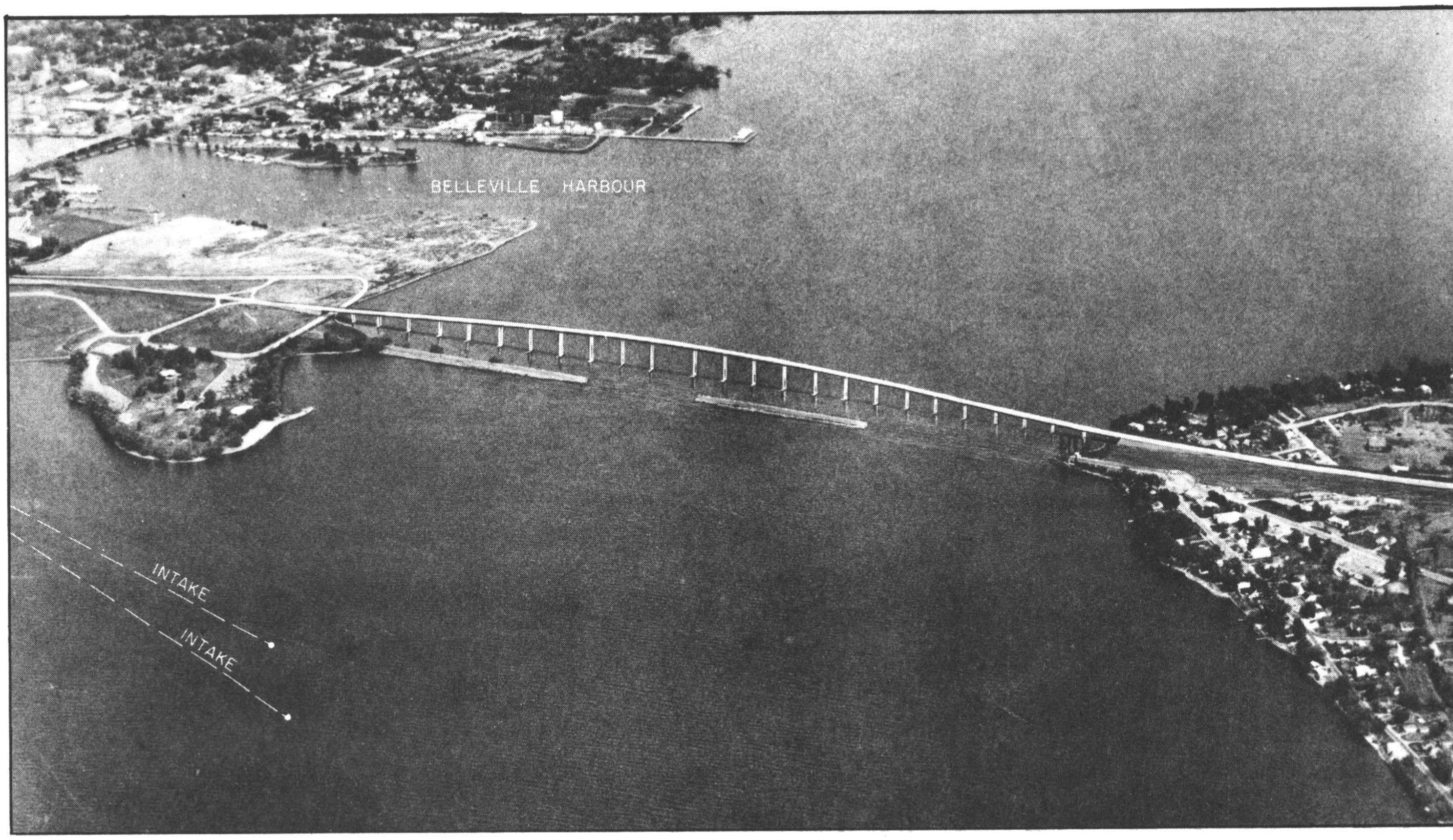
GEOCRES No: 31C-135



MARINE ENVIRONMENTAL ASSESSMENT

W.P. 134-74-01 HIGHWAY 14
BAY OF QUINTE BRIDGE
BELLEVILLE TO ROSSMORE
DISTRICT 8, KINGSTON

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS

JUNE 1979

SH

STEVENSON HARDTKE Consulting Engineers

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August 31, 1979

Mr. W.G. Wigle,
Regional Director,
Ministry of Transportation
and Communications,
Postal Bag 4000,
KINGSTON, Ontario
K7L 5A2

Re: W.P. 134-74-01, Highway 14,
Bay of Quinte Bridge
Environmental Study
Agreement No. 4212-4078-94

Dear Mr. Wigle:

In accordance with our referenced Contract, we are pleased to enclose thirty (30) copies of our Report "Marine Environmental Assessment".

This Final Report includes the extra sampling, probing and underwater inspection requested by the Ministry of Environment and the recommendations regarding dredging and dredgeate disposal are in accordance with our discussions with that Ministry.

We would be pleased to meet with your staff to further discuss our Report, at their convenience.

Yours very truly,

STEVENSON HARDTKE ASSOCIATES LIMITED,

C.A. Stevenson, P. Eng.,
Consulting Engineer

MARINE

ENVIRONMENTAL ASSESSMENT

W.P. 134-74-01 HIGHWAY 14

BAY OF QUINTE BRIDGE

BELLEVILLE TO ROSSMORE

DISTRICT 8 KINGSTON

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August 1979

SHAL PROJECT NO. 78-7201

Copy No. 5

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APPENDICES

SECTION 1

INTRODUCTION

SECTION 1 - INTRODUCTION

The construction of a new high-level bridge and approaches, approximately 100 feet east of the existing Highway 14 causeway and swing bridge between Belleville and Rossmore, spanning the Bay of Quinte, is to be undertaken shortly by the Ministry of Transportation and Communications.

The Bay of Quinte (Enclosure A-1) is a marine route from Lake Ontario to the Trent River System and the Murray Canal, and is used predominantly by pleasure craft. At regular intervals during the summer months the swing bridge is opened to permit boats to pass. In addition, the water supply of the City of Belleville is obtained from this Bay at a point northwest of the causeway (Enclosure A-2).

Concerns identified during the planning stage for this Project included the environmental influence which the bridge construction, partial causeway removal, and channel dredging may have in causing rougher wave conditions east of the present causeway, increased siltation of the boating channels, and degradation of the water quality in Belleville Harbour and at the Belleville water intakes.

This Study has been divided into three phases:

Phase I - Project Environmental Assessment

Phase II - Construction Monitoring

Phase III - Post Construction Monitoring

In carrying out Phase I, the Marine Environmental Assessment, which is the subject of this Report, four Working Papers were produced, submitted and discussed with Ministry of Transportation and Communications. These documents, which were titled:

Working Paper No. 1 - Detailed Study Design
Working Paper No. 2 - Assessment of Existing Conditions
Working Paper No. 3 - Assessment of Project
Working Paper No. 4 - Assessment of Construction Conditions

were prepared during the period from September, 1978, to March, 1979, and have been compiled herein to form this Final Report - Marine Environmental Assessment.

SECTION 2
STATEMENT OF CONCERNS

SECTION 2 - STATEMENT OF CONCERNS

Concerns identified during the planning stage for the Project included the environmental influence which the bridge construction, partial causeway removal and channel dredging might have in causing rougher wave conditions east of the present causeway, increased siltation of the boating channels, and a degradation of the water quality in Belleville Harbour and at the Belleville Water Intakes.

During the execution of the Study other concerns, such as the influence of the Project on the adjacent shorelines, on ice movements and on the fish population, were identified and included in the program for evaluation. Also included, is an evaluation of the impact of total causeway removal.

This Section states each factor of these concerns and presents in summary form the findings of this Study as to whether a positive or negative impact will result upon completion of the Project.

Further, the conditions during construction are evaluated and the resultant impact stated.

2.1 WAVE CONDITIONS

Concern No. 1: Upon completion of the Project, will the conditions be significantly different to adversely affect the boats normally using Belleville Harbour?

Evaluation: The wave climate will be essentially unchanged, and the slight increase undetectible to even small boat users.

Impact: Negligible

Concern No. 2: Upon completion of the Project, will conditions be significantly different to adversely affect the water quality at the Belleville Intakes?

Evaluation: The wave climate will be essentially unchanged, and the slight increase undetectible in terms of water quality.

Impact: Negligible

Concern No. 3: Upon completion of the Project, will conditions for boating and recreation be affected?

Evaluation: The wave conditions at the new navigation channel will be identical to those experienced at the location of the old channel. However, the new channel will provide a wider unobstructed passageway.

Impact: Positive

2.2 SEDIMENTATION AND SCOUR

Concern No. 1: Upon completion of the Project, will there be a change in the pattern of scour and sedimentation at the Belleville Intakes?

Evaluation: No change will be experienced due to the Project.

Impact: None

Concern No. 2: Upon completion of the Project, will there be a change in the pattern of scour and sedimentation at the Belleville Harbour entrance channel or the mouth of the Moira River?

Evaluation: No change will be experienced due to the Project.

Impact: None

Concern No. 3: Upon completion of the Project, will there be a change in the pattern of scour and sedimentation at the existing navigation channel?

Evaluation: The bed of the former channel area close to the causeway and beneath the proposed bridge will generally be kept swept clean of the soft organic mud as it is at present. Some initial scouring will take place at Piers No. 1 and 2.

Impact: None

Concern No. 4: Upon completion of the Project, will there be a change in the pattern of scour and sedimentation at the new navigation channel?

Evaluation: Scouring of the new navigation channel at the causeway will progress at a decreasing rate with time until the eroding material has been self-armoured. Initial scouring will also take place at Piers No. 6 and 7, and the scoured material will be deposited about 100 to 300 meters east of the causeway.

Impact: No impact from scour. Sedimentation may in rare instances require some maintenance dredging, therefore, potential negative impact.

Concern No. 5: Upon completion of the Project, will sediments be transported eastward and have an affect on water quality and benthic organisms?

Evaluation: The composition and character of sediments, and the type of known benthic organisms, are similar on either side of the causeway. Upon completion of the Project, and after a period of stabilization, there should be no increase in the quantity of sediment transported. During the stabilization period, the effect of additional sediment transport will be much less than the present effects of sediment transport on benthic organisms caused by storm waves.

Impact: Negligible

2.3 WATER QUALITY

Concern No. 1: Upon completion of the Project, will there be any change in the water quality at the Belleville Intakes?

Evaluation: Water quality is essentially the same on either side of the causeway, thus increased mass water transfer will not result in any change. Similarly, the micro increase in wave energy will not result in any noticeable quality change. The re-location of the main boat channel will result in a larger proportion of boats passing closer to the Intakes and thereby increase the possibility of water quality degradation from petroleum products in the channel area. However, any such degradation would be surface oriented, and well dispersed before reaching the area of the underwater intakes.

Impact: Minimal

Concern No. 2: Upon completion of the Project, will there be any change in the water quality in Belleville Harbour?

Evaluation: Water quality is essentially the same on either side of the causeway, thus increased mass water transfer will not result in any change. Similarly, the micro increase in wave energy will not result in any quality change, nor will the slightly closer passage of increased boat traffic.

Impact: None

2.4 SHORELINE

Concern No. 1: Upon completion of the Project, will there be any adverse effects induced along the shoreline?

Evaluation: Under existing conditions there are no areas along the adjoining shoreline subject to successive periods of erosion or accretion. Other than the immediate area of the approaches, the Project will not alter the present shoreline conditions.

Impact: None

2.5 ICE

Concern No. 1: Upon completion of the Project, will there be a change to the ice regime?

Evaluation: The enlargement of the overflow channel and removal of the existing bridge piers will probably accelerate the clearance of ice from the west side of the causeway during the spring.

Impact: Positive

2.6 FISH

Concern No. 1: Upon completion of the Project, will there be a change to the fish habitat or sports-fishing?

Evaluation: Migrating fish will easily be able to find their way through either of the two openings in the causeway, and there are no spawning grounds within the Project area. The abandoned causeway will provide new opportunities for sports-fishermen.

Impact: Positive

2.7 PROJECT CONSTRUCTION

Concern No. 1: Will the construction activities during the implementation of the Project cause either short or long-term effects?

Evaluation: Increased local turbidity and local sedimentation will be experienced during the construction period but to a lesser degree than that induced by storm waves.

Impact: Negligible

Evaluation: Water quality in the immediate area of construction will decline, however, the influence will not nearly extend to the Belleville Intakes or Belleville Harbour.

Impact: None

Evaluation: Local benthic communities will be temporarily disturbed by construction equipment and suspended solids. Re-establishment would be effected within a year or so, and the resultant affect would not likely be evident.

Impact: Negligible

Evaluation: Fish migration might be slightly restrained by the construction activity although no spawning beds will be affected.

Impact: Negligible

Evaluation: Boating and recreational use of the Project area will be restricted during the construction period. However, the restricted area represents only a micro portion of the total water recreational area, and navigation through the causeway will be continuously maintained.

Impact: Negligible

Evaluation: Excavation of the granular materials forming part of the existing causeway and which will be placed alongside will cause only local turbidity which will quickly settle and not disburse.

Impact: Negligible

Evaluation: Dredging of the soft bottom materials will cause considerable localized turbidity, as will disposal alongside the causeway. However, this small volume of material disturbance is insignificant compared to that disturbed due to storms. Once the dredgeate has been placed, no adverse effects will be experienced.

Impact: Temporarily negative.

2.8 TOTAL CAUSEWAY REMOVAL

Concern No. 1: Would the total causeway removal improve the local conditions of the Bay by returning it to the condition which naturally existed some sixty years ago?

Evaluation: Water levels would show slightly lower and higher extreme values of micro proportions.

Impact: Negligible

Evaluation: Wave exposure of the adjacent water areas, especially the east side of the causeway which includes the Public Wharf and the mouth of the Moira River, would increase.

Impact: Negative

Evaluation: Shore-fast ice would take longer to form, breakup would be earlier, thicker, larger and more mobile ice floes would occur.

Impact: Negative

Evaluation: Mass water movement would increase and much lower local current velocities would be distributed more or less uniformly across the whole width of the Bay.

Impact: None

Evaluation: Sediment regime would undergo significant changes, the Bay mud would become more widely distributed and more frequently disturbed with increased siltation in the navigation channels.

Impact: Negative

Evaluation: Water quality would be influenced by the increased activity of the bottom sediments, which would increase the level of total suspended and dissolved solids.

Impact: Negative

Evaluation: Benthic Organisms would be subject to short-term re-stabilization to suit the new regime and long-term effects of increased sediment mobility.

Impact: Negative

Evaluation: Shorelines would be subject to increased frequency of wave attack from previously sheltered directions, producing increased erosion or accretion.

Impact: Negative

SECTION 3
SUMMARY AND CONCLUSIONS

SECTION 3 - SUMMARY AND CONCLUSIONS

3.1 SUMMARY

This Environmental Assessment Study has examined and established the marine oriented Existing Base Data Conditions in the Project Area which could be affected by the Project. From this, Project Data Conditions have been established and the impact of the Project identified. Conditions likely to exist during the construction implementation of the Project have also been established and the related impact identified. Further, consideration has been given to the desirability of modifying the proposed Project to include total causeway removal, and that impact has been identified.

Table 3.1 following summarizes this marine oriented Environmental Assessment in terms of positive, negative or no impact for each of the categories investigated.

Table 3.2 summarizes the impact on the marine environment during the construction period.

Finally, Table 3.3 evaluates the impact on the marine environment of Total Causeway Removal.

TABLE 3.1

PROJECT IMPACT

Category	Belleville Harbour	Belleville Intakes	Existing Channel	New Channel	Boating & Recreation	Project Area
Wave Conditions	Negligible	Negligible	None	Positive	Positive	None
Sedimentation	None	None	None	Potential Negative	NA	Negligible
Scour	None	None	None	None	NA	Negligible
Water Quality	None	Minimal	None	None	None	None
Shorelines	NA	NA	NA	NA	NA	None
Ice	None	None	NA	NA	NA	Positive
Fish	NA	NA	NA	NA	NA	Positive
Benthic Organisms	NA	NA	NA	NA	NA	Negligible

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TABLE 3.2

PROJECT CONSTRUCTION - IMPACT

Category	Belleville Harbour	Belleville Intakes	Boating & Recreation	Project Area
Sedimentation	None	None	NA	Negligible
Scour	None	None	NA	Negligible
Water Quality	None	None	Negligible	Negligible
Fish	NA	NA	NA	Negligible
Benthic Organisms	NA	NA	NA	Negligible

TABLE 3.3

TOTAL CAUSEWAY REMOVAL - IMPACT

Category	Effect
Water Levels	Negligible
Wave Exposure	Negative
Ice	Negative
Mass Water Movement	None
Scour	None
Sedimentation	Negative
Water Quality	Negative
Benthic Organisms	Negative
Shoreline	Negative

3.2 CONCLUSIONS

3.2.1 The Project

Analysis and evaluation of the effects of the Project on the marine environment indicates that it will have no adverse effect on the Project Area of the Bay of Quinte.

The possible occasional need for maintenance dredging in the eastern approaches to the new navigation channel has been the only negative impact identified.

Upon completion, the new wider, unobstructed channel is a positive impact for boating, as is the access to further fishing areas along the abandoned causeway to sports-fishermen.

3.2.2 Project Construction

Based on an analysis and evaluation of the construction procedures described in this Report, and provided that due care is taken during construction, the Study has identified no categories having a significant positive or negative impact on the marine environment.

3.2.3 Total Causeway Removal

Should the proposed Project be modified to include total causeway removal, the marine environment would suffer a significant negative impact in the categories of wave exposure, ice, sedimentation, water quality, benthic organisms and shoreline. No categories of positive impact were identified.

3.2.4 Monitoring

This Study has been divided into three Phases:

Phase I - Project Environmental Assessment
Phase II - Construction Monitoring
Phase III - Post Construction Monitoring

In carrying out the analysis and evaluation for this Report, which fulfills the requirements of Phase I, it has been apparent that the total marine oriented environmental impact of the Project and its implementation will be insignificant and probably difficult to quantify. In view of this, and based on the premise that the Project will be implemented as described and with due care, it is unnecessary to develop a special program of monitoring either during or after construction.

However, should it be considered desirable to prove the foregoing conclusions, such an investigation could easily be carried out using this Study for the data base.

SECTION 4

EXISTING CONDITIONS

SECTION 4 - EXISTING CONDITIONS

4.1 INTRODUCTION

This Section is concerned with establishing the base data conditions which exist within the influence of the causeway and proposed construction.

For this purpose, information regarding the following subjects was required:

- winds and waves
- water levels
- ice
- currents and mass water movements
- bathometry
- sediments
- water quality
- Benthic Organisms
- geotechnical
- shoreline changes

To achieve this, all existing relevant data was obtained from:

- Project Quinte - water quality trends, physical limnology, sediments, hydrodynamics

- Ministry of Environment - river flow measurements and water quality

- Public Works Canada and National Research Council - hydraulic modelling of Belleville Harbour and wave forecasts

- Transport Canada - historic hydrographic surveys, maintenance dredging records and wind records

- Ministry of Transportation and Communications - historic records of Highway 14 causeway, hydrographic records and geotechnical data

- Ministry of Natural Resources - shoreline aerial photography and shoreline changes mapping

- Canada Centre for Inland Waters - wave records, current measurements, sediments and water quality

- City of Belleville - shoreline changes, intake water quality records

This information was supplemented by additional sediment sampling and analysis, water sampling and analysis and current measurements carried out for this Study.

The Bay of Quinte is a Z-shaped body of water 225 km^2 in area, 96 km long and less than 1.5 km wide over much of its length. It is located on the northwestern shore of Lake Ontario and almost completely separates Prince Edward County from the mainland (Figure 4.1).

The upper Bay consists of several connected basins, 4 to 8 m deep in mid-channel. It extends northeastward for 48 km from Trenton to Deseronto, forming the top of the "Z". The middle Bay (Long Beach) extends southwesterly for 13 km from Deseronto to Picton and is from 6 to 17 m deep in mid-channel. The lower Bay (Adolphus Reach), which forms the bottom of the "Z", stretches northeasterly for about 20 km.

The maximum depth increases from 17 m to 55 m towards the Bay mouth.

The Bay of Quinte is situated in Ordovician bedrock and Pleistocene glacial deposits. The watershed totals 1,200 km^2 at the Bay mouth (Johnson and Owen, 1970). The major tributaries (Trent, Moira, Salmon and Napanee Rivers) originate in the Pre-Cambrian Shield to the north but traverse limestone and clays over much of their courses. They enter along the north shore of the upper Bay.

The greatest human development is along the north shore of the upper Bay, which includes the municipalities of Trenton, Belleville, Deseronto and Napanee. The town of Picton is the largest centre bordering the middle and lower Bays.

Appendix E contains a listing of persons contacted during the execution of this Study.

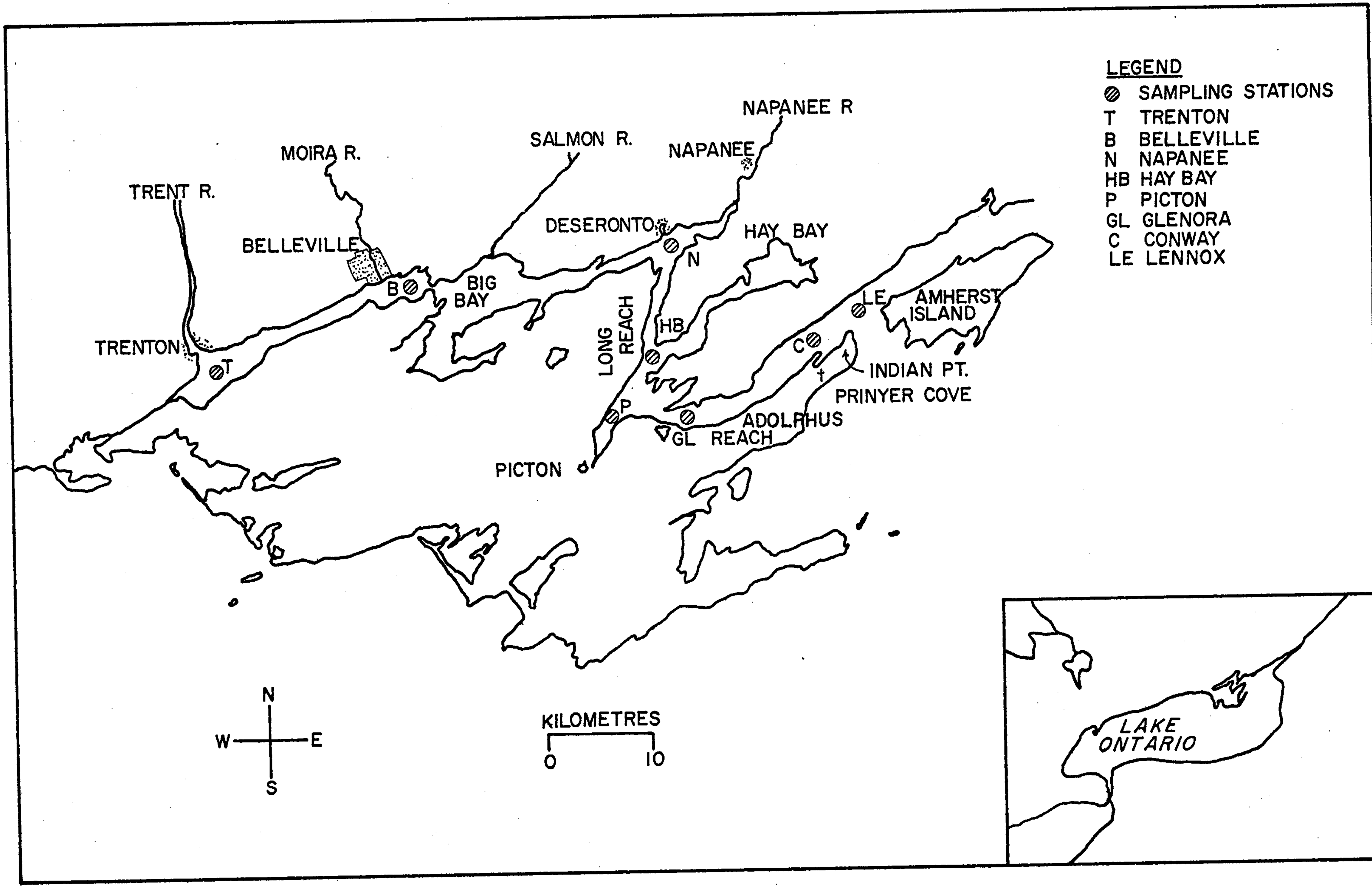

Figure 4.1: Bay of Quinte locations for chemical and biological sampling and primary productivity for "Project Quinte".
Taken from Ontario Ministry of the Environment 1976 Report.

4.2 WAVES AND ICE

4.2.1 Wave Hindcasting

Wave measurements have not been made in the Bay of Quinte and, therefore, it was necessary to rely on the empirical, but well-established, procedures of hindcasting to estimate wave conditions from wind records in the Project area.

To provide an initial indication of the typical range of existing wave conditions in the Belleville area, a summary of the results of a previous wave hindcasting exercise is presented. Next an analysis of the local wind climate is presented. This is followed by a detailed analysis of severe wave conditions at the mouth of the Moira River and at the Belleville Water Intake for the specific directions which could be affected by subsequent widening of the overflow channel.

4.2.1.1 Typical Average Wave Conditions - The hindcast wave frequency tables used were obtained from the Small Craft Harbour Study prepared for Fisheries and Environment Canada by Public Works Canada, Marine Directorate (PWC, 1978). Although they refer to conditions in the area of Belleville Public Wharf (Enclosure A-4), they may be considered generally representative of wave conditions in the area. These tables were based on ten years of wind measurements from Trenton Airport, obtained from the Atmospheric Environment Service, Fisheries and Environment Canada.

The hindcast was restricted to the boating season (defined as May 1st to October 31st) and to waves approaching from directions E, SE, S and SW, plus all these directions combined. The entries in each table indicate the estimated number of hours of occurrence per ten years of waves of "significant" height (in feet) and "peak" period (in seconds). These tables show that there is very little wave action in the area, at least during the normal boating season. Wave frequencies for all directions combined indicate that waves very seldom exceed 0.6 m (2 feet) in height, and that waves of 3.0 to 3.5 seconds period are to be expected for only two hours per ten years.

The foregoing description and tables apply to wave conditions on the east side of the causeway crossing. Waves on the west side of the causeway are somewhat more energetic because of the dominance of westerly winds. However, here too the general level of wave attack is very low, due both to the limitations of the wave generation area and its shallow depth.

4.2.1.2 Wind Climate - Data from analysis of twenty-four years of the Trenton wind records carried out by Baird, of Public Works Canada, was obtained for this Study. The analysis indicated that winds are stronger and maximum speeds are most likely to occur from November through April and to come from westerly directions, in particular from the southwest.

Table 4.1 was derived from the same source data and analysed by extreme wind speed and duration. The values for one hour duration in Table 4.1 correspond closely to those given by Baird for the southwesterly direction, but are somewhat lower than Baird's results for all directions combined.

TABLE 4.1

Estimated Extreme Wind Speeds by Duration
for All Directions and All Months Combined
(from Trenton 1955 - 74)

Frequency	Duration of Wind (in hours)				
(per Year)	1	2	3	6	12
1/10	54	51	48	44	37
1/20	58	55	52	47	40
1/50	64	61	57	51	43
1/100	68	65	61	54	46

Note: Speeds are in miles per hour, averaged over the duration shown.

4.2.1.3 Wave Generating Areas and Methodology - In order to explain the proposed definitions of wave generating areas (or fetches), it is necessary to introduce a general description of the assumptions underlying the methodology. Attention has been concentrated on the specific directions of wave attack which permit waves passing through the overflow channel to impinge on the intake or the mouth of the Moira River. Wave conditions elsewhere will be insignificant and need not be considered.

Due to the relative position of Intake, overflow channel and the mouth of the Moira River, Enclosure A-2, it was necessary to consider fetches corresponding to intermediate wind direction not directly represented in the wind records or wind analysis. This required modification of wind data to obtain wind frequencies representative of the actual fetch directions.

Four fetches have been defined - two in each direction - the two westerly fetches W1 and W2, centered at Azimuth 245°, and the two easterly fetches E1 and E2, centered at Azimuth 75°, Enclosure A-2.

The need for and use of the four fetches is based on the following methodology and assumptions:

(a) Wave conditions in the two areas of interest are most affected by waves propagating through the overflow channel in the appropriate directions; that is within a 45° sector centered on the azimuths previously stated.

(b) Two fetches are needed in each direction because waves will be generated independently on both sides of the causeway by the same wind condition. Further, the easterly fetches are used only for intake area conditions. Likewise, the westerly fetches only are considered in connection with wave conditions in the Moira mouth area.

(c) Wave conditions in the areas considered may consist of two superimposed wave trains, (i.e. for the intake):

- Those waves generated on the east side of the causeway in Fetch E1, which propagate through the overflow channel and diffract into the area on the west side, including the intake area.

- Those waves generated in Fetch E2 wholly on the west side of the causeway. These waves will be of shorter period than those from Fetch E1.

For the Moira River mouth area the same situation will exist with respect to the westerly Fetches W1 and W2, Enclosure A-2.

(d) In both cases there will be some interaction between the two wave trains, which is taken into account by adding the energy fluxes of the two individual wave trains.

Because of the irregular shape of the Bay and the consequent restriction of fetch width, the effective lengths of Fetches E1 and W1 were computed using the method given in CERC (1973). Enclosure A-1 shows the assumed limits of these wave generating areas as used in the calculation of effective fetch. The effective lengths of the smaller Fetches E2 and W2 were taken to be equal to their physical lengths because they are not restricted in width. The results obtained for the four fetches are shown in Table 4.2.

TABLE 4.2

Estimated Effective Fetch Lengths

Fetch	Effective Length	
	Ft.	m
E1	14,400	4,400
E2	2,000	600
W1	12,800	3,900
W2	3,000	1,100

The effective water depths in each of the four wave generating areas varies according to the lake level which varies seasonally. Hence, it is convenient to determine the ice-free season before determining mean effective wave generating depths.

4.2.1.4 Wave Generation Season - Freeze-up take place late December or early January and break-up late March or early April. Freeze-up may be earlier than average to the west of the causeway and break-up later. Evidence for later break-up is provided by March 1978 photographs.

For purposes of this analysis it is assumed water is ice-covered from January to March inclusive. Hence, wave conditions should be considered for the remaining months; namely, from the beginning of April until the end of December inclusive.

4.2.1.5 Lake Levels and Wave Generation Depths - Review of forecasting charts indicates that normal seasonal variation in lake levels will not significantly affect wave conditions. Hence, water depths for wave generation are based on the average lake surface elevation for ice-free months: April to December (Table 4.3).

TABLE 4.3

Lake Ontario Water Levels for Ice-Free Months

April	74.66 m above IGLD
May	74.80
June	74.85
July	74.81
August	74.41
September	74.58
October	74.47
November	74.39
December	74.37
Mean	74.59 (244.67 ft) above IGLD = 2.25 ft. above Chart Datum

(Note: Data from Marine Environmental Data Service, Fisheries and Environment Canada)

Fetch depths with allowance for Lake level were then estimated as shown:

TABLE 4.4

Estimated Effective Fetch Depths

E1	=	15.0 feet
E2	=	17.5 feet
W1	=	17.5 feet
W2	=	10.0 feet (actual depth varies 6 ft. to 20 ft.)

