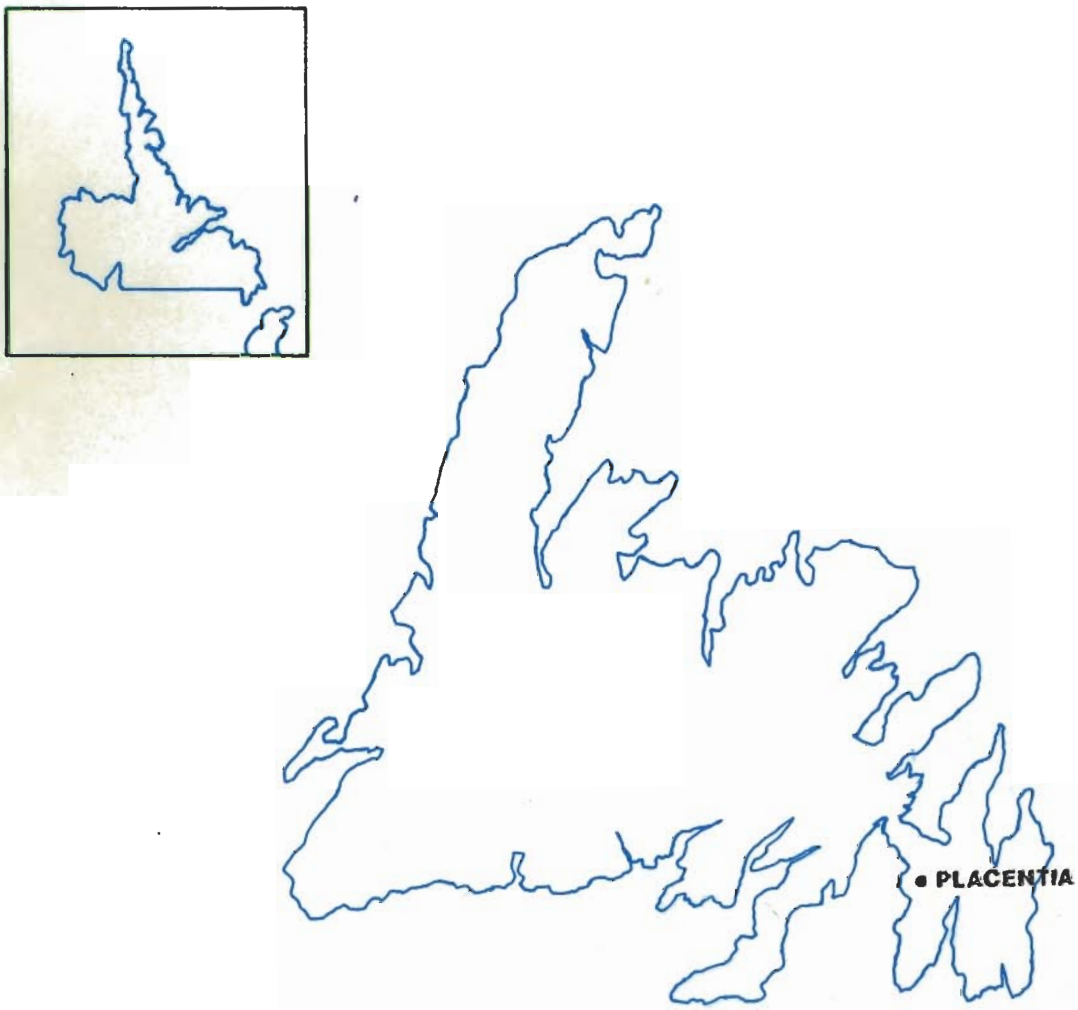
Canada – Newfoundland
Flood Damage Reduction Program

Hydrotechnical Study of the Placentia Area Flood Plain

MAIN REPORT

Volume 1 of 2



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SHAWMONT - MARTEC LIMITED
OCEAN SCIENCE AND ENGINEERING CONSULTANTS

Department of Environment

Environment Canada

CANADA - NEWFOUNDLAND

FLOOD DAMAGE REDUCTION PROGRAM

ENVIRONMENT CANADA　　　　　DEPARTMENT OF ENVIRONMENT

MAIN REPORT
ON
HYDROTECHNICAL STUDY
OF THE
PLACENTIA AREA FLOOD PLAIN

(Volume 1 of 2)

Prepared By:

SHAWMONT NEWFOUNDLAND LIMITED

SMR-28-84　　　　　April, 1985

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Canada-Newfoundland Flood Damage Reduction Program
Department of Environment
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Dear Dr. Ullah:

We are pleased to submit Volume 1 of 2 of our final report on the "Hydrotechnical Study of the Placentia Area Flood Plain". This volume contains the main report on the study and incorporates, where possible, the Technical Committee's comments on the previous interim and draft reports.

The methodology employed, the findings of the investigations and the resulting conclusions and recommendations are discussed herein. Additional information on the field program is being provided in a separate supplementary report (Volume 2 of 2).

We have enjoyed working on this very interesting study and take this opportunity to gratefully acknowledge the assistance provided by members of the Technical Committee, by Council personnel, business owners and residents of the Towns of Placentia and Jerseyside, and by various government officials during the course of the study.

Yours very truly,

ADPeach

A.D. Peach, P. Eng.,
Vice President

DHB/cb

SUMMARY

SUMMARY

The purpose of this study was to assess the flooding problem in the Placentia area, to determine the 1 in 20 year and 1 in 100 year flood contours and to recommend alternative flood control measures to minimize future flood damage.

Review of available information indicated that the entire Town of Placentia and a small part of the Town of Jerseyside are located on a low and relatively flat beach formed at the head of an inlet off Placentia Bay, called Placentia Road. The beach is almost surrounded by water with Placentia Road to the west and an estuarial system to the east comprising Northeast Arm, Swan Arm, Southeast Arm and interconnecting channels.

In late 1983, a field program was carried out to collect topographic and hydrographic detail in the study area and to determine the history of flooding in the area. It was concluded that flooding is common and occurs from two sources:

i) High water levels in Northeast Arm, Swan Arm and Southeast Arm, and

ii) Waves overtopping the beach to the west.

It was determined that, of the two Towns, Placentia experiences most of the flood damage. All of Placentia is built on the beach whereas most of Jerseyside is built on higher land north of the beach. The most recent flood events: January 10 and 16, 1982 and December 22 and 25, 1983 were the worst floods experienced.

A series of oceanographic, hydrologic and hydraulic studies were undertaken together with the field program to determine causitive factors of the flooding and to develop measures for flood control. These studies determined that the major contributor to flooding was tidal with some influence from storm surge, freshwater inflow and local wind effects on the Arms. It was determined that high water levels in the Arms are caused by the occurrence of extremely high water levels in Placentia Road which, in turn, are caused by the combined effect of tide and surge in Placentia Bay. The maximum water levels attained in each of the Arms is dependant upon the hydraulic behavior of the estuarial system acting in response to tidal forcing.

Annual high water level events in Placentia Road were estimated from statistical correlation with available tidal data from Argentia. The freshwater inflows into the Arms during these events were simulated from Argentia precipitation data using a hydrologic model (HYMO), (for periods prior to 1979) and from flow records on Northeast River (after 1979). The freshwater inflow data from the hydrologic model for these events was input into a hydraulic model, DWOPER, together with physiographic and

hydrographic data collected during the field program, to develop an annual series of maximum water levels in each of the different geographical regions in the study area.

A frequency analysis (using the Gumbel distribution metho^d) was carried out on the annual series of maximum water levels generated by DWOPER. From this analysis, the 1 in 20 year and 1 in 100 year water levels for Northeast Arm, Swan Arm, and Southeast Arm were derived. The DWOPER model was not able to compute the effect of wind on the Arms so this factor was reviewed separately and the results added to the DWOPER results to provide the 1 in 20 year and 1 in 100 year flood contours.

The sensitivity of water levels in the Arms to variations in the input parameters of the hydraulic model were analysed and where necessary, the flood contours were adjusted to provide the recommended values presented in Table 6.4.1.

Several structural options for flood control were reviewed. The viable options were considered separately or in combination with others to provide alternative flood control measures for each geographical region. A preliminary benefit-cost analysis was carried out on each alternative. The following table summarizes the structural alternatives and the results of the benefit-cost analysis for each alternative for both the 1 in 20 year and 1 in 100 year flood events.

Region	Alternative	Description	Benefit-Cost Ratio 1:20	Benefit-Cost Ratio 1:100
1	1A	Raise and/or floodproof Buildings.	.88	.87
	1B	Sheet pile wall along Narrows plus wave wall on beach west of Placentia.	.42	.38
	1C	Raise Riverside Drive plus wave wall on beach west of Placentia.	2.72	1.39
2	2A	Raise and/or floodproof buildings.	.10	.02
	2B	Wavewall west of Jerseyside plus breastwork east of Jerseyside.	.01	.01
3	3A	Raise and/or floodproof buildings.	.10	.04
	3B	Dyke in Southeast Arm plus wave wall on beach west of Placentia.	.01	.01
4	4A	Raise and/or floodproof buildings.	---	.03
	4B	Wave wall on beach west of Placentia.	---	.01

Non-structural options were also considered. These are not flood control measures as such but are discussed herein as ways to reduce future flood damage. The non-structural options considered were:

(a) Early warning (flood forecasting) and contingency planning measures, and

(b) Property acquisition and zoning.

The findings of the study to this point were then submitted to the Technical Committee for review and selection of the alternatives to be investigated in more detail.

Following the review by the Technical Committee, three alternatives were selected for more detailed investigation of costs. A more detailed benefit-cost analysis was carried out to re-evaluate the selected alternatives. The following table summarizes the selected structural alternatives and the results of the benefit-cost analysis for both the 1 in 20 year and 1 in 100 year flood events.

1 in 20 Year Event

Region	Alternative	Description	Benefit-Cost Ratio
1	1	Raise Riverside Drive and 5 Buildings	3.1
2 & 3	2	Raise 4 Buildings in Regions 2 and 3	0.3
1,2,3&4	3	Alternative 1 and 2 combined	3.0

1 in 100 Year Event

Region	Alternative	Description	Benefit-Cost Ratio
1	1	Raise Riverside Drive, 5 Buildings and Construct Wave Wall	2.2
2 & 3	2	Raise Selected Buildings in Region 2 and 3	0.2
1,2,3&4	3	Alternative 1 and 2 combined	2.1

The benefit-cost ratios given are for a 10% social discount rate, although the study also looked into the sensitivity of the findings to variation in the discount rate.

Non-structural alternatives were investigated in more detail, including suggestions for the implementation of a monitoring and flood forecasting program. The non-structural measures also included zoning, building regulation and contingency planning.

The intangible benefits of flood damage reduction were also investigated. These included the following:

i) improved safety for local residents,
ii) reduced health risk,
iii) improved accessibility to coastal areas,
iv) improved quality of living standards,
v) reduced inconvenience associated with flood events,
vi) employment during construction of damage reduction measures,
vii) improved general economic development, and
viii) reduced erosion and sediment deposition problems.

The intangible benefits are those which cannot be assigned a monetary value so were not included in the economic analysis. These benefits should, however, be considered in the evaluation of the flood damage reduction alternatives.

The final recommendations of this study are:

1. Implement Alternative 1 for the 1 in 100 year flood event. This includes raising Riverside Drive and constructing a wave wall on Placentia Beach to eliminate flooding in Regions 1 and 4.

2. Implement Alternative 2 for the 1 in 100 year flood event. This includes raising buildings in Regions 2 and 3 and will reduce flood damages in these Regions.

3. In addition, implement the non-structural measures of monitoring/flood forecasting and contingency planning for the next flood season and continue these measures after the structural measures are completed, to reduce flood damage for flood events in excess of the 1 in 100 year return period. Implement the non-structural measures of zoning and building regulations immediately.

4. It is recommended that semi-annual inspections be made of the Placentia beach and annual inspection made of the Gut and Narrows. These inspections should include beach profile surveys and inspection of beach conditions for signs of erosion.

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PART ONE

INTRODUCTION

1.1 PROJECT AUTHORIZATION

On 1983 10 04, ShawMont Martec Limited was authorized by the Newfoundland Department of Environment, on behalf of the Canada-Newfoundland Flood Damage Reduction Program, to carry out a hydrotechnical study of the Placentia Area flood plain.

1.2 SCOPE OF WORK

The purpose of the study was to assess the flooding problem in the Placentia area, to determine the 1:20 year and 1:100 year flood contours and to recommend remedial measures (structural and non-structural) to minimize future flood damage. The study was carried out in two phases.

Phase I is the investigative and preliminary analyses phase with suggested alternative measures for flood damage reduction. In particular, this phase of the study included the following items of work:

1. Collection and review of existing information,
2. A field program including:
   -- topographic and hydrographic surveys,
   -- investigation of the flooding history,
3. An evaluation of the significance of factors affecting flooding,
4. Office studies as necessary to fill data voids and to develop an annual series of water levels at various locations throughout the study area,
5. Determination of the 1 in 20 and 1 in 100 year flood lines and delineation of these on available mapping,
6. An analysis of the sensitivity of the 1 in 20 and 1 in 100 year flood lines to variations in the factors which affect flooding,
7. An evaluation of the impact, if any, of ice on the extent of flooding,
8. An evaluation of existing town by-laws and building regulations with respect to land use and construction in the flood plain,

9. Evaluation of the effectiveness of existing structures in minimizing flood damage,

10. A detailed sensitivity analysis of flood levels,

11. An assessment of previous and potential future flood damage,

12. A preliminary benefit/cost analysis of alternative methods for flood control, and

13. Preparation of an Interim Report presenting all alternative flood control measures with recommendations on which alternatives should be studied further.

Phase II, the detailed analysis phase of the study was completed after review of Phase I by the Technical Committee and upon receipt of direction from the Technical Committee as to which alternative or alternatives was to be developed in detail. In particular, this phase of the study included the following items of work:

1. A more detailed benefit/cost analysis of the selected alternatives for flood control,

2. A discussion of non-structural alternatives,

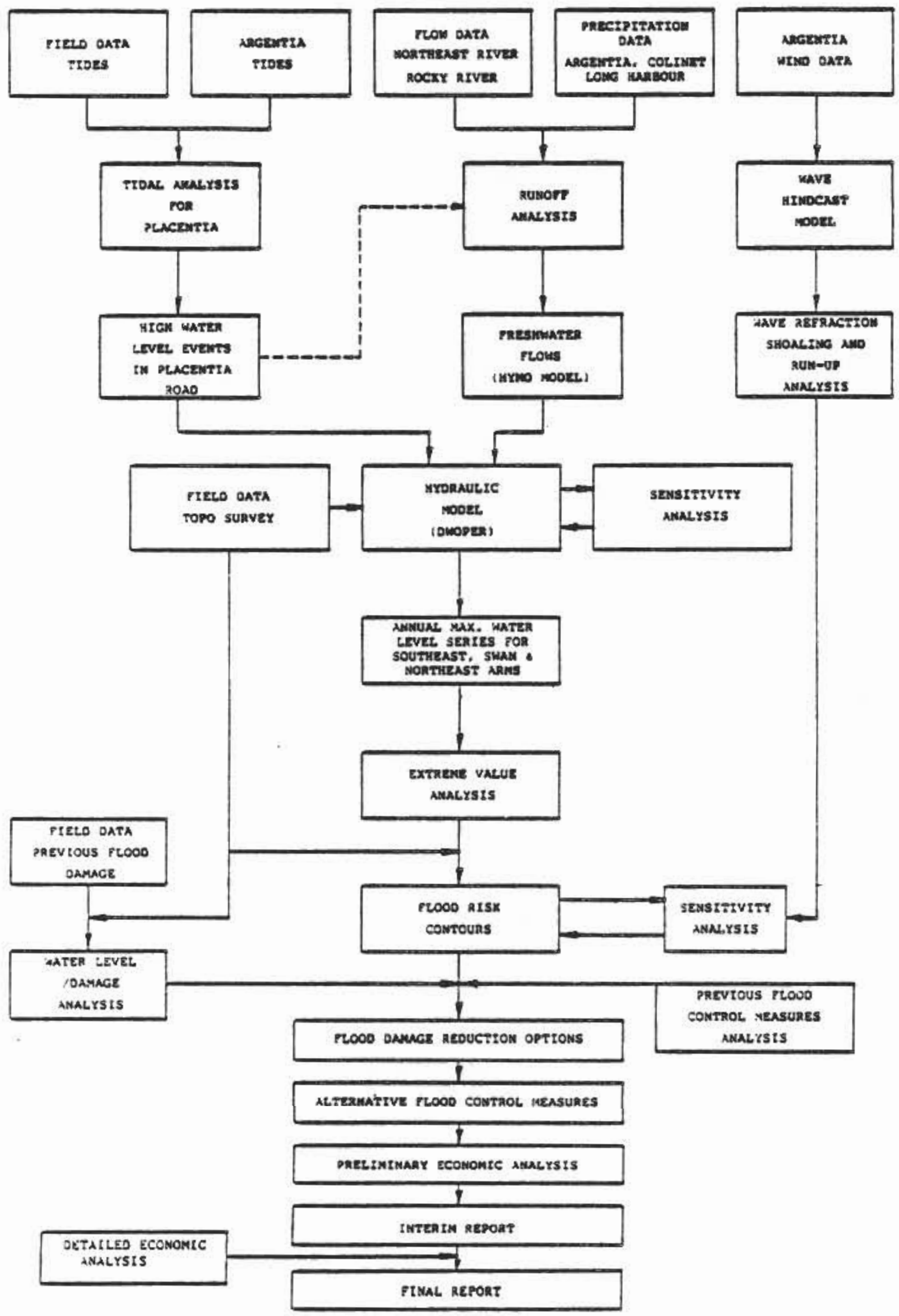
3. Preparation of a final report to provide recommendations on the scheme of development for minimizing flood damage.

The relationships between the above steps in the study are summarized in the flow chart shown in Figure 1.2.1.

1.3 METHODOLOGY

Tides, wind, waves and freshwater inflow into Northeast Arm, Swan Arm and Southeast Arm combine in a complex manner to produce high water levels and flooding in portions of the Towns of Placentia and Jerseyside. The interrelationship between some of the causitive factors are probably independent (eg. tides and freshwater flow) while a degree of interrelationship may exist between other factors - for instance, wind setup and storm rainfall. These interrelationships pose difficult methodological problems for the analysis which attempts to examine each factor independently and then predict occurrence of extreme water levels by joint probability. The approach chosen for this study attempts instead, to preserve the combinations of these factors that have actually occurred

FIGURE 1.2.1



PLACENTIA HYDROTECHNICAL STUDY

FLOW CHART

1.3 METHODOLOGY (Cont'd)

in nature. This involved the estimation of a series of annual water level maxima from historic, tide, weather and flow records in the area. An extreme value (frequency) analysis was then carried out using the annual maxima from which 1 in 20 year and 1 in 100 year flood levels were determined.

Estimation of past annual maxima was done by a computer model which simulates the hydraulic behavior of the Placentia estuarial system in response to given tidal water levels in Placentia Road, and freshwater inflows to each of the Arms. The effect of waves and wave run up for the same period was then superimposed to estimate maximum water levels at selected locations in the study area. These simulations were carried out so as to produce a series of annual maximum water levels for a study period of twelve years, 1972 - 1983.

In the absence of measured data for input to this model, the required input data was derived by correlation with data from other locations. To do this a series of oceanographic, hydrologic and hydraulic studies was required in addition to a field measurement program and review of the history of flooding in the area.

Upon completion of these investigative studies, a study of alternative flood control measures was undertaken.

A program of work to meet the objectives of this study is outlined below.

1.4 PROGRAM OF INVESTIGATIONS

A three part program of work was undertaken as follows:

- Field Program
- Investigative Studies
- Evaluation of Alternative Flood Control Measures.

1.4.1 Field Program

Detailed topographic and hydrographic surveys were carried out to confirm details shown on available mapping and to determine the interrelationship of water levels throughout the study area. The history of flooding in the area was investigated through consultation with residents and various officials.

1.4.1 Field Program (Cont'd)

The detailed results of the field program are outlined in the Field Program Report (Volume 2). The results of the field program are discussed herein and were used in the preparation of the various sections of this report.

1.4.2 Investigative Studies

The following investigative sub-studies were carried out for Phase I of this study:

(i) The hydrology of the region was studied to determine the influence of freshwater inflow and its effect on water levels of the study area. Fresh water inflows were based on stream flow data collected on Northeast River for the period 1979 - 1983. Prior to 1979 stream flow sequences were generated by a hydrologic computer model (HYMO) utilizing precipitation data collected in the Placentia area. The hydrologic model was used to determine the freshwater inflow into the Arms during particular high water level events. The HYMO output was then input into the hydraulic model of the system to provide the influence of the freshwater.

(ii) The oceanography of Placentia Bay in general, and Placentia Road in particular, was studied to determine an annual series of maximum water levels in Placentia Road. The long term tide and weather data collected at Argentia was accessed and correlated to Placentia using the one month of tide and wind records collected at Placentia during the field program. A long term record of tide levels was then developed for Placentia.

(iii) The effect of waves on the beach at Placentia was determined by an analysis of the wave climate of the region. A long term record of possible waves in Placentia Road was developed from historical records by numerical computations, called hindcasting of waves. Waves were hindcast for that sector of Placentia Bay from which the waves could propagate into Placentia Road. A wave refraction, shoaling and run-up analysis was then carried out to determine the run-up on the beach of the significant wave for the 1 in 20 and 1 in 100 year events.

1.4.2 Investigative Studies (Cont'd)

(iv) A hydraulic model (DWOPER) was developed to model the hydrodynamics of the region with tidal boundaries input from the oceanography study and freshwater inflow input from the hydrology study. Physical parameters were input from field measurements and data taken from hydrographic charts of the region. An annual series of maximum water levels was determined for Northeast Arm, Swan Arm and Southeast Arm, from which a frequency analysis determined the 1 in 20 and 1 in 100 year water levels.

(v) The sensitivity of the various factors affecting the predicted water levels was analysed and the 1 in 20 year and 1 in 100 year flood risk contours established on available mapping.

(vi) An assessment of damage from previous floods was made, previous remedial measures evaluated and flood control options identified. A preliminary benefit cost analysis was carried out and alternative remedial flood control measures recommended.

Following review of the Phase I work and the recommended alternative remedial flood control measures by the Technicial Committee, the following investigative sub-studies were carried out for Phase II of this study:

i) A more detailed benefit/cost analysis of selected alternatives for flood control.

ii) A review of non-structural alternatives for flood control.

iii) An investigation into the intangible benefits of flood damage reduction.

PART TWO

INVESTIGATION OF FLOODING PROBLEM

2.1 GENERAL

The Town of Placentia, and to a lesser extent, the Town of Jerseyside, has experienced minor flooding as a common occurrence over the decades. The first flooding noted was in 1904 but, without doubt, flooding was experienced before then. There are few historic records of flooding events in Placentia. Conclusions reached on historical flooding events were based on information provided by the residents of the area.

The larger amount of damage, as a result of flooding, has occurred in Placentia with only a small number of Jerseyside residents being affected. This is because most of Placentia is built on low lying land whereas most of Jerseyside is on higher ground. As noted in the Field Program Report, the value of property damage has generally increased with each succeeding flood because new construction has increased the number of buildings affected and a rise in the standard of living has increased the amenities to which residents have become accustomed.

2.2 PHYSICAL DESCRIPTION OF STUDY AREA

The terrain in the Placentia area is generally very rugged with high hills sloping steeply to sea level. The Town of Placentia and a small part of the Town of Jerseyside are located on a wide, low lying and generally flat expanse of beach at the eastern end of an inlet off Placentia Bay, called Placentia Road. The larger part of Jerseyside is located on higher ground just to the north of the beach. Drawing 1 in Appendix VI shows a general layout of the area.

The beach is almost completely surrounded by water with Placentia Road to the west and three bodies of water to the east. These bodies of water, called Northeast Arm, Swan Arm and Southeast Arm are interconnected by channels. Northeast Arm and Swan Arm are connected by a long, narrow and relatively shallow channel called The Narrows, which is along the east side of the beach. Swan Arm is connected to Southeast Arm by a short and relatively deep channel called MacDonald Gut. A narrow opening through the beach near its north end, called The Gut, allows the exchange of tidal water between Placentia Road and Northeast Arm and thence into Swan and Southeast Arms. A three span steel bridge across The Gut connects the Towns of Placentia and Jerseyside by road. At the south end of the beach, in an area called the Blockhouse or The Neck, a narrow section of beach separates Placentia Road from Southeast Arm and joins the beach to the higher ground to the southwest.

2.2 PHYSICAL DESCRIPTION OF STUDY AREA (Cont'd)

Two large rivers with Atlantic Salmon populations flow into the Arms from the east. Northeast River with a drainage area of 89.6 km^2 flows into Northeast Arm and Southeast River with a drainage area of 143.2 km^2 flows into Southeast Arm. A smaller river with a drainage area of 12 km^2 flows into Swan Arm.

The beach was apparently created by a depositional process caused by the interaction between Placentia Road and the Arms. The beach is comprised of a variable mixture of fine sand and rounded cobbles ranging from 100 percent fine sand, with a relatively low permeability, to 100 percent rounded cobbles, with a relatively high permeability. The western side of the beach, toward Placentia Road, is in a dynamic zone, being exposed to intense wave action created by winds from the west. This side of the beach is relatively high, being about 3 - 4 metres above mean sea level. The beach slopes uniformly down towards the three Arms where it is about one metre above mean sea level. Drawing 2 in Appendix VI shows a detailed layout of the study area complete with surface contours depicting elevations related to Geodetic Datum.

The extent of development in the study area is also shown on Drawing 2. The original development in Placentia was concentrated on the east side of the beach, along The Narrows. This is where most of the older buildings are located. Later development spread to the southwest, near the cental part of the beach. The latest development is in the area bounded by High Road, Blockhouse Road and Beach Road, to the southwest of Dixon Hill. A general paucity of developable land in Placentia is forcing development southeast of Blockhouse Road on the low lying land bordering Southeast Arm. In Jerseyside, there are a small number of buildings on the low lying beach bordering Northeast Arm. The majority of development in Jerseyside has been on the higher ground to the north.

2.3 INVESTIGATION OF PREVIOUS FLOOD EVENTS

To develop a history of previous flood events, officials from various regulatory agencies and a cross-section of residents from all areas of Placentia and the affected area of Jerseyside were interviewed. A detailed list of those interviewed is contained in Appendix I of the Field Program Report.

The earliest flooding reported was on February 3, 1904. Other floods, recalled by some of the more senior residents, occurred in September, 1955 and the winter of 1960. More recently, there was flooding on March 18, 1976, January 20, 1977, January 10 and 16, 1982 and December 22 and 25, 1983.

2.3 INVESTIGATION OF PREVIOUS FLOOD EVENTS (Cont'd)

The Town of Placentia has experienced the greatest amount of damage due to flooding in the area. Only a small amount of damage has occured in the Town of Jerseyside. Residents in Jerseyside reported that, during violent storms, waves overtop the seawall on the seaward side to the west of the Town with the water then flowing across the area to Northeast Arm. They also report that high water levels in Northeast Arm have caused flooding in the homes adjacent to the shoreline of Northeast Arm, just north of the bridge.

The worst flooding experienced by the Towns of Placentia and Jerseyside was during the more recent events. The Field Program Report describes these flood events. However, to apprise the reader, these events are described below.

January 10, 1982

Residents reported that on this date an unusually high tide, accompanied by generally southeasterly winds, resulted in waves overtopping the breastwork along the east side of Placentia. The event occurred between 10:00 AM and 1:00 PM and in the central part of the Town the water rose to the lower floor level of most homes in a matter of minutes. The duration of flooding of homes varied, depending on their elevation, but was approximately 1 to 3 days. In the north end of Placentia, and the southeast end near Swan Arm, the water ponded in the lowest areas. In some cases water was present for up to two weeks due to the residual water from this flood being supplemented by a second flood a week later. The flooded area extended over the central and eastern areas of the Town, westward to the Highroad. This road provided a natural barrier and limited the areal extent of flooding. Several homes near The Neck were also flooded at this time. These homes apparently have a recurring problem of flooding from Southeast Arm even when most parts of the Town are dry.

Based on eye witness reports, the maximum water levels during this flood were established as:

Location	Maximum Water Level (metres above geodetic)
Northeast Arm	1.7 m
Swan Arm	1.7 m
Southeast Arm	1.3 m

January 10, 1982 (Cont'd)

The approximate areal extent of flooding during this event is shown on Drawing 2 in Appendix VI. The Field Program Report contains photographs of this event.

January 16, 1982

On this date, a winter storm with strong southwesterly winds pounded the beach to the west of Placentia. At approximately midnight, the waves broke through the beach and washed across the Beach Road in the area of Laval High School. Water flowed northward and eastward across the low ground between the Beach Road and the Highroad and across the lower sections of the Town to the north and east. The higher ground near the toe of Dixon Hill provided a natural divide and water flowed southward into Southeast Arm and eastward into The Narrows and Swan Arm. The water ponded in the lowest areas which, in some cases, were still underwater from the flood a week earlier. Some residents reported residual water in their homes for up to two weeks.

On the same night, the beach was also overtopped by waves near the southwest end of the Town between the District Vocational School and Aylward's Mall. Also, waves overtopped the seawall west of Jerseyside. The water flowed across the ballpark and through this section of the Town where it flowed into Northeast Arm. Some minor flooding of homes was experienced, but for the most part, water flowed past the homes and businesses.

From the comments received during the interviews, it appears that the type of flood experienced on January 16 is a very rare occurrence. One resident, who is 83 years old, said it was the first time he had seen such a flood.

Because of the nature of flooding from the ocean side of Placentia during this particular storm, a maximum water level could not be ascertained. A very large area of Town was affected but, except for the lower sections of Town where water ponded, the water flowed away from the homes relatively quickly.

Subsequent to the two flood events of January, 1982, a representative of the Placentia Town Council was reported to have contacted all of the flood victims. The flood victims were asked to estimate the damage to their property and to identify the height of water adjacent to and inside their homes during the floods. Approximately 130 households responded to the survey and the estimated damage reported was in the order of \$ 300,000.

2.3 INVESTIGATION OF PREVIOUS FLOOD EVENTS (Cont'd)

It was subsequently learned that not all of the flood victims were contacted and that none of the businesses or the hospital were contacted for estimates of damage. The nature of the questionnaire was such that it was impossible to distinguish between damages experienced in each of the January flood events. This is discussed later under benefit/cost analysis in Section 8.

A problem highlighted by most of the residents was that of power outages caused by the flooding. Because flooding occurred during January, the loss of power caused additional hardships to those already affected. Frozen pipes and loss of heat even plagued those fortunate enough to be living in the dry parts of the Town.

December 22, 1983

At approximately 11:00 AM on this date, the water level rose in The Narrows and overtopped the breastwork, initially near the southern end. The water continued to rise rapidly for an hour or two and then receded before late afternoon. It was reported that approximately 75 homes were affected of which 30 had to be evacuated. It was also reported that the water levels experienced during this event were the highest ever experienced.

Based on eye witness reports, the maximum water levels during this flood were established as:

Location	Maximum Water Level (metres above geodetic)
Northeast Arm	1.9 m
Swan Arm	1.9 m
Southeast Arm	1.5 m

The approximate areal extent of flooding during this event is shown on Drawing 2 in Appendix VI. The Field Program Report contains photographs of this event.

There was no formal estimate of damage made following this flood event. The Town advised, however, an approximate estimate of $ 500,000 would be reasonable considering the greater number of buildings affected compared to the January 10, 1982 flood.

December 25, 1983

Only three days after the record high water levels, the breastwork along The Narrows was again overtopped.

2.3 INVESTIGATION OF PREVIOUS FLOOD EVENTS (Cont'd)

It was reported that the maximum water levels reached were similar to those experienced during the flood of January 10, 1982. On this occasion, the situation was aggrevated by frequent power outages caused by storm force winds.

2.4 FACTORS AFFECTING FLOODING

Based on the investigation of past flood events as described in the previous section, it can be concluded that flooding in the Placentia area has occurred from two sources:

(i) high water levels in the Arms (Northeast, Swan and Southeast), and
(ii) waves overtopping the beach to the west.

2.4.1 High Water Levels in the Arms

High water levels in the Arms occur with an extremely high water level in Placentia Road. The high water level in Placentia Road results from the combined effect of astronomical and atmospheric forces. Fresh water inflow into the Arms could be a contributor to high water levels in the Arms, especially if this inflow was high and occurred at the same time as a high water level in Placentia Road. It is expected, however, that the contribution of freshwater inflows is of secondary importance in comparison with tidal effects, since high freshwater inflow has not occurred during past flooding events.

Generally, an extremely high water level in Placentia Road is caused by the simultaneous occurrence of a monthly high tide, and a surge caused by atmospheric pressure and a strong onshore wind. Initial flooding of the Town of Placentia has been experienced with a high water level in Swan Arm and The Narrows, resulting from the high water level in Placentia Road, accompanied by a strong southeasterly wind. The southeasterly wind creates waves in Swan Arm and The Narrows which appear to have worsened the flooding conditions.

Therefore, although high onshore winds initially contribute to high water levels in the Arms, a local wind effect may aggrevate the flooding condition. The effect of winds in the Arms, as a contributing factor to flooding is discussed in Section 6.3.4.

Another factor affecting the water levels in the Arms is the "throttling effect" of the controls in the hydraulic system, that is the tendency of sections with a small flow

2.4.1 High Water Levels in the Arms (Cont'd)

area, such as The Narrows, to restrict the flow of water through the controls. This restriction of flow reduces the fluctuation of water levels. This subject is discussed in detail in Part 5. It is noted, however, that this throttling effect is dependent upon such factors as sedimentation, infilling and dredging, and the coefficients of friction within the system.

2.4.2 Waves Overtopping the Beach

Waves will overtop the beach if the wave run-up on the beach is sufficiently high to exceed the elevation of the top of the beach or to cause erosion and, therefore, a lowering of the top of the beach. The wave run-up is dependent upon the wave height, wave period and water level in Placentia Road. The wave height and period are initially functions of the wind speed and direction in Placentia Bay and the refraction of the deep water waves as they approach Placentia Road. Once in Placentia Road, the wave height and period are mainly dependent upon the water level and the slope of the beach. These factors and how they relate to run-up are discussed in detail in Part 4.

2.4.3 Other Factors

Although not discussed above as a contributing factor to the high water levels experienced during flooding from either of the two principal sources, it is possible that the high water levels could be caused by physiographic uplift. This is considered unlikely, however, since the only possible evidence of this was the recorded earthquake activity in New Brunswick the day before the January 10, 1982 flood. No earthquake activity was reported prior to the other flood events. It is most likely that the occurrence of the January, 1982 flood event the day after the New Brunswick earthquake was coincidental.

PART THREE

HYDROLOGY

3.1 GENERAL

The hydrological investigations for this study were undertaken with the objective of determining the impact of fresh water runoff on the water levels in the study area. To do this it was necessary to develop an appropriate methodology for quantifying the runoff. The runoff was then used in conjunction with the results of the oceanographic investigations (Part 4) as input to the hydraulic modelling of the study area (Part 5).

The freshwater inflow into the study area is from three subbasins which drain into Northeast Arm, Swan Arm and Southeast Arm. Within each of the subbasins is a river which drains a major proportion of the subbasin. As shown in Table 3.1.1, these rivers drain a total of 88% of the study area. Northeast River is the only river within the study area which is gauged, with a recording gauge which was established in 1979.

TABLE 3.1.1

Drainage Areas Within Study Area

Subbasin	Drainage Area(km^2)	River in Subbasin	Drainage Area(km^2)	% of sub-basin total
Northeast Arm	98.1	Northeast River	89.6	91
Southeast Arm	165.2	Southeast River	143.3	87
Swan Arm	16.1	Swan Arm River	12.0	75
Total	279.4	-	244.9	88

3.2 SIGNIFICANCE OF FRESHWATER FLOW

In order to determine the appropriate level of accuracy required for evaluating the freshwater runoff it was first necessary to determine the significance of freshwater flow in past flooding events.

3.2.1 Freshwater Flow During Previous Flooding

Early in the study, four previous flooding events were identified for which an examination of freshwater flow was made. The flow records on Northeast River and Rocky River showed that there was no significant runoff at the time of each flooding event. In fact, the flows during all events were in recession. For the purposes of comparison, a significant event was defined as one representing flow at or close to the recorded annual maximum flow. As shown in Table 3.2.1, all recorded flooding occured during low

3.2.1 Freshwater Flow During Previous Flooding (Cont'd)

freshwater runoff, and it was therefore concluded that freshwater runoff did not appear to have a significant impact on previous flooding events.

The comparison of runoff during each event was made based on Northeast River for all events except the 1977 event which occured prior to gauging on Northeast River. The flow comparison for the 1977 event was based on flow on Rocky River which is not within the study area, but is nearby and has been assumed to have similar flow behavior.

Table 3.2.1

Runoff Characteristics During Previous Flood Events

River	Flood Event	Daily Discharge: Maximum During Event (m^3/s)	Annual Maximum (m^3/s)	% of Annual Maximum
Northeast	Dec.22/83	1.15	43.8	2.6
Northeast	Dec.25/83	1.60	43.8	3.7
Northeast	Jan.10/82	5.90	40.2	15.0
Rocky	Jan.20/77	7.65	76.5	10.0

3.2.2 Impact of High Runoff

Although examination of previous floods indicated that freshwater flow did not significantly contribute to flooding in the past, further investigations were made to determine if a high runoff during a flood event could be significant.

Using regional flood frequency analysis results for the Island of Newfoundland (Canada - Newfoundland Flood Damage Reduction Program, 1984) estimates of the 1 in 20 and 1 in 100 year return period peaks were made for Northeast River, Southeast River and Swan Arm River as shown in Table 3.2.2. A typical storm hydrograph of hourly flow from Northeast River (July 15-17, 1981) was scaled on a direct ratio of drainage area and peak flow to give an approximate hydrograph for use in the initial investigations. Table 3.2.3 shows the development of the factors used in scaling the hydrograph and the hourly calculated flows are given in Table 3.2.4.

Table 3.2.2

Regional Flood Frequency Analysis

	N.E. RIVER	S.E. River	Swan Arm River
Drainage Area (sq. km)	89.6	143.3	12
Mean Annual Runoff (mm)	1557	1557	1557
% Controlled by Lakes or Swamps	67.5	90	50
Shape	1.754	1.824	1.455
Region	South	South	South
Maximum Instantaneous Flood Flow (m^3/s)			
1 in 2 year	92.5	90.5	31.4
1 in 10 year	153.9	141.3	57.9
1 in 20 year	178.1	160.6	69.0
1 in 100 year	235.5	204.7	97.4

Table 3.2.3

Development of Flow Factors

Subbasin	N.E. Arm	S.E. Arm	Swan Arm
Subbasin Area (km^2)	98.1	165.2	16.1
River Basin (km^2)	89.6	143.3	12.0
D.A. Ratio	1.095	1.153	1.342
20 year recurrence Peak Flow (m^3/s)	178.06	160.61	69.04
Factor	2.70	2.57	1.28
100 year recurrence Peak Flow (m^3/s)	235.54	204.71	97.40
Factor	3.57	3.27	1.81

Note: Factor for Peak based on Northeast River maximum hourly flow of 72.2 m^3/s from July 15, 1981 storm event.

Examination of flows on Northeast River and other rivers with similar sized drainage areas show that the instantaneous peak flow is within 1% of the average hourly peak flow.

$$\text{DA ratio} = \frac{\text{DA (subbasin area)}}{\text{DA (river basin)}}$$

$$\text{Factor} = \frac{\text{Peak Flow}}{72.2} \times \text{DA ratio}$$

TABLE 3.2.4

PLACENTIA FLOOD CALCULATION

HOUR	NE RIVER FLOWS	NE ARM	SE ARM	SWAN ARM	1:20 YR TOTAL	NE ARM	SE ARM	SWAN ARM	1:100 YR TOTAL	
		2.70	2.57	1.28		3.57	3.27	1.81		<-- FACTOR
1	5.20	14	13	7	34	19	17	9	45	
2	5.42	15	14	7	36	19	18	10	47	
3	6.15	17	16	8	40	22	20	11	53	
4	7.07	19	18	9	46	25	23	13	61	
5	8.17	22	21	10	54	29	27	15	71	
6	9.33	25	24	12	61	33	31	17	81	
7	10.70	29	27	14	70	38	35	19	93	
8	13.50	36	35	17	88	48	44	24	117	
9	17.60	48	45	23	115	63	58	32	152	
10	22.30	60	57	29	146	80	73	40	193	
11	28.00	76	72	36	183	100	92	51	242	
12	35.00	95	90	45	229	125	115	63	303	
13	40.90	110	105	52	268	146	134	74	354	
14	44.80	121	115	57	293	160	147	81	388	
15	51.60	139	132	66	338	184	169	93	447	
16	63.10	170	162	81	413	225	206	114	546	
17	69.20	187	178	89	453	247	226	125	599	
18	72.20	195	185	93	473	258	236	131	625	
19	66.90	181	172	86	438	239	219	121	579	
20	60.20	163	155	77	394	215	197	109	521	
21	52.90	143	136	68	346	189	173	96	458	
22	46.50	126	119	60	305	166	152	84	402	
23	40.90	110	105	52	268	146	134	74	354	
24	36.00	97	92	46	236	129	118	65	312	
25	31.40	85	81	40	206	112	103	57	272	
26	27.70	75	71	36	181	99	91	50	240	
27	25.90	70	66	33	170	93	85	47	224	
28	24.10	65	62	31	158	86	79	44	209	
29	22.50	61	58	29	147	80	74	41	195	
30	20.80	56	53	27	136	74	68	38	180	
31	19.40	52	50	25	127	69	63	35	168	
32	18.70	51	48	24	122	67	61	34	162	
33	18.00	49	46	23	118	64	59	33	156	
34	17.30	47	44	22	113	62	57	31	150	
35	16.60	45	43	21	109	59	54	30	144	
36	15.90	43	41	20	104	57	52	29	138	
37	15.30	41	39	20	100	55	50	28	132	
38	14.60	39	37	19	96	52	48	26	126	
39	13.90	38	36	18	91	50	45	25	120	
40	13.40	36	34	17	88	48	44	24	116	
41	13.00	35	33	17	85	46	43	24	112	
42	12.60	34	32	16	83	45	41	23	109	
43	12.30	33	32	16	81	44	40	22	106	
44	12.00	32	31	15	79	43	39	22	104	
45	11.60	31	30	15	76	41	38	21	100	

TABLE 3.2.4 (Cont'd)

PLACENTIA FLOOD CALCULATION (Cont'd)

HOUR	NE RIVER FLOWS	NE ARM	SE ARM	SWAN ARM	1:20 YR TOTAL	NE ARM	SE ARM	SWAN ARM	1:100 YR TOTAL
46	11.30	31	29	14	74	40	37	20	98
47	11.00	30	28	14	72	39	36	20	95
48	10.60	29	27	14	69	38	35	19	92
49	10.30	28	26	13	67	37	34	19	89
50	10.10	27	26	13	66	36	33	18	87
51	10.00	27	26	13	65	36	33	18	87
52	9.95	27	26	13	65	36	33	18	86
53	9.86	27	25	13	65	35	32	18	85
54	9.77	26	25	13	64	35	32	18	85
55	9.67	26	25	12	63	35	32	18	84
56	9.58	26	25	12	63	34	31	17	83
57	9.49	26	24	12	62	34	31	17	82
58	9.40	25	24	12	62	34	31	17	81
59	9.30	25	24	12	61	33	30	17	80
60	9.16	25	24	12	60	33	30	17	79
61	8.99	24	23	12	59	32	29	16	78
62	8.84	24	23	11	58	32	29	16	76
63	8.70	23	22	11	57	31	28	16	75
64	8.55	23	22	11	56	31	28	15	74
65	8.40	23	22	11	55	30	27	15	73
66	8.25	22	21	11	54	29	27	15	71
67	8.11	22	21	10	53	29	27	15	70
68	7.96	21	20	10	52	28	26	14	69
69	7.81	21	20	10	51	28	26	14	68
70	7.66	21	20	10	50	27	25	14	66
71	7.54	20	19	10	49	27	25	14	65
72	7.46	20	19	10	49	27	24	14	65

NOTE: ALL FLOWS IN CUBIC METRES/SEC

3.2.2 Impact of High Runoff (Cont'd)

It should be noted that the methods applied here are approximate, and would tend to be conservative, since the actual rise of flow on Southeast River would tend to be slower than Northeast River, so the peaks would not necessarily be coincident as has been assumed. Also the base flow has been factored by the same ratios, rather than being extracted and calculated separately.

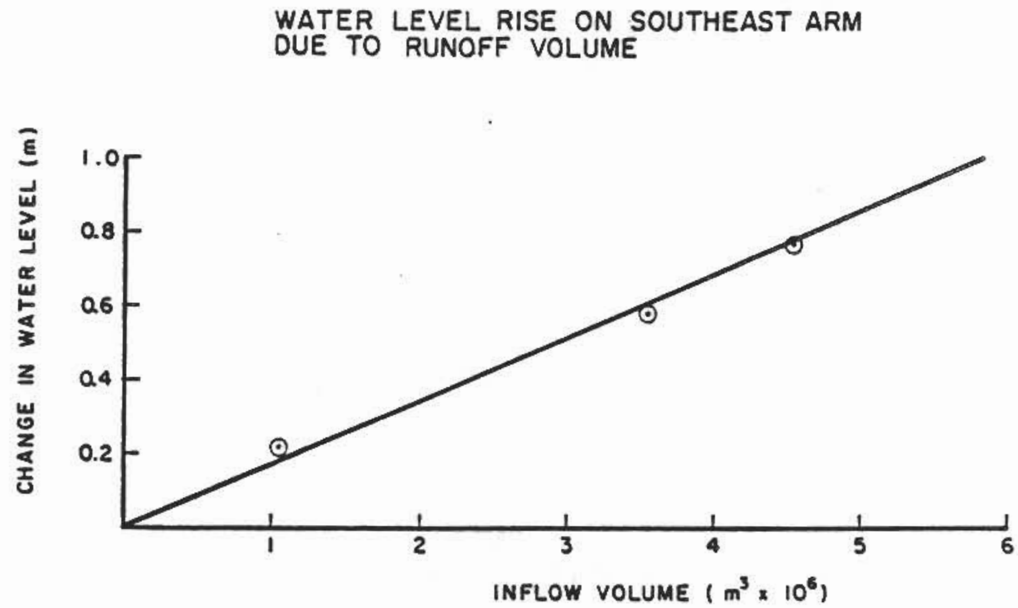
The approximate 1 in 20 and 1 in 100 year storm runoff hydrographs were applied to a normal tidal cycle, as defined in Section 6.3.3, using the hydraulic model. The results of this modelling allowed the plotting of curves for the estimated impact of runoff on waterlevels in the study area (Figure 3.2.1 and Figure 3.2.2). These curves can be used to assess the accuracy required in estimation of the freshwater flows to maintain the objective accuracy of $\pm$ 0.1 m for the flood risk levels. As the volume of a flood increases, the impact of an error in the estimate of the flood volume will increase.

Events within the simulation period of 1972 to 1983 are not major flood runoff events (Section 3.5), and therefore, the accuracy required on the hydrologic flows is not great. Table 3.2.5 shows the impact of variation of the volume estimate for a mean (2 year return) flood event on Northeast River scaled to represent the flow from the entire study area. The behavior of Swan Arm will be very similar to that of Southeast Arm. The method used does not account for an increase or decrease of the surface as the water level changes, but the change in water level is so small that this effect would be negligable. As a result of the analysis of the impact of volume change, an error of 20% has been accepted in this study for the estimation of the freshwater runoff.

Table 3.2.5
Impact of Volume Error in Freshwater Flow

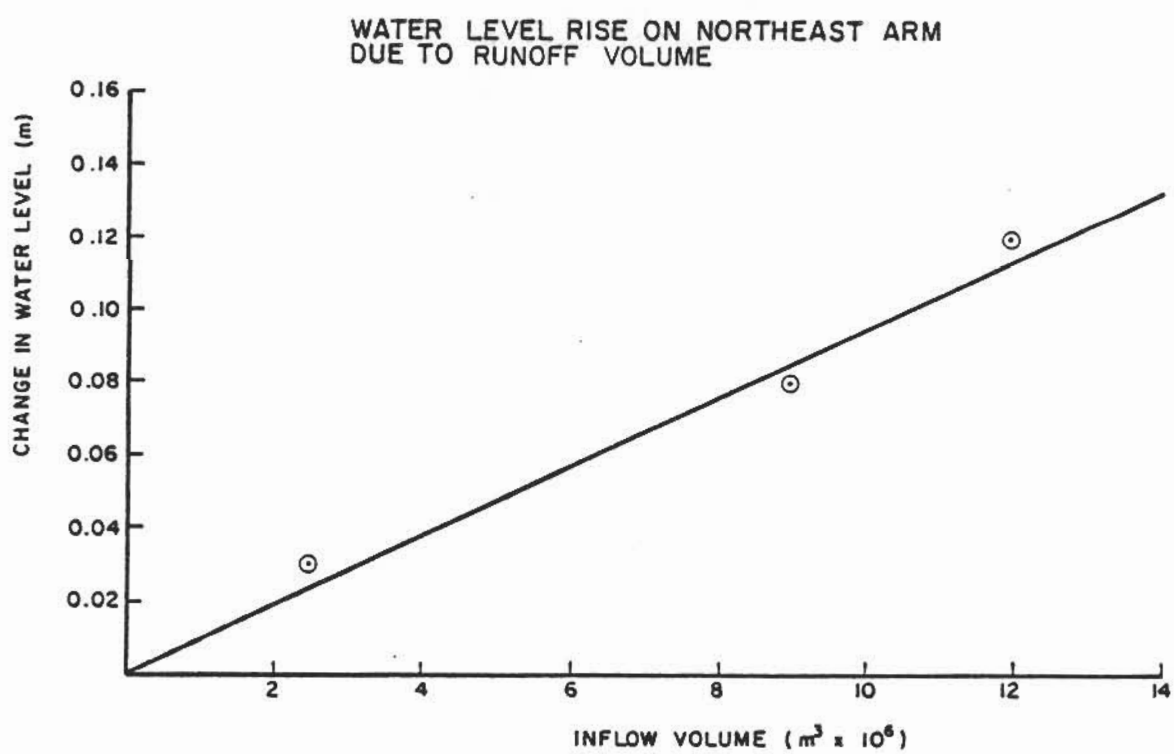
Region:	2 yr peak (m^3/s)	6 hr. volume (x 10^6 m^3)	waterlevel (m)
Southeast Arm	= 121.5	2.3	0.38
	+10%	2.5	0.41
	-10%	2.1	0.35
	+20%	2.8	0.46
	-20%	1.8	0.30
	$\pm$20% approximately 0.38 $\pm$ 0.08 m		
	$\pm$10% approximately 0.38 $\pm$ 0.03 m		
Northeast Arm	247.0	5.4	0.05
	+10%	5.9	0.056
	-10%	4.9	0.047
	+20%	6.5	0.062
	-20%	4.3	0.042
	$\pm$20% approximately 0.05 $\pm$ 0.01		
	$\pm$10% approximately 0.05 $\pm$ 0.005		

FIGURE 3.2.1



WATER LEVEL RISE ON SOUTHEAST ARM

FIGURE 3.2.2



WATER LEVEL RISE ON NORTHEAST ARM

3.3 HYDROLOGIC MODELLING

The freshwater runoff has been shown not to have a significant impact on flooding at Placentia. Its impact, in conjunction with a normal tide, will not produce extreme water levels in the study area. In accordance with the method adopted for extreme value analysis (Part 6), the freshwater runoff was calculated for periods corresponding to the high tide plus storm surge events from the period of record of tidal gauging. These events were identified as part of the oceanographic investigations.

The runoff corresponding to the high water level events are recessional in many cases, as was the case for previous flooding events (Section 3.1.1). The runoff, however, was not expected to be recessional in all cases, and therefore a strategy for producing inflow hydrographs was required. Since Northeast River is gauged, consideration was given to assuming hydrographs based on this river for the entire study area. However, the topographic features of Southeast River and Northeast River are sufficiently different to warrant a method which takes the differences into account.

A hydrologic model which could account for the physical differences between the basins within the study area was required. The HYMO Model (Williams & Hann, 1972) was selected for this purpose. Hymo is a simple, physically based model developed by the U.S. Dept. of Agriculture and represents an extension of the S.C.S. flood hydrograph procedures. The program was developed for use with drainage basins of up to 1,000 km^2. It has been used by ShawMont in previous hydrologic studies in Newfoundland, and has had wide application in Ontario (Perks, 1983). The input for the model includes precipitation, the drainage area, basin slope, and a curve number (representing runoff potential based on the soil/cover characteristic of the basin). All of the required input was derived from available information. The base flow is calculated and added separately to give the total flow.

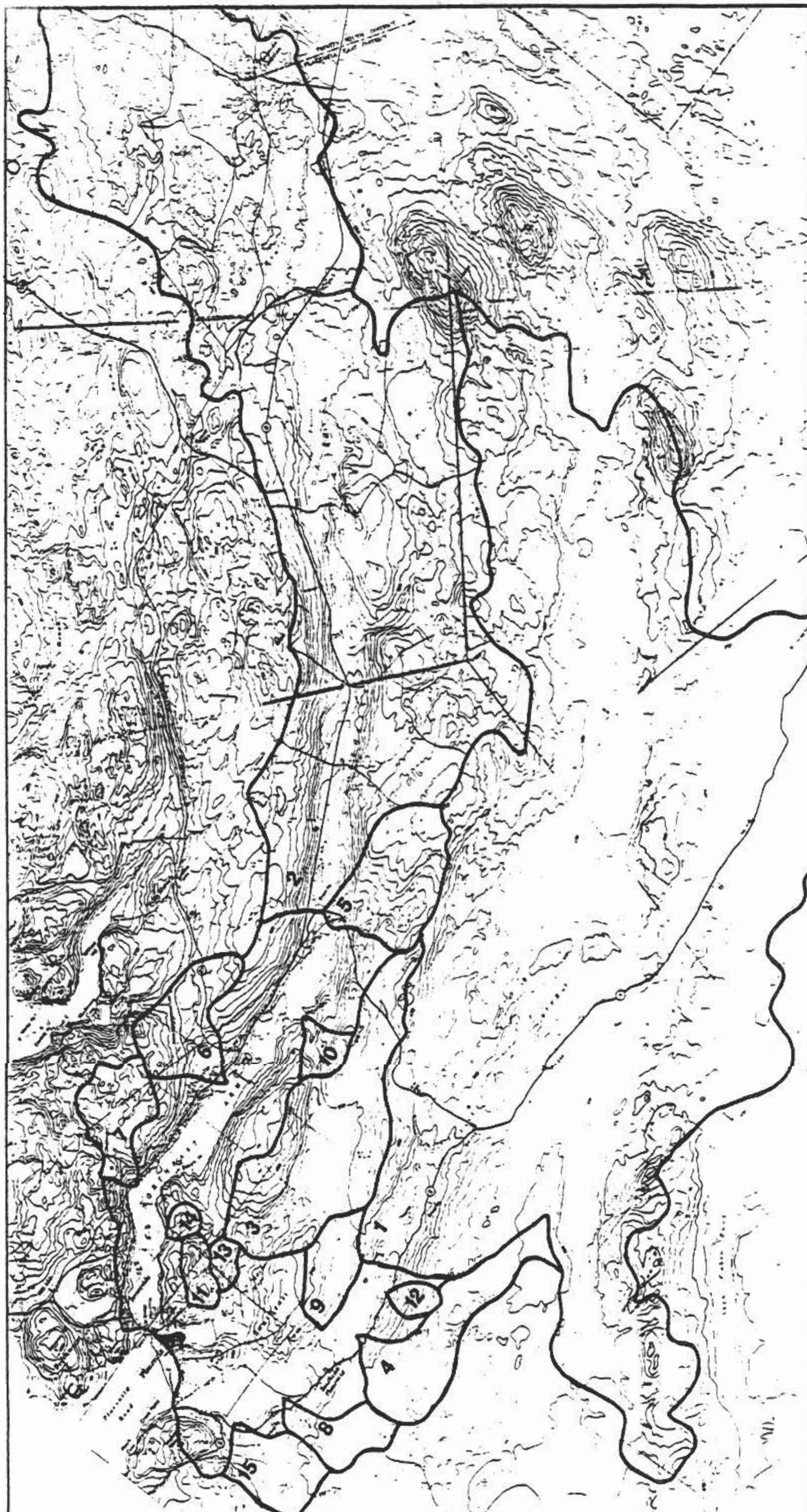
The hydrologic model was used to produce hydrographs for rivers flowing into the study area. It was applied for high water level events within the simulation period for which significant precipitation occured which could cause a high runoff. The significance of the precipitation was to be based on whether it caused a noticable peak on gauged rivers in the area, ie. Northeast River and Rocky River. Precipitation records from Argentia were examined to determine the precipitation for use in the analysis.

3.3.1 Basin Parameters

The physical parameters of the basins were derived from topographical maps of 1:50,000 scale. The freshwater inflow into the study area is from three subbasins which flow into Northeast Arm, Swan Arm and Southeast Arm. Within each of these subbasins is a river which drains a major portion of the basin. In addition, smaller rivers also flow into the Arms, for a total of 15 river draining the study area, as shown in Figure 3.3.1. The drainage areas, length of the main river channel, height from highest point in the basin to the mouth of the river were derived from the maps for input to the HYMO program. The SCS curve number was estimated based on the methods described in SCS National Engineering Handbook, (1972) and available maps of surficial hydrogeology (Altantic Development Board, 1968). The surfical hydrogeology map (Figure 3.3.2) showed the entire study region could be classed hydrologic soil group 'D', that is soils having a high runoff potential. The land use of the area can be described as wood or forest land, thin stand, poor cover, no mulch. From Table 3.3.1 the curve number was estimated at 83. The antecedent moisture condition accounts for the amount of rainfall in the previous 5 to 30 days and, as shown in Table 3.3.2, the CN can vary from 67 to 93.

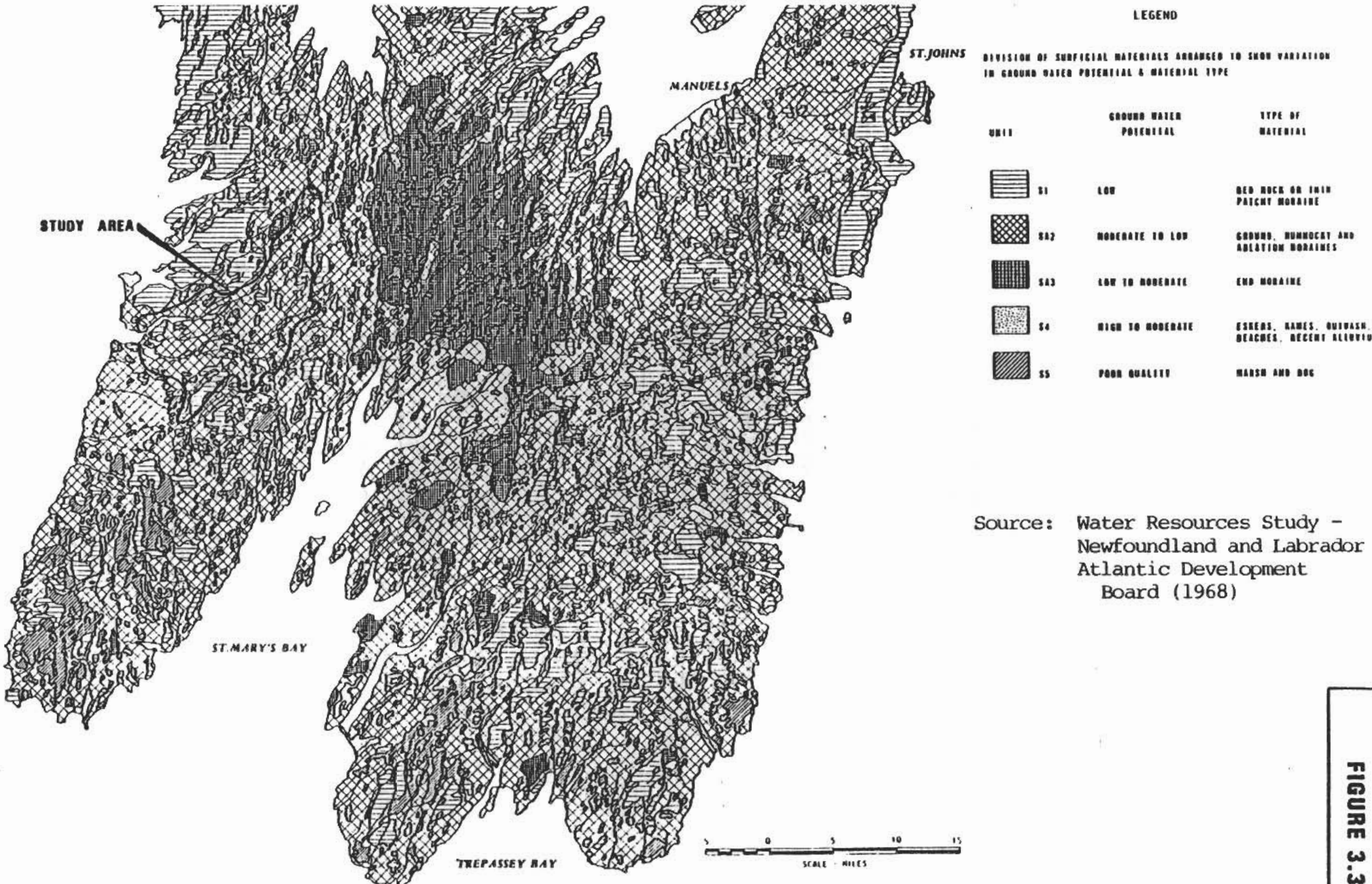
The precipitation records must, therefore, be used to determine the antecedent moisture conditions. The compilation of inflow hydrographs for use by the hydraulic model is done in three sections, representing the flow into Northeast Arm, Swan Arm and Southeast Arm. All small basins, fringe areas and direct runoff are related to these major areas. Preliminary application of formulae used by HYMO showed that, of the 15 basins identified, only three had a time to peak of greater than 1 hour. Basins with a time to peak of less than about 20 minutes were considered to behave as direct runoff on the runoff was calculated using the formula given in Section 3.3.3. This left nine basins to be modelled, three for Northeast Arm, five for Southeast Arm and one for Swan Arm. Table 3.3.3 shows the basin parameters used for HYMO, and the time to peak estimates.

FIGURE 3.3.1



DRAINAGE BASINS OF THE PLACENTIA STUDY AREA

FIGURE 3.3.2



Source: Water Resources Study - Newfoundland and Labrador Atlantic Development Board (1968)

SURFICIAL HYDROGEOLOGY OF THE PLACENTIA STUDY AREA

Table 3.3.1

Runoff Curve Numbers

Runoff curve number for selected agricultural, suburban, and urban land use. (Antecedent moisture condition II, and $I_a = 0.2S$)

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land : without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch	45	66	77	83
good cover	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential:				
Average lot size — Average % Impervious				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads:				
paved with curbs and storm sewers	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

Source : National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972

Table 3.3.2

Curve Numbers (CN) Range for Antecedent Moisture Conditions (AMC)

1	2	3	4	5	1	2	3	4	5
CN for condition II	CN for conditions I	III	S values*	Curve* starts where P =	CN for condition II	CN for conditions I	III	S values*	Curve* starts where P =
			(inches)	(inches)				(inches)	(inches)
100	100	100	0	0	60	40	78	6.67	1.33
99	97	100	.101	.02	59	39	77	6.95	1.39
98	94	99	.204	.04	58	38	76	7.24	1.45
97	91	99	.309	.06	57	37	75	7.54	1.51
96	89	99	.417	.08	56	36	75	7.86	1.57
95	87	98	.526	.11	55	35	74	8.18	1.64
94	85	98	.638	.13	54	34	73	8.52	1.70
93	83	98	.753	.15	53	33	72	8.87	1.77
92	81	97	.870	.17	52	32	71	9.23	1.85
91	80	97	.989	.20	51	31	70	9.61	1.92
90	78	96	1.11	.22	50	31	70	10.0	2.00
89	76	96	1.24	.25	49	30	69	10.4	2.08
88	75	95	1.36	.27	48	29	68	10.8	2.16
87	73	95	1.49	.30	47	28	67	11.3	2.26
86	72	94	1.63	.33	46	27	66	11.7	2.34
85	70	94	1.76	.35	45	26	65	12.2	2.44
84	68	93	1.90	.38	44	25	64	12.7	2.54
83	67	93	2.05	.41	43	25	63	13.2	2.64
82	66	92	2.20	.44	42	24	62	13.8	2.76
81	64	92	2.34	.47	41	23	61	14.4	2.88
80	63	91	2.50	.50	40	22	60	15.0	3.00
79	62	91	2.66	.53	39	21	59	15.6	3.12
78	60	90	2.82	.56	38	21	58	16.3	3.26
77	59	89	2.99	.60	37	20	57	17.0	3.40
76	58	89	3.16	.63	36	19	56	17.8	3.56
75	57	88	3.33	.67	35	18	55	18.6	3.72
74	55	88	3.51	.70	34	18	54	19.4	3.88
73	54	87	3.70	.74	33	17	53	20.3	4.06
72	53	86	3.89	.78	32	16	52	21.2	4.24
71	52	86	4.08	.82	31	16	51	22.2	4.44
70	51	85	4.28	.86	30	15	50	23.3	4.66
69	50	84	4.49	.90					
68	48	84	4.70	.94	25	12	43	30.0	6.00
67	47	83	4.92	.98	20	9	37	40.0	8.00
66	46	82	5.15	1.03	15	6	30	56.7	11.34
65	45	82	5.38	1.08	10	4	22	90.0	18.00
64	44	81	5.62	1.12	5	2	13	190.0	38.00
63	43	80	5.87	1.17	0	0	0	infinity	infinity
62	42	79	6.13	1.23					
61	41	78	6.39	1.28					

*For CN in column 1.

AMC I = SOILS ARE DRY BUT NOT TO WETTING POINT
AMC II = THE AVERAGE CASE
AMC III = HEAVY OR LIGHT RAINFALL AND LOW TEMPERATURES HAVE OCCURRED DURING PREVIOUS FIVE DAYS

Source: National Engineering Handbook, Section 4, Hydrology, Chapter 9, Aug. 1972

Table 3.3.3

HYMO PARAMETERS

Northeast River Basins:

Basin #	Drainage Area (km^2)	Basin Height (m)	Channel Length (km)	Time to Peak*(Hrs)
2	89.6	251	24.0	4.17
5	5.4	206	3.13	0.46
7	3.1	166	3.0	0.42
Fringe	19.2	---	---	----
Direct	6.7	---	---	----
Basin Total	124.0			
Swan Arm Basins				
3	12.0	160	9.0	1.41
Fringe	4.1	---	----	----
Direct	2.5	---	----	----
Basin Total	18.6			
Southeast River Basins				
1	143.2	335	26.6	4.51
4	5.6	145	5.3	0.81
8	2.2	107	2.8	0.45
9	2.3	91	2.4	0.47
15	3.0	107	1.7	0.48
Fringe	8.9	---	----	----
Direct	3.0	---	----	----
Basin Total	168.2			

Study Area Total = 310.8 km^2

Time to Peak calculated using formulae from HYMO Model

3.3.2 Precipitation

There are three meteorologic stations in the vicinity of the study area, these are Argentia, Colinet and Long Harbour. An intial study of the relevance of the precipitation data was made by comparing the runoff volume on Northeast River to the precipitation at each of these stations. This comparison revealed considerable disparity between storm precipitation measurements at each station and also with observed runoff on Northeast River as shown in Table 3.3.4. These differences are due to the sparse coverage of rainfall data in the area (one gauge per 2000 km^2) in comparison with the size of the drainage area of Northeast River (89.6 km^2) and the absence of a rain gauge in the Northeast River basin itself. These disparities frustrated efforts to interpolate between these stations to estimate precipitation on the Northeast River, whether using the Theissen polygon method or multiple regression analysis, most likely resulting from the effect of localized orography and micro-climate. It was finally concluded that the best prediction of rainfall in the Northeast River was the Argentia gauge, which has, accordingly, been used to develop the required storm hydrographs. Comparison on a longer term (monthly) basis between runoff on Northeast River and precipitation at Argentia confirms the consistency of this choice. Although variances of $\pm$ 20% are still possible on the estimated runoffs of individual storms.

Straight line average of the precipitation over 24 hours has been used to derive the small increments of precipitation required by HYMO. This approach is supported by an examination of precipitation records at Argentia which show that most major storms have a duration of 24 hours or more. Use of storm rainfall distributions developed by the Atmospheric Environment Service (Pugsley, 1981) were considered, however, the difference from the straight line was not significant, so the straight line average was used.

3.3.3 Antecedent Conditions

Examination of the hydrographs for the Northeast River indicated that, typically a storm hydrograph would be superimposed on the receding limb of a prior storm hydrograph and rarely on a "true" base flow. To establish these complex antecedent flow conditions by hydrograph simulation alone, would require modelling of extended periods of time prior to each storm event used in the study. Therefore, the approach adopted was to estimate the base flow separately and add it to the storm hydrograph. Comparison of the average monthly discharges on Northeast River and Rocky River indicated similarity of flow

Table 3.3.4

Comparison of Precipitation and Runoff

Event	Precipitation (mm) Argentia	Colinet	Long Harbour	N.E. River Runoff (mm)
79 10 14	69.3	43.9	80.1	34.5
80 12 31	60.7	27.4	90.9	31.8
80 17 07	82.8	62.5	89.9	34.5
80 11 06	77.5	67.3	86.1	59.7
81 10 17	112.0	93.2	145.8	88.4
81 04 22	22.6	22.6	19.1	20.1
82 12 07	48.3	50.3	36.3	22.1

3.3.3 Antecedent Conditions (Cont'd)

patterns with an average correlation coefficient of r = .976 for the 12 months, as shown in Table 3.3.5. This suggested that the observed recession (or base) flows on Rocky River could be used to predict antecedent flows on Northeast River for the period prior to 1979. This observation was confirmed by a regression analysis of Northeast River and Rocky River baseflows using daily discharge which gave a correlation coefficient of r = 0.92 which is significant at the 1% level. This approach has been adopted for the estimation of antecedent conditions in the study, using the regression equation given below:

$$\text{N.E. River Base Flow} = 0.286\ (\text{Rocky River Base Flow})^{1.197}$$

where flow is in m^3/s

The base flow was calculated for Northeast River and then estimated for the other basins within the study area on a m^3/s per km^2 basis. Table 3.3.6 shows the data points used in the baseflow comparison, this is plotted in Figure 3.3.3.

The antecedent moisture condition is based on the SCS methods. Rainfall prior to the storm being modelled, increases the runoff potential of the soil and results in an increase in the curve number. A CN value of 90 has been used where there has been rainfall in the previous 5 days, which was the case in all events modelled.

The snowpack and snowmelt were not considered in the modelling because the periods which required modelling did not occur while there was snow on the ground within the drainage basins.

Table 3.3.5

Correlation of Northeast River and Rocky River*

Northeast River Flow	Linear Regression Formula Rocky River Flow	r^2
January	.9765 + .3 Jan	.992
February	3.0617 - .3808 Jan. + .6746 Feb.	.923
March	8.3194 - .3814 Feb. + .1084 Mar.	.955
April	-4.837 + .1379 Mar. + .6934 Apr.	.955
May	-3.1289 + .2735 Apr. + .4307 May	.991
June	-.2220 + .0450 May + .3205 Jun	.999
July	-2.1306 + .1918 Jun. + .7433 Jul	.938
August	2.4784 - .1078 Jul. + .1913 Aug.	.857
September	1.3158 - .2124 Aug. + .4312 Sept.	.984
October	2.0002 - .2426 Sept. + .4327 Oct.	.818
November	-.9712 + .0872 Oct. + .3138 Nov.	.986
December	6.2233 - .2957 Nov. + .2395 Dec.	.998
AVERAGE		.953

* Based on mean monthly values for 1979-1983.

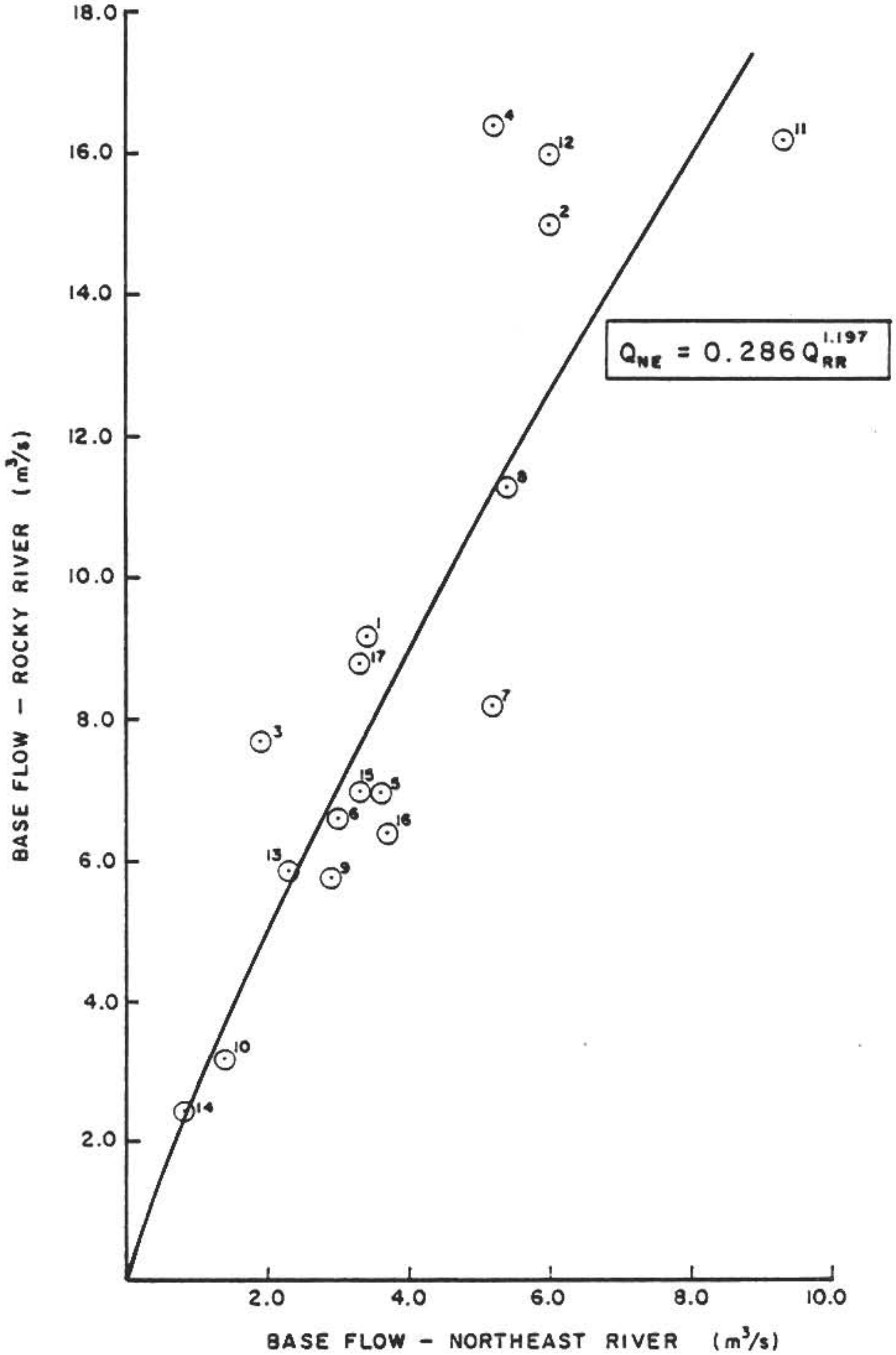
Table 3.3.6
Comparison of Base Flows

Event	Northeast River		Rocky River		
	Base Flow	Peak Flow	Base Flow	Peak Flow	I.D.*
79 12 18	3.44	25.8	9.21	39.4	1
79 10 14	6.01	21.7	15.1	63.1	2
79 01 30	1.93 E	18.8	7.66	45.6 E	3
80 12 31	5.23	23.5	16.4	52.6	4
80 10 07	3.63	23.0	7.01	61.9	5
80 11 06	2.95	22.6	6.62	36.5	6
81 10 17	5.18	36.3	8.20	107	7
81 07 15	5.35	33.4	11.3	41.0	8
81 02 10	2.90	20.3	5.75	87.7	9
82 03 09	1.37 B	40.2	3.20 B	59.0	10
82 04 22	9.28	22.6	16.2	44.1	11
82 12 07	6.07	18.6	16.0	42.3	12
82 07 01	2.32	16.3	5.90	55.7	13
82 11 26	0.842	10.1	2.45	50.7	14
81 07 07	3.34	20.2	7.05	39.3	15
81 10 05	3.73	24.0	6.42	41.7	16
80 03 10	3.27	22.0	8.79	46.3	17

Note - Base Flow is last diminishing value prior to start of rising limb of hydrograph.

* I.D. refers to point numbers of data plotted in Figure 3.3.3.

FIGURE 3.3.3



NOTE:

DATA FOR NUMBERED EVENTS GIVEN IN TABLE 3.3.6

COMPARISON OF BASE FLOWS
NORTHEAST & ROCKY RIVERS

3.4 MODEL CALIBRATION AND VERIFICATION

An initial HYMO model was set up representing the Northeast River, for which stream flow data were available, and the model was calibrated and verified to reproduce the observed behavior of this watershed. The hydrograph parameters obtained were then applied to the remaining sub-basins.

The calibration trials showed the runoff volume to be fairly sensitive to the SCS curve number, which indicates the runoff potential of the soil. A range of 83 to 93 for the curve number was initially selected based on the estimated ground water potential and ground cover. A value of 90 was found to give a good fit on volume for the calibration storm. The built in formulations used by HYMO were found to give a good fit to the initial rise of the hydrograph on Northeast River, but did not accurately model the recession limb of the curve. Factoring the recession constant, which influences the length of the recession limb, was found to provide a good fit on both the hydrograph peak and the recession limb, and it was found that the same factor provided a good fit on the verification hydrograph, as well.*

The model requires that the time interval for precipitation be less than 25% of the time to peak and therefore the 24 hour precipitation had to be divided into smaller increments (between 0.1 and 1.0 hours) for calculation of the runoff.

The high flow event of June 30, 1982 was selected for calibration. It was found that the best fit occured using 24 hour precipitation and a curve number of 90. The peak flow was found to be 30.1 m^3/s vs 28.9 m^3/s measured, runoff volume was found to be 21 mm vs 24.4 mm measured. The same parameters were taken and applied to other storms to verify the model. The events selected for verification were major solitary rainfall storms (no snow melt). Verification was made on a 24 hour average flow basis, using total 24 hour precipitation as input. Verification was performed by modelling 3 other storms, the comparison of flows was made on a 24 hour average flow basis, rather

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* Long recession periods have been observed on several Newfoundland rivers in previous studies by ShawMont Newfoundland Limited and are thought to be due to lake (and swamp) recession characteristics which are not directly considered in HYMO.

than a more rigourous hourly basis, but the hydrographs produced are shown to be quite close to the recorded, and, in view of the small impact that an average high flow event will have on the flooding at Placentia, using 24 hour average flows was considered satsifactory. Table 3.4.1 shows the calibration and verification results. Appendix I contains the output from the model for the calibration and verification runs, as well as graphs showing the comparison to the measured runoff.

3.5 STORM COMPILATION

As shown in section 3.2 comparison of the effects of tidal and freshwater inflows in producing flood conditions indicated that freshwater flow was not the prime contributor to such conditions. Accordingly, to produce the annual series of maximum water levels the three major tidal events for each year of the study period, 1972 - 1983 were identified and the corresponding fresh water inflows derived. Many of these events did not correspond with rainfall or snowmelt floods in the area. In such cases the prevailing recession (or base flow) was estimated using the method described in Section 3.4.3 and assumed to remain constant through the modelling period.

