CANADA-NEWFOUNDLAND

FLOOD DAMAGE REDUCTION PROGRAM

GLENWOOD-APPLETON FLOOD STUDY REPORT

Prepared by:

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SMR-09-89

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ShawMont

ShawMont Newfoundland Limited

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1989 11 30

File: NDE 8508-1

Canada Newfoundland Flood Damage Reduction Program
c/o Department of Environment and Lands
Confederation Building Annex (Fourth Floor)
St. John's, NF

Attn: Mr. R. Picco, P. Eng.

Dear Mr. Picco:

Re: Glenwood Appleton Flood Study Report

We take pleasure in submitting fifty (50) copies of the final report on this interesting hydrotechnical study.

We trust the findings of the study will provide a sound basis for future municipal planning and will help reduce future losses from flooding in both Glenwood and Appleton.

We wish to express our appreciation of input provided by members of the Technical Committee.

Yours very truly,

Albert D. Peach, P. Eng.
President

P.C. Helwig, P. Eng.
Study Manager

PCH/gar

EXECUTIVE SUMMARY

The main purpose of this study, as outlined in the Terms of Reference, was "to provide estimates of the 1:20 and 1:100 year recurrence interval flood levels, determine the extent of flooding associated with each level and to plot these levels on base mapping provided (by the Client)". The Terms of Reference also required that the techniques employed in the study comply with technical standards established by Environment Canada for flood risk mapping studies.

To meet these requirements a study program was organized, focusing on the problem of open water flooding; since historic flooding was due to this cause. This program comprised the following five tasks:

(i) collection of documentary data on climate, hydrology and information on past flooding,

(ii) a field program to establish elevation controls and measure river cross-sections for input to the HEC-2 computer model. Also included in the field program was an interview survey to obtain information on past flooding, and in particular to establish the extent of flooding associated with the January 1983 flood,

(iii) an hydrology study to establish the 1:20 year and 1:100 year flood flows,

(iv) an hydraulic study using the HEC-2 water profile model to establish the water levels associated with the 1:20 year and 1:100 year floods,

(v) preparation of a report summarizing the methods and findings of the study and suggesting possible remedial measures.

The main findings in the study were:

-- the 1 in 20 year flood, Q_{20} = 898 m^3/s
-- the 1 in 100 year flood, Q_{100} = 1060 m^3/s

These design floods were input to the HEC-2 model for the study reach to determine design flood profiles. These flood profiles were then used for establishing flood risk contours. The resulting flood risk maps are included in the envelope pocket at the end of this report.

The following recommendations are suggested to minimize the damage from future flooding in the study area:

(i) The communities of Appleton and Glenwood should implement zoning regulations to control development in flood prone areas as delineated on the flood risk maps produced in this study.

(ii) Water supply pumping and treatment plants should be flood proofed to safeguard their operation during floods. It is suggested that the design of flood proofing measures be based upon flood levels for the 1 in 100 year flood.

(iii) Sewage pump stations and treatment plants should be flood proofed to protect equipment vulnerable to water damage. It is likewise suggested that the design of flood proofing measures be based upon water levels for the 1 in 100 year flood.

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Flood Risk Maps are in envelope pockets at back of report

PART ONE

INTRODUCTION

1. INTRODUCTION

1.1 BACKGROUND

The Towns of Glenwood and Appleton, located on the banks of Gander River just downstream from Gander Lake, were subjected to extensive flooding in January 1983, due to an unprecedented winter rainstorm. Low lying areas in both towns are also subject to frequent minor flooding, mainly during spring runoff periods. Records also exist of a major flood in May 1926 having a similar magnitude to the January 1983 event.

In response to this problem, the Governments of Canada and Newfoundland commissioned a flood risk mapping study of the area under the auspices of the Canada-Newfoundland Flood Damage Reduction Program.

The purpose of this study was to establish flood risk contours for this area, showing the extent of flooding associated with floods having recurrence intervals of twenty (20) and one hundred (100) years. These contours would be plotted on detailed base mapping. The flood risk maps thus produced should enable the town councils in Glenwood and Appleton to better regulate development in flood prone areas and thereby minimize the losses from future flooding.

1.2 AUTHORIZATION

In May 1988 a request for proposals was published giving the Terms of Reference for flood risk mapping studies in Glenwood-Appleton and Glovertown. The proposal submitted by ShawMont Newfoundland Limited was selected. The study contract was subsequently awarded, on behalf of the Canada-Newfoundland Flood Damage Reduction Program, by a letter from the Department of Environment and Lands (Newfoundland) signed by the Minister, the Hon. James Russell, and dated October 11, 1988.

1.3 ACKNOWLEDGEMENTS

The interest and co-operation of members of the Technical Committee: L. Langley - Environment Canada, Dr. W. Ullah and R. Picco - Environment and Lands (Newfoundland), is greatly appreciated.

The principal members of ShawMont's study team were:

1.3 ACKNOWLEDGEMENTS (Cont'd)

P.C. Helwig - Study manager, responsible for technical direction of the study.

H.J. Keats - Hydrology, hydraulic computations and field program.

PART TWO

APPROACH

2. APPROACH

2.1 LOCATION OF STUDY AREA

Glenwood and Appleton are located on opposite banks of Gander River, with Glenwood on the west bank and Appleton on the east bank. The towns occupy the portions of the river banks between the area known as the "Outflow" and the Trans Canada Highway Bridge, as shown in Figure 2.1.

2.2 CLIMATE

The climate in Newfoundland may be classified as a temperate-maritime climate where the extremes of temperature are moderated by the ocean. Further from the ocean, towards the interior of Central Newfoundland, the climate exhibits a more continental character having greater variations in temperature between winter and summer seasons. Gander River which originates in Central Newfoundland, normally behaves as a nordic river, with an annual pattern of flows which would be associated with a continental climatic regime.

Based on data collected at Gander International Airport, it is estimated that approximately 40% of annual precipitation is in the form of snow, mainly occurring during the months of December - April. Annual flood peaks generally occur during the spring period (April-May), a result of snowmelt or the combined effects of rainfall and snowmelt. Minimum flows are usually experienced during summer, and on rare occasions, during winter. Infrequently, the area is subjected to vigorous winter storms of oceanic origin arriving via the eastern seaboard of the United States. These storms bring abundant precipitation in combination with unseasonably warm temperatures and are capable of producing major winter floods, as happened in January 1983.

Table 2.1 summarizes the main climatic parameters, typical of the study area. This data is taken from the climatological station at Gander International Airport which is about 23 km east of Glenwood-Appleton.

2.2 CLIMATE (Cont'd)

TABLE 2.1

Climatic Normals for Gander International Airport

MEAN OF TEMPERATURE, PRECIPITATION, HUMIDITY, PRESSURE, WIND AND SUNSHINE													
	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
TEMPERATURE (C)													
MAXIMUM	-2.4	-2.6	0.4	4.5	11.0	17.3	21.9	20.3	15.9	9.8	5.0	-0.4	8.4
MINIMUM	-9.9	-10.9	-7.4	-2.7	1.3	6.2	11.2	10.8	6.9	2.2	-1.4	-7.1	-0.1
MEAN	-6.2	-6.8	-3.5	0.9	6.2	11.8	16.5	15.6	11.4	6.0	1.8	-3.8	4.2
EXTREME MAXIMUM	11.7	12.8	14.5	21.7	28.3	32.8	35.6	33.4	28.9	24.4	20.6	14.4	35.6
EXTREME MINIMUM	-27.2	-31.1	-25.6	-15.6	-8.9	-2.8	0.6	-1.1	-1.7	-6.3	-13.9	-26.1	-31.1
PRECIPITATION													
RAINFALL (MM)	33.0	23.6	37.3	42.6	55.5	77.3	69.0	97.3	81.1	91.3	74.8	38.8	721.6
EXTREME IN 24 HR	34.3	42.9	60.2	65.3	45.0	37.1	96.3	98.3	57.2	50.0	58.2	40.6	98.3
SNOWFALL (CM)	78.7	76.2	72.3	47.1	13.1	2.8	0.0	0.0	0.1	12.2	31.8	70.9	405.2
EXTREME IN 24 HR	36.2	47.8	41.4	37.8	16.5	21.8	T	T	5.1	20.8	43.8	45.7	47.8
TOTAL (MM)	109.1	99.7	110.1	93.2	70.0	80.3	69.0	97.3	81.2	104.7	107.3	108.2	1130.1
EXTREME IN 24 HR	35.1	57.9	60.2	65.3	45.0	37.1	96.3	98.3	57.2	50.0	58.2	45.7	98.3
HUMIDITY													
VAPOUR PRESSURE (KPA)	0.36	0.34	0.41	0.53	0.72	1.02	1.37	1.35	1.06	0.79	0.62	0.43	0.75
DEW POINT (C)	-8.8	-9.7	-6.6	-2.6	1.6	6.6	11.2	11.0	7.2	2.8	-0.8	-6.2	0.5
RELATIVE HUMIDITY (%)	81	80	80	81	77	75	75	78	78	81	84	83	79
PRESSURE													
SEA LEVEL (KPA)	100.7	100.8	100.9	101.1	101.3	101.3	101.4	101.3	101.4	101.3	101.2	100.9	101.1
WIND													
PREVAILING DIRECTION	W	W	W	NNW	W	SW	SW	WSW	W	WSW	W	W	W
SPEED (KM/H)	24.4	23.9	23.4	21.6	19.7	18.7	17.3	17.2	18.9	20.6	21.8	22.8	20.8
PEAK WIND (KM/H)	161	145	135	116	114	116	105	113	116	129	129	159	161
SUNSHINE													
BRIGHT SUNSHINE (H)	85.1	98.7	104.4	115.8	162.3	183.6	214.2	186.3	146.0	110.7	66.6	68.5	1542.2
% OF POSSIBLE	31.4	34.5	28.4	28.2	34.3	38.0	43.9	41.8	38.5	32.9	24.1	26.5	34.5

[from Reference 1]

2.3 HISTORY AND DESCRIPTION OF FLOODING

Jan 1-14, 1983 Storm

"The same storm which caused extensive damage at Bishop's Falls and other areas in Central Newfoundland also resulted in serious flooding in the Glenwood-Appleton area as waters from the Gander River flooded these communities to a depth of 2 m, marooning houses, cars, boats, etc., and covering roads, back and front yards and driveways with broken ice sheets. In all, forty basements were flooded and twelve families, ten in Glenwood and two in Appleton were forced to evacuate their homes. The Glenwood sewage treatment plant was partially submerged, making it inoperable. The (Glenwood) water supply system also became inoperable due to the high water levels. A sawmill in Appleton was also submerged by the flood waters" (cited from Reference 2).

2.3 HISTORY AND DESCRIPTION OF FLOODING (Cont'd)

At the peak of the flood, water levels rose to within 1.0 m of the girders of the railway bridge, that are normally 4.2 m above river level. The Queen Elizabeth Bridge (at the Trans Canada Highway) was similarly menaced with water rising to within 1.7 m of the bottom of the bridge beams (information from local residents).

While the January 1983 flood was the highest in recent memory, flow records from the old Gander River Gauge (02YQ002), indicate that a flood of similar magnitude occurred in May 1926 (no recollections of this event were mentioned by individuals contacted during the interview survey, conducted as part of this study).

From discussions with the mayors of Glenwood and Appleton and residents of the area, it was learned that minor flooding occurs almost every year in low lying areas. This is viewed as a significant problem in Glenwood where the riverside zone is relatively low.

According to information from observers in the area, flooding is always due to high open water flows. Flooding due to backwater effects behind ice jams is not seen as a problem as there are no obstacles in the river for ice to hang up against. The only perceived risk associated with river ice would be damage to the railway and highway bridges from impacts with ice floes. This condition could only arise with flood levels in excess of those experienced in January 1983 and would therefore be a very unlikely event.

2.4 METHODOLOGY

The basic purpose of this study, as outlined in the Terms of Reference, was "to provide estimates of the 1:20 and 1:00 year recurrence interval flood levels, determine the extent of flooding associated with each level and to plot these levels on base mapping provided (by the Client)". The Terms of Reference also required that the techniques employed in the study comply with guidelines establishing technical standards for flood risk mapping studies (3).

Historically, flooding in the Study Area has been caused by open water flood events; accordingly, the study program focused on this cause. The study program comprised the following five tasks:

(i) collection of documentary data on climate, hydrology and information on past flooding,

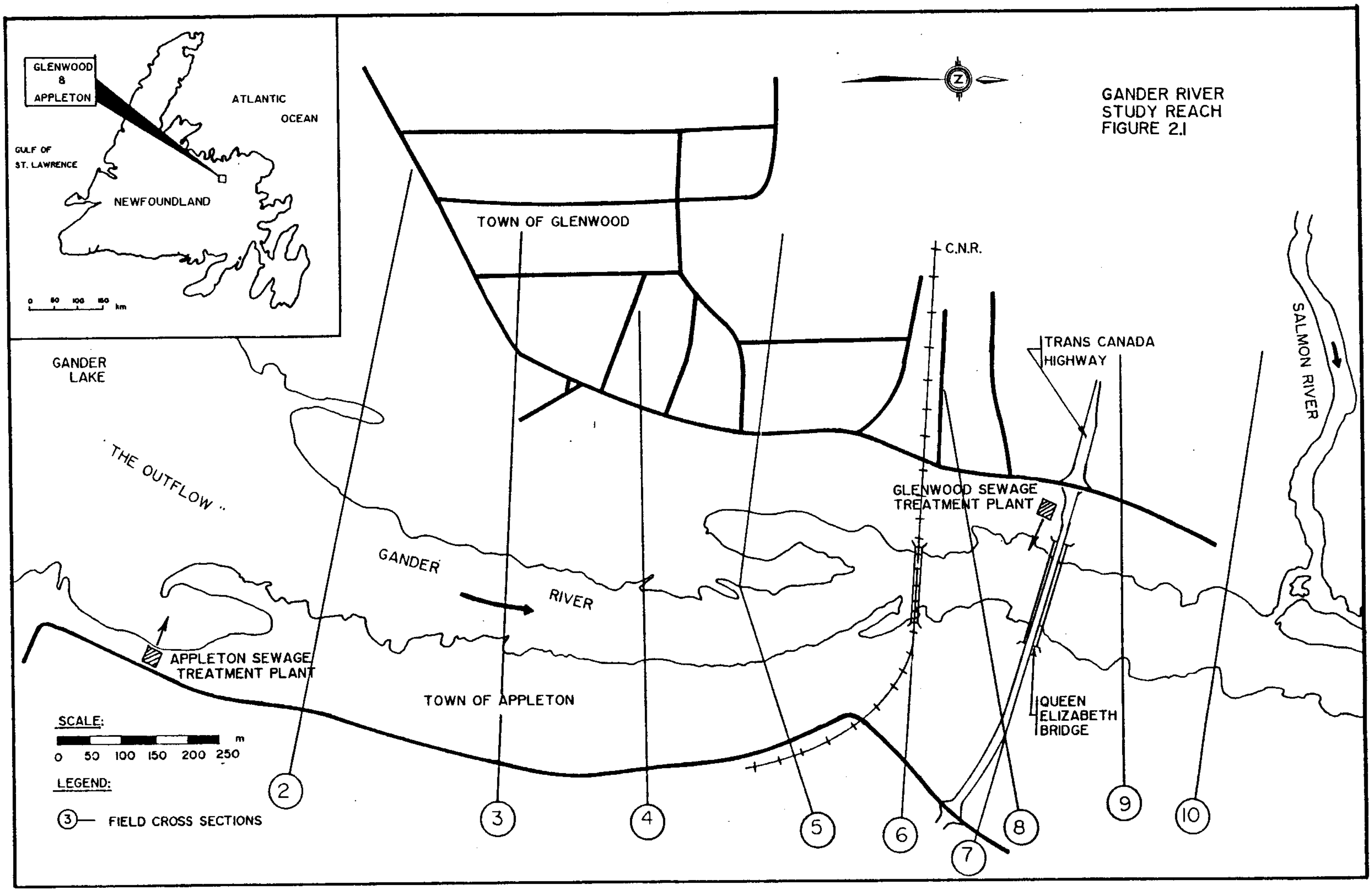
2.4 METHODOLOGY (Cont'd)

(ii) a field program to establish elevation controls and measure river cross-sections for input to the HEC-2 computer model. Also included in the field program was an interview survey to obtain information on past flooding, and in particular to establish the extent of flooding associated with the January 1983 flood,

(iii) a hydrology study to establish the 1:20 year and 1:100 year flood flows,

(iv) a hydraulic study using the HEC-2 water profile model to establish the water levels associated with the 1:20 year and 1:100 year floods,

(v) preparation of a report summarizing the methods used in, and findings of, the study and suggesting possible remedial measures.