<u>4.2.1.6 Modification of Wind Data</u> - Since the azimuths of the relevant Fetches (E1, E2, W1 and W2) do not correspond to the eight compass points used for the Trenton wind recording, it was necessary to re-partition the wind frequencies given in the wind analysis to correspond to winds over 45° wide sectors, centered on the Fetch directions.

This was accomplished as follows for the E1 and E2 Fetch:

Frequency for Az 75° - 1/3 (Frequency NE) + 2/3 (Frequency E)

In a similar manner the wind speed frequencies for W1 and W2 (Az 245°) can be approximated as:

Frequency Az 245° = 5/9 (Frequency SW) + 4/9 (Frequency W)

For the month of April the wind frequencies for the two defined Fetch directions are then as shown in Table 4.5.

Data for Table 4.5 was computed from Baird's Wind Frequency Analyses.

Table 4.5 indicates that in an average April, for 10.3% of the time, waves from the east may pass through the overflow channel, headed towards the intake area, while for 16.7% of the time waves from the west may pass through, headed towards the mouth of the Moira River.

The application of the procedure has been limited to the month of April, when the Bay is normally ice-free, high winds occur and lake levels are relatively high. Inferences to be drawn from part of these data show that it is not necessary to develop similar tables for other months of the year.

<u>4.2.1.7 Hindcast Wave Conditions</u> - For the purposes of this Study, it is sufficient to estimate wave conditions corresponding to typical severe winds to illustrate the limits of existing conditions and subsequently to demonstrate the effect of enlarging the causeway overflow channel. For this reason only the highest wind speeds shown in Table 4.5 will be used; thus a wind speed of 30 mph will be used for Fetches E1 and E2, and 40 mph for W1 and W2. Reference to extreme wind speed analyses for NE, E, SW and W directions (Wind Speed Recurrence Analysis by the Gumbel Method) indicates that the recurrence intervals for these wind speeds lie between 2 and 8 years. They, therefore, represent severe conditions. Assumed wave hindcasting conditions are summarized in Table 4.6.

TABLE 4.5

Estimated Wind Frequencies by Speed Classes for April for Octants Centered on Fetch Directions

Speed (mph)	E1, E2 (Az 75°) %	W1, W2 (Az 245°) %
1 - 2	0.13	0.16
3 - 4	0.97	0.90
5 - 6	1.53	1.56
7 - 8	1.72	2.41
9 - 10	1.65	2.18
11 - 12	1.34	1.81
13 - 14	1.08	1.99
15 - 16	0.94	2.47
17 - 18	0.56	1.33
19 - 20	0.25	0.82
21 - 22	0.10	0.37
23 - 24	0.03	0.31
25 - 26	-	0.29
27 - 28	-	0.18
29 - 30	0.01	0.10
31 - 32	-	0.04
33 - 34	-	0.01
35 - 36	-	0.04
37 - 38	-	0.01
39 - 40	-	0.01
41 - 42	-	0.01
Totals	10.31	16.73

TABLE 4.6

Assumed Wave Hindcasting Conditions

Fetch	Depth (ft)	Length (ft)	Wind Speed (mph)
E1	15.0	14,400	30
E2	12.5	2,000	30
W1	17.5	12,800	40
W2	10.0	3,000	40

Wave conditions were estimated for the tabulated conditions based on Bretschneider's empirical shallow water wave hindcast equations. Each chart gives significant wave heights and (spectral peak) wave periods as a function of fetch and wind speed for a particular water depth. Charts are given for depths at 5 foot increments. Conditions for E2 and W1 were each obtained by interpolating results from two charts.

The results of wave hindcasts are shown in Table 4.7.

TABLE 4.7

Wave Hindcast Results

Fetch	Wave Height (ft)	Wave Period (sec)
E1	1.90	2.40
E2	0.90	1.70
W1	2.45	2.65
W2	1.50	2.00

Note: The above results are based on April winds with a frequency of occurrence of about 0.01% corresponding to recurrence intervals in the range 2 to 10 years.

The height and period found for the easterly Fetch E1 can be correlated directly with wave climate data given in Section 4.2.1.1. The values for W1 are somewhat greater, reflecting the higher level of wave energy prevailing to the west of the causeway.

4.2.2 Wave Refraction and Diffraction

The total wave effect at the intake under the influence of winds in the direction of E1 and E2 (Az 75°) results from the superposition of waves generated in E1 (which penetrate the overflow channel) on waves generated west of the causeway in Fetch E2. Likewise the total effect of the westerly fetches near the mouth of the Moira is due to superposition of waves which pass through the overflow channel from W1 onto waves generated east of the causeway in Fetch W2.

The propagation and combination of wave trains under the circumstances envisaged requires consideration of three aspects of wave phenomena: wave refraction, wave diffraction and wave energy flux (or wave power). The following three sections deal with these phenomena and their application to the given case in order to determine the wave conditions at the intake and at the mouth of the Moira River.

4.2.2.1 Wave Refraction - Wave refraction is a process by which wave length, propagation speed, direction and height are caused to vary by variations in water depth.

In deep water the length of a wave (measured for example from one wave crest to the next) is related only to the wave period and is independent of the water depth. The deep water wave length, denoted L_o, is equal to 5.12 T^2 (ft) or 1.56 T^2 (m), where T is the wave period in seconds. As the water depth decreases to a value less than $L_o/2$, the wave length begins to decrease. The rate of decrease is at first slight so that at depths of $L_o/4$ the wave length is only 7% less than L_o.

On the basis of the earlier comments, it will be apparent that wave refraction can only be significant (a) if the water depth is less than $L_o/2$ and (b) if the depth is variable or irregular.

Table 4.8 lists the water depths of the Fetches, together with hindcast wave periods and corresponding deep water wave lengths L_o, and values of $L_o/2$ and $L_o/4$.

TABLE 4.8

Comparison of Water Depths and Hindcast Wave Lengths

Fetch	Depth ft.	Period sec.	L_o ft.	$L_o/2$ ft.	$L_o/4$ ft.
E1	15.0	2.40	29.5	14.8	7.4
E2	17.5	1.70	14.8	7.4	3.7
W1	17.5	2.65	36.0	18.0	9.0
W2	10.0	2.00	20.5	10.3	5.1

From Table 4.8 it is apparent that wave refraction is insignificant for all four Fetches for waves generated within each Fetch. On the other hand, waves generated in Fetches E1 and W1 propagate into Fetches E2 and W2 respectively after passing through the overflow channel, so it is also necessary to make corresponding cross comparisions in Table 4.8. These show that waves from E1, propagating into E2, will not be affected by refraction, but that waves from W1, propagating into W2, may be affected to some extent because the depth of W2 is little more than a quarter of the length of the waves generated in W1. A review of the bathymetry indicates that the effect of refraction in W2 would be to disperse rather than concentrate wave energy, so that ignoring wave refraction will produce a somewhat high estimate of wave conditions. Thus it is concluded that wave refraction effects need not be taken into account in this Study.

4.2.2.2 Wave Diffraction - Wave diffraction is the process by which wave energy propagates into "shadow areas", that is areas sheltered behind breakwaters or, as in this case, areas sheltered behind the causeway on either side of the overflow channel.

While most of the wave energy propagated through the gap continues to propagate straight ahead, part is deflected to the sides, causing the wave crests to extend and curve into the shadow areas. At some distance beyond the gap the plan form of the entire wave front in practice (though not in theory) assumes an almost semi-circular form with radial dispersion of wave energy taking place due to the curvature of the wave fronts.

The analysis used in this Study depends on standardized theoretical diffraction solutions up to a distance of twenty wave lengths from the gap (the overflow channel) and assumes that radial dispersion occurs at greater distances. The theoretical diffraction solutions used are those presented in graphical form in CERC (1973). These graphs or charts depict numerous examples of gaps or entrances and breakwater ends in which the geometry of the system is depicted in terms of wave lengths and angles of wave approach, and data is given in terms of lines of equal diffraction coefficient. (A diffraction coefficient defines the local wave height as a fraction of the incident wave height.)

Application of the theoretical solutions requires matching the geometry and wave lengths of the site conditions to the most similar theoretical configuration. In this case directions of wave approach to the causeway are nearly normal and hence diffraction solutions for 90° incident angle have been used.

The effective width of the overflow channel now (without widening) is about 53 m (174 ft.). In order to apply theoretical solutions, this width must be expressed in terms of wave lengths of waves generated in Fetches E1 and W1 respectively. The applicable local wave lengths are determined from the deep water wave lengths (L_o) previously given, and the local water depth (d) by means of an implicit relationship which has been reduced to tabular form. Wave diffraction conditions are given in Table 4.9.

TABLE 4.9

Summary of Wave Diffraction Conditions

	Fetch E1	Fetch W1
Incident wave height (ft)	1.90	2.45
Incident wave period (s)	2.40	2.65
Deep Water length (ft)	29.5	36.0
Water Depth (ft)	17.5	15
Local Wave Length L (ft)	29.5	35.6
Gap Width (ft)	174	174
Relative Gap Width (B)	6L (approx)	5L (approx)
Distance to areas of interest (E2, W2) (ft)	2,000	3,000
Relative Distance to area of interest (L)	68L (approx)	84L (approx)
Angle of Incidence (deg.)	90	90

Enclosures A-5 and A-6 depict the situations considered. The first figure referring to existing conditions for the easterly Fetch (waves from E1, diffracting into E2) is based on mirror-imaging two theoretical solutions ($B > 5L$) derived from the case of diffraction at the end of a semi-infinite breakwater. The second figure for the westerly Fetches utilizes a gap-type solution as $B \leqslant 5L$.

The results of the diffraction - dispersion analysis were as follows:

(a) Wave height at the Intake, due to diffraction of 1.9 foot high waves through the overflow channel, is o.8 feet. The period of 2.4 seconds remains unchanged.

(b) Wave height at the mouth of the Moira River due to diffraction of 2.45 foot high waves through the overflow channel is 1.2 feet and, again, the period of 2.65 seconds remains unchanged.

These results would be valid in the absence of the locally generated waves in Fetches E2 and W2.

4.2.2.3 Combined Wave Effect - The resultant wave conditions (at each of the two areas considered) result from the superposition of two wave trains as follows:

At the Intake:

Wave diffracted from E1	0.8 ft. height, 2.4 s. period
Wave generated in E2	0.9 ft. height, 1.7 s. period

At the Moira River mouth:

Wave diffracted from W1	1.2 ft. height, 2.65 s. period
Wave generated in W2	1.5 ft. height, 2.0 s. period

The resultants listed below were determined in each case by adding the wave energy fluxes and taking the period of the longer wave of each pair as the period of the resultant.

Combined wave at the Intake	1.1 ft. height, 2.4 s. period
Combined wave at the mouth of the Moira River	1.8 ft. height, 2.65s. period

The results obtained approximate the maximum influence of the overflow channel on waves at the Intake and mouth of the Moira River under present conditions. These examples represent severe conditions likely to be experienced in April with a relative frequency of 0.01%, corresponding to return periods of 2 to 8 years.

4.2.3 Ice Conditions

Available evidence indicates that ice conditions are not severe in the Bay of Quinte. For this reason it has been agreed that information on ice conditions could be limited to general observations of those familiar with the area.

Enquiries directed to Transport Canada revealed that no specific site data was available and that routine observations were made in the area, since it was not on a commercial shipping route.

The ice conditions were observed directly on March 25, 1979, and in addition air photographs provided by MTC were available.

The following observations are based mainly on information from Mr. Bruce Cooper, manager of the Bay of Quinte Fisheries, which has a cold storage facility at the Belleville end of the existing crossing, and from Mr. A.A. Carmichael of James Street, Belleville, an ice fisherman with thirty years of local experience.

Ice is normally present in the Bay of Quinte for a little more than four months, typically from November 25 until about April 5. In the early part of the season the ice often breaks and re-freezes once or twice before it becomes solid and shorefast. Once it remained broken until early February. The western part of the Bay of Quinte usually becomes solid first.

Typical ice thickness is 0.6 m, except in the vicinity of the causeway channels where the ice is thin or an open lead persists. Exceptionally, as in February, 1979, the ice reaches 0.85 m in thickness. The ice sheet is usually relatively weak, since the upper half of the ice sheet is composed of what is locally termed "snow ice". Ice formed after heavy snowfall causes submergence and freezing of snow overlying clear ice.

The ice usually begins to melt and soften in place before the breakup commences. The ice begins to break in March and to move to and fro, according to wind conditions. Ice clears from the east side of the causeway some days earlier than on the west side. Most of the ice to the west of the causeway melts in place. However, during the peak of the spring runoff of the Trent River, a continuous stream of ice floes is disgorged through the causeway channels.

There are no reports of significant rafting of ice on the side slopes of the causeway and neither of the correspondents have ever observed ice rafted to the level of the highway. The ice is said to be too weak and soft to exert much pressure, even when driven hard by wind and current.

4.3 SAMPLING AND MEASUREMENTS

4.3.1 Sediment Sampling

Surficial grab samples were taken on October 24, 1978, by use of a Power Dredge and probe.

4.3.1.1 Existing Boat Channel - Sediment samples (10, 11, 23, 24, 25, 28, 29) were taken east and west of the swing bridge (Enclosure A-9). The samples adjacent to the scour channel (23, 24, 25, 28, 29) were silty mud. Two samples (10, 11) on the apparent scour channel were obtained. Both were close packed, felt sandy and clayey and were fairly stiff. Small cobbles, etc. were recovered in the scour channel. No sampling was done immediately beneath the bridge because of the presence of submerged cables.

4.3.1.2 Belleville Water Intakes - At this location the samples (1, 2, 12, 13) were silty muds, with distinct light and dark brown colourations similar to an oil slick. Some organic matter appeared to be present in the samples.

4.3.1.3 Entrance to Belleville Harbour - The first sampling area was in and adjacent to the dredged boat channel to the Government Wharf. There was no large discernible difference in the two samples (8, 32) taken, both being fine silty muds. It was noted that the depth of the channel at the sample point (13.25' - 1.6' = 11.65') is less than the charted depth.

The second sampling area was the shallow flats at the mouth of the Moira River (d = 4.5'). Here the samples (9, 31) were definitely sandy with dark silts and loose bark, etc.

Thirdly, the overflow channel was approached from the east and sampled at periodic intervals. Again the samples (30) were predominantly a muddy silt. However, approximately 150 feet downstream (east) from the overflow channel, the bottom was hard and no sample was obtained except for some stones, (30, 22, 21).

4.3.1.4 Overflow Channel - At the channel and immediately east and west of it (approximately 100 to 150 feet), the bottom was hard and no sample obtained. Stones and small pebbles were predominant. Adjacent to the channel, to the west, both north and south, the samples (17, 3) were very similar to those obtained at the intake, except for an apparent increase in organic matter.

Subsequently, on March 9, 1979, MTC carried out Borehole No. 100, through the section of causeway proposed for removal (Enclosure A-11). This indicated the causeway fill to be a sand with gravel and trace of silt, numerous cobbles overlying sandy gravel to gravelly sand, trace of silt.

Following preliminary examination of these samplings by MOE, additional samplings and probings in the dredging area of the new navigation channel were recommended. This program was carried out jointly on July 17, 1979, by the Consultant and representatives of MOE. Six bottom samples were recovered by divers on either side of the causeway at locations indicated on Enclosure A-16 as samples 101 to 112.

In addition, diver probings were taken at locations also indicated on Enclosure A-16. In general, the area was found to be covered by bay mud in excess of one meter, decreasing to zero thickness towards the east, central part of the overflow channel.

The bay mud generally consisted of 0.3 meters of very fine material of an organic, silt consistency. Once disturbed, visibility is reduced to zero. The diver's arm could easily penetrate up to two feet into the bay mud with no discernible change in texture. In some areas, loose, thin horizontal layers of shells or small stones could be felt within the bay mud.

The probings were done with standard, metal survey range pole, and the diver was able to penetrate the bay mud using a steady push with one hand. In all cases the probe was stopped by firm resistance.

During the same period, soundings of the overflow channel area were taken by MTC staff and are also indicated on Enclosure A-16. It is estimated that the sounding line probably penetrated at least 0.3 to 0.5 meters into the bay mud.

<u>4.3.1.5 Diver Inspection</u> - The Water Purification Plant at the base of Sidney Street has two intakes:

- an old 30" diameter steel intake pipe 1,440 feet long with a bellmouth;

- a new 36" diameter concrete intake pipe with two bellmouths installed in 1974 (1,600 feet long).

Total pumping capacity is in the order of 12,000,000 USGPD. The older intake is a gravity flow to an intake well, whereas the new intake is pumped by a vertical pump at the shore end. Both intakes are marked by large spar buoys. The older 30" diameter intake is the one shown on the hydrographic charts. The new 36" diameter intake is to the west of it.

The first dive was at the old 30 inch diameter intake. Material around the intake is a slimy silt. The intake is covered with a large circular slab (approximately 9 feet in diameter), supported on vertical members of the bellmouth. The general condition of the intake is good. The buoy is approximately 15 feet south of the intake.

The second dive was at the new 36 inch diameter intake. The bottom material is a very loose slimy silt. This intake is 20 feet north of the buoy. There are two bell-mouths for the intake, in line on the pipe, approximately 10 feet apart, and approximately six feet above the bottom. The openings are large (i.e. 15" by 48"). There was an overall slight turbulent effect in the area but there were no discernible flow patterns in the bottom.

The third and fourth dives were at the inverts of the overflow channel and the swing bridge channel. Both channels contain small stones and miscellaneous debris. The exposed bottom between the stones was hard - the material was a coarse grained sediment, difficult to penetrate more than one inch with the sample jar. Going both east and west, the number of stones decreased and the bottom sediments were more uniform and coarse grained. Samples (4, 5, 6) were taken both east and west of the bridge and immediately beneath it. There was a very slight west to east current.

The results at the swing bridge were very similar. However, there were small patches of a soft, very cohesive clayey silt material. At the most northerly of the five spans there were old timber piles cut off just a few feet beneath the water surface, a definite hazard to small boat navigation; otherwise, the bottom was covered with boulders and stones with coarse granular material in between. The bottom material changes to finer particles further away from the bridge.

4.3.2 Water Sampling

On October 24, 1978, water samples were taken at points on both sides of the existing causeway and bridge (Enclosure A-9). In total, twelve samples were taken, six on the west side and six on the east side. The sampling points were reached by boat.

Samples were taken essentially one to two feet below the surface by inverting 500 ml Nalgene polyethylene sample bottles. The capped, labelled bottles were returned to the laboratory the following day for immediate analysis of conductivity, total phosphorus, total Kjeldahl nitorgen, free ammonia, alkalinity, suspended solids and iron.

Temperature, dissolved oxygen, Secchi disc water clarity and pH were measured in the field.

4.3.3 Current Measurements

4.3.3.1 Drogue Tracking Measurements - Velocity observations were made on the afternoon of October 25, 1978, at each of the seven bridge spans over the two channels through the existing causeway. (Two spans at the overflow channel and five at the boat channel.) At the time there was a moderate southerly breeze, which could not have had any significant effect on the currents. The observations were made by releasing a tethered float from a boat moored centrally under each of the bridge spans in turn. Two observations were taken at all but one of the spans where four determinations were made. In each case the drogue body (crossed aluminum vanes) was set four feet below the surface, suspended from a small cylindrical polystyrene foam float attached to a thin nylon line. The current speed was determined by timing the pay-out of a measured length of the line (kept slack floating on the water). In all cases the direction of flow was from west to east - characteristic of the long-term average run-off induced flow from the Trent River System.

The results are listed in accompanying Table 4.10, from which it will be seen that observed current speeds varied from 0.29 ft/s to 0.88 ft/s.

TABLE 4.10

Drogue Tracking Current Observations
October 25, 1978

Location	Current Speed (ft/sec)
Overflow Channel:	
North Span	0.35, 0.37
South Span	0.42, 0.56, 0.71, 0.88
Boat Channel:	
North Span	0.56, 0.57
Next Span	0.53, 0.29
Swing Span	0.50, 0.59
Next Span	0.49, 0.59
South Span	0.62, 0.61

Little can be said of the results, except that the speeds observed fall into the range of values to be expected from the effects of run-off alone.

The observed variation in speed is probably due mainly to local boundary effects, but might also reflect an oscillating flow component caused by antecedent wind conditions - (oscillations not yet damped out by bottom friction, even though not generated by the wind condition observed while the measurements were in progress).

The collection and analysis of wind records taken during the drogue tracking were considered pointless. This was because at the time the wind was blowing gently at right-angles to the axis of the Bay and could have had little influence on the observed currents.

4.3.3.2 Supplementary Current Measurements - In addition to these cursory drogue measurements, the Ministry of Environment, including CCIW and Project Quinte, expressed a keen interest in carrying out precise current and water level measurements at the causeway to add to their resource data. In view of the desirability of having more precise information for future reference and comparison, the progress of this supplementary current measurement program is reported herein.

Four recording current meters were installed in the vicinity of the existing causeway as follows (Enclosure A-8):

	Location	Latitude	Longitude
#2001	Overflow Channel	44:08:54	77:23:17
#2002	Navigation Channel	44:08:39	77:23:12
#2003	East of Causeway	44:08:54	77:22:48
#2004	West of Causeway	44:08:30	77:22:36

The instruments were deployed by staff of the Ministry of Environment, Water Resources Branch, on October 30, 1978, and recovered on November 28, 1978. All four instruments record speed, direction and temperature nominally at 10 minute intervals, although one instrument (location code #2002) was subsequently found to have triggered somewhat irregularly at a mean interval of 7.82 minutes. Instrument location code #2001 also displayed a dominant flow direction almost diametrically opposed to that expected and actually recorded by instrument location code #2002. Apart from these problems which can, if necessary, be allowed for in subsequent analyses, the output from all four instruments was judged to be good and valid.

In addition, Fisheries and Oceans Canada supplied and deployed one float well water level recorder on Belleville Public Wharf, and two pressure type water level recorders: one each on the same moorings as current meters, location codes #2003 and #2004. Data from these three instruments is not yet available.

4.4 ANALYSIS

4.4.1 Wave Climate

There are, in the context of this Study, two types of wave action to be considered. The first and most pervasive is due to wind generated waves, while the second concerns waves generated by the passage of boats. Although the term "wave climate" normally applied to wind generated waves, in this Study it has been stretched to also include boat generated waves.

4.4.1.1 Summary of Wind Wave Conditions - The area east of the causeway near the Belleville Public Wharf and the mouth of the Moira River is subject to wave attack from directions east through southwest. The wave climate is mild and waves exceeding 0.6 m (2 ft.) significant height and 3.0 to 3.5 seconds period are rare. It is noted, however, that the east side of the causeway is sheltered from the dominant westerly winds, except for the very limited area affected by waves propagating through the causeway channels.

The total wave climate on the west side of the causeway, in the vicinity of the intake, has not been developed in detail, however, it is apparent from the limited depth and effective lengths of wave generating areas, that it too is mild. However, due to its exposure to westerly winds, waves exceeding two feet in height will occasionally be experienced.

The special case of waves generated on one side of the causeway being propagated through the overflow channel to reinforce waves locally generated on the opposite side has been investigated in detail since it forms the basis for evaluating changes if the channel is subsequently enlarged. Computations outlined previously demonstrate that local wave energy can be increased as a result of waves propagating and diffracting through the channel. However, these occurrences of increased wave energy levels can only occur when winds blow within narrowly defined directional sectors. For the intake area the total frequency of occurrence of the critical direction is about 10%, and for the mouth of the Moira River about 16%.

For severe conditions, with recurrence intervals in the range 2 to 8 years, the resultant significant wave characteristics at the intake would be 1.1 feet and 2.4 seconds, and at the Moira River 1.8 feet and 2.65 seconds.

4.4.1.2 Boat Generated Waves - Boat generated waves are relevant to this assessment because it is necessary to determine whether the eventual relocation of the main navigation route will have any impact - for example, on the Belleville City Water Intake, which lies close to the proposed approach channel.

Boats propelled through the water generate a complex system of waves which propagate outwards on either side of the vessel in the familiar "arrow-head-herringbone" or echelon pattern. These are the so-called Kelvin waves which emanate from the bow, from the stern and from any abrupt change in vessel form near the waterline. In addition, there is a less obvious system of longer waves, remaining closer to the vessel, sometimes referred to as the Bernoulli Contour Wave system, caused by the impulsive flow of water displaced by the hull as the vessel advances.

Boat generated waves have been quite intensively investigated and it has been found that, while vessel speed is the most important factor determining wave energy, numerous other factors also influence such waves. These factors include vessel displacement, length, beam, draft, hull form, water depth and proximity of the banks as well as the form of bed and banks.

In Canada the usual source of information on boat generated wave problems is Public Works Canada, which is responsible for implementing Canadian government policy on shore erosion caused by ship waves. Contact with the Ontario Region Office of Public Works Canada indicated that no complaints had been received from the Belleville-Quinte area regarding adverse effects of boat generated waves and that no information was available for the area. The absence of information from this source is a possibly favourable indication, although it means that this Study will have to rely on indirect methods for assessment of existing boat wave conditions.

Since small power boats will be unaffected by the new bridge and will presumably continue to negotiate the causeway through either opening as at present, they need not be considered. Therefore, only the larger vessels which at present require the opening of the swing bridge, and which in future will use the new navigation channel at the present overflow channel, need be considered.

Records and projections based on the swing bridge operator's records (MTC, 1976) are therefore of interest. These are summarized in Table 4.11.

TABLE 4.11

Boat Traffic Using the Swing Bridge (1)

Year	No. of Boats
1970	2900
1971	2900
1972	3200
1973	3400
1974	3600
1975	4000
1976 (2)	4300
1977 (2)	4400
1978 (2)	4600

Notes: (1) Based on MTC report W.P. 134-74-00, December 1976.
(2) Predictions.

While there is a considerable mass of experimental and field measurement data available on ship waves, none of it appears to apply to sailing craft under power. Furthermore, because of the wide range of parameters which affect ship waves, it is difficult to find examples in the literature which are directly applicable to the present situation. However, the data given in Tables 4.12 and 4.13 provides some idea of boat generated waves for some sizes of vessels which could (in principle at least) be found in the Bay of Quinte. Though most of the data is for water much deeper than Quinte, the second Table provides some indication of the effect of shallow water.

The Tables also show that wave heights diminish rapidly with distance from the sailing line, so that the significance of boat generated waves depends on the distance from the navigation channel to the area of interest. A second factor is vessel speed and whether any speed limit is applied.

For the present purposes it suffices to suggest that waves up to three feet in height might be expected at a distance of 100 feet from the sailing line, while at 500 feet wave heights might approach one foot. Periods associated with the Kelvin wave system are unlikely to exceed two seconds. Periods of the Bernoulli Contour Wave are much longer: 10 to 20 seconds.

4.4.1.3 Comparison of Wind and Boat Generated Waves - Wind generated waves, when present, build up over several hours and persist at least as long as the wind continues to blow. Boat generated waves are transitory. They typically consist of an initial group of four or five large waves, followed by ten or more waves of much lesser height. Underlying these waves, which form the Kelvin wave system, there is the much longer Bernoulli Contour Wave, which is normally only apparent close to the vessel or on sloped shorelines of appropriate inclination. The total duration of a typical single boat wave system, such as could be expected in the project area, is thus up to 10 seconds for the largest waves and about 30 seconds overall - excluding minor ripples and reflections which may persist for a minute or two.

Therefore, a traffic volume of about 5,000 vessels per year will produce "large" bow generated waves, with an aggregate duration of about 14 hours and smaller following waves with a total duration of about 28 hours - neglecting ripples and reflections. For a six month long boating season (May to October inclusive) the frequency of occurrence of boat generated waves thus amounts to less than 1%.

TABLE 4.12

Boats and Boat Generated Waves

For a speed of 10 knots in water 35 feet deep

					Distance from Sailing Line			
					100 ft		500 ft	
Boat	Length (ft)	Beam (ft)	Draft (ft)	Displacement (tons)	H Max ft.	T/2 sec	H Max ft.	T/2 sec
Cabin Cruiser	23	8.25	1.66	3	1.1	-	0.8	-
Coast Guard Cutter	40	10	3.5	10	1.6	1.0	1.0	1.0
Tugboat	45	13	6	29	1.6	1.2	0.9	1.2
Fishing Boat	64	12.8	3	35	1.8	1.0	0.7	1.0
Fireboat	100	28	9-12	343	1.6	1.3	1.0	1.3

Source: Johnson, J.W., "Ship Waves in Shoaling Water", International Coastal Engineering Conference, 1968
ASCE pp 1489-1498

The Table shows results of field measurements made in Oakland Bay, California.

TABLE 4.13

Heights of Waves Generated by an Auxiliary Supply Vessel

Vessel Speed (kn)	Distance from Vessel (ft)	Max. Wave Heights (ft) Depth 12.3 ft	Depth 27 ft
6	80	0.2	-
	400	-	-
	600	-	-
8	80	1.2	0.3
	400	0.5	0.1
	600	0.4	-
9	80	3.4	0.6
	400	1.7	0.2
	600	1.3	0.1
10	80	*	1.2
	400	*	0.5
	600	*	0.4

Notes: (1) - denotes negligible wave height.
* denotes beyond possible speed range.

(2) Results are derived from model tests of a vessel with the following characteristics:

Length O/A	156 ft
Beam	36.3 ft
Draft	9 ft
Displacement	1,000 tons

Source: Hay, D., "Ship Waves in Navigable Waterways", International Coastal Engineering Conference, 1968, pp 1472-1487

It should be noted that the largest boat generated waves may be higher than the highest wind generated waves at distances less than 100 feet from the sailing line. However, boat wave periods are generally less than those of wind waves of similar height. Hence, boat waves have less effect on the bottom and are less likely to disturb bottom sediments, although of course the jet from the propellers of a deep draft vessel might well have a much greater effect.

4.4.2 Sedimentation

Surficial bottom samples (numbers 1 to 11 inclusive), (Enclosure A-9), were subjected to particle size analysis.

Samples 4, 5 and 6 contained a mixture of pebbles, shells and stones. The other samples were of a more uniform mud-like consistency.

Portions of the pebbles, shells and stones of samples 4, 5 and 6 were filtered onto Whatman white filter paper and dried. They were then placed in a petri dish on the epidiascope as an input to the Quantimet 720 Image Analysing Computer. Equivalent circular diameters were calculated from the projected areas of the particles at a screen magnification of 8X.

Samples 4, 5 and 6 were then stirred and portions of the liquid from the samples which contained suspended particles were withdrawn and placed on microscope slides. A coverslip was put in place and the particles were examined under the Leitz Ortholux miscroscope in transmitted illumination. Diameters were calculated on the Quantimet at a screen magnification of 146X.

For the rest of the samples, the mud was stirred and a portion was placed on a microscope slide. A few drops of water were added to disperse the particles and a coverslip was put in place. The particles were examined under the microscope in transmitted illumination and analysed on the Quantimet at a screen magnification of 146X.

The Sediment Sample characterizations are given in Table 4.14. The data on analysis are presented in Appendix B.