In the events which coincided with rainfall/snowmelt storms, freshwater inflows were generated at one hour intervals for input to the hydraulic model.

As shown in this Table 3.5.1 only one of the periods selected for hydraulic modelling had a corresponding runoff event. For this event the HYMO model was applied to each of the 9 subbasins within the study area, and the runoff added to produce the inflows for the hydraulic model. Table 3.6.2 shows the compiled flood. Appendix I, contains the output from HYMO for these basins. The flow into Southeast Arm shows a peak of 61.6 m^3/s, with an approximate 6 hour inflow volume of 1.2 x 10^6 m^3, this *corresponds to an estimated impact on Southeast Arm* of 0.19 m (Figure 3.2.2). An error of $\pm$ 20% on the volume estimate, would result in a change in this impact of only $\pm$ 0.03 m, showing that an error in the estimate of runoff in this case is not very significant.

The characteristics of the freshwater flows used in the hydraulic model (Part 5) are summarized in Table 3.5.1. The flows were calculated at hourly intervals for inclusion in the hydraulic model.

Table 3.4.1

CALIBRATION AND VERIFICATION OF HYMO

Storm Peak Date	Peak Flow Actual	Peak Flow Model	Diff	Runoff Actual	Runoff Model	Diff	Ppt
	m^3/s	m^3/s	%	mm	mm	%	mm
June 30/82	28.9	30.1	+ 4.2	24.4	21.1	- 13.5	43.2
Oct. 7/80	23.0	21.0	- 8.7	34.5	37.1	+ 7.5	59.2
Oct. 17/81	36.3	37.5	+ 3.31	88.4	81.3	- 8.0	101.4
Oct. 14/79	21.6	24.0	+ 11.1	34.5	32.5	- 5.8	56.6

Note: June 30/82 storm peak used for calibration of model, other storms were used for the verification.

Table 3.5.1

SUMMARY OF RUNOFF CHARACTERISTICS

Computer Run	Start Date	Duration (Hrs.) *	Inflow (m^3/s) ** N.E.	Swan	S.E.	Storm Runoff (mm)
PRO 21	25-02-72	144	5.18	0.71	7.31	Recession Flow Only
PRO 17	18-01-73	144	3.23	0.45	4.53	Recession Flow Only
PRO 10	08-01-74	120	2.46	0.34	3.45	Recession Flow Only
PRO 23	28-01-75	120	2.12	0.28	2.97	Recession Flow Only
PRO 6	05-10-75	168	49.72	9.37	61.59	38
PRO 14	15-03-76	120	7.14	0.99	10.05	Recession Flow Only
PRO 16	16-12-76	120	7.45	1.02	10.48	Recession Flow Only
PRO 18	18-01-77	144	4.56	0.62	6.43	Recession Flow Only
PRO 24	30-11-78	120	3.14	0.42	4.42	Recession Flow Only
PRO 22	27-01-79	144	17.47	2.41	24.58	Recession Flow Only
PRO 2	03-01-80	120	4.70	0.65	6.71	Recession Flow Only
PRO 13	15-02-80	120	5.49	0.76	7.73	Recession Flow Only
PRO 12	08-12-81	168	6.29	0.85	8.83	Recession Flow Only
PRO 11	08-01-82	120	8.44	11.61	11.89	Recession Flow Only
PRO 25	20-12-83	192	2.01	0.28	2.83	Recession Flow Only

* Duration is the number of hours for which flows were synthesized, based on the period required for modelling water levels with the hydraulic model (Part 5)

** Inflow is the highest inflow during the period.

TABLE 3.5.2

COMPILED STORM RUNOFF

NORTHEAST ARM FLOWS (M↑3/S)

22.02	22.41	22.75	23.15	23.52	23.89	24.25	25.70
27.14	28.58	30.05	31.50	32.94	34.47	36.00	37.50
39.03	40.55	42.08	43.33	44.60	45.87	47.15	48.42
49.69	45.56	41.43	37.30	33.17	29.06	24.93	23.83
22.72	21.62	20.55	19.44	18.34	17.63	16.95	16.27
15.57	14.89	14.18	13.78	13.33	12.90	12.48	12.08
11.66	11.40	11.18	10.95	10.70	10.44	10.22	9.96
9.85	9.74	9.62	9.51	9.42	9.37	9.31	9.25
9.20	9.14	9.08	9.03	8.97	8.91	8.86	8.83
8.77	8.72	8.69	8.63	8.57	8.55	8.49	8.46
8.43	8.38	8.35	8.32	8.26	8.24	8.21	8.18
8.15	8.12	10.92	8.07	8.04	8.01	7.98	7.95
7.92	7.90	7.87	7.84	7.81	7.78	7.75	7.73
7.70	7.67	7.64	7.61	7.58	7.56	7.53	7.50
7.47	7.44	7.41	7.39	7.36	7.33	7.30	7.27

SWAN ARM FLOWS (M↑3/S)

4.84	4.92	5.01	5.07	5.15	5.24	5.32	5.63
5.91	6.23	6.51	6.82	7.10	7.33	7.56	7.78
8.04	8.26	8.49	8.63	8.77	8.91	17.57	9.23
9.37	8.26	7.16	6.08	4.98	3.88	2.77	2.60
2.46	2.29	2.12	1.98	1.81	1.75	1.73	1.67
1.61	1.58	1.53	1.50	1.47	1.44	1.39	1.36
1.33	1.30	1.27	1.25	1.22	1.19	1.16	1.13
1.10	1.08	1.05	1.02	0.99	0.96	0.93	0.93
0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93

TABLE 3.5.2 (Cont'd)

COMPILED STORM RUNOFF

SOUTHEAST ARM FLOWS (M↑3/S)

16.56	17.09	17.60	18.14	18.68	19.19	19.73	22.07
24.39	24.39	26.74	29.09	31.41	33.76	36.31	38.83
41.37	43.92	46.44	48.99	51.08	53.18	55.27	57.36
59.46	61.55	57.93	54.31	50.69	47.06	43.44	39.82
37.92	36.00	34.10	32.18	30.28	28.38	27.20	26.01
24.82	23.63	22.44	21.25	20.55	19.84	19.13	18.45
17.74	17.04	16.67	16.30	15.96	15.59	15.23	14.86
14.69	17.35	14.35	14.15	13.98	13.81	13.73	13.61
13.53	13.41	13.33	13.22	13.13	13.05	12.96	12.85
12.76	12.68	12.59	12.54	12.45	12.37	12.31	12.23
12.17	12.11	12.06	11.97	11.91	11.86	11.80	11.74
11.69	11.63	11.57	11.52	11.46	11.40	11.35	11.29
11.24	11.18	11.12	11.07	11.01	10.95	10.90	10.84
19.27	10.73	10.67	10.61	10.56	10.50	10.44	10.39
10.33	10.27	10.22	10.16	10.10	10.05	9.99	9.93

Note: All flows at hourly intervals.

PART FOUR

OCEANOGRAPHY

4.1 GENERAL

The objective of the oceanographic component of the study was to establish a description of the oceanographic factors that affect the flooding of the Towns of Placentia and Jerseyside.

The two main oceanographic parameters of interest have been identified (from previous flood events) as: (1) extreme water levels in the Narrows and Swan Arm region; and (2) waves on the beach side of the town.

The dynamics of the fluctuations in water levels are analyzed by hydraulic modelling of a portion of Placentia Road and the inner Arms. Input to the hydraulic model are the fluctuations in water elevation in Placentia Road determined in the oceanographic program.

The wave history and wave dynamics on the beach were also determined as part of the oceanographic program.

4.2 OCEANOGRAPHIC INVESTIGATIONS

At the start of the study, a number of alternative methods were considered for the determination of the water level fluctuations in Placentia Road. Separate tide and storm surge mathematical modelling of the Placentia Bay and Placentia Road region was considered. The tides in the Placentia Bay region could be modelled but there was not sufficient tide data at the outer boundry to calibrate a model with sufficient accuracy. In addition, the storm surge in Placentia road was directly driven by the surge in Placentia Bay. Modelling of surge in the Placentia Bay region required more data than was available. Collection of field data was planned only for the Placentia Road region. With the data from the planned field program, in Placentia Road, one could utilize this local data along with long term water elevation data from Argentia to carry out the tide and storm surge studies. Thus after investigation of the data sets available for calibration of models of Placentia Bay, it was decided that more accurate results could be achieved by the use of statistical analysis of available data from Argentia. This data was transferred to Placentia by a comparison with water level data collected in Placentia Road during the field program (ShawMont Martec, 1984). The details of the statistical analysis and the application of Argentia data to the hydraulic model input is described in the subsequent sections.

Wave heights experienced on the beach at Placentia Road have been determined by modelling the wave generation process using wind records as the driving functions. Thirty years of historical wind data were utilized for the generation process and the resulting waves were described in terms of distribution of wave height and period. Long term extrapolations were also carried out and wave run-up characteristics on the beach determined.

4.3 WATER LEVELS

The changes in sea level in Placentia Road can be generally described in terms of the following driving forces:

i) Astronomical forcing (tides); and

ii) Atmospheric forcing (storm surges).

Astronomical tides are a regular rhythmic phenomena and the amplitudes of these water level fluctuations can readily be assessed by analyzing relatively short time series of data. Table 4.3.1 shows the results of such an analysis carried out on data gathered at four locations in Placentia Bay; Argentia, St. Brides, Arnold's Cove, and Come By Chance. From these results the exact tidal ranges can be computed both for the average tides as well as large (spring) tides. Each of the five tidal constituents analyzed and presented in Table 4.3.1 represent the predominant components of the tidal regime and are classified by their frequency of oscillation. They are described in terms of amplitude and phase which can be used to determine the size and the timing of the water level variations, respectively. The lunar semi-diurnal constituent, M_2, is the major contributor to the tidal system (due to its large amplitude). The phase difference in the M_2 constituent among the tide stations in Placentia Bay is at a maximum of 13 degrees (between Argentia and Arnold's Cove) which indicates that throughout this area the time of the M_2 tidal highs and lows occurs within seven minutes. The Canadian Tide and Current Tables (Canadian Hydrographic Service, 1984) for the Atlantic coast of Newfoundland show that tidal stations along the east coast of Placentia Bay experience high water within ten minutes and low water within five minutes of each other.

Based on the information presented in Table 4.3.1 it can be summarized that the water level due to astronomical forcing does not vary signficantly along the open coastlines of Placentia Bay.

Water level may also change in response to atmospheric forcing. This can be caused by the direct effect of atmospheric pressure on the water surface, or by the piling up of water against a coast by the wind. The time scale for the response of the sea level to wind forcing is of several days, therefore only relatively persistent wind will be effective in causing elevation changes. The results of atmospheric forcing at separate points along a

TABLE 4.3.1

TIDAL INFORMATION AT PLACENTIA BAY

		Argentia		St. Brides		Arnold's Cove		Come By Chance	
Location		47°18'N, 53°59'W		46°55'N 59°11'W		47°45'N 54°0'W		47°49'N 54°01'W	
Length of record (days)		362		15		53		55	
Mean tidal range (m)		1.6		1.6		1.7		1.6	
Large tidal range (m)		2.5		2.5		2.5		2.5	
Tidal constituents*		H	g	H	g	H	g	H	g
Principal Lunar	O_1	9.0	117	7.6	118	8.3	117	7.5	108
Luni-solar diurnal	K_1	7.7	131	5.7	124	8.3	114	7.7	125
Principal Lunar	M_2	69.6	237	69.4	231	71.5	224	69.8	236
Principal Solar	S_2	20.0	271	24.9	265	17.7	260	18.3	275
Quarter-diurnal	M_4	4.9	175	3.6	143	5.2	142	5.5	174

*Note: The water level amplitude, H, is in cm.
The phase lag, g, is in degrees relative to Newfoundland Standard Time (i.e. GMT + 3 1/2).

Source: Bedford Institute of Oceanography, Tidal Division.

coastline will be similar if the sites are located such that both are influenced by the same winds and the same high or low atmospheric pressure system. The proximity of Argentia to Placentia indicates that the effects of atmospheric forcing on the water level at one location will similarily effect the water level at the other.

4.3.1 Sources of Water Level Data

Ideally, a study of the flooding risk at the Town of Placentia would be based on sea level data recorded in Placentia Road over a long period of time. A data set such as this, however, does not exist and information gathered by the nearby tide gauge at Argentia was substituted. Although, as described above, the water levels at Argentia and Placentia Road are similar it is necessary to quantify the extent to which they agree and detail how, and why, they differ. Having established this correlation it is then valid to apply the long term Argentia water level data to Placentia Road and calculate extreme water elevations expected at long return periods.

The water level recording device installed at Argentia is maintained, and the data archived, by the Marine Envrionmental Data Service (MEDS). The Argentia gauge functioned intermittently during the period 1972-1982 (see Table 4.3.2) providing a total data return of 86 percent. Additional data from 1983 was added to the data set as it became available. A subset of this 1983 water level record was compared with a simultaneous record of hourly water elevations measured at Placentia Road in November 1983. Using a four point differential estimator and interpolation scheme (Abramowitz and Stengun, 1964) the correlation between the two water level records was found to reach a maximum of 0.981 when Argentia leads Placentia Road by 0.15 hours (9 minutes). The regression coefficient (Argentia to Placentia) at this optimal lag is 0.96 which accounts for all but one component in the water level regime of 10.7 cm which is present at Placentia Road but not at Argentia. This variation can be attributed to the difference in the effects of wind set-up in Placentia Road and Argentia Harbour due to the dissimilarity of the geometry and orientation of the two inlets. A second contributing factor results from the way in which the water level data was collected at each of the two locations. The Argentia gauge measures the actual surface elevation of the sea whereas the Placentia Road gauge rested on the sea floor and measured the changes in pressure and related it to water level. The Placentia Road gauge was thus affected by the low frequency variations in air pressure and because it recorded the sum of both sea level and air pressure, the record will exhibit some variance with the Argentia gauge.

TABLE 4.3.2

PERCENTAGE OF MISSING WATER LEVEL DATA
FROM ARGENTIA GAUGE
1972-1982

	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Annual Mean
1972													
1973							48	7					5
1974	58					3							5
1975													
1976	1						37	13	82	7	90	35	22
1977	10	5	54	100	100			100	100	100	100	100	64
1978	100	6											9
1979		3		1				36				100	12
1980		1	100	100	8		15	4	11				20
1981								14		52	51		10
1982	10	13	3						71				8
Monthly Mean	16	2	14	18	10	1	9	16	24	14	22	21	14

4.3.1 Sources of Water Level Data (Cont'd)

The analysis of the water level records at Placentia Road and Argentia indicate that, as expected, the semi-diurnal tides, primarily the M_2 constituent, dominate the astronomical variations and strongly influence the value of the lag and regression coefficient given above. To determine if these parameters were applicable at other frequencies a cross spectral analysis was carried out. It was determined that the values are generally consistent with the exception of a slight decrease in the regression coefficient as the frequency of the tidal constitutents under examination became higher. This can be explained by the more energetic shallow water tides in Argentia Harbour.

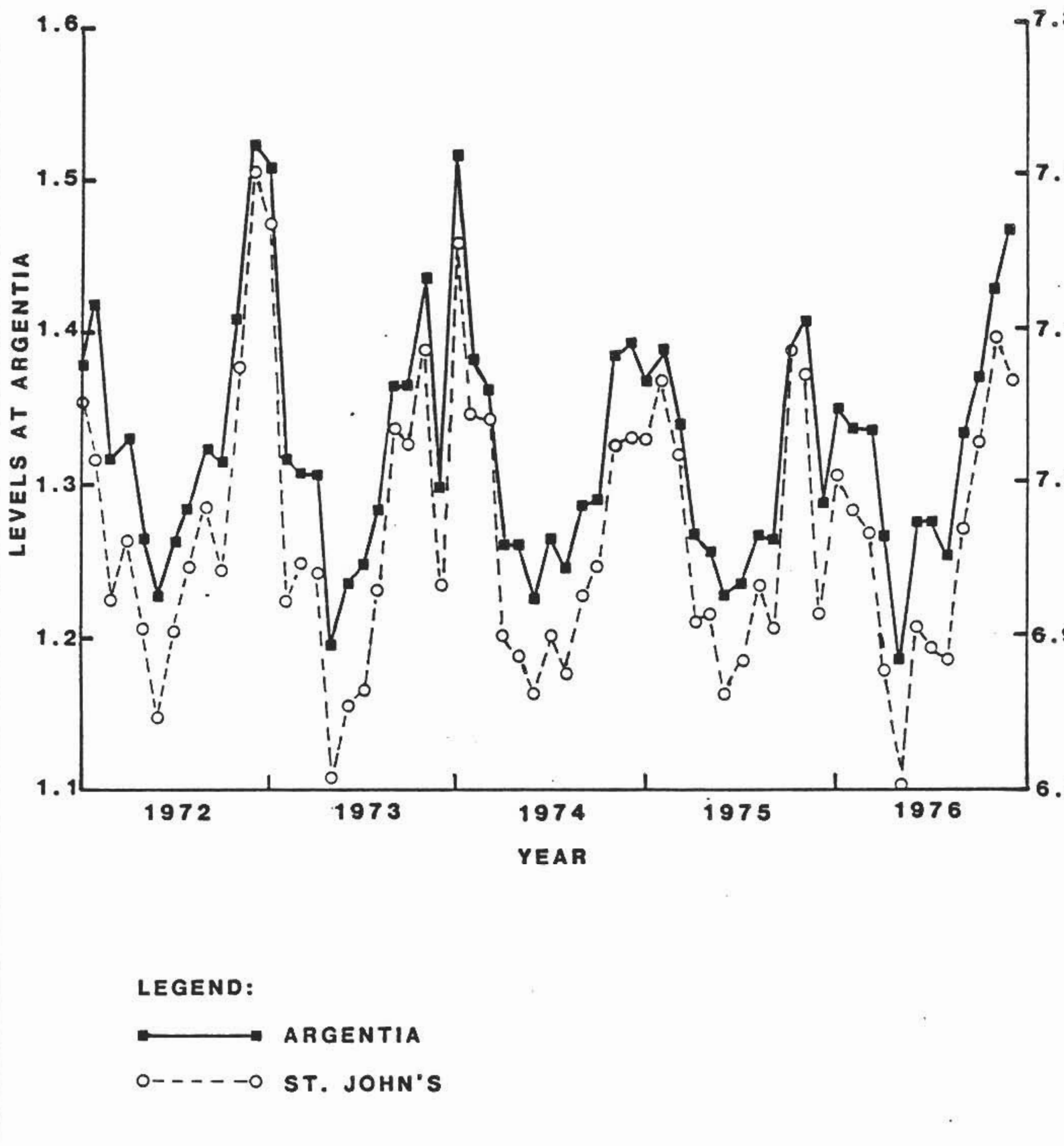
On a longer time scale, comparison of the monthly sea levels at Argentia with those recorded at St. John's show similar characteristics (see Figure 4.3.1). The dominant feature in each record is a seasonal oscillation with an amplitude of almost 10 cm due mainly to density changes on the adjacent continental shelf (Petrie and Anderson, 1983). Aperiodic variations result from the influence of large scale meteorological changes. The annual changes of sea level at Argentia are small (in the order of 2 cm) and without a pronounced trend over the period 1972-1982. The gauge, situated on the sea bottom, thus appears to be well sited with respect to monthly and longer period variations and is responsive to regional changes of sea level but not affected by localized subsidence or emergence of the land. The information gathered at this point should provide an accurate baseline of data from which to calculate extreme water level values.

4.3.2 Extremal Analysis

Having established the correlation between the water level regime at Argentia to that at Placentia Road, the longer record of the former can be used, with confidence, in Placentia Road. Since extreme values of water level that can be expected to occur only once in say 20 or 100 years are of interest (i.e. the 1 in 20 or 1 in 100 year water level) the available water level data must be extrapolated mathematically. One of the most widely accepted techniques, the Gumbel Method (Gumbel, 1954), has been employed as well as a more recently developed, and perhaps more appropriate alternative, the Joint Probability Method (Pugh and Vassie, 1980).

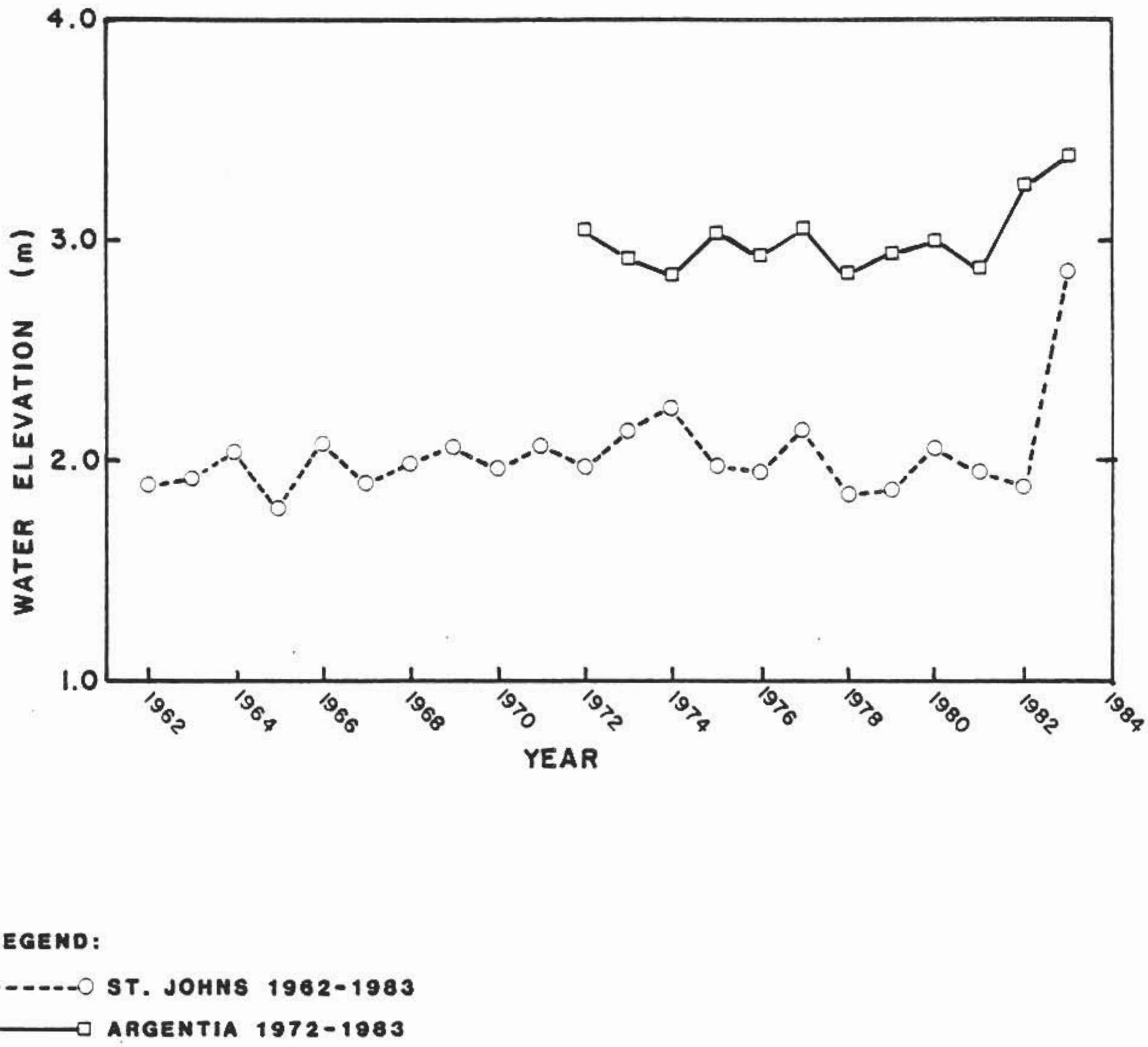
In accordance with the Gumbel method the annual instantaneous sea level maxima were determined from the available 12 year record 1972-1983. The values which are shown in Figure 4.3.2, vary by typically 20 cm from year to year and do not exhibit a marked trend. The highest water level observed during the record was associated with a large storm surge on the December 25, 1983. Figure 4.3.3 was generated by ranking the annual water level maxima

FIGURE 4.3.1



MONTHLY SEA LEVELS
AT ARGENTIA AND ST. JOHN'S

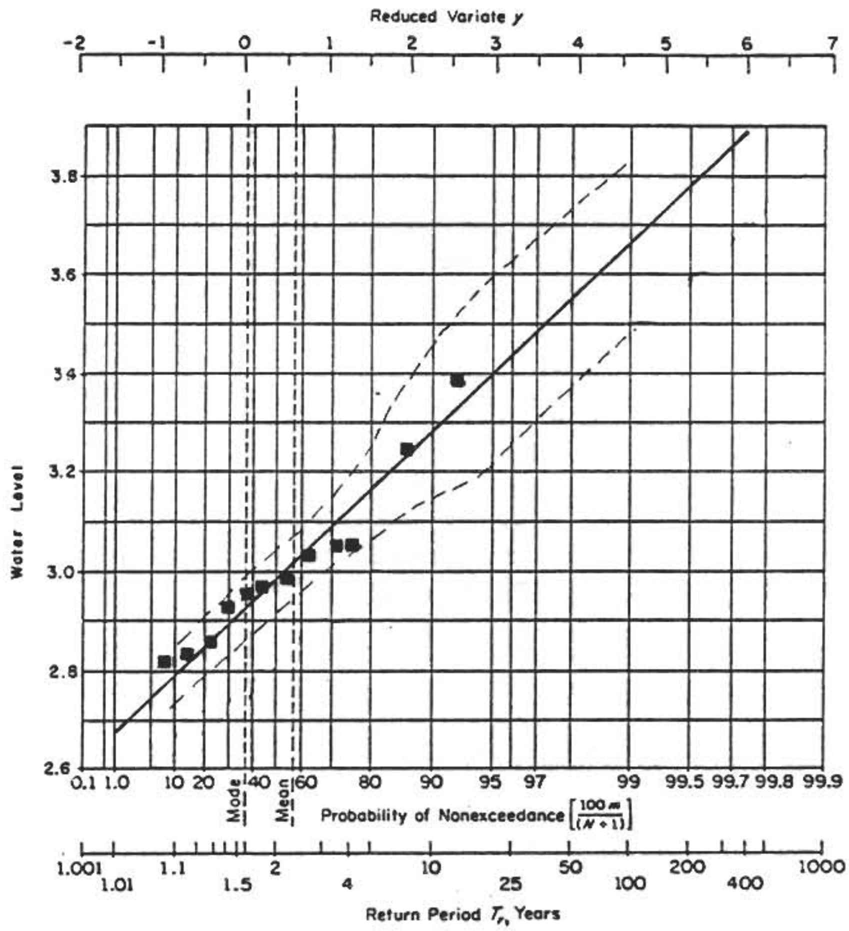
FIGURE 4.3.2



NOTE: WATER LEVEL IS RELATED TO ELEVATION ABOVE CHART DATUM AT EACH OF THE TWO LOCATIONS

ANNUAL MAXIMUM INSTANTANEOUS RECORDED WATER ELEVATIONS

FIGURE 4.3.3



---- 68% CONFIDENCE LIMITS

NOTE: GUMBEL DATA ARE INSTANTANEOUS WATER LEVEL MAXIMA

CHART DATUM = ELEVATION -1.36 m GEODETIC

GUMBEL DISTRIBUTION OF WATER LEVELS AT ARGENTIA

(Y_i, i=1,12) and plotting them on extremal probability paper with the corresponding abscissa X_i, given by i/13. Clearly the points lie close to a straight line, the slope of which is calculated from the ratio of the standard deviation of the extremes to that of the abscissae. This corresponds to a 'neutral regression' (Garrett and Petrie, 1981) and equals the geometric mean of the two regression slopes obtained by minimizing the vertical and horizontal errors by a conventional least squares technique. The extreme water levels for several return periods calculated using the Gumbel method are given in Table 4.3.3.

The determination of extreme values using the Joint Probability Method can be used effectively on relatively short data records. The method involves filtering the time series of hourly water levels such that the effects of astronomical forcing (tides) and atmospheric forcing (surge) are separated. Using standard computer analysis the tidal component of the data can be undertaken to allow for a complete nodal modulation (i.e. constituents with periods up to 19 years can be resolved) and a probability density function (pdf), P_T, determined. The residual component of the water level, that is the original record minus the tidal component, is also described in terms of a pdf, P_R. Convolution of these two pdf's, P_T and P_R, results in a pdf that is representative of the water surface elevation. The return period of a given surface elevation is then determined by calculating the exceedance probability based on a selected sampling interval of the record.

In a given time series of water elevations the instances of extremely high levels do not necessarily coincide when the tide is at a maximum. The Joint Probability Method, by determining a pdf for the tide and surge separately and then combining the two, leads to a better estimate of the pdf of the total sea surface than can be derived from the original data. The underlying assumption for this method is that the tide and surge do not interact in shallow water as a result of non-linear hydrodynamic processes. Comparision of the residual variance for different states of the ebbing and flooding tides were undertaken and as can be seen in Table 4.3.4 the differences are negligible. It can therefore be concluded that interaction is not important and the Joint Probability Method approach is valid.

Figure 4.3.4 shows the graphical results of the extreme value analysis using the Joint Probability Method. Estimates of water elevation at the various return periods are given in Table 4.3.3.

It can be seen that the extreme values calculated by the Joint Probability Method are consistently lower than those

TABLE 4.3.3

EXTREME WATER ELEVATIONS RELATIVE TO CHART DATUM AT ARGENTIA
(m)

	Return Period (years)			
Method	1 in 10	1 in 20	1 in 50	1 in 100
Gumbel Distribution using instantaneous maxima	3.28	3.39	3.55	3.67
Joint Probability Method using 1 hour time step	3.11	3.20	3.31	3.38
Gumbel Distribution using hourly maxima	3.13	3.23	3.33	3.42

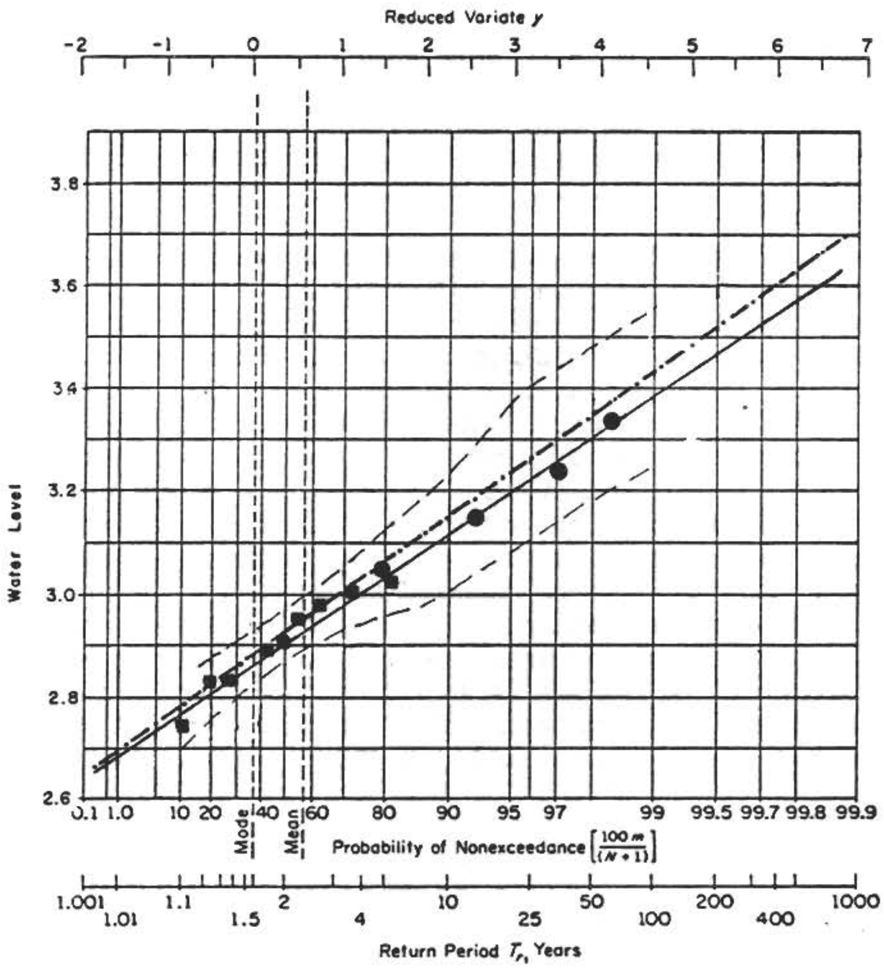
TABLE 4.3.4

TIDE/SURGE INTERACTION ANALYSIS

State of Predicted Tide	Standard Deviation of Residual Water Level (cm)	Upper and Lower 95% Confidence Limits (cm)	
Flooding	15.5	17.0	14.2
Ebbing	15.4	16.9	14.1
High Water	16.1	17.7	14.8
Low Water	16.2	17.8	14.9

Note: The determination of the residual standard deviation as a function of the state of the predicted tide was carried out for a test period of Argentia Sea level data from 1 January 1972 to 4 May 1972. The confidence limits assume 240 degrees of freedom which represents the number of predicted tidal maxima during the four month period.

FIGURE 4.3.4



LEGEND:

- ● JOINT PROBABILITY METHOD DATA
- ——— JOINT PROBABILITY METHOD ESTIMATE
- ■ GUMBEL DATA
- —·—·— GUMBEL ESTIMATE
- – – – – 68% CONFIDENCE LIMITS OF GUMBEL DISTRIBUTION

NOTES: GUMBEL DATA ARE HOURLY MAXIMA

JOINT PROBABILITY METHOD USES TIME STEPS OF ONE HOUR

CHART DATUM = ELEVATION −1.36m GEODETIC

EXTREME VALUE ANALYSIS USING JOINT PROBABILITY METHOD AND GUMBEL DISTRIBUTION

4.3.2 Extremal Analysis (Cont'd)

generated using the Gumbel distribution. This offset can be attributed to the one hour time step applied in the Joint Probability Method which in effect gives maximum water levels averaged over one hour intervals. The Gumbel results shown in Figure 4.3.3 give the instantaneous extreme water levels whereas the Joint Probability Method results in Figure 4.3.4 show the hourly extreme water levels. A comparison was made between the two techniques by using maximum hourly data and applying the Gumbel extremal analysis technique. These results are given in Table 4.3.3 and Figure 4.3.4 and show agreement between the Gumbel and Joint Probability methods.

The annual maximum instantaneous water elevation for the Argentia gauge has been plotted in Figure 4.3.2. A second curve, that of the annual maximum instantaneous water elevation for the St. John's gauge, has also been included. It can be seen that the December 1983 value in both locations indicates that the water level was exceptionally high along the south and east coasts of Newfoundland. It was in fact the highest water level ever recorded at both sites. Examination of the Halifax water level regime shows that although the peak water elevation for 1983 occurred in the same time frame (i.e. 25th December) its magnitude was relatively diminished and cannot be considered as an exceptional event. This agreement between Argentia and St. John's (rather than Argentia and Halifax) was used as an argument for taking the 22 years of water elevation data gathered at St. John's and applying the Gumbel extremal method. Results of the analysis are shown in Figure 4.3.5 and graphically describe the anomolous nature of the water level associated with the December 1983 storm (2.9 m). In statistical terms this water level represents the upper 90% limit of the 1 in 500 year event and clearly demonstrates that the vagaries of natural phenomena, such as water levels, must be kept in context when extrapolating recorded data to obtain extreme values.

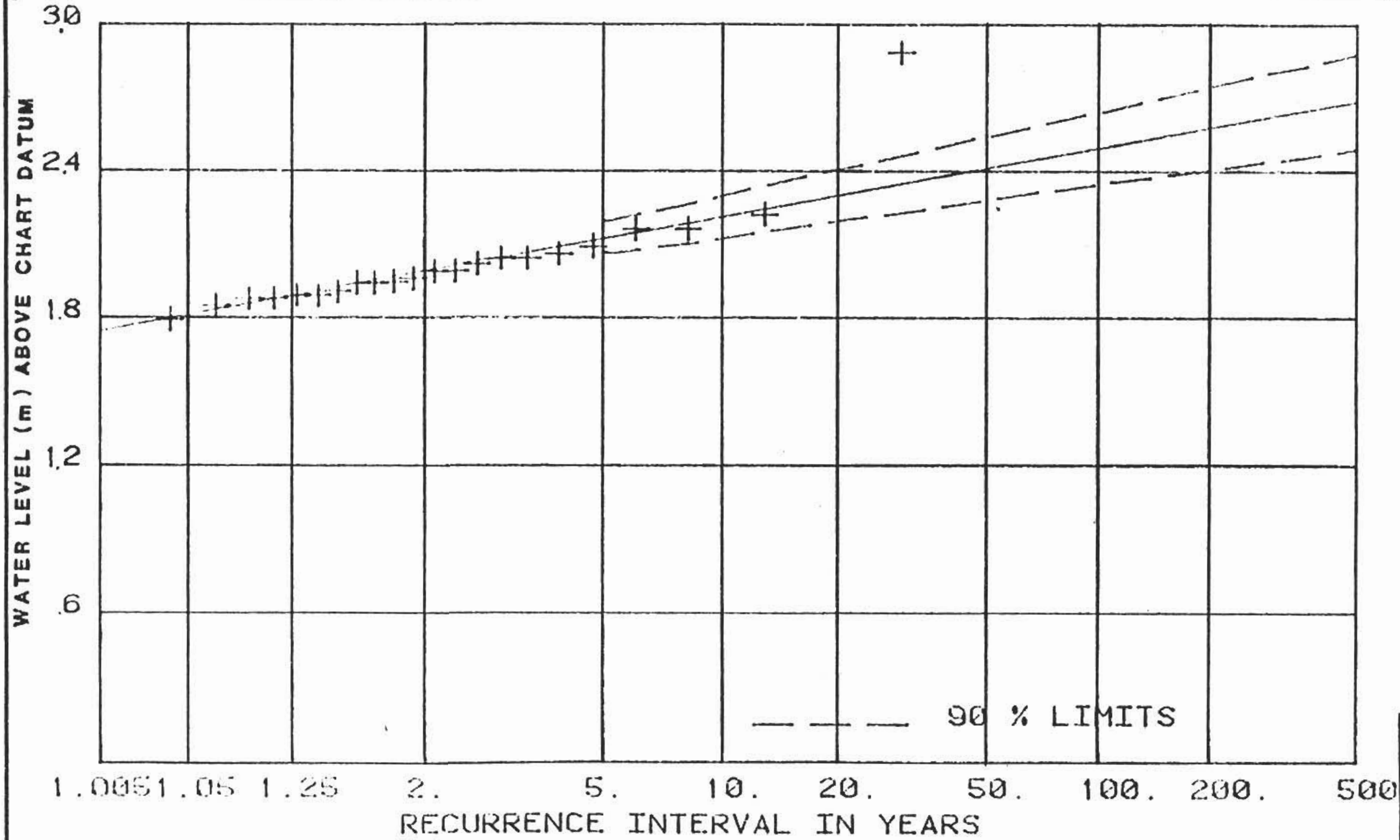
4.4 WAVE ANALYSIS

4.4.1 Introduction

To determine the effect of waves on flooding in the Placentia area, an analysis was carried out of the wave history of the region and a projection made for long term events.

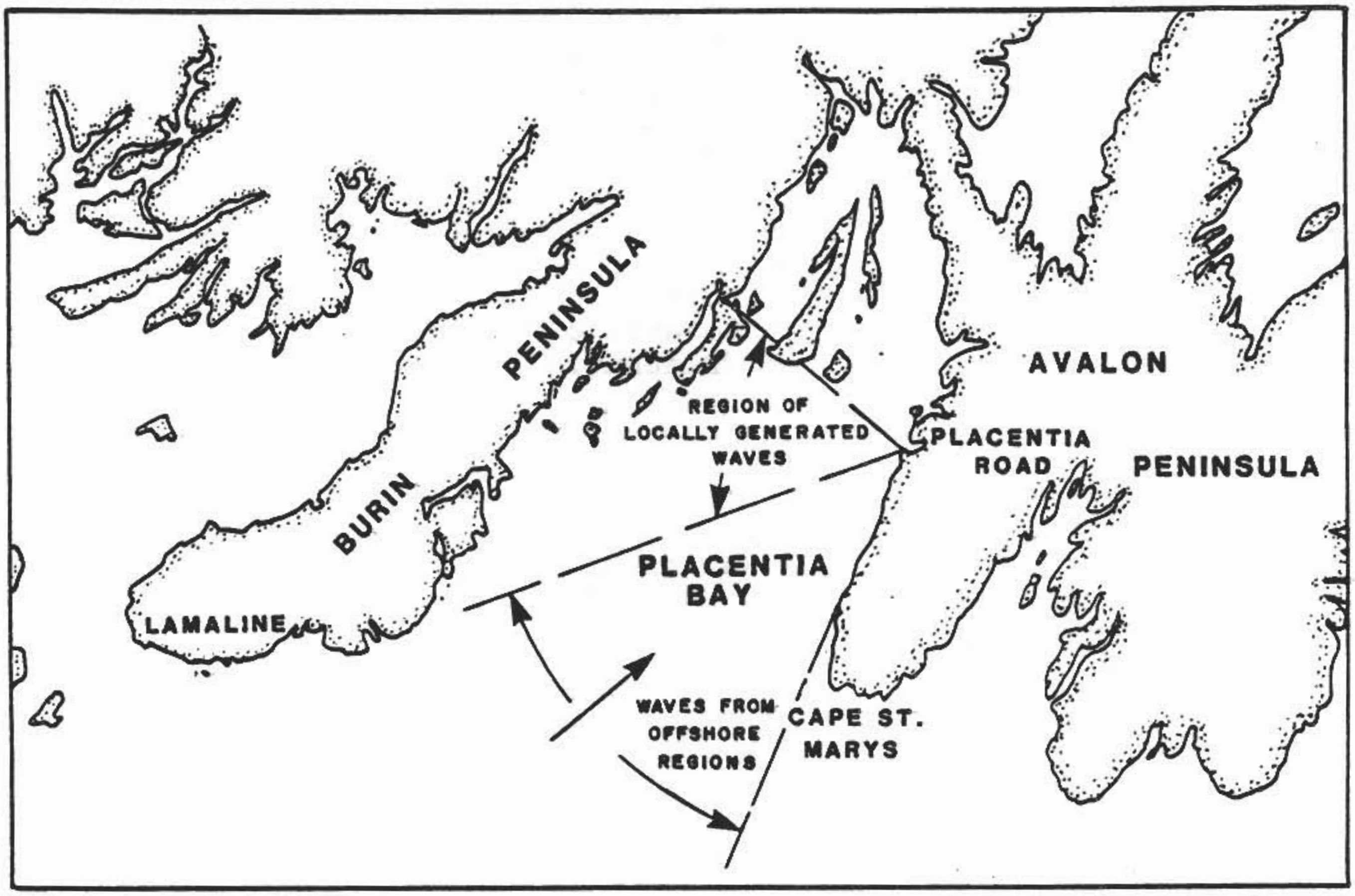
Placentia Road is situated in an east-west orientation (see Figure 4.4.1). Placentia Bay has its longest axis in a north-south direction, and hence Placentia Road is sheltered from waves generated outside of Placentia Bay. The distance from Placentia Road entrance to the southerly tip of the Burin Penninsula is approximately 50 nautical

FIGURE 4.3.5



GUMBEL DISTRIBUTION OF WATER LEVELS AT ST. JOHNS

FIGURE 4.4.1



MAP OF PLACENTIA BAY DELINEATING REGIONS OF LOCALLY GENERATED SEAS PROPAGATING INTO PLACENTIA ROAD AND DIRECTION OF WAVES FROM OFFSHORE REGIONS

4.4.1 Introduction (Cont'd)

miles. This limits the locally generated waves (i.e. those within Placentia Bay) to a sector from the southerly tip of Burin Peninsula to the northern section. An analysis was carried out (see Section 4.4.5) to show that other waves (generated outside of Placentia Bay) do not enter Placentia Road.

To determine the effect of waves on the beach area, it is necessary to determine the local wave climate from wind conditions. In order to have a long time history of the possible waves in Placentia Road, (i.e. 20 to 30 years) it is necessary to access historical wind data for that period of time and to predict from these records the generated waves by means of numerical computations (called "hindcasting of waves"). This data base of waves enables one to determine the long term effect of waves on the beach area. The subsequent sections detail the computational procedures to arrive at the wave effects in regards to flooding in the Placentia area.

4.4.2 Wave Climate

Waves were hindcast for the sector in Placentia Bay as shown in Figure 4.4.1 where the waves could propagate into Placentia Road. Waves were hindcast using wind data tapes obtained from the Atmospheric Environment Service (AES) for the following locations and dates:

Wind Station	From	To
Argentia A	Jan. 1, 1953	May 31, 1970
Placentia	Nov. 1, 1970	Dec. 31, 1975
Argentia A	May 1, 1976	Dec. 31, 1982
Argentia A	Nov. 20, 1983	Dec. 31, 1983

It will be shown subsequently, under Section 4.4.5 - Refraction and Shoaling, that waves propagating from far offshore from the southerly direction do not enter the Placentia Road area. These longer period waves will propagate in the northerly direction and will break against the northeastern side of the Burin Peninsula, Merasheen Island, Red Island, or the southwest tip of the Avalon Peninsula.

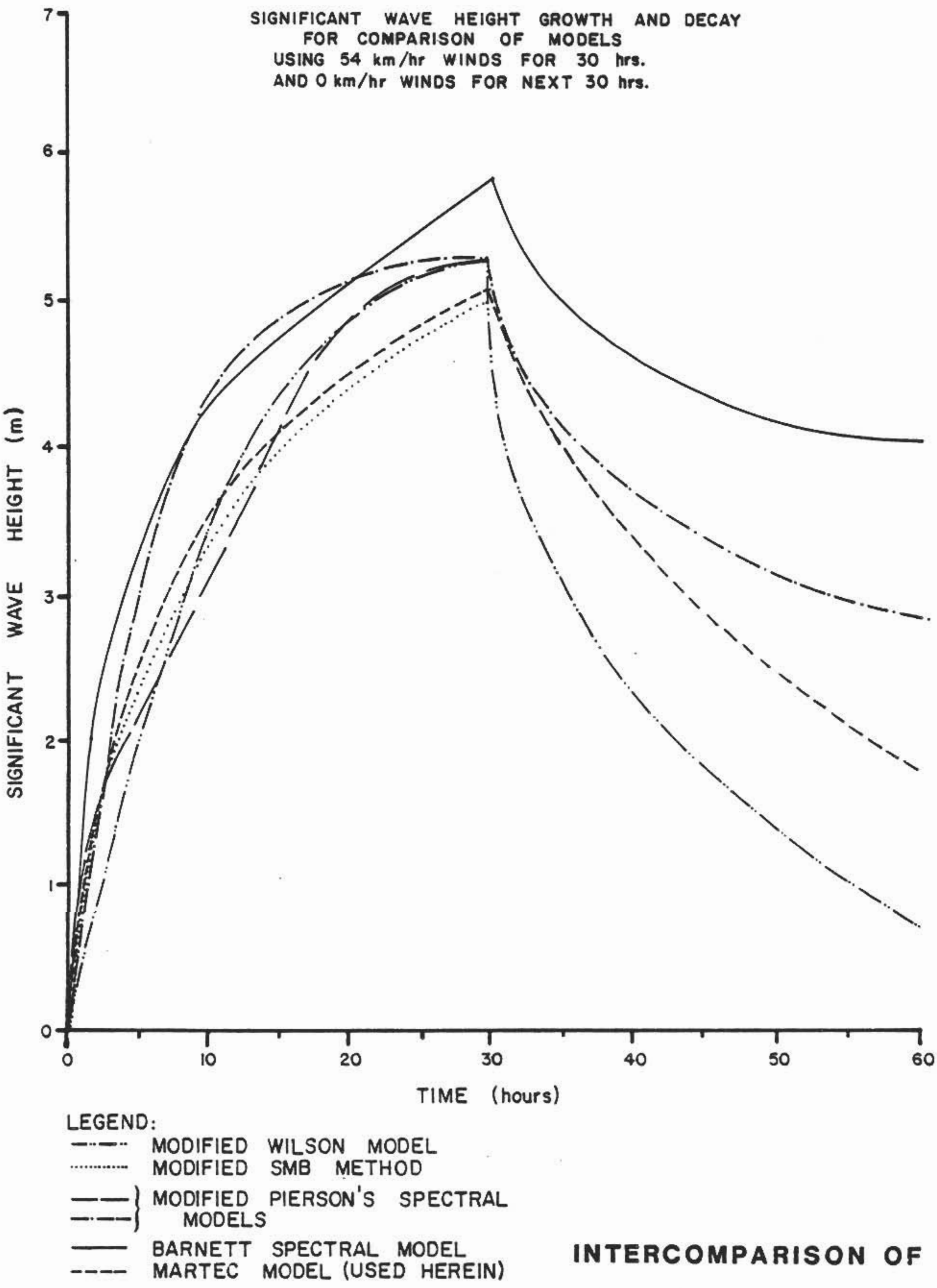
The data base, noted above, consisted of approximately 30 years of wind data (supplied by AES on magnetic tape) and was used to generate local seas which are able to propagate into Placentia Road. The wave climate for this area was developed from the wind data using the Wilson-Bretscheider Method (Wilson, 1966), modified by Patterson (1971). This method has proven effective for the development of wave growth when using the appropriate fetch length for each direction. The wave hindcasting procedure depends upon empirical relationships that have been devised for relating wave height, period, wave velocity, wind velocity and fetch length over the entire range of wave growth. The progress of the waves are followed from an arbitrary mesh point in a grid and takes account of the increasing velocity of waves (which determines their space-time track) and their progression into areas of different generating wind velocities. At all points along the track, the significant wave height and period of the waves are calculated. The method used here is a computerized version of a modified Patterson method noted above, and identified as the Martec model in Figure 4.4.2. All of the various model results are shown during the growth phase (from 0 to 30 hours). The decay portion of the curves for the two modified Pierson's Spectral Model were identical, thus only one of those decay curves is shown. Also, the SMB method was used only for the growth phase and thus there is no decay phase data for that model.

The significant wave height and significant wave period distribution were determined from the prediction equation using a one-hour time interval. The accuracy of the method used in this study for determining wave parameters compared favourably with other hindcast procedures as shown in the results of the test of the Martec hindcast model carried out for conditions shown in Figure 4.4.2.

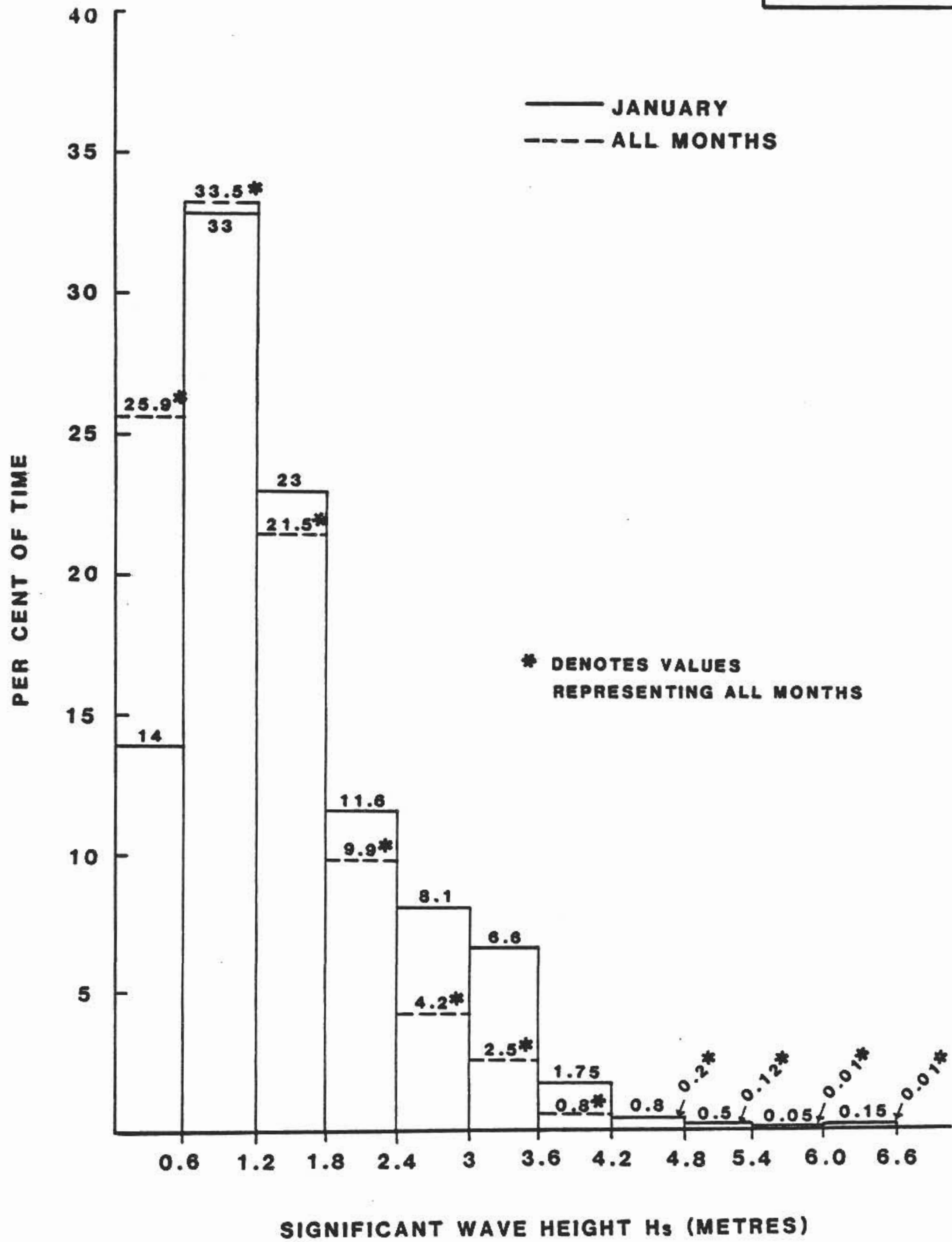
Figures 4.4.3 and 4.4.4 show a histogram of significant wave heights and periods, respectively, for Placentia Road for the 11-year period 1971-1982 (omitting 1976 since a full year of wind data was not available for that year). The period 1971-1982 was chosen for the histogram to coincide with the period during which water levels were available at the start of this study. As wind data from 1983 became available it was added to the data set and included as input to the wave hindcast analysis (waves were hindcast for the entire period 1953 to 1983). A histogram was obtained for the entire 11-year dataset and also for the 11 Januarys (January was plotted since it is one of the more severe months. It is noted that the significant wave heights have a Rayleigh-type distribution while the significant wave periods appear to be normally distributed with a mean of about 4 to 5 seconds. Also, the wave heights for the all-data set have a higher occurrence of the smaller height, low-period waves, while the 11 Januarys data set shows the higher occurrence of larger height high period waves. A computer listing of the distributions is included in Appendix II.

Histograms of the wave heights and periods are plotted in Figures 4.4.3 and 4.4.4. The wave height histogram shows

FIGURE 4.4.2

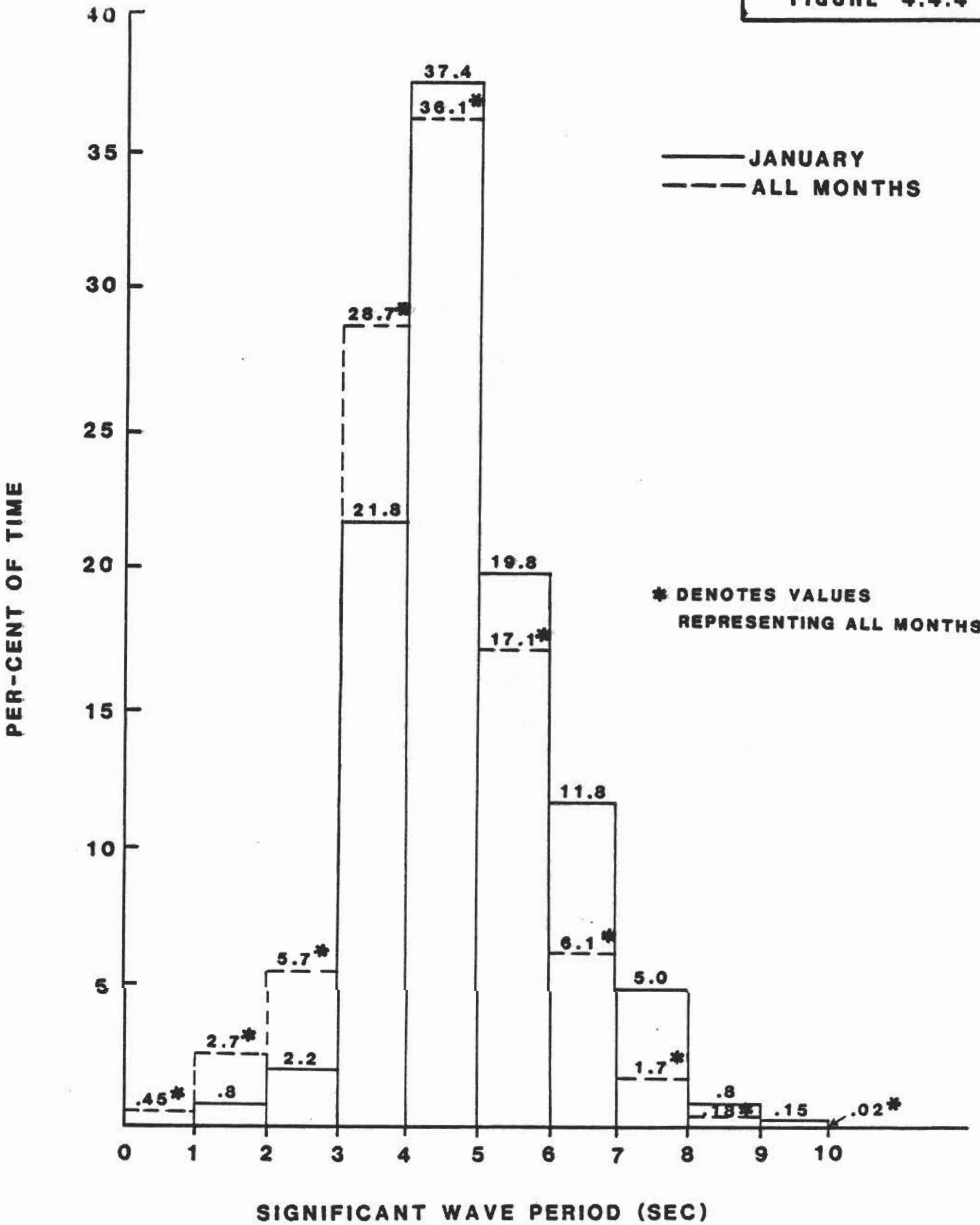


INTERCOMPARISON OF HINDCAST WAVES USING VARIOUS MODELS



HISTOGRAM OF HINDCAST SIGNIFICANT WAVE HEIGHTS FOR PLACENTIA BAY

FIGURE 4.4.4



HISTOGRAM OF HINDCAST WAVE PERIODS IN PLACENTIA BAY the percent of time that the wave height is in a given range. For example for all Januarys, it was found that 23% of the waves occurring in that month had a height range between 1.2 and 1.8 m.

It can be seen from the distribution of wave heights for Placentia Bay shown in Figure 4.4.3, that the largest locally generated waves occur a very low percentage of the time. For example in January, it would be expected that the maximum waves of 6 to 6.6 metres would occur 0.15 percent of the time (i.e. one hour, on average, in January).

In fact, based on January data approximately 90 percent of the time waves are less than 3 metres in Placentia Bay and are most likely to be in the range of 0.6 to 1.8 metres with a wave period of three to six seconds.

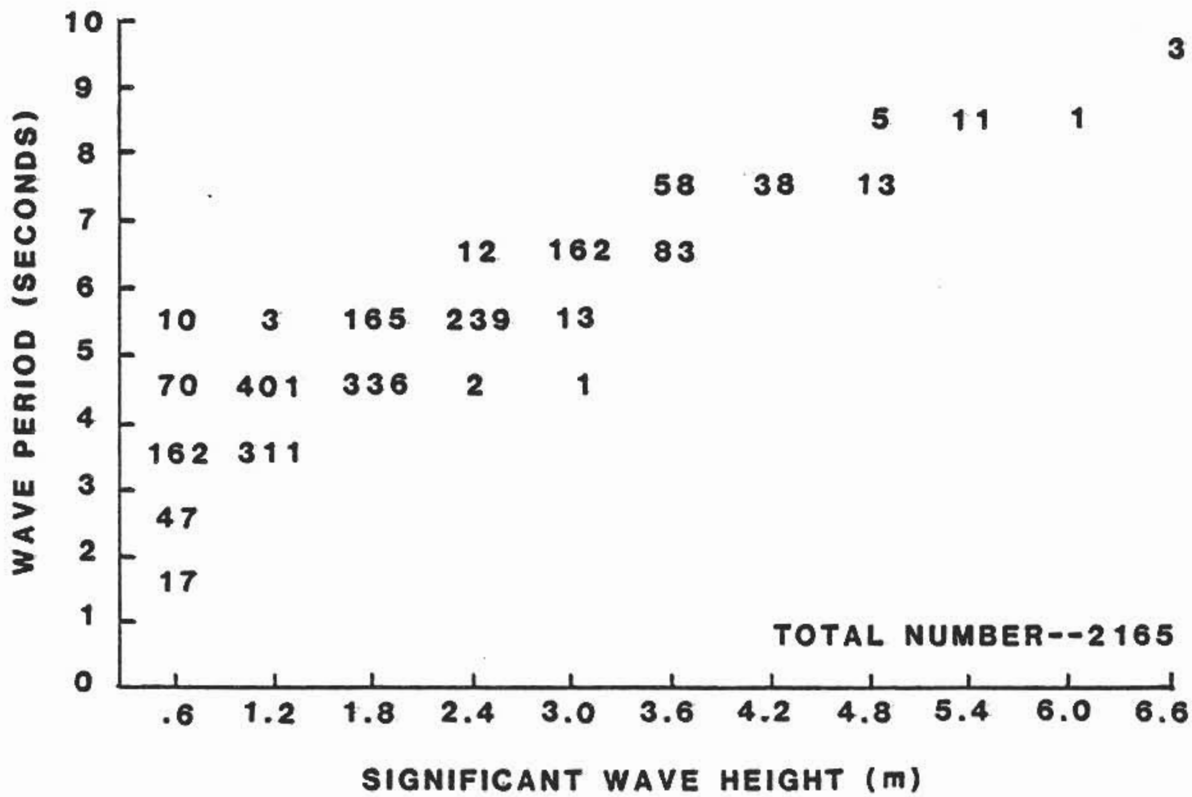
A distribution of significant wave height versus period for 11 Januarys is shown in Figure 4.4.5. The larger wave heights are seen to occur with the larger wave period. These longer period, large waves have a greater wave run-up potential, but since their occurrence is small, the overtopping effect is negligible.

The cumulative frequency distribution for the significant wave heights for the 11 Januarys is shown in Figure 4.4.6. The cumulative distribution curve becomes quite flat at the upper end, graphically showing the low number of significant waves greater than 4 metres.

4.4.3 Extreme Event Analysis

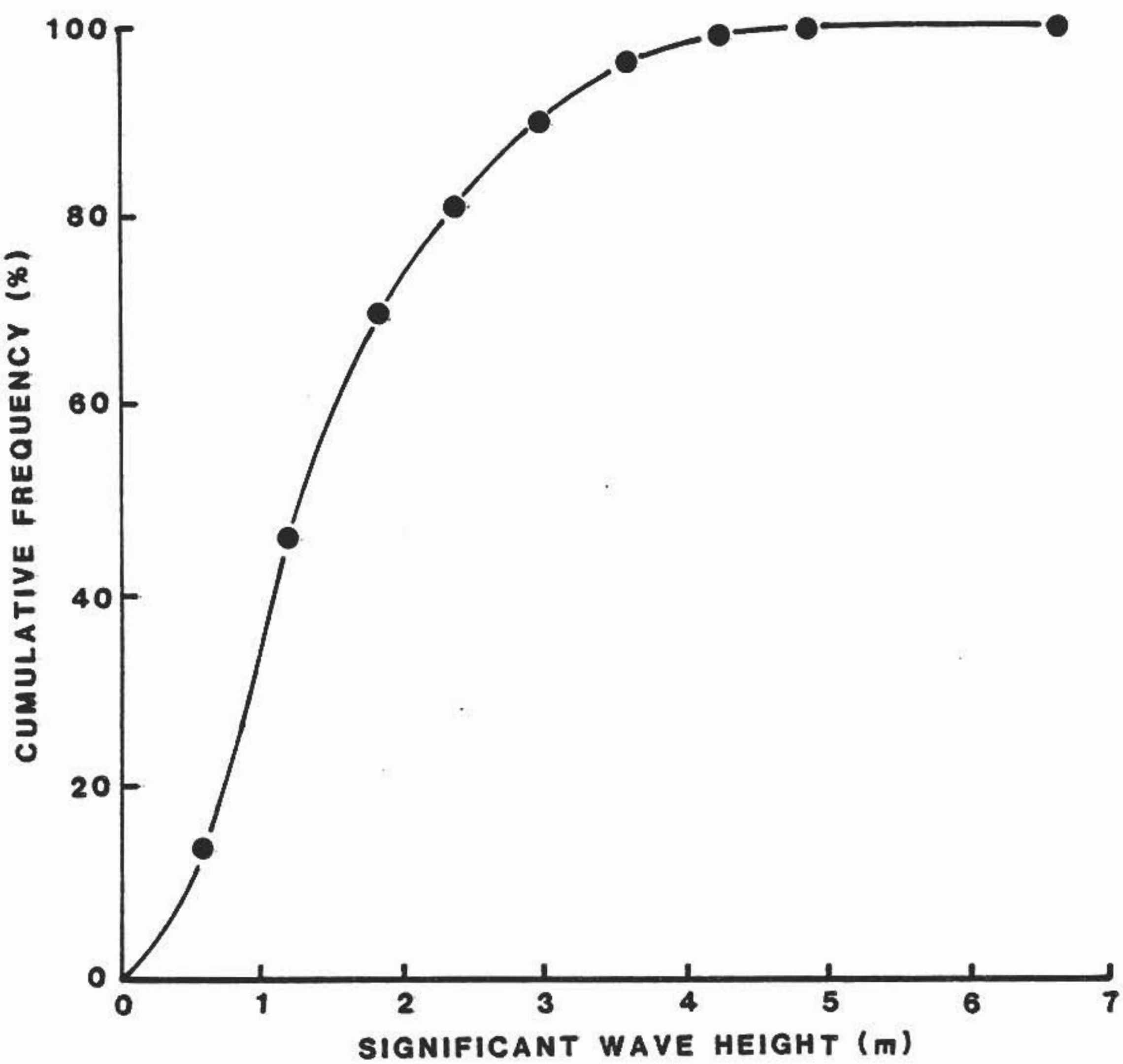
An extreme event analysis was carried out for significant wave heights using the 30 years of wave hindcast data. The extreme wave for each year was obtained and a Gumbel distribution of wave heights was performed. Figure 4.4.7 shows the results of the analysis and the computer generated listing is shown in Table 4.4.1. The dotted lines in Figure 4.4.7 show the 90% confidence limits. In other words, for a 50-year event one is 90% confident that the wave heights will be no greater than 7.8 metres and no less than 5.8 metres and 'most probably' will be about 6.8 metres as given by the full curve.

For the 100-year event the design-significant wave height would be appproximately 7.5 metres. It is to be noted that the largest significant wave generated in the 30-year hindcast period was 6.5 metres. It is also to be noted that these wave heights refer to deep-water waves in Placentia Bay and are not the waves that impinge on the beach area, as these waves go through the processes of wave refraction and wave shoaling as they propagate into



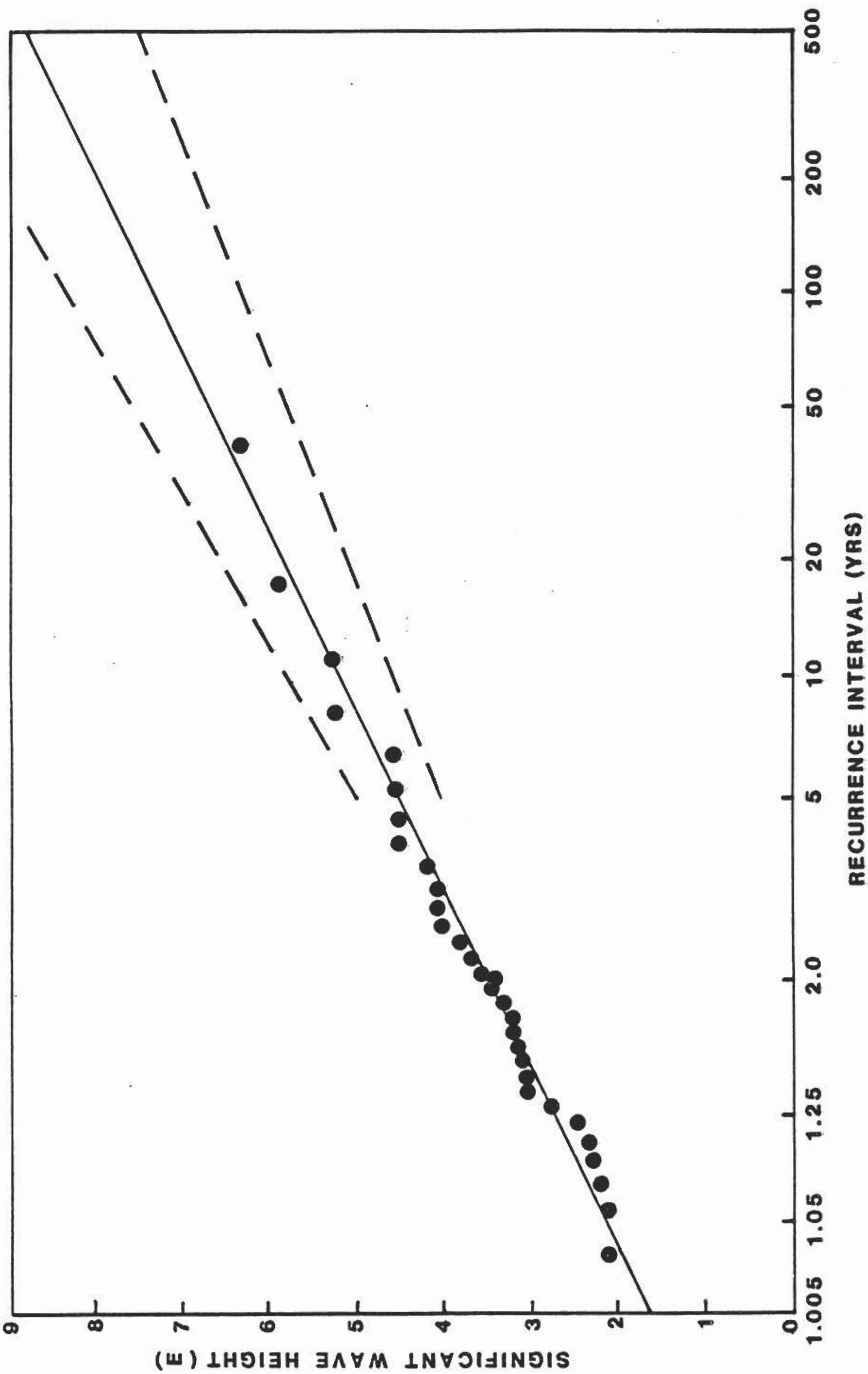
DISTRIBUTION OF SIGNIFICANT WAVE HEIGHT VS. PERIOD FOR ELEVEN JANUARYS

FIGURE 4.4.6



CUMULATIVE FREQUENCY DISTRIBUTION
OF SIGNIFICANT WAVE HEIGHTS
FOR ELEVEN JANUARYS

FIGURE 4.4.7



LEGEND

— — 90% CONFIDENCE LIMITS

GUMBEL DISTRIBUTION OF
EXTREME SIGNIFICANT WAVE HEIGHTS

the shallow-water areas of Placentia Road. The effects of refraction and shoaling are described in Sections 4.4.5 and 4.4.6.

TABLE 4.4.1

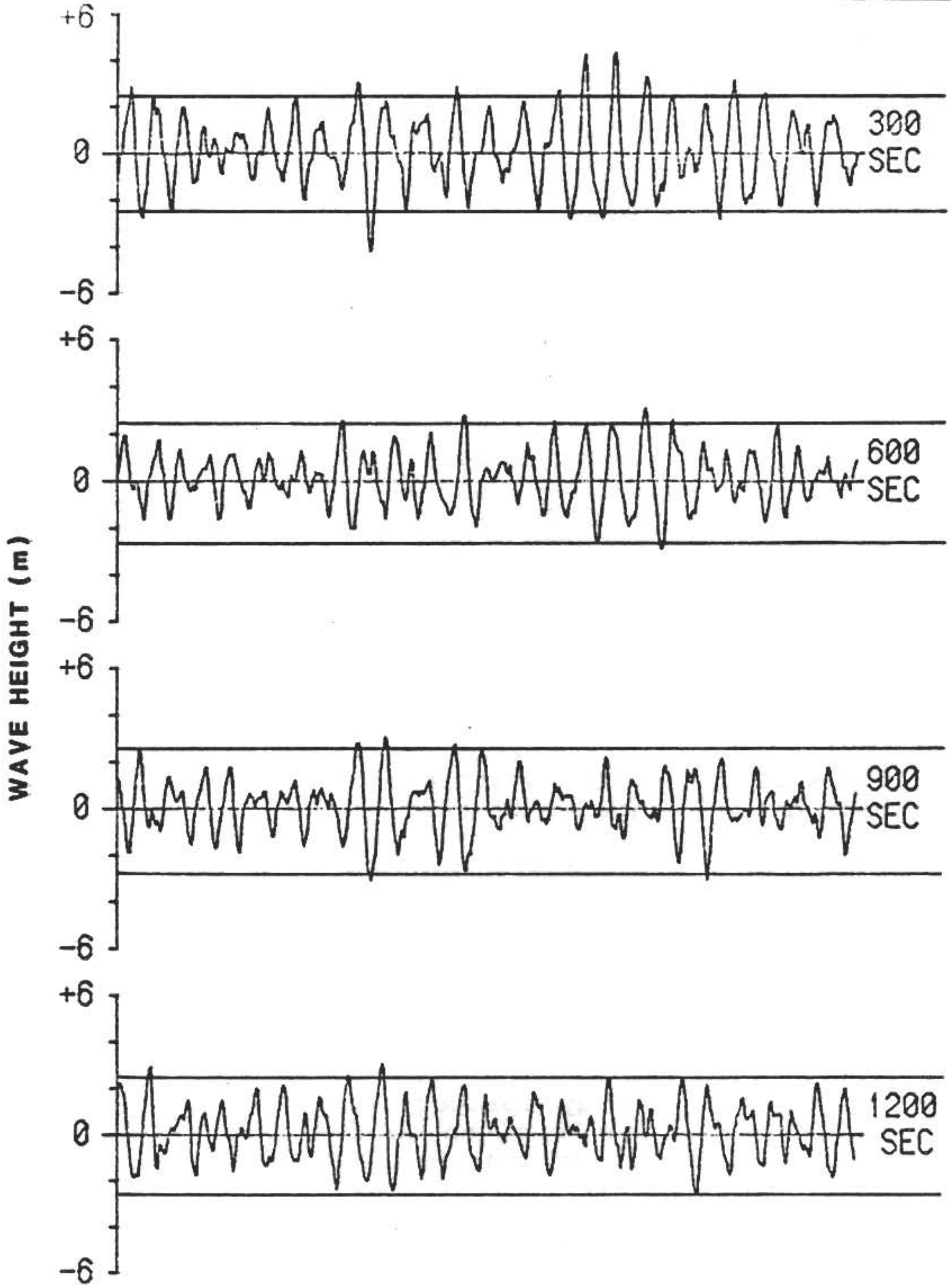
GUMBEL I DISTRIBUTION

Return Period (years)	Wave Height Estimate (metres)	Lower Limit (metres)	Upper Limit (metres)
1.0	1.7		
1.1	2.2		
1.3	2.8		
2.0	3.6		
5.0	4.6	4.1	5.1
10.0	5.3	4.7	5.9
20.0	6.0	5.2	6.7
50.0	6.8	5.9	7.8
100.0	7.5	6.3	8.6
200.0	8.1	6.8	9.4
500.0	8.9	7.5	10.4

This extreme event analysis gives a range of extreme waves that may occur in the area and hence be refracted and shoaled as they move into Placentia Road.

4.4.4 Wave Grouping and Energy Distribution

While the significant wave analysis does provide pertinent information for coastal processes it is important to consider this method in terms of the actual wave height distribution. It should be emphasized that the significant wave is not an actual wave but 'represents' a large number of actual waves that occur over a time interval; in this case, one hour. Thus, the significant wave height used in this analysis is actually the average of the highest one-third of the waves that occur for one hour, and similiarly the significant wave period is the average period of these particular waves. Figure 4.4.8 shows an actual 20-minute wave record having a significant wave height of 5.12 metres. For convenience, the twenty minute wave record has been broken up into four 300 second sections. The annotation at the end of the abscissa identifies the sections as ending at 300, 600, 900 and 1,200 seconds. The position of the abscissa represents the mean of the record and is defined as the zero of the surface elevation. The horizontal lines, drawn at +2.56 metres and -2.56 metres ($\pm 5.12 \div 2$) from the mean water level are drawn to show how the wave record deviates from



TYPICAL WAVE HEIGHT RECORD OVER A PERIOD OF TWENTY MINUTES this significant height. There is one instance of four consecutive waves around 200+ seconds, which are all higher than the significant wave height, and a few instances where two consecutive waves are higher than the significant height. It can be seen that most of the wave heights are contained within the two horizontal lines and hence this significant wave height is then representative of wave heights over this time interval.

We can define a group as an occurrence of consecutive waves greater than or equal to H_s and the number of waves (N) in that group (G) are the number of consecutive waves $\geq H_s$.

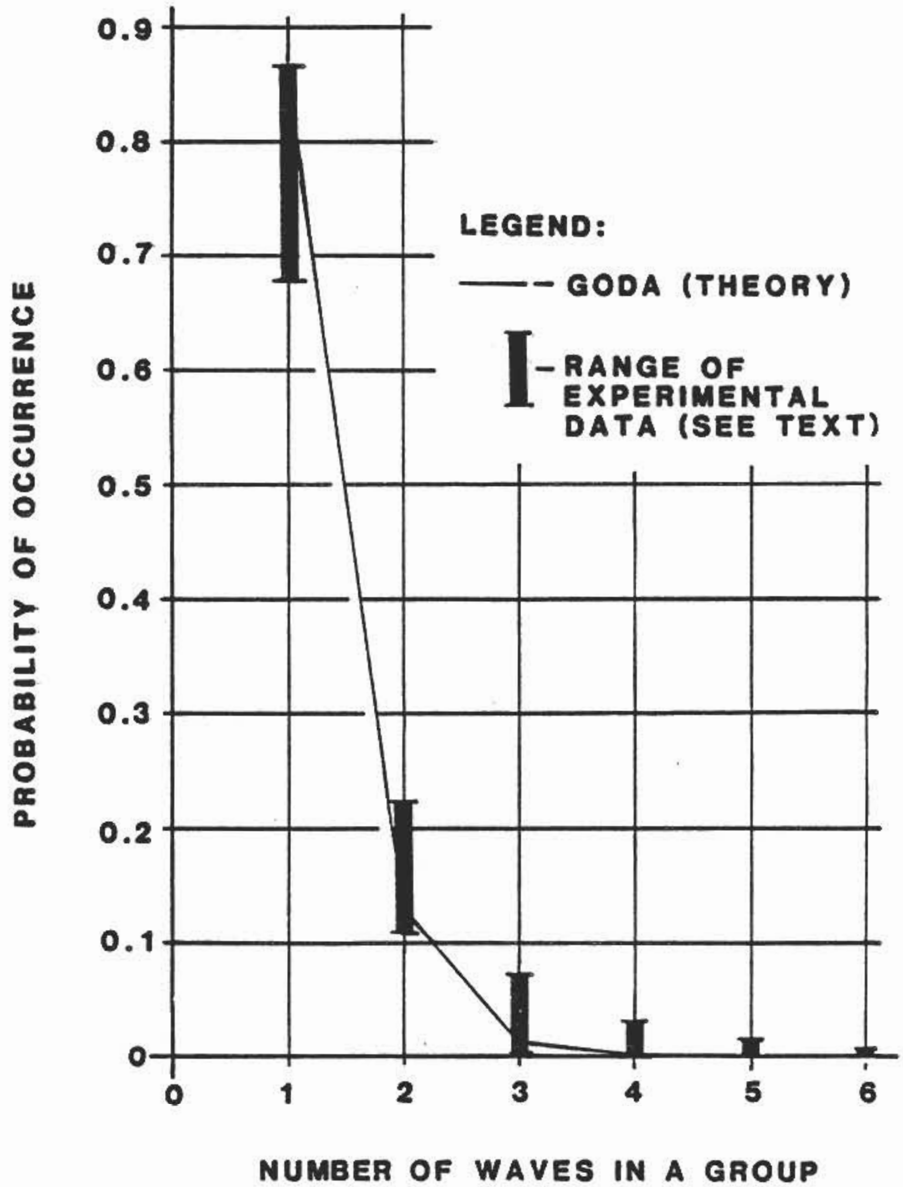
It has been shown (Siefert, 1976) that most (70-80%) groups have N=1, i.e. the occurrence of a wave $\geq H_s$ was immediately preceded and followed by waves smaller than H_s. Also, 10 to 20% of the runs had N=2, up to 7% of the runs had N=3 and up to 2.5% of the groups had N=4 (i.e. 4 waves in a row $\geq H_s$).