GANDER RIVER
STUDY REACH
FIGURE 2.1
GLENWOOD & APPLETON
ATLANTIC OCEAN
GULF OF ST. LAWRENCE
NEWFOUNDLAND
0 50 100 150 km
GANDER LAKE
THE OUTFLOW
TOWN OF GLENWOOD
C.N.R.
TRANS CANADA HIGHWAY
SALMON RIVER
GLENWOOD SEWAGE TREATMENT PLANT
GANDER
RIVER
APPLETON SEWAGE TREATMENT PLANT
TOWN OF APPLETON
QUEEN ELIZABETH BRIDGE
SCALE:
0 50 100 150 200 250 m
LEGEND:
3 FIELD CROSS SECTIONS
2
3
4
5
6
7
8
9
10

PART THREE

HYDROLOGY

3. HYDROLOGY

3.1 PREAMBLE

An ample set flow records is available from the Big Chute streamflow gauging station (2YQ001) covering thirty-eight years from 1949 to 1987. The availability of this data set, at a location just downstream of the study area, made this phase of the study relatively straight forward, namely to carry out a frequency analysis of annual peak flows on Gander River - and estimate the 1:20 year and 1:100 year flood flows.

3.2 AVAILABLE DATA

A streamflow gauging station was established on Gander River at Big Chute (Station No. 2YQ001) in 1949 and has been in continuous operation since October 1, 1949. This station is located 1.8 km downstream of the study area and measures the runoff from a drainage area of 4400 km^2. The drainage area to the outlet of Gander Lake, and including the study area is 4150 km^2. The difference between these two drainage areas is mainly due to the drainage area of Salmon River which flows into Gander River just downstream of the study area. The contribution of flows from Salmon River to major peak flows measured at Big Chute, should be minimal since the times-to-peak of floods from Gander Lake would be very much different from times to peak for the corresponding flood on Salmon River. Accordingly, it was decided to base frequency analysis for flood flows in the study area on unadjusted flows measured at Big Chute. Implications of this decision are further examined in Section 3.4.

It should also be noted that sixteen years of record, between 1923 and 1939, are also available for a station located at the outlet of Gander Lake (Station No. 2YQ002). These data are considered less reliable than data from Gander River at Big Chute since the hydraulic characteristics of this location, from a flow gauging point of view, are inferior to those at Big Chute. These data were not used in the principal analysis, but were considered in the sensitivity analyses, Section 3.4.

Incorporating historical data into the frequency analysis, as suggested in Gerard and Karpuk (9), was considered inappropriate at this location due to unavailability of witnesses or records; simply put, one could not be assured that major floods occurring either prior to establishment of the first stream flow gauging station, or during the period when flow gauging lapsed (1939 to 1949) would be remembered.

3.3 FREQUENCY ANALYSIS

Frequency analysis was carried out using the Consolidated Frequency Analysis Package (CFA), for microcomputers developed by Pilon, Condie and Harvey (4). This program fits input data to the following types of extreme value distributions:

-- Generalized Extreme Value (GEV) Distribution,
-- Three-Parameter Lognormal Distribution,
-- Log Pearson Type III, Distribution,
and
-- Wakeby Distribution

The program tests for outliers using the Grubbs and Beck outlier test. Outliers identified in this test are identified in a warning message. The program provides the option of adjusting the number of low outliers included in the analysis, but always retains high outliers.

The following non parametric statistical tests are also provided:

-- Spearman Test for independence,
-- Spearman Test for trend,
-- Run Test for general randomness,
and
-- Mann-Whitney Split Sample test for homogeneity.

Frequency analysis was carried out on a thirty-eight year (1950-1987) set of annual maximum peak flows measured at Big Chute (Station No. 02YQ001) on Gander River. The data set was first examined for independence, trend, homogeneity and randomness using the data screening features of the CFA program. The results of these tests are summarized in Tables 3.1 (a), 3.1 (b), 3.1 (c) and 3.1 (d).

Table 3.1(a)

Gander River at Big Chute, 1950 - 1987
Spearman Test for Independence

Spearman Rank Order Serial Correlation Coefficient	=	0.063	D.F. = 35
Corresponds to Students T	=	0.375	
Critical T Value at 5% level	=	1.691	(not significant)
Critical T Value at 1% level	=	2.440	(not significant)

Interpretation: The null hypothesis is that the serial (lag-one) correlation is zero.

3.3 FREQUENCY ANALYSIS (Cont'd)

At the 5% level of significance, the correlation is not significantly different from zero. That is, the data do not display significant serial dependence.

Table 3.1(b)

Gander River at Big Chute, 1950 - 1987
Spearman Test for Trend

Spearman Rank Order Serial Correlation Coefficient	= -0.342	D.F. = 36	
Corresponds to Students T	= -2.181		
Critical T Value at 5% level	= -2.029		(significant)
Critical T Value at 1% level	= -2.722		(not significant)

Interpretation: The null hypothesis is that the correlation is zero.

At the 5% level of significance, the correlation is significantly different from zero, but is not so at the 1% level of significance. That is, the trend is significant, but not highly so.

Table 3.1(c)

Gander River at Big Chute, 1950 - 1987
Mann-Whitney Split Sample Test for Homogeneity

Split by time span,	subsample 1 sample size	=	19
	subsample 2 sample size	=	19

Mann-Whitney U	= 119.5	
Critical U value at 5% significant level	= 123.0	(significant)
Critical U value at 1% significant level	= 101.0	(not significant)

Interpretation: The null hypothesis is that there is no location difference between the two samples.

At the 5% level of significance, there is a significant difference in location, but not so at the 1% level. That is, the location difference is significant, but not highly so.

Table 3.1(d)

Gander River at Big Chute, 1950 - 1987
Run Test for General Randomness

The number of observations above and below the median (RUNAB)	=	18
The number of observations above the median (N1)	=	19
The number of observations below the median (N2)	=	19

Range at 5% level of significance: = 14 to 26 (not significant)

Interpretation: The null hypothesis is that the data are random

At the 5% level of significance, the null hypothesis cannot be rejected. This is, the sample is significantly random.

These tests indicate that the data, at a 1% level of confidence, are independent and free of trend, are from a homogeneous population and are randomly distributed in time. However, the statistics at a 5% level of confidence, indicate a significant serial correlation for trend and a significant location difference (non-homogeneity) between each sub-set of data. Since the evidence of trend and non-homogeneity in the sample are not confirmed at the 1% level of significance, the significance of these factors is judged to be marginal and not sufficient grounds for rejecting the data. Accordingly frequency analysis was carried out on the entire data set, as tabulated in Table 3.2.

The data were fitted to the four extreme value distributions provided by the CFA program package, the Generalized Extreme Value (GEV), Three Parameter Lognormal, Log Pearson Type III and Wakely distributions. From an inspection of the frequency graphs, it was decided that the Three Parameter Log Normal Distribution gave the best fit for the data. The numerical results of this analysis together with the relevant sample statistics are given in Table 3.3. Figure 3.1 displays the results graphically.

The design flood flows, recommended for use in preparing flood risk contours are:

Q_{20}, the 1:20 year flood = 898 m^3/s
Q_{100}, the 1:100 year flood = 1060 m^3/s

TABLE 3.2

ANNUAL FLOODS

WSC STATION NO=02YQ001
WSC STATION NAME=GANDER RIVER AT BIG CHUTE

TOTAL TIME SPAN, YT= 38 YRS. FLOW THRESHOLD = 100.000
OBSERVED PEAKS, N= 38 HISTORIC PEAKS ABOVE THRESHOLD, NHA=38

OBSERVED PEAKS ABOVE THRESHOLD, NA= 38
OBSERVED PEAKS BELOW THRESHOLD, NB= 0
MISSING PEAKS BELOW THRESHOLD, NC= 0

MONTH (1)	YEAR (2)	FLOOD (3)	DESCENDING ORDER (4)	RANK M (5)	RANK ADJ. (6)	CUM. PROB. (7)	RET.PERIOD YEARS (8)
5	1950	578.000	1190.000	1	1.00	1.57	63.67
4	1951	648.000	933.000	2	2.00	4.19	23.87
5	1952	629.000	810.000	3	3.00	6.81	14.69
4	1953	462.000	762.000	4	4.00	9.42	10.61
3	1954	501.000	742.000	5	5.00	12.04	8.30
1	1955	436.000	742.000	6	6.00	14.66	6.82
4	1956	620.000	731.000	7	7.00	17.28	5.79
5	1957	416.000	691.000	8	8.00	19.90	5.03
3	1958	248.000	689.000	9	9.00	22.51	4.44
4	1959	334.000	685.000	10	10.00	25.13	3.98
5	1960	648.000	682.000	11	11.00	27.75	3.60
5	1961	682.000	677.000	12	12.00	30.37	3.29
4	1962	742.000	674.000	13	13.00	32.98	3.03
5	1963	549.000	668.000	14	14.00	35.60	2.81
4	1964	810.000	664.000	15	15.00	38.22	2.62
5	1965	564.000	648.000	16	16.00	40.84	2.45
12	1966	490.000	648.000	17	17.00	43.46	2.30
5	1967	731.000	640.000	18	18.00	46.07	2.17
5	1968	399.000	629.000	19	19.00	48.69	2.05
2	1969	685.000	620.000	20	20.00	51.31	1.95
5	1970	742.000	578.000	21	21.00	53.93	1.85
4	1971	640.000	575.000	22	22.00	56.54	1.77
5	1972	515.000	566.000	23	23.00	59.16	1.69
5	1973	668.000	564.000	24	24.00	61.78	1.62
5	1974	459.000	549.000	25	25.00	64.40	1.55
5	1975	566.000	515.000	26	26.00	67.02	1.49
4	1976	677.000	501.000	27	27.00	69.63	1.44
12	1977	575.000	490.000	28	28.00	72.25	1.38
5	1978	453.000	487.000	29	29.00	74.87	1.34
2	1979	487.000	462.000	30	30.00	77.49	1.29
4	1980	691.000	459.000	31	31.00	80.10	1.25
5	1981	674.000	453.000	32	32.00	82.72	1.21
5	1982	664.000	436.000	33	33.00	85.34	1.17
1	1983	1190.000	416.000	34	34.00	87.96	1.14
2	1984	689.000	415.000	35	35.00	90.58	1.10
5	1985	415.000	399.000	36	36.00	93.19	1.07
4	1986	933.000	334.000	37	37.00	95.81	1.04
4	1987	762.000	248.000	38	38.00	98.43	1.02

TABLE 3.3

FREQUENCY ANALYSIS - THREE-PARAMETER LOGNORMAL DISTRIBUTION

GANDER RIVER AT BIG CHUTE 02YQ001

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	604.526	171.038	0.283	0.871	5.995
LN X SERIES	6.365	0.289	0.045	-0.465	4.719

X(MIN)= 248.000
X(MAX)= 1190.000
LOWER OUTLIER LIMIT OF X= 269.611

TOTAL SAMPLE SIZE= 38
NO. OF LOW OUTLIERS= 1
NO. OF ZERO FLOWS= 0

AFTER REMOVAL OF ZEROES AND/OR LOW OUTLIERS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	614.162	162.605	0.265	1.165	6.484
LN X SERIES	6.388	0.255	0.040	0.114	3.823
LN(X-A) SERIES	6.692	0.189	0.028	0.342	4.135

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

PARAMETERS OF THE 3LN WHICH DUPLICATES THE CONDITIONAL FUNCTION:
A= -206.303 M= 6.672 S= 0.204

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	245.00
1.050	0.952	356.00
1.250	0.800	459.00
2.000	0.500	584.00
5.000	0.200	731.00
10.000	0.100	819.00
20.000	0.050	898.00
50.000	0.020	994.00
100.000	0.010	1060.00
200.000	0.005	1130.00
500.000	0.002	1210.00

TABLE 3.4

FREQUENCY ANALYSIS - THREE-PARAMETER LOGNORMAL DISTRIBUTION

GANDER RIVER AT OUTLET OF GANDER LAKE 02YQ002

SAMPLE STATISTICS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	612.187	242.019	0.395	0.343	4.780
LN X SERIES	6.327	0.475	0.075	-1.298	6.521

X(MIN)= 152.000
X(MAX)= 1180.000
LOWER OUTLIER LIMIT OF X= 189.478

TOTAL SAMPLE SIZE= 16
NO. OF LOW OUTLIERS= 1
NO. OF ZERO FLOWS= 0

AFTER REMOVAL OF ZEROES AND/OR LOW OUTLIERS

	MEAN	S.D.	C.V.	C.S.	C.K.
X SERIES	642.867	215.921	0.336	0.825	5.242
LN X SERIES	6.414	0.336	0.052	-0.080	3.438

SOLUTION OBTAINED VIA MAXIMUM LIKELIHOOD

DISTRIBUTION IS UPPER BOUNDED AT (U+A/K)= 0.3290E+04
PARAMETERS OF THE GEV WHICH DUPLICATES THE CONDITIONAL FUNCTION:
U= 530.69 A= 181.110 K= 0.066

FLOOD FREQUENCY REGIME

RETURN PERIOD	EXCEEDANCE PROBABILITY	FLOOD
1.003	0.997	193.00
1.050	0.952	321.00
1.250	0.800	443.00
2.000	0.500	596.00
5.000	0.200	789.00
10.000	0.100	910.00
20.000	0.050	1020.00
50.000	0.020	1150.00
100.000	0.010	1250.00
200.000	0.005	1340.00
500.000	0.002	1450.00

3.4 DISCUSSION OF RESULTS

The availability of a thirty-eight year set of data is ample for estimation of the 1:20 year and 1:100 year floods for rivers in a temperate climate. The evidence of trend and non-homogeneity at the 5% level of significance, although of marginal significance, could be indicative of changes with time of the characteristics of the basin or stage-discharge relationship with time. The latter effect is considered unlikely, as the Water Survey of Canada regularly gauge and update the stage-discharge relationships at each streamflow station. However, the area has been subject to several major forest fires and tree harvesting for pulp and paper, during the period of record which could alter basin characteristics and result in behaviour showing non-homogeneity and trend.

