Environment Canada

AN EVALUATION OF FLOODING IN STEPHENVILLE, NEWFOUNDLAND

APPENDICES

WATER PLANNING AND MANAGEMENT BRANCH,
INLAND WATERS DIRECTORATE,
ATLANTIC REGION,
HALIFAX, NOVA SCOTIA,
DEC. 1975.

FORWARD

During the preparation of the report entitled An Evaluation of Flooding in Stephenville, Newfoundland, December 1975, it was decided that additional documentation would be desirable. Since much of this documentation was of a technical nature, and not directly pertinent to the conclusions and recommendations embodied in that report, it was further decided to prepare Appendices to which those interested in further detail concerning the methodologies employed could refer. This volume presents these Appendices. Use of the technical Appendices should only be made with reference to the main text where more explanation of the rationale underlying the methodologies is provided.

AN EVALUATION OF FLOODING

IN

STEPHENVILLE, NEWFOUNDLAND

APPENDIX A

RESIDENT SURVEY

Water Planning and Management Branch
Inland Waters Directorate
Atlantic Region
Environment Canada
Halifax, Nova Scotia

December, 1975

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1. INTRODUCTION

As a part of the process of acquiring baseline information about the flooding problem in the Stephenville area, a survey of residential attitudes, perceptions, and other socio-economic characteristics was undertaken. The questionnaire was designed to provide information about those floodplain managers who actually live with the hazard on a day-to-day basis. In some flood hazard areas, it is appropriate to investigate attitudinal differences between those living off the floodplain and those living on it. However, in the Stephenville case, there did not appear to be sufficient reason to do this, so only those people living in or very close to the vulnerable zone were interviewed.

The survey was administered over a three-day period in October, 1974 by a single interviewer. As far as possible, sample selection was made on a random basis. The approximate hazard area was outlined on a map of the Town and houses were then chosen within this area. Twenty-three interviews were carried out in both development areas 8 and 10, as well as at Wheeler's where a flooding problem also exists. Figure A-1 illustrates the approximate location of the sampled homes. The randomness of the sample was undermined by the fact that sometimes people were not home when the interviewer called. Also, in several cases people were new residents of the area. In many of these cases, people would not answer our questions because they felt they had no concept of the flooding problem in the area. It proved to be very difficult to explain that they were part of a random survey and as such their answers would indeed be of value to us. However, since

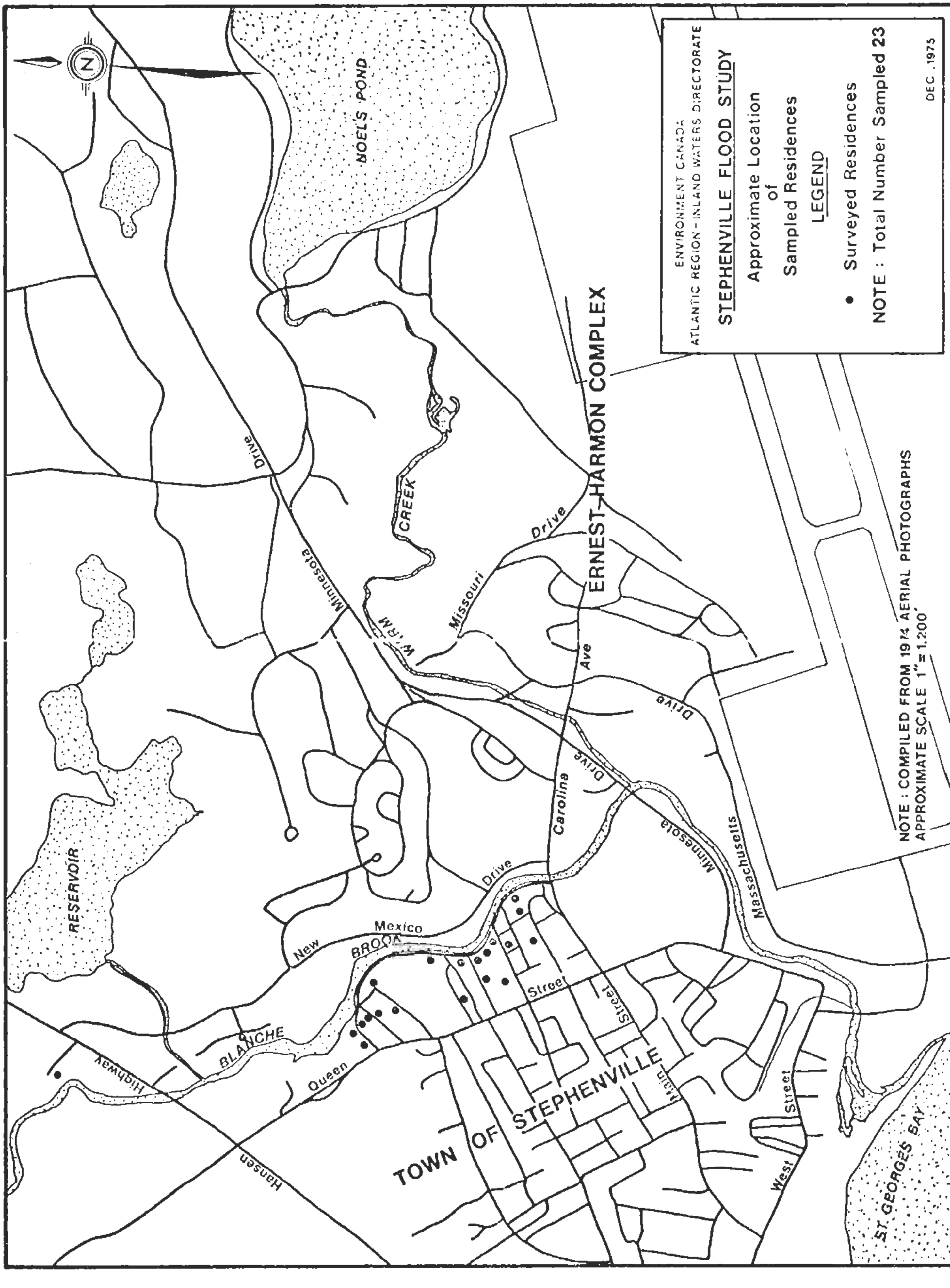
ENVIRONMENT CANADA
ATLANTIC REGION - INLAND WATERS DIRECTORATE
STEPHENVILLE FLOOD STUDY
Approximate Location
of
Sampled Residences
LEGEND
• Surveyed Residences
NOTE : Total Number Sampled 23
DEC. 1975
NOTE : COMPILED FROM 1974 AERIAL PHOTOGRAPHS
APPROXIMATE SCALE 1"=1,200'
N
NOEL'S POND
ERNEST HARMON COMPLEX
TOWN OF STEPHENVILLE
RESERVOIR
ST. GEORGE'S BAY
WARM CREEK
BLANCHE BROOK
Minnesota Drive
Missouri Drive
Ave
Carolina Drive
Drive
Massachusetts
New Mexico Drive
Street
Street
Main
Queen
Hansen Highway
West Street

FIGURE A-1

many of these potential respondents could not be convinced, our sample was accordingly biased towards those who have lived in the area long enough to have formed some concept of both the nature and extent of the hazard. In fact, three-quarters of those eventually interviewed had lived on the floodplain more than five years. Although a majority of the floodplain occupants probably have lived in the area more than five years, it must be recognized that a bias does exist in the results. The interviews were carried out informally, with questions being asked (and often answered) indirectly at times. Since no perceptible differences occurred between the answers received in the different hazard areas sampled, it was felt that it would be appropriate to collate the information in only one summary rather than doing a separate summary for each hazard area. This was in fact done, and the resultant summary is included with this Appendix (See Annex 1).

2. Interpretation of Survey Results:

Part of the survey included questions and observations concerning the property itself. Most of the residences are one storey single family dwellings. Most have half basements in which the furnace is normally located. The houses are, for the most part, over ten years old with about 62 percent being more than 15 years of age. The foundations are either concrete (73%) or wood (27%) and all are in good to fair condition. Virtually all respondents are on town sewer and water mains, with only about 25 percent having any sewer or water outlets located below the first floor. Only 13 percent have finished basements. Hot air heating predominates throughout the study area.

From this brief descriptive summary, it is possible to draw a few generalizations: 1) Basement flooding is not likely to cause very severe problems in the area in terms of dollar damages. The furnace was usually the only item of value located in the basement, although in a minority of cases a washer and dryer were also present. Since only 13 percent of the houses have finished basements, the potential for damage is not particularly great: 2) Sewer and water service disruptions on the floodplain are minimal since 95 percent of the respondents indicated they had never had any flood related problems with these facilities: 3) Given the generally good conditions of most of the foundations in the hazard area, it is not anticipated that structural damages would be very severe. Floodwaters might be expected to cause damage to building materials and utilities, but they would not cause pressure breaks or cave-ins of structures except under the severest of conditions.

A few questions were asked regarding the socio-economic characteristics of floodplain residents. As it turned out, most of the respondents were women whose ages varied so much that no bias was obvious. The length of residence on the floodplain is quite long, since 3/4 of the respondents had been there more than fifteen years. This inference is substantiated by a review of the history of the Town, from which it may be seen that the flood hazard area on Blanche Brook is the oldest established residential area in the vicinity. Further evidence is found in the fact that a majority of people in the floodplain own clear title to their land while about another 22 percent are currently paying on a mortgage.

The final observation that can be made concerns the nature of employment of breadwinners on the floodplain. Interpretation of the results of question 4 would lead to the conclusion that residents of the study area are primarily engaged in low to middle income employment.

Questions 6 to 15 were designed to reveal various aspects of residential attitudes and perceptions. Several interesting features became apparent from the answers to these questions:

(a) Flooding is not perceived to be a large enough problem to cause most people to seriously think of moving from their present dwellings, although about 13 percent indicated they had given substantial thought to the possibility;

(b) In spite of this apparent immobility on the part of the residents, a large majority did <u>not</u> know about the hazard before moving to the area. Of those who did know only a few were very concerned about the flood problem. This leads to the probable conclusion that flooding is simply not, at present, a very large problem in the eyes of those who live on the floodplain;

(c) The frequency of flooding was seen to be quite high among the survey sample. Flooding once in every five years was forecast by about 60 percent of those surveyed. Apparently, in the residential areas frequent minor flooding is the dominant trend;

(d) Clogged river channels, combined with heavy rainfall were the most often cited causes of flooding in the area. However, rapid snow melt and ice jams were mentioned quite frequently as well. Structures, especially bridges, were seen as traps for debris in summer floods and for ice in winter floods;

(e) A recurrent observation in surveys of this type is that people feel that solving the problem is someone else's responsibility. Such appears to be the case in Stephenville as well. The large majority of people felt that the Town of Stephenville should bear the responsibility for their personal infringement of the floodplain;

(f) An interesting feature of the observation in(5) above is that it is the Town authorities rather than the provincial or federal levels of government that people felt should be handling local adjustments to flooding. Since it is the municipal government which has historically been involved in the flood reaction in this area, support is found for the theory that where particular institutions have become involved in flood reaction over a number of years, these institutions tend to be expected to perform these informal functions in the future;

(g) When pressed for a specific answer, concerning which adjustments to the hazard should be made, a majority indicated they had not thought about it and thus had no opinion. Of the remainder of those surveyed, most felt that flooding would not happen again, since the Town had dyked the stream banks. Adjustments suggested by respondents were structurally, rather than non-structurally, oriented, possibly because structural adjustments had historically been made by the Town.

Information concerning historical flooding was sought in questions 16-27. From the answers to these questions for following inferences may be drawn:

(h) A majority of respondents did experience some flooding in the August, 1973 flood, although most suffered no damage whatsoever.

(i) By far the most significant economic costs inflicted were in terms of avoidance and cleanup. There was also damage in some cases to furnishings and other contents, although this was not very severe in terms of dollars. Structural damage was minimal.

(j) Virtually no post-flood effects were experienced in the floodplain except for some minor structural damages.

(k) Although about 64 percent of the properties were flooded in the summer of 1973, only 45 percent suffered any damage at all. With one exception this damage was minimal. Flooding occurred on only the outside property about 30 percent of the time, while approximately 30 percent of the homes incurred no flooding whatever. In the remaining 40 percent of the homes, water levels ranged from 4 inches to 3 feet above the basement floor. In no cases sampled did the water reach as high as the first floor. Little damage was incurred since over three quarters of the houses have half-basements or crawl spaces rather than full basements.

(l) Advance warning time prior to inundation appears to have been at most one hour and in many cases no warning was received at all. Almost everyone whose property was affected replied that no one had informed them that there would be a flood. Indications are that the bulk of the floodwaters in the 1973 flood had subsided within a day and by the time two days had passed virtually all the excess water had run off. This fact, in combination with the small amount of lead warning time, attests to the "flashy" nature of the watershed.

(m) Reaction to the 1973 flood consisted primarily of moving items to higher places, a bit of trench digging, and pumping of water from basements. Residents appear to be quite complacent about the hazard, possibly because severe damages are rarely, if ever, inflicted on any large scale. There is no discernible tendency on the part of floodplain occupants to consider any substantial adjustments to the hazard. Over the years, minor adjustments have been made (e.g. sump pumps, "slung" furnaces, raising appliances, etc.) but these have not been very major or very costly.

3. CONCLUSIONS

The most obvious impressions obtained from the survey concerned the extent of the problem. It appears that in residential areas, damages are perceived to be insignificant, except in a very few cases. The people are not, for the most part, very concerned about the hazard and there is a general feeling that since the dyke has been built, the bulk of the problem has been solved.

Those exposed to the hazard either do not think about it at all or they are generally quite complacent about its potential for endangering their property.

ANNEX 1

SUMMARY OF RESPONSES

RESIDENT SURVEY

Location: ______________

The following questions are designed to provide basic information about the flooding problem in Stephenville on Cold Brook. Please indicate the one answer in each question which best approximates the truth. Answers to all questions will be held in strictest confidence and it is vital that all questions be answered by the same person. Please fill the form out carefully as the validity of the study's conclusions depends on your efforts.

1. Sex: Male 2 (9%) Female 19 (83%) Both 2 (9%)

2. How long have you lived here?
   - less than one year 1 (4%)
   - 1 yr. to 2 yrs. 2 (9%)
   - 2 yrs. to 5 yrs. 3 (13%)
   - 5 to 10 years 5 (22%)
   - more than 10 yrs. 12 (52%)

3. How old are you?
   - Under 35 9 (39%)
   - 35-50 8 (35%)
   - over 50 6 (26%)

4. Occupations of breadwinner and respondent, if applicable:

	Respondent	Breadwinner
professional		2 (14%)
proprietor, sales, manager		5 (33%)
clerical		1 (7%)
labourer	1 (4.5%)	7 (47%)
fisherman, woodsman		
housewife	20 (87%)	
other (specify)		

	Respondent	Breadwinner
No Answer	1 (4.5%)	
Unemployed	1 (4.5%)	
Some are retired, disabled, dead.		8 (35%)

5. Do you: own your house with no mortgage 13 (57%)

   own your house with a mortgage 5 (22%)

   rent your house 5 (22%)

   other (specify) ______

6. Has the threat of flooding caused you to think at all about moving from this house?

   a lot 3 (13%)

   some 1 (4.5%)

   hardly at all 1. (4.5%)

   not at all 18 (78%)

   other (specify) ______

7. How often do you think flooding is apt to occur in Stephenville?

   once a year 8 (35%)

   once every 3 years 3 (13%)

   once every 5 years 3 (13%)

   once every 10 years 1 (4.5%)

   less often 2 (9%)

   unpredictable 3 (13%)

   don't know 2 (9%)

   never 1 (4.5%)

   other (specify) ______

8. Did you know about the flood hazard here when you moved in?

   Yes 6 (26%) No 17 (74%) N/A ______

9. When you were deciding to move here did this knowledge bother you or influence your decision?

a lot	2 (9%) →	33%
some		
a little	1 (4.5%) →	16%
none	3 (13%) →	50%
do not know		
N/A	17 (74%)	
other (specify)		

10. Knowing what you do now about the flooding hazard, would you move here again if you had it to do over?

Yes	10 (43%)
No	10 (43%)
Do not know	2 (9%)
N/A	
other (specify)	1 (4.5%) No. Ans.

11. What do you feel has been the major contributing factor to the flooding problem in Stephenville down through history? If you feel more than one of the answers below apply, please rank them as 1st in importance, 2nd. in importance, and so on.

RESPONSES		RESPONSE RANKED (1)	(2)	(3)
1	No Answer			
9 (22%)	clogged river channels	4 (10%)	4 (10%)	1 (2%)
1 (2%)	cutting of trees in river basin		1 (2%)	
	more storm drains being put in			
8 (20%)	heavy rainfall	5 (12%)	3 (7%)	
5 (12)	rapid snow melt	4 (10%)	1 (2%)	
3 (7%)	ice jams	1 (2%)	2 (5%)	
4 (10%)	structures in the river course (dams, bridges, etc.)		4 (10%)	

RESPONSES			RESPONSES RANKED (1)	RESPONSES RANKED (2)
4 (10%)	11. (cont.)	Don't know	4 (10%)	
7 (17%)		other (specify)	6 (15%)	1 (10%)

1. River course changes
2. Brook too low, won't flood again
3. Made worse by work done on brook
4. Roads not taken care of -rocks back in brook.
5. D.O.H. - 3 bridges plugged, unplugged on furtherest from mouth in 1973.

12. Who is most responsible for protecting people from floods? (Please rank your answers)

	RANK 1	RANK 2
it is the individual's responsibility		
town authorities	19 (86%)	
provincial government		1 (4.5%)
federal government		
other (specify)		
No answer 3		
don't know	2 (9%)	

13. Who should be held responsible for damages caused by floods?

	RANK 1	RANK 2
individual owners of damaged property	3 (14%)	
town	16 (73%)	1 (4%)
province	1 (4%)	
federal	1 (4%)	
combination of federal, provincial		
other (specify)		
No Answer 3		

(No answer for questions 14-27 from one resident)

14. What do you feel is the best way to deal with the flooding problem here in Stephenville? Comments?

Don't know/No answer	14 (61%)
Deepen Stream	-- 2 (9%)
Won't Flood Again/Town Did good job	3 (13%)
Manholes Upstream for Runoff	1 (4%)
Buildup Streets with Trenches on Each Side	1 (4%)
Fix Bridges to Prevent Clogging	1 (4%)
Finish Banking Upstream, Lower Culverts	1 (4%)

(A number of people indicated that flooding was an act of God, over which they had no control.)

Interviewer: Categorize answers into one or more ranks as below. Try to get a feel for the respondent's comprehension of the concepts which were not mentioned.

Possibilities:	Little or No Concent	Unnessary	Rank 1	Rank 2	Rank 3
compulsory flood insurance	14 (70%)	1 (5%)	1 (5%)	2 (10%)	2 (
voluntary flood insurance	14 (77%)			1 (5%)	3 (
flood proofing	14 (67%)	3 (14%)	3 (14%)	1 (5%)	
loss bearing	5 (38%)	6 (46%)		1 (8%)	1 (
land use regulation	18 (81%)	3 (14%)			1 (5
watershed treatment	15 (83%)	2 (11%)		1 (6%)	
channelization	4 (19%)	5 (24%)	9 (43%)	2 (10%)	1 (
dyking/levees/dams, etc.	5 (23%)	6 (27%)	5 (23%)	4 (18%)	2 (9
emergency forecasting	7 (50%)	6 (43%)			1

(Percentages represent attitudes toward each possibility, i.e. Read %'s across)

15. Would you pay for flood insurance on your property?

Yes 10 (45%) No 7 (32%) Maybe 5 (23%)

If so, how much per year? Less than $100.00 or Don't know

SUMMER 1973 FLOOD

16. Was your property flooded in 1973?

Yes 14 (64%) No 8 (36%)

12 suffered no damage at all (55%)

17. Please indicate the nature of the damage that you suffered in the 1973 flood and your estimate of the dollar value.

tal Dam-ages incurred s result of 73 flood)

Types	Describe	Value
Structural	1 - tile, gyproc wet and moldy 1 - concrete driveway cracked and fell	
Furnishings	1 - furnace switch, water heater coating tools, mechanical equipment, electric saw 1 - boxes of books, trunks of clothing, ringer washer	
Avoidance	6 - raise articles off basement flood - dig trenches	90 man hrs. total
Cleanup	considerable clean-up otherwise, nothing	50 man hours

Interviewer: Make quantitative judgements of values where possible. Attention to detail and consistency should be given.

18. Have you noticed any damage to your property since the flood that could be attributed to the Flood? If so, specify the nature and estimated cost of this damage.

None: 13 (59%)

Yes: 1 (5%)-walls cracked and warped

1 (5%)-house was settled in middle, nothing serious

7 (32%) N. Applicable

19. How high did the water reach inside the house (to the nearest inch) during the 1973 flood?

56% 9 (39%) (4" to 3') inches above basement floor or ground - 4", 4", 8" 1', 2', 2', 2½', 3'

44% ______ inches above ground floor

7 (30%) only on outside property

7 (30%) N. Applicable

20. How long before the flood occurred did you know there would be a flood?

3 (19%) (approximately 1 hour)

13 (81%) (no time or don't know)

7 (no answer)

21. Who told you there would be a flood? Experienced; watched water rise.

No one 13 (87%)

Other (specify) 1 (7%) neighbour, 1 (7%) radio (afterfact)

Not Applicable 7

No answer 1

22. How long was it before the flood waters subsided?

less than 1 day 8 (57%)

1-2 days 5 (36%)

1-2 weeks 1 (7%)

Don't know 1

Not applicable 7

No answer 1

23. Describe in detail your reaction to the 1973 flood. What did you do to avoid damage? What would you do if another similar flood occurred now? How did the water enter the house? How long did it stay? Generally describe how it affected your household.

Avoidance - 2 trenched around house

2 lifted items off basement floor; stayed up all night mopping and pumping

23. (con't)

Generally, homes are not flooded; half-basements are prevalent with nothing but furnace (if that) in them; residents simply watch waters rise and take flooding in stride.

24. What steps have you taken regarding the flood hazard since the 1973 flood?

The general feeling is that nothing further can or should be done to avoid flooding of property. Any steps toward lessening damage have been taken before 1973.

OTHER YEARS' FLOOD DATA

The August 1973 flood was somewhat different from most floods that occur in Stephenville, since flooding usually occurs in the late winter or early spring months rather than in the summer.

25. How does a winter flood affect your household? Indicate any particular problems winter is likely to bring:

    Similar to summer flood 7 (32%)

    other (specify) 5 (23%) - summer floods were worse, especially 1969

    4 (18%) - resident for less that 2 years; no floods experienced.

    5 (23%) - never any flooding of their property

    1 (5%) - 1973 flood was only one in 50 years to hit their property.

    1 No answer.

26. Can you remember dates and time of year for any previous floods on Blanche Brook?

    1972 - 1 (10%)

    1969 - 6 (60%)

    1964 - 2 (20%)

    1961 - 1 (10%)

    No - 9

    Resident Less than 2 years - 4

    Yearly seepage - 3

27. Over the years have you made any adjustments to the floodiing problem on your property?

No 15 (65%)

Yes 5 (35%)

What type 2 (25%) raised house; 2 (25%) raised washer-dryer; 1 (12%) installed sump pump; 1 (12%) no longer store anything in half-basement; 1 (12%) installed "slung" furnace; 1 (12%) more fill on ground.

28. General features of property:

- single family dwelling 20 (87%) duplex 3 (13%) mobile home _____ other _____

- number of floors (exclude basement) 1 - 15 (65%)

  1½ - 3 (13%

  2 - 5 (22%)

- is there a basement? full 6 (26%)

  half 17 (74%)

- is the basement finished? Yes 5 (22%) No 18 (78%) Furnished? Yes 3 (13%) No 2 (9%)

  Not App. 18 78%

- age of house 10-15 years - 8 (35%) 38%

  15-20 years 10 (43%) 48%

  more than 20 years 3 (13%) 14%

  Don't know 2 (9%)

- type of heating system: electric 2 (9%) hot air 14 (64%) hot water 2 (9%) other 4 (

  No answer - 1

- sewage facilities: city main 21 (91%) septic tank 1 (4%) other 1 (4%) (outhouse

- do you have water or sewage outlets in the basement? Yes 5 23% No 17 77% No Ans

- has your sewage water system ever been affected by flooding?

  Yes 1 (5%) No 21 (95%) How? ___ When? ___ No answer 1

- is the foundation concrete blocks 10 (43%) form concrete 7 (30%) wood 6 (27%) other

- in what condition is the foundation? poor ___ fair 5 (22%) good 18 (78%)

- is the furnace located below ground level? Yes 18 (78%) No 1 (4%) no furnace 4 (17%)

AN EVALUATION OF FLOODING

IN

STEPHENVILLE, NEWFOUNDLAND

APPENDIX B

FLOOD MAGNITUDES

Water Planning and Management Branch
Inland Waters Directorate
Atlantic Region
Environment Canada
Halifax, Nova Scotia

December, 1975

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LIST OF FIGURES

LIST OF TABLES

1. GENERAL

This report describes the hydrology studies carried out to develop flood flows for use in computing the water surface profiles on Blanche Brook and Warm Creek under flood conditions of varying magnitudes.

2. DESCRIPTION OF THE AREA

Blanche Brook watershed is located in the southerly portion of the west coast of Newfoundland. Its location and topography are shown on Figure B-1. The total watershed area is 51.2 square miles. The brook rises in the Indian Head Range at an elevation of 1,550 feet and flows southwesterly over a distance of 15.3 miles to its outlet into St. George's Bay. Warm Creek, the largest tributary of Blanche Brook, drains the eastern half of the drainage area and enters Blanche Brook within the Town of Stephenville, approximately one mile above the river's mouth. Warm Creek also flows to the southwest from its source in the Indian Head Range and drops a total of 1,350 feet over a length of 12 miles to its confluence with Blanche Brook.

Physiographic parameters for the basin as a whole and for the two major branches above their confluence are given in Table B-1.

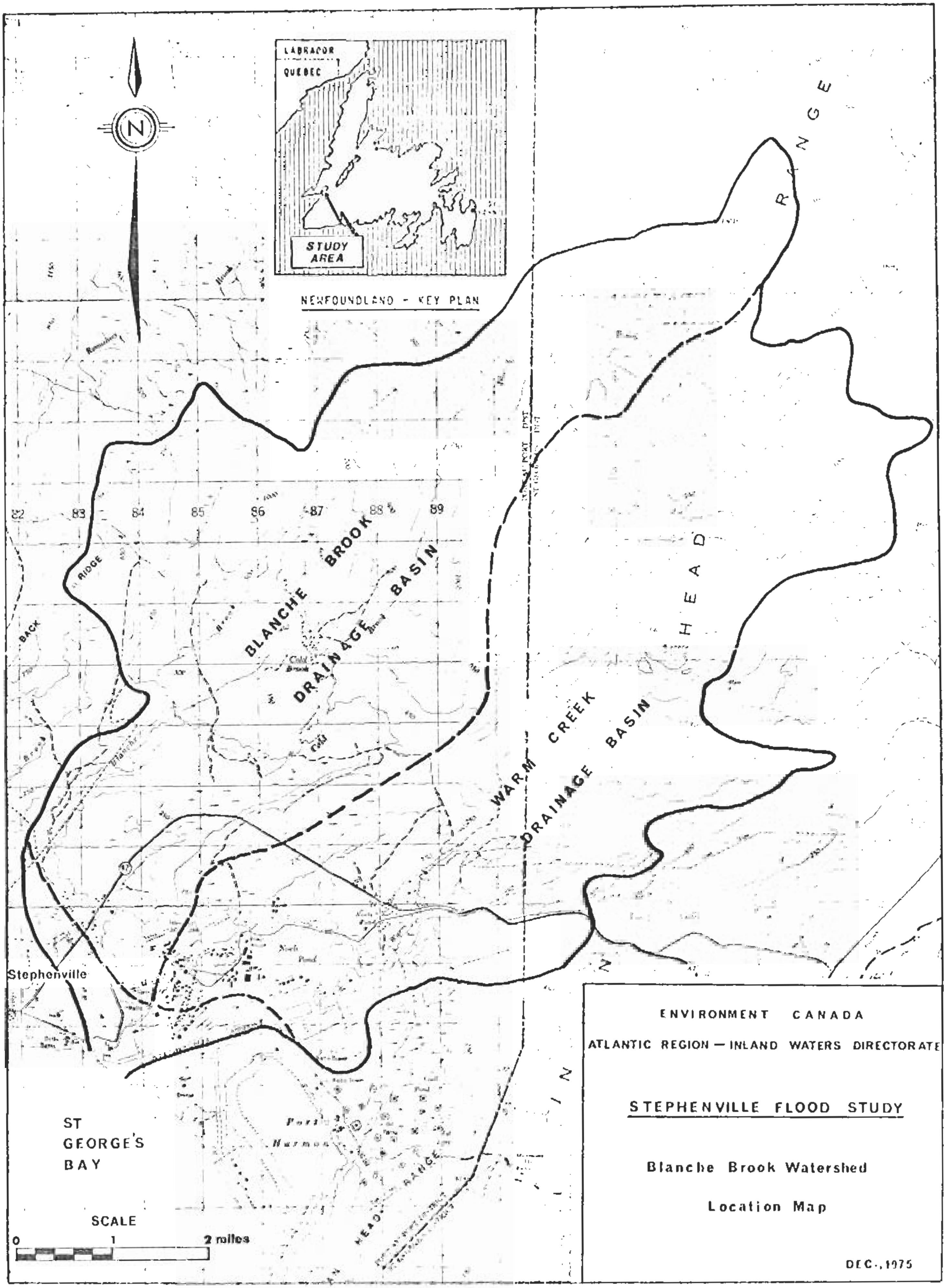
LABRADOR
QUEBEC
STUDY AREA
NEWFOUNDLAND - KEY PLAN
N
BLANCHE BROOK DRAINAGE BASIN
WARM CREEK DRAINAGE BASIN
INDIAN HEAD RANGE
BACK RIDGE
82
83
84
85
86
87
88
89
Stephenville
ST GEORGE'S BAY
SCALE
0
1
2 miles
ENVIRONMENT CANADA
ATLANTIC REGION — INLAND WATERS DIRECTORATE
STEPHENVILLE FLOOD STUDY
Blanche Brook Watershed
Location Map
DEC., 1975

FIGURE B-1

TABLE B-1

Selected Physiographic Properties

Blanche Brook Watershed

	Total Basin	Blanche Brook*	Warm Creek*
Drainage Area	51.2 sq. mi.	24.3 sq. mi.	25.2 sq. mi.
Length of Channel	15.3 miles	14.3 miles	12 miles
Difference in elevation, source to mouth	1,550 feet	1,530 feet	1,350 feet
Slope	101 ft./mile	107 ft./mile	112 ft./mile
Percent of area controlled by lakes and swamps	57%	21.7%	95%

* Parameters for basin upstream of confluence of Blanche Brook and Warm Creek.

Most of the basin is forested with extensive swampy areas in the upper reaches. The bedrock geology of the area consists of granitic and basic rocks of Precambrian age. In the highlands there is little or no cover of unconsolidated earth materials. The lowlands are covered with glacial drift and outwash deposits. As a result, the streams have developed a meandering pattern, are generally shallow and have coarse gravel and cobble beds.

3. CLIMATE

Meteorological records have been kept at the Stephenville airport since early 1942. It is considered that these data are not representative of the climatic conditions of the entire basin because of the station's proximity to the coast. However, the record at the station provides the best information available for the area.

The mean daily temperature for the period 1942-70 at Stephenville is 40.9°F. The coldest month is February with a mean daily temperature of 21.6°F. The mean daily maximum and mean daily minimum temperatures for the period are 47.3°F and 34.5°F respectively.

The mean annual precipitation at Stephenville for the period 1942-70 is 42.5 inches. The mean annual rainfall for the period is 31.4 inches and the mean snowfall is 128.6 inches.

4. FLOOD MAGNITUDES

Analysis of the flooding problem in Stephenville requires estimates of the magnitude of flood flows to be expected for various recurrence intervals. Since no streamflow records are available for Blanche Brook or Warm Creek, a review of the data from several hydrometric stations in western Newfoundland was carried out. The application of these data to a regional flood frequency analysis using a method developed by Poulin (1971)[1] was examined.

4.1 Hydrometric Data

Recorded hydrometric data in the Stephenville area is extremely sparse. The closest hydrometric station is on the Harry's River below the Highway 47 crossing. This gauge has only been in operation since 1968 and therefore has too short a record to be useful in flood frequency studies.