Figure 4.4.9 shows the probability of consecutive wave occurrence $\geq H_s$. Waves larger than H_s do occur, but the number that occur as groups with N > 2 is quite small. Thus when estimating the effect of waves on the beach in terms of run-up and overtopping, the significant wave is a reasonable parameter to use.

The hindcast procedure gives significant wave height for the region under the effect of the various wind conditions. As can be seen from Figure 4.4.8 the actual wave height distribution varies considerably over the one-hour represented by the significant wave height. Waves of different height and period occur duing this time. This wave height distribution is often represented on a wave spectra diagram as shown in Figure 4.4.10. The ordinate is given as energy (proportional to wave height-squared) and the abscissa is given as frequency (2π/period). The *energy distribution changes as the sea becomes more fully* developed. Due to short fetch length, the seas in Placentia Bay do not become fully developed. There is thus a continual change in the spectral distribution of waves entering Placentia Road.

Since, as is seen in the spectral diagrams, there is a wide variation in period associated with the wave field, the analysis of the transformation of waves from Placentia Bay, as they propagate into nearshore regions, is carried out using a wide range of possible wave periods. The dependency on wave period of wave transformation by refraction and shoaling is described in subsequent sections.

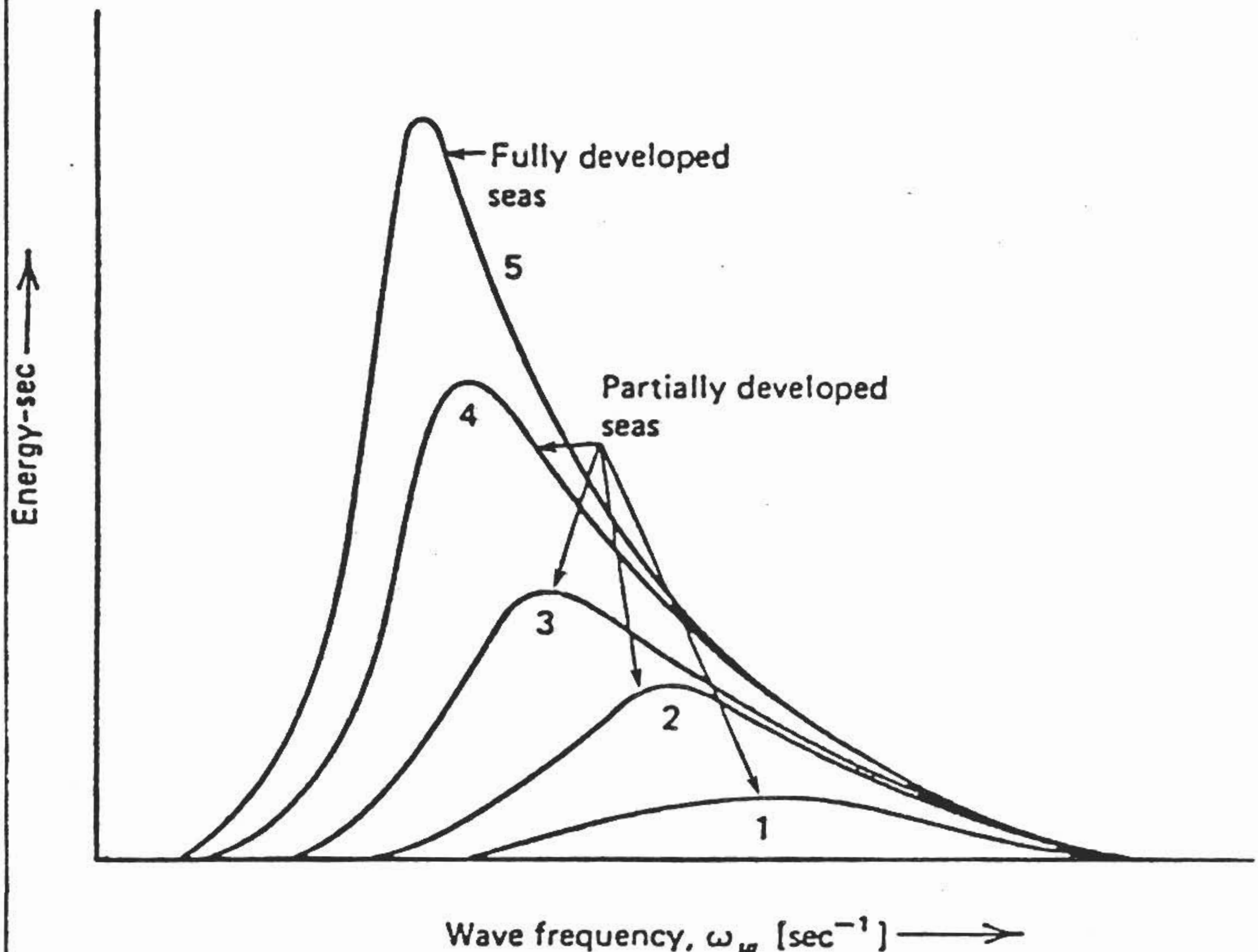
FIGURE 4.4.9



SOURCE: SIEFERT, 1976.

PROBABILITY OF WAVE GROUP FORMATION,
WAVES GREATER THAN H_{sig}.

FIGURE 4.4.10



ENERGY BUILD-UP OF PARTIALLY AND FULLY DEVELOPED SEAS

4.4.5 Wave Refraction

As waves approach shallow water, their characteristics change, among other things, due to refraction which is caused by waves slowing down in shallower water. If the bottom contours are not parallel to the wave front then each part of the wave front will travel at different speeds (slower speed for shallower water) and hence the wave front will change direction (i.e. "bend" or "refract") as it propogates into shallow water. The basic equation used to determine this bending of the wave front is that used in geometric optics, namely Snell's Law. Snell's law gives the relationship between wave speed c and the angle the wave crest makes with the bottom contour

$$\frac{c_1 = \sin\alpha_1}{c_2 = \sin\alpha_2}$$

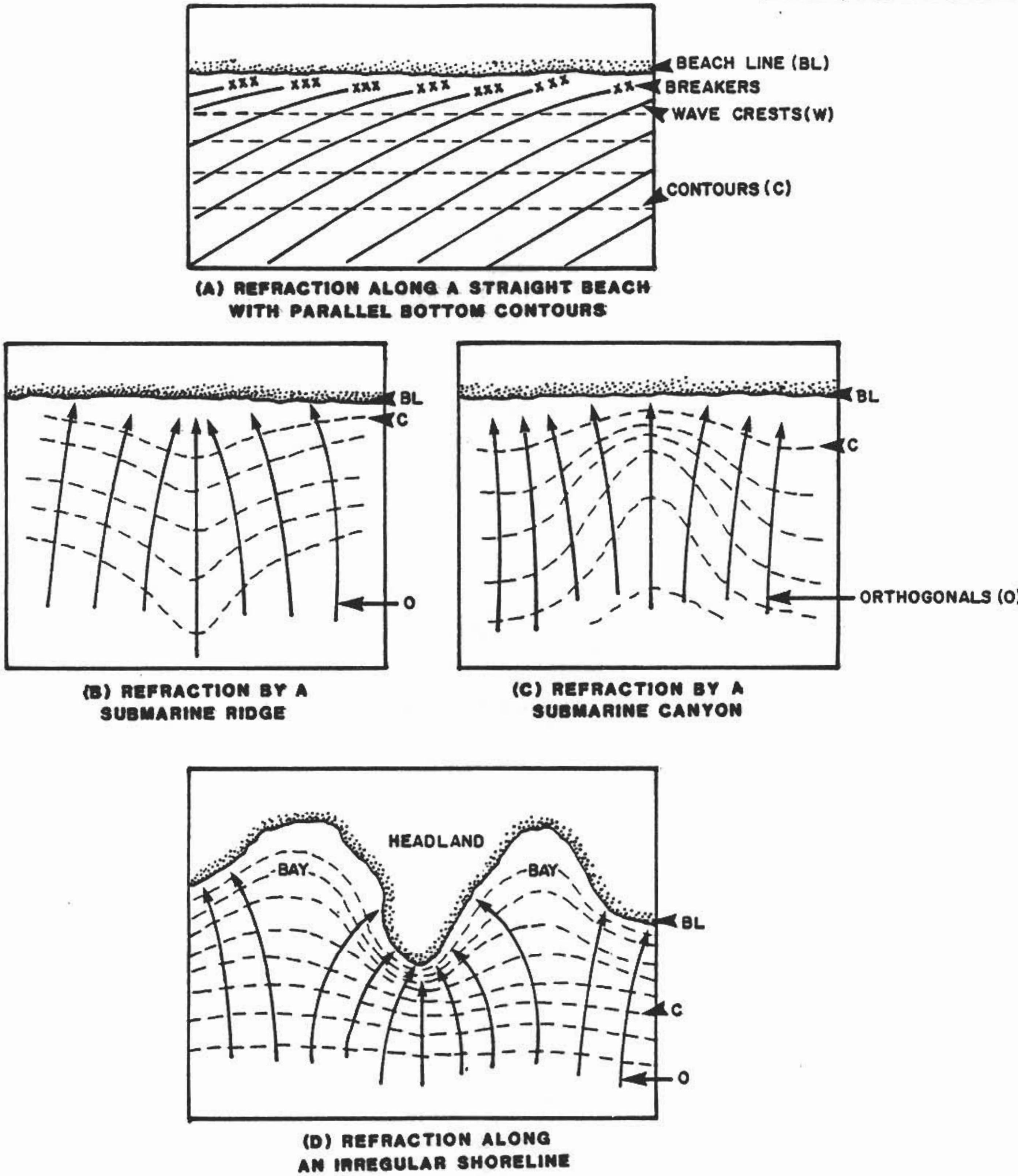
where the subscripts denote the appropriate regions. This relationship is the basis for the development of various numerical or geometric schemes for tracing paths of waves orthogonals from deep water to shoaling water in accordance with the contours describing a particualr region.

To construct these refraction diagrams computer programs are used extensively today. The procedure involves using Snell's law locally at each contour line in the offshore bathymetry that must be known. The data used for the refraction analysis was obtained from Canadian Hydrographic Service bathymetric charts for Placential Bay and Placentia Road. The wave front is started in "deep-water" (i.e. $d/L_o > 0.5$ where d = water depth and L_o = deep water wave length) with a known wave period. Wave rays (sometimes called orthogonals, i.e. at right angles to the wave crest) are equally spaced along this wave crest and then are propogated to shore starting at a particular direction.

Figure 4.4.11 shows several wave refraction diagrams and illustrates the wave refraction process over various bottom bathymetry.

The change in wave height in these refraction diagrams, can, in general, be related to the change in distance of the orthogonal lines from one another. If adjacent orthogonals converge, (e.g. the headland of Figure 4.4.11(d)) the wave height increases; if the rays remain relatively equidistant along its path, the wave height remains relatively constant and if there is a divergence of rays, the wave height decreases (as in the bay of Figure 4.4.11(d)).

FIGURE 4.4.11



TYPICAL WAVE REFRACTION DIAGRAMS

4.4.6 Discussion of Wave Refraction Results

Waves with periods ranging from 6 seconds to 15 seconds from directions between 230° to 300° were refracted in Placentia Bay and then into Placentia Road. Waves from other directions do not propagate into Placentia Bay. The bottom bathymetry in Placentia Bay causes the waves from areas outside the 230° to 300° segment to refract to the coastline on the northeastern side of the Burin Peninsula, Merasheen Island, Red Island, or the southwest tip of the Avalon Peninsula. As an example of this refraction, Figure 4.4.12 shows this wave refraction to the coastline for a wave from the 240° direction. Because of the spreading of the wave rays, very little of the energy reaches the entrance to Placentia Road.

Figures 4.4.13, 4.4.14 and 4.4.15 illustrate the refraction effects at the entrance region to Placentia Road using 6, 8 and 10 second waves. A large part of the wave energy refracts to the headlands at the entrace to Placentia Road with a reduced amount of energy propagating further into Placentia Road.

Upon entering Placentia Road, the waves are refracted further as they cross the variable bathymetry. The enlarged sketches of Placentia Road shown in Figures 4.4.13, 4.4.14 and 4.4.15 illustrate refraction in Placentia Road. The areas where wave energy is concentrated is readily identified.

For each of the cases shown, wave energy is seen to concentrate on the southern end of the beach. The concentration at this location is extreme.

The refraction analysis follows the wave into shallow water without considering the effect of wave breaking in the shallower regions. This effect is quantified in Section 4.4.7 - Wave Shoaling and Wave Run-up.

As the waves propagate into shallow water, the wave crest refracts at different rates along its length. It is seen in the refraction diagrams that the wave orthogonals cross at several locations. At these points, the wave energy increases and wave heights will increase. The process which ocurs is a complex non-linear interaction. If the incident wave is large enough, the result is a breaking wave at the intersection point with loss of energy and the reformation of a wave beyond the breaking point of lower amplitude. If breaking does not occur, the wave travels as shown in the refraction diagram, propagating to shallower water until its breaking depth is reached.

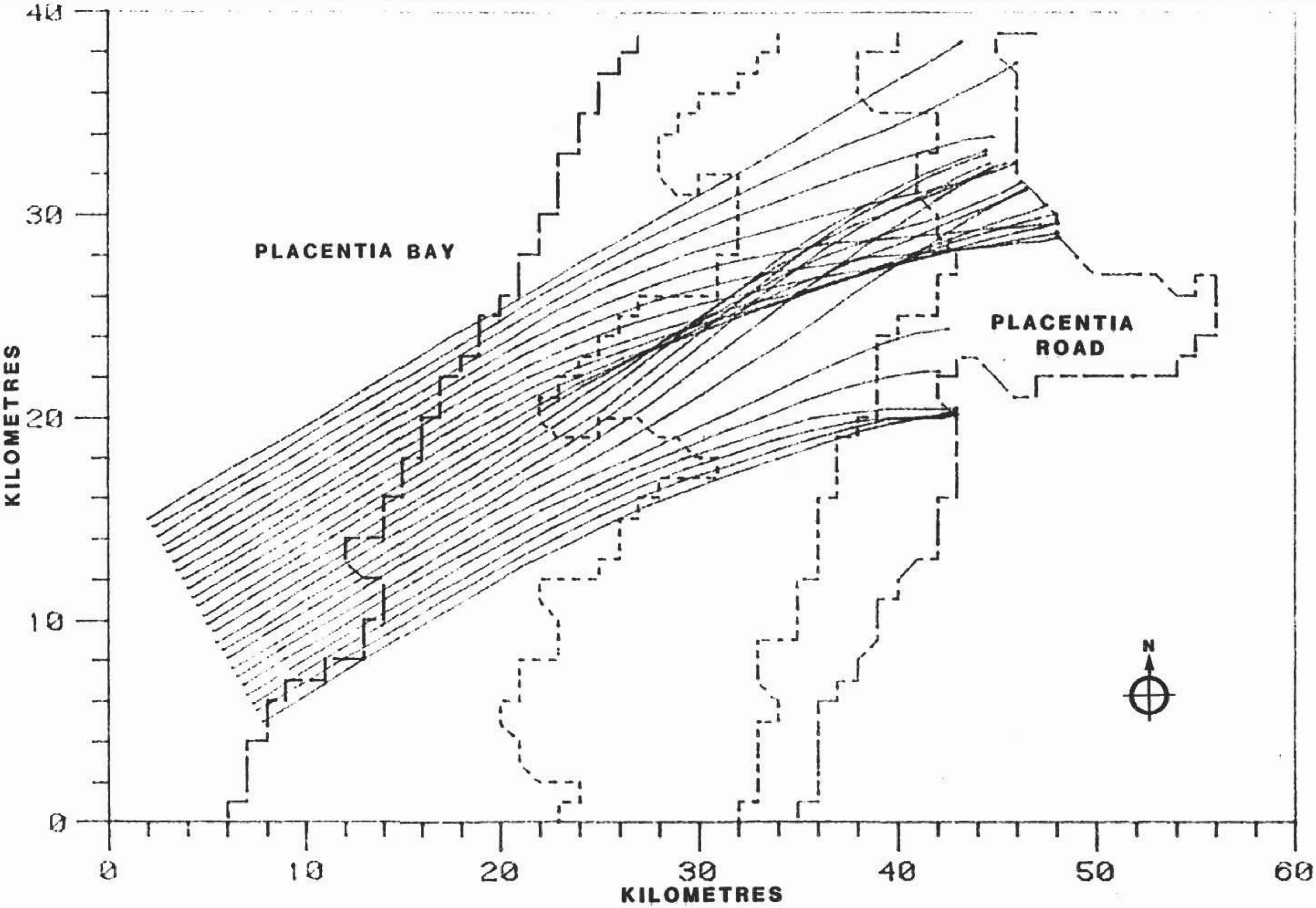
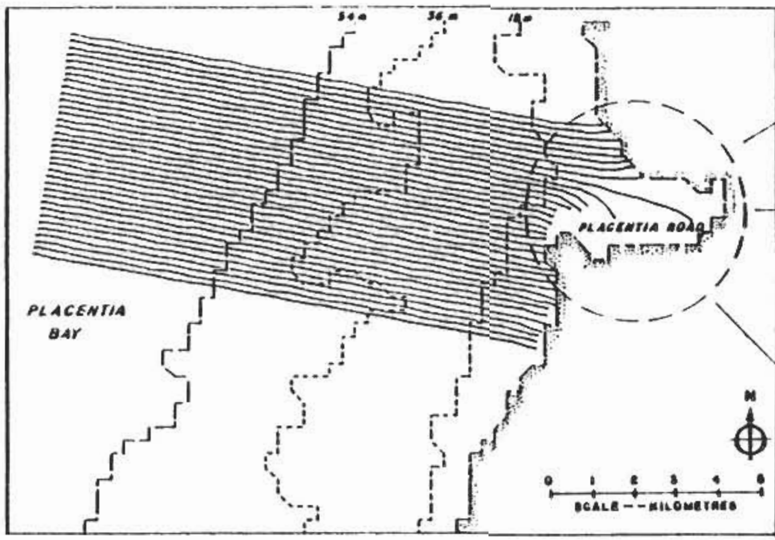


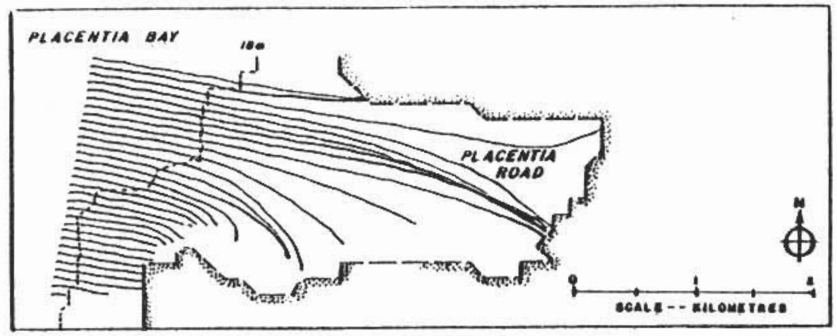
FIGURE 4.4.12

WAVE ORTHOGONALS IN PLACENTIA BAY FOR
10-SECOND WAVE AT 240° FROM NORTH

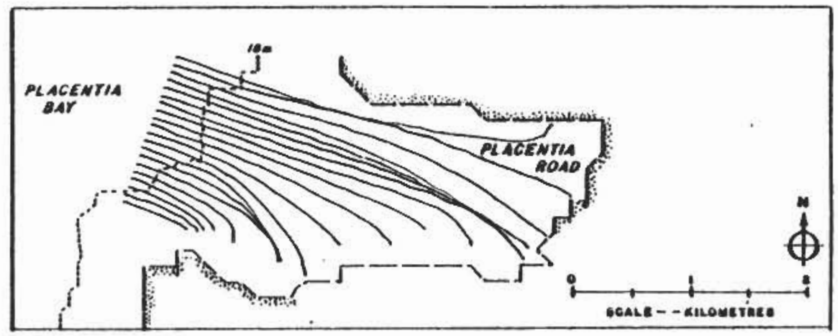
FIGURE 4.4.13



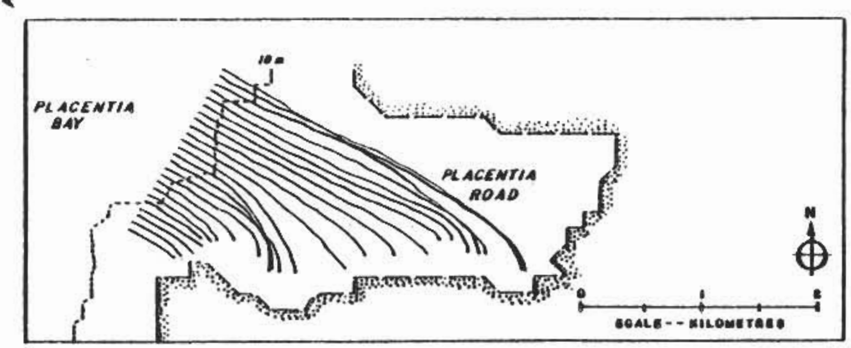
(A) PLACENTIA BAY: PERIOD 6 SECONDS AT 280° FROM TRUE NORTH



(B) PLACENTIA ROAD: PERIOD: 6 SECONDS AT 280° FROM TRUE NORTH

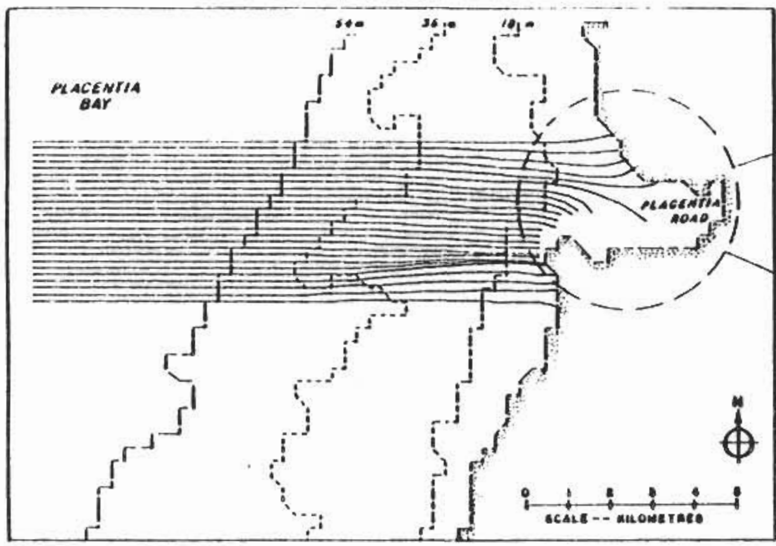


(C) PLACENTIA ROAD: PERIOD 6 SECONDS AT 290° FROM TRUE NORTH

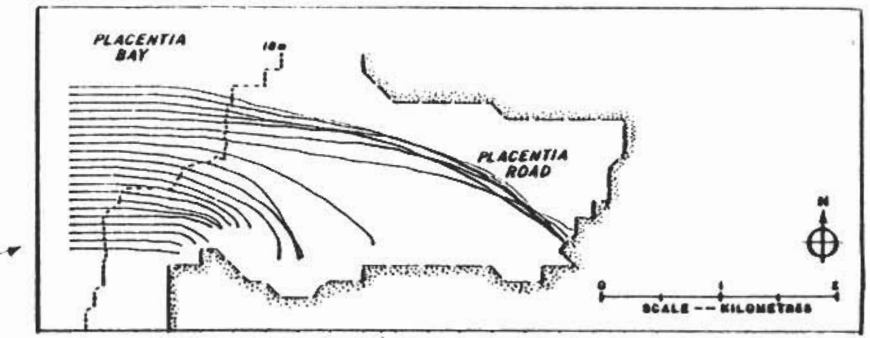


(D) PLACENTIA ROAD: PERIOD 6 SECONDS AT 300° FROM TRUE NORTH

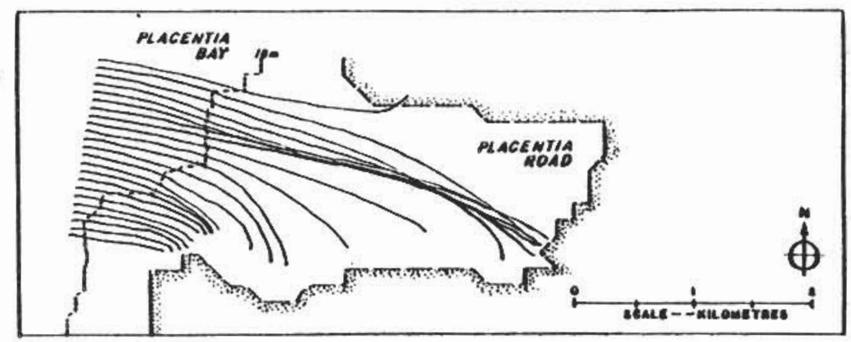
WAVE ORTHOGONALS IN PLACENTIA BAY AND PLACENTIA ROAD FOR 6 SECOND WAVE PERIOD



(A) PLACENTIA BAY: PERIOD 8 SECONDS AT 270° FROM TRUE NORTH



(B) PLACENTIA ROAD: PERIOD 8 SECONDS AT 270° FROM TRUE NORTH

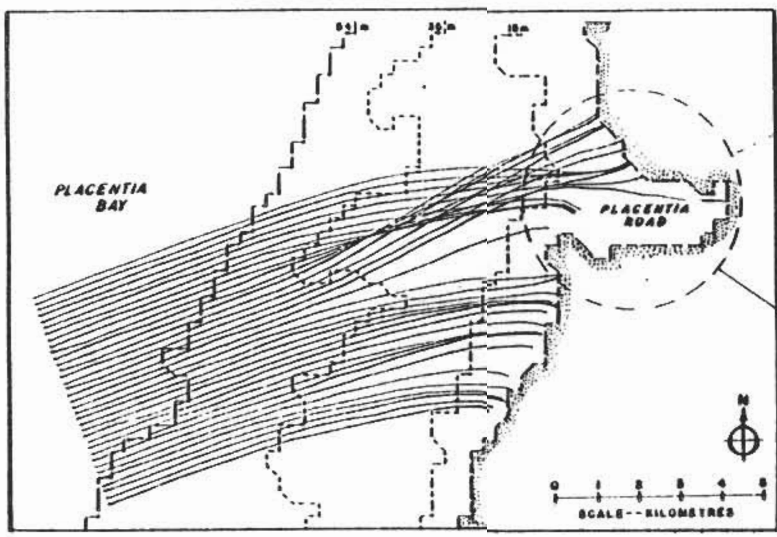


(C) PLACENTIA ROAD: PERIOD 8 SECONDS AT 280° FROM TRUE NORTH

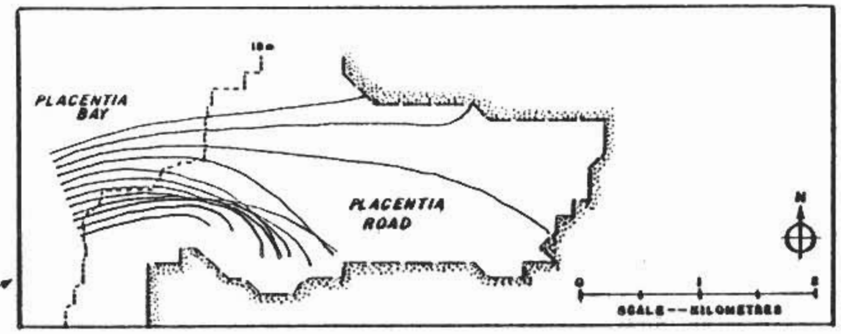
FIGURE 4.4.14

WAVE ORTHOGONALS IN PLACENTIA BAY AND
PLACENTIA ROAD FOR 8 SECOND WAVE PERIOD

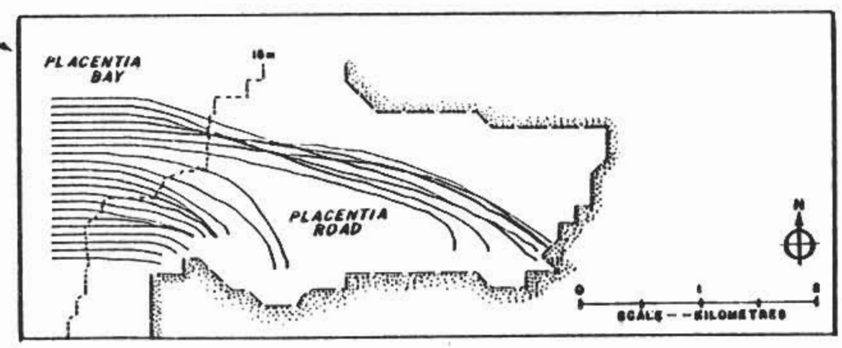
FIGURE 4.4.15



(A) PLACENTIA BAY: PERIOD 10 SECONDS AT 250° FROM TRUE NORTH



(B) PLACENTIA ROAD: PERIOD 10 SECONDS AT 250° FROM TRUE NORTH



(C) PLACENTIA ROAD: PERIOD 10 SECONDS AT 270° FROM TRUE NORTH

WAVE ORTHOGONALS IN PLACENTIA BAY AND PLACENTIA ROAD FOR 10 SECOND WAVE PERIOD

4.4.7 Wave Shoaling and Wave Run-up

i) Wave Shoaling

The maximum height of a wave travelling in deep water is limited by a maximum wave steepness for which the wave form can remain stable. This limiting steepness is approximately $H_o/L_o = 0.142$ where H_o is the deep water wave height and L_o is the wave length in deep water. When the waves move into shallow water, this limiting steepness decreases being a function of both the relative depth d/L (where d = water depth and L = wave length) and the slope of the bottom. As a wave moves into shallow water, it will steepen up to a point where wave breaking will commence and a rough estimate of the breaking wave height, H_b, to the depth at the point of breaking, d_b, is given as $H_b/d_b = 1.28$. The actual ratio depends upon the beach slope (m), as noted above and the full analysis has been used in determining the breaker wave height. In this analysis (Weggel, 1972),

$$H_b = k\, d_b$$

where $k = b(m) - a(m)$

$$a(m) = 43.8\ (1.0 - e^{-19m})$$

$$b(m) = 1.56\ (1.0 + e^{-19.5\ m})^{-1}$$

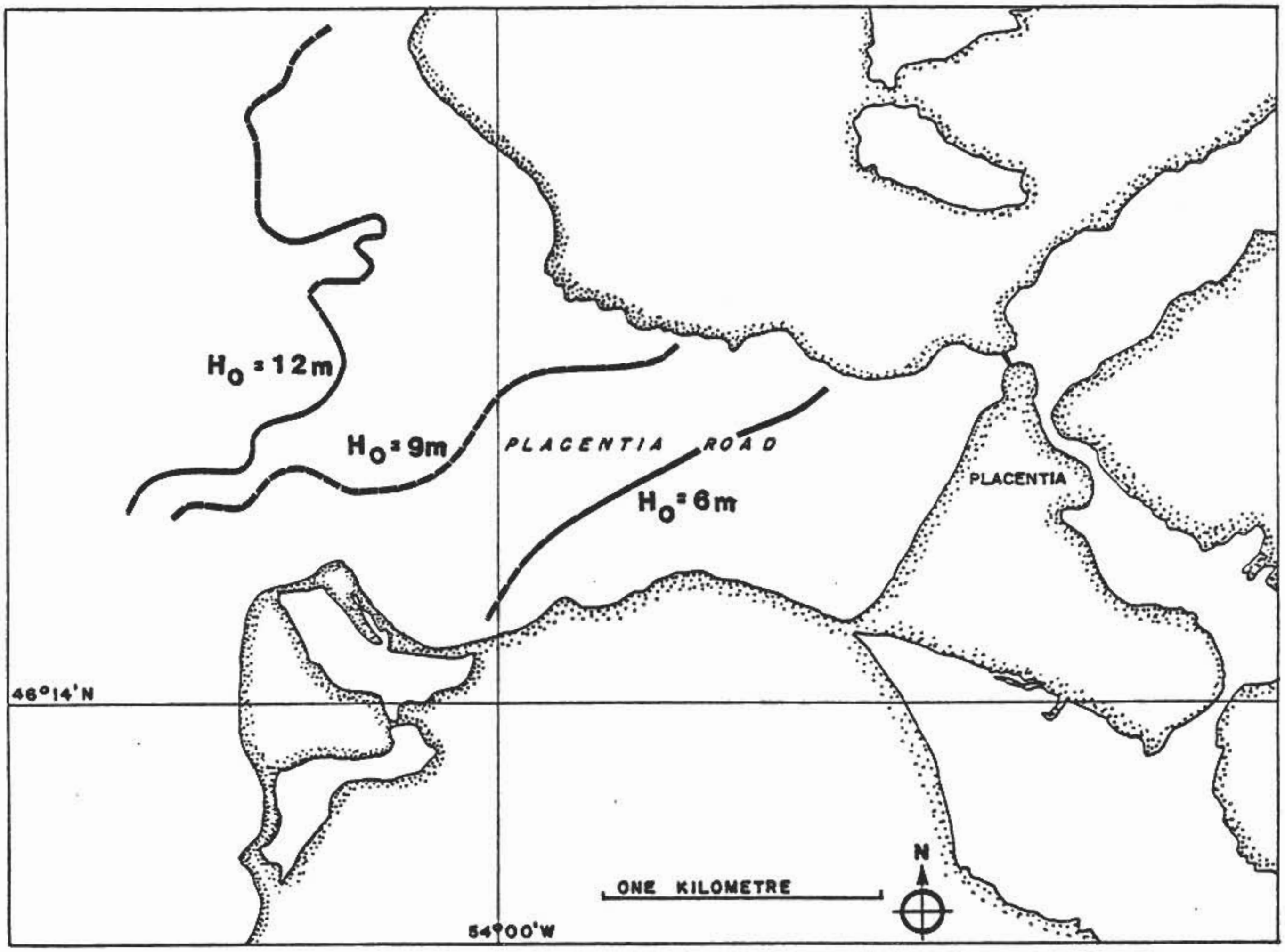
These results approach k = 1.28 as the beach slope m approaches zero.

The determination of the wave run-up on the beach involves the establishment of the location of breaking and the wave height at breaking.

The waves of different period are being affected by shoaling at different rates. However, it has been shown by Karlsson (1969) that by the time shallow water is reached, the shoaling effect on the wave spectrum is essentially the same as that of a monochromatic wave with a period equal to the maximum period of the spectrum. Thus the breaking and dissipation of the many wave trains of the spectrum could be based upon such a wave using the height of the significant wave (Silvester, 1974).

An example of the breaking location for various wave heights of 8 second period is shown in Figure 4.4.16. It is seen that the large waves (approx. 6 meters in height and 8 second period) will break mid-way out in Placentia Road at low tide. If refraction increases the wave height beyond 6 metres in Placentia Road, the waves break further out in Placentia Road. Even under high tide conditions, the extremely large waves (greater than 6 meters in height) will break near the outer limits of Placentia

FIGURE 4.4.16



LOCATION OF BREAKING REGION
FOR 8 SECOND WAVES AT
THREE DIFFERENT WAVE HEIGHTS

Road.

ii) Wave Run-up

The wave runup, R, is defined as the vertical distance above the existing water level that the wave propagating to shore will reach. Based upon field data and hydrographic charts, beach profiles were determined for three sections along the beach, and for analysis purposes an average representative cross-sectional profile was determined. Figure 4.4.17 shows this profile and a wave runup for a particular water elevation and wave condition for illustrative purposes.

To determine the wave run-up, R, the procedure followed is outlined below (U.S. Army, Corps of Engineers, 1977; Silvester, 1974):

1. A water elevation is chosen.

2. A wave with a particular unrefracted deep-water wave height, H_o', and wave period, T_o, was selected. The deepwater wavelength is calculated by $L_o=5.12\ T_o^2$.

3. From (1) and (2) the ratio of the breaker height, H_b, to the unrefracted deep-water wave height, H_o', is obtained for the given nearshore slope (approx. 1:30). This is normally referred to as the shoaling coefficient K_s.

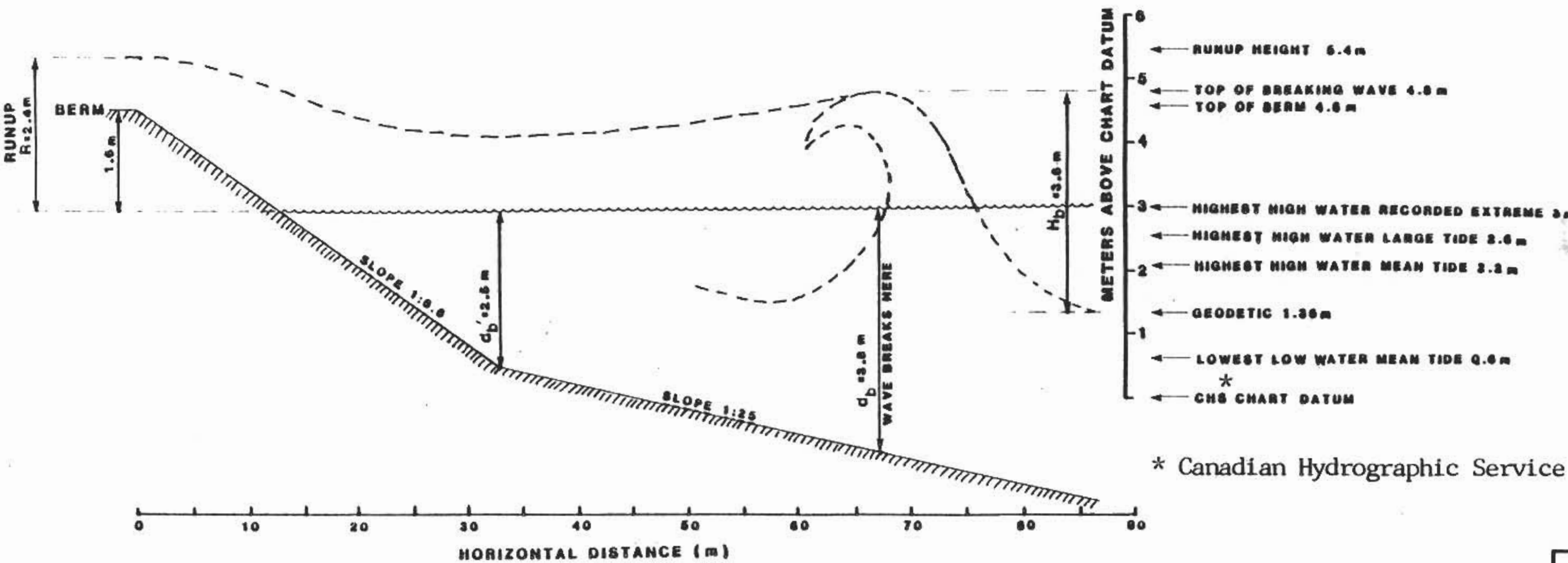
4. From the known breaker height, H_b, the location where the wave breaks is determined by finding the water depth at breaking, d_b. The location at which waves of various height break as they move into Placentia Road are shown in Figure 4.4.16 for 8 second waves.

5. For the particular water elevation chosen, the wave was propagated to the toe of the steeper beach slope. During the passage of the surging wave to this location, a reduction in wave height from H_b to H_b' will occur.

6. This reduced wave height, H_b', is then converted to an equivalent deep-water wave height, H_{oe}. From this the run-up is then calculated using the equation

   $R = (H_{oe}\ L_o)^{1/2}\ (\alpha\pi/180)$ where $\alpha = 8.2°$.
   where
   R = run-up, metres
   H_{oe} = equivalent deep water wave height, m

FIGURE 4.4.17



* Canadian Hydrographic Service

ILLUSTRATION OF BREAKING AND RUN-UP OF 9 SECOND WAVE

L_o = deep water wave length
α = angle of beach profile from horizontal

7. The procedure is repeated for other wave heights and periods at this particular water elevation (i.e. start again at (2)).

8. A new water elevation is chosen as per Step (1) and the process repeated.

Using the analysis methods described above, a set of graphs were plotted to enable a relatively fast method of determining if wave run-up presents a problem for a particular situation.

Consider Figure 4.4.17 as an illustrative example of the use of one of these plots to determine if wave runup occurs:

1. The water level chosen was HHWRE (Higher High Water Recorded Extreme), which was 3 metres above chart datum.

2. A wave having H_o' = 3 metres and T = 9 seconds was chosen.

3. Using tables in the above references, H_b/H_o' = 1.18. The breaker height H_b is then 3.6 metres.

4. The water depth at breaking was determined to be 3.8 metres.

5. The depth at the toe of the beach slope, d_b', is 2.6 metres at this water elevation. During propagation of the wave, which breaks in 3.8 metres of water, the wave height will decrease from 3.6 metres at breaking to a height of 2.5 metres at the toe. So H_b' = 2.5 metres.

6. The equivalent deep-water wave height, H_{oe}, is calculated to be 2.1 metres and has a period of 9 seconds. This equivalent wave is then 'runup' the beach slope of 1:7 and the runup R is given by

$$R = \sqrt{H_{oe} L_o}\,(.15) = 2.4 \text{ metres}$$

Since the elevation of the top of the beach is about 1.6 metres above the water level chosen it is seen that this wave will well overtop the beach top by about 0.8 metres and water will rush over this area.

This analysis was then repeated for different wave heights and periods for this particular elevation. Figure 4.4.18 shows the result of this for 9-second and 7-second waves of varying wave heights. For this water elevation it is readily seen that overtopping will occur for all waves above the horizontal line of 1.6 metres.

For a water elevation of 1.7 metres above chart datum Figure 4.4.19 shows the wave run-up for the same waves as for HHWRE. The horizontal line reprsents the minimum run-up, R, required for overtopping (i.e., approx. 3 metres). The depth of the toe, d_b', is about 1.2 metres below the chosen water line. It is seen that for this water elevation there are no waves generated which can overtop the beach front.

For all cases of wave run-up it is noted that as the wave height increases beyond 4.5 to 6 metres the wave run-up R changes very little due to the fact that wave breaking occurs further and further from shore as the wave height increases and hence has much further to travel before it reaches the toe of the beach. This will then limit the wave run-up.

Figure 4.4.20 illustrates the waves required to just reach the top of the beach for various water elevations. Consider a water elevation of 2.4 metres above chart datum which is the average of HHWLT (Higher High Water Low Tide) and HHWMT (Higher High Water Mean Tide).

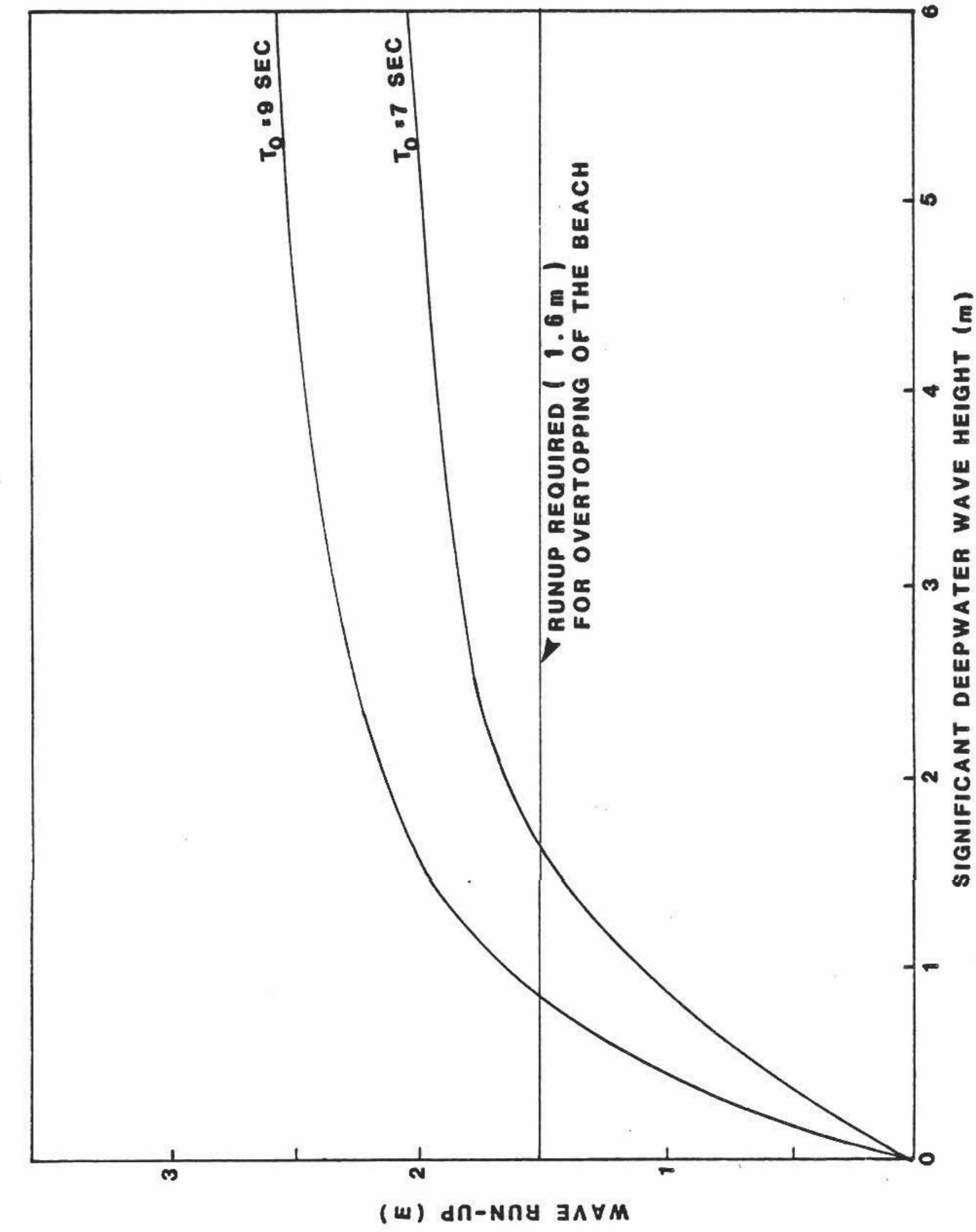
The run-up is dependent upon the wave height, wave period and water elevation. Figure 4.4.20 shows the interrelationship for these three variables for Placentia beach.

The wave period and wave heights are given as ordinates in the figure, and the curves are lines of constant run-up for a particular wave elevation.

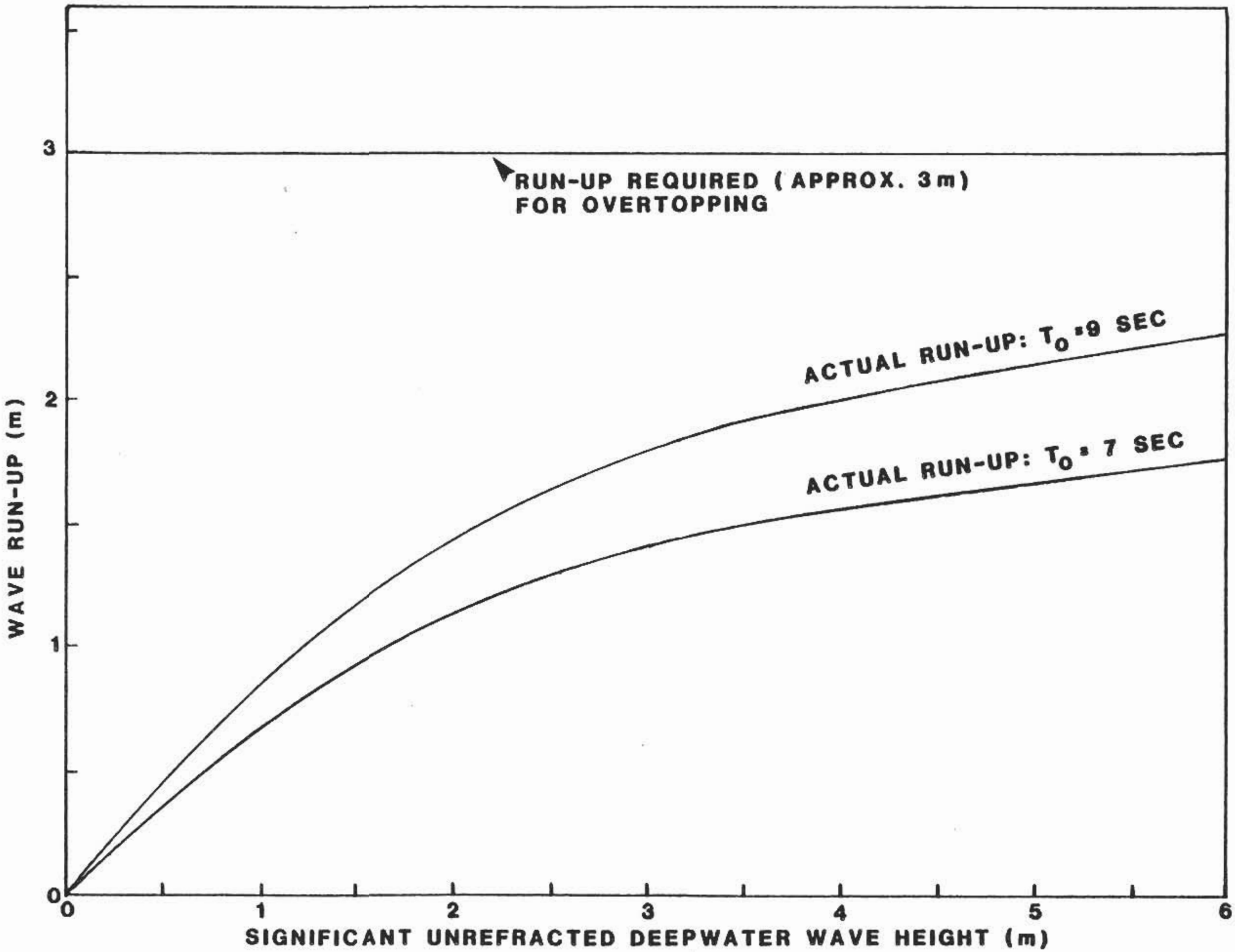
For any water elevations not explicitly shown, one can extrapolate between the curves. Thus one can determine whether or not a particular wave (height and period) will cause overtopping for any water elevation.

Thus when there is a water elevation of 3 metres in Placentia Road, the run-up required to reach the berm is 1.7 metres. Any wave height and wave period which lies on this curve will result in this critical run-up value (R_{crit}). For example, a wave height of 3 metres with a period of 6 seconds will reach the berm (when the water elevation is 3 metres). In this 3 metre wave, any period less than 6 seconds will not reach the top while any period greater than 6 seconds will cause overtopping.

FIGURE 4.4.18

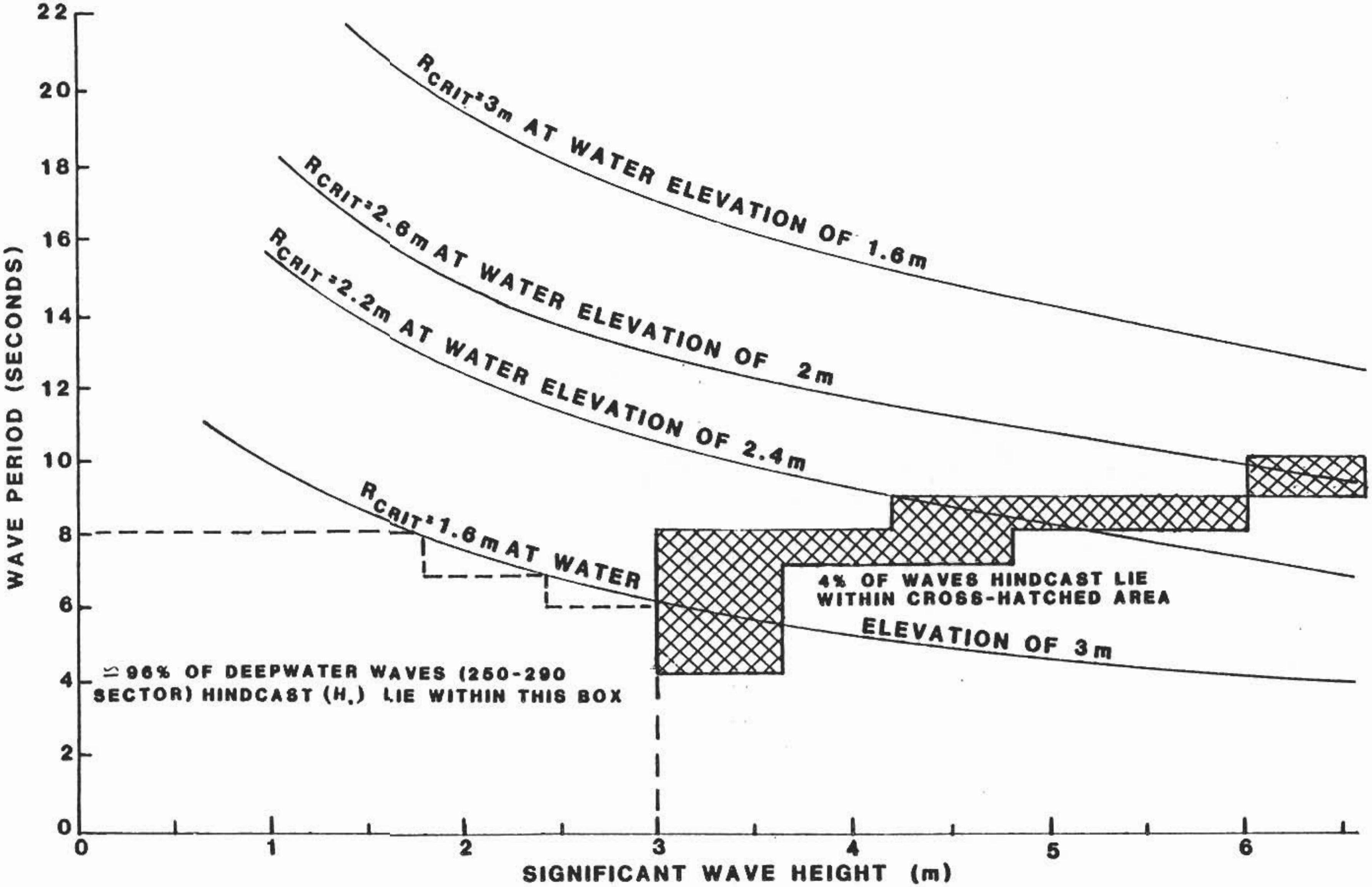


WAVE RUN-UP FOR 7 & 9 SECOND WAVES FOR WATER ELEVATION AT HIGHER HIGH WATER RECORDED EXTREME



WAVE RUN-UP FOR 7 AND 9 SECOND WAVES
FOR WATER ELEVATION 1.7m ABOVE CHART DATUM

FIGURE 4.4.19



WAVE HEIGHTS AND PERIODS REQUIRED FOR RUNUP TO REACH BERM AT FOUR WATER ELEVATIONS

FIGURE 4.4.20

4.4.8 Specific Case Studies

Two cases will be presented here involving two specific flood histories.

Case I - January 20, 1977

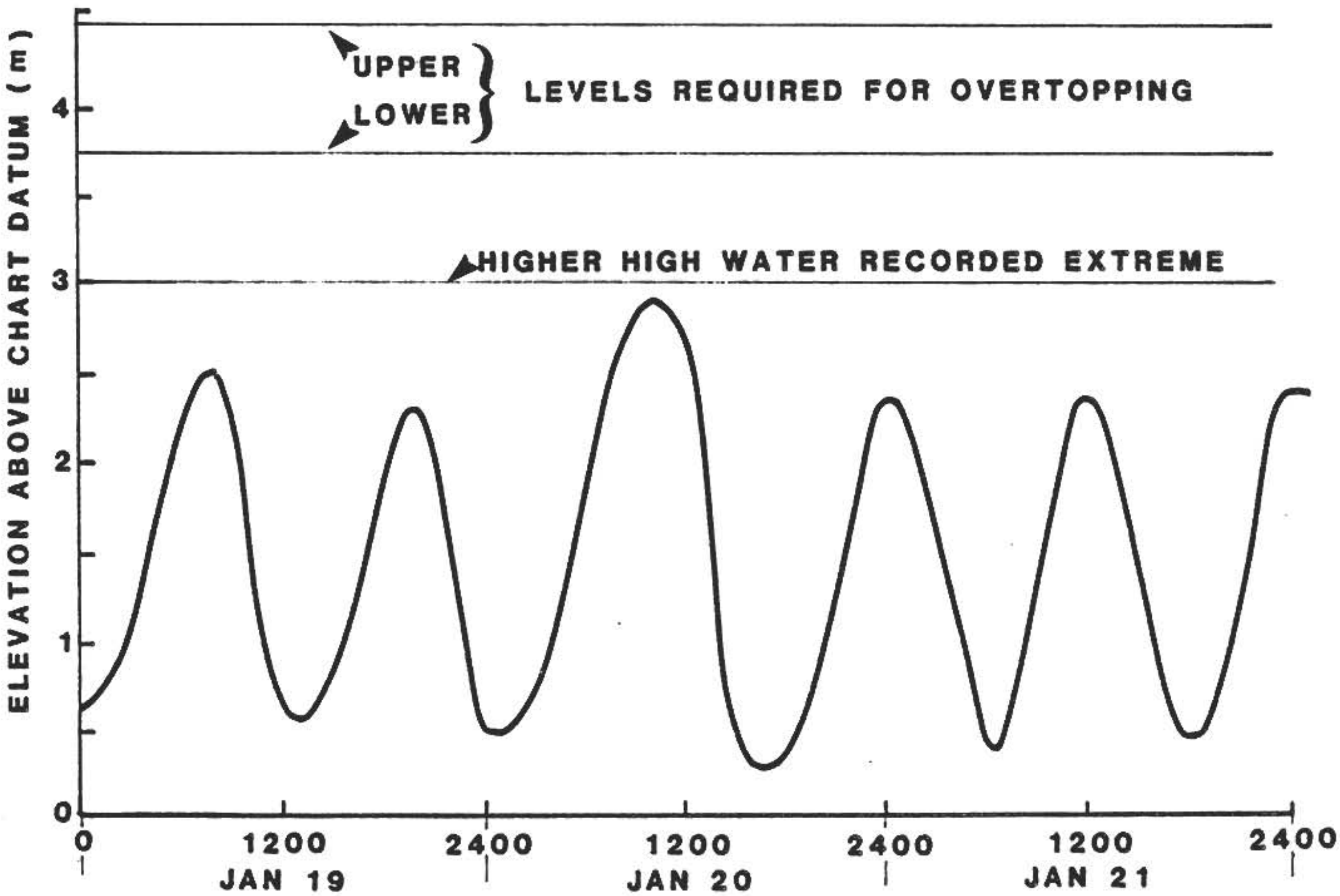
Figure 4.4.21 shows the water elevation record for the period Jan. 19, 1977 to Jan. 21, 1977. As can be seen, the water elevation was very high (2.97 metres above chart datum) around 1000 hours on Jan. 20, 1977. However, at this time the winds were relatively light from the east and hence there was no problem of wave runup. The water elevation was high again (2.5 metres) between 1000 and 1200 hours on Jan. 21, 1977. At this time there were very high waves impinging on the beach for a period before, during, and after this high-water level occurrence. For a 2.5 metre water elevation the top of the beach was about 2 metres above this. In the large waves propagating to the area at this time the run-up was such that there would be some overtopping for a couple of hours. However, if the upper beach elevations were at somewhat lower (0.6 to 1.2 metres) values than at present there would have been more or less continuous overtopping for these few hours.

January 1982 Flood

Figure 4.4.22 shows the water elevation record for Jan. 16, 1982 to Jan. 18, 1982. It was reported that flooding occurred due to runover around midnight Jan. 16, 1982. The winds were not that severe at this particular time but were very high from the south-southwest the previous night, and then began to pick up again the following evening. On further checking it was determined that the top of the beach was at a somewhat lower elevation than at present and, due to the high waves the previous evening, breaching occurred permitting the water to run over the road. However, with the beach berm area at its present elevation (rock was piled along the beach from the dredging operations in 1982), similar environmental conditions to those that occurred during the January 16, 1982 flood would not produce any overtopping due to wave runup.

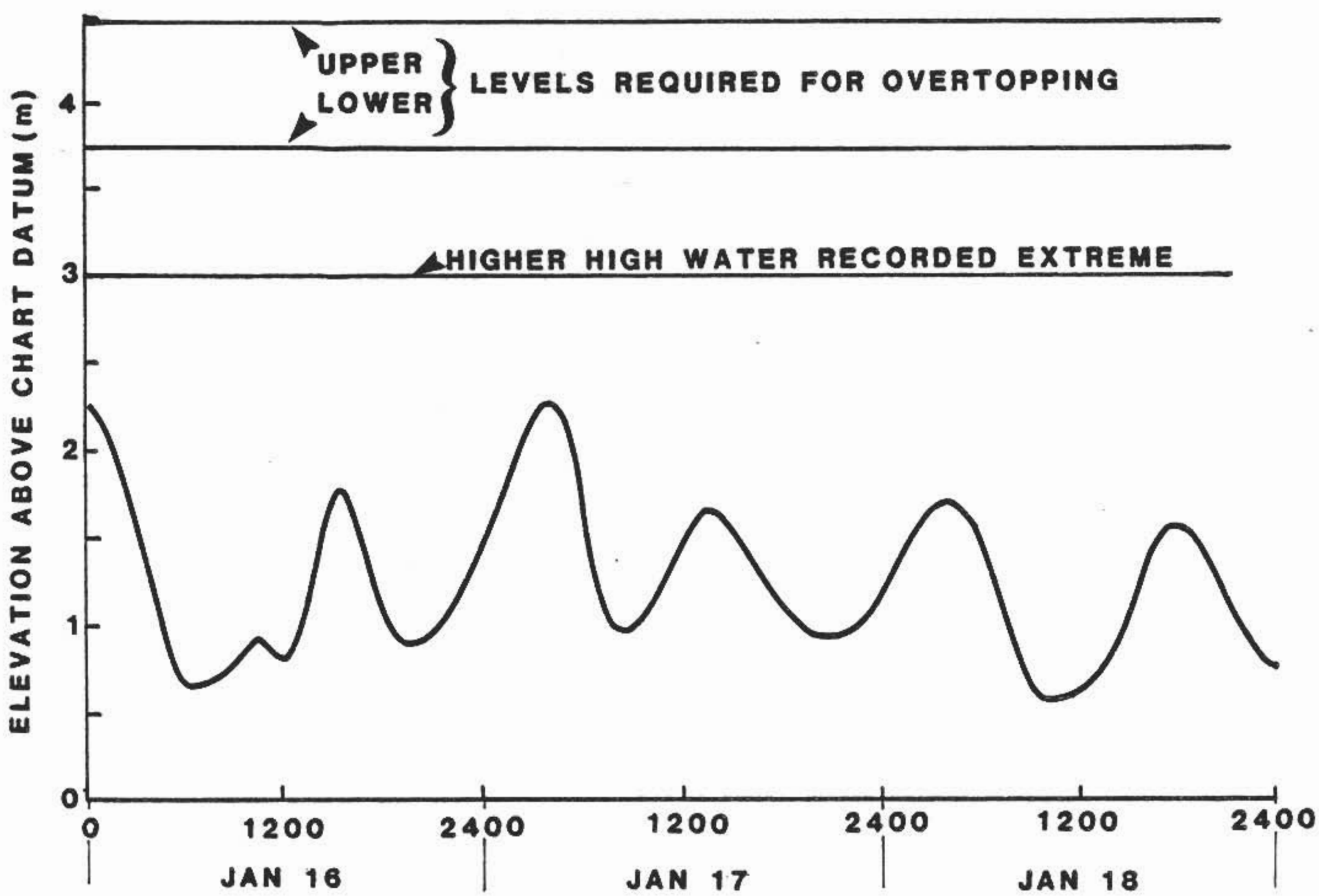
4.5 PHYSIOGRAPHIC CHANGES

Sediment transport on the beach at Placentia has been shown (Woodward - Clyde, 1982) to be from the South to North, towards the Gut. The installation of two groynes at the Northern end of the beach have clearly shown that a substantial volume of sediment is transported in the nearshore region. These groynes have in fact become filled with sand and it is now necessary to remove the deposited sediment to allow the groynes to continue to



WATER ELEVATION RECORD
JANUARY 19-21, 1977

FIGURE 4.4.22



WATER ELEVATION RECORD
JANUARY 16-18, 1982

operate as originally intended, otherwise Placentia Gut will become a deposition region.

The changes that have occurred in Placentia Gut are shown in Figure 6.3.1. At the time of the field program in 1983, the Gut appeared to have stabilized with a coarse boulder bottom layer.

The beach itself is subject to intense wave action and subsequent readjustment of profile with large storms. There is no apriori method to establish this readjustment for a boulder beach of this type. Monitoring of beach profiles is required over several seasons and concurrent wave and water level measurement during storm conditions would be required to establish an explicit relationship for the long term changes in the beach.

PART FIVE

HYDRAULICS

5.1 GENERAL

As noted in Part 2 of this report, and the Field Program Report (Volume 2), flooding in the Placentia Area has occured from two sources:

i) high water levels in the Arms, and
ii) waves overtopping the beach to the west.

This part of the report will deal with the first source of flooding, high water levels in the Arms.

The aim of the hydraulic study of the area was to apply a mathematical model to the area which would enable Placentia Road tidal levels, and other hydrometeorological factors such as wind, and freshwater runoff, to be used to hindcast an annual series of water levels in the three Arms (Northeast, Swan and Southeast).

5.2 HYDRAULICS OF THE STUDY AREA

As mentioned in Section 2.1 of this report, Northeast Arm is connected to Placentia Road by a channel, called The Gut and to Swan Arm by a long, narrow and relatively shallow channel, called The Narrows. Also mentioned was the fact that Swan Arm is connected to Southeast Arm by a short and relatively deep channel, called MacDonald Gut. The water levels in the three Arms are under tidal influence and vary quickly, following the diurnal cycle of the tides in Placentia Bay.

On a rising tide in Placentia Bay, water flows from Placentia Road, through The Gut and into Northeast Arm. From there it flows through The Narrows into Swan Arm and hence through MacDonald Gut into Southeast Arm. As the tide falls in Placentia Bay, the flow reverses and the water flows back through The Gut into Placentia Bay via the Road.

The Gut, The Narrows and MacDonald Gut are hydraulic controls in the system. The tidal exchange between Placentia Road and the three Arms (Northeast, Swan and Southeast), through the hydraulic controls, results in differential water levels in all four areas. The water level differentials result from the throttling effect on the flow of water provided by each control, with the consequential attenuation of the water level fluctuation downstream of the control. The more throttling of flow that is provided by a control, the greater is the attenuation of water level fluctuation beyond the control. This attenuation of water levels occurs with the flow of water in either direction through a control. Figure 5.2.1 illustrates the attenuation of water levels in the three Arms. Note the resulting water surface profile of diminishing water levels from Placentia Road through the controls and water passages to Southeast Arm and vice versa when the flow is in the opposite direction.

The Towns of Placentia and Jerseyside are connected by road over a bridge across The Gut. The bridge is a three span steel structure supported by abutments on shore and by two wide piers in the channel. The channel is deepest between the two piers (approximately 7m below mean water level) and there is a noticeable channelization of flow between the piers. Since 1960 cross-sections have been taken at the Bridge. These are shown on Figure 5.2.2. During the field program, three additional cross-sections were taken in the Gut. These are shown in the Field Program Report.

The Narrows is about 800 metres long, generally very shallow and is obviously subject to sediment deposition. The deepest water in the channel is along the side opposite the Town while portions of the side adjacent to the Town are dry at low tide. The original channel has been infilled along the side adjacent to the Town as part of the Town's development. A timber crib breastwork retains the fill along the entire length of The Narrows and around the northwest corner of Swan Arm. During 1982, a section of The Narrows was dredged which increased the flow area. During the field program, six cross-sections were taken in The Narrows, some of which were in the dredged section. These are shown in the Field Program Report.

MacDonald Gut is a relatively short, deep channel in an apparently stable location. It does not appear to be subject to sediment deposition. During the field program, four cross-sections were taken in this area and these are shown in the Field Program Report.

5.3 HYDRAULIC MODEL

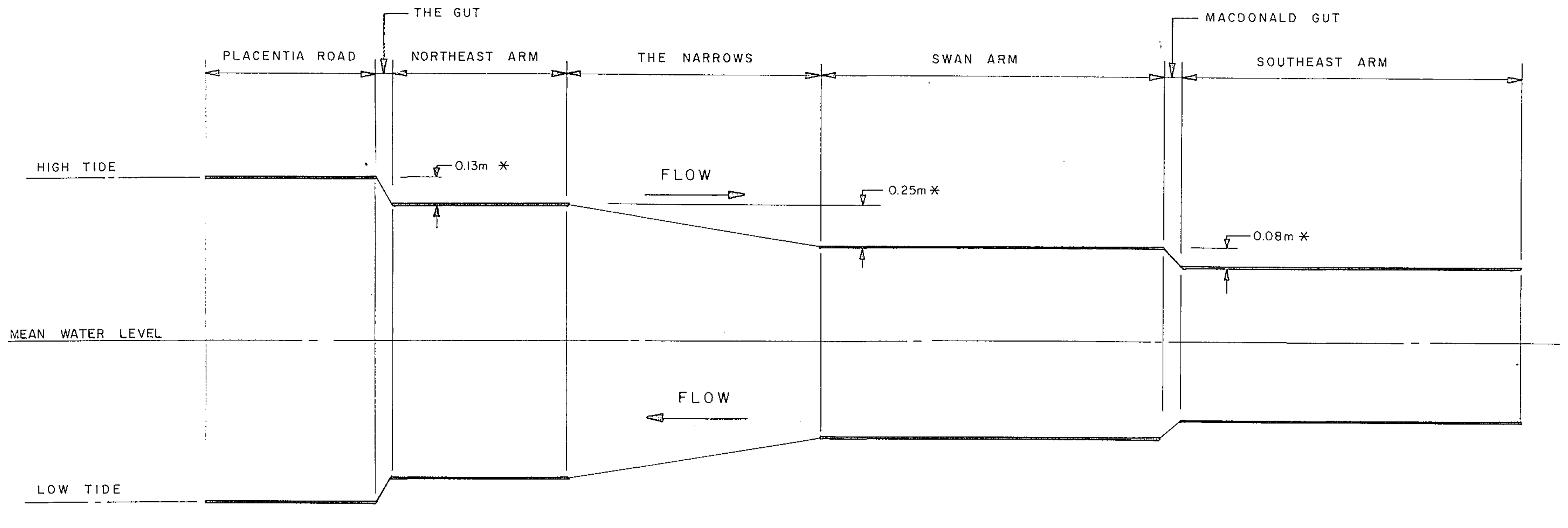
5.3.1 Model Selection

The following hydraulic models were considered to represent the water level fluctuations within Placentia Road and through the Gut to Northeast Arm, Swan Arm and Southeast Arm.

1. The HEC-5 Model - U.S. Army Corps of Engineers.

2. DWOPER - The Dynamic Wave Operational Model of the U.S. National Weather Service (Fread, 1978).

FIGURE 5.2.1



* AVERAGE ATTENUATION OF MAXIMUM WATER LEVELS FROM TABLE 5.5.1

(NOT DRAWN TO SCALE)

HYDRAULIC PROFILE

PLACENTIA ROAD - SOUTHEAST ARM

5.3.1 Model Selection (Cont'd)

3. 1-D Model - One Dimensional Hydrodynamic Model (Environment Canada, 1982).

Each of the above models was reviewed for its applicability to this study. The HEC-5 was irrelevant for this application since it was designed primarily to investigate the behavior of multiple reservoir systems and could not handle the dynamic tidal boundary and the reversing flow condition.

DWOPER solves the one-dimensional nonlinear flow equations for a dendritic channel system. It can include the effects of varying surface elevation at the open ocean end of the system, which will cause periodic flow reversals. It also simulates the effect of wind and of water storage in inundated areas and bays. The two-dimensionality of the basin geometry is recognized by allowing specification of cross-sectional shape. It is therefore possible to assess the effect of changes in topography caused by dredging or natural redistribution of materials. From a practical numerical point of view, DWOPER uses a finite difference implicit solution scheme which is simple to set up and is computationally very efficient. This model has been successively applied to many river systems (Fread, 1982) including the Saint John (New Brunswick) and Columbia (British Columbia) river systems which, as the Placentia inlet system, are strongly influenced by tides.

The 1-D model solves the one-dimensional nonlinear flow equations for a dendritic channel system similar to DWOPER and consideration was given to using either 1-D or DWOPER for this study. Previous studies of dynamic modelling techniques evaluated DWOPER and 1-D. DWOPER was chosen because of its comprehensiveness, availability and documentation (Perks et al 1983) (MacLaren Atlantic Limited, 1979). It was felt, therefore, that DWOPER would be the most suitable model for this study.

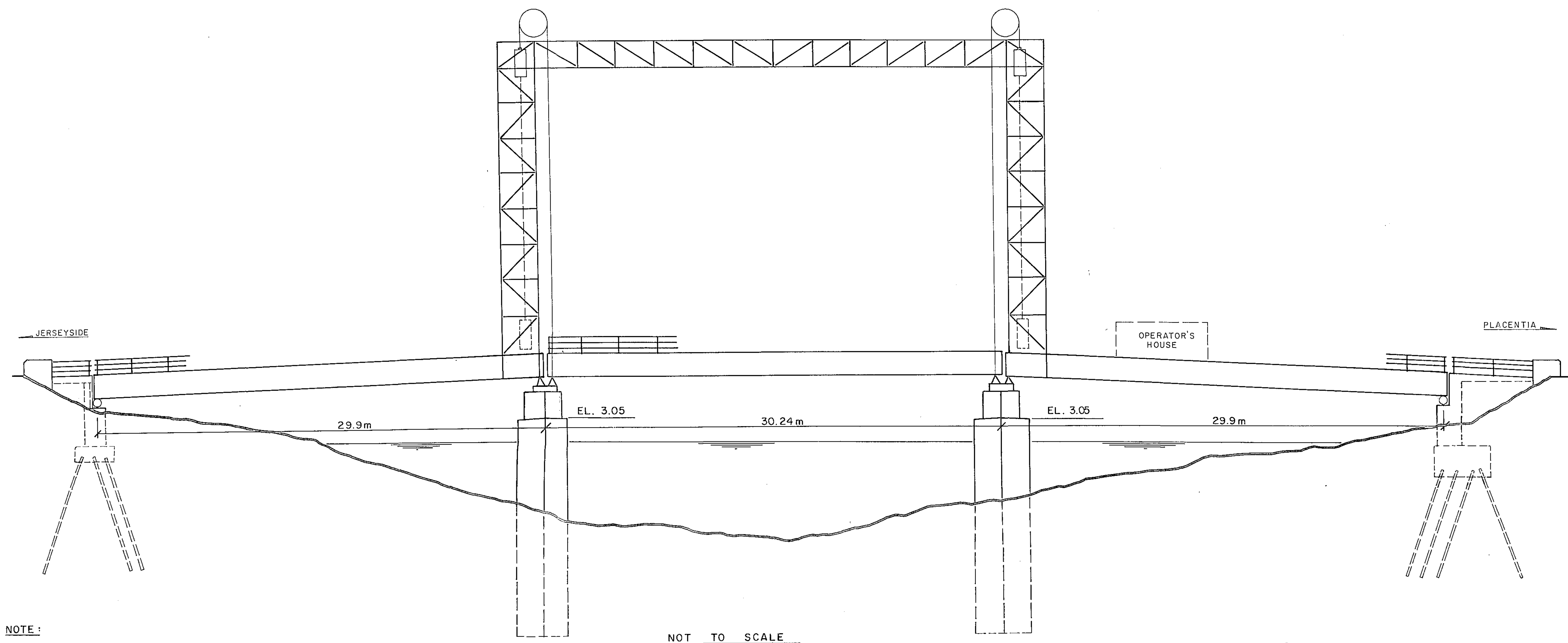
The DWOPER model has the following characteristic features which were factors in its selection:

- DWOPER is based on an implicit finite difference solution of the complete one-dimensional unsteady flow equations.

- Wind stress is easily entered into the program.

- DWOPER has an automatic calibration feature for determining optimum roughness coefficients for either a single channel or a system of interacting channels.

FIGURE 5.2.2



NOTE:
ELEVATIONS ARE IN METRES BASED ON GEODETIC DATUM

SIR AMBROSE SHEA LIFT BRIDGE

5.3.1 Model Selection (Cont'd)

- Overbank storage is accounted for.
- DWOPER can accept cross-sections spaced at irregular intervals along the river system.
- DWOPER is generalized for wide applicability to rivers of varying physical features, i.e. irregular geometry, variable roughness parameters, lateral inflows, flow diversions, off-channel storage, local head losses and wind effects.