For comparison purposes, a frequency analysis was also carried out on the data from Gander River at the Outlet of Gander Lake (Station No. 2YQ002). These data cover the sixteen year period from 1924 to 1939. The results of this analysis, are shown in Table 3.4 and Figure 3.2.

The comparison of the results of both analyses is shown in Table 3.5 (below).

Table 3.5

Comparison of Results of Frequency Analysis
on Gander River
Big Chute versus Outlet Gander Lake

RETURN PERIOD (Years)	FLOOD FLOWS - m^3/s		% DIFFERENCE
	AT BIG CHUTE	AT OUTLET GANDER LAKE	
2	584	593	2%
20	898	1050	17%
100	1060	1330	25%
Period of Record: Big Chute 1950-1987 = 38 years Outlet Gander Lake 1924-1939 = 16 years			

3.4 DISCUSSION OF RESULTS (Cont'd)

Good agreement is shown on the mean annual flood, but increasing differences appear at greater return periods, the consequence of differing values for the coefficient of variation (shown as differing slopes on the graphical plots in Figures 3.1 and 3.2).

Given the longer period of record and higher quality of data, the results based on data from Big Chute are preferred. The differences between both analyses are representative of the variations in estimates using differing data set. As a test of the practical significance of such differences, it is suggested that impact on the flood levels for the 1:20 year and 1:100 year flood be examined in sensitivity analyses utilizing flow variations of ± 8.5% and 12.5%, respectively:

giving Q_{20} = 898 ± 76 m^3/s
and Q_{100} = 1060 ± 133 m^3/s

It is perhaps worth noting that the close agreement in the mean annual floods at both stations supports the assumption, noted in Section 3.2, that peak flows in the study area would be sensibly the same as flows measured at Big Chute. As an outside limit, the effect of differing drainage areas can be estimated by assuming peak flows are proportional to the drainage area raised to an exponent of 0.76*; thus giving an adjustment of

$$\left[\frac{4150\ km^2}{4440\ km^2}\right]^{0.76} = 0.96,$$

This reduction of only 4% has little significant when compared with the inherent errors in determining Q_{20} and Q_{100}.

Finally, it should be noted that the peak flows observed in the study area (1190 m^3/s in January 1983 and 1180 m^3/s in May 1926) are both somewhat larger than Q_{100}. In the case of the 1983 flow, there is much evidence to suggest it was a flood of extremely rare occurrence - at least in areas where its impact was centred (the Salmon River - Bay d'Espoir and Exploits River watersheds). In these areas the return period of the January, 1983 storm was estimated to exceed 500 years (2). Although its impact was less severe on Gander River - this storm is still considered to be very rare, with

* From drainage area exponent in regional flood frequency analysis - Reference 10.

3.4 DISCUSSION OF RESULTS (Cont'd)

a return period somewhat greater than one hundred years. In the case of the May, 1926 storm there is insufficient information to assess its impact or to evaluate the reliability of its determination. However, the preceding arguments would apply, suggesting it also was a very rare storm.

In situations such as this, an approach sometimes applied is to base zoning regulations, bridge and/or spillway design on the flood of record rather than accepting a smaller flood of specified return period. The implications of this approach were tested by sensitivity tests as explained in Section 4.4.

FIGURE 3.1 GANDER RIVER AT BIG CHUTE (02YQ001)

FREQUENCY ANALYSIS

THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD

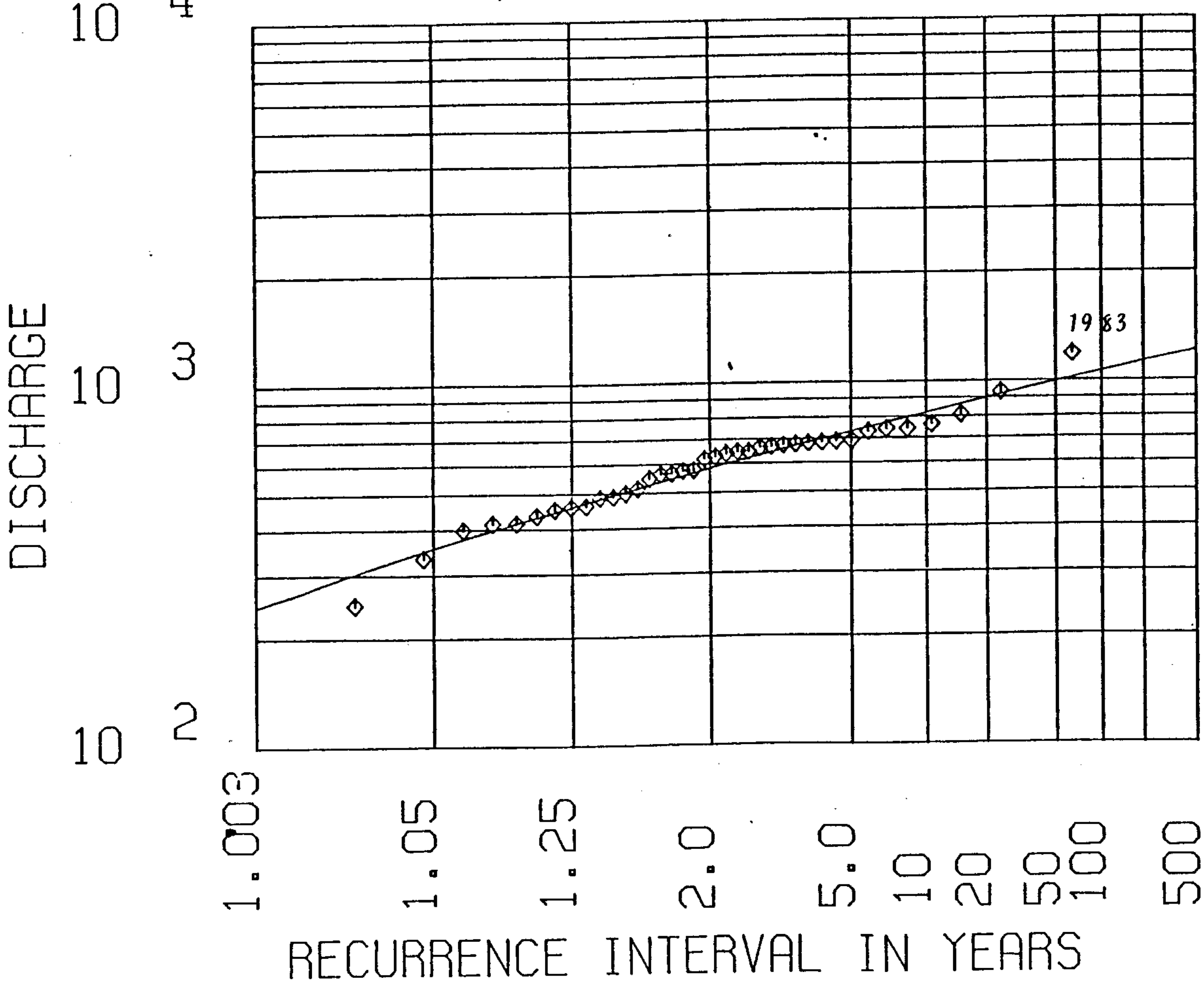


FIGURE 3.2 GANDER RIVER AT OUTLET GANDER LAKE (02YQ002)

FREQUENCY ANALYSIS

THREE PARAMETER LOGNORMAL-MAX LIKELIHOOD

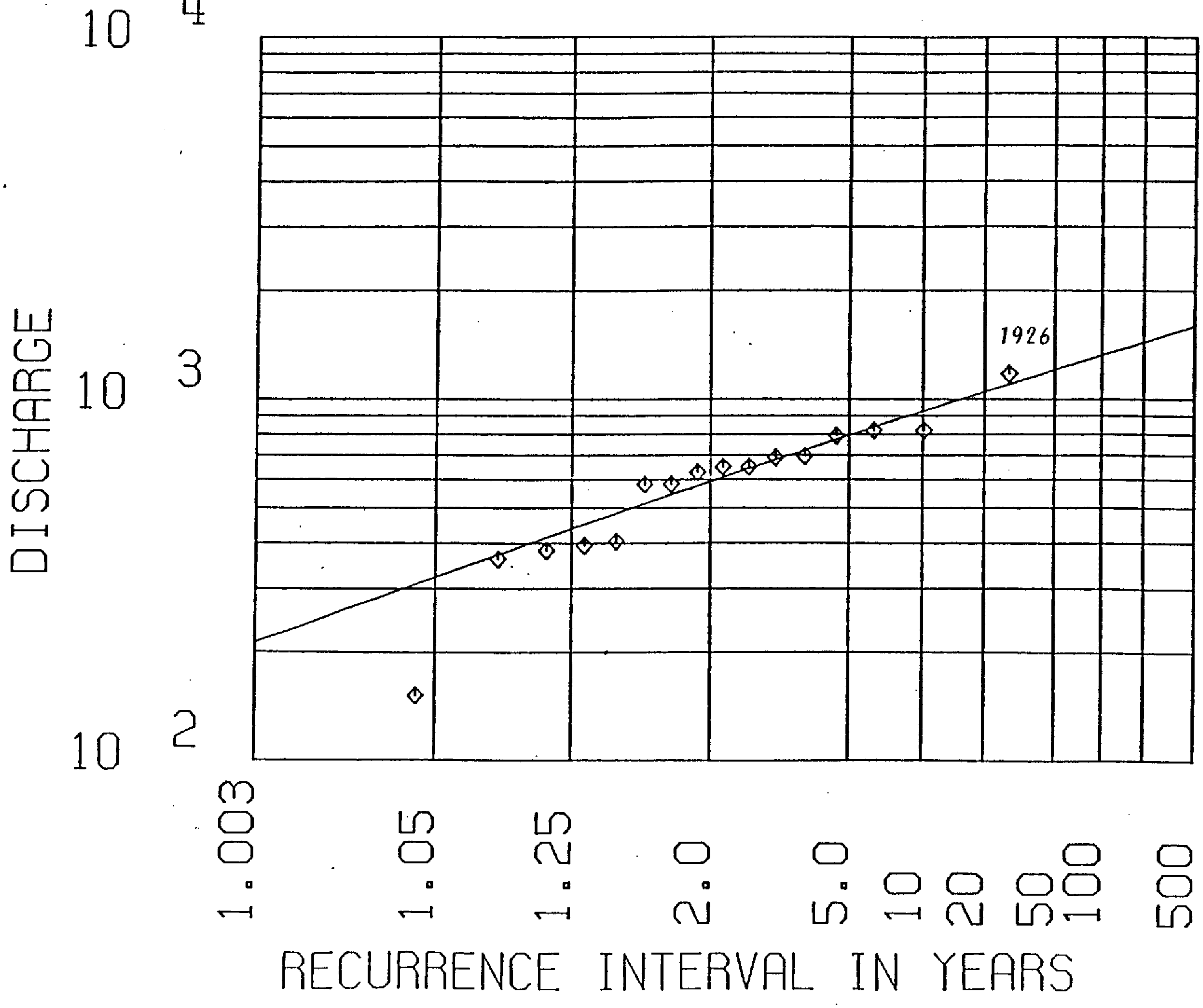


FIGURE 3.2

PART FOUR

HYDRAULICS

4. HYDRAULICS

4.1 PREAMBLE

The purpose of the hydraulics phase of the study was to determine water level profiles through the study reach for the 1:20 year and 1:100 year floods and to use these profiles to establish flood risk contours delineating the extent of flooding.

4.2 SETTING UP THE HEC-2 MODEL

Computation of water profiles was undertaken using the HEC-2 Water Profile Model developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. This program was selected because it is well tested and is widely used for back water calculations by consultants and government authorities. HEC-2 uses the standard step procedure to compute backwater profiles for a pre-selected flow starting from a cross-section at which the water level is known. In general, head losses are determined between adjacent sections using Manning's Equation. The program incorporates procedures for handling composite sections, with river bed and over bank zones having different roughness coefficients [Manning's "n" values]. At singularities, such as bridges, head losses are computed using alternative formulae. Further information on the features of this program can be found in Reference 5.

Prior to setting up the model, the study reach was inspected on foot and ten cross-sections were selected to describe the geometrical features of the river. In general, these cross-sections were selected above and below "rattles", or fast water zones, where transitions between subcritical and supercritical flow were suspected.

During the field program these river cross-sections were measured, using an electronic depth sounder, to measure depth and determining the distance traversed from the elapsed time of travel across the river. Cross-section widths were estimated by stadia and water levels by differential levelling from a geodetic bench mark in the area. Cross-section shape above water was generally determined from the contours shown on the 1:2500 scale base maps (the accuracy of the contours on the base map were verified during the field program by spot checks at appropriate locations, see Field Report (6) for further details). River flows occurring at the time of the cross-section survey were derived from gauge readings at Big Chute (Station No. 02YQ001).

4.2 SETTING UP THE HEC-2 MODEL (Cont'd)

During the interview survey, residents were asked to point out high water marks which occurred during the January 1983 flood. These locations were noted on the base mapping and their elevations were subsequently measured by differential levelling. Further details on the physical and interview surveys are given in the Field Report (6).

An additional, synthetic cross-section (#11), was added downstream to provide a starting point for the model. This cross-section was placed at the head of a rapids section 270 m downstream of cross-section 10. Its width and side slopes were estimated from 1:50,000 topo mapping, its depth was assumed to be critical and its invert elevation was chosen from consideration of head losses between cross-sections 10 and 11. In essence, this cross-section was tailored to fit the conditions observed in the field during the field survey - and was then checked for the January 1983 flood to see if the calculated profile was consistent with the profile observed. Both theoretical and observed profiles were found to be similar for the January 1983 flood; hence it was concluded that cross-section 11, as estimated, provided a suitable starting point for the model. Locations of the cross-sections are shown in Figure 4.1, and cross-section details in Figures 4.2 to 4.12.