The locations of several other hydrometric stations in the western portion of Newfoundland are shown on Figure B-2. The periods

1. Poulin, R. Y. Flood Frequency Analysis for Newfoundland Streams, Water Planning and Operations Branch, Department of Environment, Ottawa, 1971

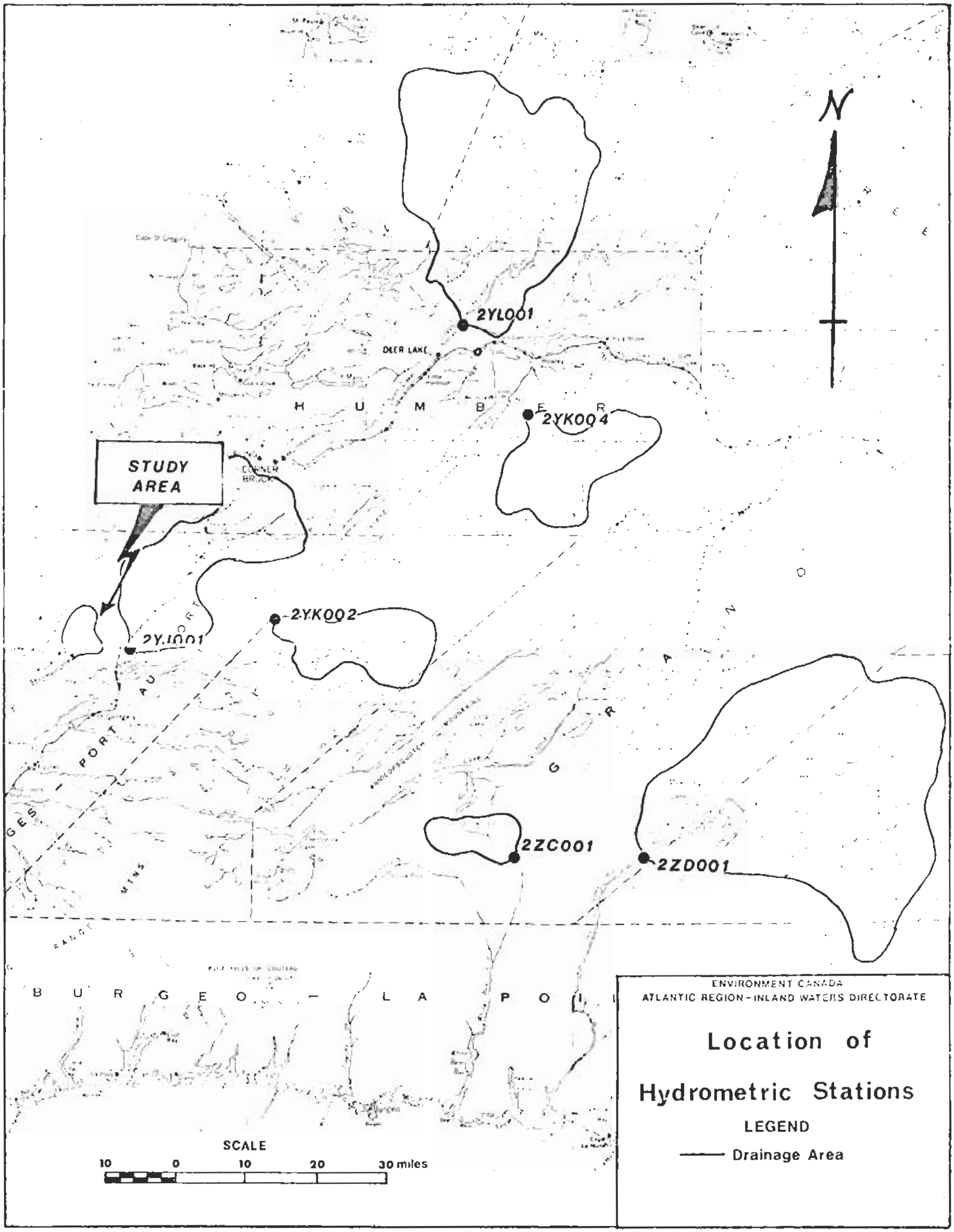
N
STUDY AREA
2YL001
DEER LAKE
H U M B E R
2YK004
CORNER BROOK
2YK002
2YJ001
PORT AU PORT
G R A N D
2ZC001
2ZD001
B U R G E O - L A P O I
SCALE
10 0 10 20 30 miles
ENVIRONMENT CANADA
ATLANTIC REGION - INLAND WATERS DIRECTORATE
Location of Hydrometric Stations
LEGEND
Drainage Area

FIGURE B-2

of record and drainage areas for these hydrometric stations are given in Table B-2. Daily mean flows and instantaneous peak flows for each day at these locations are published annually by the Inland Waters Directorate, Environment Canada, Ottawa.

TABLE B-2

HYDROMETRIC STATIONS

Station No.	Station	Drainage Area	Period of Record
02YJ001	Harry's River below Highway Bridge	245	1968-73
02YK002	Lewaseechjeech Brook at Little Grand Lake	180	1952-67
02YK004	Hinds Brook Near Grand Lake	200	1959-73
02YL001	Upper Humber River at Seal Pond	812	1950-73*
02ZC001	White Bear River at White Bear Lake	308	1964-69
02ZD001	Grey River near Pudops Lake	379	1958-67

* Record dates back to 1937, however, only considered reliable since 1950.

4.2 Regional Flood Frequency Analysis

The regional flood frequency relation presented by Poulin (1971) can be used to predict the magnitudes of floods of moderate return intervals on streams in Newfoundland. The regional flood frequency curve relating recurrence interval and the ratio of the flood magnitude to the mean annual flood is shown on Figure B-3. This curve was developed by analysis of streamflow records at 17 gauging stations in Newfoundland. In order to estimate

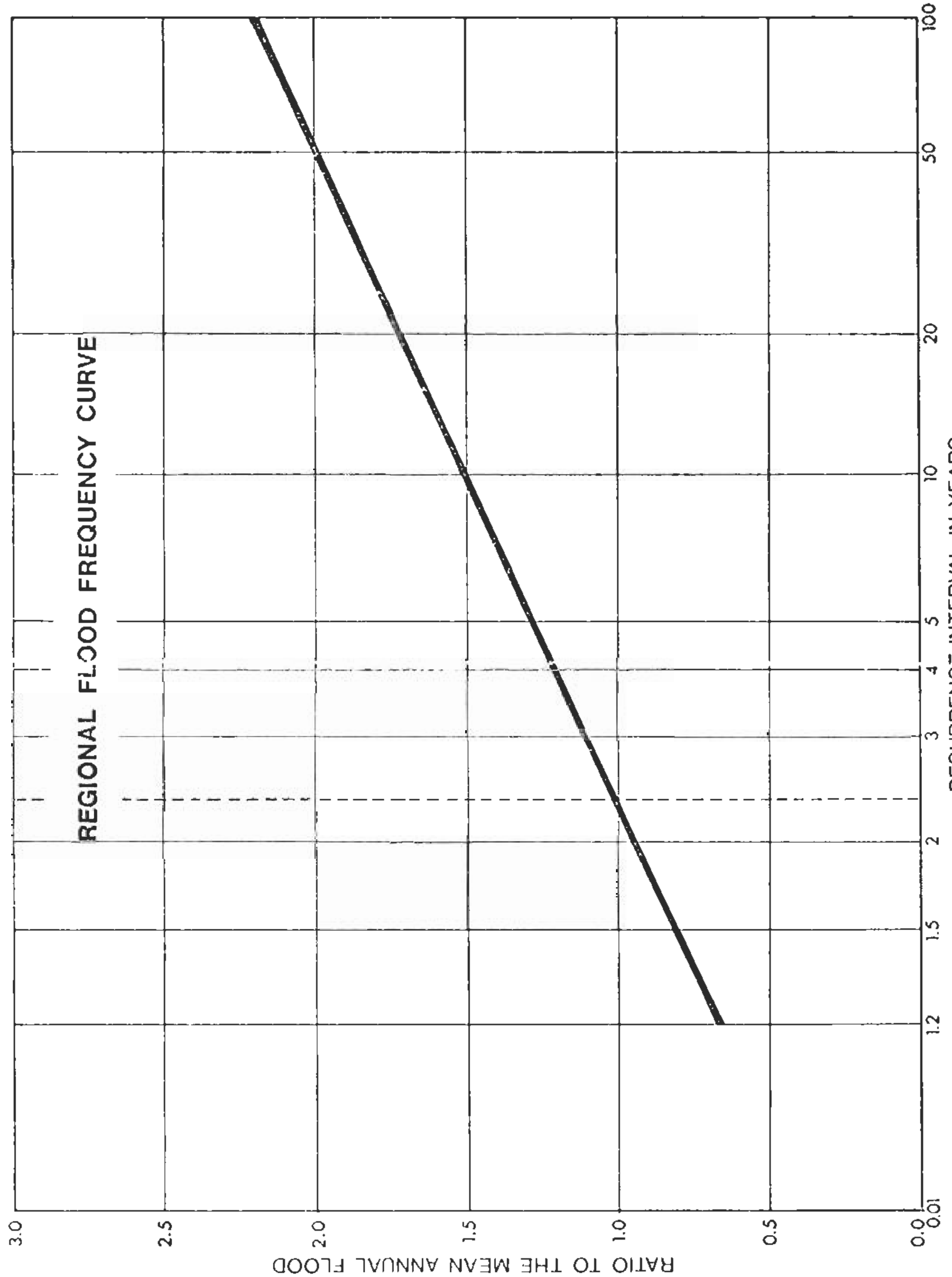
REGIONAL FLOOD FREQUENCY CURVE
RATIO TO THE MEAN ANNUAL FLOOD
0.0
0.5
1.0
1.5
2.0
2.5
3.0
RECURRENCE INTERVAL IN YEARS
1.01
1.2
1.5
2
3
4
5
10
20
50
100

FIGURE B-3

the magnitude and frequency of floods in a basin, it is necessary to compute the mean annual flood at the site. This is done by application of an equation developed by multiple stepwise linear regression which has the following form:

$$MAF = 5.563\ (X1)^{0.99252} \cdot (X2)^{1.07620} \cdot (X3)^{-1.08832} \cdot (X4)^{0.24023}$$

Where: MAF = mean annual flood
X1 = drainage area in square miles
X2 = mean annual runoff in inches
X3 = percentage of drainage area controlled by lakes and swamps
X4 = channel slope in ft./10^6 ft.

The following limitations are placed on the use of the equation to reflect the data used in the derivation of the formula:

- drainage areas between 75 and 1,700 square miles
- runoff between 25 and 85 inches
- percentage of drainage area controlled greater than 50%
- main channel slope between 1,000 and 13,000 ft./10^6 ft.

As can be seen from the basin characteristics (Table B-1) the drainage basin area and the channel slope of Blanche Brook fall outside the limits of the analysis. Caution must, therefore, be exercised in its application.

4.3 Determination of Flood Magnitudes

Application of the regional formula yields a mean annual flood of 1964 cfs. or 38.3 cfs./sq. mi. for Blanche Brook at its mouth. For comparative purposes, the mean annual flood for each of the hydrometric stations listed in Table B-2 has been computed from recorded data as well as from the regional formula. The results are given in Table B-3.

The percentage difference between the recorded and estimated mean annual floods indicates that the regional formula can be used with a fair degree of reliability in western Newfoundland. Based on the regional analysis the estimated mean annual flood discharge ranges from 8.3 cfs./sq. mi. for the Grey River to 32.4 cfs./sq. mi. for Harry's River. Compared to these values, the estimated mean annual flood unit discharge of 38.3 cfs./ sq. m. obtained for Blanche Brook seems realistic. The higher unit discharge for Blanche Brook is attributed to a smaller drainage area, a much steeper channel slope and a lower degree of control provided by lakes and swamps.

Table B-3

COMPARISON OF RECORDED AND ESTIMATED

MEAN ANNUAL FLOOD

Station	Drainage Area sq. mi.	Mean Annual Flood (mean daily discharge) Recorded (cfs.)	Estimated (cfs.)	% Diff.	Recorded cfs./sq.mi.	Estimated cfs./sq.mi.
Harry's River below Highway Bridge	245	7,500	7,938	+5.8	30.6	32.4
Lewaseechjeech Brook at Little Grand Lake	180	2,800	3,531	+ 26.1	15.5	19.6
Hinds Brook near Grand Lake	200	3,100	3,012	-2.8	15.5	15.1
Upper Humber River at Seal Pond	812	19,500	20,663	+6.0	24.0	25.4
White Bear River at White Bear Lake	308	6,332	4,916	-22.4	20.6	16.0
Grey River near Pudops Lake	379	3,175	3,144	-1.0	8.4	8.3
Blanche Brook at Mouth	51.2	-	1,964	-	-	38.3

It would also be possible to estimate flood magnitudes for Warm Creek and for Blanche Brook upstream of the confluence with Warm Creek, by applying the regional mean annual flood regression equation. However, since the drainage areas of both of these sub-basins are far below the lower limit of drainage area imposed on the use of the equation, it was decided to accept the mean annual unit flood discharge (38.3 cfs. per sq. mi.) as also being applicable to the two sub-basins. This yields mean annual flood discharges of 965 and 930 respectively for Warm Creek and Blanche Brook immediately above the confluence. These figures can be used in connection with Figure B-3 to derive the estimated flood discharges shown in Table B-4.

Table B-4

MEAN DAILY FLOOD DISCHARGE (cfs.)

Location	Recurrance Interval				
	MAF	10 yr.	20 yr.	50 yr.	100 yr.
Blanche Brook at Confluence	930	1,395	1,600	1,841	2,046
Warm Creek at Confluence	965	1,448	1,660	1,911	2,123

4.4 Adjustments to Instantaneous Peak Discharge

Since the regional analysis was developed on the basis of mean daily flows at each of the hydrometric stations, the estimates in Table B-4 also represent mean daily flows. For flood studies the maximum discharge or instantaneous discharge is required rather than the mean daily discharge.

One means of adjusting estimates of mean daily flood flow to the corresponding instantaneous flow is provided by Fuller's equation[2]:

$$\frac{Q_p}{Q} = (1 + 2A^{-0.3})$$

Where:

Q_p = instantaneous flood flow

Q = maximum mean daily flow

A = drainage area in square miles

A comparison of the factor Q_p/Q given by Fuller's equation with similar factors computed for several hydrometric stations in western Newfoundland where instantaneous peak discharges are recorded is shown in Table B-5. Fuller's equation appears to yield reasonable results for stations without significant natural or artificial storage, but consistently over estimates the ratio for highly controlled stations.

TABLE B-5

RATIO OF INSTANTANEOUS TO MEAN DAILY PEAK DISCHARGES

	Ratio Q_p/Q		
Station	Fuller Equation	Hydrometric Records	Degree of Control
02YL001	1.26	1.13	75%
02YK004	1.40	1.00	95%
02YK002	1.40	1.02	100%
02ZD001	1.33	1.03	99%

2. Creager and Justin. Hydro-Electric Handbook, 2nd Edition, John Wiley and Sons, Inc., page 61, 1950 and Handbook on the Principles of Hydrology, Canadian National Committee for the International Hydrological Decade, D. M. Gray (ed.), 1970, p. 8.20.

Because of the differences in runoff response of the two sub-basins, Blanche Brook and Warm Creek, separate adjustment factors were applied to each. Warm Creek is highly controlled by natural lakes and swamps and by Noel's Pond which is an artifically controlled water supply reservoir. The Blanche Brook branch on the other hand, is only 22 percent controlled. Thus, its runoff response is much quicker and peak flows are greater than for the Warm Creek branch.

The mean daily flows on Blanche Brook were increased by applying an adjustment factor of 1.768 estimated from Fuller's equation. For Warm Creek, the mean daily flows were adjusted arbitrarily by a factor of 1.2 which reflects the much greater control in the sub-basin. The resulting instantaneous peak discharges for various recurrence intervals for each sub-basin are given in Table B-6

The summation of these figures which represents the discharge of Blanche Brook below the confluence is also given. A frequency curve for Blanche Brook downstream of the confluence is shown on Figure B - 4.

Table B-6

INSTANTANEOUS PEAK DISCHARGE (cfs.)

Location	Recurrence Interval				
	MAF	10 yr.	20 yr.	50 yr.	100 [illegible]
Blanche Brook at Confluence	1,643	2,464	2,828	3,256	3,61[illegible]
Warm Creek at Confluence	1,157	1,736	1,992	2,294	2,54[illegible]
Blanche Brook Below Confluence	2,800	4,200	4,820	5,550	6,16[illegible]

FREQUENCY CURVE
MAXIMUM INSTANTANEOUS DISCHARGE BLANCHE BROOK AT MOUTH

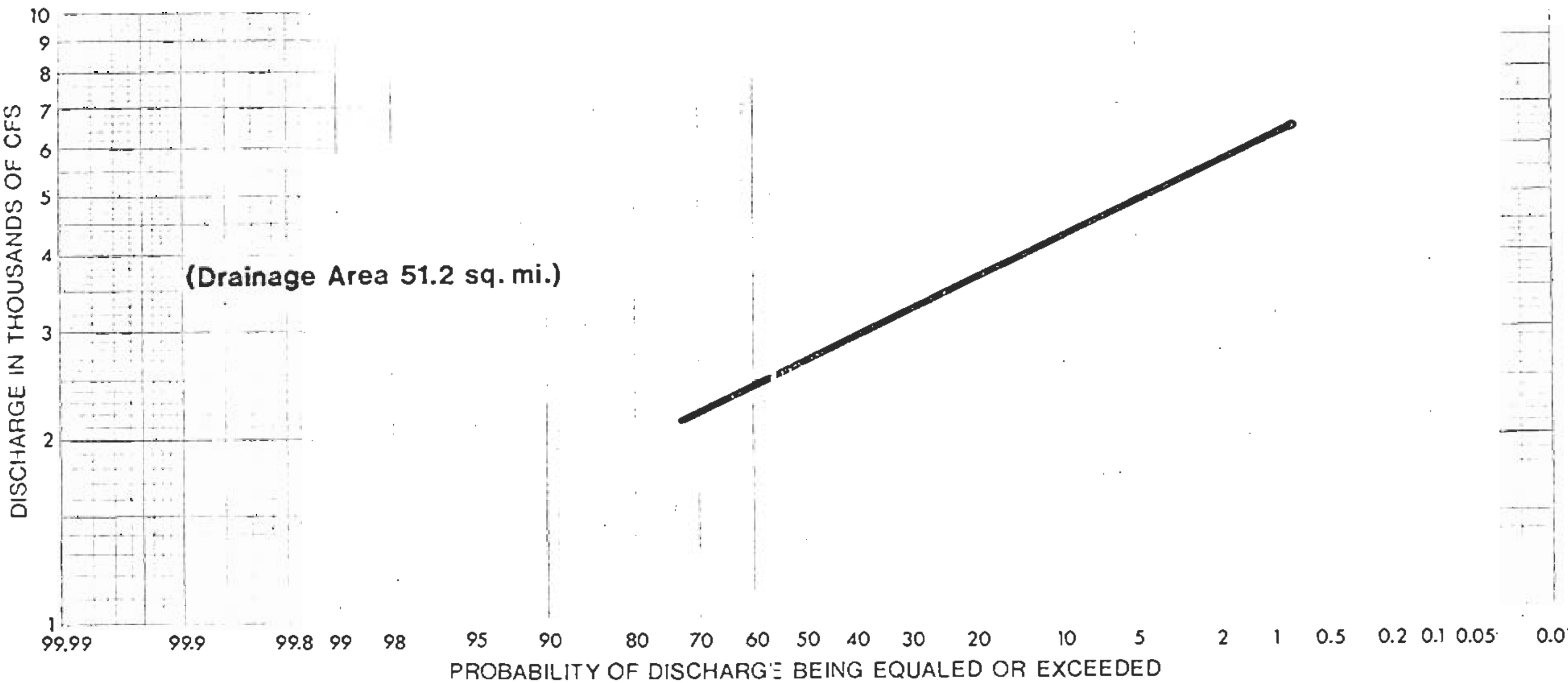

FIGURE B-4

5. CONCLUSIONS

Flood discharge values for various recurrence intervals have been esitmated for Blanche Brook and Warm Creek. The basis for development of the flows was a regional analysis for Newfoundland streams which was developed by Poulin (1971). The estimated flows were adjusted to give peak instantaneous values for use in estimating flood stages in the Town of Stephenville. The combined flow below the confluence of the two branches is as follows:

Recurrence Interval	Discharge (cfs.)
Mean Annual Flood	2,800
10 year	4,200
20 year	4,820
50 year	5,550
100 year	6,160 (174 m³/sec)

Of these totals, Blanche Brook contributes approximately 59 percent of the total peak discharge while Warm Creek contributes 41 percent.

AN EVALUATION OF FLOODING

IN

STEPHENVILLE, NEWFOUNDLAND

APPENDIX C

DEFINITION AND STRUCTURAL

EVALUATION OF THE

FLOOD HAZARD

TECHNICAL ASPECTS

Water Planning and Management Branch
Inland Waters Directorate
Atlantic Region
Environment Canada
Halifax, Nova Scotia

December, 1975

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BLANCHE BROOK

List of Symbols

Symbols	Definitions
A	Cross-section area of flow
α	Velocity (energy) coefficient
C	Coefficient of discharge
Δh	Change in water level between any two sections
Δx	Length of reach between any two sections
Dc	Critical depth of flow
D_t	Height of outlet works tunnel
Fr_1 & Fr_2	Froude number at section 1 & 2
g	The acceleration of gravity
h_e	Eddy losses in a reach
H_1 & H_2	Total energy head at sections 1 & 2
Ho	Difference in water level between upstream and downstream of orifice
h_f	Friction losses in a reach
Hu	Upstream distance from bottom of tunnel to water surface (head)
k	Coefficient of contraction or expansion
K_e	Entrance loss coefficient at tunnel
L	Length of weir
n	Mannings roughness coefficient
L_t	Length of tunnel
Q	Volumetric rate of flow
R_t	Hydraulic radius of outlet works tunnel
R_1 & R_2	Hydraulic radius of sections 1 and 2
S_f	Average friction slope of any two sections
S_{f_1} & S_{f_2}	Friction slope at sections 1 and 2
θ	Inclination angle of tunnel with the horizontal
V	Average velocity of flow through a section
Y_1 & Y_2	Distance from channel bottom to water surface at sections 1 & 2
Z_1 & Z_2	Distance from a datum to channel bottom at section 1 & 2

1. INTRODUCTION

The Water Planning and Management Branch through its Atlantic Regional Office undertook a study of the flooding problem in the Stephenville area in the fall of 1974. This appendix summarizes the hydraulic and conceptual design analyses in support of the study.

The objectives of the hydraulic studies are threefold: (1) to delineate the areal extent of the mean annual and the 1:100 year recurrence interval floods based on existing hydraulic conditions, (2) to establish criteria for various structural and non-structural alternatives aimed at minimizing future flood damages, and (3) to evaluate the effectiveness of each alternative or combination thereof.

The basic objectives of the conceptual design investigations were to compare costs and hydraulic effectiveness of the various structural alternatives in protecting land and property from the effects of flooding.

In this appendix a description is given of the following main topics: (1) Field surveys. This embodies the determination of cross sections at several hydraulically strategic positions along the watercourses involved, including sections near constrictions. (2) Backwater analyses. The purpose of these analyses is the determination of water surface elevations and profiles for the mean annual, 1:20 year and 1:100 year flooding events. (3) Flood hazard map. On this map are depicted the areas along the water courses inundated under the conditions of the mean annual and 1:100 year floods. (4) Dam and reservoir. This is one of the structural alternatives studied for the purpose of flood control to reduce water levels in the water courses during periods of potential flooding. (5) Dykes. This is one of the other structural alternatives to prevent flood waters from overflowing the banks of the water course onto plains which are considered to be of some economic and social importance. The dykes are analysed with respect to design, type, slope protection, materials and cost. (6) Channel re-alignment. It is proposed for this scheme, to divert the flow in Blanche Brook away from the river bank adjacent to the brewery, by excavating a new channel to carry the full flow. The cost and hydraulic effectiveness have been analysed. (7) Dredging. Two plans were investigated for lowering the water levels to such an extent to enable the water courses to pass a 1:100 year flood, with two feet of freeboard, from a point 500 feet above the bridge near the Recreational Center, up to a point a few hundred feet below the Carolina Avenue Bridge over Warm Creek. Both plans were examined with respect to hydraulic effectiveness and cost. (8) Diversion. Two schemes were studied, one for Blanche Brook, and the other for Warm Creek. The object was to divert the flow from the natural channels to new excavated channels so as to by-pass the flood plains of Blanche Brook and Warm Creek. Quantities and cost estimates are presented.

2. FIELD SURVEYS

2.1 General

The purpose of the field program was to obtain accurate data on the physical properties of the channel and adjacent flood plain of both Blanche Brook and Warm Creek in the Stephenville-Ernest Harmon area. This information would then provide a physical basis for hydraulic analyses. In addition, the data would also serve to verify and/or modify existing topographic mapping of the area. Field work was carried out during the period October 21-December 7, 1974.

The location of the field surveys is shown on Figure C-1. A total of 34 cross-sections were obtained at strategic hydraulic locations along both streams. Riverbed profiles of each stream were also obtained. In total, approximately 9.4 miles of level lines were run comprising 3.8 miles of river profile and 5.6 miles of cross-sections.

2.2 Field Procedures

A field reconnaissance was carried out to identify appropriate sites for cross-sections. These were identified on the following detailed topographic maps: Stephenville, Scale 1:2,400, Contour Interval - 5 feet, Ernest Harmon Complex, Scale 1:4,800, Contour Interval - 5 feet. Horizontal control was limited to identifiable features on the topographic maps. Sections are related to each other by chaining along the riverbank.

Seven brass plugs were placed, one on each of the six highway bridges and the other on the weir at the outlet of Noel Pond, for the purpose of vertical control. Temporary bench marks were established at each section. Three level nets, originating at a Geodetic Survey of Canada Monument located near the confluence of Blanche Brook and Warm Creek (Carolina Avenue Bridge), were run. Two of these were closed on the same monument while the other, W1 to W15, was closed on another GSC monument. In all cases, the level of accuracy was within ±.2 foot. In most instances, cross-sections were closed on a temporary bench mark, or on one of the seven brass plugs. Only on a few, closure was not made due to time constraints.

Each section was run perpendicular to the stream and in a straight line. This necessitates line cutting on occasion. The under-water portion

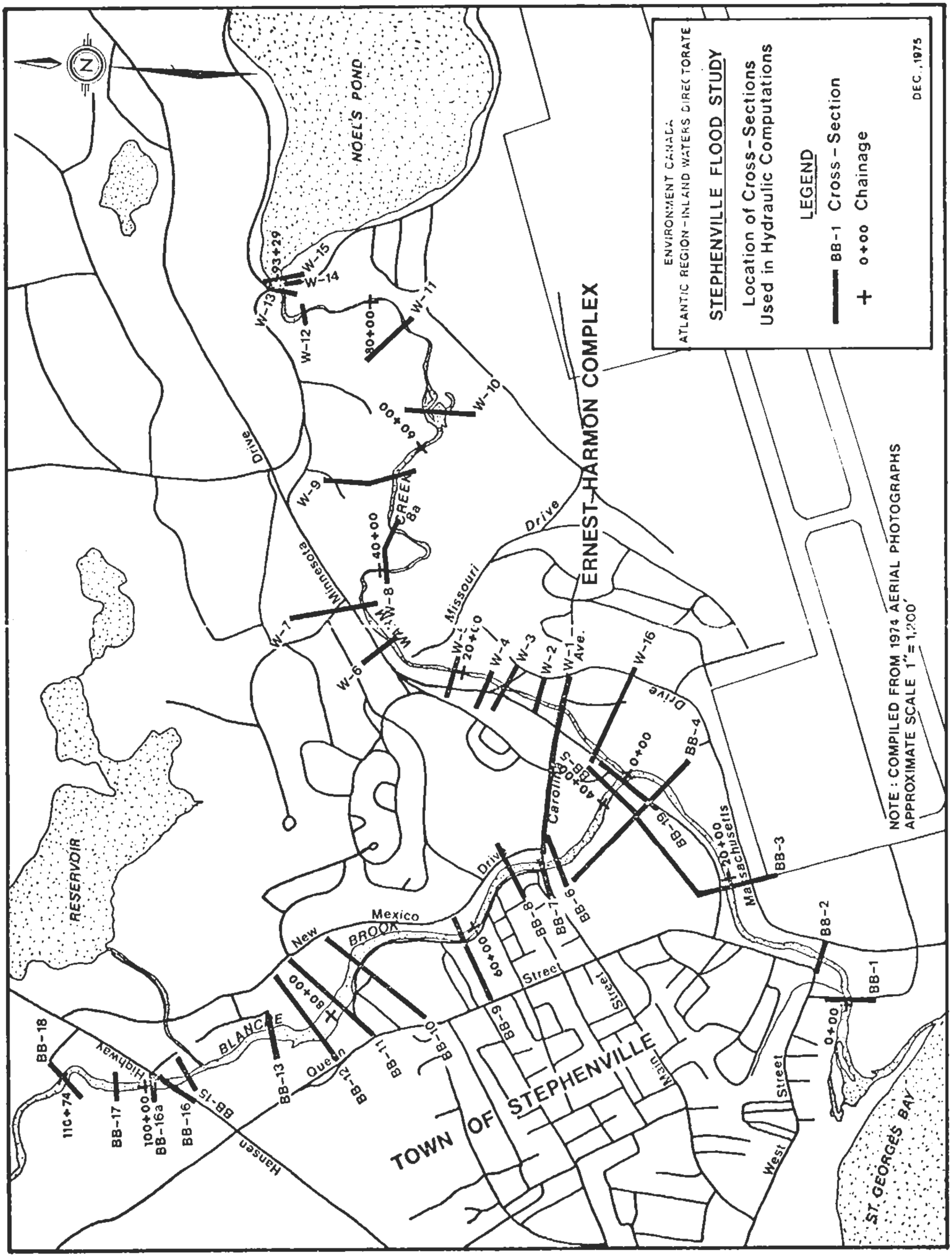
ENVIRONMENT CANADA
ATLANTIC REGION - INLAND WATERS DIRECTORATE
STEPHENVILLE FLOOD STUDY
Location of Cross-Sections
Used in Hydraulic Computations
LEGEND
BB-1 Cross-Section
0+00 Chainage
DEC. 1975
NOTE: COMPILED FROM 1974 AERIAL PHOTOGRAPHS
APPROXIMATE SCALE 1" = 1,200'
N
NOEL'S POND
ERNEST HARMON COMPLEX
RESERVOIR
TOWN OF STEPHENVILLE
ST. GEORGES BAY
BLANCHE BROOK
CREEK 8a
Minnesota Drive
Missouri Drive
Carolina Ave.
Massachusetts Drive
New Mexico Drive
Hansen Highway
Queen Street
Main Street
West Street
W-1
W-2
W-3
W-4
W-6
W-8
W-9
W-10
W-11
W-12
W-13
W-14
W-15
W-16
BB-1
BB-2
BB-3
BB-4
BB-5
BB-6
BB-7
BB-8
BB-9
BB-10
BB-11
BB-12
BB-13
BB-15
BB-16
BB-16a
BB-17
BB-18
BB-19
0+00
20+00
40+00
60+00
80+00
100+00
110+74
93+29

FIGURE C-1

of each cross-section was taken from a shore station using a cloth tape for horizontal measurement and the level rod for vertical measurement.

2.3 Physical Description of Streams

2.3.1 Riverbed Profiles

A centerline profile of the riverbed was obtained during the course of the field program for Blanche Brook and Warm Creek. The riverbed profiles for Blanche Brook and Warm Creek are shown on Figures C-2 and C-3 respectively. In general, both riverbeds within the study area are gently sloping, with few exceptions, and as such subcritical flow conditions prevail. In terms of channel slope, Blanche Brook can be categorized into two distinct reaches, from its mouth to its confluence with Warm Creek with a slope of 0.00575 foot/foot,and from the confluence to the Hanson Highway Bridge with a slope of 0.0115 foot/foot.

On the other hand, Warm Creek contains four distinct reaches in terms of channel slope as follows: (1) from its confluence with Blanche Brook (mouth) to the culvert bridge (section W3) with a slope of 0.00683 foot/foot, (2) from the culvert bridge to the wier opposite the thermal generating plant (section W6) with a slope of 0.015 foot/foot, (3) from the wier to the railraod causeway (section W9) with a slope of 0.00583 foot/foot and, (4) from the railroad causeway to Noel's Pond with a slope of 0.00132 foot/foot.

2.3.2. Channel Cross-Sections

In general, the 34 cross-sections were taken perpendicular to the general flow of the river and included the flood plain on either side to verify existing topographical mapping and to aid in the delineation of the flood hazard area. A sample of such a cross-section is provided in Figure C-4.