TABLE 4.14

Sediment Sample Characterization

Sample	Visual Characterization
1 to 11	See Appendix B
12	Silt; as in No. 8 and 2
13	Mostly silt with a few small clay lumps, like No. 8 or 2
14	Silt; as in No. 8 or 2
15	Silt; as in No. 8 or 2
16	Predominantly clay, small percentage silt, as in No. 10
17	Sandy-silt with snail shells
18	No samples
19	No samples
20	Silt; as in No. 8 or 2
21	No samples
22	No samples
23	Mostly clay, some silt, much organic matter; somewhat like No. 11
24	Sand and small stones, snail shells, like No. 4 but stones much smaller
25	No sample
26	Silty - Clay on top and sandy-gravel at bottom; top layer like No. 11, bottom layer somewhat like No. 4 but smaller stones
27	Predominantly clay, small percentage silt, like No. 10
28	Silt; like No. 8 or 2
29	Clay with some silt; like No. 10
30	No samples
31	Sand; bark and snail shells
32	Mostly silt; some clay, like No. 8 or 2

The additional bottom samples (Nos. 101 to 112, Enclosure A-16) of the proposed dredging area of the new navigation channel which were taken on July 17, 1979, were taken by the diver scooping the firmer surface material into a sample jar and sealing before bringing to the surface. Six of these samples (Nos. 101, 102, 105, 108, 110 and 111) were subjected to laboratory analysis by the Consultant and the results are indicated on Table 4.15.

TABLE 4.15

ANALYSIS OF SEDIMENT SAMPLES
BELLEVILLE - ROSSMORE CAUSEWAY
(SAMPLES TAKEN JULY 17, 1979)

SAMPLE NO.	% MOISTURE	% LOSS ON IGNITION	COD (mg/gm)	TKN (mg/gm)	Zn (ug/gm)	Pb (ug/gm)	Mg (mg/gm)	VISUAL OBSERVATIONS
101	73.2	23.33	438	10.45	65.31	27.34	4.32	Clay-silt, fine particles, some small stones a few mm. in size
102	73.7	20.70	454	10.35	59.25	13.47	5.49	Clay-type sediment; some insect larvae present
105	11.7	1.13	19	0.37	0.36	4.20	1.30	A mixture of sand and pebbles; some small stones to 2 cm. in length; some old snail shell and arthropods present
108	50.7	10.37	239	4.51	44.30	29.53	6.64	Silt and coarse sand; some organic matter in the form of arthropods and detritus
110	77.8	22.75	557	12.60	122.70	95.41	5.06	Fine silt; some arthropods present
111	25.8	3.41	52	1.03	23.52	14.26	2.48	Coarse sand and small stones to 1 cm. in size; some arthropods present

N.B. All results expressed as dryweight.

The samples were analysed for the same parameters as MOE recorded in their May 31st, 1979, report. Unfortunately, that analysis reported Mg (magnesium) instead of mercury (Hg) and, as a result, the present laboratory analysis was therefore done for magnesium instead of mercury. The results indicate that only Samples No. 105 and 111 meet the criteria for permissible levels. The remaining samples appear very organic in nature and do not meet the criteria for volatility and total nitrogen, but they come somewhat closer to meeting the criteria for the heavy metals, lead and zinc.

It would appear that the majority of the surface area is predominated by organic, silty sand with some non-organic areas. The non-organic areas appear to be located where the surface silt layer has been scoured off.

Samples No. 107, 108, 110 and 111 were submitted to laboratory testing by MOE. However, the results of this analysis are not yet available.

4.4.3 Mass Water Movement

4.4.3.1 Estimated Causeway Channel Currents Due to Run-off - These currents, which flow eastwards through the causeway channels, result almost exclusively from the discharge from the Trent River System. The long-term average rate of flow was estimated as 125 m^3/s, producing an average causeway channel velocity of 0.11 m/s. Extreme runoff discharge rates were derived from areal extrapolation from a Fisheries and Environment Canada Study of the upper 73 percent of the 12,500 km^2 Trent River Basin. The values obtained in this manner ranged from 0.5 m/s once in ten years to 0.65 m/s in a century, corresponding to mass flow rates from 560 to 770 m^3/s.

Subsequently, MTC drew attention to the existence of another data source (Trent Canal Authority) giving extreme flow rates for the whole Trent River Basin. These new data indicated that the areal extrapolation must have produced low estimates of the extreme discharges entering the Bay of Quinte. On the other hand, it was also reasoned that the large surface of the Bay of Quinte "upstream" of the causeway provides a large buffer storage, so that the instantaneous extreme discharges and velocities due to run-off to be expected at the causeway would not be correspondingly increased. It was therefore concluded that the run-off component of causeway channel flow need not be further investigated or adjusted, since it is in any case not the major cause of the most extreme velocities which can occur.

4.4.3.2 Wind-Driven Reversing Flows - There is good reason to assume that wind shear force, acting on the water surface of the Bay is capable of setting up complex patterns of oscillation, which will produce flow rates and velocities through the causeway channels considerably higher than those due to run-off alone. These currents are essentially unsteady in character, a form of long wave action. They are difficult to quantify, except by means of advanced finite difference methods requiring a considerable expenditure of computer time and supporting effort. Since such an approach is not considered warranted, a greatly simplified method of estimating extreme wind generated currents was adopted. This method involves the determination of flows associated with the development of wind set-down under the influence of extreme winds blowing from the west along the axis of the Bay.

The combination of the wind-set-down effect and a moderately large run-off effect, together with a considerable number of assumptions, yielded the results, summarized in Table 4.16

TABLE 4.16

Estimated Extreme Eastgoing Causeway Channel Velocities Due to Wind Effects and Run-Off

Frequency (per Year)	Velocity (m/s)	
	Mean	Maximum
1/10	1.2	1.7
1/20	1.4	2.0
1/50	1.6	2.2
1/100	1.7	2.5

Source: Preliminary Evaluation of Bridge Pier Hydraulic Report.

Note:
- Mean velocities are averages over the assumed 9-hour period required to develop a maximum set-down.
- Maximum velocities are peak values achieved during the 9-hour period.
- Westgoing currents would be lower due to run-off currents opposing the wind effect.

4.4.3.3 Typical Bay Currents - Away from the causeway channels, currents are much lower than the values previously discussed. For example, currents due to run-off will normally average 0.02 m/s and will reach 0.10 m/s only under extreme flood conditions. Wind induced bottom currents will be of the order of 1% of the wind speed. They will reach vlues of the order of 0.3 m/s only under the most extreme wind conditions, or due to very rapid changes in wind.

4.4.3.4 MOE Current Measurements - The data from the four current meters has been transcribed and partially analysed by B. Kohli of the Water Resources Branch of the Ministry of the Environment of Ontario. The following data are available for each of the four meters:

- Listings of edited current, speed direction, water temperatures, time and date. The edited data set is followed by a smoothed data set from the same listing (using fifth order binomial coefficients).

- Two-dimensional frequency of occurrence for current, speed and direction.

- One-dimensional frequency of occurrence of water temperatures.

- Auto-correlations for north-south and east-west currents along with the relative plots.

- Variance density spectra for north-south and east-west currents along with the relative plots.

Cross-correlations between results from pairs of meters were not performed however.

The two-dimensional frequency data for current, speed and direction are presented in Tables 4.17 to 4.20.

Temperature frequency distributions were very similar for each of the four instruments, so only that for location code #2004 is indicated in Table 4.21.

The listings of current measurements exhibit the reversing flow pattern which had been anticipated. This is the indirect result of wind shear acting on the water surface. The variance density spectra show that the most common period of the flow reversal cycle is 5.45 hours for all four meters. It is understood that this corresponds to one of the periods of Lake Ontario which was also found by Environment Canada at Glenora.

It is suggested that the measurements are not inconsistent with the estimates of extreme flow rates previously developed, but this has not been proved conclusively. No provision has been made for additional analysis or evaluation of this data.

TABLE 4.17

Location Code:	2001		Period:	Nov. 7
Area:	8 Bay of Quinte	OVERFLOW CHANNEL	Latitude:	44 08 54 N
Lake:	Ontario		Longitude:	77 23 17 W

FREQUENCY TABLE

Speed (cm/s)	Direction (Coming From) in Degrees 337.50-22.49	22.50-67.49	67.50-112.49	112.50-157.49	157.50-202.49	202.50-247.49	247.50-292.49	292.50-337.49	Row Sums
1.00 - 2.99	4.05	7.84	3.55	1.84	1.65	0.81	1.50	2.69	23.94
3.00 - 4.99	3.43	14.97	2.38	0.60	1.03	1.10	2.60	2.96	29.07
5.00 - 6.99	1.17	13.90	1.88	0.31	0.26	1.00	2.79	1.19	22.51
7.00 - 8.99	0.55	7.46	0.95	0.12	0.14	0.38	1.65	0.55	11.80
9.00 - 10.99	0.14	4.51	0.52	0.14	0.02	0.24	1.05	0.31	6.94
11.00 - 12.99	0.10	2.00	0.31	0.07	0.0	0.19	0.48	0.02	3.17
13.00 - 32.99	0.0	1.45	0.24	0.0	0.0	0.10	0.74	0.05	2.58
Column Sums	9.44	52.15	9.85	3.08	3.10	3.81	10.80	7.77	100.00

Resultant Current is 3.02 cm/s at 30 degrees from north
Mean Current is 5.43 cm/s
Maximum Current is 32.99 cm/s
Minimum Current is 1.00 cm/s

Total No. Readings 4194
Persistence is 0.56
Readings taken every 10 min

Meter Operations

Meter operated at 2.7 m from bottom in 4.6 m of water

Started at 11.45 hrs. on 30th day of 10th month 1978
Ended at 14.25 hrs. on 28th day of 11th month 1978

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TABLE 4.18

Location Code: 2002
Area: 8 Bay of Quinte
Lake: Ontario

NAVIGATION CHANNEL

Period: Nov. 7
Latitude: 44 08 39 N
Longitude: 77 23 12 W

FREQUENCY TABLE

Speed (cm/s	Direction (Coming From) in Degrees 337.50-22.49	22.50-67.49	67.50-112.49	112.50-157.49	157.50-202.49	202.50-247.49	247.50-292.49	292.50-337.49	Row Sums
1.00 - 4.99	2.25	2.25	2.60	2.75	2.75	8.61	12.20	2.79	37.49
5.00 - 8.99	0.62	2.64	1.59	0.71	1.52	18.26	3.26	0.43	29.02
9.00 - 12.99	0.39	2.17	0.45	0.17	0.79	11.96	0.71	0.02	16.65
13.00 - 16.99	0.13	1.61	0.04	0.07	0.32	7.39	0.56	0.0	10.12
17.00 - 20.99	0.0	0.32	0.0	0.07	0.07	3.26	0.06	0.0	3.78
21.00 - 24.99	0.0	0.09	0.07	0.04	0.02	0.88	0.0	0.0	1.10
25.00 - 62.99	0.0	0.07	0.02	0.0	0.0	1.74	0.0	0.0	1.83
Column Sums	3.39	10.46	4.77	3.82	5.46	52.09	16.78	3.24	100.00

Resultant Current is 5.04 cm/s at 233 degrees from north
Mean Current is 8.10 cm/s
Maximum Current is 62.11 cm/s
Minimum Current is 1.04 cm/s

Total No. Readings 5345
Persistence is 0.62
Readings taken every 7.82 min

Meter Operations

Meter operated at 4.3 from bottom in 8.5 m of water
Started at 12.52 hrs. on 30th day of 10th month 1978
Ended at 13.22 hrs. on 28th day of 11th month 1978

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TABLE 4.19

Location Code: 2003
Area: 8 Bay of Quinte EAST OF THE CAUSEWAY
Lake: Ontario

Period: Nov. 7
Latitude: 44 08 54 N
Longitude: 77 22 48 W

FREQUENCY TABLE

Direction (Coming From) in Degrees

Speed (cm/s)	337.50- 22.49	22.50- 67/49	67.50- 112.49	112.50- 157.49	157.50- 202.49	202.50- 247.49	247.50- 292.49	292.50- 337.49	Row Sums
1.00 - 2.99	5.36	3.99	4.69	3.27	2.93	4.18	6.42	5.41	36.23
3.00 - 4.99	3.96	3.12	4.44	2.45	2.52	1.87	4.52	5.57	28.47
5.00 - 6.99	2.57	3.32	2.93	0.86	1.32	1.75	3.22	4.06	20.04
7.00 - 8.99	0.96	1.83	0.86	0.12	0.48	0.96	1.01	1.83	8.05
9.00 - 10.99	0.60	0.46	0.14	0.02	0.46	0.22	0.41	0.89	3.20
11.00 - 12.99	0.48	0.17	0.07	0.05	0.29	0.29	0.19	0.29	1.83
13.00 - 29.99	0.26	0.43	0.10	0.19	0.60	0.38	0.22	0.0	2.19
Column Sums	14.20	13.31	13.24	6.97	8.60	9.66	15.98	18.04	100.00

Resultant Current is 0.88 cm/s at 332 degrees from north
Mean Current is 4.54 cm/s
Maximum Current is 29.20 cm/s
Minimum Current is 1.00 cm/s

Total No. Readings 4162
Persistence is 0.19
Readings taken every 10 min

Meter Operations

Meter operated at 2.0 m from bottom in 5.6 m of water
Started at 21.24 hrs. on 30th day of 10th month 1978
Ended at 18.54 hrs. on 28th day of 11th month 1978

STEVENSON HARDTKE ASSOCIATES LIMITED

TABLE 4.20

Location Code: 2004
Area: 8 Bay of Quinte WEST OF THE CAUSEWAY
Lake: Ontario

Period:
Latitude: 44 08 30 N
Longitude: 77 23 36 W

FREQUENCY TABLE

Direction (Coming From) in Degrees

Speed (cm/s)	337.50-22.49	22.50-67.49	67.50-112.49	112.50-157.49	157.50-202.49	202.50-247.49	247.50-292.49	292.50-337.49	Row Sums
1.00 - 2.99	5.72	18.45	12.71	4.83	3.77	4.18	4.08	4.95	57.40
3.00 - 4.99	0.94	11.46	7.57	0.60	0.24	1.03	2.23	2.19	26.26
5.00 - 6.99	0.07	4.97	2.40	0.10	0.02	0.12	0.43	0.70	8.82
7.00 - 8.99	0.22	0.43	1.35	0.19	0.02	0.07	0.48	0.58	3.34
9.00 - 10.99	0.05	0.31	0.55	0.14	0.05	0.05	0.36	0.50	2.02
11.00 - 12.99	0.0	0.0	0.58	0.07	0.05	0.0	0.10	0.05	0.84
13.00 - 20.99	0.02	0.17	1.01	0.02	0.0	0.02	0.02	0.05	1.32
Column Sums	5.72	35.80	26.17	5.96	4.16	5.48	7.71	9.01	100.00

Resultant Current is 1.77 cm/s at 59 degrees from north
Mean Current is 3.43 cm/s
Maximum Current is 20.41 cm/s
Minimum Current is 1.00 cm/s

Total No. Readings 4162
Persistence is 0.51
Readings taken every 10 min

Meter Operations

Meter operated at 2.0 m from bottom in 5.6 m of water
Started at 20.55 hrs. on 30th day of 10th month 1978
Ended at 18.25 hrs. on 28th day of 11th month 1978

STEVENSON HARDTKE ASSOCIATES LIMITED

TABLE 4.21

Temperature Frequency

Bay of Quinte 2004 Nov. 78 West of Causeway

Temperature (Deg. C)	Frequency (%)
0.00 - 0.99	3.62
1.00 - 1.99	2.32
2.00 - 2.99	7.10
3.00 - 3.99	9.13
4.00 - 4.99	3.77
5.00 - 5.99	4.06
6.00 - 6.99	21.45
7.00 - 7.99	5.65
8.00 - 8.99	35.65
9.00 - 100.00	7.25
Total	100.00

Record Mean	6.46 Deg. C.
Record Std. Dev.	2.51 Deg. C.
Min. Value	0.0 Deg. C.
Max. Value	9.20 Deg. C.
Series Length	693

4.4.3.5 Consistency of Bottom Sediment Texture - The textural distribution of surficial bottom sediments is consistent with the deduced velocity conditions as described in the preceding sub-sections.

In the causeway channels and within a few hundred feet of them, the Bay bottom is armoured with coarse material, in places apparently artificial, but in general more likely as a result of the selective erosion of fine material. The sizes of bottom material range from rocks greater than 0.3 m, to cobbles, to coarse gravel. The finer material is more distant from the channel throats.

4.4.4 Water Quality

Analyses of all water samples for chemical parameters were carried out according to the methodology outlined in "Standard Methods", with iron being measured by the atomic absorption method.

Temperature and dissolved oxygen were measured in the field using a YSI (Yellow Springs Instrument Co.) model 54 oxygen meter. Field measurements of pH were made with an I.L. 175 portable pH meter.

All the analyses were done in accordance with the Standard Methods for the Examination of Water and Wastewater, 14th Edition: Publishers: American Health Association; American Water Works Association; Water Pollution Control Federation.

Table 4.22 shows summary Project Quinte data where the "B" and "T" stations correspond to the B and T locations in Figure 4.1.

Table 4.23 shows the results of the analysis of water samples taken for this Study on October 24, 1978, and analysed on October 26, 1978. Stations 1, 2, 3, 4, 5 and 6 were spaced along the west side of the existing causeway and stations 7, 8, 9, 10, 11 and 12 were spaced along the east side (Enclosure A-9).

Additional water quality data for the Belleville side were available through daily measurements of water quality carried out for the Belleville Utilities Commission. These data are shown in Appendix C.

TABLE 4.22

RANGES OF ANNUAL MEANS FOR THE EUPHOTIC ZONE AT

STATIONS T (TRENTON) AND B (BELLEVILLE) DURING THE PERIOD 1972-78

("Project Quinte" Data)

Parameter / Station		Secchi Disc (m)	Chloro a (µg/l)	Phosphorus (mg/l)		Nitrogen (mg/l)				Iron (mg/l)	Silica (mg/l)	Sodium (mg/l)	* Cond. (umhos/ cm^2)	Sus. Solids (mg/l)	Chloro b (µg/l)	** Alk. (mg/l)	*** Chloride (mg/l)
				Total	Sol. React.	NH_3	TKN	NO_2	NO_3								
T	Max.	1.4	28.3	67	15	95	860	3	32	0.21	4.2	6.2	266	11.4	2.1	106	8.3
	Min.	0.9	10.3	42	03	30	610	2	10	0.13	0.74	3.2	224	7.1	0.5	91.2	8.3
B	Max.	1.4	37.6	90	19	87	900	3	26	0.26	4.8	5.9	268	13.2	3.1	109	7.3
	Min.	1.0	12.5	48	05	37	630	2	5	0.14	0.72	3.3	222	6.5	0.4	94.4	7.3

* 1974-78

** 1976-78

*** 1978 only

TABLE 4.23

RESULTS OF 1978 WATER ANALYSES

(Carried out by Pollutech Limited)

Parameter	Sampling Station 1	2	3	4	5	6	7	8	9	10	11	12
pH	7.6	7.7	7.7	7.7	7.7	7.7	7.8	7.7	7.7	7.7	7.8	7.8
Conductivity (m mhos/cm)	0.20	0.24	0.24	0.23	0.24	0.24	0.25	0.19	0.20	0.20	0.23	0.25
Total Phosphorus (mg/l)	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Total Kjeldahl Nitrogen (mg/l)	0.85	0.74	0.90	0.77	1.06	0.98	0.88	1.54	0.85	0.85	0.80	0.77
Free Ammonia (mg/l)	0.30	0.38	0.36	0.28	0.20	0.28	0.33	0.28	0.23	0.23	0.28	0.23
Carbonate Alkalinity (mg/l)	88	106	104	104	102	104	104	100	100	100	104	102
Suspended Solids (mg/l)	4	4	3	4	4	3	4	3	5	5	4	4
Iron (mg/l)	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
Dissolved Oxygen (mg/l)	10.4	10.1	10.0	10.4	10.2	10.4	10.6	10.5	10.5	10.5	10.2	10.4
Secchi Disc (feet)	4	6	7	6.5	6.5	6.8	7	6.5	6.8	7.5	7	to bottom (4.5)
Temperature (^{o}C)	9	9	9	9	9	9	9	9	9	9	9	9

4.4.5 Fish

There are some fifteen to twenty species of fish known to inhabit the waters of the Bay of Quinte in the Belleville area. The species tend to consist mostly of warm-water species such as the centrarchid (Sunfish) and percid (Perch) groups. However, other species found in this area in some considerable numbers are Garpike, Bowfin, Catfish and Bullheads. To a lesser extent, species such as Alewife, Herring, Chinook Salmon and Coho Salmon are found in the Belleville area.

Walleye migrate through the Bay of Quinte in an easterly direction starting in the spring. Some of the Walleye migrate back in a westward direction towards Trenton in the fall. Alewife migrate within the Bay and White Perch migrate from the lower Bay to the upper Bay in late spring.

Presently, there are no known spawning grounds for Rainbow Trout and Chinook Salmon in the Bay or in the causeway area. There are possibly some spawning beds for Trout or Salmon in the Moira River but these beds remain uncharted. If there are spawning beds for percid or centrarchid species in the vicinity of the causeway, these are not well charted.

4.5 BASE DATA CONDITIONS

4.5.1 Boating and Recreation

4.5.1.1 Wind Generated Waves - The wind wave climate in the project area can be characterized as mild, with generally relatively low levels of wave energy, due to the shallow water depths and restricted width of the wave generating areas.

To the east of the causeway waves seldom exceed two feet significant height and 3.0 to 3.5 seconds spectral peak period. To the west waves are somewhat larger, due to the exposure to dominant westerly winds, but will seldom exceed three feet in height and 4 seconds period.

It is possible under appropriate conditions that some wave energy propagates through the causeway channels, after which it is dispersed by the process of diffraction. This effect can locally increase the heights and periods of locally generated waves. Wave conditions in the area of the intake and the mouth of the Moira River, including the effects of waves propagated through the overflow channel, have been investigated. Under severe conditions, with wind in the appropriate directions, wave energies in the areas mentioned can be locally increased to a significant extent. However, the "increased" wave energies must still be lower than they would have been prior to the construction of the causeway. The effects mentioned only occur with frequencies from 10 to 16 percent, when the winds are blowing in the appropriate directions.

4.5.1.2 Boat Generated Waves - Very little is known about the distribution of vessel types and sizes, although the traffic is overwhelmingly of recreational craft. A vessel height survey made in 1975 and summarized in MTC (1976) showed that 38% of vessels which required bridge opening were 20 feet or less in height. The remaining 62%, of greater height than 20 feet, must have been almost all sailing craft.

There is no local information on boat generated waves and no evidence that they have caused problems. Boat waves rapidly decrease in height with distance from the sailing line. At a distance of 100 feet waves up to three feet in height might exceptionally occur. At 500 feet distance waves should be substantially smaller than wind waves.

Because boat waves are transitory and of short duration, a traffic level of 5,000 larger pleasure craft per year will not cause a frequency of boat wave action exceeding 1% when averaged over the six month boating season.

Boat wave effects are localized and, because boat wave periods are shorter than those of wind waves of similar height, they cause less disturbance of the bottom than wind waves.

4.5.2 Shoreline

A number of major storms in 1972 and 1973 resulted in extensive shoreline damage on the Great Lakes. These storms arrived at a time when Lake levels were at an abnormally high stage, which compounded the extent and intensity of the damage incurred to shore property owners.

As a result of these occurrences, Environment Canada and the Ontario Ministry of Natural Resources undertook a Shore Damage Survey - 1973. Although this survey was completed during only one annual high water level period, significant conclusions can be drawn from the data collected. This source is the prime reference for establishing base data conditions.

The north shore adjoining the causeway, and extending some one-half mile in either direction, is municipally owned and used for open space recreational purposes. To the west, the shoreline is formed by a glacial deposit. East of the causeway, the shoreline has been extended with artificial fill at a relatively low level. The immediate shoreline is considered to have no real value.

The whole of the south shoreline adjoining the causeway is privately owned. Extending westward for almost one mile, it is occupied by both seasonal and permanent residences. East of the causeway it is generally under-developed except for seasonal residences located within five hundred feet of the causeway. The residential areas both east and west of the causeway have a shoreline value of between $200 to $500/m (1973), and the underdeveloped lands, a shoreline value of less than $200/m (1973). Many of the residential properties have constructed individual shoreline protection structures. The shoreline consists of glacial deposits with outcroppings of bedrock.

The Great Lakes Shore Damage Survey - 1973 indicates no erosion damage along the north shore, adjacent to the causeway. Only moderate inundation damage is recorded around Belleville Harbour.

Along the south shore, only a length of about two hundred feet west of the causeway is noted as having moderate erosion damage. No inundation damage is noted.

There are no highly dynamic areas which may be subject to successive periods of erosion and accretion noted along either of the adjoining shorelines.

4.5.3 Sedimentation

The bottom of the Bay is covered with a very soft - almost liquid - very mobile organic clay or "baymud", probably containing a high fraction of silt. Exceptions are the sandy bars at the mouth of the Moira River, and the channels through the causeway, which are described in the following sub-sections.

4.5.3.1 The Overflow Channel - The overflow channel located towards the northern end of the present causeway is effectively about 53 meters wide. The least depth, caused by a ridge or sill - directly under the bridge spans, is 3.01 meters below IGLD (Lake Ontario Datum). On either side of the causeway the approach channel is significantly deeper than under the bridge, reaching 5.81 meters on the east side and 5.01 meters on the west side. The ridge under the bridge suggests the presence of harder non-erodible material, which may have prevented the present channel from scouring to a natural equilibrium depth.

Since it is quite normal in constricted channels of this type for the maximum depth to occur at the point of constriction, it may be inferred that the unrestricted depth in the overflow channel would be in excess of 6 meters if the bottom were as easily erodible as that at either side of the causeway.

Observations made by the Consultant during bottom sediment sample collection indicate that the bottom throughout the area of influence of the overflow channel has been swept clean of bay mud and is paved with coarse gravel and cobbles. Apart from the ridge mentioned above, the paving or armouring of the bottom surface could be the result of selective removal of finer, more erodible material.

According to the MTC (1977) geotechnical feasibility report dated November 2, 1977, the bottom at the depths indicated would be expected to consist of sandy gravel containing at least 50 percent gravel. This was confirmed by the results of Borehole No. 100, taken at Station 20+797.5 on March 9, 1979, through the section of causeway to be removed (Enclosure A-11). Thus, it is assumed that the bottom has been naturally or artifically armoured and that the sub-bottom material comprises gravelly sand, which will be exposed after proposed excavation and dredging have been implemented.

4.5.3.2 The Existing Boat (Swing Bridge) Channel - The existing boat channel (with swing bridge) has a total effective width of about 183 meters. It contains four piers, including a cylindrical swing span pier, which support the present bridge. The water depth directly under the present bridge varies from as little as 1.1 meter to as much as 7.8 meters. The wide range and extreme irregularity of the bottom strongly suggests that parts of the new pier foundations of the first bridge (c. 1890-1920) remain submerged between the piers of the present bridge.

As in the case of the overflow channel, the greatest water depth is found at the approaches to the bridge rather than between the spans. On both the east and west sides of the bridge maximum depths exceed 10 meters relative to IGLD.

Bottom and sub-bottom conditions are also very variable to the east of the present bridge, on the alignment of the proposed new bridge. They vary from sandy gravel near the south shore, to exposed limestone bedrock, to very soft organic clay overlying the bedrock, and sandy gravel immediately north of the causeway channel. The scour resistant materials extend to near the centre pier of the swing span, while the portion to the north is composed of the soft organic clay.

On the western side of the causeway the bottom and sub-bottom conditions are similar to those on the east, except for a somewhat larger deposit of silt and organic clay on the surface, due to the presence of the causeway and the dredging and landfill operations carried out in this area in the past.

It seems probable that the very soft - in fact almost liquid - organic clay or baymud is the product of recent sedimentation in pits dredged in the gravel 60 years ago, to provide fill for the causeway construction. This very soft sediment can be classified as "easily erodible material" following Neill (1973).

4.5.3.3 Bay of Quinte - Beyond the areas of influence of flow concentrations of the causeway channels, the Bay bottom is coated with a fine-grained, soft - in places almost liquid - mud. This is typical of protected bay and estuarine environments subject to low currents and moderate wave action. Sediment texture analysis results presented elsewhere show that 50% to 70% of the material lies on the borderline between silt and clay sized material (2 to 12 microns).

The sediment textural distribution is consistent with the inferred velocity and mass movement conditions previously presented.

4.5.4 Water Quality

The vast majority of the information regarding the physical, biological and chemical characteristics of the Bay of Quinte has arisen as a result of investigations carried out under the auspices of "Project Quinte".

Project Quinte, which completed its seventh year in 1978, is a team project with participation by the Ontario Ministry of the Environment, the Ontario Ministry of Natural Resources, the Department of the Environment (CCIW), Queen's University and the University of Guelph. The project is aimed at determining and understanding the response of the aquatic ecosystem to the phosphorus - removal program and at improving the understanding of the dynamics of aquatic systems.

During the first three years of Project Quinte seven sampling stations were frequented on a weekly basis form May to mid-October; in 1975 an eighth station (LE) was added to determine the possible effects of waste heat from the Lennox generating station.

Operations carried out at each station included measurement of physical parameters, collection of chemical and biological samples and estimation of primary production. The sequence of operations was the same (with minor variations) at each sampling station and was repeated at approximately the same time of day from week to week and from season to season.

Water quality parameters which have been measured during the seven-year history of Project Quinte include: total and dissolved reactive phosphorus, total Kjeldahl nitrogen, free ammonia, nitrate and nitrite nitrogen, conductivity, alkalinity, silica, iron, sodium and suspended solids and chlorophyll a.

A summary of the water quality conditions in the Bay of Quinte up to the point of publication of the Ontario Ministry of the Environment (1975) most recent report is as follows:

- A gradient from extreme eutrophy in the upper Bay to mesotrophy near the Bay mouth exists in the Bay of Quinte.

- Loading of suspended solids from the major tributaries (especially the Trent River) and re-suspension of sediments due to wind action may affect Secchi disc, chlorophyll a and nutrient relationships in the upper Bay.

- The concentration of phosphorus and nitrogen does not appear to have altered since the mid-sixties. N:P ratios (weight) have remained relatively constant since 1972.

- There have been no trends among relevant trophic indicators (primary production, Secchi disc, chlorophyll a, total phosphorus, total nitrogen and total Kjeldahl nitrogen) since 1972 which would indicate a change in the trophic status of the Bay.

- The shallow upper Bay warms quickly in the summer and is well mixed by prevailing winds. Weak or unstable thermal stratification may occur during times of prolonged calm weather.

- The middle Bay often experiences pronounced thermal stratification, generally during the period mid-May to mid-July. This appears to be related to internal seiche action in the lower Bay which may force cold oxygenated water from the lower Bay into the middle Bay.

- The lower Bay, which deepens to 55 meters near the Bay mouth, experiences persistent thermal stratification during the summer months but, even at the deepest point, the temperature gradient from surface to bottom is a gradual one. Bottom dissolved oxygen concentrations decrease steadily over the summer and may be reduced as low as 2 mg O_2/l by late summer.

Since 1975, Project Quinte investigations have continued, although no official data have been published.

Analysis of the detailed sampling program results of Project Quinte and of this Consultant's sampling program of October 1978 indicate the water quality to be the same (within the bounds of experimental error and true variations) on both sides of the causeway.

4.6 ENVIRONMENTAL ASSESSMENT

4.6.1 Wave Climate

The existing conditions throughout the part of the Bay of Quinte which includes the site of the proposed new bridge can be generally characterized as relatively benign with generally low levels of hydrodynamic energy typified by the presence of the layer of soft, almost liquid, mud.

Only in the area of the causeway channels (which were created over 60 years ago by the construction of the existing crossing) does the evidence point to comparatively high energy levels. These higher energy levels, mainly from the local concentration of currents, are caused by wind action and, to a lesser extent, by run-off from the Trent River.