It was anticipated that critical flow sections would be found at the head of each "rattle" and that separate HEC-2 models would be required to represent the hydraulics of the river between each critical section. In order to identify such sections, the critical flow profile was first determined for the flow observed during the field program. It was then compared to the observed water levels. Contrary to expectations this comparison showed that flow was sub-critical throughout the reach. This greatly simplified computations and permitted the use of a single HEC-2 model for the entire study reach.

4.3 CALIBRATION AND VERIFICATION OF THE MODEL

Only two coincident sets of flow and water level data were available for calibration and verification of the model, the mean flow for November 15-17, 1983 and the January 1983 flood flow. These flows were:

109 m^3/s on November 16, 1988

and

1190 m^3/s on January 15, 1983

4.3 CALIBRATION AND VERIFICATION OF THE MODEL (Cont'd)

Calibration of the model was based on the October 16, 1988 flow. This primarily involved selection and adjustment of values for Manning's "n" roughness coefficient, until the computed and measured water level profiles generally agreed within a tolerance of $\pm$ 10 cm.

Preliminary trials with the model indicated that a Manning's "n" value in the order of 0.050 would be required to reproduce the observed water profile. This value was higher than the expected value of 0.040, based on Ven T. Chow's classification (7). The high roughness value exhibited by the river was evidently due to the fact that the depth of flow and dimensions of roughness elements were of the same order of magnitude. The initial roughness value was therefore estimated using the Keulegan-Bray Equation [Reference 8]. This equation provides an explicit relationship between Darcy-Weisbach's friction factor "f", a characteristic bed roughness dimension D_{84} and the hydraulic radius (R), as below:

$$f = \left(2.21 + 2.03 \log \frac{R}{3.5 D_{84}}\right)^{-2}$$

and,

$$\frac{1}{\sqrt{f}} = \frac{1}{\sqrt{8g}} \cdot \frac{R^{1/6}}{n}$$

hence,

$$n = \frac{R^{1/6}}{\sqrt{8g}} \cdot \frac{1}{\left[2.21 + 2.03 \log \frac{R}{3.5 D_{84}}\right]}$$

where:

- D_{84} = bed material size such that 84% of particles are smaller than this size.
- R = hydraulic radius
- n = Manning's roughness coefficient
- f = Darcy-Weisbach's friction factor

The above formula was used to estimate Manning's "n" value for the initial HEC-2 trial runs.

4.3 CALIBRATION AND VERIFICATION OF THE MODEL (Cont'd)

Measured water levels from the October 1988 field survey were not all measured at the same time, but over a three day period. During this period of time the flow in the river was falling, as below:

117 m^3/s on November 15, 1988
109 m^3/s on November 16, 1988
107 m^3/s on November 17, 1988

To account for this variation in flow, the observed flow depths were adjusted, as below, to correspond to the depths (water levels) at a flow of 109 m^3/s.

The adjustment factors were determined as follows:

$$\text{Since } Q = \frac{1}{n} \cdot A\, R^{2/3} \cdot s^{1/2} \qquad \text{(Manning's Equation)}$$

For a wide river:

$$A \sim y.B, \quad \text{where } y = \text{mean depth, } B = \text{width}$$

$$R \sim y$$

$$\text{Thus} \quad Q \sim \frac{1}{n} \cdot y^{5/3}.s.^{1/2}$$

For a small change in depth, the width, slope and Manning's "n" may be assumed to remain constant, thus:

$$Q = k.y^{5/3}$$

$$\text{or} \quad y = \frac{Q^{3/5}}{k}$$

Therefore, for a change in flow, the new depth y, can be determined by:

$$y_1 = y_o \cdot \frac{Q_1}{Q_o}^{3/5}$$

where y_o = measured mean depth

y_1 = adjusted mean depth, for Q = 109 m^3/s

The resulting adjustments to measured water levels were found to be minimal, generally less than 4 cm.

4.3 CALIBRATION AND VERIFICATION OF THE MODEL (Cont'd)

Since flow restrictions at each bridge were small it was not necessary to model bridge losses explicitly. Adjustments to Manning's "n" for the reaches containing each bridge were found to reproduce observed water levels within the desired accuracy.

A number of trial runs were made and "n" values adjusted until close agreement, generally within ± 10 cm, was obtained at all stations. Table 4.1 summarizes the results of the calibration run, which is also shown graphically in Figure 4.13.

Table 4.1

Calibration Run, Comparison of Measured and Computed Water Levels November, 1988, Q = 109 m^3/s

CROSS-SECTION	CHAIN-AGE (m)	MANNINGS "n"	HEC-2 W.L. (m)	ACTUAL W.L. (m)	DIFFERENCE HEC2-ACTUAL (m)
1	0	0.050	25.40	25.39	+0.01
2	685	0.050	25.40	25.38	+0.02
3	945	0.050	25.37	25.36	+0.01
4	1155	0.060	25.29	25.33	-0.04
5	1325	0.050	25.09	25.00	+0.09
6	1585	0.050	24.39	24.48	-0.09
7	1680	0.050	24.40	24.22	+0.18
8	1790	0.050	24.01	24.22	-0.21
9	1920	0.050	24.09	24.02	+0.07
10	2080	0.050	24.05	24.02	+0.03
11	2350	0.060	23.48	-	-

As can be seen from Table 4.1 and Figure 4.13, the agreement between the computed and measured profiles is excellent except in the vicinity of Cross-Sections 7 and 8. These discrepancies probably relate to an incomplete representation of the local hydraulic effects at the Trans

4.3 CALIBRATION AND VERIFICATION OF THE MODEL (Cont'd)

Canada Highway Bridge (Cross-Section 8). The fact that the depth of flow at Cross-Section 8 is just marginally above critical level means that water level estimates would tend to be very sensitive to errors in estimates of area, etc. Since these effects are localized, it was not considered of sufficient practical importance to attempt further adjustments to the model.

For the verification run the Manning's "n" values determined from the calibration process were adjusted using the Keulegan-Bray equation to account for the radical differences in flow depths between the condition for October 1988 (very low flows) and those of January 1983 (record high flows). The Manning's "n" values thus obtained were assumed to apply in overbank areas as well. The HEC-2 model was adjusted, as above and the verification profile computed assuming the January 15, 1983 flood flow of 1190 m^3/s. Table 4.2 summarizes the results of the verification run, while Figure 4.14 provides a graphical comparison of observed and computed flood level.

As can be seen from Figure 4.14, the computed and measured profiles are in reasonably close agreement, especially when one considers that the flood levels are based on personal recollections, almost six years after the event. With the exception of points 2, 3, 4 and 9,all other points are within a tolerance of $\pm$ 15 cm. These four points could be effected, by errors in observation or lapses of memory.

A complete verification of the model was not possible because no large flows occurred within the study period, hence overland "n" values could not be directly verified. The significance of this problem was examined by a sensitivity analysis, as discussed in the next section of this report. This analysis showed that large changes in overbank "n" values did not produce significant changes in water levels. Hence this single verification run is considered to provide a practical confirmation of the reliability of the HEC-2 model.

Table 4.2

Verification Run, Comparison of Actual and Computed 1983 Flood Levels, Q = 1190 m^3/s

CROSS-SECTION	CHAIN-AGE (m)	MANNINGS "n"	HEC-2 W.L. (m)*	ACTUAL W.L. (m)	DIFFERENCE HEC2-ACTUAL (m)
1	0	0.050	28.56		-
	10	-	(28.51)	28.51	0.00
2	685	0.050	28.52		-
	885	-	(28.41)	28.71	-0.30
3	945	0.050	28.41		-
	1085	-	(28.30)	28.07	+0.23
	1135	-	(28.26)	28.71	-0.45
4	1155	0.060	28.30		-
	1225	-	(28.21)	28.17	+0.04
5	1325	0.050	28.24		-
	1335	-	(28.03)	28.00	+0.13
6	1585	0.050	27.71		-
7	1680	0.050	27.54		-
	1685	-	(27.40)	27.49	-0.09
	1720	-	(27.35)	27.33	-0.02
	1760	-	(27.17)	27.39	-0.22
8	1790	0.050	27.18		-
9	1920	0.050	27.16		-
10	2080	0.050	27.03		-
11	2350	0.060	25.45		-

* Values shown in brackets are estimated by interpolation between elevations calculated using the HEC-2 computer model.

4.4 DETERMINATION OF FLOOD RISK CONTOURS

Having calibrated and verified the HEC-2 model of the study reach, design flood profiles were determined by inputting Q_{20} (= 898 m^3/s) and Q_{100} (= 1060 m^3/s). The resulting profiles are shown in Figure 4.15. Water levels from the HEC-2 model calculations were then used to plot the 1 in 20 year and 1 in 100 year flood risk contours on the base maps (see envelope pocket at end of report).

4.5 SENSITIVITY ANALYSES

The effects on the determination of the 1 in 20 year and 1 in 100 year flood levels due to errors in the main parameters were evaluated in the following sensitivity analyses.

Test 1: Errors in determination of Q_{20}. Evaluated by computing flood levels for:

Q_{20} = 898 $\pm$ 76 m^3/s ($\pm$ 8.5%)

Test 2: Errors in determination of Q_{100}. Evaluated by computing flood levels for:

Q_{100} = 1060 $\pm$ 133 m^3/s ($\pm$ 12.5%)

Test 3: Errors due to variations in overbank roughness. Evaluated by varying overbank Manning's "n" values, as below:

	Mannings "n" Values		
Case	Left Bank	Right Bank	Channel
Case 1 - Base Case	0.050	0.050	0.050
Case 2 - Test Case	0.070	0.070	0.050

The results of these comparisons are summarized in Tables 4.3, 4.4 and 4.5.

4.5 SENSITIVITY ANALYSES (Cont'd)

Table 4.3
Test 1: Comparison of 1 in 20 Year
Flood Levels, Q_{20}, = 898 ± 76 m^3/s

Station	Chainage (m)	Flood Water Levels Q_{20} = 898 (m^3/s)	Q_{20} = 974 (m^3/s)	Diff.	Q_{20} = 822 (m^3/s)	Diff.
11	2350	25.07	25.17	+ 0.10	24.96	- 0.11
10	2080	26.50	26.64	+ 0.14	26.35	- 0.15
9	1920	26.61	26.76	+ 0.15	26.45	- 0.16
8	1790	26.64	26.79	+ 0.15	26.49	- 0.15
7	1680	26.94	27.10	+ 0.16	26.78	- 0.16
6	1585	27.12	27.28	+ 0.16	26.95	- 0.17
5	1325	27.57	27.75	+ 0.18	27.38	- 0.19
4	1155	27.64	27.83	+ 0.19	27.46	- 0.18
3	945	27.78	27.96	+ 0.18	27.60	- 0.18
2	685	27.90	28.07	+ 0.17	27.72	- 0.18
1	0	27.94	28.11	+ 0.17	27.76	- 0.18
Columns (1)	(2)	(3)	(4)	(4)-(3)	(6)	(6)-(3)

Table 4.4
Test 2: Comparison of 1 in 100 Year
Flood Levels, Q_{100}, = 1060 ± 133 m^3/s

Station	Chainage (m)	Flood Water Levels Q_{100} = 1060 (m^3/s)	Q_{100} = 1193 m^3/s)	Diff.	Q_{100} = 927 (m^3/s)	Diff.
11	2350	25.28	25.42	+ 0.14	25.11	- 0.17
10	2080	26.79	27.02	+ 0.23	26.55	- 0.22
9	1920	26.91	27.15	+ 0.24	26.66	- 0.25
8	1790	26.95	27.18	+ 0.23	26.70	- 0.25
7	1680	27.28	27.53	+ 0.25	27.00	- 0.28
6	1585	27.45	27.71	+ 0.26	27.18	- 0.27
5	1325	27.95	28.24	+ 0.25	27.64	- 0.31
4	1155	28.02	28.31	+ 0.29	27.71	- 0.31
3	945	28.15	28.43	+ 0.28	27.85	- 0.30
2	685	28.25	28.53	+ 0.28	27.96	- 0.29
1	0	28.30	28.57	+ 0.27	28.00	- 0.30
Columns (1)	(2)	(3)	(4)	(4)-(3)	(6)	(6)-(3)

4.5 SENSITIVITY ANALYSES (Cont'd)

Table 4.5
Test 3: Impact of Variation in
Bank Roughness for Q_{1983}, = 1190 m^3/s

Station	Chainage (m)	W.L. for Selected Bank Manning's "n" Values Banks = 0.05 (m^3/s)	Banks = 0.70 (m^3/s)	Difference in W.L. (m^3/s)
11	2350	25.45	25.50	+0.05
10	2080	27.03	27.14	+0.11
9	1920	27.16	27.27	+0.11
8	1790	27.18	27.30	+0.12
7	1680	27.54	27.60	+0.06
6	1585	27.71	27.79	+0.08
5	1325	28.24	28.33	+0.09
4	1155	28.30	28.41	+0.11
3	945	28.41	28.54	+0.13
2	685	28.52	28.66	+0.14
1	0	28.56	28.70	+0.14
Columns	(1)	(2)	(3)	(3)-(2)

The following observations can be made from Tables 4.3 to 4.5.

(i) An error of ± 76 m^3/s (± 8.5%) in estimating Q_{20} would produce a change in water elevation of less than ± 0.20 m.