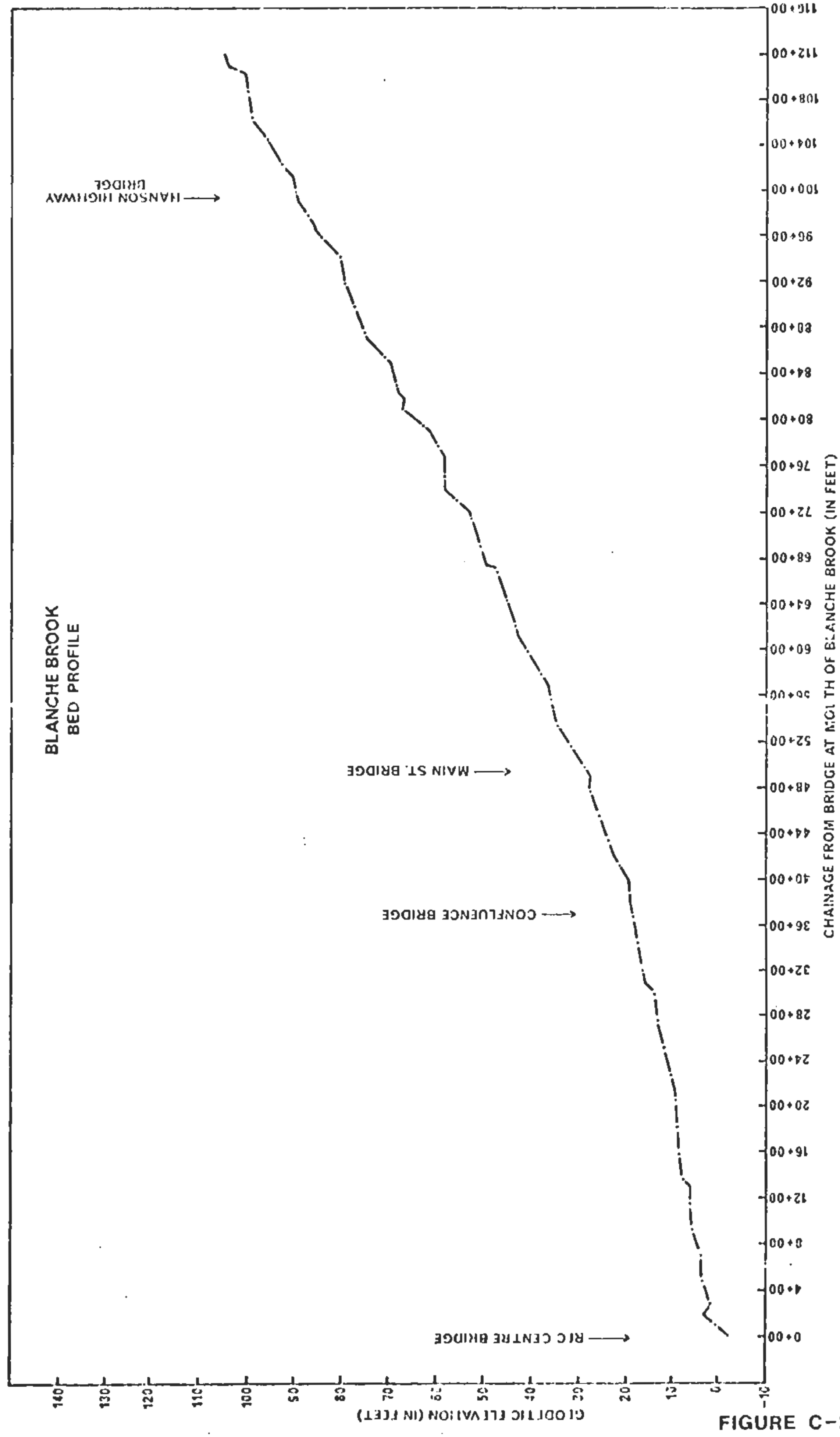
BLANCHE BROOK
BED PROFILE
HANSON HIGHWAY BRIDGE
MAIN ST. BRIDGE
CONFLUENCE BRIDGE
R/C CENTRE BRIDGE
140 130 120 110 100 90 80 70 60 50 40 30 20 10 0 -10
GEODETIC ELEVATION (IN FEET)
0+0 4+00 8+00 12+00 16+00 20+00 24+00 28+00 32+00 36+00 40+00 44+00 48+00 52+00 56+00 60+00 64+00 68+00 72+00 76+00 80+00 84+00 88+00 92+00 96+00 100+00 104+00 108+00 112+00 116+00
CHAINAGE FROM BRIDGE AT MOUTH OF BLANCHE BROOK (IN FEET)

FIGURE C-2

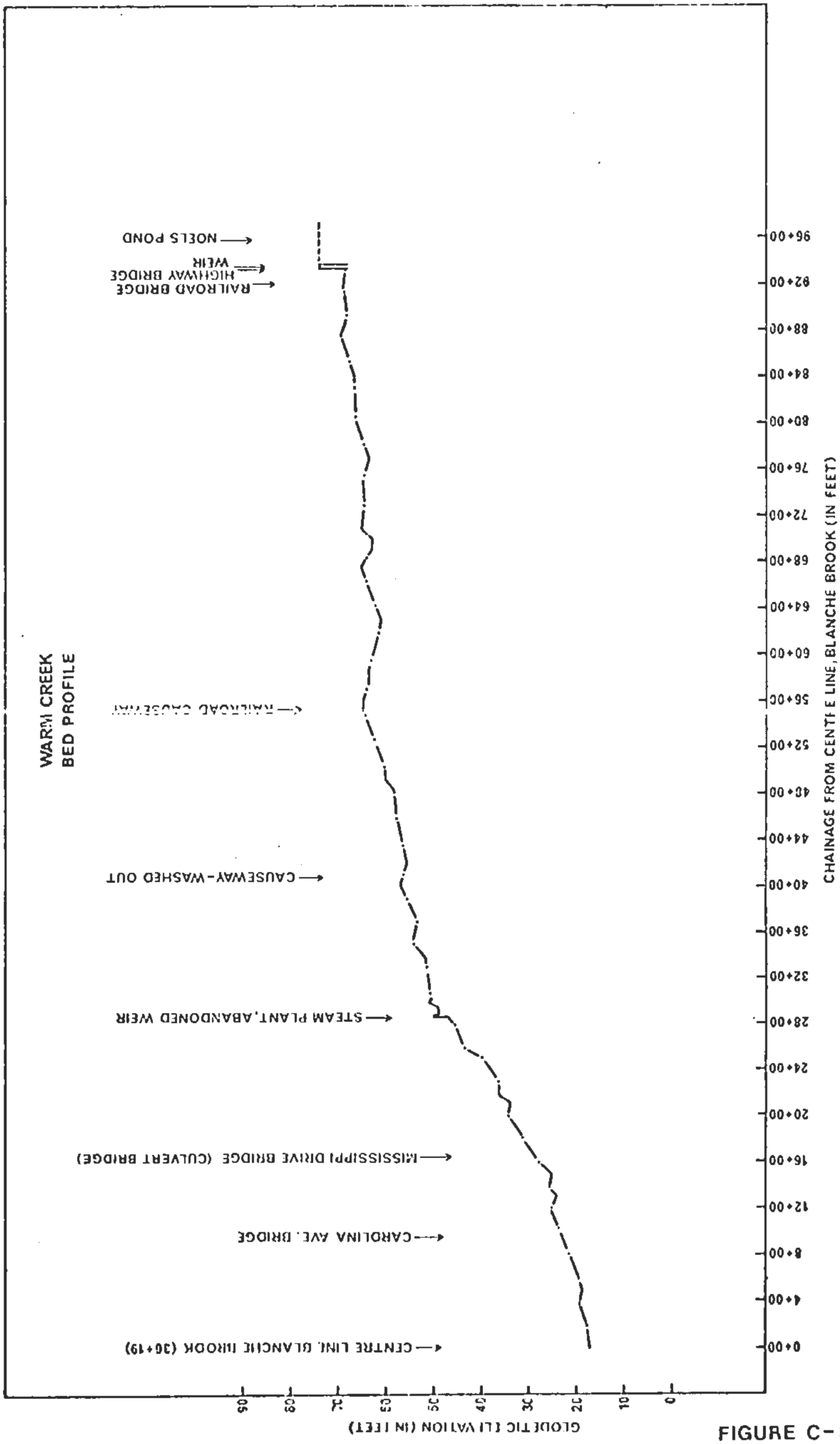
WARM CREEK
BED PROFILE
CENTRE LINE BLANCHE BROOK (30+19)
CAROLINA AVE. BRIDGE
MISSISSIPPI DRIVE BRIDGE (CULVERT BRIDGE)
STEAM PLANT, ABANDONED WEIR
CAUSEWAY-WASHED OUT
RAILROAD CAUSEWAY
RAILROAD BRIDGE
HIGHWAY BRIDGE
WEIR
NOELS POND
CHAINAGE FROM CENTRE LINE, BLANCHE BROOK (IN FEET)
GEODETIC ELEVATION (IN FEET)
0+00
4+00
8+00
12+00
16+00
20+00
24+00
28+00
32+00
36+00
40+00
44+00
48+00
52+00
56+00
60+00
64+00
68+00
72+00
76+00
80+00
84+00
88+00
92+00
96+00
0
10
20
30
40
50
60
70
80
90

FIGURE C-3

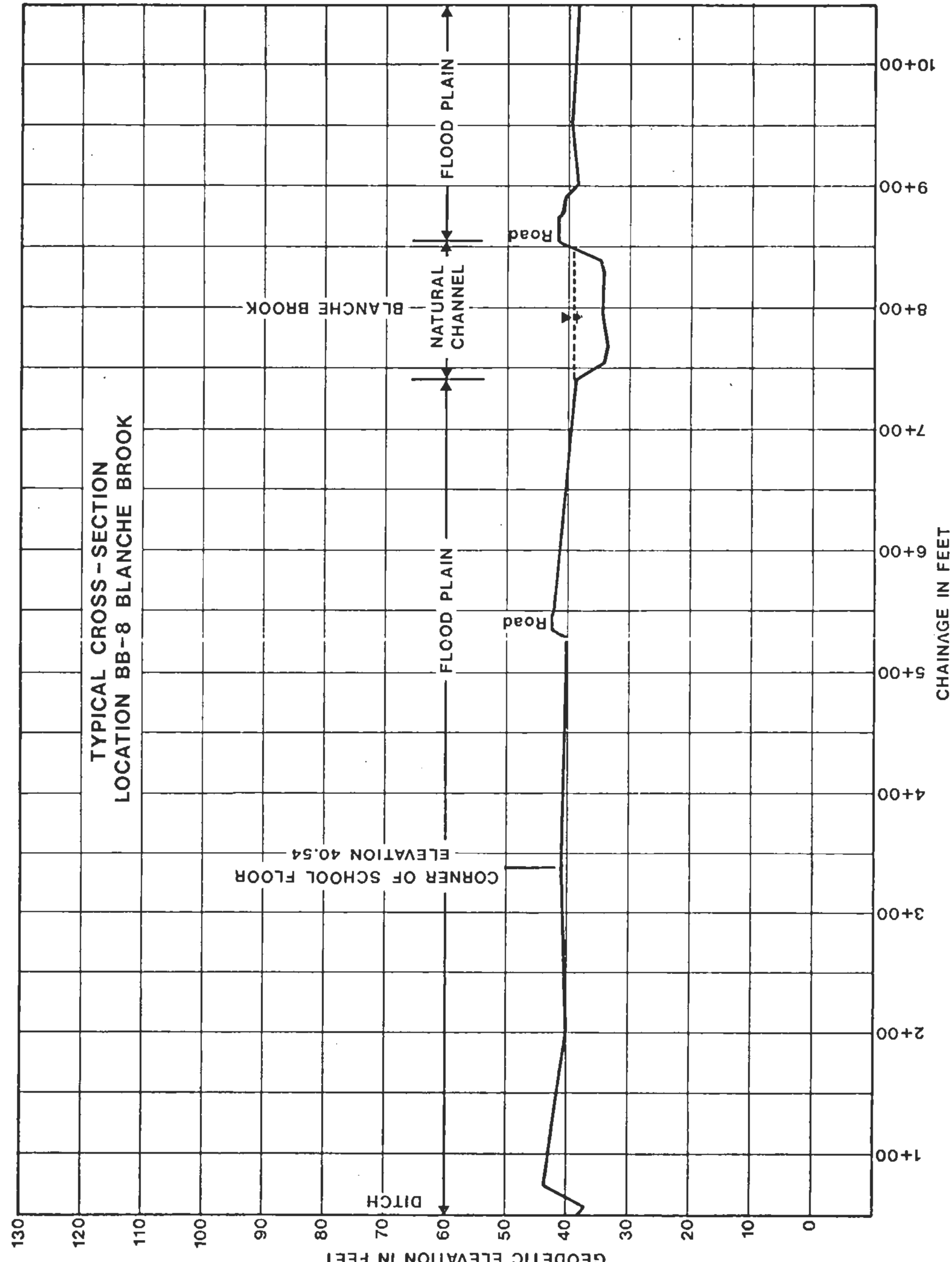
TYPICAL CROSS-SECTION
LOCATION BB-8 BLANCHE BROOK
FLOOD PLAIN
NATURAL CHANNEL
BLANCHE BROOK
FLOOD PLAIN
Road
Road
CORNER OF SCHOOL FLOOR
ELEVATION 40.54
DITCH
10+00
9+00
8+00
7+00
6+00
5+00
4+00
3+00
2+00
1+00
CHAINAGE IN FEET
130
120
110
100
90
80
70
60
50
40
30
20
10
0
GEODETIC ELEVATION IN FEET

FIGURE C-4

The main channel of Blanche Brook is typically 80-140 feet wide with a maximum depth of approximately 5-10 feet relative to its flood plain. On Warm Creek, the defined channel is somewhat smaller with typical widths of 50-80 feet and maximum depths of 4-5 feet.

2.3.3 Constrictions/Obstructions

Special consideration was given to the various man-made structures which cross the stream and in some instances act as constrictions, thus reducing the hydraulic capacity of the river. The major structures are summarized as follows:

(a) Recreation Center Bridge (near mouth of Blanche Brook)

This bridge is a clear span which hydraulically resembles a short box culvert. Under high flow conditions, the channel upstream of the bridge is approximately 250 feet wide, 52 feet at the upstream extremity of the wing walls and 31 feet at the narrowest section at the bridge.

(b) Confluence Bridge (on Blanche Brook just upstream of confluence of Blanche Brook and Warm Creek)

This bridge crosses Blanche Brook in three spans each being approximately 24'-8" long, giving a total width of 73'-8". The piers are at a slight angle to the flow. Two of the openings have a gravel deposit. The approximate width of the natural channel upstream of the bridge is 115 feet.

(c) Main Street Bridge (Blanche Brook)

This bridge crosses Blanche Brook in three spans each being 24'-6" long for a total width of 73'-6". The piers are splayed at an angle of approximately 20^{0} to the flow. Gravel deposits on the western side of the channel have considerably reduced the hydraulic capacity of one span. It is felt that the recent construction of a street into the river upstream of the bridge will add to this problem. Severe ice jamming at this bridge has resulted in significant flooding problems in the past. The approximate width of the natural channel upstream of the bridge is 95 feet.

(d) Natural Constriction (Blanche Brook, BB-9)

Bedrock outcrops act as a natural constriction to flood flows by reducing the average channel width from 120 feet to about 70 feet.

(e) Hanson Highway Bridge (Blanche Brook)

This bridge crosses Blanche Brook at an angle of approximately 45^{0} to the direction of flow. However, the bridge piers are roughly parallel to the principle direction of flow. The bridge consists of four spans varying in width from 20' to 23' 6" measured along the bridge for a total width of 80 feet. The width of the channel upstream of the bridge is approximately 265 feet. Severe jamming, caused by ice and debris, has occurred at this location in the past.

(f) Carolina Avenue Bridge (Warm Creek)

This bridge crosses Warm Creek in a single span. The channel upstream of the bridge is approximately 80 feet wide, 56 feet at the upstream end of the wing walls and 33 feet at the narrowest point at the bridge.

(g) Culvert Bridge (Warm Creek-Mississippi Drive)

This is a causeway structure containing 8 culverts at differing heights above the river bed. Seven culverts are three feet in diameter while the remaining one is 5 feet in diameter. The hydraulic capacity of the culverts is low in relation to flood flows. This has led to damages in the past as a result of overtopping and erosion of the highway bed.

(h) Abandoned Wier (Warm Creek - opposite Thermal Plant)

This sturcture has been abandoned for the past several years and serves no useful purpose at the present time. The headpond has been infilled with sand and gravel. During high flow, water is diverted around the wier resulting in significant flooding.

(i) Washed-Out Causeway (Warm Creek)

At one time this causeway served as a bridge across Warm Creek. However, it was washed out by some prior flood. This constriction does not pose any significant problems related to flooding.

(j) Railraod Causeway (Warm Creek)

This earth - fill causeway, constructed in recent years, has four - four-foot diameter corrugated metal culverts, each 61 feet in length. The culverts have been constructed above the natural bed and as such create a ponding effect. While the hydraulic capacity of the culverts is low, the only potential flooding damage is to the structure itself.

(k) Railroad Bridge (Warm Creek, just downstream of Noels Pond)

This bridge consists of four narrow openings each about 6 feet wide between massive piers which are splayed at an angle of 45^0 to the flow of water. This bridge effects the water level of Noels Pond during flooding conditions owing to its low hydraulic capacity.

(l) Highway Bridge - Noels Pond (Warm Creek)

This bridge, constructed in the early 1970's crosses Warm Creek just downstream of Noels Pond in two spans each of which is approximately 35 feet long. It is felt that this bridge does not significantly affect the hydraulic capacity of Warm Creek.

2.3.4 Channel Morphology

The lower reach of Blanche Brook, below its confluence with Warm Creek, is an artificial channel constructed just prior to 1952. The channel bed in this reach consists of coarse to medium gravel overlain by estuarian clay and silt deposits. Further upstream, bedrock is overlain with thin deposits of coarse gravel and cobbles. In the upper reach bounded by the Hanson Highway Bridge, the channel consists of a thin mantle of coarse gravel and cobbles underlain by bedrock which is exposed at several locations. The bedrock is classified as greyish sandstone of the Carboniferous period.

2.3.5 Hydraulic Properties

In view of the nature of the channel bed as described above, a Mannings Roughness Coefficient, "n", of 0.04 and 0.03 was selected for Blanche Brook and Warm Creek, respectively. In cases of significant overbank flow, Manning's "n" was adjusted upwards as was the velocity distribution coefficient "a".

For each cross-section, the areas and hydraulic radii were computed for various depths of flow. A sample (BB-8) is illustrated in Figure C-5.

2.3.6 Man's Influence on the River Regime

Over the years, man has dramatically influenced the regime of the river system through various activities including encroachment on the flood plain, the construction of bridges/causeways, and major alteration in the course of the river. With the construction of the U.S. Air Force Base in 1940, the lower reaches of Blanche Brook and Warm Creek were diverted into drainage ditches bypassing the site. Since that time, the Base was expanded several times with attendant drainage/diversion ditches being constructed on each occasion. The result of this activity was to alter the course of the

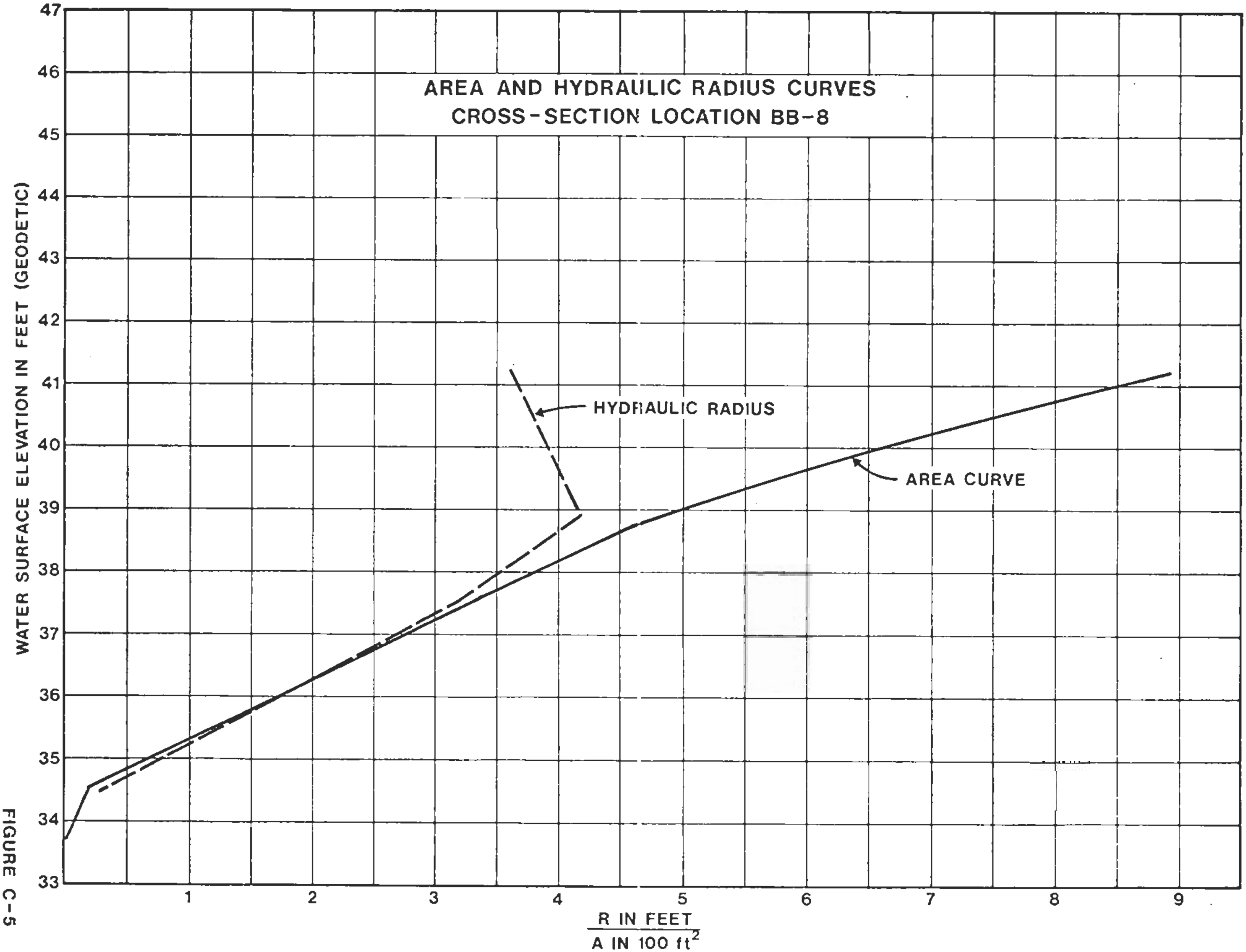
AREA AND HYDRAULIC RADIUS CURVES
CROSS-SECTION LOCATION BB-8
HYDRAULIC RADIUS
AREA CURVE
WATER SURFACE ELEVATION IN FEET (GEODETIC)
47
46
45
44
43
42
41
40
39
38
37
36
35
34
33
1
2
3
4
5
6
7
8
9
R IN FEET
A IN 100 ft²

FIGURE C-5

Blanche Brook and Warm Creek significantly. The site of the present day brewery is very nearly situated on the old river bed and as such directly in line with the natural course of the river.

Of course, other less dramatic changes have also occurred. Development has been encroaching on the flood plain and in view of present plans, there is every indication that this will continue. The implications of this are two-fold. Firstly, new development of the flood plain has the potential of increasing flood damages. Secondly, new development has the potential of increasing flood magnitudes by reducing the capacity of the river to carry flood flows. The construction of highway and railway bridges/causeways have in some instances reduced the flow capacity of the river by creating constrictions and obstructions.

3. BACKWATER ANALYSIS

The standard step method for natural channels as described in Chow[1] was used to compute water surface profiles under the flow conditions resulting from mean annual, 1:20 and 1:100 year recurrance interval floods as determined in Appendix B. Special consideration was given to the effect of bridge piers and culvert bridges on the hydraulic regime. The water surface profiles were determined on the basis of 50% and 100% blockage due to ice and debris at each constriction. Appropriate profiles were selected in each case based on rational selection of the most likely condition.

3.1 Methodology

3.1.1 Open Channel Flow

The standard step method of backwater analysis was applied using the following input data; (1) Riverflow, (2) Initial Starting Elevation,

1. Chow, Van Te, Open Channel Hydraulics, McGraw-Hill Book Co., N.Y., 1959.

(3) Hydraulic properties of each cross-section, and (4) Manning's Roughness Coefficient. Backwater calculations were carried out in a downstream to upstream direction when the flow regime was subcritical and vice versa for a supercritical flow regime.

The procedure is an interative or trial and error process. Referring to Figure C-6 and following the principle of continuity, the following may be written:

$$Z_1 + Y_1 + \alpha_1 \frac{V_1^2}{2g} = Z_2 + Y_2 + \alpha_2 \frac{V_2^2}{2g} + h_f + h_e \quad \ldots\ldots\ldots\ldots\ldots\ldots(1)$$

In the above equation, it was assumed that eddy losses, h_e, would be minimal and as such was set to zero. This assumption is valid in that (1) the sections were chosen such that the reach they represented was relatively uniform and (2) the value of Manning's "n" was selected such that eddy losses would be accounted for in the friction loss component, h_f.

The friction losses, h_f, was determined by applying the following emperical relationship:

$$h_f = S_f\,\Delta x = \left(\frac{S_{f1} + S_{f2}}{2}\right) \cdot \Delta x = \frac{n^2 \cdot \Delta x}{4.44} \left(\frac{Y_1^2}{R_1^{4/3}} + \frac{Y_2^2}{R_2^{4/3}}\right) \ldots\ldots\ldots\ldots(2)$$

Referring to figure C-6, the following relationships can be drawn:

$$H_2 = Z_2 + Y_2 + \frac{\alpha V_2^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3)$$

and

$$H_2 = H_1 - h_f \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

Using the above relationships, two estimates of H_2 are determined given an estimate of Y_2. If the estimates of H_2 differ more than the level of accuracy required, a new value of Y_2 is assumed. This process is repeated until a common solution is arrived at, thus satisfying the conservation of energy principle as described in equation 1.

TYPICAL CHANNEL REACH FOR DERIVATION OF THE STANDARD STEP METHOD

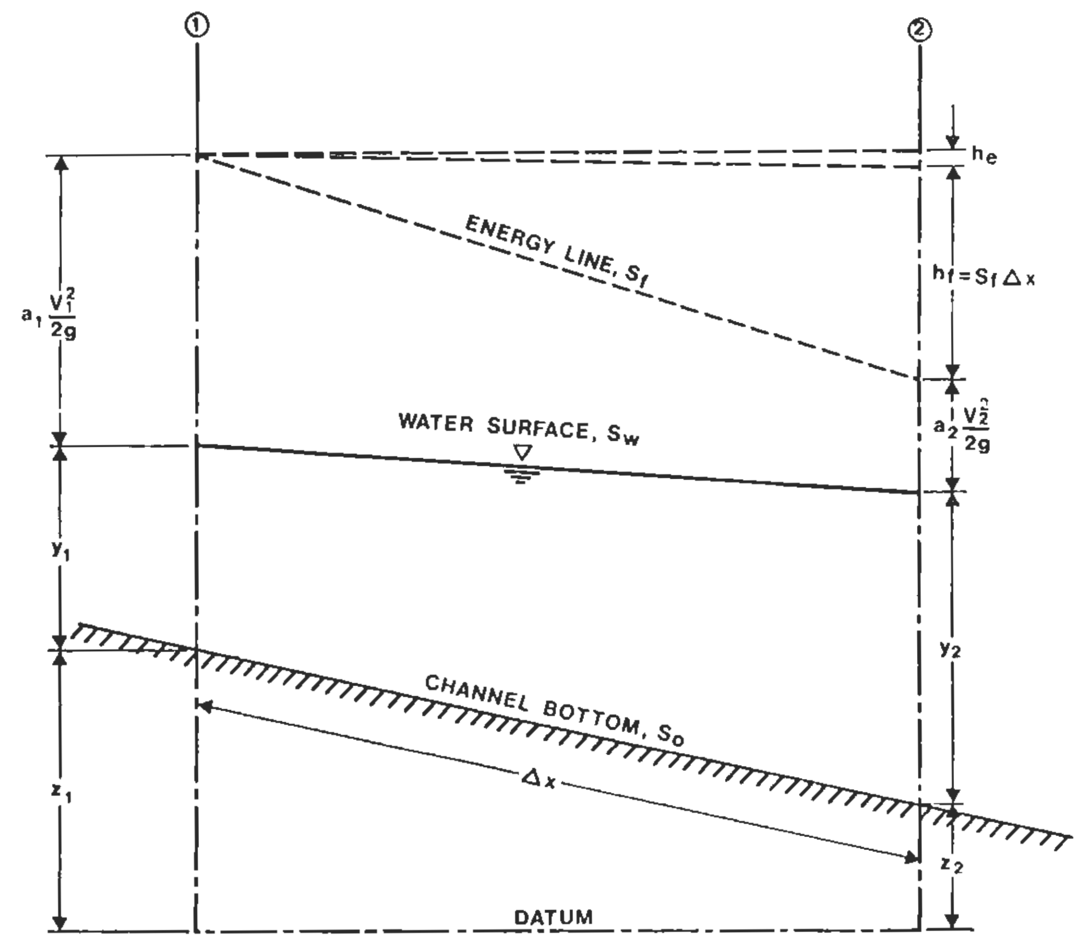

FIGURE C-6

Another approach which was used to minimize the number of iterations was to determine a value, ΔY_2;

$$\Delta Y_2 = \frac{\Delta H_2}{1-Fr_2^{\,2} + \frac{3S_{f_2}\Delta X}{2R_2}} \qquad \text{(5)}$$

which was applied to the previous estimate of Y_2.

At each section, it was determined whether the flow regime was supercritical, critical, or subcritical, to ensure that backwater calculations were being carried out in the proper direction. This was accomplished through application of the following equation:

$$A\sqrt{D_c} = Q\sqrt{\alpha/g} \qquad \text{(6)}$$

This may be solved by a trial and error process whereby an estimate of D_c is made from which both sides of the equation are estimated. This is continued until such time as both sides of the equation balance.

The energy coefficient, α, used in the above equations was assumed to be 1.3 which represents a median value as reported in the literature. In the case of overbank flow, this value was adjusted upward to account for a wider variation in velocity.

3.1.2 Constrictions

There are 12 structures within the study area which in one way or another influence the hydraulic characteristics of the river. These include two bridges with clear span, 5 bridges with piers, 3 culvert bridges (one of which is partially removed) and 2 wiers.

Contraction and expansion losses resulting from bridge abutments were estimated using the following emperical formulae:

$$\Delta h = k\left(\frac{V_1^{\,2} - V_0^{\,2}}{2g}\right) \qquad \text{(7)}$$

The coefficient, "k", of contraction was assumed to be 0.2 while the coefficient, "k", of expansion was assumed to be 0.3 based on a review of the literature.

The losses due to bridge piers were determined based on methods developed by Yarnell, Koch, and Carstanien as detailed in the literature[2]. The methodology has not been repeated here for the sake of brevity. In addition, pier losses are relatively minor in relation to other losses in this instance.

There are two wiers within the study area, one at the outlet of Noels Pond and the other near the existing thermal electric plant on Warm Creek. In addition, flows over streets and bridges were determined based on the wier concept. The state of flow over a broad crested wier may be described by:

$$Q = CLH^{3/2} \quad \ldots\ldots (8)$$

Historically flooding has been associated with blockages by ice and/or debris at the various constrictions. To evaluate the effect of blockages, a sensitivity analysis was carried out. This involved the determination of the backwater effect of the various constrictions under three conditions; (1) free flowing, (2) each constriction blocked 50% and (3) total blockage at each constriction. Should the super-structure of any constriction be overtopped, the resulted water surface profile was calculated assuming that the flow regime resulting from flowing under the super-structure was analogous to a submerged orifice while the water flowing over the super-structure may be approximated by assuming that a broad crested wier

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2. Hydraulic Design Criteria, Waterways Experimental Station, U.S. Army Corps of Engineers.

approximates the flow regime. The flow regime resulting from a submerged orifice may be approximated by the following:

$$Q = CA \sqrt{2gH_0} \quad \ldots\ldots (9)$$

3.1.3 Starting Elevation

A small amount of ground truth data exists dealing with the hydraulic regime in the downstream portion of Blanche Brook. The starting point adopted for this study was the bridge near the town's recreation center. The following steps were carried out in the determination of a starting level:

(i) The critical depth was found to be 12 feet (9 feet Geodetic) and 10.2 feet (7.2 feet Geodetic) for the 1:100 and 1:20 recurrence flood interval, respectively.