The overall effect of the existing causeway has been to reduce average wave energy levels in the area, since it substantially shelters each side from waves which approach from the other side of the causeway. It is not known whether this sheltering effect has affected bottom sediment, but it is probable that it has, encouraging a more rapid sedimentation of the mud than would otherwise have occurred.

4.6.2 Current and Mass Water Movements

Apart from the causeway channel areas, currents in nearby areas of the Bay are generally very low. In the western approach to the crossing area currents are typically in the order of 0.02 m/s and under exceptional wind conditions bottom currents could rise to 0.3 m/s. Local concentrations of higher than average velocity may result from wind-driven circulations. However, the maximum velocity concentrations anywhere in the Bay are found in the causeway channels. Here the average net flow due to run-off from the Trent is estimated to be 0.11 m/s and the maximum wind-induced current 2.5 m/s. These velocities correspond to mass flow rates 125 m^3/s and 2,840 m^3/s respectively.

The wind-induced component of mass flow is reversible, varying according to wind speed and direction. The extent of mixing of waters on the two sides of the causeway depends to a significant extent on the periods of the reversing component of flow. The longer the periods, the more effective the mixing.

Preliminary evaluation of the MOE current measurements confirms the presence of the reversing flow patterns through the causeway opening, which had been anticipated. The most common period of flow reversal cycle is 5.45 hours, which is understood to correspond with one of the periods of Lake Ontario which was also found by Environment Canada at Glenora.

4.6.3 Sedimentation

Data presented elsewhere in this Report shows that the bottom areas in and close to the causeway channels have been stabilized partly as a result of artificial armouring in the case of the overflow channel, but mainly as a result of natural armouring due to selective erosion of fine overlying material.

The remainder of the Bay bottom is almost everywhere covered with a very soft, almost liquid clay-silt mud, which must be relatively stable under normal conditions, though frequently disturbed under severe storm conditions, probably mainly by wave action rather than currents.

Striking evidence of the mobility of the mud is the fact that a dredged borrow pit, dredged over half a century ago, about 40 feet deep, located on the east side of the causeway near the swing bridge, is now completely filled with the bay-mud.

The Bay bottom in the area of the Belleville City Water Intakes was also covered in the type of mud described above. Diver observation revealed no signs of local sediment disturbance due to the flow into the intakes.

Bottom sediment conditions in the vicinity of the mouth of the Moira River are somewhat variable. Although soft mud still predominates, there is also a large shallow bar of dense sand immediately west of the River mouth. The sand bar is probably composed of material deposited from the Moira River. Since it is less mobile than the surrounding mud, it can be assumed that the sand bar is stable - though perhaps accreting due to annual additions from the Moira River.

4.6.4 Water Quality

The concensus among those conducting research on the Bay of Quinte is that the Bay of Quinte is in an advanced state of eutrophication. Water quality at present is such as to seriously impair recreational use of the Bay and to restrict its use as a source of water supply. The recreational and commercial fisheries, which were once among the most productive in Ontario, have declined drastically.

The results of Project Quinte up to 1975 have been published. Results since that date are available from the various investigators only piecemeal, and the complete data base will not be available until the next round of published papers (expected in 1979).

The indications are, however, that the overall conditions within the Bay have not changed appreciably within the last three years compared to the first four years that they were studied (in Project Quinte).

Further, the seven years of continuous water sampling of Project Quinte and the cursory water sampling carried out in October, 1978, by this Consultant indicate that within the bounds of experimental error and true variations, the water quality is the same on both sides of the causeway.

This similarity of water quality on either side of the causeway is of particular significance as it reduces the need to understand the complexities of mass water movement through the causeway openings.

4.6.5 Fish

The fifteen to twenty species of fish known to inhabit the waters of the Bay are mostly warm water species such as the centrarchid (Sunfish) and percid (Perch) groups. Other species found in this area are Garpike, Bowfin, Catfish and Bullheads, and to a lesser extent Alewife, Herring, Chinook Salmon and Coho Salmon. Walleye migrate easterly through the Bay in the spring with some returning in the fall. Within the Bay both Alewife and White Perch migrate from the lower Bay to the upper Bay.

There are no known spawning grounds for either Rainbow Trout or Chinook Salmon in the causeway area, although there are possibly some uncharted spawning beds for Trout or Salmon in the Moira River.

SECTION 5
PROJECT CONDITIONS

SECTION 5 - PROJECT CONDITIONS

5.1 INTRODUCTION

This Section is concerned with establishing the conditions which will exist within the influence of the causeway upon completion of the Project.

The Project involves the construction of a new high-level bridge structure and approaches, located approximately 30 meters east of the existing Highway 14 causeway and swing bridge between Belleville and Rossmore (Enclosure A-11). This construction will be accompanied by the removal of approximately 40 meters of the causeway south of the fixed bridge, together with the necessary dredging to develop the new navigation channel.

The proposed new high-level bridge will provide a vertical navigation clearance of 22.5 meters and a maximum roadway grade of five percent. The superstructure is to be supported by vertical concrete piers founded on concrete pile caps. The substructure of the pile cap will consist of steel tubes of 1,219 mm diameter driven to rock. Inside each tube are three HP310 steel piles, which will be driven into rock or socketed into rock as required. The tubes will then be filled with concrete. The concrete bridge abutments will also be founded on steel piles driven through a granular core.

The removal of the section of causeway south of the fixed bridge will involve excavation of gravelly sand, with a trace of silt. The dredging associated with the removal of the causeway will encounter organic clay, sand and gravel.

The preliminary foundation investigation Report by the Ministry of Transportation and Communications, November 1977, indicates that the subsoil condition beneath the Bay and beneath the adjoining lands are somewhat different, subsoil beneath the land being slightly more competent. The adjoining lands were found to be composed of a conglomerate of granular fill, sanitary landfill and topsoil. As this fill material might be detrimental to the stability of the embankment approaches, the Study recommended that the fill material and organic clay be removed entirely within the plan land limits of the proposed embankment, and replaced by a granular type of acceptable fill material. This procedure will not require the in-water excavation of any in-situ material as the existing in-water fill or organic material will be displaced by dumping rock fill from shore.

Enclosure A-11 shows the location of the new bridge piers in relation to the existing causeway and the limits of the new navigation channel.

For purposes of this Study, two aspects of the proposed bridge layout are relevant:

- the location of the pier relative to the causeway channels, and
- the extent of the enlargement of the existing overflow channel.

The effective width of the existing overflow channel is 53 meters. To provide adequate clearance for the proposed navigation channel, the overflow channel will be widened on its south side, by some 40 meters, and the new navigation channel deepened to 3.7 meters (12 feet) below lake datum, for a bottom elevation of 70.3 meters (230.8 feet). For purposes of this Study, it is assumed that the effective width of the overflow-navigation channel is 100 meters.

With this arrangement, Piers 1, 2, 6 and 7 will be subject to the effects of currents and ice flowing through the causeway openings.

5.2 ANALYSIS

In the following Sub-sections, the Project Conditions relative to waves, sedimentation, mass water movement, water quality and fish are examined.

5.2.1 Waves

5.2.1.1 Wave Refraction - The analysis previously given for neglecting wave refraction in consideration of Existing Conditions are also applicable to the present consideration of Project Conditions. In areas where refraction does occur to a small extent, it will tend to cause dispersion of wave energy, and hence the neglect of refraction will result in some overestimation of wave heights. Thus, the determination of wave propagation through the enlarged overflow channel may safely be based on wave diffraction alone.

5.2.1.2 Wave Diffraction - The procedure for determining the characteristics of waves refracted through the enlarged overflow channel is identical to that previously described for Existing Conditions, except that the gap width is assumed to be increased and standard diffraction solutions applicable to the new width are applied (CERC, 1973). Further, the same fetch designations and the same initial wave conditions apply.

Enclosures A-1 and A-2 show that the waves generated in easterly fetch E1 are diffracted through the gap towards the City Intake area, and waves generated in the westerly fetch W1 are diffracted towards the Belleville Harbour area near the mouth of the Moira River. Table 5.1 summarizes wave diffraction effects for the Project Conditions.

Enclosures A-12 and A-13 depict the conditions considered. In both cases the inner part of the diffraction zone is represented by mirror image solutions applicable to refraction at the end of a semi-infinite jetty (from CERC, 1973). This solution extends to 10L by extrapolating the K'=1 line. Beyond 20L it is assumed that radial dispersion of wave energy occurs. On this basis the following results are obtained.

(a) Wave height at the City Intake due to diffraction of 1.90 foot high waves of 2.4 seconds period through the new enlarged overflow-navigation channel is 1.08 feet. The period is unchanged.

(b) Wave height at the mouth of the Moira River due to diffraction of 2.45 foot high waves of 2.65 seconds period through the new enlarged overflow-navigation channel is 1.20 feet. The period is unchanged.

These results would be valid in the absence of locally generated waves from fetches E2 and W2, which must next be superimposed on the above waves.

TABLE 5.1

Summary of Wave Diffraction Conditions

	Fetch El	Fetch Wl
Incident wave height (ft)	1.90	2.45
Incident wave period (s)	2.40	2.65
Deep water wave length (ft)	29.5	36.0
Water depth (ft)	17.5	15.0
Local wave length L (ft)	29.5	35.6
Gap width (ft)	328	328
Relative gap width (B)	11L (approx)	9L (approx)
Distance to areas of interest (E2, W2) (ft)	2000	3000
Relative distance to areas of interest (L)	68L (approx)	84L (approx)
Angle of incidence (deg.)	90	90

5.2.1.3 Combined Wave Effect - The resultant wave condition (at each of the two areas considered) was determined by superposition of the following pairs of wave trains:

At the Intake:

Wave diffracted from El	1.1 ft height	2.4 s period
Wave generated in E2	0.9 ft height	1.7 s period

At the Moira River Mouth:

Wave diffracted from W1	1.2 ft height	2.65 s period
Wave generated in W2	1.5 ft height	2.0 s period

Waves generated in fetches E2 and W2, listed above, are from Table 4.7 of Existing Conditions.

The resultant wave conditions listed below were obtained by adding the wave energy fluxes for each pair and taking the longer period of each pair as the period of the resultant.

Combined wave at the Intake	1.3 ft height	2.4 s period
Combined wave at the mouth of the Moira River	1.9 ft height	2.65 s period

These results indicate the maximum influence of the new enlarged overflow-navigation channel on waves at the City Intake and the mouth of the Moira River. These examples represent severe conditions likely to be experienced in the month of April, with relative frequency of about 0.01% corresponding to return periods of 2 to 8 years.

5.2.1.4 Wave Climate - The wind wave climate under project conditions will be identical to that under existing conditions, except for changes due to the enlargement of the overflow channel to form the new navigation channel, which only affect the Intake and the mouth of the Moira River.

At the 0.01% frequency level for the month of April, the existing and project combined wave conditions have been estimated as follows:

TABLE 5.2

Comparison of Extreme Conditions at the Intake and the Mouth of the Moira River

		Existing Conditions	Project Conditions
Intake:	height (ft)	1.1	1.3
	period (s)	2.4	2.4
Moira River Mouth:	height (ft)	1.8	1.9
	period (s)	2.65	2.65

The waves under Project Conditions are still small. They are smaller than the largest waves which can reach the Intake and Moira River mouth areas without diffraction under different wind conditions. For example, the largest waves at the Intake under westerly winds approach 3 foot height and 3.5 second period, while the largest waves at the mouth of the Moira under easterly and southeasterly winds approach 2 feet and 3 second period.

5.2.2 Sedimentation

The only change in bottom sediments under Project Conditions will be that resulting from the removal of part of the causeway and the dredging to form the new navigation channel.

The initial effects of these changes will be to:

(a) increase the bottom width of the overflow channel;

(b) remove the existing armouring of coarse erosion-resistant material from its bottom, and

(c) expose the underlying sandy gravel which is presumed to be more easily erodible.

The presence of the gravel is indicated in the MTC geotechnical feasibility report of November 2, 1977. Borehole No. 100, taken March 9, 1979, through the portion of the causeway to be removed (Enclosure A-11), indicates fill material of sand with gravel and trace silt with numerous cobbles over sandy gravel to gravelly sand with trace silt to elevation 64.4 meters.

The exposed sandy gravel will be subject to scour to a depth and extent dependent on two factors. First, the maximum flow velocity in the navigation channel, and secondly whether the sandy gravel contains larger sized material in sufficient quantity to re-armour the bed and thereby halt bottom scour.

5.2.3 Mass Water Movement

The following account of currents and mass water movement under Project Conditions has been derived from the few ad hoc tethered drogue observations made on October 25, 1978, and the cursory examination of the MOE current measurement results, and is consistent with the Preliminary Evaluation of Bridge Pier Hydraulics Report.

5.2.3.1 Flow Distribution Patterns - The cross sectional area of the channels through the causeway for the existing and the Project Conditions are:

	Existing Navigation Channel (m^3)	Overflow Channel (m^3)	Total (m^3)
Existing Conditions	1,000 (84%)	190 (16%)	1,190 (100%)
Project Conditions	1,000 (69%)	450 (31%)	1,450 (100%)

For the Project Condition, the total flow area is increased by 22 percent.

It will be recalled that two types of water movement can occur: the one-way west to east flow, due to the Trent River runoff, and the reversing wind-induced flow. The enlargement of the overflow channel to form a new navigation channel could have a somewhat different effect on each of these two types of flow. However, for the purpose of defining changes in flow distribution patterns, it will be assumed that discharges associated with both runoff and wind-induced flows are unaltered, and that discharge distribution is proportional to the distribution of cross sectional areas.

On this basis the average velocities at any given moment will be the same in the two channels, and the change in velocity following enlargement of the overflow channel will be inversely proportional to the change in the total causeway channel cross-section.

The enlargement of the overflow channel will therefore change the percentages of the total flow which passes through the two channels in proportion to the relative change in cross sectional area. The proposed new navigation channel will therefore pass about 31 percent of the total discharge and the old navigation channel the remaining 69 percent, compared to the corresponding existing proportions of 16 percent and 84 percent. Thus, the discharge through the proposed navigation channel will be 94 percent greater than that through the existing overflow channel, while the discharge of the abandoned navigation channel will be reduced by about 18 percent. The change in flow distribution between the two causeway channels will induce some changes in the flow patterns approaching and leaving the causeway area. Enclosures A-10 and A-14 indicate in two-dimensional terms typical Existing and Project Conditions for an easterly flow.

Apart from the cross-wind effects, the assumption that the velocity at any given time is the same in both causeway channels will be correct if the boundary resistances (friction, and pierform resistances) of the two channels are similar. If there was any difference in these resistances, it would be more likely to affect existing conditions than Project conditions, and it is possible that the overflow channel carried a slightly smaller proportion of the flow under existing conditions than estimated.

5.2.3.2 Estimated Channel Currents Due to Runoff - The long-term average runoff from the Trent River System was previously estimated to be $125m^3/s$ ($4,410ft^3/s$). Under Project Conditions this corresponds to a long-term average eastgoing velocity of 0.09m/s (0.28ft/s) through the causeway channels. Corresponding runoff discharges and velocities for recurrence intervals of ten years and 100 years were found to be: $560m^3/s$ ($19,800ft^3/s$) for a velocity of 0.39m/s (1.27ft/s), and $770m^3/s$ ($27,170ft^3/s$) for a velocity of 0.53m/s (1.74ft/s). These velocities will exist only under completely calm conditions. However, the flow due to runoff will usually be combined with a reversing flow component due to wind or atmospheric pressure changes.

5.2.3.3 Wind Driven Reversing Flows - The mechanisms for wind driven reversing flows were described in the Preliminary Evaluation of Bridge Pier Hydraulics Report and reviewed in Sub-section 4.4.3 herein, where estimates of currents generated by extreme westerly winds under existing conditions were presented (Table 4.15). Accompanying Table 5.3 contains corresponding figures for the Project condition in which all velocities have been reduced to 82 percent of existing condition values.

This very simple method of predicting project velocities is predicated on the following assumption:

> Under existing conditions the total cross-section of the causeway channels is sufficiently large relative to the cross-section of the Bay to ensure that the flow resistance of the channels is but a small fraction of the total resistance to motion (inertial and frictional) of the whole Bay under the wind set-down condition assumed.
>
> Therefore, the large-scale flow pattern associated with the development of the assumed wind set-down condition, depends mainly on the Bay as a whole. Hence, it follows that the total discharge through

the causeway channels will not be substantially altered by a 22% increase in the cross-sectional area of the channels.

TABLE 5.3

Project Conditions

Estimated Extreme Eastgoing Causeway Channel Velocities Due to Wind Effects and Runoff

Frequency (per year)	Velocity (m/s) Mean	Velocity (m/s) Maximum
1/10	1.0	1.4
1/20	1.1	1.6
1/50	1.3	1.8
1/100	1.4	2.1

Note:
- Mean velocities are averages over the assumed 9-hour period required to develop a maximum set-down.
- Maximum velocities are peak values achieved during the 9-hour period.
- Westgoing currents would be lower because runoff currents oppose the wind effect.

It should be added that the foregoing assumption is probably only valid or approximately valid for the extreme condition considered. This assumed simultaneous undirectional flow along the whole length of the Bay, both west and east of the causeway, can be related to the fundamental longitudinal quarter wave length (= Bay length) mode of oscillation. For other wind induced reversing flows with progressively shorter wave lengths, the channel enlargement could result in increasing discharges and mass exchange flow. Hence, it seems probable that, while the extremal velocities will be somewhat reduced under Project conditions, the average or normal discharges and hence the overall mass exchange flow between the two sides of the causeway will not be significantly different than under existing conditions. However, it is not possible to quantify the increase at this time.

5.2.3.4 Typical Bay Currents - Away from the causeway channels and their areas of influence current patterns will not be affected by the enlargement of the overflow channel to form the new navigation channel.

5.2.4 Water Quality Analysis

The baseline condition of the water in the Bay as a whole and in the vicinity of the present causeway were presented in Section 4.5.4.

The baseline water quality data supports the mass water movement proposition that mixing (exchange) by currents, seiche flow or other modes does occur from one side of the existing causeway to the other.

Therefore, it follows that enlarging the causeway opening will allow for some increased exchange and mass water transfer from one side of the causeway to the other. However, the result will be no significant difference which will have no significant effect.

5.2.5 Fish

No adverse effects on fish migration and spawning are expected from the proposed alteration of the causeway and construction of the high-level bridge. No critical migration pathways will be blocked and migrating fish will easily be able to find their way through either of the two openings that will exist in the causeway.

5.3 PROJECT DATA CONDITIONS

5.3.1 Boating and Recreation

Information on boat traffic and boat generated waves is presented in Section 4.4.1. Implementation of the Project will not alter the characteristics or the frequency of occurrence of such waves and, hence, all of the information given is applicable.

The principal effect of the Project on boat waves is to transfer a traffic volume of larger vessels, currently about 5,000 vessel passages per year, from the existing navigation channel near the south shore of the Bay, to the proposed new navigation channel at the existing overflow channel. In actual fact, however, upon completion of the Project, a substantial number of smaller local boats will continue to use the existing (old) navigation channel, as a matter of convenience.

Boat generated waves can be larger than wind generated waves at distances of about 100 feet from the sailing line, but they are relatively unimportant at distances of 500 feet or more. Up to the time of writing, the exact locations and orientations of the channel in the approaches to the navigation span have not been given. However, examination of the layout indicates that the channel will be less than 500 feet from the Belleville City Water Intake.

5.3.2 Shoreline

The Great Lakes Shore Damage Survey (1973) indicated that no erosion damage took place along the north shore adjacent to the causeway and only moderate inundation damage recorded around Belleville Harbour. The south shore experienced only moderate erosion damage.

There are no highly dynamic areas which may be subject to successive periods of erosion and accretion along either of the adjoining shorelines. As the wind climate under Project conditions will be almost identical to that under existing conditions, it is not anticipated that other than the immediate shoreline will be affected by the Project.

It will, of course, be necessary to re-armour the exposed end of the excavated causeway. This should be done with some of the armouring material initially removed, and it should be placed in the same fashion as the existing, adjoining armour to provide a proper aesthetic effect.

In addition, the Project bridge approaches will require armouring, designed for the 100 year flood level plus the 30 year extreme wave condition, or the 100 year flood level plus the extreme boat generated wave, as indicated on Enclosure A-15.

5.3.3 Sedimentation

The depth of the existing overflow channel is constrained by the natural and artificial armouring of the channel bottom. The enlargement required to accommodate the proposed navigation channel will result in the exposure of underlying sandy gravel which will be subject to scour. The procedure for estimation of this scour is similar to that described in the Preliminary Evaluation of Bridge Pier Hydraulics, except that it is now known that the overflow channel must be widened as well as deepened (only deepening was assumed in the Report mentioned).

The estimation of scour is carried out in four steps:

- General scour in the enlarged overflow channel.

- General scour in the vicinity of the proposed bridge adjacent to both the proposed and existing navigation channels.

- Local scour adjacent to piers of the proposed and existing navigation channels.

- Local scour at the end of the excavated section of causeway.

5.3.3.1 General Scour Due to Overflow Channel Enlargement

General scour of the proposed navigation channel where it cuts through the causeway has been estimated using the trial and error competent velocity method of Neill (1973) under the following conditions:

(a) The maximum discharge through both causeway channels is about 3,000m^3/s (106,000ft^3/s).

(b) The existing navigation channel at the present swing bridge will be unaffected by the changed flow condition.

(c) The bottom of the enlarged overflow channel consists of non-cohesive sandy gravel with an effective grain size of 5 mm and there is insufficient larger material to naturally armour the bottom.

On this basis the average depth in the proposed navigation channel would be increased to about 6m referred to mean lake level or about 5.5m below datum. Since the average depth of the proposed cross-section is 4.6m, this represents an average scour depth of 0.9m. Scour is not usually uniform and maximum general scour will usually be 50 percent to 100 percent greater than average value. Therefore, if the maximum scour is 1.8m and if, furthermore, it occurs in the portion of cross-section dredged to 4.6m, then the maximum general scour could locally increase the depth in the proposed navigation channel to about 6.4m below datum.

The 1/100 per year maximum velocity would be reduced from 2.1m/s to about 1.9m/s as a result of the channel enlargement due to general scour. Since this maximum velocity only occurs in the case of eastgoing flow, it follows that there will be very little scour immediately west of the causeway and that the scour, though diminishing, will extend eastwards under the proposed new bridge.

However, this scour depth will be reduced in proportion to the degree which the scouring of the in-situ material becomes self-armouring.

5.3.3.2 General Scour Under Proposed Bridge Near the New Navigation Channel - After emerging from the constructed section of the channel at the causeway, the eastgoing stream of water will expand laterally, entrain water from each side and decelerate. Therefore, the average velocity encountered by a pier of the proposed new bridge would be somewhat less than the velocity within the overflow channel. A possible consequence of the reduced velocity could be that the depth of general scour, between the piers of the bridge, would be somewhat less than that estimated for the overflow channel. On the other hand, the extent of the velocity and scour reduction will depend on the proximity of the new piers to the causeway channel and their location relative to edges of the emerging stream of water.

For the purposes of this estimate, velocity and scour conditions 30m downstream of the causeway will be considered. At this distance the edges of the emerging stream would have begun to decelerate, but the centre velocity would be very little different from its value in the causeway construction.

With the proposed arrangement, the piers lie near the edges of the emerging stream, about 30 to 35m to the east of the causeway. At this distance piers 6 and 7 would lie in the zones of turbulent diffusion at either side of the main flow, but the centre core of the flow would not be significantly decelerated. The zone of general scour may, therefore, be expected to extend under the proposed bridge between piers 6 and 7, though depth of scour would be reduced at this distance. It seems unlikely that any scour will occur more than about 100m east of the causeway, or about 50m east of the bridge.

On this basis the volume of bottom material displaced by general scour is unlikely to exceed 5,000m^3, consisting entirely of sandy gravel. This volume will be displaced progressively as higher velocities occur. The scour hole will not be refilled with similar material since there is no upstream supply. As in the case of the bottom of the existing overflow channel, bottom velocities will frequently be too high to permit significant deposition of mud from the surrounding bay bottom in the scoured area.

5.3.3.3 Local Scour Adjacent to Piers Near the Causeway Channels - Only piers 1, 2, 6 and 7 opposite the existing and proposed future navigation channels will be subject to local scour. Other piers will be sheltered by the remaining portion of the causeway. Local bridge pier scour depends on the size and shape of the bridge pier, and, somewhat surprisingly, is usually assumed to be independent of both the grain size of the bottom material and of the velocity of flow. Neill (1973) gives a number of typical bridge pier geometries and the corresponding allowances for local scour. Unfortunately, none of these examples correspond to the configuration proposed in this case.

For a single cylindrical pier the recommended scour allowance is 1.5 times the pier diameter. For a cluster of closely spaced cylindrical caissons the allowance must be greater than for a cylinder of the same diameter as a single caisson, but less than the allowance for a solid pier of the same overall dimensions as the cluster of caissons. For a solid pier, tapered towards the top at 20° or more, the scour allowance is equal to the width of the pier.

However, the upward tapering pier might not be a suitable analogy, because the combination of the cluster of piles topped by a relatively massive concrete platform, only a few meters above the bed, might act in a manner similar to a pier which is wider at the top than at the bottom, for which the recommended scour allowance is twice the pier width.

Application of the three geometries described would lead to local scour allowances of about 1.80m, 6, and 12m. In the absence of more detailed information, a local scour allowance of 6m will be assumed. This local scour depth could not occur at pier 1 due to proximity of bedrock, but might conceivably occur at piers 2, 6 and 7.

The volume of bottom sediment (sandy gravel) displaced by local scour is unlikely to exceed 200m^3 per pier of 600m^3 in total. It is probable that cavities due to local scour will be partly filled with mud during periods of low velocity and scoured when higher velocities occur in spring and fall.

5.3.3.4 Local Scour at the End of the Excavated Causeway

Some local scour is possible at the excavated end of the causeway on the south side of the proposed navigation channel. This can be prevented by extending the slope protection beyond the toe of the slope.

5.3.4 Water Quality

The testing results of both Project Quinte and those carried out for this Study indicate that the water quality is similar on both sides of the causeway. Hence, the future water quality conditions should be such that they remain equal on both sides and for the most part, the same in the entire upper Bay. There is no evidence of unique "pooling" or isolation of a "pocket" of water in, say, Belleville Harbour at present, that will be disturbed by opening up the causeway and allowing freer mass water transport. Since the chemical parameters show relative homogeneity throughout the upper Bay, this condition should prevail and there should be no long-term adverse effects.

5.3.5 Fish

The proposed Project will have no adverse effects on fish migration and spawning as migrating fish will easily be able to find their way through either of the two openings in the causeway, and there are no known spawning grounds within the Project area.

5.4 PROJECT EVALUATION

5.4.1 Sedimentation, Shoreline, Boating and Recreation

The evaluation of Project conditions is arranged according to affected areas rather than according to phenomena. The following paragraphs deal first with any local changes associated with each part of the proposed bridge and secondly with any changes which might be more widely felt. In dealing with the local effects the account proceeds northwards from the south shore, dealing with effects associated with both the proposed bridge and the remaining parts of the causeway.

5.4.1.1 The South Shore and South Abutment - The south shore is close to the existing navigation channel and hence subject to boat waves. With the proposed new bridge, about 5,000 vessel passages (all the larger vessels) will use the new navigation channel between piers 6 and 7. This change will reduce any existing tendency for boat waves to cause erosion along the south shore and reduce any localized concentrations of pollutants associated with boating. The reduction in boat-generated waves may be offset by slightly increased wind-wave action along the shore, resulting from the removal of the piers of the old bridge, which will permit slightly more wave energy to penetrate the old navigation channel when waves approach obliquely from northeasterly and northwesterly directions. However, this effect is too small and too localized to warrant serious consideration. Overall, it appears that the Project will have a small beneficial impact on the south shore.

5.4.1.2 Former Navigation Channel and Piers 1 and 2 - The former navigation channel (still existing) will convey somewhat lower volumes of water than under existing conditions due to the re-distribution of flow resulting from the enlargement of the overflow channel. However, peak velocities will still be high enough to cause local scour at Pier No. 2. This could result in displacement of up to 200m^3 of sediment, mostly sandy gravel. Pier No. 1 is expected to be relatively immune to local scour due to proximity of bedrock.

The bed of the former channel area close to the causeway and beneath the proposed bridge will generally be kept swept clean of the soft organic mud as it is at present. It is possible that some mud will temporarily fill the scour holes near Pier No. 2 during periods of relatively low flow. This will be removed at the next occurrence of a high flow rate and will not be environmentally significant because it will occur only when overall levels of turbidity in the Bay are already very high from other unrelated causes.

The re-distribution of flow might result in a reduction in the mass exchange flow through the former navigation channel, which could have a slight effect on mixing in the area. However, since there are apparently neither localized pollution sources, nor sensitive areas, such an effect, if it should occur, would not cause any impact. In any case, the overall effect will be compensated by an increase in the exchange through the other causeway channel.

The removal of the piers of the old bridge should improve clearance of ice, a positive impact.

5.4.1.3 Piers 3 and 5 - Piers 3 and 5 are located close to the east side of the section of causeway between the two channels which, therefore, becomes an island after removal of the existing bridges.

No Project effects are expected in this area, since the piers are sheltered from the effects of currents, waves and ice by the causeway.

5.4.1.4 The New Navigation Channel and Piers 6 and 7 - In this area the existing overflow channel through the causeway will be widened to 100m and deepened to 4.6m (15 ft) depth over a width of at least 45.72m towards the south side of the widened channel to form the new navigation channel. Piers 6 and 7 of the proposed bridge located 30 to 35 meters east of the causeway, will be subject to the action of waves, water and ice flowing through the causeway channel.

The enlargement of the causeway channel will induce a substantial increase in the proportion of total discharge, conveyed by the channel. This change is accompanied by a less-than-proportional reduction in velocity. Scour is expected because the channel enlargement will result in the removal of an erosion resistant layer of rubble and cobbles from the existing bottom. The scouring will occur progressively under extreme flow conditions and following each increment of scour the bottom will be stable until a more extreme flow condition is experienced, leading to additional scouring. In all, general scour could result in the displacement of about 5,000m^3 of sandy gravel and a maximum depth locally as much as 2m greater than the dredged depth of 4.6m. Under normal conditions the bottom of the channel will be swept clear of the soft organic mud which covers the Bay bottom elsewhere. In addition, local scour is expected in the vicinity of Piers 6 and 7. Up to $400m^3$ of sandy gravel could eventually be displaced in this manner. The scour holes at the piers might become partly filled with organic mud during periods of low flow, but this would be removed by high flows when the general level of turbidity would be high.

Material displaced by general and local scour will be deposited in surrounding areas where the flow velocity is lower. However, a significant proportion of the total will be deposited in the navigation channel about 100 to 300 meters east of the causeway.

It might be possible to predict the distribution of scour and to perform sufficient overdredging initially to eliminate the need for subsequent maintenance. This might be the most economical solution or, alternatively, when maintenance dredging is required, at that time carry out a historic analysis and develop a program of overdredging.

The erosion and re-deposition of sandy gravel will make a negligible contribution to water quality impairment for two reasons: first, because trace chemicals are not usually associated with coarse grained cohesionless materials and, second, because scouring of the sandy gravel will only take place at times when the prevailing levels of turbidity are very high and when most of that turbidity will be caused by suspended organic mud, which will carry much higher concentrations of trace chemicals.