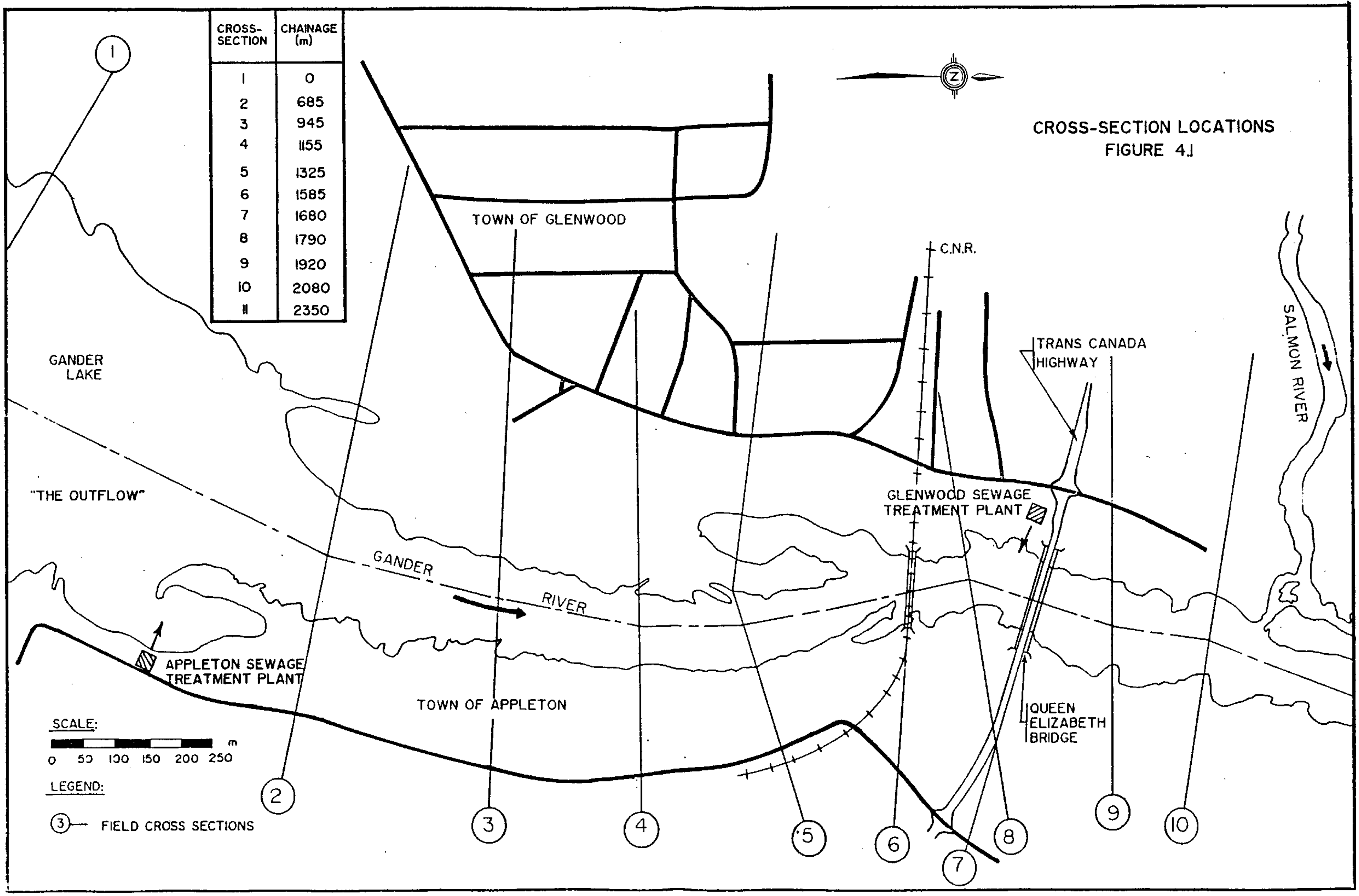
(ii) An error of ± 133 m^3/s (± 12.5%) in estimating Q_{100} would produce a change in water elevation generally less than 0.30 m.

The impact on the extent of flooding due to water level changes of these magnitudes was examined by plotting the revised (upper bound) water levels on the base map. Increases in flooded area on the right bank (Appleton) was found to be negligible for both cases. Increases in flooded area on the left bank (Glenwood) were also found to be small. In the case of Q_{100} = 1193 m^3/s, there is risk of overtopping River Road in Glenwood, in the vicinity of cross-section 3 since flood level and road elevation are similar. The additional flooded area would be covered by a very shallow depth of water that should be more a nuisance than a menace. Three or four houses could be affected.

4.5 <u>SENSITIVITY ANALYSES (Cont'd)</u>

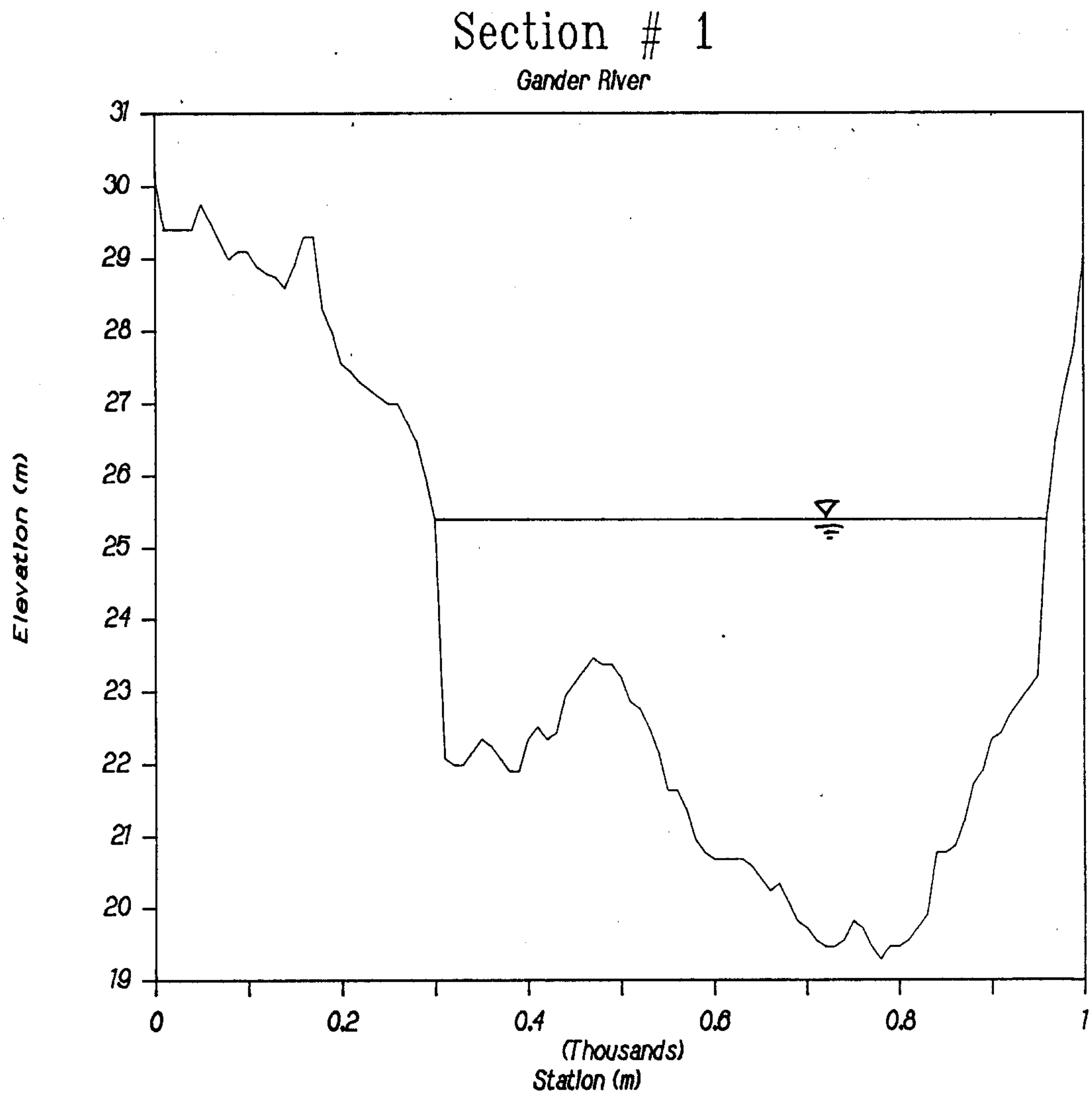
(iii) To check the effect of bank roughness, Manning's "n" values of 0.070 were assumed for overbank areas based on data from Ven T. Chow (1959). This change from the verification run, in which an overbank Manning's "n" of 0.05 was implicitly assumed, produced minimal increases in water level generally less than 0.15 m. This fact suggests that most of the flow in major floods is handled through the main river channel.

The overall observation was that the sensitivity analyses did not reveal any startling impacts which would require further investigation, and thus confirmed the suitability of using the unadjusted 1 in 20 year and 1 in 100 year flood profiles for preparing flood risk maps.