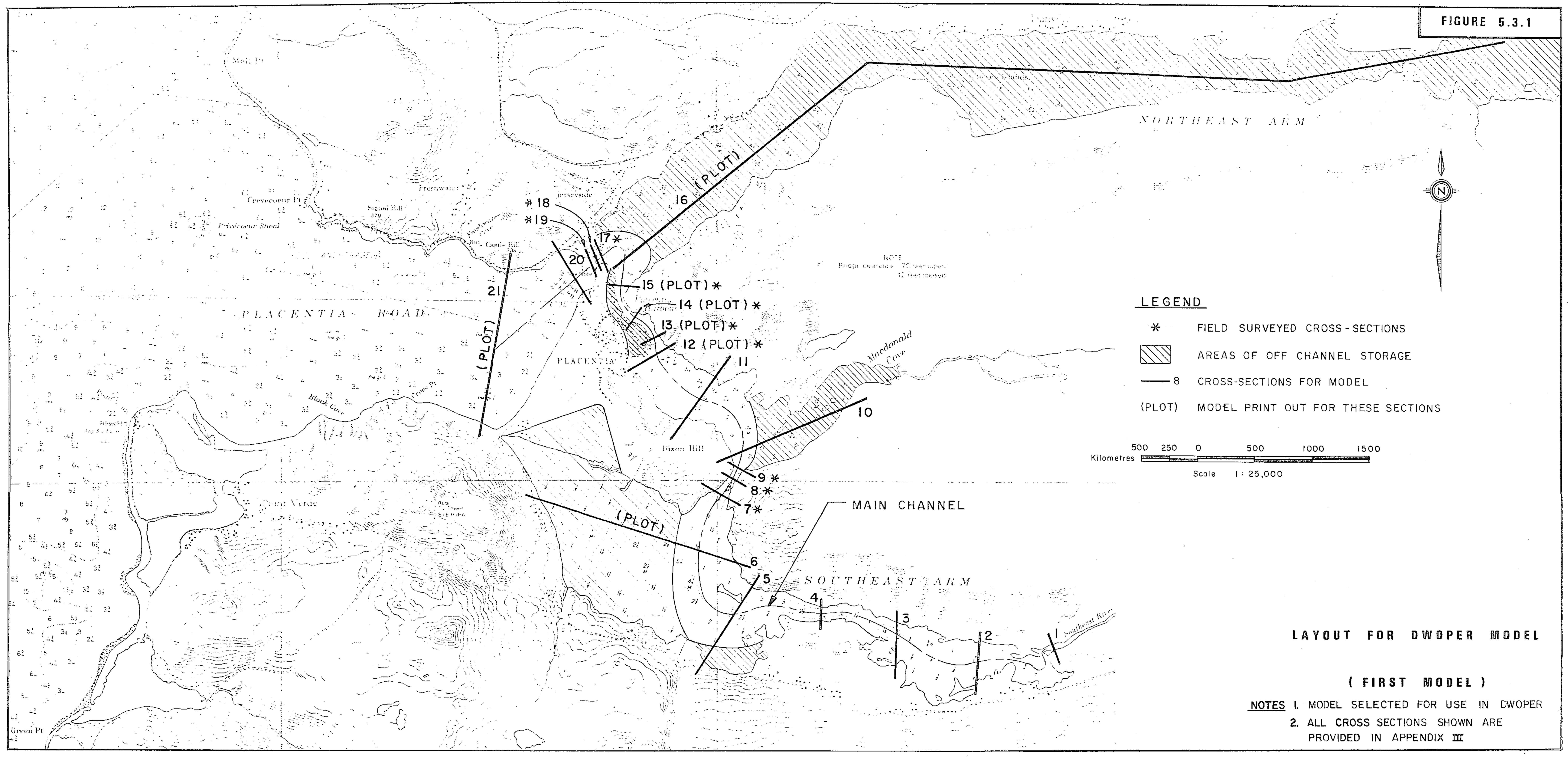
The boundary conditions for DWOPER were obtained from river flow at the upstream end using the methods described in Section 3. On the open ocean boundary the 11 year water level data set discussed earlier was used. The model was run for all major events during this 11 year period thus providing the necessary data for statistical analysis of water levels in the Placentia inlets.

5.3.2 Model Setup

The DWOPER model input data was developed for modelling of river flow, based on a setup with a main channel which has tributaries and/or off channel storage. The geographic features of the region are input into the model in the form of cross-sections from within the area. Areas which control the flow, i.e. small flow areas, are most important for input accuracy and are described in greater detail by cross-sections at more frequent intervals.

The model was initially setup for the Placentia region on this basis, with its main channel extending from Placentia Road to Southeast River. In this model, Northeast Arm, Southeast Arm, the widening of Swan Arm towards MacDonald Cove and the low lying area of Placentia, adjacent to The Narrows, were treated as off channel storage. Off channel storage in DWOPER is handled as an area which stores water but has essentially zero flow across it.

The available mapping of the area was studied and potential control sections were identified for accurate measurement during the field program. A total of eleven cross-sections were identified and measured in the field. Other sections were determined from previous soundings taken in the study area by others or from hydrographic charts. A total of 21 cross-sections, spaced at between 100 metres and 750 metres apart were used to define the system for the model. Cross-sections in The Narrows were extended across the low lying area of Town, using field survey data to define the off-channel storage in this area. Figure 5.3.1 shows the model layout and the location of cross-sections. The cross-sections used are given in Appendix III. The layout of input data is described in detail for the calibration run 'PLAC 21', contained in Appendix III.



LAYOUT FOR DWOPER MODEL

(FIRST MODEL)

NOTES 1. MODEL SELECTED FOR USE IN DWOPER
2. ALL CROSS SECTIONS SHOWN ARE PROVIDED IN APPENDIX III

5.3.2 Model Setup (Cont'd)

In order to examine the validity of this setup, a second layout of the DWOPER model input data was developed which defined the main channel as extending from Placentia Road to Northeast River. In this setup, Placentia Narrows through to Southeast River was treated as a tributary. Northeast Arm was represented by five cross-sections derived from hydrographic charts. The water levels computed by this model were insignificantly different from those computed by the first model varying by a maximum of 0.002m. This model, however, proved to be less efficient of computer time than the first model, therefore, the first model was used for all subsequent studies. Figure 5.3.2 shows the layout of the second model and the location of cross-sections.

5.3.3 Modelling Period and Time Step

To develop an annual series of maximum water levels, it was necessary to provide the model with periods of tidal variations of sufficient length to enable the model to stabilize the tidal variations prior to modelling particular events. It was felt that a minimum of one day should be added at the beginning of any period to be modelled, to enable the tidal variations to stabilize. Since freshwater inflow was to be input into the model, and because an examination of freshwater flows indicated that three to four days were required for the flow to peak and recede to near base flow, a duration of five days was selected for modelling purposes.

The computational time step used in the modeling was set at one hour. Calculation of the desirable time step and section length for the DWOPER runs were based on the DWOPER computation by Fread (1974), which found that using a time-step of one-quarter of the time scale of the event being considered yielded errors below one percent (Fread, 1974). In our case using six hours as the time scale (one-half a tidal cycle), a time step of one hour would be appropriate. DWOPER, however, allows for computation at a smaller time step. Trial runs were made using half hour and fifteen minute time steps during an early stage in the data setup for the model. These trial runs showed insignificant changes (less than .01 m) in the computed water levels, confirming that a one hour time step was valid. Therefore, the calibration and verification of DWOPER was carried out using the one hour time step.

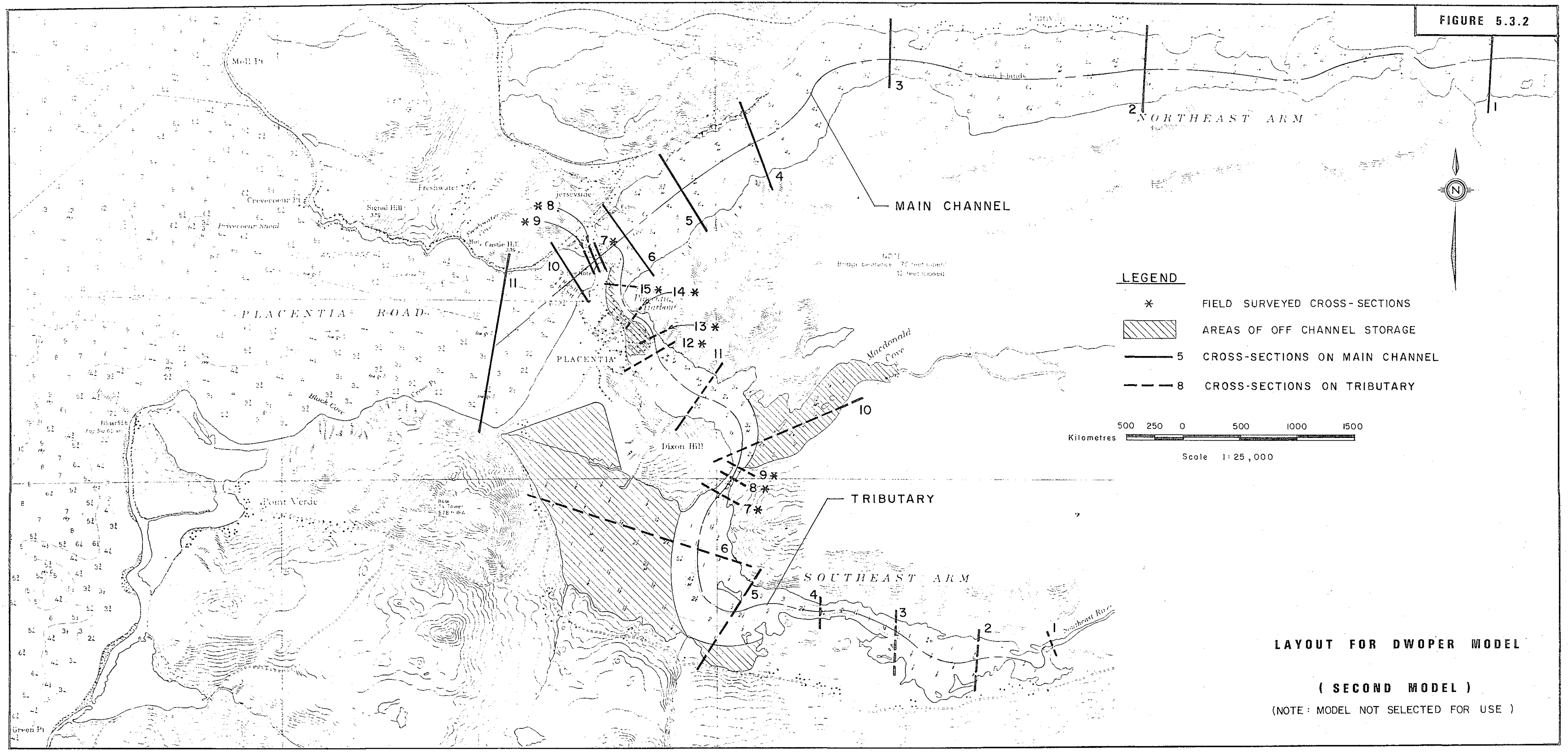


FIGURE 5.3.2
MAIN CHANNEL
TRIBUTARY
NORTHEAST ARM
SOUTHEAST ARM
PLACENTIA ROAD
PLACENTIA
Dixon Hill
Castle Hill
Signal Hill
Point Verde
Macdonald Cove
LEGEND
* FIELD SURVEYED CROSS-SECTIONS
AREAS OF OFF CHANNEL STORAGE
5 CROSS-SECTIONS ON MAIN CHANNEL
8 CROSS-SECTIONS ON TRIBUTARY
Kilometres 500 250 0 500 1000 1500
Scale 1:25,000
LAYOUT FOR DWOPER MODEL
(SECOND MODEL)
(NOTE: MODEL NOT SELECTED FOR USE)

5.3.4 Model Datum

The DWOPER model brings together hydrographic and geographic data and therefore a common datum had to be selected. The datum selected for this study was geodetic zero and all elevation information was related to it. The datum of the model was selected at 1.6 m below geodetic zero. This level was chosen to ensure that all numbers produced were positive.

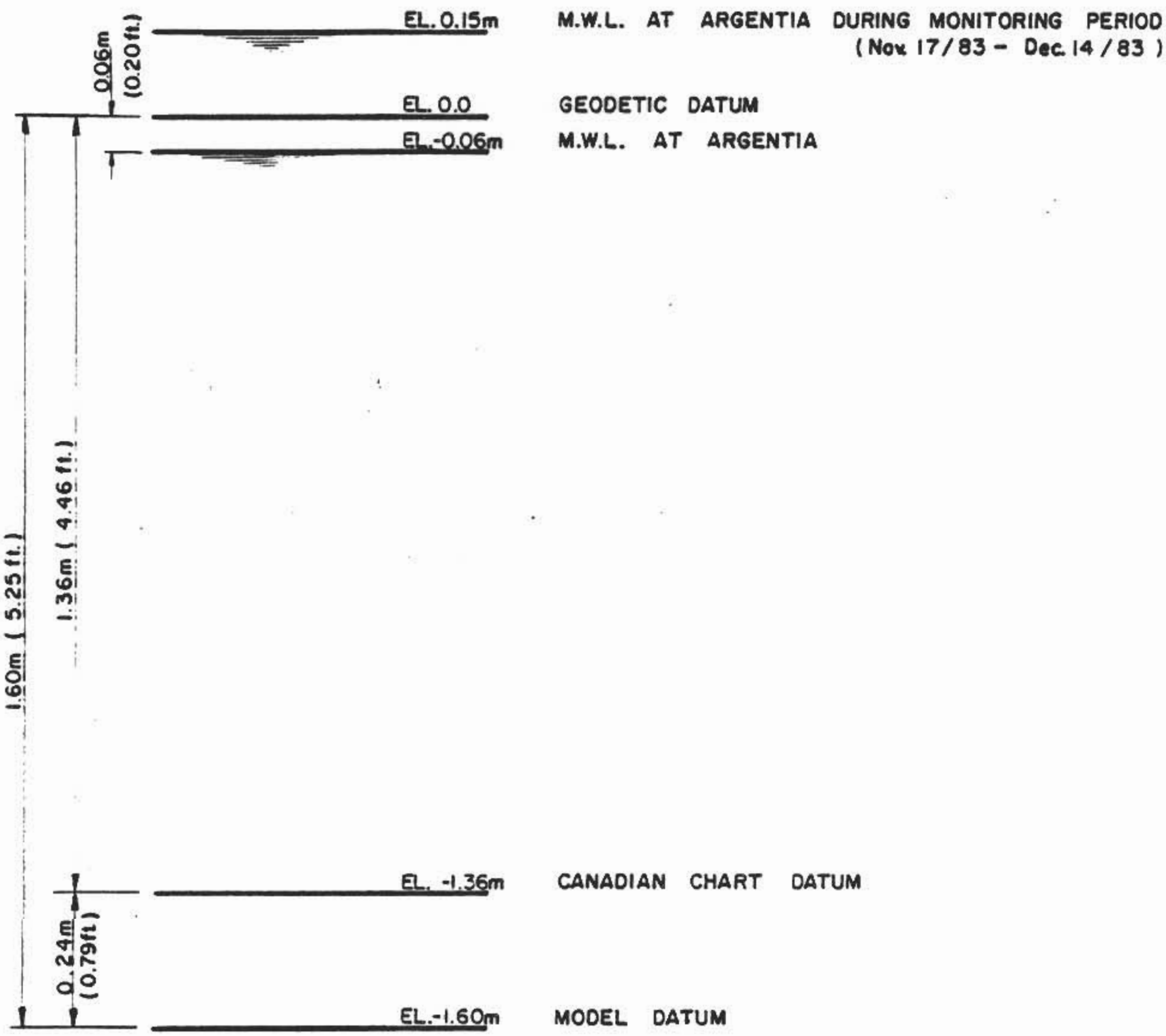
All cross-sections taken were converted to the model datum. To interpret the model output waterlevels as geodetic, 1.6 m must be subtracted from all values. The tide gauge at Argentia has a datum of 1.36 m below geodetic zero. Therefore 0.24 m must be added to all Argentia water levels to convert them to the model datum. The datum of the tide gauge placed in Placentia Road during the field program was found by comparing the water levels recorded with those of Argentia. The measurements were normalized by subtracting the period average from each value, thus the mean matched that at Argentia, this results in a requirement that 1.75 m be added to each normalized value for input to the model. The above relationships are shown in Figure 5.3.3. The tide gauges in Northeast Arm and Swan Arm were not tied into geodetic datum, but application of the model allowed a datum for these gauges to be derived, by adjusting the datum of the recorded data until the RMS errors between the observed and calculated water levels in Swan Arm and Southeast Arm were at a minimum.

5.3.5 Freshwater Inflow

Fifteen drainage basins of greater than 0.5 square kilometres flow into the Placentia study area. Of these, Northeast River and Southeast River account for 75% of the 311 square kilometre total drainage area. A hydrologic model (HYMO) was developed (Part 3) to represent the runoff flowing into the Arms. The rivers do not all flow in at one point, but are spread around the inlets. However, for the purposes of the model they were considered as three inflows; Northeast River (representing all flow into Northeast Arm), Southeast River (representing all flow into Southeast Arm up to MacDonald Gut) and Swan Arm (representing all flow into the system between MacDonald Gut and Placentia Narrows).

The record of flows on Rocky River as well as precipitation at Argentia and flow on Northeast River (post 1979) were examined to determine the freshwater runoff during the periods used in producing the series of annual maximum water levels (See Appendix III). In most cases the runoff was low, and precipitation at Argentia was low, being less than 13 mm. In these cases, a base flow computed from Rocky River or Northeast River was applied (Part 3). One case, October, 1975 showed high precipitation which could have produced a peak flow at the same time as a high tide in the Arms, for this case fresh water inflows were generated by HYMO for input to the hydraulic model - See Section 3.5.

FIGURE 5.3.3



DATUM RELATIONSHIPS

5.3.6 Model Accuracy and Sensitivity

Since the Placentia Townsite is low and flat, it was necessary for the model to produce water levels which were as accurate as possible. The aim of the calibration of the hydraulic model was to obtain results within 0.1 m of actual. This degree of accuracy was most important at high water levels where a difference of 0.3 m could have resulted in a considerable difference in the areal extent of flooding. A full discussion of the sensitivity testing is given in Section 6, however, the results are briefly discussed in the following paragraphs.

Analysis of the sensitivity of the model to changes in the parameters used in the calibration gave an indication of the precision required for input data to the model. The model was found to be most sensitive to the flow area in Placentia Narrows. Since The Narrows are changeable, being subject to the effects of dredging and sedimentation, the results of this study are true only for the channel sections measured during the field program (November, 1983). For example, when the model was run using the cross-sections measured during the field program and then the cross-sections taken prior to the June, 1982 dredging program, the model indicated a 0.07 m drop in the maximum water level computed for Southeast Arm with the pre-June, 1982 sections.

The magnitude of fresh water inflow for most of the annual maxima study was small since these events did not coincide with major river floods. However, this was not the case in October, 1975 when high inflow occurred during the period of the high tidal water levels. The effect of fresh water inflows to this event was tested by running the hydraulic model with and without fresh water inflows. This showed that the effect of high inflows was small and for the October, 1975 event only added 0.20 m to the maximum water level in Southeast Arm.

The model was found to be insensitive to wind and to inaccuracy in the large flow area sections, such as Northeast Arm and Placentia Road. The model was found to be especially sensitive to Manning's "n" at the control section but not sensitive to Manning's "n" at the other sections. The Manning's "n" parameter was the main variable used in calibration of the model.

5.3.7 Model Calibration and Verification

In order to use any mathematical model, it is necessary to calibrate it and to verify the calibration by using known data. Since there were no water level records for the study area, tide gauges were installed during the field program to provide a record of water levels over a complete lunar cycle (1 month).This tide gauge record is included in the Appendices of the Field Program Report. The records from these gauges gave the relationships between water level variations and phase lags in Placentia Road, Northeast Arm, Swan Arm and Southeast Arm and were used for calibration and verification of the model. *

The calibration period selected from the tide gauge records was the period December 1 - 6, 1983 (hour 360 to hour 480 of the tide gauge record). This period was selected because it contained the tidal cycle with the greatest range between high tide and low tide in the period of record. It also contained the highest tide in the period of record.

After initial setup of the model, the records from the tide gauges were input into the model. The records for Placentia Road were input as the tidal boundary and as such, provided the driving force for the model. The records for Northeast Arm and Swan Arm were input as observed values.

Several trial runs were made with the model, using the tide gauge data for the selected calibration period. Initial estimates for Manning's "n" were selected by the procedures suggested by Ven T. Chow (Chow, 1959). These initial values were then adjusted until observed and calculated water levels were in close agreement. The final calibration run is labeled PLAC 21 and a copy of the computer printout of this run is included in Appendix III. Table 5.3.1 shows the analysis that was done on the results of the calibration run for Swan Arm. The values in the computed and observed columns were taken from the computer printout as the values for the peaks and the troughs of the water level curve (expressed in metres). The average difference on the peak water level was found to be -0.02m. This was deemed to be acceptable considering that the desired level of accuracy was ± 0.10 m.

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* Data collected from these gauges were also used to establish a correlation between the tidal characteristics at Argentia and Placentia Road.

TABLE 5.3.1

ANALYSIS OF DWOPER CALIBRATION - PLAC 21

Calibration Period: Dec. 1/83 to Dec. 6/83

Calibration is based on computed and observed water levels in Swan Arm (See PLAC 21 in Appendix III).

WATER LEVELS* (metres)

COMPUTED	OBSERVED	DIFFERENCE
0.68	0.85	-0.17**
-0.20	-0.20	0.00**
0.70	0.71	-0.02
-0.17	-0.17	0.00
0.86	0.89	-0.03
-0.11	-0.17	0.06
0.69	0.66	0.02
-0.14	-0.23	0.09
0.91	0.97	-0.07
-0.11	-0.09	-0.02
0.78	0.80	-0.02
0.02	0.04	-0.02
1.07	1.08	-0.02
-0.05	-0.08	0.03
0.67	0.68	-0.02
-0.14	-0.16	0.02
0.90	0.95	-0.05
-0.02	-0.01	-0.02

Average difference - peaks = -0.02m
- troughs = 0.02m

* Based on Geodetic Datum

** Not included in averaging due to initial instability of model.

TABLE 5.3.2

ANALYSIS OF DWOPER VERIFICATIONS - PLAC 20

Verification Period: Nov. 23/83 to Nov. 28/83

Verification is based on computed and observed water levels in Swan Arm (See PLAC 20 in Appendix III).

WATER LEVELS* (metres)

COMPUTED	OBSERVED	DIFFERENCE
0.89	0.92	-0.03**
-0.10	-0.16	0.06**
0.52	0.53	-0.01
-0.11	-0.24	0.12
0.64	0.69	-0.05
-0.23	-0.30	0.07
0.38	0.45	-0.07
-0.18	-0.22	0.04
0.66	0.74	-0.08
-0.15	-0.20	0.05
0.52	0.63	-0.11
-0.18	-0.14	-0.05
0.79	0.99	-0.20
0.15	0.21	-0.06
1.02	1.11	-0.10
0.15	0.20	-0.05
0.82	0.88	-0.06
-0.14	-0.19	0.05

Average difference - peaks = -0.09m
- troughs = +0.02m

* Based on Geodetic Datum

** Not included in averaging due to initial instability of model.

TABLE 5.3.3

ANALYSIS OF DWOPER VERIFICATION - PLAC 22

Verification Period: Dec. 6/83 to Dec. 11/83

Verification is based on computed and observed water levels in Swan Arm (See PLAC 22 in Appendix III).

WATER LEVELS* (metres)

COMPUTED	OBSERVED	DIFFERENCE
0.93	0.96	-0.04**
0.03	-0.06	0.09**
0.50	0.49	0.01
-0.28	-0.31	0.02
0.55	0.63	-0.08
-0.14	-0.11	-0.03
0.56	0.68	-0.12
-0.06	-0.01	-0.05
0.77	0.84	-0.07
-0.05	0.04	-0.09
0.51	0.57	-0.06
-0.05	0.03	-0.08
0.75	0.74	0.01
-0.10	-0.14	0.04
0.36	0.34	0.02
-0.35	-0.30	-0.05
0.26	0.36	-0.10
0.05	0.13	-0.08

Average difference - peaks = -0.05m
- troughs = -0.04m

* Based on Geodetic Datum

** Not included in averaging due to initial instability of model.

5.3.7 Model Calibration and Verification (Cont'd)

With the model calibrated for one period of recorded data, it was verified on two other periods. The first verification period was November 23 - 28, 1983, (hour 200 to hour 320 of the tide gauge record) and was selected because it contained the tidal cycle with the lowest range and lowest tide in the period of record. This first verification run was labeled PLAC 20. The second verification period was between December 6 - 11, 1983 (hour 480 to hour 600 of the tide gauge record) and was selected to provide another verification based on recorded data without extremes of high or low tides. The second verification run was labeled PLAC 22. Copies of these runs are included in Appendix III. Tables 5.3.2 and 5.3.3 show the analysis done on the results of each. The average difference on the peak water level was found to be -0.09 m and -0.05 m for PLAC 20 and PLAC 22 respectively. Again, this was deemed to be acceptable considering that ± 0.10 m was the desired level of accuracy.

Examination of the overall calibration and verification results showed that DWOPER underestimated the peak water levels by an average of 0.05m. On the recommendation of the Technical Committee, this value has been added to all peak water levels output by DWOPER for use in the extreme value analysis. The Placentia Road water levels are unaffected by this change since they are the driving force, and not calculated by the DWOPER model.

5.4 MODEL RESULTS

With the DWOPER model calibrated and verified to within an accuracy of 0.10 metres on recorded tide data, the model was run for the January 10, 1982, the December 22, 1983 and the December 25, 1983 flood events. Figure 5.4.1 shows the water level variations for the three events, over the tidal cycle of each event with the highest water level. A copy of the computer printout for the December 22, 1983 flood event (PRO 25) is included in Appendix III.

Noted on Figure 5.4.1 are the various water levels at which flooding started in each of the Arms, together with the time sequence in which the flooding started. For example, for the December 22, 1983 flood event, flooding first occured from Swan Arm when its water level was about 1.0 m geodetic, followed approximately a half hour later by flooding from Northeast Arm when its water level was about 1.6 m geodetic. The low lying land adjacent to Southeast Arm then started to flood when Southeast Arm's water level was about 0.9 m geodetic, approximately a half hour after the flooding started in Swan Arm. Figure 5.4.1 also shows the approximate duration of flooding in each area.

The maximum computed water level for each of the Arms for each flood event was compared with the water level which was reported to have been experienced during each flood. The reported water level was based on eye witness reports collected from local residents and officials, although these reports were of variable quality and could not all be considered accurate, an effort was made to determine an estimate of the actual levels during the flooding events. The following table summarizes the water levels for this comparison:

TABLE 5.4.1

	MAXIMUM WATER LEVELS (metres above geodetic)					
	N.E. ARM		SWAN ARM		S.E. ARM	
FLOOD EVENT	Comp.	Repor.	Comp.	Repor.	Comp.	Repor.
Jan. 10/82	1.73	1.70	1.57	1.70	1.53	1.30
Dec. 22/83	1.80	1.90	1.58	1.90	1.49	1.50
Dec. 25/83	1.65	1.70	1.46	1.70	1.39	1.30

Where:

Comp. = computed water levels

Repor. = reported water levels.

The computed and reported water levels have relative differences which require explanation.

-- In comparing the computed and reported water levels in Northeast Arm, it is noted that the differences are within 0.10 m, which is within the model accuracy.

-- It is noted that the difference in the computed and reported water levels in Swan Arm is 0.13 m, 0.32 m and 0.24 m for the January 10, December 22 and December 25 flood events, respectively. This comparison between computed and reported water levels in Swan Arm, indicated that local hydraulic effects were important.

5.4 MODEL RESULTS (Cont'd)

-- This possibility may be explained by the fact that once Northeast Arm started to flood the Town from the northeast, water would have flowed southward across the Town, discharging into Swan Arm. This would have caused a temporary high water level in the southeast part of the Town similar to that in Northeast Arm. At the same time as the water level in the southeastern part of the Town was rising due to this effect, the water level in Northeast Arm was dropping as the tide ebbed, causing Swan Arm to flow into Northeast Arm. Therefore, the situation could exist wherein a higher water level was experienced in the southeastern part of the Town, adjacent to Swan Arm, than in Swan Arm itself. Albeit this would be for a very short period of time. Although there are no direct eyewitness reports of this phenomena, the period of inundation of the town during the December 1983 flood events corresponds to the time immediately after the high tide.

-- In comparing the computed and reported water levels in Southeast Arm, it is noted that the levels for the two December floods were within the level of accuracy of the model. The difference in the water levels for the January, 1982 flood could be partially attributable to the effect of dredging in The Narrows which was completed after the January, 1982 flood. The computed water level was based on cross-sections taken in The Narrows during the field program, which was after the dredging program in 1982. Computed water levels based on post-dredging cross-sections would be higher than water levels based on pre-dredging cross-sections because of the reduced throttling effect of The Narrows as a control.

5.5 ANNUAL MAXIMUM WATER LEVELS

The investigations undertaken as part of the Oceanographic studies (Part 4) and the Hydrologic studies (Part 3) indicated that the high water levels in the study area were mainly due to high tides. Therefore, to develop an annual series of maximum water levels, the period of record for the Argentia tide-gauge, from 1972 to 1983 was examined and the three highest water level events were selected for each year. In some cases, two or three of the events occured within a day of each other so the simulation period was selected to include all the high water level events. The second and third highest events were included in the analysis in case the DWOPER model showed that the influence of other factors, besides the high tide, produced higher levels in the Arms from these events. For each event two days of recorded water levels before and after the event were used in the model to provide periods of at least five days duration to allow the tidal variations in the model to stabilize and therefore to allow the model to respond properly to water level changes.

The DWOPER model was then run to hindcast a series of water levels at eight points throughout the hydraulic system from Placentia Road to Southeast Arm for the highest water level event(s) each year. The resultant series of water levels is summarized in Table 5.5.1. These water levels include all the factors that affect the water level in Placentia Road, ie. wind, waves, storm surge, setup, etc., since the actual water level record used as the open ocean boundary for DWOPER. However, the local wind and wave effects in each of the Arms, are not included.

The wind effect in the Arms will result in wind setup and waves with associated surge. These factors will result in slightly higher water levels than those given in Table 5.5.1, if an on-shore wind occurs at the same time as the high water level event. The significance of wind effects is discussed in Section 6.3.

Figure 5.5.1 is a graphical presentation of the data in Table 5.5.1. Also included on Figure 5.5.1 are the water levels corresponding to the 1 in 20 and 1 in 100 year recurring floods, as calculated in Section 6.

It can be determined from Table 5.5.1 that the average drop in maximum water levels throughout the system, as a result of attenuation through the controls are as follows:

-- from Placentia Road to Northeast Arm = 0.13 m
-- from Northeast Arm to Swan Arm = 0.24 m
-- from Swan Arm to Southeast Arm = 0.08 m
-- from Placentia Road to Southeast Arm = 0.46 m

TABLE 5.5.1

ANNUAL SERIES OF MAXIMUM WATER LEVELS

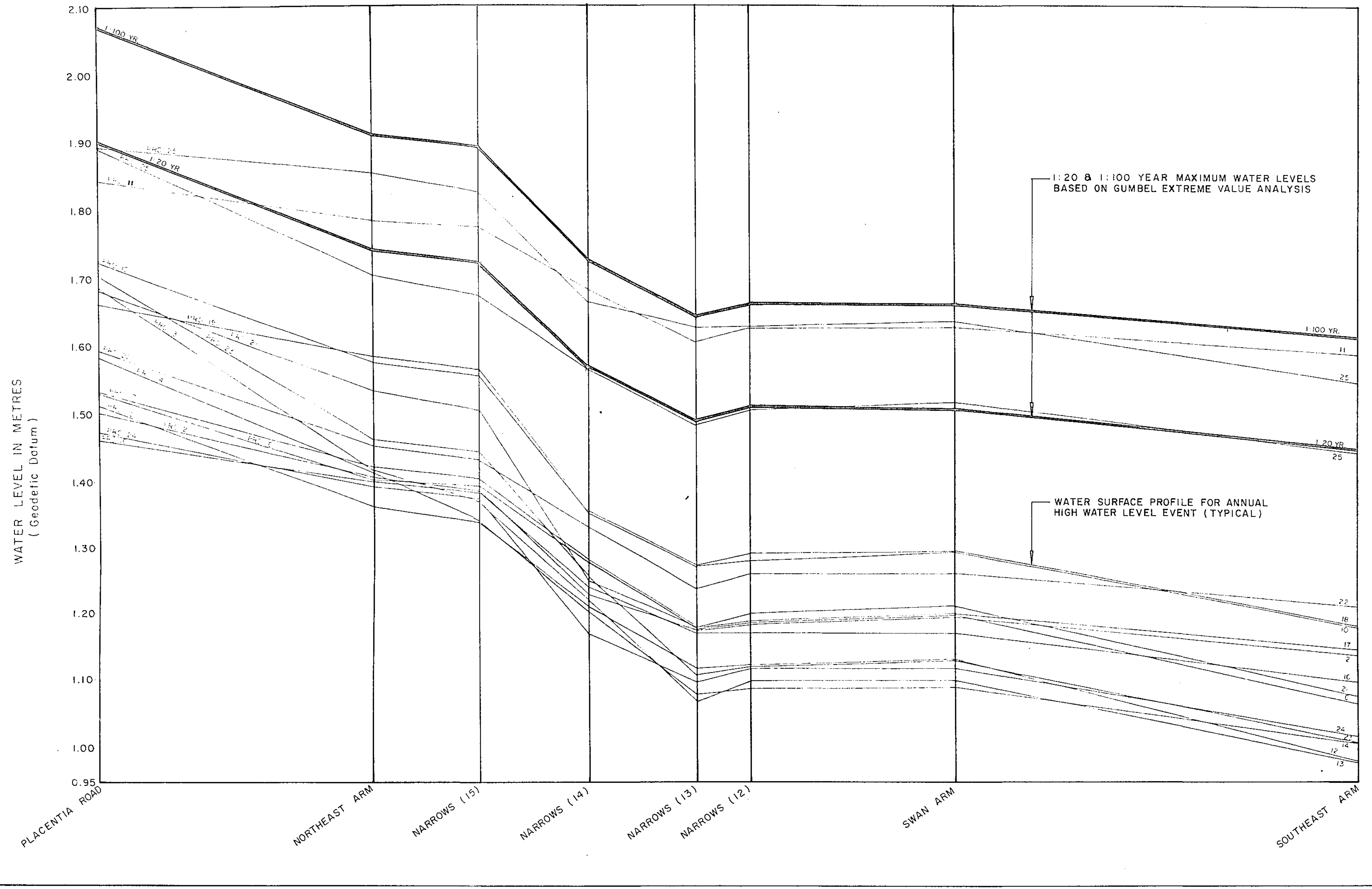
Computer Run	Start Date	Time of Peak	S.E. Arm Section 6	Swan Arm Section 11	Narrows				N.E. Arm Section 16	Placentia Rd Section 21
					Section 12	Section 13	Section 14	Section 15		
PRO21	25-02-72	0900-27	1.08	1.21	1.20	1.18	1.25	1.50	1.53	1.63
PRO17	18-01-73	2200-21	1.15	1.20	1.19	1.18	1.28	1.40	1.42	1.48
PRO10	08-01-74	1100-10	1.18	1.29	1.28	1.27	1.35	1.55	1.57	1.67
PRO23	28-01-75	1200-30	1.01	1.13	1.12	1.11	1.25	1.44	1.46	1.65
PRO6	05-10-75	1200-09	1.17	1.20	1.19	1.18	1.28	1.39	1.40	1.48
PRO14	15-03-76	2300-17	1.01	1.09	1.09	1.08	1.20	1.34	1.41	1.53
PRO16	16-12-76	0600-18	1.10	1.17	1.17	1.17	1.24	1.38	1.40	1.41
PRO18	18-01-77	1100-20	1.18	1.29	1.29	1.27	1.35	1.56	1.58	1.61
PRO24	30-11-78	1100-01	1.02	1.12	1.12	1.10	1.17	1.37	1.39	1.42
PRO22	27-01-79	1100-30	1.21	1.26	1.26	1.24	1.33	1.43	1.45	1.54
PRO2	03-01-80	1200-05	1.14	1.20	1.19	1.18	1.23	1.38	1.40	1.45
PRO13	15-02-80	1000-17	0.98	1.10	1.10	1.07	1.22	1.37	1.41	1.63
PRO12	08-12-81	0800-10	1.08	1.13	1.12	1.12	1.21	1.34	1.36	1.46
PRO11	08-01-82	1100-10	1.58	1.62	1.62	1.60	1.68	1.77	1.78	1.79
PRO25	20-12-83	1200-22	1.54	1.63	1.62	1.62	1.66	1.82	1.84	1.84
	20-12-83	1300-25	1.44	1.51	1.50	1.48	1.56	1.67	1.70	1.84

All waterlevels in metres above geodetic datum.

NOTES: (1) Date is start of water level series for each high water level event.
(2) Time of peak eg. 0900-27 means peak water level occured on Southeast Arm at 0900 hours on 27th.

Only single event picked for each year if that event was highest on both Southeast and Northeast Arms (except PRO25).

FIGURE 5.5.1



ILLUSTRATED WATER PROFILES FOR ANNUAL MAXIMUM WATER LEVEL EVENTS

PLACENTIA ROAD – SOUTHEAST ARM

PART SIX

FLOOD RISK CONTOURS

6.1 GENERAL

The annual series of maximum water levels was used in a frequency analysis to determine the 1 in 20 and 1 in 100 year recurring water levels for each region of the study area. The sensitivity of these water levels to variations in the input parameters of the hydraulic model was reviewed and assessed in establishing the 1 in 20 and 1 in 100 year flood risk contours.

6.2 FREQUENCY ANALYSIS FOR 1:20 AND 1:100 YEAR WATER LEVELS

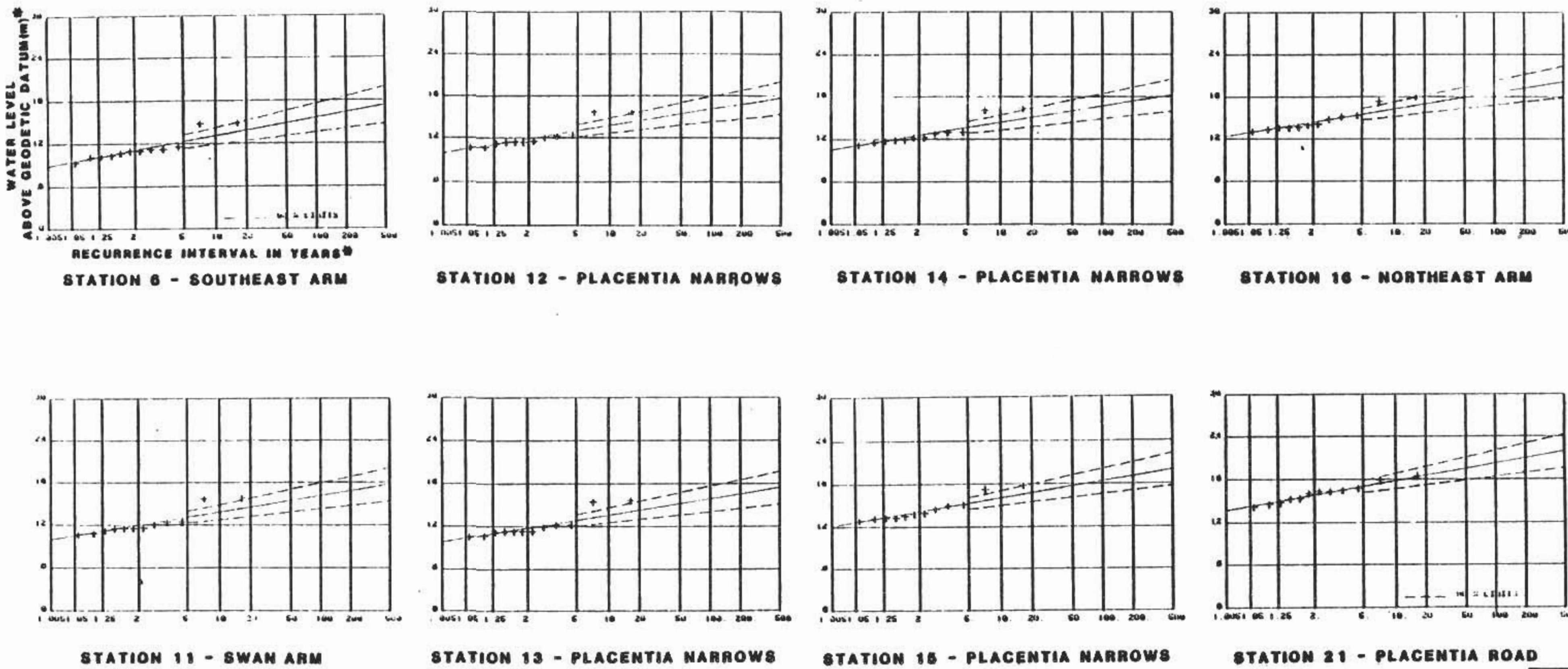
Four probability distributions were considered for this frequency analysis:

i) Extreme Value (Gumbel I),
ii) Lognormal,
iii) Three-Parameter Lognormal, and
iv) Log Pearson Type 3.

All of these distributions were used to evaluate the long term distributions. The methodology and computer programs were those developed by Condie et al (1981). As described in that report, in flood frequency analysis it is not yet possible to state categorically which distribution must be used, although it is suggested that the distribution should be fitted to the data using maximum likelihood theory. The programs give the flood frequency regime as obtained using each of the four distributions, employing the maximum likelihood method of fitting.

All of the distributions were plotted and were used as a guide to show how well the distribution has been fitted to the data sets.

Evaluation of the results has indicated the Gumbel distribution gives the best overall fit throughout the region. Therefore using the results of maximum water levels, generated by the DWOPER model and presented in Section 5.5, the Gumbel Extreme Value technique (Gumbel, 1954) was used and water elevation at the eight reference stations for various return periods are given in Table 6.2.1. Figure 6.2.1 shows the water level data from the eight stations plotted on extremal probability paper. It is evident that in all the plots there are two data points that are anomalously high. These values refer to the water levels resulting from the floods in January of 1982 and December of 1983. These two events are observed data points, measured at the time of floods at Placentia. Similar high water levels were measured at St. John's at the same time. These data points represent very extreme events compared to the rest of the data population but must be included in the analysis.



---- 90% CONFIDENCE LIMITS

*ALL GRAPHS

Note: Add 0.05 m to all water levels determined from above graphs for all stations, except Station 21, for any particular recurrence interval.

GUMBEL EXTREME ANALYSIS OF DWOPER MODEL RESULTS AT EIGHT STATIONS

FIGURE 6.2.1

TABLE 6.2.1

GUMBEL EXTREME VALUE ANALYSIS OF DWOPER MODEL RESULTS

(Water Levels are above Geodetic Datum)

Location	Return Period (yrs)	Water Level (m)	90% Lower Limit (m)	90% Upper Limit (m)
Southeast Arm	5	1.28	1.19	1.37
	10	1.36	1.24	1.48
	20	1.44	1.29	1.58
	50	1.54	1.36	1.72
	100	1.61	1.41	1.82
Swan Arm	5	1.36	1.27	1.44
	10	1.43	1.32	1.54
	20	1.50	1.37	1.63
	50	1.59	1.43	1.76
	100	1.66	1.47	1.85
Station 12	5	1.35	1.26	1.44
	10	1.42	1.32	1.53
	20	1.50	1.36	1.63
	50	1.59	1.42	1.75
	100	1.66	1.47	1.84
Station 13	5	1.34	1.25	1.42
	10	1.41	1.30	1.52
	20	1.48	1.35	1.61
	50	1.57	1.41	1.73
	100	1.64	1.45	1.83
Station 14	5	1.42	1.33	1.50
	10	1.49	1.38	1.60
	20	1.56	1.43	1.69
	50	1.65	1.48	1.81
	100	1.72	1.53	1.91
Station 15	5	1.58	1.49	1.66
	10	1.65	1.54	1.76
	20	1.72	1.59	1.86
	50	1.82	1.65	1.98
	100	1.89	1.69	2.08
Northeast Arm	5	1.60	1.52	1.69
	10	1.67	1.57	1.78
	20	1.74	1.61	1.88
	50	1.84	1.67	2.00
	100	1.91	1.72	2.10
Placentia Road	5	1.70	1.61	1.79
	10	1.78	1.66	1.89
	20	1.85	1.71	1.99
	50	1.95	1.77	2.12
	100	2.02	1.82	2.22

6.3 SENSITIVITY ANALYSIS OF WATER LEVELS

The hydraulic modelling described in Section 5 has resulted in the ability to numerically simulate the physical reaction of the Placentia system (i.e. the change in water levels) to changes in the input parameters of the model. Implicit in the structure of the numerical model are generalizations of the geometry of the channels, and assumptions as to the behavior of the water within the system. These generalizations and assumptions were minimized to remain consistent with the level of accuracy required. In order to confirm that variations in certain parameters do not inordinately affect the model results, a sensitivity analysis was carried out on the following parameters:

i) Friction Coefficient - Manning's "n",
ii) Flow Area,
iii) Freshwater Inflow,
iv) Wind, and
v) Off-Channel Storage.

In addition to the above, the effect of ice in the study area on the water levels was reviewed.

All sensitivity analyses have been carried out using the input data for the December 1983 flooding events (PRO25). The testing of sensitivity has been carried out by varying only one parameter at a time while all other parameters were kept at the values used in the calibrated model.

6.3.1 Friction Coefficient - Manning's "n"

During the initial setup of the DWOPER model, a Manning's "n" of 0.03 was assumed throughout the hydraulic system. (See Section 5.3.7). The values of Manning's "n" which were finally selected as giving the best calibration of the model were as follows:

Placentia Road	-	0.03
Northeast Arm	-	0.03
Swan Arm	-	0.03
Southeast Arm	-	0.03
Placentia Gut	-	0.13*
The Narrows	-	0.04
MacDonald Gut	-	0.06

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* This "n" value is an equivalent "n" value incorporating contraction and expansion losses for flow through the Placentia Bridge. DWOPER would not handle these losses in a more rigorous manner with occurrence of two way flow.

6.3.1 Friction Coefficient - Manning's "n" (Cont'd)

The value of Manning's "n" in the wide sections of the hydraulic system, namely the three Arms and Placentia Road remained unchanged from the originally assumed value of 0.03. The flow area in these sections is so large that the effect of changes in boundary conditions is insignificant.

It can be expected that variations in Manning's "n" in sections which exert control on the flow in the system would effect water levels in the system. To test this, the flood event of December 22, 1983 (PRO 25) was used and Manning's "n" was varied at each of the controls. For each value of Manning's "n", corresponding water levels throughout the system were determined by the model. In each case, the value of Manning's "n" in other areas was unchanged from the values used in the calibration of the model and given in the list above. The following table summarizes the variations in Manning's "n" tested and the resultant water levels throughout the system.

TABLE 6.3.1 Sensitivity to Manning's "n"

		WATER LEVELS (metres above geodetic)			
Control	n	Placentia Road	Northeast Arm	Swan Arm	Southeast Arm
Placentia Gut	0.12	1.84	1.81	1.59	1.51
	0.13	1.84*	1.80*	1.58*	1.49*
	0.14	1.84	1.78	1.56	1.47
The Narrows	0.03	1.84	1.78	1.65	1.57
	0.04	1.84*	1.80*	1.58*	1.49*
	0.05	1.84	1.81	1.49	1.42
MacDonald Gut	0.05	1.84	1.79	1.57	1.51
	0.06	1.84*	1.80*	1.58*	1.49*
	0.07	1.84	1.80	1.58	1.47

* Original water levels for the December 22, 1983 flood event (PRO 25) based on the values of Manning's "n" used in the calibration of the model.

From the above table, it is noted that, generally, as the Mannings "n" is decreased at a control an increase in water level occurs downstream of the control and a decrease in water level occurs upstream of the control. This is true in all cases except for Placentia Gut where Placentia Road is upstream, as expected, Placentia Road water level does not change since this water level is the open ocean boundary of the hydraulic model. Similarly, when Manning's "n" is increased the water level downstream of the control is reduced and an increase in water level occurs upstream of the control. In the case of MacDonald Gut, for an increase in Manning's "n" from 0.06 to 0.07, the water levels in Northeast Arm and Swan Arm did not change. This can be attributed to the inherent accuracy of the hydraulic model. The region most sensitive to the variation in Mannings "n" is The Narrows where changes in water level of up to 0.09 m (approximately 6 percent change in water level) were simulated.

6.3.2 Flow Area

The flow of water through the study area is controlled by the natural and man made constrictions throughout the area. The Gut, The Narrows and MacDonald Gut, in particular, are controls in the hydraulic system because of the relatively small flow area at each. The open water bodies, such as Placentia Road, Northeast Arm, Swan Arm and Southeast Arm, with their relatively large flow areas, do not significantly affect or constrict the flow of water.

Furthermore, erosion and sedimentation in the area will affect the flow areas throughout the system. The Woodward-Clyde Report, 1982 discusses this problem in detail. From this report it is apparent that there is a significant amount of sedimentation occurring in Placentia Road, The Gut and The Narrows but this process is less severe in Northeast Arm, Swan Arm, MacDonald Gut and Southeast Arm. This report also describes the erosion that is occurring at the bridge across The Gut.

Based on results obtained during the calibration of the DWOPER model, it was apparent that the flow area in The Gut and The Narrows had a significant effect on the flow of water through the system with a consequential effect on the water levels throughout the system. As previously described (Section 5.2), the smaller the flow area, the more pronounced the throttling effect on flow and therefore the greater the attenuation of water levels in the Arms.

6.3.2 Flow Area (Cont'd)

The bridge across The Gut was constructed in 1961. The infilling for the abutments on each side and construction of the supporting piers in the channel have reduced the flow area at this control such that there is about a 10 percent attenuation of the tidal range in Northeast Arm compared to Placentia Road. Figure 6.3.1 shows the detail of the bridge and the available flow area. The bottom profiles are included on this figure to illustrate how much the flow area under the bridge has changed since its construction. The flow areas used in the DWOPER model at the Gut were based on cross-sections taken during the 1983 field survey and the November 1983 survey by Professional Divers.

To illustrate the effect of the bridge on the high water levels throughout the system, the model was run for the December 22, 1983 flood event (PRO 25) with the end spans closed off. This effectively reduced the flow area by about 50 percent. The following table summarizes the effect on the maximum water levels throughout the system for that flood event.

	MAXIMUM WATER LEVELS (metres above geodetic)			
Condition	Placentia Road	Northeast Arm	Swan Arm	Southeast Arm
end spans open	1.84	1.80	1.58	1.49
end spans closed	1.84	1.64	1.46	1.38

The effect of closing the end spans of the bridge was greatest on the water level of Northeast Arm (0.16 m drop) and was least on Southeast Arm (0.11 m drop).

Other factors that affect flow area are dredging and infilling. During the 1970's, the west side of The Narrows, adjacent to the Town of Placentia was infilled, in some places as much as 50 metres from the original shoreline. This infilling decreased the flow area. In 1982, a large portion of The Narrows was dredged, the effect of which was to increase the flow area. To illustrate the effect that increasing or decreasing the flow area of The Narrows has on the maximum water levels throughout the system, the pre-1982 dredging and the post-1982 dredging cross-sections of The Narrows were input into the DWOPER model. On the average, dredging increased the flow area approximately 20 percent at mean sea level. The following table summarizes the effect of changing the flow area in The Narrows on the December 22, 1983 flood event (PRO 25).

6.3.2 Flow Area (Cont'd)

	MAXIMUM WATER LEVELS (metres above geodetic)			
Condition	Placentia Road	Northeast Arm	Swan Arm	Southeast Arm
pre-dredging	1.84	1.80	1.50	1.42
post-dredging	1.84	1.80	1.58	1.49

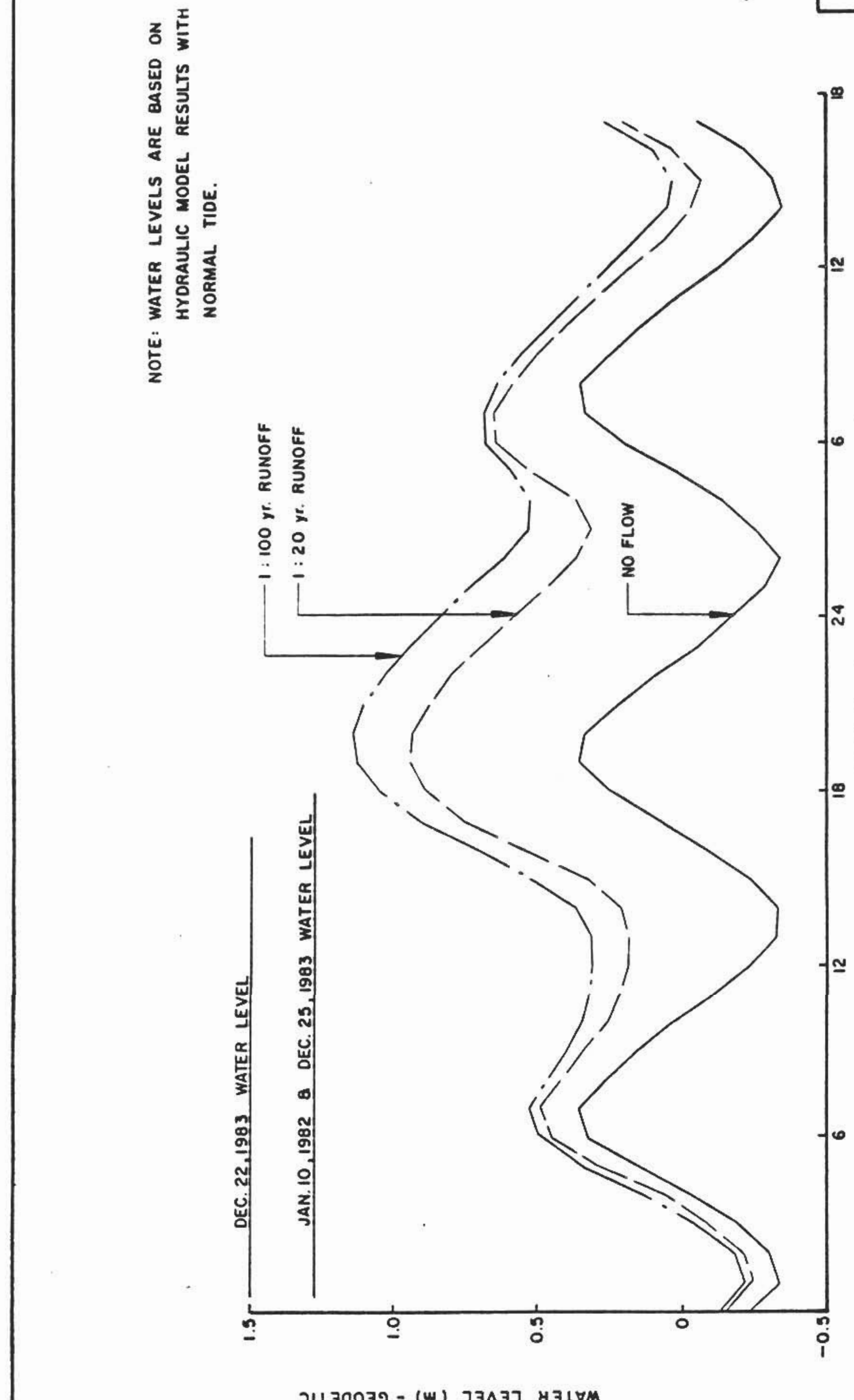
From the above it can be seen that changing the flow area of The Narrows by 20 percent, the maximum water levels in Swan Arm and Southeast Arm changed by 0.08 m and 0.07 m, respectively.

6.3.3 Freshwater Inflow

The previous tests of sensitivity in this section have tested changes to physical parameters within the Placentia study area, however, testing of the freshwater flow impact involves the testing of sensitivity of the system to a possible change to the boundary conditions. If the freshwater inflow for December 22, 1983 is changed to represent a 1 in 20 year return period inflow, for example, the model will no longer represent the conditions of December 1983, since such an event did not occur. The testing of the sensitivity of the study area to freshwater flow has therefore been carried out using freshwater inflow as the upstream boundary and a 'normal' tide as the downstream boundary. The normal tide was taken as a sinusoidal wave with a period of 12.4 hours and an amplitude of 69.6 cm, which was found to be the dominant tidal component under the oceanographic investigations (Part 4); this was assumed to cycle around the mean water level at Argentia. The effects of the freshwater inflow are shown in Figures 6.3.2 and 6.3.3 respectively. It can be seen from this figure that freshwater inflow has a greater effect on Southeast Arm than on Northeast Arm with the 1 in 20 year and 1 in 100 year inflows. It can be seen that these levels are lower than the levels experienced in previous flooding.

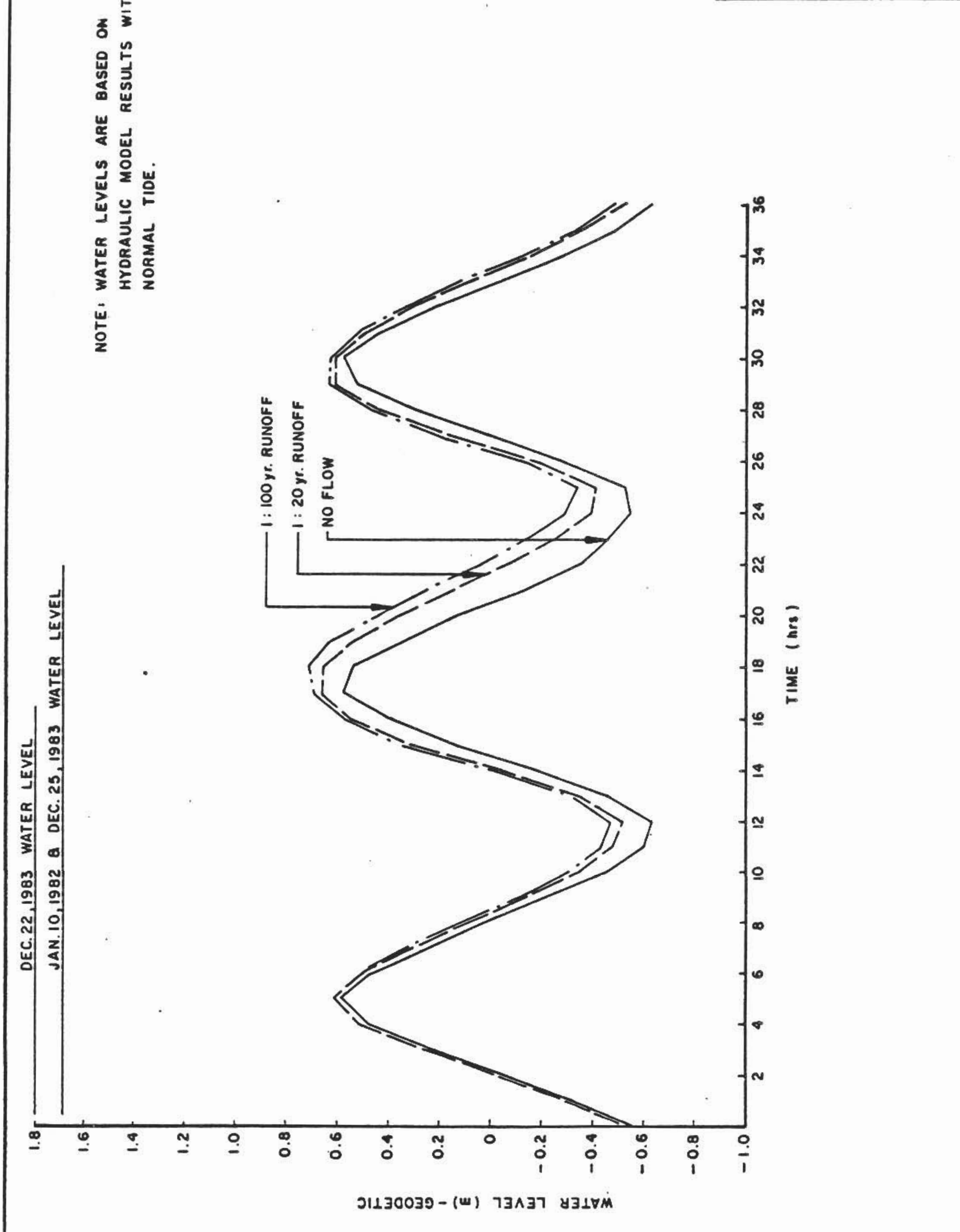
The tide rises over a period of 6 hours, the volume of freshwater flow over a six hour period coinciding with the rising tide has therefore been used in comparing the water levels output by the DWOPER model to the water level rise. A further comparison was made using the PRO 6 DWOPER run, in which the freshwater flow required use of the hydrologic model. The DWOPER model was rerun for the PRO 6

FIGURE 6.3.2



IMPACT OF FRESHWATER INFLOWS ON SOUTHEAST ARM WATER LEVELS

FIGURE 6.3.3



IMPACT OF FRESHWATER INFLOWS ON NORTHEAST ARM WATER LEVELS

6.3.3 Freshwater Inflow (Cont'd)

input data with the boundary freshwater inflow set to zero to obtain an estimate of the component of water level rise directly attributable to freshwater inflow. These levels are plotted in Figures 6.3.4 and 6.3.5 for Southeast Arm and Northeast Arm, respectively, and show the level of impact of a freshwater inflow. It can be seen from the figures that the DWOPER model is not very sensitive to freshwater inflow to Northeast Arm, but is more sensitive to the inflow to Southeast Arm.

From Figures 6.3.4 and 6.3.5 estimates of the sensitivity of the model to freshwater inflow on Southeast Arm and Northeast Arm can be made. The sensitivity will increase with the size of the freshwater storm. For a base flow condition, such as the December 22, 1983 condition, variation of $\pm$ 20% in the base flow estimate will result in a $\pm$ .002 m change in water level on Southeast Arm and a average of less than .001 m on Northeast Arm. As the freshwater runoff component increases, the magnitude of the effect of a 20% volume change will increase. Figures 6.3.4 and 6.3.5 show the envelopes of the water level changes for a 20% change in the flood volume.

6.3.4 Wind Effects in the Arms

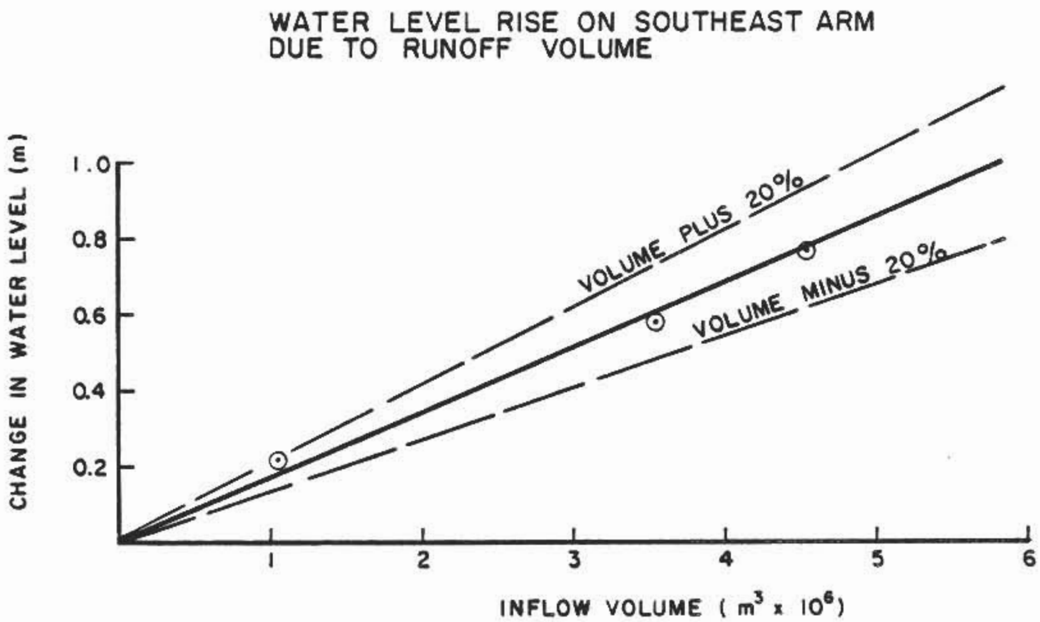
Winds blowing across the surface of a water body produce setup (or "wind tide") and waves on the windward shore. The magnitude of setup varies as a function of wind speed, fetch distance and as the inverse of depth. These effects are significant contributors to high water conditions in Placentia Bay, as discussed in Section 4, but are of much less importance in the enclosed Arms because fetch distances are relatively short.

Local wind intensities and directions are substantially altered by local topography particularly where there are abrupt changes in relief such as found in the study area. In the absence of site specific records these effects can only be approximately estimated from observation and engineering judgement.

At the start of the project, winds were considered as a possible major contributor to the local water elevation in the Placentia region. Examination of previous flood events showed that winds have not been a major contributor to flooding. In fact during several of the flood events winds were no greater than 50 km/hr.

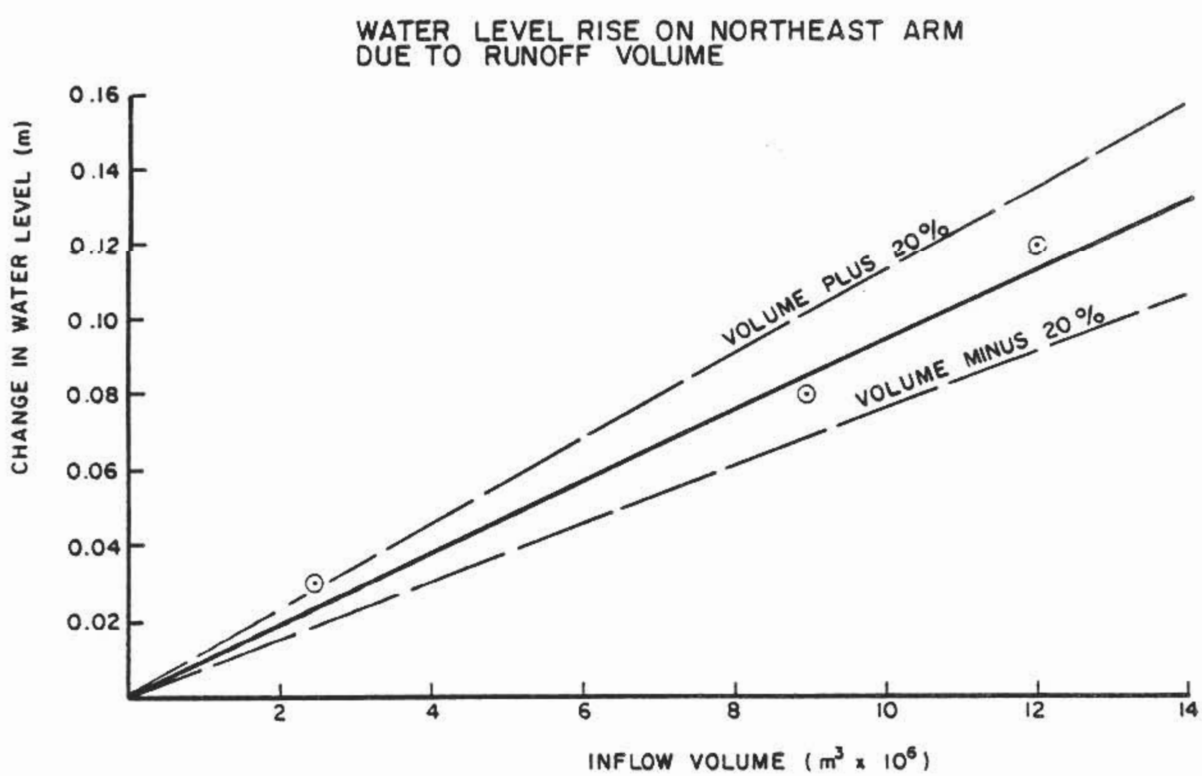
The changes in water level in the Arms due to wind stress are small. For example a sustained wind of 90 km/hr will raise the water level by .04 m, 0.13 m and 0.13 m in Swan Arm, Southeast Arm and Northeast Arm, respectively. (See Figure 6.3.6), as calculated by the following formula: (Saville, 1962).

FIGURE 6.3.4



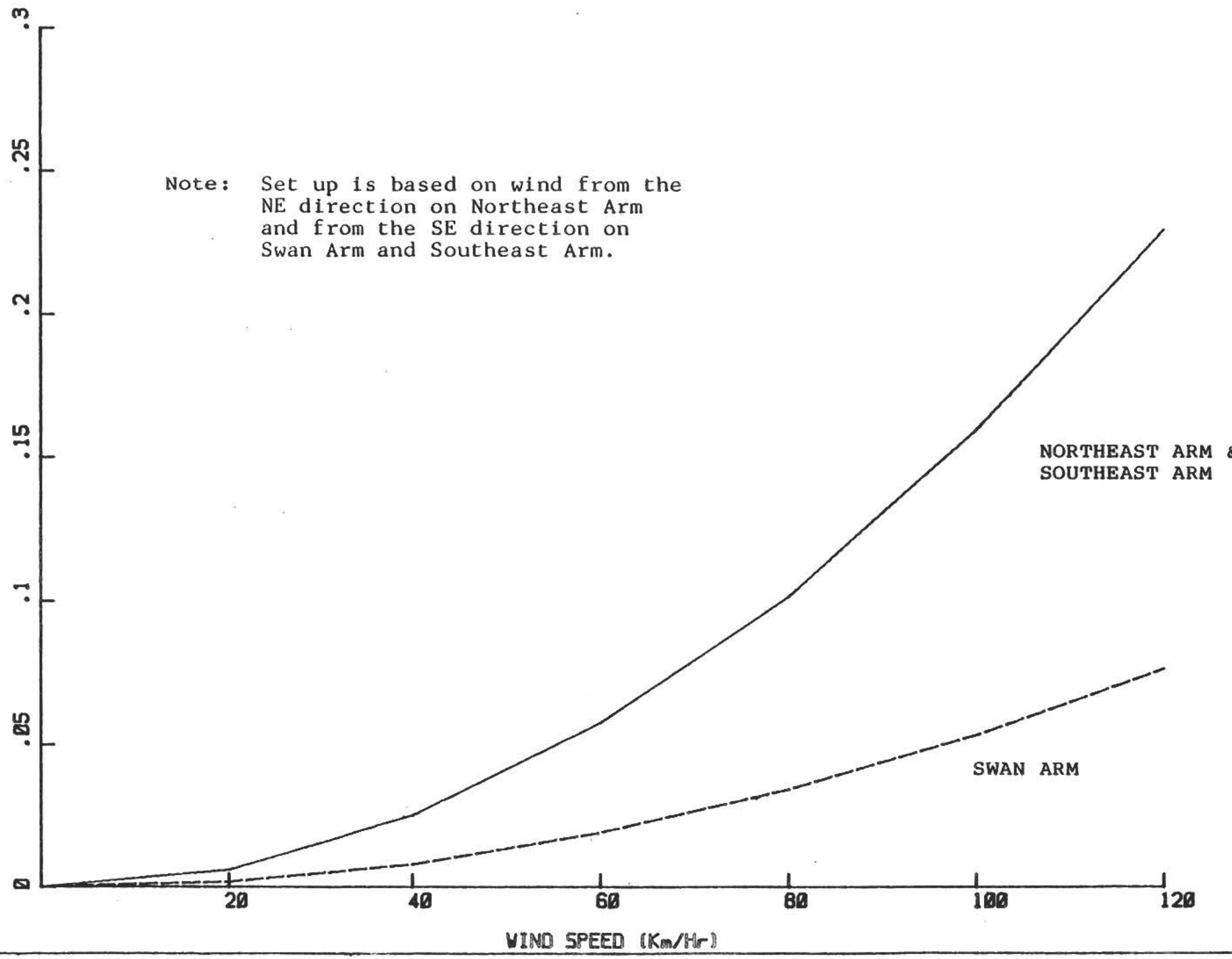
SENSITIVITY OF
WATER LEVEL RISE ON SOUTHEAST ARM

FIGURE 6.3.5



SENSITIVITY OF
WATER LEVEL RISE ON NORTHEAST ARM

FIGURE 6.3.6



WIND SET UP ON NORTHEAST, SWAN & SOUTHEAST ARMS

$$S = FU^2/63{,}000\ D$$

where S = setup in metres
F = fetch in km
U = wind speed in km/hr
D = average depth in m

These changes will only occur when there is a sustained wind blowing into the Town of Placentia or Jerseyside from across one of the Arms, as was the case during the January 10, 1982 flood event. Normally the winds which contribute to abnormally high tides blow inland across the Arms and hence away from the Towns.

6.3.5 Off Channel Storage

As mentioned previously, (Section 5.3.2) the low lying land area of Placentia, adjacent to the Narrows, and Southeast Arm were treated in the DWOPER model as areas of off channel storage. It can be expected that deletion or reduction of off channel storage in these areas would cause some increase in water levels in Swan Arm and Southeast Arm. Since elimination of some or all of the off channel storage in these areas is considered in Section 7.5 under flood control measures, it was deemed appropriate to test the sensitivity of water levels on these deletions or reductions.

To test the sensitivity of water levels to the elimination of the off channel storage adjacent to The Narrows, the flood event of December 22, 1983 (PRO 25) was used. The appropriate changes were made to the input data for the sections in The Narrows and the model was run for the new condition. There were no changes in water levels for Northeast, Swan and Southeast Arms, however, a slight increase in the water level in The Narrows of not more than 0.02 m was noted.

The sensitivity of the water level in Southeast Arm to elimination of the off channel storage in the area of the low lying land, bordering Southeast Arm and adjacent to Placentia could not be tested in the model because of the way in which the sections were taken for the model (See Figure 5.3.1). The effect of eliminating off channel storage from this area was checked manually by comparing the volume of lost storage and the total volume of storage on Southeast Arm, assuming the same volume of tidal inflow to Southeast Arm with and without the extra off channel storage. The effect noted was an increase in the Southeast Arm water level of 0.03 m when the off channel storage was eliminated from this area.

6.3.6 Effect of Ice

An ice cover forms in areas of still water while Placentia Gut, The Narrows and McDonald Gut remain open since the water velocities in these areas are too great (1.2 m/s +) to allow formation of an ice cover.

Eye witness reports indicate that, during periods of breakup, an ice jam starts to form at the bottle neck where Swan Arm flows into The Narrows. The jam starts to form on the ebb tide, and eye witnesses report seeing ice pans "diving under" the jammed ice, an action consistent with the formation of a hanging dam. This ice jam is not stable and is broken up as the water level changes on the ebb tide or on the returning tide. The existance of two way flow in this tidal system automatically clears jams of loose ice and limits problems associated with ice rafting and ice pile up which are minor phenomena because of the short fetch across the Arms. Additionally, ice moving through the Placentia Gut may interfere with the passage of fishing boats through this area - this again is believed to be a relatively minor problem of a transitory nature.

6.4 FLOOD RISK CONTOURS

As described in Section 2, flooding in the Placentia area is from two sources, viz:

i) High water level in the Arms, and
ii) Waves overtopping the beach.

This section discusses the development of the flood risk contours resulting from each source.

High Water Level in the Arms

In the foregoing sections the annual series of water levels, generated by the DWOPER model, for Placentia Road and each of the Arms was used in a Gumbel Extreme Value frequency analysis to determine the 1 in 20 year and 1 in 100 year water levels in each area. The sensitivity of these water levels to changes in the input parameters was analysed and the 1 in 20 year and the 1 in 100 year flood risk contours were developed. Table 6.4.1 summarizes the water levels generated by the DWOPER model, the sensitivity analysis on factors which would affect flood water levels and the recommended flood risk contours. The flood risk contours are shown on Drawing 4 which is included in Appendix VI.

TABLE 6.4.1

FLOOD RISK CONTOURS

ITEM	WATER LEVEL (Metres Above Geodetic)			
	Placentia Road	Northeast Arm	Swan Arm	Southeast Arm
Gumbel Frequency Analysis				
- 1 in 20 year water level	1.85	1.74	1.50	1.44
- 1 in 100 year water level	2.02	1.91	1.66	1.60
Sensitivity Results				
- Manning's "n" (1)	---	---	+0.07	+0.08
- Flow Area in The Narrows (2)	---	---	-0.07	-0.07
- Wind Setup on Arms (3)	---	+0.13	+0.04	+0.13
- Ice	---	---	---	---
Recommended Flood Risk Contours				
- 1 in 20 year water level	1.85	1.87	1.87 (4)	1.58 (5)
- 1 in 100 year water level	2.02	2.04	2.04 (4)	1.74 (5)

NOTES:

(1) Manning's "n" could decrease in time due to sedimentation, causing an increase in water level.
(2) The flow area could decrease due to sedimentation, causing a decrease in water level. This would probably offset the reduction in Manning's "n".
(3) Based on 90 km/hour wind on each Arm. (from NE direction on Northeast Arm, from SE direction on Swan Arm and Southeast Arm).
(4) For water levels above 1.6 m the water levels in Swan Arm and Northeast Arm are the same.
(5) Based on assumption high freshwater inflow and 90 km/hour wind in required direction occur simultaneously.

Figures 6.4.1 and 6.4.2 graphically show the frequency of occurrence of water levels for Northeast Arm, Swan Arm and Southeast Arm. Shown on these figures are the recurring water levels based on the Gumbel frequency analysis and the adjustment made for the results of the sensitivity analysis (Table 6.4.1) to provide the recommended water level frequency values. Figure 6.4.1 illustrates the influence that Northeast Arm has on Swan Arm water levels when the water level in Northeast Arm exceeds 1.6 m. This influence was explained in Section 5.4.

The study area was divided into four geographical regions (See Section 7.1.2). The water level frequency in Regions 1, 2 and 3 can be determined from Figures 6.4.1 and 6.4.2.

Region 1 is influenced by water levels in Swan Arm and The Narrows until the water level in Northeast Arm exceeds 1.6 m, whereupon the region is influenced by the water levels in Northeast Arm. The vertical step in the recommended curve for Region 1 on Figure 6.4.1 is a result of this influence.

Region 2 is influenced by the water level in Northeast Arm. The water level frequency for this region is obtained from the Northeast Arm curve on Figure 6.4.1.

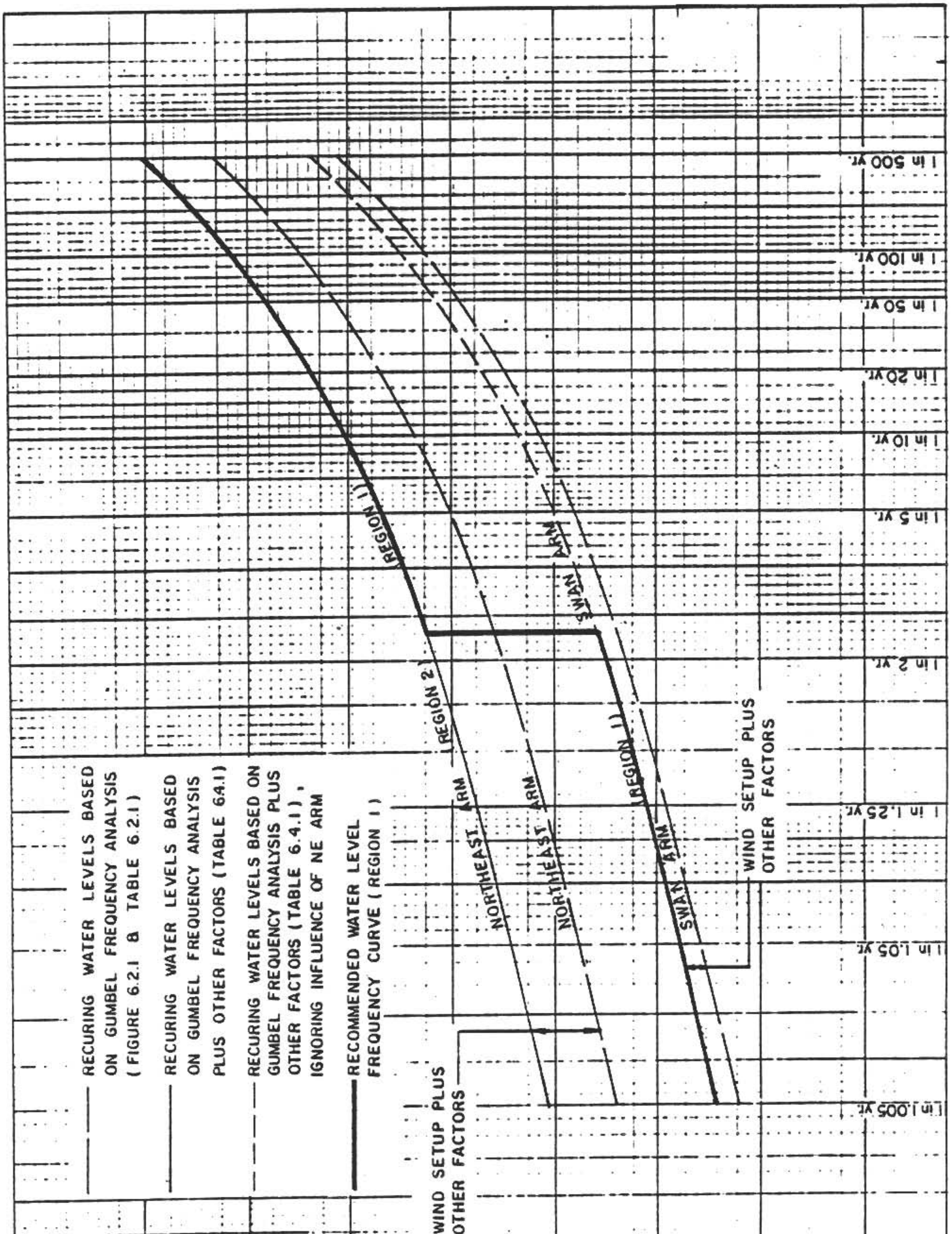
Region 3 is influenced by the water level in Southeast Arm. The water level frequency for this region is obtained from the Southeast Arm curve on Figure 6.4.2.