(ii) The tailwater elevation below the bridge under hightide would be 6-8 feet plus losses or about 9 feet. In any case it is felt that a 9 feet elevation would be attained despite the high tide condition since a well defined river outlet does not exist under 7 feet Geodetic. At elevation 9 feet, the flow regime downstream of the bridge is subcritical.

(iii) The state of flow upstream of the bridge was determined to be subcritical. Since the critical depth at the bridge was computed to be about 3 feet higher than the normal depth of the upstream channel, an M_1 profile is indicated.

(iv) The submerged orifice, equation 9 above, modified to take into account approach velocity, was applied to determine the losses through the bridge. Based on this analysis the starting level was determined to be 11 feet Geodetic and 9 feet Geodetic for the 1:100 and 1:20 feet recurrence interval flood under a free flowing condition. This was modified in the analysis by the effect of channel blockages.

4. THE FLOOD HAZARD MAP

This section of the report deals with the delineation of the extent of flooding under 1:100 and 1:20 year recurrence interval floods. It should be borne in mind that the flood hazard map as developed is based on existing conditions. The following paragraphs summarize the various steps taken in the preparation of the flood hazard map.

4.1 The Base Map

One of the prerequisites to the development of an accurate flood hazard map is the availability of a suitable and accurate topographic map. Two basic sources of mapping were employed in the development of a base map. These were; (1) Basic Layout Plan, Ernest Harmon Complex, Scale: 1" = 400 feet, Contour Interval: 5 feet compiled by Henningson, Durham and Richardson Inc., Omaha, Nebraska for the U.S. Department of the Air Force and (2) Topographic Mapping, Town of Stephenville, Scale: 1" = 200 feet, Contour Interval: 5 feet compiled by the Photographic Survey Corporation Ltd. based on 1958 photographs and revised based on May 1963 and September 1966 photographs. The former map was blown up to a scale of 1" = 200 feet and spliced with the latter to form the base map. It must be pointed out that the mapping on the Harmon Complex appears in places to be distorted and as such the final base map was not of uniform quality. However, based on field surveys, the topographic features of the flood plain were revised and brought up to a consistent and relatively accurate state.

4.2 Water Surface Profiles

Water surface profiles were developed for the Mean Annual, 1:20 year and 1:100 year recurrence interval floods under three conditions: (1) no blockage at constrictions, (2) 50% blockage at each constriction and (3) 100% blockage at each constriction.

4.2.1 Mean Annual Flood

The water surface profile developed for the mean annual flood is shown on Figure C-7 and on Table C-1 for Blanche Brook and on Figure C-8 and Table C-2 for Warm Creek. The solid line in Figure C-7 and C-8 represents the estimated water surface profiles under free flowing conditions. Because of the history of channel blockages at the various constrictions by ice and debris, the estimated water surface profiles shown on Figures C-7 and C-8 by a dashed line represents the condition whereby all constrictions are fully blocked with ice. Profiles were also developed under the condition when the cross-section would be reduced by 50% at each constriction.

4.2.2 1:20 Year Recurrence Interval Flood

The water surface profile developed for the 1:20 Year Recurrence Interval Flood shown on Figure C-9 and on Table C-1 for Blanche Brook and on Figure C-10 and on Table C-2 for Warm Creek.

4.2.3 1:100 Year Recurrence Interval Flood

The water surface profile developed for the 1:100 Year Recurrence Interval flood is shown on Figure C-11 and on Table C-1 for Blanche Brook and on Figure C-12 and on Table C-2 for Warm Creek.

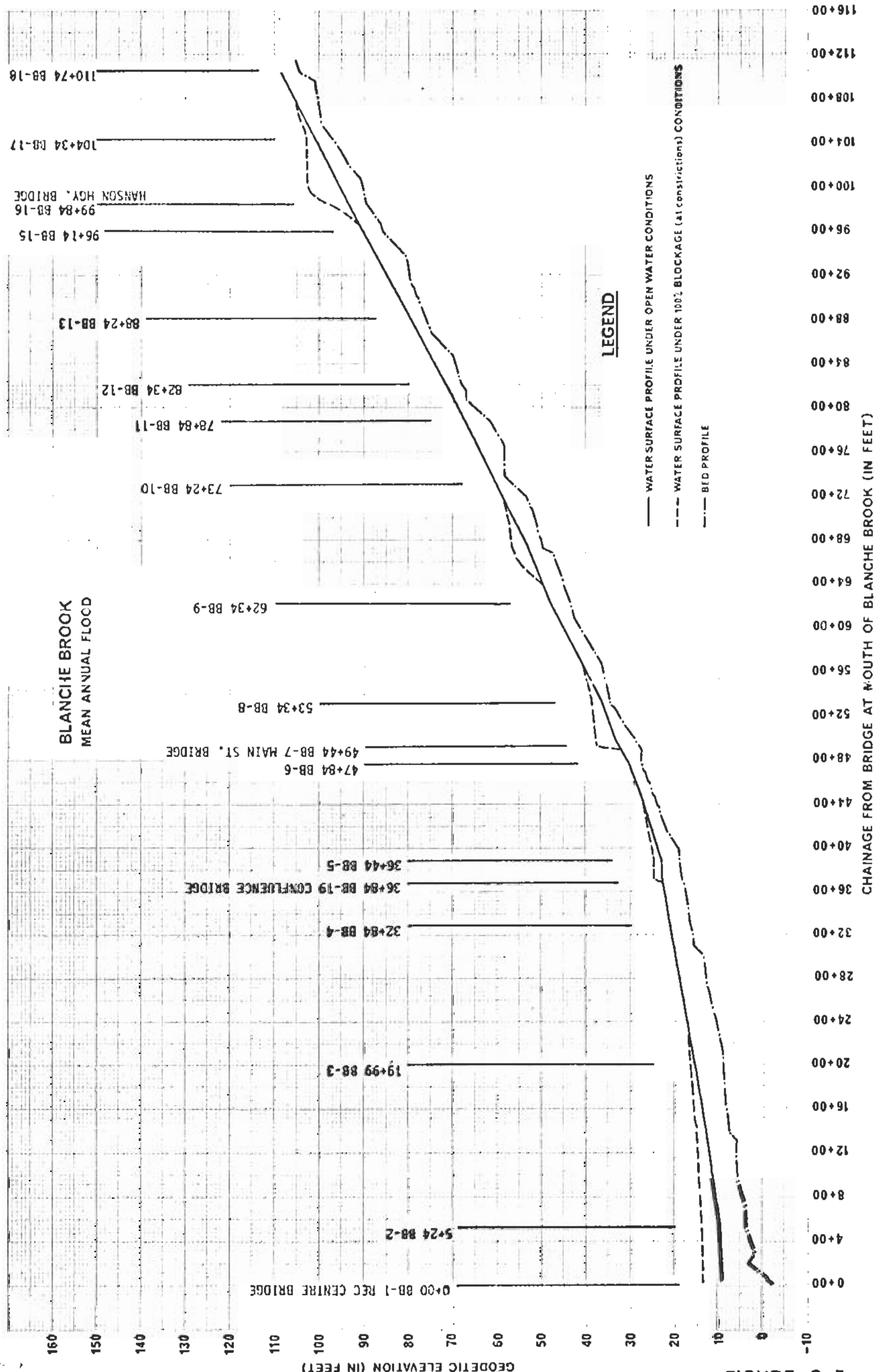

BLANCHE BROOK
MEAN ANNUAL FLOOD
LEGEND
WATER SURFACE PROFILE UNDER OPEN WATER CONDITIONS
WATER SURFACE PROFILE UNDER 100% BLOCKAGE (at constrictions) CONDITIONS
BED PROFILE
110+74 BB-18
104+34 BB-17
99+84 BB-16
HANSON HGY. BRIDGE
96+14 BB-15
88+24 BB-13
82+34 BB-12
78+84 BB-11
73+24 BB-10
62+34 BB-9
53+34 BB-8
49+44 BB-7 MAIN ST. BRIDGE
47+84 BB-6
36+44 BB-5
36+84 BB-19 CONFLUENCE BRIDGE
32+84 BB-4
19+99 BB-3
5+24 BB-2
0+00 BB-1 REC CENTRE BRIDGE
GEODETIC ELEVATION (IN FEET)
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-10
CHAINAGE FROM BRIDGE AT MOUTH OF BLANCHE BROOK (IN FEET)
0+00
4+00
8+00
12+00
16+00
20+00
24+00
28+00
32+00
36+00
40+00
44+00
48+00
52+00
56+00
60+00
64+00
68+00
72+00
76+00
80+00
84+00
88+00
92+00
96+00
100+00
104+00
108+00
112+00
116+00

FIGURE C-7

TABLE C-1

WATER SURFACE PROFILES[1]

BLANCHE BROOK

STATION	CHAINAGE (feet)	1:100 YEAR RECURRENCE INTERVAL FLOOD: OPEN WATER CONDITION (feet)	1:100 YEAR RECURRENCE INTERVAL FLOOD: 100% BLOCKAGE CHANNEL CONSTRICTIONS (feet)	1:20 YEAR RECURRENCE INTERVAL FLOOD: OPEN WATER CONDITION (feet)	1:20 YEAR RECURRENCE INTERVAL FLOOD: 100% BLOCKAGE CHANNEL CONSTRICTIONS (feet)	MEAN ANNUAL FLOOD: OPEN WATER CONDITION (feet)	MEAN ANNUAL FLOOD: 100% BLOCKAGE CHANNEL CONSTRICTIONS (feet)
BB-1	0+00						
	0+40	11.0	14.6	9.0	14.4	9.0	13.8
BB-2	5+24	11.2	14.6	10.0	14.4	9.5	13.8
BB-3	19+99	20.0	20.0	19.2	19.2	15.3	16.3
BB-4	32+84	23.9	23.9	22.8	22.8	21.0	21.0
	36+44	24.4	24.4	23.6	23.6	22.3	22.3
	37+04	24.7	25.6	23.9	25.0	22.5	24.8
BB-5	38+44	24.8	25.6	24.0	25.0	22.9	24.8
	44+84	29.1	29.1	28.5	28.5	27.3	27.5
BB-6	47+84	31.5	31.5	31.1	31.1	30.3	30.3
	48+94	33.1	33.1	32.5	32.5	31.9	31.9
BB-7	49+56	35.2	38.1	33.2	38.0	33.0	37.7
BB-8	53+34	38.3	39.0	36.8	39.0	36.2	38.7
BB-9	62+34	51.6	51.6	50.3	50.3	48.0	48.0
	67+24	55.0	58.5	54.2	57.5	53.0	56.7
BB-10	73+24	61.4	61.4	61.1	61.1	60.5	60.5
BB-11	78+84	69.0	69.0	68.4	68.4	67.3	67.3
BB-12	82+34	73.0	73.0	72.6	72.6	71.5	71.5
BB-13	88+24	81.3	81.3	80.7	80.7	79.7	79.7
BB-15	96+14	92.0	92.0	91.4	91.4	90.2	90.2
BB-16a	99+84	96.1	103.0	95.6	102.8	94.8	102.4
BB-17	104+34	102.6	103.7	101.9	103.4	100.3	102.8
BB-18	110+74	109.9	109.9	109.4	109.4	108.5	108.5

[1] ALL ELEVATIONS REFERRED TO GSC DATUM

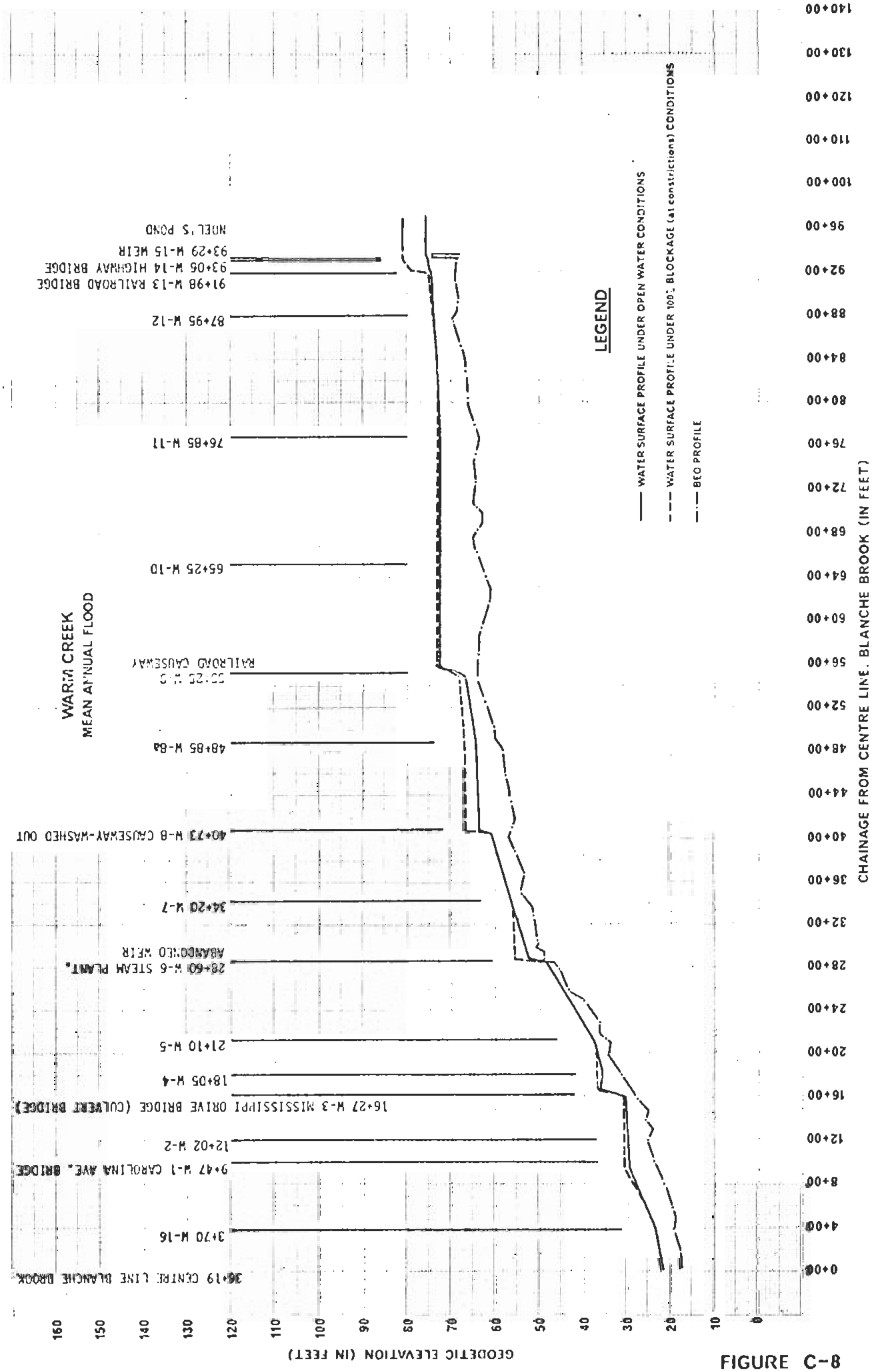

WARM CREEK
MEAN ANNUAL FLOOD
LEGEND
WATER SURFACE PROFILE UNDER OPEN WATER CONDITIONS
WATER SURFACE PROFILE UNDER 100% BLOCKAGE (at constrictions) CONDITIONS
BED PROFILE
CENTRE LINE BLANCHE BROOK
3+70 W-16
9+47 W-1 CAROLINA AVE. BRIDGE
12+02 W-2
16+27 W-3 MISSISSIPPI DRIVE BRIDGE (CULVERT BRIDGE)
18+05 W-4
21+10 W-5
28+60 W-6 STEAM PLANT.
ABANDONED WEIR
34+20 W-7
40+73 W-8 CAUSEWAY-WASHED OUT
48+85 W-8a
RAILROAD CAUSEWAY
65+25 W-10
76+85 W-11
87+95 W-12
91+98 W-13 RAILROAD BRIDGE
93+06 W-14 HIGHWAY BRIDGE
93+29 W-15 WEIR
NOEL'S POND
GEODETIC ELEVATION (IN FEET)
CHAINAGE FROM CENTRE LINE. BLANCHE BROOK (IN FEET)

FIGURE C-8

TABLE C-2

WATER SURFACE PROFILES[1]

WARM CREEK

STATION	CHAINAGE (feet)	1:100 YEAR RECURRENCE INTERVAL FLOOD		1:20 YEAR RECURRENCE INTERVAL FLOOD		MEAN ANNUAL FLOOD	
		OPEN WATER CONDITION (feet)	100% BLOCKAGE CHANNEL CONSTRICTIONS (feet)	OPEN WATER CONDITION (feet)	100% BLOCKAGE CHANNEL CONSTRICTIONS (feet)	OPEN WATER CONDITION (feet)	100% BLOCKAGE CHANNEL CONSTRICTIONS (feet)
Intersection of Warm Creek with Blanche Bk	0+00	24.4[2]	24.4[2]	23.3[2]	23.3[2]	22.0[2]	22.0[2]
W-16	3+70	25.5	25.5	25.0	25.0	23.7	23.7
W-1	9+47	30.3	31.4	30.3	31.1	29.5	30.8
W-2	12+02	31.0	31.4	31.0	31.5	29.9	30.9
	15+77	31.0	31.0	31.3	31.6	30.5	31.0
	16+17	32.2	32.2	32.0	32.0	31.6	31.6
	16+73	36.5	37.5	36.4	37.3	36.1	36.8
W-4	18+05	36.4	36.4	36.1	36.1	35.8	35.8
W-5	21+10	39.5	39.5	38.6	38.6	37.6	37.6
	28+38	51.4	51.4	50.0	50.0	48.5	48.5
	28+75	55.5	57.5	54.8	57.0	52.5	55.6
	31+76	56.2	56.2	55.4	--	54.5	55.6
W-7	34+20	59.2	59.2	58.2	58.2	56.8	56.9
	40+33	63.0	63.0	62.5	62.5	61.0	61.0
W-8	40+73	67.5	69.0	67.0	68.8	63.6	67.0
W-8a	48+85	68.2	69.2	67.5	69.0	64.4	67.2
	55+10	69.6	70.0	69.1	69.7	66.9	58.2
	55+95	73.6	73.9	73.3	73.6	72.5	73.1
W-10	65+25	73.6	73.9	73.3	73.7	72.6	73.1
W-11	76+85	73.8	74.1	73.4	73.8	72.7	73.2
W-12	87+95	75.0	75.1	74.5	74.6	73.7	74.0
	91+78	76.7	76.7	75.9	75.9	74.7	74.7
	92+31	77.2	80.3	76.5	80.1	75.1	79.7
	93+15	77.2	82.0	76.5	81.7	75.2	81.0
Noels Pond		77.7	82.2	77.2	82.0	75.9	81.2

[1] ALL ELEVATIONS REFERRED TO GSC DATUM

[2] ELEVATIONS TAKEN FROM BLANCHE BROOK FLOOD PROFILES

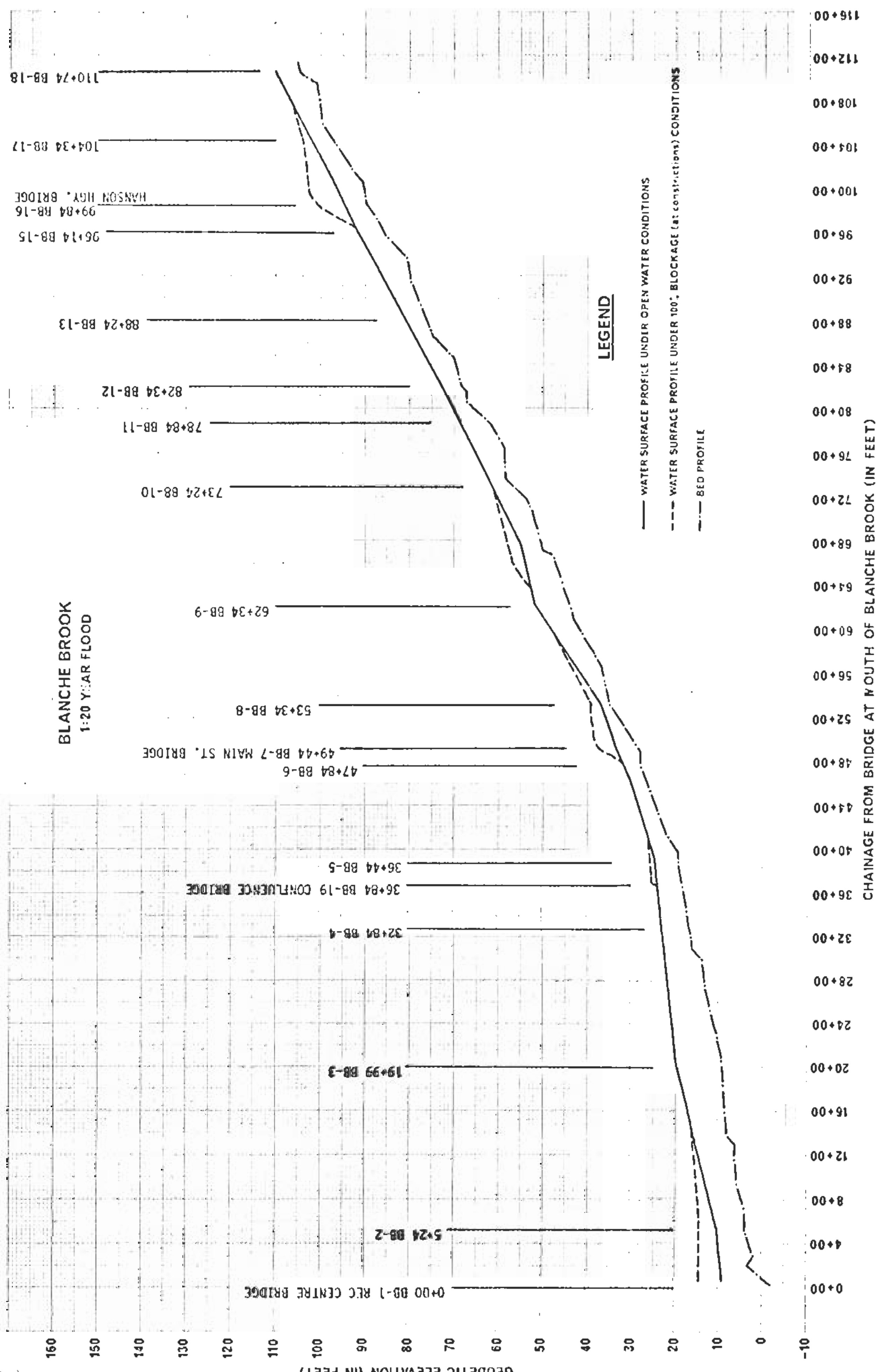

BLANCHE BROOK
1:20 YEAR FLOOD
LEGEND
WATER SURFACE PROFILE UNDER OPEN WATER CONDITIONS
WATER SURFACE PROFILE UNDER 100% BLOCKAGE (at constrictions) CONDITIONS
BED PROFILE
110+74 BB-18
104+34 BB-17
HANSON HGY. BRIDGE
99+84 BB-16
96+14 BB-15
88+24 BB-13
82+34 BB-12
78+84 BB-11
73+24 BB-10
62+34 BB-9
53+34 BB-8
49+44 BB-7 MAIN ST. BRIDGE
47+84 BB-6
36+44 BB-5
36+84 BB-19 CONFLUENCE BRIDGE
32+84 BB-4
19+99 BB-3
5+24 BB-2
0+00 BB-1 REC CENTRE BRIDGE
GEODETIC ELEVATION (IN FEET)
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-10
CHAINAGE FROM BRIDGE AT MOUTH OF BLANCHE BROOK (IN FEET)
0+0
4+00
8+00
12+00
16+00
20+00
24+00
28+00
32+00
36+00
40+00
44+00
48+00
52+00
56+00
60+00
64+00
68+00
72+00
76+00
80+00
84+00
88+00
92+00
96+00
100+00
104+00
108+00
112+00
116+00

FIGURE C-9

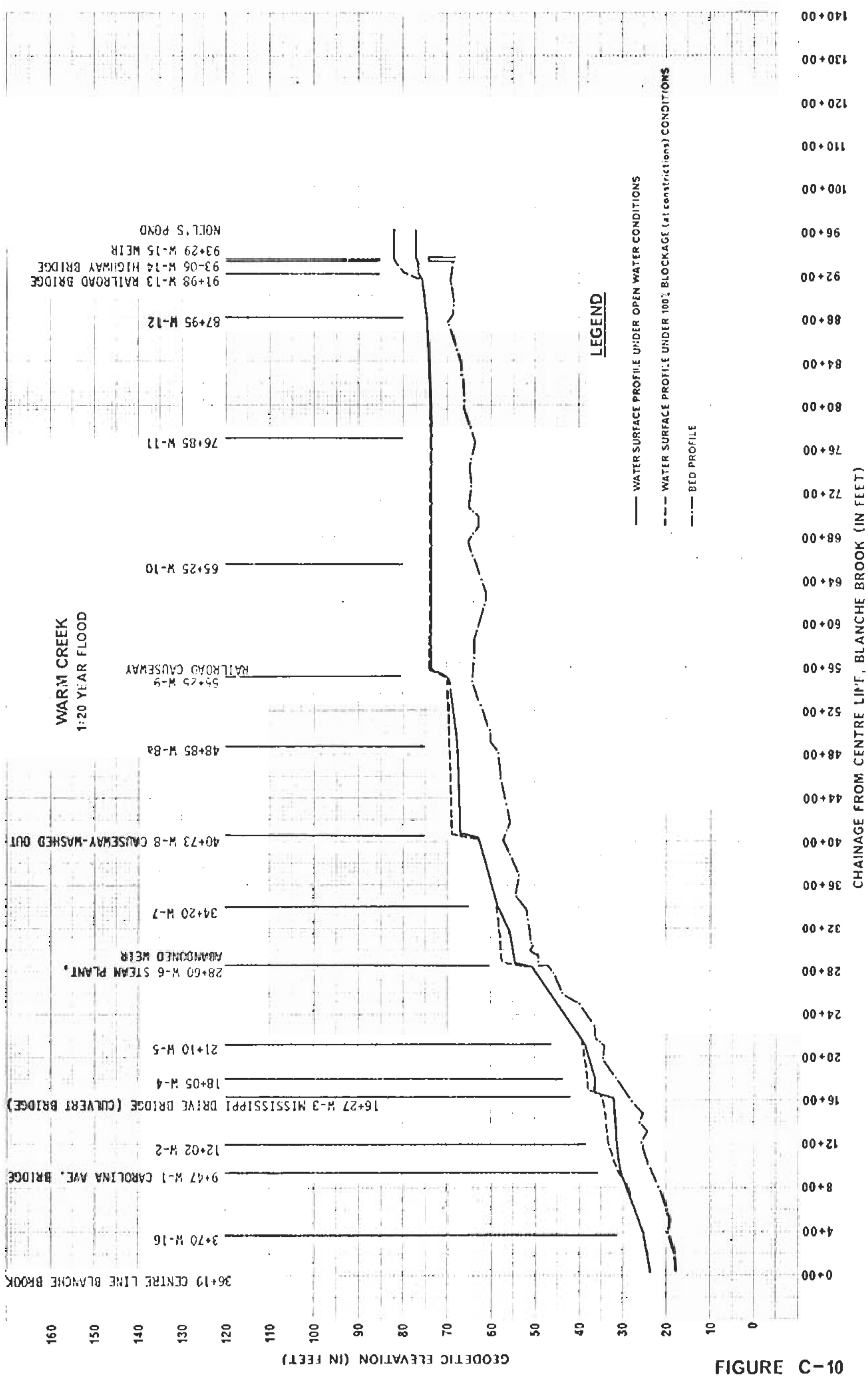

WARM CREEK
1:20 YEAR FLOOD
LEGEND
WATER SURFACE PROFILE UNDER OPEN WATER CONDITIONS
WATER SURFACE PROFILE UNDER 100% BLOCKAGE (at constrictions) CONDITIONS
BED PROFILE
NOLL'S POND
93+29 W-15 WEIR
93+06 W-14 HIGHWAY BRIDGE
91+98 W-13 RAILROAD BRIDGE
87+95 W-12
76+85 W-11
65+25 W-10
55+25 W-9
RAILROAD CAUSEWAY
48+85 W-8a
40+73 W-8 CAUSEWAY-WASHED OUT
34+20 W-7
28+60 W-6 STEAM PLANT,
ABANDONED WEIR
21+10 W-5
18+05 W-4
16+27 W-3 MISSISSIPPI DRIVE BRIDGE (CULVERT BRIDGE)
12+02 W-2
9+47 W-1 CAROLINA AVE. BRIDGE
3+70 W-16
36+19 CENTRE LINE BLANCHE BROOK
GEODETIC ELEVATION (IN FEET)
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
CHAINAGE FROM CENTRE LINE, BLANCHE BROOK (IN FEET)
0+0
4+00
8+0
12+00
16+00
20+00
24+00
28+00
32+00
36+00
40+00
44+00
48+00
52+00
56+00
60+00
64+00
68+00
72+00
76+00
80+00
84+00
88+00
92+00
96+00
100+00
110+00
120+00
130+00
140+00

FIGURE C-10

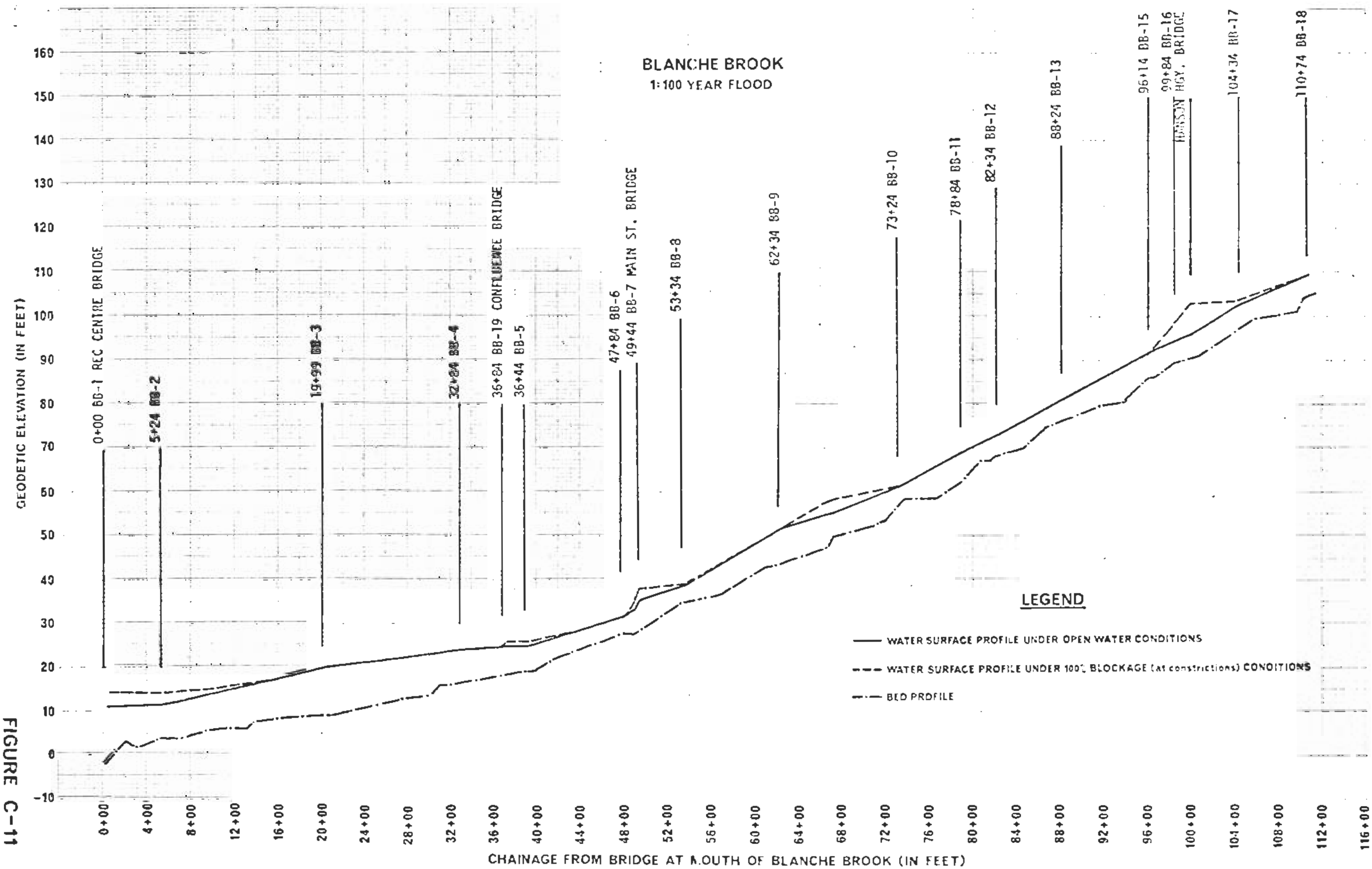

BLANCHE BROOK
1:100 YEAR FLOOD
0+00 BB-1 REC CENTRE BRIDGE
5+24 BB-2
19+99 BB-3
32+84 BB-4
36+84 BB-19 CONFLUENCE BRIDGE
36+44 BB-5
47+84 BB-6
49+44 BB-7 MAIN ST. BRIDGE
53+34 BB-8
62+34 BB-9
73+24 BB-10
78+84 BB-11
82+34 BB-12
88+24 BB-13
96+14 BB-15
99+84 BB-16
HANSON HGY. BRIDGE
104+34 BB-17
110+74 BB-18
LEGEND
WATER SURFACE PROFILE UNDER OPEN WATER CONDITIONS
WATER SURFACE PROFILE UNDER 100% BLOCKAGE (at constrictions) CONDITIONS
BED PROFILE
GEODETIC ELEVATION (IN FEET)
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
-10
0+00
4+00
8+00
12+00
16+00
20+00
24+00
28+00
32+00
36+00
40+00
44+00
48+00
52+00
56+00
60+00
64+00
68+00
72+00
76+00
80+00
84+00
88+00
92+00
96+00
100+00
104+00
108+00
112+00
116+00
CHAINAGE FROM BRIDGE AT MOUTH OF BLANCHE BROOK (IN FEET)