CROSS-SECTION	CHAINAGE (m)
1	0
2	685
3	945
4	1155
5	1325
6	1585
7	1680
8	1790
9	1920
10	2080
11	2350

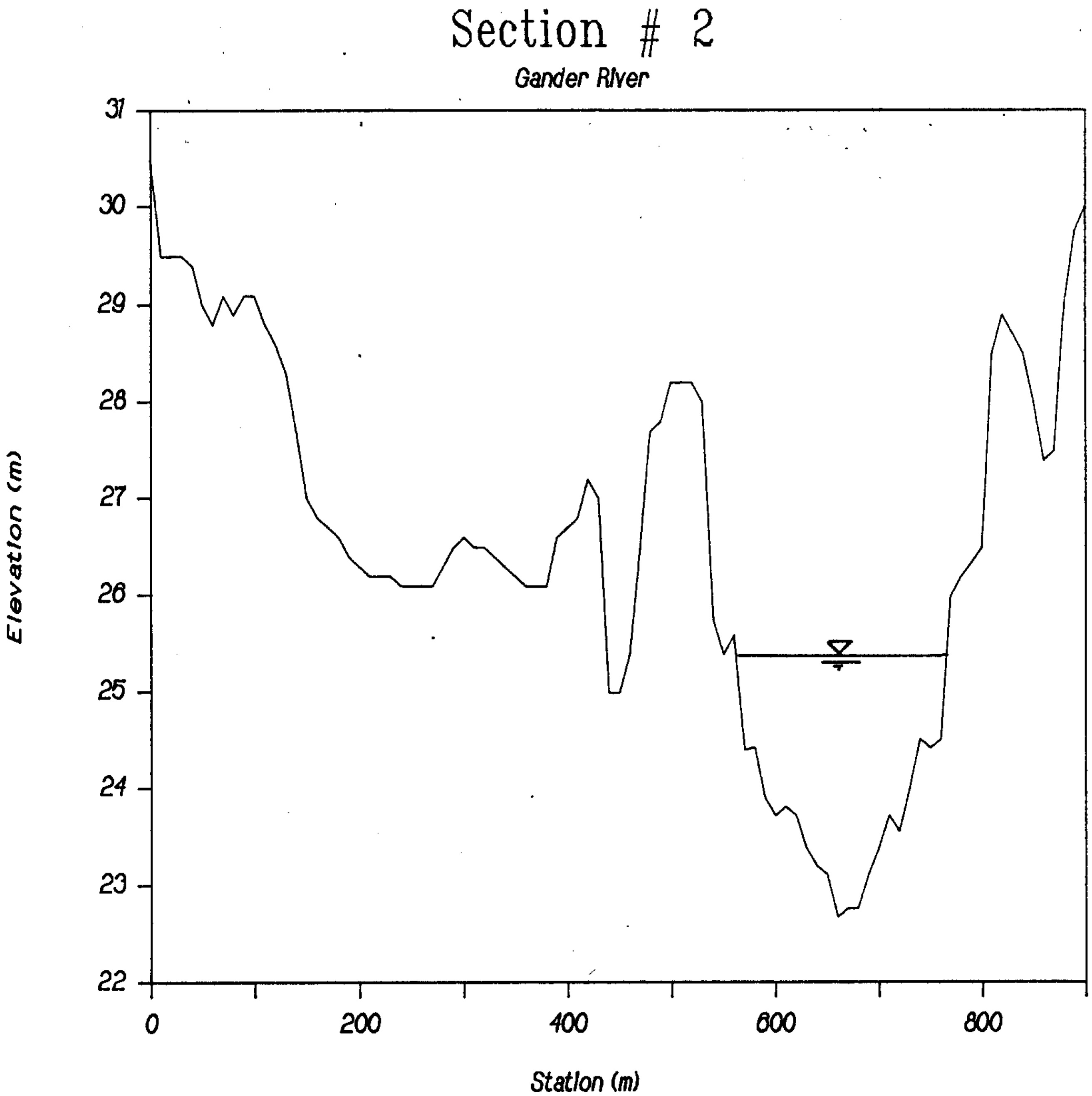
CROSS-SECTION LOCATIONS
FIGURE 4.1



NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

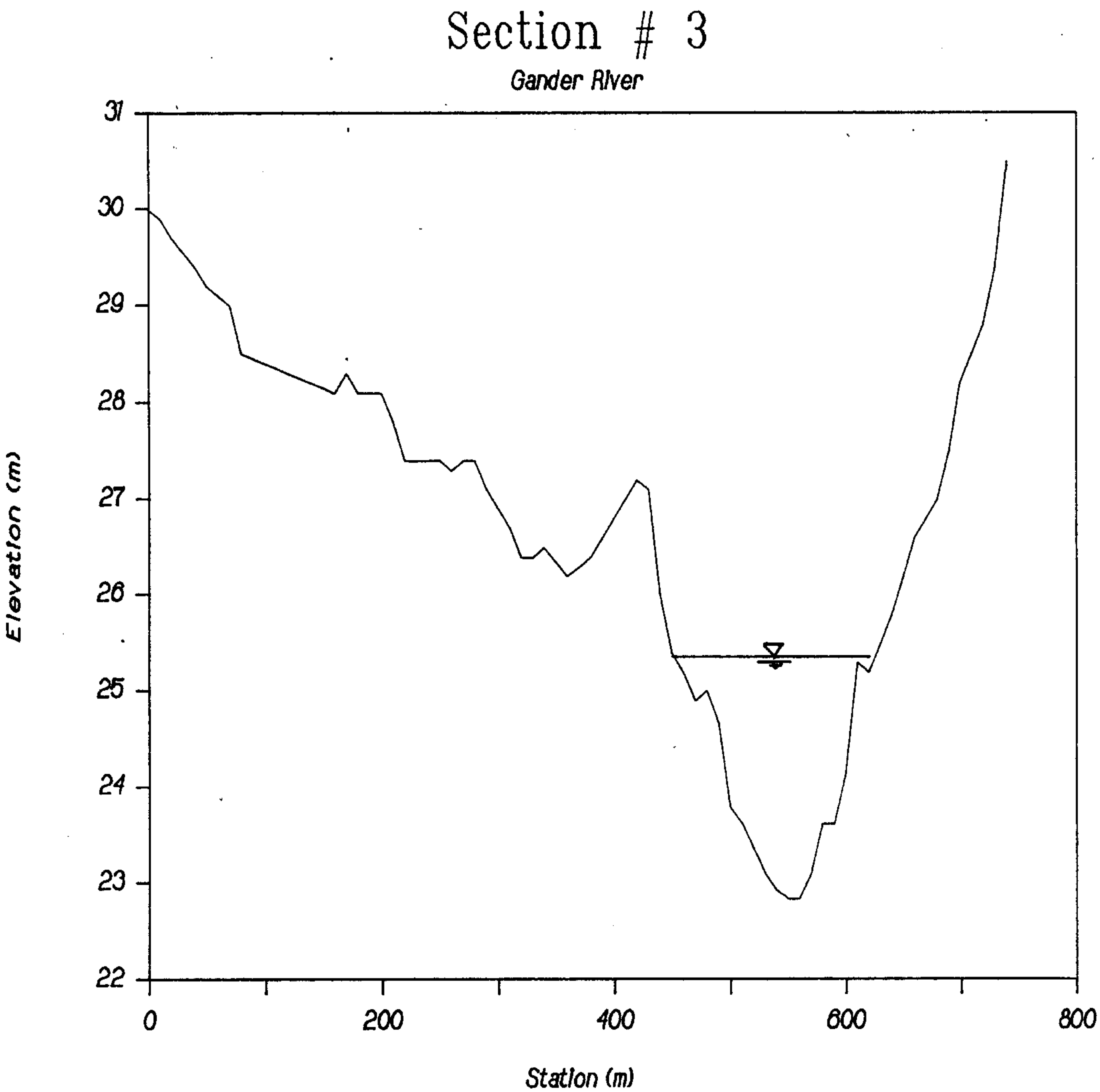
FIGURE 4.2



NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

FIGURE 4.3

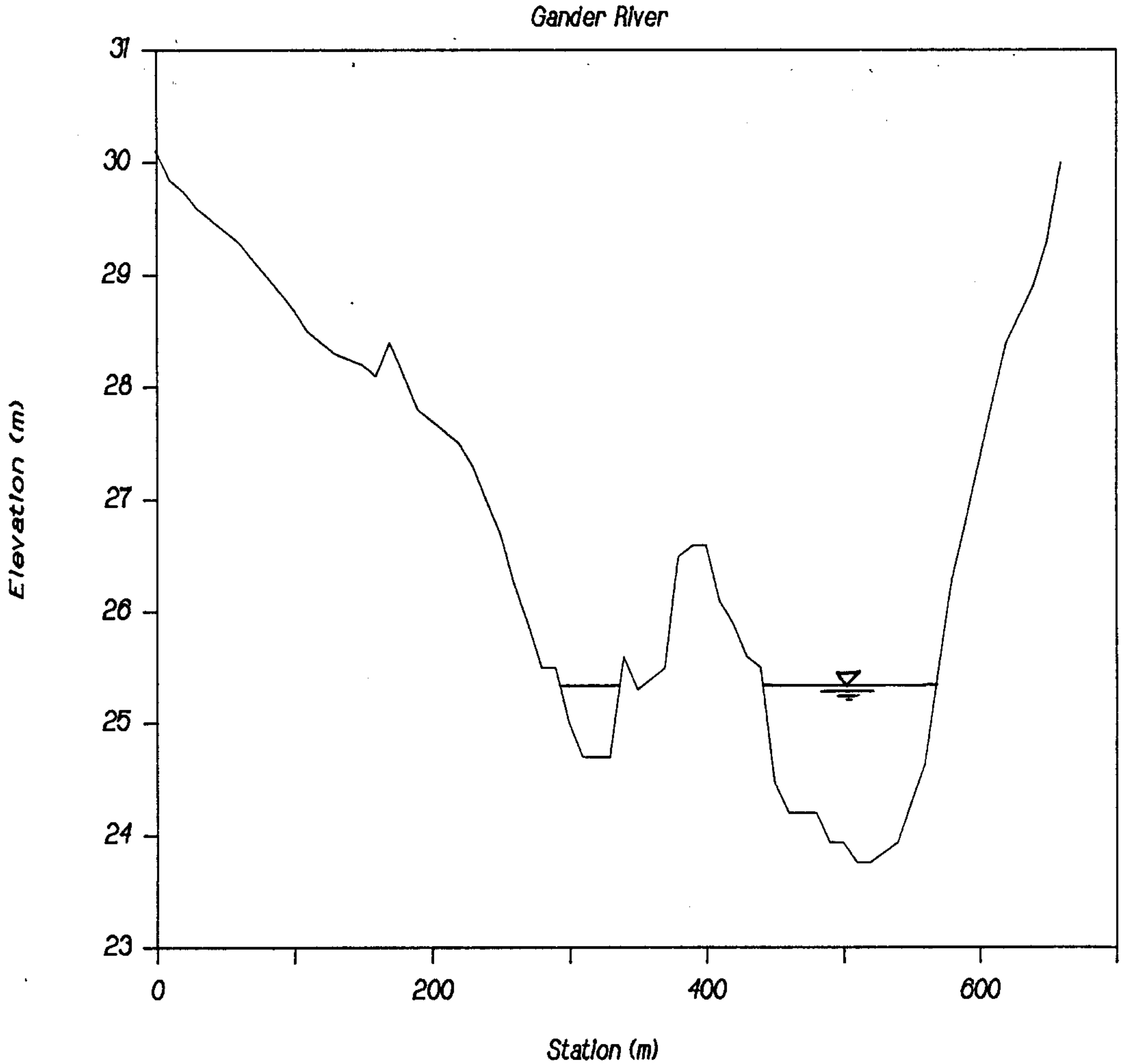


NOTE:

