GENERAL MANAGER'S C

Final Report

SHORELINE MANAGEMENT PLAN

KETTLE CREEK CONSERVATION AUTHORITY

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Abstract

This report describes the preparation of a shoreline management plan for the Lake Erie shoreline within the watershed of the Kettle Creek Conservation Authority. The purpose of the shoreline management plan is to balance the options of shoreline prevention, protection, environmental impact, monitoring, emergency response and public education in an overall management plan of the shoreline resources. This plan was prepared in accordance with the Guidelines for preparing Shoreline Management Plans prepared by the Ontario Ministry of Natural Resources in 1987. It also gives consideration to certain aspects of a pending provincial policy on shoreline management.

The report is accompanied by 5 appendices bound under separate covers.

The shoreline within the study area was divided into 6 reaches; 5 within the Village of Port Stanley and 1 including the high bluff shorelines either side of Port Stanley. The bluffs outside Port Stanley experience high rates of irreversible erosion, with estimated long term recession rates exceeding 2 metres per year in some locations. The reaches within Port Stanley include 2 sections west (updrift) of the harbour, the harbour area and 2 sections east (downdrift) of the harbour. The reaches updrift of the harbour are characterized by low fillet beaches with limited dune development. Flooding rather than erosion problems are predominant in these reaches. The two reaches downdrift of the harbour contain extensively protected low till bluffs. Flooding of these reaches is not an issue although, depending upon the level of protection provided by the individual structures, erosion can be an issue. The harbour area was not investigated in any detail.

Both preferred and alternate concepts for shoreline management are proposed for the five studied reaches. The preferred concepts include both preventative solutions such as establishing regulatory zones with development restrictions and protective solutions such as flood berms and floodproofing techniques. Prevention techniques are generally preferred to protection.

Separate chapters of the report discuss the preferred and alternate shoreline management concepts, environmental issues, emergency response, monitoring and public information components of the plan. Additional chapters also discuss the data collection and review process, generation of design data and criteria and an overview of prevention and protection techniques.

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SUMMARY

Introduction

This summary outlines the steps taken to prepare a shoreline management plan for the Lake Erie shoreline within the watershed of the Kettle Creek Conservation Authority. The purpose of the shoreline management plan is to balance the options of shoreline prevention, protection, environmental impact, monitoring, emergency response and public education in an overall management plan of the shoreline resources. This plan was prepared in accordance with the Guidelines for Preparing Shoreline Management Plans prepared by the Ontario Ministry of Natural Resources in 1987. It also gives consideration to certain aspects of a pending provincial policy on shoreline management. The key components of the shoreline management plan were identified as:

Prevention:	evaluation of and recommendations for controls and regulations governing new shoreline development
Protection:	evaluation of and recommendations for potential capital works for existing and new development
Emergency Response:	reviewing existing emergency response plans and recommending appropriate modifications
Public Information:	recommendations for dissemination of information and education of the general public regarding shoreline management
Environment:	assess environmental impact of the Shoreline Management Plan
Monitoring:	identify methods to monitor changes to local conditions affecting shoreline management

This plan was prepared specifically for the shoreline within the Kettle Creek Conservation Authority's watershed. The study area, shown in Figure S1, covers most but not quite all of the Lake Erie shoreline within the Townships of Southwold and Yarmouth. The Village of Port Stanley is located between the two townships.

The study started with a field survey to inventory all protection structures within the study area. This was later expanded to also record the location of gullies and setbacks of structures on the high bluffs. Design data and design criteria were derived to provide design wave conditions, design water levels, alongshore and offshore sediment transport rates, design uprush levels and allowable flood depths. This information was then used in preparation of the Shoreline Management Plan.

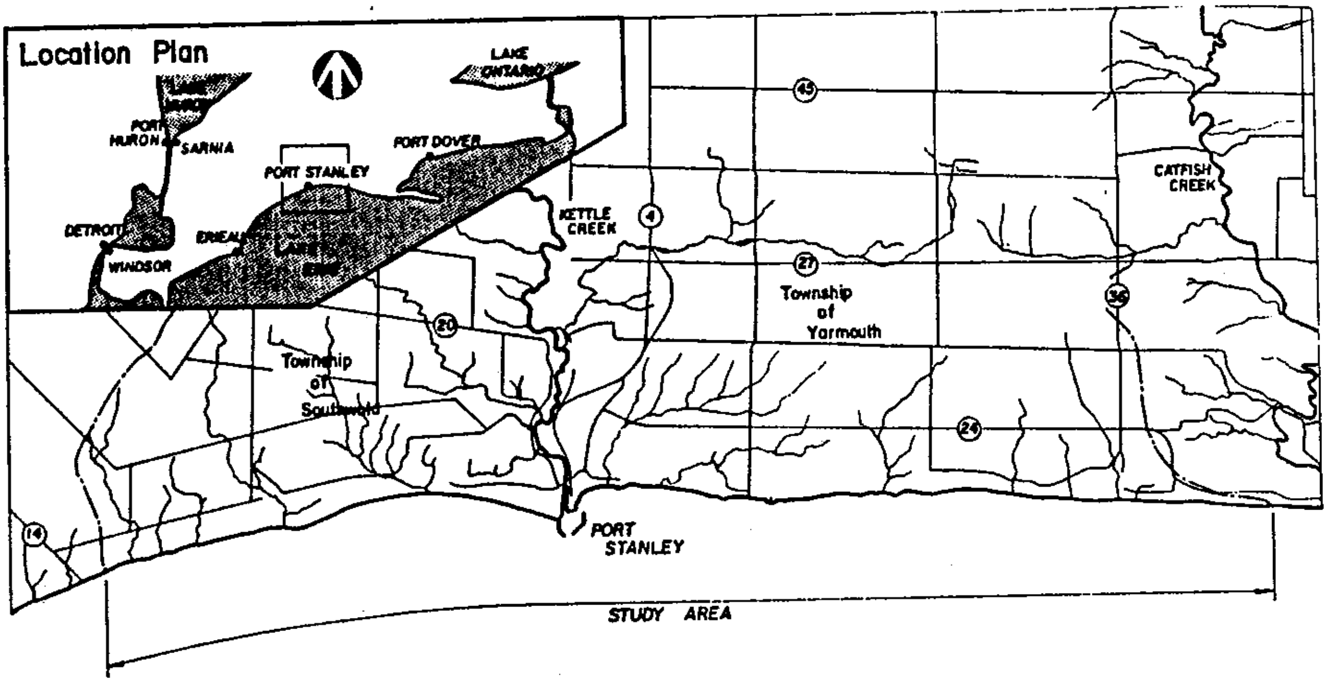


Figure S1
Site Plan

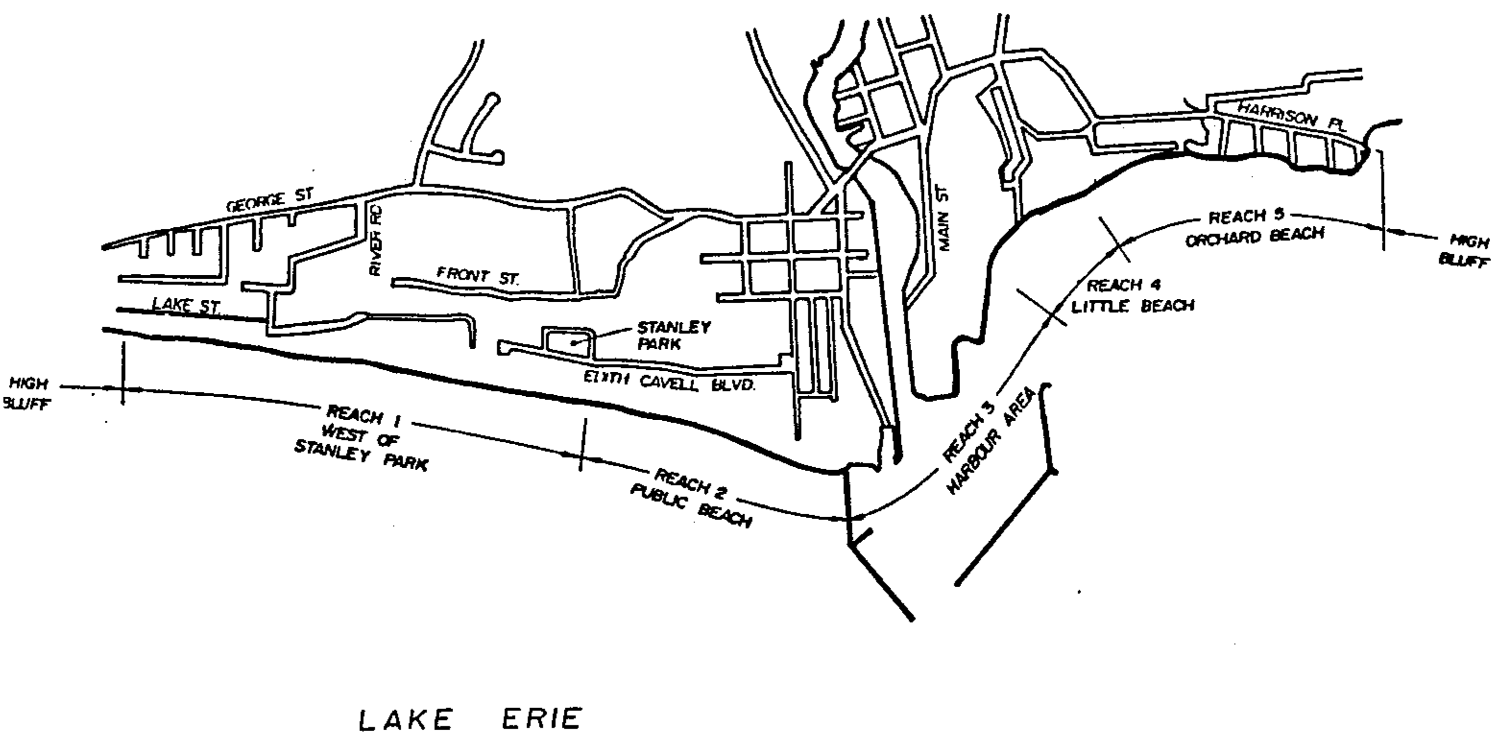


Figure S2
Port Stanley
Shoreline Reaches

Site Conditions

The shoreline within the study area was defined as being either bluff higher than 10 metres, low bluff up to 10 metres in height or sand beach shoreline. The shorelines of the two townships and the extreme eastern and western limits of Port Stanley were high bluffs. The low bluff and sand beach shorelines were found within Port Stanley.

Different preferred and alternate management plans or concepts were developed for the different segments or reaches of the shoreline. All high bluff shoreline was considered together. The Port Stanley shoreline was divided into a total of 5 reaches, based on the physical characteristics of that shoreline. These reaches are shown in Figure S2.

The high bluffs are generally uniform in height, approximately 40 metres high. Bluff stratigraphy does vary throughout the study site but in a general sense it could be described as Port Stanley Till overlain by sand, sand and silt or silt and clay layers varying in thickness from 5 to 15 metres. The shoreline is dissected by a large number of gullies varying from a few metres to a kilometer in length. The very short gullies are by far the most frequent and only a few gullies have developed to more than a couple of hundred metres in length. The formation of the very large gullies was attributed to local groundwater patterns but it was shown that many of the smaller gullies were associated with outlets from artificial drainage systems.

This section of shoreline does not experience flooding problems but is subject to extremely high rates of erosion. This erosion is directly attributable to wave attack at the toe of the bluff. Vegetation, both on the slope of the bluff and the table lands, tends to stabilize the bluff, allowing a steeper slope to form, but it does not effect the long term erosion rate. Average bluff recession rates were calculated to vary from 1.0 to 4.2 metres per year. The highest rates were found immediately to the east of reach 5 of Port Stanley. Excluding this one area the next highest average recession rate was 2.4 metres per year.

Reach 1 of Port Stanley is characterized by a low dune system which fronts a low backshore beach increasing in width from west to east. The increase in beach width occurs because this is a fillet beach held in place by the Port Stanley harbour breakwater. This area has been fairly extensively developed with mostly summer cottages although there are a number of full year residences.

Only a very few of these dwellings have been protected with man made structures. The natural dune system provides erosion and flood protection except at very high static water levels. When static water levels are high the storm surge associated with severe winds overtops the dunes and floods both properties and dwellings. Erosion to the dune system does occur but this is only temporary as the dunes are rebuilt at lower water levels and during calmer periods.

Reach 2 of Port Stanley contains the most publicly used section of shoreline within the study area. There is a slight dune formation along the crest of the beach but the dune is not as well developed as within Reach 1. The beach behind the dune is almost flat and extends well inland. There are approximately 150 structures located within the wave uprush zone. Some of these structures are flooded by waves which wash over both the public beach and the piers of the inner harbour and, in a few locations, through storm sewer intakes located below the flood water levels.

Reach 3 of Port Stanley consists of the harbour lands owned by the Federal Government. These lands are susceptible to flooding during high lake levels but the extent to which flooding takes place was not evaluated because the authority's mandate does not apply to federal lands. The existing breakwaters are holding the fillet beach in its present location.

Reach 4 of Port Stanley is characterized by low bluffs, about 3 m in height. There are a total of 10 properties located within this reach. Each of the 10 properties is presently fronted by a protection structure. The type and effectiveness of each structure varies. This protection has been constructed on an individual basis rather than as part of a concerted effort. These structures are in turn fronted by a low sand beach created by placing sand dredged from the harbour entrance. The eastern end of this beach is held in place by a small protruding headland constructed of scrap concrete. This beach will not increase in size in the long term, even if more dredgate is placed, unless the eastern headland is extended.

Reach 5 of Port Stanley is also characterized by low bluff, but here it varies in height from 4 to 6 metres. This section of shoreline has been completely protected by a number of different types of structures. A few adjacent property owners have jointly constructed concrete rubble revetments but overall there has been no effort to standardize the protection. Some of the property owners have placed a large volume of rubble fill and others have only armoured the natural bank with concrete. Still others have constructed revetments out of concrete blocks and, in one instance, some armourstone.

Design Data and Criteria

Design data and criteria were derived during the preparation of the plan. Water level data was provided by the Ministry of Natural Resources. Offshore and nearshore wave conditions and wave uprush levels were simulated by the consultant with in house numerical models. Table S1 summarizes the design data and criteria.

Table S1

100 year breaking significant wave height	2.8 m
100 year static water level	175.0 m GSC
mean annual high water level	174.8 m GSC
100 year storm surge	0.96 m
100 year flood level	175.5 m GSC
100 year uprush elevation on beach dune	177.0 m GSC
100 year uprush elevation landward of dune	176.8 m GSC
100 year uprush elevation on a sloped structure	178.7 m GSC
100 year uprush elevation on a vertical wall	181.4 m GSC
100 year dune recession width	5 m
100 year wave active beach width	40 m
minimum allowable structural opening freeboard	0.3 m
minimum allowable dyke freeboard	0.3 m
minimum allowable structural setback	based on regulatory shoreland zone

This study benefited greatly from very high quality data on shoreline recession and nearshore bottom and bluff stratigraphy and composition. This data was obtained from the Port Burwell Shore Erosion Damage Claim Study conducted by the Federal Government of Canada. This was the most detailed study of coastal processes carried out on the Great Lakes and produced the best available data. The data was used in the preparation of the shoreline management plan to calculate 100 year recession setbacks and an alongshore littoral sediment budget.

Shoreline Management Plan

The shoreline management plan examines each of the components identified within the 1987 Provincial guidelines. The plan is presented by first describing preferred shoreline management concepts then alternate concepts for each shoreline segment. This is followed by separate chapters discussing environment, emergency response, monitoring and public information.

Preferred and Alternate Concepts

Preferred and alternate shoreline concepts were developed for each of the six study areas. The recommended preferred concepts are in accordance with the pending provincial policy on shoreline management. Prevention techniques were considered preferable to protection techniques where possible. Benefit cost analyses were carried our for each of the protection concepts considered. These analyses were performed in accordance with provincial guidelines for benefit cost analyses but relied on a limited data base. The results were used to evaluate the relative merits of the alternative concepts but more detailed estimates of the associated benefits should be conducted to support implementation.

The one common aspect of the preferred plan for each shoreline segment is the definition of a regulatory shoreland zone. The method of defining that zone varies, however, from segment to segment. The regulatory shoreland zone is one of the key elements of the pending provincial policy on shoreline management. It is defined as the landward most of:

a) a flood limit considered to be the 100 year uprush limit;
b) a 100 year erosion limit considered to be 100 times the average annual recession rate plus a stable slope allowance;
c) a dynamic beach limit considered to be the landward limit of the cohesionless beach deposit.

The preferred and alternate shoreline management concepts for each shoreline segment are described following.

High Bluff Shoreline

The preferred shoreline management concepts within the high bluff shoreline segments of the study area are for the Kettle Creek Conservation Authority to:

1. Establish a regulatory shoreland zone based on the erosion setback of 100 times the estimated annual recession rate plus a stable slope allowance.

2. Restrict land uses and associated development within the regulatory shoreland zone to agriculture and passive recreation, or any other uses as may be approved by the municipality and the conservation authority.

3. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

4. Encourage owners of dwellings at risk of damage through erosion of the bluffs to relocate their dwellings to a location outside the regulatory shoreland zone.

5. Encourage the Township of Southwold, The Village of Port Stanley and the Township of Yarmouth to enact bylaws which will enable them to require the relocation or demolition and removal of a non-inhabitable structure before it is undermined by bluff erosion. A structure is to be defined as non-inhabitable when it is in imminent danger of being undermined by bluff erosion.

The proposed limit of the regulatory shoreland zone is shown on the photomosaics contained in the appendices accompanying the report. This limit was established as 100 times the calculated average recession rate plus a stable slope allowance of 3.5 times the bluff height, measured from the first landward break in slope of the bluff.

A total of 52 structures are presently located within the proposed limit of the regulatory shoreland zone. These structures were inventoried during the field survey.

Shoreline protection structures were considered as alternate concepts but none were proposed as part of the preferred plan because of the high construction costs of the structures and the relatively low value of the land. Concepts considered included shore parallel revetments and the creation of headland bays by armouring presently developing hardpoints. None of the benefit cost ratios exceed 0.3 for these alternate concepts.

Port Stanley Reach 1, West of Stanley Park

The preferred shoreline management concepts for Reach 1 of Port Stanley are for the Kettle Creek Conservation Authority to:

1. Establish a regulatory shoreland zone based on the location of the dynamic beach zone.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

3. Implement a dune management program to promote the growth and stabilization of a dune system lakeward of the line of existing development.

4. Restrict new development until the dune management program (step 3 above) has been established and, once established, only allow development compatible with that program.

5. Encourage existing landowners to floodproof their dwellings to withstand the level of flooding associated with the 100 year wave uprush level.

Before any new development is allowed within the regulatory shoreland zone landowners should have to demonstrate that they have overcome both flooding and beach instability hazards, without causing or aggravating updrift or downdrift problems. This should be demonstrated within an impact statement.

Flooding problems associated with the 100 year uprush level should be overcome using dry passive floodproofing techniques. Different minimum allowable land elevations were proposed for instances where the property is either exposed to or sheltered from direct wave uprush. Properties located north of Edith Cavell Boulevard could be raised in elevation by placing cohesive fill. Properties south of Edith Cavell Boulevard should only use sand fill with characteristics similar to those of the existing beach sand.

Because of the low frequency of occurrence of major flood events only low cost flood prevention techniques were found to be viable. A revetted flood berm and a concrete retaining wall, considered as alternate concepts, had benefit cost ratios of 0.68 and 0.31, respectively. Floodproofing individual dwellings and maintaining existing dune protection had a benefit cost ratio of 1.71.

It was also proposed that a formal dune management program be implemented to improve the flood resistance capacity of the existing dune system. The program would include both land use controls and the use of vegetation. No specific guidelines were suggested as the Ministry of Natural Resources anticipates releasing a dune development guideline during the summer of 1990.

Port Stanley Reach 2, Public Beach

The preferred concepts for Reach 2 of Port Stanley are for the Kettle Creek Conservation Authority to:

1. Establish a regulatory shoreland zone based on the location of the dynamic beach zone.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

3. Encourage existing landowners to floodproof their dwellings to withstand the level of flooding associated with the 100 year wave uprush level.

This reach is similar to Port Stanley Reach 1 in that it was recommended that development only be allowed once the hazards have been demonstrated to have been overcome, that dry passive floodproofing be used and that only sand fill be allowed south of Edith Cavell Boulevard. For this reach the benefit cost ratio for floodproofing flood prone structures was calculated to be 4.22.

A dune management program was not included with the preferred concepts for this reach because of the high level of pedestrian traffic. This reach contains a heavily used public recreational beach and it was felt that restricted access, as needed for dune development, would not be realistic. It was noted however, that if a way of creating natural dunes without interfering with the public use of the beach could be found then it should be considered.

Port Stanley Reach 3, Harbour Area

No specific shoreline management plan concepts were developed for this reach because these lands are owned by the federal government. It was recommended, however, that the conservation authority establish a line of contact with Transport Canada and request that any federal plans consider the objectives of the authority's shoreline management plan.

There are two aspects of the federal lands which are important to shoreline management. The existing harbour breakwaters are holding the fillet beach in its present location. If those harbour structures were to be removed the entire fillet beach would eventually be lost. If the harbour breakwaters were to be shortened then the beach would degrade as the lakeward end of the breakwater is anchoring the toe of the fillet beach.

Secondly, environmentally hazardous substances are sometimes stored within the 100 year wave uprush limits. The potential for contamination during flood events needs to be identified and appropriate preventative measures should be adopted.

Port Stanley Reach 4, Little Beach

The preferred shoreline management concepts for Reach 4 of Port Stanley are for the Kettle Creek Conservation Authority to:

1. Establish a regulatory shoreland zone based on a 30 metre setback plus a stable slope allowance, in accordance with the pending provincial policy.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

3. Encourage landowners to upgrade and maintain their existing shoreline protection structures.

The regulatory shoreland zone should be established as 3 times the bluff height plus 30 metres, measured from the first landward break in slope of the bluff. No development should be allowed within the regulatory shoreland zone until it has been demonstrated that the potential erosion hazards have been overcome without impacting updrift or downdrift shorelines. This should be demonstrated through a prepared impact assessment.

Because of the narrow depth of the lots in this reach it is possible that the regulatory shoreland zone could cover portions of properties landward of those located on the shoreline. The owners of those properties will not be able to address shoreline protection issues. It is therefore recommended that the conservation authority consider some flexibility while developing their guidelines for applying the Shoreline Management Plan. Flexibility could be considered in two ways; by waiving the requirement for an impact statement for properties not on the shoreline and by assessing the development setback required on a property by property basis. The setback required will depend upon the effectiveness of the shoreline protection, maintenance access to the protection structure and the overtopping characteristics of the protection structure.

Alternate concepts mentioned were a continuous armourstone revetment and extending an existing concrete rubble headland to allow artificial beach fill. Costs associated with the headland/fill concept cannot be estimated until at least some preliminary design work is carried out. The benefit cost ratio of a new armourstone revetment was estimated to be only 0.25 whereas the benefit cost ratio for maintaining the existing structures was estimated to be 3.11.

Port Stanley Reach 5, Orchard Beach

The preferred shoreline management concepts for Reach 5 of Port Stanley are for the Kettle Creek Conservation Authority to:

1. Establish a regulatory shoreland zone based on a 30 metre setback plus a stable slope allowance, in accordance with the pending provincial policy.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

3. Encourage landowners to upgrade and maintain their existing shoreline protection structures.

As with Port Stanley Reach 4 it is recommended that the regulatory shoreland zone be defined in accordance with the pending provincial policy but that the conservation authority consider some flexibility when developing their implementation guidelines.

The construction of a new armourstone revetment was also considered for this reach but, as with Reach 4, it is not cost effective. The benefit cost ratio for a new revetment was estimated to be 0.12. The benefit cost ratio for maintaining the existing structures was estimated to be 1.47.

Environment

A review of potential environmental concerns was undertaken for the study area. Information regarding the local fish population, amphibians and reptiles, birds, mammals and plant life was obtained. No environmentally sensitive areas were defined but it was noted that a number of rare plant species are located along the eroding bluffs.

None of the preferred or alternate concepts will have an adverse environmental impact as long as only clean approved fill and constructive materials are used and construction activities are planned to avoid conflict with spawning fish.

Emergency Response

The only change to the existing emergence response plans proposed was for the conservation authority to undertake their own prediction of potential storm surge floods. This would be done by performing a detailed storm surge analysis for varying wind speeds and directions. The results of this study would be a table of setup heights for any given wind condition which could be added to the static water level to estimate flood water levels.

Monitoring

The proposed monitoring program included:

1. Updating bluff erosion rates using detailed digital mapping prepared now, and then every 10 years.

2. Monitoring downcutting of the nearshore profile in front of Reaches 4 and 5 of Port Stanley by repeating detailed bathymetric soundings every 5 years.

3. Including monitoring plans within the dune management program to be developed for Reach 1 of Port Stanley.

4. Surveying wave uprush and flood levels experienced during extreme storms.

5. Conducting an annual examination of the shoreline protection structures within Reaches 4 and 5 of Port Stanley.

Public Information

Public information was collected during preparation of the plan by distributing a questionnaire to all waterfront properties during the field survey and an information leaflet with a request for comments during a public open house meeting. This meeting was held to present the draft shoreline management concepts to the public.

It was also proposed that a program be undertaken to provide future information to the public. This would be done by distributing a bulletin each spring containing: a reminder of the existence of the plan and the conservation authority's role in administering it; a reminder of the need for approvals for shoreline work; water level forecasts; and updates of recent events pertaining to shorelines and shoreline management. It was also suggested that public meetings be held during high water level periods to provide information about the existing dangers and appropriate responses.

The public information section of the report also provides information about the approvals required before any work is undertaken within the regulatory shoreland zone. It is recommended that in order to obtain approvals an impact statement be prepared. That statement would address:

- Site Location
- Site Description including environmentally significant features
- Coastal Conditions (Design Parameters)
- Littoral Transport
- Description of Proposed Works
- Design Calculations
- Construction Schedule
- Maintenance Requirements and Access
- Impact on Littoral Transport and Environment

The impact assessment should also demonstrate three key items:

1. That there will be no increase in the long term recession rate on neighbouring properties caused by the proposed development.

2. That the proposed development will not cause damage to adjacent shoreline protection structures.

3. That the proposed development will in no way have any detrimental effects on the environment.

For the average homeowner who wishes to undertake a relatively modest project with no major issues having to be resolved an impact assessment would likely cost in the order of $2,000 to $3,000.

1.0 INTRODUCTION

A Shoreline Management Review Committee was appointed in April, 1986 to study long term management of the Great Lakes shorelines. Their report was submitted to the Ministers of Municipal Affairs and Natural Resources in October, 1986. In December of 1988, the Ministry of Natural Resources announced government actions in response to the Committee's recommendations. Amongst these actions was the designation of conservation authorities as the agency responsible for implementing and administering the Ministry's shoreline policies, including overseeing the preparation of shoreline management plans.

The Conservation Authorities and Water Management Branch of the Ontario Ministry of Natural Resources had concurrently developed guidelines for developing Great Lakes shoreline management plans. Their guidelines, published in August of 1987, identified the major goals of a shoreline management plan as:

- To minimize danger to life and property damage from flooding, erosion and associated hazards along the shoreline, and

- To ensure that shoreline development adequately addresses flooding and erosion hazards through a combination of public and private management and development alternatives."

The key components of a shoreline management plan were identified as:

Prevention	-	evaluation of and recommendations for controls and regulations governing new shoreline development
Protection	-	evaluation of and recommendations for potential capital works for existing and new development
Emergency Response	-	reviewing existing emergency response plans and recommending appropriate modifications
Public Information	-	recommendations for dissemination of information and education of the general public regarding the shoreline management plan
Environment	-	assess environmental impact of the shoreline management plan

Monitoring - identify methods to monitor changes to local conditions affecting shoreline management.

The Ministry of Natural Resources together with the Ministry of Municipal Affairs are currently preparing a Provincial Policy on Shoreline Management of the Great Lakes System. To date a second draft of this policy has been prepared and circulated for review to Senior MNR personnel and conservation authority general managers. This draft policy has not been publicly released and was therefore not provided to the consultants during the preparation of the plan. However, in order to best ensure that this plan complied with the pending provincial policy, 2 aspects of that policy were discussed with the consultant. These aspects were:

i) The plan should define a regulatory shoreland zone based on the greater of:

   a) flood limit considered to be the 100 year uprush limit
   b) a 100 year erosion limit considered to be 100 times the average annual recession rate plus a stable slope allowance
   c) a dynamic beach limit considered to be the landward limit of the unconsolidated beach deposit.

ii) New development within the regulatory shoreland zone should only be considered if the hazards being used to define that zone can be overcome using methods approved by the conservation authority without aggravating or creating updrift or downdrift shoreline problems.

The report is the final product of a study which involved a site investigation to review existing conditions and collect relevant data, generation of design data and criteria, development of shoreline protection and prevention concepts and a benefit cost analysis.

The study area, shown in Figure 1.1, covers the entire Lake Erie shoreline of the Kettle Creek Conservation Authority watershed. This watershed includes most but not quite all of the Lake Erie shoreline of the Township of Southwold and the Township of Yarmouth. The Village of Port Stanley is located between the two Townships.

The shoreline within the study area can be subdivided into three classifications described as following:

i) bluffs higher than 10 metres
ii) low bluffs up to 10 metres in height
iii) sand beaches

The shorelines of the two Townships and the western limit of the Village of Port Stanley are high bluffs. The low bluff and sand beach shorelines are found within Port Stanley. For this study the shoreline sections within Port Stanley were subdivided into a total of 5 reaches, based on the location and physical characteristics of the section of shoreline. These 5 reaches are shown in Figure 1.2. The criteria used to define these reaches are discussed in the appropriate sections of Chapter 6, Preferred and Alternate Shoreline Management Concepts.

The report has been structured to first review the data collected, outline the derivation of the various design criteria established for the study then provide an overview of the various prevention and protection techniques considered. This is followed by the preferred and alternate shoreline management concepts and the remaining elements of the plan; environment, emergency response, monitoring and public information.

The preferred and alternate shoreline prevention and protection concepts (Chapter 5) are presented first for the high bluff sections of the study area then on a reach by reach basis for the remainder of the Port Stanley area. Within each of these sections the report discusses existing conditions, shoreline management preferred concepts and the alternate concepts considered.

Throughout the report tables and figures referred to in the text are presented at the end of the section with which they are associated.

The main text of the report is also accompanied by a number of appendices bound under separate cover. The appendices directly associated with the plan include:

- existing conditions photomosaics
- Port Stanley preferred and alternate concepts photomosaics
- shoreline inventory
- oblique photographs
- public questionnaires

To assist readers with the terms and definitions used within this report a glossary has been provided. This glossary provides a brief description of technical terms and explanatory sketches of a number of the definitions and is located prior to the appendices.

It is important to note that the shoreline erosion and flood protection concepts discussed within this report cannot be used for actual construction purposes. Only design concepts are presented. Actual protective works must be properly designed by a professional coastal engineer. Site specific conditions, too detailed to be evaluated within the report, must be considered.

It must also be noted that the following guidelines presented within this report do not preclude the need for approvals and permits. Any landowner who wishes to construct shoreline works must contact the Kettle Creek Conservation Authority to determine which regulations must be followed and what permits are required. The conservation authority will be able to provide a listing of the permit which may be required, the agencies which require them and contact names and phone numbers. This is discussed further in Chapter 9, Public Information.

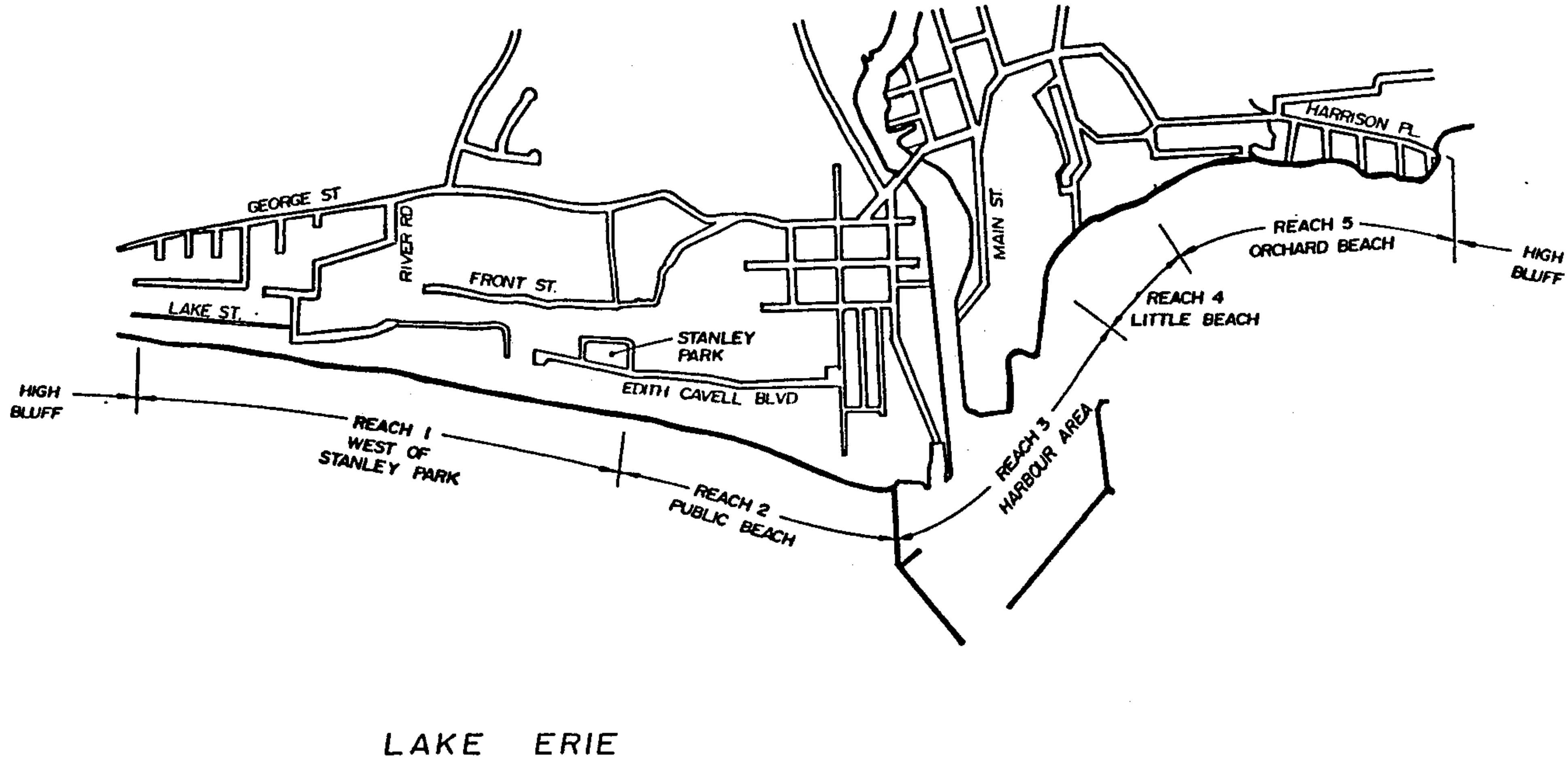


Figure 1.2
Port Stanley Shoreline Reaches

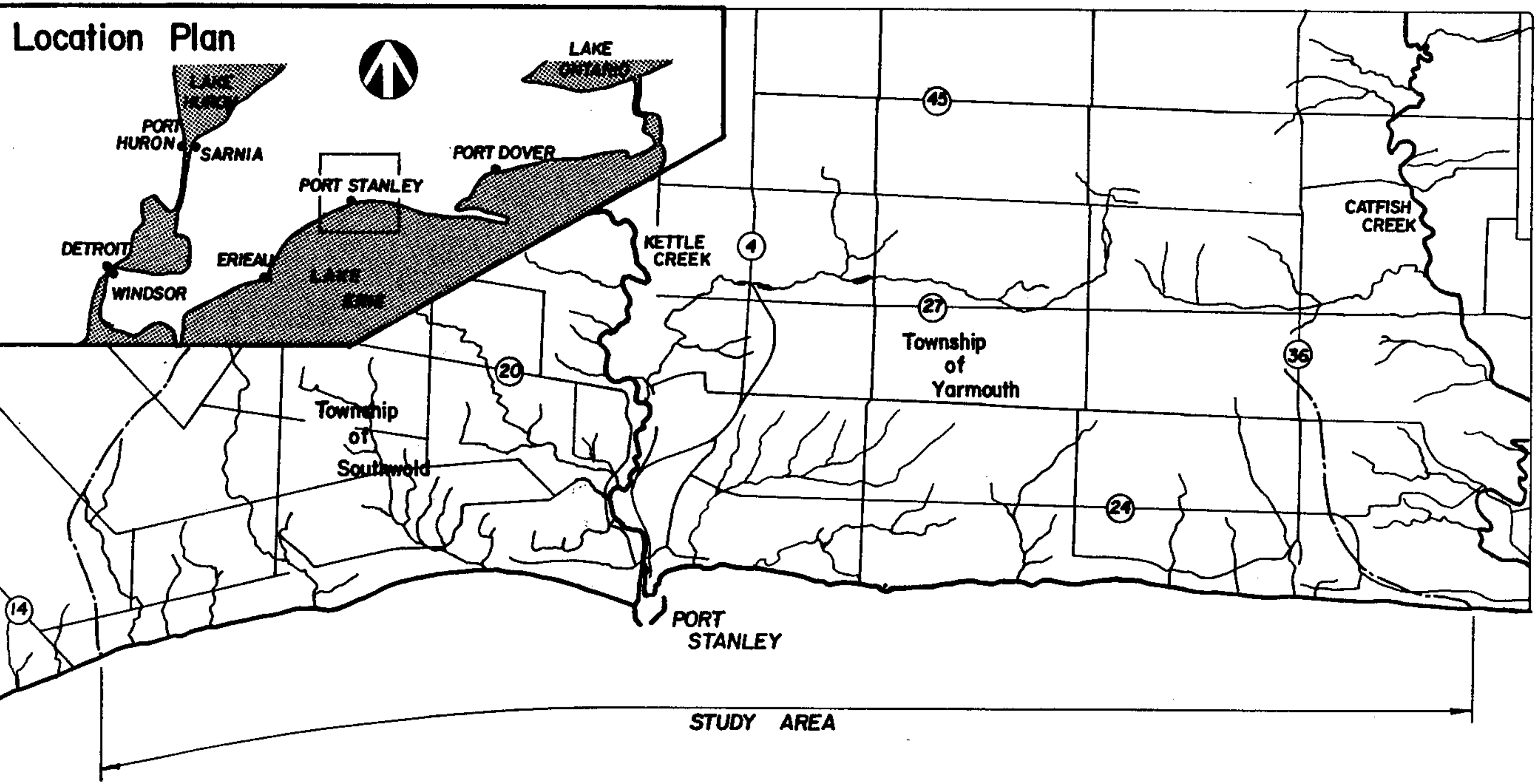


Figure 1.1
Site Plan

2.0 Data Collection and Review

This section of the report describes data collected for the study, identifies the data sources, and provides a brief comment on the utility of the data. A more detailed discussion of how the data was used is included in the appropriate sections of Chapter 3 of the report, Generation of Design Data and Design Criteria. This chapter covers:

- literature search
- official plan and zoning regulations
- resident questionnaire
- mapping and aerial photography
- environmental data
- bluff recession data
- sediment supply data
- field survey

Each of these items is separately covered in the following sections.

2.1 Literature Search

A literature search was conducted to identify and evaluate existing reports, papers and documents relevant to the shoreline management plan study. The literature study was started by reviewing the references listed within the terms of reference. These references included a list of background documents and most importantly, the Guidelines for Developing Great Lakes Shoreline Management Plans (MNR, 1987). These guidelines formed the basic framework under which this plan was prepared and, as such, were the most relevant information reviewed.

The list of background documents provided with the request for proposal included:

- revised Official Plans for the Municipalities of Port Stanley, Southwold and Yarmouth.

- Port Stanley Shoreline Protection Plan (Reinders 1986).

- Wave Hindcast Database for Ontario's Great Lakes (MNR, 1988a).

- Littoral Cell Definition and Sediment Budget for Ontario's Great Lakes (MNR, 1988b).

- Great Lakes System Flood Levels and Water Related Hazards (MNR, 1989).

- Aylmer District Land Use Guidelines (MNR, 1983).

The Port Stanley Shoreline Protection Plan (Reinders, 1986), prepared for the Kettle Creek Conservation Authority provided an overview of existing conditions for Port Stanley as well as some possible shoreline management concepts. The report was valuable as a bench mark against which to qualitatively compare the concepts derived within this study. An exact comparison of the various concepts was not carried out because of the variation between the water level and wave conditions used with the two studies. The Reinders (1986) Study used only estimates on design wave and water level conditions whereas this study utilizes the results of detailed analyses.

The Wave Hindcast Database for Ontario's Great Lakes (MNR, 1988a) was reviewed but was not used for this study because of uncertainty with the results. The wave hindcast analysis performed for this study is discussed in Section 3.3. of the report.

The Littoral Cell Definition and Sediment Budget for Ontario's Great Lakes (MNR, 1988b) was also reviewed but the results were not used because data from the Port Burwell litigation study was considered more detailed. The Port Burwell data is discussed in later within this section of the report.

Water level data, including estimates of extreme static water levels, storm surges and instantaneous water levels were obtained from the MNR (1989) report "Great Lakes System Flood Levels and Water Related Hazards". The data provided within that report was used without modification. The water levels used as design criteria are discussed in Section 3.5 of this report.

The Aylmer District Land Use Guidelines (MNR, 1983a) provide an overall design for integrated land use, including targetted levels of use for a number of activities, up to the year 2,000. These guidelines contain a number of general statements on public safety and environmental concerns within the same scope as those contained in the Shoreline Management Plan Guidelines (MNR, 1987). The shoreline of the study area was noted to contain both a significant earth science feature, the Port Stanley Till glacial deposits, and a significant life science feature, the remaining natural habitat along the bluff shoreline. Protecting significant life science and earth science features is a target of the land use guidelines. Overall these guidelines did not provide any specific information directly used within the shoreline management plan but provided a valuable overview of MNR's policies and plans for the area. These policies were considered when the preferred shoreline management plan was developed.

The most relevant technical information regarding coastal processes on the bluff shorelines was obtained from documents prepared for the Port Burwell shoreline erosion litigation defence. The Federal Government of Canada was sued for damages purported to be caused by the construction of the harbour breakwater at Port Burwell (Alton et al verses Her Majesty the Queen). In defence of this claim a detailed study was undertaken by a number of scientists, engineers and historians. In the end more than thirty five expert reports were produced from the five year study. The information contained within those reports forms the framework of the current knowledge of coastal processes on eroding cohesive shorelines. Specific data obtained from those reports are discussed later in this section of the report. A listing of the Port Burwell study reports related to shoreline processes is contained in the bibliography.

2.2 Official Plan and Zoning Regulations

Extracts of Official Plans and Zoning Regulations and By-Laws for the Township of Yarmouth, Township of Southwold and Village of Port Stanley were provided by the Central Elgin Planning Office. These documents were reviewed for specific information relating to the shoreline areas. Within Yarmouth and Southwold there are no specific by-laws dealing with shoreline protection or development. There has also been no formal adoption of the Fill, Construction and Alteration to Waterways Regulations along the Lake Erie shoreline within the Kettle Creek Conservation Authority's jurisdiction. There are, however, Official Plan policies which apply to the shoreline.

The Township of Yarmouth and Township of Southwold have separate Official Plans but the sections dealing with the Lake Erie shoreline are identical. Both Townships have defined the shoreline as Lake Erie Hazard Prone Area. The explicit policies dealing with this area are found in Section 9 of the Official Plans and have been extracted and presented in Appendix H of this plan.

The village of Port Stanley has prepared a draft new Official Plan but it has not yet been legally adopted. Relevant extracts from both the existing and draft Official Plans for Port Stanley have also been included in Appendix H.

2.3 Resident Questionnaires

2.3.1 Initial Questionnaire

During the summer field survey a questionnaire was hand delivered to all residents along the waterfronts. A total of approximately 180 questionnaires were delivered. Information was requested to assess the current public knowledge of shoreline management, and to obtain specific information on past flood and erosion problems and solutions. A copy of the questionnaire is included in Appendix E. The 71 received responses are bound under separate cover. These responses were divided into different categories, based on location and nature of problems, before the erosion and flood information was reviewed. The public knowledge information was considered irrespective of location. The results of the questionnaire are summarized following:

Public Knowledge

Eighty per cent of the 71 residents who returned questionnaires were aware that their property lies within the Kettle Creek Conservation Authority watershed but only fifty five per cent had been aware that the authority was the agency responsible for commenting on the use and development of the shoreline. Fifteen per cent of the people were aware that the authority was preparing a shoreline management plan.

Sixty five per cent of the residents indicated that they believed weather events caused the fluctuations in Lake Erie's water levels, twenty nine per cent cited government regulations effecting water levels and six per cent did not know what caused the fluctuations. A number of respondents did, however, cite more than one cause.

Sand Beach Shoreline

Thirty one of the 71 returned questionnaires (44%) were completed by landowners from the sand beach shoreline updrift of the harbour. Of those 31 a total of 18 (58%) indicated that they had experienced flooding problems in the past. Of the 42% cent who indicated that they have not had flooding problems, one half had moved there since 1985, when the last major flood event occurred. It is therefore possible that as little as 16% of the properties have not been flooded in recent years.

Two main concerns were noted by the people in this area; the volume of refuse ending up on their beach originating from people updrift who are either trying to protect their bluffs or are simply dumping over the bluffs and the flooding of land because neighbours have flattened their dunes.

Low Bluff Shoreline

Twelve of the 71 responses were from the low bluff shoreline area downdrift of Port Stanley harbour. Each of those twelve people indicated that they had experienced erosion problems in the past. The costs of correcting those problems varied from almost negligible to typically $10,000 to $20,000, although 1 individual spent $50,000. A number of people indicated that they had constructed lesser than full design structures because of the cost of those structures.

Some instances of dwelling flooding were noted but that flooding was attributed to overland runoff of rainwater and not to high lake levels.

High Bluffs Outside Port Stanley

Twenty eight of the 71 responses were from the high bluff areas of the shoreline. Of those twenty eight only two residents indicated that they had no erosion problems. One of those two had a stable bluff and the other did not indicate why he had no problem. One respondent did not answer the erosion information questions as he had just purchased the property and did not know the history of any problems.

Of the 25 respondents who indicated past erosion, 6 related the problems to high water levels and/or wave action, 5 to high ground water flow from the bluff and the remaining 14 noted both causes. Seven people tried to halt or slow the erosion by dumping fill or brush over the bluff. Four people planted vegetation and 5 installed some form of drainage system. Seven people noted that they had not tried anything because the options were too expensive.

2.3.2 Public Meeting

A public meeting was held November 2, 1989 to present the alternate shoreline management concepts. A summary of the project, including a request for comments, was distributed and is provided in Appendix E. Although approximately 80 people attended the meeting, only 4 comment sheets were returned. These are also included in Appendix E.

2.4 Mapping and Aerial Photography

Unrectified 1:2,000 and 1:5,000 scale base mapping for showing both the shoreline management plan concepts and existing information were produced from aerial photographs. The photographs were taken in late 1988 as part of the Canada/Ontario Flood Damage Reduction Program.

Bottom bathymetry data used in the wave refraction analysis was taken from the Canadian Hydrographic Service field sheets 8032 and 8048.

Land contours, spot elevations and road elevation information within Port Stanley were obtained from the conservation authority's 1985 flood risk mapping for Port Stanley.

2.5 Environmental Data

A brief review of potential environmental concerns was undertaken for the study area. Information was generally obtained from existing reports provided by MNR and the conservation authority as well as from discussions with MNR staff biologists. The information obtained is summarized in Section 6, Environment. A listing of the reports reviewed is included in the Bibliography.

2.6 Bluff Recession Data

Bluff recession data was obtained from the Port Burwell litigation study (Fleming, 1983a). Average annual recession rates were estimated at 100 m intervals for two periods; 1896 to 1936/37 and 1936 to 1968/71/75. The exact periods covered depended on specific locations. The recession rates were computed from three sets of continuous historic survey data, as described by Kolberg (1983). Several other collections of plans, charts and survey notes including shoreline data were examined and coordinated by Kolberg (1983) but none was found to be sufficiently complete or accurate for shoreline comparison.

Because of the accuracy of the continuous survey data and the close spacing of the points where the bluff recession was measured, this data set may be considered to be of the best quality and accuracy.

The data from Fleming (1983a) was used to produce the erosion setbacks used in the prevention component of the plan. The development of the setbacks is discussed in Section 3.7. The recession data from Fleming (1983a) is reproduced in Appendix G. The data coverage periods are also shown in Appendix G.

2.7 Sediment Supply Data

As part of the Port Burwell study Philpott (1983a) estimated the volumes of sediment supplied to the nearshore zone from four sources; bluff erosion, gully erosion, nearshore bottom erosion and watershed erosion. Bluff erosion, followed by nearshore bottom erosion were found to contribute the greatest volumes of sediment, contributing 70 and 29 per cent of the total sediment volume, respectively. Gully erosion and watershed erosion (sediment loading from creeks and drains) were found to be of minor importance, each contributing less than 1 per cent of the total.

All grain sizes entering the nearshore zone were grouped into one of four size categories; shingle, littoral drift, sub-littoral drift and washload. The grain size categories were determined on the basis of the behaviour of that size of material once it enters the nearshore zone. Figure 2.1 relates these categories to standardized sediment size classifications.

Bluff erosion volumes were estimated by combining bluff composition data from Zeman (1980) with the recession rates estimated by Fleming (1983a).

The stratigraphic cross-sections provided by Zeman were subdivided into 250 m long reaches of shoreline. For each reach the face areas allocated by Zeman to particular sediment samples were determined. These areas were then subdivided proportional to the grain size fractions given in the sample texture statistics from Zeman's report. The areas associated with each grain size fraction were then summed to form totals for each 250 m reach. The results were then multiplied by the averaged 1937 to to 1975 recession rates obtained from Fleming (1983a) to obtain volumetric rates of sediment input for each 250 m reach.

Finally, to provide a consistent level of data resolution, the bluff sediment yield rates for the 250 m reaches were grouped into the same reaches as used to define nearshore sediment data (Philpott, 1983a).

Estimates of volumes from gully erosion were determined by adjusting gully erosion to the procedure for determining erosion volumes for the bluff. The textural data from Zeman (1980) was first interpolated across the mouths of the gullies shown in his stratigraphic cross sections. The cross sectional area of each gully mouth was then subdivided in proportion to grain size fractions and multiplied by the appropriate recession rate from Fleming (1983a). These results were then grouped in the shoreline segments derived from the nearshore data.

Watershed erosion volumes were obtained from a report prepared for the International Joint Commission (Wall et. al., 1978). This report lists computed annual average sediment flows for individual watersheds. The annual rates for each area were related to the appropriate shoreline reach.

Nearshore bottom erosion volume estimates were obtained from the comparison of nearshore profiles (Philpott, 1983a). Particle size distribution was obtained from bottom core samples compiled by Lewis (1983).

Tables 2.1 to 2.4 show the computed sediment supply volumes from bluff erosion, nearshore bottom erosion, watershed erosion and gully erosion. The baseline used in these tables corresponds to the baseline used in the Port Burwell Study. This baseline is shown on the existing conditions photomosaics in Appendix A.

2.8 Field Survey

In July of 1989 the consultant visited the study area to review the entire site and to inventory existing protection structures. This inventory was later expanded to include all development along the high bluffs, a review of a number of the larger gullies including probable causes for the gullying and the identification of natural and artificial drainage systems. A questionnaire was also distributed to each shorefront dwelling within the study area. The questionnaire and returned responses are discussed in Section 2.3 of the report.