FIGURE C-11

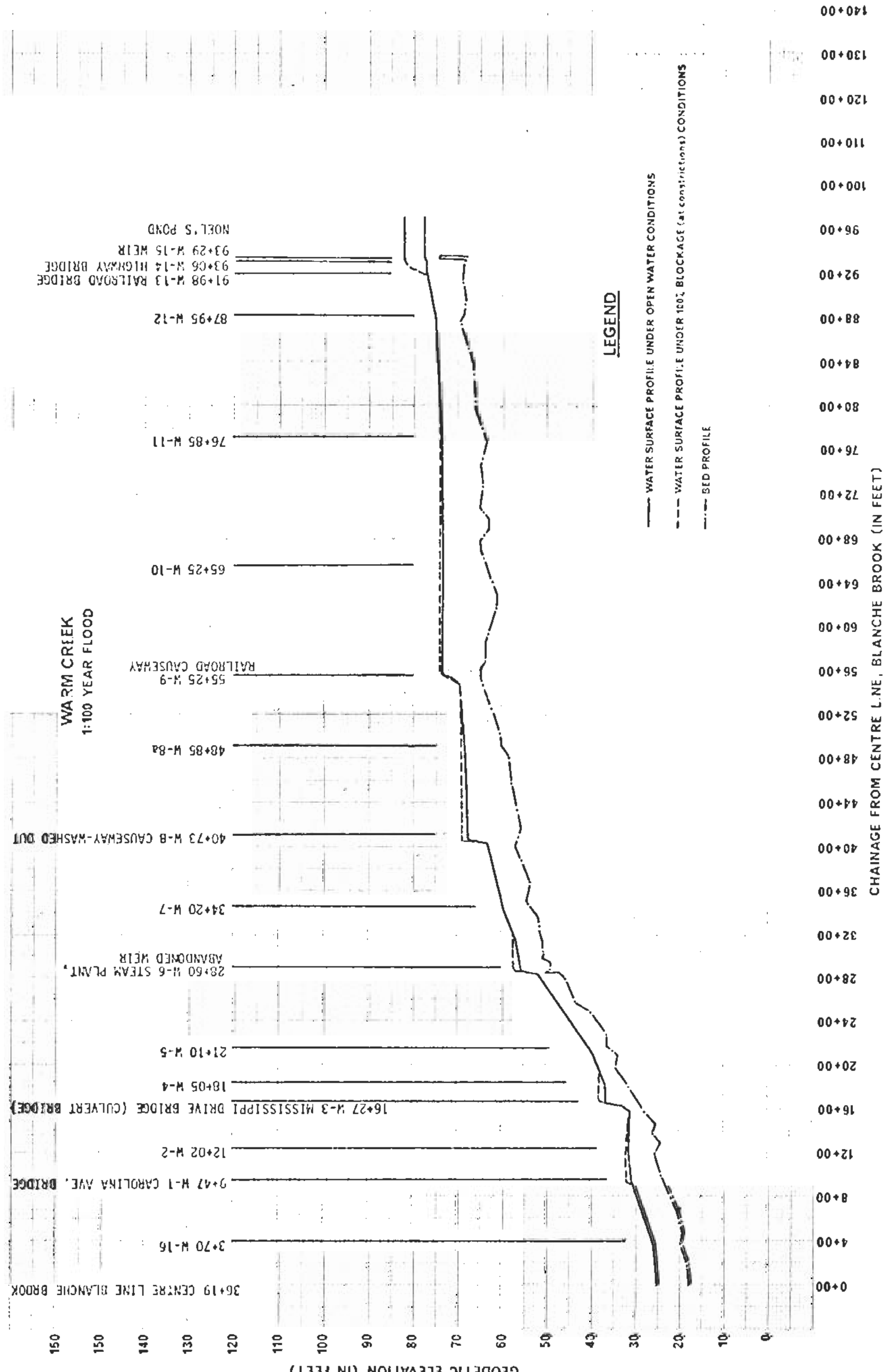

WARM CREEK
1:100 YEAR FLOOD
LEGEND
WATER SURFACE PROFILE UNDER OPEN WATER CONDITIONS
WATER SURFACE PROFILE UNDER 100% BLOCKAGE (at constrictions) CONDITIONS
BED PROFILE
36+19 CENTRE LINE BLANCHE BROOK
3+70 W-16
9+47 W-1 CAROLINA AVE. BRIDGE
12+02 W-2
16+27 W-3 MISSISSIPPI DRIVE BRIDGE (CULVERT BRIDGE)
18+05 W-4
21+10 W-5
28+60 W-6 STEAM PLANT, ABANDONED WEIR
34+20 W-7
40+73 W-8 CAUSEWAY-WASHED OUT
48+85 W-8a
55+25 W-9 RAILROAD CAUSEWAY
65+25 W-10
76+85 W-11
87+95 W-12
91+98 W-13 RAILROAD BRIDGE
93+06 W-14 HIGHWAY BRIDGE
93+29 W-15 WEIR
NOEL'S POND
CHAINAGE FROM CENTRE LINE, BLANCHE BROOK (IN FEET)
GEODETIC ELEVATION (IN FEET)

FIGURE C-12

4.3 Delineations of Flood Hazard Areas

The areal extent of existing flooding was determined by applying the water surface elevations derived for the Mean Annual and 1:100 year recurrence interval flood to the topographic information available. It must be pointed out that becuase of the limitations of existing mapping in terms of topographic detail (5 foot contour interval), the areal extent of flooding can only be defined within ±2.5 feet in the vertical. This accuracy was, however, improved substantially when the ground survey information was utilized and it is felt that the accuracy was improved to ±1 foot. Despite this fact, it was extremely difficult in some instances to descern appreciable differences between the Mean Annual and 1:100 year floods in terms of areal extent. The key element to consider when comparing the actual hazard of a particular flood situation is the stage which will reflect in the depth of flooding and in some instances the velocity of flood waters. In the final analysis, the following three factors were considered in defining the flood regime; (1) The areal extent of flooding, (2) The depth of the flood waters and (3) The velocity of flood waters.

Certain assumptions were necessary in defining the flood hazard areas which are basic to all three factors above. The following is a listing of the major assumptions with a brief discussion on each:

(1) It was assumed in this study that the fill is a temporary measure to combat flooding and as such would not provide protection from even the mean annual flood.

(2) As a general rule, it was assumed that those bridges with piers would become fully blocked under extreme floods while bridges with clear spans would be partially (50%) blocked.

(3) In certain areas, land may be flooded by waters which are diverted from the river channel upstream and which may not return to the river channel for a considerable distance downstream. This has been termed overland flooding as compared to overbank flooding. The flood hazard maps as developed differentiate between overbank flooding and overland flooding.

4.3.1 Delineation of the Aerial Extent of Flooding

Water surface elevations under the Mean Annual and 1:100 year recurrence interval flood were extracted from the water surface profiles as presented in paragraph 4.2 and applied to the topographic base map to determine the aerial extent of flooding. Additional topographic information obtained from the field surveys was also used. The shaded areas on Maps 1 and 2 represent the extent to which overbank flooding occurs under Mean Annual and 1:100 year floods respectively. The cross hatched areas represent overland flooding. As indicated in the preceding paragraph, it must be pointed out that in most circumstances the entire area would not carry overland flow and as such this is viewed to be a conservative estimate of the flood hazard.

Because of the limitations with respect to topographic detail and recognizing that design elevations may be more appropriate in terms of floodproofing new construction, water surface elevations are depicted on Maps 1 and 2 at various key locations.

4.3.2 Depth of Flooding

One of the key variables in terms of estimating existing or future flood damage is the depth of flood waters at the site of damageable goods. The relative depths of flooding at various key locations on the flood plain are shown on figure C-13. For example, it is estimated, bearing in mind the

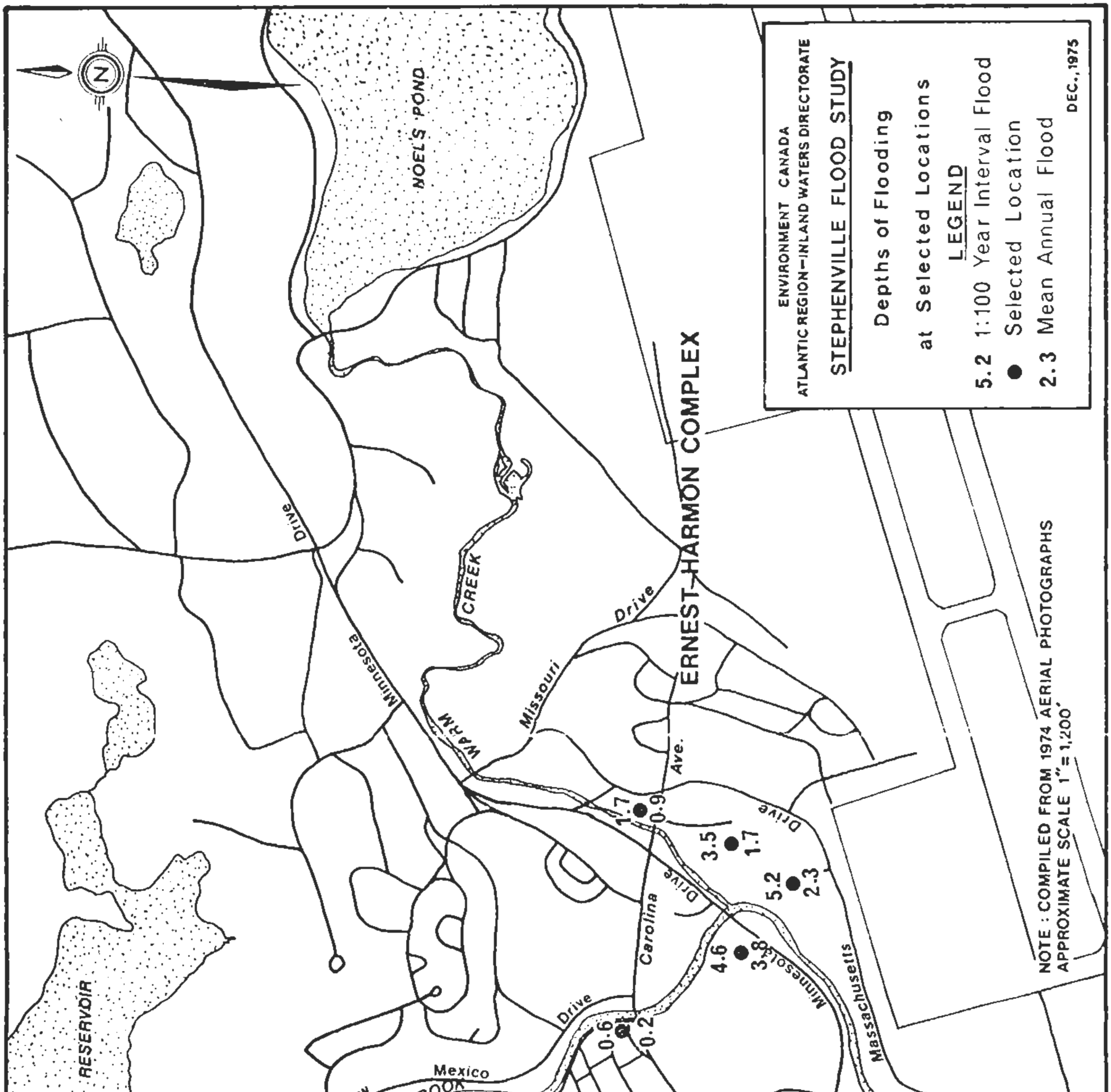

ENVIRONMENT CANADA
ATLANTIC REGION—INLAND WATERS DIRECTORATE
STEPHENVILLE FLOOD STUDY
Depths of Flooding
at Selected Locations
LEGEND
5.2 1:100 Year Interval Flood
● Selected Location
2.3 Mean Annual Flood
DEC., 1975
NOTE : COMPILED FROM 1974 AERIAL PHOTOGRAPHS
APPROXIMATE SCALE 1" = 1,200'
N
NOEL'S POND
ERNEST HARMON COMPLEX
WARM CREEK
Minnesota Drive
Missouri Drive
Ave.
Carolina Drive
Massachusetts Drive
RESERVOIR
New Mexico Drive
BLANCHE BROOK
Queen Street
Hansen Highway
TOWN OF STEPHENVILLE
Main Street
West Street
ST. GEORGE'S BAY
3.0 2.4
3.5 1.7
0.6 0.2
4.6 3.8
5.2 2.3
3.5 1.7
1.7 0.9
3.9 3.3

FIGURE C-13

assumption as stated in paragraph 4.3 above, that the brewery floor would be under five feet and two feet of water for the 1:100 year and Mean Annual floods, respectively.

4.3.3 Velocity of Flood Waters

Another key parameter in terms of damage potential is the velocity of flood waters over the flood plain. Quite logically, velocities will diminish away from the river to a point where stagnant water conditions exist. Any development on the flood plain must take cognizance of this fact. For example, a structure on the flood plain north-east of the confluence bridge located in the grey area close to the river bank may impede flow and cause flooding upstream. In general, the cross hatched areas on Maps 1 and 2 would be relatively low velocity zones with the possible exception of localized areas where the topographic features are riverine in nature. It is anticipated that velocities in the area of the brewery could be relatively damaging under a severe flood.

5. DAM/RESERVOIR ALTERNATIVE

5.1 General

The provision of a dam and its attendant reservoir for the purpose of storing flood waters for later release downstream is a well recognized structural alternative to flood control. If properly designed and operated, this alternative will normally contain all flood waters to the design flood. However, there are many disadvantages associated with this type of alternative which require careful scrutiny. The following is a partial listing of some of the disadvantages: (1) a high initial capital cost, (2) a possibility of failure should a flood greater than the design flood occur, (3) the possibility of higher damages should faliure occur, that is, new development may occur simply because the dam/reservoir provides in the minds of planners/engineers a certain security from future floods and (4) adverse environmental effects in most circumstances.

Of course, there are many advantages to a proposal of this nature which again require careful scrutinizing. These are: (1) protection from floods less than the design flood, (2) the capability for other uses such as water supply, recreation and (3) potential environmental benefits such as low flow augmentation, provision of wildlife (waterfowl) habitat etc.

5.2 Alternative Sites

Based on an analysis of available topographic mapping (NTS, 1:50,000 and 1:15,000 - 10 foot contour interval mapping as prepared by Lockwood, Kessler and Bartleth Inc.) three sites, as shown on Figure C-14, were selected on Blanche Brook for future analysis. No prospective sites were found on Warm Creek or its tributaries. There is little or no opportunity of incorporating flood control storage in Noel's Pond in view of upstream flooding problems and its existing use as a water supply reservoir.

5.2.1 Hydrologic Considerations

Based on a regional analysis of runoff as detailed in Appendix B, Flood Magnitudes, the 1:100 year recurrence interval flood was determined to be approximately 3,600 cfs on Blanche Brook upstream of its confluence with Warm Creek. This has been accepted as the design flood for the purpose of evaluating storage requirements. However, equally important in this evaluation is the timing of the flood, its antecedent conditions, its duration, etc. It was therefore essential to develope a flood hydrograph for the 1:100 year or design event. Shawmont Newfoundland Limited in their study of a Water Supply System for the Stephenville Industrial Area developed

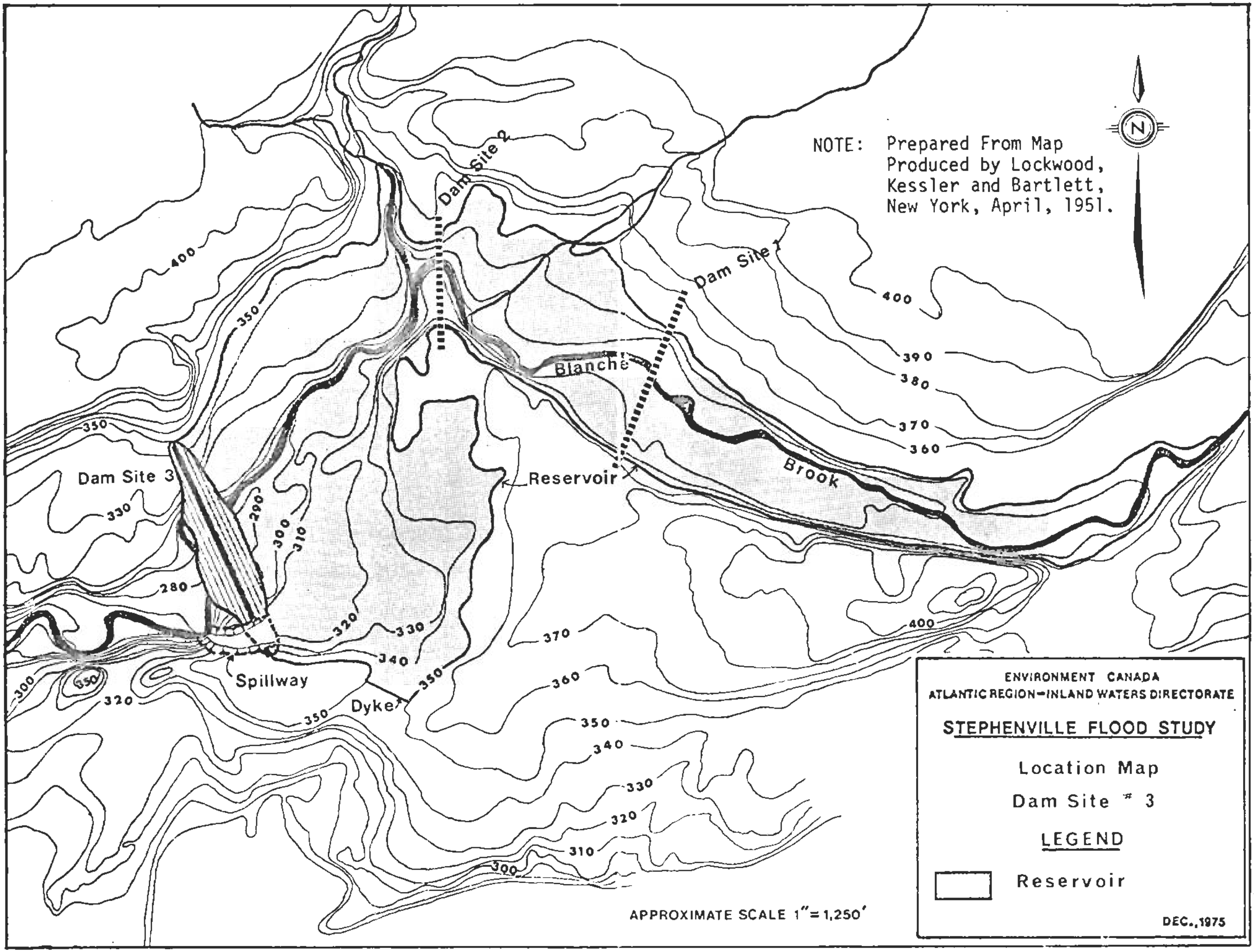

NOTE: Prepared From Map Produced by Lockwood, Kessler and Bartlett, New York, April, 1951.
N
Dam Site 1
Dam Site 2
Dam Site 3
Blanche
Brook
Reservoir
Spillway
Dyke
400
390
380
370
360
350
340
330
320
310
300
290
280
ENVIRONMENT CANADA
ATLANTIC REGION—INLAND WATERS DIRECTORATE
STEPHENVILLE FLOOD STUDY
Location Map
Dam Site # 3
LEGEND
Reservoir
APPROXIMATE SCALE 1"=1,250'
DEC., 1975

FIGURE C-14

a flood hydrograph for 1:10,000 and 1:100 recurrence interval floods utilizing the unit hydrograph technique. The flood hydrographs were determined for Warm Creek at Noel's Pond on the basis of an analysis of the worst series of consecutive six-hour rain fall periods and a base flow of 250 cfs. The 1:100 year recurrence interval flood hydrograph was determined from the 1:10,000 year recurrence interval flood hydrograph by applying an arbitrary factor of .75. The resultant 1:100 year hydrograph is reproduced here in Figure C-15.

As can be seen, the peak flow developed for Warm Creek by the regional analysis technique is about 23% lower than that determined by Shawmont for nearly the same drainage area. This discrepancy may be explained by several factors, the most notable of which is the fact that the natural and artificial storage on the Warm Creek system was taken into account in the regional analysis technique. There is no indication that storage was in Shawmont's study. In order to ensure consistency, the ordinates of the 1:100 year flood hydrograph was adjusted such that the maximum instantaneous discharge corresponded to approximately 3,600 cfs as determined by regional analysis. The resultant hydrograph for Blanche Brook at the confluence is shown on Figure C-16. The hydrograph was then transported to each site by adjusting the ordinates in relation to drainage area.

It is recognized that the methods adopted herein are relatively approximate. However, they are judged to be within the realm of accuracy required for conceptual design. Because of data limitations, there is little opportunity to improve upon these estimates in this part of Newfoundland. As the period of record for the Harrys' River matures, there will be an opportunity to improve these estimates substantially.

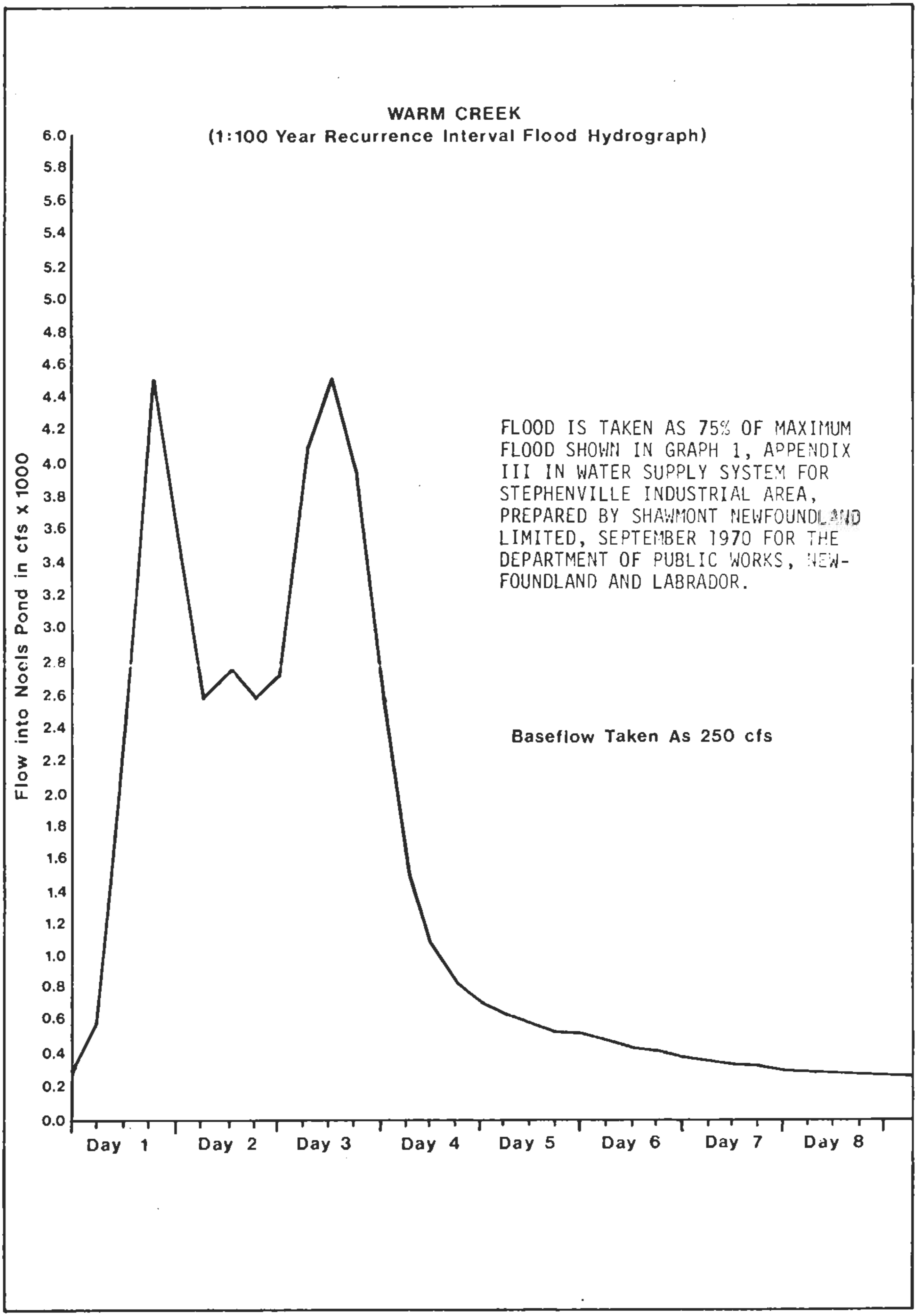

WARM CREEK
(1:100 Year Recurrence Interval Flood Hydrograph)
Flow into Noels Pond in cfs x 1000
6.0
5.8
5.6
5.4
5.2
5.0
4.8
4.6
4.4
4.2
4.0
3.8
3.6
3.4
3.2
3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Day 1
Day 2
Day 3
Day 4
Day 5
Day 6
Day 7
Day 8
FLOOD IS TAKEN AS 75% OF MAXIMUM FLOOD SHOWN IN GRAPH 1, APPENDIX III IN WATER SUPPLY SYSTEM FOR STEPHENVILLE INDUSTRIAL AREA, PREPARED BY SHAWMONT NEWFOUNDLAND LIMITED, SEPTEMBER 1970 FOR THE DEPARTMENT OF PUBLIC WORKS, NEWFOUNDLAND AND LABRADOR.
Baseflow Taken As 250 cfs

FIGURE C-15

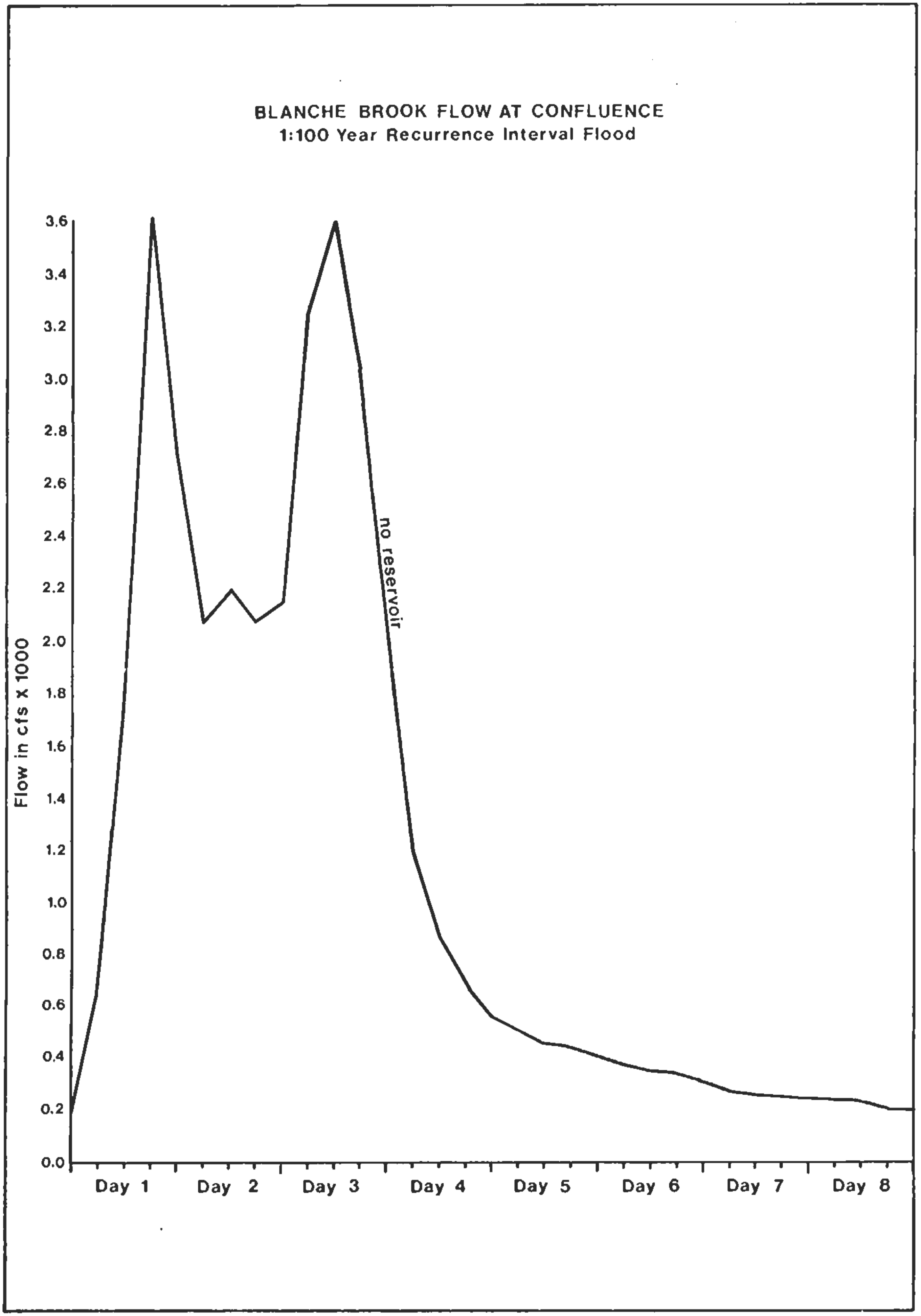
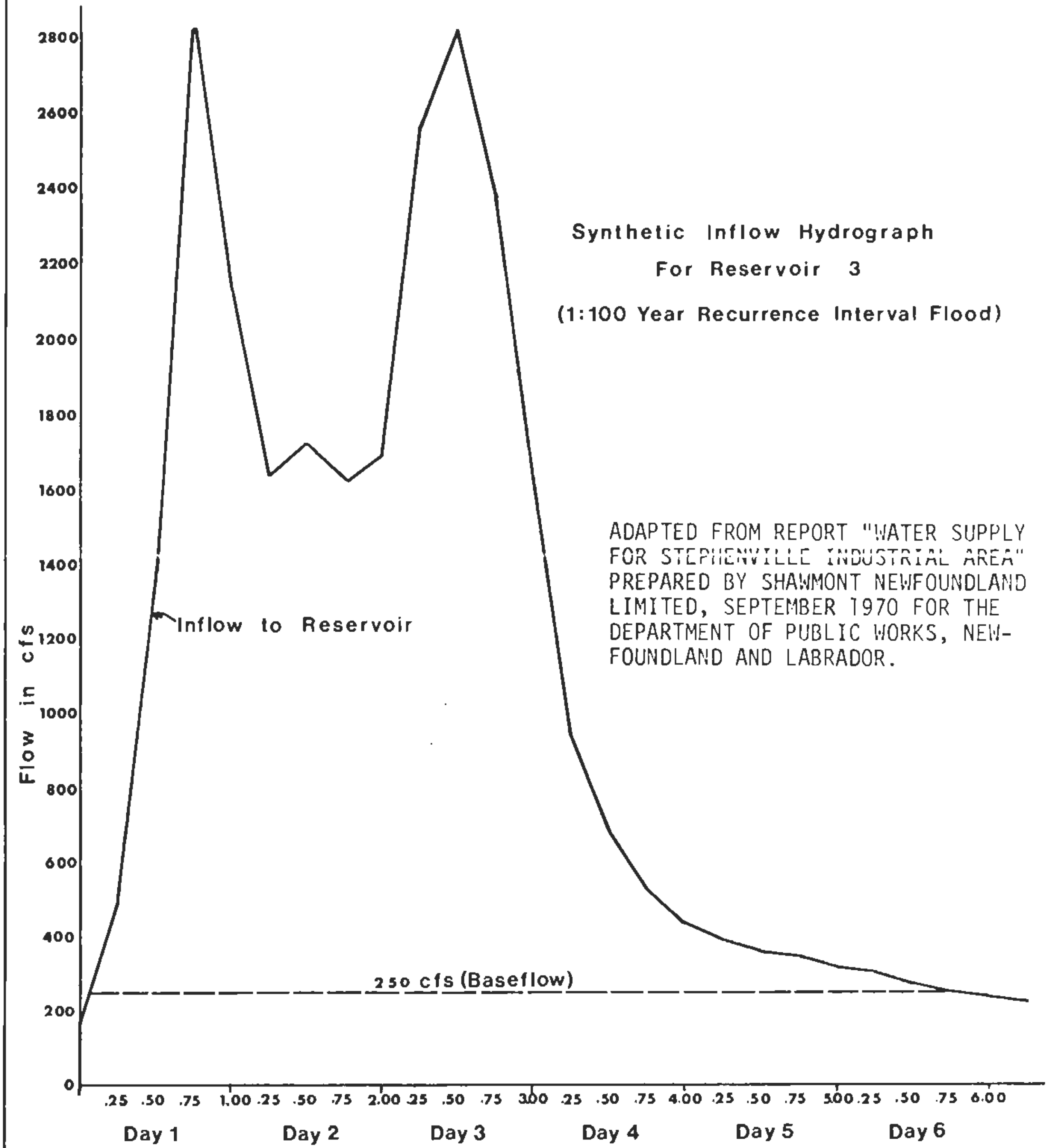

BLANCHE BROOK FLOW AT CONFLUENCE
1:100 Year Recurrence Interval Flood
Flow in cfs X 1000
3.6
3.4
3.2
3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
no reservoir
Day 1
Day 2
Day 3
Day 4
Day 5
Day 6
Day 7
Day 8

FIGURE C-16

5.2.2 Hydrologic Routing

The 1:100 year recurrence interval flood hydrograph as outlined in section 5.2.1 was adjusted to represent the inflow to the particular reservoir under review. An example of the resultant inflow hydrograph to reservoir 3 is shown on Figure C-17. The resultant outflow from the reservoir was then combined with the local inflow to Blanche Brook, downstream of the dam, to provide an estimate of the resultant flow in Blanche Brook above its confluence with Warm Creek. Downstream of the confluence, the partially regulated flow in Blanche Brook was combined with the uncontrolled flow in Warm Creek to provide an estimate of the river flow in the reach below. An analysis of the storage potential of each of the reservoirs indicated that reservoir 3 would provide substantially more storage than either of the others. The stage-storage relationship at this reservoir is shown on Figure C-18. An analysis of the inflow hydrograph, Figure C-17, reveals that the total storage required to completely regulate river flow at this location, relative to a base flow of 250 cfs, would be 12,000 acre-feet. In view of the storage capacity of the site, 8,700 acre-feet, the regulated outflow would have to be approximately 700 cfs which when combined with the local inflow between the dam site and the confluence of Blanche Brook and Warm Creek represents a flow of about 1,800 cfs in that reach of the river, or about the mean annual flood. The flow downstream of the confluence would be approximately 4,300 cfs, or about the 10 year recurrence interval flood in that section of the river. The water level at the brewery would be about elevation 22.0 feet or about 2 feet lower than the 1:100 year recurrence interval flood as determined in Section 4.2.