Waves Overtopping the Beach

In Part 4 it was noted that the 1 in 100 year wave in Placentia Road could overtop the beach to the west of the Town of Placentia. Since, in January 1982, waves overtopped the beaches west of Placentia and Jerseyside in what has been described herein as a 1 in 100 year event, it was assumed that for all 1 in 100 year events, both beaches would be overtopped.

Drawing 4 in Appendix VI shows the assumed locations of beach overtopping for the 1 in 100 year event (similar to the January 1982 event) and the estimated direction and extent of flow of the surface water as it flows through the Towns towards Southeast Arm, Swan Arm and Northeast Arm. Also shown are areas where pondage of water can be expected and the assumed water levels in these areas. Pondage of water would be temporary since the ground in this entire area is relatively previous. The depth of water ponding in the low areas would be relatively shallow, the maximum being approximately 0.4 m, in the area near Laval High School.

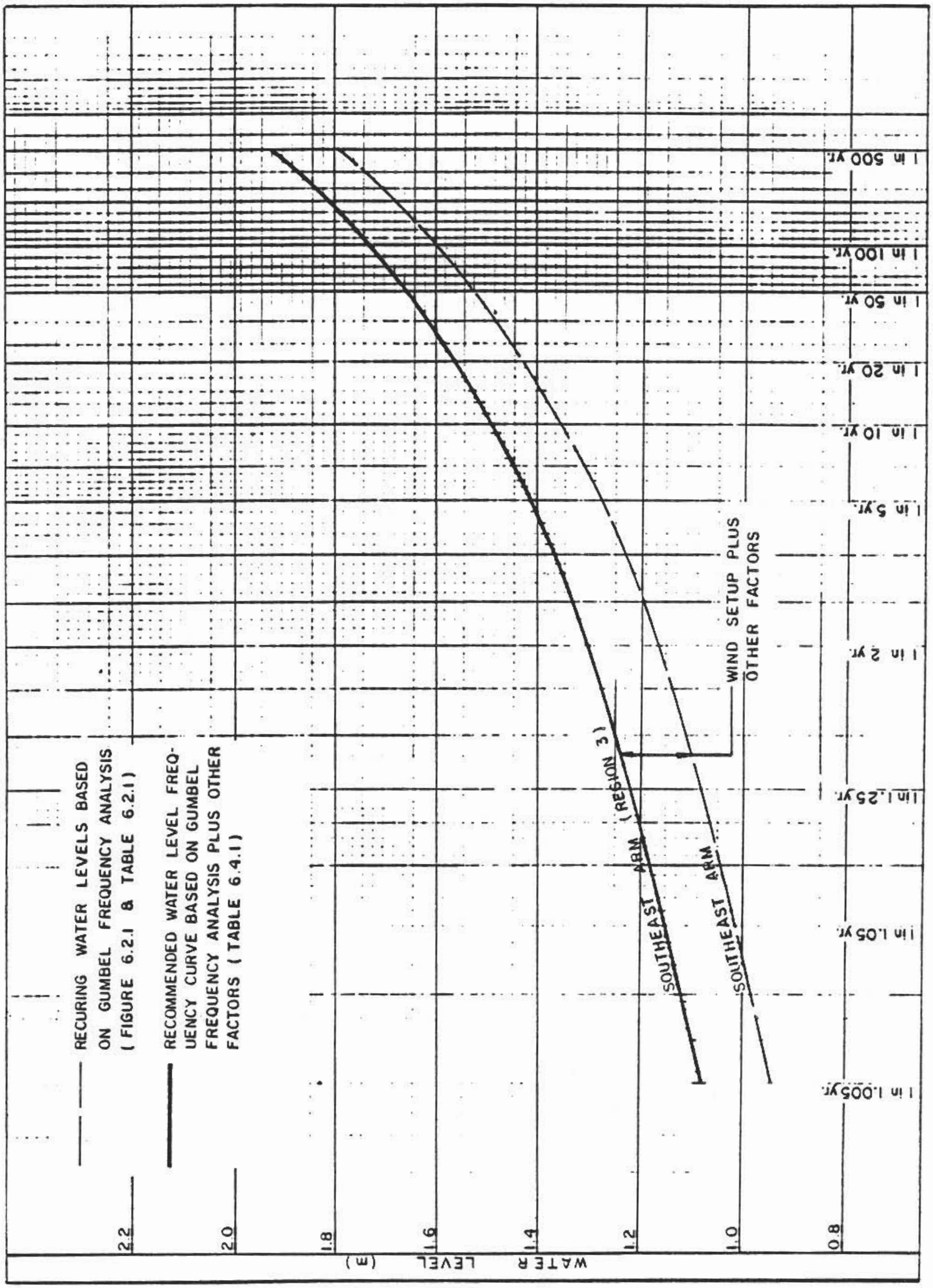
FIGURE 6.4.1



WATER LEVEL FREQUENCY

NORTHEAST ARM & SWAN ARM

FIGURE 6.4.2



WATER LEVEL FREQUENCY
SOUTHEAST ARM

PART SEVEN

FLOOD CONTROL MEASURES

7.1 ASSESSMENT OF DAMAGE FROM PREVIOUS FLOODS

As noted in Part 2 and in the Field Program Report (Volume 2) which was prepared in January, 1984 upon the completion of the field program aspect of this study, the Towns of Placentia and Jerseyside have experienced minor flooding as a relatively common occurrence over the decades. Over the years, new building construction has increased the number of buildings affected by flooding and a rise in the standard of living has increased the amenities to which residents have become accustomed. Therefore, with each succeeding flood, the value of property damage has generally increased.

It was also noted in Part 2 that the larger portion of flood damage in the area has been in Placentia with only a small amount of damage experienced in Jerseyside.

The worst flooding that has been experienced by the Towns of Placentia and Jerseyside were the most recent events of:

1) January 10, 1982
2) January 16, 1982
3) December 22, 1983 and
4) December 25, 1983

These events are described in detail in the Field Program Report and in Part 2 of this report.

The objectives of this section of the report are:

(a) to identify the damages which have occurred in the area as a result of the 1982 and 1983 flooding events;

(b) to describe and quantify the magnitude of flood damage which has occurred and which can be expected to occur should the community experience a 1 in 20 year or a 1 in 100 year flood level; and

(c) to outline and discuss the methodologies used in determining the types and the value of flood damages.

In order to accurately identify and quantify flood damages, four techniques were employed in the assessment, namely, site visits, questionnaires, background research and synthetic damage assessments. Discussed briefly, each is described as follows:

7.1 ASSESSMENT OF DAMAGE FROM PREVIOUS FLOODS (Cont'd)

-- Site visits were used as a data gathering tool for such details as: oceanographic and hydrological data; housing stock information; information on previous flood conditions and associated damages; and finally, to identify social and environmental considerations.

-- Questionnaires were distributed to local businesses which had experienced flood damages. In addition, the results of a questionnaire, developed and implemented by the Town of Placentia for the 1982 flood events, were reviewed.

-- Background research consisted of reviewing relevant reports by government departments, as well as those by consultants, in order to understand the history of flooding in the area as well as the remedial measures which have been implemented in the region. Additional background information was identified through conversations with Town Council personnel and residents of the area.

-- Synthetic damage curves were generated to evaluate the relationship between flood level and associated damages. These curves were based on data and techniques discussed in a report on flooding conditions in Southern Ontario (Acres, 1968).

7.1.1 Damage Assessment Methodologies

In an effort to assess previous flood damages in the Placentia area, two methodologies were employed:

i) Assessment of damages based on anecdotal data and questionnaires, and
ii) Use of synthetic damage curves.

Assessment of Damages Based on Anecdotal Data and Questionnaires

Data was gathered through background research (ie. conversations with local residents, utility companies and community officials) and flood damage questionnaires.

The first questionnaire used was the residential questionnaire. This was designed, implemented and compiled by the Town of Placentia. The objective of the questionnaire was to identify and quantify flood damages to residents of Placentia resulting from the January 10, 1982 and January 16, 1982 flood events. The questionnaire was distributed through the municipal council office and was designed to cover 100 percent of all residents experiencing flood damages.

The second questionnaire, designed, implemented and compiled by ShawMont Martec Limited, was aimed at the businesses (and Placentia Cottage Hospital) which were identified, by council personnel, as having experienced flood damages. This questionnaire was designed to identify flood damages to businesses by flood event and level of inundation. Examples of the business questionnaire and the residential questionnaire are included in Appendix IV.

Having performed a preliminary evaluation of the results of the questionnaire it became obvious that little confidence could be placed in the detail and accuracy of the data. Several problems were identified relating to the design of the questionnaire. The most serious being:

a) The residential questionnaire did not identify damage with geographic location or flood event. This was important since different events experienced different flood levels resulting from different driving forces.

b) The residential questionnaire did not cover 100 percent of the residents who experienced flood damage, as was first believed.

c) The business questionnaire results did not always correlate damage value and flood event, in that, events which were known to have had a lower flood level were found to be given a higher damage value than events with a higher flood level. Examples were also found of over-estimation of damage estimates.

Use of Synthetic Damage Curves

Because of the lack of confidence in the questionnaires an alternative method of assessing damages was used. This method provided an assessment of damages based on synthetic damage curves. As it turned out this method proved to confirm the relative accuracy of the questionnaire results rather than dispute them.

Using this method, a sample of approximately 30 homes, evenly distributed throughout the flood region, was selected. From this sample, housing stock details were collected. These details included height of first floor above grade level, house type classification, and whether or not houses had basements (developed or undeveloped).

7.1.1 Damage Assessment Methodologies (Cont'd)

Houses were classified as being either AW, BW or CW. A brief description of each type is as follows:

AW: An architect designed home having a wood frame and constructed of high quality materials.

BW: A typical middle class home having double walled wood frame construction.

CW: An economy home having wood frame thin walled construction.

The results of the survey sample were then extrapolated to incorporate all houses within the flood area.

Knowing the flood levels that occurred in the area it was then possible to calculate, using available mapping, the number of houses affected by particular flood levels and the expected depth of water in each house. In this regard it was necessary to assume that local topography was uniform throughout each contour interval. Only in areas where depressions were clearly identifiable from the mapping was it possible to identify the number of houses which had a greater level of water inundation.

Knowing the number of houses affected at particular water levels and the classification of houses within the flooded area, it was then possible, using the synthetic curves generated from flood data based on flooding conditions in the Galt-Cambridge area of Southern Ontario (Acres, 1968), to calculate total residential damage for particular water levels. This was done by multiplying the number of homes with a given water level (relative to the first floor) by the dollar value of damages, per house, corresponding to that water level. It should be noted at this point that houses with basements in the Placentia flood region are few and those which do exist are low (approximately 6'), undeveloped, with earthen floors. By comparison, basements in the Southern Ontario area were mainly developed with concrete floors and are often used for storage. For this reason a damage value of $300 has been assumed for basement flooding. This value is consistent with basement damage for houses in the Mill Brook area of Nova Scotia (McLaren Plansearch, 1984). Composite stage damage curves for different house types and damage types are presented in Appendix IV.

Since the Acres report was based on 1968 dollars it was necessary, using Statistics Canada inflation indices to update all damage values to 1984 dollars.

7.1.1 Damage Assessment Methodologies (Cont'd)

It was also necessary to compare property values between the Galt-Cambridge area of Southern Ontario and the Placentia region of Newfoundland. For this comparison, use was made of the "home evaluation calculator" issued by the Insurance Bureau of Canada. This publication provides a comparison of housing costs based on regional location. For the case in point, very little difference in the quality location factor was noted between the Galt-Cambridge area (2.07 for the standard type of house) and the Placentia area (2.10).

The Acres report gives synthetic damage curves for commercial establishments but it was not possible to transfer these to conditions experienced in Placentia. This was mainly due to the fact that the damage curves for the Southern Ontario region assume businesses to be homogeneous in terms of their merchandise (eg. grocery stores, furniture stores and footwear stores). In the Placentia area many stores sell a variety of items and it was therefore impossible to place a value of damage as a function of flood level. In order to identify a total commercial damage estimate a realistic value of $5,000 per business was used. This value, although subjective, is based on conversations with local officials and a review of the commercial damage questionnaire. The value was then multiplied by the number of businesses expected to be affected by a given water level based on available mapping and complemented by visits to the area.

In addition to direct damage to residential and business sectors, indirect damages were also estimated. Based on the Acres report these damages were assumed to be in the order of 10 - 15 percent for residential damage and 15 - 20 percent for commercial properties. Within these ranges, because of the low flood duration, values of 10 percent for residential and 15 percent for commercial were used to calculate indirect damages.

Utility damages were assessed as being 10 percent of direct physical damages (Inland Waters Directorate, 1979). Utilities would include such operations as energy and communications companies and municipal services. Transportation damages were assessed, based on the short duration of flooding, as 15 percent of the direct physical damages (Acres, 1968). Transportation damages include damages (direct or indirect) to highways, railroads and marine facilities and include damages to equipment and services.

Damages to agricultural and recreational sectors were addressed and no significant damages were identified. Therefore, no further discussion will be carried out on these items.

7.1.2 Results and Discussion

Viewing the flood area in a regional context the Placentia area can be subdivided into four distinct regions. As shown on Figure 7.4.1, these are as follows:

Region 1

This region encompasses the older section of the Town of Placentia and borders on Swan Arm and The Narrows. The area is the most important in terms of the flooding situation in Placentia, in that, it is the region most effected by recurring flood waters which primarily result from high tides in Swan Arm and The Narrows. The homes are mostly of the BW type (95%) with the remainder of the CW type. No AW type homes were found in this area (or any of the flood regions). Most of the Town's services such as schools, churches and the hospital are found in this region.

Region 2

This is the portion of Jerseyside immediately adjacent to the lift bridge. This area is bordered by Placentia Road to the west and Northeast Arm to the east. Flooding results from overtopping of the beach on Placentia Road and high tidal water in Northeast Arm. House classification and percentages were similar to those found in Region 1. Since conditions resulting in significant flooding are infrequent in this area the associated damages are low. Two commercial businesses were identified in this area.

Region 3

This region is the area bordering on Southeast Arm. The housing stock is similar to Region 1 although homes are newer and more basements are present (approximately 10% of the houses have basements). Housing density is much lower than in Region 1. Flood levels resulting in flood damage are infrequent in this area.

Region 4

This region is that part of the Town of Placentia adjacent to Placentia Road. This is a more recently developed area of Placentia with the majority of the homes having typical 8 feet high undeveloped basements. This region has been affected by waves overtopping the beach to the west. Flooding in this region, like Regions 2 and 3 is rare and therefore associated damages, in the long term, are low.

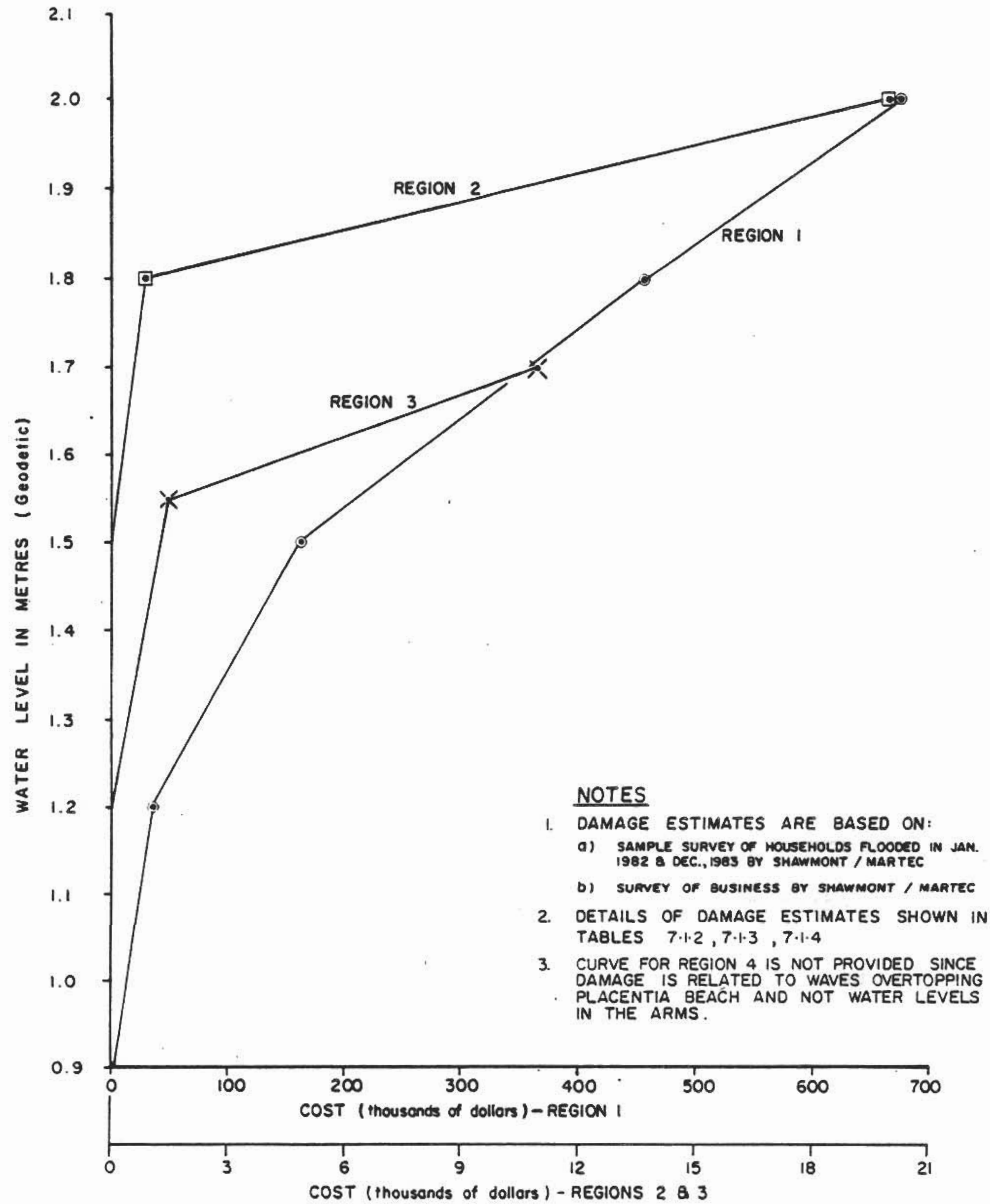
7.1.2 Results and Discussion (Cont'd)

As can be seen from Table 7.1.1 damage estimates range from \$300 for basement flooding to \$6,180 for flooding to a water level of 1.4 m (above the first floor) for BW type houses. By comparison, for the CW type houses damage estimates range from \$590 at a water level of 0.08 m (above the first floor) to \$4,890 for a water level of 1.40 m (above the first floor).

Based on the damage value for each particular water level and on the number of buildings damaged at those water levels (including indirect, commercial, utility and transportation damage) it can be seen in Tables 7.1.2, 7.1.3, 7.1.4 and 7.1.5 that Region 1 is the most affected area in terms of overall flood damage. This is more noticable in Table 7.1.6 which incorporates the probability of occurrance of particular water levels with the damage estimates. It can be seen from this table that for the 1 in 100 year flood event the average annual damage value for Region 1 (\$160,170) is much higher than Regions 2, 3 and 4 which have values of \$290, \$310 and \$635 respectively. It is also evident that the damages associated with the 1 in 20 year flood event (\$153,440) is not much less than that for the 1 in 100 year event.

To test the accuracy of the damage assessment methods, a comparison was made of the damage estimates derived from the synthetic curves with those calculated based on actual reported flood damages. For example, during the January 10, 1982 flood event for which the water level was approximately 1.70 m in Swan Arm, the actual damages were estimated to be \$275,000 as compared to the damage value of \$350,000 extrapolated from the water level damage curves (Figure 7.1.1). Therefore, the damage estimate, at this level, can be considered, with reasonable confidence, to be in the order of \$300,000. One explanation for the difference in estimates can be related to the fact that not all flood damage victims responded to the questionnaire.

A second example of the comparability of the synthetic versus the actual damage estimates is the synthetic damage value of \$575,000 for the December 22, 1983 flood event compared with the \$500,000 estimate provided by local municipal officials.



WATER LEVEL / DAMAGE CURVES

TABLE 7.1.1

REPRESENTATIVE STAGE DAMAGE VALUES FOR RESIDENTIAL PROPERTIES IN PLACENTIA*

Flooding Depth Relative to First Floor (m)	STRUCTURE CLASSIFICATION	
	BW ($) (1)	CW ($) (2)
- 1.52	300	---
- 0.76	300	---
0.08	770	590
0.23	2250	1570
0.46	3290	2810
0.77	4330	3770
1.07	5200	4530
1.40	6180	4890

* Adapted from Acres Limited, Guidelines for Analysis, 1968.

Notes:
1. BW refers to typical, middle class, double wall frame houses; 95% of all houses in the study area were of this type.
2. CW refers to economy type houses having rough frame, thin walled construction; 5% of all houses in the study area were of this type.
3. Values are in 1984 dollars.

TABLE 7.1.2

WATER LEVEL/DAMAGE SUMMARY - PLACENTIA REGION 1

(BORDERING SWAN ARM AND THE NARROWS)

Flood Level (m)	Probability of Occurence	Residential Damage [1] Number	Value ($)	Commercial Damage [2] Number	Value ($)	Utility [3] Damage ($)	Transportation [4] Damage ($)	Total Damages ($)
0.9	1.00	2	660	0	0	60	90	810
1.2	.88	9	11,470	3	17,250	2,540	3,810	35,070
1.5	.44	68	104,450	5	28,750	12,000	17,990	163,190
1.8	.11	145	335,900	6	34,500	33,540	50,300	454,240
2.0	.01	147	485,640	11	63,250	49,650	74,470	673,010

Notes:
1. Residential Damage includes 10% Indirect Damage.
2. Commercial Damage (including the Hospital) includes 15% Indirect Damage.
3. Utility Damage is calculated as 10% of Direct Residential and Commercial Damage.
4. Transportation Damage is calculated as 15% of Direct Residential and Commercial Damage.

TABLE 7.1.3

WATER LEVEL/DAMAGE SUMMARY - PLACENTIA REGION 2

(JERSEYSIDE)

Flood Level (m)	Probability of Occurence	Residential Damage Number	Residential Damage Value ($)	Commercial Damage Number	Commercial Damage Value ($)	Utility Damage ($)	Transportation Damage ($)	Total Damages ($)
0.9	1.00	---	---	---	---	---	---	---
1.2	1.00	---	---	---	---	---	---	---
1.5	.93	---	---	---	---	---	---	---
1.8	.11	2	660	0	0	60	90	810
2.0	.01	7	5,040	2	11,500	1,460	2,190	20,190

Notes:
1. Residential Damage includes 10% Indirect Damage.
2. Commercial Damage includes 15% Indirect Damage.
3. Utility Damage is calculated as 10% of Direct Residential and Commercial Damage.
4. Transportation Damage is calculated as 15% of Direct Residential and Commercial Damage.

TABLE 7.1.4

WATER LEVEL/DAMAGE SUMMARY - PLACENTIA REGION 3

(BORDERING SOUTHEAST ARM)

Water Level (m)	Probability of Occurrence	Residential Damage		Utility Damage	Transportation Damage	Total Damages ($)
		Number	Value ($)			
0.9	1.00	---	---	---	---	---
1.2	.86	---	---	---	---	---
1.5	.10	2	990	100	150	1,240
1.55	.05	2	1,180	110	160	1,450
1.70	.01	11	8,880	890	1,210	10,980

Notes:
1. Residential Damage includes 10% Indirect Damage.
2. Utility Damage is calculated as 10% of Direct Residential Damage.
3. Transportation Damage is calculated as 15% of Direct Residential Damage.
4. No Commercial Damage is expected at these water levels.

TABLE 7.1.5

WATER LEVEL DAMAGE SUMMARY - PLACENTIA REGION 4

(BORDERING PLACENTIA ROAD)

Flood Level (m)	Probability of Occurrence	Residential Damage		Utility Damage ($)	Transportation Damage ($)	Total Damage ($)
		Number	Value ($)			
Not Applicable (1)	.01 (2)	20	11,000	1,000	51,500	63,500

Notes:
1. Flood level cannot be identified because flooding was a result of waves over-topping the beach.
2. The event was a very rare occurrence and at best can be considered to occur once in 100 years.
3. Damages are calculated based on $500 per house as damage was primarily restricted to flooded basements.
4. Residential Damage includes 10% Indirect Damage.
5. Utility Damage is calculated as 10% of Direct Residential Damage.
6. Transportation Damage is calculated as 15% of Direct Residential Damage plus $50,000 for Beach Repair.
7. No Commercial Damage is expected at these water levels.

TABLE 7.1.6

AVERAGE ANNUAL DAMAGES

Region	Geographic Location	Value ($)	
		1:20	1:100
1	The Narrows/Swan Arm	153,440	160,170
2	Jerseyside	90	290
3	Southeast Arm	200	310
4	Placentia Road	---	635

7.2 EVALUATION OF PREVIOUS FLOOD CONTROL MEASURES

Over the past few years, marine and flood control related construction has taken place in the Placentia area. All this construction may not have been intended for flood control but would have had an effect on water levels in the area. The following discusses the effect each item of construction had on the flooding situation in the area.

i) Infilling and Breastwork Along The Narrows

In 1969 The Narrows along the east side of the Town of Placentia was infilled, in some places as much as 50 metres from the original shoreline. A timber crib breastwork was constructed to contain the infilling and thereby provided a docking area for small boats, which tied up alongside the breastwork.

Some concern has been expressed by residents that this infilling of The Narrows worsened the flooding condition in Placentia. It has been suggested that this infilling has constricted the flow of water through The Narrows, especially when the water was flowing from Swan Arm into Northeast Arm (tide falling in Placentia Road).

It was demonstrated in Part 5 of this report that the more that flow through a control is throttled, the greater is the attenuation of the maximum water level in the Arms. The infilling of The Narrows, therefore, has had the effect of reducing the maximum water levels in Swan Arm and Southeast Arm during flood events, thereby reducing flood damage in areas adjacent to these Arms.

ii) Wave Wall on the Beach West of Placentia

Originally, a small wave wall was constructed on the southwest end of the beach in the area known as the Blockhouse. This wall was commonly called the breakwater and was constructed to prevent waves from washing over the beach, across the road and into Southeast Arm. A severe storm sometime during the 1960's washed away this wall. Later, this wall was replaced with a properly constructed wave wall. During 1982 this wall was raised and extended a distance of 300 metres to a point opposite the District Vocational School. This construction was undertaken as a flood control measure following the storm of January 16, 1982 when waves broke through the beach. This wall, however, was not extended to the area which was breached in 1982. Instead,

ii) Wave Wall on the Beach West of Placentia (Cont'd)

material dredged from The Narrows was dumped in the area of 1982 breaching to create a berm along the crest of the beach to the same height as the wave wall.

The height of the wall is sufficient to prevent overtopping by the 1 in 100 year waves but there is some concern about the stability of the berm of dredged material.

iii) Wave Wall on the Beach West of Jerseyside

A wave wall similar to the one west of Placentia was constructed as a flood control measure to prevent waves from overtopping this area of the beach. This wall prevents smaller waves from overtopping the beach but was not adequate to prevent the larger waves, which were experienced during the January, 1982 storm, from overtopping the beach and flowing across this part of the Town of Jerseyside, into Northeast Arm.

iv) Breastwork Along Northeast Arm in Jerseyside

The shoreline of the beach on the Northeast Arm in Jerseyside has been built up behind a timber crib breastwork. It is reported by residents of the area that this construction has reduced but not eliminated damage resulting from high water levels in Northeast Arm.

v) Dredging in The Narrows

During 1982 a portion of The Narrows was dredged. This was not undertaken as a flood control measure but to increase the depth of water adjacent to the breastwork along The Narrows, to provide berthing space for boats.

Dredging would have the effect of increasing the flow area of The Narrows. As discussed in Part 5 of this report dredging in The Narrows would reduce the throttling affect of The Narrows to the flow of water into Swan Arm and Southeast Arm. Consequently, this has resulted in higher maximum water levels (or flood levels) in Swan Arm and Southeast Arm than would otherwise have been experienced. In Section 6.2, it was shown that the 1982 dredging program resulted in the water levels in Swan Arm and Southeast Arm being 0.08m and 0.07m, respectively, higher than would otherwise have been experienced.

vi) Dredging in Placentia Road

During late 1983, Placentia Road was dredged just seaward of The Gut Bridge. This was required to remove an accumulation of sediment which was causing obstruction to boats.

Theoretically, any dredging of an area which acts as a control on the flow of water will result in increased water levels on the controlled water body. Since the flow area in the reach of Placentia Road where dredging was carried out is so large, minimal control on the flow of water is provided. Therefore, dredging in this area will have had a minimal effect on water levels in the three Arms.

vii) Groynes in Placentia Road

Two groynes were consructed near the north end of the beach west of Placentia in 1983. The purpose of these groynes was to reduce the amount of littoral drift along the beach which is causing sediment deposition at points in the study area, particularly Placentia Road.

The south groyne is apparently fulfilling its intended purpose because it has trapped a large volume of sand which would otherwise have been deposited elsewhere in the system. Shore based dredging will soon be necessary to remove the accumulated sand to permit the groyne to continue to function as an efficient sand trap.

The two groynes are apparently too close to each other and the north groyne is not functioning properly. (Verbal communication - ShawMont-Martec and Small Craft Harbours Directorate - Fisheries and Oceans Canada). Severe erosion has occured around this groyne and consideration is being given to relocating it.

viii) Raising of Buildings

It was apparent during site visits to Placentia that a small number of houses had been raised above their original levels. Presumably, this was a measure taken by these few individuals to reduce flood damage. If ingress of water into a building can be prevented by raising the building high enough, then damage to the building and contents will be reduced or eliminated.

7.2 EVALUATION OF PREVIOUS FLOOD CONTROL MEASURES (Cont'd)

ix) Modifications to Utilities

One of the side effects of flooding in the Placentia area has been power outages. This problem has resulted in increased damage costs as a result of frozen water pipes and spoilage of frozen foods. It is understood from discussions with the power utility that this problem was mainly due to problems with icing of the aerial cable terminations for the transmission line at the lift bridge.

Since the January, 1982 flood events, it is understood that these terminations have been relocated and this has eliminated the problems previously experienced. Although this was not a flood control measure, it was a measure taken to reduce damage costs in future flood events.

7.3 TOWN BY-LAWS AND BUILDING REGULATIONS

Enquiries directed to the Town Council office of Placentia and the Department of Municipal Affairs indicated that there are no Town by-laws or building regulations, concerning building construction, in the Placentia area. The Department of Municipal Affairs is presently compiling a comprehensive land use plan for the Placentia area and vicinity, and are awaiting the results of this study on the extent of flooding, before identifing potential development zones.

7.4 FLOOD CONTROL OPTIONS

7.4.1 General

In considering flood control options for the Placentia area, it must be recognized, firstly, that the Town of Placentia and a portion of the Town of Jerseyside are built on a wide, low and generally flat expanse of beach. This beach was developed by a depositional process caused by the interaction between the open water of Placentia Bay to the west and the three Arms (Northeast, Swan and Southeast) to the east. The beach is comprised of a variable mixture of fine sand and rounded cobbles ranging from 100 percent fine sand, with a relatively low permeability, to 100 percent rounded cobbles, with a relatively high permeability.

7.4.1 General (Cont'd)

Secondly, it must be recognized that the tidal exchange between Placentia Road and the three Arms (Northeast, Swan and Southeast) through the hydraulic controls (The Gut, The Narrows and MacDonald Gut) results in differential water levels in all four areas. As described in Section 5.2, the water level differentials result from the throttling effect on the flow of water provided by each control, with the consequential attenuation of the water level fluctuations.

In addition to the foregoing, an understanding of the flooding process is important for an appreciation of the flood control options to be discussed. As described in Part Two and the Field Program Report, flooding in the Placentia Area has occurred from two sources:

(i) high water levels in the Arms (Northeast, Swan and Southeast), and

(ii) waves overtopping the beach to the west.

High Water Levels in the Arms

In this case, high water levels in the Arms occur with an extremely high tide in Placentia Road which, in turn, results from the combined effect of tide, wind and atmospheric pressure. It has been demonstrated that fresh water inflow into the Arms has a small effect on water levels in the Arms. With a high tide in Placentia Road, a large volume of water flows through The Gut and into Northeast Arm, causing the Northeast Arm water level to rise. At the same time, water flows through the Narrows into Swan Arm and thence through MacDonald Gut into Southeast Arm. As described in Section 5.2, the throttling effect of the controls results in a water surface profile of diminishing water levels in the direction of flow.

As the water level throughout the system rises with the tide in Placentia Road, the water level in Swan Arm reaches a level at which it overtops Riverside Drive South in the Southeast section of the Town between the Cottage Hospital and the small boat slipway. This level is about 1.0 m above Geodetic Datum. At that point in time, the water level in Northeast Arm is about 1.48 m above Geodetic. The northeast section of the Town, adjacent to Northeast Arm does not flood at this water level since the Riverside Drive South which parallels the Narrows in this area is at an elevation of about 1.6 m above Geodetic. As

7.4.1 General (Cont'd)

High Water Levels in the Arms (Cont'd)

the water levels in Swan Arm and Northeast Arm continue to rise, the east side of the Town of Placentia begins to flood from Swan Arm and the area of inundation spreads to the west and to the northwest along the Narrows. When the water level in Northeast Arm exceeds the level of about 1.6 m above Geodetic, the entire eastern section of the Town is inundated. Drawing 2 shows the areal extent of flooding experienced during the 1982 and 1983 floods. This drawing, which also shows the ground contours throughout the Town, indicates that the lowest area is adjacent to Swan Arm.

At the same time that flooding is occuring in the area of Town adjacent to Swan Arm, the Narrows and Northeast Arm, the low lying land bordering Southeast Arm at the southern end of the Town is also being inundated. Drawing 2 also shows the areal extent of flooding experienced in this area during the 1982 and 1983 floods. The peak water level experienced in this area is generally slightly lower than that in Swan Arm.

Waves Overtopping the Beach

When a high onshore wind combines with a high tide in Placentia Road, higher than normal wave runup on the beach to the west of the Town can result. With continuous pounding by the waves, the beach could be eroded to the point where waves could overtop the crest. This was the case on January 16, 1982 when the beach was eroded in two areas, near Laval High School and the District Vocational School, and permitted the waves to wash through eroded depressions in the beach crest. The water flowed east and northeast through the Town like a broad river, following the lowest land, until it discharged into Southeast Arm, Swan Arm and the Narrows. Water ponded in the lowest areas along the flood route resulting in flooding to some buildings.

7.4.2 Options

Several possible options (structural and non-structural) for flood control and relief from flood damage in the Placentia area were considered. Each option is discussed hereafter, however, some were discarded on the basis of impracticality or ineffectiveness on flood control. The effectiveness of each option on flood control varies and each offers particular advantages or disadvantages. Each option is not necessarily an effective control measure in itself and, where necessary, is combined with other options to provide alternative flood control measures which

7.4.2 Options (Cont'd)

are described in the next section. A total of seven structural options were finally considered as being practical and were ultimately included singly or in combination with other options to provide alternative flood control measures. These options are located on Figure 7.4.1. The costs associated with each alternative flood control measure are discussed in Section 7.5.

The structural options considered were:

(a) Raise/floodproof buildings within the flood plain,
(b) Excavate channel through the beach between Southeast Arm and Placentia Road,
(c) Close the two end spans of The Gut Bridge,
(d) Close the end spans of The Gut Bridge and provide a flood control gate in the middle span,
(e) Dredge The Narrows,
(f) Dredge Placentia Road and/or The Gut,
(g) Construct a groyne in The Narrows,
(h) Construct a steel sheet pile wall along The Narrows,
(i) Raise Riverside Drive and Swan's Road,
(j) Construct a dyke along the shoreline of Southeast Arm,
(k) Extend the existing wave wall on the beach to the west of Placentia for a distance of 300 m,
(l) Riprap a section of the beach to the west of Placentia for a distance of 300m,
(m) Construct additional groynes along the beach to the west of Placentia,
(n) Raise the wave wall on the beach to the west of Jerseyside, and
(o) Raise the brestwork along the shoreline of Northeast Arm to the east of Jerseyside.

The non-structural options considered were:

(p) Early warning (flood forecasting) and contingency planning measures, and
(q) Property acquisition or zoning.

A detailed description of each option follows with advantages and disadvantages identified.

Structural Options

Option (a) - Raise/Floodproof Buildings

The main concern of the flood victims in the Placentia Area is the financial loss incurred as a result of private property damage, or business loss. In so far as private property is concerned, property damage results from damage to homes and personal property contained in the homes

7.4.2 Options (Cont'd)

Structural Options (Cont'd)

Option (a) - Raise/Floodproof Buildings (Cont'd)

whereas business losses result from building damage, damage to goods, lost sales due to inaccessibility of buildings and general clean-up costs. This loss could be substantially reduced by raising the houses and buildings above the flood level or otherwise floodproofing them to prevent the ingress of water. Most of the private houses are built entirely above ground without substantial foundations. A few of the newer houses have concrete foundations and some even have basements. Buildings which are built entirely above ground would have to be raised and blocked at a higher level. Houses or buildings with more substantial foundations or basements would have to be raised off the foundations and the foundation or basement walls built up to suit the building. Otherwise the basement walls would have to be floodproofed, including sealing of any door and window openings or installing leak-proof doors and windows of sufficiently heavy material to withstand the hydrostatic pressure of, perhaps, up to a metre of water.

This option would not eliminate or reduce the extent of flooding and would not improve access to flooded properties but would substantially reduce flood damage to those buildings which were raised or floodproofed.

Option (b) - Channel Between Southeast Arm and Placentia Road

Initially, the excavation of a channel through the beach between Southeast Arm and Placentia Road, in the area of the Blockhouse, was considered as a means of reducing high water levels in Southeast Arm. This of course was based on the assumption that the high water level in the Arms was caused by high volumes of fresh water inflow from the rivers flowing into the Arms. It was also based on the assumption that the water level in Placentia Road was at all times lower than the water level in Southeast Arm.

As a result of investigations into the hydrology and the hydraulics of the area, it has been shown that the high water levels in the Arms, and particularly Southeast Arm, were not due to high volumes of fresh water inflow. It has also been shown that the high water level in Placentia Road is, on average 0.46 m above the high water level in Southeast Arm. Therefore, it can be concluded that excavation of a channel through the beach between Southeast Arm and Placentia Road would result in the higher water level of Placentia Road causing a flow of water through the

7.4.2 Options (Cont'd)

Option (b) - Channel Between Southeast Arm and Placentia Road (Cont'd)

channel into Southeast Arm, instead of out of Southeast Arm. This would result in higher water levels in Southeast Arm than those which have previously been experienced.

For this reason, this alternative is considered counter productive and was not be considered further.

Option (c) - Close the End Spans of The Gut Bridge

The effect of closing the end spans of the The Gut Bridge would result in additional throttling of the flow of water through this control. As discussed earlier in this section, throttling the flow through a control, generally, results in attenuation of the water level fluctuation of the water body downstream of the control. In other words, closing the end spans of the Gut Bridge should result in lower high water levels in Northeast Arm, and consequently in Swan and Southeast Arms, than those presently experienced. The hydraulic model showed that for the December 22, 1983 flood event the high water levels in Northeast Arm, Swan Arm and Southeast Arm would have been lowered by about 0.16m, 0.12m and 0.11m respectively, if the ends of the Gut Bridge had been closed.

This option would however, have the disadvantage of causing increased downtime for fishermen who must pass through The Gut on their way to and from the fishing grounds. Closing the end spans of the bridge would result in increased velocity of water through the middle span. Fishermen would have to wait for the velocity of flow through the bridge to drop to a tolerable level before attempting passage through The Gut.

Another disadvantage of this alternative is the increased risk of damage to the bridge piers. This damage could result from increased scouring around the base of the piers due to the higher water velocity and also from impacts with ice slabs which would be carried through the open span at higher velocities. Measures could be taken to prevent or reduce the effect of scouring, however, these measures would be extremely difficult and costly to install considering the high water velocities which would be encountered.

Considering the relatively minor reduction in water levels, the undesirable social impact and the potential for structural damage to the bridge that closing the end spans would cause, this option is not considered practical.

7.4.2 Options (Cont'd)

Option (d) - Close the End Spans of The Gut Bridge and Install a Closure Gate

This option would be similar to Option (c) in that the two end spans of the bridge would be closed. The middle span would be fitted with a mechanically or hydraulically operated closure gate which would normally be left open to permit passage of boats, fish and normal tidal enchange. During the historical flood season (December and January) the gate would be closed just prior to the onset of the predicted highest tides for each month if the other conditions necessary for flooding to occur were present.

Historically, as has been demonstrated, flooding has occurred during high wind and the occurrence of the highest tide for the particular month in which the flood occured. Also, the high tide has occurred very close to the time predicted in the Canadian Tide and Current Tables published by Fisheries and Ocean Canada. Therefore, if conditions for flooding were right, the gate could be closed just prior to the predicted high tide. The high water level, then, would be restricted to Placentia Road only and the water level in the Arms would rise only a small amount due to fresh water inflow to the Arms and a small amount of sea water which would perculate through the beach around the gate and bridge structure. Once the high tide was past (2 or 3 days) or the wind abated, the gate could be opened again.

This option would have the disadvantages quoted for Option (c), ie. the disruption to boat traffic due to increased velocity of water through the bridge and the potential for damage to the bridge piers. In addition, the fact that this area is subject to deposition of sediment could result in operating problems with the gate due to sediment accumulation around critical gate parts. The cost of the gate and modifications to the existing structure to receive the gate are expected to be too high to consider this as an economic option.

Option (e) - Dredge The Narrows

Dredging The Narrows would have the effect of increasing the cross-sectional area of the flow channel, thereby reducing the throttling effect of this hydraulic control. As a result, on an incoming tide, the water levels in Swan Arm and Southeast Arm, which are downstream of this control would rise to higher levels than they would if dredging is not done. Therefore, dredging in The Narrows would have a detrimental effect on flooding in the area of the Town of Placentia adjacent to Swan Arm and Southeast Arm. If The Narrows was dredged, water would flow out of

7.4.2 Options (Cont'd)

Option (e) - Dredge The Narrows (Cont'd)

Southeast and Swan Arms, back into Northeast Arm, faster on the outgoing tide. However, it must be remembered that it is the high water level reached during the incoming tide that has resulted in the flooding condition.

Considering this, it can be concluded that dredging of The Narrows would aggravate the flooding situation in Placentia and will, therefore, not be considered further.

Dredging the Road and/or The Gut would increase the cross sectional area of the flow channel in this area. As discussed above for The Narrows, this would have the effect of reducing the throttling effect of the approach to The Gut and The Gut itself. This would result in higher water levels downstream of The Gut (in Northeast, Swan and Southeast Arms) on an incoming tide. This option would have the advantage, however, of a reduced velocity of flow through The Gut, because of the increased flow area.

Since this option would aggravate the flooding situation, it is not recommended as an option for flood control.

Option (g) - Groyne in The Narrows

Construction of a groyne in the Narrows would have the effect of reducing the cross-sectional area of the Narrows, thereby throttling the flow of water. Initially it was thought this would result in lower maximum water levels in Swan Arm and Southeast Arm, on an incoming tide. However, model testing revealed higher maximum water levels in the Arms would result. A preliminary hydraulic assessment of this option showed that increased head losses for net outflow through the contracted section at the groyne produced increases in mean water levels in Swan and Southeast Arms which more than offset the reductions in the range of tidal fluctuations achieved by this measure. It was, therefore, concluded that this option will be completely ineffective. This option was not considered further.

Option (h) - Steel Sheet Pile Wall Along The Narrows

This option would involve the construction of a continuous steel sheet pile wall along the full length of the Narrows and around the northwest corner of Swan Arm, tying into high ground at each end, to contain the high water levels of Northeast Arm, the Narrows and Swan Arm. The sheet piling would be driven adjacent to and on the water side of the existing timber crib wall. At the north end of the Narrows the existing wharf and building presently owned by

7.4.2 Options (Cont'd)

Option (h) - Steel Sheet Pile Wall Along The Narrows (Cont'd)

Aylward's would have to be removed to allow construction of a continuous wall to tie into high ground near the south approach to the Gut Bridge. This wharf and building are presently in a state of disrepair. In the northwest corner of Swan Arm, at the southern end of the wall, the sheet piling would tie into high ground created by raising the section of Swan's Road from the small boat slipway along the west side of Swan Arm to the cemetary road. The height of this wall would be approximately one (1) metre above the adjacent Riverside Drive South.

This option would be an effective flood control measure and would not effect the existing water or ice flow patterns through the Narrows. A disadvantage of this option would be increased difficulty for people getting to their boats moored alongside the wall. Access would be provided over the wall by ladders attached to the wall. Another disadvantage of this option would be the inconvenience (small as it would be) caused by the raising of Swan's Road around Swan Arm. Another disadvantage of this option would be the increased ponding and increased length of time that water would be ponded along the east side of Placentia, if the beach to the west was ever overtopped again by waves. In January, 1982 when this happened, the water ran across Placentia and flowed into Swan Arm, the Narrows, and Northeast Arm. A wall constructed as described above would retard the runoff of water as experienced in January, 1982 and would cause more extensive ponding of water until the water could perculate through the subgrade. This option may be combined with Option (k), as described below, to minimize this problem.

Option (i) - Raise Riverside Drive and Swan's Road

This option would be similar to Option (h) above in that it would provide a barrier to high water levels in Northeast Arm, the Narrows, and Swan Arm. It would involve raising Riverside Drive South and part of Swan's Road, as described in Option (h) above, approximately one (1) metre to create a dyke around the east side of Placentia. At the north end of the Narrows, Riverside Drive South would be raised where it passes adjacent to the existing Aylward's wharf and building (the wharf and building need not be removed as in Option (h)) and would tie into the high ground on the south approach to the Gut Bridge. At the south end, around Swan Arm, the raised Riverside Drive South would tie into the raised section of Swan's Road as described for Option (h).

7.4.2 Options (Cont'd)

Option (i) - Raise Riverside Drive and Swan's Road (Cont'd)

Depending on the permeability of the subsurface materials beneath Riverside Drive South, it may be necessary to excavate a cut-off trench along Riverside Drive South, which would be backfilled with a relatively impervious material, prior to raising the road. For cost estimating, it has been assumed that such a cut-off would be necessary.

This option would also be an effective flood control measure with the same advantages and disadvantages as Option (h). One additional disadvantage in this option is the ramping that would be required to provide access, from the raised road, onto adjoining roads and private driveways. Near the north end of the Narrows it may even be necessary to raise five or six buildings and related properties, due to their close proximity to the road itself. This option may be combined with Option (k), as discussed for Option (h), to minimize water pondage due to waves overtopping the beach to the west.

Option (j) - Dyke Along Shoreline of Southeast Arm

This option was considered as a means of flood control in the area of Placentia adjacent to Southeast Arm. This area is generally low lying, flat and undeveloped. The area is somewhat marshy but the subsurface materials appear to vary from fine sand to rounded cobbles.

Construction of a dyke along the shoreline of Southeast Arm would prevent the high water levels of Southeast Arm from encroaching on this area and the area could be developed. The dyke would tie into the high ground near the blockhouse on the western end and the high ground of Dixon Hill on the eastern end.

This construction would essentially eliminate off channel storage on the low lying land bordering Southeast Arm as discussed in Section 6.3.5. This would result in insignificantly higher (0.03m) water levels in Southeast Arm and Swan Arm.

Natural drainage in this area is toward Southeast Arm and construction of a dyke as described would possibly interfere with drainage, and may require a culvert fitted with a tidal gate or pumping station to control drainage from this area. Prior to dyke construction extensive subsurface investigations would be required along the route of the dyke. This would be to determine the permeability of the subsurface materials and the requirement for a cut-off

Option (j) - Dyke Along Shoreline of Southeast Arm (Cont'd,

trench beneath the dyke to minimize seepage of water from Southeast Arm into the reclaimed area during high water levels in Southeast Arm. For this study, a 0.90 m diameter drainage culvert with flap gate has been assumed for costing purposes.

Option (k) - Extension of Wave Wall Along Beach

This option was considered as a means of preventing waves from overtopping the beach to the west of the Town of Placentia as occured on January 16, 1982, and thereby protecting the Town from the second source of flooding discussed earlier in this section. It would involve the extension of the existing timber crib wave wall a distance of approximately 300 metres to a point beyond the area of overtopping in January, 1982.

The extension would involve construction of a similar wall as constructed in 1982 by the Dept. of Transportation. It is felt that a wall of this length would prevent any future overtopping of the beach since this area of the beach, and that farther to the southwest aready protected by such a wall is the narrowest section on which wave runup to the top is possible. To the northeast, the beach widens and wave energy is dissipated before the wave runup reaches the top of the beach.

Option (l) Riprap Beach West of Placentia

This option, like option (k), was considered as a means of preventing waves from overtopping the beach to the west of the Town of Placentia. It would involve placement of heavy riprap on the existing beach profile for a distance of 300 metres north of the existing wave wall. Approximately two metres thickness of riprap would be required over an area approximately 20 metres wide and 300 metres long near the beach crest. The riprap would decrease the runup of waves and stabilize the existing beach crest.

The actual extent of riprap, riprap size and grading would require indepth study to determine its effectiveness on reducing wave runup and beach overtopping. It is felt that placement of riprap in this area would reduce the runup of the 1 in 100 year waves such that overtopping of the beach would be eliminated. However, with the volume of riprap required, the estimated cost of this option exceeds that of Option (k). Considering that Option (k) is much more definitive, as well as more economical, this option will not be considered further.

7.4.2 Options (Cont'd)

Option (m) - Additional Groynes along Beach West of Placentia

This option was considered as a means to encourage natural widening of the beach, by a depositional process, in the area where it is narrowest (a distance of about 300 metres north of the existing wave wall). At this location the beach is narrow enough that the 1 in 100 year waves can runup and overtop the beach. Widening the beach in this area would reduce the wave runup.

A total of four (4) groynes spaced a distance of approximately 100 metres apart would be required. Prior to construction of these groynes, extensive studies would be required to determine the exact locations required to prevent counterproductive effects which could result from incorrect placement.

Considering the uncertainty of the effectiveness of groynes in this area, the relatively high cost of initial construction and high maintenance costs as compared with Options (k) and (l), this option is not considered further. It is felt that Option (k) would be more effective and more economical.

Option (n) - Raise Wavewall West of Jerseyside

This option was considered as a means of preventing waves from overtopping the existing wave wall to the west of Jerseyside. During the January 16, 1982 event, waves overtopped the existing wall, flooded the ball park and caused water to flow across the highway and between houses to discharge into Northeast Arm. This occurrence is apparently very unusual, being similar in occurence to the overtopping of the beach west of Placentia. Raising the existing wall about one metre would prevent future overtopping.

Option (o) - Raise Breastwork East of Jerseyside

This option was considered as a means of reducing flood damage in Jerseyside as a result of high water levels and waves in Northeast Arm. The extent of previous flood damage in Jerseyside, as a result of high water levels in Northeast Arm has been very minor, being limited to the few houses along the shoreline of Northeast Arm, just north of the bridge.

7.4.2 Options (Cont'd)

Option (o) - Raise Breastwork East of Jerseyside (Cont'd)

Raising the existing retaining wall about one metre would not eliminate flooding in this area; at best it could be expected to reduce flood damage by preventing waves from travelling into the area. It is expected that the existing breastwork on which the extension would be constructed is not impervious and a high water level in Northeast Arm would permeate through the wall to attain a similar level inside the wall. Buildings which would otherwise be damaged by wave action, in combination with the high water level, would not receive damage if waves are prevented from entering the area.

Non-Structural Options

Option (p) - Early Warning (Flood Forecasting) and Contingency Planning Measures

Reduction in flood damages may be achieved if sufficiently early warning of an impending flood can activate pre-planned procedures to protect property. To provide early warning of an impending flood would require monitoring of conditions contributary to flooding in Placentia, during the flood prone period. Generally, current weather conditions and trends and in particular, winds and tides would have to be monitored during the period mid-December to the end of January each year.

Monitoring of conditions contributary to flooding would have to be done by trained and responsible personnel. The monitor(s) would issue early warnings so that residents and businesses could prepare for a possible flood. This warning could activate emergency plans for reduction of flood damage by allowing time for residents to raise valuable property to higher levels or to evacuate buildings in a more orderly fashion.

The success of early warning measures is dependant upon the receipt of timely and accurate flood warning and the co-operation of all people concerned.

Unfortunately, flood waters can rise to damage levels quickly in Placentia and it may not be possible to provide flood warnings sufficiently in advance of a specific flood to eliminate damage. There would undoubtly be cases where warnings would be ignored and other cases where people may not be available to carry out the appropriate actions. In some buildings it may not be possible, in the time available, to raise property to higher levels, because of their permanency or physical weight. It may be more practical to place the Town on a state of alert during the flood prone period so that the Town is in a state of preparedness with all valuable property raised to higher levels during this period.

7.4.2 Options (Cont'd)

Option (p) - Early Warning (Flood Forecasting) and Contingency Planning Measures (Cont'd)

The development of flood forecasting and contingency plans may be effective in reducing flood damages, however, considering how quickly, flood waters rise in Placentia, it is more likely that these would be implemented after the flood has started. These should be designed, therefore, to handle a flood situation which has started rather than to prepare for a specific event.

Option (q) - Property Acquisition or Zoning

Eliminating development in the area of flooding is undoubtly the best way to reduce flood damage to buildings and property. However, in Placentia a large number of existing buildings and properties are in the potential flood zone. Approximately 165 houses and 13 businesses are within the 1 in 100 year flood zone adjacent to the Arms. The buildings would have to be purchased at a price, probably well above their true value, to enable the displaced residents to build and resettle elsewhere. The buildings would then have to be demolished and the water, sewer and hydro services in the flood zone disconnected from the remaining system and, where necessary, removed.

Available land for development of new housing in the area of Placentia is scarce and displaced residents would have to resettle at some distance from their original home. This would undoubtly cause considerable social disruption and the high cost associated with this option, together with the loss in tax base for the Town, would not make this a practical flood control option.

Zoning of land within the flood zone to eliminate future construction or to regulate the type of construction would prevent an increase in flood damages for both the 1 in 20 and 1 in 100 year events. Considering the shortage of developable land in Placentia, the latter would be more desirable. In other words development within the flood zone should not be eliminated but controlled so that future flood damage is eliminated.

In particular, the undeveloped land adjacent to Southeast Arm could be developed provided measures are taken during construction of individual buildings to eliminate potential flood damage. This could be accomplished by requirements for land fill above the 1 in 100 year flood level and that no basements be allowed in new buildings. Considering that the depth of water over this land during a 1 in 100 year flood is relatively shallow, this seems to be a practical option although not a flood control measure as such.

7.4.2 Options (Cont'd)

Option (q) - Property Acquisition or Zoning (Cont'd)

Similarly, any undeveloped land in the area between Beach Road and Blockhouse Road/High Road where potential flooding, as a result of waves overtopping the beach to the west, exists should be zoned to control the type of construction.

7.5 ALTERNATIVE FLOOD CONTROL MEASURES

Several options for flood control were described in the previous section. Not all of the options could be considered as effective flood control measures. Alternative flood control measures were developed for each geographical region, comprising one or more of the options described above. The alternatives for each particular geographic region are described below:

Region 1 (Bordering on Swan Arm and The Narrows)

Alternative	Options Included	Description
1A	(a)	Raise/Floodproof Buildings
1B	(h)+(k)	Steel Sheet-Pile Wall Along Narrows + Wave Wall
1C	(i)+(k)	Raise Riverside Drive + Wave Wall.

Alternative 1A

This alternative which was described in the previous section as Option (a) is not a flood control measure as such. It would simply reduce property damage and business losses by the fact that the buildings would be raised above the flood level. It would not reduce the extent of flooding or improve access to flooded properties; nor would it reduce business losses due to lost sales, since access to the business properties flooded would be interrupted. Access to service facilities such as the hospital would be interrupted for the duration of the flooding and utility and transportation damages would still be incurred. It is estimated that this alternative would result in approximately 50 percent damage reduction.

Alternative 1A (Cont'd)

To estimate the cost of raising and/or floodproofing all the houses and buildings in the region, which are prone to flood damage, a sampling survey was made of building types and elevations. The following Table summarizes the estimated costs of this alternative to raise and/or floodproof all the structures within the area to protect them from the 1 in 20 year and 1 in 100 year flood levels. The costs are based on an average cost per structure of $5000 to raise and/or floodproof it (based on cost estimates prepared by Newfoundland and Labrador Housing Corporation for work currently in progress).

Estimated Costs - Alternative 1A

	Number of Buildings Affected			
Flood Event	Residential	Commercial	Total	Cost
1 in 20 year	145	6	151	$870,000
1 in 100 year	147	11	158	$910,000

Alternative 1B

This alternative would include construction in two areas to contain high water levels and prevent the flooding of the Town of Placentia in Region 1. It comprises two of the options previously described, viz.

(h) Construction of a steel sheet-pile wall along the Narrows, and

(k) Construction of a 300 metre extension to the existing wave wall on the beach to the west of Placentia.

Construction of the sheet pile-wall, in conjunction with raising part of Swan's Road, would provide closure of the low spots bordering Swan Arm and The Narrows, thereby eliminating high water levels from the Town in this area. To ensure 100 percent reduction in flood damage in this region for the 1 in 100 year flood event, it would be necessary to construct a wave wall extension on the beach to the west. Without this wave wall it is estimated that the 1 in 100 year waves would overtop the beach. This would result in water flowing across the Town, as it did on January 16, 1982, and ponding in low areas of the Town. The sheet pile wall would prevent this water from flowing directly into Swan Arm and The Narrows and would therefore

Alternative 1B (Cont'd)

result in increased ponding of water in the Town. Without the wave wall it is estimated that, with 1 in 100 year waves overtopping the beach, water would pond to an average depth of about one metre over a small area of the Town.

Prior to construction of the sheet-pile wall along The Narrows, detailed subsurface investigation would be required to determine the type of subsurface material and the permeability of this material along the length of the wall, as this information would influence the design depth of the wall.

The elevation of the top of the sheet-pile wall would be 2.17 m and 2.34 m Geodetic for the 1 in 20 year and 1 in 100 year recurring floods, respectively, providing a free-board of 0.3 metre in each case. At the north end of the wall, the existing Aylward's property would be removed and the wall extended through this area to tie into the high ground at the south approach to the Gut Bridge. Local infilling and ramping would be required in this area to permit vehicular access over the wall to existing properties. At the south end of the wall, around Swan Arm, the wall would tie into Swan's Road which would be raised over the section between Riverside Drive South and Cemetary Road. Here, again, local infilling and ramping would be required to permit vehicular access to the small boat slipway and adjacent properties.

During construction of the wall, the existing timber crib wall and boardwalk and the pavement on Riverside Drive would essentially be destroyed, to permit installation of wall tie-backs. For cost estimating it has been assumed that the boardwalk would not be replaced but the dry side of the wall would be backfilled with gravel and that Riverside Drive would be completely resurfaced.

Construction of the wave wall extension on the beach to the west of the Town of Placentia would be straight forward and similar to that used in the existing wall. The wall would be constructed to the same top elevation as the existing wall and backfilled with rock.

The estimated costs of this alternative are summarized in the following table and the detailed estimate is given in Appendix IV.

Alternative 1B (Cont'd)

Estimated Costs - Alternative 1B

Flood Event	Sheet-Pile Wall (Narrows)	Wave Wall (Beach)	Total
1 in 20 year	$3,504,000	---	$3,504,000
1 in 100 year	3,558,000	520,000	$4,078,000

Alternative 1C

This alternative is identical to Alternative 1B except that, instead of the steel sheet-pile wall along The Narrows, Riverside Drive North and South would be raised to create a dyke along The Narrows to prevent flooding of the east side of the Town of Placentia (Region 1).

Alternative 1C comprises the following two options previously described, viz:

(i) Raise Riverside Drive and Swans Road, and

(k) Construction of a 300 metre extension to the existing wave wall on the beach to the west of Placentia.

Similar to the sheet-pile wall, the raised road would be tied into high ground at the south approach to the Gut Bridge at one end and into the raised section of Swan's Road at the other end. The elevation of the raised road would be the same as the elevation of the top of the sheet-pile wall in Alternative 1B, ie. 2.17 m and 2.34 m Geodetic for the 1 in 20 year and 1 in 100 year recurring floods, respectively.

Since the raised road would be approximatley 0.5 - 1.3 metres above adjacent roads and properties, local ramping would be required to facilitate vehicular access onto these. Near the northern end of Riverside Drive North, in the area of Aylward's property, there are five or six houses very close to the road. These houses and respective driveways may have to be raised because of the elevation differential between the original ground and the raised road. The cost estimates which follow include costs for

Alternative 1C (Cont'd)

raising these properties. It is not expected that it would be necessary to remove the Aylward's property for this alternative since Riverside Drive is on the landward side of the property and the property should not interfere with the raising of the road.

The estimated costs of this alternative are summarized in the following table and the detailed estimate is given in Appendix IV.

Estimated Costs - Alternative 1C

Flood Event	Riverside Drive (Narrows)	Wave Wall (Beach)	Total
1 in 20 years	$535,000	---	$ 535,000
1 in 100 years	560,000	520,000	$1,080,000

Region 2 (Jerseyside - Adjacent to the Lift Bridge)

Alternative	Options Included	Description
2A	(a)	Raise/Floodproof Buildings
2B	(n) + (o)	Raise Wave Wall West of Jerseyside + Raise breastwork east of Jerseyside.

Alternative 2A

This alternative is similar to Alternative 1A for Region 1 and was described in the previous section as Option (a). As for Region 1, it would reduce flood damage by only about 50 percent.

The estimated costs of this alternative are summarized in the following table:

Estimated Costs - Alternative 2A

Flood Event	No. of Buildings Affected: Residential	Commercial	Total	Cost
1 in 20 year	2	0	2	$12,000
1 in 100 year	7	2	9	$52,000

Alternative 2B

This alternative would include construction in two areas to contain high water levels and prevent flooding of that area of the Town of Jerseyside adjacent to and north of the Sir Ambrose Shea lift bridge. It comprises two of the options previously described, viz:

(n) Raising the existing wavewall to the west of Jerseyside, and

(o) Raising the existing breastwork to the east of Jerseyside.

Raising the existing wavewall west of Jerseyside by approximately one metre would prevent the 1 in 100 year waves from overtopping the beach in this area and thence flooding the ballpark and buildings in this area, as happened on January 16, 1982.

To ensure 100 percent reduction in flood damage in this region, it would be necessary to also raise the existing breastwork east of Jerseyside. This would prevent waves on Northeast Arm, occuring with a high water level in the Arm, from travelling into the area of the buildings along the shoreline of the Arm. The existing breastwork is assumed to be previous and is not expected to eliminate water completely from this area. However, high water level alone has apparently not caused flood damage in this area. Any damage has apparently resulted from wave action on the high water level. The top of the new construction would be 2.85 m and 3.15 m Geodetic for the 1 in 20 year and 1 in 100 year recurring floods, respectively, providing a freeboard of 1.0 metre in each case.

The estimated costs of this alternative are summarized in the following table and the detailed estimate is given in Appendix IV.

Estimated Costs - Alternative 2B

Flood Event	Wave Wall	Breastwork	Total
1 in 20 year	$ ---	$165,000	$165,000
1 in 100 year	$190,000	$165,000	$355,000

7.5 ALTERNATIVE FLOOD CONTROL MEASURES (Cont'd)

Region 3 (Bordering on Southeast Arm)

Alternative	Options Included	Description
3A	(a)	Raise/Floodproof Buildings
3B	(j) + (k)	Dyke in Southeast Arm + Wave Wall.

Alternative 3A

This alternative is similar to Alternative 1A for Region 1 and was described in the previous section as Option (a). As for Region 1, it would reduce flood damage by only about 50 percent.

The estimated costs of this alternative are summarized in the following table:

Estimated Costs - Alternative 3A

	No. of Buildings Affected			
Flood Event	Residential	Commercial	Total	Cost
1 in 20 year	2	0	2	$ 12,000
1 in 100 year	11	0	11	$ 63,000

Alternative 3B

This alternative would include construction in two areas to contain high water levels and prevent flooding of the undeveloped land and a small number of buildings in the Southern area of the Town of Placentia bordering Southeast Arm. It comprises two of the options previously described, viz:

(j) Construction of a dyke along the shoreline of Southeast Arm, and

(k) Construction of a 300 metre extension to the existing wave wall on the beach to the west of Placentia.

7.5 ALTERNATIVE FLOOD CONTROL MEASURES (Cont'd)

Alternative 3B (Cont'd)

The dyke would be constructed along the shoreline of Southeast Arm from the blockhouse to Dixon Hill. It would be constructed of compacted gravel placed on the original ground. Depending on the permeability of the insitu underlying materials, a cut-off trench backfilled with compacted sand or gravel may be required to reduce the perculation of water under the dyke.

It may be possible to utilize local material for construction of the dyke by bulldozing it into a mound and shaping and compacting to form the dyke. For cost estimating, because of the lack of information on the quality of insitu materials, and to provide a conservative estimate, it was assumed that a cut-off trench would be required beneath the dyke and that gravel would be imported from a borrow pit within a radius of 3 km for the dyke construction.

The crest elevation of the dyke would be 2.6 m and 2.75 m Geodetic for the 1 in 20 year and the 1 in 100 year recurring floods, respectively, on Southeast Arm, providing a freeboard of 1 metre in each case.

As described under Alternative 1B, construction of the wave wall extension on the beach to the west of the Town of Placentia would be similar to that used in the existing wall. The wall would be constructed for the 1 in 100 year event, to the same top elevation as the existing wall and backfilled with rock.

The estimated costs of this alternative are summarized in the following table and the detailed estimate is given in Appendix IV.

Estimated Cost - Alternative 3B

Flood Event	Dyke (S.E. Arm)	Wave Wall (Beach)	Total
1 in 20 year	\$ 183,000	---	\$ 183,000
1 in 100 year	\$ 183,000	\$ 520,000	\$ 703,000

7.5 ALTERNATIVE FLOOD CONTROL MEASURES (Cont'd)

Region 4 - (Bordering Placentia Road)

Alternative	Options Included	Description
A	(a)	Raise/Floodproof Buildings
B	(k)	Wave Wall

Alternative 4A

This alternative is similar to Alternative 1A for Region 1 and was described in the previous section as Option (a). As for Region 1, it will reduce flood damage by only about 50 percent.

The estimated costs of this alternative are summarized in the following table:

Estimated Costs - Alternative 4A

	No. of Buildings Affected			
Flood Event	Residential	Commerical	Total	Cost
1 in 20 year	---	---	---	---
1 in 100 year	20	---	20	$114,000

Alternative 4B

As described under Alternative 1B, construction of the wave wall extension on the beach to the west of the Town of Placentia would be similar to that used in the existing wall. The wall would be constructed for the 1 in 100 year event, to the same top elevation as the existing wall and backfilled with rock.

The estimated costs of this alternative are summarized in the following table and the detailed estimate is given in Appendix IV.

Estimated Costs - Alternative 4B

Flood Event	Wave Wall (Beach)	Total
1 in 20 year	---	---
1 in 100 year	$520,000	$520,000

PART EIGHT

PRELIMINARY ECONOMIC ANALYSIS

8.1 GENERAL

In order to determine the economic significance of the alternative flood control measures as discussed in Part 7, each of the alternatives was assessed using benefit-cost analysis. It is important to note that this is a preliminary assessment and once the preferred alternative(s) is finalized, a more definitive investigation will be undertaken of all economic parameters for inclusion in the final report.

8.2 METHOD OF ANALYSIS

The metohd of analysis used was based on references contained in the Guidelines for the Benefit-Cost Analysis of Flood Damage Reduction Projects (Inland Waters Directorate, 1984). These references were: Joint Task Force on Water Conservation Projects in Southern Ontario, Volumes 1-3 (Acres Limited, 1968) and Benefit-Cost Analysis Guide (Treasury Board Secretariat, 1976).

Each of the alternative flood control measures was assessed based on its present value (PV) of benefits and costs, net present value (NPV) and benefit-cost ratio.

The present value of cost and benefits in any given year are found by multiplying the prospective cost or benefit by the following discount factor:

$$\frac{1}{(1 + i)^j}$$

where: j is the index of the year concerned (range $j=0$ to n)
i is the social discount rate

The discounted values of all project costs and benefits are summed to give the total present values of all project costs and benefits. The net present value is then found by subtracting the present value of project costs from the present value of project benefits. Similiarly, the benefit-cost ratio is calculated by dividing the projects present value of benefits by its present value of costs. The following are formulae (Treasury Board Secretariat, 1976) used to calculate present value benefits, present value costs, net present value and benefit-cost ratio.

8.2 METHOD OF ANAYSIS (Cont'd)

Present value of benefits (b):

$$\sum_{j=o}^{n} \frac{bj}{(1 + i)^j}$$

Present value of costs (c):

$$\sum_{j=o}^{n} \frac{cj}{(1 + i)^j}$$

Net present value:

$$\sum_{j=o}^{n} \frac{bj}{(1 + i)^j} - \sum_{j=o}^{n} \frac{cj}{(1 + i)^j}$$

Benefit-cost ratio:

$$\frac{\sum_{j=o}^{n} \frac{bj}{(1+i)^j}}{\sum_{j=o}^{n} \frac{cj}{(1 + i)^j}}$$

For the case at hand, benefits to be derived from each alternative flood control measure were defined as the dollar value of all tangible flood damages which would be avoided (or reduced) based on the 1 in 20 year and 1 in 100 year flood events. Using the probability of occurrence, estimated average annual flood damage was calculated for each of the four geographic regions (Table 7.1.6). These benefits were then discounted at the 10 percent level over the economic life of the flood control measure to arrive at the present value of benefits.

Project costs were estimated for each of the alternative flood control measures (ie. 1 in 20 and 1 in 100 year events) and included a 15 percent contingency factor. Appendix IV provides a breakdown of costs for each of the flood control measures. As with the calculation of present value of project benefits, present value of costs was found by discounting the project costs over the life of the project at the 10 percent level.

8.2 METHOD OF ANALYSIS (Cont'd)

In addition to the tangible benefits and costs associated with each of the alternative flood control measures there are important intangible benefits and costs. The intangible benefits which are likely to be gained are:

i) Improved safety for local residents,
ii) Reduced health risk associated with the backing up of septic systems, cess pools, etc;
iii) Improved accessibility to coastal land areas,
iv) Improved quality of living standards for residents,
v) Reduced inconvenience associated with flood events,
vi) Employment for residents during construction phases of the project,
vii) Improved general economic development, and
viii) Reduced erosion and sediment deposition problems.

The intangible costs associated with implementation of flood control measures are:

i) The inconvenience to local residents during construction phases of the flood control measure(s), and

ii) The reduced asthetic quality of the area which may be associated with the construction of wave walls, sheet pile walls, dykes and raising of roads.

8.3 RESULTS AND DISCUSSION

Table 8.3.1 presents a summary of the benefit-cost analysis for each of the alternative flood control measures for each of the four geographic regions. From this table it can be seen that from a purely economic standpoint the most important region to protect is Region 1, the older area of the Town of Placentia bordering on Swan Arm and The Narrows. As was shown in the section on previous damages, this area is frequently inundated with flood water and recent damages have been in the order of $500,000. In the remaining three areas flooding has been a less frequent occurrence and associated damages has been much less.

If it is assumed that a flood control measure is acceptable if its net present value is positive and its benefit--cost ratio is greater than unity, then Alternative 1C for Region 1 (Raise Riverside Drive and Construct a wave wall along the beach) is the only viable alternative. For this alternative, net present values were found to be $959,000 (Table 8.3.1) for protection from the 1 in 20 year flood event and $444,000 (Table 8.3.1) for protection from the 1 in 100 year flood event. Associated benefit-cost ratios were found to be 2.72 and 1.39 for the 1 in 20 year and 1 in 100 year events respectively.

8.3 RESULTS AND DISCUSSIONS (Cont'd)

If consideration is given to the intangible benefits, as previously discussed, then Alternative 1A for Region 1 (Raise/flood proof buildings) would be an acceptable alternative, but, it is important to note that implementation of this flood control measure would only reduce flood damages by approximately 50 percent, since only building and contents would be protected. Damages associated with transportation, utilities and indirect damage to property and people would still occur.

PART NINE

PHASE I - CONCLUSIONS AND RECOMMENDATIONS

9.1 CONCLUSIONS

Based on investigations into the history of flooding in the Placentia area and on the results of the oceanographic, hydrologic, hydraulic and economic studies, which were carried out in Phase I of this study, the following conclusions can be stated:

i) Flooding in the Placentia area has occured from two sources, namely:

- high water levels in the Arms, and
- waves overtopping the beach to the west.

Most flooding in the area occurs from the first source (ie. high water levels in the Arms). It was reported that the January 16, 1982 event was the only time that flooding has occurred when waves overtopped the beach on the west side of Placentia and Jerseyside (at least in the memory of an 83 year old resident).

ii) Most flood damage occurs in the Town of Placentia with only minor damage being experienced in the Town of Jerseyside. Of the four geographical regions identified in the study area, Region 1 (bordering Swan Arm and The Narrows) incurs the greatest damage.

iii) Flooding of the Town of Placentia will occur when the water levels in Northeast Arm and Swan Arm exceed 1.6 m and 1.0 m respectively. Flooding of the low lying land bordering Southeast Arm will occur when the water level in Southeast Arm exceeds 0.8 m. In this area which is the southeast end of the Town of Placentia, the Town will flood when the water level is approximately 1.5 m. Flooding of Jerseyside from Northeast Arm will occur when the water level in Northeast Arm exceeds about 1.7 m.

iv) Flooding of Placentia from the Arms is a result of unusually high water levels in the Arms. This condition is caused by extremely high water levels in Placentia Road which result from the combined effects of tide and storm surge. The high water levels in Placentia Road force water through The Gut into Northeast Arm and thence into Swan Arm and Southeast Arm. The throttling effect or constrictions provided by The Gut, The Narrows and MacDonald Gut result in head losses and therefore an attenuation of water levels throughout the system.

9.1 CONCLUSIONS (Cont'd)

v) Flooding of the Town from the Arms would be aggrevated by strong winds blowing across the Arms toward the Town during high water level events. A sensitivity analysis showed that 90 km/hour winds would increase the flood water levels (adjacent to the Towns) in Northeast Arm, Swan Arm and Southeast Arm by 0.13 m, 0.04 m and 0.13 m respectively.

vi) A sensitivity analysis showed that freshwater inflow into the Arms has a small effect on water levels in the Arms. The effect on Southeast Arm was found to be greater than that on Northeast Arm but high inflow is not sufficient to cause flooding without an extreme tide and storm surge event.

vii) A sensitivity analysis of the physical parameters of the system showed that the water levels in the Arms were most sensitive to variations in Manning's "n" and were less sensitive to variations in flow area and elimination of off channel storage.

viii) The 1 in 20 year and 1 in 100 year recommended flood contours for Northeast Arm, Swan Arm and Southeast Arm are as follows:

	Northeast Arm	Swan Arm	Southeast Arm
1 in 20 yr.	1.87 m	1.87 m	1.58 m
1 in 100 yr.	2.04 m	2.04 m	1.75 m

ix) The present elevation of the top of the beach on the west side of Placentia is sufficient to prevent waves overtopping due to run-up on the beach. However, because the crest of the beach is built up of relatively fine rockfill (from the dredging operation in 1982), continuous pounding by waves during storm conditions concurrent with a high water level in Placentia Road, could result in erosion of the beach crest and subsequent overtopping.

x) Flood control options which attempt to regulate flow through the "control sections" were generally found to be either ineffective (Options b, e, f and g) or too expensive (Options c and d). Instead, options to keep high water levels out of Placentia (Options h, i, j, k, l and m) were found to be more practical. Of these options (l) and (m) were found to be too expensive. Options to keep high water levels out of Jerseyside (Options n & o) were considered practical.

9.1 CONCLUSIONS (Cont'd)

xi) Option (a) has the least capital cost for each region but will not eliminate flooding or inconvenience due to flooding. It will only reduce the value of property damage.

xii) Option (p) might help reduce flood damage if sufficiently early warning of an impending flood can be given. It will not, however, eliminate flood damage. It may be more practical to place the Towns of Placentia and Jerseyside on a state of alert during the flood prone period.

Contingency planning measures could be developed to help reduce flood damages but would be implemented to handle a flood situation which is in progress.

xiii) Option (q) would be an effective way to reduce future flood damage. Property acquisition would not be practical on a large scale because of the social implications. Zoning would be a good long term measure to reduce future damage by regulating the type of construction in areas prone to flooding.

xiv) To provide 100 percent reduction in flood damage for the 1 in 100 year flood event in each region, to maximize the net present value, the following alternative flood control measures would have to be implemented:

Region	Alternative	Benefit-Cost Ratio
1	1C	1.39
2	2B	0.01
3	3B	0.01
4	4B	0.01

Of these, only alternative 1C for Region 1 is considered economically acceptable according to the Inland Waters Directorate, Guidelines for the Benefit-Cost Analysis of Flood Damage Reduction Projects, which states:

"As a rule, a preliminary engineering design should not be done unless a project shows a benefit-cost ratio of 0.5 or more, and is feasible in other aspects as well".

The economic significance of intangible benefits and costs have not been included.

xv) Approximately 50 percent reduction in flood damage could be provided by raising and/or floodproofing buildings in the flood zone of each region. The following summarizes the benefit-cost ratio of these alternatives:

9.1 CONCLUSIONS (Cont'd)

xv) (Cont'd)

Region	Alternative	Benefit-Cost Ratio
1	1A	0.87
2	2A	0.02
3	3A	0.04
4	4A	0.03

Of these, only Alternative 1A for Region 1 is economically viable and is less viable than Alternative 1C as noted in (xiv) above. It is noted, however, that Alternatives 2A, 3A and 4A are not economically viable but more viable than Alternatives 2B, 3B and 4B for Regions 2, 3, and 4 respectively, as noted in (xiv) above.

9.2 RECOMMENDATIONS

Based on preliminary economic analysis, it is recommended that Alternative 1C for Region 1 (Raise Riverside Drive and extend the Wave Wall along the beach to the west of Placentia) be looked at in greater detail in Phase II of this study.

It is also recommended that, in addition to Alternative 1C, selective raising of houses in each of the remaining three regions be looked at in more detail.

If structural flood control meausures are not constructed, zoning by-laws and building regulations should be developed by the Department of Municipal Affairs in conjunction with the Towns of Placentia and Jerseyside.

PART TEN

FINAL ECONOMIC ANALYSIS

10.1 GENERAL

Parts one to nine of this report represent Phase I of this study which, along with the determination of flood risk contours, presented several alternative flood control measures and a preliminary economic analysis of these alternatives. This Part Ten represents Phase II of the study and presents a detailed economic analysis of selected alternatives which have been identified as a means of reducing or eliminating flood damage in the Placentia area. From the preliminary investigations the following alternatives have been selected for more detailed final analysis:

For the 1 in 20 year flood event:

Alternative 1:	Raise Riverside Drive
Alternative 2:	Raise Selected Buildings in Regions 2 and 3
Alternative 3:	Alternatives 1 and 2 combined

For the 1 in 100 year flood event:

Alternative 1:	Raise Riverside Drive and Construct Wave Wall on beach west of Placentia
Alternative 2:	Raise Selected Buildings in Regions 2 and 3
Alternative 3:	Alternatives 1 and 2 combined

For each of these alternatives the final assessment herein contains: an analysis of average annual damages which have been identified as the principle project tangible benefits; an assessment of each alternative based on the net present value and benefit/cost ratio criterion. As well, a sensitivity analysis is made on the discount rate using social discount rates of 7, 10 and 13 percent. Finally, the intangible benefits resulting from the implementation of the recommended alternatives are discussed.

