between Edenvale and Jack's Lake where the flow depth is 4 to 6.5 m.

The average longitudinal channel slope is 25×10^{-5} through the Minesing Swamp. Except for a short steeper section just downstream of the confluence with Lamont and Warrington Creeks, the remainder of the streamcourse is relatively flat with an average slope of 36×10^{-6}.

4.3.2 Off-channel Storage

The Minesing Swamp represents a significant dead water storage area which has a significant attenuating effect on flood peaks entering at Angus and at major tributaries such as Willow Creek and the Mad River. Due to the extent and heavily wooded nature of the swamp it was not possible to obtain the access required for a field survey. The cross-sectional properties of the swamp had to be abstracted from available topographic mapping. 1:10000 scale maps are available for the southern one-third of the swamp, however, only 1:50000 scale maps are available for the remainder. The topographic information from the 1:50000 scale maps were supplemented by spot elevations provided by the Ontario Ministry of Natural Resources Surveys and Mapping

TABLE 4.1

CROSS-SECTION DATA, LOWER NOTTAWASAGA RIVER

	Cross-section No.	Distance from River Mouth (m)	Manning's Channel Roughness
	1	45,900	0.02
	2	44,400	0.02
	3	42,150	0.02
Storage from Minesing Swamp	4	40,900	0.035
	5	39,650	0.035
	6	38,150	0.035
	7	36,400	0.035
	8	34,650	0.035
	9	33,400	0.035
	10	31,900	0.035
	11	31,025	0.035
	12	29,910	0.035
	13	28,150	0.035
	14	26,900	0.035
	15	25,525	0.035
Doran Lake	16	24,650	0.035
	17	23,775	0.035
	18	21,400	0.035
	19	20,271	0.035
	20	19,275	0.035
Jack's Lake	21	17,275	0.035
	22	16,525	0.035
	23	15,650	0.035
	24	14,450	0.035
	25	13,400	0.035
	26	11,450	0.035
	27	5,607	0.02
	28	4,000	0.02
	29	1,750	0.02
	30	1,090	0.02
	31	750	0.02
	32	0	0.02

Branch. The banks of the Nottawasaga River exhibit levee-like forms through the Minesing Swamp. Hence, channel conveyance is confined mostly between the banks and overbank areas are designated as off-channel storage areas. A conceptual sketch of the convergence and off-channel storage areas is shown in Figure 4.2.

Off-channel storage areas are specified at cross-sections 1 to 8 (inclusive). Storage areas (specified as top width vs. elevation) for cross-section 1 to 4 were obtained from 1:10000 scale maps whereas storage areas for cross-sections 5 to 8 were obtained from the 1:50000 scale maps.

Doran Lake and Jack's Lake are two small flood storage areas downstream of Edenvale. These are not expected to have a significant impact on the hydrologic routing but are, nevertheless, included at cross-sections 16 and 21 respectively.

4.4 Boundary Conditions

4.4.1 Upstream

The upstream boundary condition was specified as a discharge hydrograph at cross-section 1 located just downstream of the Highway No. 90 bridge near Angus.

In the case of the calibration run, this discharge hydrograph consisted of daily inflows observed during the spring of 1987. For the design flood simulations inflow hydrographs at the upstream boundary were obtained from hydrologic simulations of the watershed.

4.4.2 Downstream

The downstream boundary condition is the water surface elevation of Lake Huron. The observed water level during the simulation period was used for

CONCEPTUAL SKETCH OF CHANNEL CONVEYANCE
AND OFF-CHANNEL STORAGE AREAS
IN MINESING SWAMP

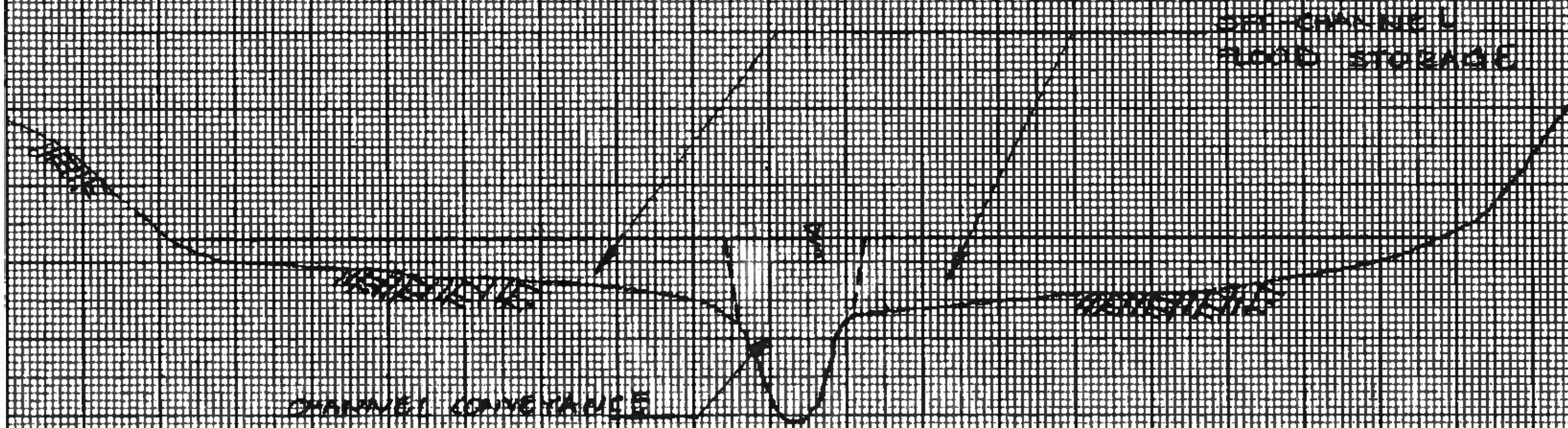


FIGURE 4.2

the calibration run and the long term average lake level during the spring months was used for the design flood simulations.