At the beginning of the field study the entire site was reviewed by boat and photographed. The photographs are presented under separate cover in Appendix D. Beach profiles were measured at 3 locations, Orchard Beach, the main beach just updrift of the breakwater and the main beach at the foot of Lake Street. The locations of these profiles are shown on the photomosaics in Appendix A. The measured profiles are shown in Figure 2.3.

Each shoreline property within Port Stanley was inventoried during the initial field survey. Information was gathered by filling in a standardized inventory sheet, as shown in Figure 2.2. This sheet was designed to minimize the amount of subjective review by requiring a number of categorized responses. The shoreline type and structure type categories were copied from the 1975 Canada Ontario Great Lakes Shore Damage Survey Coastal Zone Atlas to allow present conditions to be compared to the 1975 conditions.

Within the fillet beach portion of Port Stanley the natural dunes were considered to be shoreline protection. The dune crest elevation and setback were therefore recorded with the structure details information at the bottom of the inventory sheet. The inventory sheets from Port Stanley are presented under separate cover in Appendix C. The crest elevation of each protection structure, including the beach dunes, are shown on the existing conditions photomosaics in Appendix A.

Structures along the high bluff, located within the 100 year erosion limit, were also inventoried. Setbacks to the crest of the bluff were measured. An erosion factor for each dwelling was determined by subtracting the bluff stable slope allowance from the setback then dividing that value by the estimated annual recession rate. The erosion factor therefore represents the number of years until that dwelling is located within the stable slope allowance. It does not suggest that the dwelling is at a specified risk. A dwelling considered to have an "imminent" erosion factor is presently located within the stable slope allowance defined herein as 3.5 times the bluff height, measured from the top of the bluff. This definition is based on the pending provincial flood and erosion policy statement and is explained in the glossary.

Table 2.5 shows the results of the high bluff structure inventory. Each of the structures inventoried are shown on the existing conditions photomosaics in Appendix A. The structures have been numbered consecutively from west to east. The first 2 columns of Table 2.5 show the photomosaic number and inventory number, respectively. None of the structures inventoried had been protected by shoreline protection structures.

A further investigation was conducted to inventory large gullies, probable causes for the formation of those gullies and both natural and artificial drainage systems. The drainage systems are shown on the existing conditions photomosaics in Appendix A. The results of the gully inventory are presented in Table 2.6. As with the high bluff structures, each inventoried gully has been numbered consecutively from west to east and is shown on the existing conditions photomosaics in Appendix A. The first two columns of Table 2.6 show the photomosaic number and inventory number, respectively.

Table 2.1 Input From Bluff Erosion (cubic metres per year)

Chainage	Shingle	Littoral Drift	Sublittoral Drift
8.20	316	1240	1750
9.78	422	2012	2327
11.74	780	2883	3081
13.73	1207	2385	2848
16.35	2102	1953	2818
18.23	708	1500	1679
19.65	991	1686	1922
21.65	21	382	506
23.20	0	654	838
24.57	7	78	1084
25.88	80	367	1714
26.97	22	891	3441
29.60	895	4578	16884
36.66	0	2341	10445
43.15	84	927	4498
48.97	407	2854	4496
55.51	0	915	8586
61.21	0	264	4943
65.37	367	1029	9012
67.19	0	7937	74018
71.44	0	12306	68869
75.20	620	10158	22966
77.97	3	2795	13520
79.25	2156	4331	6405

Table 2.2 Input From Nearshore Bottom Erosion (cubic metres per year)

Chainage	Shingle	Littoral Drift	Sublittoral Drift
8.20	0	0	0
9.78	0	0	0
11.74	0	0	0
13.73	0	0	0
16.35	0	0	0
18.23	0	0	0
19.65	0	0	0
21.65	0	0	0
23.20	0	0	0
24.57	0	0	0
25.88	0	0	0
26.97	0	0	1576
29.60	0	0	3807
36.66	0	0	2988
43.15	0	0	903
48.97	0	0	655
55.51	0	0	1598
61.21	0	0	463
65.37	0	0	1364
67.19	0	0	1952
71.44	0	0	1386
75.20	0	0	1662
77.97	0	0	960
79.25	0	0	759

Table 2.3 Input From Gully Erosion (cubic metres per year)

Chainage	Shingle	Littoral Drift	Sublittoral Drift
8.20	0	0	0
9.78	0	0	0
11.74	0	15	18
13.73	8	12	14
16.35	0	0	0
18.23	18	48	51
19.65	0	0	0
21.65	0	0	0
23.20	0	7	24
24.57	0	0	0
25.88	0	0	0
26.97	0	0	0
29.60	0	102	750
36.66	0	0	3
43.15	0	7	57
48.97	0	84	173
55.51	0	289	1132
61.21	0	0	0
65.37	0	0	0
67.19	0	58	1741
71.44	0	83	880
75.20	0	52	1681
77.97	0	0	0
79.25	0	0	0

Table 2.4 Input From Watershed Erosion (cubic metres per year)

Chainage	Shingle	Littoral Drift	Sublittoral Drift
8.20	0	0	41
9.78	0	0	50
11.74	0	0	51
13.73	0	0	67
16.35	0	0	48
18.23	0	0	36
19.65	0	0	51
21.65	0	0	40
23.20	0	0	35
24.57	0	0	34
25.88	0	0	28
26.97	0	0	68
29.60	0	0	181
36.66	0	0	17
43.15	0	0	167
48.97	0	0	525
55.51	0	0	0
61.21	0	0	852
65.37	0	0	852
67.19	0	0	0
71.44	0	0	0
75.20	0	0	0
77.97	0	0	0
79.25	0	0	621

Table 2.5

High Bluff Stucture Inventory

Map Number	Inventory Number	Description of Structures	Setback (m)	Recession Rate (m/yr)	Erosion Factor (yr)
1	1	1 storey brick house	140	1.3	imminent
2	2	barn & 2 silos	260	1.33	90
2	3	2 storey brick house	200	1.37	44
2	4	2 storey brick house	160	1.37	15
2	5	house under construction	90	1.39	imminent
2	6	1 storey frame house	100	1.39	imminent
2	7	1 storey frame house	20	1.39	imminent
2	8	1 storey frame house	120	1.39	imminent
2	9	1 storey frame house & barn	120	1.39	imminent
2	10	1 storey frame house & barn	75	1.39	imminent
2	11	1 storey frame house & barn	20	1.42	imminent
2	12	1 storey frame house & barn	20	1.45	imminent
2	13	1 storey frame house & barn	95	1.5	imminent
2	14	split level frame house	35	1.5	imminent
2	15	1 storey frame cottage	30	1.5	imminent
2	16	1 storey frame house	110	1.35	imminent
2	17	2 storey frame house	60	1.19	imminent
2	18	1 storey frame house	20	1.08	imminent
2	19	1 storey frame house	15	0.97	imminent
2	20	1 storey frame house	23	0.86	imminent
2	21	1 storey frame house	20	0.86	imminent
2	22	1 storey frame house	30	0.75	imminent
2	23	1 storey frame house	20	0.65	imminent
2	24	1 storey frame house	50	0.54	imminent
2	25	1 storey frame house	40	0.54	imminent
2	26	1 storey frame house	15	0.43	imminent
2	27	1 storey frame house	10	0.43	imminent
2	28	1 storey frame house	20	0.32	imminent
2	29	2 storey brick house	140	0.32	imminent
3	30	2 storey frame house	10	4.10	imminent
3	31	2 storey frame house	30	4.10	imminent
3	32	1 storey frame house	70	4.10	imminent
3	33	1 storey frame house	100	4.10	imminent
3	34	1 storey frame house	100	4.10	imminent
3	35	1 storey frame house	40	4.10	imminent
3	36	2 storey frame house	90	4.10	imminent
4	37	1 storey frame house	220	2.33	34
4	38	barn	175	2.33	15
4	39	6 storage sheds/barn	290	2.33	64
5	40	farm-3 barns, 2 silos,2 storey house	220	1.46	55
5	41	1 storey frame house	150	1.37	7
5	42	2 story frame house	130	1.38	imminent

Table 2.5 (cont.)

High Bluff Stucture Inventory

Map Number	Inventory Number	Description of Structures	Setback (m)	Recession Rate (m/yr)	Erosion Factor (yr)
5	43	2 storey frame house & barn	160	1.25	16
5	44	1 storey frame house & poultry farm (7 barns.storage sheds)	110	1.44	imminent
5	45	1 storey frame house & garage	60	1.34	imminent
6	46	1 storey frame house	260	1.3	92
6	47	1 storey frame house	60	1.31	imminent
6	48	2 storey frame house	70	1.45	imminent
6	49	1 storey frame house & large storage shed	30	1.45	imminent
6	50	1 storey frame house, large storage shed & greenhouses	190	1.41	35
6	51	1 storey frame house & 2 greenhouses	60	1.67	imminent
6	52	1 storey frame house, 2 green-houses and 8 storage sheds	50	1.81	imminent

Table 2.6 Gully Inventory

Map Number	Inventory Number	Gully (1) Shape	Length (2) (m)	Slope (3) Vegetation	Water (4) Source
1	G1	V	50	N	G,S,D
1	G2	V	40	N	G,S,D
4	G3	U	90	U-F	G
4	G4	V	400+	F	G,S,D
4	G5	V	70	U-F	G
5	G6	V	70	W-F, E-U-P	G,D
5	G7	V	300	W-F, E-P	G,D
5	G8	V	100	P	G,S,D
5	G9	U	200	P	G,D
5	G10	V	70	P	G,S,D
5	G11	V	80	U-F	G,S
5	G12	V	800+	W-P, E-F	G,S,D
5	G13	V	35	U-F	G,S
6	G14	U	60	U-P	G,S,D
6	G15	V	90	U-F	G,S,D

Notes:

(1) Gully shapes are either V or U. A V shape gully is actively forming. A U shape gully has been formed for some time and is widening at the bottom.

(2) Gully length is taken from the shoreline to the head of the gully at the table land elevation. Very long gully lengths were not measured to the head and are indicated by a "+" after the distance.

(3) Slope vegetation:
N = no vegetation
F = full vegetation
P = partially vegetated
E - applies to east side of gully
W - applies to west side of gully
U - applies to upper part of slope only

(4) Water Sources:
G = groundwater
S = surface runoff or open ditch
D = drainage pipe or tile

	from U.S. Army (1977)				
from Philpott (1983a) used this study	Wentworth Scale (Size Description)	Phi Units ϕ *	Grain Diameter, D (mm)	U.S. Std Sieve Size	Unified Soil Classification (USC)
	Boulder				
		-8	256		Cobble
Shingle	Cobble		76.2	3"	
		-6	64.0		Coarse Gravel
			19.0	3/4"	
	Pebble		4.76	No. 4	Fine Gravel
		-2	4.0		Coarse Sand
	Granule				
		-1	2.0	No. 10	
	Sand – Very Coarse	0	1.0		Medium Sand
Littoral Drift	Sand – Coarse	1	0.5		
	Sand – Medium		0.42	No. 40	
		2	0.25		Fine Sand
	Sand – Fine	3	0.125		
Sub-littoral Drift	Sand – Very Fine		0.074	No. 200	
		4	0.0625		Silt or Clay
	Silt	8	0.00391		
Washload	Clay	12	0.00024		
	Colloid				

* $\phi = -\log_2 D$ (mm)

Figure 2.1

Sediment Particle Siz[e] Classifications

KETTLE CREEK CONSERVATION AUTHORITY
LAKE ERIE SHORELINE INVENTORY, 1989

Field personnel________ Date__________ Time______ Water Level__________IGLD

DWELLING

\# storeys____ colour______________ brick/stone/wood/aluminum deck yes/no
roof: flat/steep/average roof colour_____________ garage/shed/pool
Ownership: Private/Public Name ______________________________________
Land Use: Residential/Institutional/Commercial/Recreatinal/Industrial
Street # _____________________________ Lot____ Plan_____ Map____________

Frontage: protection ________m property ___________m
Setbacks: Dwelling ________m from top bluff/structure/water line
Top of Bluff __________m from structure/water line
Structure __________m from water line

SHORELINE (choose ONE only)

- BEACH: Bars and Spits/Beach or Dune Complex/Wetland or Marsh
- BLUFF (>3M): Glacial Drift >10m/Glacial Drift <10m/Bedrock/Artificial Fill
- LOW PLAIN: Glacial Drift/Bedrock/Artificial Fill Rock Outcrop Yes/No

BEACH / BEACH DEPOSIT AT TOE OF BLUFF OR LOW PLAIN

	slope	width	material size	thickness
nearshore	_______		Fine/Coarse/Pebble/Cobble	0 - 15 - 30 > cm.
foreshore	_______	_______	Fine/Coarse/Pebble/Cobble	0 - 15 - 30 > cm.
backshore	_______	_______	Fine/Coarse/Pebble/Cobble	0 - 15 - 30 > cm.

BLUFF: Wet/Dry/Seeping/Drained Natural/Regraded
height_______m width_______m vegetation cover 0 - 1/4 - 1/2 - 3/4 - full
material: Sand/Till/Sand-Till/Sand-Till-Tallus/Other___________________
lower slope_________ middle slope_________ upper slope__________
lower thickness________m middle thick.________m upper thick.________m

LOW PLAIN: bank height________m bank width_______m slope________________
Cover: None/Sand/Pebble/Cobble thickness 0 - 15 - 30 > cm. width_________m

STRUCTURE (choose ONE only)
Seawall/Bulkhead/Gabion/Retaining Wall/Groyne/Jetty/Breakwater/Revetment/
Bank Stabilization /Beach Replenishment/Dyke

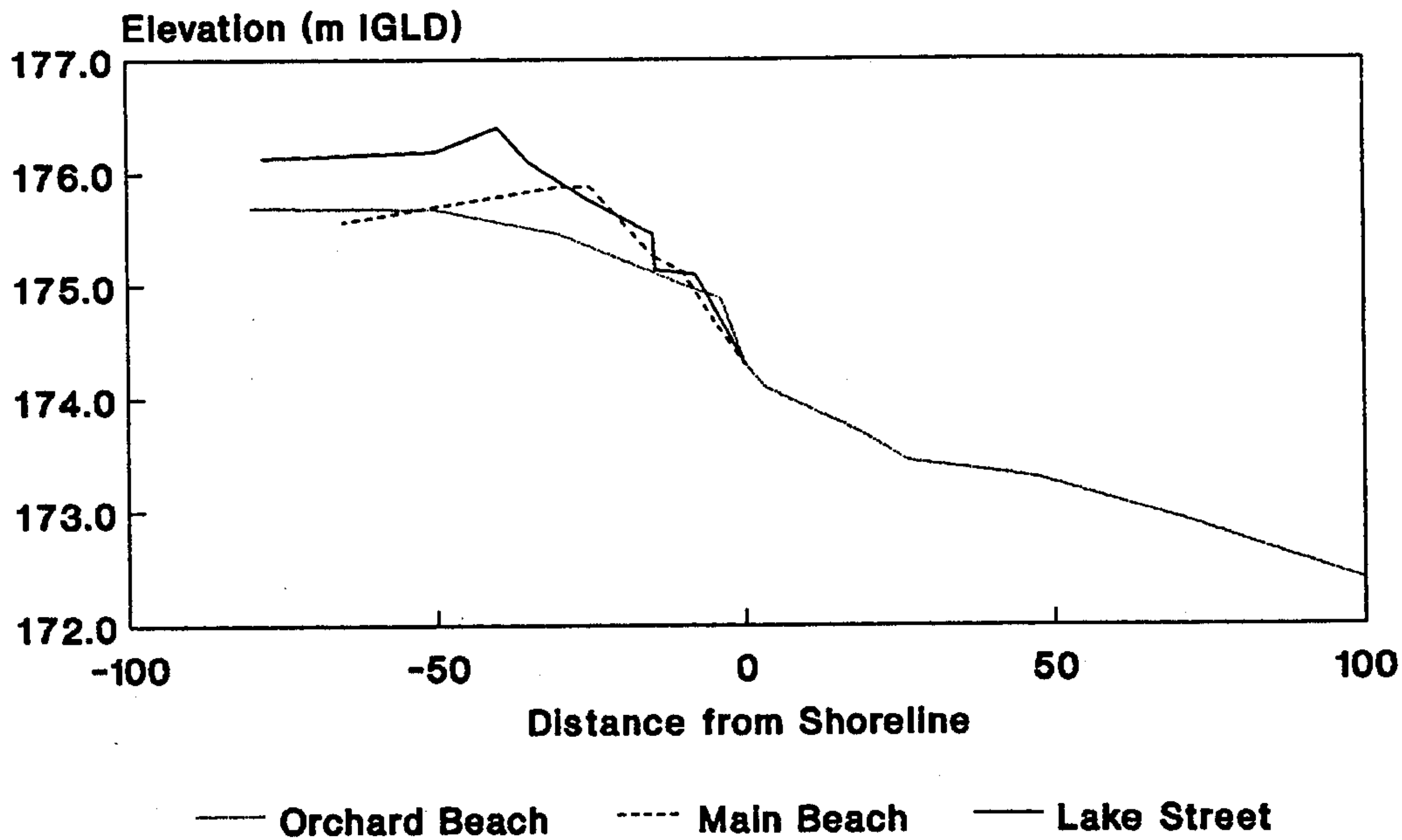
Material:	armourstone	rip rap	rubble	field stone	concrete block
size:	____________	____________	____________	____________	____________
size:	____________	____________	____________	____________	____________
Material:	**poured concrete**	**steel pile**	**timber pile**	**timber planks/ties**	
size:	____________	____________	____________	____________	
size:	____________	____________	____________	____________	
Material:	**grouted bags**	**used tires**	**other**		
size:	____________	____________	____________		
size:	____________	____________	____________		

Filter: definite no/probable no/probable yes/definite yes/cannot tell
Flank Protection: yes/no length______m same type/other__________________
Splash pad: height_____m elevation_____m width______m type_______________
Toe scour protection Yes/No size____________________________ width_________
Toe: height _______m elevation _______IGLD
Crest: height _______m elevation ______IGLD flood freeboard______________m

COMMENTS ____
____

Figure 2.2
Shoreline Inventory Sheet

KCCA SHORELINE MANAGEMENT PLAN
Measured Beach Profiles



profiles surveyed July 12, 1989
water level 174.30 m IGLD

Figure 2.3

3.0 GENERATION OF DESIGN DATA AND DESIGN CRITERIA

This section presents the design data and design criteria developed for the Shoreline Management Plan.

3.1 Summary

The design data and design criteria developed within this Section are summarized following and defined in the glossary.

100 year breaking significant wave height	2.8 m
100 year static water level	175.0 m GSC
mean annual high water level	174.8 m GSC
100 year storm surge	0.96 m
100 year flood level	175.5 m GSC
100 year uprush elevation on beach dune	177.0 m GSC
100 year uprush elevation landward of dune	176.8 m GSC
100 year uprush elevation on a sloped structure	178.7 m GSC
100 year uprush elevation on a vertical wall ...	181.4 m GSC
100 year dune recession width	5 m
100 year wave active beach width	40 m
minimum allowable structural opening freeboard .	0.3 m
minimum allowable dyke freeboard	0.3 m
minimum allowable structural setback	based on regulatory shoreland zone

3.2 Design Water Levels

Design static (still) water levels, storm surge levels and maximum water levels (flood levels) were provided by the Conservation Authorities and Water Management Branch of the Ontario Ministry of Natural Resources (MNR, 1989). Their estimates for Port Stanley were:

	10 year return period	100 year return period
Static water level	174.72 m GSC	175.00 m GSC
Storm surge	0.63 m	0.96 m
Peak water level	175.18 m GSC	175.52 m GSC

These values were estimated by performing a statistical regression of measured water level data using the Conservation Authorities and Water Management Branch in house computer program HYDSTAT (MNR, 1989). GSC refers to Geodetic Survey of Canada datum.

The static water level estimates were obtained by considering annual maximum mean water levels from 1900 to 1987, after modifying those levels to account for the effects of any water level regulation schemes. The modifications were based on a standard approach that has been utilized for some time by both Environment Canada and the U.S. Army Corps of Engineers. Basically, historic water levels are put through a hydrological routing model to produce water levels equivalent to those which would be expected to have occurred considering present day diversions and regulation plans.

The long-term estimates of storm surge levels were based on actual surge events recorded at Port Stanley. The actual storm surges were extracted from the recorded instantaneous water levels by the Inland Waters Directorate, Environment Canada, using the following procedure:

1. the difference between maximum recorded and mean monthly water level was computed for each month from January 1967 to December 1986.

2. for the largest monthly difference of each year the actual "calm" water level (rather than monthly mean) prior to the storm surge event was computed so that the actual surge level could be determined.

3. the actual surge levels from 2 above were checked against other close monthly values from 1 above to ensure that the largest surge was considered after the calm water level had been determined.

4. a HYDSTAT regression was performed on the computed surge levels.

The estimates of 10 and 100 year return period flood levels were obtained by performing a multi-variate probability analysis of the static water levels and the storm surge levels. This was also done using the program HYDSTAT. The 10 year results are included here to show the sensitivity of design water level to return period. the mean annual high water level, which is the average of the highest yearly instantaneous water level has an associated return period of 2 years.

3.3 Wave Hindcasts

Fifteen years of hourly significant wave height, peak wave period and mean wave direction were computed at a location offshore of Port Stanley. This data was generated with a wave hindcast program which considers lake bottom effects on wave growth. The model used to generate the wave data for the MNR Lake Erie wave hindcast database (MNR, 1988a) did not consider bottom effects and it was felt that both the predicted wave heights and the predicted wave periods were too high.

A hindcast calibration analysis was conducted by hindcasting to sites offshore of Point Pelee and Port Colborne where wave data had been measured and comparing the measured and hindcast wave heights and periods.

Measured wind data from Windsor, Sarnia and Simcoe as well as factored wind data provided from the MNR hindcast database files were evaluated. The Sarnia wind data produced the best calibration results. Figure 3.1 shows a sample of the hindcast calibration results for wave data measured off of Point Pelee. There are a number of occurrences of both overpredictions and underpredictions of the measured wave data but the hindcast results are within acceptable standards for estimating wave uprush elevations.

Once the hindcast procedure had been calibrated wave data was generated for a site offshore of Port Stanley. The results of this hindcast are summarized as a set of wave statistics presented in Figures 3.2 to 3.5.

3.4 Wave Refraction Analysis

Wave data generated by hindcasting (Section 3.3) was transferred to the shoreline by applying the results of a wave refraction analysis. This analysis was conducted in two steps. First, wave rays were tracked from an inshore point of interest, or node, to the offshore boundary. The ray tracks were computed by assuming linear waves propagating over a digitized numerical grid representing the lake bottom bathymetry. These wave ray paths were then used to transfer energy from offshore directional wave spectra to the nearshore point of interest. By transferring a number of representative sea states a series of wave height factors and wave direction shifts applicable to all offshore wave conditions were determined.

This wave transformation process is based on the energy within a spectrum and not individual waves. The representative wave heights used to describe the offshore and nearshore spectra are defined as four times the variance of the wave energy within the spectra and are referred to as the zero moment wave height. At the offshore boundary the zero moment wave height may be assumed to be equal to the significant wave height, defined as the average of the highest one third of the individual waves comprising the wave spectrum. At the nearshore node there may be a difference between the zero moment and significant wave heights.

Refraction analyses were performed at 2 nearshore nodes, as shown in Figure 3.6. These two nodes were selected to define the wave climate on either side of the harbour breakwaters. It was concluded that, for the purpose of this study, the wave data generated at node 1 could be assumed to apply throughout the study area, except in the immediate lee of the harbour.

The bottom bathymetry for the area was represented by a total of 8 wave refraction grids with varying grid spacing. The grids covered the area from Patrick Point to just west of Port Burwell and lakeward to a depth of 15 metres. The entire gridded area, shown on Figure 3.6 covered more than 1,000 square kilometres.

Although individual wave ray paths are directly related to water depth, the overall wave transformation process is not overly sensitive to changes in water level. If the nearshore profile were planar, then changes in water level would cause a lateral shift in the location of the breaker line but not the magnitude of the breaking wave heights. Because the nearshore profile is relatively straight, it was concluded that only one wave refraction analysis would be required to represent the range of possible water levels. A water level of 174.2 m GSC was used.

Figures 3.7 to 3.10 show the nearshore wave height exceedence and wave energy distributions at node 1 and node 2. These figures show that there is virtually no difference in wave heights at the two locations but there is a small change in wave direction. The sheltering effects of the harbour pier are not significant at the location of the nearshore nodes (bottom elevation 170.2 m GSC). The effects of sheltering would be more noticeable landward of the wave refraction node locations.

Because non linear energy losses occur during the wave refraction process, a long term statistical distribution as done offshore cannot be performed with the nearshore wave heights. Extreme nearshore wave heights are predicted by estimating extreme offshore wave heights and refracting them inshore.

Table 3.1 shows the predicted nearshore wave conditions for node 1. These values may be considered to be applicable to the entire study area.

3.5 Wave Uprush

Wave uprush levels were determined to define flood elevations behind the Port Stanley fillet beach and to establish design crest elevations for protection structures. Because of the location of the study area, the design wave conditions will occur at the same time as the design wind setup conditions. Design uprush elevations were therefore determined by computing the wave uprush levels associated with 1 in 100 year nearshore wave conditions and adding those heights to the 1 in 100 year design instantaneous water level. These design uprush levels may also be assumed to have a 100 year return period.

Uprush levels for beaches and sloped structures were computed by first applying a number of wave uprush formulas then selecting an "average" result considered to be applicable under the given circumstances. This approach was adopted because of the range of uprush levels predicted by the different methods as well as the difference in conditions under which the specific methods best apply. A total of 6 methods were considered, 5 of which were computational and 1 which was based on a graphical analysis. The 5 computational methods considered were based on the work of Ahrens and Titus (1985), Losada and Gimenez-Curto (1980), Pilarczyk and den Boer (1983), Kobayashi and Reece (1983) and Ogawa and Shuto (1984). The graphical analysis used the procedure presented by the Shore Protection Manual (CERC, 1984).

Wave uprush on beaches was calculated by averaging the results of the Pilarczyk and den Boer (1983) and the Ogawa and Shuto (1984) models. Uprush on structures was calculated by averaging the results of the other 3 computational models. Predicted uprush levels for both beaches and structures were then compared to the results predicted by the Shore Protection Manual graphical method to check that the computed results were reasonable.

Wave uprush levels for vertical walls were estimated by using the graphical methods presented in the shore protection manual. For these cases, breaking wave heights were estimated to be equal to the water depth at the toe of the wall. None of the computational runup analysis methods used for beaches and sloped walls are applicable for vertical walls.

3.5.1 Uprush on a Beach

For the beach area updrift of Port Stanley two cases must be considered; waves which run up the face of the fronting row of dunes, and waves which overtop the dune and flood into the essentially flat backshore area. The 100 year uprush level for the dunes was estimated to be 1.5 m, giving a design wave uprush elevation of 177.0 m GSC. This value was estimated using the method discussed in Section 3.5 and by assuming that the front face of the dune would have an overall composite slope of 1 to 15. This slope is slightly flatter than would be expected for a less severe storm and was determined during the cross shore transport analysis (Section 3.6.2).

This wave uprush elevation was qualitatively checked by comparison to field results. This was done by estimating the wave uprush levels which would have occurred during the December 2, 1985 storm and comparing those estimates to the flooding information provided in response to the public questionnaire (Section 2.4.1).

The December 2 storm was selected as it is believed to be the most commonly referred to storm of the last few years. It was therefore assumed that reported flood events would have occurred during this storm. The results of the comparison are shown in Table 3.2.

The estimated dune freeboard was calculated by subtracting the dune crest elevation measured during the field survey (Section 2.8) from the predicted wave uprush level for the December 2, 1985 storm (176.7 m GSC). The freeboard therefore represents the depth to which water would have passed over the dune crest. A negative freeboard indicates that the dune would not have been overtopped.

From Table 3.2 it may be seen that the differences between the reported flood depths and estimated dune freeboard vary, but the most common differences are 0.3 or 0.4 metres. This means that if the land behind the dunes was 0.4 m lower than the dune crest, this comparison would suggest that the wave uprush estimates and flood depths were similar. The land elevations were not surveyed throughout the area, but based on the profiles that were measured, this is not an unreasonable assumption.

This comparison cannot support any definite conclusions because there are too few points of comparison and too long a period between when the wave uprush occurred and when the dunes were surveyed. The comparison does, however, qualitatively suggest that the uprush estimates are reasonable.

It may be seen from the shoreline inventory notes and mapping (Section 2.8) that much of the dune crest elevation in Reach 1 of Port Stanley is below the design uprush elevation of 177.0 m GSC. Therefore, during a design storm at the design high water level, much of the dune will be overtopped. This means that waves will not be running up a slope similar to that assumed in the wave uprush analysis but over a composite slope which is flatter overall. This in turn means that the overall uprush elevation will be lower than 177.0 m GSC. The exact elevation which would be expected is difficult to assess because the land behind the dune crest is extremely flat. For much of the area, an elevation of 177.0 m is only met along the bluff north of Edith Cavell Boulevard. Typically used wave runup equations do not apply to such a flat backshore area.

One means of estimating the extent of flooding to be expected inland of the dunes is to consider the highest expected lake level. This assumes that the energy from the uprushing waves has dissipated over the flooded area and does not increase the water level. The highest level that could be considered would therefore be the 100 year instantaneous water level plus an allowance for a wave setup level. For the nearshore design conditions described in section 3.4, the setup associated with the 1 in 100 year wave would be 0.5 m. This would suggest that the highest expected flood levels well landward of the wave uprush zone would be 176.0 m, GSC.

There will, therefore, be a transition zone between the dune line and the extent of the flooded area where the flood level will be between 176.0 and 177.0 m. This transition area will extend from the dune line inland to where the uprushing waves have dissipated. The expected uprush level anywhere within this transition zone would be related to the composite slope from the water's edge to that point. It is not possible, within the general context of this plan, to establish an accurate uprush elevation which could be considered applicable throughout this area. This is due to both the variation in uprush levels predicted by the various models considered as well as a lack of knowledge of how the uprush levels will be used. A detailed analysis of a specific project could consider the range of predictions and the sensitivity of that range with respect to that project.

Because one of the objectives of the shoreline management plan is to identify areas where potential problems could occur, a relatively conservative estimate of the uprush level should be adopted. This level could be updated in the future as the methods of predicting uprush levels are better defined. As the currently used 100 year uprush level of 176.8 m GSC is at the upper end of the range of levels discussed above (ie. 176.0 to 177.0), we recommend that it also be used for this shoreline management plan.

3.5.2 Uprush on a Structure

Uprush levels on a structure are directly related to the height of the waves which reach the structure. Those wave heights are in turn related to the water depth at the toe of the structure.

Typical uprush elevations were calculated for three structure toe elevations; 171.5, 174.2 and 175.0 m GSC. Elevation 171.5 m is estimated to be the lowest that a toe should have to be placed to account for long-term erosion of the nearshore bottom. Toe elevation 174.2 was selected as representative of the existing structures in Reach 5 of Port Stanley. Elevation 175.0 m is the 1:100 year design static water level for Lake Erie and was selected to represent the toe elevation of structures placed on a beach beyond the static high water level.

Two types of structures were considered; vertical bulkheads and sloped revetments. Revetments may be either concrete or armour stone with a front slope between 1:1.75 and 1:2 (vertical to horizontal). The results of the uprush analyses are shown in Table 3.3.

3.6 Sediment Transport

Both alongshore and cross shore sediment transport analyses were carried out to evaluate the nearshore sediment regime. Cross shore transport is important as this mechanism both feeds dunes through the slow but continuous shoreward transport of sand and quickly removes that sand during storm events. These two elements of cross shore transport were examined separately, as discussed in Section 3.6.2.

Alongshore transport rates were estimated to see what role alongshore transport might have in the various shoreline management issues considered. These rates are discussed in Section 3.6.1.

3.6.1 Alongshore Sediment Transport

Alongshore sediment transport rates are supply limited throughout the study area, with the possible exception of the fillet beach immediately to the west of Port Stanley harbour. The supply of sediment to the nearshore zone is less than that which would be transported by the available wave energy. When this is the case, alongshore transport rates are estimated through a sediment budget, an accounting of the sediment sources and sinks within the nearshore zone.

For this plan a littoral sediment budget was synthesized from the sediment budget developed by Philpott (1983a). Philpott's sediment budget was derived from the erosion and bluff composition data collected during the Port Burwell Study as described in Sections 2.6 and 2.7 of this report. The purpose of this budget was to account for the littoral material, that is the portion of the eroded bluff which would be transported in the alongshore direction. Fine material transported offshore to deeper water was considered a sink to the littoral budget and the fate of that material was not evaluated.

For a sediment budget the shoreline is divided into a number of segments or reaches and the sediment sources and sinks of each segment are determined. As discussed in section 2.7, sediment supply volumes for varying grain size ranges were quantified from four sources; bluff erosion, nearshore bottom erosion, gully erosion and the watercourse supply of inland soils. Sinks considered were offshore losses of fine material, the comminution of soft sand grains, which were ultimately also lost offshore, and the increase in volume of the nearshore sediment deposit. A decrease in the nearshore sediment deposit volume would be a source. The volumetric differences between these sources and sinks is assumed to be transported alongshore. The net alongshore sediment transport at any point is found by summing the alongshore transport rates from all shoreline segments updrift of that point. Figure 3.11 shows the sources, sinks and inferred transport rates for a typical shoreline segment.

The source volumes for the littoral budget are presented in Section 2.7. The shoreline segments or reaches used to define the sediment budget were taken from Philpott (1983a) and were based on the location of available nearshore sediment deposit data. The locations of these shoreline segments are shown in Figure 3.12a. Philpott (1983b) showed that with existing data it was not possible to quantitatively assess the rate of change of the nearshore deposits. It was therefore concluded that the nearshore deposit change in the sediment budget (S in Figure 3.11) would have to be taken as zero.

The rate at which bluff material is lost offshore is based on the grain size of that material. As discussed in Section 2.7, four grain size ranges were considered; shingle, littoral drift, sub-littoral drift and washload. Philpott (1983a) suggested that for a littoral sediment budget, no shingle is lost offshore and all washload may be considered to be lost offshore. Approximately 10 per cent of littoral material is lost through comminution of soft grains and some unknown portion of sub-littoral drift is lost offshore. For this study it was assumed that fifty per cent of the sub littoral material would be lost offshore and fifty percent would be transported alongshore. This is an arbitrary division and based more on supposition than a specific interpretation of any specific data. It was selected as a "median" value to attempt to minimize the error associated with the estimate. It must be noted, however, that sub-littoral material was defined by Philpott (1983a) as that material found mainly below the level of active wave induced littoral transport and has not been shown to have a major role in shoreline management planning. This suggests that errors associated with the estimated percentage of sublittoral material bypassing the harbour are not significant.

Finally, to complete the sediment budget, the rate of bypassing of the Port Stanley harbour breakwater must be estimated. This is needed to estimate the updrift sediment supply rate input into the shoreline segment to the east of the harbour, segment 4 in Figure 3.12a. Cumming-Cockburn and Associates Limited (CCL, 1987) estimated that an average of 15,000 cubic metres per year of fine sand is deposited in the harbour approach channel but noted that there was no evidence that this material was bypassing the channel. As the harbour entrance is currently maintained through dredging, it was assumed that no sediment bypasses the breakwaters.

Based on the above noted points, a sediment budget was developed as presented in Figure 3.12. Two rates of alongshore sediment transport are shown; the littoral transport rate in Figure 3.12b which was computed by summing the inputs of all shingle material and 90 per cent of the littoral material, and the sub-littoral transport rate shown in Figure 3.12C which was computed by adding 50 per cent of the sub-littoral material to the littoral transport rates shown in Figure 3.12b.

The sediment budget, depicted in Figure 3.12, was also placed on a computer spreadsheet (on Lotus 123, Release 2.01) and provided to the conservation authority. This will allow the conservation authority to update the budget as either more detailed information becomes available or as conditions change either within the study area or updrift.

Table 3.5 shows the "results" sections of the sediment budget spread sheet. This section sums the volumetric inputs from the various sources considered. the baseline chainage used corresponds to the baseline shown on the existing conditions photomosaics presented in Appendix A. The baseline both east and west of chainage 0+00 located at the Yarmouth/Southwold Township line.

As either development of the shoreline continues or perhaps more detailed sediment budget analyses are performed the results of this sediment budget may change. In order to update this budget to reflect the new data information will likely have to be shared with the adjacent conservation authorities. Any new data obtained from the Lower Thames Valley Conservation Authority will effect the input values to this sediment budget. The results of this budget and any future changes should be provided to the Catfish Creek Conservation Authority for use in their sediment budget.

3.6.2 Cross Shore Transport

Potential cross shore sediment transport rates were estimated for the fillet beach updrift of the Port Stanley Harbour piers to determine the potential for dune erosion during severe storms and a probable recovery time for that eroded dune. These rates were computed using a numerical model which independently considers wave height transformation through the surf zone, surf zone kinematics and sediment dynamics (HRL, 1989). The numerical model used the nearshore wave data described in Section 3.4 and beach profile data measured during the field study.

Because of its computational setup, this model is capable of predicting wave decay and sediment transport phenomena both over multiple barred profiles and during storm surges. It must be noted, however, that the underlying physics of cross shore transport are still not well understood. The results of this model are considered to be state of the art, and therefore the best possible estimates, but they should not be viewed as absolutes.

Because of the shape of the beach, extreme high water levels are not believed to cause more beach erosion than lower water levels. The analysis indicated that the 1 in 100 year storm would transport about 10 percent more sand offshore if it occurred during the 1:10 year water level than if it occurred during the 1:100 year water level. This is because the 1:10 year water level would allow direct wave attack on the dune face whereas the 1:100 year water levels would cause storm waves to overtop the dune.

It was estimated that a 1 in 10 year storm would remove from the upper portion of the dune approximately 3 to 4 cubic metres of sand per metre width of beach. The 1 in 100 year storm would likely remove up to 5 cubic metres per metre width. As long as the dune is not overtopped the eroded material would be transported offshore. If the dune is overtopped the volume of material eroded would be less but a portion would be washed landward. Sediment moved offshore will be transported back onto the dune during more quiescent wave conditions but material lost landward will not naturally find its way back into the dune.

It is difficult to give an accurate assessment of the width of the active beach zone because that width is ultimately determined by the frequency with which successive severe storms occur.

During a 1 in 100 year storm we would expect a vertical change in the profile to occur over 30 metres, plus or minus 10 metres. A distance of 40 metres was therefore adopted as the 1 in 100 year "wave active beach width". In other words, up to 40 metres of beach above the instantaneous water level could be effected by wave action during a 1 in 100 year storm. The greatest horizontal recession at any point of the original profile would, however, only be in the order of 5 metres. The greatest recession occurs near the water line and recession would not be expected to be significant further than 20 metres from the instantaneous water level at the time of the storm.

Using on the average annual wave statistics it was estimated with the cross shore transport program that less than 10 cubic metres per year of sand transported offshore of the dune would be recovered over a 1 year period. Only about 1 cubic metre per year of sediment originating from littoral drift would be transported into the dune system in a given year.

3.7 Recession Rates and Erosion Setbacks

The potential recession width of the fillet beach updrift of Port Stanley Harbour from one 100 year return period design storm was estimated in Section 3.6.2 as 5 metres. This recession can occur over 40 metres of beach, landward of the still water level.

Estimates of the average annual recession rates for the high bluff shoreline sections of the study area were obtained by averaging the recession data reported by Fleming (1983a) as described in Section 2.5 of this report and as presented in Appendix G. Fleming's recession rate data was estimated at 100 metre intervals over the entire high bluff shoreline of the study area but did not consider either the low bluffs at Orchard Beach, (reaches 4 and 5 of Port Stanley) or the fillet beach updrift of the harbour piers.

The bluff recession data was mainly used to determine the development control setbacks described in general in Section 4.4 of this report and applied in the relevant sections of Chapter 5, Preferred and Alternate Shoreline Management Concepts. Because the recession data was being used to define a regulated erosion area the data had to be averaged to produce a smooth curve. The raw data, described by Fleming (1983a), displayed both temporal and spatial variations although the temporal variation was not as extreme as the spatial variation. Because of the objective of the study for which that data had been derived it was necessary to smooth the spatial variations of the recession data in order to evaluate the temporal data. This smoothing was done by performing a rolling mean averaging where the averaged recession rate at any specific point was the average of the rates over a given distance either side of that specific point. Averaging lengths of 1, 2, 3, 4 and 5 kilometers were examined. It was concluded in the Port Burwell study that averaging over 5 km, that is 2.5 km either side of the specific point being considered, was required to smooth the spatial variations to a degree that the temporal variations could be evaluated.

Figures 3.13 and 3.14 show samples of the unsmoothed and 5 km smoothed data from Fleming (1983a). Each of these figures shows the average annual recession rates from the two periods considered, 1896 to 1934 and 1936 to 1975, for a section of shoreline east of Port Stanley. The abscissa of figures 3.13 and 3.14 has chainage 0.0 km at the Port Stanley Harbour breakwater.

It may be seen from Figure 3.14 that when the spatial variations in the recession data have been smoothed that the average recession rates from 1936 to 1975 were higher than the average rates from 1896 to 1937. During the Port Burwell study it was shown that the higher recession rates from the period 1936 to 1975 corresponded to an increasing water level trend (Fleming, 1983b). Because current water levels are in the range of the 1936 to 1975 water levels and not the 1896 to 1936 levels it was concluded that the 1936 to 1975 recession data would be used for this study.

Although Fleming used 5 km averaging to smooth out the spatial trends in the recession data for the Port Burwell Study, it did not follow that 5 km averaging would have to be used for this study. The 1, 2, 3 and 4 km wide averaging results from Fleming were therefore examined, and it was concluded that 1 km averaging smoothed the data sufficiently. Figure 3.15 shows the 1 km smoothed data for the same location as Figures 3.13 and 3.14. This smoothing, however, was carried out without consideration of any potential causes for the spatial variations in the recession rates.

The recession rate data was therefore re-examined to see if there was any noticeable correlation between the rates and the bluff composition. The recession rates were plotted at a large scale and visually compared to mapping based on the bluff stratigraphy data prepared by Zeman (1980) and discussed in Section 2.7. That comparison showed that it was possible to identify locally high recession rates associated with gullying but it was not possible to correlate changes in bluff stratigraphy with the spatial variation in the recession rates. Some qualitative similarities existed but it was not possible to definitely conclude that any given section of shoreline should have a locally higher or lower recession rate because of the local stratigraphy.

A final data set of estimated annual average recession rates was derived by averaging the raw recession data from Fleming (1983a) for the period 1936 to 1975, excluding the individual data points which could be shown to be locally high due to gullying. These data are representative of the overall bluff recession rates, without being biased by the occurrence of gullies. Due to the nature of the data used, these recession rates should be considered to be amongst the most accurate long term rates available on the Great Lakes. Table 3.4 shows the data.

The baseline chainages shown in Table 3.4 correspond to the baseline used by Fleming (1983a). The relationship between Fleming's baseline and the one used in this study is shown on the existing conditions photomosaics presented in Appendix A.

The revised data set was then used to develop setbacks from the bluffs for the prevention component of the shoreline management plan. Following the proposed procedure for the pending provincial policy, the erosion limit was calculated as 100 times the estimated annual erosion rate plus a stable slope allowance. The stable slope allowance was taken at 3.5 times the bluff height, measured landward of the top of the bluff. The 100 year recession limit is also shown in Table 3.4

3.8 Flood Depth Criteria

Wherever there is flooding, there is a risk to life and a risk of property damage when water depths are too deep or flow velocities are too high. In order to reduce these risks to acceptable levels maximum allowable flood depths should be established. In order to derive these depths we reviewed the flood criteria presented in MNR and MMA (1988). Concepts presented in that document considered relevant to this study include:

- a product of depth and velocity less than or equal to 0.4 m2/s defines a low risk hazard providing that the depth does not exceed 0.8 m and the velocity does not exceed 1.7 m/s.

- ingress and egress from a floodproofed area by most "typical" automobiles will be halted by flood depths above 0.3 - 0.5 m.

- 0.8 m depth is the safe upper limit for floodproofing the above ground/superstructure of conventional brick, brick veneer and concrete block buildings using closures and seals.

- floodproofing by elevation on fill becomes complex at flood depths beyond about 1.8 - 2.4 m.

- floodproofing design should be carried out by a professional engineer skilled in floodproofing measures:

    . where any wet floodproofing is proposed (irrespective of any other circumstances);

. where dry floodproofing is proposed and:

  . where the product of flood depth and velocity is equal to or greater than 0.4 m2/s or where flood depth exceeds 0.8 m or where flood velocity exceeds 1.7 m/s;

  . where floodproofing through the use of fill exceeds depths of 1.8 m or velocities between 0.8 - 1.5 m/s depending on soil type, vegetation cover and slope;

  . where berms and floodwalls in excess of 1 m in height are proposed.

  . where piles, columns and posts and proposed.

When flooding is due to wave uprush it is possible to estimate potential flow velocities based on the water depth. This is done by assuming that uprushing waves propagate with a celerity equal to the square root of the product of the water depth and acceleration due to gravity. Using this relation it is possible to relate the above mentioned criteria to flood depth alone. Thus a flood velocity of 1.7 m/s may be represented by a flood depth of 0.3 m and a product of depth and velocity equal to 0.4 m2/s may be represented by a flood depth of 0.25 m. If it is assumed, therefore, that flooded areas will experience the full potential flow velocity, then the maximum allowable flood depth will be 0.25 m below the 100 year flood level. This provides safe conditions with respect to each of the instances considered by MNR and MMA (1988).

Based on the above we therefore propose that the maximum allowable flood depth be considered to be 0.25 m unless site specific studies are undertaken to demonstrate that flow velocities will be lower than assumed herein. Such studies would therefore have to demonstrate that the area under consideration <u>cannot</u> be exposed to direct wave uprush but can only be flooded via some indirect path. If that can be proven then the more liberal flood depth criteria proposed by MNR and MMA (1988) could be adopted; ie the flood depth could be up to 0.8 m as long as the flood velocity does not exceed 1.7 m/s and the product of that flood depth and velocity does not exceed 0.4 m2/S.

These initial recommendations should be reviewed during the preparation of the guidelines for applying the Shoreline Management Plans.

Table 3.1 Summary of Extreme Wave Heights Calculated for Nearshore Node 1

Return Period (years)	Hs off[1] (m)	Hmo in[2] (m)	Hs in[3] (m)
1	3.8	1.7	2.1
10	4.4	2.0	2.5
20	4.6	2.1	2.6
50	4.8	2.2	2.7
100	4.9	2.2	2.8

1) Offshore significant wave height from Weibull analysis
2) Nearshore zero moment wave height from refraction analysis
3) Theoretical maximum nearshore significant wave height.

Table 3.2 Comparison of Flood Levels

Address on Edith Cavell Blvd.	(A) Reported Flood Depth on Property (m)	(B) Estimated Dune Freeboard (m)	Difference (A-B) (m)
394	0.6	0.2	0.4
426	0.6	0.3	0.3
476	0.4	0.1	0.3
496	0.3	-0.6	0.9
506	0.2	-0.1	0.3
509	0.6	0.2	0.4
549	0.3	0.2	0.1
559	0.6	0.0	0.6

Table 3.3 1:100 Year Uprush on Structures

	Toe Elevation (m GSC)	171.5	174.1	175.0
	Design Wave Height[1] (m)	3.2	1.6	1.0
Vertical Bulkhead	Uprush Level (m)	5.9	3.3	2.4
	Uprush Elevation[2] (m GSC)	181.4	178.8	177.9
Sloped Revetment	Uprush Level (m)	5.1	2.8	2.1
	Uprush Elevation[2] (m GSC)	180.6	178.3	177.6

Notes: 1) design wave height is based on water depth
2) 1:100 year uprush elevation equals 1:100 year instantaneous water level (175.5 m) plus uprush level.

Table 3.4

Bluff Erosion Rates and Setbacks

Baseline Chainage (Km)	Measured Recession Rate (m/yr)	1 Km Avg. Recession Rate (m/yr)	100 Year Setback from Top of Bluff * (m)	Location
55.190	1.23	1.25	265	- Dunwich/Southwold Township Line
55.286	1.19	1.23	263	
55.383	1.19	1.22	262	
55.479	1.16	1.22	262	
55.575	1.45	1.20	260	
55.671	1.42	1.20	260	
55.767	1.26	1.20	260	
55.863	1.32	1.20	260	
55.959	1.26	1.20	260	
56.055	0.65	1.20	260	
56.151	1.10	1.14	254	
56.247	1.15	1.14	254	
56.344	1.21	1.12	252	
56.441	1.26	1.13	253	
56.538	1.13	1.14	254	
56.635	0.77	1.23	263	
56.829	1.48	1.28	268	
56.926	1.03	1.28	268	
57.023	1.35	1.29	269	
57.120	1.39	1.30	270	
57.217	1.68	1.35	275	
57.314	1.68	1.42	282	
57.412	1.10	1.40	280	
57.606	1.35	1.41	281	
57.703	1.39	1.39	279	
57.800	1.58	1.40	280	
57.897	1.55	1.37	277	
57.994	1.29	1.34	274	
58.091	1.16	1.33	273	Southwold Township
58.188	1.16	1.31	271	
58.289	1.45	1.27	267	
58.390	1.39	1.21	261	
58.490	1.32	1.14	254	
58.591	1.03	1.12	252	
58.692	1.06	1.11	251	
58.793	1.03	1.08	248	
58.894	0.87	1.01	241	
58.994	0.77	0.97	237	
59.095	1.10	0.99	239	
59.398	1.03	1.06	246	
59.498	0.87	1.06	246	
59.599	0.65	1.09	249	

* 100 year setback includes stable slope allowance

Table 3.4 (cont.)

Bluff Erosion Rates and Setbacks

Baseline Chainage (Km)	Measured Recession Rate (m/yr)	1 Km Avg. Recession Rate (m/yr)	100 Year Setback from ~~Top~~ Toe of Bluff * (m)	Location
59.700	0.94	1.12	252	
59.801	1.58	1.16	256	
59.902	1.74	1.19	259	
60.002	1.13	1.21	261	
60.103	1.26	1.24	264	
60.204	1.23	1.30	270	275
60.297	1.26	1.35	275	
60.390	1.45	1.33	273	
60.484	1.19	1.31	271	
60.577	1.23	1.33	273	Southwold
60.670	1.29	1.35	275	Township
60.763	1.52	1.34	274	
60.857	1.32	1.33	273	
60.950	1.52	1.32	272	
61.043	1.35	1.34	274	
61.136	1.45	1.35	275	
61.230	1.16	1.37	277	
61.323	1.19	1.39	279	
61.416	1.32	1.42	282	
61.509	1.35	1.39	279	→ Grand Canyon Area
61.603	1.39	1.39	279	
61.696	1.55	1.45	285	
61.789	1.71	1.49	289	
62.169	1.61	1.51	291	
62.269	1.21	1.21	261	
64.640				- Southwold/Yarmouth
66.160	4.19	4.19	559	Township Line
66.259	4.45	4.17	557	
66.358	3.87	4.09	549	
66.358	3.87	4.02	542	
66.457	4.06	3.80	520	
66.556	4.13	3.59	499	
66.655	3.58	3.44	484	
66.754	3.23	3.32	472	
66.853	2.81	3.23	463	
66.952	2.45	3.13	453	
67.051	2.81	3.01	441	
67.150	2.61	2.86	426	Yarmouth
67.249	3.06	2.77	417	Township
67.348	2.90	2.71	411	
67.447	2.74	2.71	411	
67.546	2.74	2.68	408	
67.645	2.52	2.60	400	

* 100 year setback includes stable slope allowance

Table 3.4 (cont.)