Assuming that upstream storage sites provided the additional 3,300 acre-feet required to completely regulate flows between reservoir 3, the flood



Synthetic Inflow Hydrograph
For Reservoir 3
(1:100 Year Recurrence Interval Flood)
ADAPTED FROM REPORT "WATER SUPPLY FOR STEPHENVILLE INDUSTRIAL AREA" PREPARED BY SHAWMONT NEWFOUNDLAND LIMITED, SEPTEMBER 1970 FOR THE DEPARTMENT OF PUBLIC WORKS, NEWFOUNDLAND AND LABRADOR.
Inflow to Reservoir
250 cfs (Baseflow)
Flow in cfs
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
0
.25 .50 .75 1.00 .25 .50 .75 2.00 .25 .50 .75 3.00 .25 .50 .75 4.00 .25 .50 .75 5.00 .25 .50 .75 6.00
Day 1
Day 2
Day 3
Day 4
Day 5
Day 6

FIGURE C-17

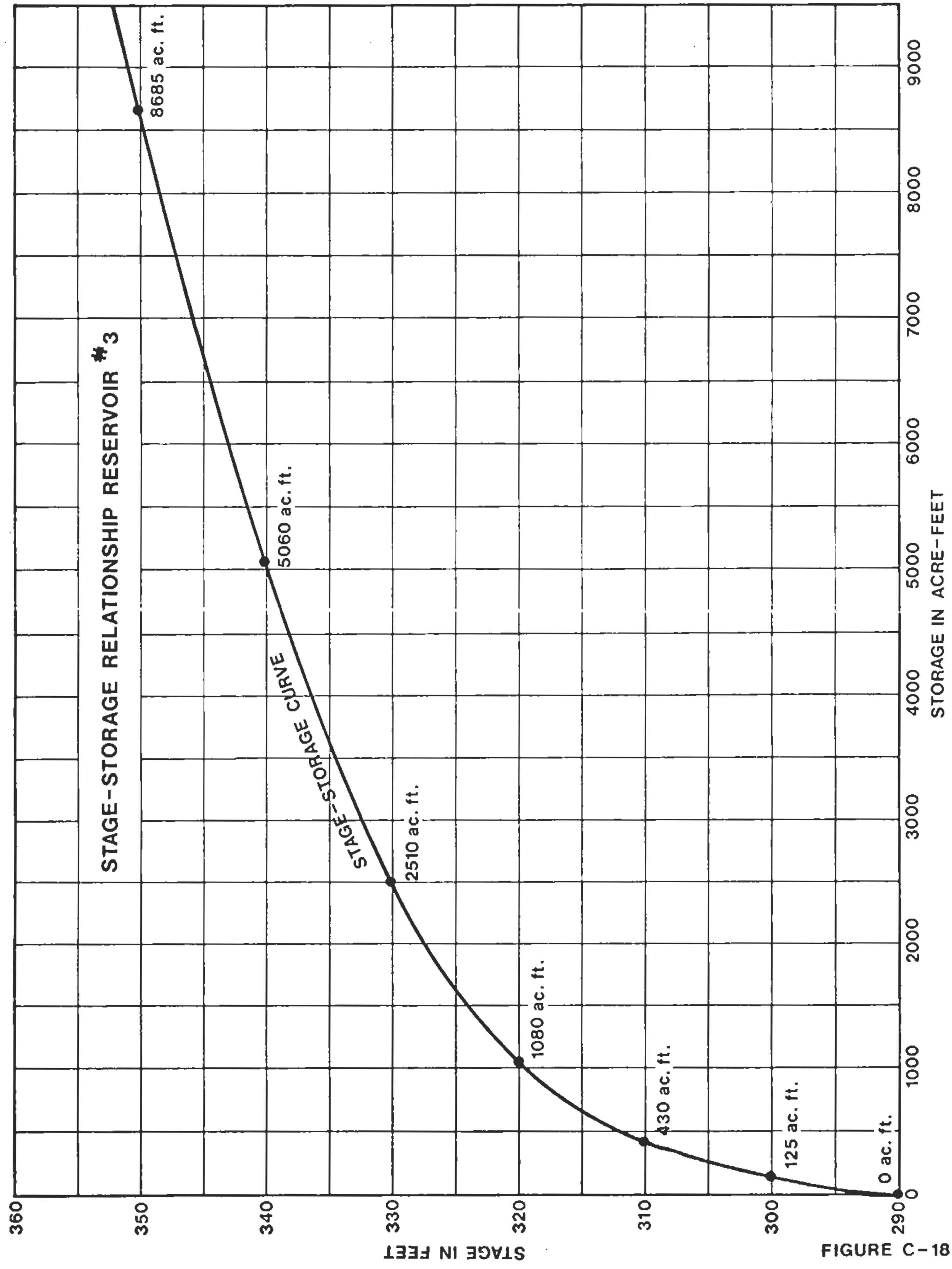

STAGE-STORAGE RELATIONSHIP RESERVOIR #3
STAGE-STORAGE CURVE
8685 ac. ft.
5060 ac. ft.
2510 ac. ft.
1080 ac. ft.
430 ac. ft.
125 ac. ft.
0 ac. ft.
STORAGE IN ACRE-FEET
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
STAGE IN FEET
290
300
310
320
330
340
350
360

FIGURE C-18

flows in Blanche Brook upstream of the confluence would be reduced to about 1,100 cfs thus virtually eliminating flooding in that part of the river. However, because Warm Creek would not be controlled, the flood flows at the brewery would still be as high as 3500 cfs which represents an elevation of about 21.5 feet at the brewery, or about 2.5 feet lower than the 1:100 year recurrence interval flood elevation as determined in Section 4.2, or about 0.5 foot higher than mean annual flood elevation.

Based on the foregoing, it was concluded that some protection would be achieved by a storage development and that a preliminary cost estimate of such works was warranted to put this alternative in perspective. If reservoir site 3 looked promising, it would be necessary to evaluate other sites in the headwaters of Blanche Brook for additional storage.

5.3 Conceptual Design

5.3.1 Description of the Site

Reservoir 3, as denoted in this report, is strategically located below the confluence of four major tributaries to Blanche Brook. At that location, approximately 19 mi^2 or 79% of the Blanche Brook watershed would be controlled by the proposed dam/reservoir. A profile along the axis of the dam is shown in Figure C-19. To provide maximum storage at the site, the crest elevation was taken at 350 feet. The maximum height of the dam would be 70 feet, and the total overall length, including the small dyke section, would be 3,800 feet.

No detailed information was available or obtained with regard to the surficial and bedrock geology of the area. Published geologic maps of the area indicate that the bedrock is overlain with a thick deposit of

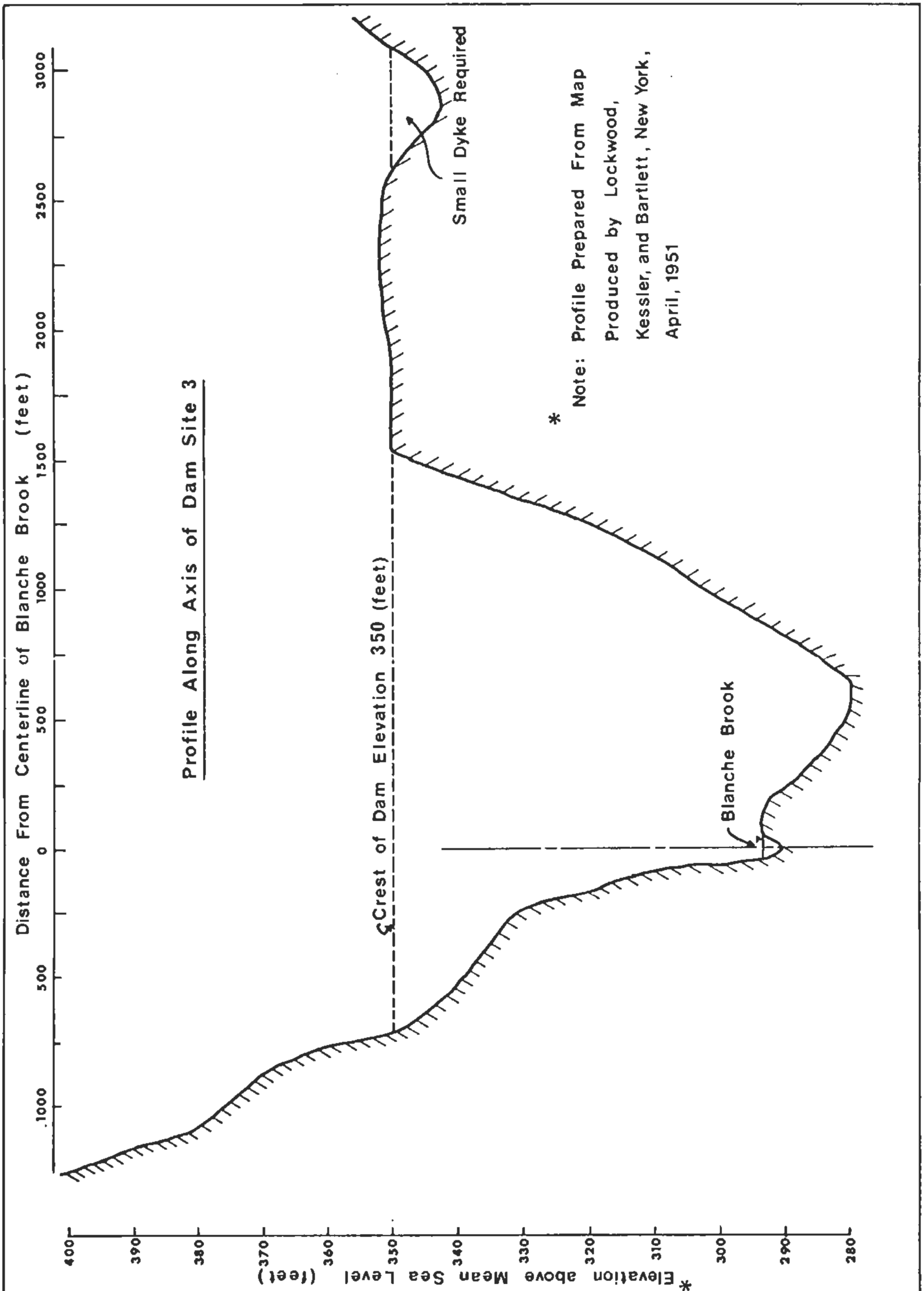

Profile Along Axis of Dam Site 3
Distance From Centerline of Blanche Brook (feet)
3000
2500
2000
1500
1000
500
0
500
1000
*Elevation above Mean Sea Level (feet)
400
390
380
370
360
350
340
330
320
310
300
290
280
Crest of Dam Elevation 350 (feet)
Small Dyke Required
Blanche Brook
* Note: Profile Prepared From Map
Produced by Lockwood,
Kessler, and Bartlett, New York,
April, 1951

FIGURE C-19

glacial drift consisting of minor gravel, sand, silt and clay. The bedrock geology has not been classified although it would appear, in view of outcrops upstream and downstream of the site, to be a grey limestone with minor shale of lower and middle Ordovician age. There are indications that a possible fault exists near the proposed dam site. In view of the foregoing, any further design work should not be carried out until more information as to the geology of the area is determined.

5.3.2 Foundation

In view of the apparent geology of the site, it was assumed that the foundation consists of a deep relatively pervious layer of glacial drift. Because of this, a partial cut-off trench supplemented by an upstream blanket of impervious material was adopted to reduce and/or control seepage. An impervious downstream embankment zone was incorporated into the design to prevent and relieve uplift. It was assumed that the foundation would be stable and that settling problems would be minor.

5.3.3 Embankment

In view of the limited quantity of impervious material available, a zoned embankment dam was selected. A cross-section is shown in Figure C-20. The basic core adopted consists of silt, clay and marine sand compacted in layers, and ranging from 10 feet at the top to about 100 feet at the base at the deepest point. The side slopes of the impervious core would be 1:1. Both upstream and downstream sides would be comprised of local materials consisting of gravels, sands, clays and silt and are judged to be relatively pervious. The core material may have to be obtained from the Port au Port area, 20 miles away. The upstream slope was selected at 3:1 while the downstream slope was

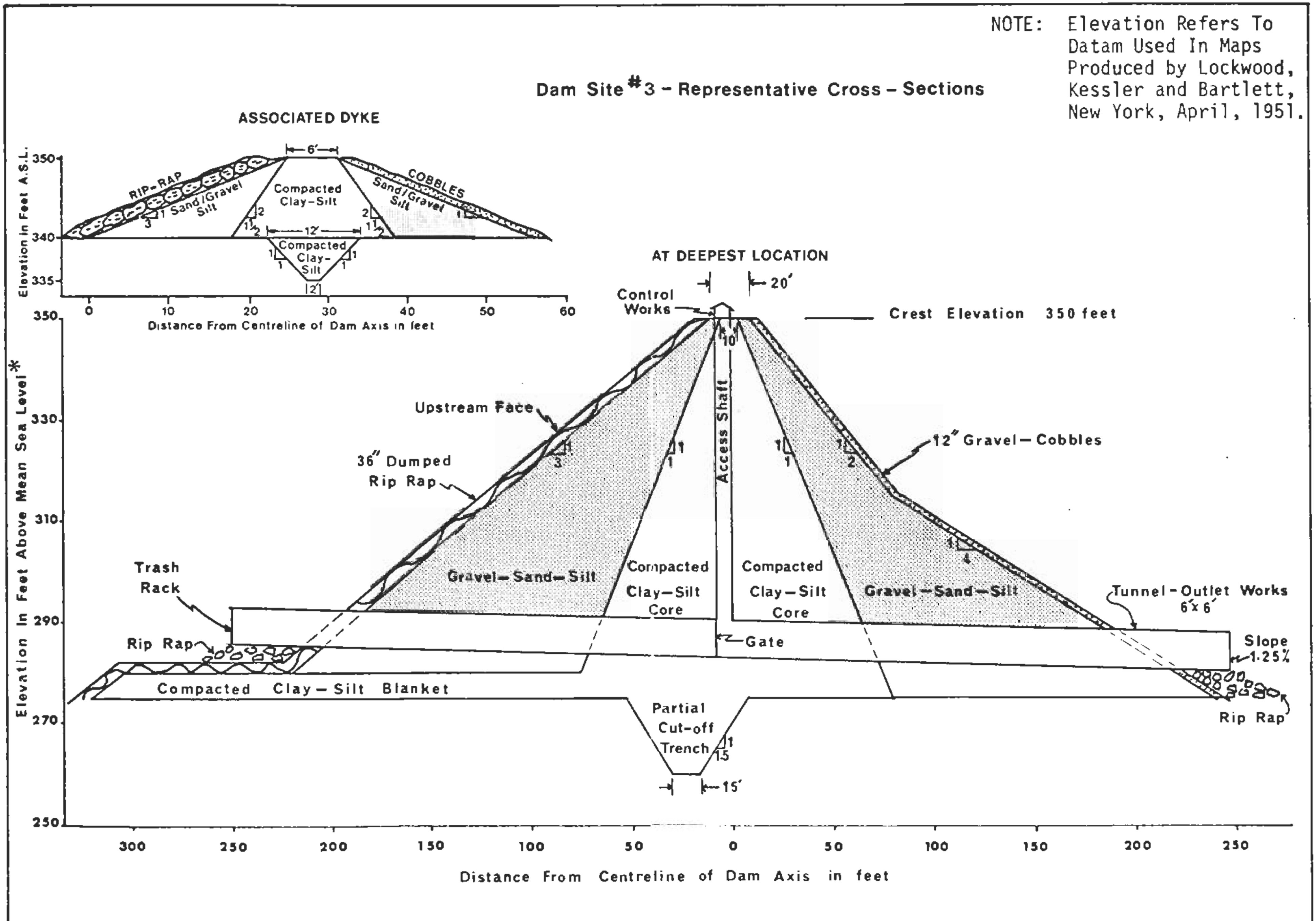

NOTE: Elevation Refers To Datam Used In Maps Produced by Lockwood, Kessler and Bartlett, New York, April, 1951.
Dam Site #3 - Representative Cross - Sections
ASSOCIATED DYKE
RIP-RAP
Sand/Gravel Silt
Compacted Clay-Silt
COBBLES
Sand/Gravel Silt
Compacted Clay-Silt
Elevation in Feet A.S.L.
Distance From Centreline of Dam Axis in feet
AT DEEPEST LOCATION
Control Works
Crest Elevation 350 feet
Access Shaft
Upstream Face
36" Dumped Rip Rap
12" Gravel - Cobbles
Gravel - Sand - Silt
Compacted Clay - Silt Core
Compacted Clay - Silt Core
Gravel - Sand - Silt
Trash Rack
Rip Rap
Gate
Tunnel - Outlet Works 6' x 6'
Slope 1.25%
Rip Rap
Compacted Clay - Silt Blanket
Partial Cut-off Trench
Elevation In Feet Above Mean Sea Level*
Distance From Centreline of Dam Axis in feet

FIGURE C-20

selected at 2:1 gradually increasing to 4:1 at the heel. The crest width was taken to be 20 feet. The upstream slope could be protected against wave action by the use of one to three feet quarried rock from the Stephenville area. The rock would be placed by dumping and would be about three feet thick. A one-foot bed of natural gravel was selected as a filter between the rip-rap and the fill. The downstream slope should be protected against wind and rain fall actions by a 24 inch layer of cobbles.

To complete the closure of the reservoir at elevation 350, a small dyke would be required measuring 500 feet long (approximately) 10 feet high and sloped 3:1 on the upstream face and 2:1 on the downstream face. It would be of the zone embankment type using clay for the impervious core and key. A cross-section is shown in Figure C-20.

5.3.4 Spillway

The design criterion for the spillway of passing a 1:10,000 year flood was adopted. An estimated flow of 5,900 cfs, based on the investigations done by Shawmont Newfoundland Limited, was selected. Using the relationship: $Q = CLH^{3/2}$ and selecting a C value of 3.78, and a head of 3 feet over the crest, a wier length of 300 feet was calculated.

The layout for the spillway is shown on Figure C-14. Based on foundation and cost considerations, it was decided to locate the spillway structure near the right embankment. The sill elevation was selected at el. 346 feet. The spillway would be constructed of reinforced concrete, 1 foot thick, with side walls four feet in height. An anchor wall would be constructed at the entrance to the spillway, extending down five feet, to provide stability against sliding. The downchute, a continuation of the spillway through the dam, extends from elevation 346 to elevation 314 down the slope face of the dam. Its length is 72 feet, and the fall is 32 feet. The side walls vary from 4 feet to 3 feet in height. A cross-sectional view of the spillway is shown on Figure C-21.

At elevation 314, a horizontal stilling basin[3] was inserted to dissipate some of the flow energy. Based on this method, the dimensions of the stilling basin were 300 feet wide and 28 feet long. There would be 300 chute blocks, 6 inches wide, spaced at 6 inch centers, and 160 dentated sill blocks. A sketch is shown on Figure C-22.

5.3.5 Outlet Works

The outlet structure would consist of a gated tunnel located near the lowest part of the dam, as shown on Figure C-20. An access shaft would be incorporated to house the gate mechanism. The gate would be designed to pass a flow of 1,000 cfs under a head of 15 feet to allow drawdown to occur rapidly in anticipation of a flood peak. This represents, considering lateral inflow between the dam site and the town, a safe discharge under flood conditions.

The hydraulic design of the outlet works was based on the following relationship:[4]

$$Q = A\sqrt{2g}\left[\frac{H_u + L_t \sin\theta - \frac{D_t}{2}}{1 + K_e + \frac{29.1\, h^2 L_t}{Rt^{4/3}}}\right]^{\frac{1}{2}} \qquad \ldots\ldots\ldots(10)$$

The slope was arbitarily selected at .0125, due to site conditions, the entrance loss coefficient taken at .05, n at .015 and the length at 500 feet and using these data and assumptions the discharge was calculated to be 1,260

3. Hydraulic Design of Stilling Basins and Energy Dissipators, United States Department of the Interior, Bureau of Reclamation, Monograph No. 25, Type 2.

4. Design of Small Dams, United States Department of the Interior, Bureau of Reclamation.

TYPICAL CROSS-SECTION OF SPILLWAY

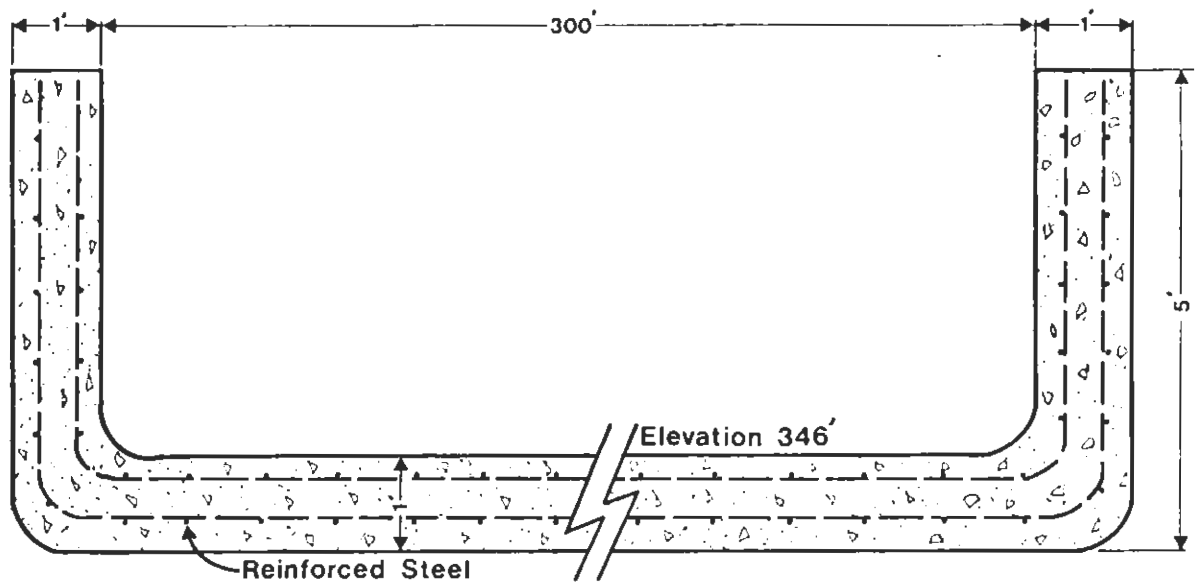


FIGURE C-21

STILLING BASIN

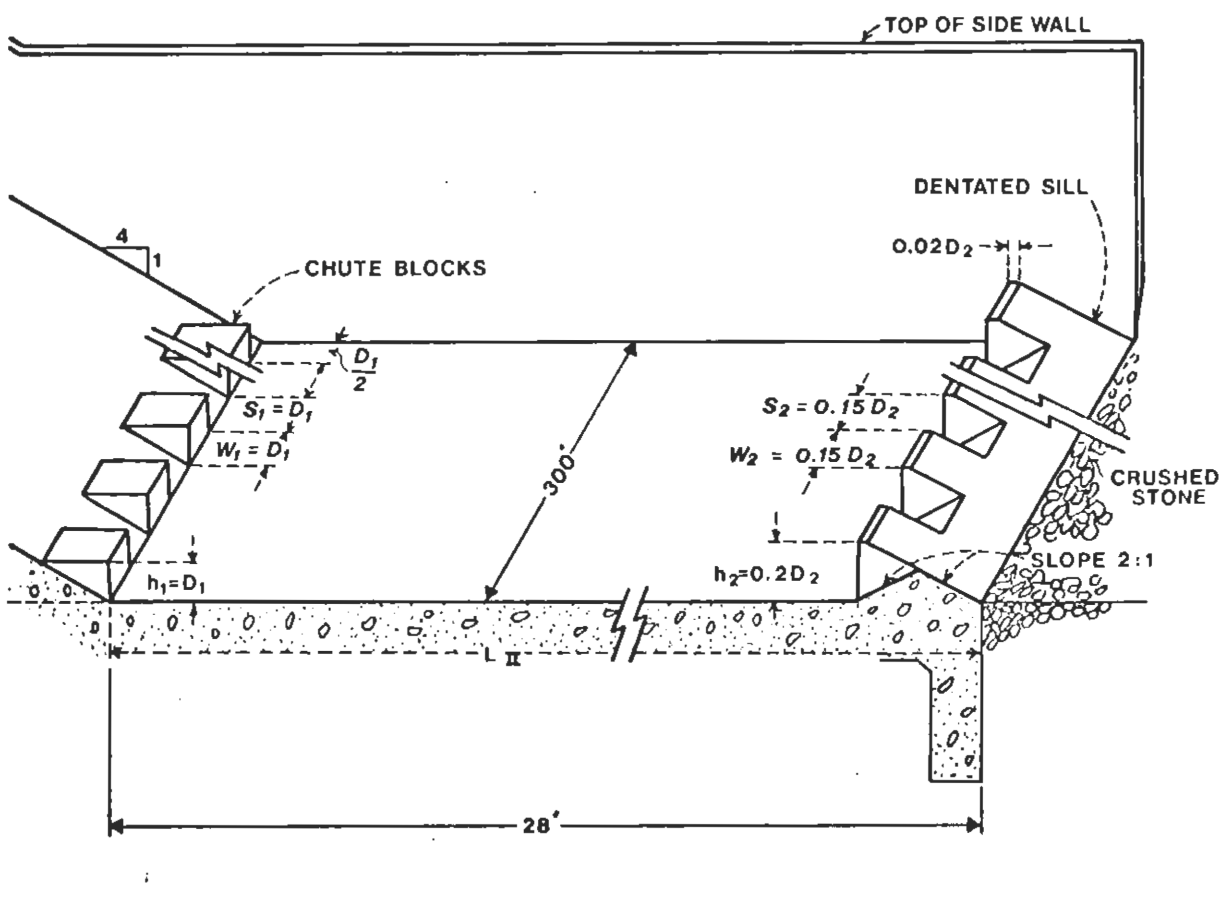


FIGURE C-22

cfs at a cross-section area of 36 feet, or a tunnel measuring 6 feet by 6 feet , inside measurements.

It is assumed the box culvert is flowing full and control is at the outlet.

The structure would be made from reinforced concrete and all walls 1 foot thick. It would be laid on a filter consisting of pervious gravel or crushed stone to prevent uplift forces from acting on it. Pressure relief would be afforded lengthwise through the porous material.

5.3.6 Intake Structure

The entrance to the outlet works should be rounded or bell mouthed to reduce hydraulic losses. If a trash rack should be installed to prevent ice and debris from interferring with the control mechanism, then considera-tion should be given to minimizing hydraulic losses.

5.3.7 Terminal Structures and Dissipating Devices

The discharge from the outlet structure will issue at a high velocity usually in a nearly horizontal direction. Deflectors might be installed to direct the high velocity flow away from the outlet structure and pass the downstream toe of the dam if erosion-resistant bedrock is at shallow depths in the downstream channel. If softer foundations are prevalent, then a dissipating device might be provided to absorb the energy of flow before it is returned to the brook.

5.3.8 Outlet Channel

An outlet channel is required to convey discharges from the end of the outlet works to the brook or watercourse downstream. The channel should be excavated to stable slopes and to dimensions of size to prevent scour. Rip-rap may be required for this type of protection.

5.3.9 Control & Access Shafts

The size of the access shaft should be of only sufficient size in dry well installations to provide operating room at the bottom of the shaft. A 6 feet by 6 feet reinforced concrete manhole type of access shaft is shown on the drawing. A smaller one, either directly above or offset from the chamber, and just large enough to permit passage of removable and replacable gate parts could be considered in final design work.

Selection of outlet work components should be based on the use of commercially available gates or valves of relatively simple design rather than on the use of special devices which will involve expensive design and fabrication costs. Cast iron slide gates, used for control gates, are available for both rectangular and circular openings for design hydraulic heads in excess of 50 feet.

A housing around the outlet works is sometimes installed when operating equipment would otherwise be exposed, or adverse weather conditions would prevail during operating periods. They are usually made large enough to accommodate auxiliary equipment such as ventilating fans, heaters, air pumps, and small power-generating sets.

5.4 Quantities and Cost Esitmates

The total length of the dam was divided into eleven panels along its axis. The mean depth of each length was then determined from a crest elevation of 350 above sea level. After these parameters were established, the average area or average volume per lineal foot was then calculated.

The volume for each sectional length was calculated by multiplying the volume per lineal foot by the length of the section. The resulting volumes were added to give the total volume of material for each component, as shown in Table C-3.

An estimate shown in Table C-4 is the total first cost of the dam/associated dyke/flood control reservoir system for a storage capacity of 8,700 acre-feet. These estimates were increased by 20% for contingencies, and a further 10% to allow for final design, construction, supervision, administration, etc. The unit costs were derived from various sources, including contractors and consulting engineers engaged in business in the western area of Newfoundland, and other areas of the Atlantic Region. The total first cost was estimated at $8.1 million. Obviously, these costs need to be revised periodically, depending on changes in the economic situation.