The increased discharge due to the channel enlargement will improve local mixing. This is not necessarily a beneficial impact. However, it will have a smaller overall effect because the increased mixing caused by the new navigation channel is largely offset by reduced mixing at the southern channel.

The enlargement of the overflow channel and removal of the existing bridge pier may have a noticeable effect in accelerating the clearance of ice from the west side of the causeway in the latter part of the winter. This is probably a positive or beneficial impact.

Alteration of the patterns of wave action close to the channel and the adjacent parts of the proposed bridge is not significant.

5.4.1.5 Piers 8, 9, 10 and Northern Abutment - The situation of Piers 8, 9 and 10, and the northern abutment is similar to that of Piers 3 to 5 and here again no significant Project induced effects or impacts are likely.

5.4.1.6 The North Shore Near the Northern Abutment - The north shore of the Bay of Quinte in the vicinity of the proposed bridge will not be subject to any significant Project-induced effects or impacts for the following reasons:

(a) Wind wave conditions are unaltered because the shore is sheltered from the effect of increased wave energy propagated through the enlarged overflow channel.

(b) The shore is more than 500 feet from the probable alignment of the new navigation channel and, therefore, not subject to a significant increase in total wave attack due to boat induced waves.

5.4.1.7 The Belleville City Water Intakes - The Intakes (there are two) are located about 2,000 feet west of the causeway, more or less in line with the proposed new navigation channel.

The detailed analysis of wind generated waves showed that waves generated east of the causeway by winds in the appropriate direction can be propagated through the enlarged overflow channel, causing a small increase in wave height in the intake area. This increase is not considered significant for the following reasons:

(a) Wind blows in the critical easterly direction for only ten percent of the time.

(b) The extreme condition actually considered did not produce waves sufficiently large to produce significant bottom velocities, or significant disturbance of bottom sediment.

(c) Westerly winds generate larger waves, unaffected by the Project.

Boat generated waves on the other hand might pose a problem, depending on the distance between the Intake and the navigation channel alignment. Since the alignment of the channel has not yet been defined, it is not possible at this time to do more than note that the channel should certainly be located more than 100 feet from the Intake and, preferably, several hundred feet. Localized pollution due to boat operation, for example oil and gasoline spills and leaks, etc., provide another reason for maximizing the distance between the navigation channel and the Intake.

Apart from the potential for adverse impact due to boat waves and localized pollution, the Intakes will also feel the effects, if any, of the local increased mixing induced by the enlargement of the overflow channel. Since water quality analysis carried out by both Project Quinte and for this Study, confirm that there is no significant difference on either side of the causeway, there should be no negative effects by the increased mixing. However, since the City of Belleville and the Moira River represent the principal local sources of pollution dangers, should any unusual pollutants be contributed by these sources, then under Project conditions there is more of a likelihood that they might effect water quality at the City Intakes.

5.4.1.8 Belleville Harbour and the Mouth of the Moira River - If there should be any higher level of pollution east of the north end of the causeway from the Harbour or the Moira River, the increased flow through the enlarged overflow channel will reduce the effect.

Boat wave effects and slightly increased wind wave energy (when the wind blows in exactly the right direction) are quite insignificant and warrant no further comment.

5.4.2 Water Quality

The overall water quality condition in the Bay of Quinte can be described as "eutrophic". The "trend" in the water quality over the past few years is that the quality has remained relatively static.

Because there is not likely to be any isolation of water masses on one side of the causeway compared to the other, one cannot expect any significant change in water quality as a result of the Project. Therefore, the input of nutrients from rivers of the western end of the Bay can be expected to be transferred and generally mixed with waters on the east side of Belleville rather than being isolated on the west side.

Furthermore, the bridge will not alter the rate of eutrophication of the Bay since this is strictly a function of nutrient inflow via streams and rivers running into the Bay of Quinte.

Johnson and Brinkhurst (1971) have demonstrated the existence of three major invertebrate associations in the Bay of Quinte, Prince Edward Bay and the adjacent area of Lake Ontario. They are as follows:

> A chironomid association of the eutrophic inner and middle bays; an association of many species of sphaeriids, oligochaetes, chironomids, and crustaceans in the mesotrophic lower Bay of Quinte and Prince Edward Bay; and a cold-stenotherm association of the oligotrophic deep basin of Lake Ontario. Species diversity in Lake Ontario associations apparently was related to degree of eutrophy and to water temperature.

Since there will continue to be a free exchange of water through the causeway, the pattern for benthic biota will follow that for water quality, i.e. the impact of the overall Project on the benthic biota of the Bay of Quinte will be negligible. Only in local backwaters within the immediate vicinity of the new bridge and causeway might there be any shift in species diversity or association.

Since the overall state of eutrophy of the water and the temperature of the water will not be changed by the Project, the overall benthic community will not be changed.

5.5 ENVIRONMENTAL ASSESSMENT

The completion of the Project as contemplated will result in:

(a) The construction of a bridge approximately 30 meters east of the existing causeway.

(b) The removal of approximately 30 meters of causeway south of the fixed bridge where the present overflow channel exists.

(c) The dredging of a new navigation channel at the present overflow channel location.

(d) New embankments on either side of the Bay to accommodate the approaches for the bridge.

The Ministry of Environment's concern, therefore, is whether or not the works contemplated would result in rougher wave conditions, increased siltation of the boating channels and degradation of the water quality.

5.5.1 Wave Conditions

The opening of the existing overflow channel from 53 to almost 90 meters will result in an insignificant increase in wave height and wave energy at the water intake and at the mouth of the Moira River. This occurs for only 0.01 percent of the time during April, representing a return period of two to eight years.

Under these extreme conditions, the waves at the water intake and the mouth of the Moira River, for existing conditions, are 1.1 and 1.8 feet respectively. Under Project conditions, the heights are computed to be 1.3 feet at the intake and 1.9 feet at the mouth of the Moira River.

However, the largest waves at the water intake, which are caused by westerly winds, approach three feet height, while the largest waves at the mouth of the Moira River, under easterly and southeasterly winds, approach two feet in height.

Thus, the Project will have no significant effect or negative impact on the wave climate of the referenced areas.

5.5.2 Boating and Recreation

There will be no significant change in the environmental conditions affecting boating and recreation by the Project. The new navigation channel represents an improvement on the existing situation, with a larger, deeper and better aligned channel, which will be able to accommodate a greater number of vessels.

The City Water Intake is so located that the new navigation channel would be positioned approximately 100 meters (328 feet) to the south. Boat generated waves can be larger than wind generated waves at distances up to 30 meters (100 feet). The effect of these waves, therefore, on the Intake should not be very significant if this location of the navigation channel is maintained. If the channel is oriented closer to the Intake, the effect of boat generated waves could be felt. The occurrence of spills from boats, although rare, could provide harmful pollution effects at the Intake.

After considering all the factors associated with the influence of boating, it is concluded that the negative environmental impact should be insignificant, as long as the new boat channel west of the causeway is located a minimum of 100 feet, and preferably 500 feet, distance from the City Water Intake.

5.5.3 Mass Water Movements

It has been established that there will be an increase of flow through the existing overflow channel after widening and dredging to make it the new navigation channel. Flow patterns will be altered and velocities should not be significantly changed, except under conditions of extreme westerly winds when the velocities would be reduced to 82 percent of existing condition values. Wind induced reversing flow creating mass water movement should not be significantly different from the Existing Conditions.

5.5.4 Sedimentation and Scour

The removal of a section of the causeway and the dredging necessary to create the new navigation channel will result in the removal of some of existing armour of coarse erosion resistant material now present, and the exposure of the more easily erodible sand and gravel layers.

There should be very little scour occurring west of the causeway because the maximum velocity occurs with the eastgoing flow. The general scour will diminish downstream from the causeway. The bottom velocities should be sufficiently high to prevent the deposition of mud from the surrounding bay bottom.

Local scour will occur at the piers adjacent to the navigation channels. Local bridge pier scour depends on the size and shape of the bridge pier, and for the proposed design a local scour allowance of 6 meters is proposed. Pier No. 1 is near bedrock and, therefore, relatively free from scour, but piers 2, 6 and 7 will be subjected to local scour. At the excavated end of the causeway it will be necessary to extend the slope protection beyond the toe of the slope to prevent scour.

The causeway excavation and channel dredging for this Project should produce no harmful environmental affects where sedimentation is concerned, especially west of the causeway. The section of the bay east of the causeway will experience some scour in the navigation channel, but this would be decreased downstream as the velocities decrease and sediments are deposited.

5.5.5 Water Quality

The relatively free exchange of water through the causeway will mean that the completed Project will not cause a significant impact on the overall water quality or benthic species associations in the Bay of Quinte.

There will be some local impact due to construction activities.

Because currents and water movement patterns might change locally (i.e. in regions near the causeway), local benthic community associations might shift perceptably but one would probably have to find a peculiar microenvironment in order to detect such a local change.

Similarly, there will be no negative impact on the water quality at the Belleville Intakes.

SECTION 6
CONSTRUCTION CONDITIONS

SECTION 6 - CONSTRUCTION CONDITIONS

6.1 INTRODUCTION

The Project involves the construction of a new high-level bridge structure and approaches, located approximately 30 meters east of the existing Highway 14 causeway and swing bridge btween Belleville and Rossmore. This construction will be accompanied by the removal of approximately 40 meters of the causeway south of the fixed bridge, together with the necessary dredging to develop the new navigation channel.

The proposed new high-level bridge will provide a vertical navigation clearance of 22.5 meters and a maximum roadway grade of five percent. The superstructure is to be supported by vertical concrete piers founded on concrete pile caps. The substructure of the pile cap will consist of steel tubes of 1,219 mm diameter driven to rock. Inside each tube are three HP310 steel piles, which will be driven into rock or socketed into rock as required. The tubes will then be filled with concrete. The concrete bridge abutments will also be founded on steel piles driven through a granular core.

The preliminary foundation investigation Report by the Ministry of Transportation and Communications, November 1977, indicates that the subsoil conditions vary along the Project length. These can be summarized as:

North Approach (Land)	- fill material (sanitary) over organic clay and sand over clay and till over bedrock
North Approach (Water)	- organic clay over sandy gravel over clay over glacial till
Causeway	- sand and gravel fill with cobbles to boulders
Bay Bottom	- organic clay up to 46 feet deep
South Approach	- sand and gravel overlying bedrock

The Soils Report recommends:

North Approach	- removal of the fill material and organic clay within the land

plan limits of the embankment. No in-water excavation. Back-filling with rock fill from shore to displace any in-water fill or organic material. Armour side slopes.

South Approach - construction of the embankment by the placement of rock fill to one foot above water level. Armour side slopes.

6.2 CONSTRUCTION METHODS

6.2.1 Bridge Piers

The bridge piers consist of a reinforced concrete pile cap, located below water level, supported by end bearing steel tube piles. MTC will allow the Contractor to choose the method of construction, of which two are envisaged:

- steel sheet pile cofferdam, or
- precast concrete "float in place" pile cap.

The cofferdam method would require the complete encasement of the pier or abutment area with steel sheet piling suitably braced and waled to allow partial dewatering for placement of the concrete pile cap. During driving of the sheet piling some minor local scouring of silt will commence in response to the growing obstruction caused by the cofferdam to the prevailing currents. This scouring will be almost negligible at all pier locations except 1, 2, 3, 7 and 8, where their proximity to the currents through the causeway openings will result in greater scour. However, it is unlikely that during this period of construction, the resultant scour would approach the maximum scour projected for the finished pier under extreme conditions.

During excavation of the piles an average volume of about 13 cubic yards of sands, clays and tills will be removed per pile to reach bedrock (some thirty feet below). The excavation of this native, subterranean material will total about 160 cubic yards per bridge pier, and the material will be replaced on the Bay bottom adjacent to the construction.

The "float in place" pile cap method only requires the driving of some temporary "fixing" piles to secure the precast pile cap in place until the permanent steel tube bearing piles have been driven. During the driving of these piles, some minor local scour might occur, although for less than the maximum scour projected for the finished pier under extreme conditions.

Construction of the north embankment will require the prior excavation and off-site disposal of the on-land sanitary fill and organic clays. This will be accomplished by utilizing land-based excavation equipment. No in-water excavation is anticipated. Backfilling with granular material from shore will displace any silt, weak fill and organic material which will form a mud wave at the toe of the embankment. Although this method will cause some disturbance to the in-situ material, and result in some material being placed in suspension, the lack of hydrodynamic activity will limit the influence to a very local area.

Construction of the south embankment will consist of placing granular material over the sand-gravel subsoil.

Although this area is exposed to a higher hydrodynamic level than the north embankment area, the activity should be of such a low level that it should only cause local and temporary influence.

For both embankments, wave protection should be carried to elevation 77.5 meters with uprush protection above.

6.2.2 Causeway Excavation

The excavation of the causeway will first require the removal of the armour stone, retaining sufficient quantities on-site to be used for the protection of the exposed end of the excavated causeway and the embankments.

The material to be excavated from within the causeway consists of sand and gravel with cobbles and boulders. Land-based equipment is both the most economic and best method of excavation. Although the operation will cause some disturbance of bottom silts and suspension of material, the effects will be both temporary and localized.

Enclosure A-16 attached, indicates the half meter contours inferred by the MTC survey. The volume of material termed "excavation" is that quantity of the old causeway which can be removed to navigation depth of 70.3 meters IGLD (12 feet), with side slopes of 3:1 below water and 2:1 above, using land-based equipment. Allowing for 0.3 meters of overdredging (1 foot), the total volume is about 6,500 cubic meters. This volume consists of the armour stone, rubble filter and the sand-gravel causeway core.

Although no samples of these materials have been tested, the materials should have no significant contaminants and should be suitable for use in the new approach construction or placed alongside the existing causeway.

6.2.3 Dredging

The material to be dredged consists of sand and gravel adjacent to the causeway, becoming fine silt and clay in the channel east of the new bridge (Borehole No. 100).

Considering the small quantity of material to be removed, it would not be economical to mobilize a hydraulic dredge with its attendant problems of slurry disposal. A scow-mounted clamshell, discharging onto a scour would be both more efficient and economic.

Although any dredging operation will cause considerable disturbance and suspension of materials, the short period required for dredging and the lack of any significant hydrodynamic activity will result in an insignificant and localized effect. This effect can be minimized by carrying out the dredging during the quieter summer months of July and August.

Without further time-consuming and expensive investigation it is difficult to correlate the relationship between sounding depths and probing depths. However, it can be assured that the sounding lead would not penetrate to the limit of the probing, and it would seem reasonable to assume a sounding lead penetration of 0.3 to 0.5 meters in most areas.

Based on the results of the sounding survey, and assuming a dredged depth to 70.3 meters (IGLD), with an allowance of 0.3 meters for overdredging, the total dredging volume is 4,500 cubic meters. A substantial portion of this volume (the bay mud) could never be removed by mechanical means.

It is recommended that the "firm" material, estimated to be in the order of 2,500 to 3,500 cubic meters, be disposed of underwater, along the east side of the existing causeway.

6.3 ANALYSIS

Considering the small volume and localized character of the dredging and the large area of the Bay, there appears to be negligible risk of significant impact due to either dredging or sub-aqueous disposal. Selection of a dredge material disposal site would pose no special problem if it were carried out within a restricted area, for example, adjacent to the toe of the slope of the causeway, or in the deeper areas adjacent to the existing navigation channel.

The excavated material from the causeway will be predominantly sand and gravel and cobbles, or large armour stones from the toe of the fill and on the surface of the overflow channel. It has been suggested by MOE that the surplus causeway excavation material be placed to form a low berm between the existing causeway and Piers No. 5 and 6. This would then form an embayment for the dredgeate material and would somewhat restrict its further transport into the new navigation channel.

With regard to the on-land excavation of existing sanitary landfill and organic clay, it is recommended that disposal sites be negotiated with the municipal authorities and be located inland away from influence of the Bay.

During earth-moving and construction activities, some local excessive turbidity might be created, followed by a fallout of silt which could blanket any very nearby spawning beds, but their effect would not be any greater than that produced by wave action during one of the frequent "rough days" of storms in the area. No critical migration pathways will be blocked and migrating fish will easily be able to find their way through either of the two openings that will exist in the causeway.

6.4 ENVIRONMENTAL ASSESSMENT

Assuming the recommended methods of construction are adopted, the impact of construction on water quality will be temporary. The major impact will result from the disturbing of sediments and the moving of earth which will result in an increase in suspended solids (and perhaps dissolved solids) in the region during the construction operations. When the construction ceases, the local turbidity of the water will (after a lag period of several days) disappear. This turbidity in any case is not likely to be greater in extent or cause a greater impact than the disturbance caused by a moderate storm.

Since construction activities will be confined to the eastern section of the Bay, any local disturbance of the water quality would have very little, if any, effect at the Intake.

Any trenching or mounding on the floor of the Bay resulting from construction operations should also result in minimal long-term impact on the natural environment. Equilibrium conditions would be expected to be re-established relatively quickly.

Similarly, perturbation of local benthic communities by direct physical contact of construction equipment, or by "fallout" of suspended solids should be local and temporary in nature. One would expect, again, the local benthic system to be equilibrated within the space of a few months (or a year or so) and for most colonies to become re-established or adjusted to the "new" condition. In any event, the impact on the benthos or on the water quality would not likely be evident.

The methods of construction envisaged should cause no significant removal or transport of sediments causing siltation or severe scouring. Should the contractor elect to use the cofferdam technique, the construction should be programmed so that the cofferdams for Piers No. 1, 2, 3, 7 and 8 are not in place during the spring run-off period of higher flow velocities through the causeway openings.

SECTION 7
TOTAL CAUSEWAY REMOVAL

SECTION 7 - TOTAL CAUSEWAY REMOVAL

It has been suggested that the construction of a high-level bridge provides the opportunity to completely remove the causeway and thereby return the area of the Bay to the condition which naturally existed some sixty years ago.

However, the present physical environmental conditions are significantly different from those existing prior to 1920. Therefore, significant environmental impacts would accompany the process of re-adjustment if the causeway were totally removed.

This Section examines this impact.

7.1 WATER LEVELS

The causeway creates a semi-enclosed basin at the western end of the Bay, which is capable of responding to disturbances due to wind shear or lake level oscillation, by seiching at its own natural periods.

The removal of the causeway would eliminate some of these resonant frequencies and would reduce or eliminate the tendency of the western end of the Bay to oscillate independently of the remainder. This would reduce the amplitude of oscillation in the area of the crossing, slightly lowering maximum water levels and raising minimum water levels. These changes would not in themselves be of any practical importance and would have no negative impact.

7.2 WAVES

Removal of the causeway would increase the exposure of the adjacent water areas, especially the east side of the causeway which includes the Public Wharf at the mouth of the Moira River. This would cause a substantial increase in the frequency of occurrence of wave action and a substantial increase in its severity in the area east of the crossing, including the Public Wharf at the mouth of the Moira River. This would result in a definite negative impact.

7.3 ICE

The ice would become far more mobile if the causeway was removed. Ice would take longer to freeze shore to shore in the early winter and would break up earlier in the spring. For these reasons thicker, larger and stronger ice floes would occur than under present conditions, and would have some negative impact.

7.4 MASS WATER MOVEMENT

Currents and mass water movements at some distance from the causeway location (say one or two kilometers) would be only very slightly affected in magnitude by the causeway removal. On the other hand, periodicity of the wind induced reversing component of flow would be affected in the same way that water level variations are altered.

Close to the existing causeway there would be a very considerable effect on the distribution of currents. The existing concentrations of currents in and near the two causeway channels would disappear, to be replaced by very much lower current velocity, distributed more or less uniformly across the whole width of the Bay, with any remaining velocity variation being dependent mainly on wind conditions. Thus, where the channels are now located there would be a substantial velocity reduction, and where the causeway now blocks easterly and westerly flow, unimpeded flow would occur. The presence of the causeway must have somewhat increased the hydraulic resistance of the Bay to easterly and westerly flow. Therefore, the removal of the causeway would correspondingly increase the mass exchange of water which by itself should not have a negative impact.

7.5 SEDIMENTS

Significant changes in the sediment regime of the Bay in the vicinity of the crossing could be expected to result from the total removal of the causeway.

The soft mud covering most of the Bay bottom would become more widely and more frequently disturbed since the whole area would be fully exposed to wave action due to both westerly and easterly winds, with no shelter from either. The east side of the crossing, including the area of the Public Wharf and the mouth of the Moira River, would be most affected by this change.

Further, the dredged navigation channel would silt with bay mud and would probably require repeated, possibly frequent, dredging. The resultant effect would be a definite negative impact.

7.6 WATER QUALITY

The present level of mass water exchange suffices to produce approximately uniform water quality with respect to dissolved solids under normal conditions. However, the increased activity of the bottom sediments, resulting from removal of the causeway, would occasionally increase the level of total dissolved solids which in turn could cause small changes in the dissolved solids content due to adsorption or release of trace substances to or from the suspended solids. This could result in a negative impact due to an increase in suspended solids and possibly dissolved solids.

7.7 BENTHIC ORGANISMS

The disturbance of the bottom sediments promulgated by the entire removal of the causeway would undoubtedly disturb the benthic community, resulting in changes across the Bay. These changes, however, would be temporary in nature, and would soon be stabilized. However, the long-term general increase in suspended solids would have a localized negative impact.

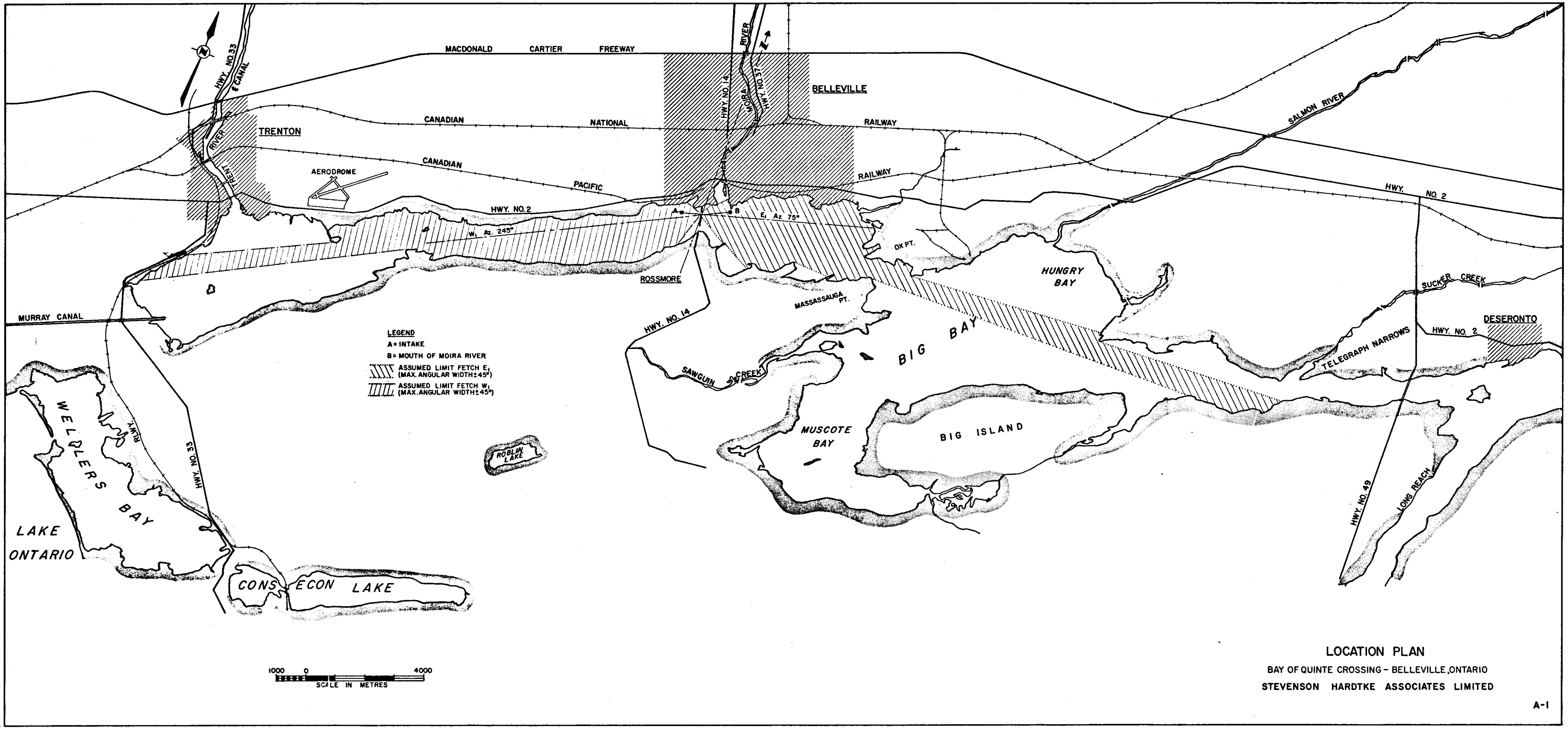
7.8 SHORELINE

The increased frequency of wave attack resulting from elimination of the shelter now provided by the causeway would produce a perceptible increase in the tendency for shore erosion and would result in a negative impact.

This Final Report, which was prepared by C.A. Stevenson, P. Eng., was based on Working Papers prepared by E.A. Ffolkes, P. Eng., and K.L. Philpott, P. Eng. The wave, mass water movement and sediment analysis was carried out by K.L. Philpott, P. Eng., and the water quality analysis by Dr. I. Cappon.

APPENDIX A Enclosures

A-1	Location Plan
A-2	Fetch Directions
A-3	Bathymetry of Study Area
A-4	Bathymetry of Belleville Harbour
A-5	Easterly Wave Diffraction - Existing
A-6	Westerly Wave Diffraction - Existing
A-7	Diffracted and Local Waves
A-8	Locations of Recording Instruments
A-9	Locations of Water & Sediment Samples
A-10	Easterly Flow Pattern - Existing
A-11	Proposed Location of Bridge Piers
A-12	Easterly Wave Diffraction - Project
A-13	Westerly Wave Diffraction - Project
A-14	Easterly Flow Pattern - Project
A-15	Armouring - Approach Embankments

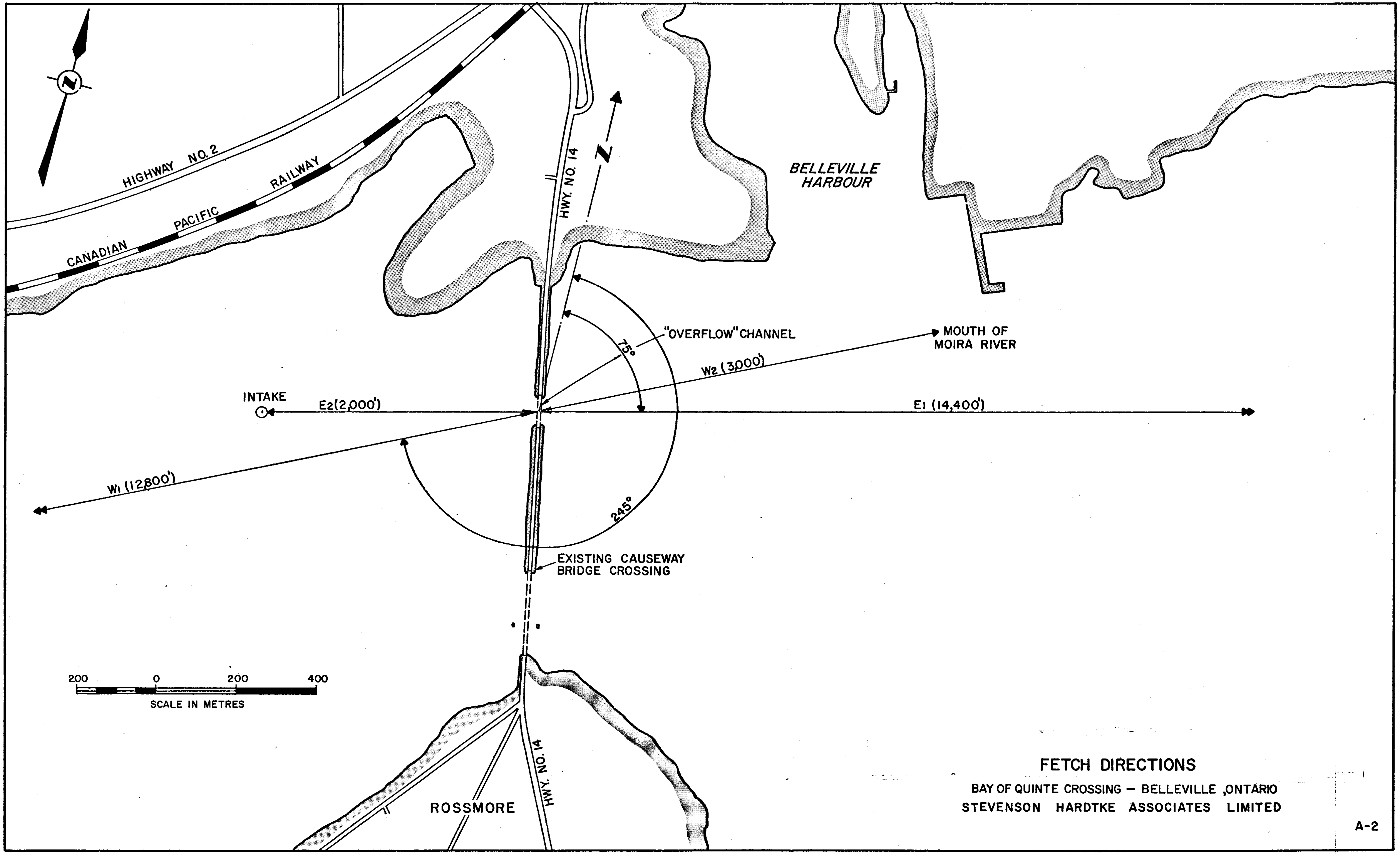


LOCATION PLAN

BAY OF QUINTE CROSSING – BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED

A-1

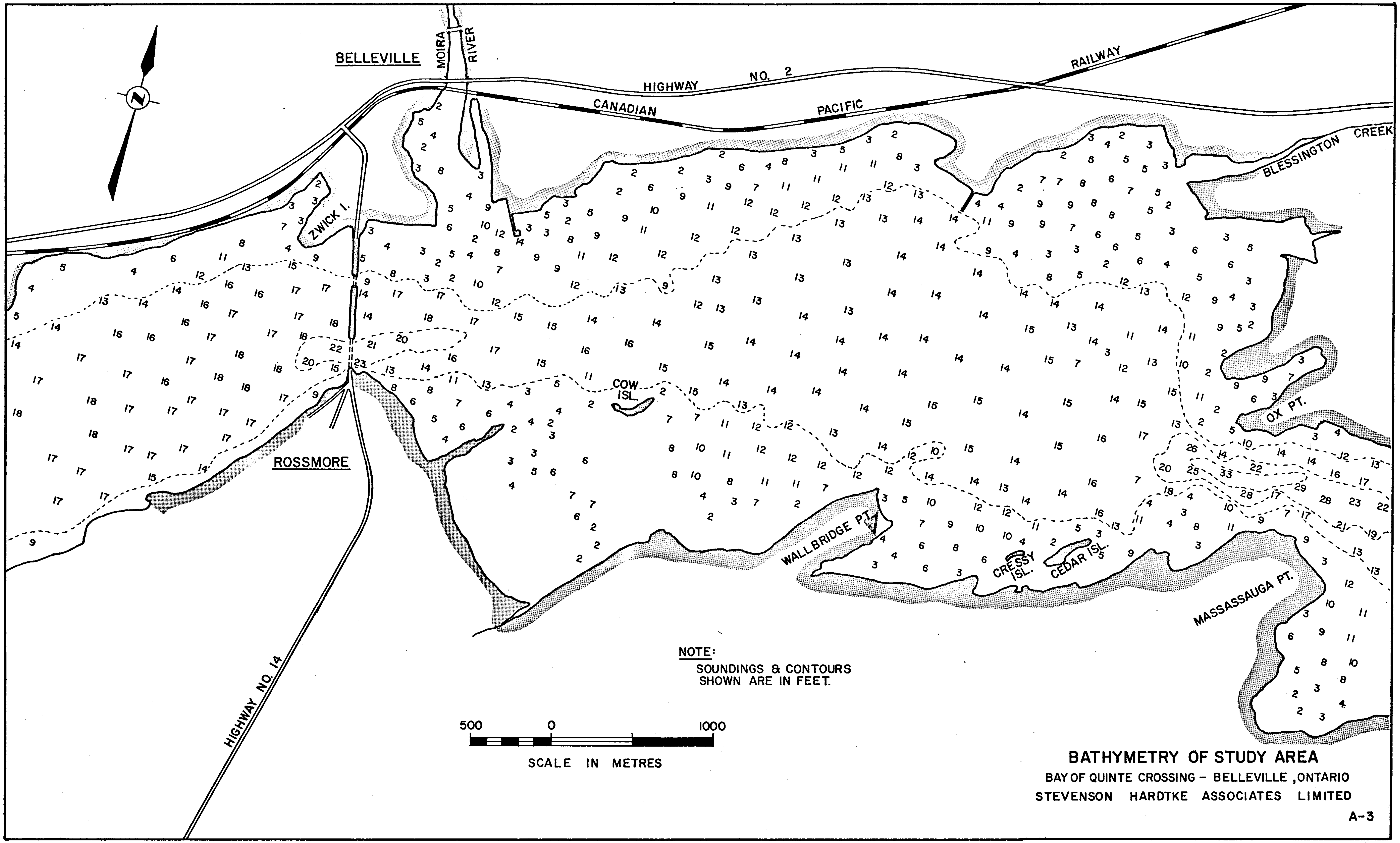


FETCH DIRECTIONS

BAY OF QUINTE CROSSING – BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED

A-2

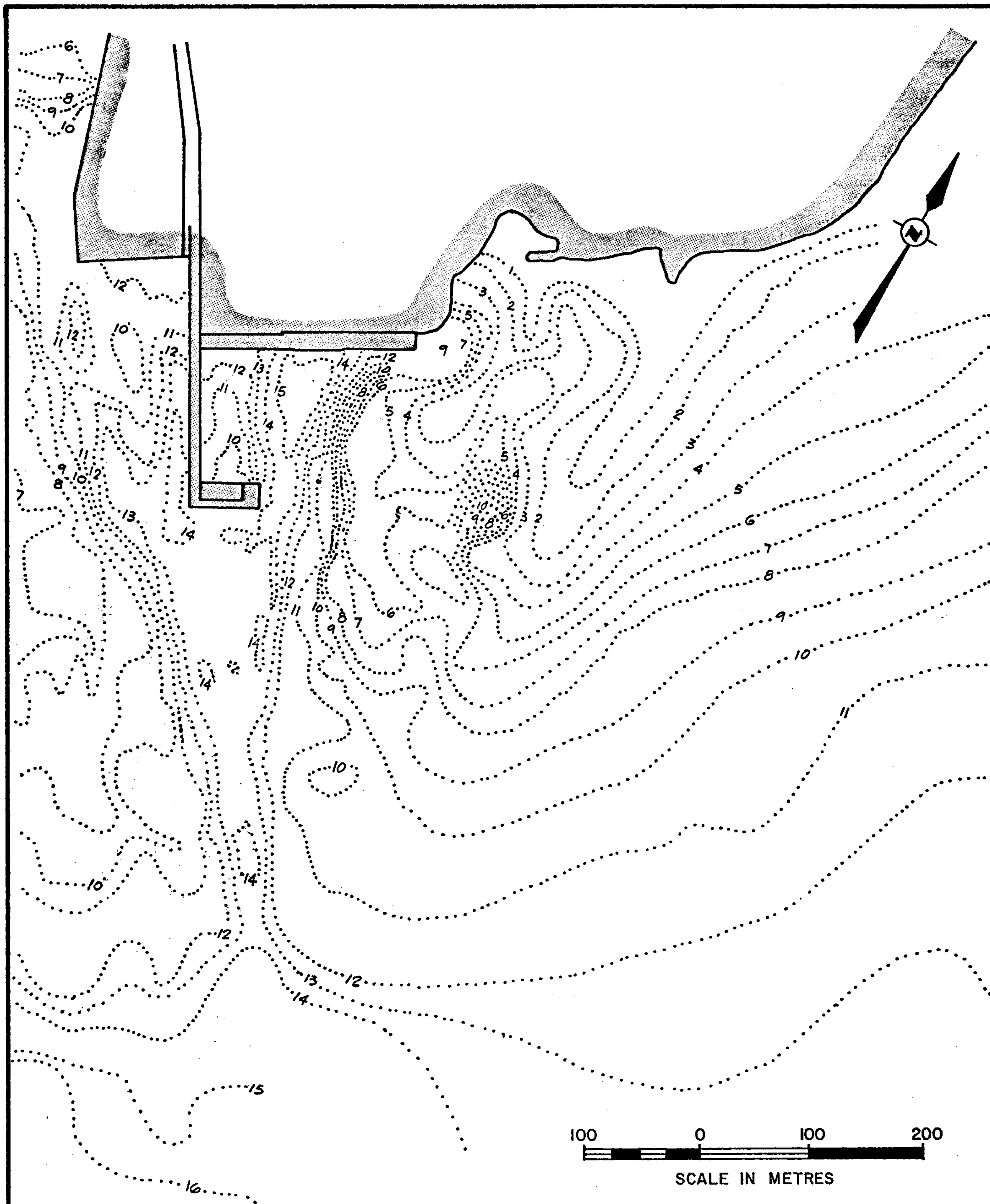


BATHYMETRY OF STUDY AREA

BAY OF QUINTE CROSSING – BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED

A-3



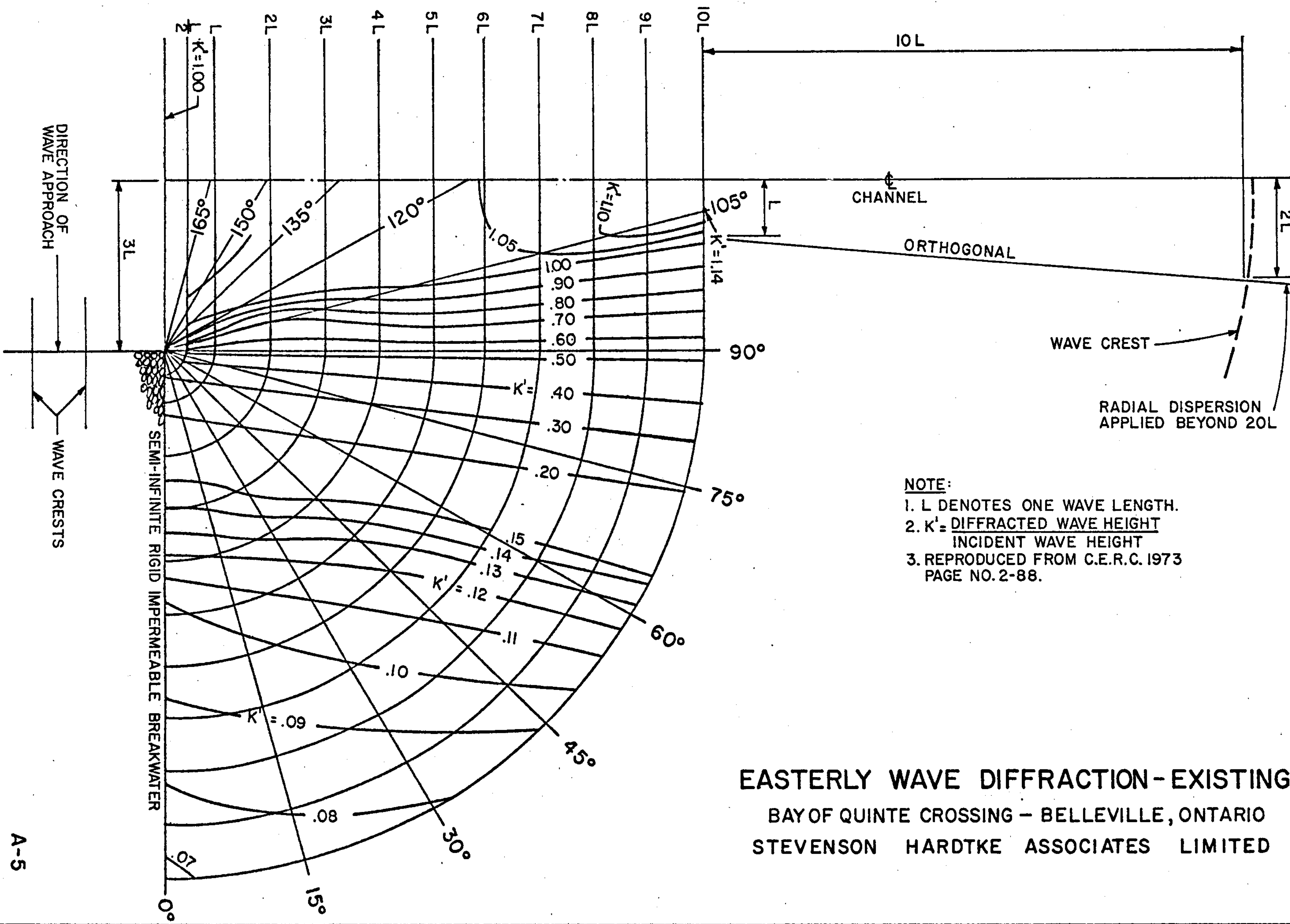
NOTE
CONTOURS SHOWN ARE IN FEET.

BELLEVILLE HARBOUR BATHYMETRY

BAY OF QUINTE CROSSING – BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED

A-4

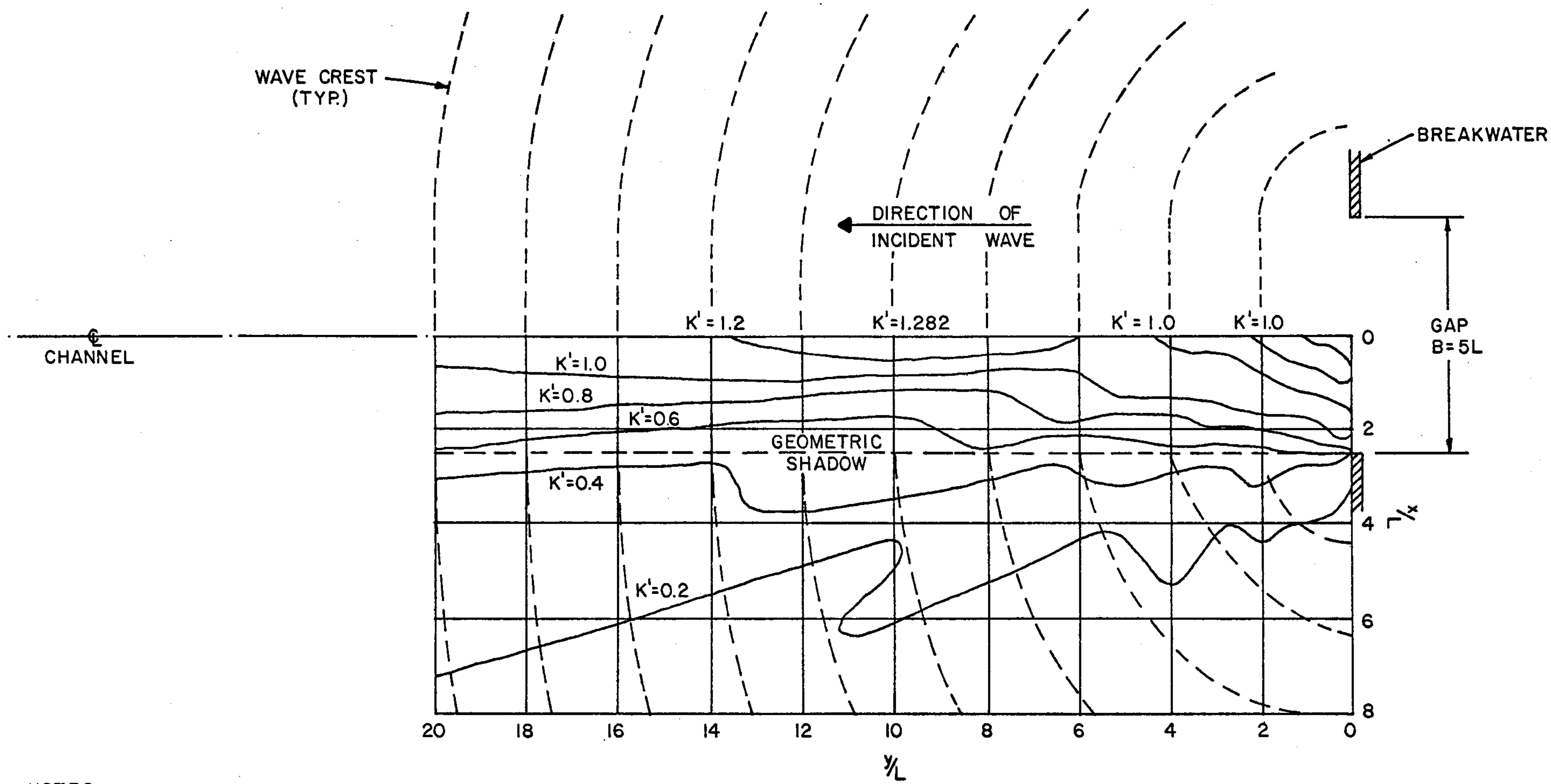


NOTE:
1. L DENOTES ONE WAVE LENGTH.
2. $K' = \frac{\text{DIFFRACTED WAVE HEIGHT}}{\text{INCIDENT WAVE HEIGHT}}$
3. REPRODUCED FROM C.E.R.C. 1973 PAGE NO. 2-88.

EASTERLY WAVE DIFFRACTION - EXISTING

BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED



NOTES:

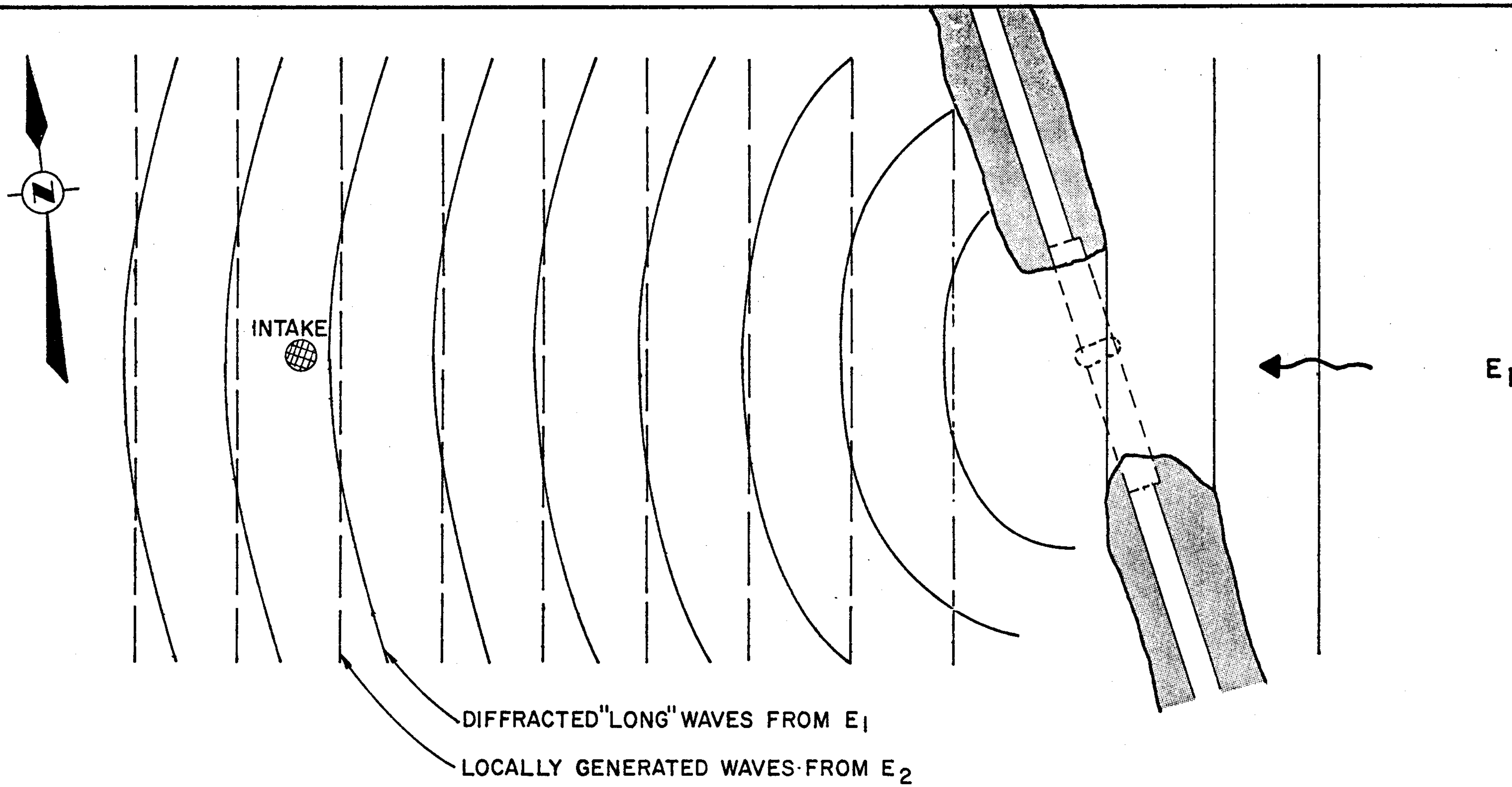
1. L DENOTES ONE WAVE LENGTH.
2. $K' = \frac{\text{DIFFRACTED WAVE HEIGHT}}{\text{INCIDENT WAVE HEIGHT}}$
3. REPRODUCED FROM C.E.R.C. 1973 PAGE NO. 2-104.

WESTERLY WAVE DIFFRACTION - EXISTING

BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED

A-6

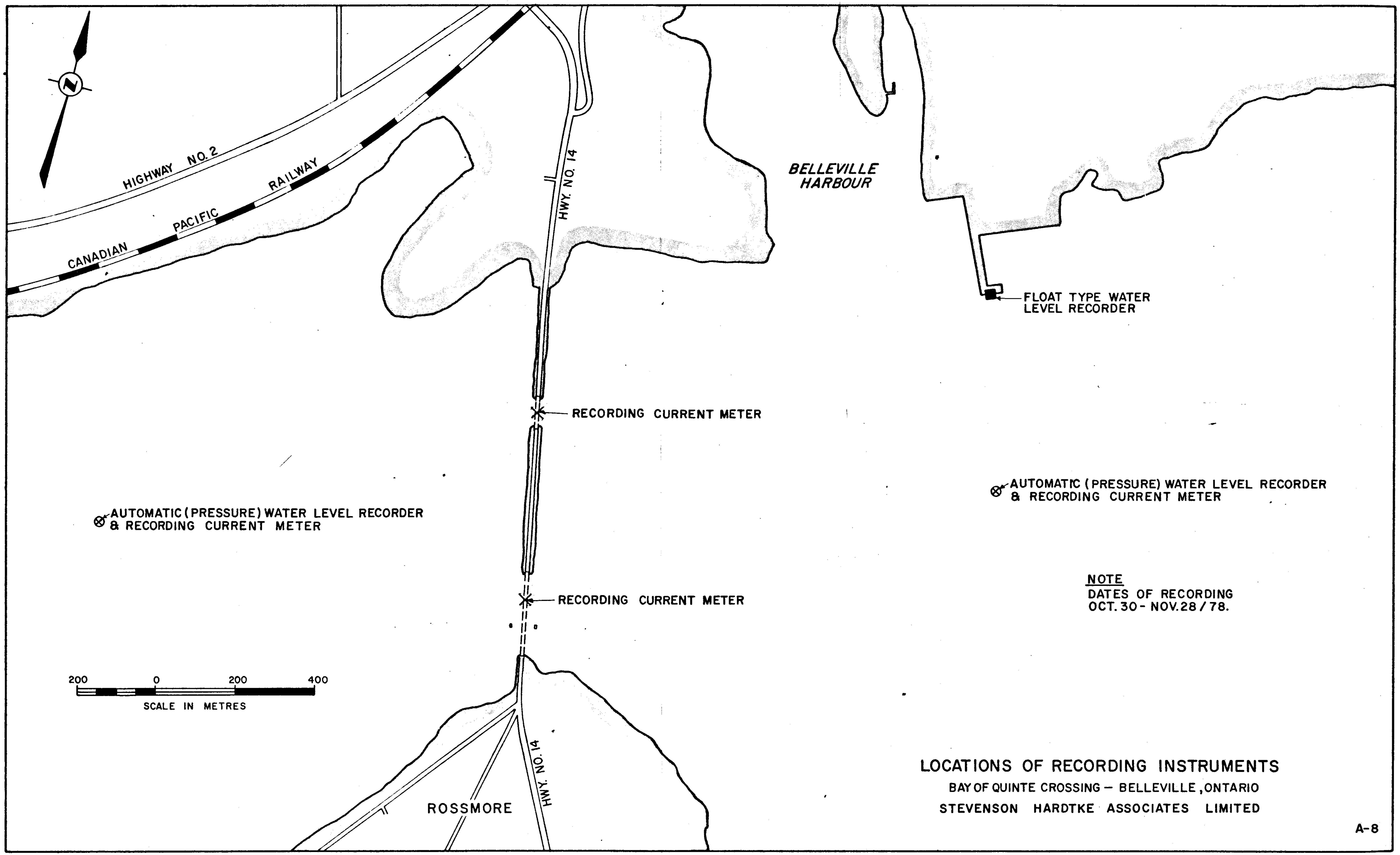


DIFFRACTED & LOCAL WAVES

BAY OF QUINTE CROSSING – BELLEVILLE, ONTARIO

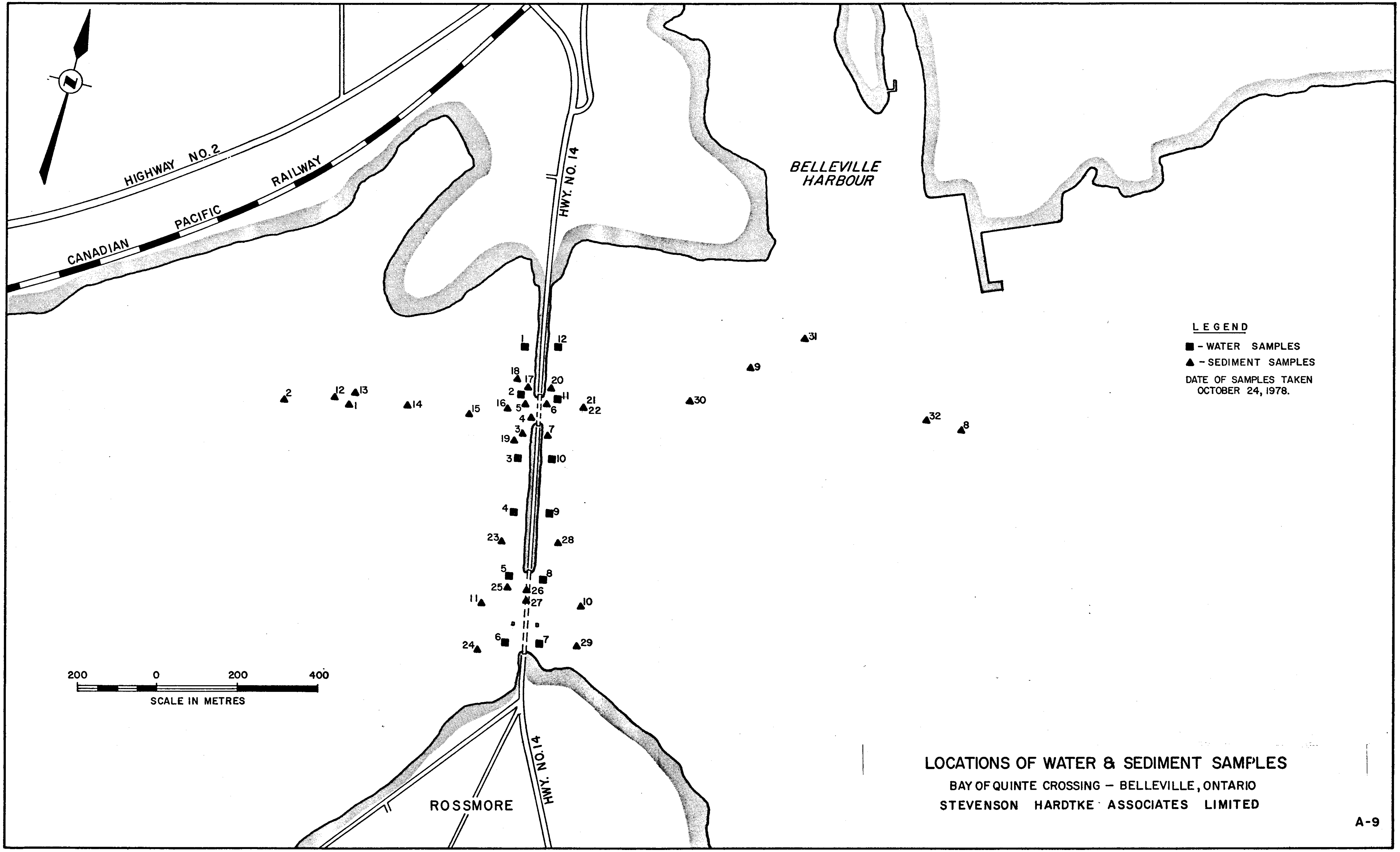
STEVENSON HARDTKE ASSOCIATES LIMITED

A-7



NOTE
DATES OF RECORDING
OCT. 30 - NOV. 28 / 78.

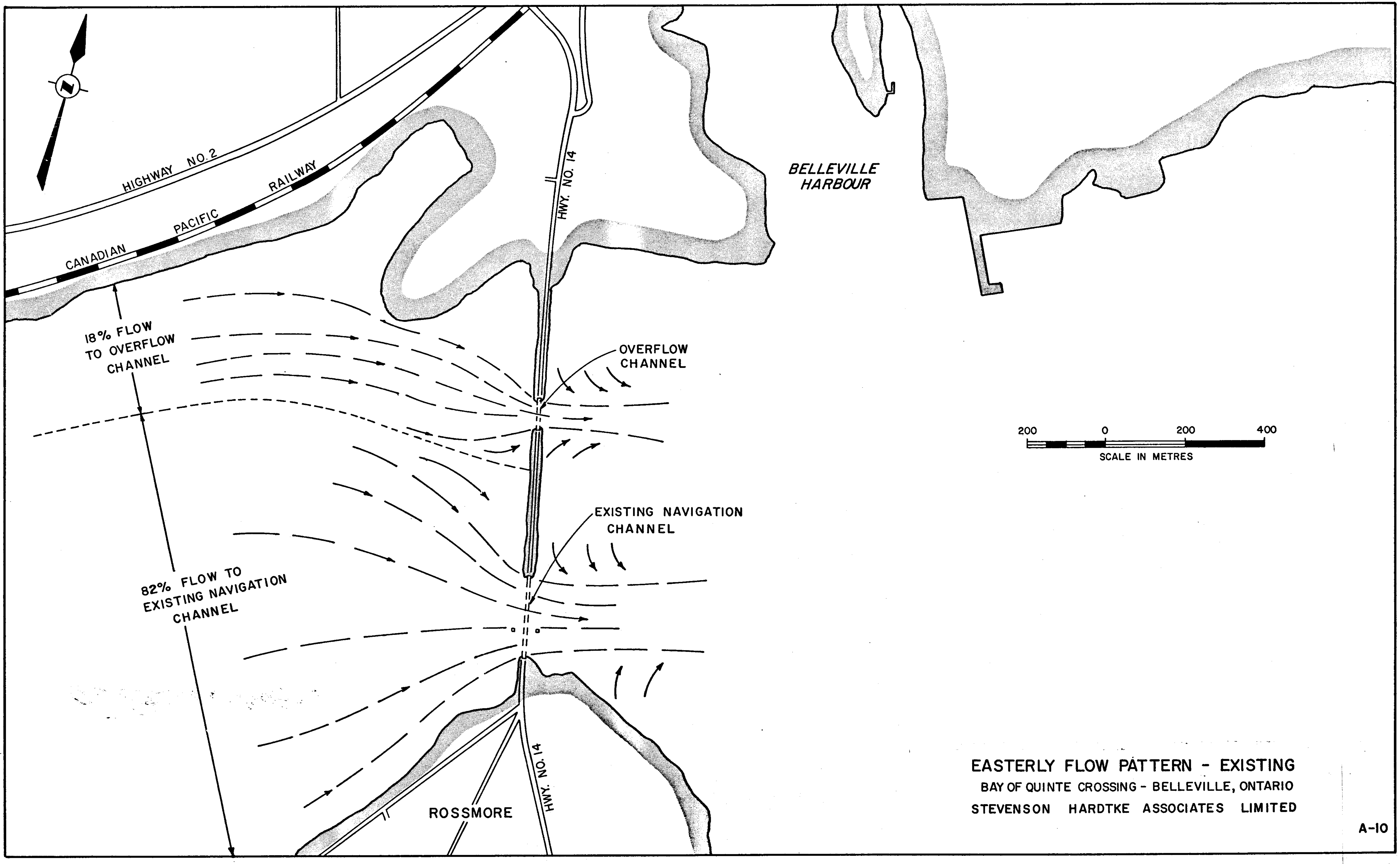
LOCATIONS OF RECORDING INSTRUMENTS
BAY OF QUINTE CROSSING – BELLEVILLE, ONTARIO
STEVENSON HARDTKE ASSOCIATES LIMITED