Bluff Erosion Rates and Setbacks

Baseline Chainage (Km)	Measured Recession Rate (m/yr)	1 Km Avg. Recession Rate (m/yr)	100 Year Setback from Top of Bluff * (m)	Location
67.744	2.58	2.52	392	
67.843	2.58	2.39	379	Yarmouth
67.942	2.77	2.30	370	Township
68.041	2.16	2.18	358	
68.140	1.97	2.05	345	
68.240	1.74	1.97	337	
68.339	1.58	1.90	330	
68.537	1.90	1.84	324	
68.637	1.39	1.75	315	
68.736	1.39	1.70	310	
68.835	1.61	1.68	308	- Elgin Area Water
68.934	1.77	1.72	312	Treatment Plant
69.034	1.97	1.75	315	
69.133	1.74	1.67	307	
69.232	1.68	1.66	306	
69.331	1.71	1.68	308	
69.431	2.13	1.72	312	
69.530	1.97	1.76	316	
69.629	1.00	1.76	316	
69.728	1.29	1.83	323	
69.828	1.61	1.88	328	
69.927	2.03	1.93	333	
70.027	2.19	1.91	331	
70.127	2.06	1.92	332	
70.228	2.42	2.03	343	
70.328	2.29	2.10	350	
70.428	2.19	2.15	355	
70.528	1.94	2.17	357	
70.628	2.13	2.15	355	
70.729	2.16	2.17	357	
70.829	2.10	2.19	359	
71.029	2.16	2.22	362	
71.129	2.23	2.27	367	
8 km → 71.229	2.00	2.36	376	
71.330	2.29	2.38	378	
71.430	2.58	2.41	381	
71.530	2.68	2.40	380	
71.630	2.74	2.41	381	
71.730	2.94	2.40	380	Yarmouth
71.831	2.26	2.40	380	Township
71.931	2.55	2.33	373	
7 kms 72.230	1.97	2.25	365	

* 100 year setback includes stable slope allowance

Table 3.4 (cont.)

Bluff Erosion Rates and Setbacks

	Baseline Chainage (Km)	Measured Recession Rate (m/yr)	1 Km Avg. Recession Rate (m/yr)	100 Year Setback from Top of Bluff * (m)		Location
	72.329	2.29	2.14	354		
	72.429	2.08	2.03	343		
	72.528	1.97	1.87	327		
	72.628	1.61	1.77	317		
duplicate	72.727	1.63	1.65	305		
data	72.827	1.55	1.57	297		
	72.926	1.45	1.48	288	Duplicate	
	73.026	1.16	1.44	284	Data	
	73.126	1.21	1.39	279		
	73.225	1.26	1.38	278		
	73.325	1.05	1.36	276		
	73.424	1.34	1.32	272		
	73.623	1.63	1.34	274		
	73.723	1.37	1.33	273		
	72.329	2.29	2.14	354		
	72.429	2.08	2.03	343		
	72.528	1.97	1.87	327		
	72.628	1.61	1.77	317		
	72.727	1.63	1.65	305		
	72.827	1.55	1.57	297		
	72.926	1.45	1.48	288		
	73.026	1.16	1.44	284		Yarmouth
	73.126	1.21	1.39	279		Township
6km	73.225	1.26	1.38	278		
	73.325	1.05	1.36	276		
	73.424	1.34	1.32	272		
	73.623	1.63	1.34	274		
	73.723	1.37	1.33	273		
	73.822	1.55	1.36	276		
	73.922	1.42	1.37	277		
	74.022	1.11	1.39	279		
	74.123	1.63	1.38	278		
5km	74.223	1.03	1.34	274		
	74.324	1.55	1.33	273		
	74.424	1.39	1.30	270		
	74.525	1.29	1.28	268		
	74.625	1.24	1.27	267		
	74.826	1.11	1.22	262		
	74.926	1.34	1.24	264		
	75.127	1.16	1.25	265		
4km	75.228	1.18	1.26	266		
	75.328	1.03	1.29	269		

* 100 year setback includes stable slope allowance

Table 3.4 (cont.)

Bluff Erosion Rates and Setbacks

Baseline Chainage (Km)	Measured Recession Rate (m/yr)	1 Km Avg. Recession Rate (m/yr)	100 Year Setback from Top of Bluff * (m)	Location
75.429	1.08	1.30	270	
75.529	1.26	1.34	274	
75.730	1.66	1.34	274	
75.831	1.50	1.35	275	
75.931	1.66	1.35	275	Harding Smith Line 270m + 55
76.031	1.29	1.40	280	
76.130	1.58	1.44	284	
3km – 76.230	1.32	1.46	286	
76.330	1.29	1.43	283	Yarmouth
76.430	1.18	1.40	280	Township
76.529	1.53	1.35	275	
76.629	1.58	1.33	273	
76.729	1.50	1.31	271	
76.829	1.32	1.30	270	
76.928	1.13	1.32	272	
77.028	1.13	1.34	274	
77.128	1.11	1.33	273	
2km – 77.228	1.34	1.31	271	
77.327	1.24	1.30	270	
77.427	1.45	1.30	270	
77.527	1.42	1.32	272	
77.627	1.37	1.37	277	
77.726	1.42	1.43	283	
77.826	1.37	1.42	282	
77.926	1.29	1.41	281	
78.027	1.42	1.42	282	
78.128	1.66	1.45	285	
1km – 78.229	1.71	1.50	290	
78.330	1.24	1.55	295	
78.430	1.16	1.61	301	
78.531	1.61	1.67	307	
78.632	1.74	1.72	312	
78.733	1.92	1.74	314	
78.834	1.97	1.75	315	
78.935	2.03	1.81	321	
79.036	1.89	1.87	327	
79.137	1.95	1.90	330	
79.238	1.87	1.90	330	– Yarmouth/Malahide Township Line

* 100 year setback includes stable slope allowance

Table 3.5

ALONGSHORE SEDIMENT TRANSPORT RATES

based on sediment budget approach

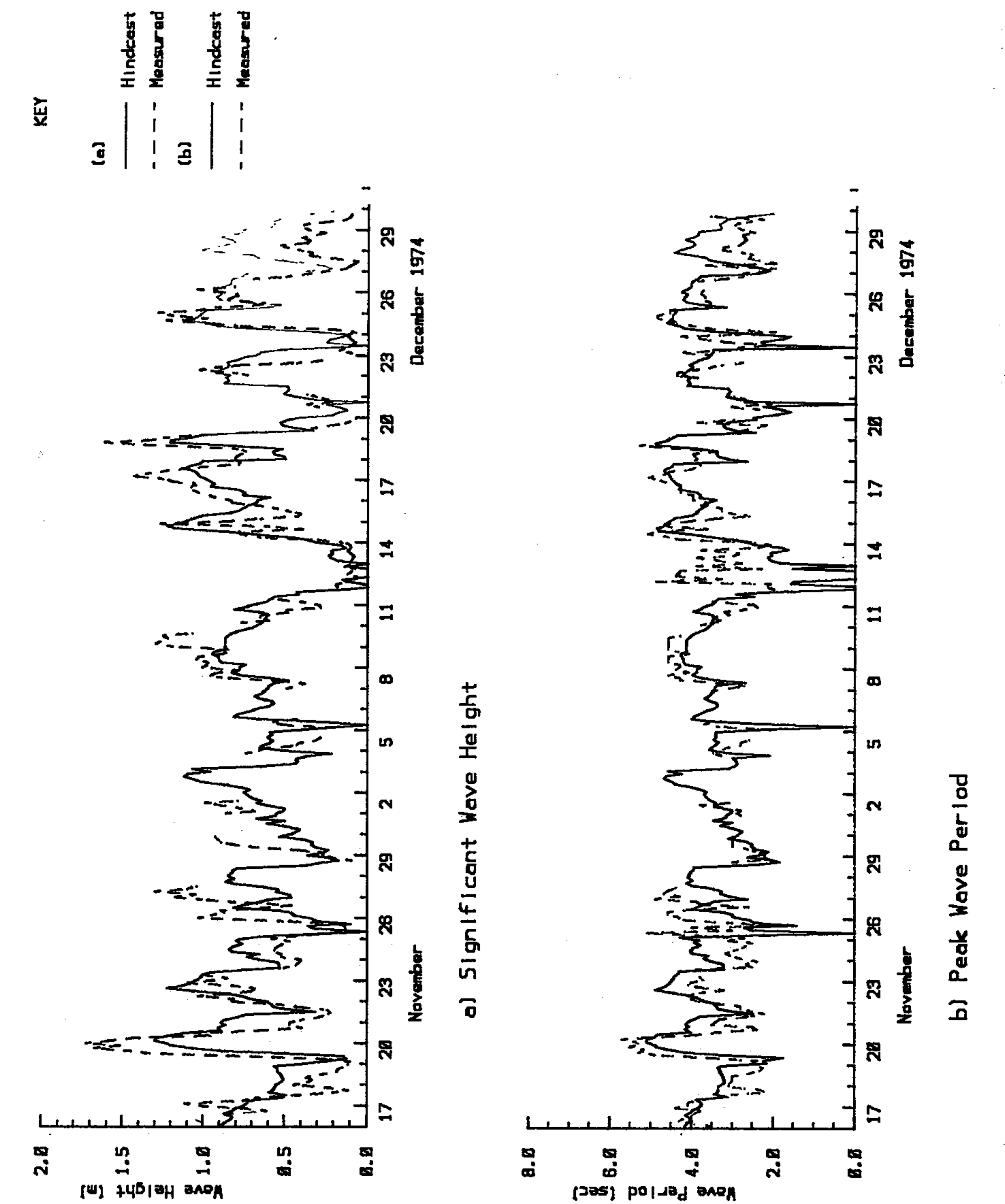
Alongshore Transport Rates
(thousands of cubic metres per year)

Burwell Chainage	Shingle (A)	Littoral Drift (B)	Sublittoral Drift (C)	Sum A+B	Sum A+B+C
8.20	0	1	1	1	2
9.78	1	3	2	4	6
11.74	2	6	4	7	11
13.73	3	8	5	10	16
16.35	5	9	7	14	21
18.23	6	11	7	16	24
19.65	7	12	8	19	27
21.65	7	13	9	19	28
23.20	7	13	9	20	29
24.57	7	13	10	20	30
25.88	7	14	11	20	31
26.97	7	15	13	21	34
29.60	8	19	24	26	50
36.66	8	21	31	28	59
43.15	8	22	33	29	63
48.97	8	24	36	32	69
55.19	Western end of study limit				
55.51	8	25	42	33	76
61.21	8	26	45	34	79
64.64	Port Stanley harbour breakwater				
65.37	0	1	6	1	7
67.19	0	8	44	8	53
71.44	0	19	80	20	100
75.20	1	28	93	29	123
77.97	1	31	100	32	132
79.25	3	35	104	38	142
79.24	Eastern end of study area				

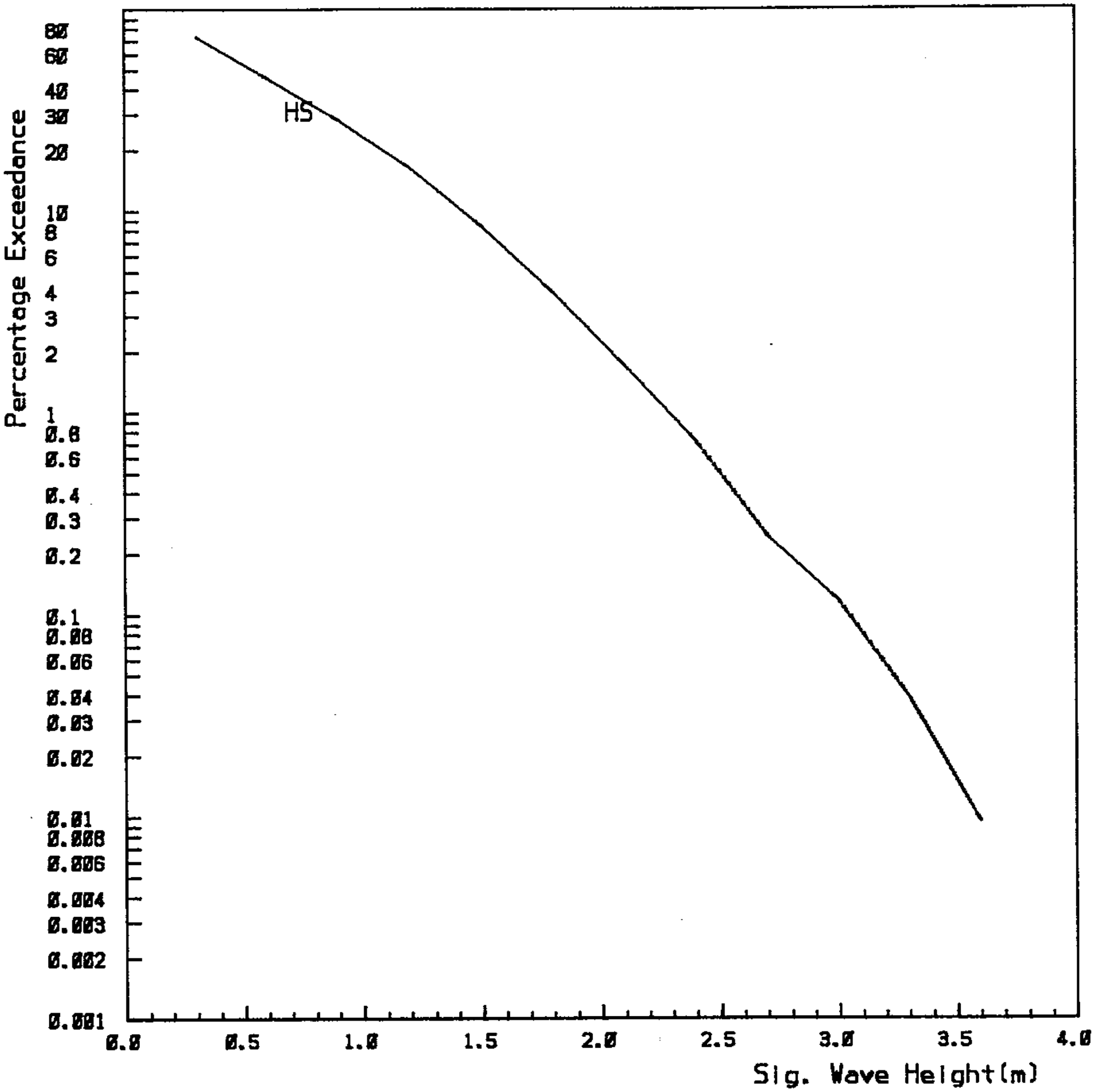
Adjustable Parameters

0% shingle lost
10% littoral drift lost
50% sublittoral drift lost
0% sublittoral drift bypassing Port Stanley harbour breakwater

FIGURE 3.1



KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 10 Sep 89
	Scales as shown
Wave Hindcast Calibration Results	Philpott Associates Coastal Engineers



Direction Limits:0.0 TO 360.0

Calms:27.0 %

Dates: 3/1/73 to 25/11/87

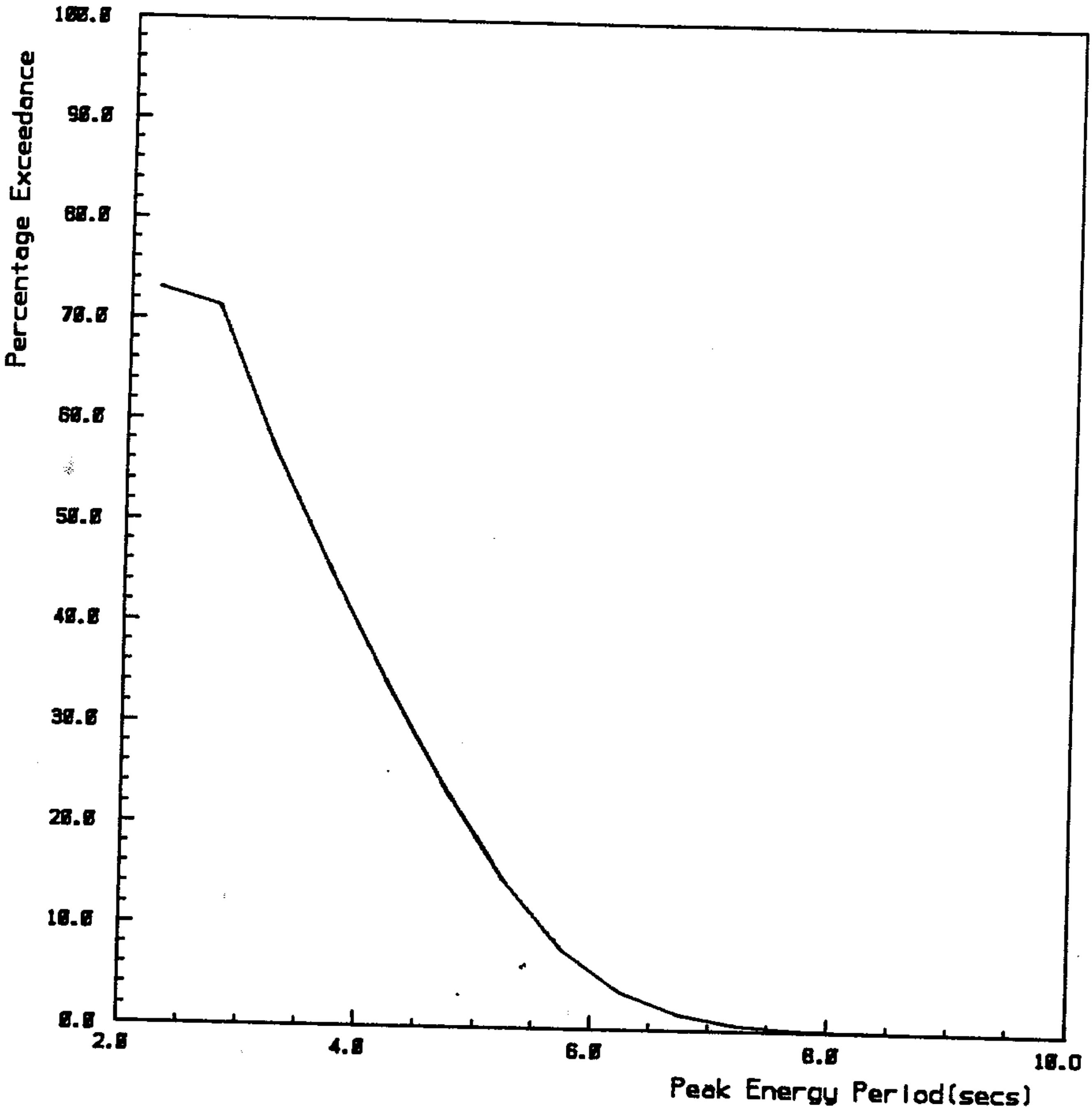
Season:Open water

Total No. of Records:106560

Waves hindcast with Sarnia wind data

KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Sep 89
	Scales as shown
Deep Water Wave Height Exceedance	Philpott Associates Coastal Engineers

FIGURE 3.3



Direction Limits:0.0 TO 360.0

Calms:27.0 %

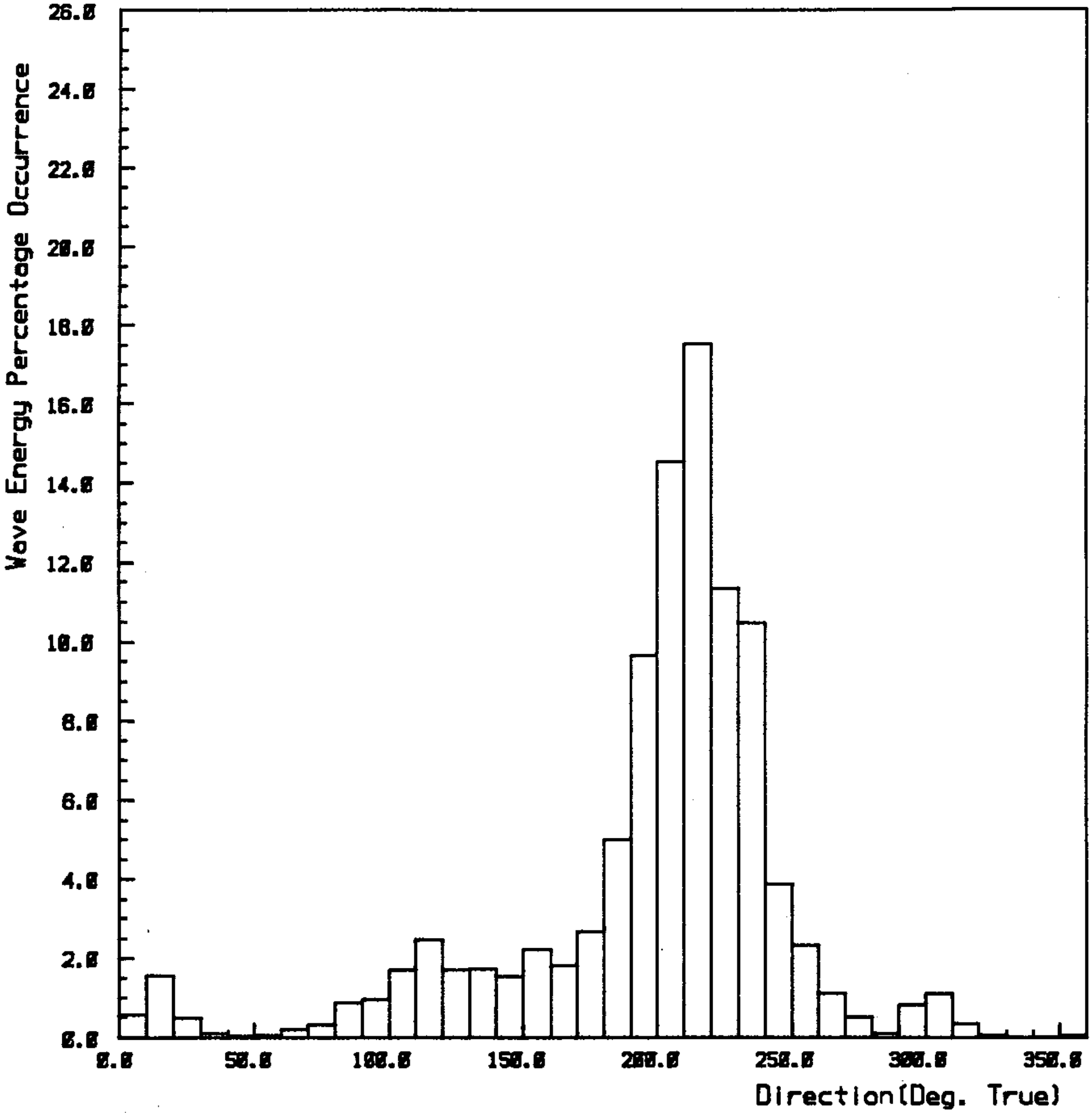
Dates: 3/1/73 to 25/11/87

Season:Open water

Total No. of Records:106560

Waves hindcast with Sarnia wind data

KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Sep 89
	Scales as shown
Deep Water Wave Period Exceedance	Philpott Associates Coastal Engineers



Direction Limits:0.0 TO 360.0

Calms:27.0 %

Total wave energy equals 47930 KJ/m/m

Dates: 3/1/73 to 25/11/87

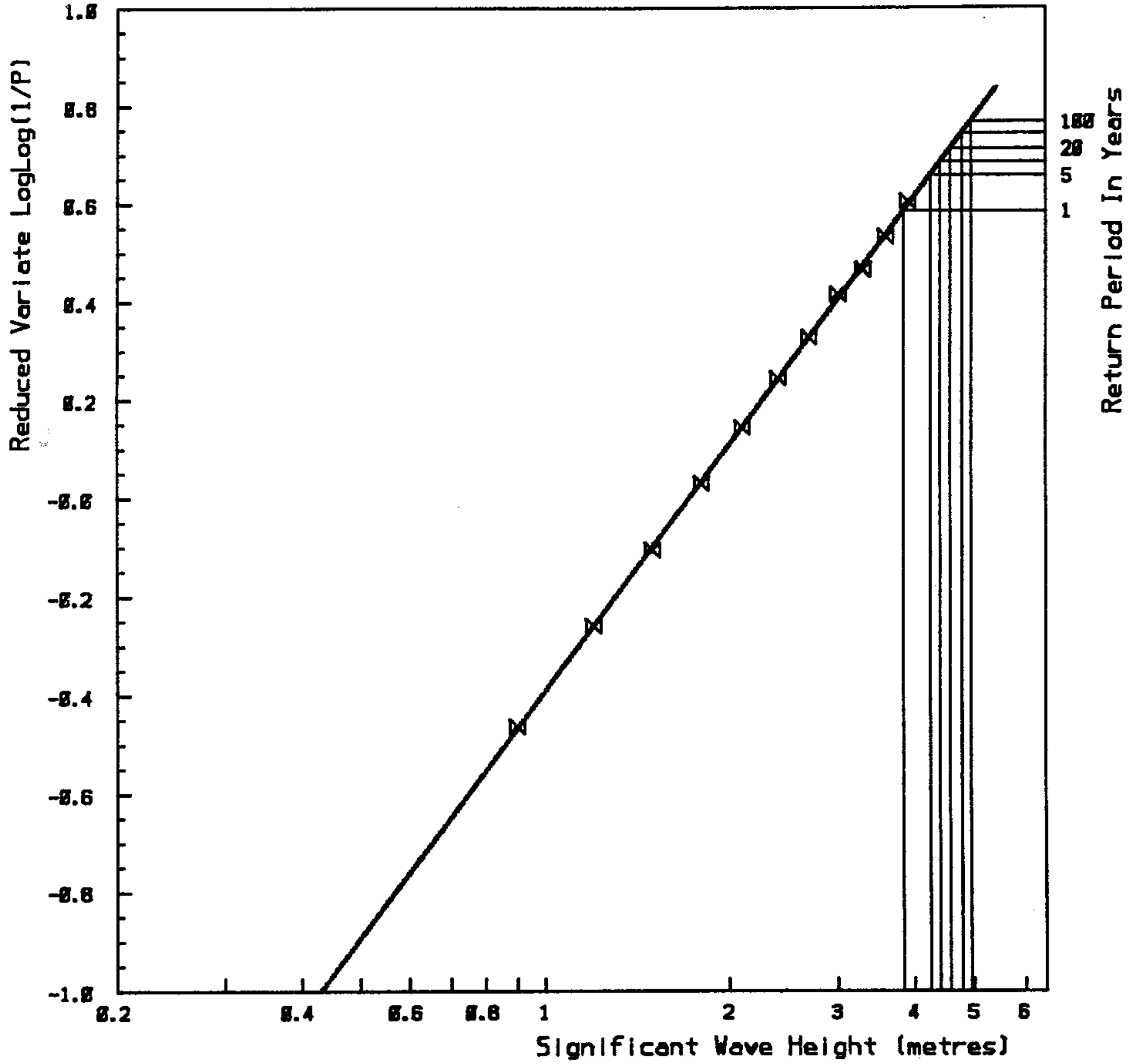
Season:Open water

Total No. of Records:106560

Waves hindcast with Sarnia wind data

KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Sep 89
	Scales as shown
Deep Water Wave Energy Distribution	Philpott Associates Coastal Engineers

FIGURE 3.5



Direction Limits:0.0 TO 360.0

Calms:27.0 %

Dates: 3/1/73 to 25/11/87 , # years: 14.9

Season:Open water

Total No. of Records:106560

Waves hindcast with Sarnia wind data

KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 . ep 89
	Scales as shown
Weibull Long Term Distribution of Wave Heights	Philpott Associates Coastal Engineers

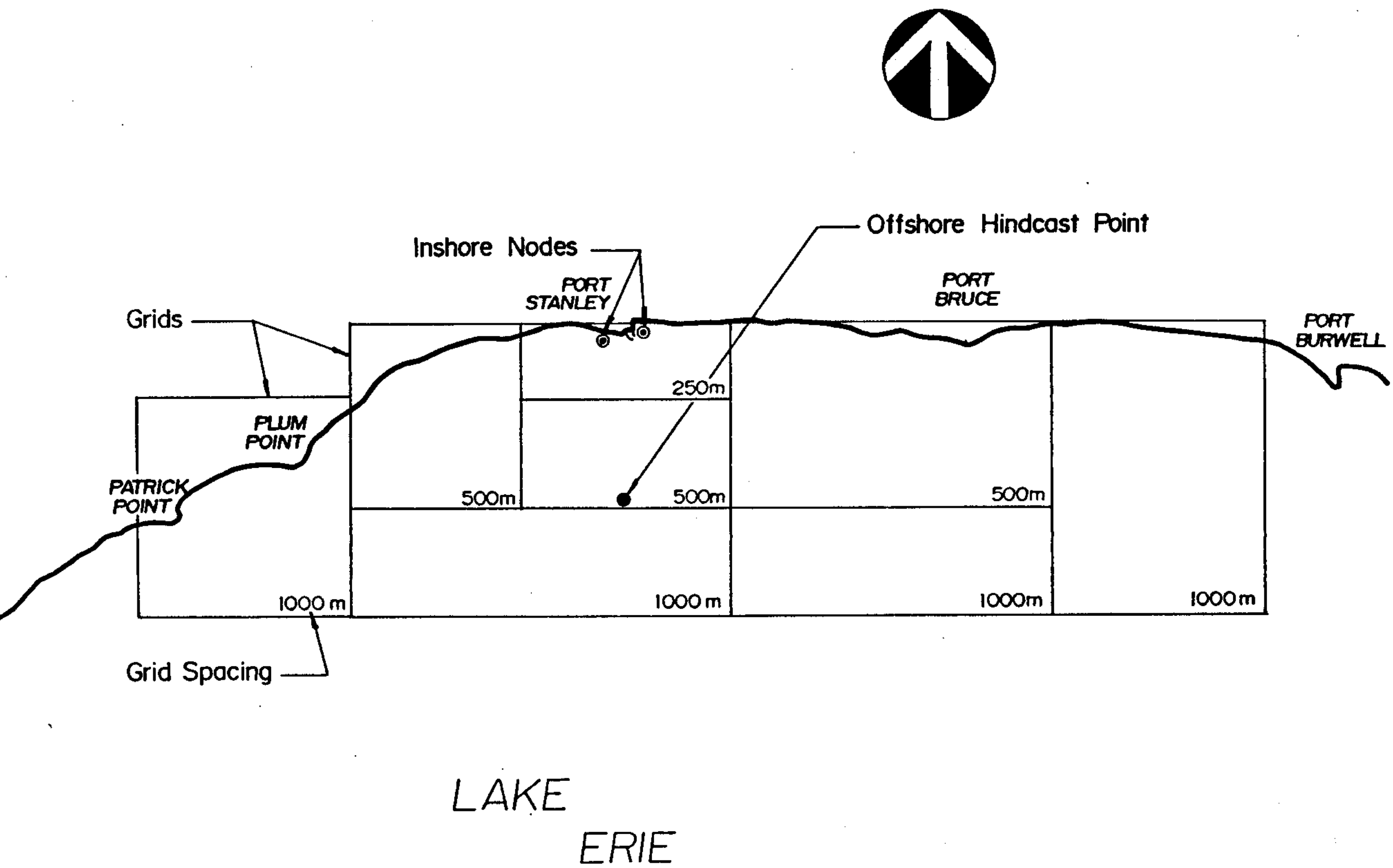
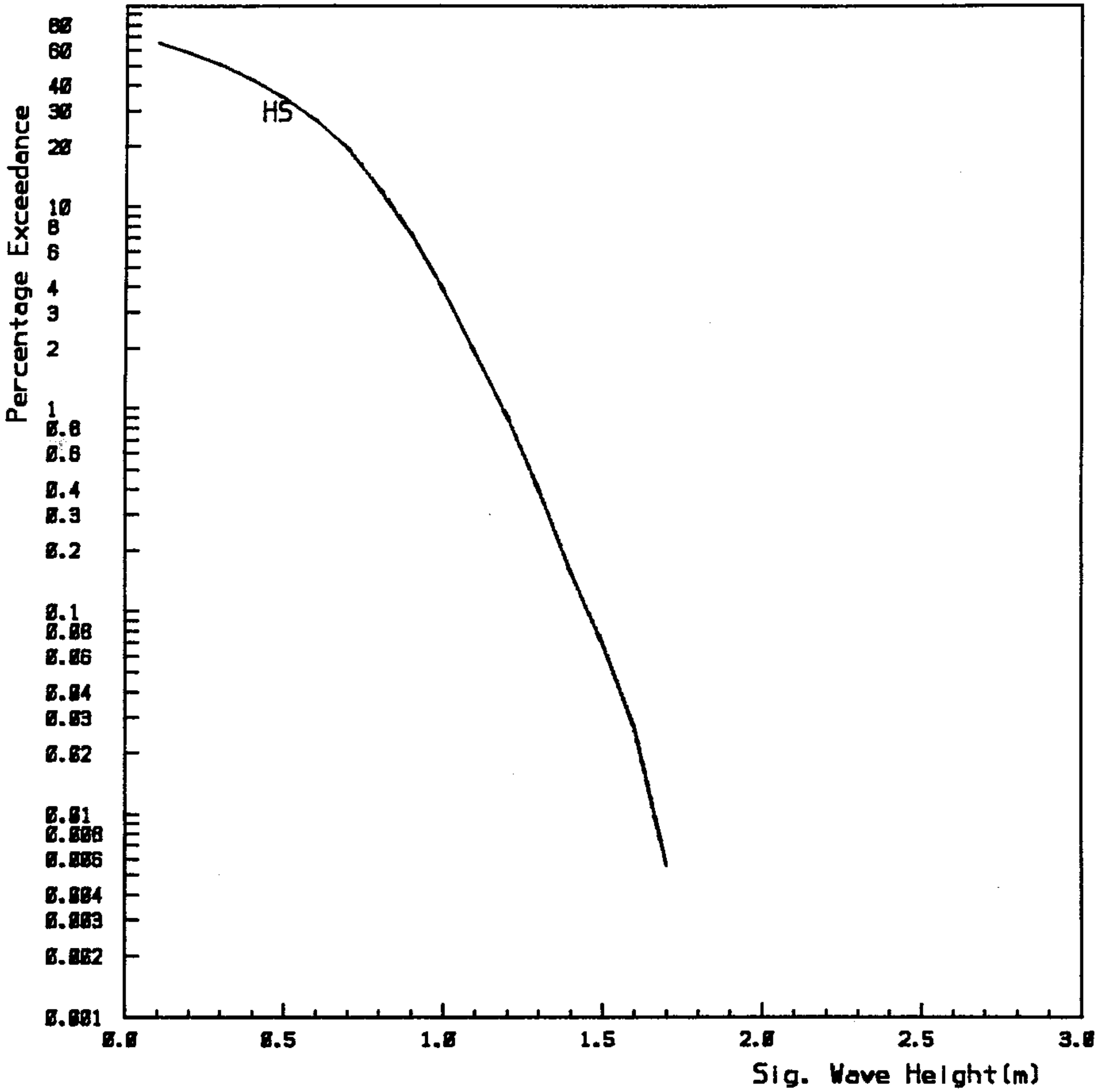


Figure 3.6
Wave Refraction Setup

FIGURE 3.7



Direction Limits:90.0 TO 270.0

Calms:34.8 %

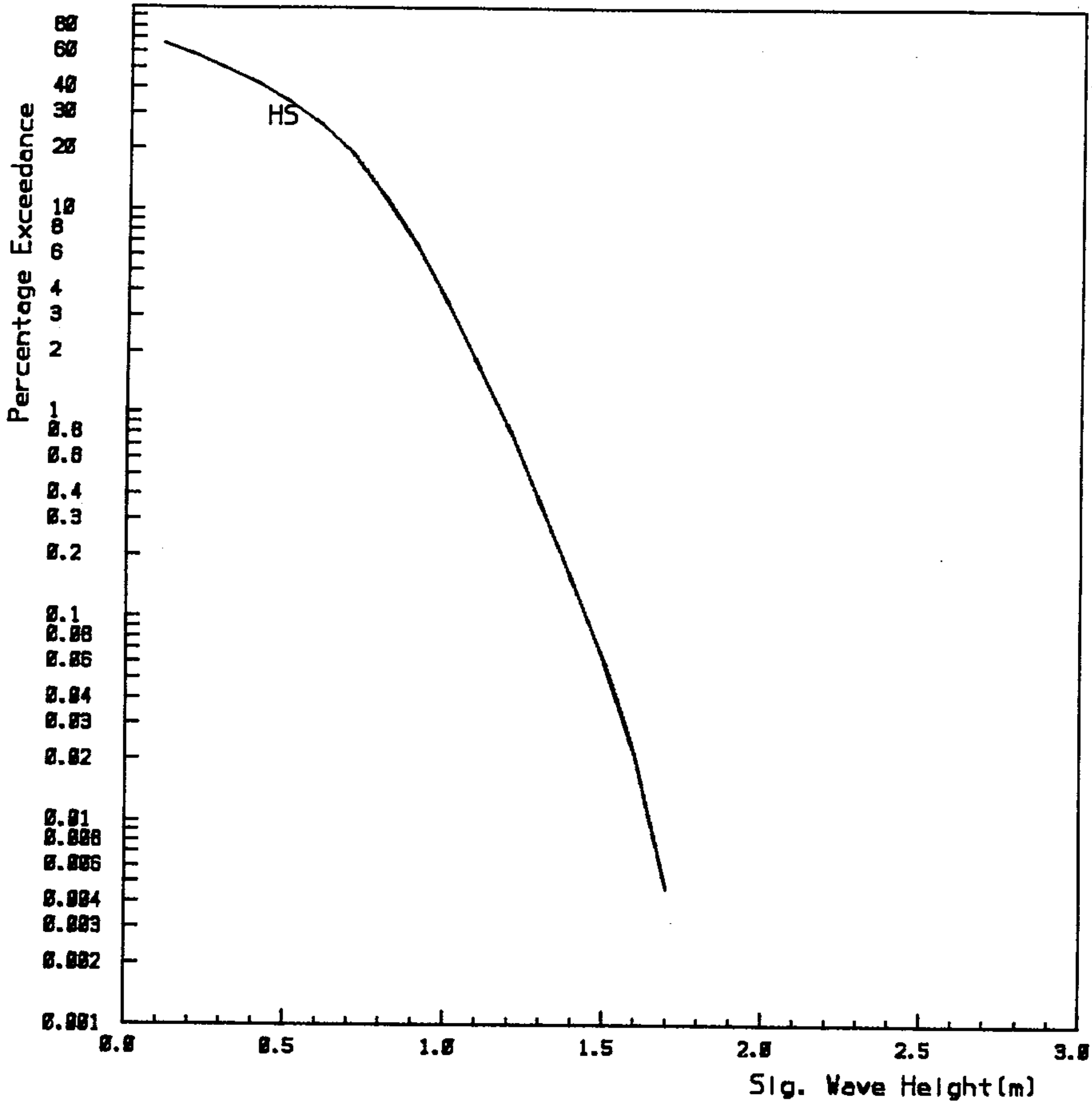
Dates: 3/1/73 to 25/11/87

Season:Open water

Total No. of Records:106560

Waves hindcast with Sarnia wind data

KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Sep 89
	Scales as shown
Inshore Wave Height Exceedance - Node 1	Philpott Associates Coastal Engineers



Direction Limits:90.0 TO 270.0

Calms:34.5 %

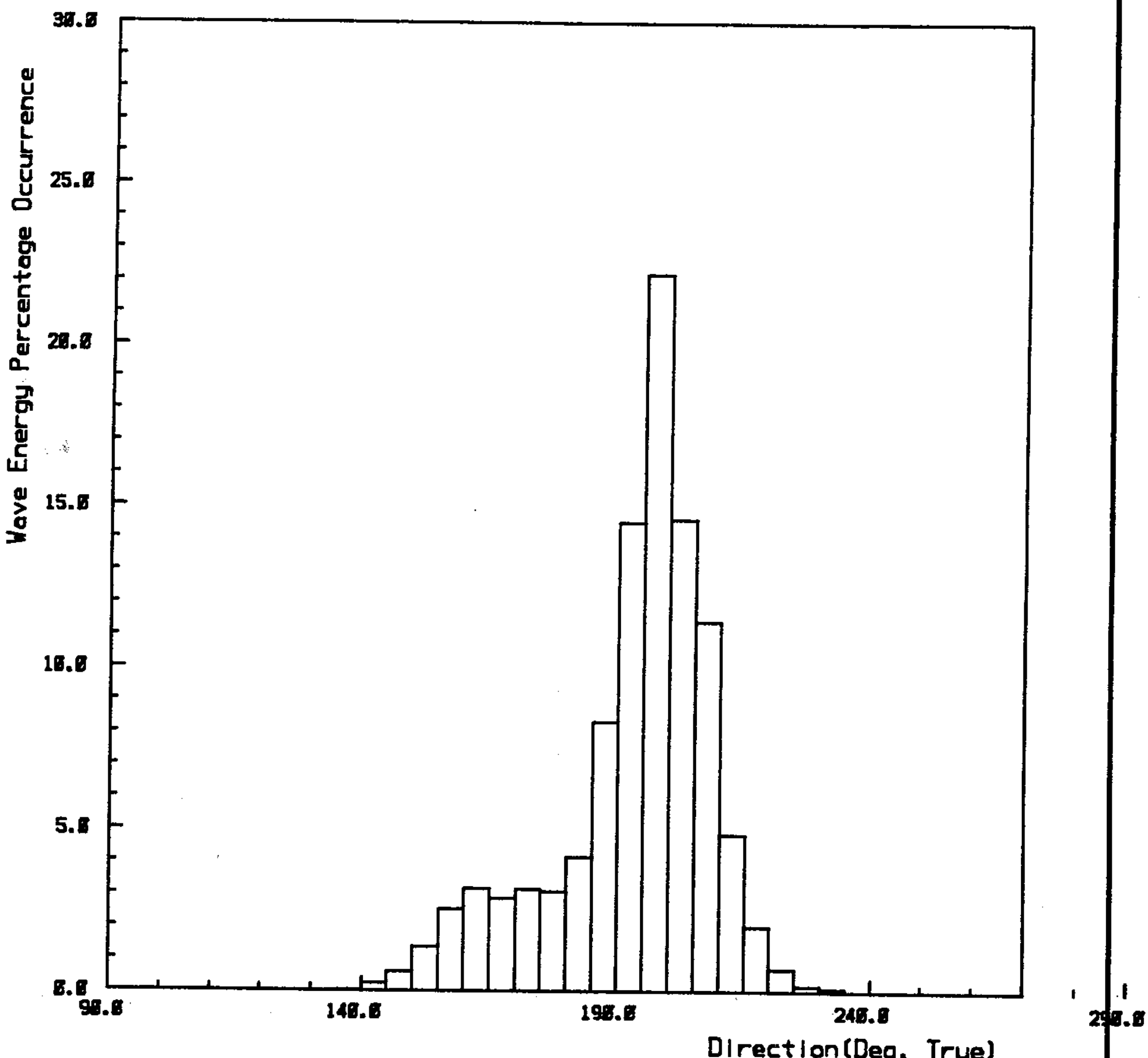
Dates: 3/1/73 to 25/11/87

Season:Open water

Total No. of Records:106560

Waves hindcast with Sarnia wind data

KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Sep 89
	Scales as shown
Inshore Wave Height Exceedance - Node 2	Philpott Associates Coastal Engineers



Direction Limits:90.0 TO 270.0

Calms:34.6 %

Dates: 3/1/73 to 25/11/87

Season:Open water

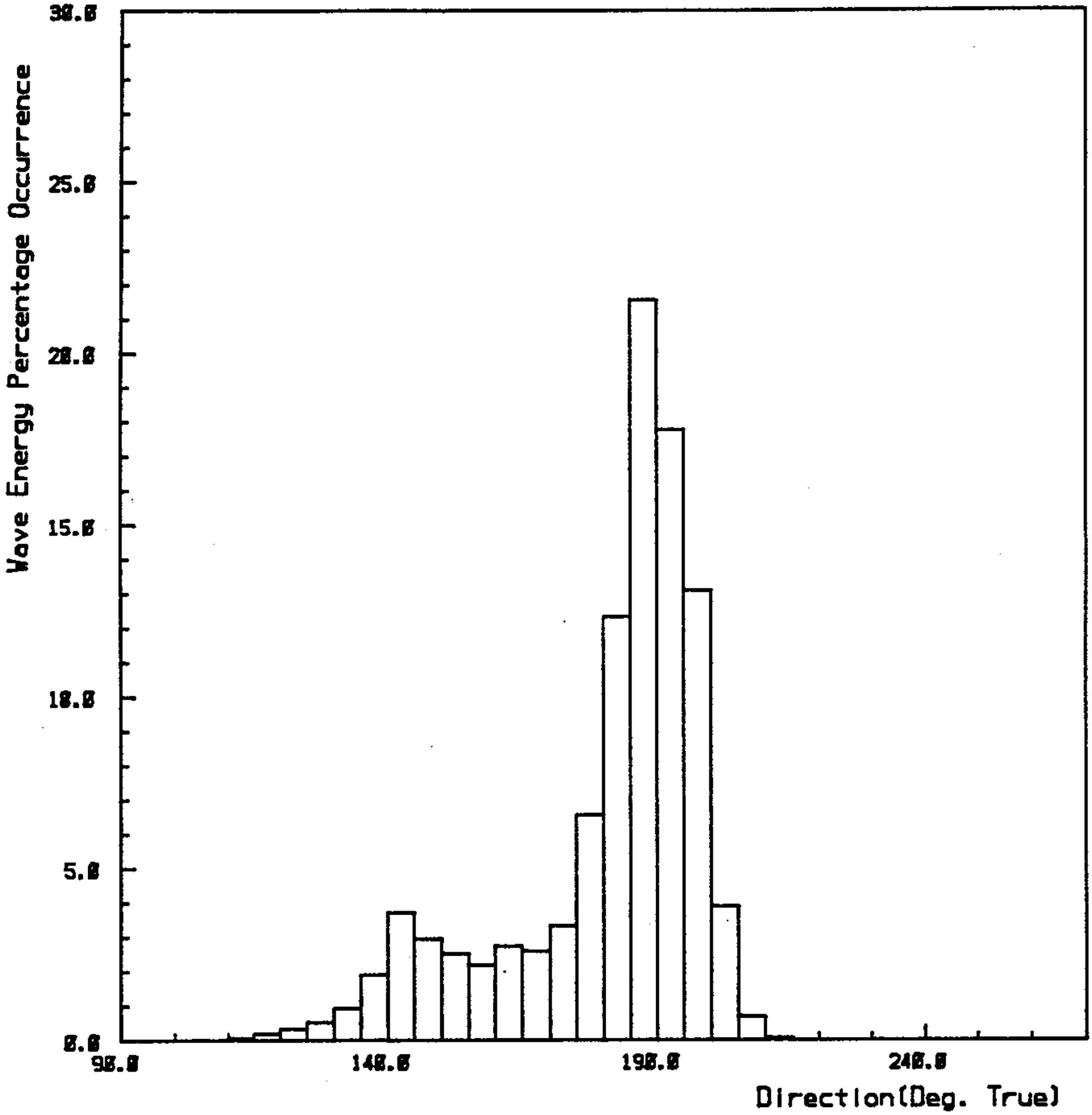
Total No. of Records:106560

Waves hindcast with Sarnia wind data

Total wave energy equals 16019 KJ/m/m

KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Sep 89
	Scales as shown
Inshore Wave Energy Distribution - Node 1	Philpott Associates Coastal Engineers

FIGURE 3.10



Direction Limits:90.0 TO 270.0

Calms:34.5 %

Total wave energy equals 15625 KJ/m/m

Dates: 3/1/73 to 25/11/87

Season:Open water

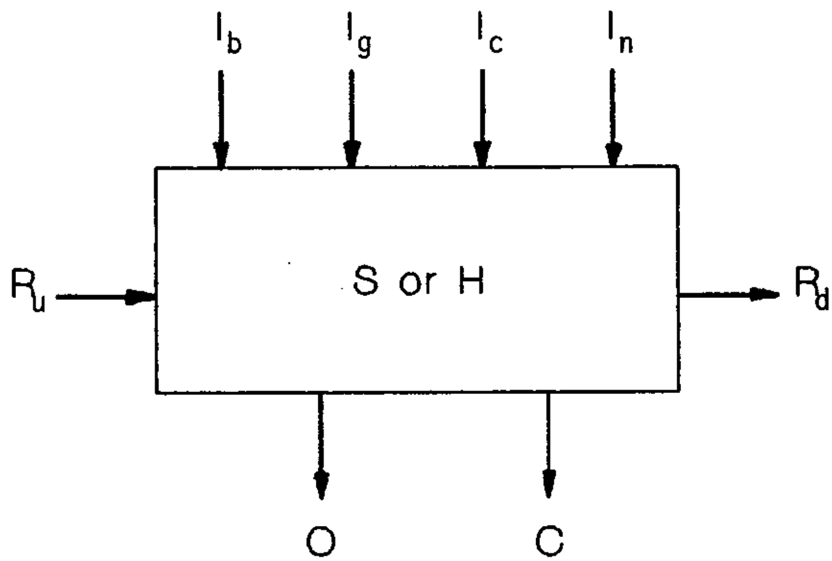
Total No. of Records:106560

Waves hindcast with Sarnia wind data

KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Sep 89
	Scales as shown
Inshore Wave Energy Distribution - Node 2	Philpott Associates Coastal Engineers

Figure 3.11

Typical Shoreline Segment in Sediment Budget Analysis



$$R_d = I_b + I_g + I_c + I_n - S - H - O - C$$

$$R_N = \Sigma R_u + R_d$$

where:

I_b = input from bluff erosion
I_g = input from gully erosion
I_c = input from creeks and rivers
I_n = input from nearshore bottom erosion

O = offshore losses
C = comminution of soft grains (also lost offshore)
S = gain in nearshore deposit (negative for loss
H = gain in harbour deposit (negative for loss)
R_u= sediment transport rates from updrift segments
R_d= sediment transport rate to downdrift segment
R_N= net alongshore sediment transport rate