The downstream boundary condition turned out not to have a significant effect on the Lower Nottawasaga River since its influence was found to only extend approximately 6 km upstream of the river mouth (ie. to Schoonerstown).

4.4.3 Lateral Inflows

Inflow from major tributaries such as the Mad River and Willow Creek and runoff from subcatchment draining overland to the Nottawasaga River were specified as lateral inflow hydrographs at appropriate locations along the streamcourse. The inflow from the Mad River is specified at cross-section 7 and the inflow from Willow Creek is specified at cross-section 8.

4.5 Calibration

4.5.1 Objective

Although DWOPER is a physically based model, calibration is required to adjust model parameters such as the Manning's roughness coefficient. In this application the calibration exercise will also enable the refinement of the stage-storage volume relationship for the Minesing Swamp.

The calibration exercise concentrated on reproducing the lagging and attenuation effects of the Minesing Swamp on peak flood flows observed during the period of March 15 to April 15, 1987.

4.5.2 Calibration Data

Model calibration was based on daily steamflows recorded at Water Survey of Canada gauges at:

- the Nottwasaga River at Baxter (Station No. 02ED003)
- the Mad River near Glencairn (Station No. 02ED005)
- Willow Creek near Midhurst (Station No. 02ED010).

These flows were used to specify inflow hydrographs to the Lower Nottawasaga river system. Streamflow data from the Ontario Ministry of Natural Resources gauge on the Pine River were not available.

Additional data were obtained by Water Survey of Canada who measured flow rates in the Nottawasaga River at the Highway No. 90 bridge near Angus and at the Highway No. 26 bridge at Edenvale on April 6,9,10 and 15, 1987.

Water level data were obtained at Edenvale and at the Schoonertown bridge near the river mouth during the period of April 7-15, 1987 from staff gauges installed by the Consultant.

The upstream boundary of the DWOPER model is at the Highway No. 90 bridge near Angus. It was, therefore, necessary to adjust the flow record at Baxter to reflect the additional drainage area at the model limit and the flow contribution from the Pine River whose confluence with the Nottawasaga River is just upstream of the Highway No. 90 bridge. The daily flows at Baxter were compared with the flow measurements carried out by Water Survey of Canada at the Highway No. 90 bridge. It was determined that, on average,the Nottawasaga River flows at Highway No. 90 were 1.6 times the flows at Baxter. Hence, the upstream boundary flow hydrograph was obtained by multiplying the Baxter flow record by 1.6. The observed daily streamflow at the WSC gauges on the Mad River and Willow Creek were also adjusted to account for the additional drainage area between the gauge location and the confluences of the tributaries with the Nottawasaga River. Since measured flows at the confluences were not available, it was not possible to adjust the flows in a manner similar to that carried out for the Nottawasaga River. The tributary flows were simply prorated on a drainage area basis to account for additional drainage to the tributaries as well as local inflow to the Swamp.

The downstream boundary condition was specified as the water surface elevation of Lake Huron as recorded during the simulation period at the Environment Canada water level gauge at Collingwood. During the period of March 15 to April 15, 1987, the water level fluctuations of Lake Huron were minor varying from 177.03 M (GSC) to 177.07 M (GSC). Hence, the downstream boundary condition was specified as a constant water level of 177.05 M representing the mean water level during that period.

The inflow hydrographs used in the calibration exercise are presented in Table 4.2.

4.5.3 Bridge Losses

Head losses at six bridges along the stream channel were simulated by assigning appropriate head loss coefficients at the bridge locations. Based on procedures described in the Users Manual for the HEC-2 computer model, a head loss coefficient of 0.8 was assigned at bridge sections 12,17,19,23,27 and 30. This head loss coefficient represented the sum of contraction and expansion head loss coefficients of 0.3 and 0.5 respectively.

Since the dynamic wave simulation is primarily intended for flow routing, it was not necessary to utilize the internal boundary rating curve capability of the DWOPER.

4.5.4 Results

The dynamic wave modelling was carried out in order to properly account for flow routing effects through the Lower Nottawasaga River system. Accordingly, the calibration exercise concentrated on reproducing the lagging and attenuation of the peak flood flows.

The calibration was conducted by first adjusting the stage-storage relationship of the Minesing Swamp near the ground surface where the available

TABLE 4.2 CALIBRATION DATA
Daily Streamflows (m^3/s)

Date (1987)	Mad River near Glencairn	Mad River at[1] Confluence with Nottawasaga River	Willow Creek near Midhurst	Willow Creek[2] at Confluence with Nottawasaga River	Nottawasaga River at Baxter	Nottawasaga River[3] at Highway No. 90 bridge near Angus
March 15	4.1	6.4	1.8	4.7	21.2	33.9
" 16	3.6	5.6	1.7	4.5	17.5	28.0
" 17	3.7	5.8	1.6	4.2	15.5	24.8
" 18	3.7	5.8	1.6	4.2	16.2	25.9
" 19	4.3	6.7	1.6	4.2	18.3	29.3
" 20	4.6	7.2	1.7	4.5	21.4	34.2
" 21	5.1	8.0	2.0	5.3	23.6	37.8
" 22	5.9	9.2	2.4	6.3	27.2	43.5
" 23	7.7	12.0	3.4	8.9	33.5	53.6
" 24	9.7	15.1	4.6	12.1	35.2	56.3
" 25	12.6	19.7	6.2	16.3	34.5	55.2
" 26	16.3	25.4	7.4	19.5	38.7	61.9
" 27	14.1	22.0	8.0	21.0	29.0	62.4
" 28	12.7	19.8	7.3	19.2	26.0	41.6
" 29	11.9	18.6	6.5	17.1	21.1	33.8
" 30	19.5	30.0	6.1	16.0	19.9	31.8
" 31	14.8	23.1	6.5	17.1	21.1	33.8
April 1	13.1	20.4	6.0	15.8	32.8	52.5
" 2	12.5	19.5	5.5	14.5	30.9	49.4
" 3	11.4	17.8	5.1	13.4	30.3	48.5
" 4	10.5	16.4	4.8	12.6	29.6	89.6*
" 5	22.6	35.3	5.8	15.3	71.1	122.0*
" 6	29.1	45.4	7.7	20.3	106.0	169.6
" 7	21.3	33.2	7.0	18.4	71.9	115.0
" 8	16.6	25.9	5.8	15.3	56.0	89.6
" 9	14.9	23.2	5.0	13.2	39.3	60.1*
" 10	13.1	20.4	4.3	11.3	30.8	50.8*
" 11	11.8	18.4	3.7	9.7	22.0	35.2
" 12	11.0	17.2	3.2	8.4	17.7	28.3
" 13	10.0	15.6	2.8	7.4	17.8	28.5
" 14	-	-	2.4	6.3	17.0	27.2
" 15	-	-	2.2	5.8	18.0	28.0*