LOCATIONS OF WATER & SEDIMENT SAMPLES

BAY OF QUINTE CROSSING – BELLEVILLE, ONTARIO

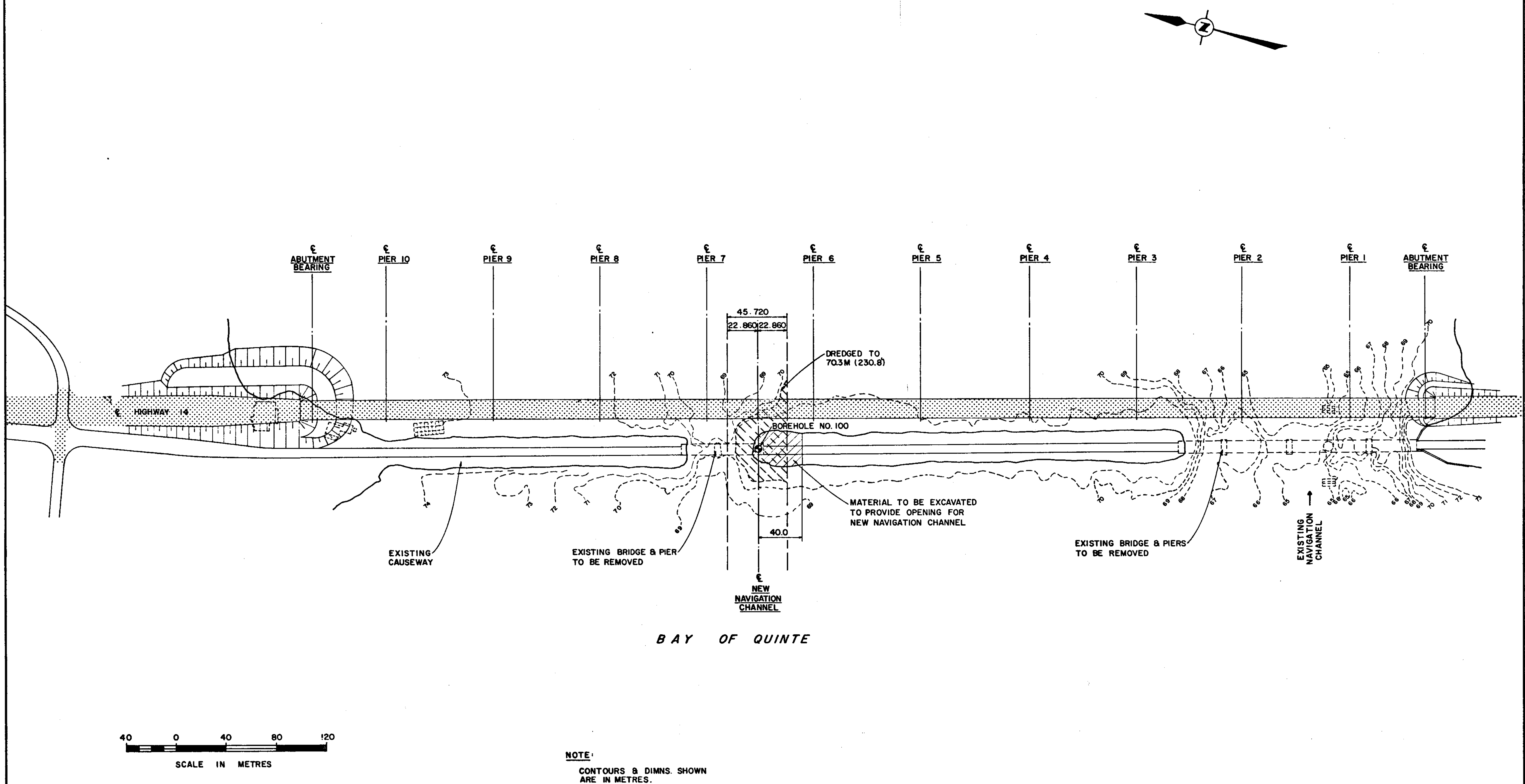
STEVENSON HARDTKE ASSOCIATES LIMITED



EASTERLY FLOW PATTERN - EXISTING

BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO

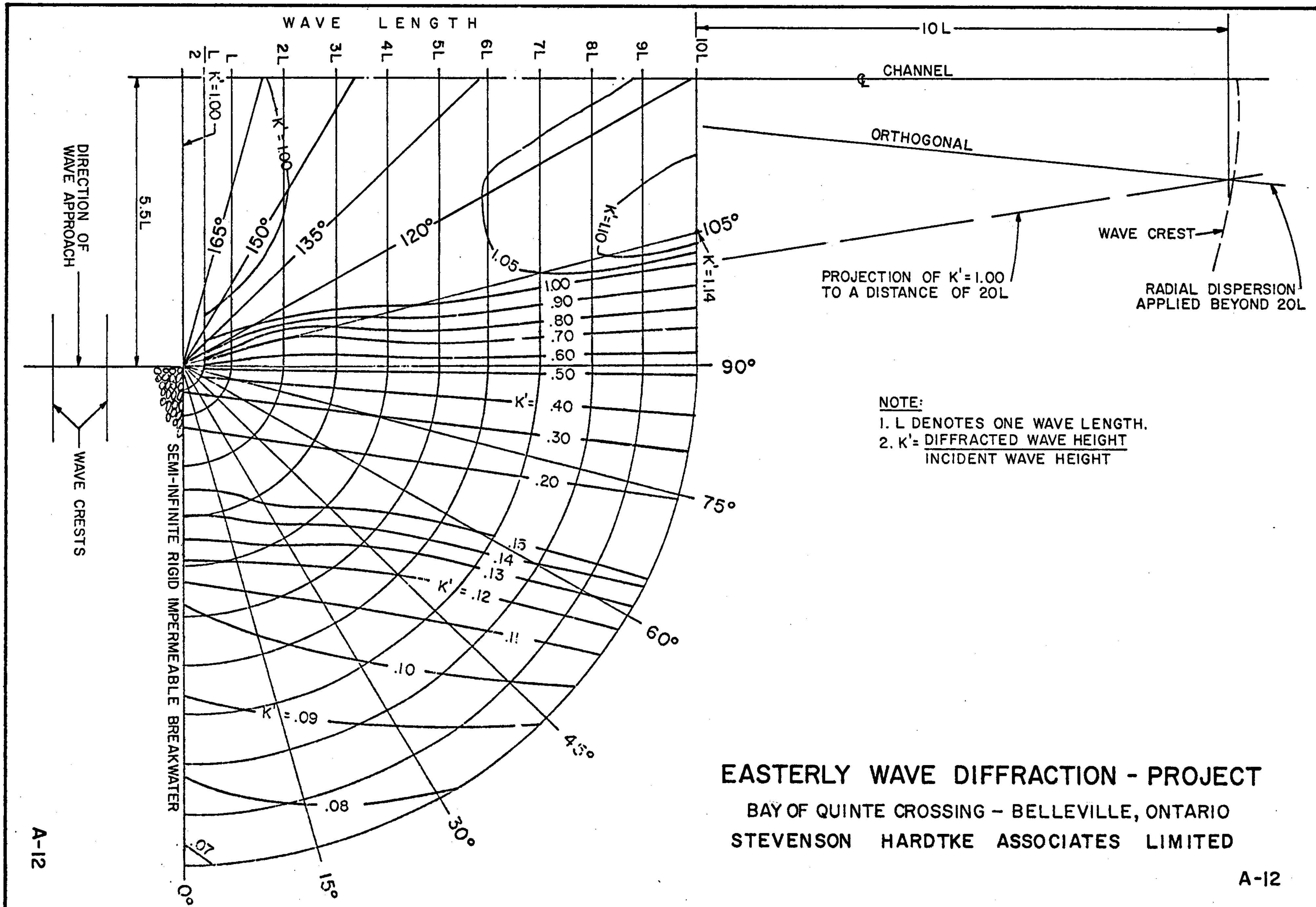
STEVENSON HARDTKE ASSOCIATES LIMITED



PROPOSED LOCATION OF BRIDGE PIERS

BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO

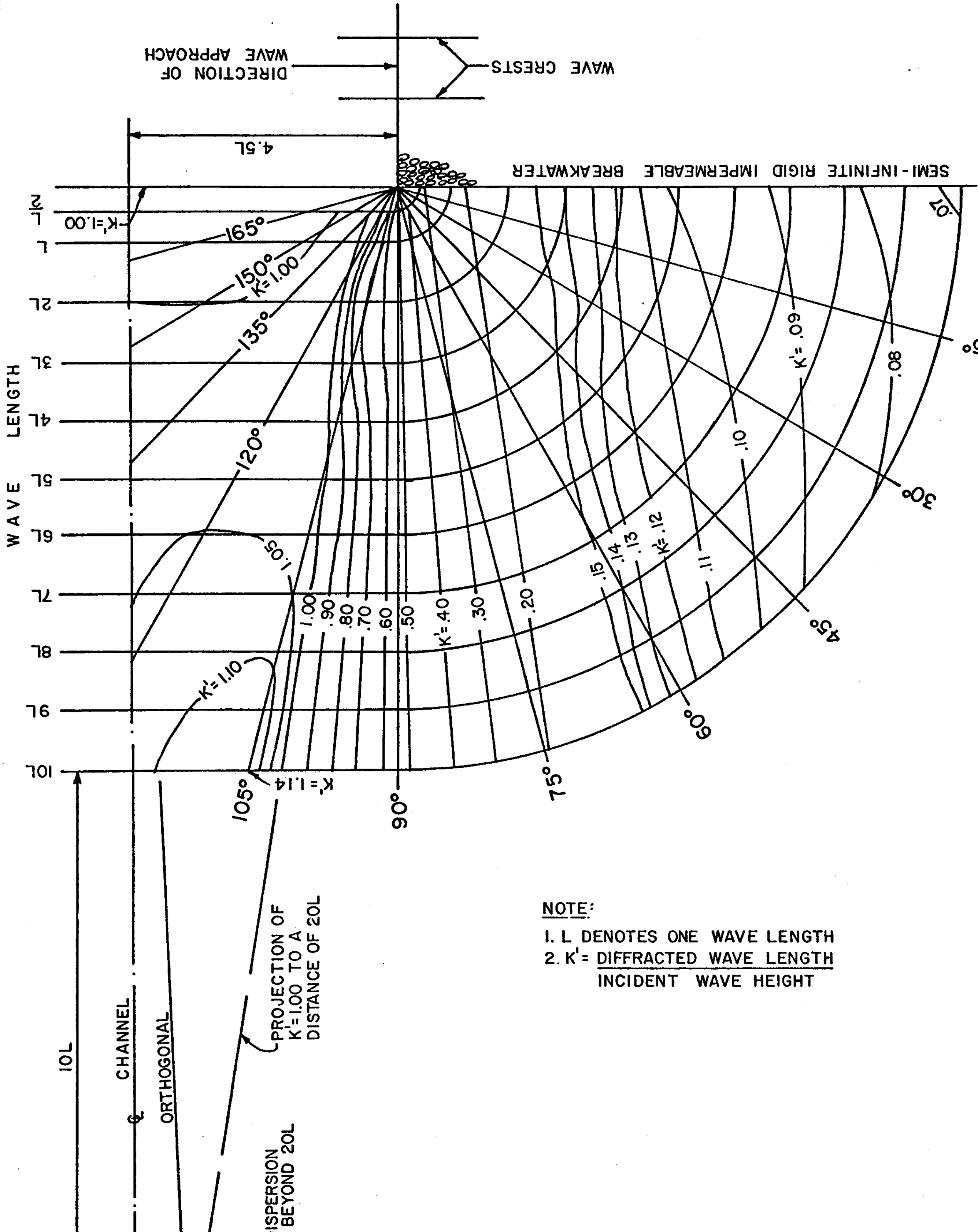
STEVENSON HARDTKE ASSOCIATES LIMITED



EASTERLY WAVE DIFFRACTION - PROJECT

BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED

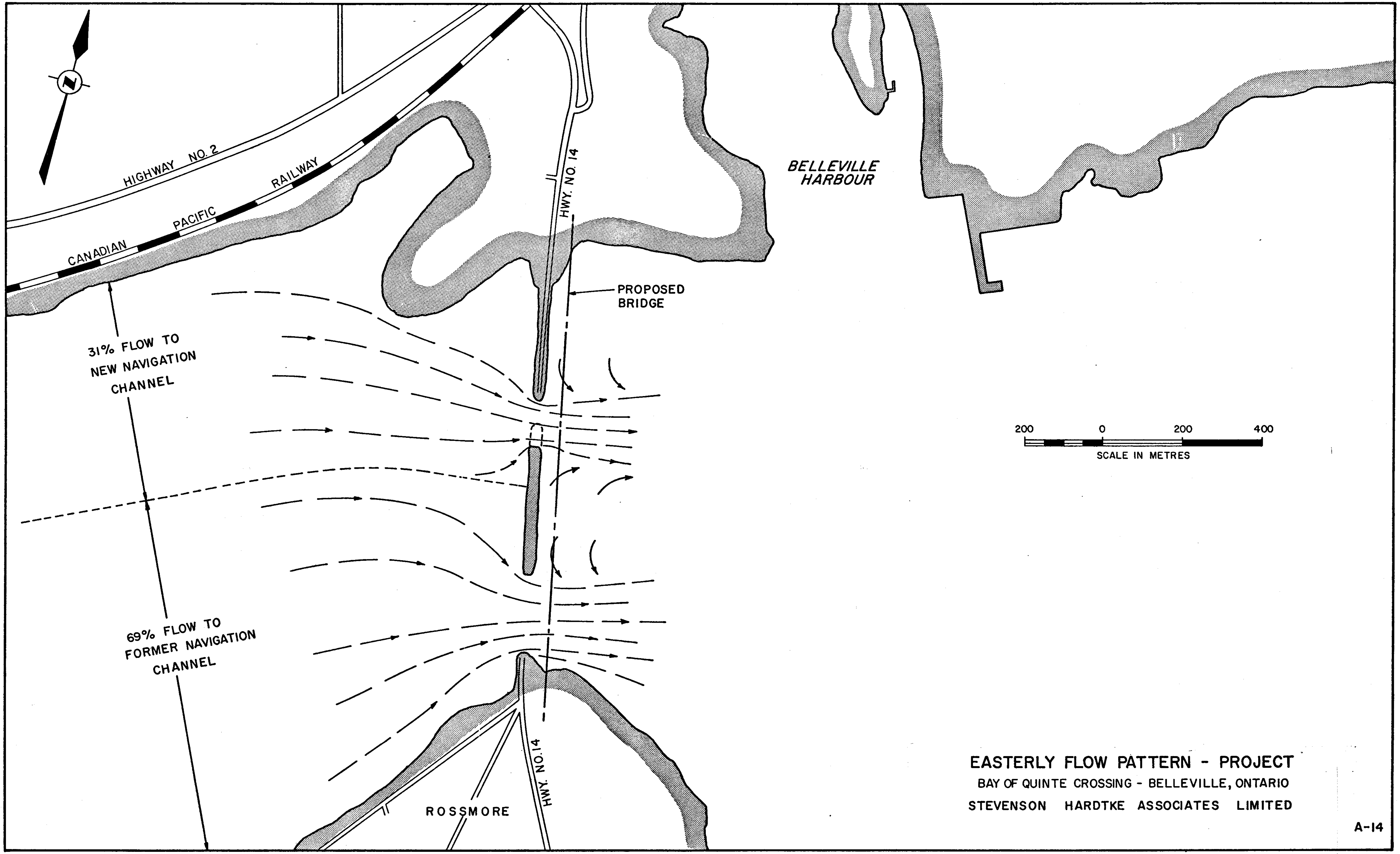


NOTE:
1. L DENOTES ONE WAVE LENGTH
2. $K' = \frac{\text{DIFFRACTED WAVE LENGTH}}{\text{INCIDENT WAVE HEIGHT}}$

WESTERLY WAVE DIFFRACTION - PROJECT

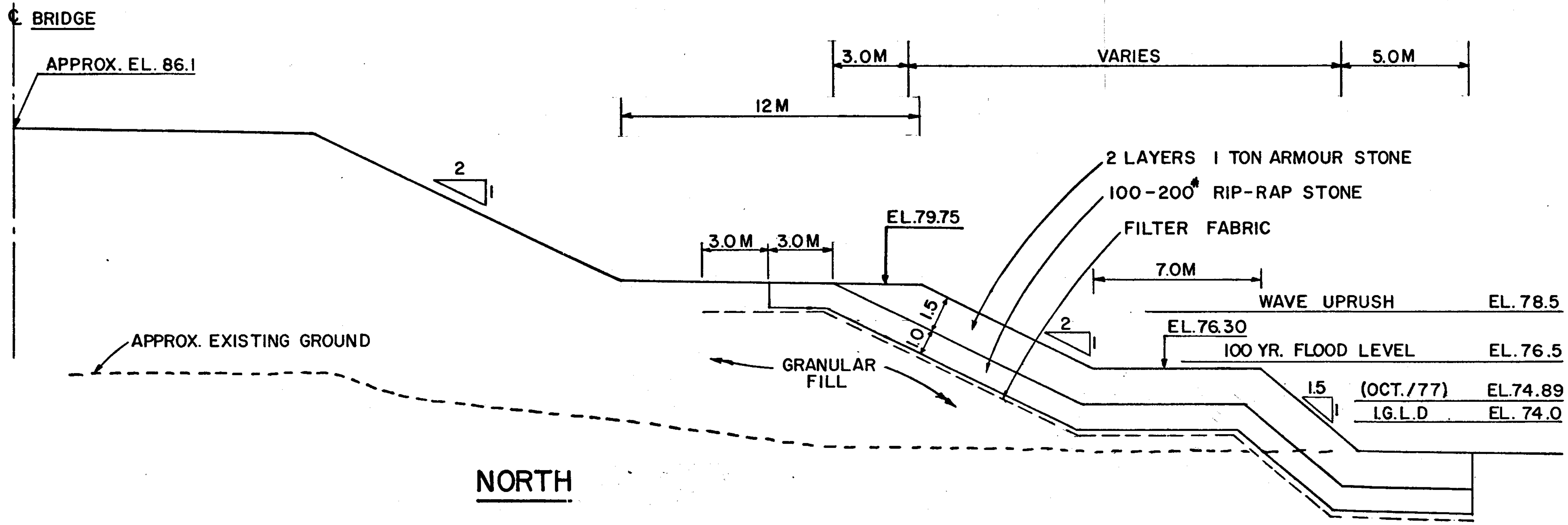
BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO

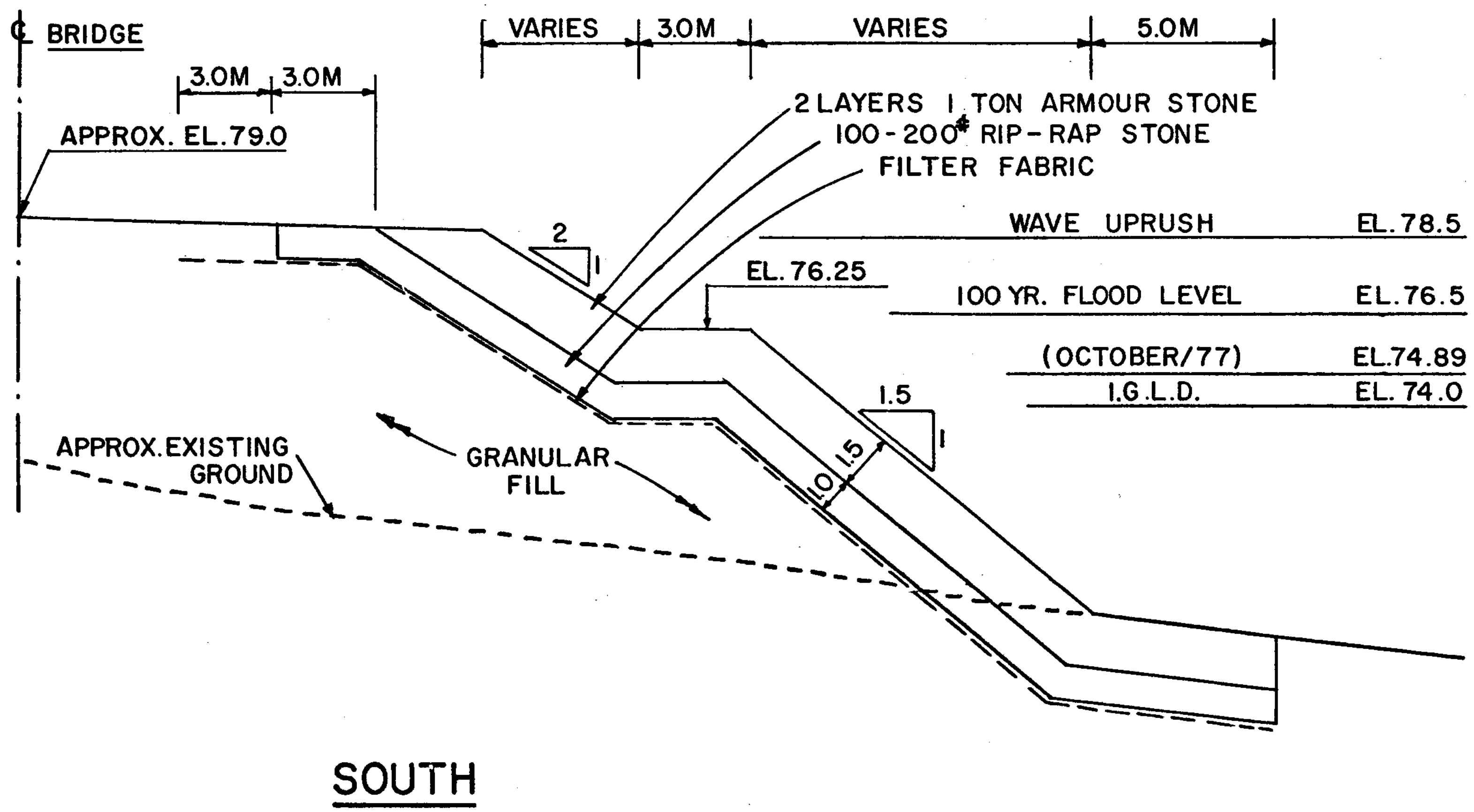
STEVENSON HARDTKE ASSOCIATES LIMITED



EASTERLY FLOW PATTERN - PROJECT
BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO
STEVENSON HARDTKE ASSOCIATES LIMITED

A-14





NOTES:

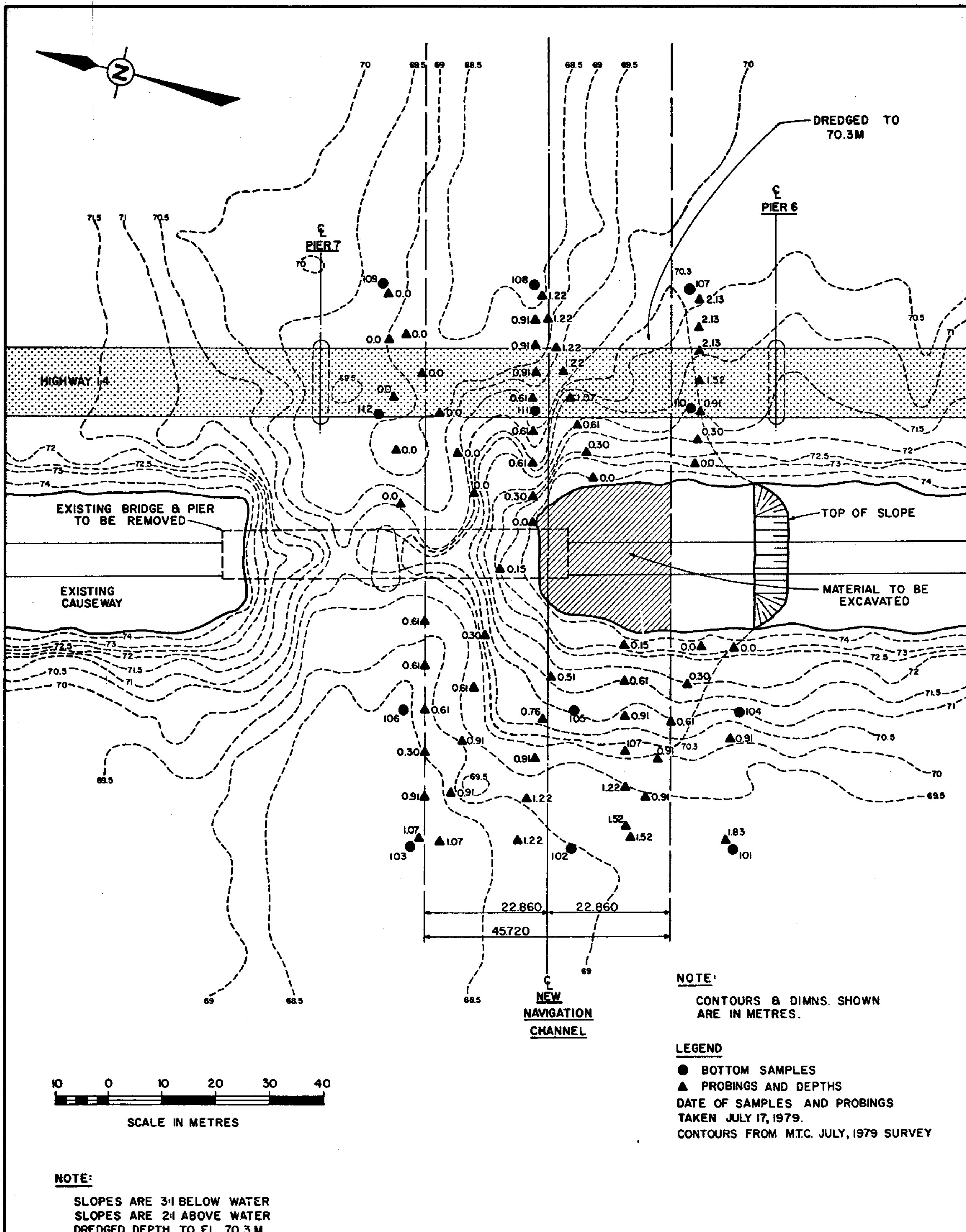
1. DESIGN WAVE HEIGHT 1.2M (4FT.)
2. GENERAL ARRANGEMENT OF SECTIONS AS PER M.T.C. DWG. NO. W.P. 134-74-01 JAN./79

ARMOURING - APPROACH EMBANKMENTS

BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED

A-15



NAVIGATION DREDGING

BAY OF QUINTE CROSSING - BELLEVILLE, ONTARIO

STEVENSON HARDTKE ASSOCIATES LIMITED

A-16

APPENDIX B

Sediment Samples - Particle Size

Table 1 - 11

(Sample Locations Enclosure A-9)

APPENDIX B

TABLE I

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE #1

DIAM. CLASS INTERVAL (MICRON)	MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00- 12.00	7.00	775	514.3	51.4
12.00- 22.00	17.00	384	254.8	76.9
22.00- 32.00	27.00	142	94.2	86.3
32.00- 42.00	37.00	85	56.4	92.0
42.00- 52.00	47.00	52	34.5	95.4
52.00- 62.00	57.00	13	8.6	96.3
62.00- 72.00	67.00	16	10.6	97.3
72.00- 82.00	77.00	9	6.0	97.9
82.00- 92.00	87.00	7	4.6	98.4
92.00- 102.00	97.00	3	2.0	98.6
102.00- 112.00	107.00	3	2.0	98.8
112.00- 122.00	117.00	8	5.3	99.3
122.00- 132.00	127.00	4	2.7	99.6
132.00- 142.00	137.00	2	1.3	99.7
142.00- 152.00	147.00	3	2.0	99.9
152.00- 162.00	157.00	0	0.0	99.9
162.00- 172.00	167.00	0	0.0	99.9
172.00- 182.00	177.00	0	0.0	99.9
182.00- 192.00	187.00	1	0.7	100.0
192.00- 202.00	197.00	0	0.0	100.0
202.00- 212.00	207.00	0	0.0	100.0
212.00- 222.00	217.00	0	0.0	100.0
222.00- 232.00	227.00	0	0.0	100.0
232.00- 242.00	237.00	0	0.0	100.0
242.00- 252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1507

APPENDIX B

TABLE II

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE #2

DIAM. CLASS INTERVAL (MICRON)	MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00- 12.00	7.00	784	503.9	50.4
12.00- 22.00	17.00	292	187.7	69.2
22.00- 32.00	27.00	136	87.4	77.9
32.00- 42.00	37.00	78	50.1	82.9
42.00- 52.00	47.00	57	36.6	86.6
52.00- 62.00	57.00	51	32.8	89.8
62.00- 72.00	67.00	27	17.4	91.6
72.00- 82.00	77.00	18	11.6	92.7
82.00- 92.00	87.00	17	10.9	93.8
92.00- 102.00	97.00	10	6.4	94.5
102.00- 112.00	107.00	22	14.1	95.9
112.00- 122.00	117.00	10	6.4	96.5
122.00- 132.00	127.00	12	7.7	97.3
132.00- 142.00	137.00	18	11.6	98.5
142.00- 152.00	147.00	5	3.2	98.8
152.00- 162.00	157.00	0	0.0	98.8
162.00- 172.00	167.00	0	0.0	98.8
172.00- 182.00	177.00	0	0.0	98.8
182.00- 192.00	187.00	5	3.2	99.1
192.00- 202.00	197.00	3	1.9	99.3
202.00- 212.00	207.00	8	5.1	99.8
212.00- 222.00	217.00	1	0.6	99.9
222.00- 232.00	227.00	2	1.3	100.0
232.00- 242.00	237.00	0	0.0	100.0
242.00- 252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1556

APPENDIX B

TABLE III

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE #3

DIAM. CLASS INTERVAL (MICRON)	MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00- 12.00	7.00	853	540.9	54.1
12.00- 22.00	17.00	333	211.2	75.2
22.00- 32.00	27.00	144	91.3	84.3
32.00- 42.00	37.00	66	41.9	88.5
42.00- 52.00	47.00	57	36.1	92.1
52.00- 62.00	57.00	41	26.0	94.7
62.00- 72.00	67.00	17	10.8	95.8
72.00- 82.00	77.00	15	9.5	96.8
82.00- 92.00	87.00	8	5.1	97.3
92.00- 102.00	97.00	8	5.1	97.8
102.00- 112.00	107.00	7	4.4	98.2
112.00- 122.00	117.00	5	3.2	98.5
122.00- 132.00	127.00	8	5.1	99.0
132.00- 142.00	137.00	5	3.2	99.4
142.00- 152.00	147.00	3	1.9	99.6
152.00- 162.00	157.00	0	0.0	99.6
162.00- 172.00	167.00	0	0.0	99.6
172.00- 182.00	177.00	0	0.0	99.6
182.00- 192.00	187.00	3	1.9	99.7
192.00- 202.00	197.00	1	0.6	99.8
202.00- 212.00	207.00	2	1.3	99.9
212.00- 222.00	217.00	0	0.0	99.9
222.00- 232.00	227.00	0	0.0	99.9
232.00- 242.00	237.00	1	0.6	100.0
242.00- 252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1577

APPENDIX B

TABLE IV

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE #4

DIAM. CLASS INTERVAL (MM)		MEAN OF INTERVAL (MM)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
0.04-	0.05	0.05	0	0.0	0.0
0.05-	0.06	0.06	105	67.6	6.8
0.06-	0.08	0.07	113	72.7	14.0
0.08-	0.10	0.09	93	59.8	20.0
0.10-	0.13	0.11	62	39.9	24.0
0.13-	0.16	0.14	79	50.8	29.1
0.16-	0.20	0.18	103	66.3	35.7
0.20-	0.25	0.23	117	75.3	43.2
0.25-	0.32	0.29	141	90.7	52.3
0.32-	0.40	0.36	144	92.7	61.6
0.40-	0.50	0.45	129	83.0	69.9
0.50-	0.63	0.57	109	70.1	76.9
0.63-	0.80	0.72	106	68.2	83.7
0.80-	1.00	0.90	78	50.2	88.7
1.00-	1.26	1.13	70	45.0	93.2
1.26-	1.59	1.43	41	26.4	95.9
1.59-	2.00	1.80	25	16.1	97.5
2.00-	2.52	2.26	26	16.7	99.2
2.52-	3.18	2.85	3	1.9	99.4
3.18-	4.00	3.59	3	1.9	99.5
4.00-	5.04	4.52	1	0.6	99.6
5.04-	6.34	5.69	2	1.3	99.7
6.34-	7.98	7.16	3	1.9	99.9
7.98-	10.05	9.01	1	0.6	100.0
10.05-	12.65	11.35	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1554