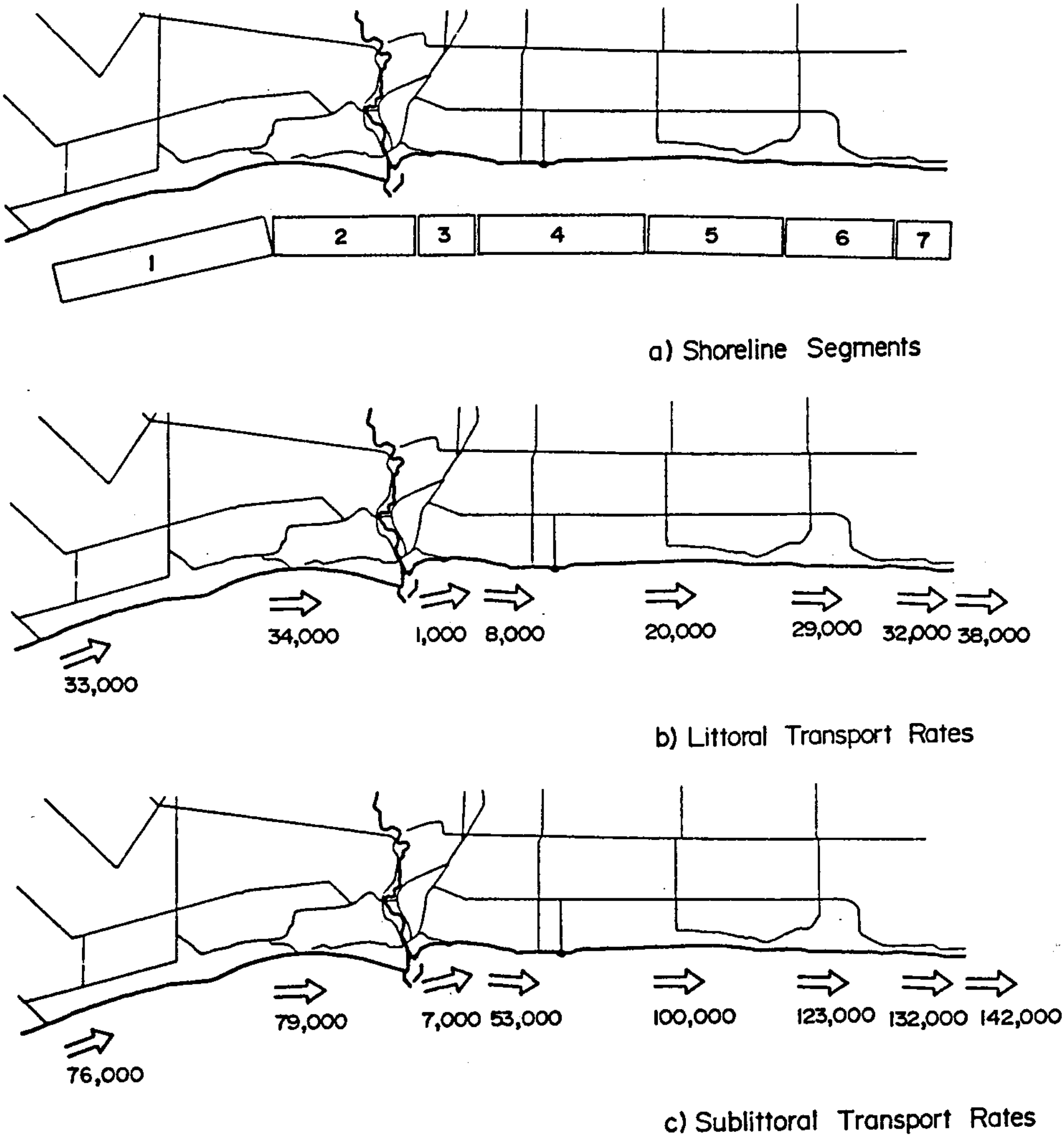
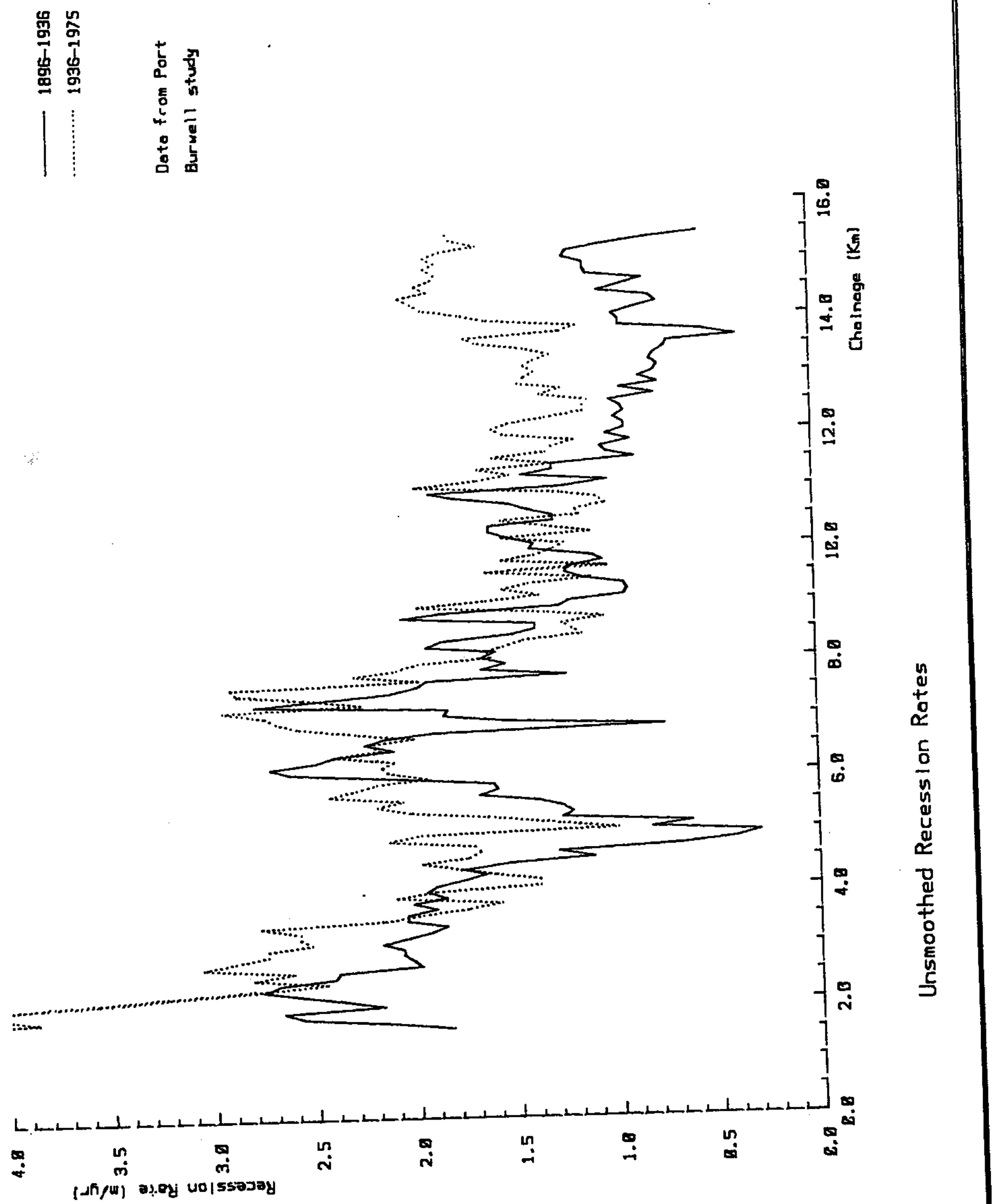
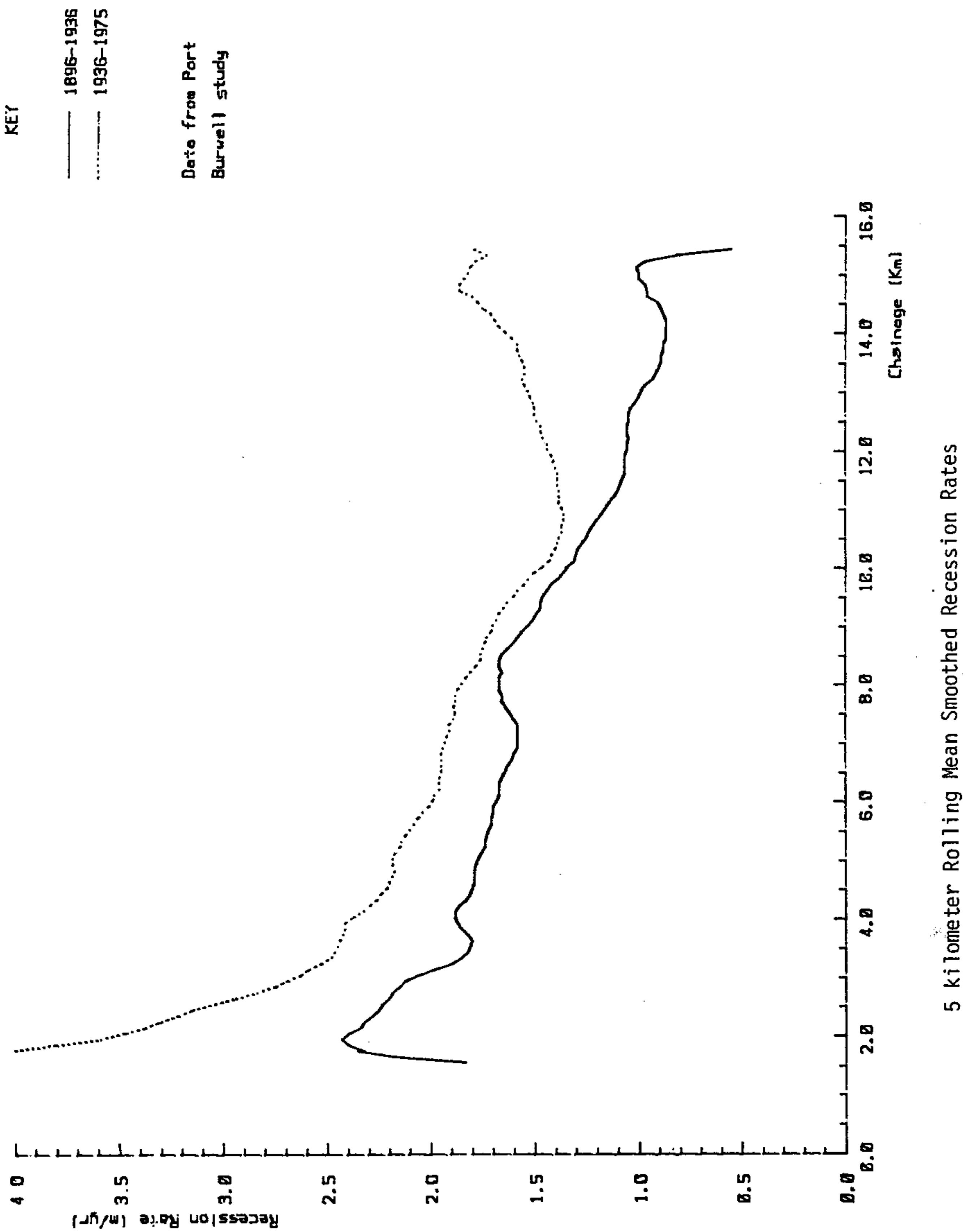


Figure 3.12
Alongshore Littoral Sediment Budget

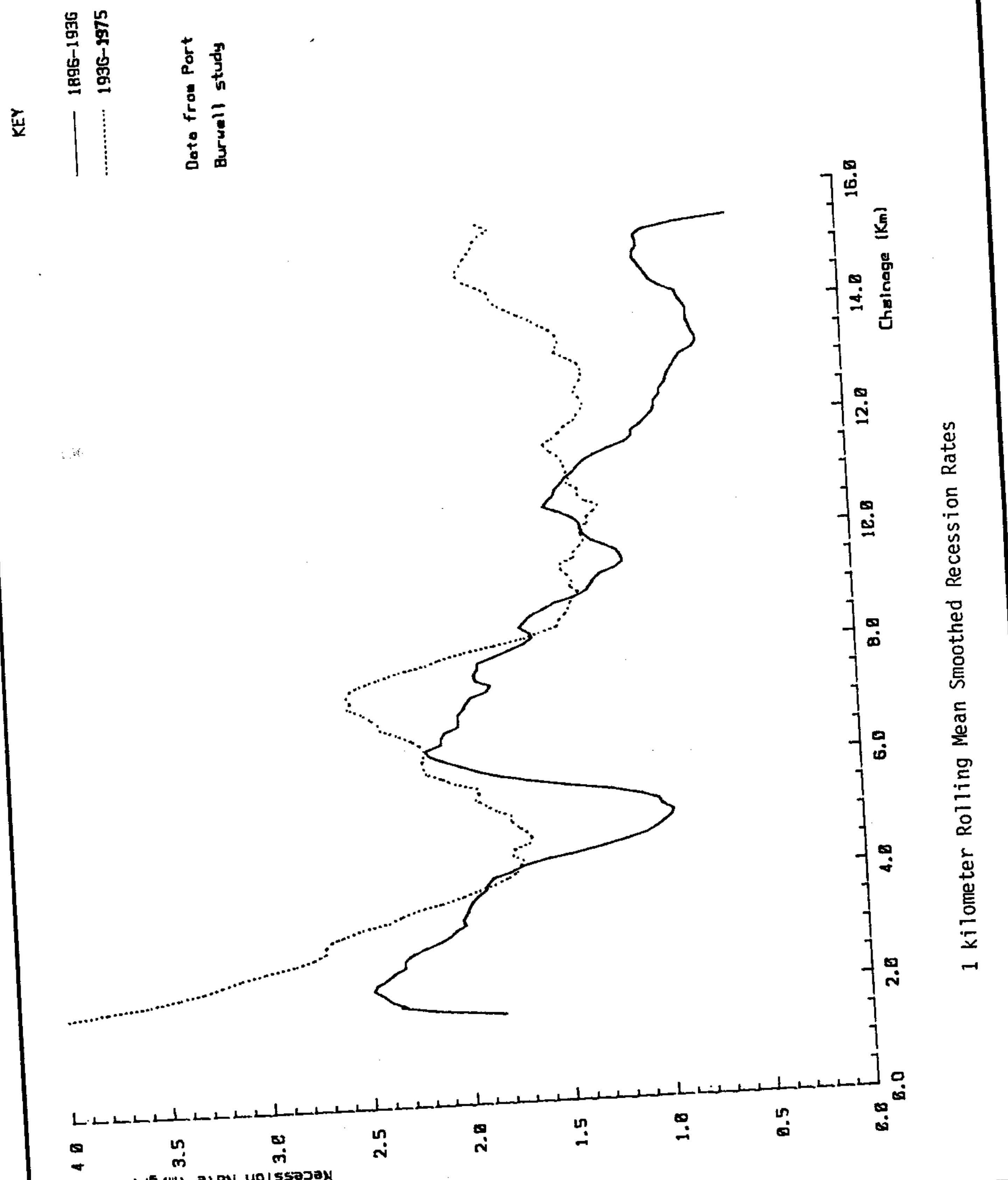


KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Jul. 89
	Scales as shown
Comparison of Recession Rates	Philpott Associates Coastal Engineers

FIGURE 3.14



KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Jul. 89
	Scales as shown
Comparison of Recession Rates	Philpott Associates Coastal Engineers



KETTLE CREEK SHORELINE MANAGEMENT PLAN	Date: 7 Jul. 89
	Scales as shown
Comparison of Recession Rates	Philpott Associates Coastal Engineers

4.0 Overview of Prevention and Protection

This chapter of the report provides an overview of possible shoreline erosion and flooding prevention and protection methods for the study area. Concept designs are presented for the protection methods considered viable within the study area. Common protection methods not considered viable within the study area are briefly reviewed and the reasons they are not considered applicable to this area are presented. Chapter 5 of the report, which discusses the preferred and alternate shoreline management concepts for each reach of shoreline, outlines which of the methods discussed within this chapter are considered applicable to that specific reach. Within the context of this shoreline management plan, prevention is considered to be the implementation of controls, regulations and land uses to avoid the risk of flooding or erosion to new development. Protection is considered to be the implementation of capital works for existing development. This would include for example, structural measures such as constructing revetments or floodproofing a dwelling by sealing all openings below a given flood level, or non structural methods such as dune vegetation or sand fill.

Depending upon the specific circumstances of a given section of shoreline either protection, prevention or a combination of both methods may be viable. Within this chapter and chapter 5, Preferred and Alternate Shoreline Management Concepts, a specific distinction is not necessarily made between whether a given solution is considered to be protection or prevention. It should be noted, however, that prevention is preferable to protection in that it is wiser to avoid having a problem now than it is to allow development that will need protecting in the near future. This in turn gives the most cost effective approach in the long term as well as reducing the risk of loss of life or property.

Essentially there are two types of protection responses to existing shoreline erosion and flooding problems; applying measures to hold back flood waters and wave action, and applying measures to allow the shoreline to withstand waves and high water without exceeding design levels of damage. These remedial measures may be divided into two groups; structural and non-structural methods. Generally, non-structural methods are the most desirable form of shoreline protection but they carry a higher risk of failure to perform during design conditions. Structural methods, on the other hand, can be constructed to withstand design conditions. Both structural and non-structural protection will, however, require maintenance throughout its design life.

4.1 Design Criteria

This shoreline management plan was developed using 100 year design criteria for rating erosion and flooding hazards and solutions. Erosion setbacks were established to allow for 100 years of erosion on a cohesive bluff, plus a stable slope allowance or for the erosion associated with a 100 year return period design storm on a sand beach. Flood levels were selected based on the 100 year wave uprush elevation, which was computed by adding the wave uprush from a 100 year return period design storm to the 100 year maximum instantaneous water level. The 100 year instantaneous water level was computed from the combined probabilities of occurrence of high static water levels and severe wind set-up. Derivation of the 100 year design conditions and criteria are covered in Chapter 3 of the report.

There is a clear distinction between the 100 year event used to define bluff erosion and the 100 year event used to define beach erosion and flooding. Bluff erosion setbacks are based on the 100 year recession limit, calculated as 100 times the estimated annual recession rate plus a stable slope allowance. Although not entirely correct, bluff erosion may, within the context of this plan, be viewed as a certainty. It may be assumed that erosion to the 100 year setback will not occur within the next few years. It may also be assumed that erosion to the 100 year setback will occur in +/- 100 years.

Flood levels and beach erosion, on the other hand, are computed based on a 100 year return period storm. This is a storm with a severity which is expected to occur once in 100 years. It is also the severity of storm of which there is a 1 per cent chance of occurrence within the next year. This design storm event cannot be considered as a certainty: it must be viewed on its probability of occurrence. It could quite conceivably occur within the next year (1 per cent chance) or it may not occur at all within the next 100 years (36 per cent chance), 150 years (22 per cent chance) or 200 years (13 per cent chance).

The use of 100 year designs, both for bluff erosion setbacks and probability of occurrence of severe storms, was adopted to comply with expected MNR policy. Although there is at present no policy under the planning act addressing flood and erosion issues along the Great Lakes shoreline, the province has followed a formal position since 1979. A new policy has been drafted by the province and is currently being reviewed by MNR and conservation authorities. This new policy which includes the elements of the former policy will eventually become a policy statement under the planning act. The draft policy calls for 100 year erosion setbacks, including a stable bluff allowance, and calls for the design of structures and flooding setbacks to be based on 100 year storm events. The use of a 100 year storm event as a design standard is consistent with existing provincial policy dealing with riverine flood events and is a well accepted practice.

4.2 No Action

In most problem cases some action must be taken, so the no action or do nothing alternative is mostly a decision making aid that can be used to evaluate various other alternatives. Because even minor protective measures can be quite costly, it is preferable to estimate potential losses assuming the no action alternative, particularly if no structures or lives would be at risk. This is particularly true when one examines the way that long term water level fluctuations effect the statistical assumptions used to produce design elevations and setbacks. For example, consider the 100 year design surge and flood levels developed for this study. With a 100 year return period there is a 1 per cent change that the event will occur at any given time. In Section 9.2 it is shown that the design storm surge and flood level for the study site are 0.96 metres and 175.52 metres G.S.C., respectively. If one is contemplating a corrective action with a 100 year return period design, (designed to withstand events with a 1 per cent chance of occurrence) then that action does not need to be taken until the static water level approaches 175.52 - 0.96 = 174.56 metres G.S.C.

If the static water level was at a relatively low elevation, say 173.7 metres G.S.C. then the do nothing action would be a viable alternative for the time being. One would have to realize, however, that as the water level rises to 174.5 metres the no action alternative becomes no longer viable and some action must be taken to maintain only a 1 per cent chance of damage, or a 100 year level of protection. It is important to note, therefore, that while this reasoning applies to the do nothing alternative, it does not necessarily indicate that taking some action to a lower design level is appropriate. It must also be noted that the preceding discussion is based on statistical theory and must be treated as only theory and not a certainty.

As a second example, consider a house located within the 100 year recession limit but still 60 years from imminent risk of destruction. There would be little benefit at present relocating the house beyond the 100 year setback. Only at the time that safe occupancy or use of the house and property becomes jeopardized is action beneficial. The potential future need for relocation should however be considered with any plans to modify the existing dwelling, sever the land etc.

An option similar to the do nothing alternative in action but quite different in consequence is the temporary abandonment of a property or structure. This is usually a last resort action taken when the cost of remedying a flooding problem is out of proportion to its significance. This is not the same as evacuation, which usually implies a short term response to an emergency situation. Occupation of the property or structure would be resumed at lower static lake levels rather than when surge induced flood levels subside.

4.3 Relocation

For most sites within the study area the do nothing alternative will not solve the problem, and some corrective measure will be required. In some of these cases relocating existing shoreline protection structures, dwellings and roadways would be less expensive than either constructing new or improving existing erosion and flood protection. The main objective of relocation would thus be to allow the present erosion or flooding problem to be ignored.

Relocation can be to an entirely different site, to a greater setback at the existing site or to a higher elevation at the existing setback. When the relocation alternative is exercised it is critical that the structure or roadway be relocated to a sufficient elevation and/or setback. Relocation is usually expensive and one does not want to have to repeat that expense because the original relocation was not sufficient.

When a structure is relocated it should be relocated either outside the regulatory shore land zone or to a location within that zone where the flood and erosion hazards have been overcome. When assessing whether or not a hazard has been overcome, it should be remembered that setbacks and elevations discussed within this plan are minimum design values. If possible homeowners should be encouraged to setback greater distances and raise to higher elevations above flood levels.

The cost associated with relocating a structure is related to the size of the structure, the structure foundation and the distance which the structure must be moved. The least expensive relocation would be the raising or jacking up of a small structure supported by but not connected to concrete blocks. Relocating a structure with a poured concrete foundation and floor slab would be much more expensive.

When a structure is threatened and relocation is contemplated the cost of that relocation will generally determine whether or not it is done. If the cost of relocation is considered to be too high in relation to the value of the threatened structure then abandonment of that structure may appear to be a reasonable solution. Temporary abandonment of a flood prone structure is acceptable, as discussed in Section 4.2. Permanent abandonment of either a flood prone or erosion prone structure is not acceptable.

A permanently abandoned structure in an erosion prone area will eventually fall into the lake. A permanently abandoned structure in a flood prone area will remain in place but it will deteriorate to that point that it cannot be inhabited even if the flood threat subsides. Neither of these cases should be allowed to happen as these abandoned structures will be environmental hazards as well as potential threats to public safety. The municipalities should ensure that such structures are dealt with before they become a concern.

This could be done by adopting by-laws which would allow a dwelling to be designed as "non -inhabitable" and requiring that such a structure be removed. The municipality would then either remove the structure itself or require the property owner to remove it. Again a by-law could be passed to ensure that this is done. Removal of the structure should include all infrastructure and servicing.

The decision of when a dwelling should be considered non-inhabitable depends upon the nature of the risk to that dwelling. If a structure is left in a flood prone but non-eroding environment then there is no urgent need to remove it. We therefore recommend that the individual municipalities determine the extent to which they will allow such dwellings to deteriorate before they require their removal. A structure threatened by erosion, on the other hand, could be at imminent risk but not be deteriorated to the extent that the municipalities would require its removal. We therefore recommend that the conservation authority take the responsibility of determining when such a structure is not inhabitable. That determination should be based on a more detailed analysis of the bluff or bank stability than has been undertaken for this study. Such a study would be able to consider local conditions such as bluff composition and ground water conditions. We would suggest that a detailed bluff stability analysis should be undertaken once the bluff erodes to a point that a dwelling is within a 2.5:1 slope (horizontal to vertical) from the toe of the slope. If the detailed bluff stability analysis showed that the dwelling was at risk, the conservation authority could then notify the municipality that the structure should be removed, as specified in the adopted municipal by-law.

If we assume that the average existing bluff slope is in the order of 1 to 1 (this is a rough estimate only) then a 2.5:1 slope from the toe of the bluff would intersect the table land about 60 m from the bluff crest. Twenty five of the fifty two structures covered in the high bluff structure inventory (Table 2.5) are within 60 metres of the bluff crest.

4.4 Minimum Setbacks and Elevations

Minimum setbacks and elevations are regulated to locate new development out of problem areas. They are also used as preferred standards for relocating existing structures. There is a difference, however, in the way in which setbacks and minimum elevations are used. Recession setbacks are used to keep development outside the limits of where that development would be at risk within approximately 100 years. If the calculated average annual recession rates, which were based on past erosion, are representative of the average recession rates over the next 100 years, then new development presently located just beyond the 100 year recession limit will be at risk in 100 years. Because the average annual recession rates used to define the 100 year recession limit are only estimates, one cannot assume that new development will be at risk in exactly 100 years. It will be some time until that development is at risk from erosion, and we do not know exactly when that time will be, but it will occur. This is because erosion of a bluff is a continually irreversible process.

Setbacks can also be used to keep development out of flood prone areas, although under certain specific circumstances development can occur within those areas if it is constructed above a certain flood elevation. Both the setbacks and elevations in this instance are based on the 100 year return period design storm.

Because erosion of the Port Stanley fillet beach is a reversible process, erosion setbacks for reaches 1 and 2 of Port Stanley must also be based on the design 100 year storm event. In Section 3.6.2 it was estimated that the 100 year storm could cause a recession of 5 metres near the waterline and that up to 40 metres of beach, from the still waterline, could receed. The recession setback for 1 100 year storm would therefore be 40 metres. It is possible, however, that recession could be experienced over a greater width of beach due to a number of successive less storms or a more severe storm. The extent to which this may occur was not investigated. Because these values are based on a statistical analysis there is a chance that they could be exceeded. For this reason, design storm related setbacks and elevations should be viewed as minimums only and homeowners should be encouraged to setback further than or elevate higher than these minimum values.

4.5 Non-structural Protection

Non-structural measures are generally the least expensive forms of protection but, conversely, they do not work in serious problem areas. The non-structural techniques considered here; sand fill, vegetation, use control and dune management, are different techniques but they are closely related and generally work better when combined than when considered separately.

4.5.1 Natural Beach and Dune Protection

Wave action transports sand onshore forming a sand berm. Wind blows the sand towards the backshore, forming a dune. The portion of the berm within the wave uprush zone forms the beach. The beach together with the dune form the active beach zone. The sand berm acts as a barrier separating the backshore from Lake Erie.

The sand veneer provides cover to the underlying clay bottom. This cover is critical since erosion to clay is irrecoverable and results in long term recession of the shoreline. Although individual sand grains increase erosion of the clay through abrasion, a thick enough deposit within the nearshore zone acts as a buffer to prevent exposure of the clay and subsequent erosion thereof.

In order to fully protect a shoreline, dunes must store a considerable volume of sand. In most locations a series of dunes, not just one dune, are required to provide this protection. Dunes will provide protection to the backshore only as long as there is a sufficient volume of sand and a sufficient width of beach to properly form the dunes. The volume needed depends upon the duration of the storm which itself is a random event with a certain probability of occurrence.

The sand beach updrift of the Port Stanley harbour breakwaters behaves distinctly different than the sand deposits in front of the high bluffs. Although the beach has been artificially formed, through construction and extension of the breakwaters, it is wide enough that natural dune formations do occur.

The shoreline is largely developed with most of the permanent development located beyond the active beach zone. However, non-structural encroachment onto the dunes has reduced the flood protection of many areas. This encroachment has generally taken place through the flattening of the dune crests to provide a better view and access to the lake. If at all possible, one should strive to enhance or recreate the natural dune protection, not flatten the dunes. Where recreating natural protection is beyond hope, however, relocation or structural methods will be required.

The most effective means of enhancing the natural dune protection are placing sand fill, vegetating the dunes, effective use controls and proper dune management. These methods are discussed in more detail in the following sections.

4.5.2 Sand Fill

The placement of sand fill along a portion of shoreline is a non-structural erosion control technique used to either protect existing sand beaches or to create new ones. Where feasible this is an attractive solution because not only do beaches provide an additional recreational benefit, but, when stable, beaches also provide the best erosion protection to a shoreline. Sand fill will also provide protection against flooding by storm surges but such fill must be placed landward of the back of the beach beyond the point where wave uprush during design storms can adjust the beach profile.

Sand fill may be applied in two ways; it can be placed directly on the beach that it is to protect or it can be placed updrift of the beach so that it is transported into place through natural littoral processes. In order for sand fill to be effective, however, it must be carefully placed. Large piles of sand dumped on a beach will tend to act like a groyne and an "erosion wave" will propagate downdrift, leading the beach feeding material.

If sand fill is to be used as a long term protection measure then a guaranteed supply of an adequate grade of sand is required. Annual maintenance costs must also be considered when evaluating such a procedure because they can represent a significant proportion of the initial cost. Annual replenishment volumes are very site specific and relate to the alongshore gradient in potential sediment transport rates.

Unless other measures are taken in conjunction with sand fill, such as placing groynes, then sand fill should only be viewed as providing a temporary buffer zone. Little Beach, (Reach 4 in Port Stanley) is a good example of this. Much of the sand fill was lost offshore when the water level rose after the sand was placed because there were no structures placed to retain the sand fill.

4.5.3 Vegetation

A planting program designed to introduce certain species of vegetation to the upper portion of a beach and backshore area can be an inexpensive means of increasing the shoreline flood resistance. This is accomplished by both decreasing the volume of sand moved offshore during storms and decreasing the loss of wind blown sand landward of the dune.

Depending on where stabilization is required, species from one of two general groups should be selected to ensure adequate growth (U.S. Army, 1981). These groups are marsh plants and upland species. Marsh plants are not suitable for shoreline protection throughout the study site because of the severity of the wave climate.

Upland species, such as trees, shrubs and grasses are especially adaptable to growing in the low moisture, low nutrient environment characteristic of the upper portion of beach dunes. While the primary purpose of planting this vegetation is usually to trap sand to stabilize the shoreline, it also improves the beauty of the shoreline, resists erosion due to rainwater runoff and provides wildlife habitat. Vegetation, however, cannot alone prevent dune erosion due to the significance of wave action.

The U.S. Army (1981) suggests that reed canary grass, big bluestem, little bluestem and witchgrass are all suitable grasses for stabilizing dunes on the Great Lakes. Various ground cover may also be planted. Local vegetation experts should be contacted for information about the suitability of using various grasses. Local feed stores can provide information about the availability of the different grasses.

4.5.4 Use Controls

Controlling the use of shorelines in order to avoid interfering with erosion protection or aggravating previously damaged areas is another form of non-structural flood and erosion protection control. The main objective of use controls is therefore to avoid causing or having a problem rather than actively correcting a problem. It must be recognized, however, that proper use controls can also allow for a natural recovery of a problem shoreline.

Under this approach shorelines could be classified as: limited access areas, limited construction areas, specified construction only areas, or specified setback areas. These methods are not usually applied to properties where existing development has already caused sufficient damage to the shoreline that greater protection efforts are required. However, this is not always the case and use controls can be applied to existing properties where less severe problems exist.

The application of use controls to protect developed properties may be demonstrated along the beach updrift of Port Stanley harbour. The dune system within Reach 1 provides close to 100 year flood protection in some places but much less protection in others. The low protection areas in many cases result from property owners flattening the dunes to provide easier access and a better view of the lake. If dune growth is to be encouraged then flattening of the dunes must be stopped and pedestrian access to the worst areas should be prevented until the dunes have been established. Both preventing flattening of the dune and restricting pedestrian access are forms of use controls.

An alternate form of use control can be illustrated within Reach 2 of Port Stanley, where the existing dunes are well below the 1:100 year flood protection elevation and significant inland areas can be flooded. The likelihood of a flood even within this area must be considered when various land uses are being contemplated. For example it may be acceptable to keep a car in a garage in a flood prone area but the storage of paints, chemicals and deleterious materials would not be advisable. Restricting the types of items kept in a flood prone area is another type of use controls.

4.5.5 Dune Management

The last type of non-structural protection is dune management. Under this technique, either new dunes are constructed or existing dunes are reinforced to increase their level of shoreline protection. Dunes are formed parallel to and behind the shoreline and retain sand that is transported onshore by wind action and wave uprush.

Dunes are an extremely critical component of a stable shoreline as they provide a reservoir of sand for the beach system. Dunes are eroded during storms, providing sand for the formation of offshore breaker bars. During calmer wave conditions sand is transported from the offshore bars back into the dune system. Because dunes act as dynamic reservoirs and flood barriers, they adopt to varying wind and wave conditions and long term fluctuations in water levels.

Dune management is best achieved in conjunction with the other described non-structural protection techniques; sand fill, vegetation and use controls. Constructing or replenishing dunes is in fact a form of sand fill. Vegetation is required along the surface of dunes to help reduce landward losses of beach sand. Until a firm cover of vegetation exists snow fences may be used to physically retain blowing sand. Finally, use controls are strongly recommended in the areas of developing dunes. Sand removal, improper construction and even an excess of pedestrian traffic can do irreparable damage to a dune.

The Ministry of Natural Resources is currently preparing a design manual for the artificial construction of beach dune systems. This manual will provide details about dune width and height, locations on the beach, suitable types of vegetation, etc. It is estimated that this document will be completed during the summer of 1990.

4.5.6 Summary of Non-Structural Protection

Non-structural protection is generally preferable to structural protection. Where it is effective, non-structural protection tends to complement the natural coastal processes rather than resist them. Non-structural techniques tend to provide a more natural setting which in turn leads to increased vegetation cover and wildlife habitation. Non-structural methods are also considered by most people to provide a more aesthetic waterfront than provided by structural protection. Although preferable to structural protection, non-structural protection is viable in fewer locations.

Generally non-structural protection cannot withstand severe wave action. The exception to this is a stable fully developed beach with an adequate dune. It is therefore likely that non-structural techniques can only be relied upon to provide 100 year design protection within Reach 1 of Port Stanley. Non-structural techniques will not work in front of the high bluffs and are not considered to be a practical solution in the high pedestrian traffic area of the beach in Reach 2 of Port Stanley.

4.6 Structural Protection

This section discusses a number of various types of structures which, under the proper circumstances, can be used to provide effective protection against both erosion and flooding. These structures include revetments, bulkheads, flood berms, groynes, headland bays and breakwaters. The following sections describe these structures in varying levels of detail, depending upon their utility within the study area.

4.6.1 Revetments

Revetments are perhaps the most successful type of structures used for erosion protection on the Great Lakes. Essentially a revetment is a sloped structure, supported by a natural bank or artificial fill, with an erosion resistant facing.

In order to construct structural protection the individual landowner will have to demonstrate that the structure will not have a significant impact on either updrift or downdrift coastal processes. This issue is discussed in more detail following.

The primary purpose of a revetment is to prevent erosion of the shoreline although a revetment will reduce flooding amounts if it is high enough to prevent significant overtopping. A revetment itself is not water tight and therefore will not hold back water below the flood level. To be successful a revetment must be able to meet the main criteria:

1) stability and durability of the armour layer;
2) overtopping scour protection;
3) toe scour protection;
4) flank protection;
5) no significant impact on coastal processes.

The first four of these criteria is discussed immediately following. The last criteria is discussed in Section 4.6.8

As long as these conditions are met a vast number of materials may be used to construct revetments. More common types of material include quarried stone, concrete rubble, interlocking concrete blocks, stacked bags and gabion baskets. These materials are also discussed following.

Stability and Durability of the Armour Layer

The armour layer, which is the lakeward surface of a revetment, must be stable both during design storm conditions and while subjected to extreme ice forces. Unfortunately it is not possible to quantify the destructive ice forces with the same degree of accuracy as wave forces and hence a conservative estimate of the armour sizing must be made. The armour material, as well as other materials within the revetment, must also be durable enough to guarantee a reasonable design life to the revetment. Ideally, a design life of at least 100 years is desired but in reality there is no shoreline structure that should be expected to last 100 years without maintenance, with the possible exception of an excessively heavy gauge steel pile wall. The component materials within a revetment must be durable enough on their own that they can at least last 50 years. The revetment structure as a whole must be properly maintained throughout its life.

With durability in mind neither gabion baskets nor stacked bags (either sand or grout filled) are recommended for permanent revetment construction. Gabion baskets exposed to waves and rafting ice do not usually last more than a few years. Stacked bags also have a relatively low service life but that service life depends on a number of factors which cannot be generalized here (such as bag material, fill material and construction technique). Both gabions and stacked bags do have the advantages of lower cost and ease of construction but these advantages are outweighed by the disadvantage of a short design life.

Two of the key features of a revetment are that they are flexible and porous structures. Increased porosity increases the revetment's dissipation of incoming wave energy. Flexibility allows for differential settlement along the length of the revetment without adversely effecting the revetment. For these reasons continuously formed poured concrete revetments should not be constructed. Non-interlocking concrete blocks may be used as primary armour on a revetment if they are large enough. Such blocks should be of the same size as quarried armourstone and, for the purpose of this discussion, may be treated the same as quarried stone. They must, however, be made with a reasonable strength of concrete.

A large number of designs of interlocking concrete blocks exist on the market today. This includes, but is not limited to, Erco Blocks, Gobi Blocks, Jumbo Blocks, Lok-Gard Blocks, Turf Blocks, Nami rings, Shiplap Blocks, and Terra-Fix Blocks (U.S. Army, 1981). The authors of this report, however, strongly recommend that only interlocking blocks which are cabled together as part of the block design be used for for shoreline revetments. A number of failures of interlocking block revetments have apparently been caused by a loss of stability of neighbouring blocks following the failure of individual blocks. By cabling blocks together the risk of this mode of failure is greatly reduced.

It should also be noted that, given the severity of the wave climate within the study area, the only places that interlocking blocks could be used would be along the upper portion of a beach. For example, a dyke constructed along the back of the main beach could be protected by cabled interlocking blocks. Interlocking blocks would not work in front of the bluffs.

On the basis of the preceding discussion, it may be concluded that quarried stone and concrete are the most suitable material for the primary armouring of a revetment. The concrete may be either poured blocks or large rubble and, for conceptual design purposes, may be considered similar to stone.

Figures 4.1 and 4.2 show 2 similar typical revetment cross sections, one for a single layer revetment and 1 for a multi layer revetment. The single layer revetment has one layer of primary armour, with each armour stone weighing between 5 and 7 tonnes (5,000 and 7,000 kilograms). This primary layer is placed on top of a layer of 225 to 450mm diameter rip rap. The rip rap increases the porosity of the revetment and protects the filter layer.

A multi-layer revetment, on the other hand, is constructed with two layers of primary armour, a secondary armour layer then rip rap. The second primary armour and the secondary armour provide protection to the revetment should the layer of primary armour stone fractured or dislodge. Less care is required in the placement of the individual armour stones in the multi-layer revetment than in the single layer revetment. A single layer revetment is much cheaper to construct than a multi-layer revetment, but will also have a higher annual maintenance cost. Cost estimates of the various types of protection are provided in Section 4.8.1.

To ensure stability of the armour layer, a revetment should not be constructed steeper than 1.5 horizontal to 1 vertical. A slope of 2:1 is preferred. The toe of the revetment should be excavated into the bottom till and the largest armourstones used within the revetment should be reserved for use as toe stones.

Figures 4.1 and 4.2 show a crest elevation of 180.6 m GSC. This is the design wave uprush level for a revetment having a toe elevation of 171.5 m as derived in Section 3.5.2. A toe elevation of 171.5 m would be required for a revetment being constructed offshore of the bluff, and was selected as an elevation equal to the expected long term erosion depth for the cohesive profile. A revetment with a higher toe elevation and a lower crest elevation could be constructed if located inland from the water's edge, but one must be careful to consider the expected long term downcutting of the foreshore and not just its position at the time of construction.

Overtopping Scour Protection

Waves that overtop and scour the land or bank behind shoreline protection is one of the most common causes of failure of protection on private properties. It is critical that a proper filter layer be placed between the bank and stone revetment. This could be either a graded stone filter or a synthetic filter fabric as shown in Figures 4.1 and 4.2. Filter fabrics are generally easier to use when backfill material is required behind the revetment, as would be the case through most of the study area.

Depending on the crest elevation of the revetment varying volumes of water will overtop the structure. This water will gouge deep scour holes if it lands on a sand or soil surface so stone protection must be carried landward. The width of this splash protection depends upon the crest elevation and is a design detail.

Toe Scour Protection

Scouring and undercutting of the toe of the revetment must be prevented by constructing proper toe protection. Figures 3.1 and 3.2 show the revetment toe excavated into the lake bottom till and fronted by an additional armourstone. The excavation into the toe allows the natural long term downcutting of the foreshore to occur without undermining the revetment. This excavation will be filled with sand except during storm conditions. The toe stone provides lateral resistance to sliding and hence settlement of the sloped armour and prevents any scouring directly under the sloped stones. Some scouring can occur under the lakeward edge of this horizontal toe stone without reducing the stability of the sloped armourstones.

Flank Protection

The ends of a segment of a revetment are the most vulnerable and require special attention. If neighbouring properties are not properly protected it will be necessary to reinforce the end of the protection by turning it landward. If not protected by flank protection, the land will eventually erode behind the revetment, causing progressive failure. Return sections can be provided either during the original construction or later as erosion progresses. Revetments must usually be progressively lengthened as erosion to adjacent lands continues but some initial flank protection should be included with the original construction.

Different measures should be taken depending on existing and planned future site conditions. Several possible situations are addressed here.

1. When the neighbouring property is likely to remain unprotected, it is necessary to reinforce the end of the revetment by turning the end back landward.

2. If the owner of the neighbouring property intends to construct his own revetment in the near future, it is necessary to leave enough extra filter cloth beyond the end of the revetment being constructed to ensure that the neighbour can achieve a proper overlap without disturbing the revetment.

3. If the neighbour has a revetment of rock already in place, it may be necessary to obtain the owner's approval to dismantle the end of their revetment in order to achieve a good smooth connection.

   If they have a different design and have used filter fabric, it may require some ingenuity to connect the revetments while maintaining continuity of filter fabric protection with a proper overlap.

4. If the neighbouring property already has a rock revetment but without filter fabric there are two choices;

   a) turn the new revetment back into the shore as though there was no revetment on the next property.

   b) obtain the neighbours approval to dismantle at least 5 metres of their revetment and to reinstall it with proper use of filter cloth.

Note that to simply abut a revetment to a neighbours revetment that was built without a filter layer is to risk outflanking of the new revetment when the neighbour's revetment fails.

5. If the neighbour has already installed some other form of retaining wall special caution will be required. Many retaining walls presently installed are susceptible to sudden failure which could lead to simultaneous outflanking of an abutting revetment.

   If a retaining wall is soundly designed it will suffice to secure the ends of the filter cloth under the revetment to the side of the retaining wall and to pile rock against it and around the corner of the retaining wall.

   If the retaining wall is not soundly founded with adequate toe protection the owner of the wall should be advised to take appropriate steps to secure the wall. The steps would include the placement of filter cloth and a slope of rocks in front of the wall. The revetment could then be aligned so that the toe of the revetment joins the toe of the rock slope that is placed in front of the retaining wall.

   In general the builder of a revetment cannot compel the owner of an adjoining property to take any steps to secure the area of lakefront at their common boundary. However, it is almost invariably to their mutual advantage to cooperate and preferably to join forces in the protection of their properties.

4.6.2 Bulkheads

Bulkheads are vertical retaining walls which retain an area of landfill and protect it from wave action. If a bulkhead is water tight it will also provide protection against flooding from wave action and, if properly designed, against flood water levels. They may be used in areas where land reclamation is desired in front of the existing shoreline and may be used to provide a depth of water adjacent to a section of land.

A major disadvantage with bulkheads is that the vertical face reflects much more wave energy than does a revetment. This often leads to an excessive amount of scouring at the toe of the bulkhead. Existing fronting beaches can be lost due to this scouring effect. A second disadvantage, which is less common but actually more critical, is that a bulkhead which is breached and fails in one spot will rapidly fail altogether. This does not typically occur with a flexible structure like a stone revetment.

Bulkheads may be either cantilevered, anchored or gravity structures. A cantilevered bulkhead must have a sufficient penetration into the bottom soil that the soil strength can resist the loading forces applied to the bulkhead. It is only the resistance of this soil that prevents a bulkhead failure. If a cantilevered bulkhead is used it is critical that the possibility of toe scour be considered when the wall is designed.

Anchored bulkheads also require an adequate toe penetration but not as deep as cantilevered bulkheads. Most of the bulkhead strength is developed through the anchoring system but toe penetration is still required. Because scouring causes a reduction in the penetration depth it must be prevented.

Gravity structures eliminate the need for pile driving but they require considerably more width. A gravity structure develops its strength through friction between the structure and the lake bottom. They must be excavated into the lake bottom but not usually to a great enough depth to utilize any soil resistance.

Within each of the types of bulkheads there are also a number of different designs and materials which can be used. Typical types of bulkheads, commonly found on the Great Lakes include: cantilevered and anchored steel pile; anchored wood pile; post supported; cantilevered and gravity structure concrete; and cribs. Figure 4.3 shows typical cross sections for 4 of these bulkheads. Figure 4.4 shows 2 types of bulkheads which may be used in very low wave energy locations.

It is important to note however, that there are very few locations within the study area where a bulkhead could be recommended for protection of residential properties. The design wave and ice forces to which that bulkhead could be subjected will also limit the various types of materials and construction that are feasible. For example, based on cost alone, protection of a section of high bluff would best be done with a stone revetment. If however, someone insisted on installing a bulkhead then only heavy gauge steel sheet piling could be used to obtain the design level of protection.

As discussed in Section 2.2 a low wall could be constructed along the north side of Edith Cavell Boulevard to hold back surge water levels. In this instance any of the bulkheads shown in Figure 4.3 could be used but a cantilevered concrete wall would likely be most appropriate.

As with revetments a number of criteria must be considered in the design of a proper bulkhead. These include retention of the backfill material, prevention of toe scour, flank protection, durability, backfill drainage, resistance to design forces, and impact on coastal processes. These criteria are also discussed following.

Sheet Pile Bulkheads

Both steel and timber sheet piles may be used to construct either cantilevered or anchored bulkheads. Cantilevered timber bulkheads, however, would not likely be feasible within the study area because of the required length of penetration into the bottom till. Cantilevered steel sheet piles would be most feasible because of the difficulty associated with placing anchors when the backshore area is a bluff. An anchored bulkhead could be used to retain a backfilled area but there is no location within the study area where we would recommend that this be done.

Concrete Bulkheads

Concrete may be used to construct both gravity bulkheads and cantilevered bulkheads. Figure 4.4 shows both a poured concrete cantilevered bulkhead and a poured concrete gravity bulkhead. Gravity bulkheads may be either a poured concrete wall or a backfilled concrete container such as large pipes.

Large concrete pipes, must be backfilled with granular soil and properly filtered and must be constructed on a stone pad trenched into the lake bottom. A pipe bulkhead would not usually last as long as a poured concrete bulkhead because of deterioration of the pipe.

It would not be practical to use a concrete bulkhead to protect an exposed bluff section because of the mass of concrete that would be required. A vertical wall exposed to full design conditions would have to be 8.5 m high (Section 9.5.2). A concrete bulkhead would be suitable within Reach 4 of Port Stanley if steps are taken to ensure retention of the sand beach. A cantilevered concrete bulkhead such as that shown in Figure 4.4 a would be suitable for constructing the low wall described for Reach 1 of Port Stanley (Section 5.2).

Post Supported Bulkheads

Post supported bulkheads consist of regularly spaced posts, driven into the ground, supporting a wall facing. Typical post supported walls are either timber posts with timber facing spiked to the back of the posts, or steel H pile posts with railway ties placed between the piles. This latter type of bulkhead is shown in Figure 4.3 (d). When timber issued it is important that a proper tie back and/or anchoring system be used to prevent flotation of the timber.

The U.S. Army (1981) also suggests that post supported bulkheads may be constructed out of wire fencing and stacked bags. Fencing may be attached to properly anchored posts and used to support sand or grout filled bags stacked on the landward side of the fence. This method would require trenching of the posts and bags, proper scour protection and a proper filter. Backfill would not be required behind the sand bags. Overall this type of wall is not appropriate for long term shoreline protection but may well form one of the best types of emergency response bulkheads for a relatively low wave environment area, such as the updrift section of fillet beach in Reach 1 of Port Stanley. Figure 4.4 b shown a wire fence and stacked bag bulkhead.

Retention of Backfill Material

Retention of backfill material is essential for the stability of any bulkhead using tie backs and anchors and for cantilevered concrete bulkheads. Other bulkheads, (ie. gravity bulkheads and cantilevered sheet pile bulkheads) can withstand some loss of backfill material. However, as one of the functions of a bulkhead is to retain the backfill material, it is clear that even if a bulkhead remains standing if it has not retained the backfill, then it is not working properly.

If a bulkhead might be overtopped, and any bulkhead within the study area should be designed as if it will be, then a splash apron will be required. Typically a splash apron is constructed by placing rip rap size stone (200 to 450 mm diameter) for a distance of 2 to 5 metres behind the back wall of the bulkhead. This stone must be placed on filter cloth so that the soil underneath cannot wash up between the stones.

The extent of the splash protection required is related to the crest elevation of the bulkhead. Bulkheads designed to provide a 100 year level of protection will usually incorporate a splash apron at least 2 to 3 metres wide. Lower elevation bulkheads will require a wider and preferable heavier (i.e. larger stones) splash apron as those aprons will be more susceptible to damage from overtopping water. Overtopping water, defined as the "green" water which passes over the structure crest, has a much higher damage potential than the "white water", or splash, which passes over a full height crest. The design crest elevation of a bulkhead is directly related to the highest waves which strike the structure, which depends upon the water depth in front of the structure. The potential for future downcutting of the nearshore profile must be considered when estimating the toe water depth. This issue is covered in more detail in Section 4.5 where wave uprush levels are derived.

If the bulkhead is intended to provide flood protection then special design consideration is required. If the bulkhead is required to withhold water below the flood level then no drains or weep holes should be placed through the bulkhead. This in turn requires the wall to be designed assuming fully saturated backfill material. If only wave uprush is to be held back or if flood prevention is not a requirement of the bulkhead then drains should be placed along the bottom of the bulkhead. These drains must be filtered so that backfill material is not lost through the drains.

If drains are used then it is most likely that a wedge of free draining backfill material will be placed behind the bulkhead (see Figure 4.4). A filter layer may be required between the free draining wedge and other backfill material.

Prevention of Toe Scour

Toe scour protection must be included with any bulkhead because of the amount of energy reflected from vertical surfaces. As with overtopping scour protection, toe scour protection is provided by placing stone on filter cloth. Toe scour protection stones, however, generally have to be larger than overtopping protection stones, say 300 to 600 mm diameter. The width of the toe scour protection varies depending on the location of the bulkhead.

Ensuring that proper toe protection is provided is extremely important for the existing bulkheads within the study area. For example, the bulkheads constructed in Reach 5 of Port Stanley have a toe elevation of about 174.0 m GSC. If the nearshore till profile is not protected, then it is expected that in the long term the till will erode to about elevation 171.5 m. It is shown in Section 4.5.2 that the crest elevation for a vertical wall with toe elevation 171.5 m has to be 2.5 metres higher than for a wall with toe elevation 174.0 m. For these bulkheads (and the revetments in the area) overtopping of the crest will occur in the future if the bottom is not armoured to prevent gradual downcutting of the till. This armouring could be accomplished with quarried stone or concrete.

The toe protection should, if possible, be placed in an excavated trench so that it lies flat. If the toe is not excavated enough to place the protection flat, then the protection should be sloped against the bulkhead, but not steeper than 2 horizontal to 1 vertical.

Flank Protection

The ends of a bulkhead are the most vulnerable and require special attention. If neighbouring properties are not protected, or are poorly protected, it will be necessary to reinforce the end of the bulkhead by turning it landward. If not prevented by flank protection, the land will eventually erode behind the bulkhead, increasing the chance of failure. Once a section of bulkhead fails during storm conditions, the remainder usually collapses soon after.

Unlike revetments it is not usually a simple matter to continue the flank protection landward as neighbouring erosion progresses. A sufficient length of flank protection should therefore be included with the original construction. If the neighbouring property does erode past the flank protection then the protection must be lengthened. This additional protection does not necessarily have to be the same sort of protection as the original bulkhead.

Durability and Resistance to Design Forces

Bulkheads must be designed with enough strength that they can withstand both wave and ice forces. They should be designed so that the bulkhead itself can withstand these forces without relying on the passive resistance of the backfill soil. Design wave forces should be determined assuming waves break directly on the bulkhead. Design wave heights are presented in Section 3.5.

Unfortunately it is not possible to determine accurate design ice loading forces on open sections of the Great Lakes. Experience has shown that structures designed to withstand wave forces on exposed sections of the Great Lakes will generally survive ice forces but this should not be blindly relied upon. This essentially means that designing for ice forces within the study area should be based on local experience, that is by reviewing which structures have withstood ice forces and which structures have failed.

Durability of steel and concrete bulkheads, if built with proper materials, is generally not an issue. Wooden structures on the other hand can rapidly deteriorate if subjected to repeated cycles of wetting and drying for significant durations. This applies to both pressure treated and untreated timbers although untreated timbers will decay faster.