The non-structural alternatives are also discussed, these apply to all regions for both the 1 in 20 and 1 in 100 year flooding events. The non-structural alternatives include:

1) Zoning and Building Regulations
2) Contingency Planning
3) Monitoring/Flood Forecasting

An evaluation of the effectiveness of existing structures in providing flood control and a detailed study into the sensitivity of water levels were completed in Phase I (Sections 7 and 6, respectively). No further analysis is required into effectiveness of existing structures or water level sensitivity for the completion of Phase II and is therefore, not included in this section.

10.2 FLOOD CONTROL MEASURES

Following a review by the Technical Committee of the alternative flood control measures presented in Phase I of this study, a more detailed analysis has been preformed on the recommended alternatives. A description of the flood control measures is presented here for measures which provide protection against the 1 in 20 and 1 in 100 year flooding events. A review of the cost estimates has been made and any price adjustment has been included in the final economic analysis. All estimates include the cost of pre-engineering studies, site investigations, design costs, inspection services during construction, equipment, materials and labour and contractor mobilization and demobilization at the site.

The non-structural measures do not provide direct protection against flooding, but help reduce potential damage from any future floods. The non-structural measures can be implemented immediately to reduce flood damage until the recommended structural measures are completed. Implementation of zoning and building regulations will also reduce or eliminate future damage to any construction completed after these measures are implemented.

The costs associated with each alternative flood control measure and the region to which they apply are summarized in Table 10.2.1.

10.2.1 Structural Measures

Region 1

1 in 20 Year Flood Event

To eliminate flood damage in Region 1 for the 1 in 20 year flood event, it is recommended that Riverside Drive and Swan Road be raised to an elevation of 2.17 m to provide a dyke along The Narrows against high water levels in Northeast Arm and Swan Arm as described under Alternative 1C in Phase I of this study (Section 7.5). The extension to the wave wall on the beach west of Placentia, as described under Alternative 1C, is not required for this flood event.

Sub-surface site investigations would be required prior to construction to determine the permeability of the insitu material. For estimating purposes, it was assumed a cut-off trench would be necessary to reduce the permeability of the insitu material. Raising of the road surface will require ramping of adjacent road and driveways and necessitate the raising of 5 houses and the associated properties because of their close proximity to Riverside Drive.

TABLE 10.2.1

COST OF RECOMMENDED STRUCTURAL FLOOD CONTROL MEASURES*

1 in 20 Year Flood Event

Alternative	Region	Action	Cost
1	1	Raise Road and 5 Buildings	$ 695,000
2	2 & 3	Raise 4 Buildings	$ 26,000
	4	No action required	---
3	1,2,3 & 4		$ 721,000

1 in 100 Year Flood Event

Alternative	Region	Action	Cost
1	1	Raise Road, 5 Buildings and Construct Wave Wall	$ 1,249,000
2	2 & 3	Raise 20 Buildings	$ 116,500
	4	No Action required	---
3	1,2,3 & 4		$ 1,365,000

* Note: Details of cost estimates are given in Appendix V.

10.2.1 Structural Measures (Cont'd)

1 in 20 Year Event (Cont'd)

Details of the construction required to raise Riverside Drive and the location of the properties to be raised are shown on Figure 10.2.1.

1 in 100 Year Event

To eliminate flood damage in Region 1 for the 1 in 100 year flood event, it is recommended that Riverside Drive and Swan Arm be raised to an elevation of 2.34 m geodetic to provide a dyke along The Narrows against high water levels in Northeast Arm and Swan Arm and to construct an extension to the wave wall along the beach to the west of Placentia as described under alternative 1C in Phase I of this study (Section 7.5). Without the wave wall, it is estimated that the 1 in 100 year waves would overtop the beach, and flow across the town, causing water to pond to as much as one metre deep in the lowest areas of Region 1.

Subsurface investigations would also be required along Riverside Drive for this alternative to determine the permeability of insitu materials. This alternative will also necessitate the raising of 5 houses and their associated properties to permit access onto the raised road.

Regions 2 and 3

1 in 20 Year Event

To reduce flood damages in Regions 2 and 3 for the 1 in 20 year flood event it is recommended that houses be raised and/or flood proofed. As described under Alternatives 2A and 3A in Phase I of this study (Section 7.5), two houses in Region 2 and two houses in Region 3 would have to be raised. This would not result in a complete elimination of flood damage to these regions, as explained in Section 7.4.2, option (a), since a flood would still impede access to the buildings and cause utility, transportation and indirect damages.

1 in 100 Year Event

To reduce flood damages in Regions 2 and 3 for the 1 in 100 year flood event, it is recommended that houses be raised and/or flood proofed. This would involve a total of 20 buildings within the regions, 7 houses and 2 commercial buildings in Region 2 and 11 houses in Region 3. As described for the 1 in 20 year event, this would not result in the complete elimination of damage in the regions due to flooding, but would protect the buildings and their contents.

10.2.1 Structural Measures (Cont'd)

Region 4

1 in 20 Year Event

There is no action required to reduce or eliminate flood damage in Region 4 for the 1 in 20 year event. This region is located above the level which could be affected by high water levels in Northeast, Swan or Southeast Arms, and with the existing beach profile the 1 in 20 year waves will not overtop the beach.

1 in 100 Year Event

Elimination of flood damage in Region 4, for the 1 in 100 year flood event, would necessitate construction of an extension to the existing wave wall on the beach to the west of Placentia. This is not economically feasible when considered for Region 4 alone. However, this wall is necessary for elimination of flood damage in Region 1 and its construction for Region 1 will provide the necessary protection for Region 4.

Regions 1, 2, 3 and 4, Combined

1 in 20 Year Event

The recommended flood control measures discussed above for each region eliminate or reduce flood damages in each particular region. To reduce flood damages in the entire study area for the 1 in 20 year flood event, all the measures recommended for each region for this event would be necessary. That is:

Region 1	- Raise Riverside Drive and 5 Buildings
Regions 2 and 3	- Raise 4 Buildings
Region 4	- no action required.

It is not possible to eliminate flood damages in Regions 2 and 3 by the raising of buildings only since flooding will still occur which will result in utility, transportation and indirect damages.

1 in 100 Year Event

To reduce flood damages in the entire study area for the 1 in 100 year event, all the measures recommended for each region for this event would be necessary. That is:

Region 1	- Raise Riverside Drive, 5 buildings and construct wave wall,
Regions 2 and 3	- Raise 20 Buildings
Region 4	- No action required.

10.2.1 Structural Measures (Cont'd)

1 in 100 Year Event (Cont'd)

As for the 1 in 20 year event, it is not possible to eliminate flood damages in Regions 2 and 3 by the raising of buildings only because of the utility, transportation and indirect damages that will still occur.

10.2.2 Non-Structural Measures

A reduction in flood damages can be realized by the implementation of non-structural methods. Since reduction of flood damage in Regions 2 and 3 involves flood proofing of buildings, it is imperative that building and zoning regulations be developed so that any future construction includes flood proofing if the construction is within the flood risk area. As described in Phase I (Section 7.4.2, Option (q)) zoning would not prohibit construction, but ensure that any new construction follows regulations for flood proofing so that an increase in damage would not result.

The area below the 1 in 100 year flood contour (which includes the area below the 1 in 20 year flood contour) should be designated the flood risk area. Since the nature of flooding in Regions 2 and 3 for the 1 in 20 year and 1 in 100 year flood events is such that land is inundated by slow rising flood water, without high flow velocity, and with a short duration, construction in this area can be permitted if the regulations are followed. The building and zoning regulations should require fill in the development area to be above the 1 in 100 year flood level and require the elimination of basements or strengthening and sealing of basement walls to prevent cracking and leakage under hydrostatic pressure.

A reduction in flood damages can be realized in the study area by the implementation of early warning (flood forecasting) and contingency planning measures. As explained in Phase I (Section 7.4.2, Option (q)) a proper monitoring program may allow a warning of flood danger. Due to the nature of the flooding at Placentia, this warning may be as little as a couple of hours. It is important, therefore, that a contingency plan for action by the Town, in the event of a flood, be put in place. Such a plan should follow the guidelines of the Emergency Measures Organization and should be approved by that Organization.

10.2.2 <u>Non-Structural Measures (Cont'd)</u>

A proper monitoring program would require the installation of a water level recording gauge in Placentia Road and monitoring of meteorogical conditions in the vicinity. An observer would look for conditions which could lead to extreme water levels, such as low pressure systems in conjunction with sustained high onshore winds and spring tides. This would not require a separate weather station; the data collected at the Argentia station should be sufficient, provided it is immediately available to the observer.

The monitoring program should be conducted annually by a trained and responsible person during the flood prone period mid-December to the end of January. Ideally, the monitoring would begin early in December to ensure all systems are operational by mid-December.

The following should be included in the monitoring program:

- Monitor regional weather patterns for conditions which may cause storm surge in Placentia Bay.
- Monitor local weather conditions for low pressure systems and sustained onshore winds.
- Monitor weather forecasts for changes or deterioration of weather at flood risk time.
- Monitor tide tables for time and date of highest tides predicted.
- Monitor tidal variation in Placentia Road on a real time basis to note difference between predicted and recorded levels - this will provide indication of surge.
- Monitor the water level in Swan Arm - a very high level may indicate possible flood at next high tide.
- Monitor precipitation - extreme precipitation in conjuction with extreme tidal levels may worsen flood levels in Southeast Arm.

In addition to the monitoring program during the flood prone period, semi-annual inspections should be made of Placentia beach (to the west of Placentia) and annual inspections made of the Gut and Narrows. The following should be included in these inspections:

- Survey beach profile at particular locations - this will identify profile changes which could result in extreme runup and consequent overtopping of the beach with the occurence of extreme waves. It will identify the need for remedial work to prevent overtopping.
- Inspect beach condition for signs of erosion or deterioration of beach and undercutting of wave wall which could result in failure of the wave wall.

10.2.2 Non-Structural Measures (Cont'd)

- Monitor depth of flow at The Gut and The Narrows for excessive sedimentation or erosion which may cause a change in present throttling effect on water levels.

10.3 AVERAGE ANNUAL DAMAGE ASSESSMENT

Following a review of the method of analysis used for assessment of average annual damages in Phase I, the Technical Committee requested that a method based on a study by Day, Bugliarello et al, (1969) be used for Phase II.

The average annual damages for each of the flood regions has been reassessed using the methodology discussed by Day, Bugliarello et al, (1969). In their method, expected annual loss can be calculated using the following equation:

$$E\,(D) = \sum_{i - 1}^{n} p_i D_i$$

where:

p_i = probability of a flood occuring between stage;

D_i = community damage associated with flood level at the particular stage. D_i is a function of the warning time, type of action, and response to warning;

E (D)= expected annual loss;

n = number of contour intervals required to approach flood plain limit. It also represents a flood recurrence interval.

Based on this approach, the expected annual damages for each of the regions, as presented in Tables 10.3.1 to 10.3.4, was found to be \$225,911, \$664, and \$1015 for Regions 1, 2 and 3 respectively for the 1 in 20 year flood event. Region 4 has experienced damages resulting from a rare flooding situation which, at best, is believed to occur 1 in 100 years, therefore the 1 in 20 year event is not applicable for this Region. For the 1 in 100 year event, expected average annual damage estimates were calculated to be \$293,212, \$4,038, \$1,454 and \$635 for Regions 1, 2, 3 and 4, respectively.

10.3 AVERAGE ANNUAL DAMAGE ASSESSMENT (Cont'd)

The damages can be considered as project benefits, given 100 percent reduction in flooding damages within the region. As was mentioned in the preliminary assessment (Section 7.1.1) no damages were identified within the agricultural and recreational sectors, this was verified in the final assessment. In addition to the tangible benefits resulting from flood damage reduction, several intangible benefits have been identified. These are addressed in the following section.

TABLE 10.3.1

STAGE - DAMAGE SUMMARY - REGION 1

Stage * (m)	Flood Probability, p	Flood Stage Probability, p_i	D_i ($)	$p_i D_i$ ($)	E(D) ($p_i D_i$)
0.9	1.00	---	8.0		
1.2	0.88	1.00-0.88 = 0.12	35,070	4,208	4,208
1.5	0.44	0.88-0.44 = 0.44	163,190	71,804	76,012
1.8	0.11	0.44-0.11 = 0.33	454,240	149,899	225,911
2.0	0.01	0.11-0.01 = 0.10	673,010	67,301	293,212

D_i = the flood damage due to water level at particular stage.

$p_i D_i$ = the damage associated with the flood probability between stages.

E(D) = the expected average annual damage.

* Stage means water level expressed in metres above Geodetic datum.

TABLE 10.3.2

STAGE - DAMAGE SUMMARY - REGION 2

Stage * (m)	Flood Probability, p	Flood Stage Probability, p_i	D_i ($)	p_i D_i ($)	E(D) ($p_i D_i$)
0.9	1.00	---	---	---	---
1.2	1.00	---	---	---	---
1.5	0.93	---	---	---	---
1.8	0.11	0.93-0.11 = 0.82	810	664	664
2.0	0.01	0.11-0.01 = 0.10	20,190	2,019	2,683

D_i = the flood damage due to water level at particular stage.

$p_i D_i$ = the damage associated with the flood probability between stages.

E(D) = the expected average annual damage.

* Stage means water level expressed in metres above Geodetic datum.

TABLE 10.3.3

STAGE - DAMAGE SUMMARY - REGION 3

Stage * (m)	Flood Probability, p	Flood Stage Probability, p_i	D_i (\$)	p_i D_i (\$)	E(D) ($p_i D_i$)
0.9	1.00	---	---	---	---
1.2	0.86	---	---	---	---
1.5	0.10	0.86-0.10 = 0.76	1,240	942	942
1.55	0.05	0.10-0.05 = 0.05	1,450	73	1,015
1.70	0.01	0.05-0.01 = 0.04	10,980	439	1,454

D_i = the flood damage due to water level at particular stage.

$p_i D_i$ = the damage associated with the flood probability between stage.

E(D) = the expected average annual damage.

* Stage means water level expressed in metres above Geodetic datum.

TABLE 10.3.4

SUMMARY OF EXPECTED AVERAGE ANNUAL DAMAGE

Region	Geographic Location	Expected Average Annual Damages ($) (p_iD_i)	
		1 : 20	1 : 100
1	The Narrows/Swan Arm	225,911	293,212
2	Jerseyside	664	2,683
3	Southeast Arm	1,015	1,454
4	Placentia Road	---	635*

* Region 4 damages are based on a total damage estimate of $63,500 and a flood probability of 0.01.

10.4 INTANGIBLE BENEFIT ASSESSMENT

The Guidelines for the Benefit-Cost Analysis of Flood Damage Reduction Projects (Inland Waters Directorate, 1984) states that intangible benefits are those which cannot be assigned a mometary value but that these should still be considered in the evaluation of flood damage reduction alternatives. To reiterate the findings of the preliminary assessment, the intangible benefits expected to arise as a result of implementing the flood control measures are:

i) improved safety for local residents,
ii) reduced health risk associated with the backing up of septic systems, cess pools, etc.,
iii) improved accessibility to coastal areas,
iv) improved quality of living standards for residents,
v) reduced inconvenience associated with flood events,
vi) employment for residents during construction phases of the project,
vii) improved general economic development, and
viii) reduced erosion and sediment deposition problems.

In addition, further investigation indicates that the marketability of homes within the flood prone area may be improved.

Each of these benefits is briefly discussed as follows:

Improved Safety

Safety elements such as electrical shock, drowning and miscellaneous injury related to falls, car accidents and fires are included in this benefit category. Although past flooding situations did not result in any of these mishaps, the possibility does exist that serious injury or death could occur related directly or indirectly to the flooding situation.

Reduced Health Risk

Several homes experienced septic system or cess pool back up. This was of serious concern to many of the area residents not only with regard to the physical damages incurred but more importantly with regard to possible health risk. Although not a major problem in the area, since much of the region is on a water and sewer system, this problem has been identified by local residents and is perceived as a potential health risk.

Improved Coastal Accessibility

This benefit relates to accessibility of local residents and tourists to the coastal areas during and, for a period, following flood events. Although this has not been identified as a serious problem it has been pointed out and is therefore noted.

Improved Quality of Living and Reduced Inconvenience

These benefits are closely associated. Many residents, including towns people, municipal officials and business people, identified inconveniences associated with the flood events. These included such things as:

- having to leave their homes - some elderly or sick residents had to be carried from their homes by friends, relatives or emergency personnel due to the high water levels,
- the time and inconvenience associated with cleaning up after flooding,
- the temporary loss or reduction of services resulting from the flood - included were: hydro service, telephone communication, shopping facilities, and municipal services,
- the reduced interest to purchase high quality items or material such as floor covering due to the potential for loss or damage to these items.

These conditions and perceptions were intensified during the December, 1983 flood events which occurred during the Christmas season; especially during the flood which occurred on Christmas Day.

Local Employment

Recent statistics place the unemployment in the area of Placentia in the order of 17% (Statistics Canada). The flood control measures which have been recommended would provide temporary employment in the area for the duration of construction by utilizing local labour, services and materials.

Improved General Economic Development

In general, with the risk of flooding reduced or eliminated the potential for general economic development should be improved. Present and future business interests would be encouraged to expand or locate in the area due to the increased availability of land and reduced threat of flood damage.

10.4 INTANGIBLE BENEFIT ASSESSMENT (Cont'd)

Reduced Erosion and Sediment Deposition Problems

Many residents complained of the erosion of land or, alternatively, the deposition of eroded material as the flood waters moved across the community. In general, this problem was manifested in the reduced asthetics of the area, clean up problems and associated inconvenience.

Increased Real Estate Marketability

During the field investigations several residents believed that property valves had fallen drastically as a result of the flooding. Discussions with real estate appraisers indicate that although the short term marketability of real estate in the flood prone area is reduced following a flood event, the long term property values do not decline based on flood risk alone. This finding is in agreement with that of Babcock and Mitchell (1980) who assessed property value changes in the Galt-Cambridge area of Southern Ontario. Although long term property values are not affected, the perception of reduced property values by local residents may result in a reduced resale value. Therefore, increased marketability of real estate in the area or even perception of increased real estate value would be a definite benefit. Due to the various factors involved and the complex nature of property value it is impossible to quantify this benefit to produce a tangible benefit estimate, hence, it is considered an intangible benefit.

10.5 COST ASSESSMENT

The final cost estimates for each of the alternatives are documented in Appendix V. The total estimated construction cost for each is given in Table 10.2.1.

In addition to the construction costs, maintenance costs have been estimated to be $5,000 every two years for the guard rail on Riverside Drive and $2,000 every two years for the wave wall. No maintenance costs for the raising of buildings has been identified.

10.6 ECONOMIC ANALYSIS

Based on the tangible benefits and costs described in the previous sections, an economic analysis was completed for the selected alternative flood control measures. The present value of benefits and costs were derived using social discount rates of 7%, 10% and 13%. The net present value was found by subtracting the present value of costs from the present value of benefits and the benefit - cost ratio was found by dividing the present value of benfits by the present value of costs.

10.6 ECONOMIC ANALYSIS (Cont'd)

The analysis is based on the following assumptions:

1. Alternatives have a 50 year economic life; for the 1 in 100 year flood event this will involve adding the cost of reconstructing the wave wall at year 30; for other components no reconstruction costs are foreseen (ie. their physical life and economic life are equal to 50 years);

2. alternatives have a 1 year construction period which for the purposes of analysis will be considered year 0;

3. benefits are expected to commence on the completion of construction of the flood control measure and are assumed to occur over the life of the structure, that is, years 1 to 50 inclusive;

4. raising of buildings will result in 50 percent damage reduction in Regions 2 and 3;

5. raising of Riverside Drive and construction of wave wall will result in a 100 per cent damage reduction in Regions 1 and 4.

The results of the economic analysis are shown in Table 10.6.1 and the following observations are noted:

- based on net present value, the best alternative would be Alternative 1 for the 1 in 100 year flood event (raise Riverside Drive and construct wave wall). This alternative provides the highest net present value of $2,694,000 at a discount rate of 7%. In order of decreasing net present value the alternatives are:

Net Present Value	Alternative	B/C Ratio	Discount Rate
$ 2,694,000	1 (1:100)	3.0	7%
2,607,000	3 (1:100)	2.8	7%
2,390,000	1 (1:20)	4.3	7%
2,375,000	3 (1:20)	4.1	7%
1,606,000	1 (1:100)	2.2	10%
1,521,000	1 (1:20)	3.1	10%
1,505,000	3 (1:100)	2.1	10%
1,503,000	3 (1:20)	3.0	10%
1,021,000	1 (1:20)	2.4	13%
1,001,000	3 (1:20)	2.3	13%
965,000	1 (1:100)	1.7	13%
865,000	3 (1:100)	1.6	13%

10.6 ECONOMIC ANALYSIS (Cont'd)

- Alternative 2 (raising selected buildings in Regions 2 and 3) for either the 1 in 20 or 1 in 100 year flood events is not economically viable. That is, the B/C ratios are less than unity;

- Alternative 3 - combination of Alternatives 1 and 2 (Raise Riverside Drive/construct wave wall and raise selected houses in Regions 2 and 3) is economically viable having B/C ratios of 2.8, 2.1 and 1.6 for discount rates of 7, 10 and 13 percent respectively, for the 1 in 100 year flood event;

- although net present values, and B/C ratios increase with decreasing social discount rates, neither alternative changes from being economically non viable to economically viable by reducing the discount rate (ie. alternatives are not sensitive to discount rate).

TABLE 10.6.1

ECONOMIC ANALYSIS SUMMARY - PLACENTIA FLOOD REGION

Alternative	Economic Criteria	EVENT 1:20 Social Discount Rate 7%	EVENT 1:20 Social Discount Rate 10%	EVENT 1:20 Social Discount Rate 13%	EVENT 1:100 Social Discount Rate 7%	EVENT 1:100 Social Discount Rate 10%	EVENT 1:100 Social Discount Rate 13%
Alternative 1: Raise Riverside Drive (1)	Present Value Costs	728 (2)	719	713	1,353	1,307	1,285
	Present Value Benefits	3,118	2,240	1,734	4,047	2,913	2,250
	Net Present Value	2,390	1,521	1,021	2,694	1,606	965
	B/C Ratio	4.3	3.1	2.4	3.0	2.2	1.7
Alternative 2: Raise Selected Buildings in Regions 2 and 3	Present Value Costs	26	26	26	116	116	116
	Present Value Benefits	12	8	6	29	21	16
	Net Present Value	-14	-18	-20	-87	-95	-100
	B/C Ratio	0.5	0.3	0.2	0.3	0.2	0.1
Alternative 3: Combination of Alternatives 1 and 2	Present Value Costs	754	754	739	1,469	1,423	1,401
	Present Value Benefits	3,129	2,248	1,740	4,075	2,928	2,266
	Net Present Value	2,375	1,503	1,001	2,607	1,505	865
	B/C Ratio	4.1	3.0	2.3	2.8	2.1	1.6

(1) For the 1:100 Year Event this alternative includes construction of a wave wall on the beach to the West of Placentia.

(2) All values except for B/C Ratios are in thousands of dollars.

PART ELEVEN

PHASE II RECOMMENDATIONS AND CONCLUSIONS

11.1 CONCLUSIONS

Based on the investigative and economic studies carried out in Phase I of this study and the more detailed cost analysis carried out in Phase II, the following conclusions are made:

- Region 1 experiences the most frequent and extensive damages; expected average annual damages were found to be $225,911 and $293,212 for the 1 in 20 year and 1 in 100 year flood events, respectively;

- Regions 2 and 3 were found to have less frequent and lower value damages than Region 1; expected average annual damages were found to be $664 (1:20) and $4,038 (1:100) for Region 2 and $1,015 (1:20) and $1,454 (1:100) for Region 3;

- flood damages in Region 4 were found to be minimal ($635 per year) and very infrequent; at most, damage can be expected to occur 1 percent of the time,

- Alternatives 1 and 3 were found to be economically viable at all discount levels for both the 1 in 20 and 1 in 100 year flood events; given that these conclusions were based solely on the tangible benefits they can be expected to be enhanced if the intangible benefits are included;

- raising selected buildings in Regions 2 and 3 by itself, even when considering intangible benefits, does not appear to be a viable alternative.

11.2 RECOMMENDATIONS

1. It is recommended that Alternative 1 for the 1 in 100 year flood event (raise Riverside Drive and construct wave wall on the beach to the west of Placentia) be implemented for elimination of flood damage in Regions 1 and 4.

2. It is also recommended that Alternative 2 (raising selected houses in Regions 2 and 3) be implemented in conjunction with Alternative 1. This will reduce the benefit-cost ratio of alternative 1 slightly but the benefit-cost ratio of the combination of Alternatives 1 and 2 is still attractive.

   This recommendation will reduce but not eliminate flood damages in Regions 2 and 3. It is made considering the social impact which would result if no flood reduction action is taken in these areas.

11.2 RECOMMENDATIONS (Cont'd)

3. It is recommended that the non-structural alternatives of monitoring/flood forecasting and contingency planning, in addition to zoning and building regulations, be implemented to reduce further flood damage to the study area. The monitoring and flood forecasting program in conjunction with contingency planning should be implemented as soon as possible to reduce potential damage to the study area. After the recommended structural flood control measures are completed, it is recommended that the monitoring/flood forecasting and contingency planning remain in effect to help reduce future flood damages in Regions 2 and 3 by providing early warnings of flood events.The monitoring/flood forecasting program would also provide early warning to Region 1 for events in excess of the 1 in 100 year return period.

4. It is recommended that semi-annual inspections be made to Placentia beach (to the west of Placentia) and annual inspections be made of the Gut and The Narrows. The inspections should include a survey beach profile at particular locations to identify profile changes which could result in extreme runup and consequent overtopping of the beach with the occurence of extreme waves. It will also identify the need for remedial work to prevent overtopping. Inspections should also be made of the beach undercutting of wave wall which could result in failure of the wave wall.

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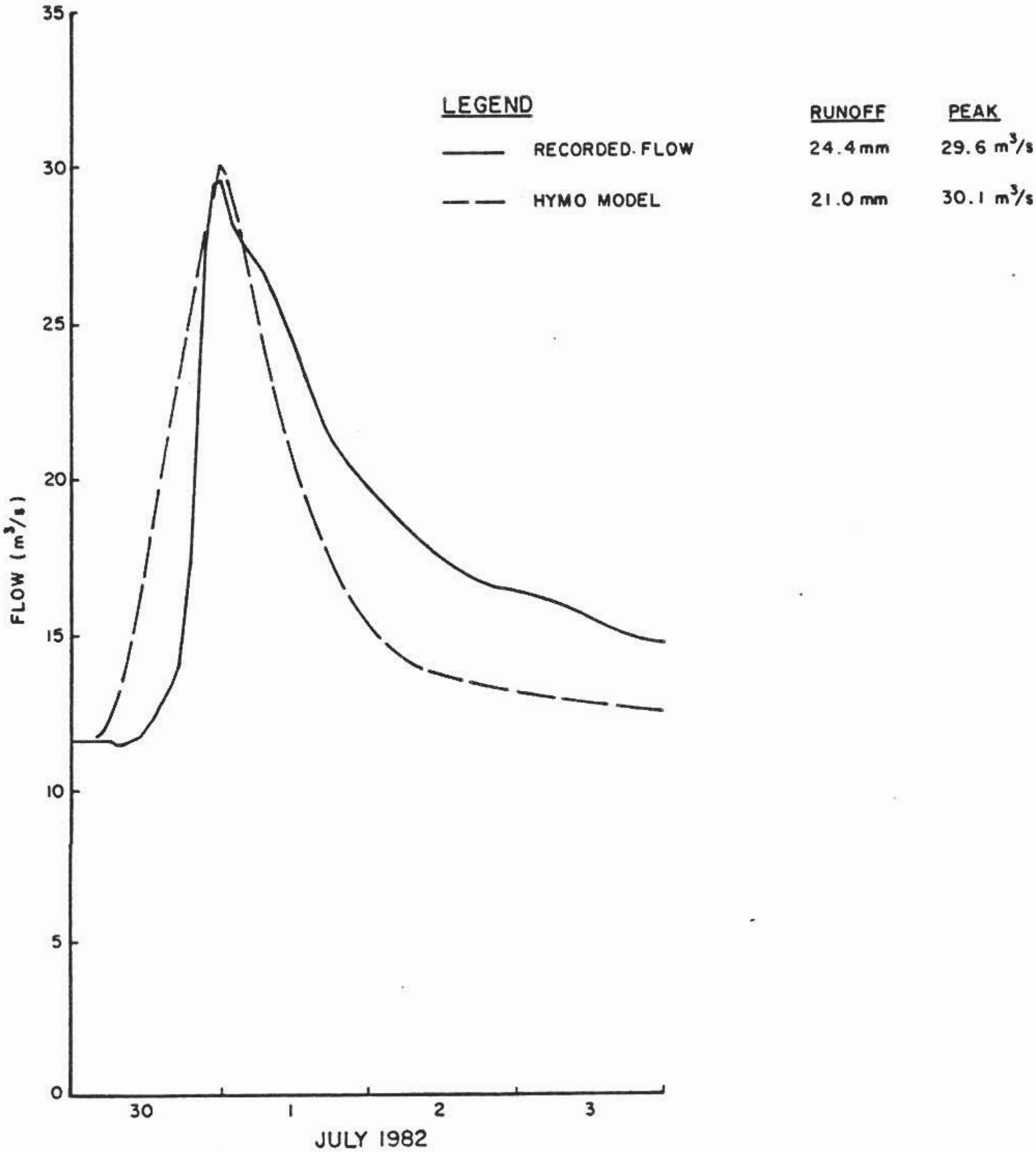
APPENDIX I

HYMO CALIBRATION AND VERIFICATION RUNS

HYMO OUTPUT FOR OCT. 5, 1979 EVENT (PRO 6)



NORTHEAST RIVER HYDROLOGIC MODEL

CALIBRATION RUN

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: NORTHEAST RIVER — 30 JUN/82
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 1
DRAINAGE AREA: 89.61399 KM↑2 MAX HEIGHT OF BASIN: 251.46 M
LENGTH OF RIVER: 23.989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.173864 K= 13.63126

MASS RAINFALL: (mm)

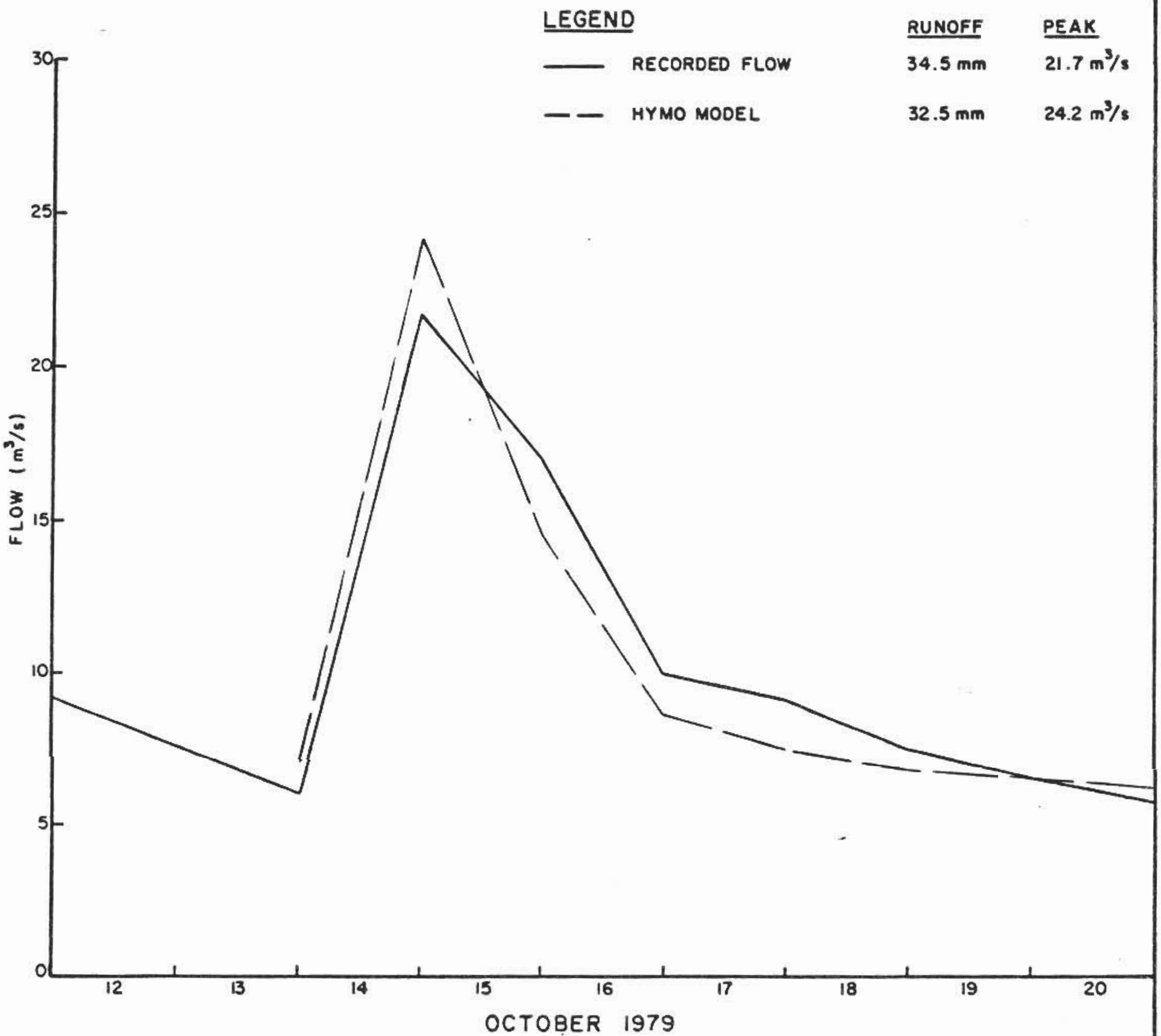
0.00	1.79	3.57	5.36	7.15	8.93	10.72	12.51	14.29	16.08
17.86	19.65	21.44	23.22	25.01	26.80	28.58	30.37	32.16	33.94
35.73	37.52	39.30	41.09	42.88					

SHAPE CONSTANT, N= 1.52443
UNIT PEAK= 28.91824 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	52.0	3.1	103.0	0.7	154.0	0.2	205.0	0.1
2.0	0.0	53.0	2.9	104.0	0.7	155.0	0.2	206.0	0.1
3.0	0.0	54.0	2.8	105.0	0.7	156.0	0.2	207.0	0.1
4.0	0.0	55.0	2.6	106.0	0.7	157.0	0.2	208.0	0.1
5.0	0.1	56.0	2.5	107.0	0.7	158.0	0.2	209.0	0.1
6.0	0.3	57.0	2.4	108.0	0.7	159.0	0.2	210.0	0.1
7.0	0.7	58.0	2.3	109.0	0.6	160.0	0.2	211.0	0.1
8.0	1.3	59.0	2.2	110.0	0.6	161.0	0.2	212.0	0.1
9.0	2.0	60.0	2.2	111.0	0.6	162.0	0.2	213.0	0.1
10.0	2.8	61.0	2.1	112.0	0.6	163.0	0.2	214.0	0.0
11.0	3.8	62.0	2.0	113.0	0.6	164.0	0.2	215.0	0.0
12.0	4.8	63.0	2.0	114.0	0.6	165.0	0.2	216.0	0.0
13.0	5.8	64.0	1.9	115.0	0.6	166.0	0.2	217.0	0.0
14.0	6.9	65.0	1.9	116.0	0.5	167.0	0.2	218.0	0.0
15.0	8.1	66.0	1.8	117.0	0.5	168.0	0.2	219.0	0.0
16.0	9.2	67.0	1.8	118.0	0.5	169.0	0.1	220.0	0.0
17.0	10.3	68.0	1.8	119.0	0.5	170.0	0.1	221.0	0.0
18.0	11.4	69.0	1.7	120.0	0.5	171.0	0.1	222.0	0.0
19.0	12.5	70.0	1.7	121.0	0.5	172.0	0.1	223.0	0.0
20.0	13.6	71.0	1.6	122.0	0.5	173.0	0.1	224.0	0.0
21.0	14.6	72.0	1.6	123.0	0.5	174.0	0.1	225.0	0.0
22.0	15.6	73.0	1.6	124.0	0.4	175.0	0.1	226.0	0.0
23.0	16.6	74.0	1.5	125.0	0.4	176.0	0.1	227.0	0.0
24.0	17.5	75.0	1.5	126.0	0.4	177.0	0.1	228.0	0.0
25.0	18.5	76.0	1.4	127.0	0.4	178.0	0.1	229.0	0.0
26.0	18.2	77.0	1.4	128.0	0.4	179.0	0.1	230.0	0.0
27.0	17.5	78.0	1.4	129.0	0.4	180.0	0.1	231.0	0.0
28.0	16.7	79.0	1.3	130.0	0.4	181.0	0.1	232.0	0.0
29.0	15.7	80.0	1.3	131.0	0.4	182.0	0.1	233.0	0.0
30.0	14.8	81.0	1.3	132.0	0.4	183.0	0.1	234.0	0.0
31.0	13.8	82.0	1.3	133.0	0.4	184.0	0.1	235.0	0.0
32.0	12.9	83.0	1.2	134.0	0.4	185.0	0.1	236.0	0.0
33.0	12.0	84.0	1.2	135.0	0.3	186.0	0.1	237.0	0.0
34.0	11.1	85.0	1.2	136.0	0.3	187.0	0.1	238.0	0.0
35.0	10.4	86.0	1.1	137.0	0.3	188.0	0.1	239.0	0.0
36.0	9.6	87.0	1.1	138.0	0.3	189.0	0.1	240.0	0.0

37.0	8.9	88.0	1.1	139.0	0.3	190.0	0.1	241.0	0.0
38.0	8.3	89.0	1.1	140.0	0.3	191.0	0.1	242.0	0.0
39.0	7.7	90.0	1.0	141.0	0.3	192.0	0.1	243.0	0.0
40.0	7.2	91.0	1.0	142.0	0.3	193.0	0.1	244.0	0.0
41.0	6.7	92.0	1.0	143.0	0.3	194.0	0.1	245.0	0.0
42.0	6.2	93.0	1.0	144.0	0.3	195.0	0.1	246.0	0.0
43.0	5.8	94.0	0.9	145.0	0.3	196.0	0.1	247.0	0.0
44.0	5.4	95.0	0.9	146.0	0.3	197.0	0.1	248.0	0.0
45.0	5.0	96.0	0.9	147.0	0.3	198.0	0.1	249.0	0.0
46.0	4.6	97.0	0.9	148.0	0.2	199.0	0.1	250.0	0.0
47.0	4.3	98.0	0.8	149.0	0.2	200.0	0.1	251.0	0.0
48.0	4.0	99.0	0.8	150.0	0.2	201.0	0.1	252.0	0.0
49.0	3.8	100.0	0.8	151.0	0.2	202.0	0.1	253.0	0.0
50.0	3.5	101.0	0.8	152.0	0.2	203.0	0.1	254.0	0.0
51.0	3.3	102.0	0.8	153.0	0.2	204.0	0.1	255.0	0.0

RUNOFF VOLUME= 20.99212 mm.
PEAK DISCHARGE= 18.45349 M↑3 ,   AT TIME= 24 HRS.



NORTHEAST RIVER HYDROLOGIC MODEL

VERIFICATION RUN

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: NORTHEAST RIVER -- OCT 14/79
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 24
DRAINAGE AREA: 89.61399 KM↑2 MAX HEIGHT OF BASIN: 251.46 M
LENGTH OF RIVER: 23.989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.173864 K= 13.63126

MASS RAINFALL: (mm)
0.00 10.92 56.64
SHAPE CONSTANT, N= 1.52443
UNIT PEAK= 28.91824 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
24.0	7.1	96.0	8.7	168.0	6.5	240.0	6.1	312.0	6.0
48.0	24.2	120.0	7.5	192.0	6.2	264.0	6.0	336.0	6.0
72.0	14.6	144.0	6.8	216.0	6.1	288.0	6.0	360.0	6.0

RUNOFF VOLUME= 32.54222 mm.
PEAK DISCHARGE= 25.19181 M↑3 , AT TIME= 48 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: NORTHEAST RIVER — OCT 14/79
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 1
DRAINAGE AREA: 89.61399 KM↑2 MAX HEIGHT OF BASIN: 251.46 M
LENGTH OF RIVER: 23.989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.173864 K= 13.63126

MASS RAINFALL: (mm)

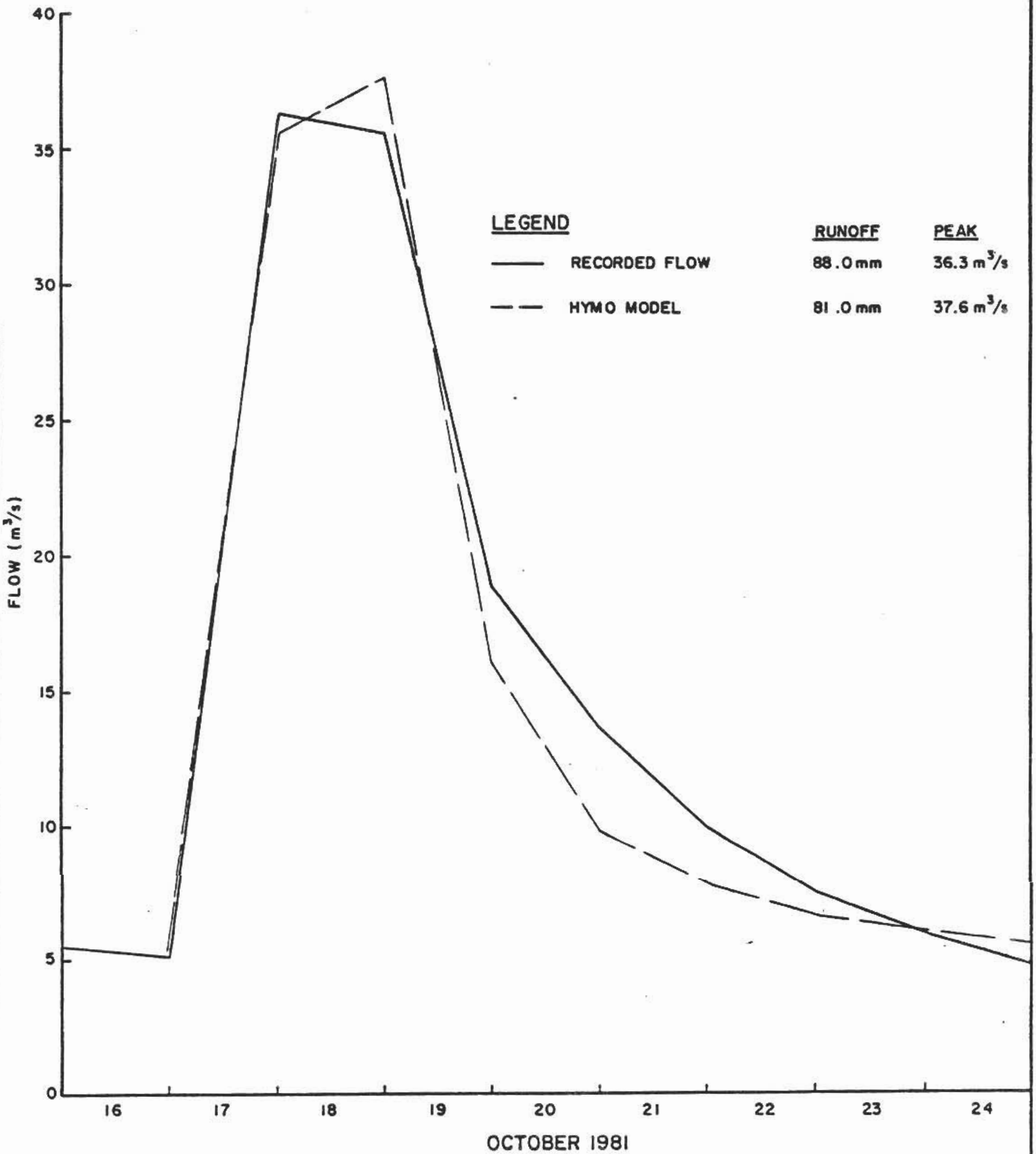
0.00	0.46	0.91	1.37	1.82	2.28	2.73	3.19	3.64	4.10
4.55	5.01	5.46	5.92	6.37	6.83	7.28	7.74	8.19	8.65
9.10	9.56	10.01	10.47	10.92	12.83	14.73	16.64	18.54	20.45
22.35	24.26	26.16	28.07	29.97	31.88	33.78	35.69	37.59	39.50
41.40	43.31	45.21	47.12	49.02	50.93	52.83	54.74	56.64	

SHAPE CONSTANT, N= 1.52443
UNIT PEAK= 28.91824 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	57.0	16.0	113.0	1.5	169.0	0.4	225.0	0.1
2.0	0.0	58.0	14.9	114.0	1.5	170.0	0.4	226.0	0.1
3.0	0.0	59.0	13.8	115.0	1.5	171.0	0.4	227.0	0.1
4.0	0.0	60.0	12.9	116.0	1.4	172.0	0.4	228.0	0.1
5.0	0.0	61.0	11.9	117.0	1.4	173.0	0.4	229.0	0.1
6.0	0.0	62.0	11.1	118.0	1.4	174.0	0.3	230.0	0.1
7.0	0.0	63.0	10.3	119.0	1.3	175.0	0.3	231.0	0.1
8.0	0.0	64.0	9.6	120.0	1.3	176.0	0.3	232.0	0.1
9.0	0.0	65.0	8.9	121.0	1.3	177.0	0.3	233.0	0.1
10.0	0.0	66.0	8.3	122.0	1.2	178.0	0.3	234.0	0.1
11.0	0.0	67.0	7.8	123.0	1.2	179.0	0.3	235.0	0.1
12.0	0.0	68.0	7.2	124.0	1.2	180.0	0.3	236.0	0.1
13.0	0.0	69.0	6.8	125.0	1.1	181.0	0.3	237.0	0.1
14.0	0.0	70.0	6.3	126.0	1.1	182.0	0.3	238.0	0.1
15.0	0.0	71.0	5.9	127.0	1.1	183.0	0.3	239.0	0.1
16.0	0.0	72.0	5.6	128.0	1.1	184.0	0.3	240.0	0.1
17.0	0.1	73.0	5.2	129.0	1.0	185.0	0.3	241.0	0.1
18.0	0.1	74.0	4.9	130.0	1.0	186.0	0.3	242.0	0.1
19.0	0.2	75.0	4.7	131.0	1.0	187.0	0.3	243.0	0.1
20.0	0.3	76.0	4.4	132.0	1.0	188.0	0.2	244.0	0.1
21.0	0.4	77.0	4.2	133.0	0.9	189.0	0.2	245.0	0.1
22.0	0.5	78.0	4.0	134.0	0.9	190.0	0.2	246.0	0.1
23.0	0.6	79.0	3.8	135.0	0.9	191.0	0.2	247.0	0.1
24.0	0.7	80.0	3.6	136.0	0.9	192.0	0.2	248.0	0.1
25.0	0.8	81.0	3.5	137.0	0.9	193.0	0.2	249.0	0.1
26.0	1.4	82.0	3.4	138.0	0.8	194.0	0.2	250.0	0.1
27.0	2.1	83.0	3.2	139.0	0.8	195.0	0.2	251.0	0.1
28.0	3.0	84.0	3.1	140.0	0.8	196.0	0.2	252.0	0.1
29.0	4.0	85.0	3.0	141.0	0.8	197.0	0.2	253.0	0.0
30.0	5.1	86.0	3.0	142.0	0.8	198.0	0.2	254.0	0.0
31.0	6.3	87.0	2.9	143.0	0.7	199.0	0.2	255.0	0.0
32.0	7.5	88.0	2.8	144.0	0.7	200.0	0.2	256.0	0.0
33.0	8.7	89.0	2.8	145.0	0.7	201.0	0.2	257.0	0.0
34.0	9.9	90.0	2.7	146.0	0.7	202.0	0.2	258.0	0.0

35.0	11.2	91.0	2.6	147.0	0.7	203.0	0.2	259.0	0.0
36.0	12.4	92.0	2.6	148.0	0.7	204.0	0.2	260.0	0.0
37.0	13.6	93.0	2.5	149.0	0.6	205.0	0.2	261.0	0.0
38.0	14.7	94.0	2.4	150.0	0.6	206.0	0.2	262.0	0.0
39.0	15.9	95.0	2.4	151.0	0.6	207.0	0.2	263.0	0.0
40.0	17.0	96.0	2.3	152.0	0.6	208.0	0.2	264.0	0.0
41.0	18.0	97.0	2.3	153.0	0.6	209.0	0.1	265.0	0.0
42.0	19.1	98.0	2.2	154.0	0.6	210.0	0.1	266.0	0.0
43.0	20.0	99.0	2.2	155.0	0.6	211.0	0.1	267.0	0.0
44.0	21.0	100.0	2.1	156.0	0.5	212.0	0.1	268.0	0.0
45.0	21.9	101.0	2.1	157.0	0.5	213.0	0.1	269.0	0.0
46.0	22.8	102.0	2.0	158.0	0.5	214.0	0.1	270.0	0.0
47.0	23.6	103.0	2.0	159.0	0.5	215.0	0.1	271.0	0.0
48.0	24.4	104.0	1.9	160.0	0.5	216.0	0.1	272.0	0.0
49.0	25.2	105.0	1.9	161.0	0.5	217.0	0.1	273.0	0.0
50.0	24.6	106.0	1.8	162.0	0.5	218.0	0.1	274.0	0.0
51.0	23.6	107.0	1.8	163.0	0.5	219.0	0.1	275.0	0.0
52.0	22.4	108.0	1.7	164.0	0.4	220.0	0.1	276.0	0.0
53.0	21.1	109.0	1.7	165.0	0.4	221.0	0.1	277.0	0.0
54.0	19.8	110.0	1.7	166.0	0.4	222.0	0.1	278.0	0.0
55.0	18.5	111.0	1.6	167.0	0.4	223.0	0.1	279.0	0.0
56.0	17.2	112.0	1.6	168.0	0.4	224.0	0.1	280.0	0.0

RUNOFF VOLUME= 32.54222 mm.
PEAK DISCHARGE= 25.19183 M↑3 ,   AT TIME= 48 HRS.



NORTHEAST RIVER HYDROLOGIC MODEL

VERIFICATION RUN

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: NORTHEAST RIVER -- OCT 17/81
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 24
DRAINAGE AREA: 89.61399 KM↑2 MAX HEIGHT OF BASIN: 251.46 M
LENGTH OF RIVER: 23.989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.173864 K= 13.63126

MASS RAINFALL: (mm)
0.00 12.70 98.55
SHAPE CONSTANT, N= 1.52443
UNIT PEAK= 28.91824 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
24.0	5.4	96.0	16.1	168.0	6.6	240.0	5.4	312.0	5.2
48.0	35.6	120.0	9.9	192.0	6.0	264.0	5.3	336.0	5.2
72.0	37.6	144.0	7.8	216.0	5.6	288.0	5.2	360.0	5.2

RUNOFF VOLUME= 81.40521 mm.
PEAK DISCHARGE= 53.90352 M↑3 , AT TIME= 48 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: NORTHEAST RIVER — OCT 17/81
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 1
DRAINAGE AREA: 89.61399 KM↑2 MAX HEIGHT OF BASIN: 251.46 M
LENGTH OF RIVER: 23.989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.173864 K= 13.63126

MASS RAINFALL: (mm)

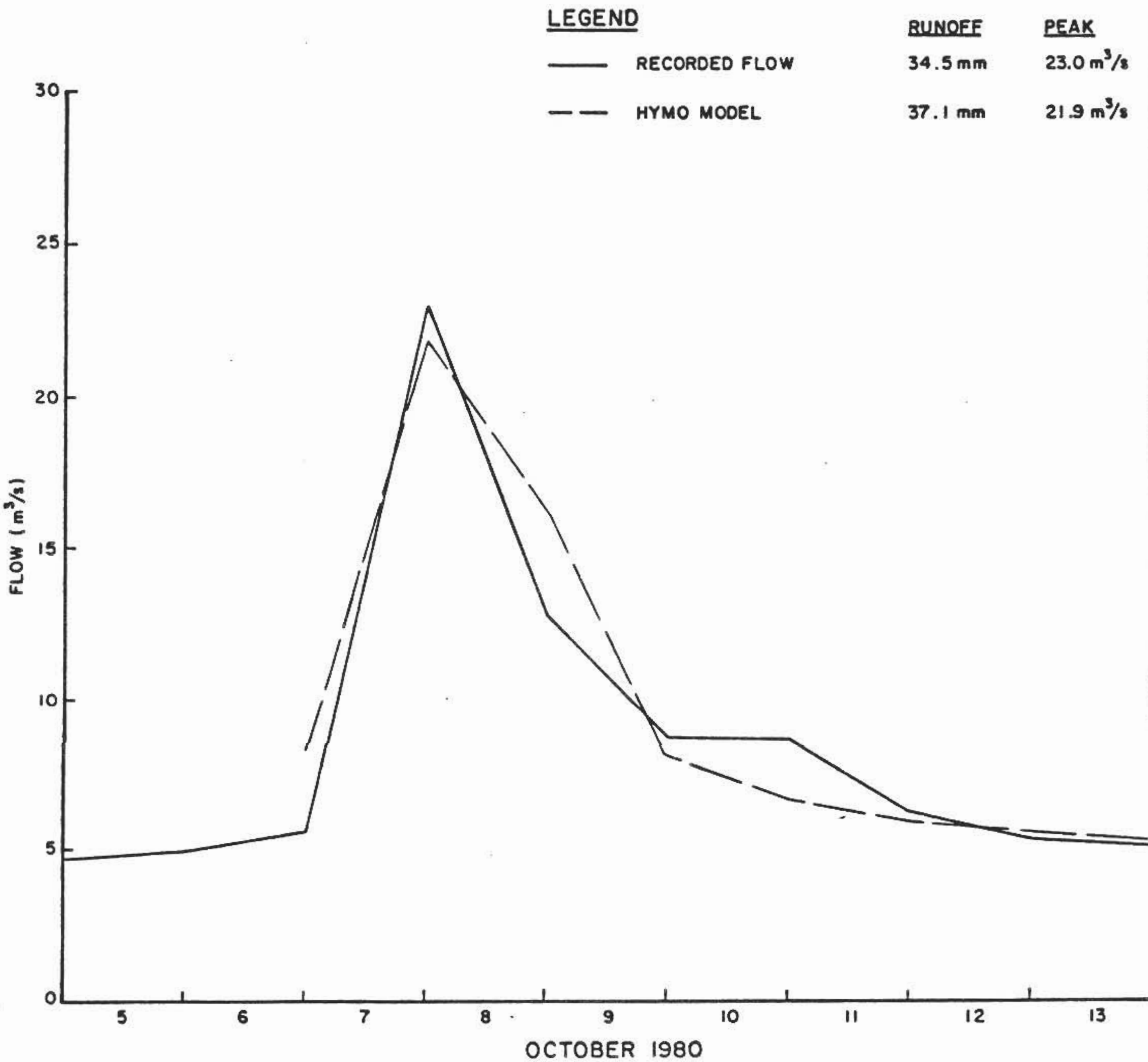
0.00	0.53	1.06	1.59	2.12	2.65	3.18	3.70	4.23	4.76
5.29	5.82	6.35	6.88	7.41	7.94	8.47	9.00	9.53	10.05
10.58	11.11	11.64	12.17	12.70	16.28	19.85	23.43	27.01	30.59
34.16	37.74	41.32	44.89	48.47	52.05	55.63	59.20	62.78	66.36
69.93	73.51	77.09	80.67	84.24	87.82	91.40	94.97	98.55	

SHAPE CONSTANT, N= 1.52443
UNIT PEAK= 28.91824 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	57.0	34.1	113.0	3.3	169.0	0.8	225.0	0.2
2.0	0.0	58.0	31.7	114.0	3.2	170.0	0.8	226.0	0.2
3.0	0.0	59.0	29.4	115.0	3.2	171.0	0.8	227.0	0.2
4.0	0.0	60.0	27.4	116.0	3.1	172.0	0.8	228.0	0.2
5.0	0.0	61.0	25.4	117.0	3.0	173.0	0.8	229.0	0.2
6.0	0.0	62.0	23.6	118.0	2.9	174.0	0.7	230.0	0.2
7.0	0.0	63.0	22.0	119.0	2.9	175.0	0.7	231.0	0.2
8.0	0.0	64.0	20.4	120.0	2.8	176.0	0.7	232.0	0.2
9.0	0.0	65.0	19.0	121.0	2.7	177.0	0.7	233.0	0.2
10.0	0.0	66.0	17.7	122.0	2.7	178.0	0.7	234.0	0.2
11.0	0.0	67.0	16.5	123.0	2.6	179.0	0.7	235.0	0.2
12.0	0.0	68.0	15.4	124.0	2.5	180.0	0.6	236.0	0.2
13.0	0.0	69.0	14.4	125.0	2.5	181.0	0.6	237.0	0.2
14.0	0.0	70.0	13.5	126.0	2.4	182.0	0.6	238.0	0.2
15.0	0.1	71.0	12.7	127.0	2.4	183.0	0.6	239.0	0.2
16.0	0.2	72.0	11.9	128.0	2.3	184.0	0.6	240.0	0.1
17.0	0.3	73.0	11.2	129.0	2.2	185.0	0.6	241.0	0.1
18.0	0.4	74.0	10.5	130.0	2.2	186.0	0.6	242.0	0.1
19.0	0.5	75.0	10.0	131.0	2.1	187.0	0.5	243.0	0.1
20.0	0.6	76.0	9.4	132.0	2.1	188.0	0.5	244.0	0.1
21.0	0.8	77.0	9.0	133.0	2.0	189.0	0.5	245.0	0.1
22.0	0.9	78.0	8.5	134.0	2.0	190.0	0.5	246.0	0.1
23.0	1.1	79.0	8.1	135.0	1.9	191.0	0.5	247.0	0.1
24.0	1.2	80.0	7.8	136.0	1.9	192.0	0.5	248.0	0.1
25.0	1.4	81.0	7.5	137.0	1.8	193.0	0.5	249.0	0.1
26.0	2.6	82.0	7.2	138.0	1.8	194.0	0.5	250.0	0.1
27.0	4.4	83.0	7.0	139.0	1.8	195.0	0.4	251.0	0.1
28.0	6.6	84.0	6.8	140.0	1.7	196.0	0.4	252.0	0.1
29.0	9.1	85.0	6.6	141.0	1.7	197.0	0.4	253.0	0.1
30.0	11.7	86.0	6.4	142.0	1.6	198.0	0.4	254.0	0.1
31.0	14.5	87.0	6.3	143.0	1.6	199.0	0.4	255.0	0.1
32.0	17.3	88.0	6.1	144.0	1.6	200.0	0.4	256.0	0.1
33.0	20.2	89.0	6.0	145.0	1.5	201.0	0.4	257.0	0.1
34.0	23.0	90.0	5.8	146.0	1.5	202.0	0.4	258.0	0.1

35.0	25.7	91.0	5.7	147.0	1.4	203.0	0.4	259.0	0.1
36.0	28.4	92.0	5.5	148.0	1.4	204.0	0.4	260.0	0.1
37.0	31.0	93.0	5.4	149.0	1.4	205.0	0.3	261.0	0.1
38.0	33.4	94.0	5.3	150.0	1.3	206.0	0.3	262.0	0.1
39.0	35.8	95.0	5.1	151.0	1.3	207.0	0.3	263.0	0.1
40.0	38.1	96.0	5.0	152.0	1.3	208.0	0.3	264.0	0.1
41.0	40.2	97.0	4.9	153.0	1.2	209.0	0.3	265.0	0.1
42.0	42.3	98.0	4.8	154.0	1.2	210.0	0.3	266.0	0.0
43.0	44.2	99.0	4.7	155.0	1.2	211.0	0.3	267.0	0.0
44.0	46.1	100.0	4.6	156.0	1.2	212.0	0.3	268.0	0.0
45.0	47.8	101.0	4.4	157.0	1.1	213.0	0.3	269.0	0.0
46.0	49.5	102.0	4.3	158.0	1.1	214.0	0.3	270.0	0.0
47.0	51.0	103.0	4.2	159.0	1.1	215.0	0.3	271.0	0.0
48.0	52.5	104.0	4.1	160.0	1.0	216.0	0.3	272.0	0.0
49.0	53.9	105.0	4.0	161.0	1.0	217.0	0.3	273.0	0.0
50.0	52.5	106.0	3.9	162.0	1.0	218.0	0.3	274.0	0.0
51.0	50.3	107.0	3.8	163.0	1.0	219.0	0.2	275.0	0.0
52.0	47.7	108.0	3.7	164.0	1.0	220.0	0.2	276.0	0.0
53.0	44.9	109.0	3.7	165.0	0.9	221.0	0.2	277.0	0.0
54.0	42.1	110.0	3.6	166.0	0.9	222.0	0.2	278.0	0.0
55.0	39.3	111.0	3.5	167.0	0.9	223.0	0.2	279.0	0.0
56.0	36.6	112.0	3.4	168.0	0.9	224.0	0.2	280.0	0.0

RUNOFF VOLUME= 81.40521 mm.
PEAK DISCHARGE= 53.90352 M↑3 , AT TIME= 48 HRS.



NORTHEAST RIVER HYDROLOGIC MODEL

VERIFICATION RUN

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: NORTHEAST RIVER — OCT 7/80
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 24
DRAINAGE AREA: 89.61399 KM↑2 MAX HEIGHT OF BASIN: 251.46 M
LENGTH OF RIVER: 23.989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.173864 K= 13.63126

MASS RAINFALL: (mm)
0.00 28.91 61.93
SHAPE CONSTANT, N= 1.52443
UNIT PEAK= 28.91824 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
24.0	8.3	96.0	8.1	168.0	5.5	240.0	5.0	312.0	5.0
48.0	21.9	120.0	6.6	192.0	5.2	264.0	5.0	336.0	5.0
72.0	16.3	144.0	5.9	216.0	5.1	288.0	5.0	360.0	5.0

RUNOFF VOLUME= 37.15608 mm.
PEAK DISCHARGE= 22.00158 M↑3 , AT TIME= 48 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: NORTHEAST RIVER — OCT 7/80
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 1
DRAINAGE AREA: 89.61399 KM↑2 MAX HEIGHT OF BASIN: 251.46 M
LENGTH OF RIVER: 23.989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.173864 K= 13.63126

MASS RAINFALL: (mm)

0.00	1.20	2.41	3.61	4.82	6.02	7.23	8.43	9.64	10.84
12.04	13.25	14.45	15.66	16.86	18.07	19.27	20.47	21.68	22.88
24.09	25.29	26.50	27.70	28.91	30.28	31.66	33.03	34.41	35.78
37.16	38.54	39.91	41.29	42.66	44.04	45.42	46.79	48.17	49.54
50.92	52.29	53.67	55.05	56.42	57.80	59.17	60.55	61.93	

SHAPE CONSTANT, N= 1.52443
UNIT PEAK= 28.91824 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	57.0	13.8	113.0	1.5	169.0	0.4	225.0	0.1
2.0	0.0	58.0	12.9	114.0	1.5	170.0	0.4	226.0	0.1
3.0	0.0	59.0	12.0	115.0	1.5	171.0	0.4	227.0	0.1
4.0	0.0	60.0	11.2	116.0	1.4	172.0	0.4	228.0	0.1
5.0	0.0	61.0	10.5	117.0	1.4	173.0	0.4	229.0	0.1
6.0	0.0	62.0	9.8	118.0	1.4	174.0	0.3	230.0	0.1
7.0	0.1	63.0	9.1	119.0	1.3	175.0	0.3	231.0	0.1
8.0	0.2	64.0	8.5	120.0	1.3	176.0	0.3	232.0	0.1
9.0	0.5	65.0	8.0	121.0	1.3	177.0	0.3	233.0	0.1
10.0	0.8	66.0	7.5	122.0	1.2	178.0	0.3	234.0	0.1
11.0	1.2	67.0	7.0	123.0	1.2	179.0	0.3	235.0	0.1
12.0	1.6	68.0	6.6	124.0	1.2	180.0	0.3	236.0	0.1
13.0	2.1	69.0	6.2	125.0	1.1	181.0	0.3	237.0	0.1
14.0	2.7	70.0	5.9	126.0	1.1	182.0	0.3	238.0	0.1
15.0	3.3	71.0	5.5	127.0	1.1	183.0	0.3	239.0	0.1
16.0	3.9	72.0	5.2	128.0	1.1	184.0	0.3	240.0	0.1
17.0	4.5	73.0	5.0	129.0	1.0	185.0	0.3	241.0	0.1
18.0	5.2	74.0	4.7	130.0	1.0	186.0	0.3	242.0	0.1
19.0	5.8	75.0	4.5	131.0	1.0	187.0	0.3	243.0	0.1
20.0	6.4	76.0	4.3	132.0	1.0	188.0	0.2	244.0	0.1
21.0	7.1	77.0	4.1	133.0	0.9	189.0	0.2	245.0	0.1
22.0	7.7	78.0	3.9	134.0	0.9	190.0	0.2	246.0	0.1
23.0	8.3	79.0	3.7	135.0	0.9	191.0	0.2	247.0	0.1
24.0	8.9	80.0	3.6	136.0	0.9	192.0	0.2	248.0	0.1
25.0	9.5	81.0	3.5	137.0	0.9	193.0	0.2	249.0	0.0
26.0	10.2	82.0	3.3	138.0	0.8	194.0	0.2	250.0	0.0
27.0	10.9	83.0	3.2	139.0	0.8	195.0	0.2	251.0	0.0
28.0	11.6	84.0	3.1	140.0	0.8	196.0	0.2	252.0	0.0
29.0	12.3	85.0	3.1	141.0	0.8	197.0	0.2	253.0	0.0
30.0	13.0	86.0	3.0	142.0	0.8	198.0	0.2	254.0	0.0
31.0	13.6	87.0	2.9	143.0	0.7	199.0	0.2	255.0	0.0
32.0	14.3	88.0	2.8	144.0	0.7	200.0	0.2	256.0	0.0
33.0	14.9	89.0	2.8	145.0	0.7	201.0	0.2	257.0	0.0
34.0	15.5	90.0	2.7	146.0	0.7	202.0	0.2	258.0	0.0

35.0	16.1	91.0	2.6	147.0	0.7	203.0	0.2	259.0	0.0
36.0	16.6	92.0	2.6	148.0	0.7	204.0	0.2	260.0	0.0
37.0	17.1	93.0	2.5	149.0	0.6	205.0	0.2	261.0	0.0
38.0	17.6	94.0	2.5	150.0	0.6	206.0	0.2	262.0	0.0
39.0	18.1	95.0	2.4	151.0	0.6	207.0	0.2	263.0	0.0
40.0	18.6	96.0	2.3	152.0	0.6	208.0	0.2	264.0	0.0
41.0	19.0	97.0	2.3	153.0	0.6	209.0	0.1	265.0	0.0
42.0	19.5	98.0	2.2	154.0	0.6	210.0	0.1	266.0	0.0
43.0	19.9	99.0	2.2	155.0	0.6	211.0	0.1	267.0	0.0
44.0	20.3	100.0	2.1	156.0	0.5	212.0	0.1	268.0	0.0
45.0	20.6	101.0	2.1	157.0	0.5	213.0	0.1	269.0	0.0
46.0	21.0	102.0	2.0	158.0	0.5	214.0	0.1	270.0	0.0
47.0	21.4	103.0	2.0	159.0	0.5	215.0	0.1	271.0	0.0
48.0	21.7	104.0	1.9	160.0	0.5	216.0	0.1	272.0	0.0
49.0	22.0	105.0	1.9	161.0	0.5	217.0	0.1	273.0	0.0
50.0	21.3	106.0	1.8	162.0	0.5	218.0	0.1	274.0	0.0
51.0	20.4	107.0	1.8	163.0	0.5	219.0	0.1	275.0	0.0
52.0	19.3	108.0	1.7	164.0	0.4	220.0	0.1	276.0	0.0
53.0	18.2	109.0	1.7	165.0	0.4	221.0	0.1	277.0	0.0
54.0	17.0	110.0	1.7	166.0	0.4	222.0	0.1	278.0	0.0
55.0	15.9	111.0	1.6	167.0	0.4	223.0	0.1	279.0	0.0
56.0	14.8	112.0	1.6	168.0	0.4	224.0	0.1	280.0	0.0

RUNOFF VOLUME= 37.15608 mm.
PEAK DISCHARGE= 22.00158 M↑3 , AT TIME= 48 HRS.

HYMO OUTPUT FOR OCT. 5, 1979 EVENT (PRO 6)

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 1 (SOUTHEAST RIVER)
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 1
DRAINAGE AREA: 143.227 KM↑2 MAX HEIGHT OF BASIN: 335.28 M
LENGTH OF RIVER: 26.565 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.509012 K= 12.7249

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.630899
UNIT PEAK= 48.31645 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	53.0	5.8	105.0	1.3	157.0	0.3	209.0	0.1
2.0	0.0	54.0	5.5	106.0	1.3	158.0	0.3	210.0	0.1
3.0	0.0	55.0	5.2	107.0	1.3	159.0	0.3	211.0	0.1
4.0	0.0	56.0	5.0	108.0	1.2	160.0	0.3	212.0	0.1
5.0	0.3	57.0	4.8	109.0	1.2	161.0	0.3	213.0	0.1
6.0	1.0	58.0	4.6	110.0	1.2	162.0	0.3	214.0	0.1
7.0	2.0	59.0	4.5	111.0	1.1	163.0	0.3	215.0	0.1
8.0	3.4	60.0	4.3	112.0	1.1	164.0	0.3	216.0	0.1
9.0	5.1	61.0	4.2	113.0	1.1	165.0	0.3	217.0	0.1
10.0	7.0	62.0	4.1	114.0	1.1	166.0	0.3	218.0	0.1
11.0	9.1	63.0	4.0	115.0	1.0	167.0	0.3	219.0	0.1
12.0	11.3	64.0	3.9	116.0	1.0	168.0	0.3	220.0	0.1
13.0	13.6	65.0	3.8	117.0	1.0	169.0	0.2	221.0	0.1
14.0	15.9	66.0	3.7	118.0	0.9	170.0	0.2	222.0	0.1
15.0	18.3	67.0	3.6	119.0	0.9	171.0	0.2	223.0	0.1
16.0	20.6	68.0	3.5	120.0	0.9	172.0	0.2	224.0	0.1
17.0	22.9	69.0	3.4	121.0	0.9	173.0	0.2	225.0	0.1
18.0	25.2	70.0	3.3	122.0	0.9	174.0	0.2	226.0	0.1
19.0	27.4	71.0	3.3	123.0	0.8	175.0	0.2	227.0	0.1
20.0	29.5	72.0	3.2	124.0	0.8	176.0	0.2	228.0	0.1
21.0	31.6	73.0	3.1	125.0	0.8	177.0	0.2	229.0	0.1
22.0	33.6	74.0	3.0	126.0	0.8	178.0	0.2	230.0	0.1
23.0	35.5	75.0	2.9	127.0	0.8	179.0	0.2	231.0	0.0
24.0	37.3	76.0	2.9	128.0	0.7	180.0	0.2	232.0	0.0
25.0	39.1	77.0	2.8	129.0	0.7	181.0	0.2	233.0	0.0
26.0	38.6	78.0	2.7	130.0	0.7	182.0	0.2	234.0	0.0
27.0	37.2	79.0	2.6	131.0	0.7	183.0	0.2	235.0	0.0
28.0	35.5	80.0	2.6	132.0	0.7	184.0	0.2	236.0	0.0
29.0	33.4	81.0	2.5	133.0	0.6	185.0	0.2	237.0	0.0
30.0	31.3	82.0	2.4	134.0	0.6	186.0	0.2	238.0	0.0
31.0	29.2	83.0	2.4	135.0	0.6	187.0	0.2	239.0	0.0
32.0	27.1	84.0	2.3	136.0	0.6	188.0	0.2	240.0	0.0
33.0	25.1	85.0	2.3	137.0	0.6	189.0	0.1	241.0	0.0
34.0	23.2	86.0	2.2	138.0	0.6	190.0	0.1	242.0	0.0
35.0	21.4	87.0	2.1	139.0	0.5	191.0	0.1	243.0	0.0
36.0	19.8	88.0	2.1	140.0	0.5	192.0	0.1	244.0	0.0

37.0	18.3	89.0	2.0	141.0	0.5	193.0	0.1	245.0	0.0
38.0	16.9	90.0	2.0	142.0	0.5	194.0	0.1	246.0	0.0
39.0	15.7	91.0	1.9	143.0	0.5	195.0	0.1	247.0	0.0
40.0	14.5	92.0	1.9	144.0	0.5	196.0	0.1	248.0	0.0
41.0	13.4	93.0	1.8	145.0	0.5	197.0	0.1	249.0	0.0
42.0	12.4	94.0	1.8	146.0	0.5	198.0	0.1	250.0	0.0
43.0	11.5	95.0	1.7	147.0	0.4	199.0	0.1	251.0	0.0
44.0	10.6	96.0	1.7	148.0	0.4	200.0	0.1	252.0	0.0
45.0	9.9	97.0	1.6	149.0	0.4	201.0	0.1	253.0	0.0
46.0	9.2	98.0	1.6	150.0	0.4	202.0	0.1	254.0	0.0
47.0	8.5	99.0	1.6	151.0	0.4	203.0	0.1	255.0	0.0
48.0	7.9	100.0	1.5	152.0	0.4	204.0	0.1	256.0	0.0
49.0	7.4	101.0	1.5	153.0	0.4	205.0	0.1	257.0	0.0
50.0	6.9	102.0	1.4	154.0	0.4	206.0	0.1	258.0	0.0
51.0	6.5	103.0	1.4	155.0	0.4	207.0	0.1	259.0	0.0
52.0	6.1	104.0	1.4	156.0	0.4	208.0	0.1	260.0	0.0

RUNOFF VOLUME= 27.39083 mm.
PEAK DISCHARGE= 39.06867 M↑3 ,   AT TIME= 24 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 2 (NORTHEAST RIVER)
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ 1 HRS, OUTPUT@ 1
DRAINAGE AREA: 89.61399 KM↑2 MAX HEIGHT OF BASIN: 251.46 M
LENGTH OF RIVER: 23.989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 4.173864 K= 13.63126

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.52443
UNIT PEAK= 28.91824 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	52.0	4.0	103.0	1.0	154.0	0.3	205.0	0.1
2.0	0.0	53.0	3.8	104.0	0.9	155.0	0.3	206.0	0.1
3.0	0.0	54.0	3.6	105.0	0.9	156.0	0.3	207.0	0.1
4.0	0.0	55.0	3.4	106.0	0.9	157.0	0.3	208.0	0.1
5.0	0.2	56.0	3.2	107.0	0.9	158.0	0.3	209.0	0.1
6.0	0.6	57.0	3.1	108.0	0.9	159.0	0.2	210.0	0.1
7.0	1.3	58.0	3.0	109.0	0.8	160.0	0.2	211.0	0.1
8.0	2.2	59.0	2.9	110.0	0.8	161.0	0.2	212.0	0.1
9.0	3.2	60.0	2.8	111.0	0.8	162.0	0.2	213.0	0.1
10.0	4.3	61.0	2.7	112.0	0.8	163.0	0.2	214.0	0.1
11.0	5.6	62.0	2.6	113.0	0.8	164.0	0.2	215.0	0.1
12.0	6.9	63.0	2.6	114.0	0.7	165.0	0.2	216.0	0.1
13.0	8.3	64.0	2.5	115.0	0.7	166.0	0.2	217.0	0.1
14.0	9.7	65.0	2.4	116.0	0.7	167.0	0.2	218.0	0.1
15.0	11.1	66.0	2.4	117.0	0.7	168.0	0.2	219.0	0.1
16.0	12.5	67.0	2.3	118.0	0.7	169.0	0.2	220.0	0.1
17.0	13.9	68.0	2.3	119.0	0.7	170.0	0.2	221.0	0.1
18.0	15.2	69.0	2.2	120.0	0.6	171.0	0.2	222.0	0.1
19.0	16.6	70.0	2.2	121.0	0.6	172.0	0.2	223.0	0.1
20.0	17.9	71.0	2.1	122.0	0.6	173.0	0.2	224.0	0.1
21.0	19.1	72.0	2.1	123.0	0.6	174.0	0.2	225.0	0.0
22.0	20.3	73.0	2.0	124.0	0.6	175.0	0.2	226.0	0.0
23.0	21.5	74.0	2.0	125.0	0.6	176.0	0.2	227.0	0.0
24.0	22.6	75.0	1.9	126.0	0.6	177.0	0.2	228.0	0.0
25.0	23.7	76.0	1.9	127.0	0.5	178.0	0.2	229.0	0.0
26.0	23.2	77.0	1.8	128.0	0.5	179.0	0.2	230.0	0.0
27.0	22.4	78.0	1.8	129.0	0.5	180.0	0.1	231.0	0.0
28.0	21.3	79.0	1.7	130.0	0.5	181.0	0.1	232.0	0.0
29.0	20.1	80.0	1.7	131.0	0.5	182.0	0.1	233.0	0.0
30.0	18.9	81.0	1.7	132.0	0.5	183.0	0.1	234.0	0.0
31.0	17.6	82.0	1.6	133.0	0.5	184.0	0.1	235.0	0.0
32.0	16.4	83.0	1.6	134.0	0.5	185.0	0.1	236.0	0.0
33.0	15.3	84.0	1.5	135.0	0.4	186.0	0.1	237.0	0.0
34.0	14.2	85.0	1.5	136.0	0.4	187.0	0.1	238.0	0.0
35.0	13.2	86.0	1.5	137.0	0.4	188.0	0.1	239.0	0.0
36.0	12.3	87.0	1.4	138.0	0.4	189.0	0.1	240.0	0.0

37.0	11.4	88.0	1.4	139.0	0.4	190.0	0.1	241.0	0.0
38.0	10.6	89.0	1.4	140.0	0.4	191.0	0.1	242.0	0.0
39.0	9.8	90.0	1.3	141.0	0.4	192.0	0.1	243.0	0.0
40.0	9.2	91.0	1.3	142.0	0.4	193.0	0.1	244.0	0.0
41.0	8.5	92.0	1.3	143.0	0.4	194.0	0.1	245.0	0.0
42.0	7.9	93.0	1.2	144.0	0.4	195.0	0.1	246.0	0.0
43.0	7.3	94.0	1.2	145.0	0.3	196.0	0.1	247.0	0.0
44.0	6.8	95.0	1.2	146.0	0.3	197.0	0.1	248.0	0.0
45.0	6.4	96.0	1.1	147.0	0.3	198.0	0.1	249.0	0.0
46.0	5.9	97.0	1.1	148.0	0.3	199.0	0.1	250.0	0.0
47.0	5.5	98.0	1.1	149.0	0.3	200.0	0.1	251.0	0.0
48.0	5.2	99.0	1.1	150.0	0.3	201.0	0.1	252.0	0.0
49.0	4.8	100.0	1.0	151.0	0.3	202.0	0.1	253.0	0.0
50.0	4.5	101.0	1.0	152.0	0.3	203.0	0.1	254.0	0.0
51.0	4.3	102.0	1.0	153.0	0.3	204.0	0.1	255.0	0.0

RUNOFF VOLUME= 27.33039 mm.
PEAK DISCHARGE= 23.65686 M↑3 ,   AT TIME= 24 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 3 (SWAN ARM)
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ .2 HRS, OUTPUT@ 1
DRAINAGE AREA: 11.9917 KM↑2 MAX HEIGHT OF BASIN: 160.02 M
LENGTH OF RIVER: 8.999899 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= 1.410203 K= 5.717079

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.400637
UNIT PEAK= 9.543147 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.5	21.0	4.0	41.0	0.6	61.0	0.2	81.0	0.1
2.0	0.0	22.0	4.2	42.0	0.6	62.0	0.2	82.0	0.1
3.0	0.0	23.0	4.3	43.0	0.6	63.0	0.2	83.0	0.1
4.0	0.0	24.0	4.5	44.0	0.5	64.0	0.2	84.0	0.0
5.0	0.1	25.0	4.4	45.0	0.5	65.0	0.2	85.0	0.0
6.0	0.3	26.0	3.8	46.0	0.5	66.0	0.1	86.0	0.0
7.0	0.6	27.0	3.3	47.0	0.4	67.0	0.1	87.0	0.0
8.0	0.9	28.0	2.8	48.0	0.4	68.0	0.1	88.0	0.0
9.0	1.2	29.0	2.4	49.0	0.4	69.0	0.1	89.0	0.0
10.0	1.5	30.0	2.0	50.0	0.4	70.0	0.1	90.0	0.0
11.0	1.8	31.0	1.7	51.0	0.4	71.0	0.1	91.0	0.0
12.0	2.1	32.0	1.5	52.0	0.3	72.0	0.1	92.0	0.0
13.0	2.4	33.0	1.3	53.0	0.3	73.0	0.1	93.0	0.0
14.0	2.6	34.0	1.2	54.0	0.3	74.0	0.1	94.0	0.0
15.0	2.9	35.0	1.0	55.0	0.3	75.0	0.1	95.0	0.0
16.0	3.1	36.0	0.9	56.0	0.3	76.0	0.1	96.0	0.0
17.0	3.3	37.0	0.8	57.0	0.3	77.0	0.1	97.0	0.0
18.0	3.5	38.0	0.8	58.0	0.2	78.0	0.1	98.0	0.0
19.0	3.7	39.0	0.7	59.0	0.2	79.0	0.1	99.0	0.0
20.0	3.9	40.0	0.7	60.0	0.2	80.0	0.1	100.0	0.0

RUNOFF VOLUME= 27.32132 mm.
PEAK DISCHARGE= 4.526488 M↑3 , AT TIME= 24.00002 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 4
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ .2 HRS, OUTPUT@ 1
DRAINAGE AREA: 5.5944 KM↑2 MAX HEIGHT OF BASIN: 144.78 M
LENGTH OF RIVER: 5.2969 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= .8062009 K= 3.307844

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.394754
UNIT PEAK= 7.708452 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.5	15.0	1.7	29.0	0.8	43.0	0.2	57.0	0.0
2.0	0.0	16.0	1.8	30.0	0.7	44.0	0.1	58.0	0.0
3.0	0.0	17.0	1.9	31.0	0.6	45.0	0.1	59.0	0.0
4.0	0.0	18.0	1.9	32.0	0.5	46.0	0.1	60.0	0.0
5.0	0.1	19.0	2.0	33.0	0.4	47.0	0.1	61.0	0.0
6.0	0.3	20.0	2.1	34.0	0.4	48.0	0.1	62.0	0.0
7.0	0.4	21.0	2.2	35.0	0.4	49.0	0.1	63.0	0.0
8.0	0.6	22.0	2.2	36.0	0.3	50.0	0.1	64.0	0.0
9.0	0.8	23.0	2.3	37.0	0.3	51.0	0.1	65.0	0.0
10.0	1.0	24.0	2.4	38.0	0.3	52.0	0.1	66.0	0.0
11.0	1.1	25.0	2.2	39.0	0.2	53.0	0.1	67.0	1.5
12.0	1.3	26.0	1.7	40.0	0.2	54.0	0.0	68.0	1.5
13.0	1.4	27.0	1.3	41.0	0.2	55.0	0.0	69.0	1.6
14.0	1.5	28.0	1.0	42.0	0.2	56.0	0.0	70.0	1.6

RUNOFF VOLUME= 27.16062 mm.
PEAK DISCHARGE= 2.38457 M↑3 , AT TIME= 24.00002 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 5
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ .1 HRS, OUTPUT@ 1
DRAINAGE AREA: 5.3872 KM↑2 MAX HEIGHT OF BASIN: 205.74 M
LENGTH OF RIVER: 3.1073 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= .4606131 K= 1.451389

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.548616
UNIT PEAK= 16.23158 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	10.0	1.3	19.0	2.3	28.0	0.5	37.0	0.1
2.0	0.0	11.0	1.5	20.0	2.3	29.0	0.4	38.0	0.0
3.0	0.0	12.0	1.6	21.0	2.4	30.0	0.3	39.0	0.0
4.0	0.1	13.0	1.7	22.0	2.4	31.0	0.3	40.0	0.0
5.0	0.2	14.0	1.8	23.0	2.5	32.0	0.2	41.0	0.0
6.0	0.5	15.0	2.0	24.0	2.5	33.0	0.2	42.0	0.0
7.0	0.7	16.0	2.0	25.0	2.1	34.0	0.1	43.0	0.0
8.0	0.9	17.0	2.1	26.0	1.2	35.0	0.1	44.0	0.0
9.0	1.1	18.0	2.2	27.0	0.8	36.0	0.1	45.0	0.0

RUNOFF VOLUME= 27.3236 mm.
PEAK DISCHARGE= 2.552477 M↑3 , AT TIME= 24.00006 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 7
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ .1 HRS, OUTPUT@ 1
DRAINAGE AREA: 3.108 KM↑2 MAX HEIGHT OF BASIN: 167.64 M
LENGTH OF RIVER: 2.9946 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= .4203046 K= 1.544846

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.452372
UNIT PEAK= 9.0178 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	10.0	0.7	19.0	1.3	28.0	0.3	37.0	0.0
2.0	0.0	11.0	0.8	20.0	1.3	29.0	0.3	38.0	0.0
3.0	0.0	12.0	0.9	21.0	1.4	30.0	0.2	39.0	0.0
4.0	0.0	13.0	1.0	22.0	1.4	31.0	0.2	40.0	0.0
5.0	0.1	14.0	1.1	23.0	1.4	32.0	0.1	41.0	0.0
6.0	0.3	15.0	1.1	24.0	1.5	33.0	0.1	42.0	0.0
7.0	0.4	16.0	1.2	25.0	1.2	34.0	0.1	43.0	0.0
8.0	0.5	17.0	1.2	26.0	0.7	35.0	0.1	44.0	0.0
9.0	0.6	18.0	1.3	27.0	0.5	36.0	0.1	45.0	0.1

RUNOFF VOLUME= 27.21038 mm.
PEAK DISCHARGE= 1.46363 M↑3 , AT TIME= 24.00006 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 8
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ .1 HRS, OUTPUT@ 1
DRAINAGE AREA: 2.2015 KM↑2 MAX HEIGHT OF BASIN: 106.68 M
LENGTH OF RIVER: 2.8014 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= .4462215 K= 1.975576

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.359529
UNIT PEAK= 5.135615 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	11.0	0.5	21.0	0.9	31.0	0.2	41.0	0.0
2.0	0.0	12.0	0.6	22.0	1.0	32.0	0.1	42.0	0.0
3.0	0.0	13.0	0.7	23.0	1.0	33.0	0.1	43.0	0.0
4.0	0.0	14.0	0.7	24.0	1.0	34.0	0.1	44.0	0.0
5.0	0.1	15.0	0.7	25.0	0.9	35.0	0.1	45.0	0.0
6.0	0.2	16.0	0.8	26.0	0.6	36.0	0.1	46.0	0.0
7.0	0.2	17.0	0.8	27.0	0.4	37.0	0.1	47.0	0.1
8.0	0.3	18.0	0.9	28.0	0.3	38.0	0.0	48.0	0.1
9.0	0.4	19.0	0.9	29.0	0.2	39.0	0.0	49.0	0.1
10.0	0.5	20.0	0.9	30.0	0.2	40.0	0.0	50.0	0.1

RUNOFF VOLUME= 27.09029 mm.
PEAK DISCHARGE= 1.013736 M↑3 , AT TIME= 24.00006 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 9
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ .1 HRS, OUTPUT@ 1
DRAINAGE AREA: 2.3051 KM↑2 MAX HEIGHT OF BASIN: 91.43999 M
LENGTH OF RIVER: 2.3989 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= .4337228 K= 1.90898

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.362129
UNIT PEAK= 5.560199 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.3	11.0	0.6	21.0	1.0	31.0	0.2	41.0	0.0
2.0	0.0	12.0	0.6	22.0	1.0	32.0	0.1	42.0	0.0
3.0	0.0	13.0	0.7	23.0	1.0	33.0	0.1	43.0	0.0
4.0	0.0	14.0	0.7	24.0	1.1	34.0	0.1	44.0	0.0
5.0	0.1	15.0	0.8	25.0	0.9	35.0	0.1	45.0	0.0
6.0	0.2	16.0	0.8	26.0	0.6	36.0	0.1	46.0	0.0
7.0	0.3	17.0	0.9	27.0	0.4	37.0	0.1	47.0	0.1
8.0	0.4	18.0	0.9	28.0	0.3	38.0	0.0	48.0	0.1
9.0	0.4	19.0	0.9	29.0	0.2	39.0	0.0	49.0	0.1
10.0	0.5	20.0	1.0	30.0	0.2	40.0	0.0	50.0	0.1

RUNOFF VOLUME= 27.10552 mm.
PEAK DISCHARGE= 1.065039 M↑3 , AT TIME= 24.00006 HRS.