APPENDIX B

TABLE IVA

PARTICLE SIZE DISTRIBUTION FOR POLLUTECH SAMPLE #4 IN SUSPENSION

DIAM. CLASS INTERVAL (MICRON)	MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00- 12.00	7.00	1084	696.7	69.7
12.00- 22.00	17.00	312	200.5	89.7
22.00- 32.00	27.00	85	54.6	95.2
32.00- 42.00	37.00	30	19.3	97.1
42.00- 52.00	47.00	17	10.9	98.2
52.00- 62.00	57.00	9	5.8	98.8
62.00- 72.00	67.00	8	5.1	99.3
72.00- 82.00	77.00	6	3.9	99.7
82.00- 92.00	87.00	2	1.3	99.8
92.00- 102.00	97.00	1	0.6	99.9
102.00- 112.00	107.00	1	0.6	99.9
112.00- 122.00	117.00	0	0.0	99.9
122.00- 132.00	127.00	1	0.6	100.0
132.00- 142.00	137.00	0	0.0	100.0
142.00- 152.00	147.00	0	0.0	100.0
152.00- 162.00	157.00	0	0.0	100.0
162.00- 172.00	167.00	0	0.0	100.0
172.00- 182.00	177.00	0	0.0	100.0
182.00- 192.00	187.00	0	0.0	100.0
192.00- 202.00	197.00	0	0.0	100.0
202.00- 212.00	207.00	0	0.0	100.0
212.00- 222.00	217.00	0	0.0	100.0
222.00- 232.00	227.00	0	0.0	100.0
232.00- 242.00	237.00	0	0.0	100.0
242.00- 252.00	247.00	0	0.0	100.0

OTAL NUMBER OF PARTICLES = 1556

APPENDIX B

TABLE V

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE #5

DIAM. CLASS INTERVAL (MM)		MEAN OF INTERVAL (MM)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
0.04-	0.05	0.05	0	0.0	0.0
0.05-	0.06	0.06	43	24.2	2.4
0.06-	0.08	0.07	63	35.4	6.0
0.08-	0.10	0.09	31	17.4	7.7
0.10-	0.13	0.11	50	28.1	10.5
0.13-	0.16	0.14	35	19.7	12.5
0.16-	0.20	0.18	56	31.5	15.6
0.20-	0.25	0.23	77	43.3	20.0
0.25-	0.32	0.29	132	74.2	27.4
0.32-	0.40	0.36	209	117.5	39.1
0.40-	0.50	0.45	205	115.2	50.6
0.50-	0.63	0.57	219	123.1	63.0
0.63-	0.80	0.72	178	100.1	73.0
0.80-	1.00	0.90	139	78.1	80.8
1.00-	1.26	1.13	126	70.8	87.9
1.26-	1.59	1.43	71	39.9	91.8
1.59-	2.00	1.80	56	31.5	95.0
2.00-	2.52	2.26	51	28.7	97.9
2.52-	3.18	2.85	7	3.9	98.3
3.18-	4.00	3.59	19	10.7	99.3
4.00-	5.04	4.52	7	3.9	99.7
5.04-	6.34	5.69	3	1.7	99.9
6.34-	7.98	7.16	1	0.6	99.9
7.98-	10.05	9.01	1	0.6	100.0
10.05-	12.65	11.35	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1779

APPENDIX B

TABLE VA

PARTICLE SIZE DISTRIBUTION FOR POLLUTECH SAMPLE # 5 IN SUSPENSION

DIAM. CLASS INTERVAL (MICRON)	MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00- 12.00	7.00	1120	735.4	73.5
12.00- 22.00	17.00	243	159.6	89.5
22.00- 32.00	27.00	66	43.3	93.8
32.00- 42.00	37.00	28	18.4	95.7
42.00- 52.00	47.00	19	12.5	96.9
52.00- 62.00	57.00	11	7.2	97.6
62.00- 72.00	67.00	12	7.9	98.4
72.00- 82.00	77.00	4	2.6	98.7
82.00- 92.00	87.00	4	2.6	98.9
92.00- 102.00	97.00	5	3.3	99.3
102.00- 112.00	107.00	1	0.7	99.3
112.00- 122.00	117.00	7	4.6	99.8
122.00- 132.00	127.00	2	1.3	99.9
132.00- 142.00	137.00	1	0.7	100.0
142.00- 152.00	147.00	0	0.0	100.0
152.00- 162.00	157.00	0	0.0	100.0
162.00- 172.00	167.00	0	0.0	100.0
172.00- 182.00	177.00	0	0.0	100.0
182.00- 192.00	187.00	0	0.0	100.0
192.00- 202.00	197.00	0	0.0	100.0
202.00- 212.00	207.00	0	0.0	100.0
212.00- 222.00	217.00	0	0.0	100.0
222.00- 232.00	227.00	0	0.0	100.0
232.00- 242.00	237.00	0	0.0	100.0
242.00- 252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1523

APPENDIX B

TABLE VI

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE # 6

DIAM. CLASS INTERVAL (MM)		MEAN OF INTERVAL (MM)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
0.04-	0.05	0.05	0	0.0	0.0
0.05-	0.06	0.06	92	59.1	5.9
0.06-	0.08	0.07	143	91.8	15.1
0.08-	0.10	0.09	99	63.5	21.4
0.10-	0.13	0.11	90	57.8	27.2
0.13-	0.16	0.14	95	61.0	33.3
0.16-	0.20	0.18	81	52.0	38.5
0.20-	0.25	0.23	63	40.4	42.6
0.25-	0.32	0.29	59	37.9	46.3
0.32-	0.40	0.36	61	39.2	50.3
0.40-	0.50	0.45	72	46.2	54.9
0.50-	0.63	0.57	101	64.8	61.4
0.63-	0.80	0.72	87	55.8	66.9
0.80-	1.00	0.90	95	61.0	73.0
1.00-	1.26	1.13	104	66.8	79.7
1.26-	1.59	1.43	69	44.3	84.1
1.59-	2.00	1.80	74	47.5	88.9
2.00-	2.52	2.26	77	49.4	93.8
2.52-	3.18	2.85	13	8.3	94.7
3.18-	4.00	3.59	40	25.7	97.2
4.00-	5.04	4.52	20	12.8	98.5
5.04-	6.34	5.69	13	8.3	99.4
6.34-	7.98	7.16	6	3.9	99.7
7.98-	10.05	9.01	4	2.6	100.0
10.05-	12.65	11.35	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1558

APPENDIX B

TABLE VIA

PARTICLE SIZE DISTRIBUTION FOR POLLUTECH SAMPLE #6 IN SUSPENSION

DIAM. CLASS INTERVAL (MICRON)	MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00- 12.00	7.00	1067	699.7	70.0
12.00- 22.00	17.00	240	157.4	85.7
22.00- 32.00	27.00	88	57.7	91.5
32.00- 42.00	37.00	34	22.3	93.7
42.00- 52.00	47.00	15	9.8	94.7
52.00- 62.00	57.00	13	8.5	95.5
62.00- 72.00	67.00	9	5.9	96.1
72.00- 82.00	77.00	8	5.2	96.7
82.00- 92.00	87.00	8	5.2	97.2
92.00- 102.00	97.00	2	1.3	97.3
102.00- 112.00	107.00	7	4.6	97.8
112.00- 122.00	117.00	13	8.5	98.6
122.00- 132.00	127.00	6	3.9	99.0
132.00- 142.00	137.00	4	2.6	99.3
142.00- 152.00	147.00	5	3.3	99.6
152.00- 162.00	157.00	0	0.0	99.6
162.00- 172.00	167.00	0	0.0	99.6
172.00- 182.00	177.00	0	0.0	99.6
182.00- 192.00	187.00	2	1.3	99.7
192.00- 202.00	197.00	1	0.7	99.8
202.00- 212.00	207.00	2	1.3	99.9
212.00- 222.00	217.00	0	0.0	99.9
222.00- 232.00	227.00	0	0.0	99.9
232.00- 242.00	237.00	1	0.7	100.0
242.00- 252.00	247.00	0	0.0	100.0

OTAL NUMBER OF PARTICLES = 1525

APPENDIX B

TABLE VII

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE #7

DIAM. CLASS INTERVAL (MICRON)		MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00-	12.00	7.00	1091	688.8	68.9
12.00-	22.00	17.00	264	166.7	85.5
22.00-	32.00	27.00	72	45.5	90.1
32.00-	42.00	37.00	32	20.2	92.1
42.00-	52.00	47.00	18	11.4	93.2
52.00-	62.00	57.00	7	4.4	93.7
62.00-	72.00	67.00	15	9.5	94.6
72.00-	82.00	77.00	9	5.7	95.2
82.00-	92.00	87.00	11	6.9	95.9
92.00-	102.00	97.00	5	3.2	96.2
102.00-	112.00	107.00	15	9.5	97.2
112.00-	122.00	117.00	11	6.9	97.9
122.00-	132.00	127.00	8	5.1	98.4
132.00-	142.00	137.00	7	4.4	98.8
142.00-	152.00	147.00	8	5.1	99.3
152.00-	162.00	157.00	0	0.0	99.3
162.00-	172.00	167.00	0	0.0	99.3
172.00-	182.00	177.00	0	0.0	99.3
182.00-	192.00	187.00	1	0.6	99.4
192.00-	202.00	197.00	3	1.9	99.6
202.00-	212.00	207.00	2	1.3	99.7
212.00-	222.00	217.00	2	1.3	99.8
222.00-	232.00	227.00	3	1.9	100.0
232.00-	242.00	237.00	0	0.0	100.0
242.00-	252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1584

APPENDIX B

TABLE VIII

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE # 8

DIAM. CLASS INTERVAL (MICRON)		MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00-	12.00	7.00	930	600.8	60.1
12.00-	22.00	17.00	347	224.2	82.5
22.00-	32.00	27.00	75	48.4	87.3
32.00-	42.00	37.00	54	34.9	90.8
42.00-	52.00	47.00	32	20.7	92.9
52.00-	62.00	57.00	18	11.6	94.1
62.00-	72.00	67.00	12	7.8	94.8
72.00-	82.00	77.00	17	11.0	95.9
82.00-	92.00	87.00	8	5.2	96.4
92.00-	102.00	97.00	12	7.8	97.2
102.00-	112.00	107.00	10	6.5	97.9
112.00-	122.00	117.00	8	5.2	98.4
122.00-	132.00	127.00	11	7.1	99.1
132.00-	142.00	137.00	5	3.2	99.4
142.00-	152.00	147.00	3	1.9	99.6
152.00-	162.00	157.00	0	0.0	99.6
162.00-	172.00	167.00	0	0.0	99.6
172.00-	182.00	177.00	0	0.0	99.6
182.00-	192.00	187.00	3	1.9	99.8
192.00-	202.00	197.00	0	0.0	99.8
202.00-	212.00	207.00	3	1.9	100.0
212.00-	222.00	217.00	0	0.0	100.0
222.00-	232.00	227.00	0	0.0	100.0
232.00-	242.00	237.00	0	0.0	100.0
242.00-	252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1548

APPENDIX B

TABLE IX

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE #9

DIAM. CLASS INTERVAL (MICRON)		MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00-	12.00	7.00	1027	662.6	66.3
12.00-	22.00	17.00	249	160.6	82.3
22.00-	32.00	27.00	98	63.2	88.6
32.00-	42.00	37.00	54	34.8	92.1
42.00-	52.00	47.00	29	18.7	94.0
52.00-	62.00	57.00	13	8.4	94.8
62.00-	72.00	67.00	17	11.0	95.9
72.00-	82.00	77.00	12	7.7	96.7
82.00-	92.00	87.00	15	9.7	97.7
92.00-	102.00	97.00	3	1.9	97.9
102.00-	112.00	107.00	7	4.5	98.3
112.00-	122.00	117.00	1	0.6	98.4
122.00-	132.00	127.00	8	5.2	98.9
132.00-	142.00	137.00	1	0.6	99.0
142.00-	152.00	147.00	4	2.6	99.2
152.00-	162.00	157.00	0	0.0	99.2
162.00-	172.00	167.00	0	0.0	99.2
172.00-	182.00	177.00	0	0.0	99.2
182.00-	192.00	187.00	2	1.3	99.4
192.00-	202.00	197.00	4	2.6	99.6
202.00-	212.00	207.00	5	3.2	99.9
212.00-	222.00	217.00	0	0.0	99.9
222.00-	232.00	227.00	1	0.6	100.0
232.00-	242.00	237.00	0	0.0	100.0
242.00-	252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1550

APPENDIX B

TABLE X

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE # 10

DIAM. CLASS INTERVAL (MICRON)	MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00- 12.00	7.00	919	580.9	58.1
12.00- 22.00	17.00	343	216.8	79.8
22.00- 32.00	27.00	143	90.4	88.8
32.00- 42.00	37.00	71	44.9	93.3
42.00- 52.00	47.00	39	24.7	95.8
52.00- 62.00	57.00	20	12.6	97.0
62.00- 72.00	67.00	13	8.2	97.9
72.00- 82.00	77.00	9	5.7	98.4
82.00- 92.00	87.00	4	2.5	98.7
92.00- 102.00	97.00	3	1.9	98.9
102.00- 112.00	107.00	4	2.5	99.1
112.00- 122.00	117.00	3	1.9	99.3
122.00- 132.00	127.00	4	2.5	99.6
132.00- 142.00	137.00	3	1.9	99.7
142.00- 152.00	147.00	3	1.9	99.9
152.00- 162.00	157.00	0	0.0	99.9
162.00- 172.00	167.00	0	0.0	99.9
172.00- 182.00	177.00	0	0.0	99.9
182.00- 192.00	187.00	1	0.6	100.0
192.00- 202.00	197.00	0	0.0	100.0
202.00- 212.00	207.00	0	0.0	100.0
212.00- 222.00	217.00	0	0.0	100.0
222.00- 232.00	227.00	0	0.0	100.0
232.00- 242.00	237.00	0	0.0	100.0
242.00- 252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1582

APPENDIX B

TABLE XI

PARTICLE SIZE DISTRIBUTION DATA FOR POLLUTECH SAMPLE # 11

DIAM. CLASS INTERVAL (MICRON)	MEAN OF INTERVAL (MICRON)	FREQUENCY	FREQUENCY PER 1000	CUMULATIVE PER CENT UNDERSIZE
2.00- 12.00	7.00	894	569.4	56.9
12.00- 22.00	17.00	337	214.6	78.4
22.00- 32.00	27.00	143	91.1	87.5
32.00- 42.00	37.00	68	43.3	91.8
42.00- 52.00	47.00	41	26.1	94.5
52.00- 62.00	57.00	23	14.6	95.9
62.00- 72.00	67.00	15	9.6	96.9
72.00- 82.00	77.00	15	9.6	97.8
82.00- 92.00	87.00	4	2.5	98.1
92.00- 102.00	97.00	4	2.5	98.3
102.00- 112.00	107.00	7	4.5	98.8
112.00- 122.00	117.00	6	3.8	99.2
122.00- 132.00	127.00	3	1.9	99.4
132.00- 142.00	137.00	6	3.8	99.7
142.00- 152.00	147.00	0	0.0	99.7
152.00- 162.00	157.00	0	0.0	99.7
162.00- 172.00	167.00	0	0.0	99.7
172.00- 132.00	177.00	0	0.0	99.7
182.00- 192.00	187.00	1	0.6	99.8
192.00- 202.00	197.00	2	1.3	99.9
202.00- 212.00	207.00	1	0.6	100.0
212.00- 222.00	217.00	0	0.0	100.0
222.00- 232.00	227.00	0	0.0	100.0
232.00- 242.00	237.00	0	0.0	100.0
242.00- 252.00	247.00	0	0.0	100.0

TOTAL NUMBER OF PARTICLES = 1570

APPENDIX C

Belleville Public Utilities Commission

Water Quality Data

Microbiological Report

APPENDIX C

MICROBIOLOGICAL REPORT

Raw Water
(Per 100 ml)

(Belleville P.U.C. Data)

Date	Fecal Coliforms	Background Colonies	Coliform Bacteria
Jan. 8/74	2	190	102
Jan.15/74	24	640	220
Jan.22/74	30	1,420	248
Jan.29/74	100	2,400	3,200
Feb. 5/74	32	1,240	566
Feb.12/74	34	860	700
Feb.19/74	Nil	194	128
Feb.26/74	Nil	1,060	286
Mar. 5/74	6	800	392
Mar.12/74	52	3,800	800
Mar.19/74	Nil	283	68
Mar.26/74	Nil	940	12
Apr. 2/74	2	720	120
Apr.16/74	4	860	12
Apr.23/74	2	480	156
May 7/74	2	620	100
May 14/74	30	1,020	230
May 21/74	8	660	214
May 28/74	10	840	6
May 30/74	Nil	620	17
Jun. 4/74	Nil	Lab Acc	Lab Acc
Jun.11/74	6	9,900	4,100
Jun.17/74	12	5,200	3,900
Jun.25/74	0	9,900	700
Jul. 2/74	0	6,800	1,100
Jul. 9/74	0	88,000	3,500
Jul.16/74	0	10,200	12,000
Jul.23/74	2	14,100	20,700
Jul.30/74	2	15,600	8,300

APPENDIX C

MICROBIOLOGICAL REPORT (Cont'd)

Raw Water
(Per 100 ml)

(Belleville P.U.C. Data)

Date	Fecal Coliforms	Background Colonies	Coliform Bacteria
Aug. 6/74	0	5,600	7,400
Aug.13/74	0	43,000	200
Aug.19/74	2	25,000	1,100
Aug.27/74	0	8,100	300
Sep. 3/74	0	10,800	100
Sep.10/74	4	11,000	800
Sep.17/74	4	7,400	200
Sep.24/74	14	13,000	400
Oct. 1/74	8	0	0
Oct. 8/74	8	1,600	400
Oct.15/74	2	3,800	1,800
Oct.22/74	0	1,800	1,700
Oct.29/74	0	16,400	1,000
Nov. 5/74	2	1,600	400
Nov.12/74	4	1,260	298
Nov.18/74	70	19,000	7,200
Nov.26/74	0	620	392
Dec. 3/74	0	162	122
Dec.10/74	16	1,020	320
Dec.17/74	16	1,060	90
Jan.14/75	28	360	322
Jan.21/75	6	12,400	40
Jan.28/75	6	1,000	136
Feb. 4/75	4	518	116
Feb.18/75	4	1,060	130
Mar. 4/75	30	1,960	254
Mar.11/75	18	1,900	220
Mar.18/75	14	2,200	600
Mar.25/75	60	6,600	400

APPENDIX C

MICROBIOLOGICAL REPORT (Cont'd)

Raw Water
(Per 100 ml)

(Belleville P.U.C. Data)

Date	Fecal Coliforms	Background Colonies	Coliform Bacteria
Apr. 1/75	0	480	106
Apr. 8/75	0	282	82
Apr.15/75	0	204	18
Apr.22/75	0	1,000	214
Apr.29/75	10	920	294
May 6/75	10	600	32
May 13/75	4	2,080	38
May 20/75	0	920	34
Jun. 3/75	0	11,400	60
Jun.10/75	2	80,000	14
Jun.24/75	0	3,400	20
Jul. 8/75	0	15,800	8
Jul.15/75	0	5,700	0
Jul.22/75	0	36,000	0
Aug. 5/75	0	13,300	1,000
Aug.19/75	0	5,800	6
Aug.20/75	6	8,000	60
Sep. 2/75	0	11,600	288
Sep.10/75	20	9,800	20
Sep.30/75	24	7,500	76
Oct. 7/75	6	6,800	88
Oct.14/75	8	5,600	76
Oct.28/75	2	1,900	32
Nov. 4/75	16	5,300	232
Nov.18/75	56	7,700	1,600
Dec.16/75	48	6,100	700
Jan.13/76	440	2,100	9,100
Jan.20/76	30	2,100	300
Jan.27/76	190	8,500	1,600

APPENDIX C

MICROBIOLOGICAL REPORT (Cont'd)

Raw Water
(Per 100 ml)

(Belleville P.U.C. Data)

Date	Fecal Coliforms	Background Colonies	Coliform Bacteria
Feb. 3/76	50	1,240	254
Feb.10/76	32	2,700	800
Feb.18/76	26	900	458
Feb.24/76	36	5,700	800
Mar. 2/76	46	10,600	1,700
Mar.16/76	56	4,600	2,300
Mar.23/76	72	6,100	1,800
Apr.20/76	0	860	76
Apr.27/76	0	26,000	272
May 4/76	0	16	2
May 11/76	0	1,100	300
May 18/76	28	2,200	400
Jun. 1/76	8	960	120
Jun. 8/76	6	31,000	300
Jun.15/76	0	8,900	200
Jun.29/76	0	33,000	56
Jul. 5/76	0	36,000	100
Jul.13/76	0	15,000	14
Jul.23/76	0	1,600	100
Aug. 3/76	0	32,000	100
Aug.10/76	0	39,000	8
Aug.17/76	0	97,000	0
Aug.24/76	4	5,500	4
Aug.31/76	0	90,000	10
Sep.14/76	0	15,300	6
Sep.21/76	4	4,400	22
Oct.12/76	6	800	100
Oct.15/76	2	1,100	52
Oct.19/76	18	2,200	34
Oct.26/76	14	700	170

APPENDIX C

MICROBIOLOGICAL REPORT (Cont'd)

Raw Water
(Per 100 ml)

(Belleville P.U.C. Data)

Date	Fecal Coliforms	Background Colonies	Coliform Bacteria
Nov. 2/76	6	3,900	520
Nov. 9/76	2	538	220
Nov.16/76	16	1,800	300
Nov.23/76	12	286	110
Dec. 7/76	<2	1,100	106
Dec.21/76	40	640	156
Jan. 4/77	<2	180	48
Jan.18/77	8	386	70
Jan. 1/77	282	6,200	1,300
Feb. 1/77	110	4,500	2,000
Feb. 8/77	62	1,100	300
Feb.15/77	146	15,000	2,100
Feb.22/77	10	2,000	240
Mar. 1/77	208	3,000	1,300
Mar. 8/77	10	7,400	700
Mar.15/77	58	25,000	400
Mar.22/77	34	318	238
Mar.29/77	0	1,700	360
Apr. 5/77	10	1,500	800
Apr.12/77	2	620	464
Apr.19/77	<2	2,500	640
Apr.26/77	6	1,900	100
May 3/77	<2	2,000	600
May 10/77	2	1,900	100
May 17/77	<2	9,600	88
May 24/77	<2	5,500	120
May 31/77	6	15,000	500
Jun. 7/77	<2	8,100	<4
Jun. 6/77	<2	87,000	8

APPENDIX C

MICROBIOLOGICAL REPORT (Cont'd)

Raw Water
(Per 100 ml)

(Belleville P.U.C. Data)

Date	Fecal Coliforms	Background Colonies	Coliform Bacteria
Jun.21/77	<2	10,700	100
Jun.28/77	<2	30,000	10
Jul. 5/77	2	70,000	300
Jul. 7/77	4	2,000	10
Jul.19/77	<2	15,000	20
Jul.26/77	<2	2,700	400
Aug. 2/77	<2	15,000	300
Aug. 9/77	<2	1,700	4
Aug.16/77	12	15,000	300
Aug.23/77	6	1,000	16
Aug.30/77	26	15,000	33
Sep. 7/77	<2	4,400	2
Sep.13/77	10	5,000	128
Sep.20/77	2	43,000	24
Sep.27/77	4	17,000	100
Oct.25/77	12	1,200	78
Nov. 2/77	12	132	22
Nov. 8/77	-	1,800	<100
Nov.23/77	-	1,500	240
Dec.10/77	28	20,000	108
Dec.22/77	300	53,000	5,100
Dec.14/77	12	23,000	144
Jan. 3/78	20	G 15,000	900
Jan.10/78	300	G 15,000	13,000
Jan.17/78	28	G 15,000	450
Jan.24/78	6	G 15,000	400
Jan.31/78	32	9,400	440
Feb. 7/78	16	1,600	700
Feb.15/78	8	16,000	820

APPENDIX C

BELLEVILLE PUBLIC UTILITIES COMMISSION

WATER QUALITY DATA

MICROBIOLOGICAL REPORT (Cont'd)

Raw Water
(Per 100 ml)

(Belleville P.U.C. Data

Date	Fecal Coliforms	Background Colonies	Coliform Bacteria
Feb.22/78	18	2,200	800
Feb.28/78	10	1,000	196
Mar. 7/78	<2	520	100
Mar.14/78	8	2,800	200
Mar.21/78	10	500	200
Mar.28/78	42	3,400	1,200
Apr. 4/78	34	2,400	440
Apr.11/78	34	4,700	400
Apr.18/78	8	1,400	226
Apr.25/78	2	680	84
May 2/78	<2	320	24
May 9/78	<2	120	24
May 16/78	–	42	0
May 23/78	<2	640	22
May 30/78	<2	5,700	300
Jun. 6/78	<2	18,000	C4
Jun.13/78	<2	59,000	C100
Jun.20/78	<2	G 15,000	C36
Jun.27/78	<2	1,000	4
July.4/78	<2	600	<2
Jul.11/78	<2	20,000	C4
Jul.18/78	<2	3,400	<2
Aug.15/78	2	G 15,000	C12
Aug.22/78	<2	22,000	C22
Aug.29/78	2	27,000	C12
Sep.12/78	<2	21,000	C560

APPENDIX C

WATER QUALITY

(Belleville P.U.C. Data)

Date	Colour		Turbidity		T.O.N.		
	Raw	Tap	Raw	Tap	Raw	Tap	Algae (Raw)
Jan/73	10-30	<5	2.0-15.0	1.0	30-40	8-10	Nil
Feb/73	10-30	<5	1.5-12.0	1.0	20-25	6-8	Nil
Mar/73	15-50	<5	2-15	1	35	12	Nil
Apr/73	20	<5	4.0	<1	40-50	12-15	Nil to Light
May/73	20	<5	2-4	<1	80-90	12-15	2,600 ASU
Jun/73	15-30	<5	2.0-7.2	<1.0	Nil	Nil	Mod. to Heavy
Jul/73	30-40	<5	4.0-10.0	1.0	130	30	10,000 ASU
Aug/73	25-35	<5	3.3-8.0	<1	Nil	Nil	12,000 ASU
Sep/73	30-45	<5	3.5-10.5	1.0	200+		12,000 ASU
Oct/73	35-20	<5	2-9	<1	200	25	11,079 ASU
Nov/73	10-15	<5	2-10	<1	100	25	3,000 ASU
Dec/73	10	<5	5.0-2.0	<1	25	15	Nil
Jan/74	15	<5	1.0-5.2	<1.0	25	12	Nil
Feb/74	15	<5	1.3-5.3	<1.0	25	15	Nil
Mar/74	15-25	<5	2-10	<1	20	10	Nil
Apr/74	20-30	<5	12.0-3.0	<1	25	10	4,000 ASU
May/74	15-25	<5	4.0-10.0	<1.0	60	12	6,000 ASU
Jun/74	15-30	<5	2.0-9.0	<1	100	40	6,500 ASU
Jul/74	25-45	<5	4-11	1.0	200	40	10,000 ASU
Aug/74	20-50	<5	6-12	1.0	200+	40-60	17,800 ASU
Sep/74	30-45	<5	7.5-12.0	1.0	200+	40-50	12,000 ASU
Oct/74	35-20	<5	10-5	1.0	200	20	12,000-3,000 ASU
Nov/74	15-30	<5	4-9	1.0	40	10	Light
Dec/74	10-15	<5	2.0-6.0	<1	20	10	Nil
Jan/75	10	<5	1.3-3.5	1.0	25	8	Nil
Feb/75	10-30	<5	1.8-8.0	<1	25	10	Nil
Mar/75	15-35	<5	3.0-15.0	1.0	20-25	10-15	Nil
Apr/75	15-40	<5	3.0-14.0	1.0	135	20-25	2,000 ASU
May/75	15-35	<5	3-12	<1.0	250+	20-30	4,000 ASU
Jun/75	20-30	<5	5.4-9.0	1.0	200+	35	6,000 ASU

APPENDIX C

WATER QUALITY (Cont'd)

(Belleville P.U.C. Data)

Date	Colour		Turbidity		T.O.N.		
	Raw	Tap	Raw	Tap	Raw	Tap	Algae (Raw)
Jul/75	20-50	<5	4.2-14.5	1.0	200-250	40-50	18,000 ASU
Aug/75	40-50	<5	9.0-14.0	1.0	200+	40	12,000 ASU
Sep/75	20-35	<5	5-10	1.0	200-250	20-40	13,000 ASU
Oct/75	20-30	<5	3.3-9.5	1.0	200	15-20	5,000 ASU
Nov/75	20	<5	4.0-7.0	1.0	135	15	2,900 ASU
Dec/75	20-10	<5	7.5-2.0	1.0	30	10	500 ASU
Jan/76	15-10	<5	3.3-1.9	1.0	15-20	10	Nil
Feb/76	10-20	<5	2.0-8.0	1.0	30	15	Nil
Mar/76	10-25	<5	2.0-12.5	1.0	15	6	Nil
Apr/76	15-25	<5	2.0-9.0	1.0	70	20	Nil
May/76	15-50	<5	2.5-10.6	1.0	150-200	20	2,442 ASU
Jun/76	15-30	<5	3.4-8.3	1.0	150-200	25	4,000 ASU
Jul/76	25-35	<5	5.6-10.0	1.0	150	30	10,000 ASU
Aug/76	30-40	<5	14.4-6.9	.7-1.0	150-200	20-30	10,436 ASU
Sep/76	25-40	<5	7.1-12.6	1.0	200	60	10,000-21,000ASU
Oct/76	20-35	<5	10.4-4.2	.8-.5	150-70	20-35	Mod.
Nov/76	20-10	<5	2.6-7	1.0	20	10	1,000 ASU
Dec/76	10	<5	1.6-4.2	1.0	10	8	Nil
Jan/77	10	<5	1.2-2.0	<1.0	8	6	Nil
Feb/77	10	<5	1.2-2.9	<1.0	13	8	Nil
Mar/77	10-35	<5	1.5-11.8	1.0	20	8	Nil
Apr/77	15-30	<5	3.0-9.8	<1.0	50	25	6,000 ASU
May/77	15	<5	3.5	1.0	80-100	30-40	5,000 ASU
Jun/77	15-30	<5	3.0-8.0	<1.0	150	40	6,000 ASU
Jul/77	20-50	<5	5-17	1.0	200+	40	10,000-15,000ASU
Aug/77	25-50	<5	5.6-14.8	1.0	200+	30	8,000 ASU
Sep/77	20-30	<5	4.7-9.8	<1.0	200	20	10,000 ASU
Oct/77	10-25	<5	10.0-2.4	<1.0	200	20	4,000 ASU
Nov/77	10-20	<5	2.0-4.5	<1.0	140	10	Light
Dec/77	10-20	<5	2-5	<1.0	10	4	Light to Nil

APPENDIX C

WATER QUALITY (Cont'd)

(Belleville P.U.C. Data

Date	Colour		Turbidity		T.O.N.		
	Raw	Tap	Raw	Tap	Raw	Tap	Algae (Raw)
Jan/78	5-15	<5	1.0-3.0	<1.0	10	6	Nil
Feb/78	10-15	<5	1.0-3.5	<1.0	10	8	Nil
Mar/78	10-20	5	0.8-2.4	<1.0	8	6	Nil
Apr/78	10-25	<5	1.4-5.0	<1.0	10	8	Nil

APPENDIX D

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APPENDIX D - REFERENCES

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APPENDIX E

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