4.6.3 Flood Berms

A flood berm is a relatively impervious structure designed to prevent flooding due to short term events such as storm surges and wave uprush. If the berm is exposed to direct wave action then it must be protected against erosion. This is usually done by constructing a revetment on the exposed face of the berm. Figure 4.5 shows one possible cross section for a flood berm. This section would apply to a berm constructed within Reach 1 of Port Stanley, as proposed in Section 5.2.

The berm shown in Figure 4.5 should have a core of relatively fine grained fill material compacted to reduce the rate at which water flows through it. The berm is not intended to withhold high static levels and has been assumed to be constructed above the 100 year static water level. The berm crest elevation (178.3 m GSC), toe elevation (174.0 m) and armour stone revetment shown are based on the assumption that the beach in front of the revetment will be eroded during the design storm. This could occur along the very western limit of Reach 1 but would not, assuming only one storm, occur closer to the harbour. In that area the revetment toe could be as high as 175.0 m, the crest could be as low as 177.5 m, and the berm could be protected with a rip rap revetment rather than armourstone.

4.6.4 Groynes

A groyne (U.S. sp. groin) is a narrow structure projecting from the shoreline, normally at right angles, to hold beach material in place. Groynes are used to:

1) Create or promote the build up of beaches on eroding shores where beaches do not naturally occur.

2) Hold existing beaches in place when they would otherwise erode.

3) Increase the width and height of existing beaches.

The Port Stanley harbour breakwater and the Elgin area water treatment plant are good examples of structures which acts as groynes.

Groynes have always been an attractive form of shoreline protection on the Great Lakes because where they are successful they can create or enhance recreational beaches whereby greatly increasing the value of shoreline property. Unfortunately, there are also many risks and problems associated with groynes. They may not work as intended and they may cause damage to other properties.

The interactions of groynes with the natural coastal process is complicated and still not fully understood. However, there are some important principles that are quite clear.

Groynes build beaches by trapping coarse sand and gravel that would otherwise have been transported past the area by wave action. **If there is no natural alongshore transport of suitable beach material groynes will not work.** Groynes do not work well at places where waves break straight on the shore because the main directions of movement of beach material are then directly onshore and offshore rather than alongshore. Groynes require a continuing supply of suitable new beach material to fill the groyne cells when they are first built and to replace the inevitable losses that occur after filling is complete.

The normal source of supply is the natural movement of littoral drift along the shore caused by waves breaking obliquely on the shoreline. Not all littoral drift is coarse enough to be retained by groynes. The particle sizes that can be held depend on local conditions, wave intensity and the length of the groynes. Coarse material is more easily retained than fine material. **Groynes do not "attract" beach material.** They can, at best, entrap only a portion of the material that is being moved past them by the waves and currents. This contradicts a common misconception. Unless suitable beach material is already present at the shoreline and is moving along the shore, it cannot be captured by the groynes.

An important reason why groynes work in some area is because shore erosion is allowed to continue in other areas. In the Great Lakes the main source of littoral sediment is erosion of the shoreline itself. Therefore, in order to maintain a groyne protection scheme in one area it is necessary that a sufficiently large part of the updrift shoreline remain unprotected and continues to erode.

Unless artificially filled, groynes cannot be expected to work east of Port Stanley. The volume of material being naturally transported along that stretch of shoreline is too small to fill a groyne within a reasonable time frame. For example the Elgin Area water treatment plant, built in 1967, has been trapping sediment on its updrift (western) side. The trapped beach would not, at present prevent erosion of the backshore during the design storm event.

There is sufficient material moving alongshore to fill groynes west of Port Stanley but those groynes would have to be slowly lengthened to avoid trapping too much material at once. Trapping too much material would cause erosion downdrift of the groynes. These groynes, however, would not be capable of providing protection to the backshore during a design storm event and should therefore not be considered for use.

When constructed along shorelines where there is a sufficient supply of littoral material to fill the groyne there is a very high probability that construction of a groyne would either cause or exasperate a downdrift erosion problem. This happens when the stability of the downdrift shoreline depends upon the supply of the material which is retained by the groyne. Artificially filling the groyne so that it bypasses littoral drift will prevent a larger scale downdrift erosion problem but a very local effect may still be experienced.

4.6.5 Headland Bays

Headland bays are stable sections of shoreline between two armoured points referred to as headlands. The shoreline between the two headlands can achieve a static equilibrium shape where no input littoral drift is required to stabilize the shoreline. It can also obtain a dynamic equilibrium shape where it will bypass some of the material entering the system from updrift. Headland bays may be artificially constructed by armouring appropriate sections of shoreline. This armouring may be done with any of the previously discussed shoreline protection methods.

Headland Bays also occur frequently in nature. There are a number of locations of these on the Great Lakes, including Crystal Beach, Long Beach and Camelot beach along the north eastern shore of Lake Erie.

Many people have studied these natural shoreline features and a number of means of predicting their formation have been derived. One of the most widely recognized of these people is Silvester. Figure 3.6, which shows the main components of a headland bay (termed crenulate shape bay by Silvester) has been taken from Silvester (1976). The bay indentation, headland spacing and orientation of the tangent sections of the bay may all be related to the incident wave climate.

The headland bay protects the backshore by forming a beach between the two headlands. This beach dissipates the incoming wave energy and prevents erosion of the backshore. The downdrift headland must therefore extend far enough into the lake that it can anchor the toe of the beach. The extent to which the toe extends depends on the sediment size of the beach material. For a headland bay on an eroding bluff shoreline, as being considered here, the size of the beach material depends upon the composition of the bluff and the extend of sediment bypassing the updrift headland into the bay.

One of the biggest advantages of headland bays is that they will stabilize the shoreline once the headlands have formed. For a shoreline management plan this means that directing development to certain strategically selected locations will, if that development requires armouring of the shoreline (either at the time of development or later), have a much broader impact. For example, by armouring the pumping station at the bottom of the bluff at the Elgin area water treatment plant a hardpoint downdrift of Port Stanley was formed. Parker and Quigley (1980) show that the shoreline between Orchard Beach and the treatment plan is presently forming a headland bay shape. This shoreline will eventually stabilize.

4.6.6 Breakwaters

Breakwaters are constructed parallel to the shoreline at some distance offshore. They either prevent the passage of waves, thus protecting the shoreline, or they dissipate some portion of the wave energy to decrease potential sediment transport rates. Decreasing sediment transport potential causes the build up of sand deposits in the lee of the breakwater, protecting the shoreline with a beach. For sedimentation to occur, however, the reduced sediment transport potential must be less than the sediment supply rate.

Breakwaters may be either fixed or floating. Fixed breakwaters are constructed directly on the lake bottom and must be designed to criteria similar to that for a revetment; structural stability of the armour, overtopping, and toe scour. Floating breakwaters will not work within this study area because of the incident wave periods. Fixed breakwaters could be made to work within the study area but the resulting structures would have to be so large that they would be impractical. Because the sediment transport rates are supply limited the offshore breakwaters would have to dissipate the vast majority of the incoming wave energy to produce potential sediment transport rates below the supply rates. The waves breaking on the breakwaters would be larger than those breaking on a shoreline revetment and so the wave runup would be higher, leading to a higher required crest elevation. This in turn would require a tremendous volume of material to construct the breakwaters, leading to high construction costs. The high volume of material is directly related to the water level fluctuations in that extreme water levels must be considered for the design, giving significant water depths and hence a significant breakwater width at the base.

It may therefore be concluded that offshore breakwaters are not an appropriate shoreline protection method within the study area.

4.6.7 Maintenance of Structures

Maintenance of any structural protection is a fundamental requirement if that structure is to have a significant design life. Even structures designed in accordance with the 100 year design criteria (Section 4.1) will not last anywhere close to 100 years if they are not maintained. The life expectancy of a typical structure cannot be generalized because of the specific nature of the need for maintenance.

There are essentially two types of maintenance which will need to be undertaken; a general ongoing repair of the structure with time and the specific repair of structures damaged by severe storms. Because, statistically, almost any severity of storm can occur there is always a risk of a storm more severe than the 100 year return period storm. Such a storm could damage a structure designed to the 100 year standard. This type of damage cannot be reasonably prevented because it is not usually reasonable to design a structure to withstand more extreme return period storms.

Ongoing maintenance is generally required even when there has been no specific damage associated with a severe storm. If neighbouring properties are not protected then flank protection will probably have to be extended. Armourstones can crack from frost and need replacement. The level of maintenance needed will likely increase with time. A revetment which is 60 years old will need to have more stones replaced, on an annual basis, than a similarly constructed 5 year old revetment.

It is feasible to reduce the level of ongoing maintenance required by overdesigning the structure at the time of construction. For example, consider a two layer revetment being constructed instead of a single layer revetment. In Section 5.7 it is shown that the construction costs of a multi layer and single layer revetment are estimated to be $4,700 and $3,100 respectively, a difference of $1,600 per metre. It was also estimated that the annual maintenance costs of a single layer revetment would be in the order of $5 per metre whereas a multi layer revetment could be assumed to have no annual maintenance cost. Considering the $5 annual maintenance cost over 100 years, discounted to present value using the methods employed during the benefit cost analysis (Section 5.7) would add only $71/m to the present value of the single layer revetment. This is much less than the extra $1,600 per metre it would cost to build the multilayer revetment. In fact it would require an annual maintenance cost exceeding $120 per metre before the present values of the single layer and multi layer revetments were the same.

Based on the preceding example it may be seen that it is much more economical to construct protection which requires ongoing maintenance than it is to overdesign the protection so that such maintenance is not required. It must be noted that the above example does not consider the potential for a higher degree of damage from a severe storm to a single layer revetment than to a multi layer revetment. It may be assumed, however, that as both types of revetments would be designed for the 100 year return period storm that the cost savings of the single layer revetment outweighs this potential.

The exact form of maintenance required depends on the type of structure as well as site specific circumstances. Generally armourstone and possibly poured concrete block revetments will require the greatest amount of maintenance as the armour or blocks are susceptible to fracturing from frost. Once a stone or block has been cracked, excluding minor cracks around the periphery, it should be replaced. Typically revetments require replacement of individual armour units every 5 years or so. Irrespective of the type of structure in place, homeowners should perform a visual inspection of their shoreline protection structure at least twice annually. These inspections would be best done in the early spring and late fall.

Four main items should be examined, the soundness of the primary armouring, the splash apron, the toe and the structure flanks. The primary armour, which includes steel and concrete for bulkheads and wood frames for cribs, etc., should be stable and appear to be sound. If the splash apron has been damaged it may indicate that either larger stones or a wider apron are required. The toe of the structure should be inspected for any signs of undermining. Apparent losses of backfill material may indicate undermining. Finally, the flanks should be viewed to see if they need to be extended to protect against increased exposure due to eroding adjacent land.

If a homeowner is unsure about any of the results of his/her inspection then they should be able to consult with a conservation authority engineer. We also recommend that homeowners be encouraged to consult the conservation authority before they authorize a contractor to proceed with any maintenance work on a protection structure.

In order to allow access to the structure we recommend that a maintenance access width of 10 metres be provided. This access should extend along the length of the structure to a municipal road. It is also important that the access route not be obstructed. This includes obstruction by trees and shrubs even if the homeowner indicates that he would be willing to remove them, as they might prevent the correction of a problem which the homeowner feels is not serious enough to warrant removal of the obstruction.

The conservation authority should also work with the municipalities to develop plans to secure access right of ways to the shoreline. This may require securing right of ways across the front of properties, along driveways, etc. A planned access program should be in place for consideration as approval for new development is granted.

4.6.8 Impact of Structures on Coastal Processes

If protection of a segment of shoreline is to be permitted, then the protection must be designed to minimize its impact on adjacent and downdrift shorelines. Generally, two types of impact will have to be considered: local impacts associated with altered nearshore wave or current patterns and the interruption of littoral drift; and regional impacts associated with the interruption or starvation of littoral drift within the littoral cell. Littoral starvation is defined as the reduction or elimination of potential littoral drift material by protecting an otherwise eroding bluff. The individual property owner should be responsible for demonstrating that there will be negligible local impact.

While considering local impacts it must be realized that the effects of a structure need to be minimized, not prevented. It is not possible to construct protection which will have no impact on local coastal processes. The extent to which impact will be tolerated must be clearly set out so that individual property owners are able to determine what will not be permitted. We are proposing within this plan that in order to obtain the necessary approvals to construct shoreline protection structures that property owners be required to submit an impact statement to the conservation authority. Such statements will be prepared in direct response to detailed terms of reference provided by the authority to the property owner. Section 9.4 provides a more detailed description of the elements of the proposed impact statement.

With respect to the impact of structures on coastal processes the property owner's local impact assessment should be required to demonstrate three key items. These are:

1. That there will be no increase in the long term erosion rate on neighbouring properties caused by the proposed structure.

2. That the proposed structure will not cause damage to adjacent structures.

3. That the proposed structure will in no way have any detrimental effects on the environment.

The length of shoreline considered to be "local" to a proposed structure depends upon the exact nature of that structure and the specific shoreline conditions. Each impact assessment should therefore contain a definition of the length of shoreline it covers. In general, however, local impacts will normally be experienced over a distance of up to about +/-5 times the "length" of the structure. The typical length of a structure depends upon the type and location of that structure and would have to be addressed within the local impact assessment.

The conservation authority should, through this plan and other more detailed studies, decide which, if any, regional impacts will be permitted. The issues of regional impact are more far reaching than should be considered by an individual property owner. There are a number of issues which must be examined, such as the extent to which littoral starvation can take place before its effects are relevant and what, if any, mitigative efforts should be required.

The criteria required to be used to resolve these issues are not defined. For example, the extent to which an eroding shoreline is developed plays a major role in accessing the regional impact of littoral starvation. On its own a small protection structure will not effect the regional coastal processes within most of the littoral sub cells within the study area. If, however, many structures are present the cumulative effect of littoral starvation may well be noticed. Questions which must be considered are whether a specific project should be judged on its own merits or as part of an overall plan? If it is the latter then should potential future development be considered or not? If yes then how much?

One possible way of addressing these issues is to shorten the time frame over which the decision making process is applied. For example, the plan could recommend that, for the time being, people be allowed to protect eroding bluffs as long as local impacts have been adequately considered, but that the decision to allow the construction of new protection be reviewed every 5 years. If the bluffs were to be protected to the extent that it were felt that further development might cause regional effects then protection of the bluffs would have to be halted.

One disadvantage of this method, is that it creates a first come first serve situation. The alternative, however, is to deny someone the option to protect their eroding shoreline on the basis that protecting the entire littoral cell would have detrimental impacts when the protection of that cell will not happen within the foreseeable future.

Whether or not being able to protect an eroding shoreline is an option to which people should be entitled, even if it might add to a cumulative regional impact cannot be clearly resolved within this plan. The conservation authority must choose its own planned course of action based on its own interpretation of this issue, any required legal advice and any direction provided by the province in its pending shoreline management policy.

The authors of this report, however, favour the principle of allowing protection to proceed now, monitoring the shoreline within the littoral cell, reviewing the potential for regional impacts on a more frequent basis, and disallowing further protection once it is judged that there is a risk of regional impacts. Unfortunately, this cannot be realistically done given the current state of knowledge of coastal processes on cohesive shorelines. Opinions can be provided discussing some of the effects of protecting significant proportions of a cell but it will not be possible to provide a detailed assessment of the potential for regional impacts until more is known about cohesive coastal processes. For example, after some detailed study reasonable estimates could be made about the probable effect of littoral starvation on the beaches at littoral cell sinks but no one can yet accurately predict what would happen to the shoreline between the protected source and the sink.

4.7 Floodproofing Structures and Properties

Floodproofing may be defined as structural changes and/or adjustments incorporated into the basic design and/or construction or alteration of individual buildings, structures or properties to protect them from flood damage (MNR and MMA, 1988). There are two general types of floodproofing, defined as follows:

". dry floodproofing
- the use of fill, columns, or design modifications to elevate openings in buildings or structures above the regulatory flood level

or

- the use of water tight doors, seals, berms/floodwalls to prevent water from entering openings below the regulatory flood level.

. wet floodproofing

- the use of materials, methods and design measures to maintain structural integrity and minimize water damage

- buildings or structures designed to intentionally allow flood waters to enter.

There are two techniques to floodproofing, defined as:
. active floodproofing

- floodproofing techniques which require some action prior to any impending flood in order to make the flood protection operational, i.e. closing of water tight doors, installation of waterproof protective coverings over windows, etc.

. passive floodproofing

- floodproofing techniques which are permanently in place and do not require advance warning and action in order to make the flood protection effective."

Within their guidelines MNR and MMA (1988) state that dry passive flood protection should be provided where at all possible. They note that this may not always be possible but should be implemented to the fullest possible extent. Wet floodproofing may be applied to non residential structures, such as garages. Minor alternatives or additions to existing buildings are considered to be the only development which could be floodproofed to a level less than the regulatory flood, but, as a minimum, the alternation or addition should not be more flood vulnerable than the existing structure. Although these policies were not developed to be applied to lake induced flooding we consider them to be sound and have chosen them to be applied as guidelines for dealing with the flooding encountered here.

The only areas susceptible to flooding are reaches 1, 2 and 3 within Port Stanley. From the field survey (Section 8.8) it was found that only a few dwellings had visible structural openings below the design flood levels. It was evident from the public questionnaires, however, that a number of the dwellings had basements that flooded. It is likely that only a few of the dwellings would experience flooding of the main floor during the design storm. A number of the properties are also flood prone.

Dwellings with potentially flood prone main floors should be floodproofed with dry passive methods. The most effective way of doing this is by raising the dwelling and surrounding land although not all dwellings can be raised easily. If this can be done, then the dwelling should be raised so that the lowest opening is at least 30 cm above the estimated flood level.

Whether or not it is feasible to raise a dwelling depends upon the construction of the dwelling. For example, it would be much easier to elevate a small cottage supported by piles or blocks than a house with a concrete foundation. Homeowners should consult with a qualified professional to determine if their dwelling can be raised.

The land around the dwelling should be raised by importing suitable fill material. This will further reduce the risk of flooding the dwelling and, depending on how high the land is raised, will reduce damage to the land during a flood. If the land is not also raised to at least the design flood elevation, then the dwelling should be raised so that only the support columns are below a 30 cm freeboard above the design flood elevation. By raising all parts of the dwelling to this height, flood waters as well as floating debris will be able to pass under the dwelling without obstruction.

Irrespective of whether or not fill is placed the footings of the raised structure need to be properly designed. This design must consider a stable base to resist erosion by flood water and rainwater runoff. This could be accomplished, for example, by placing the footings on a crushed stone pad rather than the native sand.

While it is best that the land be raised to an elevation higher than the flood level, eliminating the potential for damage during the design flood, it must also be recognized that this is not always possible for all property owners. If the land around a dwelling is not raised then, as a minimum, both pedestrian and vehicular access and egress should be provided. The minimum allowable elevations for pedestrian and vehicular egress is 176.55 m GSC (0.25 m below the flood elevation) if the area is subject to direct wave uprush. Flood depth criteria are discussed in Section 3.8.

If a structure cannot be raised then dry passive floodproofing may be provided by permanently sealing or welding all exterior openings below the flood level or by constructing a flood berm around the structure. If the structure is to be sealed or welded, then the building footings and foundation walls must be of waterproof construction or must be capable of being waterproofed. For example a building must have a sound, sufficiently heavy, concrete base slab, or must be well founded on bedrock.

Special attention must be given to any concrete floor slab when a building is being floodproofed. Floors are generally designed assuming loading from the surface only. If a building is floodproofed, however, then it must be assumed that seepage below the slab can occur and that the slab is therefore being subjected to hydrostatic forces equal to the head of water from the slab grade to the water surface. This hydrostatic pressure can apply a significant upward loading to the bottom of the floor slab. If the floor slab is not reinforced to withstand this loading it will crack. MNR and MMA (1988) suggest that sealing not be considered when the flood depths exceed 0.8 m., as discussed in Section 3.8. If sealing is carried out for such a depth then the design should be reviewed by a professional engineer.

Structures with the main floor above the flood level but with a basement below the flood level must be considered separately. Placing fill around the structure to raise the elevation above the flood level will reduce the risk of flooding but will not ensure that the basement will be flood free. Considering the local soil conditions one could expect the ground water level within the flood prone area to approach the elevation of the instantaneous water level on the lake (ie surge but no uprush). The land up to this elevation could easily be saturated by the combination of flood waters, rain water and potentially from ground water originating from the bluffs.

If the basement floor elevation is more than 0.8 m lower than the 1:100 year surge elevation of 175.5 m GSC then wet floodproofing methods should be considered. This essentially means that a flood event should be expected and the use of that space should be carefully planned.

Floodproofing design should be carried out by a professional engineer skilled in floodproofing measures where the following conditions apply:

- any wet floodproofing is proposed (irrespective of any other circumstances);
- dry floodproofing is proposed and:
  - . the product of flood depth and velocity is equal to or greater than 0.4 m2/m or where flood depth exceeds 0.8 m or where flood velocity exceeds 1.7 m/s;
  - . floodproofing through the use of fill exceeds depths of 1.8 m or velocities exceed 0.8 m/s;
  - . where berms and floodwalls in excess of 1 m in height are proposed.
- . where piles, columns or posts and proposed.

4.8 Summary of Prevention and Protection

There are two basic responses which may be taken with respect to shoreline erosion and flooding problems; relocating endangered structures and roadways, and taking actions to remedy the existing problem. Remedial solutions may be classified as structural or non structural. Non structural solutions include sand fill, use controls and dune management. The recommended structural solution is a stone or concrete revetment.

Non structural protection is preferred to structural protection, but is viable in fewer places. Either revetments or bulkheads can provide good structural protection but revetments are preferred. Bulkheads will have a detrimental effect on fronting beaches because of wave reflection. Revetments can sustain partial damage without a total failure, but once a bulkhead begins to fail total failure soon follows. Finally, if less than a full design level of protection is initially constructed and improvement to the protection is later required, a revetment can be upgraded much more easily than a bulkhead.

Before constructing any shoreline protection structures the landowner must obtain, as a minimum, approval from the Conservation Authority. As part of the process of obtaining this approval an impact statement should be required. Approvals may also be required by the provincial and federal governments.

Headland bays are an effective means of stabilizing longer reaches of shoreline but are not economical solutions for individual property owners. However, if future development of the shoreline is strategically planned and appropriate sections of the shoreline are protected, headland bays can form. This will provide the long term benefit of a stable shoreline.

Floodproofing of a structure and property may be achieved with either passive or active, wet or dry floodproofing methods. Dry passive methods are recommended for the main floor of all dwellings within the flood prone area. Depending on floor elevations wet floodproofing may be required for some basements.

Minimum setbacks and elevations form two of the bases of the prevention component of the shoreline management plan. Erosion setbacks for a bluff are based on estimated annual erosion rates. Erosion of a bluff, unless protected, may be looked upon as a certainty. It may also be said with certainty that development setback for 100 years of erosion will not have an erosion problem for a number of years. On the other hand erosion setbacks and minimum elevations on a beach do have a risk of occurrence within any given year. It is not certain, however, that the design conditions will occur. Beach recession setbacks based on the 100 year storm event could be exceeded by a number of successive storms of less severity. Successive storms would not effect the design flood elevations.

Finally, it must be remembered that the protection methods discussed within this chapter must be properly designed before they are built. The services of a professional engineer are recommended. The cross sections shown in this report cannot be used for construction purposes.

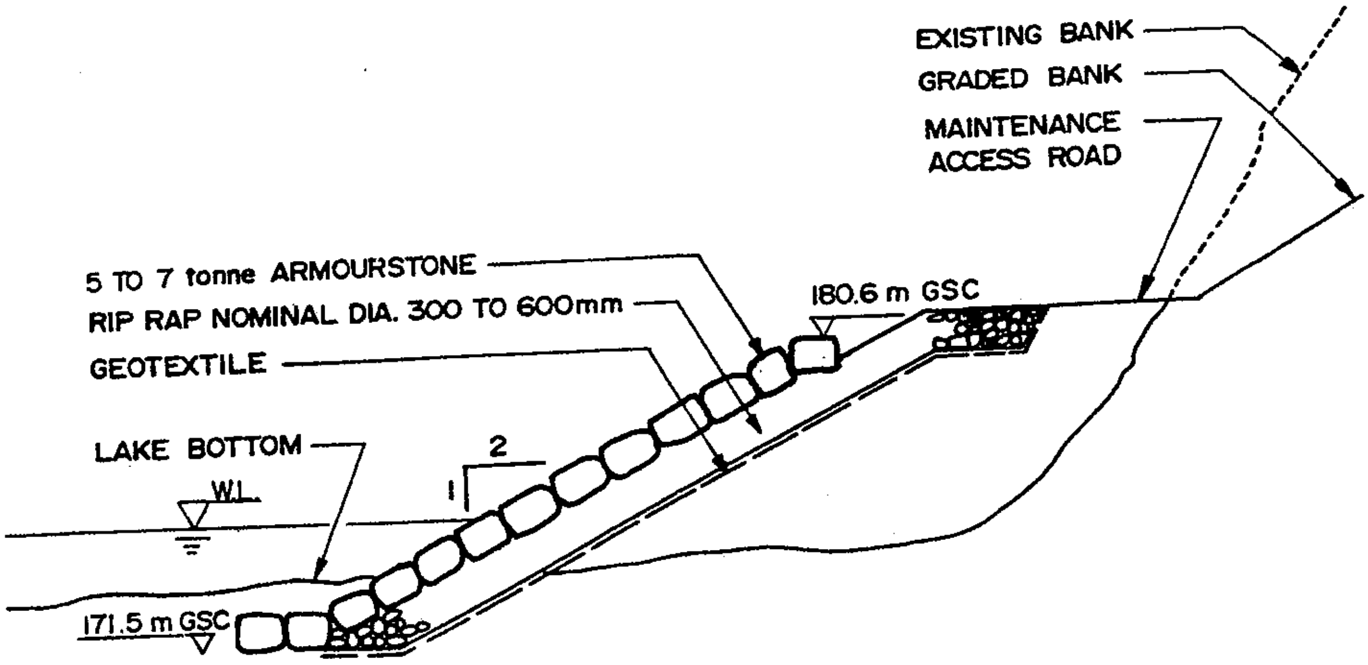


Figure 4.1 Typical Single Layer Revetment

(Not for Construction)

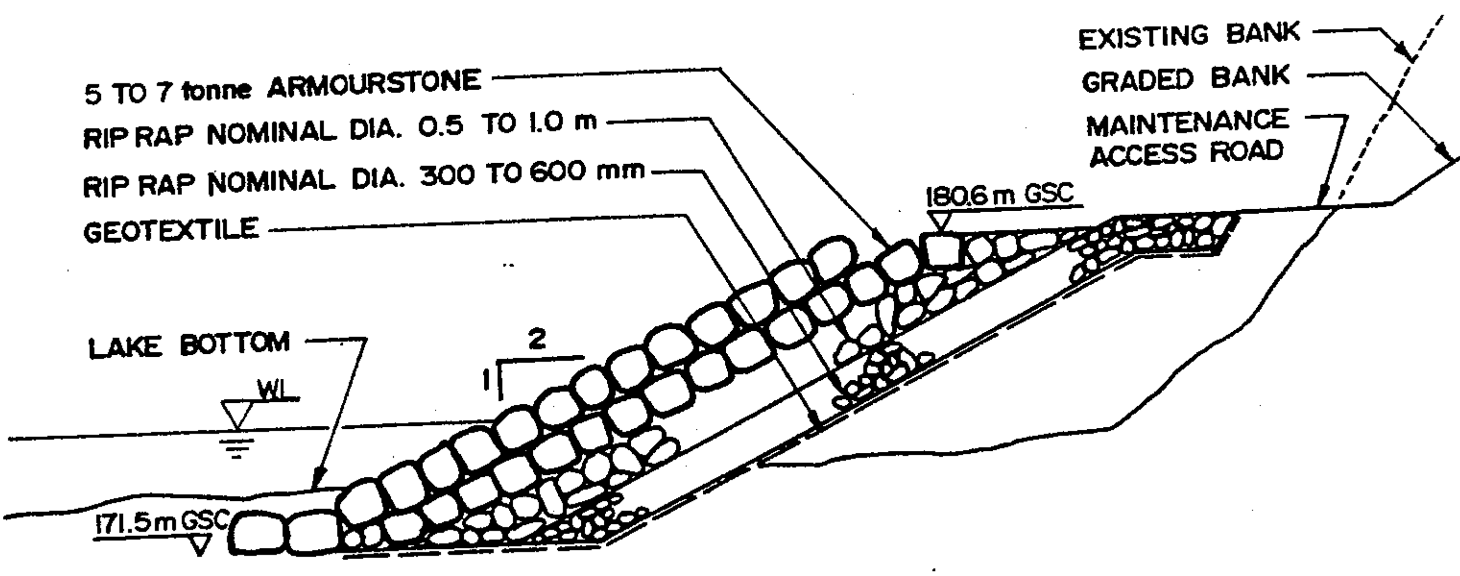
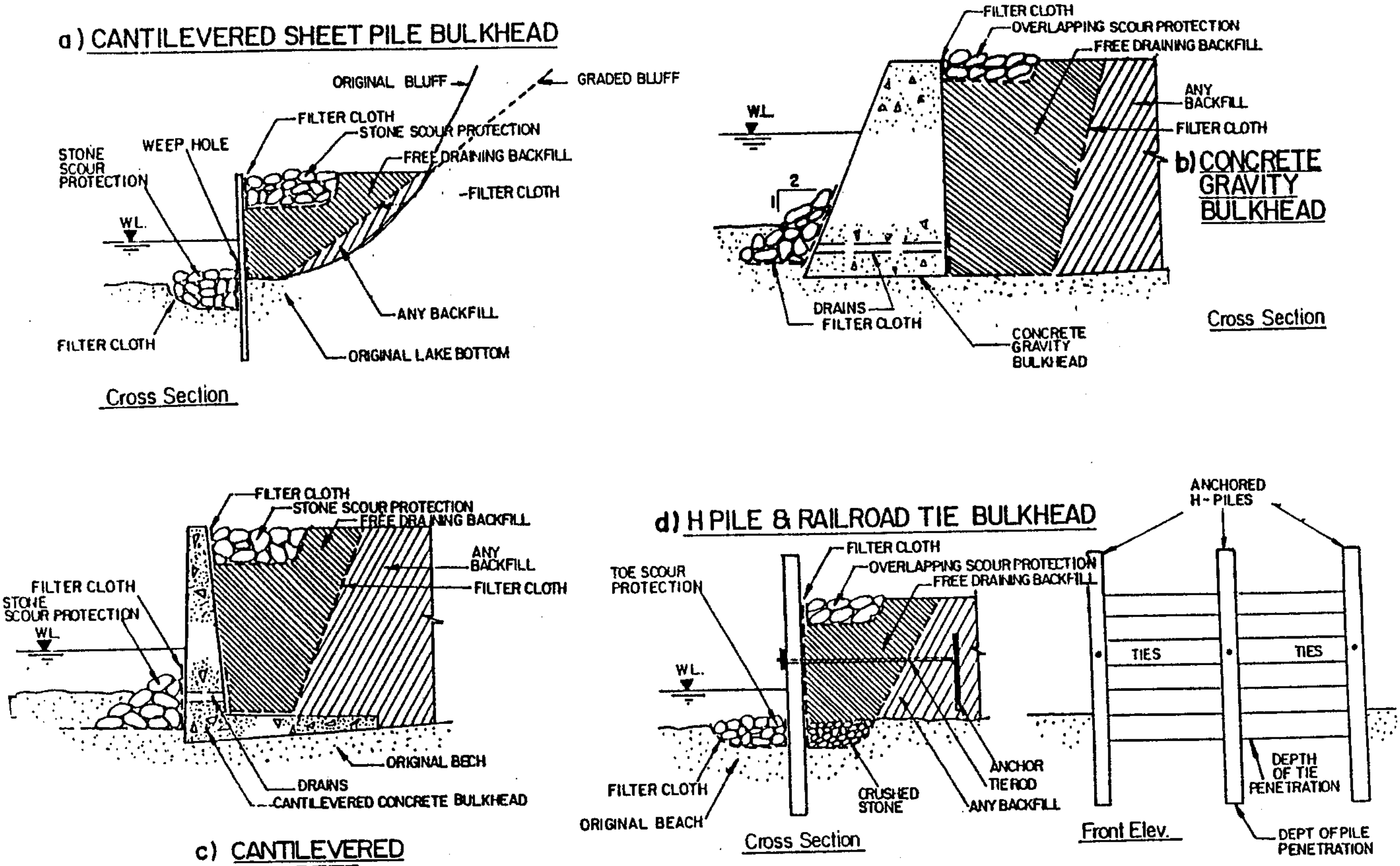


Figure 4.2 Typical Multi-Layer Revetment

(Not for Construction)

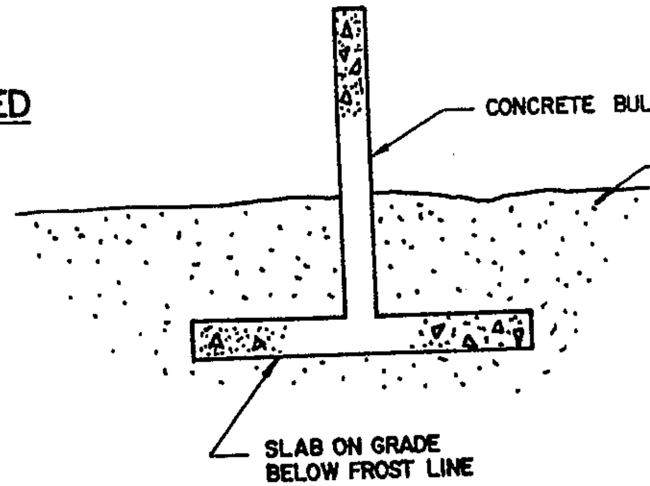


(Not for Construction)
N.T.S.

Figure 4.3
Typical Bulkheads

a) CANTILEVERED CONCRETE BULKHEAD

Cross Section



b) WIRE FENCE & STACKED BAG BULKHEAD

Cross Section

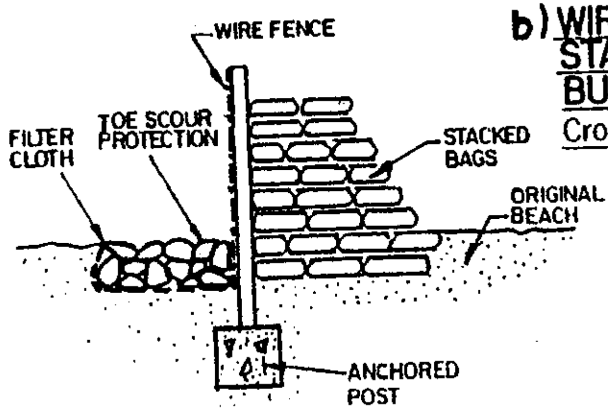


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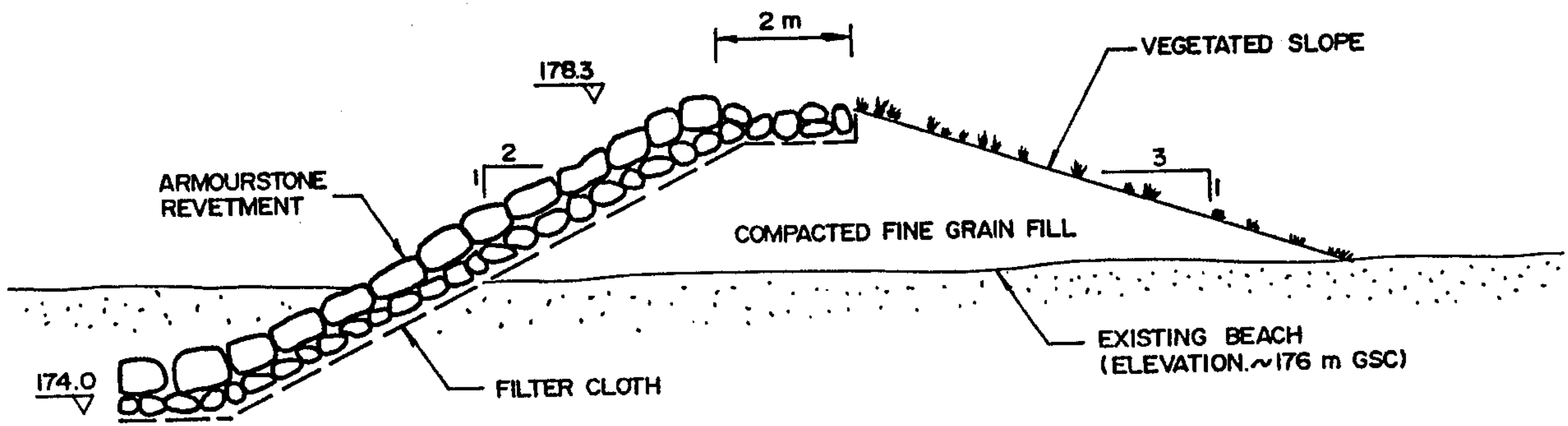
Figure 4.4
Typical Bulkheads for
Low Wave Energy Locations



(Not for Construction)

Figure 4.5
Typical Flood Berm

5.0 Preferred and Alternate Shoreline Management Concepts

The shoreline within the study area has been divided into 6 segments, one considering all high bluffs within the entire study area and 5 adjacent reaches within Port Stanley. The selection of these reaches was based on the physical characteristics of the shoreline. Figure 1.2 (from Section 1) shows the reaches within Port Stanley.

This section of the report discusses for each shoreline segment the existing conditions, the preferred shoreline management plan, alternate concepts considered, and the results of the benefit cost analysis. The preferred and alternate plans discuss both prevention and protection concepts, as applicable. These concepts are presented on a site specific basis but the concepts are only discussed in general terms. Chapter 3 of the report provides a more detailed overview of the different prevention and protection techniques.

The preferred plan for each shoreline segment is presented as a list of recommendations of specific actions to be implemented by The Kettle Creek Conservation Authority. This list is then followed by a broader explanation of those recommendations.

5.1 High Bluff Shorelines

This section considers the high bluff shorelines within the study area, which includes the Township of Southwold, the Township of Yarmouth and both the east and west ends of the Village of Port Stanley. These different sections of bluff have been grouped together because the viable shoreline management options are similar.

Structural shoreline protection is not considered to be economically viable for the high bluffs. The unit cost of protecting a section of bluff shoreline is very high but the bluffs have not been developed to the extent that the land and structures are worth the cost of the protection. Relocation and abandonment are the best protection concepts. These items are discussed in detail in the following sections.

5.1.1 Existing Conditions

As part of the Port Burwell Study Zeman (1980a) prepared a detailed description of the stratigraphy and textural composition of the bluffs between Rondeau and Long Point. The sections of his report which describe the bluffs within the Townships of Southwold and Yarmouth have been reproduced in Appendix F. As well, oblique photographs of the bluffs were taken during the field study. These photographs have been bound under separate cover in Appendix D.

The table land has been extensively developed but mostly as farmland. There are, however, a number of isolated locations where the land has not been cleared. Here the bluffs tend to have a somewhat steeper stable slope, giving the impression that these areas are more erosion resistant than the developed sections of the bluff. This is not true in the long term as it is the rate of wave erosion of the nearshore bottom which ultimately dictates the bluff recession rates. This is demonstrated by the relatively uniform recession of the shoreline, whether it is vegetated or not.

There are some residences and farm buildings located within the 100 year erosion setback from the top of the bluff. A description of these buildings is included in the shoreline inventory, discussed in Section 2.8.

Gullies identified from the aerial photographs were inventoried to identify the probable cause of gullying. This is also discussed in Section 2.8. The majority of the gullies may be described as being in the order of 10 to 200 metres long or a kilometer or more in length. The gullies measured in terms of kilometers have been present since at least 1951, and quite possibly since early in the nineteenth century (Wood, 1951).

If this is true then it must be concluded that these major gullies are related to natural drainage patterns. The field survey showed that a number of the smaller gullies were directly related to man made drainage systems. This may be seen by reviewing the existing conditions photomosaics and oblique photographs presented in Appendices A and D , respectively. It may also be seen, however, that there are a number of small gullies where artificial drainage systems were not identified. The occurrence of this type of gullies was also noted by Wood in 1951.

These small natural gullies are formed due to local groundwater conditions and bluff stratigraphy. They appear to form, stabilize, then disappear as the surrounding bluffs recede. Although they represent a sometimes significant local recession of the bluff crest they do not seem to contribute to the long term recession rate of the bluff. As such these gullies were not considered when the 100 year recession line was determined. It is not possible to determine with any certainty which, if any, of these small gullies could form into larger gullies kilometers in length. Zeman (1980b) concluded that gullies within the study area were receding at rates less than or equal to the bluff recession rate.

Gullies attributed to artificial drainage systems could become significant if that drainage system is constructed such that it continues to provide water to the head of the gully. There is no conclusive evidence, however, that artificial drainage systems will be certain to cause a serious gullying problem.

5.1.2 Preferred Shoreline Management Concepts

The preferred concepts of the shoreline management plan within the high bluff shoreline segments of the study area are for the Kettle Creek Conservation Authority to:

1. Establish a regulatory shoreland zone based on the erosion setback of 100 times the estimated annual recession rate plus a stable slope allowance.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

3. Restrict land uses and associated development within the regulatory shoreland zone to agriculture and passive recreation, or any other uses as may be approved by the municipality and the conservation authority.

4. Encourage owners of dwellings at risk of damage through erosion of the bluffs to relocate their dwellings to a location outside the regulatory shoreland zone.

5. Encourage The Township of Southwold, The Village of Port Stanley and the Township of Yarmouth to enact bylaws which will enable them to require the relocation or demolition and removal of a non-inhabitable structure, as defined herein, before it is undermined by bluff erosion. A structure is to be considered non-inhabitable when it is in imminent danger of being undermined by bluff erosion.

The location of the proposed limit of the regulatory shoreland zone is shown on the existing conditions photomosaics presented in Appendix A. This limit has been established using the calculated average recession rates discussed in Section 3.7 and a stable bluff allowance of 3.5 times the bluff height. The limit was measured from the first landward break in the bluff slope. This allowance was selected to comply with the pending provincial policy on shoreline management. A bluff height of 40 metres was assumed to apply throughout the study area so that the regulatory shoreland zone could be delineated. Actual bluff heights are not significantly different.

A total of 52 structures are presently located within the proposed limit of the regulatory shoreland zone. These dwellings were identified during the field survey discussed within Chapter 2 of the report. As shown in Section 2.8, 41 of the 52 structures are within the stable slope allowance. They may not yet be at imminent risk due to erosion of the bluff but because they are within the stable slope allowance their present risk should be quantified. The benefit cost analysis assumed an average value of $70,000 per dwelling. This estimate was provided by a local real estate agent.

No construction of shoreline protection structures have been proposed within the preferred plan because of the very high cost of those structures. As shown in the benefit cost analysis, the value of the table lands and structures within the regulatory shoreland zone do not even approach the cost of adequate shoreline protection.

5.1.2 Alternate Concepts

Alternate protection concepts were considered for 3 developed areas; Grand Canyon, between Orchard Beach and the Elgin Area Water Treatment Plant and in front of an exposed section of County Road 24 near the Yarmouth/Malahide Township line. Each of these areas is discussed separately below.

Grand Canyon

Protection of existing development either side of Grand canyon as well as a concept for stabilizing a larger area west of Grand Canyon were considered. These concepts are shown in Figures 5.1 and 5.2, respectively. Figure 5.1 shows two revetments, one either side of the mouth of Grand Canyon. The eastern revetment would cost $2.8 million and bank stabilization would cost a further $3.6 million but together these would protect only 9 dwellings. It is also possible that 7 of these 9 dwellings would still have to be moved, depending on the stable slope of the graded bluff. The western revetment would cost $1.8 million, bluff stabilization would cost a further $1.8 million and 7 of the 8 existing dwellings presently within the 100 year erosion setback could be saved. The cost benefit ratios for these revetments were estimated to be 0.15 east of Grand Canyon and 0.05 west of Grand canyon. Based on the benefit cost analysis there would be no justification for constructing these revetments.

Figure 5.2 shows a headland bay concept located to make use of the naturally protruding point west of Grand Canyon. This concept is not being proposed as a viable alternative for protecting the development at Grand Canyon because, as shown in Figure 5.2, those structures would be lost. This concept does show, however, the long term benefit which would result in the armouring of that section of bluff. Essentially, the bluffs either side of that hardpoint would be stabilized, although that could take up to 4 centuries, based on the estimated annual erosion rate. This sort of protection would not be constructed for short term gains. The benefit cost ratio for constructing the headland was estimated to be 0.00.

East of Port Stanley

The armouring of Orchard Beach (reach 5 of Port Stanley) and the Elgin Area Water Treatment Plant to the east has produced the headlands required for the formation of a headland bay system, as discussed in Section 4.6.5. Figure 5.3 shows the predicted long term shoreline position if it is assumed that the Orchard Beach and Elgin Area Water Treatment Plant headlands remain in place. That will most likely be the case.

Figure 5.3 also shows the possible shoreline position that could be stabilized if a series of smaller intermediate headlands were to be constructed. These headlands have been positioned to make the best use of potential access routes. This concept would cost more than $10 million, would preserve only an additional 70 hectares of land over a 100 year period and would have a benefit cost ratio of 0.05. This does not support construction of the intermediate headlands.

County Road 24

Approximately 1.8 kilometres of County Road 24 lie just within the 100 year recession limit. The roadway is also within the 100 year recession limit in the Catfish Creek Conservation Authority watershed but this has not been considered here. Protecting this roadway with an armourstone revetment would cost in excess of $6.5 million. Even considering that a number of farm buildings would be protected by this revetment the benefit cost ratio for building the revetment is still only 0.04. This was calculated assuming that the exposed portion of the roadway could be replaced at a cost of $2.5 million. Therefore, even if the road were at risk of erosion now rather than in 100 years it would be more cost effective to relocate the road.

5.2 Port Stanley Reach 1 West of Stanley Park

Reach 1 of Port Stanley extends from the western limit of the fillet beach to the end of the single dwelling structures approximately south of Stanley Park. This reach was considered separately from the public beach to the east because of the shape of the backshore dunes. There are essentially no dunes within the adjacent reach. The location of reach 1 is shown in Figure 1.2 (Chapter 1) and on the Port Stanley preferred and alternate concepts photomosaics in Appendix B.

5.2.1 Existing Conditions

This section of the shoreline is characterized by a low dune system which fronts a flat backshore beach increasing in width from west to east. A typical section was measured through the centre of this reach and is presented in Figure 2.3 in Chapter 2. The increase in beach width occurs because this is a fillet beach held in place by the Port Stanley harbour breakwater. The eastern half of this reach is much more densely developed than the western half. From the start of the fillet beach until almost due south of River Road, there are no dwellings to the south of Edith Cavell Boulevard. From south of River Road to the eastern end of this reach there is quite an extensive amount of development south of Edith Cavell Boulevard. The flat backshore west of Lake Street is lightly developed, with only 7 dwellings.

The vast majority of these properties have not been protected by man made structures. Three of the dwellings north of Edith Cavell Boulevard and west of Lake Street were protected by a low sandbag wall constructed along the southern edge of Edith Cavell Boulevard. The western most property owner has constructed a low beach curb out of telephone poles but the crest elevation is below the 100 year wave uprush elevation.

The beach throughout this reach is composed of a fine to medium sized sand with a thin pebble to cobble sized veneer at the water line. The beach slope from the waters edge to the dune crest varies from an almost straight 1:10 slope along the western limit to a composite profile with an overall slope of between 1:15 and 1:20 along the eastern end. The crest elevation of the dune varies considerably through this area, mostly because the dune crest has been flattened by property owners. The crest elevation in front of each property, measured during the field survey, is shown on the existing conditions photomosaics in Appendix A .

5.2.2 Preferred Shoreline Management Concepts

The preferred shoreline management concepts for reach 1 of Port Stanley are for the Kettle Creek Conservation Authority to :

1. Establish a regulatory shoreland zone based on the location of the dynamic beach zone.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place.

3. Implement a dune management program to promote the growth and stabilization of a dune system lakeward of the line of existing development.

4. Restrict new development until the dune management program (step 3 above) has been established and, once established, only allow development compatible with that program.

5. Encourage existing landowners to floodproof their dwellings to withstand the level of flooding associated with the 100 year were uprush level.

The regulatory shoreland zone should be based on the location of the dynamic beach zone because it extends either as far as or further landward than the limit of the 100 year wave uprush limit. The dynamic beach zone limit is shown on the Port Stanley preferred and alternate concepts photomosaics presented in Appendix B.

Before any new development is allowed within the regulatory shoreland zone landowners should have to demonstrate that they have overcome both flooding and beach instability hazards, without causing or aggravating updrift or downdrift problems. This should be demonstrated within an impact statement, as discussed in Section 9.2

Flooding problems associated with the 100 year uprush level should be overcome using dry passive floodproofing techniques, as discussed in Section 4.7. For properties which are or may be directly exposed to wave uprush the minimum allowable land elevations should be 176.55 m GSC. This is based on the criterion of a maximum flood depth of 0.25 m, as discussed in Section 3.8. Development not directly exposed to wave uprush may employ lower elevations as long as the flood depths do not exceed 0.8 m. and access and egress flood depths do not exceed 0.3 m. The landowners impact statement must demonstrate that the area cannot be exposed to direct wave uprush in order to use these greater flood depths.

Beach stability problems associated with the dynamic beach zone should be overcome using accepted engineering techniques and must be approved by the conservation authority. Properties north of Edith Cavell Boulevard may overcome the risk of beach instability by placing cohesive fill as long as it does not cause drainage problems on neighbouring properties. This fill be vegetated and/or protected by retaining structures. Properties south of Edith Cavell Boulevard should only use sand fill with similar characteristics to the existing beach sand.

Because serious flooding only occurs when lake levels are high the frequency of flood events, in the long term, is relatively low. This in turn means that the benefit cost ratio for protection measures will only be greater than 1 for low cost measures. Dune management will be the lowest cost alternative for this location.

Generally a dune management program will involve dune construction techniques to promote dune growth and land use controls to reduce the damage to those dunes. Use controls and dune management are discussed in more detail in Section 3.5 of the report.

The dune management program must determine an optimum width of dune formation program for the given wave and beach characteristics. Until this dune formation program has been developed, two development restrictions should be imposed; no new dwellings should be constructed south of the line of existing development and no flood barriers or flood retaining walls should be constructed south of Edith Cavell Boulevard. Once the dune management program has been established, both dwellings and barriers or walls could be allowed in locations where they will not effect dune development.

Prior to the development of the dune management program, and thus the definition of the dune development zone, we anticipate that if the recommendations of this plan are adopted and applied then little development would be allowed to take place south of Edith Cavell Boulevard. It is not likely possible, on an individual property basis, to overcome potential dynamic beach instability risks by placing only approved sand fill without any sort of fill retaining structures. It is possible, however, that these risks could be overcome by a large scale development project encompassing a number of adjacent properties.

Irrespective of the dune management program owners of existing dwellings at risk to flooding should be encouraged to floodproof their dwellings. This floodproofing should employ dry passive techniques, as discussed in Section 4.7, and assume a flood level of 176.8 m GSC. The benefit cost ratio for floodproofing existing development is 1.71. The benefit cost analysis is presented in Section 5.7.

The Village of Port Stanley's existing bylaws and Official Plan do not at present specify any specific development setbacks or elevations associated with the Lake Erie Hazard Prone Area. The village's draft new Official Plan does however state that all proposed development within the flood limit be floodproofed to the 100 year wave uprush elevation of 176.8 m GSC. The O.P. also states that "a future implementing zoning by law shall provide for adequate setbacks for buildings and structures within or adjacent to hazard land designations".

The conservation authority should work with the village to ensure that these new bylaws are in accordance with the authorities implementation plans for their new flood and fill line regulations. However, as these bylaws are unlikely to apply to existing development, the conservation authority should also ask the village to add statements to their new Official Plan encouraging property owners to floodproof existing structures to the levels required for new development.