[1],[2], Prorated on a drainage area basis to include local inflow to Minesing Swamp plus the catchment area between the WSC gauge and confluence with the Nottawasaga River.
[3], Adjusted based on measured flows at Hwy. 90 bridge near Angus.
* Measured at Hyw. No. 90.

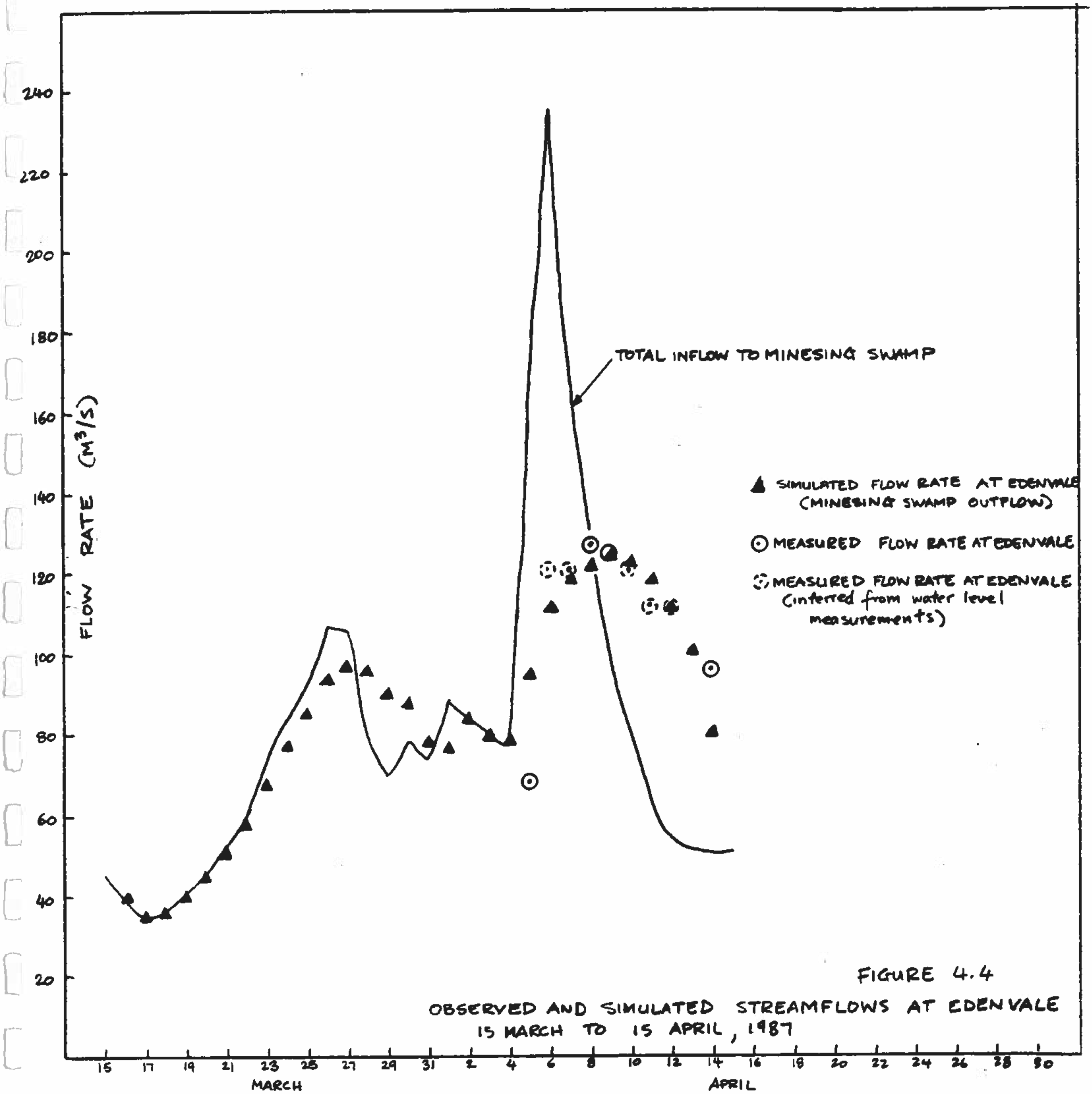


FIGURE 4.4

OBSERVED AND SIMULATED STREAMFLOWS AT EDENVALE
15 MARCH TO 15 APRIL, 1987

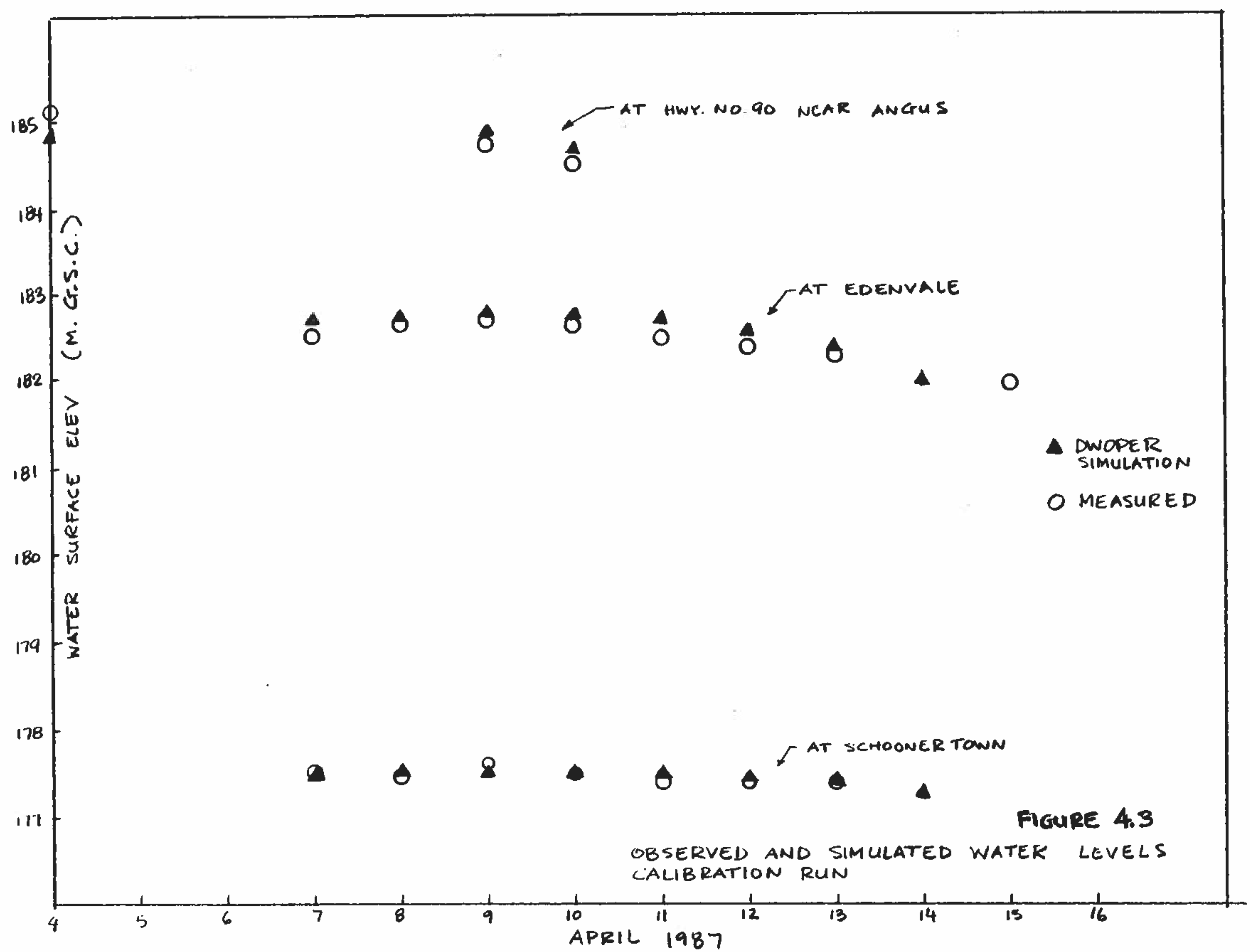


FIGURE 4.3

OBSERVED AND SIMULATED WATER LEVELS
CALIBRATION RUN

topographic information was the most uncertain. When reasonable agreement between observed and simulated peak flows were obtained (at Edenvale) the channel roughness coefficients were then adjusted to reproduce the water levels observed at Edenvale, Schoonertown and at the Hwy. No. 90 bridge. The reach from the Schoonertown bridge to the river mouth was found to have a Manning's roughness coefficient of 0.020, the reach from Schoonertown to the confluence with the Mad River was found to have a Manning's roughness coefficient of 0.035, and the remainder of the channel was found to have a Manning's roughness coefficient of 0.02 (refer to Table 4.1). The calibration simulations were conducted at time steps of 24 hours.

Observed and simulated discharge hydrographs at Edenvale representing the outflow from the Minesing Swamp are presented in Figure 4.3. The comparison of water levels is shown in Figure 4.4. There is excellent agreement with regard to both discharge and water levels, indicating that the dynamic wave model is capable of reliably simulating the lagging and routing effects of the Lower Nottawasaga River and Minesing Swamp.

4.6 Design Floods - Minesing Swamp and Lower Nottawasaga River

Peak design flows along the Lower Nottawasaga River were determined by specifying the appropriate inflow hydrographs to the calibrated DWOPER model. These inflows consist of the Nottawasaga River where it enters the Minesing Swamp(the upstream boundary condition for the model) and a number of lateral inflows representing tributaries and subcatchments contributing overland runoff directly to the streamcourse. A schematic representation of the DWOPER model is shown in Figure 4.5.

Design flows used in the simulations consist of the 1 in 5, 1 in 10, 1 in 20, 1 in 50 and 1 in 100 year return period flows and the Regional Flood. The design flow hydrographs comprising the inflows to the Minesing Swamp and the Lower Nottawasaga River were described in Section 3.3.5 and plots of the return period events are presented in Appendix J.