1. All evaluations·geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

FIGURE 4.4

Section # 4

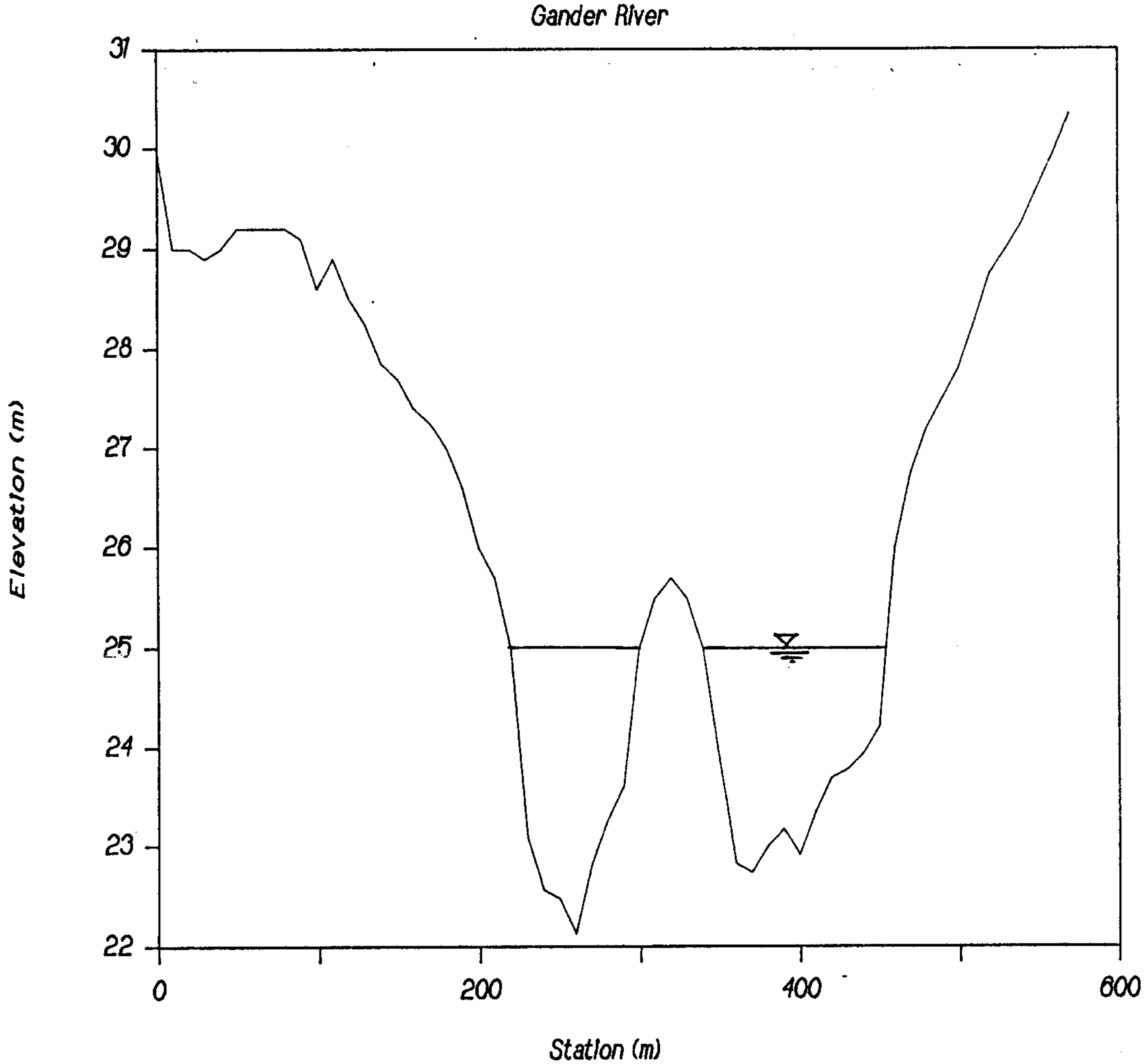


NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

FIGURE 4.5

Section # 5

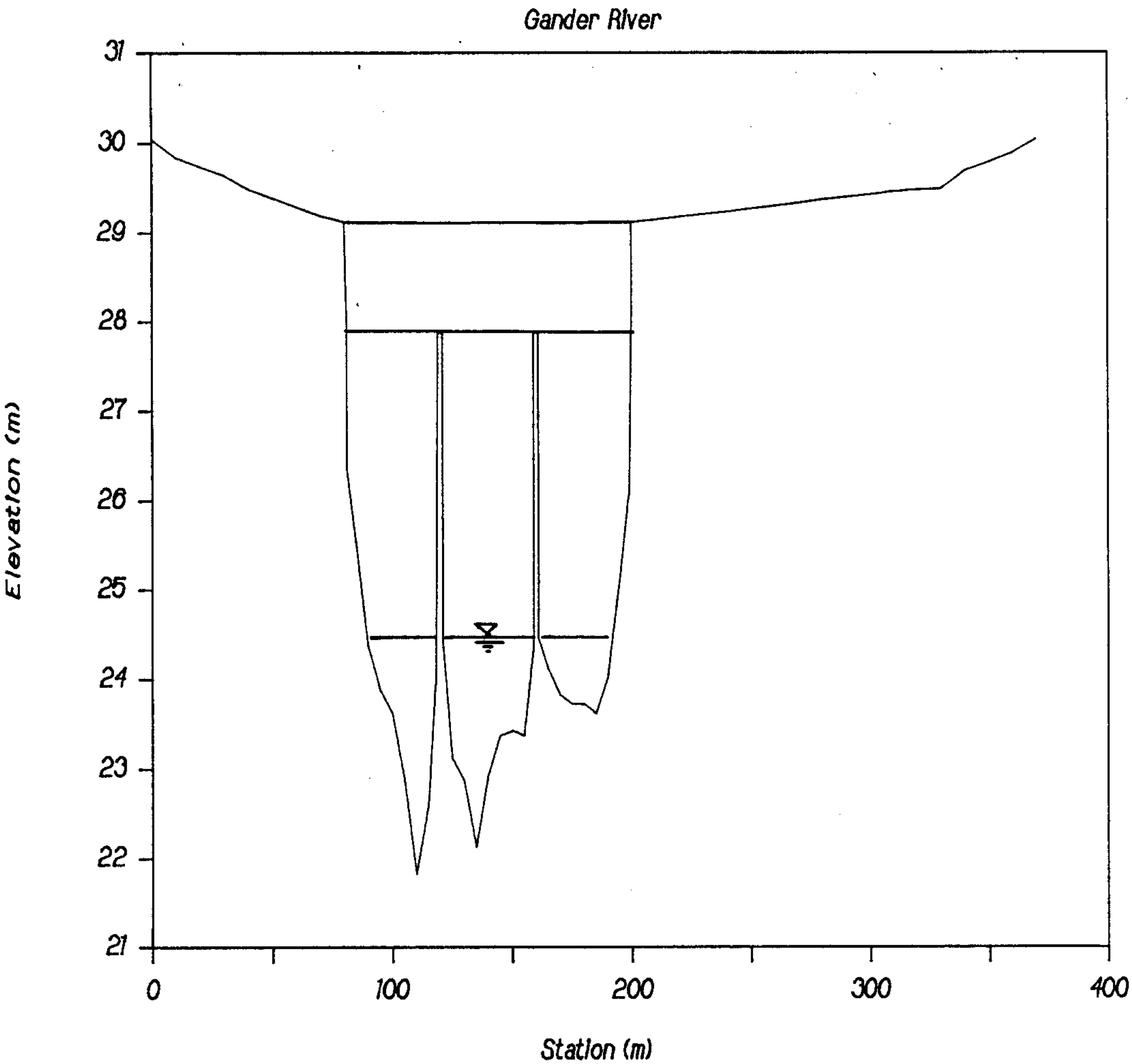


NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

FIGURE 4.6

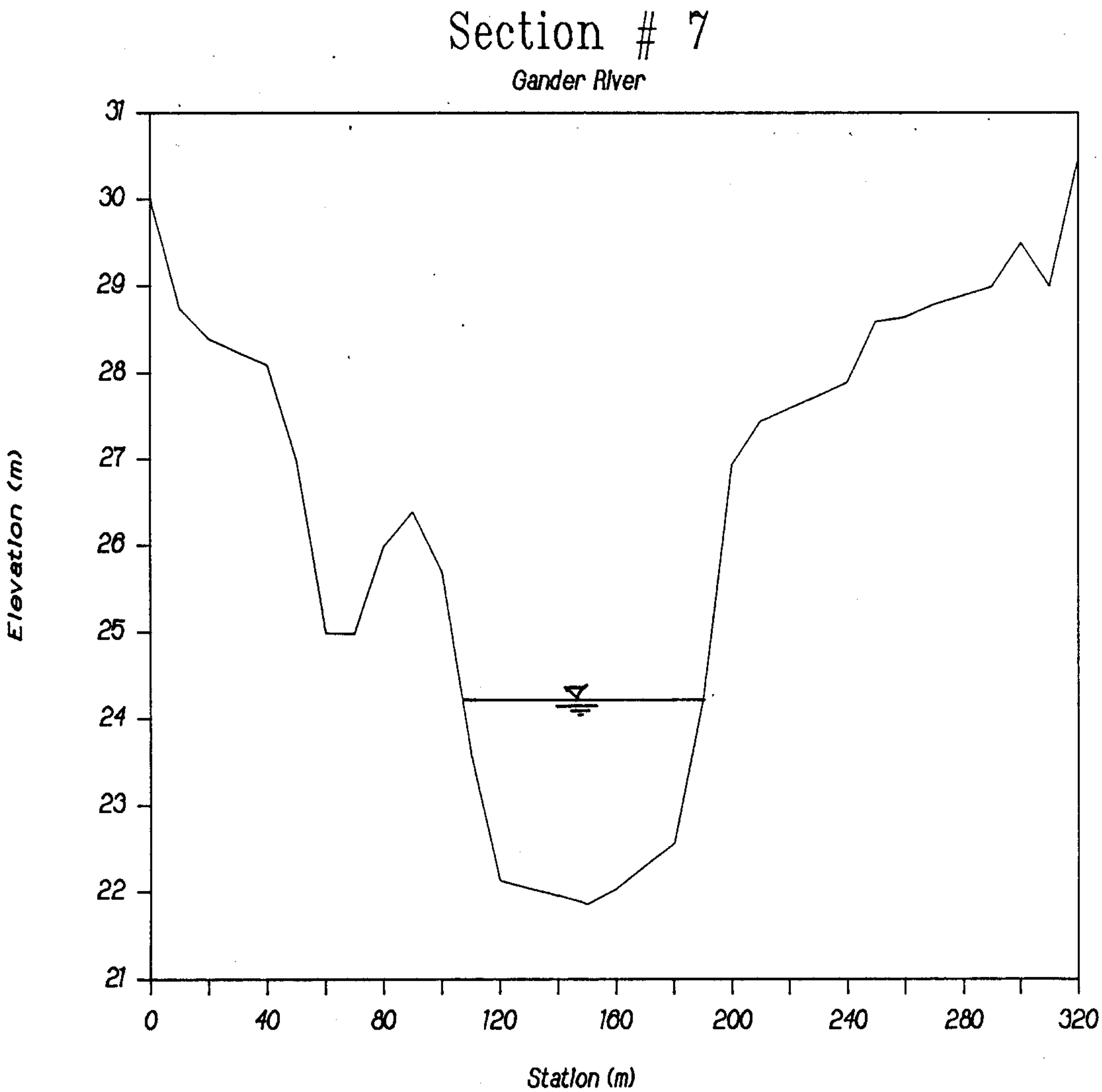
Section # 6



NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

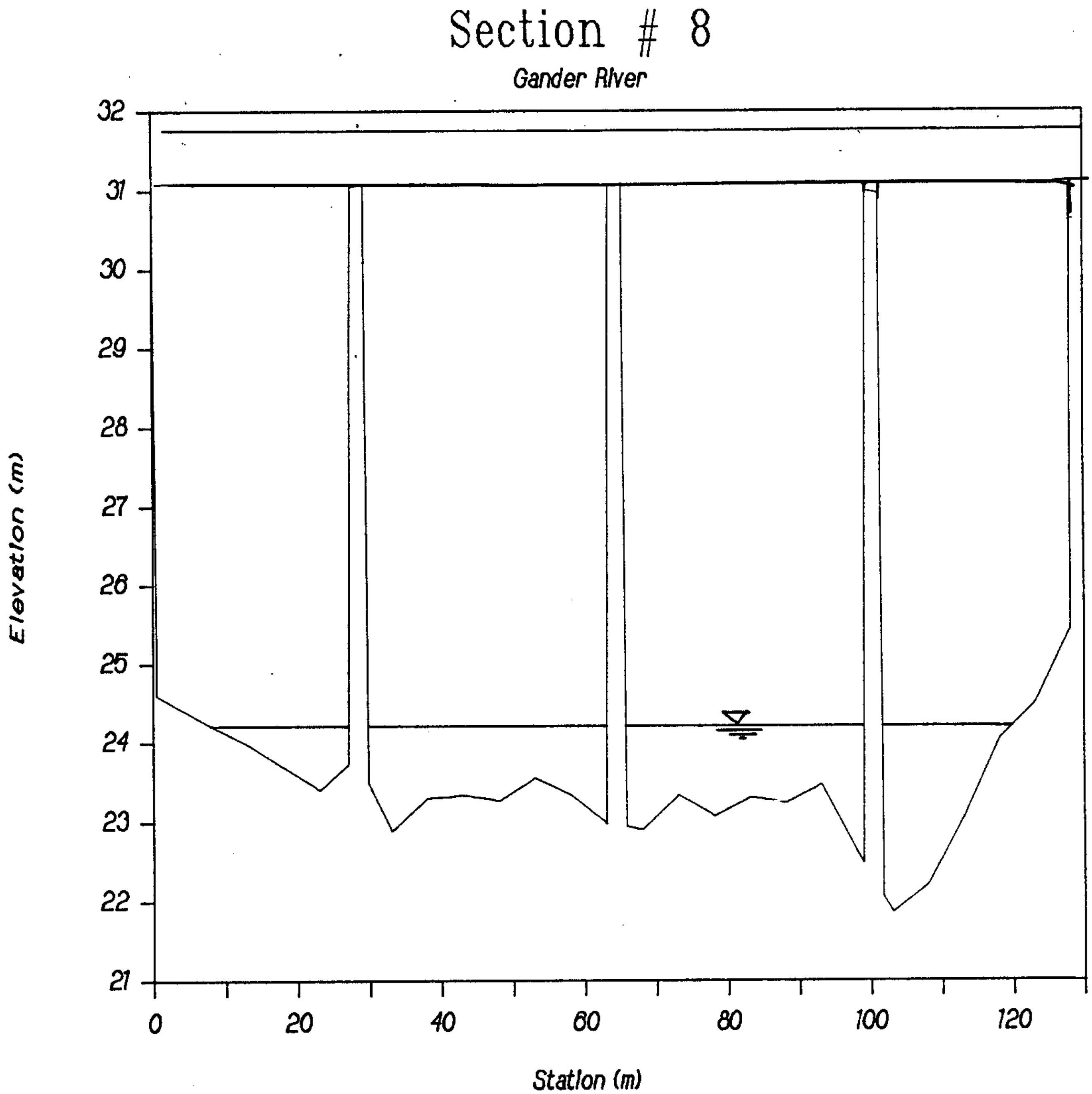
FIGURE 4.7



NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

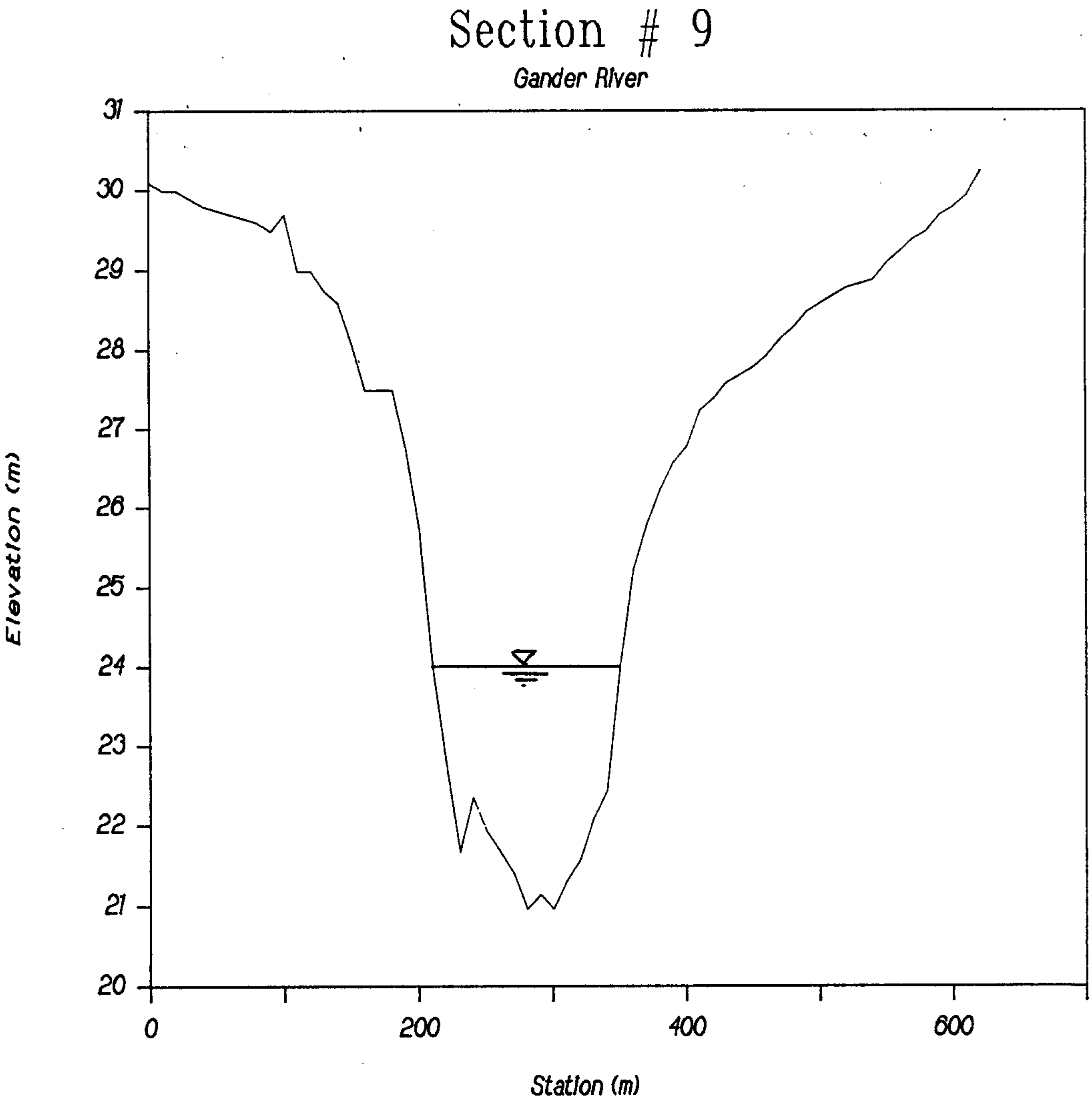
FIGURE 4.8



NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

FIGURE 4.9



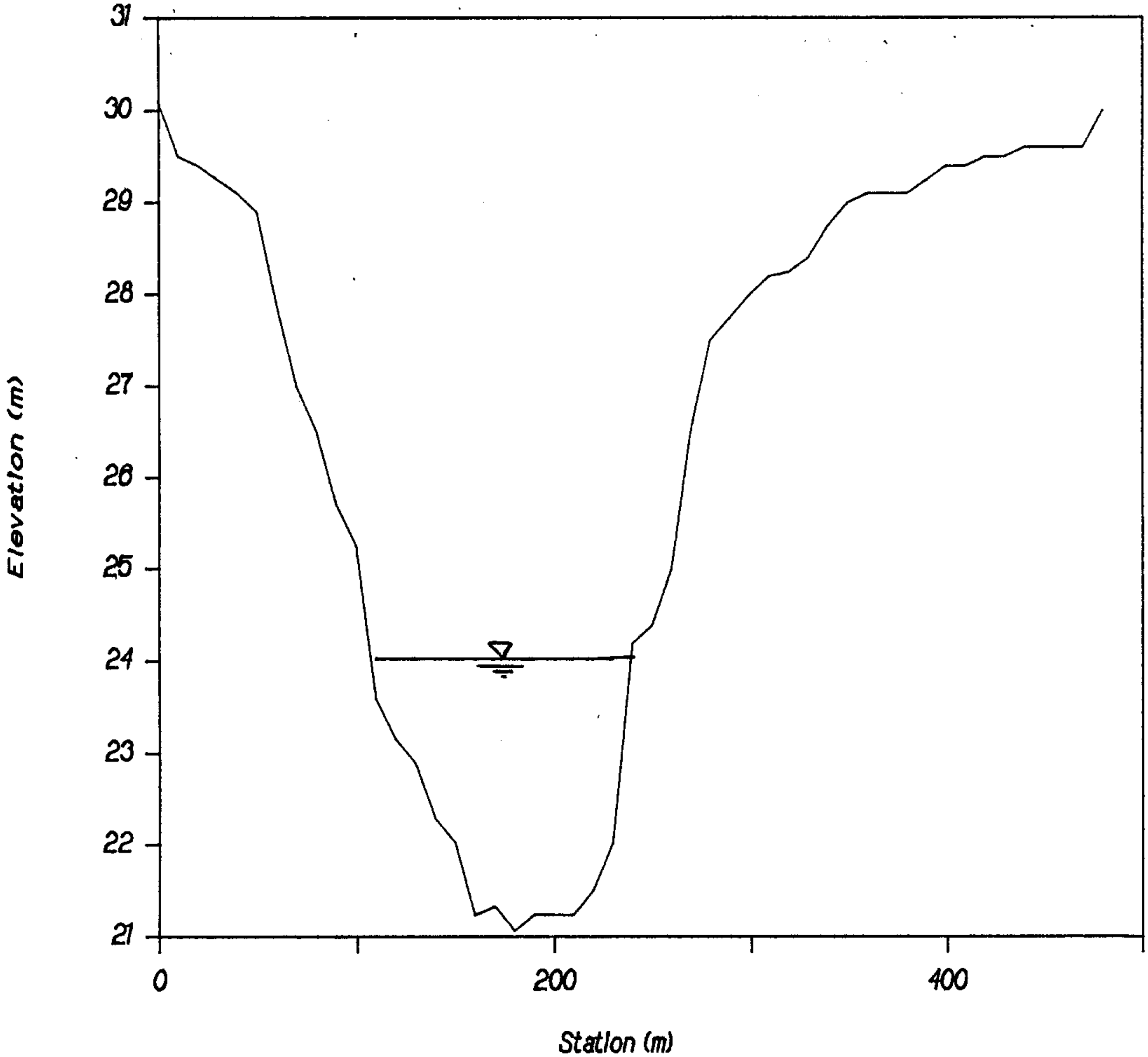
NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