5.2.3 Alternate Concepts

No alternative prevention concepts were developed for this reach of shoreline. Alternate protection concepts considered include a flood berm, a flood retaining wall and relocation of structures. None of these alternatives produce benefit cost ratios which suggest that the alternative is viable.

A flood berm could be constructed along the alignment of the existing dune crest from the western limit of the fillet beach to the harbour breakwater. A flood berm is a relatively impervious structure designed to prevent flooding from short term rises in the water level due to storm surge and wave uprush. A brief description of flood berms, including a typical cross section, is presented in Section 4.6.3. The alignment of the berm is shown in Figure 5.4 and the Port Stanley Alternate Concepts Photomosaics presented in Appendix A .

The berm would replace the dune management option proposed under the preferred plan for this reach. It offers the advantage that flood protection would be provided once the berm was built, whereas a dune management system does not produce immediate protection. Although not shown in the typical cross section in Section 4.6.3, a walkway could be constructed along the berm crest.

The berm would not present too much of an obstacle to access to the water when the water level is low and sand has blown up against the berm. At higher water levels, however, wave uprush would expose the revetment armouring the berm. This would be a significant disadvantage as the exposed armour would impede or prevent much of the pedestrian access to the water. The berm would tend to divide the beach into sections, one above the berm and one below.

The berm would also be an expensive way of reducing flooding, considering that flooding only occurs at high lake levels. Constructing the berm along the reach 1 portion of the alignment shown in the photomosaics, would cost in excess of $820,000 and would have a benefit cost ratio of 0.68.

If this berm were shortened, however, to protect only the more densely developed section of shoreline east of the River Road right of way then the cost would be $440,000 but the benefit cost ratio would rise to 1.01. It should be noted, however, that these cost estimates were derived assuming that the berm could be protected with rip rap only, and not armourstone. Placing armourstone would significantly increase the cost and reduce the benefit cost ratio. Whether or not rip rap would provide sufficient armouring depends upon the final location of the berm. This is a design detail that cannot be resolved within a concept design.

A flood retaining wall could also be constructed along part of this reach as an alternative to developing a dune system. The wall would be constructed along the southern edge of Edith Cavell Boulevard from the western most dwelling to the gas pumping station south of River Road. The alignment of this wall is shown in Figure 5.4, and the Port Stanley preferred and alternate concepts photomosaics presented in Appendix B. This wall is intended to prevent flooding of properties and dwellings along the north side of Edith Cavell Boulevard. The wall would terminate at the gas pumping station where the development south of Edith Cavell Boulevard starts. This wall would have to tie in to the dune system or flood berm, described above, to prevent flood waters from coming around the end of the wall.

This flood retaining wall would be similar in cross section to the cantilevered concrete bulkhead shown in Figure 4.4a. The footing has to be deep enough so that it is below the frost line. The top of wall elevation would be 177.1 m GSC, in places more than 1 m higher than the land elevation. Depending upon the effectiveness of the existing road drainage system improved swales or perhaps a lateral toe drain would be required to prevent the retention of rain water.

This wall would offer the advantage of immediate flood control, rather than waiting for a dune system to develop.
Disadvantages include a restricted access to the beach, a high cost per property protected and an "unnatural" appearance.
Steps would have to be constructed at a number of locations to

facilitate access to the beach. This would be a major disadvantage at the foot of Lake Street where there is a parking area intended to encourage access to the beach at this area.

Because this part of reach 1 has not been extensively developed, only a limited number of structures would be protected by the wall. This would include 7 dwellings to the west of Lake Street and 6 to the east. The wall would cost $380,000 and was estimated to give a benefit cost ratio of only 0.30.

If a proper flood berm or flood retaining wall were to be constructed then , theoretically, there would be no need to floodproof the dwelling protected by the berm or the wall. There is, however, no guarantee that there will be absolutely no flooding with a berm and wall and hence floodproofing is still advisable as an added precaution. Details of floodproofing are discussed within the preferred concept and in Section 4.7.

5.3 Port Stanley Reach 2 Public Beach

Reach 2 of Port Stanley extends from the eastern limit of reach 1 to the harbour breakwater. This area was considered separately from reach 1 because it does not have the existing backshore dune profile typical of the beach within reach 1.

5.3.1 Existing Conditions

The beach of reach 2 is the most publicly used section of shoreline within the study area. There is a slight dune formation along the crest of the beach but the dune is not as well developed as within reach 1. The beach behind the dune crest is relatively flat and extends well inland. A typical profile was measured through the centre of this reach and is shown in Figure 2.3 in chapter 2. The beach is regularly cleaned by the Village of Port Stanley to maintain its appearance. Part of this cleaning involves raking the beach by tractor.

There are approximately 150 structures, (both residential and commercial) within the area bounded by the 100 year uprush elevation of 176.8 m GSC. These structures are flooded by waves which wash over both the public beach and the piers of the inner harbour and, in a few locations, through storm sewer intakes located below the flood water levels.

5.3.2 Preferred Shoreline Management Concepts

The preferred concepts for Reach 2 of Port Stanley are for the Kettle Creek Conservation Authority to :

1. Establish a regulatory shoreland zone based on the location of the dynamic beach zone.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

3. Encourage existing landowners to floodproof their dwellings to withstand the level of flooding associated with the 100 year wave uprush level.

The regulatory shoreland zone should be based on the location of the dynamic beach zone because it extends further landward than the 100 year wave uprush limit. The dynamic beach zone limit is shown on the Port Stanley preferred and alternate concepts photomosaics presented in Appendix B.

Before any new development is allowed within the regulatory shoreland zone, landowners will have to demonstrate that they have overcome both flooding and beach instability hazards, without causing or aggravating updrift or downdrift problems. This should be demonstrated within an impact statement, as discussed in Section 9.2.

Flooding problems associated with the 100 year uprush level should be overcome using dry passive floodproofing techniques, as discussed in Section 4.7. For properties which are or may be directly exposed to wave uprush the minimum allowable land elevations should be 176.55 m GSC. This is based on the criterion of a maximum flood depth of 0.25 m, as discussed in Section 3.8. Development not directly exposed to wave uprush may employ lower elevations as long as the flood depths do not exceed 0.8 m. and access and egress flood depths do not exceed 0.3 m. The landowner's impact statement must demonstrate that the area <u>cannot</u> be exposed to direct wave uprush in order to use these greater flood depths.

The benefit cost ratio for floodproofing dwellings within Reach 2 was estimated to be 4.22. The benefit cost analysis is presented in Section 5.7. It should be noted that, as discussed in Section5.7, this benefit cost analysis is based on estimated costs only and while it may be used for comparing alternations a more detailed analysis should be undertaken to support implementation.

Beach stability problems associated with the dynamic beach zone should be overcome using techniques approved by the conservation authority. Properties north of Edith Cavell Boulevard could come over the risk of beach instability by placing fill. This fill may be either vegetated or protected by retaining structures. It should be a cohesive material such as clay or clay/silt with sufficient cohesion or a cohesionless material with sufficient weight that it will not be transported by wind if it is no vegetated. The fill should be placed to the elevations discussed above. This fill, however, should only be placed if it can be shown that it will not cause a drainage problem on neighbouring properties. This would have to be addressed in the impact assessment required to obtain the necessary conservation authority approvals to place the fill. It is unlikely that filling an individual property will cause drainage problems on adjacent properties but development of a larger area could. If a large area is to be developed the conservation authority could give consideration to requiring a master plan be prepared as part of the development proposal.

Properties south of Edith Cavell Boulevard should only use sand fill similar in characteristics to the existing beach sand. This fill should not be protected by retaining structures which may in any way be exposed to wave uprush. Property owners should be required to show within their impact statement that any structures placed here will be above the flood limit.

5.5.3 Alternate Concepts

No alternate prevention concepts were developed for this reach of shoreline. An alternate method of protecting existing development with a flood retaining wall was considered. This wall would be similar to the flood retaining wall proposed as an alternative for protecting the western end of reach 1.

Under this alternate concept the wall would be constructed along the alignment shown in Figure 5.5 and on the Port Stanley preferred and alternate concept photomosaics in Appendix B. The western end of the wall must be tied into the flood protection method utilized within reach 1, ie a dune system or flood berm. This will prevent flood waters from getting in behind the wall from the west. The wall would have to be discontinued where it crosses William Street. That gap would have to be closed with sand bags or some other temporary barriers during periods of high lake levels.

This flood retaining wall would be similar in cross section to the cantilevered concrete bulkhead shown in Figure 4.4a. The footing has to be deep enough so that it is below the frost line. The top of wall elevation would be 177.1 m GSC. Depending upon the effectiveness of the existing road drainage system a lateral toe drain may be required to prevent the retention of rain water.

The wall would offer the advantage of immediate flood control, rather than waiting for individuals to floodproof their own properties. Disadvantages include restricted access to the beach and a high cost per property protected because it would also encompass a number of dwellings not at risk of flooding. Access through or over the wall would have to be constructed at a number of locations to accommodate pedestrian traffic. If access were provided through the wall it would have to be closed during storm events.

It was estimated that this wall would cost $739,000 and would yield a benefit cost ratio of 1.77. Details of the benefit cost analysis are presented in Section 5.7 of the report. It must be noted, however, that this analysis was done without detailed information regarding the elevations of the dwellings within this reach. The analysis was performed assuming the same distribution of elevations as surveyed along the beach. A more detailed survey of these properties, including a revised benefit cost analysis would be recommended before acting based upon the results of this analysis.

If a flood retaining wall were to be constructed then, theoretically, there would be no need to floodproof the dwelling protected by the wall. There is, however, no guarantee that there will be absolutely no flooding with the wall in place and hence floodproofing is still advisable as an added precaution. Details of floodproofing are discussed within the preferred concept and in Section 4.7.

A dune management program as proposed within the preferred concept for reach 1 was not considered for the preferred concept of this reach. This was not done because it was felt that the level of pedestrian traffic was too high to allow proper dune formation. People would have to be kept off the forming dunes. This would leave a strip of beach, in the order of 10 to 20 m wide, right across the middle of the public beach, where people would not be allowed. It was felt that this would not be acceptable to the Village of Port Stanley. Port Stanley's Official Plan places a high value on the recreational aspects of this beach.

If a way of creating permanent dunes without interfering with the public use of the beach could be found then this would be preferred.

5.4 Port Stanley Reach 3
Harbour Area

Reach 3 of Port Stanley consists of the harbour lands owned by the federal government. This reach, shown on the Port Stanley preferred and alternate concepts photomosaics in Appendix B covers land on either side of Kettle Creek. Within the Official Plan and zoning bylaws this area is designated as an industrial land use area. Existing land uses include, however, the Village of Port Stanley's roads department garage and an open space area not included within a designated land use area. This is because it is an artificial land base created after the adoption of the Official Plan.

No specific shoreline management plan concepts have been developed for this reach because these lands are owned by the federal government. We do recommend, however, that the conservation authority establish a line of contact with Transport Canada and request that any federal plans consider the objectives of the authority's shoreline management plan.

There are two aspects of the federal lands which are important to shoreline management. The existing harbour breakwaters are holding the fillet beach in its present location. If those harbour structures were to be removed the entire fillet beach would eventually be lost. If the harbour breakwaters were to be shortened then the beach would degrade. Although it may not appear to be so to the casual observer the lakeward end of the breakwater is actually anchoring the toe of the fillet beach.

Secondly, environmentally hazardous substances are sometimes stored within the 100 year wave uprush limits. The potential for contamination during flood events needs to be identified and appropriate preventative measures, as needed, should be adopted.

5.5 Port Stanley Reach 4 Little Beach

Reach 4 of Port Stanley is located immediately to the east of the harbour area. This reach has been considered separately from the reach to its east because it is fronted by a sandy beach.

5.5.1 Existing Conditions

This section of shoreline is characterized by low bluffs, about 3 m in height. There are a total of 10 properties located within this reach. These properties are fronted by a low sand beach created by placing sand dredged from the harbour entrance. A typical profile measured on this beach is shown in Figure 2.3 in Chapter 2. The eastern end of this beach is held in place by a small protruding headland constructed of scrap concrete. This beach will not increase in size in the long term, even if more dredgate is placed, unless the eastern headland is extended.

Each of the 10 properties is presently fronted by a protection structure. The type and effectiveness of each structure varies, as noted on the shoreline inventory notes described in Section 2.8 and presented, under separate cover, in Appendix C. This protection has been constructed on an individual basis rather than as part of a concerted effort.

It appears as if each of these properties was once protected by a vertical wall but that some problems have been experienced with those walls. The property owners on either end of the reach have placed concrete rubble in front of their previously existing structures but the central property owners have not. These central walls display signs of overtopping and settlement. The settlement is likely indicative of toe scour and a loss of backfill material.

5.5.2 Preferred Shoreline Management Concepts

The preferred shoreline management concepts for reach 4 of Port Stanley are for the Kettle Creek Conservation Authority to:

1. Establish a regulatory shoreland zone based on a 30 metre setback plus a stable slope allowance, in accordance with the pending provincial policy.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

3. Encourage landowners to upgrade and maintain their existing shoreline protection structures.

A regulatory shoreland zone based on a 30 metre setback plus a stable slope allowance has been proposed in accordance with the pending provincial policy on shoreline management. This setback should be measured from first lakeward break in slope of the bluff. The stable slope allowance should be taken as 3 times the bluff height considered to be the height from the nearshore break in slope to the bluff crest. Under the provincial policy no development should be allowed within the regulatory zone until it has been demonstrated that the potential erosion hazards have been overcome without impacting updrift or downdrift shorelines. Shoreline property owners who wish to redevelop should therefore have to demonstrate that the erosion hazards will be overcome by preparing an impact statement. Details of impact statements are provided in Section 9.2.

Because of the depth of the lots in this reach the defined regulatory shoreland zone will also cover portions of the properties located north of Edward Street. These people will not be able to address shoreline protection issues because they do not own shoreline property. The conservation authority should therefore view Little Beach area as a location where some flexibility could be incorporated into their guidelines for applying the Shoreline Management Plan. Flexibility in the application guidelines is also warranted at this site because of the extend of development which already exists. For example, the definition of the regulatory erosion standard includes a development setback of 30 metres. This distance includes, amongst other things, provision for access to the dwellings from the lakeward side of the dwelling and continuous access to the shoreline protection structures from parallel to the shoreline. However, each of the shoreline properties within Little Beach already have access from the landward side of the lot and the existing level of the development already prevents continuous access parallel to the shoreline. The conservation authority should therefore consider some flexibility in the minimum setback requirements.

There are two areas where some flexibility could be applied. First, the conservation authority could consider waiving the need for an impact statement for development on properties north of Edward Street as these people have no control over shoreline erosion protection. Secondly, allowable setbacks in this area could be determined on a property by property basis, as the need arises. Individual property owners could be given the flexibility to request a lesser setback than required within the regulatory shoreland zone. They should have to demonstrate, however, that there is a proper provision for maintenance access to their shoreline protection structure that the bluff is adequately protected against erosion and that their dwelling is setback far enough that it will not be damaged by overtopping water, wind blown spray and either small stones, ice or floating debris thrown by wave action. They would also have to show that their shoreline protection structures are sound and will prevent erosion of the low bluff. These items could be covered in their impact statement.

If maintenance access is to be provided from the landward side of the protection structure then no dwellings, trees, shrubs etc. should be placed within 10 metres of the protection structure. If maintenance is to be provided along Little Beach then the impact statement must demonstrate that such access is feasible and will be permitted by the Village of Port Stanley. The village must also recognize that if it agrees to permit maintenance access along this beach then this must be a long term agreement. If a dwelling is setback so that it is only 10 metres from the shoreline protection structure then the impact assessment must demonstrate that the dwelling is safe from spray and debris damage. The protection structure in this instance must be designed for no overtopping.

It is stated in Section 4.6 that sloped revetments are generally considered to be the preferred type of shoreline protection. This is also true here. The size of waves which strike these structures are determined by the water depth in front of the structure. That water depth is currently limited by the sand beach, making the beach an important component of the shoreline protection. Vertical walls will reflect greater amount of wave energy than will revetments. This in turn causes the loss of a greater amount of beach sand.

Most of the structures within this reach need some improvements to provide protection against the 100 year design storm. Individual suggestions are provided in the field inventory notes. Overall, these property owners should not rely upon the existence of the sand beach. That beach will greatly diminished at high water levels.

It must be noted, however, that most of these properties are not at serious risk at present water levels. This means that these shoreline protection structures do not need to be upgraded now. It is felt that only 1 of the structures would likely suffer damage if a severe storm were to occur now. This situation would change, however, with an increase in the water level. It must also be remembered however that one of the long term goals of Port Stanley is to develop a small craft marina somewhere in the area. If such a marina is built it would most likley be placed adjacent to the outer harbour, in front of reach 4. It seems likely that a marina will be constructed sometime but is impossible to estimate whether or not that will be in the near future. If a marina were constructed offshore of reach 4 then these protection structures would not have to be upgraded.

The benefit cost ratio for maintaining the existing protection structures within Reach 4 of Port Stanley was estimated to be 3.11. The benefit cost analysis is presented in Section 5.7. This analysis was undertaken for the existing conditions and gives no consideration to a potential marina.

5.5.3 Alternate Concepts

The level of shoreline protection provided by the beach could be increased by increasing the size of the beach. This would also improve the recreational utility of the beach, as desired by the Village of Port Stanley. The size of the beach could be increased by adding more sand but first the rubble "headland" anchoring the eastern toe of the beach would have to be extended. Any sand placed on the beach without extending the headland will eventually be lost.

If the headland were to be extended and additional beach material were to be placed then there would be less need to improve the existing shoreline protection structures. The level of improvements needed would be related to the extent to which the beach size is increased. The extent to which the beach size could be increased can only be determined by a more detailed analysis. This would depend in part upon the potential local impacts identified within the impact statement which would have to be prepared.

The protection structures within this reach could be uniformly upgraded to a consistent level of protection by constructing a continuous armourstone revetment across the reach. A description of armourstone revetments is provided in Section 4.6.1. the cost benefit ratio for a revetment was, however, only estimated to be 0.25.

5.6 Port Stanley Reach 5
Orchard Beach

Reach 5 of Port Stanley extends from the scrap concrete point which retains the end of Little Beach eastward to the end of the low bluff portion of Port Stanley.

5.6.1 Existing Conditions

This section of shoreline has been completely protected by a number of different types of structures. A few adjacent property owners have jointly constructed concrete revetments, but overall there has been no effort to standardize the protection. Some of the property owners have placed a large volume of rubble fill and others have only armoured the natural bank with concrete. Still others have constructed revetments out of concrete blocks, and in one instance, some armourstone. Details of each property including an estimate of the level of protection offered and deficiencies of the protection are included in the shoreline inventory presented in Appendix C.

5.6.2 Preferred Plan

The preferred shoreline management concepts for reach 5 of Port Stanley are for the Kettle Creek Conservation Authority to:

1. Establish a regulatory shoreland zone based on a 30 metre setback plus a stable slope allowance, in accordance with the pending provincial policy.

2. Require written approval from the municipality and the conservation authority prior to new land uses and associated development taking place within the regulatory shoreland zone.

3. Encourage landowners to upgrade and maintain their existing shoreline protection structures.

A regulatory shoreland zone based on a 30 metre setback plus a stable slope allowance has been proposed in accordance with the pending provincial policy on shoreline management. This setback should be measured from the first lakeward break in slope of the bluff. The stable slope allowance should be taken as 3 times the bluff height, considered to the height from the nearshore break in slope to the bluff crest. Under the provincial policy no development should be allowed within the regulatory zone until it has been demonstrated that the potential erosion hazards have been overcome without impacting updrift or downdrift shorelines.

Shoreline property owners who wish to redevelop should therefore have to demonstrate that the erosion hazards will be overcome by preparing an impact statement. Details of impact statements are provided in Section 9.2.

Because of the depth of the lots in this reach the defined regulatory shoreland zone will cover portions of some properties located north of Kitchener Street and some properties along Queen Street and Adelaide Street. These people will not be able to address shoreline protection issues because they do not own shoreline property. The conservation authority should therefore also view the Orchard Beach reach as a location where some flexibility could be incorporated into their guidelines for applying the Shoreline Management Plan.

The reasons for considering flexibility along this reach of shoreline are mostly related to the extent of existing development. The regulatory shoreland zone in this area will be based on the regulatory erosion standard rather than a flood standard. The erosion standard stipulated by the pending provincial policy is based on a stable bluff allowance equal to three times the bluff height, measured from the top of the bluff, plus a setback of 30 metres. In this shoreline reach, however, all of the bluff has been protected to some extent and that protection is carried some height up the bluff. Any erosion which occurs to the unprotected portions of the bluff will not produce a slope as gentle as the one included in the regulatory erosion standard. As well, the 30 metre setback distance includes, amongst other things both provision for access to the dwellings from the lakeward side of the dwelling and provision for continuous alongshore access to the shoreline protection structures. However, each of the shoreline properties in this reach already assessed from the landward side of the lot and the existing level of development already prevents continuous access parallel to the shoreline.

There are two areas where some flexibility could be applied in the application of the Shoreline Management Plan. First, the conservation authority could consider waiving the need for an impact statement for development on non-shoreline properties located within the regulatory shoreland zone. This would apply to any property where existing lakeward properties have already been developed. Secondly, the minimum allowable development setbacks in Orchard Beach could be determined on a property by property basis, as the need arises. Individual property owners could be given the flexibility to request a lesser setback than required within the regulatory shoreland zone.

The reduced setback could be based on both a reduced stable slope allowance and a reduced additional setback, but the property owners should have to demonstrate that their proposed setback is sufficient. A reduced stable bluff slope allowance would depend upon the exact form of the bluff protection and the bluff material. As a general rule, however, we would not expect this allowance to exceed two bluff heights measured from the toe of the bluff. To reduce the additional setback now proposed to be 30 metres, the landowner should have to demonstrate that there is proper provision for maintenance access to their shoreline protection structure. In addition, the landowner should demonstrate that the dwelling is set back far enough that it will not be damaged by overtopping water, wind blown spray, and either small stones, ice or floating debris thrown by wave action. This reduced setback, however, should not be less than 10 metres. All of this could be covered in their impact statement.

For maintenance access no dwellings, trees, shrubs etc. should be placed within 10 metres of the protection structure. If a dwelling is setback so that it is only 10 metres from the shoreline protection structure then the impact assessment must demonstrate that the dwelling is safe from spray and debris damage. The protection structure in this instance must be designed for no overtopping.

Most of the structures within this reach need some improvements to provide protection against the 100 year design storm. Individual suggestions are provided in the field inventory notes. Overall, there appears to be a lack of a proper filter layer for most of the structures. This is a serious deficiency which should be overcome. Revetments which front a native material bank tend to be too low to provide protection against the 100 year design storm. Properties which have been protected by dumping large volumes of concrete are considered to be fairly well protected. Serious damage to the concrete fill during a design storm should be expected as this fill has been placed at a steep slope. Based on the information obtained from the field survey, however, we do not expect major erosion of the native bluff behind this fill.

The benefit cost ratio for maintaining the protection structures within Reach 5 was estimated to be 1.47. The benefit cost analysis is presented in Section 5.7.

5.6.3 Alternate Concepts

The structures within this area could be uniformly upgraded to a consistent level of protection by constructing a continuous armourstone revetment across the reach, except at the mouth of Little Creek. This revetment would not be a cost effective solution.

Protection could also be uniformly upgraded by encouraging the ongoing dumping of scrap concrete as it becomes available. This would ultimately result in a continuous revetment, as proposed above, but it would look much cruder than the armourstone. This would also be much cheaper than constructing an armourstone revetment.

5.7 Benefit Cost Analysis

The costs and benefits of the proposed shoreline protection plan have been estimated in 1989 dollars and have been prepared in accordance with the guidelines published by MNR (1983b). All costs and benefits are estimates only and are based on preliminary design and approximate site information.

For each of the shoreline reaches, alternative protection concepts have been proposed. The protection concepts mitigate the loss of land and future destruction of homes due to long term recession of the bluff, damage to properties due to flooding from Lake Erie and the risk of destruction of homes due to the potential failure of existing shore protection structures. For the analysis a 100 year project life has been considered. Land loss spanning 100 years, all homes within the 100 year erosion limit and all homes within the 100 year flood limit have been included within the tangible benefits. The annual cost of land loss and flood damage and the future cost of destruction have been discounted to present value at a 7% rate of return.

Presented are the details of the benefit cost analysis for each reach of shoreline where protection measures have been proposed. Possible impacts of structures and intangible benefits have not been included as direct costs or benefits within the analysis.

5.7.1 Cost

Table 5.1 presents in detail the quantity and cost estimates for materials for each capital expense specified within the protection concepts. All costs are presented on either a per metre, per dwelling or lump sum basis.

The total cost for each alternative protection concept for each shoreline reach is presented in Table 5.2. Added to the construction cost are annual maintenance cost and engineering and supervision costs. The construction, engineering and supervision costs are presented at present value in 1989 dollars. The annual maintenance fee is also presented at present value by discounting the annual cost at a 7 per cent rate of return over the 100 year project life.

The annual maintenance costs for the shore protection structures are based on an assumed level of damage for a given storm event. The levels of damage are estimates only, based on experience. No solid data or literature is available to more accurately quantify the maintenance costs.

5.7.2 Benefits

Four direct tangible benefits were computed for each reach: reduced damage due to flooding, reduced land loss due to erosion, prevention of destruction due to erosion and reduced risk of destruction due to failure of an existing shore protection structure. For the shoreline reaches characterized by high bluffs, damage due to flooding is not of concern due to the elevation of the bluff. Only land loss and destruction due to erosion have been included as tangible benefits for such reaches. For the shoreline reaches characterized by beaches flood damage reduction is the most significant tangible benefit although erosion control benefits have been included where appropriate. For areas protected by an existing structure reduction of risk due to failure of that structure has been included where appropriate.

The flood damage reduction evaluation for Port Stanley is presented in Table 5.3 and 5.4. In order to access the damage reduction, each home was categorized based on door sill elevation. For homes along the shoreline at Port Stanley the door sill elevations were determined from the shoreline inventory notes presented in Appendix C. For inland homes, the number of flood prone homes was determined from available mapping. The door sill elevation of these homes was determined by assuming a similar distribution of elevation to the homes along the shoreline. A more detail inventory is required to more accurately access the flood potential for the inland areas.

For each category the annual average damage was determined as a percentage of the property and content value. The total annual damaged was based on a summation of the probability of occurrence of the percentage damage for a given flood level above the door sill elevation. The percentage damage was obtained using Tables B.1 and B.2 from the guidelines (MNR, 1983b), and assumed all houses to be one storey with no basement and a content value of 30 per cent of the property value. The value of each home, not including land cost, was assumed to be $70,000. The flood levels were obtained using the design water level analysis described in Chapter 3.

Annual average cost of land lost due to long term bluff recession is presented in Table 5.5. In order to determine the cost of lost land, the entire project area was divided into 26 reaches. Each reach was characterized by an average recession rate and an average land value. Recession rates for each reach were determined by averaging the rates presented in Section 3.7. Land values were determined based on the present market value for the particular use of land. Land use was categorized as being a wood lot, farmland, low density residential, and high density residential.

The total annual cost associated with long term bluff recession for each reach is determined from the product of recession rate, length of sector and unit cost of land for that sector. Under existing conditions it is estimated that the cost associated with the loss of land due to long term recession is approximately $150,000 per year and is due to an annual loss of roughly 3.5 hectares of land is lost annually over the study area.

For the reaches where protection structures have been proposed, the erosion control benefits are based on the area of land preserved. For headlands or revetments used for headland protection, the area preserved extends beyond the structure to include the wedge of land defining the headland bay. Table 5.6 presents the estimated annual erosion control benefits for each of the proposed bluff protection concepts discussed in Section 5.1.

The future cost of destruction of houses due to long term bluff recession is presented in Table 5.7. Each house within the 100 year erosion limit was included. The setback distance for each house from the top of the bluff was measured from aerial photographs and the years to destruction was estimated. Assuming a replacement cost of $70,000 for each house, the present value was determined by discounting the cost of future loss of the house from the years until destruction. Over the entire study area approximately 30 homes are at risk within the next 100 years. This relates to a present value cost of roughly $375,000. Of the total cost, roughly $330,000. will be incurred with the destruction of 13 homes within the next 10 to 20 years.

The cost associated with the reduced risk of destruction of a house due to failure of an existing shore protection structure is presented in Table 5.8. This benefit applies to areas where a house would potentially be at risk of being damaged or destroyed if the existing protection structure failed. As there is no available data or literature to provide an accurate assessment of the associated damage estimates were determined based on local experience.

5.7.3 Benefit Cost Ratio

A summary of total costs, benefits and the resulting benefit cost ratios is presented in Table 5.9. For all alternatives utilizing structural protection methods, the cost greatly exceeds the benefit. The only capital expense which may be justified on the basis of the tangible benefits would be relocation of dwellings when they are at imminent risk of damage and preservation and management of the natural beach dunes at Port Stanley.

Due to the limited data base from which this benefit cost analysis has been prepared the results should only be used for comparative purposes and not for implementation. Further assessment is required to more accurately determine the associated benefits of each alternative.

Table 5.1

Cost Estimates for Structural Protection (Benefit Cost Analysis)

	Estimat Quantity	Unit	Unit Price	Price/ Metre
A. Typical Single Layered Revetment				
Armourstone	48	tonne	$40	$1,920
Rip Rap	35	tonne	$25	$875
Geotextile	22	sq.metre	$10	$220
Grading Preparation	0.5	hours	$100	$50
Total Price Per Linear Metre				$3,065
B. Typical Double Layered Revetment				
Armourstone (primary)	79	tonne	$40	$3,160
Armourstone (secondary)	22	tonne	$30	$660
Rip Rap	26	tonne	$25	$650
Geotextile	22	sq.metre	$10	$220
Grading Preparation	0.5	hours	$100	$50
				$4,740
C. Road Access Cut				
Excavation	500000	cu.metres	$4	$2,000,000
Total Price Per Access Road				$2,000,000
D. Typical Headland				
Road Access Cut	1	lump sum	$2,000,000	$2,000,000
Dbl. Layered Revetment	340	metres	$4,700	$1,598,000
Total Price Per Headland				$3,598,000
E. Earth Berm				
Excavation	9	cu.metres	$4	$36
Earth Fill	10	cu.metres	$4	$40
Geotextile	9	sq.metres	$10	$90
Rip Rap	19	tonnes	$25	$475
Total Price Per Linear Metre				$641
F. Concrete Retaining Wall				
Excavation	4	cu.metres	$4	$16
Concrete	1.2	cu.metres	$500	$600
Total Price Per Linear Metre				$616

Table 5.2

Construction Cost Estimates
(Benefit Cost Analysis)
(High Bluff Shoreline)

Item	Quantity Estimate	Units	Unit Price	Total Price
Grand Canyon - Alternative 1, Two Revetments at Grand Canyon				
1. Eastern Revetment				
- Access Road	500	metres	500	$250,000
- Berm	450	metres	300	$135,000
- Single Layer Revetment	450	metres	3100	$1,395,000
- Bank Stabilization	450	metres	4000	$1,800,000
2. Western Revetment				
- Single Layer Revetment	900	metres	3100	$2,790,000
- Bank Stabilization	900	metres	4000	$3,600,000
		Subtotal		$9,970,000
Grand Canyon - Alternative 2, Headland Bay				
- Access Road	600	metres	500	$300,000
- Single Layer Revetment	600	metres	3100	$1,860,000
- Bank Stabilization	600	metres	4000	$2,400,000
		Subtotal		$4,560,000
East of Port Stanley - Headland Bays				
- Road Access Cut	3	lump sum	2000000	$6,000,000
- Headlands	3	lump sum	1598000	$4,794,000
		Subtotal		$10,794,000
County Road 24 - Revetment at Exposed Road				
- Single Layer Revetment	2100	metres	3100	$6,510,000
		Subtotal		$6,510,000

Table 5.2 (cont.)

Construction Cost Estimates
(Benefit Cost Analysis)
(Within Port Stanley)

Item	Quantity Estimate	Units	Unit Price	Total Price
Reach 1, West of Stanley Park - Alternative 1				
- Flood Proofing Homes	15	houses	10000	$150,000
- Dune Management	1300	metres	75	$97,500
		Subtotal		$247,500
Reach 1, West of Stanley Park - Alternative 2, Full Earth Berm				
- Earth Berm	1300	metres	640	$832,000
Reach 1, West of Stanley Park - Alternative 3, Short Earth Berm				
- Earth Berm	700	metres	640	$448,000
Reach 1, West of Stanley Park - Alternative 4, Short Retaining Wa				
- Retaining Wall	600	metres	620	$372,000
Reach 2, East of Stanley Park - Alternative 1, Flood Proof Houses				
- Flood Proofing Homes	35	houses	10000	$350,000
Reach 2, East of Stanley Park - Alternative 2, Concrete Wall				
- Retaining Wall	1200	metres	620	$744,000
Reach 4, Little Beach - Alternative 2, Revetment				
- Single Layer Revetment	130	metres	3100	$403,000
Reach 5, Orchard Beach - Alternative 2, Revetment				
- Single Layer Revetment	800	metres	3100	$2,480,000

Table 5.3

Annual Flood Damage Factors
(Benefit Cost Analysis)

Door sill elevation 175.7 m GSC

flood level	flood depth	% damage	cumul. dist.	return period	prob. exceed.	prob. occurr.	aver. damage	annual damage
175.8	0.1	11.5	0.50	2.0	0.50			
						0.300	14.30	4.29
176.1	0.4	17.1	0.80	5.0	0.20			
						0.100	18.20	1.82
176.2	0.5	19.3	0.90	10.0	0.10			
						0.060	22.70	1.36
176.4	0.7	26.1	0.96	25.0	0.04			
						0.030	29.50	0.89
176.6	0.9	32.9	0.99	100.0	0.01			
						0.005	34.60	0.17
176.7	1.0	36.3	1.00	200.0	0.01			
							total	8.53

Door sill elevation 176.2 m GSC

flood level	flood depth	% damage	cumul. dist.	return period	prob. exceed.	prob. occurr.	aver. damage	annual damage
175.8	-0.4	0.0	0.50	2.0	0.50			
						0.300	3.00	0.90
176.1	-0.1	6.0	0.80	5.0	0.20			
						0.100	8.00	0.80
176.2	0.0	10.0	0.90	10.0	0.10			
						0.060	11.50	0.69
176.4	0.2	13.0	0.96	25.0	0.04			
						0.030	15.05	0.45
176.6	0.4	17.1	0.99	100.0	0.01			
						0.005	18.20	0.09
176.7	0.5	19.3	1.00	200.0	0.01			
							total	2.93

Table 5.3 (cont.)

Annual Flood Damage Factors
(Benefit Cost Analysis)

Door sill elevation 176.7 m GSC

flood level	flood depth	% damage	cumul. dist.	return period	prob. exceed.	prob. occurr.	aver. damage	annual damage
175.8	-0.9	0.0	0.50	2.0	0.50			
						0.300	0.00	0.00
176.1	-0.6	0.0	0.80	5.0	0.20			
						0.100	0.00	0.00
176.2	-0.5	0.0	0.90	10.0	0.10			
						0.060	0.00	0.00
176.4	-0.3	0.0	0.96	25.0	0.04			
						0.030	3.00	0.09
176.6	-0.1	6.0	0.99	100.0	0.01			
						0.005	8.00	0.04
176.7	0.0	10.0	1.00	200.0	0.01			
							total	0.13

Door sill elevation 177.2 m GSC

flood level	flood depth	% damage	cumul. dist.	return period	prob. exceed.	prob. occurr.	aver. damage	annual damage
175.8	-1.4	0.0	0.50	2.0	0.50			
						0.300	0.00	0.00
176.1	-1.1	0.0	0.80	5.0	0.20			
						0.100	0.00	0.00
176.2	-1.0	0.0	0.90	10.0	0.10			
						0.060	0.00	0.00
176.4	-0.8	0.0	0.96	25.0	0.04			
						0.030	0.00	0.00
176.6	-0.6	0.0	0.99	100.0	0.01			
						0.005	0.00	0.00
176.7	-0.5	0.0	1.00	200.0	0.01			
							total	0.00

Table 5.4

Estimated Annual Flood Damage Cost (Benefit Cost Analysis)

	door sill elevation, (m GSC) 175.7	176.2	176.7	177.2	177.7	annual damage	annual cost
Percentage Damage	8.53	2.93	0.13	0.00	0.00		
Port Stanley, Reach 2	6	5	4	10	23	0.66	$46,445
Port Stanley, Reach 1	14	11	10	24	52	1.53	$107,065
Total	20	16	14	34	75	2.19	$153,510

Table 5.5

Annual Land Loss Cost (Benefit Cost Analysis)

Reach	Sector	Sector Lenght (m)	Recession Rate (m/yr)	Land Loss (m2/yr)	Unit Cost ($/m2)	Land Cost ($/yr)
W. Grand Canyon	1	880	1.26	1109	0.5	$554
W. Grand Canyon	2	1530	1.22	1867	0.5	$933
W. Grand Canyon	3	270	1.44	389	0.2	$78
W. Grand Canyon	4	740	1.33	984	0.5	$492
W. Grand Canyon	5	790	0.98	774	0.5	$387
W. Grand Canyon	6	260	0.85	221	0.2	$44
W. Grand Canyon	7	330	1.42	469	0.5	$234
W. Grand Canyon	8	690	1.25	863	0.5	$431
Grand Canyon	9	760	1.35	1026	10.0	$10,260
Grand Canyon	10	320	1.40	448	10.0	$4,480
Grand Canyon	11	380	1.71	650	0.2	$130
Grand Canyon	12	770	1.41	1086	35.0	$38,000
P.Stanley	13	3050	0.00	0	35.0	$0
E. Orchard Beac	14	330	4.19	1383	35.0	$48,395
E. Orchard Beac	15	1440	3.24	4666	0.2	$933
E. Orchard Beac	16	950	2.07	1967	0.5	$983
Pump Station	17	400	1.69	676	0.5	$338
E. Pump Station	18	1030	1.73	1782	0.5	$891
E. Pump Station	19	860	2.16	1858	0.5	$929
E. Pump Station	20	930	2.43	2260	0.5	$1,130
E. Pump Station	21	1940	1.63	3162	0.5	$1,581
E. Pump Station	22	1060	1.31	1389	0.2	$278
E. Pump Station	23	540	1.14	616	0.2	$123
County Road 24	24	1250	1.46	1825	0.5	$913
County Road 24	25	600	1.21	726	10.0	$7,260
County Road 24	26	1870	1.60	2992	10.0	$29,920
TOTAL		23970		35184		$149,697

Table 5.6

Annual Erosion Control Benefits
(Benefit Cost Analysis)

Reach	Alt.	Land Loss Reduction (m2/yr)	Land Loss Reduction ($/yr)
1.a Grand Canyon	- Revetments	2953	$50,304
1.b Grand Canyon	- Headland	1077	$472
2.a E. Port Stanley	- Maintenance	1659	$830
2.b E. Port Stanley	- Headlands	6687	$38,071
3. County Road 24	- Revetment	3299	$15,653

Table 5.7

Future Destruction of Houses
(Benefit Cost Analysis)

Reach	Alt.	No. of Houses	Years to Risk (yr)	Future Value ($)
1.a Grand Canyon	- Revetments	9	10	$630,000
		7	75	$490,000
1.b Grand Canyon	- Headland	1	65	$70,000
2.a E. Port Stanley	- Maintenance	0		$0
2.b E. Port Stanley	- Headlands	0		$0
3. County Road 24	- Revetment	10	50	$700,000

Table 5.8

Cost of Future Destruction of Houses on Bluff
(Benefit Cost Analysis)

Years to Distructn	No. of Houses	Unit Cost	Future Cost	Discount Factor	Present Value
10-19	13	$70,000	$910,000	0.3620	$329,420
20-29	0	$70,000	$0	0.1840	$0
30-39	2	$70,000	$140,000	0.0937	$13,118
40-49	8	$70,000	$560,000	0.0476	$26,656
50-59	1	$70,000	$70,000	0.0242	$1,694
60-69	3	$70,000	$210,000	0.0123	$2,583
70-79	1	$70,000	$70,000	0.0063	$441
80-89	2	$70,000	$140,000	0.0032	$448
90-99	0	$70,000	$0	0.0016	$0
Total	30				$374,360

Table 5.9

Benefit Cost Analysis

Grand Canyon - Alternative 1, Two Revetments at Grand Canyon

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$9,570,000
Annual Maintenance	$12,950		$184,784
Engineering and Super., (10%)			$957,000
Total Costs			$10,711,784
TANGIBLE BENEFITS			
Flood Damage Reduction	$0		$0
Future Destruction, 10 yrs		$630,000	$320,229
Future Destruction, 75 yrs		$490,000	$3,087
Erosion Control Benefits	$50,304		$717,788
Indirect Benefits, (10%)			$72,087
Total Benefits			$1,113,191
Net Project Benefits			($9,598,592)
Benefit-Cost Ratio			0.10

Grand Canyon - Alternative 2, Headland Bay

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$4,560,000
Annual Maintenance	$9,200		$131,275
Engineering and Super., (10%)			$456,000
Total Costs			$5,147,275
TANGIBLE BENEFITS			
Flood Damage Reduction	$0		$0
Future Destruction, 65 yrs		$70,000	$861
Erosion Control Benefits	$472		$6,735
Indirect Benefits, (10%)			$760
Total Benefits			$8,356
Net Project Benefits			($5,138,919)
Benefit-Cost Ratio			0.00

Table 5.9 (cont.)

Benefit Cost Analysis

East of Port Stanley - Alternative 1, Maintain Headland

	ANNUAL	CURRENT	PRESENT
COSTS			
Construction			$0
Annual Maintenance	$3,100		$44,234
Engineering and Super., (10%)			$0
Total Costs			$44,234
TANGIBLE BENEFITS			
Flood Damage Reduction	$0		$0
Future Destruction		$0	$0
Erosion Control Benefits	$830		$11,843
Indirect Benefits, (10%)			$1,184
Total Benefits			$13,028
Net Project Benefits			($31,206)
Benefit-Cost Ratio			0.29

East of Port Stanley - Alternative 2, Headland Bays

	ANNUAL	CURRENT	PRESENT
COSTS			
Construction			$10,794,000
Annual Maintenance	$28,200		$402,386
Engineering and Super., (10%)			$1,079,400
Total Costs			$12,275,786
TANGIBLE BENEFITS			
Flood Damage Reduction	$0		$0
Future Destruction		$0	$0
Erosion Control Benefits	$38,071		$543,235
Indirect Benefits, (10%)			$54,324
Total Benefits			$597,559
Net Project Benefits			($11,678,227)
Benefit-Cost Ratio			0.05

Table 5.9 (cont.)

Benefit Cost Analysis

County Road 24 - Revetment at Exposed Road

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$6,510,000
Annual Maintenance	$16,700		$238,292
Engineering and Super., (10%)			$651,000
Total Costs			$7,399,292
TANGIBLE BENEFITS			
Future Destruction, 50 yrs		$700,000	$23,730
Future Destr. of Road, 100yrs		$2,500,000	$3,000
Erosion Control Benefits	$15,653		$223,353
Indirect Benefits, (10%)			$25,008
Total Benefits			$275,091
Net Project Benefits			($7,124,201)
Benefit-Cost Ratio			0.04

Reach 1, West of Stanley Park - Alternative 1, Floodproofing

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$247,500
Annual Maintenance, (dune)	$9,750		$139,123
Engineering and Super., (10%)			$24,750
Total Costs			$411,373
TANGIBLE BENEFITS			
Flood Damage Reduction	$46,445		$640,987
Future Destruction		$0	$0
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$64,099
Total Benefits			$705,086
Net Project Benefits			$293,713
Benefit-Cost Ratio			1.71

Table 5.9 (cont.)

Benefit Cost Analysis

Reach 1, West of Stanley Park - Alternative 2, Full Earth Berm

	ANNUAL	CURRENT	PRESENT
COSTS			
Construction			$832,000
Annual Maintenance	$8,320		$118,718
Engineering and Super., (10%)			$83,200
Total Costs			$1,033,918
TANGIBLE BENEFITS			
Flood Damage Reduction	$46,445		$640,987
Future Destruction		$0	$0
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$64,099
Total Benefits			$705,086
Net Project Benefits			($328,832)
Benefit-Cost Ratio			0.68

Reach 1, West of Stanley Park - Alternative 3, Short Earth Berm

	ANNUAL	CURRENT	PRESENT
COSTS			
Construction			$448,000
Annual Maintenance	$4,480		$63,925
Engineering and Super., (10%)			$44,800
Total Costs			$556,725
TANGIBLE BENEFITS			
Flood Damage Reduction	$37,200		$513,397
Future Destruction		$0	$0
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$51,340
Total Benefits			$564,737
Net Project Benefits			$8,012
Benefit-Cost Ratio			1.01

Table 5.9 (cont.)

Benefit Cost Analysis

Reach 1, West of Stanley Park - Alternative 4, Short Retaining Wall

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$372,000
Annual Maintenance	$3,720		$53,081
Engineering and Super., (10%)			$37,200
Total Costs			$462,281
TANGIBLE BENEFITS			
Flood Damage Reduction	$9,300		$128,349
Future Destruction		$0	$0
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$12,835
Total Benefits			$141,184
Net Project Benefits			($321,096)
Benefit-Cost Ratio			0.31

Reach 2, East of Stanley Park - Alternative 1, Flood Proofing

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$350,000
Annual Maintenance	$0		$0
Engineering and Super., (10%)			$35,000
Total Costs			$385,000
TANGIBLE BENEFITS			
Flood Damage Reduction	$107,065		$1,477,604
Future Destruction		$0	$0
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$147,760
Total Benefits			$1,625,364
Net Project Benefits			$1,240,364
Benefit-Cost Ratio			4.22

Table 5.9 (cont.)

Benefit Cost Analysis

Reach 2, East of Stanley Park - Alternative 2, Concrete Wall

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$739,000
Annual Maintenance	$7,390		$105,448
Engineering and Super., (10%)			$73,900
Total Costs			$918,348
TANGIBLE BENEFITS			
Flood Damage Reduction	$107,065		$1,477,604
Future Destruction		$0	$0
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$147,760
Total Benefits			$1,625,364
Net Project Benefits			$707,017
Benefit-Cost Ratio			1.77

Reach 4, Little Beach - Alternative 1, Maintain Protection

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$0
Annual Maintenance (@$20/m)	$2,600		$37,099
Engineering and Super., (10%)			$0
Total Costs			$37,099
TANGIBLE BENEFITS			
Flood Damage Reduction	$0		$0
Risk Reduction	$7,350		$104,877
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$10,488
Total Benefits			$115,365
Net Project Benefits			$78,265
Benefit-Cost Ratio			3.11

Table 5.9 (cont.)

Benefit Cost Analysis

Reach 4, Little Beach - Alternative 2, New Revetment

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$403,000
Annual Maintenance (@$5/m)	$650		$9,275
Engineering and Super., (10%)			$40,300
Total Costs			$452,575
TANGIBLE BENEFITS			
Flood Damage Reduction	$0		$0
Risk Reduction	$7,350		$104,877
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$10,488
Total Benefits			$115,365
Net Project Benefits			($337,210)
Benefit-Cost Ratio			0.25

Reach 5, Orchard Beach - Alternative 1, Maintain Protection

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$0
Annual Maintenance (@$20/m)	$16,000		$228,304
Engineering and Super., (10%)			$0
Total Costs			$228,304
TANGIBLE BENEFITS			
Flood Damage Reduction	$0		$0
Risk Reduction	$21,315		$304,144
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$30,414
Total Benefits			$334,558
Net Project Benefits			$106,254
Benefit-Cost Ratio			1.47

Table 5.9 (cont.)