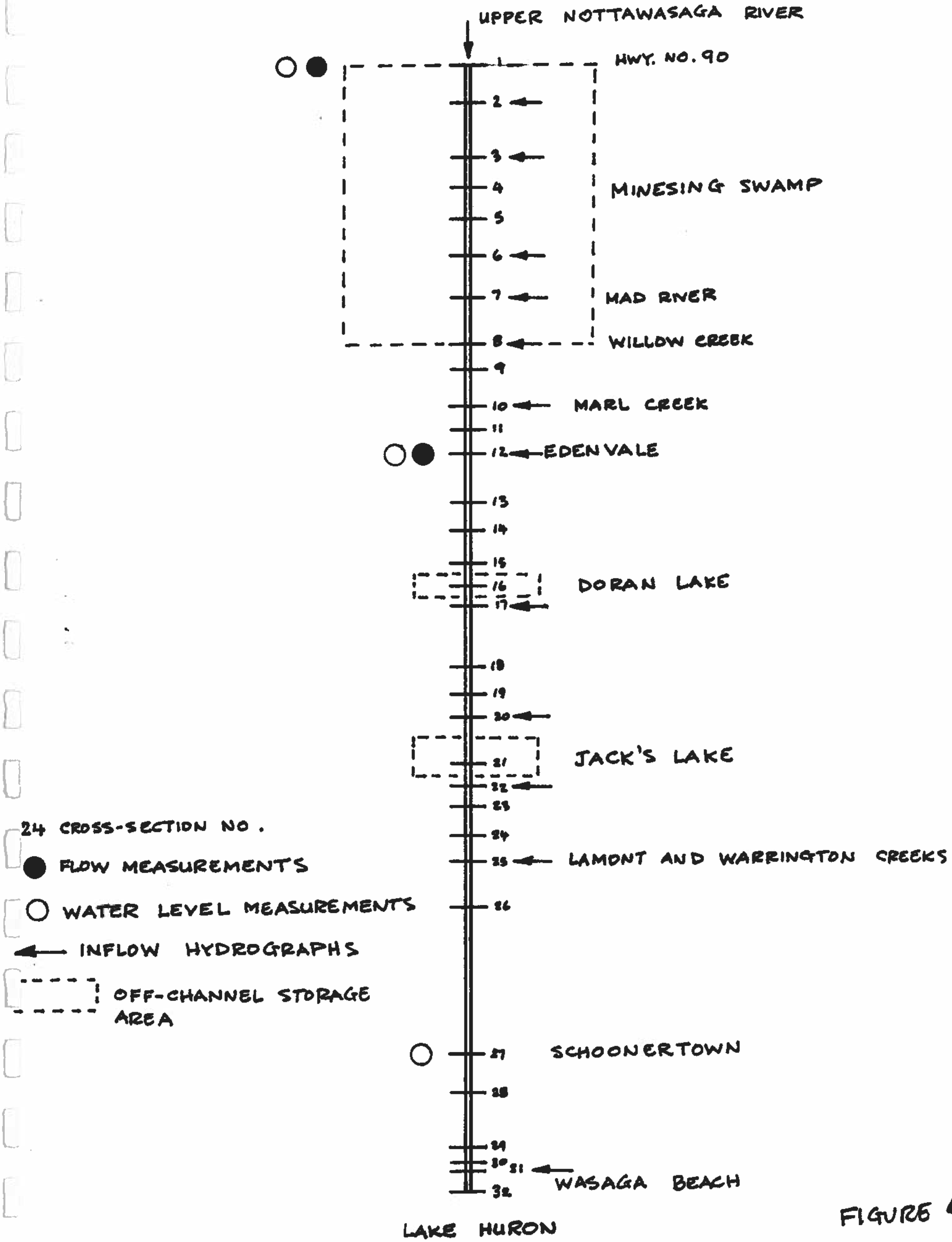


FIGURE 4.5

DWOPER SCHEMATIC

Peak flows corresponding to each of the design events obtained from the DWOPER simulations are presented in Appendix G for the Lower Nottawasaga.

Plots of the 1 in 100 year return period event and Regional Flood hydrographs are given in Figures 4.6 and 4.7 for following locations:

1. Highway No. 26 bridge near Edenvale (representing the outflow from the Minesing Swamp) (Reference No. D12).

2. The bridge just downstream of Doran Lake (Reference No. D17).

3. The bridge just downstream of Jack's Lake (Reference No. D23).

4. The Schoonertown bridge (Reference No. D27).

5. Highway No. 92 bridge at Wasaga Beach near the river mouth (Reference No. D30).

A summary of peak flows and peak stages are presented in Table 4.3 for these locations.

4.7 Discussion and Conclusions

4.7.1 Return Period Flows

The simulations showed that the Minesing Swamp provides significant peak flow attenuation during the design events. Considerable time lagging of the peak discharge was also observed. For example, the time to peak for the 1 in 100 year flood at Edenvale is approximately 130 hours compared to times to peak of 70 hours for the Nottawasaga River inflow at Highway No. 90 and 9 and 17 hours for Willow Creek and Mad River inflow hydrographs, respectively.

Downstream of the confluence with Lamont and Warrington Creeks, the effect of the local and tributary inflows become noticeable with the streamflow hydrographs exhibiting a "double peak" (see Figure 4.6) the first peak being

TABLE 4.3 - SUMMARY OF PEAK FLOW AND PEAK STAGE AT SELECTED LOCATIONS

LOCATION	CROSS SECTION NUMBER	PEAK FLOW AND ASSOCIATED STAGE						PEAK STAGE AND ASSOCIATED FLOW						
		5-YR	10 YR	20 YR	50 YR	100 YR	REGIONAL	5-YR	10 YR	20 YR	50 YR	100 YR	REGIONAL	
Edenvale	D12	161 183.46	180 183.73	201 184.00	224 184.28	247 184.49	243 185.05	159 183.46	178 183.73	199 184.00	223 184.28	244 184.49	237 185.07	flow (m^3/sec) stage (m)
Bridge D/S Doran Lake	D17	160 182.75	180 183.00	200 183.26	224 183.52	247 183.72	256 184.11	158 182.76	178 183.01	199 183.26	223 183.52	243 183.72	254 184.55	flow (m^3/sec) stage (m)
Bridge D/S Jack Lake	D23	158 182.13	178 182.36	199 182.58	223 182.81	242 182.98	340 183.98	158 182.13	178 182.36	199 182.58	223 182.81	242 182.98	330 184.01	flow (m^3/sec) stage (m)
Schooner-Town Bridge	D27	213 178.03	242 178.18	289 178.39	317 178.52	336 178.59	469 179.06	213 178.03	242 178.18	289 178.39	317 178.52	336 178.59	469 179.06	flow (m^3/sec) stage (m)
Wasaga Beach	D30	211 177.38	240 177.46	288 177.59	318 177.69	338 177.75	469 178.08	211 177.38	240 177.46	288 177.59	318 177.69	338 177.75	464 178.08	flow (m^3/sec) stage (m)