Clay is a major item in the total estimate of the reservoir system. For purposes of information and clarification, the components of the unit cost are presented. The estimated cost of purchasing, loading and hauling it 20 miles to the dam site, spreading and compacting this material, is $8.50 per cubic yard.

TABLE C-3

Estimated Volumes of Earth Materials Required for Dam

Top Elevation at 350'

MATERIAL	COMPONENT	VOLUME (CU. YD.)
Compacted clay-silt	partial cut off trench	48,015
Compacted clay-silt	core	229,730
Compacted clay-silt	upstream blanket	79,608
Gravel/sand/silt fill	upper slope	158,412
	down slope	144,744
Rock rip-rap	upper side	56,506
Gravel-cobbles	down side	11,681
	clearing & stripping	161,653
	common excavate	48,015

TABLE C-4

SUMMARY OF FIRST COST

DAM SITE 3 - BLANCHE BROOK

Item	Quantity*	Unit	Unit Cost* ($)	Amount ($)
Earth Fill Dam				
Clearing and Stripping	173,500	yd^3	3.00	520,000
Excavation	38,400	yd^3	3.00	115,200
Compacted Clay/Silt	361,000	yd^3	8.50	3,068,500
Gravel/Sand/Silt fill (Compacted)	307,000	yd^3	4.50	1,381,500
Rip-rap	58,500	yd^3	5.00	292,500
Gravel, Cobbles	12,200	yd^3	4.75	58,000
Reservoir clearing	420	acres	600.00	252,000
Reinforced Concrete				
(a) spillway	1,750	yd^3	185.00	323,800
(b) outlet works	550	yd^3	185.00	101,800
Gate and appurtenances	1	unit	12,000.00	12,000
Sub Total				6,125,800
Contingencies at 20%	-	job	-	1,225,200
Sub Total	-	job	-	7,351,000
Engineering Design, Supervision and Administration at 10%	-	job	-	735,100
TOTAL FIRST COST				$8,086,100

* All quantities and unit costs are for materials in place.

6. DYKES

6.1 General

The purpose of a dyke is to prevent flood water of a watercourse from overflowing onto plains which are considered to be of some economic and social importance. Any proposed dyke constructed along the banks of a watercourse must withstand forces of moving ice and water velocities, and erosion caused by wind, wave and rainfall actions. Specifically strong forces caused by moving water are in evidence in the area of the brewery near the confluence of Blanche Brook and Warm Creek. This area is of relatively high economic importance and remedial measures are needed for its protection. It is worthy to note that the original course of Blanche Brook passed through the area of the brewery on its way to the sea. It is necessary to construct a drainage system as well to collect seepage through and underneath the dyke and any local run-off that would be impounded by the proposed dyke. A dyke protected must also be properly maintained, because some deterioration must be expected over a period of time. Two basic dyke systems were proposed and evaluated; one is for the protection of LeBatt's brewery and Humbers Motors, and the other is to protect residences west of Brookside Drive along Blanche Brook.

6.2 Hydraulic Design Criteria

The top of the proposed dyke system; (1) the LeBatt's Brewery - Humber Motors dyke, and (2) the Brookside Drive, is two feet above the estimated water surface level under 1:100 year flood conditions. Two feet of freeboard is considered adequate since wave action will be at a minimum. It is estimated that the maximum velocity of flow normal to the proposed dyke in the vicinity of the brewery would be 10-12 feet per second (two times the mean velocity). This is a critical section of the dyke and must be adequately protected. Downstream from the brewery, the water velocity is parallel to the sides of the dyke and is approximately equal to the average velocity through any specified section, based on literature reviews. A similar situation exists most of the way upstream of the confluence on Warm Creek to the end of the dyke in the vicinity of Humber Motors. The flow in Blanche Brook, adjacent to Brookside Drive, is not parallel to its banks. There are some bends in the watercourse causing changes of direction in flow, resulting in expenditure of kinetic energy. Hence the upstream faces of the dyke in these areas must be adequately protected against erosion.

The functions of a drainage ditch are: (1) to provide structural stability for the dyke by preventing its constituents from becoming saturated and structurally unstable, and (2) to collect seepage and local run-off and transport the collected water away from the critical areas.

6.3 Construction Methods and Materials

The dyke embankment and Brookside Drive fill should be compacted in six-inch layers by use of a roller type (sheeps foot) compactor to ensure stability and tightness. Compaction would extend from the base of the dyke key to (at least) the high water mark. Rip-rap would be placed on the stream side of the dyke by truck dumping. However, armour stone, due to its weight, should be placed by crane. No armour stone is proposed for the upstream face of Brookside Drive.

Construction should be done during summer to minimize costs, when in all probability, the water levels are at a minimum.

The availability of construction materials in the immediate area has the obvious effect of moderating construction prices; whereas materials such as clay, located, for example, 8 miles from the site has the effect of increasing costs due to haulage.

The consensus is that the nearest source of sufficient quantities of suitable clay materials is at Port-au-Port, 8 miles away. There are ample supplies of sand, gravel, hard rock for rip-rap, and armour stone, and ready-mix concrete in the immediate area.

Either a clay core, or the adoption of the existing material at the immediate site as core material was proposed for the LeBatt's Brewery and Humber Motors dyke system. For the upgrading of Brookside Drive it was proposed to use the existing material at the site.

6.4 Unit Costs

Most of the cost data for the derivation of the unit price of clay in place (core and key), for example; purchasing, loading, hauling (8 miles), spreading and compacting were obtained from the region (telephone conversation with professional engineers of Lundrigan's Ltd, Corner Brook) The unit prices for sand, gravel, and rip-rap in place were derived from several sources, notably, Whitman and Benn[5] and Shawmont Newfoundland Limited[6], and adjusted to reflect current price levels. The unit price for armour stone, proposed as bank protection for the confluence area, was derived with the assistance of the Department of Public Works of Canada. Further factors of 20% and 10% were applied in succession to cover indirect cost: project management, final design, interest during construction, etc.

6.5 Conceptual Design

6.5.1 LeBatt's Brewery - Humber Motors

Two basic types of design were envisaged for the LeBatt's Brewery and Humber Motors dyke, namely: (1) a homogenous embankment type using

5. Whitman and Benn Ltd., Sackville River Study for the N.S. Department of the Environment, dated 1974.

6. Shawmont Newfoundland Limited, Water Supply System for Stephenville Industrial Area, dated September, 1970.

existing material (exclusive of slope protection) in the immediate area for its construction, and (2) a zone embankment dyke, with clay as an impervious core and flanked by local materials. See figure C-23 for a profile. and figure C-24 for a plan view and some cross-sections of the clay core and key dyke system.

For a homogenous type of dyke, the material comprising it must be sufficiently impervious to provide an adequate water barrier, and the slopes relatively flat for stability to prevent sloughing. However, the downstream face will be affected by seepage approximately equal to 1/3 the height of the water on the upstream face if maintained for a sufficiently long time. regardless of slope. Dykes controlling short duration flood peaks rarely become saturated enough to represent a hazardous condition. No drawings were made of the homogenous type of dyke system. This type would not have a key, and the side slopes could be made less or flatter. However, for cost analysis and comparisons, as discussed in section 6.6, the same dimensions (above ground) were used for both types of dykes.

6.5.2 Brookside Drive

Because of the relatively low flood damage potential in sub-area 3, expensive and extensive flood control measures are not justified. However, Brookside Drive can be considered to function as a dyke if upgraded and extended. The road could provide protection against flooding, except a small area just upstream of the Main Street Bridge. This small area would permit release of flood waters in the event Main Street Bridge became blocked with ice, thus reducing pressure on the structure. It is suggested that all openings through the road bed, storm drains, culverts, be blocked to prevent back-up.

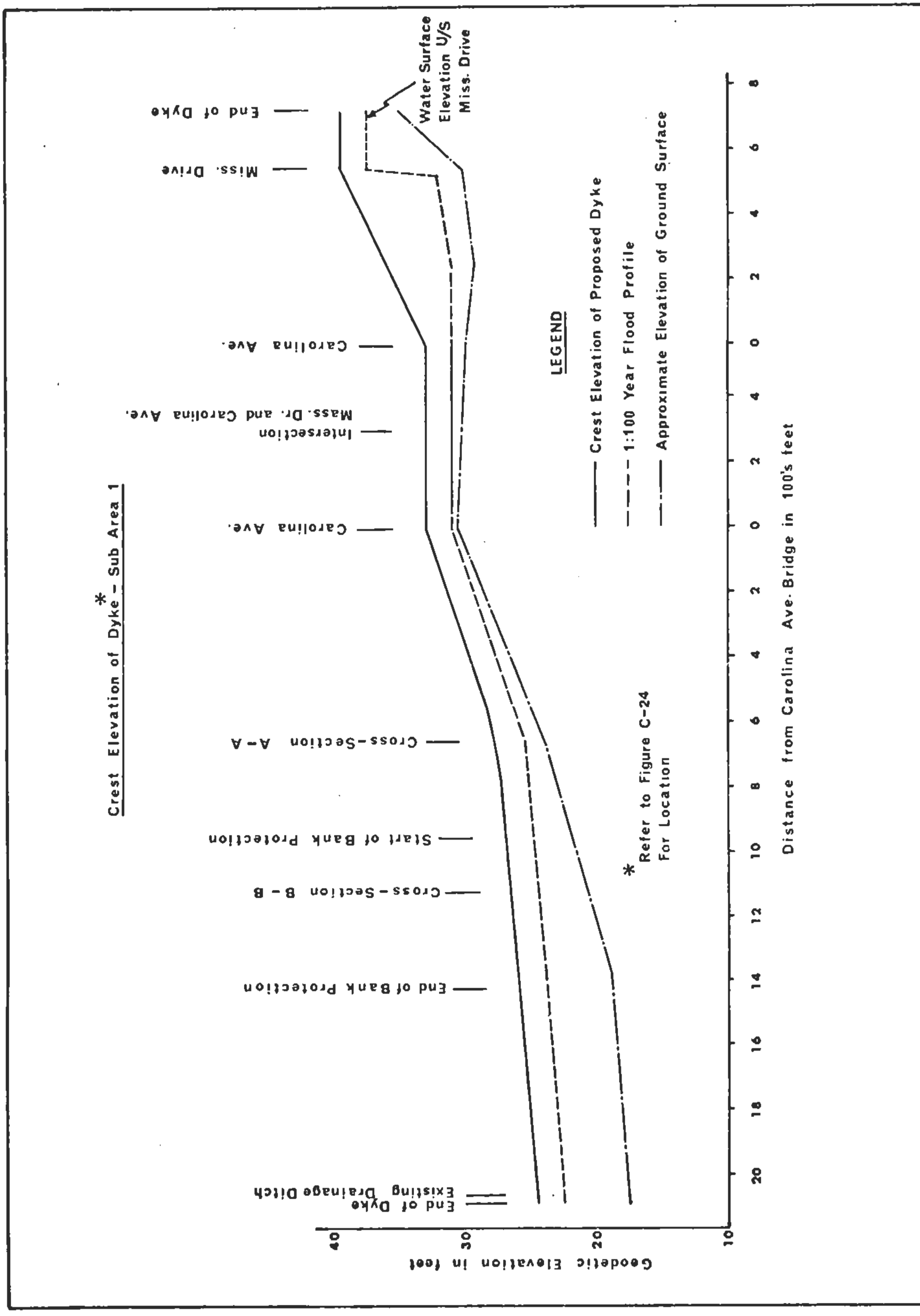

Crest Elevation of Dyke* – Sub Area 1
LEGEND
Crest Elevation of Proposed Dyke
1:100 Year Flood Profile
Approximate Elevation of Ground Surface
Water Surface Elevation U/S Miss. Drive
End of Dyke
Miss. Drive
Carolina Ave.
Intersection Mass. Dr. and Carolina Ave.
Carolina Ave.
Cross-Section A – A
Start of Bank Protection
Cross-Section B – B
End of Bank Protection
Existing Drainage Ditch
End of Dyke
* Refer to Figure C-24 For Location
Geodetic Elevation in feet
10
20
30
40
Distance from Carolina Ave. Bridge in 100's feet
8
6
4
2
0
4
2
0
2
4
6
8
10
12
14
16
18
20

FIGURE C-23

A profile of the existing Brookside Drive, including its bed profile, the 1:100 year flood line, and the existing and proposed elevation of Brookside Drive, is shown in figure C-25. In addition, a plan view showing the road, areas of rip-rap protection, and the 500 feet extension in the northerly direction to prevent flood waters from getting behind the dyke, including representative cross-sections is shown in figure C-26.

6.6 Quantities and Cost Estimates

6.6.1 LaBatt's Brewery - Humber Motors Dyke System

As already stated in section 6.5.1, the two alternatives for the dyke system studied for this project are: (1) zone embankment, using compacted clay as core and key materials, flanked by existing materials found at the site, and (2) the homogenous type, using only the existing material located at the site.

Referring to figures C-23 and C-24, the entire length of the dyke system was divided into six sections, and by use of the cross-sections shown on figure C-24 for top width, side slopes of the various material components, depth of key, etc., the quantities for the dyke system, using clay as core material were calculated.

In a similar fashion, the division of the total length of the dyke into six sections, the rip-rap quantity for protection of the upstream slope, and the area required for grass-seeding of the downstream slope were estimated. The armour stone quantity required for extra protection of the slope in the area of the brewery, a minimum length of 350 feet was used for the cost estimate, although it was conceded that 500 feet would be required for full protection as shown in figure C-24. The size of the armour stone

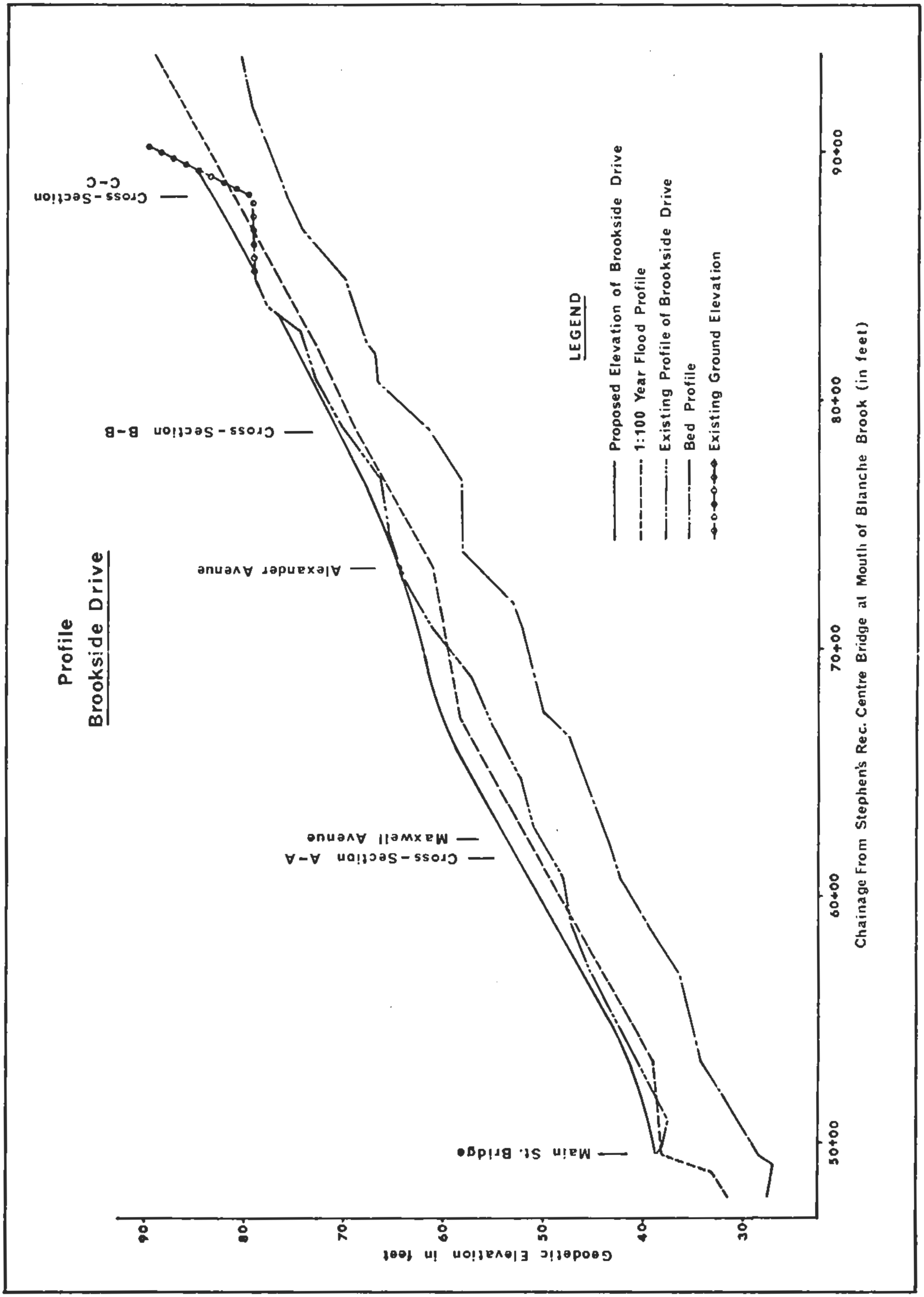

Profile
Brookside Drive
LEGEND
Proposed Elevation of Brookside Drive
1:100 Year Flood Profile
Existing Profile of Brookside Drive
Bed Profile
Existing Ground Elevation
Cross-Section C-C
Cross-Section B-B
Alexander Avenue
Maxwell Avenue
Cross-Section A-A
Main St. Bridge
Geodetic Elevation in feet
90
80
70
60
50
40
30
50+00
60+00
70+00
80+00
90+00
Chainage From Stephen's Rec. Centre Bridge at Mouth of Blanche Brook (in feet)

FIGURE C-25

must be of hard and durable quality, which is obtainable from a local quarry, and each stone should range in size from four to five tons.

Following is a summary of the various materials required for the clay-core dyke:

Clay (key and core)	-	9,000 cu. yds.
Sand and gravel	-	6,850 cu. yds.
Rip-rap	-	10,650 cu. yds.
Armour Stone (brewery protection)	-	3,400 tons
Grass seeding	-	42,300 sq. ft.

The homogenous dyke was assumed to be of the same overall size as the zone embankment type for cost-estimating purposes. The existing materials at the site were assumed to be silt, sand and gravel mixed with some marine clay, and can be compacted satisfactorily into a dyke system.

The following quantities were estimated for this particular type of dyke system:

Silt, sand and gravel	-	15,850 cu. yds.
Rip-rap	-	10,650 cu. yds.
Armour stone	-	3,400 tons

The unit cost of the clay core in place, $6.50 per cubic yard, includes: purchasing, loading, hauling (8 miles), spreading and compacting was acquired from a source in the region. The estimates of unit costs for sand, silt, gravel, and rip-rap were identical to those of the dam-reservoir system and derived from the same sources. Various alternatives were studied for

extra protection of the upstream slope of the dyke in the area of the brewery: concrete on fill, a reinforced concrete retaining wall, a sheet steel piling wall, a rock-filled timber crib and armour stone. From a structural point of view, these types of protection were considered adequate in providing the necessary protection. But armour stone was selected because it was estimated to cost the least. The unit prices for the various alternatives were obtained from the informal telephone conversations with personnel of the Department of Public Works of Canada.

A summary of the first costs for the zone embankment and homogenous type of dykes are shown in tables C-5 and C-6 respectively.

6.6.2 Brookside Drive

Approximately 9,300 cubic yards of compacted fill would be required to upgrade and extend the road, based on the plan view and cross-sections shown on figure C-26. From the same figure the rip-rap quantities were estimated to be 8,788 cubic yards.

Table C-7 shows the first cost of upgrading Brookside Drive without the substitution of cobble stones, which presently exist in and along the banks, for rip-rap protection.

7. BLANCHE BROOK CHANNEL RE-ALIGNMENT

7.1 Design Concept

For this proposal it is envisaged to divert the flow in Blanche Brook from a section 450 feet downstream of the Main Street Bridge to a section approximately 1,700 feet downstream of the confluence, by constructing a new channel to relieve the confluence area adjacent to the brewery from the

TABLE C-5

SUMMARY OF FIRST COST

PROPOSED ZONE TYPE DYKE - SUB-AREA 1

Item	Quantity[1]	Unit	Unit[1] Cost ($)	Amount ($)
Excavation	4,400	yd^3	3.00	13,200
Clay Core-Key	9,000	yd^3	6.50	58,500
Sand/Gravel/Silt	6,850	yd^3	3.00	20,550
Rip-rap	10,650	yd^3	5.00	53,250
Armour Stone	3,400	ton	11.00	37,400
Grass Seeding	42,300	ft^2	0.10	4,200
Miscellaneous	--	--	--	4,000
Sub Total				191,100
Contingencies at 20%	--	job	--	38,200
Sub Total				229,300
Engineering Design Supervision and Administration at 10%	--	job	--	22,900
TOTAL FIRST COST				$252,200

1. All quantities and unit costs are for materials in place.

TABLE C-6

SUMMARY OF FIRST COST

PROPOSED HOMOGENEOUS TYPE DYKE - SUB-AREA 1

Item	Quantity[1]	Unit	Unit[1] Cost ($)	Amount ($)
Excavation	4,400	yd^3	3.00	13,200
Sand/Gravel/Silt	15,850	yd^3	3.00	47,550
Rip-Rap	10,650	yd^3	5.00	53,250
Armour Stone	3,400	ton	11.00	37,400
Grass Seeding	42,300	ft^2	0.10	4,300
Miscellaneous	-	-	-	4,000
Sub Total				159,700
Contingencies at 20%				31,940
Sub Total				191,640
Engineering Design Supervision and Administration at 10%				19,164
TOTAL FIRST COST				$210,804

1. All quantities and unit costs are for materials in place.

TABLE C-7

SUMMARY OF FIRST COST

PROPOSED UPGRADING OF BROOKSIDE DRIVE - SUB-AREA 3

Item	Quantity[1]	Unit	Unit[1] Cost ($)	Amount ($)
Compacted Fill	9,300	yd^3	$4.50	41,850
Rip-Rap	8,788	yd^3	5.00	43,940
Sub Total				85,790
Contingencies at 20%				17,158
Sub Total				102,948
Engineering Design Supervision and Administration at 10%				10,295
TOTAL FIRST COST				113,243

1. All quantities and unit costs are for materials in place.

effects of the Blanche Brook flow. Figure C-27 shows the proposed route of the new channel in plan view. In addition three cross-sections are shown: BB-3, a section of existing main channel, BB-3(a) and BB-3(b) for the proposed new re-alignment channel, and the area where rip-rap is required for bank protection.

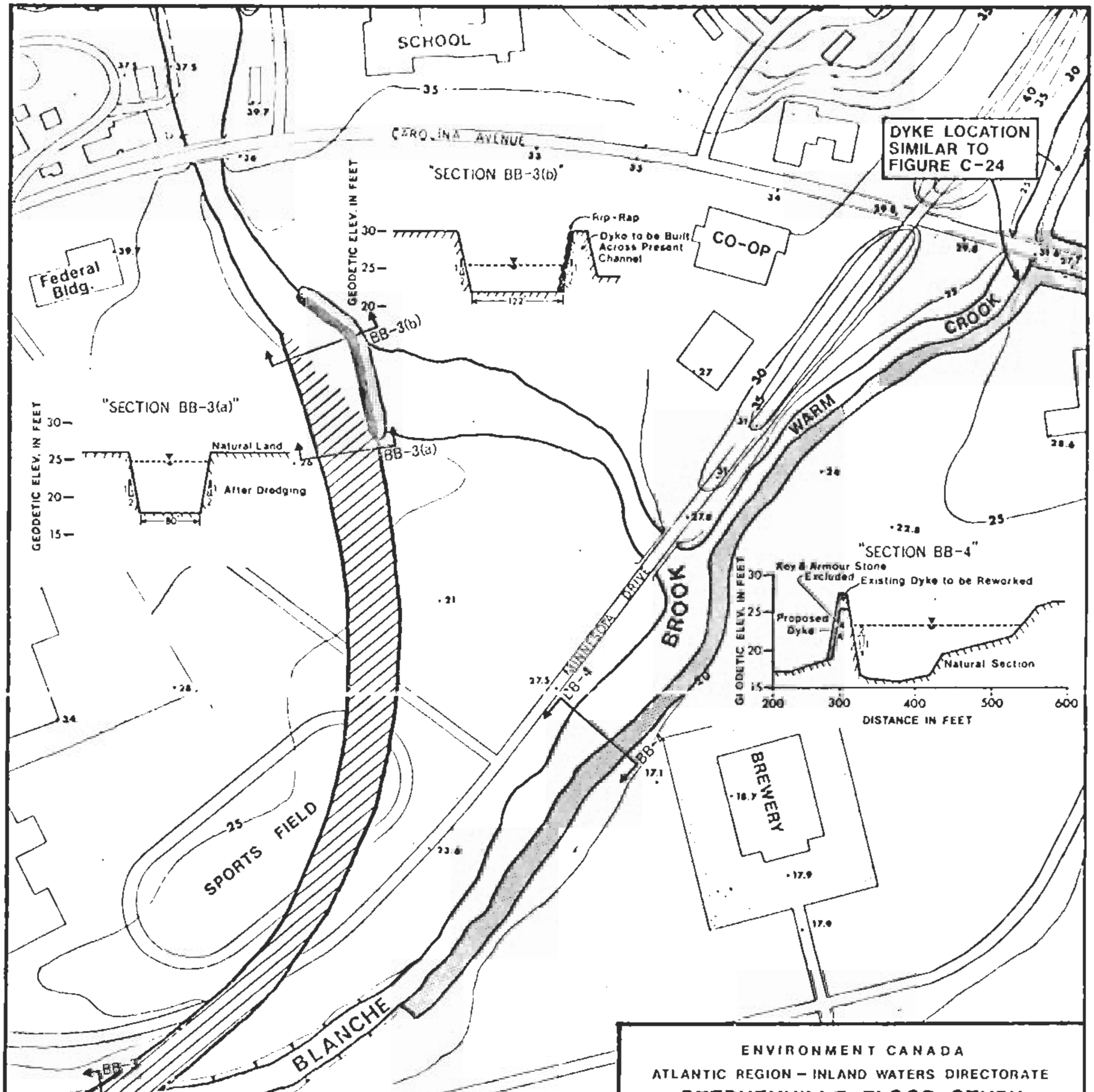
7.2 Hydraulic Effectiveness

It was determined by back water calculations that the water level at the surveyed section near the brewery, BB-4, dropped only 6 inches in a 1:100 year flooding event, when compared to the situation where the full flood flow passed through that section. But on the positive side, the scheme rules out the necessity of providing armour stone for water velocity protection in the area adjacent to the brewery.

Figure C-28 shows the profile of the proposed new re-alignment channel, the water surface line (1:100 year flood), the original ground line, the channel bottom, and the four cross-sections.

Following is a summary of the backwater calculations for the determination of water-surface elevations in the new re-alignment channel. The stepwise computations were taken upstream from BB-3 to BB-3(a) to BB-3(b), and from BB-3 to BB-4.

STATION	Z (ft)	A (ft^2)	V (fps)	Q (cfs)
BB-3	20.04	755	8.16	6,160
BB-3(a)	24.8	636	5.66	3,600
BB-3(b)	25.5	452	7.96	3,600
BB-4	23.4	1,020	2.49	2,550



ENVIRONMENT CANADA

ATLANTIC REGION – INLAND WATERS DIRECTORATE

STEPHENVILLE FLOOD STUDY

CHANNEL REALIGNMENT BLANCHE BROOK

LEGEND

PROPOSED DYKING

EXTENT OF SPECIAL BANK PROTECTION FROM HIGH VELOCITIES

PROPOSED CHANNEL ALIGNMENT

DEC., 1975

FIGURE C-27

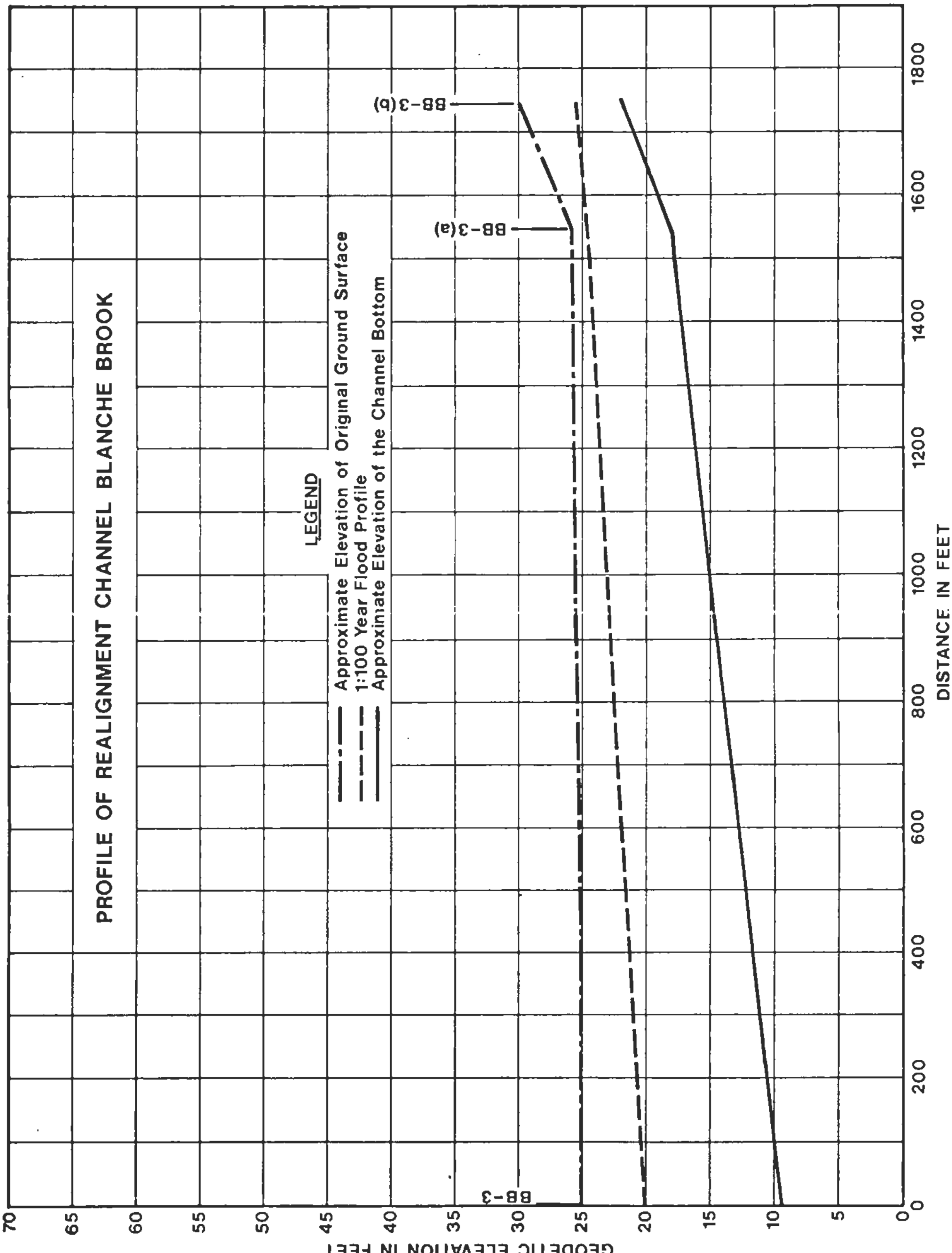

PROFILE OF REALIGNMENT CHANNEL BLANCHE BROOK
LEGEND
Approximate Elevation of Original Ground Surface
1:100 Year Flood Profile
Approximate Elevation of the Channel Bottom
BB-3(b)
BB-3(a)
BB-3
GEODETIC ELEVATION IN FEET
70 65 60 55 50 45 40 35 30 25 20 15 10 5 0
DISTANCE IN FEET
0 200 400 600 800 1000 1200 1400 1600 1800

FIGURE C-28

7.3 Quantities and Cost Estimates

Estimated quantities for excavated and other materials in addition to costs are summarized and listed in table C-8.

In conclusion, the cost to construct the re-alignment channel, excluding new bridges, was estimated at $242,400. By comparison the cost of providing armour stone for extra bank protection near the brewery was $75,000. Assuming the life expectancy of the re-alignment channel to be 10 years, due to silt deposition, the cost of this proposal is very unattractive.

TABLE C-8

SUMMARY OF FIRST COST

CHANNEL REALIGNMENT

BLANCHE BROOK

Station	Excavated Volume yd^3	Other Material yd^3	Unit Cost ($)	Amount ($)
BB(3) to BB-3(a)	57,000		4.00	228,000.
BB-3(a) to BB-3(b)	3,000		4.00	12,000.
Fill upstream of BB-3(b) to block old channel	---	600	1.50	900.
Rip-Rap upstream of BB-3(b) for bank protection	---	300	5.00	1,500.
		TOTAL FIRST COST		$242,400.