Benefit Cost Analysis

Reach 5, Orchard Beach - Alternative 2, New Revetment

COSTS	ANNUAL	CURRENT	PRESENT
Construction			$2,480,000
Annual Maintenance (@$5/m)	$4,000		$57,076
Engineering and Super., (10%)			$248,000
Total Costs			$2,785,076
TANGIBLE BENEFITS			
Flood Damage Reduction	$0		$0
Risk Reduction	$21,315		$304,144
Erosion Control Benefits	$0		$0
Indirect Benefits, (10%)			$30,414
Total Benefits			$334,558
Net Project Benefits			($2,450,518)
Benefit-Cost Ratio			0.12

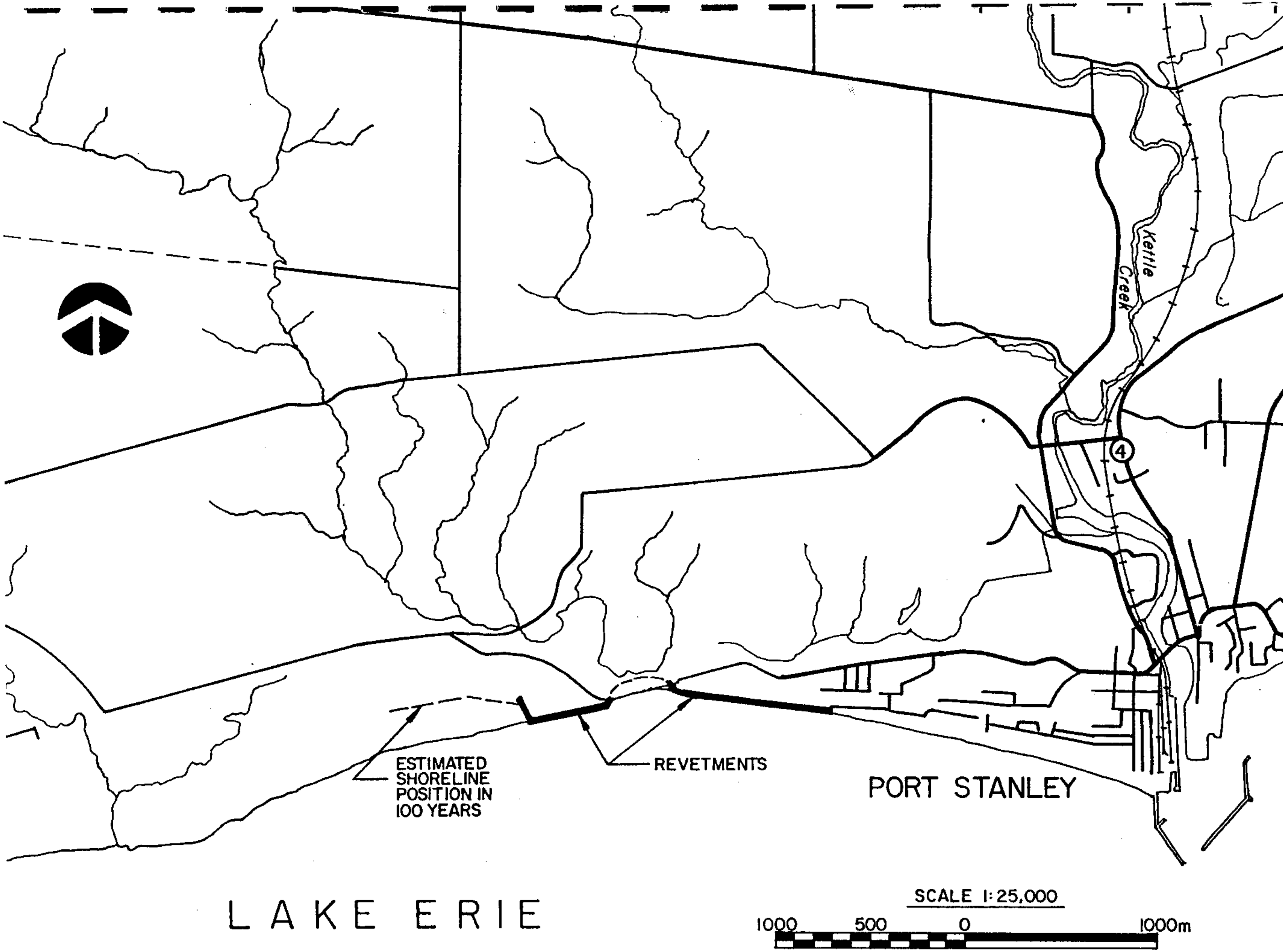


Figure 5.1 Alternate Protection Concept for Grand Canyon

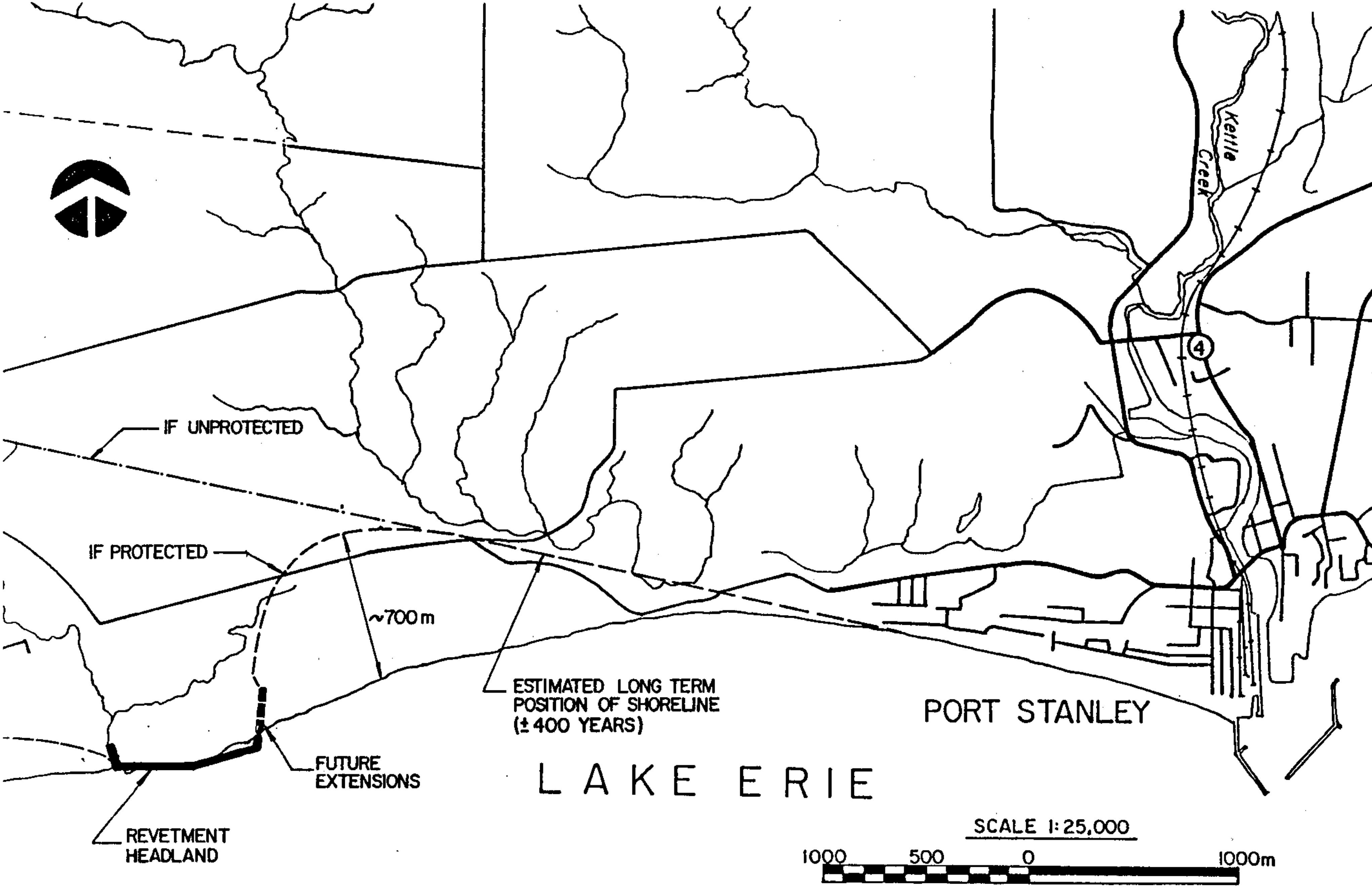


Figure 5.2 Alternate Protection Concept for West of Port Stanley

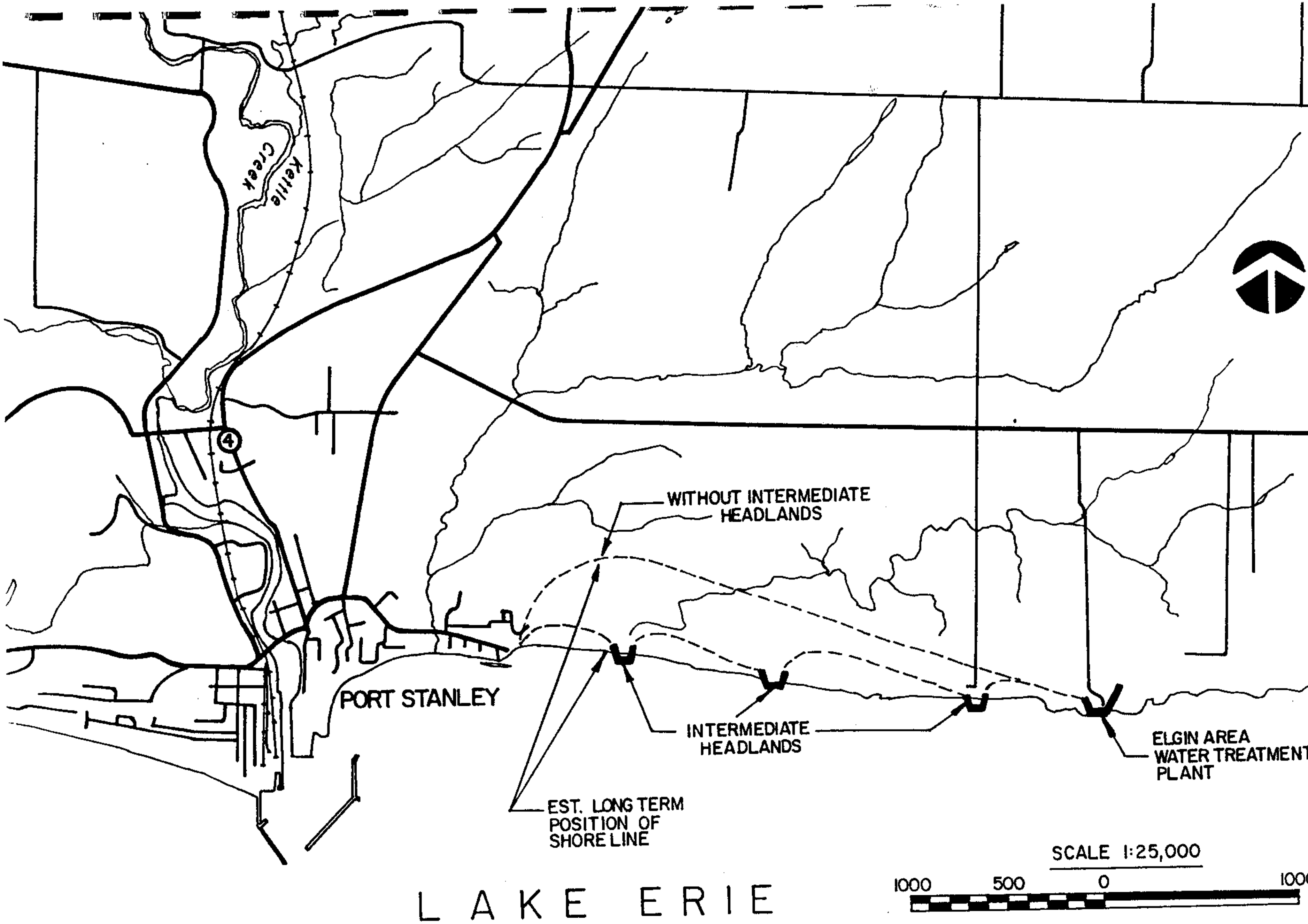
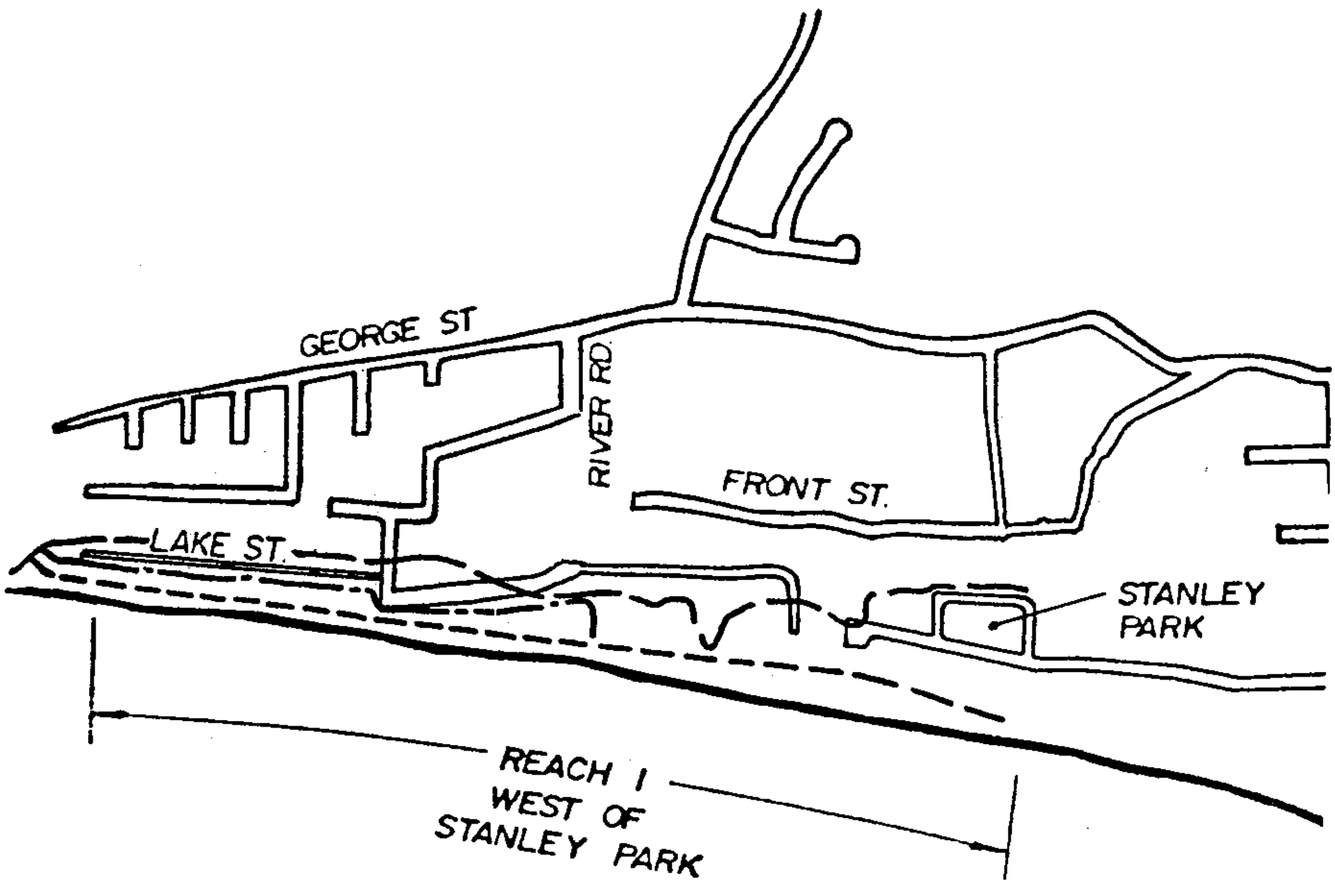


Figure 5.3 Alternate Protection Concept - East of Port Stanley



LEGEND

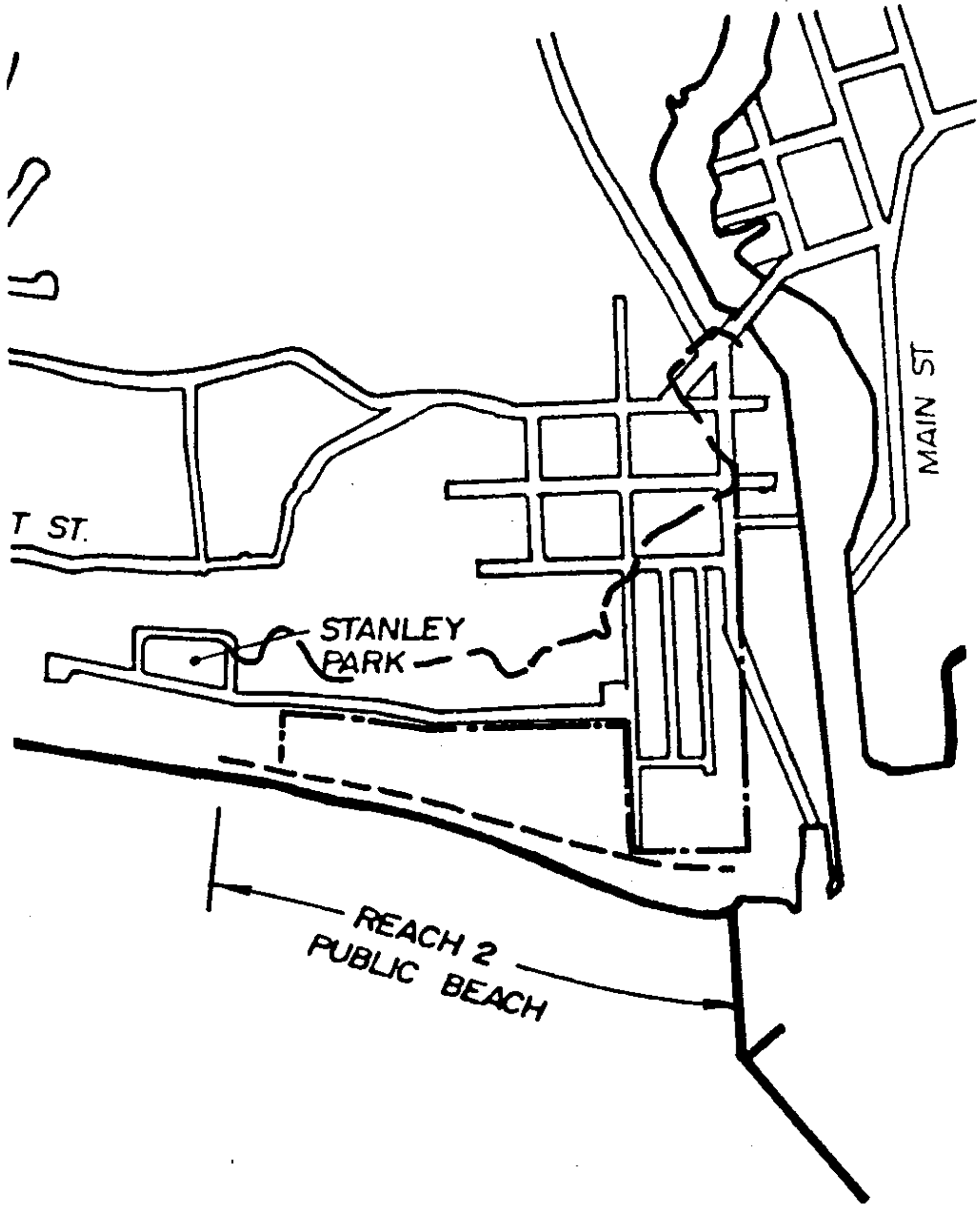
100 YEAR UPRUSH LIMIT (176.8 m GSC)

DUNE OR DYKE ALIGNMENT

FLOOD RETAINING WALL

Figure 5.4

Port Stanley
Preferred and Alternate
Concepts – Reach I



LEGEND

100 YEAR UPRUSH LIMIT (176.8 m GSC)

DUNE OR DYKE ALIGNMENT

FLOOD RETAINING WALL

Figure 5.5

Port Stanley
Preferred and Alternate

6.0 Environment

6.1 Environmental Review

A review of potential environmental concerns was undertaken for the study area. Information was obtained from existing reports provided by MNR and the conservation authority as well as from discussions with MNR staff biologists. A listing of reports examined during the review is provided in the bibliography. A summary of the environmental review is provided below. This is followed by a summary of the environmental aspects of the preferred and alternate shoreline management concepts and an overview of the environmental aspects of shoreline construction.

Fish

Rainbow trout, coho salmon and chinook salmon have been known to spawn in several tributaries along the shoreline including East Creek (Little Creek), which is located slightly east of Port Stanley at Orchard Beach. A walleye spawning run is also known to occur in Kettle Creek. In addition, rainbow trout are released each year by the Lake Erie Salmon and Trout Club Hatchery, located on Mill Pond Creek, a tributary of Kettle Creek. These fish migrate back up Kettle Creek to spawn each year. The exact dates when fish spawn vary from year to year but spring runs generally occur in March, April and May and fall runs generally occur in October, November and December.

The silver chub which is considered rare in Lake Erie, has been previously noted in the Port Stanley area, however its present status in the area is unknown.

This area is likely an important habitat for various bait and forage fish species. There is an established bait fishery for minnows and shiners on both sides of Port Stanley. Smelt may also be spawning within the study area, but no direct confirmation of recent spawning appearances have been documented. If smelt spawning is occurring, the important areas would be beaches and shoals in water depths of three metres or less.

The area below the bluffs is not considered an important area for the spawning of major commercial fish species because of the strong long-shore currents and the high water turbidity due to erosion.

Amphibians and Reptiles

None of the amphibians and reptile species listed in the Elgin-Kent County Shoreline life science inventory are considered rare in Ontario or Canada.

Birds

Bobwhites and Bald Eagles inhabit the area and are considered to be rare for the province. The Elgin-Kent County shoreline is also important as a day-use, feeding and perching area for the Bald Eagle. Bobwhites were confirmed as breeding in Essex, Kent, Lambton, Elgin and Middlesex counties in 1983.

The Hawk Cliff Banding Station, located east of Port Stanley is an internationally known location for viewing migratory raptors. Fifteen species of raptors are known to migrate along the north shore of Lake Erie, including the Peregrine Falcon, Golden Eagle and Bald Eagle, all of which are considered endangered species. The first large flight of raptors along the Lake Erie shoreline generally begins after September 10, when rainy weather is followed by a northerly high pressure system.

The north shore of Lake Erie is also a migration route for waterfowl such as the Tundra Swan, a protected species, as well as various songbirds such as the Blue Jay.

Mammals

Most of the mammals listed in the life sciences inventory for the Elgin-Kent County shoreline are considered relatively common and widespread, with two exceptions. The Southern Bog Lemming is rare in Southern Ontario and the American Badger is considered rare in Ontario. At least one badger den has been observed along the eroding bluffs of Lake Erie.

Plants

The Elgin-Kent County area has a total of 19 species currently listed as provincially and/or nationally rare; 11 species are regionally rare and at least 25 are at or near the limits of their range in Ontario. A listing of these plants is present in Table 6.1

Most of these plant species are not associated with the shoreline. Four species that are found along the eroding bluffs, however, include: a rare Sedge, Ghost-Pipe or Cancer Root, Canada Hawkweed and Hairy Rock-Cress.

Wetlands

There are no large wetlands located within the study area. Some areas of the fillet beach updrift of the harbour breakwater could potentially support interdunal ponds, but none were found at the time of the field survey.

6.2 Environmental Aspects of Preferred and Alternate Concepts

This section of the report summarizes the environmental aspects of the preferred and alternate shoreline management concepts presented in Chapter 5. Details of the environmental approval process are outlined in Section 9.2.

6.2.1 High Bluff Shorelines

The preferred plan for reducing the impacts of erosion includes the use of development restrictions, the relocation or removal of structures and the application of zoning regulations. These are not expected to result in negative environmental effects.

If no shoreline protection structures are constructed in this area, erosion of the bluffs will continue. Although no monetary value can be assigned to the flora and fauna that may be affected by continued erosion of the bluffs, it should be recognized that stands of Carolinian forest, several rare plant species, and several types of wildlife habitat exist in the areas being eroded.

If alternate concepts such as revetments or headlands are constructed to protect the natural resources associated with the high bluff shoreline, construction activities should be timed to avoid conflict with the spawning periods of the fish species which might be spawning near the construction site. There are no specific concerns about increased turbidity in front of the bluffs associated with construction activities because of the existing high turbidity levels from bluff erosion. Turbidity from construction in front of the bluffs could however effect downdrift areas. The extent to which this could apply would depend upon the exact nature of the construction. As a guideline we would suggest that particular attention be paid to any projects located within about 1 kilometer of a known spawning area.

6.2.2 Port Stanley Reach 1 West of Stanley Park

The preferred plan, which consists of the enforcement of minimum setbacks and elevations for new developments, the floodproofing of existing structures, and a dune management program, is expected to have no significant negative impacts on the environment of the area.

If a flood berm or a flood barrier wall is constructed, the fill quality should meet existing criteria, as discussed in Section 6.3. It must also be recognized that the construction of a long uninterrupted revetted flood barrier could potentially impede the land-to-water migration of amphibians or reptiles. This may not be a significant concern for small scale barriers with frequent interruptions such as those which may result from individual privately constructed barriers. If a continuous barrier were to be planned an environmental assessment should evaluate in more detail the importance of this area to amphibians or reptiles.

6.2.3 Port Stanley Reach 2 Public Beach

The preferred plan for this area is similar to that proposed for Reach 1, discussed above. As in the case of Reach 1, the negative environmental effects of the plan are expected to be minimal. Any fill material used to augment elevations in the area should meet the relevant regulatory criteria. This issue is discussed further in Section 6.3.

6.2.4 Port Stanley Reach 3 Harbour Area

No shoreline management concepts were developed for this reach and, as a result, potential impacts of proposed shoreline plans were not evaluated.

It should be noted, however, that this is a flood-prone area, and that there are several liquid storage tanks located within the zone of potential flooding. The tanks are reported to contain fuels and liquid fertilizers. The environmental risks associated with flooding and the potential leakage of these tanks have not been evaluated during this study.

6.2.5 Port Stanley Reach 4 Little Beach

There are no significant negative impacts associated with the preferred plan, which consists of proposed improvements to existing shoreline protection structures.

If elements of the alternative concept are implemented, care should be taken in the placement of supplementary beach materials to minimize turbidity. In addition, any extension of the headland should be completed outside of the spawning periods of the fish in the study area. All fill materials used in the construction of the headland and for beach material must meet the regulatory criteria for fill quality. Environmental aspects of shoreline construction are discussed in more detail in Section 6.3

6.2.6 Port Stanley Reach 5
Orchard Beach

The preferred plan for this area proposes the upgrading of existing shoreline protection structures. Any construction at this site should be completed outside of the spawning periods of the indigenous fish species, particularly walleye and the salmonids. In addition, the lake Erie Salmon and Trout Club reportedly release fish in the vicinity of Port Stanley during the spring and autumn of each year. Construction of shoreline protection structures should be timed to avoid conflict with the fish release activities of the club. Any fill placed at this site should meet regulatory fill quality criteria. Environmental aspects of shoreline construction are discussed in more detail in Section 6.3.

6.3 Environmental Aspects of Shoreline Construction

The main environmental concerns with shoreline construction relate to fill quality and the effects of construction activities. Contaminated fill material, for example, may adversely effect the water quality and the use of fine grained fill often results in the creation of turbid plumes. If a fine grained fill is used the resulting turbid plume can effect spawning fish. Fine grained sediments also tend to carry associated contaminants. To minimize the environmental effects of shoreline construction it is important that only clean fill be used and that construction activities incorporate methods to minimize the suspension of fines in the water.

All material proposed for fill must meet the regulatory standards and policies set forth by the governing agencies which have jurisdiction at the construction site. The present standards and policies are defined by the Open Water Disposal Guideline prepared by the Ontario Ministry of the Environment. The exact policies which apply are updated frequently and the conservation authority should keep aware of the current requirements. MOE will usually be given an opportunity to review proposed shoreline works (for example NWPA of Transport Canada automatically sends a copy of all submissions to MOE) but their approval of those works is not required. MOE will, however, stop work on the project if unacceptable materials are being used.

The conservation authority should, as part of their approval process, request information about the sources of any materials to be used during construction. If they have any reservations about those materials they could then notify MOE of their concerns so that MOE can monitor the construction project. Fill material which would cause concern would include materials obtained from present or former industrial or commercial sites, properties neighbouring those sites where floods or groundwater flow may have transported contaminants, sites with existing fill of unknown origin and locations with a history of fertilizer use.

The conservation authority could also consider requiring formal fill contaminant analyses for projects where large volumes of fill material will be used. As an example the Metropolitan Toronto and Region Conservation Authority automatically requires testing for any project using more than 200 cubic metres of fill material.

The need for controlling the use of fine grained materials depends upon the exact nature of the construction and time of year the work takes place. Even small amounts of silt can kill fish eggs and destroy spawning habitat so construction should not take place near to spawning areas during periods of fish spawning activities.

Table 6.1 Status of Significant Plants

	Rare In Elgin and Kent countys	Rare In Ont.	Rare In Can.	Range Limit[3]
PROVINCIALLY RARE SPECIES				
Phegopteris hexagonoptera	X	X	X	X
Cinna arundinacea		X[1]	X	X
Spehnopolis obtusata	X	X		X
Carex hirsutella	X	X	X	X
Carex prasina	X	X	X	X
Carex radiata	X	X	X	X
Carex virescens	X	X		X
Juncus acuminatus	X	X		X
Disporum lanuginosum	X	X		X
Juglans nigra		X	X	X
Liriodendron tulipifera		X	X	X
Argrimonia parviflora	X	X	X	X
Crataegus brainardii	X	X	X	X
Crataegus suborbiculata	X	X	X	X
Floerkea proserpinacoides	X	X	X	X
Cornus florida		X[1]	X	X
Myosotis macrosperma	X	X	X	X
Collinsonia canadensis		X	X	X
Vernonia gigantea	X	X	X	X
REGIONALLY RARE FLORA[2]				
Lycopodium obscurum var. obscurum	X			X
Carex amphibola var. turgida	X			X
Platanthera flava	X		X	X
Platanthera lacera	X			X
Coptis trifolia	X			X
Adlumia fungosa	X			
Arabis hirsuta	X			
Viola nephrophylla	X			
Sanicula gregaria	X			
Orobanche uniflora	X			
Hieracium canadense	X			

1 - Species will probably be delisted in future editions of the Atlas of the Rare Vascular Plants of Ontario.

2 - regionally rare means the plant was known from only 5 or less locations in the counties.

3 - species reaching or at their northern, southern, eastern or western limit in Ontario.

7.0 Emergency Response

An emergency response plan for dealing with flooding from Lake Erie now exists as part of the Kettle Creek Conservation Authority's Flood Warning and Flood Forecasting Plan. The flood warning is only applicable to the Village of Port Stanley as this is the only area low enough to be impacted by lake related flooding. As such the plan calls for the notification of the Village of Port Stanley's clerk, road superintendent, bridge operator, reeve and deputy reeve. Local radio stations should also be contacted.

The Village of Port Stanley also maintains a flood contingency plan which outlines municipal responsibilities, available resources and a list of contact people. The Village is presently updating its contact list to ensure that it remains current. Port Stanley also has a Peacetime Emergency Plan which outlines in more detail specific municipal responsibilities during emergencies, including floods.

In conjunction these two plans seem to be fairly comprehensive. Although they were developed with more consideration towards riverine flooding rather than lake induced flooding, they are still applicable to flooding from Lake Erie. The flood contingency plan should, however, be amended to specifically note that Lake Erie can be a source of flood water as well as Kettle Creek.

Structures at risk of damage from flooding can be partially protected by interim solutions such as sand bagging and pumping but there is no reason for this to be required as part of an emergency response. Within the preferred shoreline management concepts it is recommended that the dwellings at risk of flooding be properly floodproofed. If this is done then there will be no need for emergency flood protection. If dwellings are not properly floodproofed then any short term solutions should at least be carried out when static water levels are high and hence the risk of flooding is high, but before any severe storms actually occur.

The emergency response aspects of the Shoreline Management Plan should therefore concentrate on proper flood warning rather than any specific response to a flood. The probability of floods should be able to be estimated because severe floods only occur when static lake levels are high and strong winds occur. The occurrence of floods should be predicted by forecasting storms and not by monitoring progressing events such as done with riverine flooding. There is an automatic water level gauge in Port Stanley which the conservation authority can monitor via computer. Monitoring water levels can confirm predicted surges but because it provides no lead time it cannot be used to warn of upcoming floods. Once Port Stanley registers a high water level flooding will already be occurring.

In order to predict flood events, accurate forecasts of wind speed and direction are required. At present the conservation authority has access to World Weather Watch and to the ENVOY system run by the Streamflow Forecast Centre. The World Weather Watch information tends to be more accurate as it provides detailed local information from London and Exeter. ENVOY works on a more province wide basis.

During the 1985 to 1987 high water levels the Great Lakes Water Level Communication Centre of Environment Canada notified news services and the Streamflow Forecast Centre of predicted surge events. These surge events were predicted using lake wide circulation models driven by the forecast winds. As Environment Canada no longer automatically provides these warnings and to avoid dependency on these services in the future it is suggested that the conservation authority prepare a "look up" table of wind directions and speeds with corresponding surge water levels.

This would be done by performing a detailed storm surge analysis for varying wind speeds and directions. The results of this study would be a table of setup heights for any given wind condition which could be added to the static water level to estimate instantaneous water levels.

The methodology for preparing such tables would include the following:

- selection of a suitable numerical model;
- calibration of the model for use on Lake Erie;
- determination of the possible ranges of wind speed and duration of extreme events from measured wind data, recorded on Lake Erie;
- compute surge elevations at Port Stanley under changing wind direction and speeds to determine the sensitivity of the surge level to changing conditions. If sensitive, determine duration of constant wind speed and direction required to reach maximum surge level and prepare a correction table for wind durations less than that to generate maximum surge;
- select suitable range and increment of wind speed and direction for tablation of surge elevations;
- compute surge elevation for each speed and direction specified above using constant wind conditions;

The work necessary to prepare such tables is estimated to cost about $15,000 and would take roughly 30 man days.

8.0 Monitoring

The shoreline within the Kettle Creek Conservation Authority watershed is in a unique position of having excellent long term erosion data. This data is available as a result of the Port Burwell litigation defence study. This approach to assessment of erosion rates is much preferred over individual sections taken at various distant locations along the shoreline.

It is therefore recommended that future assessments of erosion rates be completed by using more recent mapping of the shoreline and comparing this data to the information generated from the Port Burwell project. It is not necessary to update the erosion file on an annual basis, as results obtained for such a short period would not be meaningful. It is suggested that such updating of erosion data be completed once every 10 years or more as accurate mapping becomes available. This process will become much more practical when a digital form of mapping is available.

There are two areas where monitoring of shoreline profiles is recommended. The first area is the east half of Port Stanley known as Orchard Beach. Here, the erosion of the shoreline has been stopped by various existing shoreline protection structures. However, it has been recognized that nearshore bottom erosion continues. As the bottom is lowered, design storm conditions at this site will worsen. Therefore, it is recommended that not less than four monitoring stations be established in the area of Orchard Beach. These stations should extend out from the top of the bank to a bottom elevation of 169.0 m GSC. Again, as this erosion is a very slow process, it is not necessary to complete this survey on an annual basis. It is likely that surveys should be spaced approximately 5 years apart. It is crucial that accurate horizontal and vertical control be provided. Potential locations of these monitoring lines are presented on the Port Stanley preferred and alternate shoreline management concepts photomosaics in Appendix B.

The second area to be monitored is the beach updrift of the harbour, within reaches 1 and 2 of Port Stanley. The monitoring program should provide data regarding the dune development as a result of programs proposed in the Shoreline Management Plans. The profile lines should extend to a bottom elevation of 169.0 m GSC. This may also provide more information regarding the shoreline beach profile changes should this data be compared with the wave climate just prior to the completion of the surveys.

Whenever a flood is reported a record of the uprush level and flood level should be established by field survey. These uprush and flood levels can usually be estimated from the position of debris, flood marks or as reported by residents. These, of course, should be verified by visual inspection of damage wherever possible. This type of flood monitoring program will add confidence to uprush level calculations.

Details of the beach monitoring program should be developed under the dune management program as recommended in Chapter 5. Details cannot be provided until the exact dune management program has been developed. This in turn will rely upon the upcoming MNR dune design manual.

The shore protection structures along Orchard Beach should be reviewed by the conservation authority on an annual basis and any rapid changes or damage to the structures should be noted. This review should be undertaken by a professional engineer experienced in the review of shoreline structures. Since the density of development on this area is high, damage to any one structure may effect other properties. This ongoing review is suggested as few of the structures have been formerly designed and their conditions and likely performance are only estimated by reviewing the above water portion of the structure.

Monitoring of other shoreline related environmental issues, such as fish habitat, should be continued by the responsible agencies so that up to date information is available for assessment of any future projects.

Private and public drains should be monitored by the conservation authority to provide early identification of any drains which may potentially lead to gully formation.

8.1 Cost of Monitoring Program

The cost of updating erosion rates along the high bluff sections of the shoreline has not been determined. The Ministry of Natural Resources is currently considering a pilot project to determine erosion rates in a manner similar to that suggested herein. Cost can be better provided by the province based on the result of the pilot project.

The manpower requirements for monitoring nearshore lowering in the Orchard Beach area is estimated to be 16 man days per survey. An additional 10 days should be allocated for the initial surveys to establish ground controls. It is our view that the staff of the authority may not be appropriately trained for precise sounding surveys as required here and that a capital expenditure for accurate depth sounding equipment may not be justified. It is therefore suggested that this work be completed by qualified personnel on a consulting contract basis. On the basis of this approach, the cost of the initial contract would be about $10,000 and subsequent monitoring would likely cost $7,000 per survey.

The cost of monitoring flood levels after major events is likely to be in the range of $2,000. This cost includes an allocation for a survey crew for three days and time for a technician to plot the result.

The cost of an annual review of structures at Orchard beach is estimated at $1,500. This is based on a one day review by a professional engineer followed by a brief letter type report.

All cost estimates are in 1989 dollars.

9.0 Public Information

There are two aspects to the public information component of a shoreline management plan. The first component is the collection of information from the public during the preparation of the shoreline management plan. The second component deals with a public information program after completion of the shoreline management plan. The second component is, in part, based on the input provided by the public during the course of this study.

9.1 Public Input During Preparation of the SMP

A questionnaire was hand delivered to all residents along the waterfront. Information was requested to assess the current public knowledge of shoreline management issues and to obtain specific information regarding past flood and erosion problems and solutions. As an introduction to this questionnaire a brief information bulletin regarding the preparation of this shoreline management plan was provided. A summary of the questionnaire is provided in Section 2.3. A copy of the questionnaire and returned responses are provided in Appendix E.

A public open house was held in Port Stanley on November 2, 1989 to obtain input from the local residents and general public. A brief information leaflet and request for comments was distributed at the meeting. A copy of the leaflet is presented in Appendix E. Although the open house was well attended by approximately 80 people, the written response to the handout was poor. Only four people submitted comments.

9.2 Future Public Information

Once the study is completed, a public information program must be implemented to ensure that residents continue to be aware of the results of the study as well as any follow up information that becomes available. The public information program is crucial as it has been our experience that many shoreline property owners recognize the dangers of waterfront development only during periods of high lake levels. Information should also be provided to potential new shoreline property owners.

Based on the review of the responses from the questionnaire, the conservation authority needs to make people aware of and continue to remind them of the existence of the shoreline management plan and the role that the conservation authority plays in shoreline management under the Provincial Policy. Although the public meeting and associated announcements have likely increased the public's awareness of the role of the conservation authority, new residents will need to be informed. Basic information regarding the causes of the long term erosion and fluctuations in water levels, also need to be provided. The questionnaire shows a lack of knowledge of the public in these areas.

It is suggested that an annual bulletin or an information leaflet be published by the conservation authority and distributed directly to shoreline property owners. This should also be available to the general public through the municipal offices and other public agencies. It is also suggested that local real estate offices and agents be provided this information and asked to distribute it to potential buyers. The objectives of this leaflet would be to both notify people that there is a shoreline management plan and to discuss any new aspects of shoreline management in the watershed. There is no doubt that publishing this information bulletin will also minimize any potential liabilities to the conservation authority and municipalities resulting from flooding and erosion problems.

It is suggested that this bulletin be distributed in the spring when many residents start to plan their "projects" for the summer and when seasonal residents begin to make use of their summer cottages. The bulletin may typically contain the following information:

- reminder of the existence of the shoreline management plan and the role of the Kettle Creek conservation Authority in shoreline management.

- reminder of approvals required for work along the shoreline.

- water level forecast (from Monthly Water Level Bulletin) for the upcoming summer.

- any new developments in the implementation of shoreline management plan such as adoption of Regulations by the conservation authority or bylaws by municipalities.

- feature article dealing with recent developments in shoreline management on the Great Lakes (research results, I.J.C. studies and resolutions, major local waterfront developments etc.)

It is further suggested that during periods of high lake levels additional action be taken. This could include public meetings to provide specific information to shoreline owners about the levels of existing dangers, action to be taken to minimize dangers etc.

It is not expected that this public information program will directly result in property owners taking immediate action to protect their property from erosion or flooding. However, property owners will be aware of the shoreline management plan and can, if they so choose, take advantage of the information available.

9.3 Cost of Public Information Program

The suggested annual bulletin is likely to take 8 to 10 man days to prepare ready for reproduction. This time allocation includes time to write the articles, assemble information, complete any necessary art work (drafting) and review by the manger of the conservation authority. The cost of the manpower will depend on cost of personnel used. The cost of reproduction and mailing is likely to be less than $2,000 annually.

9.4 Approvals

In accordance with the pending provincial policy on shoreline management a regulatory shoreland zone will be established throughout the study area. This zone will be based on the landward most limits of:

- the 100 year uprush level (Section 3.5)
- the 100 year erosion limit (Section 3.7)
- the extent of the unconsolidated beach deposit.

The regulatory shoreland zone is shown on the existing conditions photomosaics in Appendix A.

In order to construct anything within the shoreland zone approval will be required from at least the conservation authority and possibly also from the provincial and federal governments. Approvals from other agencies are discussed in Section 9.4.1.

Under the pending provincial policy the conservation authority will be encouraged to revise their existing flood and fill line regulations to include the regulatory shoreland zone. Their regulations will be somewhat general in nature and will be implemented in accordance with both the pending provincial policy and the conservation authorities own implementing policy. We recommend that the conservation authority consider within their implementing policy two different standards; one for new development and a second for construction associated with existing development.

In order to introduce new development to the regulatory shoreland zone. The landowner should have to demonstrate that all hazards associated with the definition of the shoreland zone has been overcome. If for example a specific area has been included within the shoreland zone because it is within the 100 year uprush level then the development must be floodproofed to withstand that level of uprush. The floodproofing technique used must be approved by the conservation authority. If that area is also within the 100 year erosion limit then an approved erosion control structure will be required. If approved techniques are not used to overcome these hazards then development should not be allowed.

If, however, development already exists within the regulated shoreland zone and that development is at risk from a natural hazard then any action proposed to reduce that risk should be considered. The conservation authority should encourage the landowner, for his own benefit, to take appropriate steps to overcome the hazards to the same extent as required for new development. However, the landowner may not be willing to do so. If not, then the conservation authority may consider accepting a lower level of effort providing that:

- there is a net reduction in risk to both life and/or property.
- there are no adverse environmental or coastal impact.
- no dwelling should be relocated or structure constructed if it is not capable of withstanding a 100 year design storm.

The conservation authority may request a maintenance agreement.

In order to assist the conservation authority in assessing the nature and the impact of the proposed works, it is recommended that the proponent prepare an impact statement. The impact statement should be a document that will address all aspects of the project and should be circulated by the conservation authority to all approving agencies. The impact statement should address the following issues:

- Site Location
- Site Description including environmentally significant features
- Coastal Conditions (Design Parameters)
- Littoral Transport
- Description of Proposed Works
- Design Calculations
- Construction Schedule
- Maintenance Requirements and Access
- Impact on Littoral Transport and Environment

The impact assessment should also be required to demonstrate three key items. These are:

1. That there will be no increase in the long term recession rate on neighbouring properties caused by the proposed development.

2. That the proposed development will not cause damage to adjacent structures.

3. That the proposed development will in no way have any detrimental effects on the environment.

For the average home owner who wishes to undertake a relatively modest project with no major issues having to be resolved an impact assessment would likely cost in the order of $2,000 to $3,000 (1989).

We recommend that, should the authority adopt formal regulation for the shoreland zone, all construction drawings and impact statements should carry a seal of a professional engineer licensed to practice in Ontario and at least that all construction drawings carry a seal of a licensed engineer and impact statements be completed by a recognized expert approved by the conservation authority.

9.4.1 Approvals by Other Agencies

As indicated above, the public needs to be provided with information regarding approvals for waterfront related construction. There are many existing Acts that control and regulate work near water. These Acts are federal and provincial legislations. The application of these acts and the level of review and comments provided by the approving agencies will vary depending on the size and specific details of the project. The following approvals may be required.

Public Lands Act and Amends Thereto, by the Ministry of Natural Resources.

To allow construction of works on both public and private lands. Unless otherwise indicated, all lands under the lake water is the property of the provincial crown.

Lakes and Rivers Improvement Act, by the Ministry of Natural Resources.

A permit to construct any structure in or along any stream, river or lake. Permit includes technical review and location approval.

The Beach Protection Act, by the Ministry of Natural Resources.

This act regulates the removal of sand and gravel from beaches and water of any lake or stream.

Water Resources Act, by the Ministry of the Environment.

No permit is required prior to construction, but MOE has the right to stop work that they judge adversely affects water quality.

Navigable Waters Protection Act, Ministry of Transportation.

Controls construction in navigable waters to ensure maintenance of the right of passage. Submissions under NWPA are automatically circulated to Environment Canada, the local conservation authority and the Ministry of Natural Resources for review.

In addition to these approval requirements, other acts may apply depending on the nature of the development, the size and the proponent. For example, the Environmental Assessment Act may be applied for public projects.

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GLOSSARY

- Accepted Engineering Principles refers to those principles, methods and procedures involving wave uprush and other water related hazards which are used and applied in current hydro-technical engineering practice and have been approved by the local Conservation Authority and the Ministry of Natural Resources.

- Accepted Geo-Technical Principles refer to those principles, methods and procedures involving slope stability analysis which are used and applied in current geo-technical practice and have been approved by the local Conservation Authority and Ministry of Natural Resources.

- Accretion May be either NATURAL or ARTIFICIAL. Natural accretion is the buildup of land, solely by the action of the forces of nature, on a BEACH by deposition of waterborne or airborne material. Artificial accretion is a similar buildup of land by reason of an act of man, such as the accretion formed by a groin, breakwater, or beach fill deposited by mechanical means. Also AGGRADATION.

- Alongshore Parellel to and near the shoreline; same as LONGSHORE.

- Artificial Nourishment The process of replenishing a beach with material (usually sand) obtained from another location.

- Attenuation (1) A lessening of the amplitude of a wave with distance from the origin. (2) The decrease of water-particle motion with increasing depth. Particle motion resulting from surface oscillatory waves attenuates rapidly with depth, and practically disappears at a depth equal to a surface wavelength.

- Average Annual Recession Rate refers to the average annual linear landward retreat of shore land.

- Backshore That zone of the shore or beach lying between the foreshore and the coastline and acted upon by waves only during severe storms, especially when combined with exceptionally high water. Also BACKBEACH. It comprises the BERM or BERMS.

- Bar A submerged or emerged embankment of sand, gravel, or other unconsolidated material built on the sea floor in shallow water by waves and currents.

- Bathymetry The measurement of depths of water in oceans, seas, and lakes; also information derived from such measurements.

- Beach Berm A nearly horizontal part of the beach or backshore formed by the deposit of material by wave action. Some beaches have no berms, others have one or several.
- Beach Erosion The carrying away of beach materials by wave action, tidal currents, littoral currents, or wind.
- Beach Face The section of the beach normally exposed to the action of the wave uprush. The FORESHORE of a BEACH. (Not synonymous with SHOREFACE.)
- Beach Width The horizontal dimension of the beach measured normal to the shoreline.
- Bench Mark A permanently fixed point of known elevation.
- Bluff A high steep bank or cliff.
- Breaker Depth The stillwater depth at the point where a wave breaks. Also BREAKING DEPTH.
- Breakwater A structure protecting a shore area, harbor, anchorage, or basin from waves.
- Bulkhead A structure or partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.
- Celerity Wave speed.
- Connecting Channels refers to the rivers (e.g. St. Mary's, St. Clair, Detroit, and Niagara Rivers) which convey water flows between Lakes Superior, Huron, St. Clair, Erie and Ontario. This also includes the St. Lawrence River.
- Datum The horizontal plane to which soundings, ground elevations, or water surface elevations are referred. Also REFERENCE PLANE.
- Deep Water Water so deep that surface waves are little affected by the lake bottom. Generally, water deeper than one-half the surface wavelength is considered deep water.
- Development means the construction, erection or placing of a building or structure of any kind or the making of an addition or alteration to a building or structure that has the effect of increasing the size or usability thereof, and includes such related activities as site grading and the placing or dumping of fill.
- Development Setback refers to a measured distance inland from a specified feature (e.g. top of bluff) wherein development is prohibited or restricted except for the purpose of flood and/or erosion control.

- Downdrift The direction of predominant movement of littoral materials.
- Dunes Ridges or mounds of loose, wind-blown material, usually sand.
- Dynamic Beach refers to the zone of un-consolidated sediment that extends landward to the historic limit of the beach. Commonly there is an abrupt change in slope and/or composition at the landward limit.
- Echo Sounder An electronic instrument used to determine the depth of water by measuring the time interval between emission of a sonic or ultrasonic signal and the return of its echo from the bottom.
- Erosion means a volumetric reduction of shore land by natural processes.
- Fill, Construction and Alteration to Waterways Regulation means a regulation passed pursuant to section 28 (1) of the Conservation Authorities Act, R.S.O. 1980, or its successors, whereby a Conservation Authority may, among other matters, regulate:
  - the straightening, changing, diverting or interfering in any way with the existing channel of a river, creek, stream or watercourse;
  - the construction of any building or structure in or on a pond or swamp or in any area susceptible to flooding; and
  - the placing or dumping of fill of any kind in any defined part of the area over which the Conservation Authority has jurisdiction in which, in the opinion of the Conservation Authority, the control of flooding or pollution or the conservation of land may be affected.
- Flood means a rise in the stillwater level resulting in the inundation of areas adjacent to a lake or connecting channel not ordinarily covered by water.
- Floodproofing means a combination of structural changes and/or adjustments incorporated into the basic design and/or construction or alteration of individual buildings, structures or properties subject to flooding so as to reduce or eliminate flood damages.
- Foreshore The part of the shore lying between the crest of the seaward berm (or upper limit of wave uprush) and the ordinary low water mark, that is ordinarily traversed by the uprush and backrush of the waves.

GLOSSARY
(cont.)

- Freeboard The additional height of a structure above design high water level to prevent overflow. Also, at a given time, the vertical distance between, the water level and the top of the structure.

- GSC refers to elevations above Geodetic Survey of Canada datum. This datum is 0.2 m higher than IGLD at Port Stanley.

- Gentle slope refers to a slope having a rise of less than 1 unit vertically to 2.5 units horizontally.

- Geomorphology That branch of both physiography and geology which deals with the form of the earth, the general configuration of its surface, and the changes that take place in the evolution of landform.

- Great Lakes - St. Lawrence River System refers to the major water system consisting of Lakes Superior, Huron, St. Clair, Erie and Ontario and their connecting channels, and the St. Lawrence River within the boundaries of the Province of Ontario.

- Hazardous Substances means substances which individually, or in combination with other substances, are normally considered to pose a danger to public health, safety and the environment. These substances generally include a wide range of materials that are toxic, ignitable, corrosive, reactive, radioactive or pathological

- Headland (Head) A high steep-faced promontory extending into the lake.

- Hindcasting, Wave The use of historic wind data to calculate wave characteristics that probably occurred at some past time.

- IGLD refers to elevations above the International Great Lakes Datum established in 1955. At Port Stanley IGLD is 0.2 m lower than GSC datum.

- Inshore (Zone) In beach terminology, the zone of variable width extending from the low water line through the breaker zone. SHOREFACE.

- Level of Protection means a specified elevation or setback to which new development must not be susceptible to flood and/or erosion-related damage.

- Littoral Cell see SHORELINE SEDIMENT COMPARTMENT

- Littoral Drift The sedimentary material moved in the littoral zone under the influence of waves and currents.

- Littoral Transport The movement of littoral drift in the littoral zone by waves and currents. Includes movement parallel (longshore transport) and perpendicular (cross shore transport) to the shore.

- Littoral Transport Rate Rate of transport of sedimentary material parallel to or perpendicular to the shore in the littoral zone. Usually expressed in cubic meters per year. Commonly used as synonymous with LONGSHORE TRANSPORT RATE.

- Littoral Zone In beach terminology, an indefinite zone extending seaward from the shoreline to just beyond the breaker zone.

- Load The quantity of sediment transported by a current. It includes the suspended load of small particles, and the bedload of large particles that move along the bottom.

- Local Conditions refers to the geo-physical, hydro-physical, environmental, economic and social characteristics of reaches which may affect shoreline management.

- Longshore Parellel to and near the shoreline.

- Longshore Transport Rate Rate of transport of sedimentary material parallel to the shore. Usually expressed in cubic meters per year. Commonly used as synonymous with LITTORAL TRANSPORT RATE

- 100-Year Erosion Limit means 100 times the average annual recession rate measured landward from the first break in slope.

- 100-Year Flood Level means the peak instantaneous stillwater level due to the combined occurrences of mean monthly lake levels and wind setup having a total probability of being equalled or exceeded of 1% in any year. In connecting channels and the St. Lawrence River the 100-year flood level is the peak instantaneous stillwater level having a probability of being equalled or exceeded of 1% in any year.

- Nearshore (Zone) In beach terminology an indefinite zone extending seaward from the shoreline well beyond the breaker zone.

- Other Water-Related Hazards refers to water-associated phenomena acting on shore lands other than flooding and wave uprush. This includes, but is not limited to, wave spray and ice action.

- Overtopping Passing of water over the top of a structure as a result of wave runup or surge action.

GLOSSARY
(cont.)

- Overwash That portion of the uprush that carries over the crest of a berm or of a structure.

- Profile, Beach The intersection of the ground surface with a vertical plane; may extend from the top of the dune line to the seaward limit of sand movement.

- Reach refers to a stretch of shoreline having similar physiography, geologic composition, average annual recession rate and orientation to waves.

- Recession (of a beach) (1) A continuing landward movement of the shoreline. (2) A net landward movement of the shoreline over a specified time. Also RETROGRESSION.

- Refraction (of Water Waves) (1) The process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours. (2) The bending of wave crests by currents.

- Refraction Coefficient The square root of the ratio of the spacing between adjacent orthogonals in deep water and in shallow water at a selected point. When multiplied by the SHOALING FACTOR and a factor for friction and percolation, this becomes the WAVE HEIGHT COEFFICIENT or the ratio of the refracted wave height at any point to the deepwater wave height. Also the square root of the ENERGY COEFFICIENT.

- Regulatory Erosion Standard means the approved standard(s) used to define shore land erosion limits, based on recession rates, for regulatory purposes.

- Regulatory Flood Standard means the approved standard(s) used to define shore land flood limits for regulatory purposes.

- Regulatory Shore Land Zone refers to the land, including that covered by water, between the international boundary and the furthest landward limit of the regulatory flood standard and/or the regulatory erosion standard and/or the dynamic beach zone.

- Restricted means that new development is limited to:

  - flood and/or erosion control structures;
  - facilities which by their nature must locate within or near shore lands;
  - ancillary facilities of an adjacent land use which are of a passive, non-stuctural nature and do not adversely affect the flood and/or erosion susceptibility of updrift/downdrift shore lands and developments.

GLOSSARY
(cont.)

- Revetment A facing of stone, concrete, etc., built to protect a scarp, embankment, or shore structure against erosion by wave action or currents.