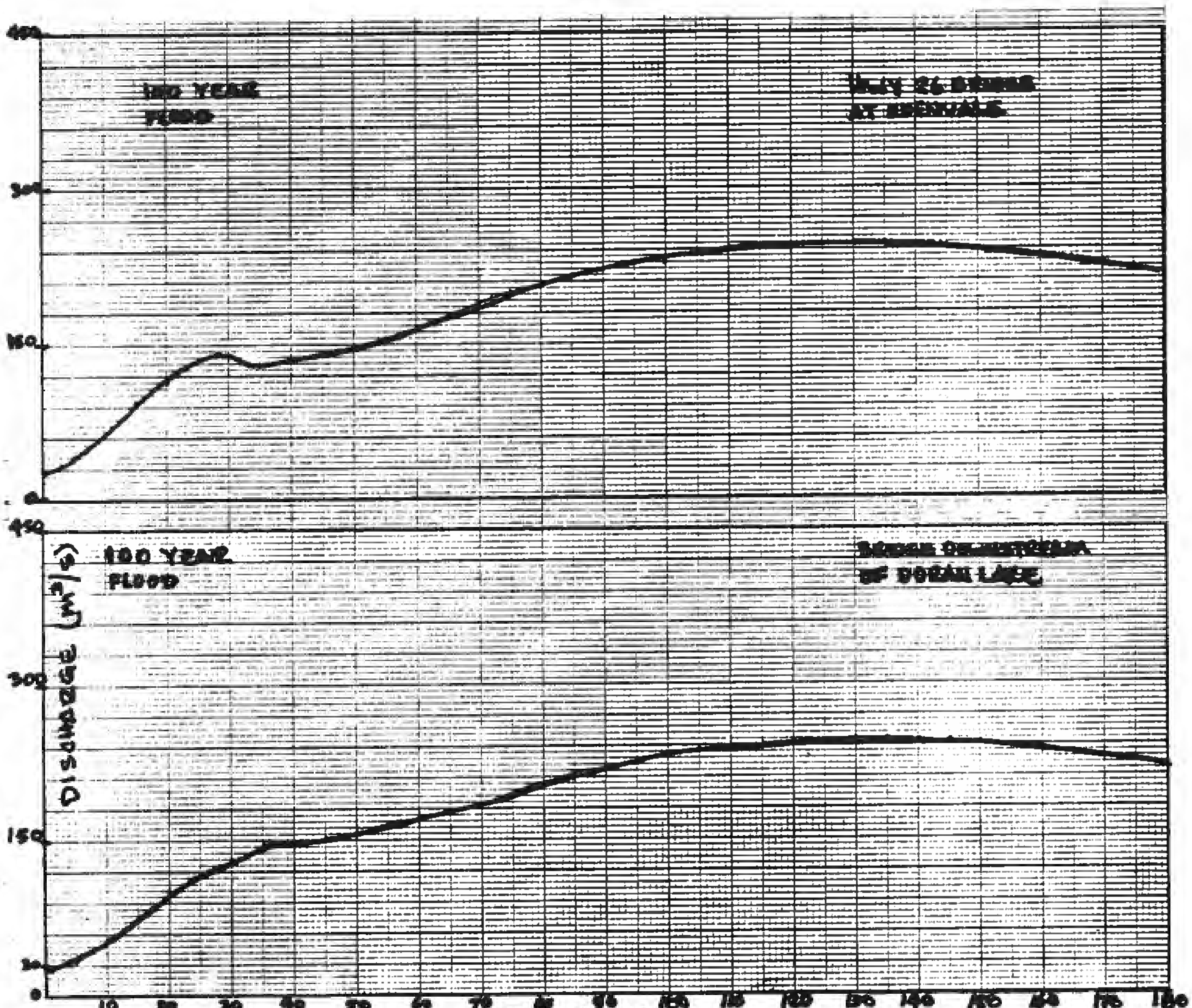

100 YEAR
FLOOD
DISCHARGE (m³/s)
0
150
300
450
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
TIME (HRS)

FIGURE 4.6 (a)