HYDROLOGIC MODELLING PROGRAM (HYMO)

ADAPTED BY SHAWMONT NFLD LTD
TO RUN ON DEC RAINBOW COMPUTER

HYDROGRAPH NAME IS: BASIN 15
TIME INTERVALS: RAINFALL INPUT @ 24 HRS. CALC @ .1 HRS, OUTPUT@ 1
DRAINAGE AREA: 3.0044 KM↑2 MAX HEIGHT OF BASIN: 106.68 M
LENGTH OF RIVER: 1.6744 KM CURVE NUMBER (CN) = 90
TIME TO PEAK (TP)= .3359729 K= 1.205144

MASS RAINFALL: (mm)

0.00	2.11	4.21	6.32	8.42	10.53	12.64	14.74	16.85	18.95
21.06	23.17	25.27	27.38	29.49	31.59	33.70	35.80	37.91	40.02
42.12	44.23	46.33	48.44	50.55					

SHAPE CONSTANT, N= 1.466317
UNIT PEAK= 11.13189 CFS

TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW	TIME	FLOW
1.0	0.0	9.0	0.7	17.0	1.2	25.0	1.1	33.0	0.1
2.0	0.0	10.0	0.8	18.0	1.3	26.0	0.6	34.0	0.0
3.0	0.0	11.0	0.8	19.0	1.3	27.0	0.4	35.0	0.0
4.0	0.0	12.0	0.9	20.0	1.3	28.0	0.3	36.0	0.0
5.0	0.2	13.0	1.0	21.0	1.3	29.0	0.2	37.0	0.0
6.0	0.3	14.0	1.1	22.0	1.4	30.0	0.1	38.0	0.0
7.0	0.4	15.0	1.1	23.0	1.4	31.0	0.1	39.0	0.0
8.0	0.5	16.0	1.2	24.0	1.4	32.0	0.1	40.0	0.0

RUNOFF VOLUME= 27.16583 mm.
PEAK DISCHARGE= 1.430728 M↑3 , AT TIME= 24.00006 HRS.

APPENDIX II

30 YEAR WAVE HINDCAST

CUMMULATIVE DISTRIBUTIONS OF

SEA HEIGHT AND PERIOD

PLACENTIA BAY

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR JANUARY

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
	SEA HEIGHT IN FEET												
0. TO 1.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
1. TO 2.	17.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	17.0	.2
2. TO 3.	47.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	47.0	.9
3. TO 4.	162.	311.	0.	0.	0.	0.	0.	0.	0.	0.	0.	473.0	2.2
4. TO 5.	70.	401.	336.	2.	1.	2.	0.	0.	0.	0.	0.	812.0	3.7
5. TO 6.	10.	3.	165.	239.	13.	0.	0.	0.	0.	0.	0.	430.0	6.3
6. TO 7.	0.	0.	0.	12.	162.	83.	0.	0.	0.	0.	0.	257.0	9.4
7. TO 8.	0.	0.	0.	0.	0.	58.	38.	13.	0.	0.	0.	109.0	12.4
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	5.	11.	1.	0.	17.0	16.6
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	3.	3.0	21.1
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	306.	715.	501.	253.	176.	143.	38.	18.	11.	1.	3.		

2171 OBSERVATIONS: 6. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 5.050

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR FEBRUARY

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
	SEA HEIGHT IN FEET												
0. TO 1.	1.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	1.0	.1
1. TO 2.	15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	15.0	.2
2. TO 3.	55.	2.	0.	0.	0.	0.	0.	0.	0.	0.	0.	57.0	1.1
3. TO 4.	192.	272.	0.	0.	0.	0.	0.	0.	0.	0.	0.	464.0	2.1
4. TO 5.	61.	251.	305.	0.	0.	0.	0.	0.	0.	0.	0.	617.0	3.8
5. TO 6.	22.	2.	194.	300.	2.	0.	0.	0.	0.	0.	0.	520.0	6.1
6. TO 7.	0.	0.	0.	13.	98.	24.	0.	0.	0.	0.	0.	135.0	9.1
7. TO 8.	0.	0.	0.	0.	0.	15.	16.	5.	0.	0.	0.	36.0	12.6
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	2.	4.	1.	0.	7.0	16.6
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	346.	527.	499.	313.	100.	39.	16.	7.	4.	1.	0.		

1865 OBSERVATIONS: 13. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 4.477

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR MARCH

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
	SEA HEIGHT IN FEET												
0. TO 1.	4.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	4.0	.1
1. TO 2.	20.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	20.0	.2
2. TO 3.	49.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	49.0	.7
3. TO 4.	152.	191.	0.	0.	0.	0.	0.	0.	0.	0.	0.	343.0	2.0
4. TO 5.	46.	278.	211.	0.	0.	0.	0.	0.	0.	0.	0.	535.0	3.7
5. TO 6.	8.	0.	85.	106.	2.	0.	0.	0.	0.	0.	0.	201.0	6.1
6. TO 7.	0.	0.	0.	5.	74.	40.	0.	0.	0.	0.	0.	119.0	9.5
7. TO 8.	0.	0.	0.	0.	0.	15.	14.	0.	0.	0.	0.	29.0	12.2
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	279.	469.	296.	111.	76.	55.	14.	0.	0.	0.	0.		

1324 OBSERVATIONS: 24. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 4.076

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR APRIL

SEA PERIOD	SEA HEIGHT IN FEET 0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
0. TO 1.	3.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	3.0	.1
1. TO 2.	43.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	43.0	.2
2. TO 3.	95.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	95.0	.8
3. TO 4.	146.	96.	0.	0.	0.	0.	0.	0.	0.	0.	0.	242.0	1.8
4. TO 5.	38.	131.	87.	0.	0.	0.	0.	0.	0.	0.	0.	256.0	3.4
5. TO 6.	4.	2.	14.	21.	0.	0.	0.	0.	0.	0.	0.	41.0	5.7
6. TO 7.	0.	0.	0.	0.	5.	0.	0.	0.	0.	0.	0.	5.0	8.9
7. TO 8.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	329.	229.	101.	21.	5.	0.	0.	0.	0.	0.	0.		

718 OBSERVATIONS: 33. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 2.320

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR MAY

SEA PERIOD	SEA HEIGHT IN FEET 0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
0. TO 1.	4.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	4.0	.1
1. TO 2.	61.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	61.0	.2
2. TO 3.	58.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	58.0	.8
3. TO 4.	103.	48.	0.	0.	0.	0.	0.	0.	0.	0.	0.	151.0	1.5
4. TO 5.	22.	57.	36.	0.	0.	0.	0.	0.	0.	0.	0.	115.0	3.4
5. TO 6.	1.	0.	26.	33.	0.	0.	0.	0.	0.	0.	0.	60.0	6.2
6. TO 7.	0.	0.	0.	1.	11.	0.	0.	0.	0.	0.	0.	12.0	8.8
7. TO 8.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	249.	105.	62.	34.	11.	0.	0.	0.	0.	0.	0.		

474 OBSERVATIONS: 13. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 2.415

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR JUNE

SEA PERIOD	SEA HEIGHT IN FEET 0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
0. TO 1.	15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	15.0	.1
1. TO 2.	58.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	58.0	.2
2. TO 3.	68.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	68.0	.7
3. TO 4.	53.	25.	0.	0.	0.	0.	0.	0.	0.	0.	0.	78.0	1.5
4. TO 5.	7.	22.	12.	0.	0.	0.	0.	0.	0.	0.	0.	41.0	3.3
5. TO 6.	1.	0.	7.	1.	0.	0.	0.	0.	0.	0.	0.	9.0	5.1
6. TO 7.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
7. TO 8.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	202.	47.	19.	1.	0.	0.	0.	0.	0.	0.	0.		

272 OBSERVATIONS: 3. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 1.318

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR JULY

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
	SEA HEIGHT IN FEET												
0. TO 1.	8.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	8.0	.1
1. TO 2.	47.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	47.0	.2
2. TO 3.	36.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	36.0	.6
3. TO 4.	39.	14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	53.0	1.5
4. TO 5.	5.	18.	4.	0.	0.	0.	0.	0.	0.	0.	0.	27.0	3.0
5. TO 6.	1.	0.	1.	0.	0.	0.	0.	0.	0.	0.	0.	2.0	2.7
6. TO 7.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
7. TO 8.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	136.	32.	5.	0.	0.	0.	0.	0.	0.	0.	0.		

182 OBSERVATIONS: 9. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 1.096

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR AUGUST

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
	SEA HEIGHT IN FEET												
0. TO 1.	6.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	6.0	.1
1. TO 2.	27.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	27.0	.2
2. TO 3.	82.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	82.0	.8
3. TO 4.	136.	117.	0.	0.	0.	0.	0.	0.	0.	0.	0.	253.0	1.8
4. TO 5.	32.	82.	74.	0.	0.	0.	0.	0.	0.	0.	0.	188.0	3.4
5. TO 6.	7.	0.	15.	45.	0.	0.	0.	0.	0.	0.	0.	67.0	5.9
6. TO 7.	0.	0.	0.	2.	0.	0.	0.	0.	0.	0.	0.	2.0	7.9
7. TO 8.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	[illegible]	[illegible]	[illegible]	42	0.	0.	0.	0.	0.	0.	0.		

646 OBSERVATIONS: 21. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 2.457

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR SEPTEMBER

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
	SEA HEIGHT IN FEET												
0. TO 1.	3.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	3.0	.1
1. TO 2.	31.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	31.0	.2
2. TO 3.	89.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	89.0	.9
3. TO 4.	174.	182.	0.	0.	0.	0.	0.	0.	0.	0.	0.	356.0	2.0
4. TO 5.	34.	249.	138.	1.	0.	0.	0.	0.	0.	0.	0.	422.0	3.6
5. TO 6.	3.	0.	55.	118.	1.	0.	0.	0.	0.	0.	0.	177.0	6.4
6. TO 7.	0.	0.	0.	3.	17.	0.	0.	0.	0.	0.	0.	20.0	8.4
7. TO 8.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	334.	431.	193.	122.	18.	0.	0.	0.	0.	0.	0.		

1113 OBSERVATIONS: 15. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 3.246

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR OCTOBER

SEA HEIGHT IN FEET

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
0. TO 1.	3.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	3.0	.1
1. TO 2.	19.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	19.0	.2
2. TO 3.	71.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	71.0	1.0
3. TO 4.	218.	237.	0.	0.	0.	0.	0.	0.	0.	0.	0.	455.0	2.0
4. TO 5.	45.	245.	160.	0.	0.	0.	0.	0.	0.	0.	0.	450.0	3.6
5. TO 6.	12.	1.	74.	73.	0.	0.	0.	0.	0.	0.	0.	160.0	5.7
6. TO 7.	0.	0.	0.	3.	25.	10.	0.	0.	0.	0.	0.	38.0	9.4
7. TO 8.	0.	0.	0.	0.	0.	4.	2.	0.	0.	0.	0.	6.0	11.9
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	368.	483.	234.	76.	25.	14.	2.	0.	0.	0.	0.		

1217 OBSERVATIONS: 15. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 3.253

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR NOVEMBER

SEA HEIGHT IN FEET

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
0. TO 1.	6.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	6.0	.1
1. TO 2.	14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	14.0	.2
2. TO 3.	59.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	59.0	1.1
3. TO 4.	177.	302.	0.	0.	0.	0.	0.	0.	0.	0.	0.	479.0	2.2
4. TO 5.	51.	282.	288.	0.	0.	0.	0.	0.	0.	0.	0.	621.0	3.8
5. TO 6.	4.	1.	123.	129.	1.	0.	0.	0.	0.	0.	0.	258.0	6.2
6. TO 7.	0.	0.	0.	5.	38.	21.	0.	0.	0.	0.	0.	64.0	9.5
7. TO 8.	0.	0.	0.	0.	0.	1.	10.	1.	0.	0.	0.	12.0	13.1
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	311.	585.	411.	134.	39.	22.	10.	1.	0.	0.	0.		

1524 OBSERVATIONS: 11. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 3.815

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR DECEMBER

SEA HEIGHT IN FEET

SEA PERIOD	0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
0. TO 1.	7.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	7.0	.1
1. TO 2.	10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	10.0	.2
2. TO 3.	44.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	44.0	.9
3. TO 4.	164.	312.	3.	0.	0.	0.	0.	0.	0.	0.	0.	479.0	2.2
4. TO 5.	75.	332.	319.	1.	0.	0.	0.	0.	1.	0.	0.	728.0	3.7
5. TO 6.	6.	3.	139.	204.	6.	0.	0.	0.	0.	0.	0.	358.0	6.3
6. TO 7.	0.	0.	0.	11.	112.	43.	0.	0.	0.	0.	0.	166.0	9.3
7. TO 8.	0.	0.	0.	0.	0.	12.	21.	0.	0.	0.	0.	33.0	12.5
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	306.	647.	461.	216.	118.	55.	21.	0.	1.	0.	0.		

1835 OBSERVATIONS: 10. OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 4.372

PLACENTIA BAY HINDCAST WAVE DATA

CUMULATIVE DISTRIBUTION OF SEA HEIGHT AND PERIOD FOR ALL DATA

SEA PERIOD	SEA HEIGHT IN FEET 0.0 TO 2.0	2.0 TO 4.0	4.0 TO 6.0	6.0 TO 8.0	8.0 TO 10.0	10.0 TO 12.0	12.0 TO 14.0	14.0 TO 16.0	16.0 TO 18.0	18.0 TO 20.0	20.0 AND OVER	TOTAL	MEAN HEIGHT
0. TO 1.	60.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	60.0	.1
1. TO 2.	362.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	362.0	.2
2. TO 3.	753.	2.	0.	0.	0.	0.	0.	0.	0.	0.	0.	755.0	.9
3. TO 4.	1716.	2107.	3.	0.	0.	0.	0.	0.	0.	0.	0.	3826.0	2.0
4. TO 5.	486.	2348.	1970.	4.	1.	2.	0.	0.	4.	0.	0.	4812.0	3.7
5. TO 6.	79.	12.	898.	1269.	25.	0.	0.	0.	0.	0.	0.	2283.0	6.1
6. TO 7.	0.	0.	0.	55.	542.	221.	0.	0.	0.	0.	0.	818.0	8.8
7. TO 8.	0.	0.	0.	0.	0.	105.	101.	19.	0.	0.	0.	225.0	9.3
8. TO 9.	0.	0.	0.	0.	0.	0.	0.	7.	15.	2.	0.	24.0	5.0
9. TO 10.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	3.	3.0	3.4
10. TO 11.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
11. TO 12.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
12. TO 13.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
13. TO 14.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
14. TO 15.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
15. TO 16.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.0	0.0
16. OR MORE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
TOTAL	3456.	4469.	2871.	1328.	568.	328.	101.	26.	16.	2.	3.		

13341 OBSERVATIONS: 173 . OCCURENCES OF CALM SEAS: MEAN SEA HEIGHT : 3.828

APPENDIX III

Freshwater Flow and Precipitation During High Water Level Events

CROSS SECTIONS FOR DWOPER MODEL

DWOPER Calibration Run - PLAC 21

DWOPER Verification Run - PLAC 20

DWOPER Verification Run - PLAC 22

DWOPER Production Run - PRO 25

Note: All cross section elevations to Geodetic Datum

FRESHWATER FLOW AND PRECIPITATION

DURING HIGH WATER LEVEL EVENTS

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
18 12 72	5.95		0.0
19 12 72	5.52		0.0
20 12 72	5.24		3.8
21 12 72	4.96		0.0
22 12 72	4.81		11.4

Baseflow: Northeast Arm: 2.78 m^3/s
Southeast Arm: 3.91 m^3/s
Swan Arm: 0.40 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
25 02 72	5.18		1.0
27 02 72	5.10		0.5
27 02 72	7.08		8.9
28 02 72	9.06		0.0
29 02 72	11.9		4.1
01 03 72	15.3		8.9

Baseflow: Northeast Arm: 5.18 m^3/s
Southeast Arm: 7.31 m^3/s
Swan Arm: 0.71 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
18 01 73	4.28		0.0
19 01 73	3.91		1.5
20 01 73	6.09		5.3
21 01 73	9.06		0.5
22 01 73	5.72		1.0
23 01 73	7.02		12.4

Baseflow: Northeast Arm: 3.23 m^3/s
Southeast Arm: 4.53 m^3/s
Swan Arm: 0.45 m^3/s

FRESHWATER FLOW AND PRECIPITATION

DURING HIGH WATER LEVEL EVENTS (Cont'd)

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
05 01 73	21.8		2.5
06 01 73	17.8		4.8
07 01 73	15.1		1.3
08 01 73	13.3		0.3
09 01 73	11.8		6.1

Baseflow:
- Northeast Arm: 10.34 m^3/s
- Southeast Arm: 14.55 m^3/s
- Swan Arm: 1.42 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
05 02 74	4.11		2.0
06 02 74	3.94		1.0
07 02 74	3.82		0.0
08 02 74	3.68		0.0
09 02 74	3.54		0.0
10 02 74	3.45		1.0
11 02 74	3.34		0.3

Baseflow:
- Northeast Arm: 1.81 m^3/s
- Southeast Arm: 2.55 m^3/s
- Swan Arm: 0.25 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
08 01 74	5.52		3.8
09 01 74	5.10		0.0
10 01 74	4.73		0.0
11 01 74	4.39		0.0
12 01 74	4.13		2.5

Baseflow:
- Northeast Arm: 2.46 m^3/s
- Southeast Arm: 3.45 m^3/s
- Swan Arm: 0.34 m^3/s

FRESHWATER FLOW AND PRECIPITATION

DURING HIGH WATER LEVEL EVENTS (Cont'd)

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
28 01 75	4.67		0.5
29 01 75	4.45		0.5
30 01 75	4.19		6.1
31 01 75	3.91		0.5
01 02 75	3.79		0.0

Baseflow: Northeast Arm: 2.12 m^3/s
Southeast Arm: 2.97 m^3/s
Swan Arm: 0.28 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
05 10 75	21.4		0.0
06 10 75	14.7		0.0
07 10 75	11.3		50.5
08 10 75	27.5		4.6
09 10 75	26.8		0.0
10 10 75	18.8		0.0
11 10 75	14.0		0.0

Baseflow: Northeast Arm: 12.88 m^3/s
Southeast Arm: 18.12 m^3/s
Swan Arm: 1.78 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
15 03 76	2.13		
16 03 76	12.1		
17 03 76	10.5		
18 03 76	7.93		
19 03 76	6.65		

Baseflow: Northeast Arm: 7.14 m^3/s
Southeast Arm: 10.05 m^3/s
Swan Arm: 0.99 m^3/s

FRESHWATER FLOW AND PRECIPITATION

DURING HIGH WATER LEVEL EVENTS (Cont'd)

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
16 12 76	14.6		0.0
17 12 76	12.6		7.6
18 12 76	13.0		0.0
19 12 76	11.9		0.0
20 12 76	8.47		0.0

Baseflow:	Northeast Arm:	7.45 m^3/s
	Southeast Arm:	10.48 m^3/s
	Swan Arm:	1.02 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
18 01 77	7.65		0.0
19 01 77	6.94		0.0
20 01 77	7.65		7.0
21 01 77	9.91		0.0
22 01 77	8.50		0.0
23 01 77	7.48		0.0

Baseflow:	Northeast Arm:	4.56 m^3/s
	Southeast Arm:	6.43 m^3/s
	Swan Arm:	0.62 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
04 02 77	6.37		0.3
05 02 77	6.09		0.0
06 02 77	5.95		7.9
07 02 77	5.92		0.0
08 02 77	5.10		0.3

Baseflow:	Northeast Arm:	3.14 m^3/s
	Southeast Arm:	4.45 m^3/s
	Swan Arm:	0.42 m^3/s

FRESHWATER FLOW AND PRECIPITATION

DURING HIGH WATER LEVEL EVENTS (Cont'd)

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
30 11 78	23.5		0.0
01 12 78	18.0		1.0
02 12 78	14.0		0.2
03 12 78	11.5		0.0
04 12 78	9.91		0.0

Baseflow: Northeast Arm: 9.88 m^3/s
Southeast Arm: 13.93 m^3/s
Swan Arm: 1.36 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
06 02 78	7.08		0.0
07 02 78	6.51		0.0
08 02 78	6.00		13.3
09 02 78	5.61		1.0
10 02 78	5.15		0.0
11 02 78	4.84		0.7

Baseflow: Northeast Arm: 3.14 m^3/s
Southeast Arm: 4.42 m^3/s
Swan Arm: 0.42 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
27 01 79	29.3	11.1	9.7
28 01 79	28.6	14.2	4.4
29 01 78	31.7	10.4	6.8
30 01 79	36.8	18.8	12.2
31 01 79	45.6	14.5	3.9
01 02 79	34.5	11.0	0.8

Baseflow: Northeast Arm: 25.80 m^3/s
Southeast Arm: 36.30 m^3/s
Swan Arm: 3.54 m^3/s

FRESHWATER FLOW AND PRECIPITATION

DURING HIGH WATER LEVEL EVENTS (Cont'd)

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
06 10 79	10.5	4.45	13.6
07 10 79	15.5	5.61	0.0
08 10 79	12.9	6.02	6.8
09 10 79	14.0	5.50	1.2
10 10 79	13.5	5.54	16.3

Baseflow: Northeast Arm: 8.30 m^3/s
Southeast Arm: 11.69 m^3/s
Swan Arm: 1.13 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
23 10 80	12.2	5.81	30.8
24 10 80	31.2	9.11	4.0
25 10 80	19.0	6.68	0.0
26 10 80	13.0	5.21	9.6
27 10 80	23.3	7.51	2.0

Baseflow: Northeast Arm: 13.31 m^3/s
Southeast Arm: 18.75 m^3/s
Swan Arm: 1.84 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
03 01 80	7.03	4.53	13.6
04 01 80	11.1	5.49	0.0
05 01 80	8.48	4.25	0.0
06 01 80	7.93	4.18	2.0
07 01 80	7.14	4.21	0.0

Baseflow: Northeast Arm: 4.70 m^3/s
Southeast Arm: 6.71 m^3/s
Swan Arm: 0.65 m^3/s

FRESHWATER FLOW AND PRECIPITATION

DURING HIGH WATER LEVEL EVENTS (Cont'd)

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
15 02 80	2.94	1.10	0.6
16 02 80	2.89	1.70	0.0
17 02 80	4.53	7.67	45.5
18 02 80	8.21	13.1	0.0
19 02 80	28.3	7.02	1.0

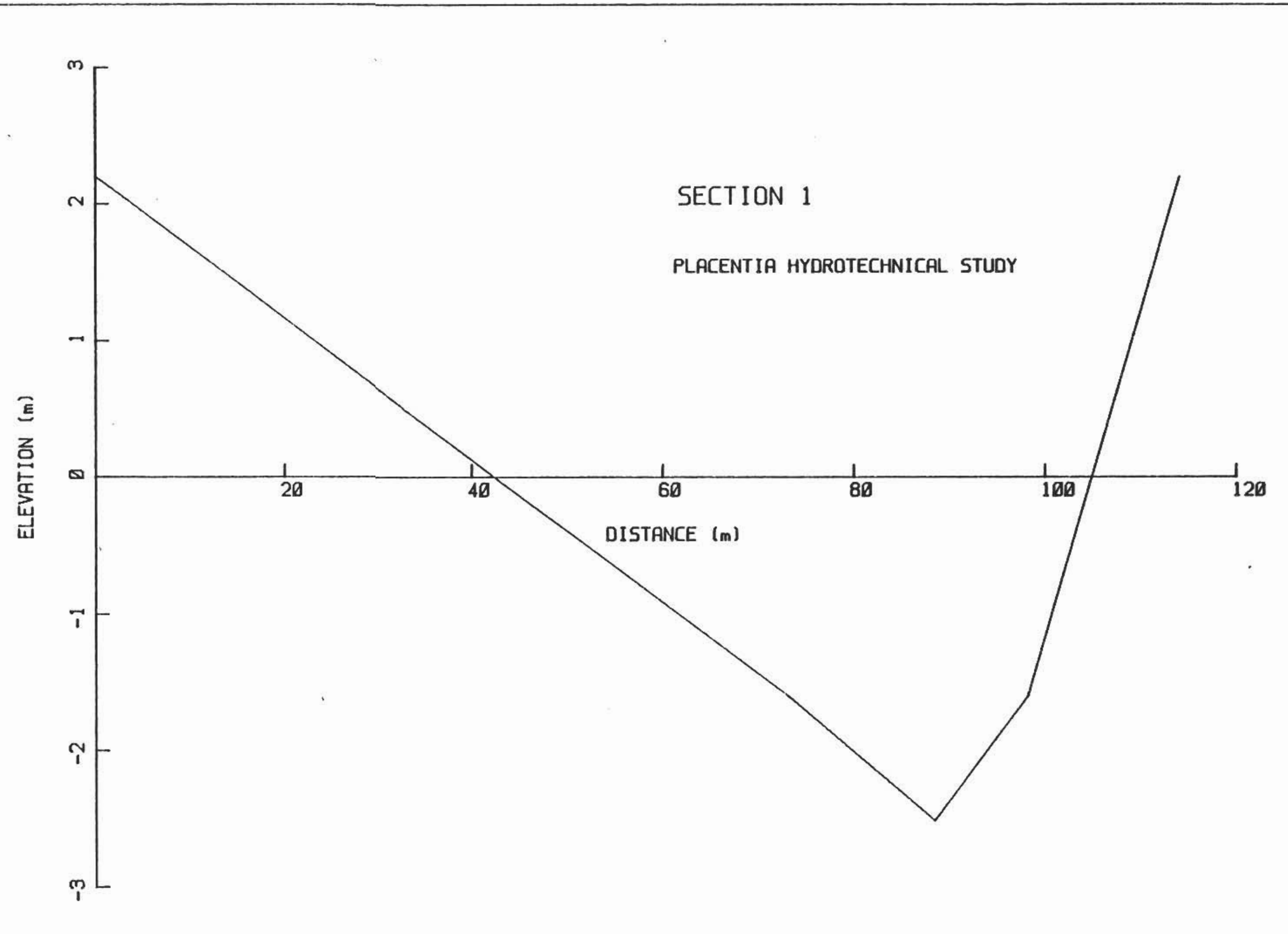
Baseflow:
Northeast Arm: 5.49 m^3/s
Southeast Arm: 7.73 m^3/s
Swan Arm: 0.76 m^3/s

Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
08 12 81	8.50	2.57	12.0
09 12 81	8.66	3.07	4.2
10 12 81	10.3	3.81	7.1
11 12 81	14.0	4.70	6.6
12 12 81	13.0	4.73	1.4
13 12 81	10.5	4.03	0.0
14 12 81	8.54	3.22	0.0

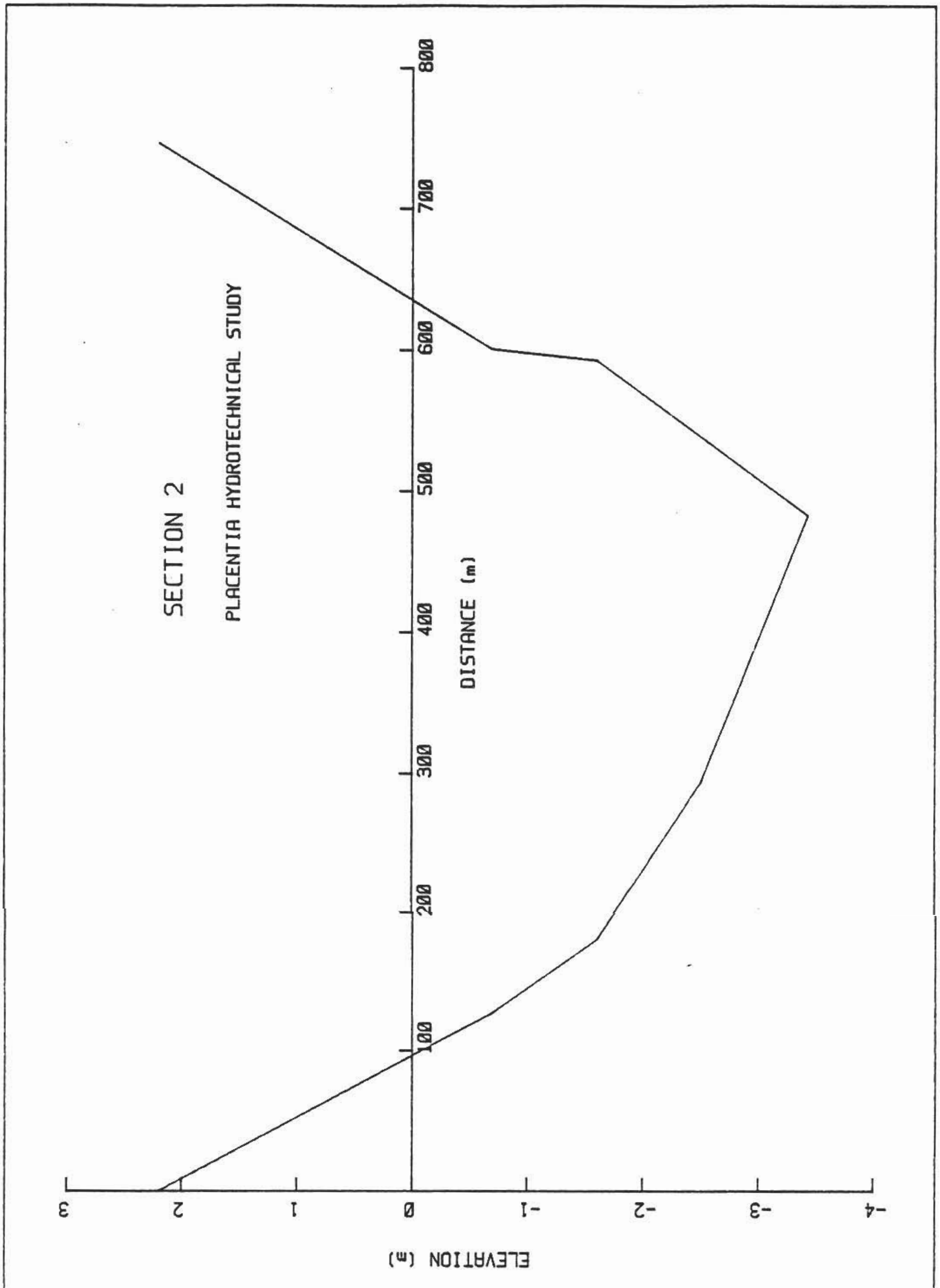
Baseflow:
Northeast Arm: 6.29 m^3/s
Southeast Arm: 8.83 m^3/s
Swan Arm: 0.85 m^3/s

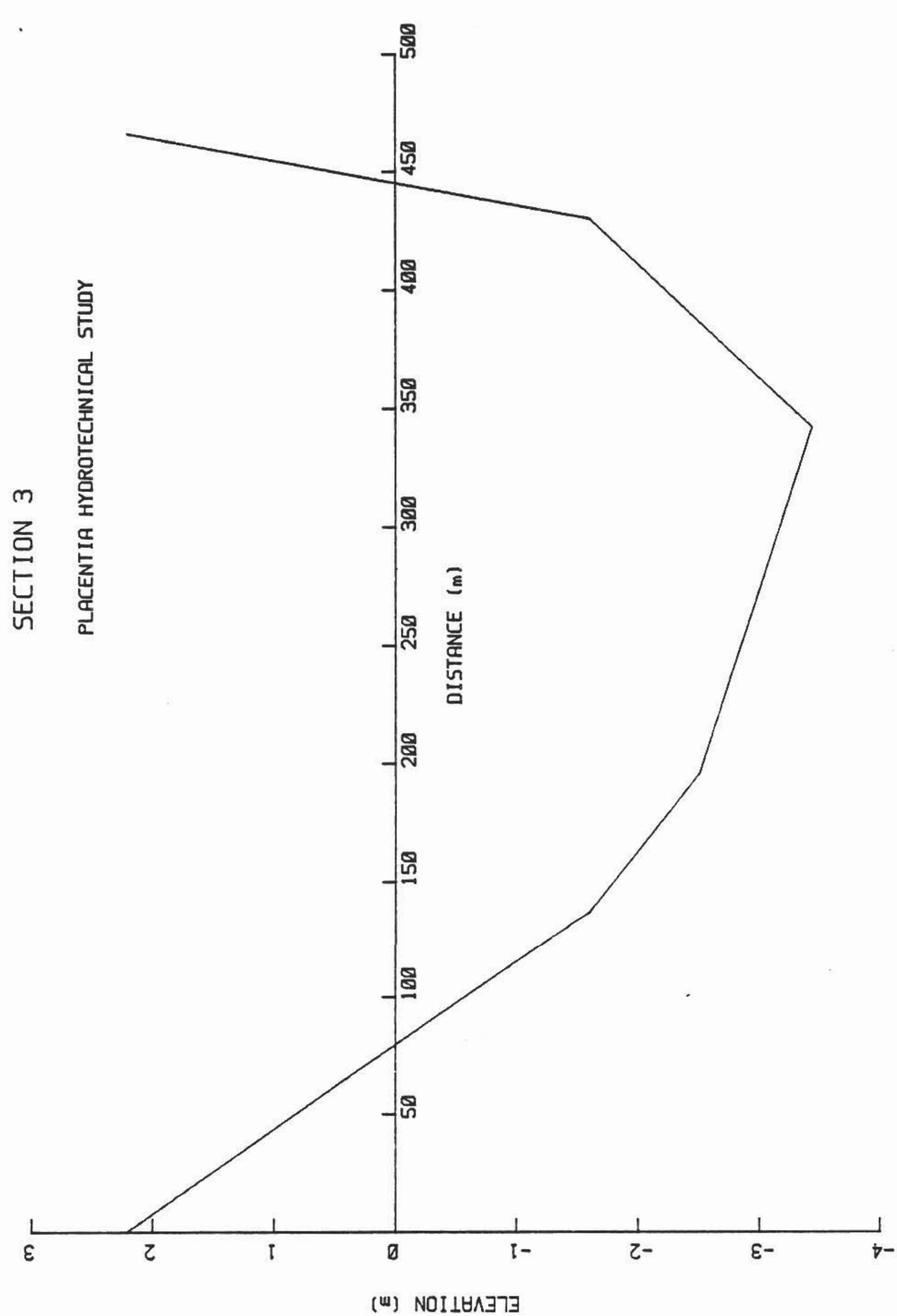
Date	Flow Rocky River (m^3/s)	Flow Northeast River (m^3/s)	Ppt Argentia (mm)
08 01 82	12.8	5.20	3.0
09 01 82	8.34	5.00	0.4
10 01 82	12.7	4.80	8.2
11 01 82	18.9	4.65	0.0
12 01 82	14.5	4.45	4.9

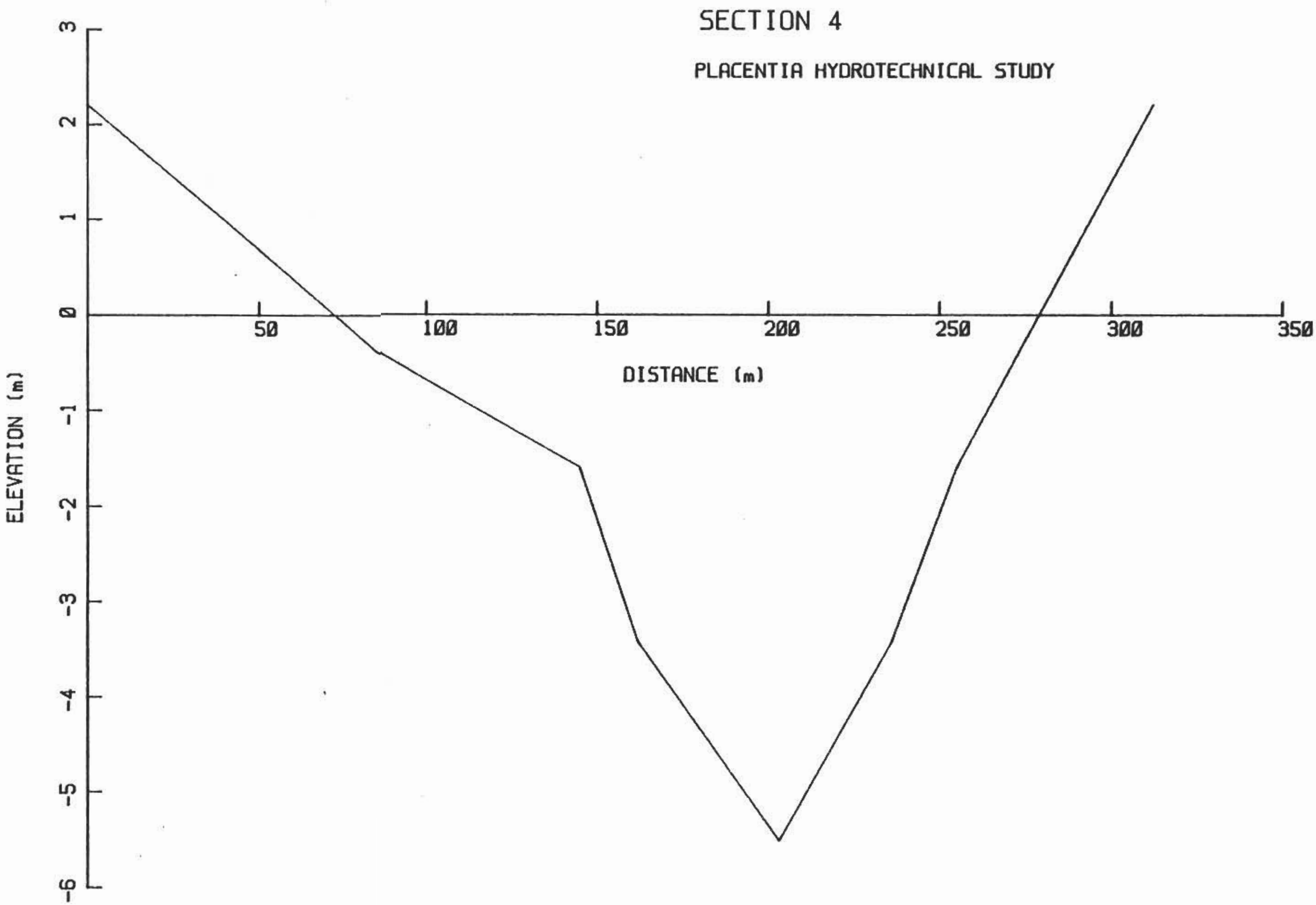
Baseflow:
Northeast Arm: 8.44 m^3/s
Southeast Arm: 11.89 m^3/s
Swan Arm: 1.16 m^3/s

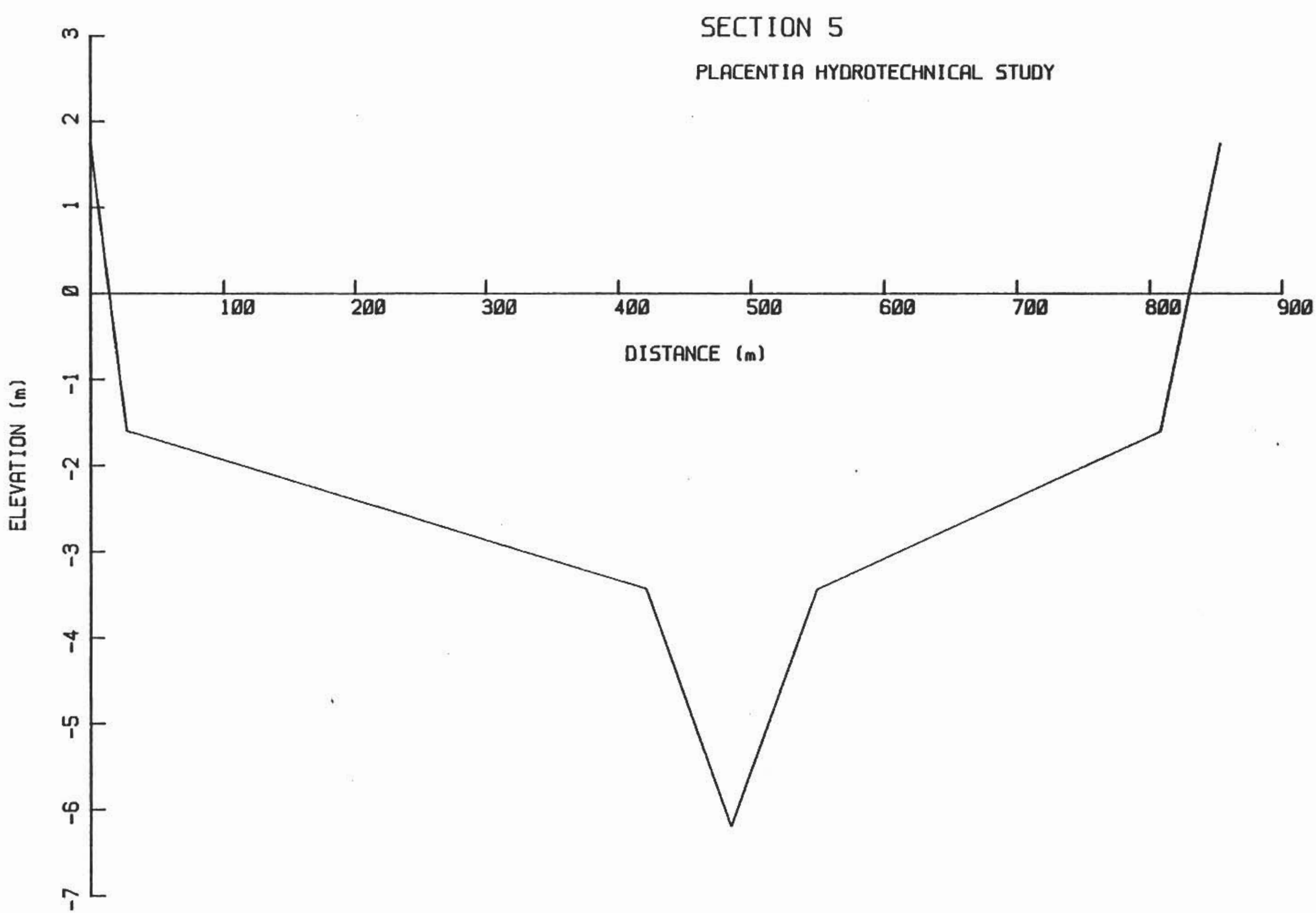


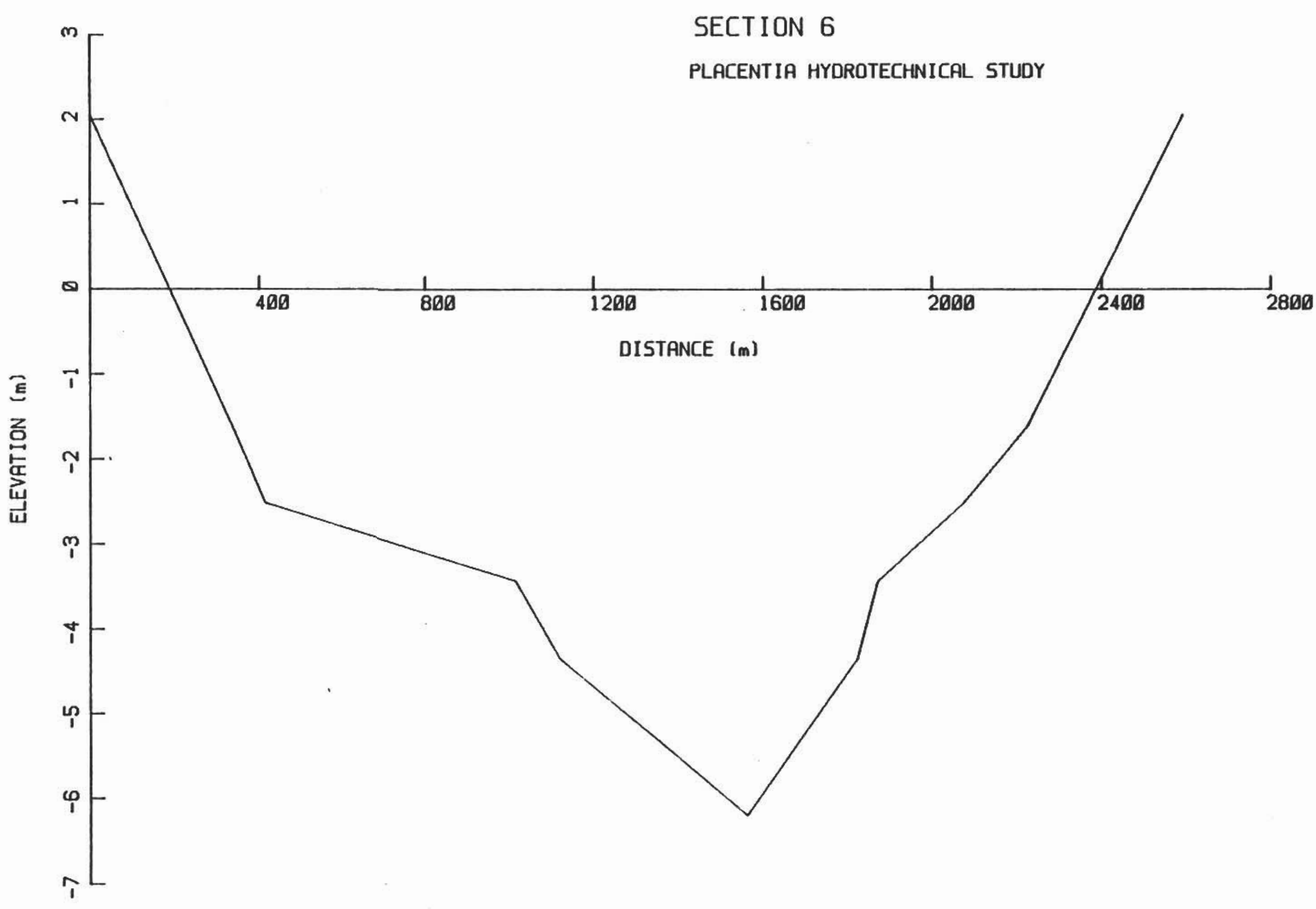
SECTION 1
PLACENTIA HYDROTECHNICAL STUDY
ELEVATION (m)
DISTANCE (m)
3
2
1
0
-1
-2
-3
20
40
60
80
100
120


SECTION 2
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
100
200
300
400
500
600
700
800
3
2
1
0
-1
-2
-3
-4

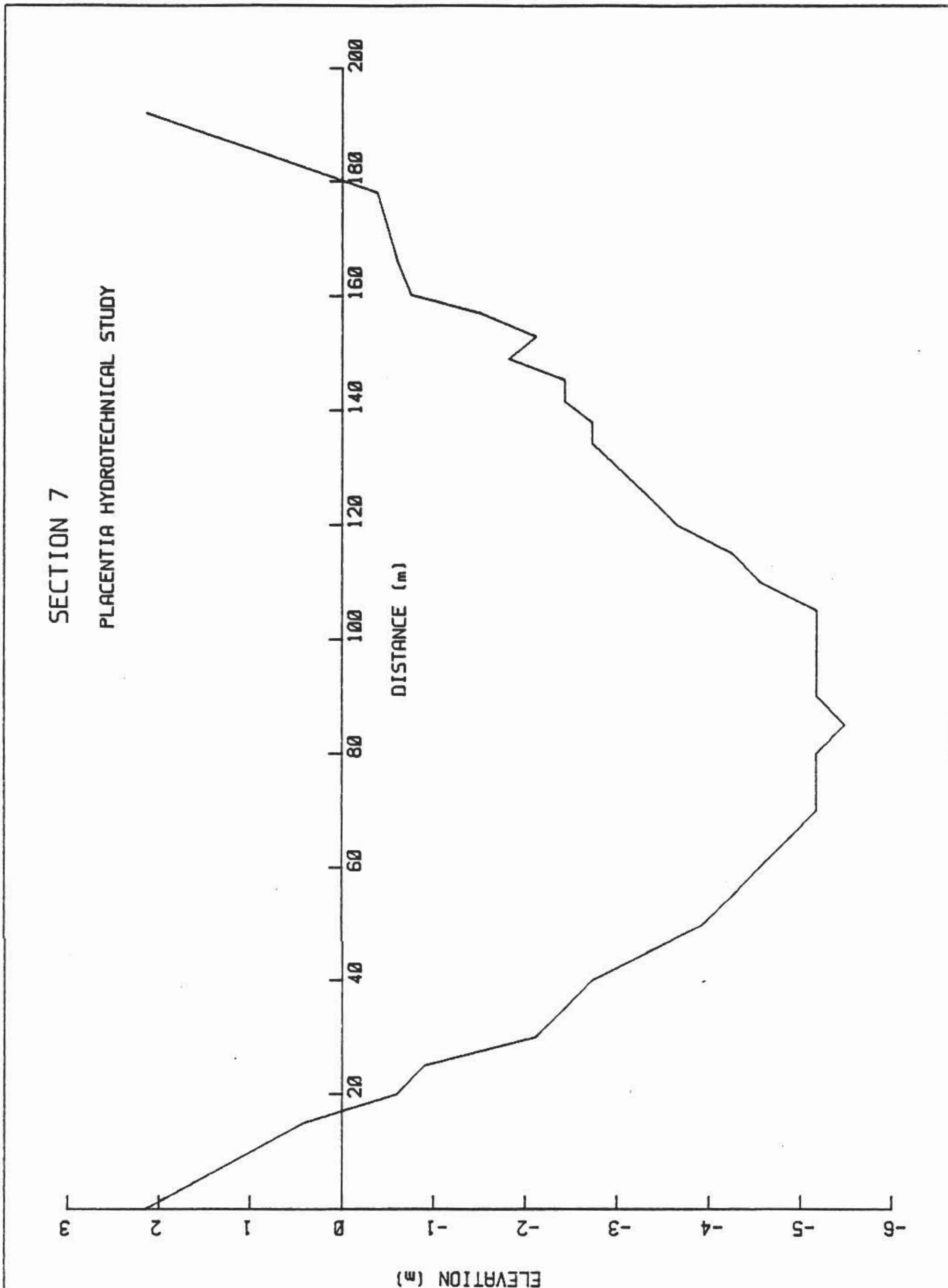

SECTION 3
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
50
100
150
200
250
300
350
400
450
500
3
2
1
0
-1
-2
-3
-4

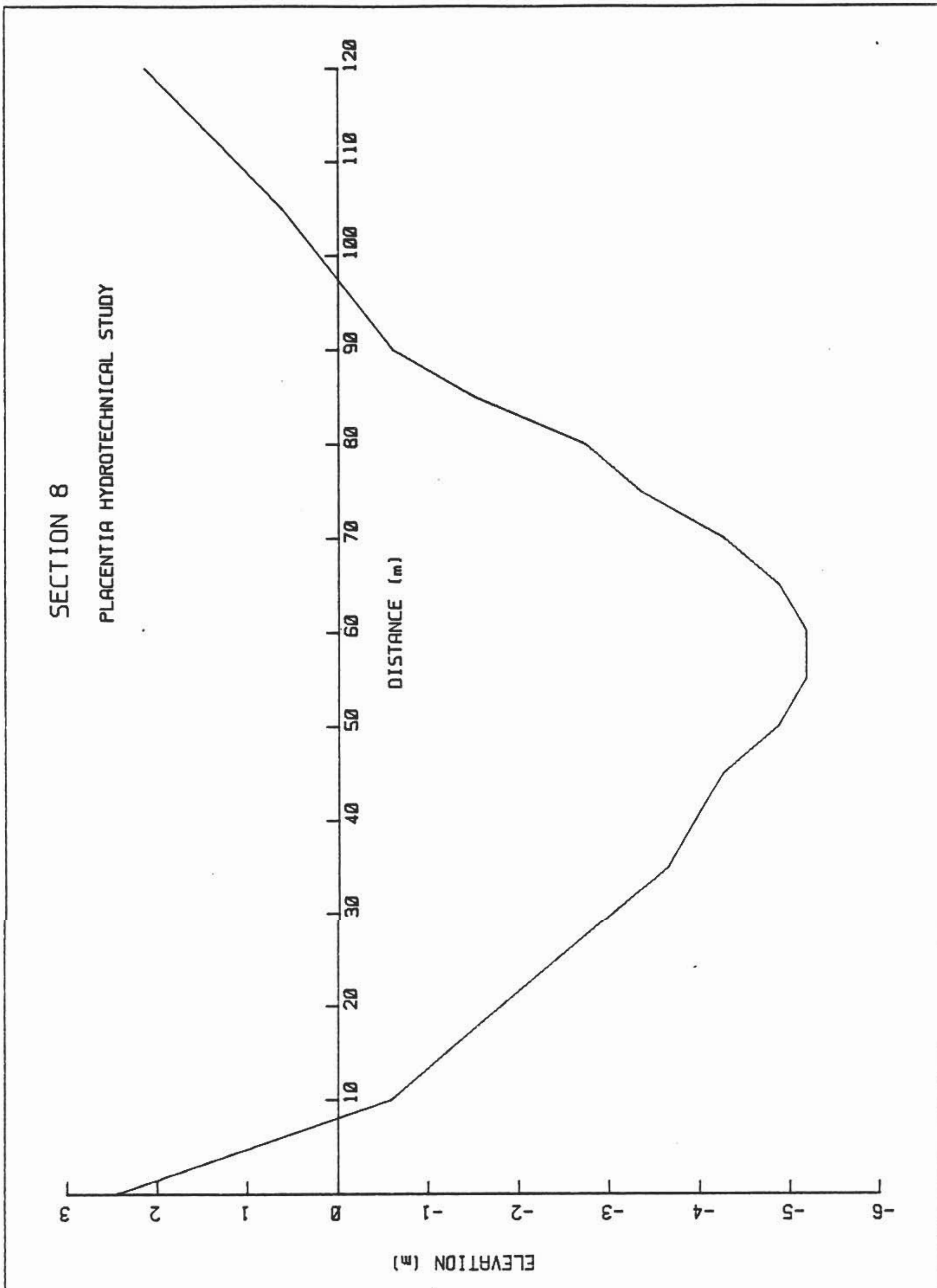

SECTION 4
PLACENTIA HYDROTECHNICAL STUDY
ELEVATION (m)
3
2
1
0
-1
-2
-3
-4
-5
-6
50
100
150
200
250
300
350
DISTANCE (m)

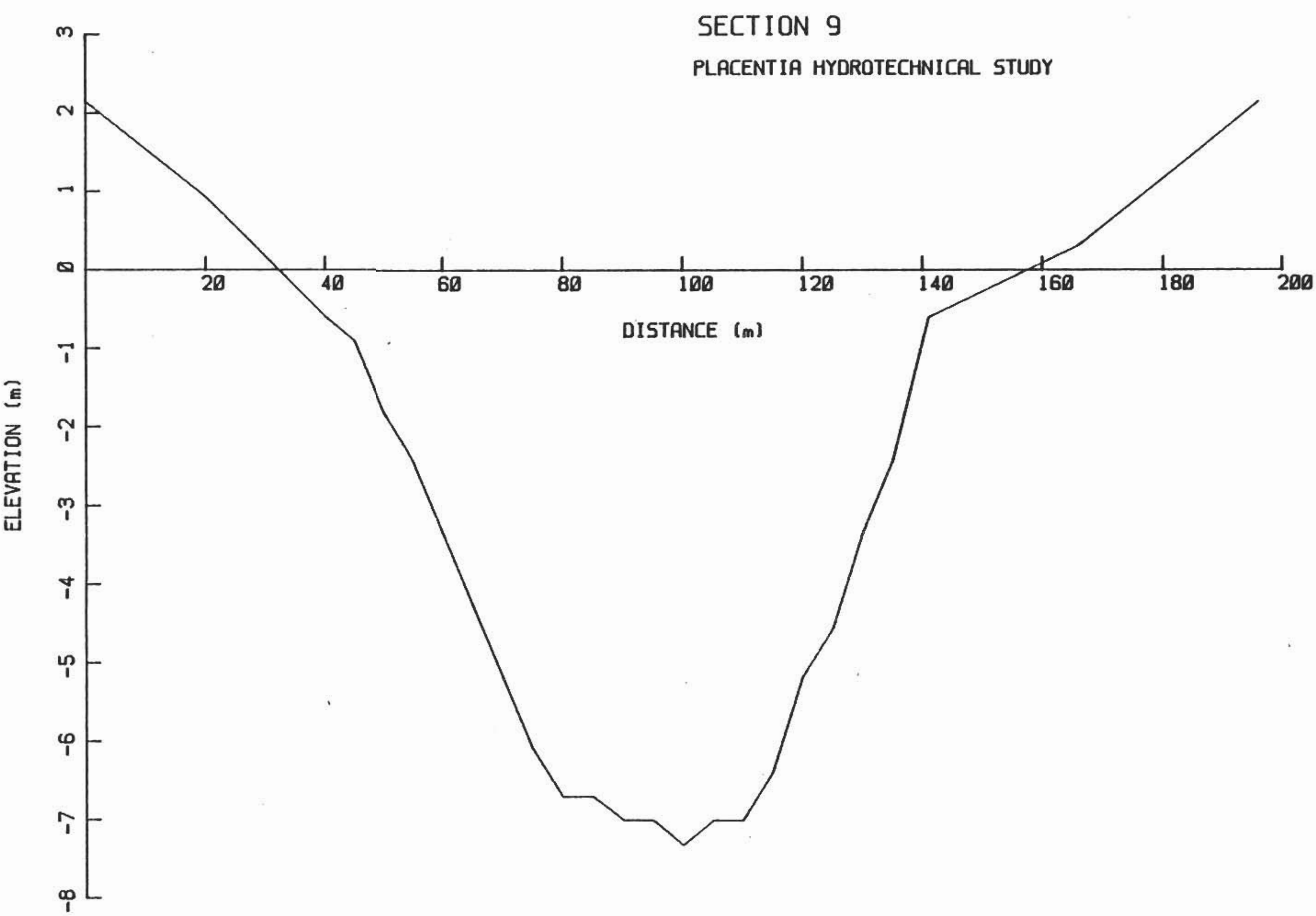

SECTION 5
PLACENTIA HYDROTECHNICAL STUDY
ELEVATION (m)
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
100
200
300
400
500
600
700
800
900
DISTANCE (m)



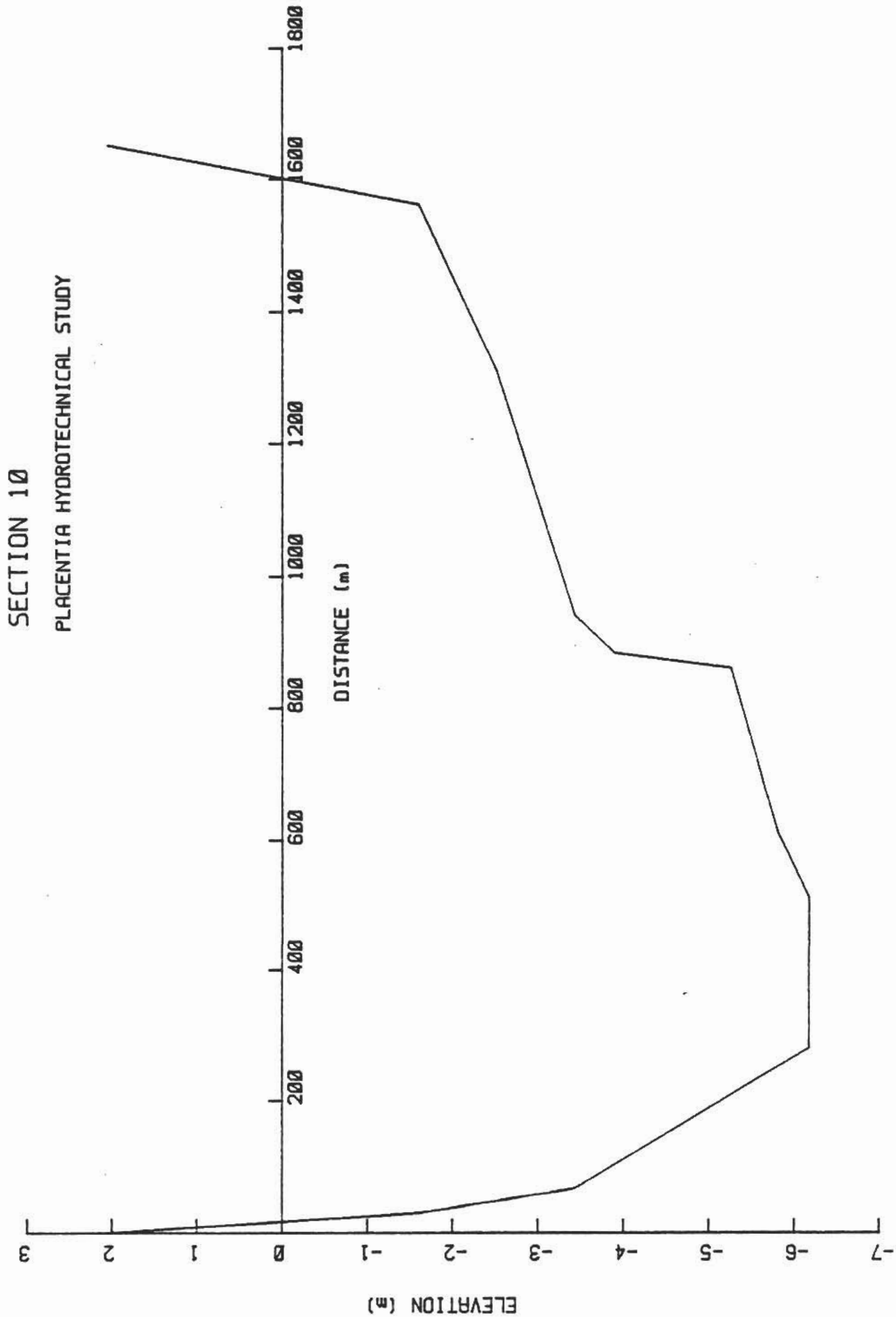
SECTION 6
PLACENTIA HYDROTECHNICAL STUDY
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
ELEVATION (m)
400
800
1200
1600
2000
2400
2800
DISTANCE (m)

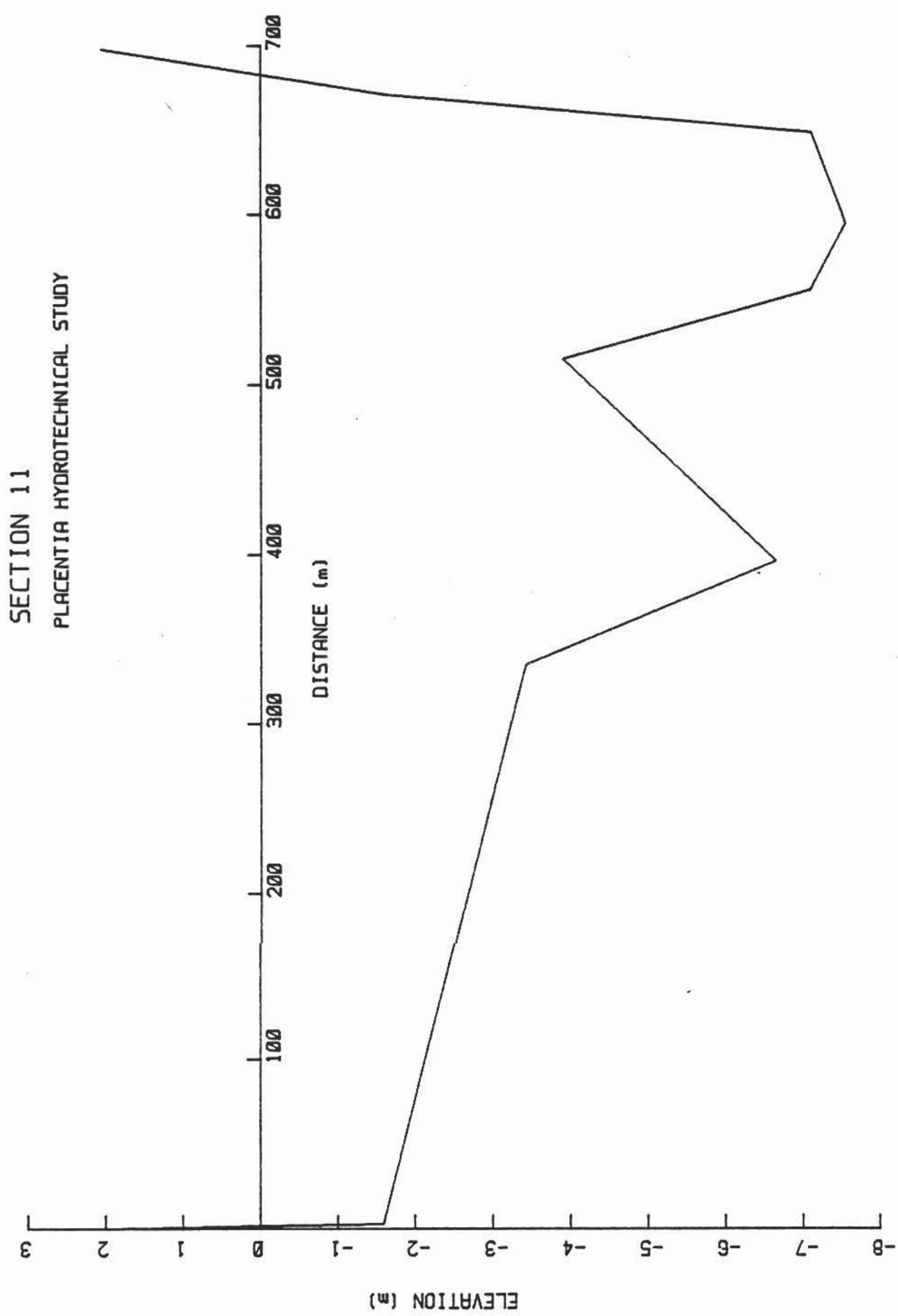

SECTION 7
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
0
20
40
60
80
100
120
140
160
180
200
3
2
1
0
-1
-2
-3
-4
-5
-6


SECTION 8
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
0
10
20
30
40
50
60
70
80
90
100
110
120
3
2
1
0
-1
-2
-3
-4
-5
-6

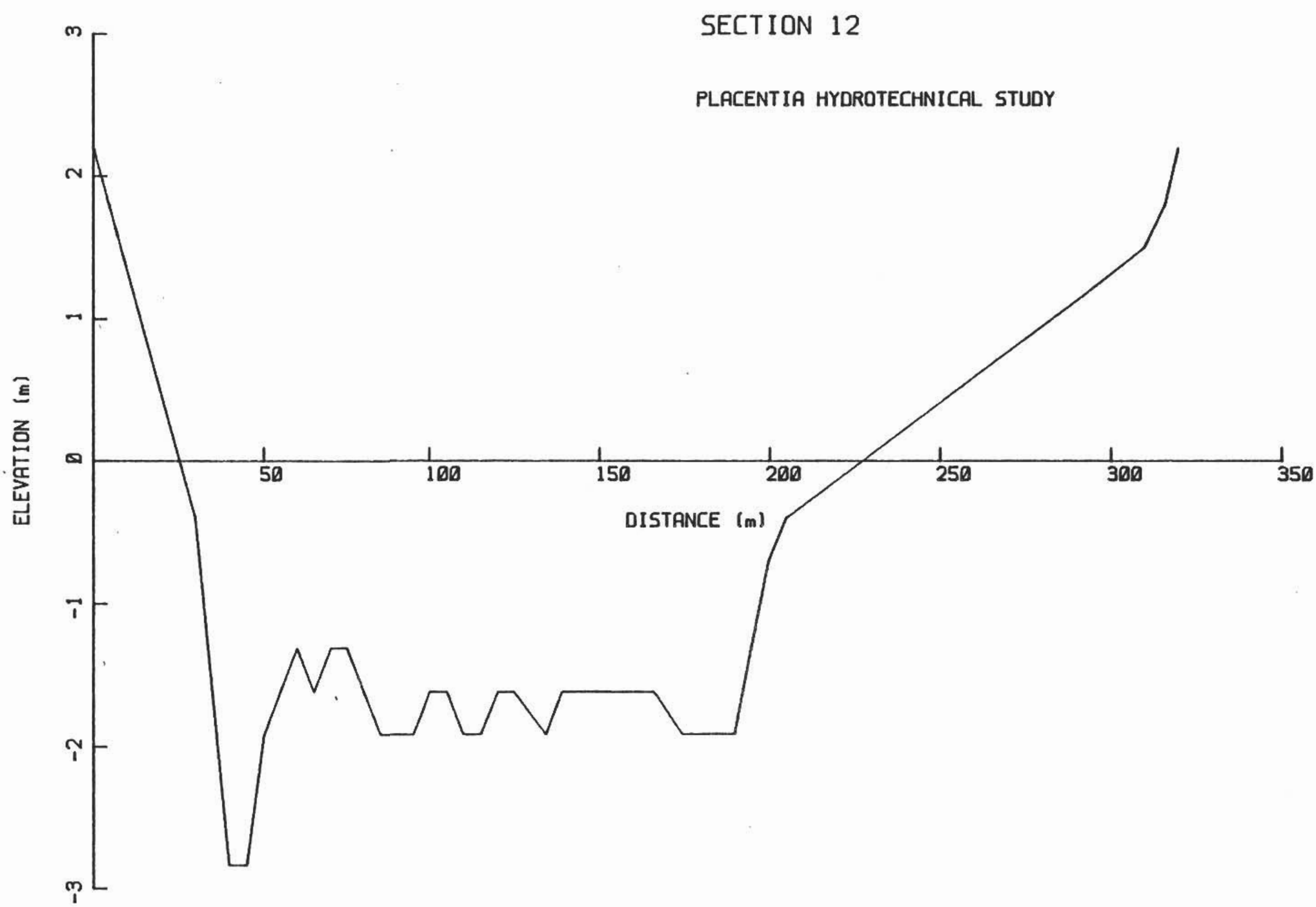


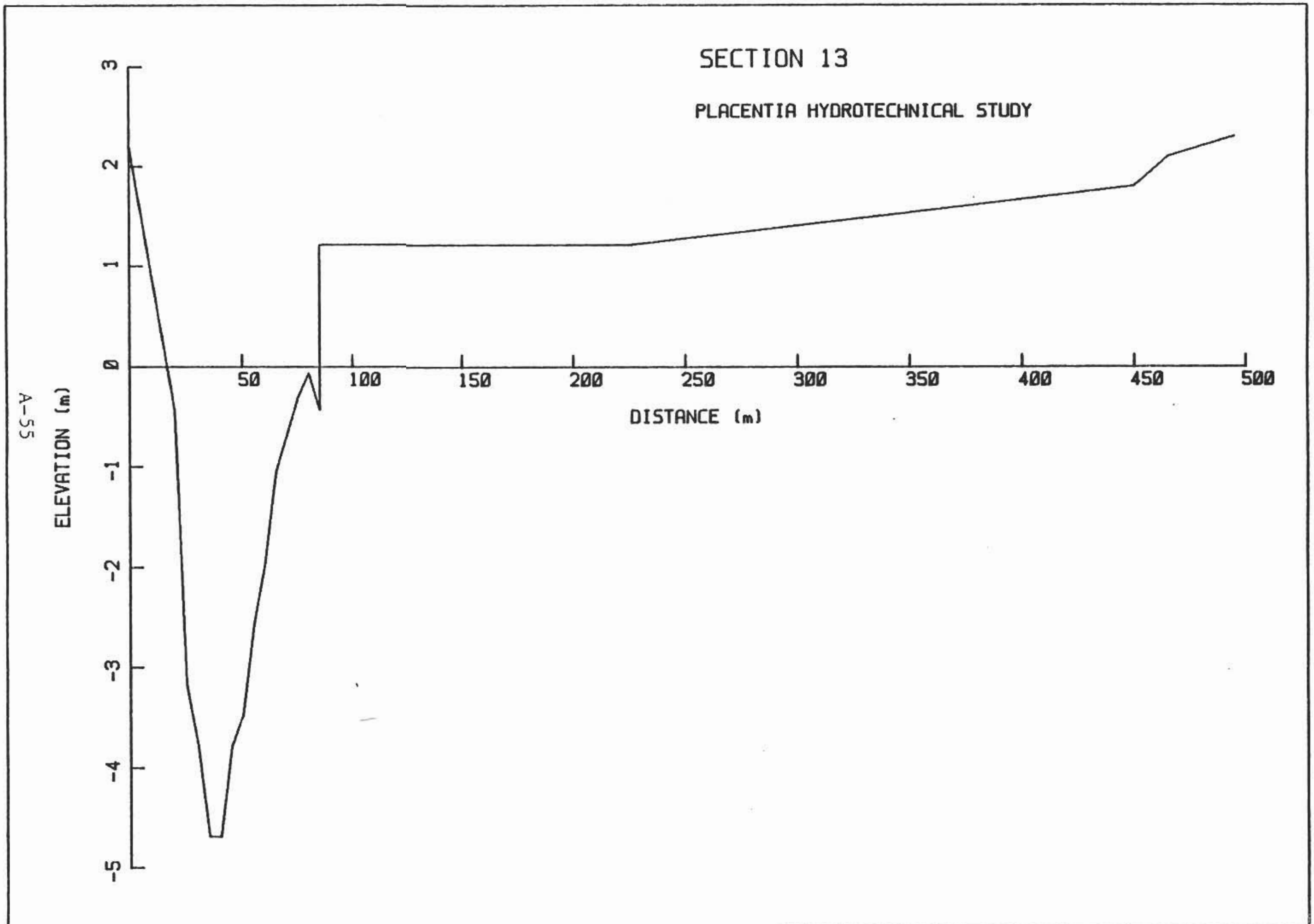
SECTION 9
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
20
40
60
80
100
120
140
160
180
200
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8