FIGURE 4.10

Section # 10

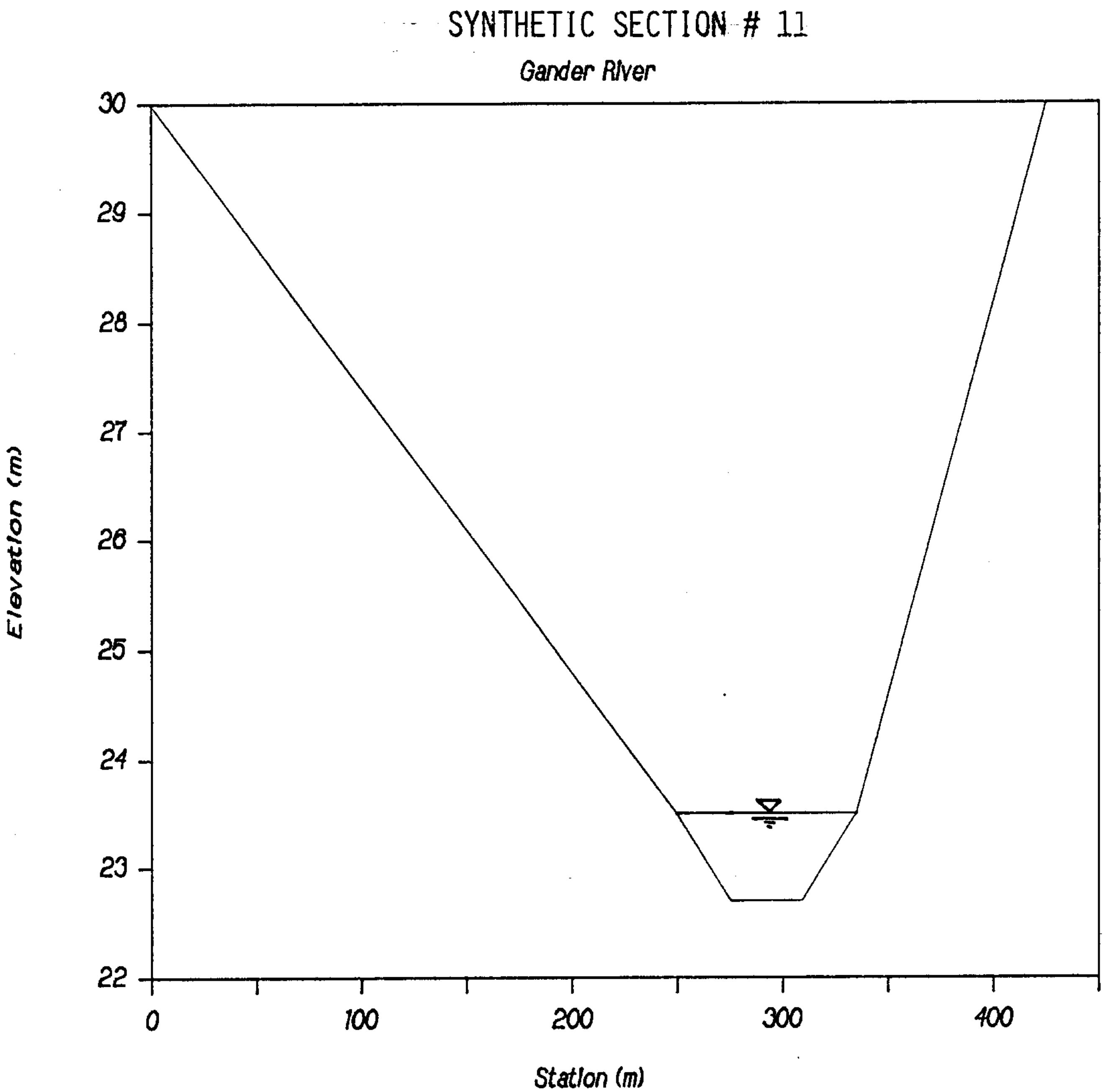
Gander River



NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

FIGURE 4.11

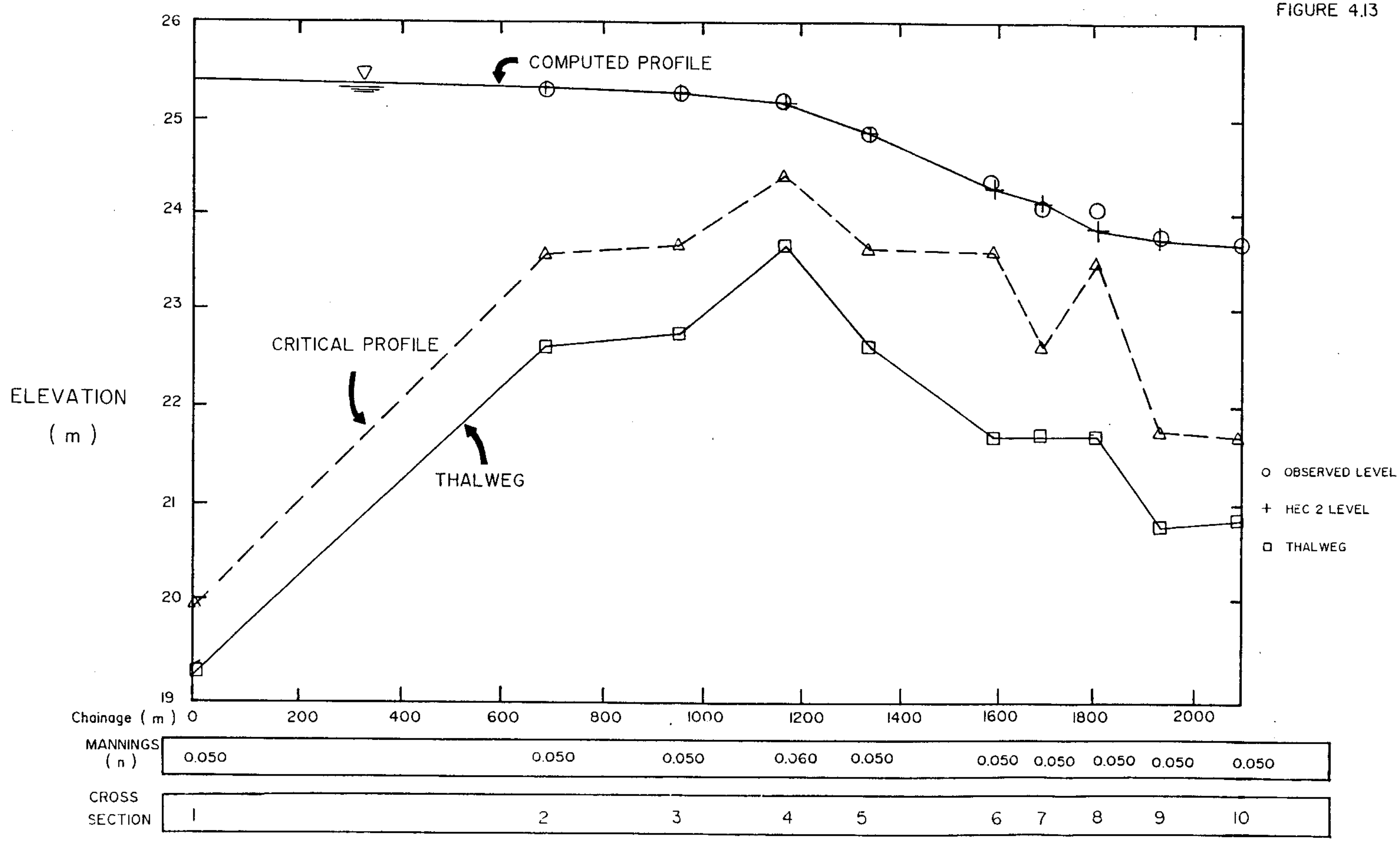


NOTE:

1. All evaluations-geodetic
2. Cross-sections are plotted looking downstream
3. Water level at time of survey

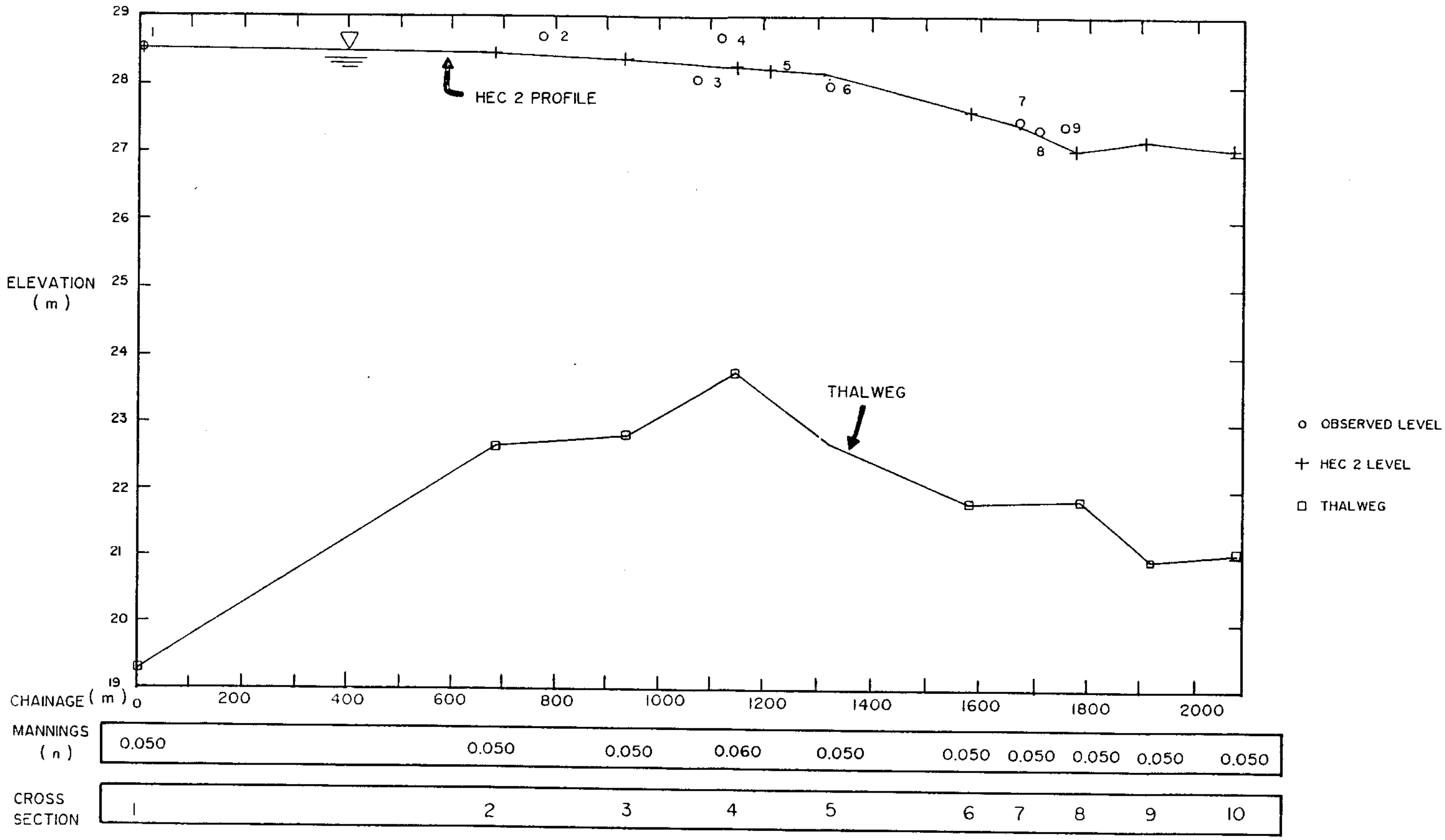
FIGURE 4.12

FIGURE 4.13



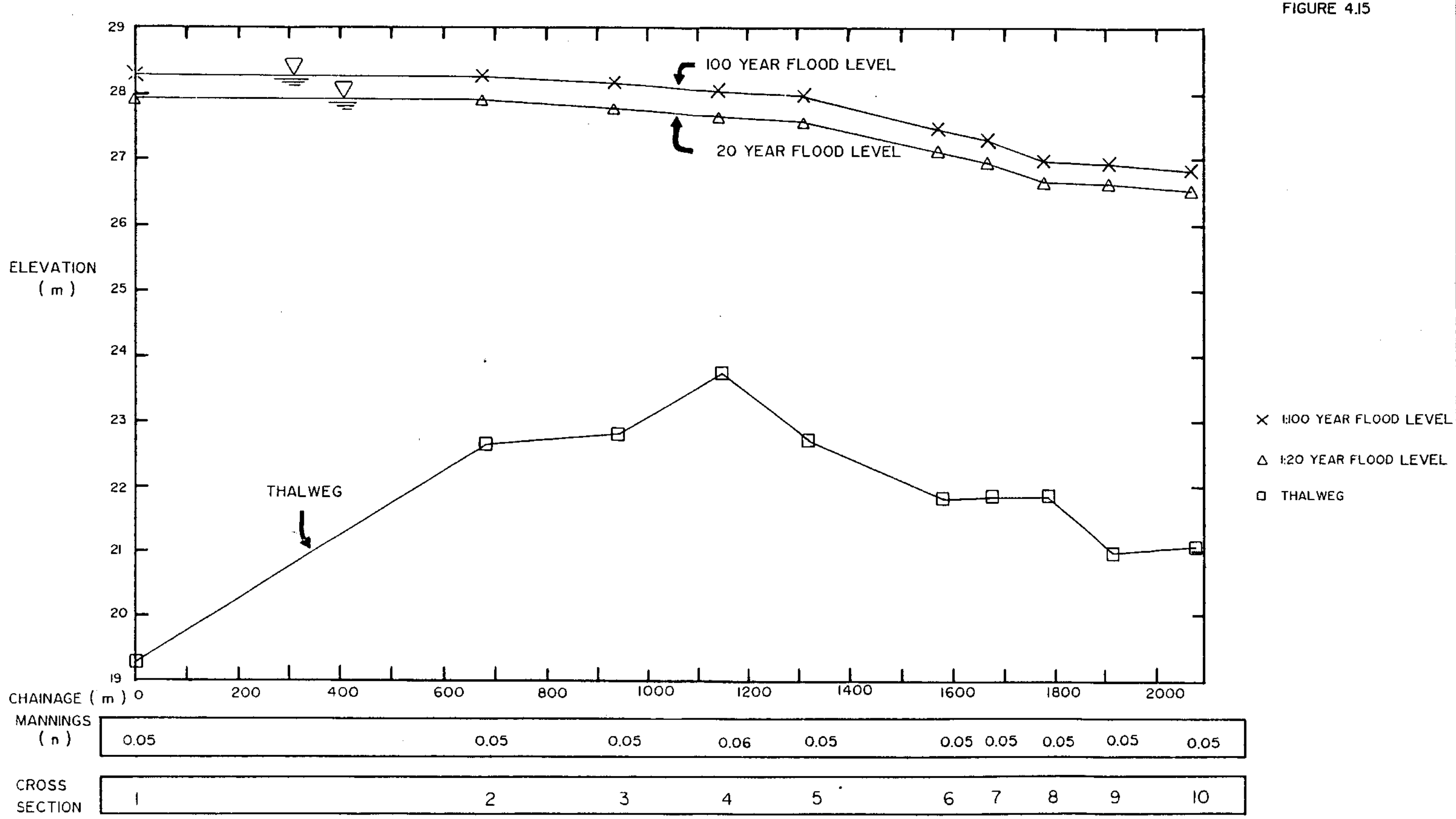
GLENWOOD / APPLETON FLOOD STUDY
MODEL CALIBRATION, Q=109 m^3/s

FIGURE 4.14



GLENWOOD / APPLETON FLOOD STUDY
MODEL VERIFICATION, $Q=1190 \, m^3/s$
(JANUARY 1983 FLOOD)

FIGURE 4.15



GLENWOOD / APPLETON FLOOD STUDY
1:20 YEAR AND 1:100 YEAR FLOOD PROFILES

PART FIVE

RECOMMENDATIONS

5. RECOMMENDATIONS

The following recommendations are suggested to minimize the damage from future flooding in the study area.

(i) The communities of Appleton and Glenwood should implement zoning regulations to control development in flood prone areas as delineated on the flood risk maps produced in this study.

(ii) Water supply pumping and treatment plants should be flood proofed to safeguard their operation during floods. It is suggested that the design of flood proofing measures be based upon flood levels for the 1 in 100 year flood.

(iii) Sewage pump stations and treatment plants should be flood proofed to protect equipment vulnerable to water damage. It is likewise suggested that the design of flood proofing measures for the sewage systems be based upon the 1 in 100 year flood water level.

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