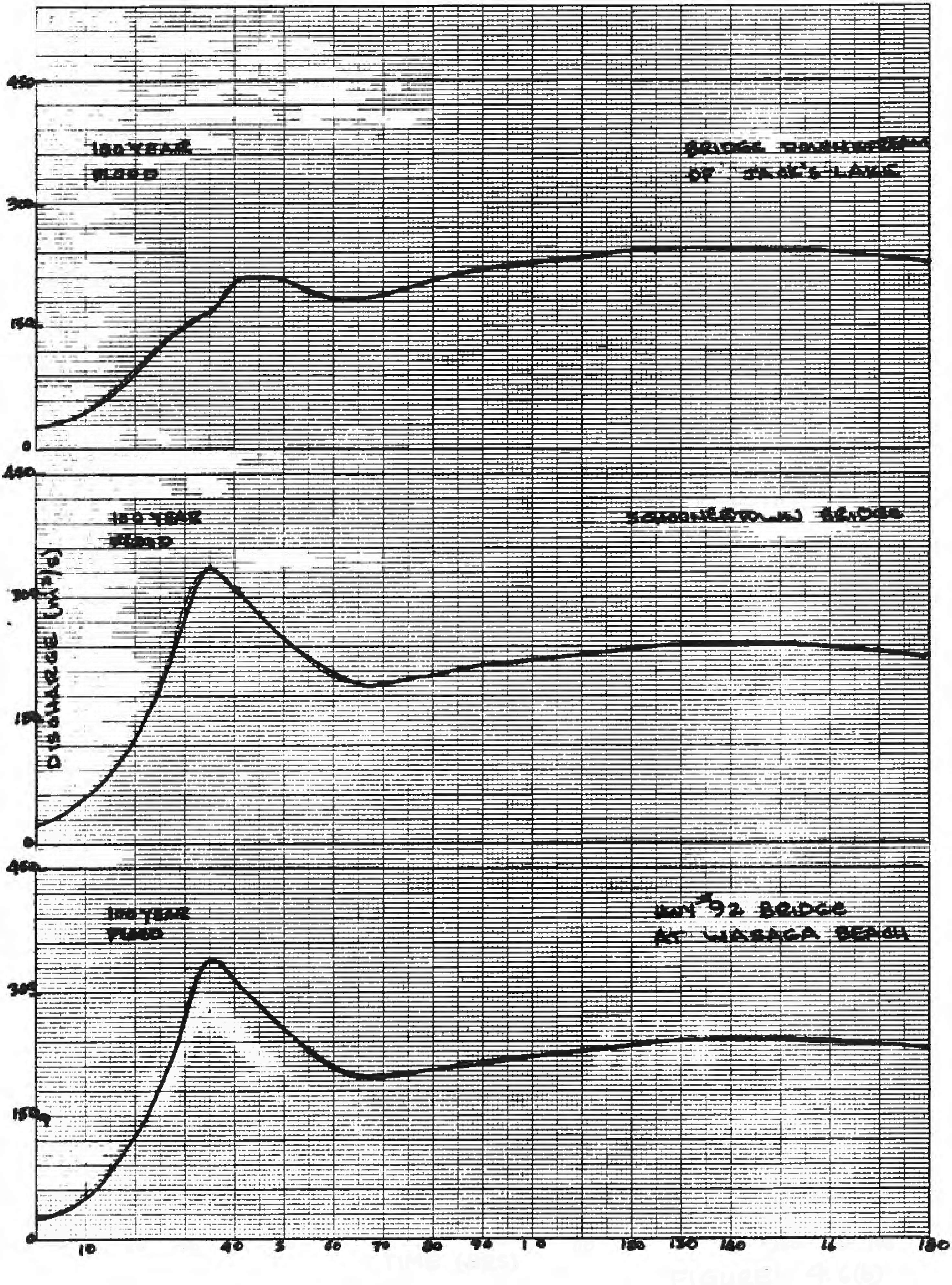

100 YEAR FLOOD
450
300
150
0
HWY 92 BRIDGE
AT WASAGA BEACH
DISCHARGE (m³/s)
TIME (HRS)

FIGURE 4.6(b)