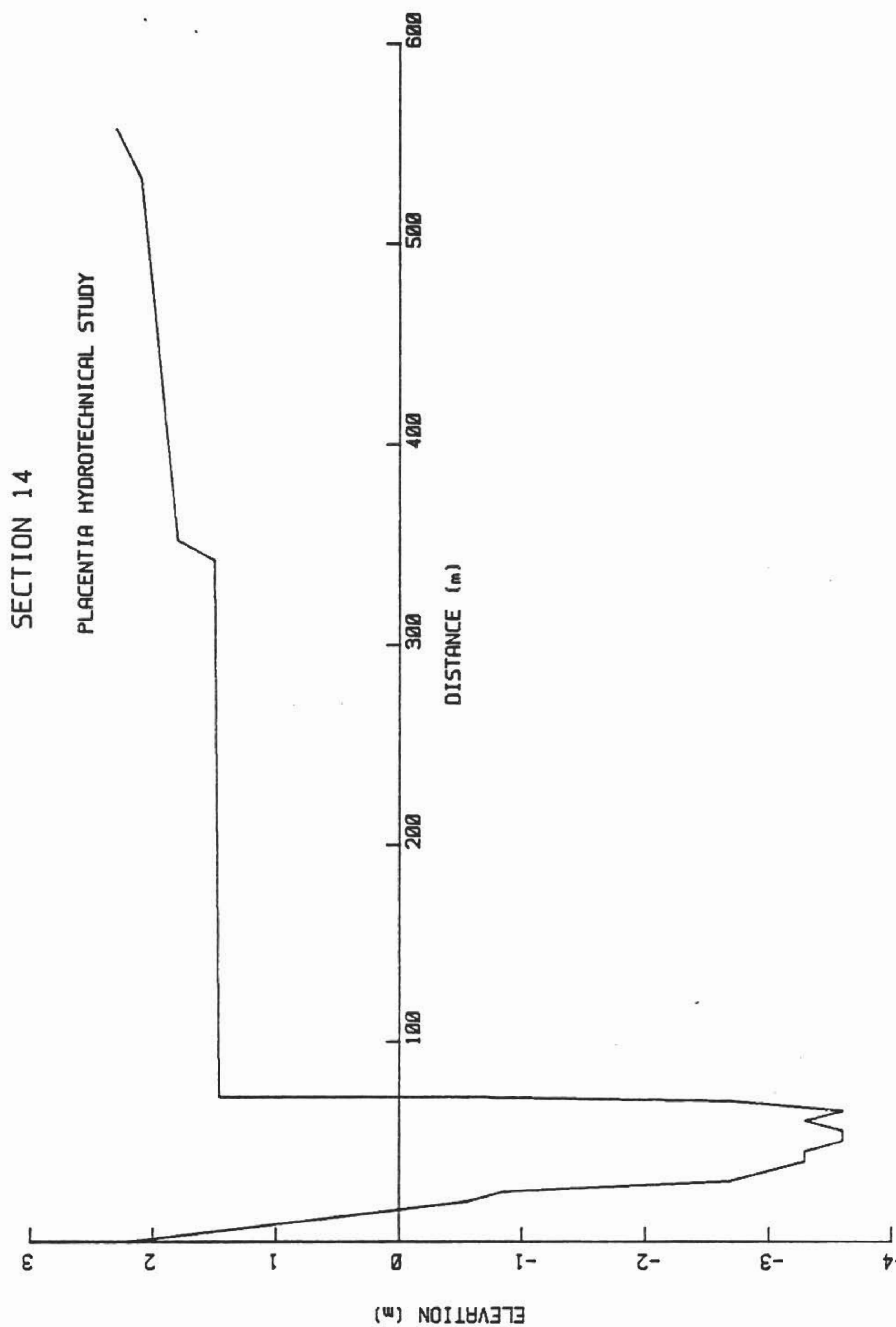

SECTION 10
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
200
400
600
800
1000
1200
1400
1600
1800
3
2
1
0
-1
-2
-3
-4
-5
-6
-7



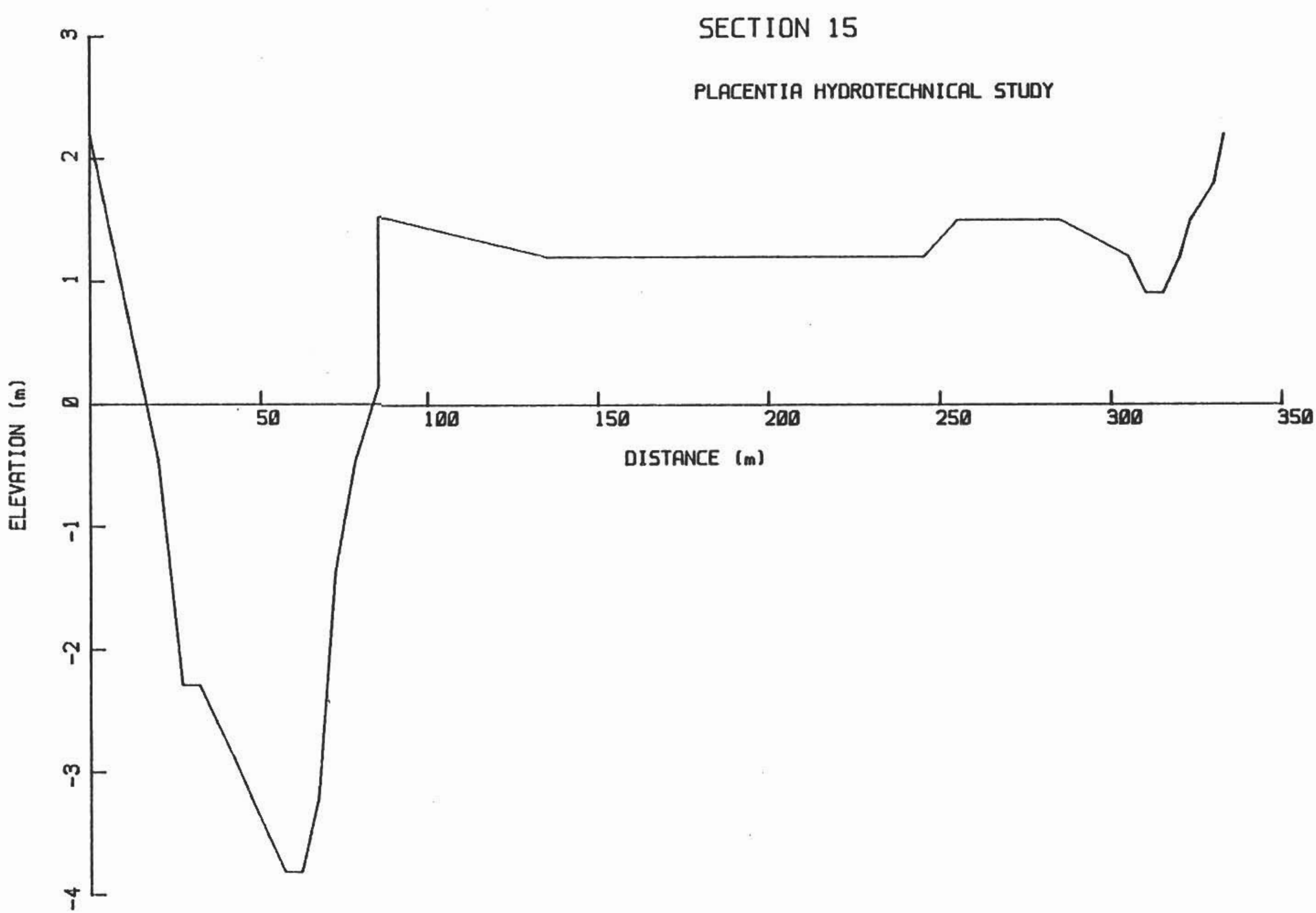
SECTION 11
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
0
100
200
300
400
500
600
700
ELEVATION (m)
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8


SECTION 12
PLACENTIA HYDROTECHNICAL STUDY
ELEVATION (m)
DISTANCE (m)
3
2
1
0
-1
-2
-3
50
100
150
200
250
300
350

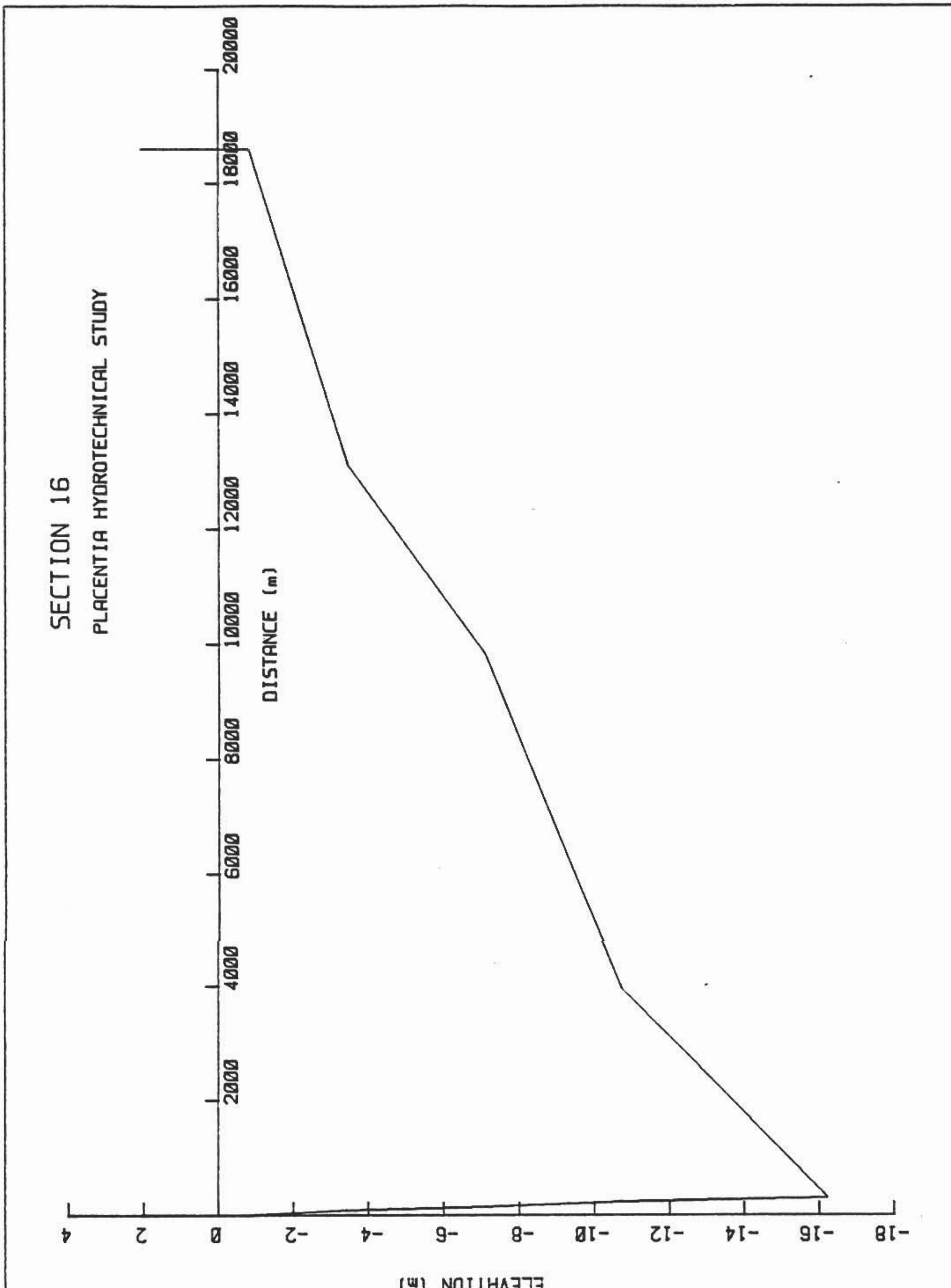

SECTION 13
PLACENTIA HYDROTECHNICAL STUDY
ELEVATION (m)
3
2
1
0
-1
-2
-3
-4
-5
50
100
150
200
250
300
350
400
450
500
DISTANCE (m)

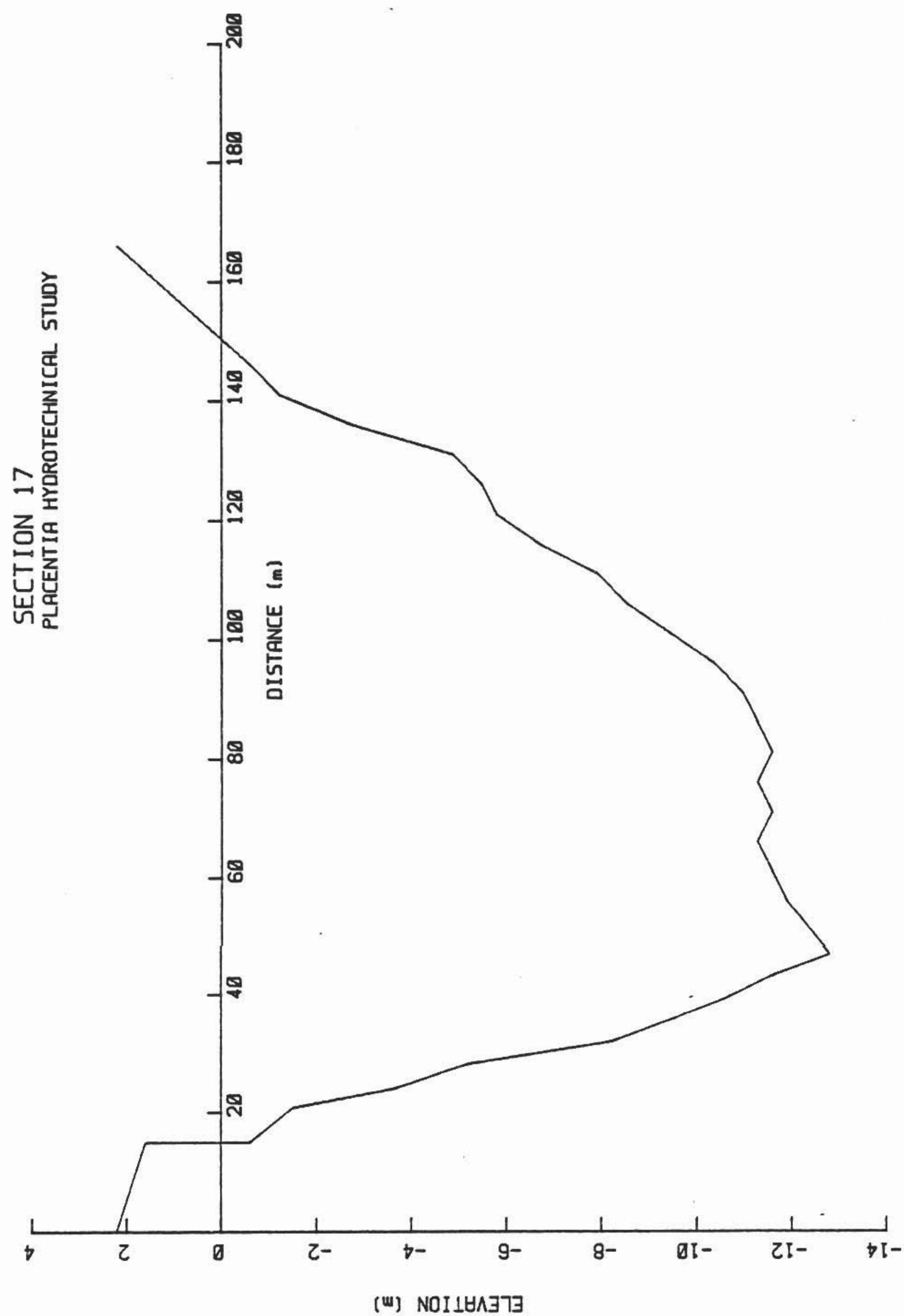


SECTION 14
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
0
100
200
300
400
500
600
3
2
1
0
-1
-2
-3
-4

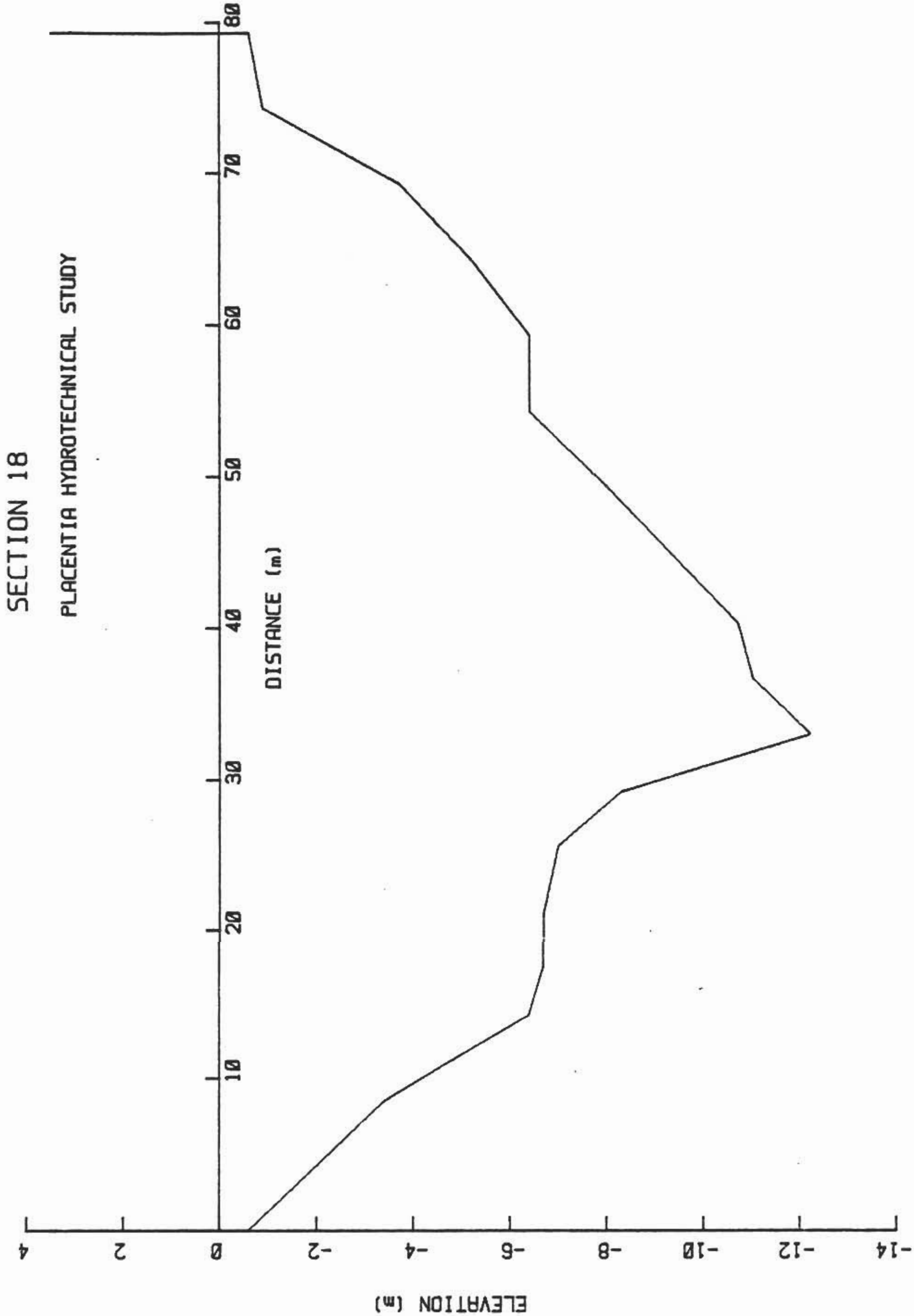


SECTION 15
PLACENTIA HYDROTECHNICAL STUDY
ELEVATION (m)
3
2
1
0
-1
-2
-3
-4
50
100
150
200
250
300
350
DISTANCE (m)

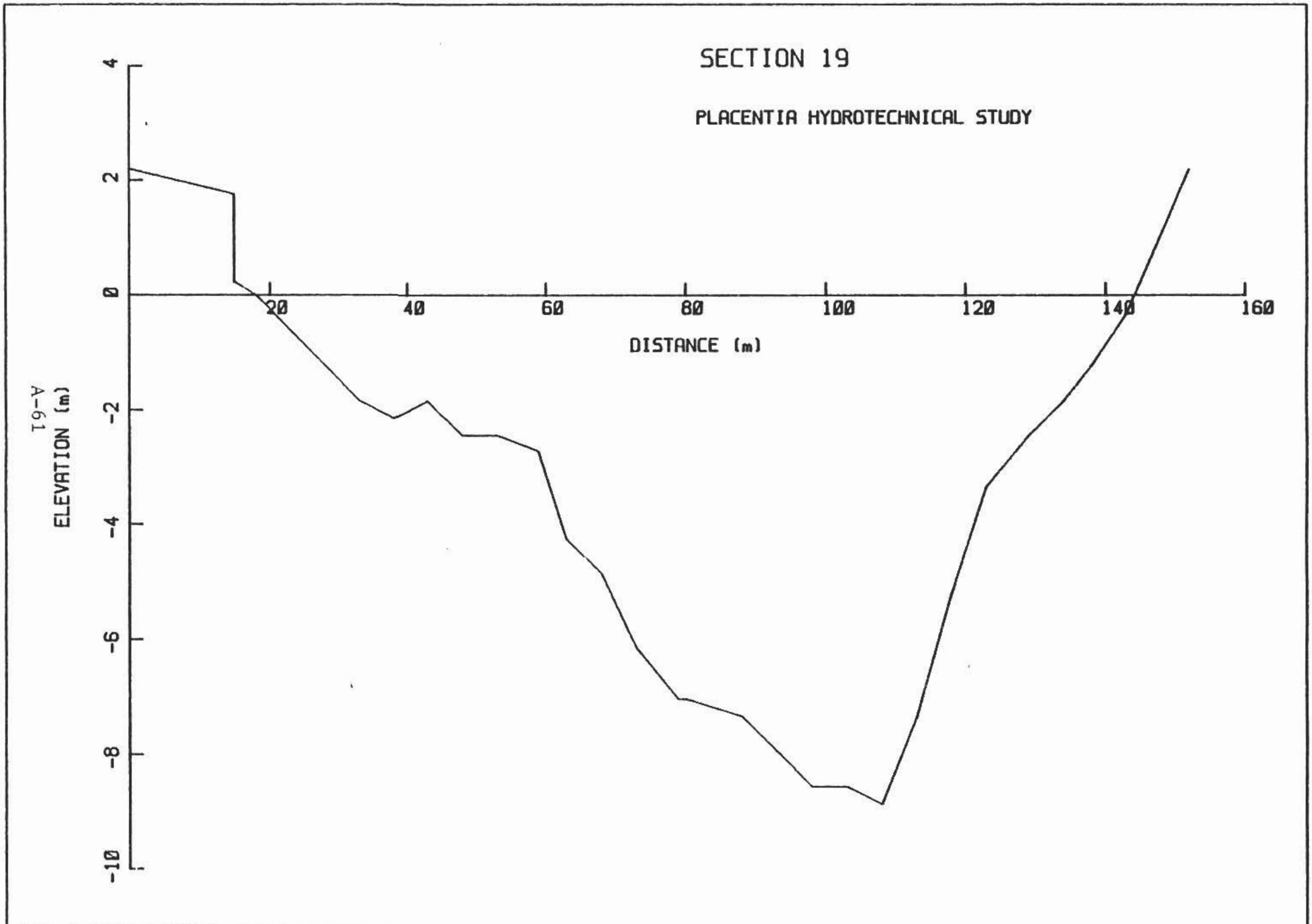

SECTION 16
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18



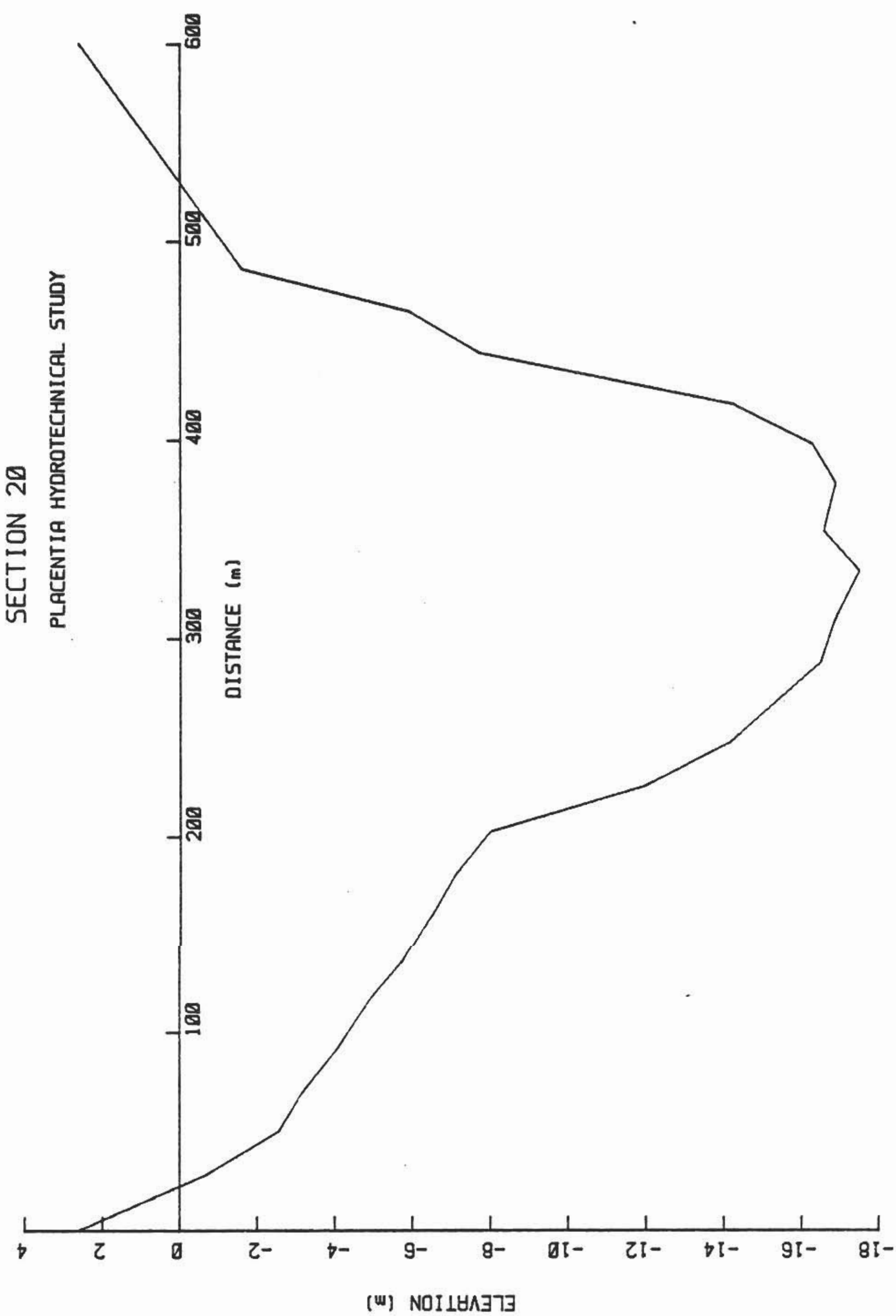
SECTION 17
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
0
20
40
60
80
100
120
140
160
180
200
4
2
0
-2
-4
-6
-8
-10
-12
-14



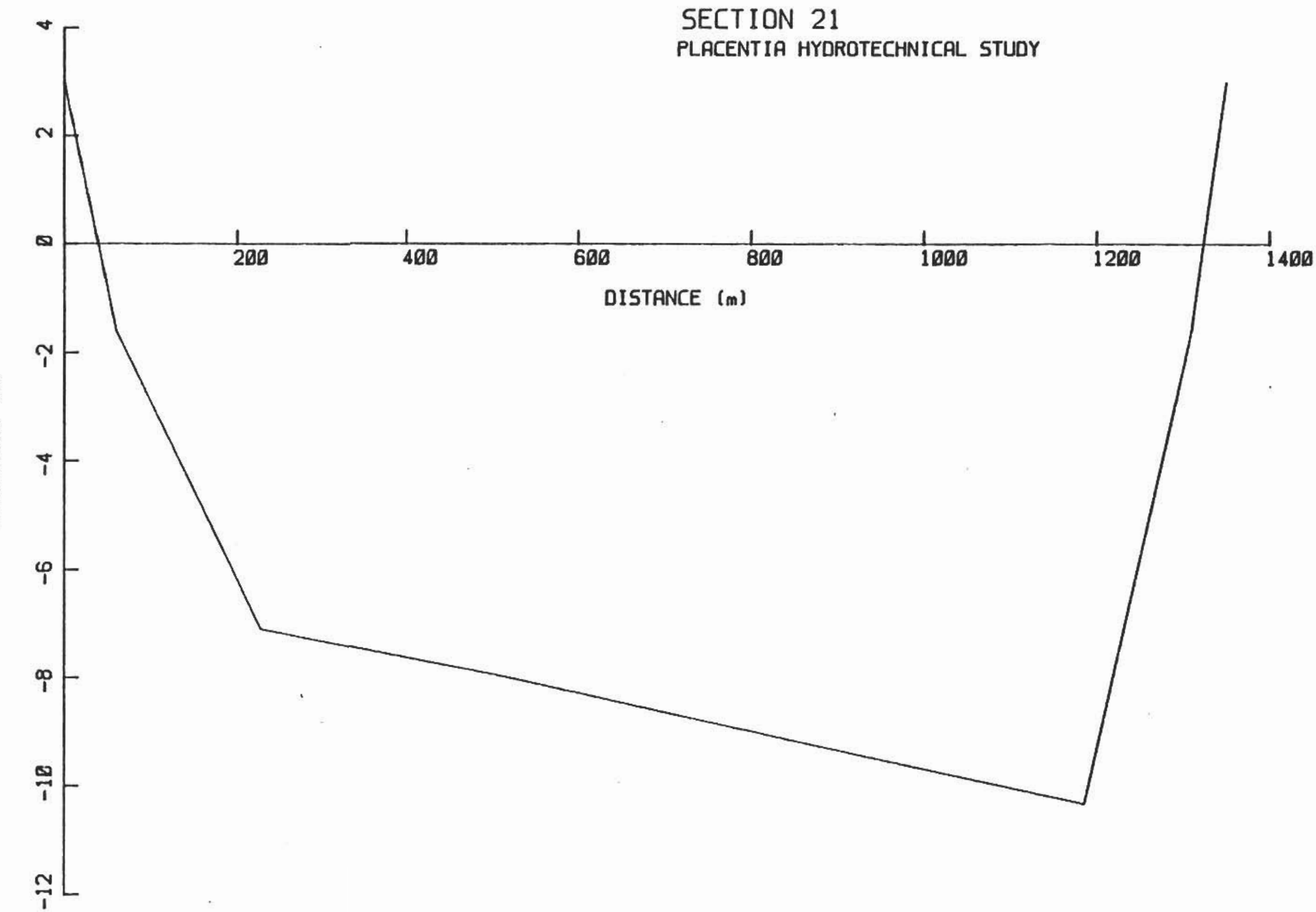
SECTION 18
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
0
10
20
30
40
50
60
70
80
4
2
0
-2
-4
-6
-8
-10
-12
-14



SECTION 19
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
4
2
0
-2
-4
-6
-8
-10
20
40
60
80
100
120
140
160



SECTION 20
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
100
200
300
400
500
600
4
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-18


SECTION 21
PLACENTIA HYDROTECHNICAL STUDY
DISTANCE (m)
ELEVATION (m)
200
400
600
800
1000
1200
1400
4
2
0
-2
-4
-6
-8
-10
-12

Setup parameters decribed in manual.

2 stations with lateral inflow

Not Used

Not Used

There are 7 stations with off channel storage.

Stations with off channel storage.

# of stations (x-sections)

upstream observations flow in cfs

downstream observation (boundary) is water level in feet.

there are 7 "N" reaches; 13 observation points; 0 tributaries.

"N" is same for sta 1-7, 7-9, 9-13, etc.

stations with observed water levels.

Datum for Swan Arm, Northeast Arm and Placentia Road

Observed water levels - Swan Arm

Observed water levels - Northeast Arm

Observed water levels - Placentia Road

Runoff - Southeast River

Observed water levels - Placentia Road Driving force - tidal Boundary

Datum

Top widths of cross-sections

Cross-section data elevations corresponding to top widths

area below lowest top width for each Cross-Section

Cross Section data for off channel storage on Southeast Arm

off channel storage on MacDonald Cove (Swan Arm)

off channel storage on Cross Section 12

off channel storage on Cross Section 13

off channel storage on Cross Section 14

off channel storage on Cross Section 15

off channel storage Northeast Arm

wind on river; coefficient, velocity (ft/s), direction

station of cross-sections in meters.

expansion and contraction coefficients

Run off into Swan Arm

Run off from Northeast River

Maximum # of observations, computation time step (1hr) and output time step (1hr)

initial water level at all stations

(initial flow at each (station

Maximum elevation to which "N" is valid

Manning's "N" values

Description of run.

Initial run off at each station.

FCST = forecast of computed

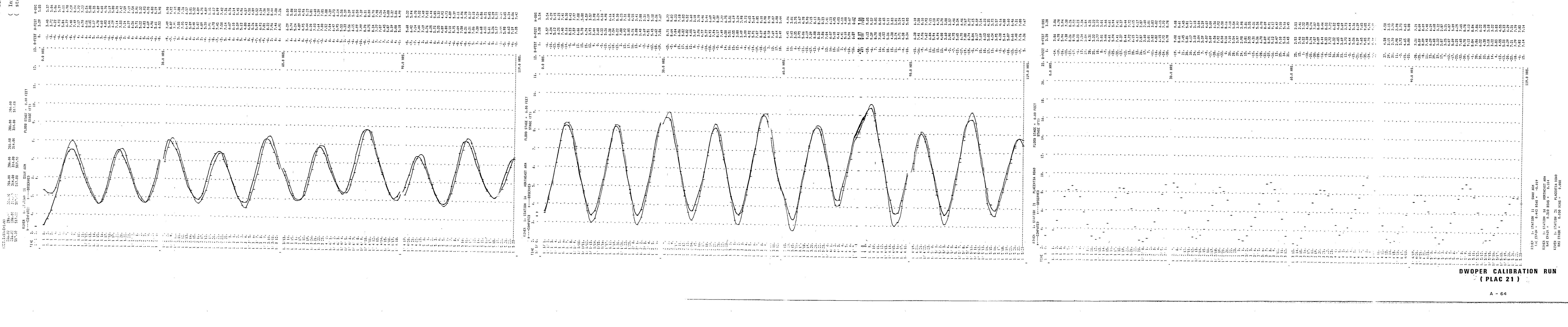
Q-FCST = computed flows ($x10^{3}$ cfs)

H-FCST = computer water levels (ft)

H-OBS = observed water level (water level - 5.25 = geodetic datum)

1/1/10 = 1/day/hour

Hours from start time.

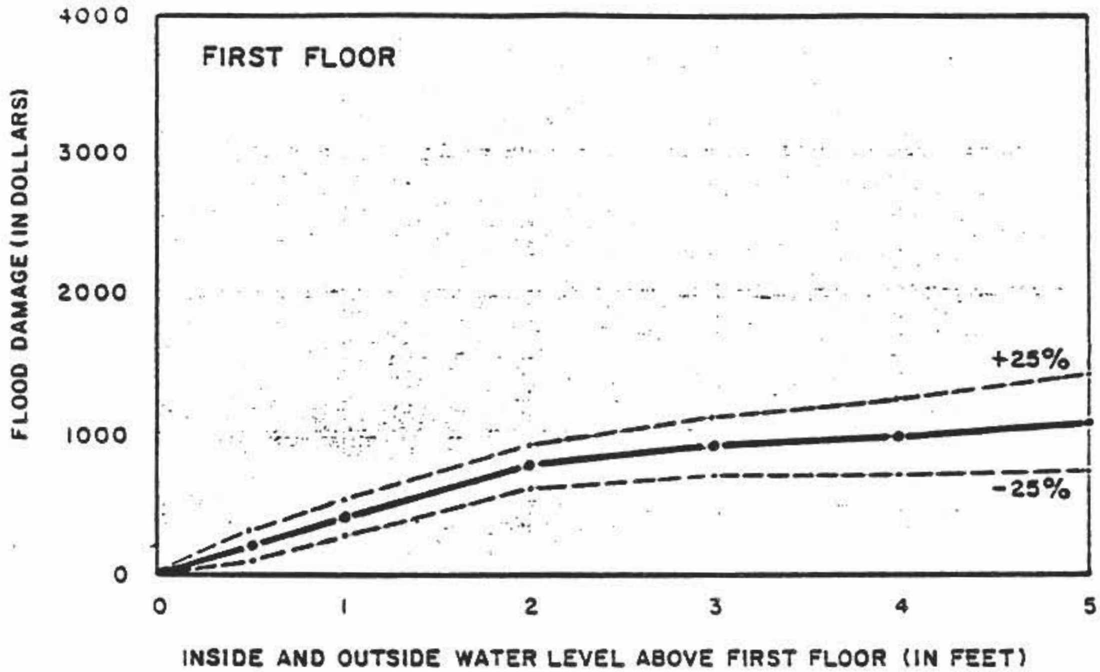


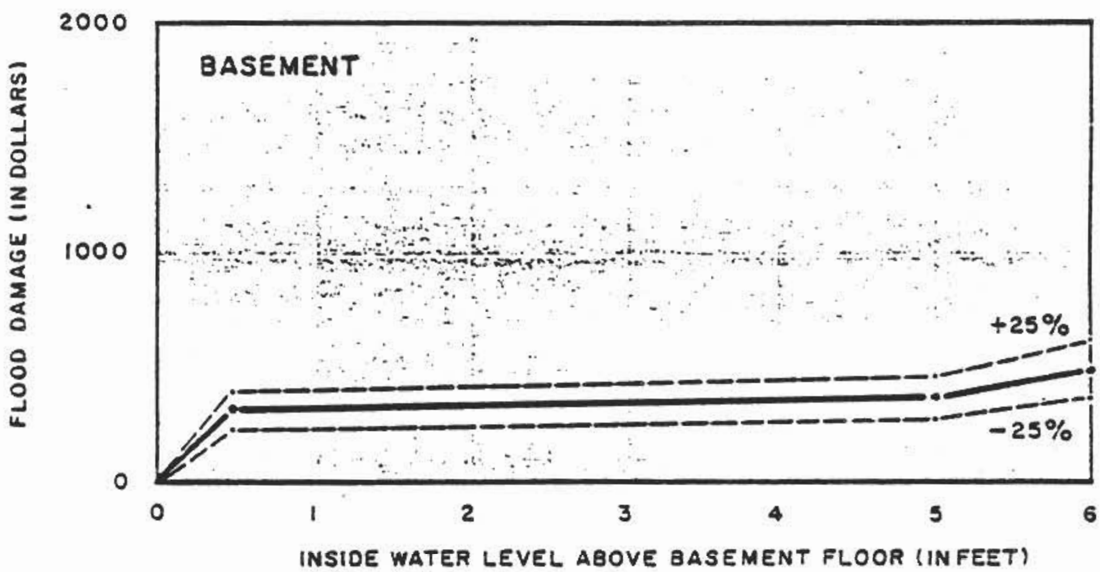
DWOPER CALIBRATION RUN
(PLAC 21)

APPENDIX IV

COST DATA - PHASE I

1. Stage Damage Curves for Different House Types.
2. Sample Questionnaires
3. Cost Estimates (Phase I)

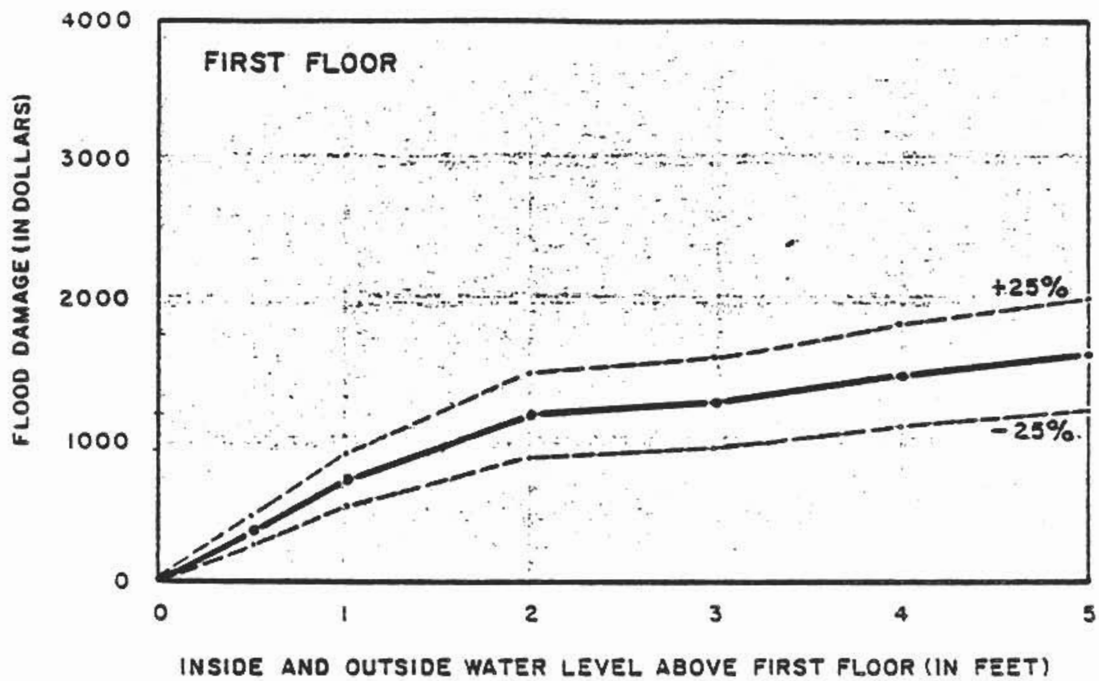


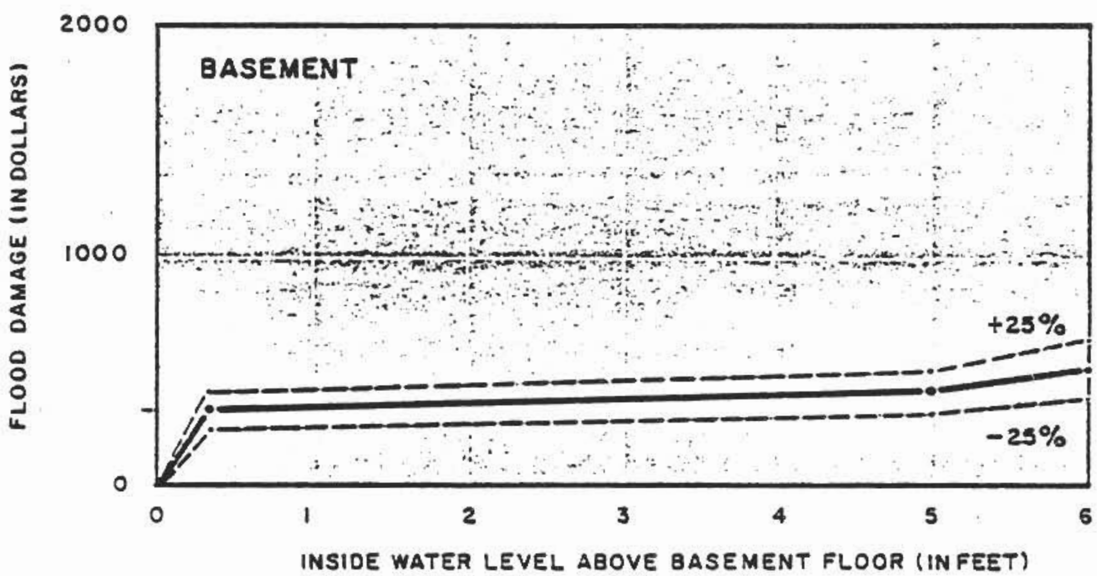


NOTE: ±25% DENOTES LEVEL OF ACCURACY

AVERAGE STRUCTURAL DAMAGES:
C W HOMES

SOURCE: ACRES LIMITED GUIDELINES FOR ANALYSIS, VOLUME 2, FLOOD DAMAGES, AUGUST 1968

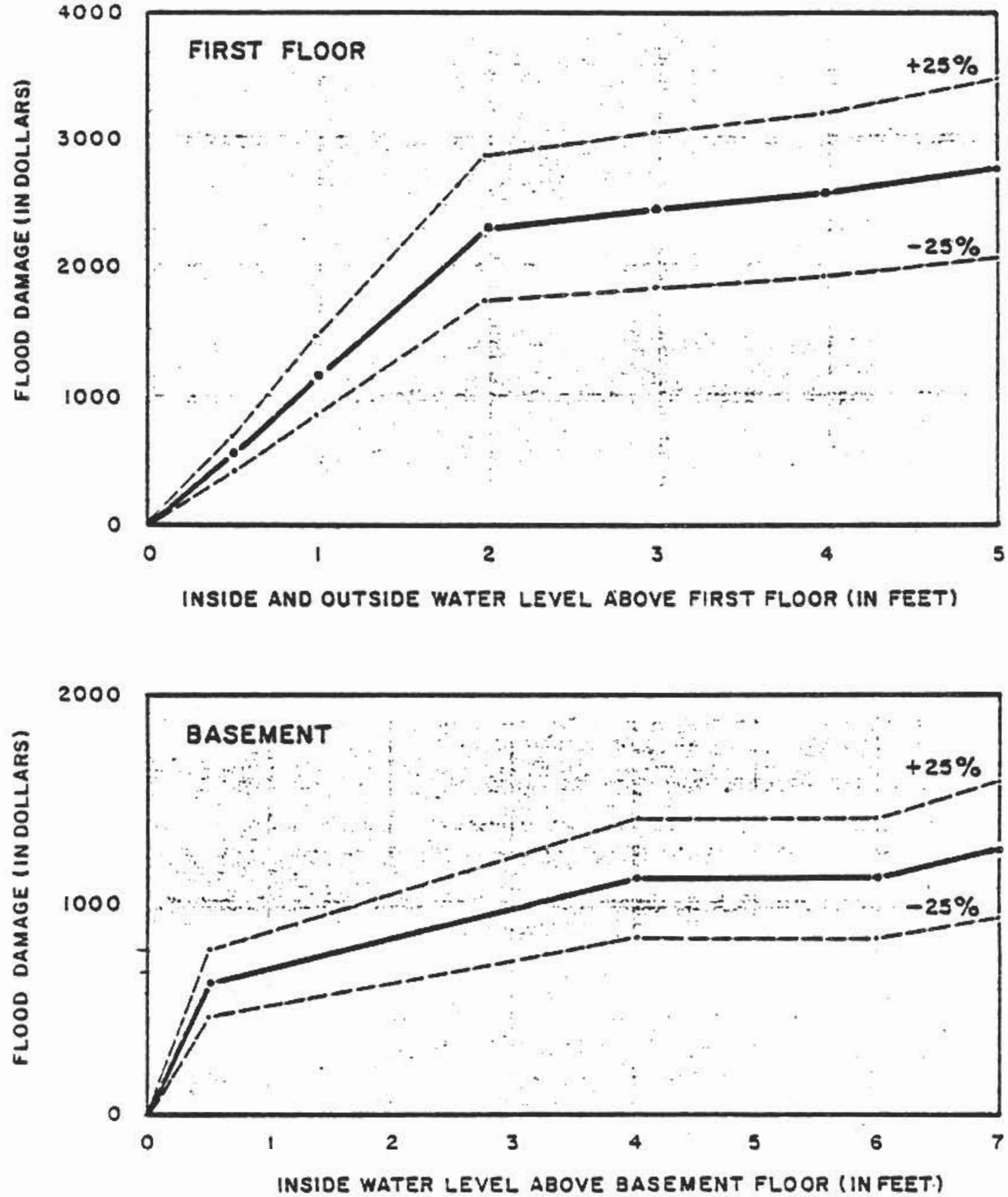




NOTE: ±25% DENOTES LEVEL OF ACCURACY

AVERAGE STRUCTURAL DAMAGES: B W HOMES

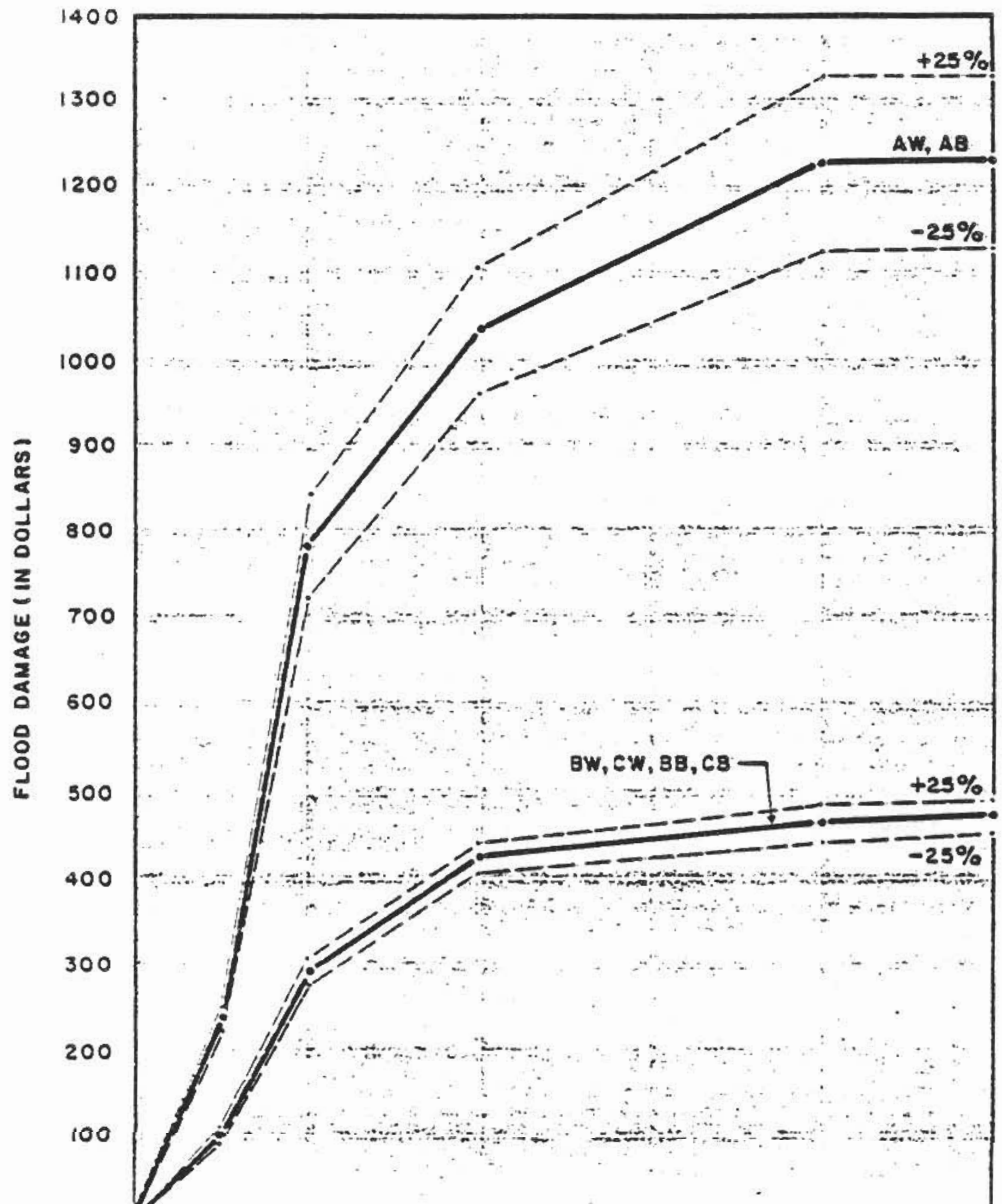
SOURCE: ACRES LIMITED GUIDELINES FOR ANALYSIS, VOLUME 2, FLOOD DAMAGES, AUGUST 1968



NOTE: ±25% DENOTES LEVEL OF ACCURACY

AVERAGE STRUCTURAL DAMAGES:
A W HOMES

SOURCE: ACRES LIMITED GUIDELINES FOR ANALYSIS, VOLUME 2, FLOOD DAMAGES, AUGUST 1968



NOTE: ±25% DENOTES LEVEL OF CONFIDENCE

AVERAGE DAMAGE TO CONTENTS OF ALL HOMES: FIRST FLOOR

SOURCE: ACRES LIMITED GUIDELINES FOR ANALYSIS, VOLUME 2, FLOOD DAMAGES, AUGUST 1968

FLOOD VICTIMS

"QUESTIONNAIRE"

I. NAME: ______________________

ADDRESS: ______________________

AGE: ______________________

OCCUPATION: EMPLOYED ☐ UNEMPLOYED ☐ RETIRED ☐

NUMBER OF OCCUPANTS: ______________

2. FLOODING

EXTERIOR ONLY	YES ☐	NO ☐	APPROX. DEPTH	______
INTERIOR ONLY	YES ☐	NO ☐	APPROX. DEPTH	______
FIRST TIME FLOODED	YES ☐	NO ☐		

NUMBER OF TIMES FLOODED IN THE PAST TEN YEARS ________

EVACUATION YES ☐ NO ☐

DURATION ______________

3. A. DAMAGES

CARPETING	YES ☐	NO ☐	DEEP FREEZE	YES ☐	NO ☐
FLOOR COVERING	YES ☐	NO ☐	WASHER	YES ☐	NO ☐
FURNACES	YES ☐	NO ☐	DRYER	YES ☐	NO ☐
ELECTRICAL	YES ☐	NO ☐	T.V.	YES ☐	NO ☐
FRIDGE	YES ☐	NO ☐	HI FI	YES ☐	NO ☐
STOVE	YES ☐	NO ☐	OTHER	______________	

B. ESTIMATE OF LOSS ______________

.......2

ITEM "DD"

PLACENTIA FLOOD STUDY

Name of Business or Institution: ____________________

____

____

Address: ____________________

____

____

Completed By: ____________________

The questionnaire is subdivided to deal with each of the following flood dates:

1. January 10, 1982

2. January 16 and 17, 1982

3. December 23, 1983

4. December 25, 1983

Please complete all sections that are applicable to you. Detailed information, although desirable, is not required—your best estimate is all that we require.

S E C T I O N 1 : FLOOD DATE - JANUARY 10, 1982

1. Was your business, institution or organization affected by the January 10, 1982 flood: YES ____ NO ____

   If "YES", please complete the remainder of this section. If "NO", continue to Section 2.

2. Amount of damage to the building: $ ______________

   Amount of damage to the furnishings and/or equipment: $ ______________

   Amount of damage to the goods in stock: $ ______________

3. Were you forced to close during the flood. YES ____ NO ____

   If "YES", how many hours were you forced to close: __________ hours

   What is the best estimate of total lost employee income if you were forced to close and employees were not paid: $ ______________

4. How deep was the water that entered the building during the peak of the flood: $ ______________

S E C T I O N 2 : FLOOD DATE - JANUARY 16 and 17, 1982

1. Was your business, institution or organization affected by the January 16 and 17, 1982 flood: YES ____ NO ____

   If "YES", please complete the remainder of this section. If "NO", continue to Section 3.

2. Amount of damage to the building: $ ______________

   Amount of damage to the furnishings and/or equipment: $ ______________

   Amount of damage to the goods in stock: $ ______________

3. Were you forced to close during the flood. YES ____ NO ____

   If "YES", how many hours were you forced to close: __________ hours

   What is the best estimate of total lost employee income if you were forced to close and employees were not paid: $ ______________

4. How deep was the water that entered the building during the peak of the flood: $ ______________

S E C T I O N   3 :   FLOOD DATE - DECEMBER 23, 1983

1. Was your business, institution or organization affected by the December 23, 1983 flood: YES ____ NO ____

   If "YES", please complete the remainder of this section. If "NO", continue to Section 4.

2. Amount of damage to the building: $ ______________

   Amount of damage to the furnishings and/or equipment: $ ______________

   Amount of damage to the goods in stock: $ ______________

3. Were you forced to close during the flood. YES ____ NO ____

   If "YES", how many hours were you forced to close: __________ hours

   What is the best estimate of total lost employee income if you were forced to close and employees were not paid: $ ______________

4. How deep was the water that entered the building during the peak of the flood: $ ______________

SECTION 4: FLOOD DATE - DECEMBER 25, 1983

1. Was your business, institution or organization affected by the December 25, 1983 flood: YES ____ NO ____

   If "YES", please complete the remainder of this section.

2. Amount of damage to the building: $ ________________

   Amount of damage to the furnishings and/or equipment: $ ________________

   Amount of damage to the goods in stock: $ ________________

3. Were you forced to close during the flood. YES ____ NO ____

   If "YES", how many hours were you forced to close: ____________ hours

   What is the best estimate of total lost employee income if you were forced to close and employees were not paid: $ ________________

4. How deep was the water that entered the building during the peak of the flood: $ ________________

ADDITIONAL COMMENTS:

Cost Estimates (Phase I)

APPENDIX IV - COST ESTIMATES (PHASE I)

REGION 1

ALTERNATIVE 1A - RAISE AND/OR FLOODPROOF BUILDINGS.

1 in 20 Year Event

Raise 151 buildings @ $4,830 ea.	$ 730,000
15% Contingency Factor	$ 110,000
Total Construction	$ 840,000
Engineering	$ 30,000
Total Estimated Cost	$ 870,000

1 in 100 Year Event

Raise 158 buildings @ $4,830 ea.	$ 763,000
15% Contingency Factor	$ 117,000
Total Construction	$ 880,000
Engineering	$ 30,000
Total Estimated Cost	$ 910,000

Notes:

- No maintenance/operating cost
- 50% reduction in flood damage
- 50 year economic life

APPENDIX IV - COST ESTIMATES (PHASE I) (Cont'd)

REGION 1

ALTERNATIVE 1B - SHEET PILE WALL + WAVE WALL

1 in 20 Year Event:

Sheet Piling	9400 m^2 @ \$272/m^2	\$ 2,557,000
Paving	7700 m^2 @ \$22/m^2	170,000
Backfilling	2400 m^3 @ \$8/m^3	19,000
Ramping	Provisional	10,000
Raise Swan's Road	280 m @ \$224	63,000
Property Acquisition	Provisional	100,000
Drainage Culvert with Flap Gate	Lump Sum	15,000
15% Contingency Factor		\$ 440,000
Total Construction		\$ 3,374,000
Engineering		\$ 130,000
Total Estimated Cost		\$ 3,504,000

1 in 100 Year Event:

Sheet Piling	9550 m^2 @ \$272/m^2	\$ 2,600,000
Pavement	7700 m^2 @ \$22/m^2	170,000
Backfilling	2500 m^3 @ \$8/m^3	20,000
Ramping	Provisional	10,000
Raise Swan's Road	280 m @ \$243/m	66,000
Property Acquisition	Provisional	100,000
Wave Wall	300 m @ \$1450/m	435,000
Drainage Culvert with Flap Gate	Lump Sum	15,000
15% Contingency Factor		\$ 512,000
Total Construction		\$ 3,928,000
Engineering		\$ 150,000
Total Estimated Cost		\$ 4,078,000

Notes:

- \$20,000 maintenance every 15 years on steel-piling.
- 100% damage reduction for 1 in 20 and 1 in 100 year events.
- 30 year economic life for sheet-piling and wave wall.

REGION 1

ALTERNATIVE 1C - RAISE RIVERSIDE DRIVE + WAVE WALL

1 in 20 Year Event:

Mass Fill	13,400 m^3 @ $8/$m^3$	$ 107,000
Pavement	7700 m^2 @ $22/$m^2$	170,000
Raising Properties	Provisional	50,000
Ramping	Provisional	25,000
Guard Rail	1280 m @ $60/m	77,000
Drainage Culvert with Flap Gate	Lump Sum	15,000
15% Contingency Factor		$ 67,000
Total Construction		$ 511,000
Engineering		$ 24,000
Total Estimated Cost		$ 535,000

1 in 100 Year Event:

Mass Fill	16,000 m^3 @ $8/$m^3$	$ 128,000
Pavement	7700 m^2 @ $22/$m^2$	170,000
Raising Properties	Provisional	50,000
Ramping	Provisional	25,000
Guard Rail	1280 m @ $60/m	77,000
Wave Wall	300 m @ $1450/m	435,000
Drainage Culvert with Flap Gate	Lump Sum	15,000
15% Contingency Factor		$ 135,000
Total Construction		$ 1,035,000
Engineering		$ 45,000
Total Estimated Cost		$ 1,080,000

Notes:

- $5,000 maintenance to guard rail every 2 years.
- 100% reduction in flood damage for 1 in 20 and 1 in 100 year events.
- 50 year economic life

APPENDIX IV - COST ESTIMATES (PHASE I) (Cont'd)

REGION 2

ALTERNATIVE 2A - RAISE AND/OR FLOODPROOF BUILDINGS

1 in 20 Year Event:

Raise 2 buildings @ $4,830 ea.	$ 10,000
15% Contingency Factor	$ 1,500
Total Construction	$ 11,500
Engineering	$ 500
Total Estimated Cost	$ 12,000

1 in 100 Year Event:

Raise 9 buildings @ $4,830 ea.	$ 43,000
15% Contingency Factor	$ 6,500
Total Construction	$ 49,500
Engineering	$ 2,500
Total Estimated Cost	$ 52,000

Notes:

- No maintenance/operation cost
- 50% reduction in flood damage
- 50 year economic life

REGION 2

ALTERNATIVE 2B - RAISE WAVE WALL WEST OF JERSEYSIDE + BREASTWORK EAST OF JERSEYSIDE.

1 in 20 Year Event:

Breastwork	220 m @ $620/m	$ 136,000
15% Contingency Factor		$ 20,000
Total Construction		$ 156,000
Engineering		$ 9,000
Total Estimated Cost		$ 165,000

1 in 100 Year Event:

Wave Wall	250 m @ $620/m	$ 155,000
Brestwork	220 m @ $620/m	$ 136,000
15% Contingency Factor		$ 44,000
Total Construction		$ 335,000
Engineering		$ 20,000
Total Estimated Cost		$ 355,000

Notes:

- 100% damage reduction for 1 in 20 and 1 in 100 year events.
- 30 economic life.

REGION 3

ALTERNATIVE 3A - RAISE AND/OR FLOODPROOF BUILDINGS

1 in 20 Year Event:

Raise 2 buildings @ $4,830 ea.	$ 10,000
15% Contingency Factor	$ 1,500
Total Construction	$ 11,500
Engineering	$ 500
Total Estimated Cost	$ 12,000

1 in 100 Year Event:

Raise 11 buildings @ $4,830	$ 53,000
15% Contingency Factor	$ 8,000
Total Construction	$ 61,000
Engineering	$ 2,000
Total Estimated Cost	$ 63,000

Notes:

- No maintenance/operation cost.
- 50% reduction in flood damage.
- 50 year economic life.

REGION 3

ALTERNATIVE 3B - DYKE IN SOUTHEAST ARM + WAVE WALL

1 in 20 Year Event:

Dyke	15,000 m^3 @ \$10/m^3	\$ 150,000
15% Contingency Factor		\$ 23,000
Total Construction		\$ 173,000
Engineering		\$ 10,000
Total Estimated Cost		\$ 183,000

1 in 100 Year Event:

Dyke	15,000 m^3 @ \$10/m^3	\$ 150,000
Wave Wall	300 m @ \$1450/m	\$ 435,000
15% Contingency Factor		\$ 88,000
Total Construction		\$ 673,000
Engineering		\$ 30,000
Total Estimated Cost		\$ 703,000

Notes:

- No maintenance/operation cost.
- 50% reduction in flood damage.
- 50 year economic life.

REGION 4

ALTERNATIVE 4A - RAISE AND/OR FLOODPROOF BUILDINGS

1 in 20 Year Event:

Not Required.

1 in 100 Year Event:

Raise 20 buildings @ $4,830 ea.	$ 97,000
15% Contingency Factor	$ 14,000
Total Construction	$ 111,000
Engineering	$ 3,000
Total Estimated Cost	$ 114,000

ALTERNATIVE 4B - WAVE WALL

1 in 20 Year Event:

Not Required.

1 in 100 Year Event:

Wave Wall	300 m @ $1,450/m	$ 435,000
15% Contingency		$ 65,000
Total Construction		$ 500,000
Engineering		$ 20,000
Total Estimated Cost		$ 520,000

Notes:

- No maintenance/operation cost.
- 50% reduction in flood damage.
- 50 year economic life.

APPENDIX V

COST DATA - PHASE II

1. Cost Estimates (Phase II)

APPENDIX V - COST ESTIMATES (PHASE II)

ALTERNATIVE I

1 in 20 Year Event (Raise Riverside Drive and 5 Buildings):

Contractor Mobilization and Demobilization	Provisional	$ 20,000
Demolition of Existing Pavement	$7700m^2$ @ $\$6/m^2$	46,000
Excavation of Cut Off Trench	$1600m^3$ @ $\$10/m^3$	16,000
Mass Fill	$13,400m^3$ @ $\$8/m^3$	107,000
Pavement	$7700m^2$ @ $\$22/m^3$	170,000
Raising Properties	Provisional	50,000
Ramping	Provisional	25,000
Guard Rail	1280m @ $60/m	77,000
Drainage Culvert with Flap Gate	Lump Sum	15,000
15% Contingency Factor		$ 79,000
Total Construction		$ 605,000
Engineering including pre-design services (site survey, sub-surface investigations), design services, administration and inspection services during construction.		$ 90,000
Total Esimated Cost		$ 695,000

APPENDIX V - COST ESTIMATES (PHASE II)

ALTERNATIVE I (Cont'd)

1 in 100 Year Event (Raise Riverside Drive and 5 Buildings and Construct Wave Wall:

Contractor Mobilization and Demobilization	Provisional	$ 20,000
Demolition of Existing Pavement	$7700m^2$ @ $\$6/m^2$	46,000
Excavation of Cut Off Trench	$1600m^3$ @ $\$10/m^3$	16,000
Mass Fill	$16{,}000m^3$ @ $\$8/m^3$	128,000
Pavement	$7700m^2$ @ $\$22/m^2$	170,000
Raising Properties	Provisional	50,000
Ramping	Provisional	25,000
Guard Rail	1280m @ $60/m	77,000
Wave Wall	300 m @ $1450/m	435,000
Drainage Culvert with Flap Gate	Lump Sum	15,000
15% Contingency Factor		$ 147,000
Total Construction		$1,129,000
Engineering including pre-design services (site survey, sub-surface investigation), design services, administration and inspection services during construction.		$ 120,000
Total Estimated Cost		$1,249,000

Notes:

- $5,000 maintenance to guard rail every 2 years.
- 100% reduction in flood damage for 1 in 20 and 1 in 100 year events.
- 50 year economic life on Riverside Drive and ~~30~~ 50 years economic life on wave wall.

APPENDIX V - COST ESTIMATES (PHASE II)

ALTERNATIVE 2

1 in 20 Year Event (Raise 4 Buildings):

Raise 4 buildings @ $4,830 ea.	$ 20,000
15% Contingency Factor	$ 3,000
Total Construction	$ 23,000
Engineering (including pre-design services, design services and inspection services during construction)	$ 3,000
Total Estimated Cost	$ 26,000

1 in 100 Year Event (Raise 20 Buildings):

Raise 20 buildings @ $4,830 ea.	$ 97,000
15% Contingency Factor	$ 14,500
Total Construction	$ 111,500
Engineering (including pre-design services, design services and inspection services during construction)	$ 5,000
Total Estimated Cost	$ 116,500

Notes:

- No maintenance/operation cost
- 50% reduction in flood damage
- 50 year economic life

APPENDIX V - COST ESTIMATES (PHASE II)

ALTERNATIVE 3

1 in 20 Year Event (Combine Alternatives 1 and 2 for this same event):

Total Estimated Cost	$ 721,000

1 in 100 Year Event (Combine Alternatives 1 and 2 for this same event):

Total Estimated Cost	$1,365,000

APPENDIX VI

DRAWINGS

DRAWING NO.	TITLE
1	General Site Plan
2	Areal Extent of 1982 and 1983 Floods
4	Flood Risk Contours

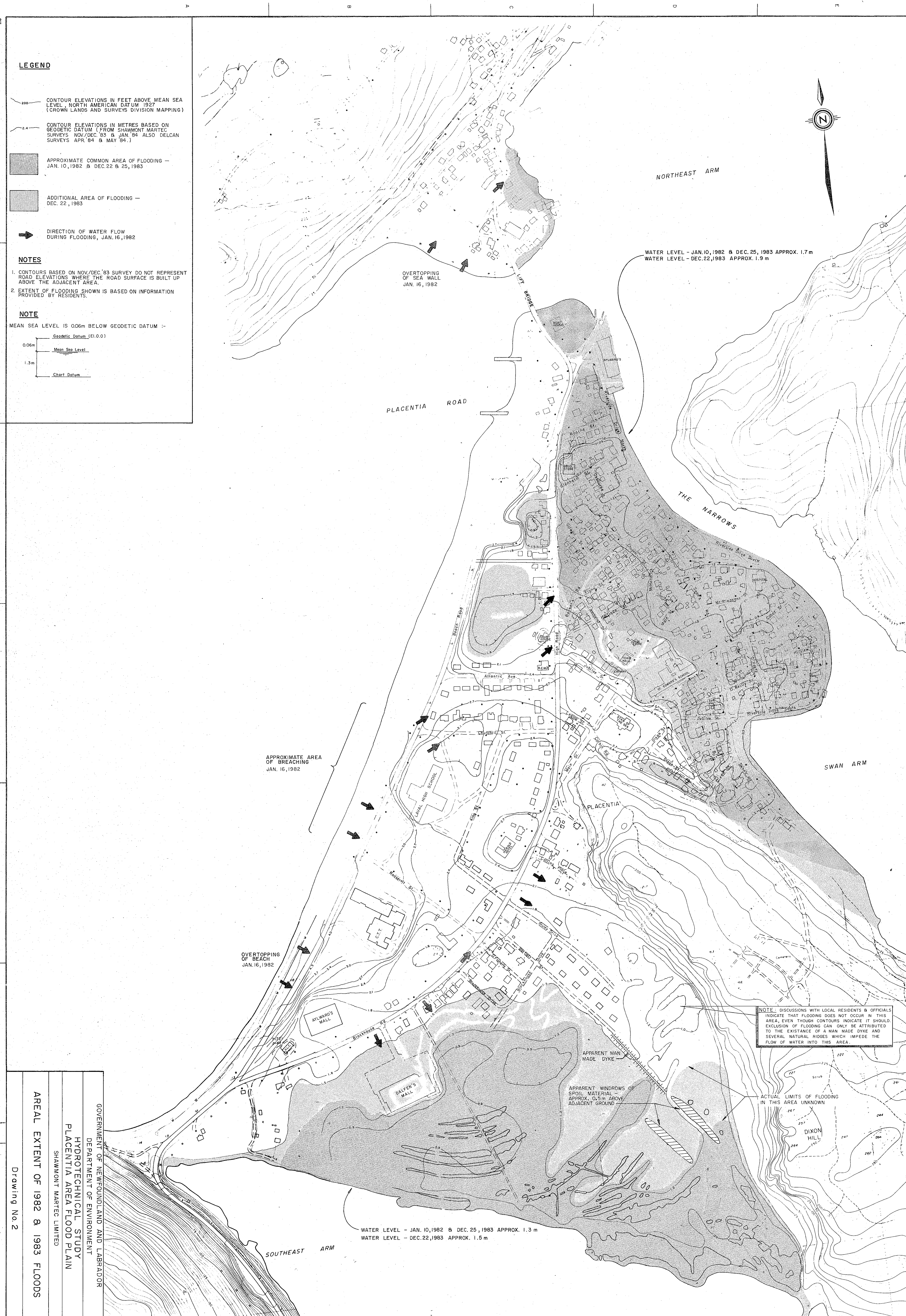
GOVERNMENT OF NEWFOUNDLAND AND LABRADOR
DEPARTMENT OF ENVIRONMENT

HYDROTECHNICAL STUDY
PLACENTIA AREA FLOOD PLAIN

SHAWMONT MARTEC LIMITED

GENERAL SITE PLAN

Drawing No. I



LEGEND
CONTOUR ELEVATIONS IN FEET ABOVE MEAN SEA LEVEL, NORTH AMERICAN DATUM 1927 (CROWN LANDS AND SURVEYS DIVISION MAPPING)
CONTOUR ELEVATIONS IN METRES BASED ON GEODETIC DATUM (FROM SHAWMONT MARTEC SURVEYS NOV./DEC. '83 & JAN. '84 ALSO DELCAN SURVEYS APR. '84 & MAY '84.)
APPROXIMATE COMMON AREA OF FLOODING — JAN. 10, 1982 & DEC. 22 & 25, 1983
ADDITIONAL AREA OF FLOODING — DEC. 22, 1983
DIRECTION OF WATER FLOW DURING FLOODING, JAN. 16, 1982
NOTES
1. CONTOURS BASED ON NOV./DEC. '83 SURVEY DO NOT REPRESENT ROAD ELEVATIONS WHERE THE ROAD SURFACE IS BUILT UP ABOVE THE ADJACENT AREA.
2. EXTENT OF FLOODING SHOWN IS BASED ON INFORMATION PROVIDED BY RESIDENTS.
NOTE
MEAN SEA LEVEL IS 0.06m BELOW GEODETIC DATUM :-
Geodetic Datum (El. 0.0)
0.06m
Mean Sea Level
1.3m
Chart Datum
NORTHEAST ARM
WATER LEVEL - JAN. 10, 1982 & DEC. 25, 1983 APPROX. 1.7 m
WATER LEVEL - DEC. 22, 1983 APPROX. 1.9 m
OVERTOPPING OF SEA WALL JAN. 16, 1982
LIFT BRIDGE
PLACENTIA ROAD
THE NARROWS
Riverside Drive South
Atlantic Ave.
SWAN ARM
APPROXIMATE AREA OF BREACHING JAN. 16, 1982
HIGH SCHOOL
PLACENTIA
OVERTOPPING OF BEACH JAN. 16, 1982
AYLWARD'S MALL
Blockhouse Rd.
NOTE: DISCUSSIONS WITH LOCAL RESIDENTS & OFFICIALS INDICATE THAT FLOODING DOES NOT OCCUR IN THIS AREA, EVEN THOUGH CONTOURS INDICATE IT SHOULD. EXCLUSION OF FLOODING CAN ONLY BE ATTRIBUTED TO THE EXISTANCE OF A MAN MADE DYKE AND SEVERAL NATURAL RIDGES WHICH IMPEDE THE FLOW OF WATER INTO THIS AREA.
APPARENT MAN MADE DYKE
APPARENT WINDROWS OF SPOIL MATERIAL - APPROX. 0.5 m ABOVE ADJACENT GROUND
ACTUAL LIMITS OF FLOODING IN THIS AREA UNKNOWN
DALFEN'S MALL
DIXON HILL
WATER LEVEL - JAN. 10, 1982 & DEC. 25, 1983 APPROX. 1.3 m
WATER LEVEL - DEC. 22, 1983 APPROX. 1.5 m
SOUTHEAST ARM
GOVERNMENT OF NEWFOUNDLAND AND LABRADOR
DEPARTMENT OF ENVIRONMENT
HYDROTECHNICAL STUDY
PLACENTIA AREA FLOOD PLAIN
SHAWMONT MARTEC LIMITED
AREAL EXTENT OF 1982 & 1983 FLOODS
Drawing No. 2


GOVERNMENT OF NEWFOUNDLAND AND LABRADOR
DEPARTMENT OF ENVIRONMENT
HYDROTECHNICAL STUDY
PLACENTIA AREA FLOOD PLAIN
SHAWMONT MARTEC LIMITED
FLOOD RISK CONTOURS
Drawing No. 4
LEGEND
CONTOUR ELEVATIONS IN FEET ABOVE MEAN SEA LEVEL, NORTH AMERICAN DATUM 1927 (CROWN LANDS AND SURVEYS DIVISION MAPPING)
CONTOUR ELEVATIONS IN METRES BASED ON GEODETIC DATUM (FROM SHAWMONT MARTEC SURVEYS NOV/DEC '83 & JAN '84 ALSO DELCAN SURVEYS APR. '84 & MAY '84)
1:20yr. & 1:100yr. FLOOD RISK CONTOURS RESULTING FROM HIGH WATER LEVEL IN THE ARMS.
DIRECTION & LIMITS OF SURFACE WATER FLOW RESULTING FROM 1:100 yr. WAVES OVERTOPPING BEACH.
LIMITS OF WATER PONDAGE RESULTING FROM 1:100yr. WAVES OVERTOPPING BEACH.
NOTE
MEAN SEA LEVEL IS 0.06m BELOW GEODETIC DATUM :-
Geodetic Datum (EL.0.0)
Mean Sea Level
Chart Datum
0.06m
1.3m
NORTHEAST ARM
THE NARROWS
SWAN ARM
SOUTHEAST ARM
PLACENTIA ROAD
PLACENTIA
DIXON HILL
LIFT BRIDGE
ASSUMED AREAS OF BEACH OVERTOPPING
APPARENT MAN MADE DYKE
APPARENT WINDROWS OF SPOIL MATERIAL APPROX. 0.5m ABOVE ADJACENT GROUND
LAVAL HIGH SCHOOL
DALTEN'S MALL
ATLANTIC MALL
1:100yr. 1.1m
1:100yr. 1.0m
1:100yr. 0.9m
1:100yr. 1.9m
1:100yr. 2.4m
1:100yr. 1.6m
1:100yr. 1.8m
1:20yr. (W.L.=1.87m)
1:100yr. (W.L.=2.04m)
1:100yr. (W.L.=1.75m)
1:20yr. (W.L.=1.58m)
NORMAN WADE CO. TRANSLAR 303