8. DREDGING ALTERNATIVE - BLANCHE BROOK

8.1 General

Improving the hydraulic capacity of a water course is one of the well-recognized structural alternatives to flood control. An analysis was therefore undertaken to determine the magnitude and effectiveness of channelization in the reach between the Recreation Center and the brewery. Presently, this channel is not adequate to pass flood flows without causing water to back up to the brewery, thus contributing to the existing flooding problems.

Blanche Brook, from its mouth to its confluence with Warm Creek, is an artifical channel constructed sometime before 1952, during the expansion of the air base. It is not known whether the hydraulic capacity of this reach has been reduced over the years by sediment disposition or not. The drop in water level over this length of reach is approximately 13.5 feet under 1:100 year flood conditions. Consequently, it was decided to investigate the feasibility of dredging this particular part of the water course.

8.2 Hydraulic Design Criteria

The principle design criterion, as in the case of the dyke proposal, was that the channel have the capability of passing a 1:100 year flood without causing significant flooding problems. A freeboard allowance of 2 feet was adopted.

Consideration was given, as in the major dyking proposal, to the protection of the river bank near the brewery at the confluence, where velocities of 10-15 feet per second tend to re-direct the flow to its natural course, directly through the brewery and airport runway.

8.3 Alternative Dredging Schemes

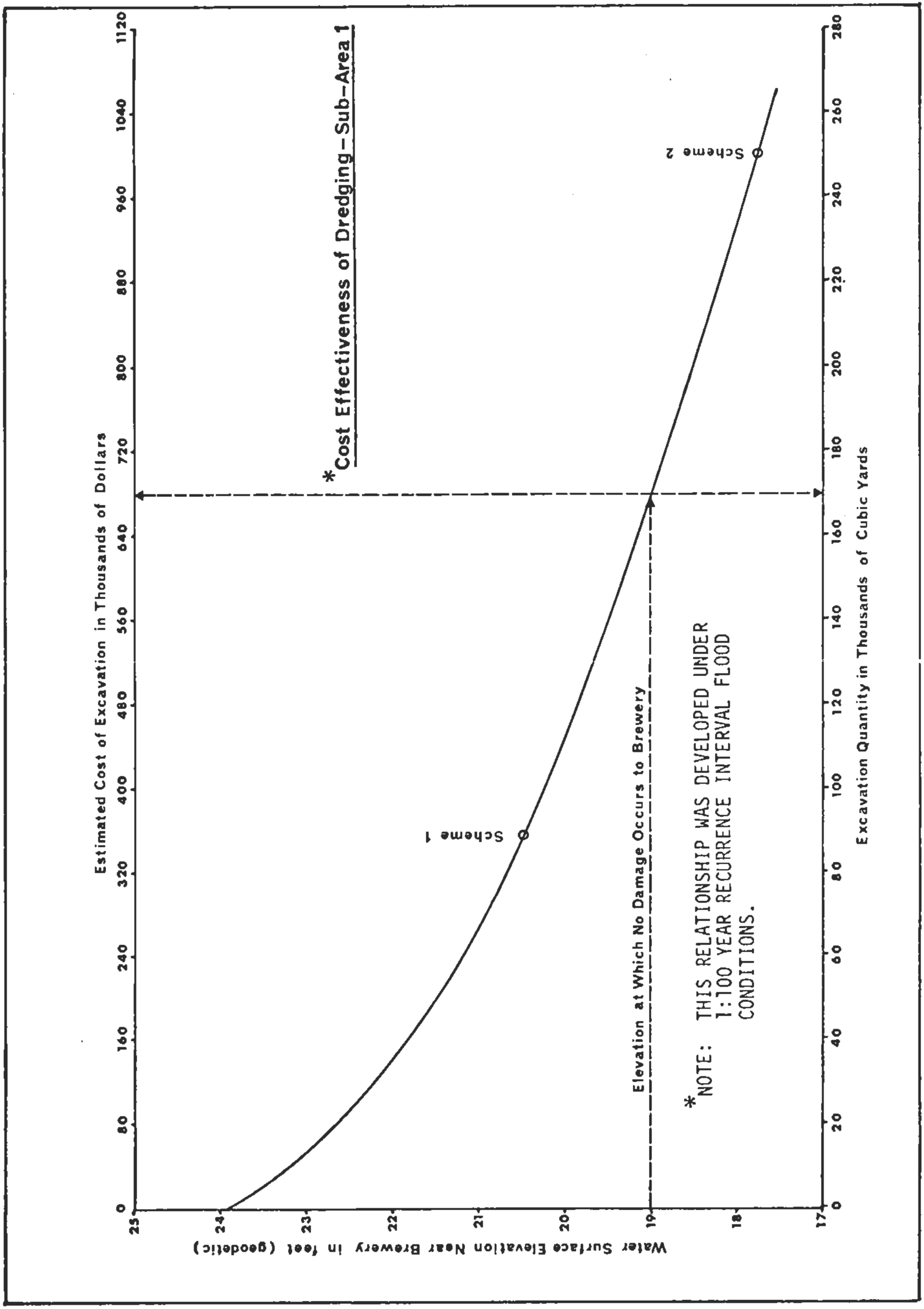
Two dredging schemes were explored in detail to assess the cost/effectiveness of the range of possible schemes. Both schemes call for lowering the existing river bed by three feet from a point approximately 500 feet above the bridge near the Recreational Center up to a point within a few hundred feet below the Carolina Avenue Bridge over Warm Creek. There were no significant changes made in bed slope for both schemes.

Scheme I involved the widening of Blanche Brook to 80 feet at the base where necessary, with side slopes of two feet horizontal to one vertical for bank stability. For scheme 2, Blanche Brook was widened to 160 feet at the base with gradual tapering to the bridges. Under both schemes, Warm Creek, from the confluence to below the Carolina Avenue Bridge, was widened to 60 feet. The areal extent and representative cross-sections of scheme I are shown in figure C-29.

8.4 Hydraulic Effectiveness

Under present channel conditions, the 1:100 year flood flow would result in a maximum water level of 23.9 feet in the brook opposite the brewery. As a result of scheme I (increasing the depth by 3 feet and providing a minimum width of 80 feet) this level could be reduced to 20.5 feet. Scheme 2 would reduce the level to slightly less than 18 feet. A relationship, as shown in figure C-30, between the volume of excavation and water surface elevations near the brewery, (BB-4) was developed from schemes 1 and 2 and from existing conditions.

If scheme I was adopted, it would be imperative to provide a low dyke extending from a point about 200 feet below the Carolina Street Bridge



*Cost Effectiveness of Dredging–Sub–Area 1
Estimated Cost of Excavation in Thousands of Dollars
0 80 160 240 320 400 480 560 640 720 800 880 960 1040 1120
Water Surface Elevation Near Brewery in feet (geodetic)
25 24 23 22 21 20 19 18 17
Excavation Quantity in Thousands of Cubic Yards
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280
Scheme 1
Scheme 2
Elevation at Which No Damage Occurs to Brewery
*NOTE: THIS RELATIONSHIP WAS DEVELOPED UNDER 1:100 YEAR RECURRENCE INTERVAL FLOOD CONDITIONS.

FIGURE C-30

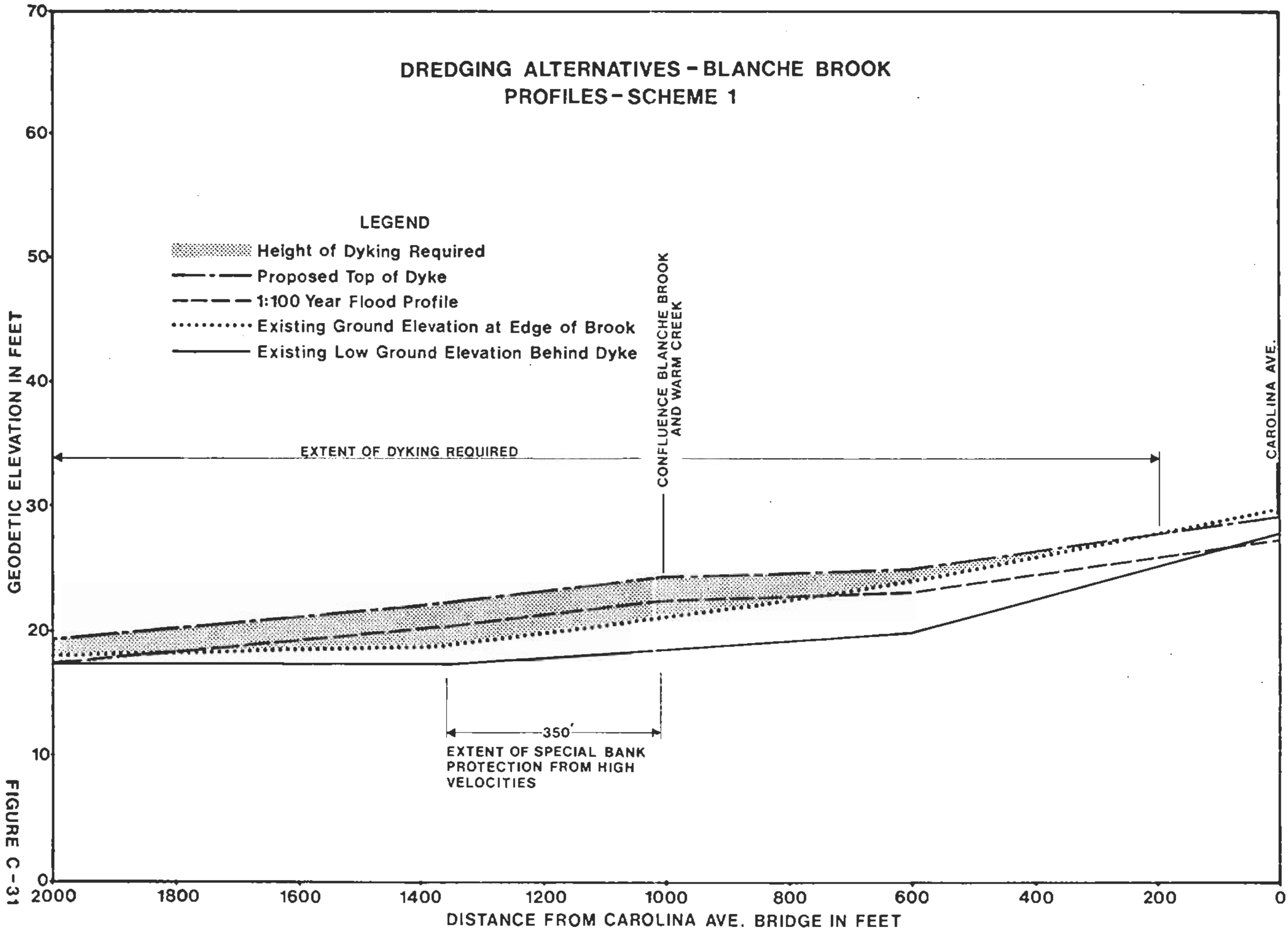

DREDGING ALTERNATIVES - BLANCHE BROOK
PROFILES - SCHEME 1
LEGEND
Height of Dyking Required
Proposed Top of Dyke
1:100 Year Flood Profile
Existing Ground Elevation at Edge of Brook
Existing Low Ground Elevation Behind Dyke
CONFLUENCE BLANCHE BROOK AND WARM CREEK
CAROLINA AVE.
EXTENT OF DYKING REQUIRED
350'
EXTENT OF SPECIAL BANK PROTECTION FROM HIGH VELOCITIES
GEODETIC ELEVATION IN FEET
0
10
20
30
40
50
60
70
2000
1800
1600
1400
1200
1000
800
600
400
200
0
DISTANCE FROM CAROLINA AVE. BRIDGE IN FEET

FIGURE C-31

to a point 600 feet downstream to the brewery. This dyking would have a maximum height of only 3.5 feet including 2 feet for freeboard, and would have to be approximately 1,800 feet long. Scheme 2 would not require any dyking.

The water surface profile resulting from scheme 1 under 1:100 year flood conditions is shown in figure C-31. The top of the proposed dyke required in conjunction with scheme 1 is also shown. Moreover, the profiles of the existing ground elevation at the edge of the brook, and the ground surface behind the dyke are also shown.

On the surface, it would appear that scheme 2 effectively solves the flooding problem at the brewery, by confining all the flow to the dredged channel with no dyking requirements. However, a consideration of the cost of providing each plan would be required.

The greatest cost is dredging. No attempt was made to optimize the quanity of material to be excavated. This should be done if the alternative appears feasible.

The protection of the bank from the confluence of Warm Creek with Blanche Brook to a section just downstream of the brewery is necessary, whether dyking is constructed or dredging is carried out. The methods of protecting this reach, approximately 350 feet in length, are discussed in the section on dykes.

Little is known about the sediment regime associated with Blanche Brook. However, based on observations on the degree of upstream bank erosion, and the fact that numerous large cobbles have settled out along the stream course, it is felt that sediment loads are relatively high, and maintenance dredging may be required. It was assumed that the effective economic life of any dredging scheme in the reach under consideration for dredging, is 10 years.

8.5 Quantities & Cost Estimates

Based on a limited analysis of unit dredging costs and informal discussions with personnel of the Federal Department of Public Works, a unit cost of $4.00 per cubic yard was assumed. The cost of dredging was included in figure C-30 (top ordinate). By examining figure C-30, it can be determined that approximately 170,000 cubic yards of material at a cost of $680,000 would be required to alleviate the problem for all flooding up to the 1:100 year flood. Additional costs for bank protection ($75,000) and a small dyke ($61,000) to provide 2 feet of freeboard, would also be incurred.

9. DIVERSIONS

9.1 General

Conceptually, diversion means the conveyance of water from one watershed to an adjacent watershed and normally involves some diversion works, for example, the construction of a small dam and an artificial canal. Presumably, no flooding problems would be created in the adjacent watershed as a result of the diversion.

Two large scale diversions were considered and examined in detail by use of existing topographical maps and intrepretations of air photos of the area.

There are several small scale possibilities for diversion on Warm Creek, while there are no apparent ones on Blanche Brook. But it was judged that the cost of these small scale diversions would far out-weigh any potential flood control benefits.

9.2 Design Concept

The design concept for both diversions was to relieve those flood prone areas of the watercourses from the ravages of flooding, particularly those in the vicinity of LeBatt's Brewery and Humber Motors.

The design criteria for both diversions was to determine the size of the canals to carry a flood of 2,500 cfs without overbank flooding. This quantity would represent a complete diversion of the flow produced on the Warm Creek watershed and seventy percent of the flow produced on the Blanche Brook watershed during a 1:100 year flooding event. The remaining flow of 1,100 cfs in Blanche Brook would not create any flooding problems at the brewery near the confluence.

9.3 Alternative Routes

The Warm Creek diversion would entail an artificial channel from Noel's Pond to Muddy Pond and finally to Port Harmon. Indication of the route is shown in plan on figure C-32. Two routes were examined for this proposal. Route 1 was considered infeasible because of steep slopes. Route 2, involving three plans or "cuts", looked more promising, and preliminary studies were carried out for it.

For the Blanche Brook diversion, an artificial channel was envisaged from the Blanche Brook watercourse to Gadon's Brook, outlined as Route 3 in figure C-32.

9.4 Quantities and Cost Estimates

By use of the standard manning formula, and assuming a value of n of 0.04, a channel width of 100 feet, a discharge rate of 2,500 cfs and adopting the slopes as shown in figures C-33 and C-34 respectively, rough estimates of excavation quantities for the Warm Creek diversion, route 2, plan 2, was 200,000 cubic yards, and the Blanche Brook diversion, route 3, was 370,000 cubic yards.

Applying an estimated unit cost of $3.00 per cubic yard, the total estimated cost of 570,000 cubic yards of excavation amounted to $1,710,000.

Moreover, it must be emphazised that the estimated excavation costs were based on the assumption that the material is unconsolidated and no rock blasting required. The inclusion of diversion works (dam), channelization, enlarging Gadon's Brook to accommodate flood discharges, and other measures, which might be required to protect houses against flooding along this watercourse may also affect the cost. The loss of valuable land (area 13), or the construction and/or relocation of multiple bridges along the Noel's Pond - Port Harmon Route were also not considered in the cost.

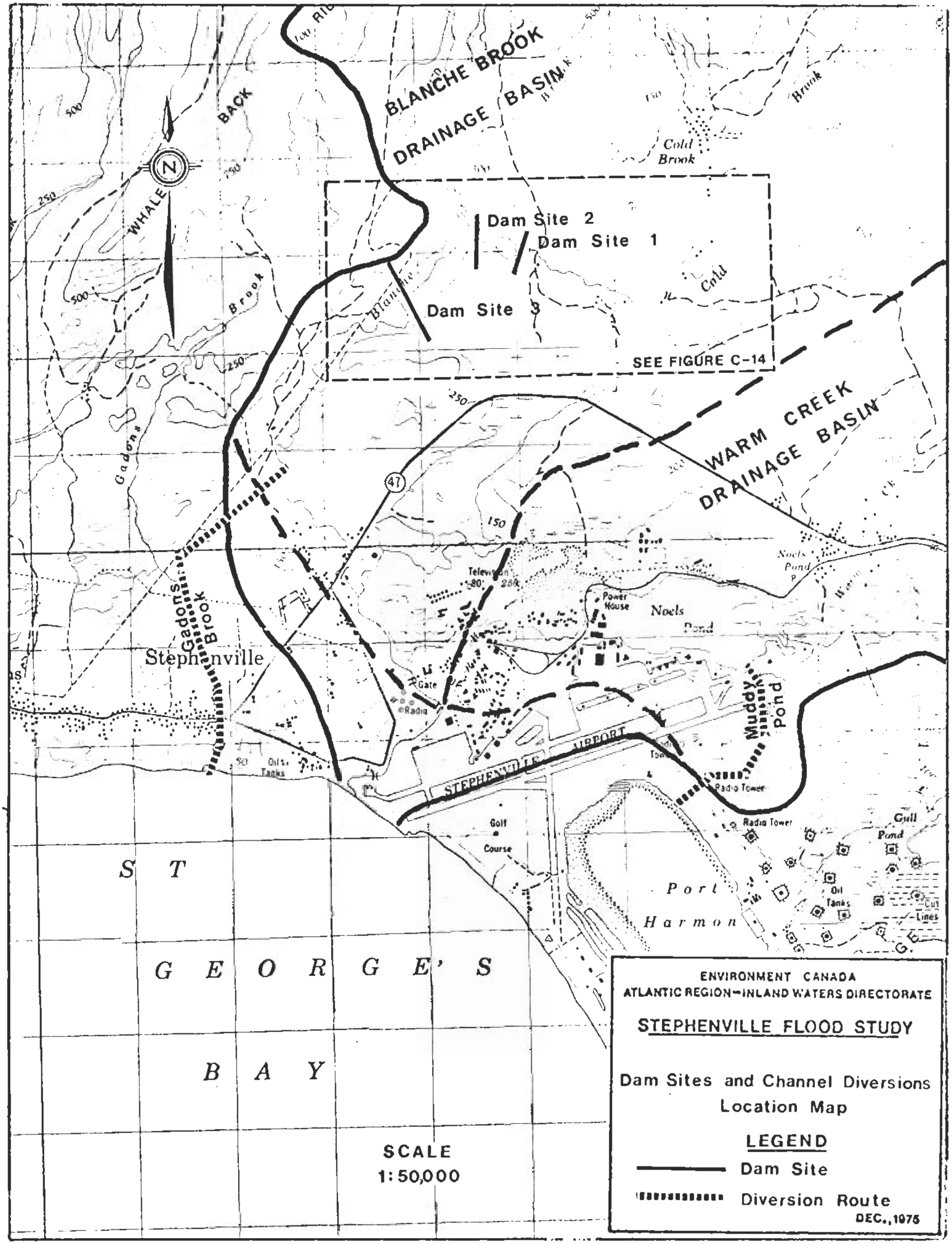

BLANCHE BROOK
DRAINAGE BASIN
BACK
WHALE
Brook
Cold
Brook
Dam Site 2
Dam Site 1
Dam Site 3
Blanche
Cold
SEE FIGURE C-14
WARM CREEK
DRAINAGE BASIN
47
Gadons
Gadons
Brook
Stephenville
Noels
Pond
Power
House
Muddy
Pond
STEPHENVILLE AIRPORT
Radio Tower
Golf
Course
Oil Tanks
Port
Harmon
Gull
Pond
S T
G E O R G E ' S
B A Y
SCALE
1:50,000
ENVIRONMENT CANADA
ATLANTIC REGION–INLAND WATERS DIRECTORATE
STEPHENVILLE FLOOD STUDY
Dam Sites and Channel Diversions
Location Map
LEGEND
Dam Site
Diversion Route
DEC., 1975

FIGURE C-32

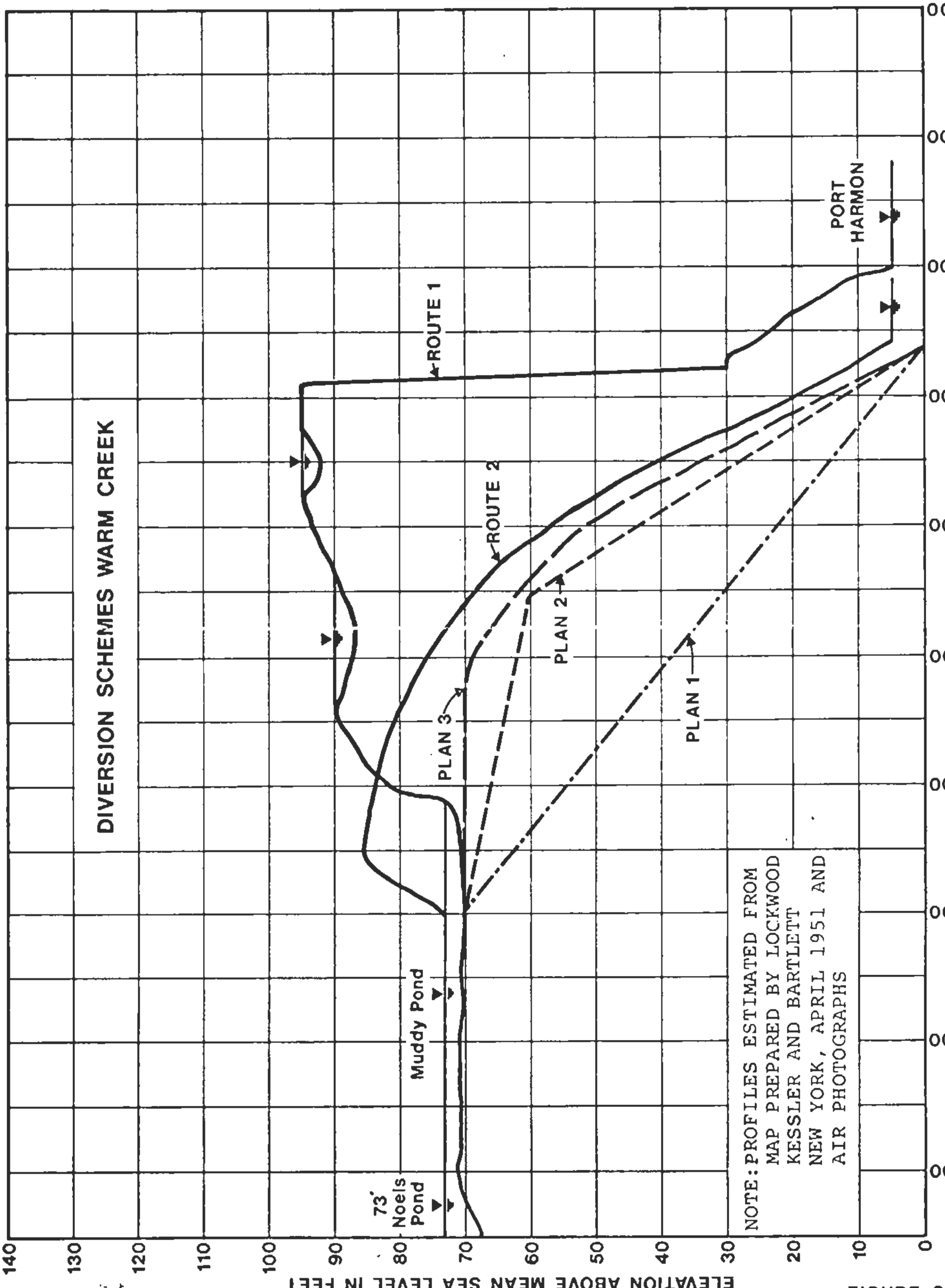

DIVERSION SCHEMES WARM CREEK
ELEVATION ABOVE MEAN SEA LEVEL IN FEET
CHAINAGE FROM NOELS POND
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
0+00
10+00
20+00
30+00
40+00
50+00
60+00
70+00
80+00
90+00
ROUTE 1
ROUTE 2
PLAN 1
PLAN 2
PLAN 3
PORT HARMON
Muddy Pond
73'
Noels Pond
NOTE: PROFILES ESTIMATED FROM
MAP PREPARED BY LOCKWOOD
KESSLER AND BARTLETT
NEW YORK, APRIL 1951 AND
AIR PHOTOGRAPHS

FIGURE C-33

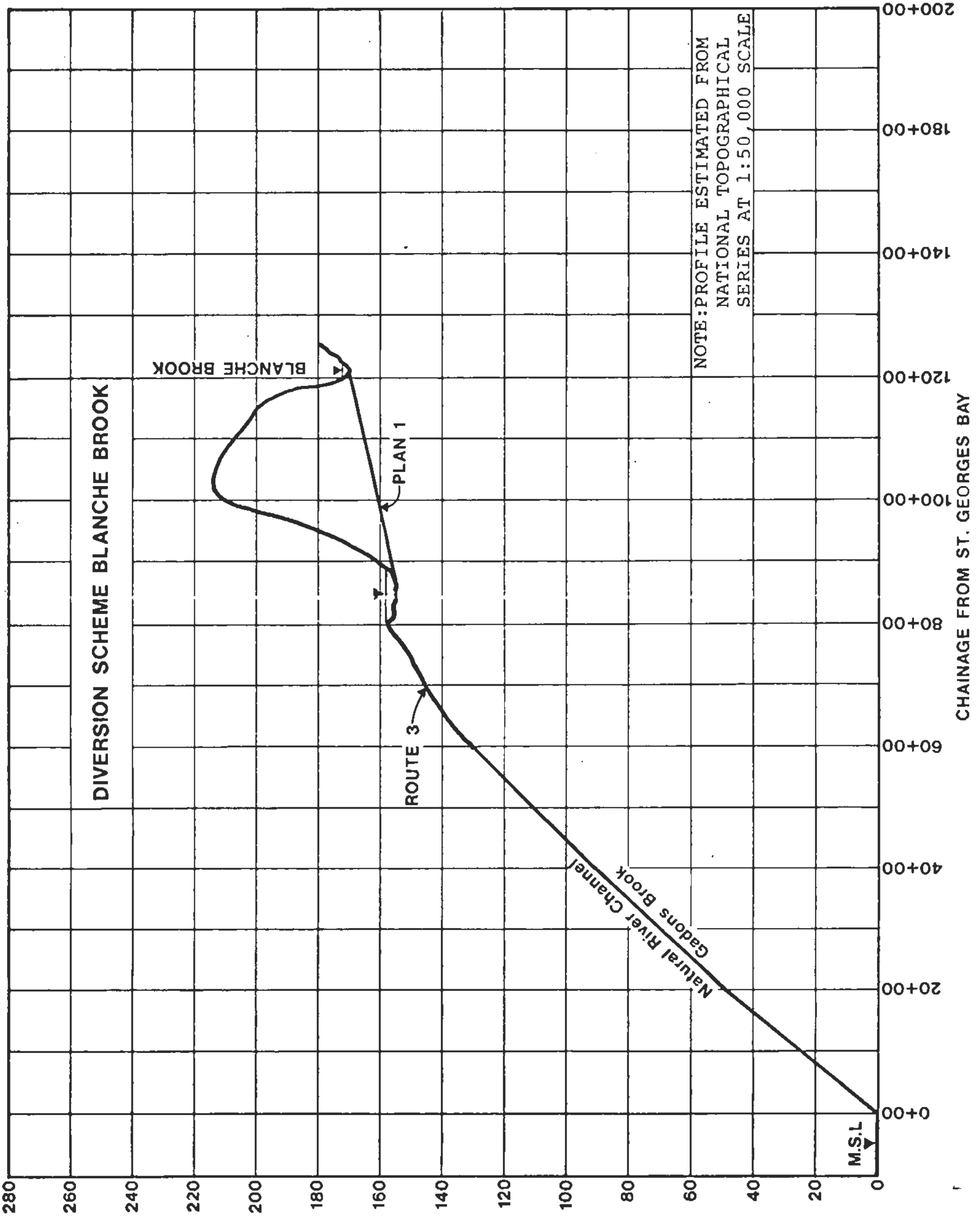

DIVERSION SCHEME BLANCHE BROOK
NOTE: PROFILE ESTIMATED FROM
NATIONAL TOPOGRAPHICAL
SERIES AT 1:50,000 SCALE
BLANCHE BROOK
PLAN 1
ROUTE 3
Natural River Channel
Gadons Brook
M.S.L
ELEVATION ABOVE MEAN SEA LEVEL IN FEET
CHAINAGE FROM ST. GEORGES BAY
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
0+00
20+00
40+00
60+00
80+00
100+00
120+00
140+00
180+00
200+00

FIGURE C-34

AN EVALUATION OF FLOODING

IN

STEPHENVILLE, NEWFOUNDLAND

APPENDIX D

TEXT OF A PRESS RELEASE BY THE MINISTER

OF THE ENVIRONMENT FOR CANADA MME. J. SAUVE

APRIL 10, 1975

Water Planning and Management Branch
Inland Waters Directorate
Atlantic Region
Environment Canada
Halifax, Nova Scotia

December, 1975

APPENDIX D - FEDERAL FLOOD DAMAGE REDUCTION PROGRAM OUTLINED

Environment Canada Minister Jeanne Sauve today invited a new federal approach to reduce the mounting toll of damages caused by floods.

The long-range approach is based on the development of a series of federal-provincial Accords to reduce potential flood damages and a national flood hazard mapping program.

"Canada has to reduce the toll of personal suffering and financial loss due to floods, " said Madame Sauve.

"We have developed a co-ordinated federal-provincial approach to water resource management through the Canada Water Act. Accords on floods will focus that co-operation on a concerted effort to reduce the ravages of floods in Canada."

The proposed Accords would be based on the principles that:

a. Flood risk areas must be clearly defined and mapped;
b. Information on flood hazards must be communicated to the public, industry, municipalities and the provinces;
c. Construction of federal facilities, federal housing loans and other grants and loans should not be made in flood risk areas or be made conditional upon adequate flood proofing or other damage reduction measures;
d. Disaster assistance should be refused for further development in identified high flood risk areas where the public has been made fully aware of the hazard; and
e. Provinces and municipalities should be encouraged to consider appropriate restrictions on land use in high flood risk areas.

Federal-provincial co-operation in keeping with these principles is evident in five pilot flood hazard mapping projects now underway. A flood risk map for Federicton, New Brunswick is almost complete and maps for Carman, Manitoba, Moose Jaw, Saskatchewan, and Oshawa and Sault Ste. Marie, Ontario are being drafted.

As a key part of this new approach to flood damage reduction, a national flood hazard mapping program has been approved. The mapping program may cost up to $20 million with costs shared equally by the Federal government and the provinces. Priorities for mapping flood risks for more than 200 rural and urban communities affected by floods will be worked out jointly.

"When this flood risk information is available, federal and provincial governments can undertake commitments to discourage further investment in flood risk areas, "Madame Sauve explained.

"However, if it is not possible to work out a mutually acceptable Accord with any province, the federal government will not be deterred from doing what it can. We will act decisively in our own areas of responsibility."

Federal disaster assistance will be withheld in areas where a high flood risk has been identified and where development has proceeded in spite of knowledge of the risk. Direct federal investment and federally financed development will not be made inside known hazard areas.

The federal government will continue to participate in traditional flood control projects such as dams where these offer the best solutions. However, a greater emphasis will be placed on a combination of structural and non-structural alternatives. Experience has shown that a flood damage reduction policy based on structural works alone may not be effective.

Alternatives to be considered include land-use, adjustments such as acquisition and zoning, flood warning and forecasting, flood routing through property easements, flood proofing of structures, upstream storage, stream straightening, flood by-passes and dykes.

"I have discussed the Accords informally with a number of provinces, " Madame Sauve said. "More detailed discussion will take place through established consultative committees. These Consultations will, I believe, lead to suitable Accords with each province."