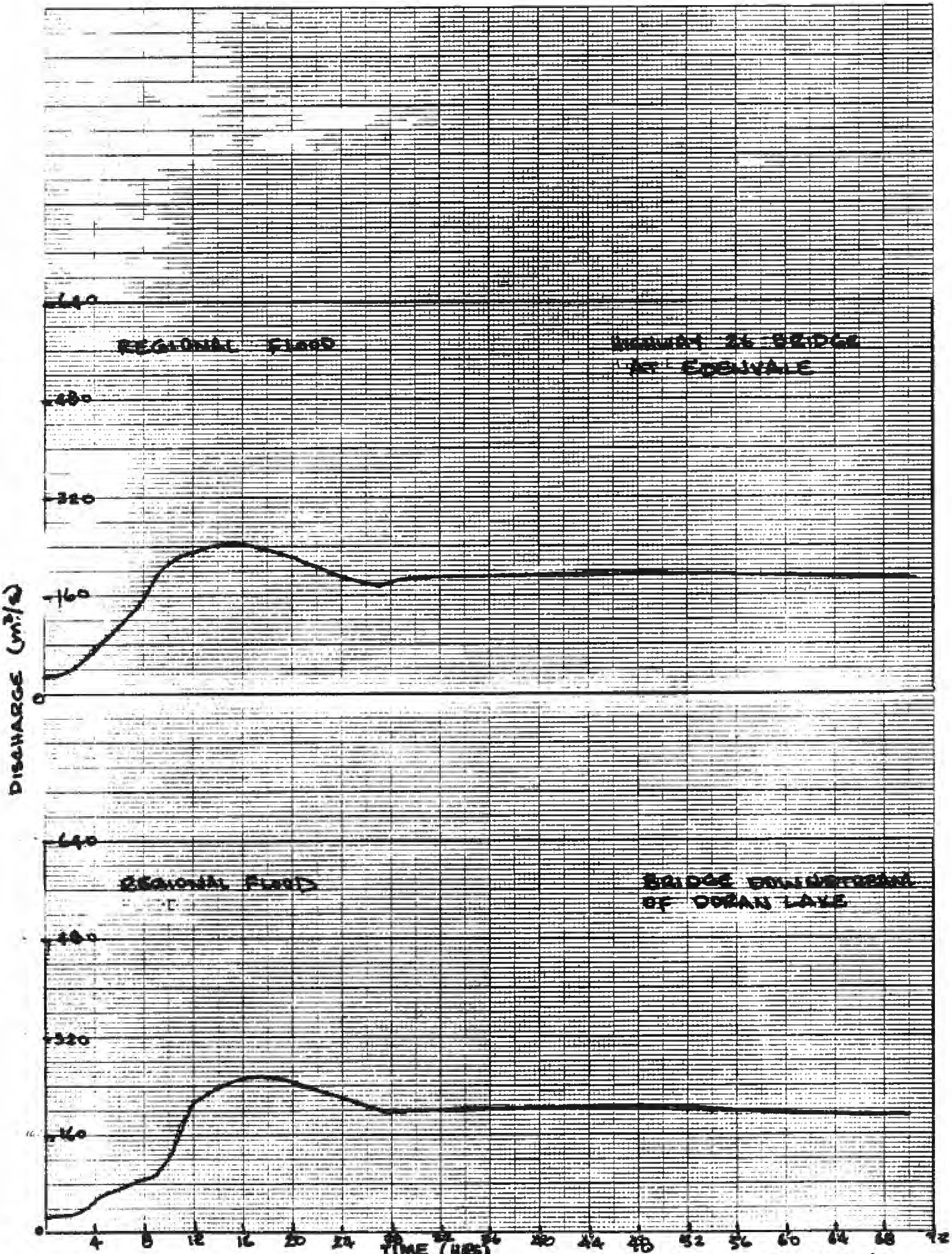
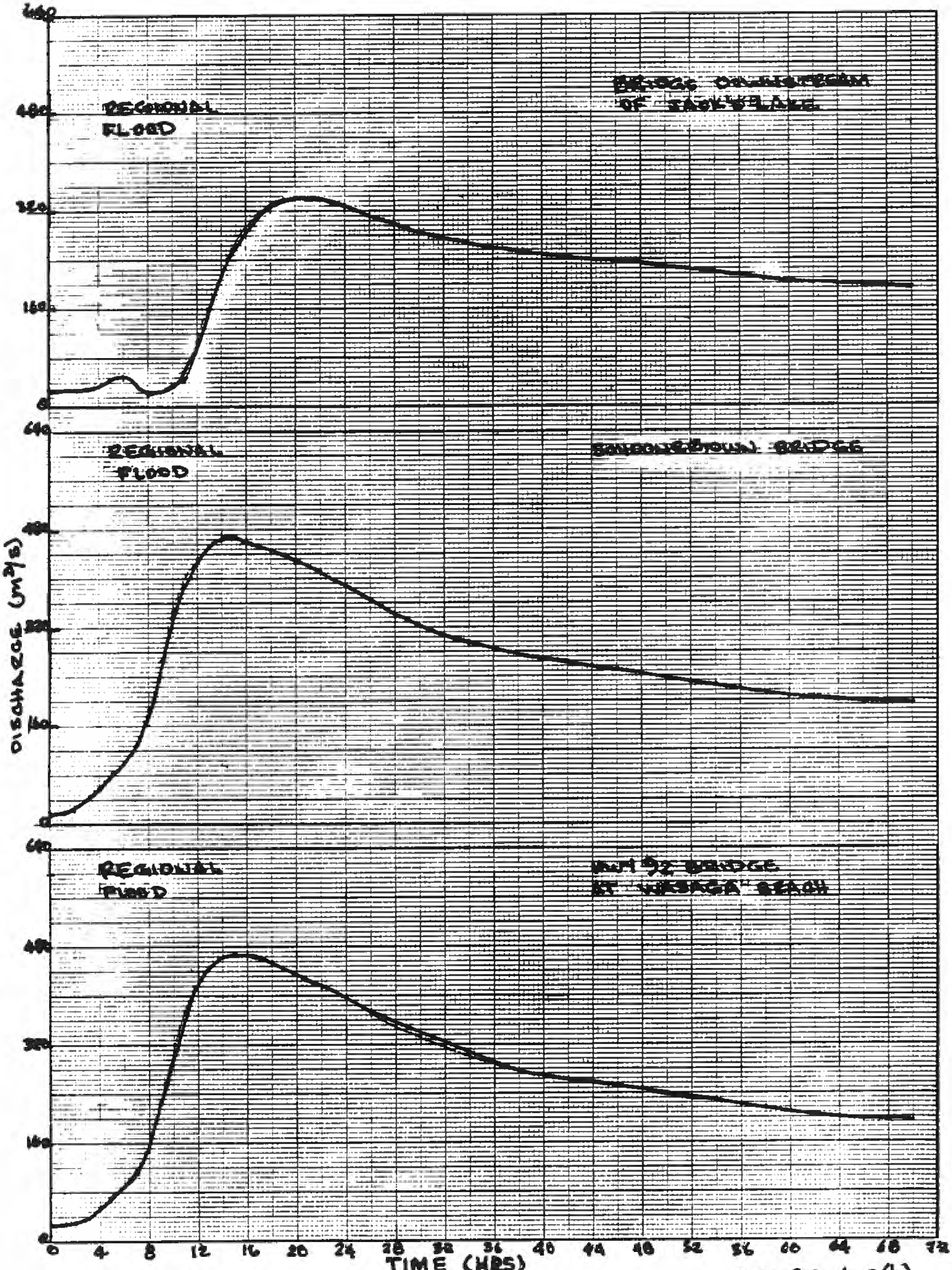

REGIONAL FLOOD
HIGHWAY 26 BRIDGE AT EDENVALE
REGIONAL FLOOD
BRIDGE DOWNSTREAM OF DORAN LAKE
DISCHARGE (m³/s)
640
480
320
160
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
TIME (HRS)

FIGURE 4.7 (a)



REGIONAL FLOOD
BRIDGE DOWNSTREAM OF JACK'S LAKE
REGIONAL FLOOD
REGIONAL FLOOD
HWY 92 BRIDGE AT WASAGA BEACH
DISCHARGE (m³/s)
TIME (HRS)
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
0
160
320
480
640

FIGURE 4.7(b)

due to local and tributary inflow and the second due to outflows from the Minesing Swamp. The highest flood levels, therefore occur at significantly different times along the streamcourse. Along the upper reach, from Edenvale to the confluence with Lamont and Warrington Creeks, the highest peak flow occurs at approximately 130 hours whereas the highest peak flow occurs at approximately 35 hours downstream of Lamont and Warrington Creeks.

Due to the significant difference in times to peak, consideration should be given to establishing flood levels using dynamic rather than steady flow water level simulations. This would require some refinement in the simulation of bridge losses as presently utilized in the model in order to improve its accuracy with regard to establishing flood elevations.

4.7.2 Regional Storm Flows

The results of the Regional Flood Simulation also indicated significant peak flow attenuation due to the Minesing Swamp. However, the times to peak along the Lower Nottawasaga only varies from 15 to 19 hours, and does not exhibit the "double peak" of the hydrographs for the return period floods. Hence, dynamic flow simulation techniques are not required to determine Regional Flood water surface elevations, and the usual steady state backwater analysis can be applied.

A mass balance calculation was carried out for the Regional Flood event to check the accuracy of the DWOPER simulations. For the period of simulations the difference between the total inflows to the model and outflows at the river mouth was found to agree with the change in channel storage within approximately one percent (1%).

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