- Riprap A layer, facing, or protective mounds of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used.

- Rubble (1) Loose angular waterworn stones along a beach. (2) Rough, irregular fragments of broken rock or concrete

- Runup The rush of water up a structure or beach on the breaking of a wave. Also UPRUSH. The amount of runup is the vertical height above stillwater level that the rush of water reaches.

- Scour Removal of material by waves, winds and currents, especially at the base or toe of a shore structure.

- Shallow Water (1) Commonly, water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than 1/25 the surface wavelength as shallow water. TRANSITIONAL WATER and DEEP WATER.

- Shoal (1) To become shallow gradually. (2) To cause to become shallow. (3) To proceed from a greater to a lesser depth of water.

- Shoaling Coefficient The ratio of the height of a wave in water of any depth to its height in deep water with the effects of refraction, friction, and percolation eliminated.

- Shore The narrow strip of land in immediate contact with the sea, including the zone between high and low water lines. A shore of unconsolidated material is usually called a beach.

- Shoreface The narrow zone seaward from the SHORELINE covered by water over which the beach sands and gravels actively oscillate with changing wave conditions. See INSHORE (ZONE).

- Shoreline The intersection of a specified plane of water with the shore or beach. (e.g., the highwater shoreline would be the intersection of the plane of mean high water with the shore or beach.)

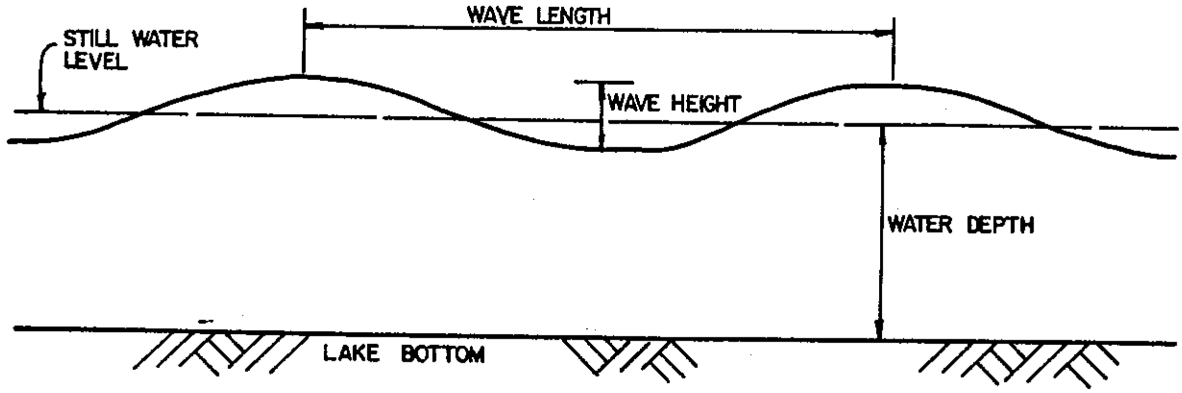
- Shoreline Sediment Compartment refers to a self-contained coastal sediment system that has no movement of sediment across its boundaries. The along-shore limits are defined by natural formations or artificial barriers where the net sediment movement changes direction or becomes zero. By definition, this system can encompass more than one littoral cell.

GLOSSARY
(cont.)

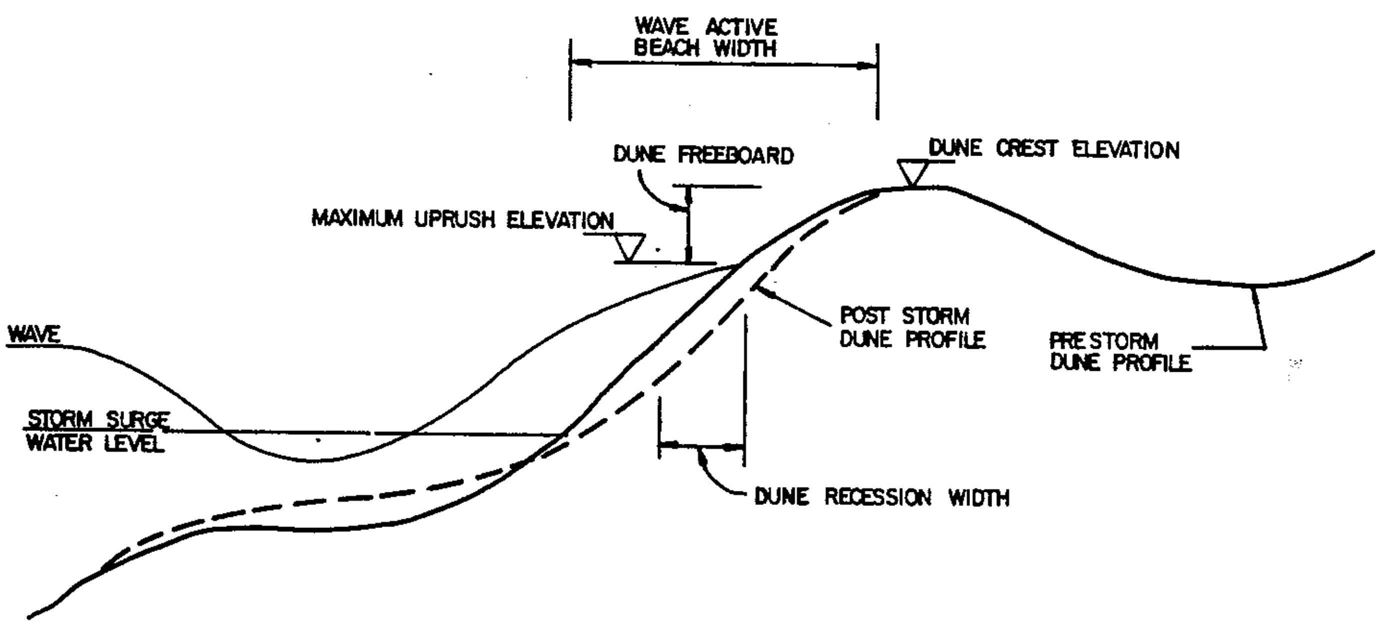
- Shore Protection refers to non-structural/structural works which are intended to reduce damages caused by flooding and/or erosion.
- Significant Wave Height The average height of the one-third highest waves of a given wave group.
- Static Water Level refers to the elevation that the surface of the water would assume if wind and other atmospheric and/or tidal forces were absent.
- Steep slope refers to a slope having a rise of greater than 1 unit vertically to 2.5 units horizontally.
- Stillwater Level refers to the elevation that the surface of the water would assume if wave action was absent.
- Storm Surge A rise above static water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress. See WIND SETUP.
- Transitional Water Water depth ranging between DEEP WATER AND SHALLOW WATER at which surface waves are first affected by the bottom topography. It is customary to consider water depths less than 1/2 and greater than 1/25 the surface wave length as transitional water.
- Updrift The direction opposite that of the predominant movement of littoral materials.
- Wave A ridge, deformation, or undulation of the surface of a liquid.
- Wave Action Beach Width is the width of beach, measured from any given still water line, which would undergo a noticeable profile change if subject to a 100 year design storm at that given still water level.
- Wave Height The vertical distance between a crest and the preceding trough. See also SIGNIFICANT WAVE HEIGHT.
- Wavelength The horizontal distance between similar points on two successive waves measured perpendicular to the crest.
- Wave Spectrum In ocean wave studies, a graph, table, or mathematical equation showing the distribution of wave energy as a function of wave frequency. The spectrum may be based on observations or theoretical considerations. Several forms of graphical display are widely used.

- Wave Uprush means the rush of water up onto the beach, bluff or structure following the breaking of a wave; for any given water level the limit of uprush is the point of farthest uprush.

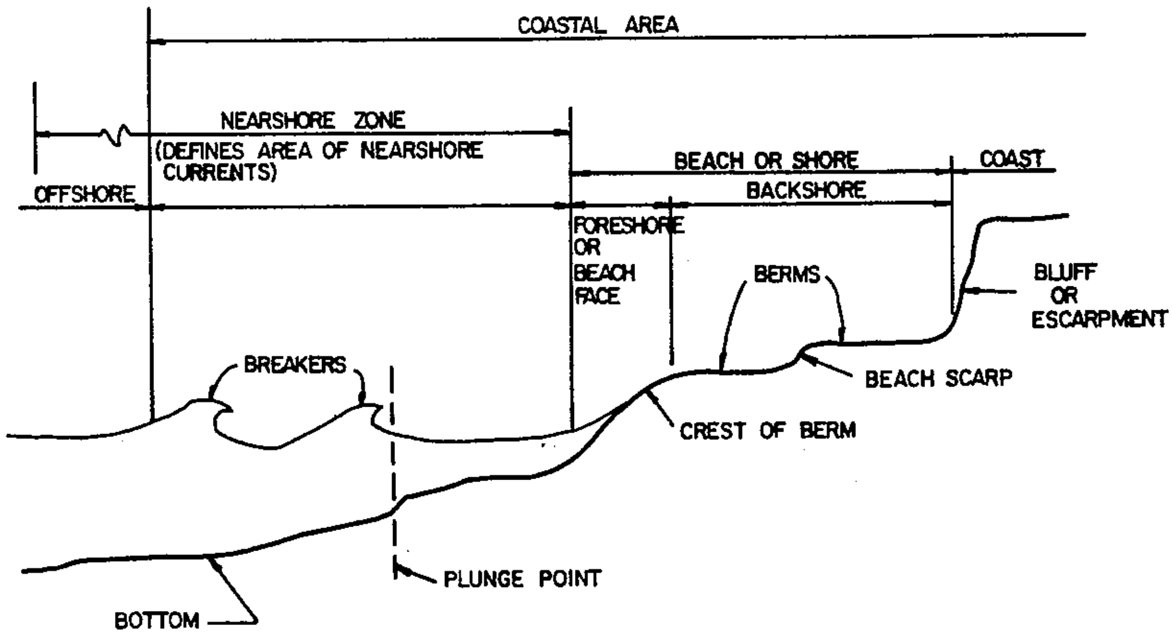
- Wind Setup (1) The vertical rise in the stillwater level on the leeward side of a body of water caused by wind stresses on the surface of the water. (2) The difference in still water levels on the windward and the leeward sides of a body of water caused by wind stresses on the surface of the water. (3) Synonymous with and STORM SURGE. STORM SURGE is usually reserved for use on the ocean and large bodies of water. WIND SETUP is usually reserved for use on reservoirs and smaller bodies of water.



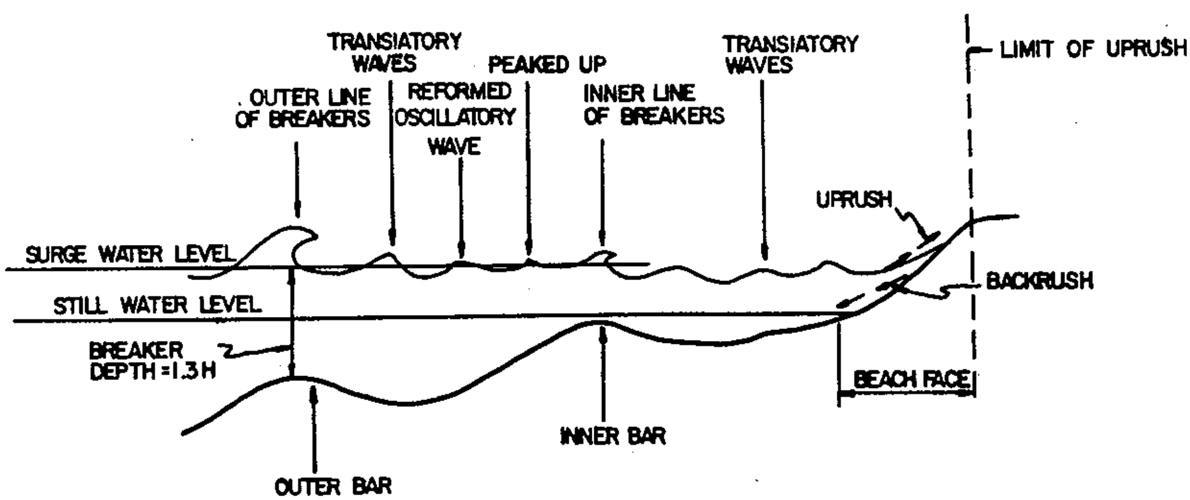
— BREAKING WAVE HEIGHT IS WAVE HEIGHT WHEN WAVE BREAKS.
— 1:100 YEAR WAVE HEIGHT IS WAVE HEIGHT STATISTICALLY EXPECTED TO OCCUR ONCE EVERY 100 YEARS.



WAVE HEIGHT AND DUNE PROFILE CHARACTERISTICS TERMS



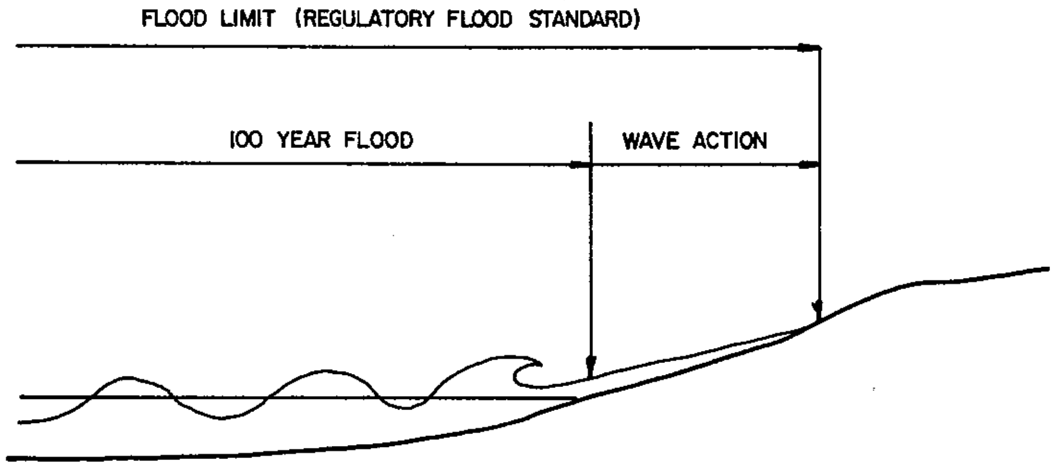
BEACH PROFILE RELATED TERMS



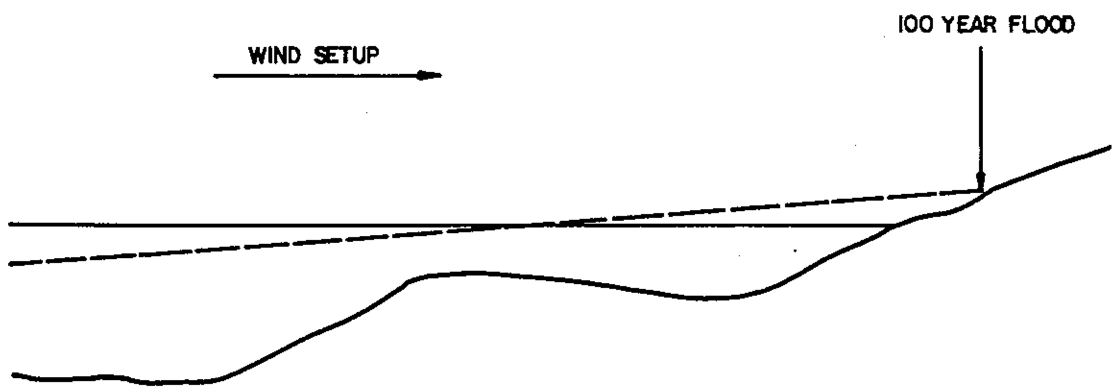
SCHEMATIC DIAGRAM OF WAVES IN THE BREAKER ZONE

: after Shore Protection Manual, U.S. Army Corps of Engineers, 1977.

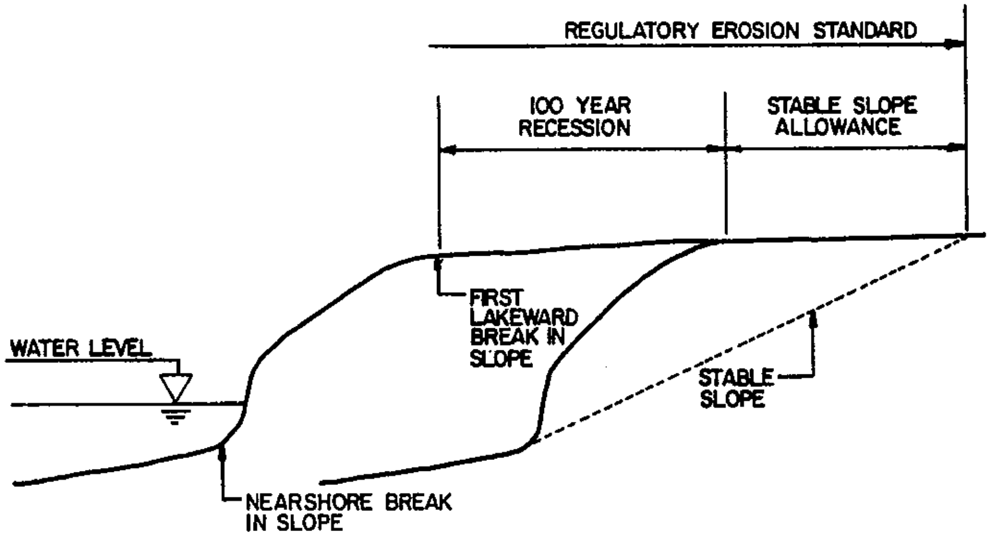
REGULATORY FLOOD STANDARD



100 YEAR FLOOD LEVEL



REGULATORY EROSION STANDARD



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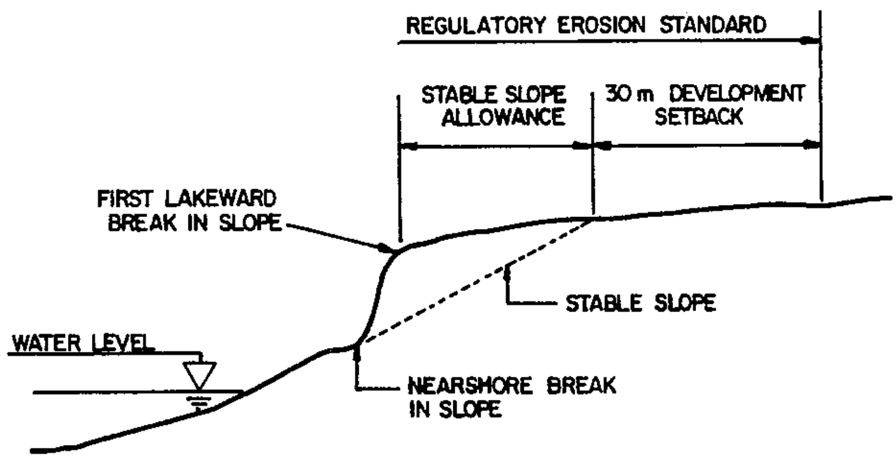

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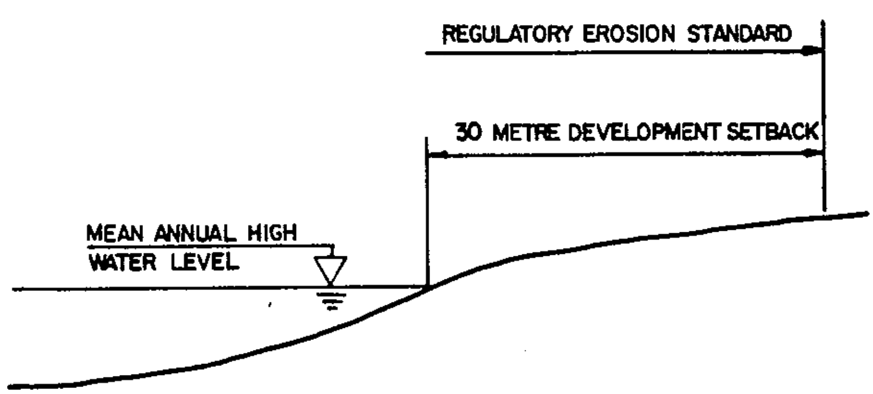
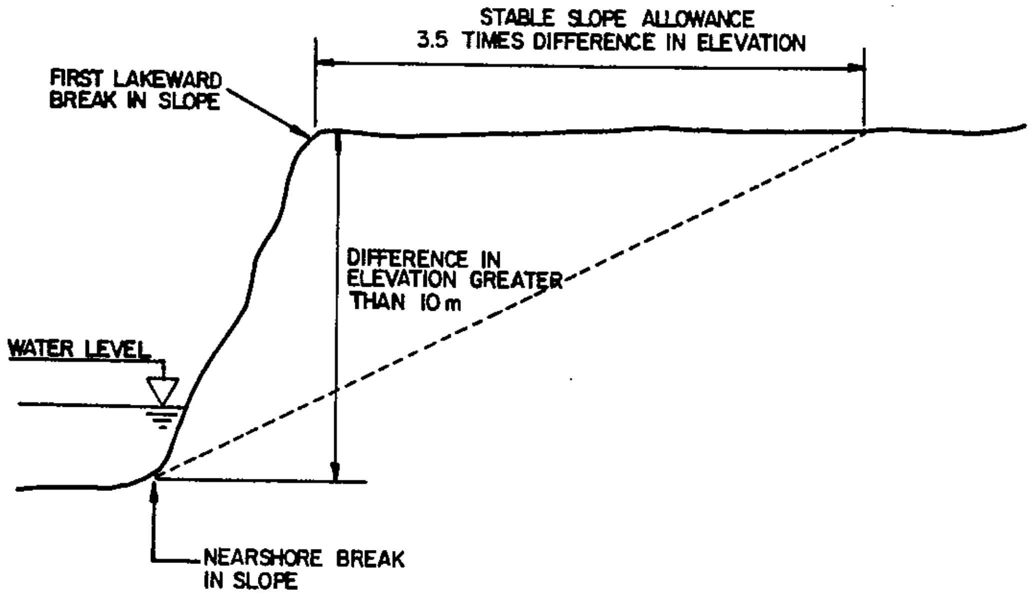
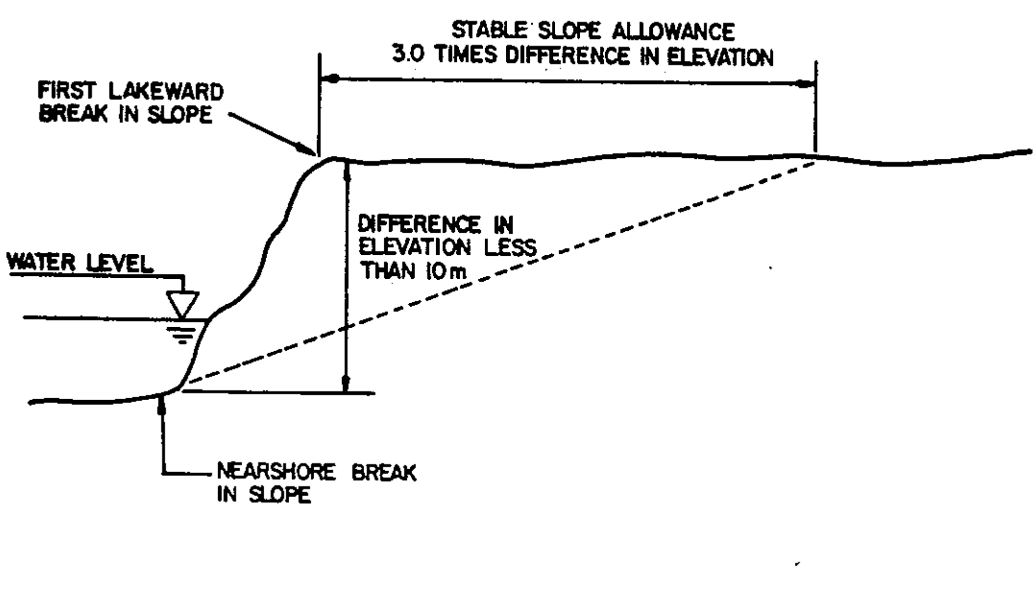


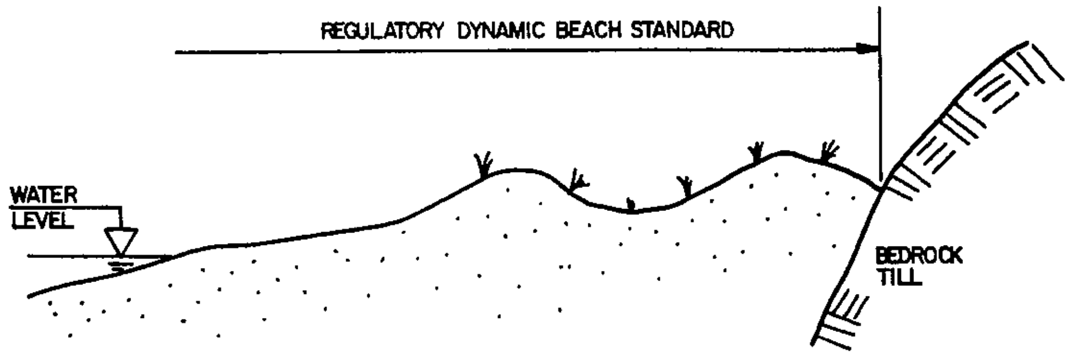
STABLE SLOPE ALLOWANCE



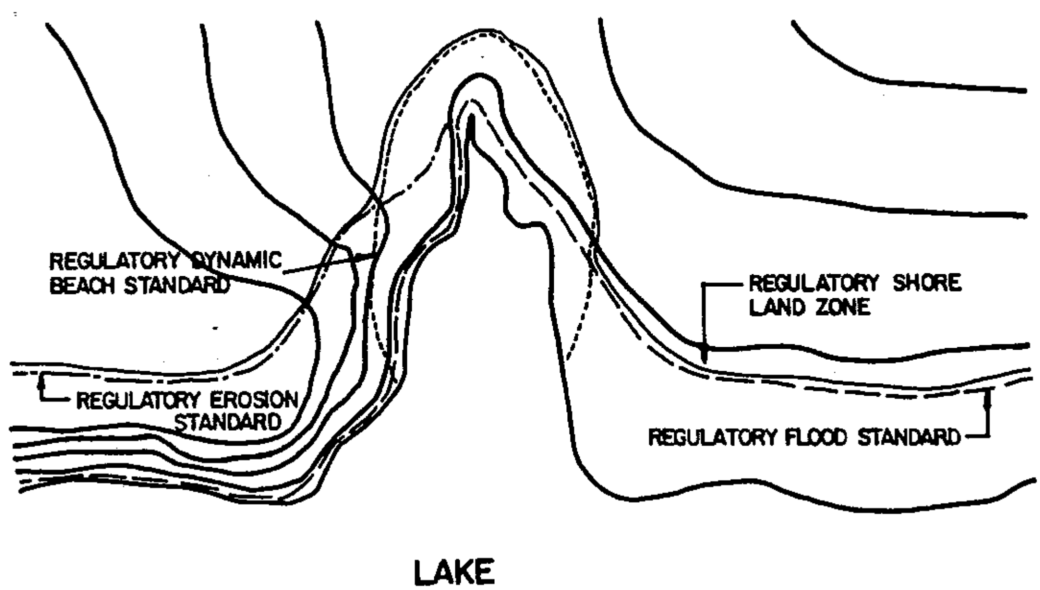
STABLE SLOPE ALLOWANCE



REGULATORY DYNAMIC BEACH STANDARD



REGULATORY SHORE LAND ZONE



July 10, 1989

LANDOWNER QUESTIONNAIRE, LAKE ERIE SHORELINE

Dear Landowner:

The Kettle Creek Conservation Authority is undertaking a study of shoreline flooding and erosion problems. As a shoreline landowner we value you opinion and would like to learn more about both the problems you have experienced and how you have solved them.

The purpose of this study is to balance the options of shoreline protection, development, environmental impact, erosion monitoring, emergency response and public education in an overall management plan of our shoreline resources. To ensure that we prepare the best possible plan we need your input.

Engineers hired by the Conservation Authority will be surveying shoreline properties in the near future as part of this study. A public meeting will be scheduled in September to present the draft plan to shoreline owners and to solicit their responses to the plan.

We appreciate your co-operation and request that you return this questionnaire in the enclosed addressed and stamped envelope by July 28, 1989. If you have any questions please feel free to contact Mr. Bryan Hall, General Manager of the Kettle Creek Conservation Authority at 631-1270.

Thank you on behalf of the Kettle Creek Conservation Authority.

Bruce M. Pinchin, P.Eng.
Project Engineer
Philpott Associates Coastal Engineers Limited.

PART 1 - GENERAL

NAME: ____________________

ADDRESS: ____________________

____

TELEPHONE: ____________________

1. HOW LONG HAVE YOU LIVED AT YOUR PRESENT RESIDENCE?

   Since 19 ___ ___ Permanent ____ Seasonal ____

2. ARE YOU AWARE THAT YOUR PROPERTY LIES WITHIN THE KETTLE CREEK CONSERVATION AUTHORITY WATERSHED?

   Yes ____, No ____

3. ARE YOU AWARE THAT THE CONSERVATION AUTHORITY HAS BEEN DESIGNATED BY THE PROVINCIAL GOVERNMENT AS THE AGENCY RESPONSIBLE FOR COMMENTING ON THE USE AND DEVELOPMENT OF YOUR SHORELINE?

   Yes ____, No ____

4. PRIOR TO RECEIVING THIS QUESTIONNAIRE WERE YOU AWARE THAT THE KETTLE CREEK CONSERVATION AUTHORITY IS PREPARING A SHORELINE MANAGEMENT PLAN THIS YEAR?

   Yes ____, No ____

5. WHICH OF THE FOLLOWING BEST DESCRIBES YOUR SHORELINE?

   - Sandy beach ____________
   - Low bluff, less than 20 feet high ____________
   - High bluff, more than 20 feet high ____________

6. WHAT DO YOU BELIEVE CAUSES WATER LEVEL FLUCTUATIONS ON THE GREAT LAKES?

   - Weather patterns ____________
   - Government Regulations (eg. as for shipping, hydro, water diversions, etc) ____________
   - Don't know ____________
   - Other (please specify) ____________

PART 2 - SHORELINE EROSION

Please only answer those questions which you feel apply to your circumstances.

7. DO YOU HAVE, OR HAVE YOU EVER HAD, A SHORELINE EROSION PROBLEM AT THIS PROPERTY?

Yes ____, No ____

8. IF YOU HAVE EXPERIENCED EROSION OF A BLUFF WHAT DO YOU FEEL WAS THE CAUSE?

a. Wave Action ____________

b. Water Seepage from the Bluff Face ____________

c. A combination of both a and b ____________

d. High water levels ____________

e. Structures built on or out from shoreline ____________

f. Some other cause (please specify) ____________

If you answered yes to either b or c, do you know where the seeping water comes from? ____________

____

9. IF YOU HAVE EXPERIENCED EROSION OF A BEACH WHAT DO YOU FEEL WAS THE CAUSE?

a. Wave Action? ____________

b. Structures built on or out from the shoreline? ____________

c. High water levels ____________

d. Removal of vegetated cover ____________

e) Some other cause (please specify) ____________

____

____

____

____

10. IF YOU HAVE EXPERIENCED SHORELINE EROSION WHAT HAVE YOU DONE (SINCE <u>YOU</u> HAVE OWNED THE PROPERTY) TO REPAIR THE DAMAGE OR TO PROTECT THE SHORELINE?

TYPE OF WORK	YEAR DONE	APPROXIMATE COST
____________________	__________	____________
____________________	__________	____________
____________________	__________	____________
____________________	__________	____________
____________________	__________	____________
____________________	__________	____________

11. IF YOU HAVE EXPERIENCED SHORELINE EROSION WHAT HAS BEEN THE APPROXIMATE COST OF YOUR LOSSES (eg. building, land, crops, etc.) SINCE YOU HAVE OWNED THE PROPERTY? __________

____

12. IN YOUR OPINION, HOW EFFECTIVE IS YOUR PRESENT SHORELINE PROTECTION STRUCTURE(S)? (please check ONE)

Excellent _______ Good _______ Fair ______ Poor ______

13. DO YOU FEEL THAT THERE IS AN IMMEDIATE EROSION THREAT TO YOUR:

LAND? Yes ____, No ____

PROTECTION STRUCTURE? Yes ____, No ____

DWELLING? Yes ____, No ____

OTHER (Please Specify) ________________________________

____

____

If yes what is the value of the land, structure, etc, at risk? ________________________________

14. HAVE YOU EVER SOUGHT OR TRIED TO SEEK ANY PROFESSIONAL OR TECHNICAL ADVICE ABOUT HOW TO DEAL WITH YOUR SHORELINE EROSION PROBLEM?

Yes ____, No ____

If yes, when and from whom? ________________________________

____

If no, are there any particular reasons why you haven't (eg. didn't know who, didn't think it would help, too expensive)? ________________________________

____

15. PLEASE PROVIDE ANY ADDITIONAL INFORMATION OR COMMENTS WHICH YOU FEEL WILL HELP US ASSESS SHORELINE EROSION PROBLEMS ________________________________

____

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PART 3 - FLOODING

Please only answer those questions which you feel apply to your circumstances.

16. DO YOU HAVE, OR HAVE YOU EVER HAD, A FLOODING PROBLEM AT THIS PROPERTY?

    Yes ____, No ____

17. IF YOU HAVE EXPERIENCED FLOODING HOW DID THE FLOOD WATER APPROACH YOUR PROPERTY (please check one or more)?

    Don't know ____________

    Your Lake Erie frontage ____________

    Your neighbour's property ____________

    Your street ____________

    Other (please specify) ____________________

    ____

18. IF YOU HAVE EXPERIENCED FLOODING WHAT HAVE YOU DONE (SINCE YOU HAVE OWNED THE PROPERTY) TO REPAIR THE DAMAGE OR TO PREVENT FUTURE FLOODING?

TYPE OF WORK	YEAR DONE	APPROXIMATE COST

19. WHAT ARE THE APPROXIMATE COSTS ASSOCIATED WITH ANY FLOOD DAMAGES NOT REPAIRED? ____________________

20. CAN YOU TELL US WHEN FLOODING HAS OCCURRED ON YOUR PROPERTY? (This will help to assess your risk of future floods).

DATE/YEAR FLOODED	ITEM FLOODED (eg. house or land)	DEPTH OF WATER
____________	____________	____________
____________	____________	____________
____________	____________	____________
____________	____________	____________

21. DO YOU FEEL THAT THERE IS AN IMMEDIATE FLOOD THREAT TO YOUR LAND, STRUCTURES, OR OTHER?

Yes ____, No ____

If yes what is the approximate value of the land, structure, etc, at risk?

____

22. HAVE YOU EVER SOUGHT OR TRIED TO SEEK ANY PROFESSIONAL OR TECHNICAL ADVICE ABOUT HOW TO DEAL WITH YOUR FLOODING PROBLEM?

Yes ____, No ____

If yes, when and from whom? ____________________

____

If no, are there any particular reasons why you haven't (eg. didn't know who, didn't think it would help, too expensive)? ____________________

____

23. PLEASE PROVIDE ANY ADDITIONAL INFORMATION OR COMMENTS WHICH YOU FEEL WILL HELP US ASSESS FLOODING PROBLEMS ____________________

____

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Leaflet from Public Meeting

November 2, 1989

Shoreline Management Plan
Kettle Creek Conservation Authority

The Kettle Creek Conservation Authority in cooperation with the Village of Port Stanley, Township of Southwold, Township of Yarmouth and the Ministry of Natural Resources, (MNR) is preparing a Shoreline Management Plan. This plan is being developed in accordance with Guidelines for Development of Great Lakes Shoreline Management Plans prepared by MNR in August 1987.

<u>Background</u>

The Guidelines set the following major goals of Shoreline Management Plans:

- To minimize danger to life and property damage from flooding, erosion and associated hazards along shorelines.

- To ensure that shoreline development adequately address flooding and erosion hazards through a combination of public and private management and development alternatives.

The key components of the Shoreline Management Plan are:

- prevention
- protection
- emergency response
- public information
- environment
- monitoring

The Conservation Authorities have been designated as the agencies responsible for the initiation and completion of these studies.

In June 1989, the Kettle Creek Conservation Authority retained Philpott Associates Coastal Engineers Limited to complete such a plan for the shoreline of Lake Erie within their watershed. The consultants have been working on the plan and have developed various concepts within the context of the Shoreline Management Plan. These concepts are being provided for your review and comments at this open house.

After the Consultants receive comments about the various concepts from shoreline owners, government agencies and other interested parties, they will be formulating a preferred plan.

We request that you review the summary presented in this hand-out, review the displays presented tonight and provide us with your comments. Since the completion of the study must be achieved within a set time table, we ask that you provide your comments on the attached sheet tonight. If you cannot return the attached sheet tonight, please mail it to the Consultant no later than November 8, 1989. This will ensure that your comments are taken into account.

Shoreline Management Plan

Prevention: The plan identifies the 100 year erosion limit. This limit is established on the basis of long term erosion data and stable slope allowance. The erosion line is plotted on aerial photography plans at a scale of 1:2000.

The erosion rate has been identified for all bluff sections with the exception of the east side of Port Stanley (Orchard Beach). In this area, existing protection is halting erosion above the water line.

The beach area of Port Stanley is not subject to permanent erosion so no erosion limit has been identified. However, 1:100 instantaneous water level and 1:100 wave uprush have been identified as 175.35 m (IGLD) and 176.6 m (IGLD) respectively.

Future development within the 100 year erosion limit and the 1:100 year uprush limit will be controlled through regulations of the Conservation Authority or the municipal planning process.

Protection: Various types of shoreline protection structures have been reviewed. Armour stone revetment structures have been identified as the most economical protection means. The cost of shore protection including provision of maintenance access road is estimated to be about $3,000/metre. This type of structure will provide protection from 1:100 year storms. The cost of bank stabilization, where required, will be in addition to this cost. Improvements to existing structures particularly in the Orchard Park area will be discussed on an individual basis.

We have completed a preliminary cost benefit analysis of shoreline protection work. With the exception of improvements in the area of Orchard Beach, the cost benefit analyses does not support the implementation of structural protection.

The Shoreline Management Plan plan concurs with the previously proposed location of a marina east of Port Stanley Harbour. This location has been identified as having the least impact on littoral transport and would provide some shoreline protection to the Orchard Beach area. Social, environmental and water quality issues must be addressed in detail prior to such a plan being developed.

Emergency Response: The Village of Port Stanley has an existing contingency program. The Shoreline Management Plan recommends that the Conservation Authority provide warning of surges and storms to the Municipality.

Public Information: The Shoreline Management Plan will recommend that annual information bulletins be issued by the Conservation Authority. More detailed involvement, such as public meetings, will be required during high lake level periods.

Environment: The concepts proposed within the Shoreline Management Plan have been reviewed with respect to impact on the natural environment. Preliminary assessment finds that potential impacts can be mitigated. A detailed terrestrial and aquatic review on a project by project basis will be recommended.

Monitoring: The Shoreline Management Plan will make specific recommendations about monitoring erosion and coastal processes. The exact recommendations cannot be made until a preferred plan is developed.

Questions and Comments:

1. Are you a shoreline property owner? Please indicate your name and location:

2. Your Comments:

Please return to: Philpott Associates Coastal Engineers Limited
111 Merton Street, Suite 202
Toronto, Ontario M4S 3A7

no later than November 8, 1989 to ensure that your concerns are taken into account.

4.0 STRATIGRAPHY AND SHORELINE CHARACTERISTICS

In this section, the stratigraphy of the bluffs is described to complement information shown on the stratigraphic cross sections (Figures 2.1 to 2.9 inclusive). The descriptions include much-summarized, diverse observations made during the course of field reconnaissance mapping and sampling. The time dependency and hence the limited significance of these observations, in particular concerning the beach width, the slope angle and the groundwater seepage, must be strongly emphasized.

4.5 Southwold Township (Figure 2.5)

The shoreline of Southwold Township extends from km 55.1 to km 64.75 of the study area. The bluff height is fairly uniform, varying from 35 to 37 m, except for a one-km stretch of protected bluffs immediately west of the Kettle Creek, which has the average height of 28 m.

From km 55.1 to about km 58 (Stations SO-7*, SO-6 and SO-5), the slope angle near the toe of the bluffs is typically close to 90°; further upslope, the bluffs are less steep and they are partially vegetated. Block falls, as a result of toe undercutting and shallow sloughing, are characteristic modes of bluff retreat. From km 58 to km 61.7 (Stations SO-4, SO-3 and SO-2), the mode of bluff retreat is characterized by rotational slumping and the formation of offshore debris fans. From km 61.7 to km 64.75 (Station SO-1), the toe of the bluffs is protected by a permanent sandy beach built up to the west of the breakwater at Port Stanley. Free degradation of the bluffs in this area results in lower overall slopes and the formation of a debris mantle, which covers the lower half of the slope.

The Port Stanley till occurs at the lake level along the entire stretch. Due to its low moisture content, the till is of hard to very hard consistency and it is frequently fissured. The texture of the till varies between silty clay and clayey silt. A continuous interlayer of lacustrine silt and clay occurs 10 to 15 m above the lake level. This unit is 3- to 4-m thick in the western half of the stretch. East of Station SO-3, the lacustrine unit becomes thicker and it is overlain by a stratum of fine sand. Strong groundwater discharge was observed at the interface between these two units. The presence of the fine sand stratum just below the mid height of the slope softens the underlying till, which may explain why rotational slumps occur in this area. The lacustrine sequence is overlain by an 8- to 12-m thick bed of the Port Stanley till. This bed is very similar in its texture and its overall appearance to the till that occurs at the toe of the bluffs. The upper portion of the bluffs is formed by a stratum of lacustrine silt and clay, up to 10 m thick, capped with an intermittent 1- to 2-m thick top stratum of rusty brown sand and silty sand.

* The site of Station SO-7, referred to as the Iona Study Site on Figure 2.5, has been subjected to extensive exploration by Quigley and his co-workers (Gelinas, 1974; Quigley, 1977; Quigley et al., 1977). Quigley et al. (1977) estimated that the cyclic pattern of bluff failure, erosion, steepening and new failure occurs at this location over a period of 10-25 years depending on the fluctuation of lake levels. Considerable steepening of a bluff profile was documented during rising lake levels.

The most prominent beach deposit within the stretch is the accretion sandy beach occurring west of the Port Stanley breakwater from km 61.7 to km 64.75. A well-developed, 100-m long gravelly beach was observed around the mouth of a gully at Station SO-5. At Station SO-4, a 50-m long gravelly beach formed on the western side of a large debris fan extending into the lake. General absence of beaches was observed in other areas.

4.6 Yarmouth Township (Figure 2.6)

The shoreline of Yarmouth Township extends from km 64.75 to km 79.25. The bluffs are about 40 m high, except for several short stretches where the bluffs are intersected by gullies and streams. The maximum height of 42 m is attained around the Elgin Area umping Station (km 68.75) and west of Barnum Gully (km 73.75). With the exception of sand dunes occurring in Houghton Township, these are the highest bluffs along the entire Lake Erie north shore.

Slope morphology is controlled to a large extent by stratigraphy of the bluffs. At the lake level the bluffs are almost vertical and directly undercut by waves. Frequent fissures in the till, block falls and surface sloughing were noted almost everywhere near the lake level. Further upslope, the bluffs have steep slopes (30^{o} to 45^{o}) in cohesive till strata while gentler slopes (30^{o} to 45^{o}) are formed in silts and fallen sand debris. Slopes are again almost vertical at the crest in a sand topstratum, which is usually not more than 1-metre thick. Huge rotational slumps are characteristic for the entire stretch, especially in areas with a thick mantle of pervious sediments in the upper portion of the bluffs. The crest of the bluffs has the undulated form of ribs and landslide scallops.

The oldest stratigraphic units are silts, clays and fine sands of lacustrine origin found at the lake level east of Port Stanley. This unit was sampled at Station YA-12 (km 66.0) and it is shown by Dreimanis (1966) to extend eastwards to about km 67.0. Bou (1975) described a 1.5-m thick discontinuous layer of dense fine sand occurring at the lake level at the site of the Elgin Area Pumping Station (km 68.9; Station YA-10), which probably represents an eastward continuation of this unit. This sand layer was not observed during the field visit to the site by the author in 1979.

scattered sand and gravel particles. Lacustrine units of sand, silt and clay occur as lenses or interlayers within the till as shown on Figure 2.6. Groundwater discharge typically occurs at the interface between the lacustrine units and the underlying, less pervious till stratum. The upper portion of the bluffs consists of a unit of lacustrine laminated sand to sandy silt capped by a topstratum of rusty brown medium sand. East of Port Stanley, (Station YA-12, Sample 12-2) the top layer in the bluffs has a character of sandy till and it probably correlates laterally with the upper stratum of the Port Stanley till.

Two sites within this stretch have been investigated in detail by other investigators. Bou (1975) and Quigley et al. (1977) studied high-velocity rotational slumps in the vicinity of the Elgin Area Pumping Station (km 68.9, Station YA-10). Processes of erosion at the University of Western Ontario Geography Station (km 76.75-km 77.4, Station YA-2) have been under study since 1965 (Zimmer, 1965; Quigley and Tutt, 1968; Gelinas, 1974; Packer, 1976; Quigley et al., 1977). Investigations at this site have documented the occurrence of huge deep-seated rotational slumps, which may develop during periods of low-water levels (Quigley et al., 1977).

The shore of Yarmouth Township is characterized by vigorous toe erosion by waves and the general absence of the beach. Short beaches, consisting of about 70 percent sand and 30 percent gravel, were noted at Station YA-11 (km 68.0), on the western side of the Pumping Station headland (km 68.9), around slumped debris fans at Station YA-8 (km 70.5), at Station YA-7 (km 71.5) and at Station YA-6 (km 72.2).

5.0 TEXTURAL COMPOSITION

For ease of reference, the results of particle size analyses for each township are summarized in Table 1 and the results of textural computations (cf. Section 3.3 above) are similarly summarized in Table 2. More detailed data pertaining to the size analyses and to the textural computations are tabulated in Appendices 1 and 2 respectively.

In Southwold Township (Figures 2.5, 3.3 and 3.4), the Port Stanley till contains, on the average, 5% sand, 46% silt and 49% clay. No gravel was detected in any of the samples collected. Two units of lacustrine silt and clay, which occur above and within the till, are very similar in texture to the till, except for coarser material found at Station SO-2 (Sample SO2–2) km 60.95). The unit of surficial sand does not contribute significantly to the overall textural composition. Silt and clay represent 94% of all shore materials (Table 2).

In Yarmouth Township (Figures 2.6 and 3.4), the Port Stanley till occupies about two thirds of the bluffs. On the average, it contains 10% sand and gravel, 35% silt and 55% clay (Table 1). The unit of lacustrine sand, silt and clay above the till, in which sand-sized and silt-sized material predominates, increases significantly the percentage of granular material available in the bluffs, which amounts to about 24% of all shore materials (Table 2). The unit of sandy till, occurring in the upper portion of the bluffs east of Port Stanley, has a highly variable texture characterized by the predominance of sand-sized and clay-sized material. The total unit volumes determined for this township are, on the average, higher than anywhere else within the study area due to the extreme height of the bluffs (Table 2).

TABLE 1 AVERAGE VALUES OF PARTICLE SIZE ANALYSES

	No. of Samples	Stratigraphic Unit	Gravel %	Sand %	Silt %	Clay %
5.		SOUTHWOLD TOWNSHIP				
Mean	8	Port Stanley	0.00	5.42	46.03	48.80
St. Deviation		Till		5.73	6.64	5.49
Mean	10	Lacustrine Silt	0.00	9.29	48.39	42.33
St. Deviation		and Clay		11.82	20.29	20.28
	1	Sand	0.00	86.48	5.92	7.60
6.		YARMOUTH TOWNSHIP				
Mean	22	Port Stanley	0.28	9.52	35.58	54.64
St. Deviation		Till	0.60	6.61	12.37	10.28
Mean	4	Sandy	8.83	41.46	18.11	31.60
St. Deviation		Till	14.47	16.50	11.24	10.89
Mean	21	Lacustrine	0.26	41.28	38.00	20.46
St. Deviation		Sand, Silt	0.82	29.88	24.57	23.90
		and Clay				
Mean	21	Sand	0.03	90.44	6.37	3.16
St. Deviation			0.12	5.17	3.53	2.99

TABLE 2 AVERAGE TEXTURAL COMPOSITION OF SHORE IN EACH TOWNSHIP OF THE STUDY AREA

No. of 250-m Segments*	Average Area in One Segment				
	Total m^2	Gravel m^2	Sand m^2	Silt m^2	Clay m^2
5.	SOUTHWOLD TOWNSHIP				
31	8537.78 (100%)	0 (0%)	491.74 (5.76%)	4257.97 (49.87%)	3788.07 (44.37%)
6.	YARMOUTH TOWNSHIP				
58	9335.43 (100%)	41.26 (0.44%)	2216.90 (23.75%)	3418.74 (36.62%)	3658.53 (39.19%)

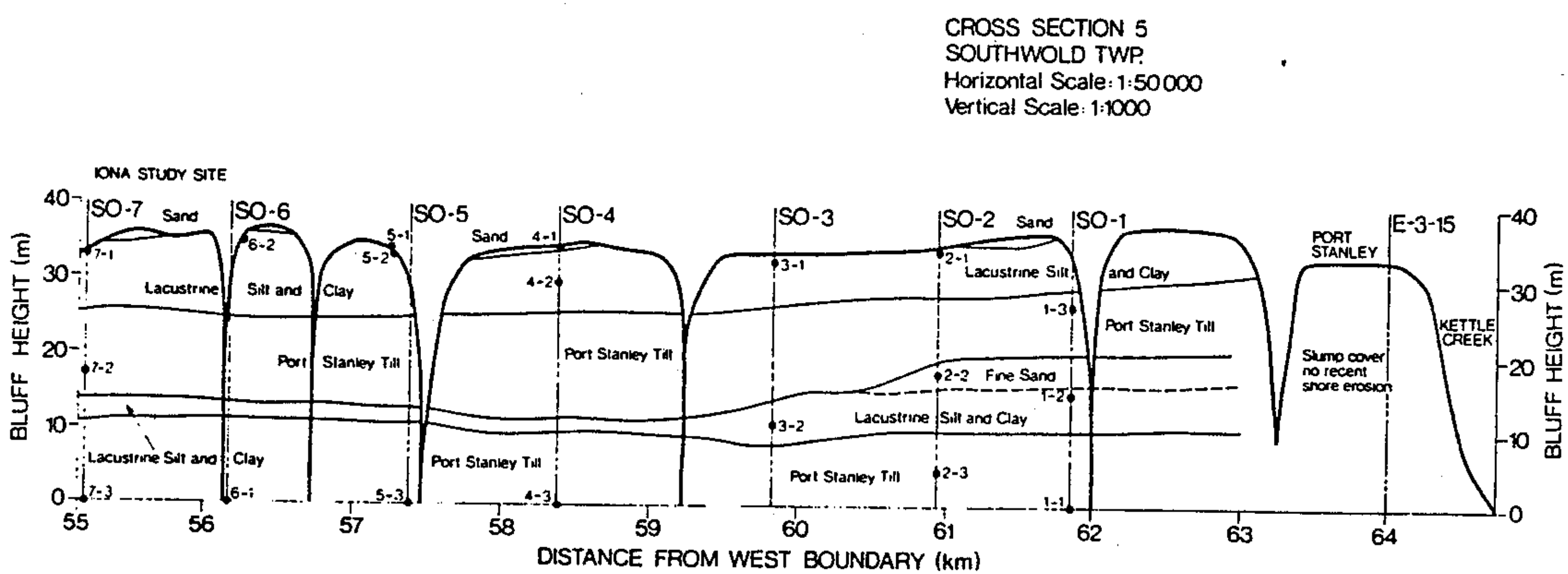

Fig. 2.5 Stratigraphic Cross Section 5, Southwold Township

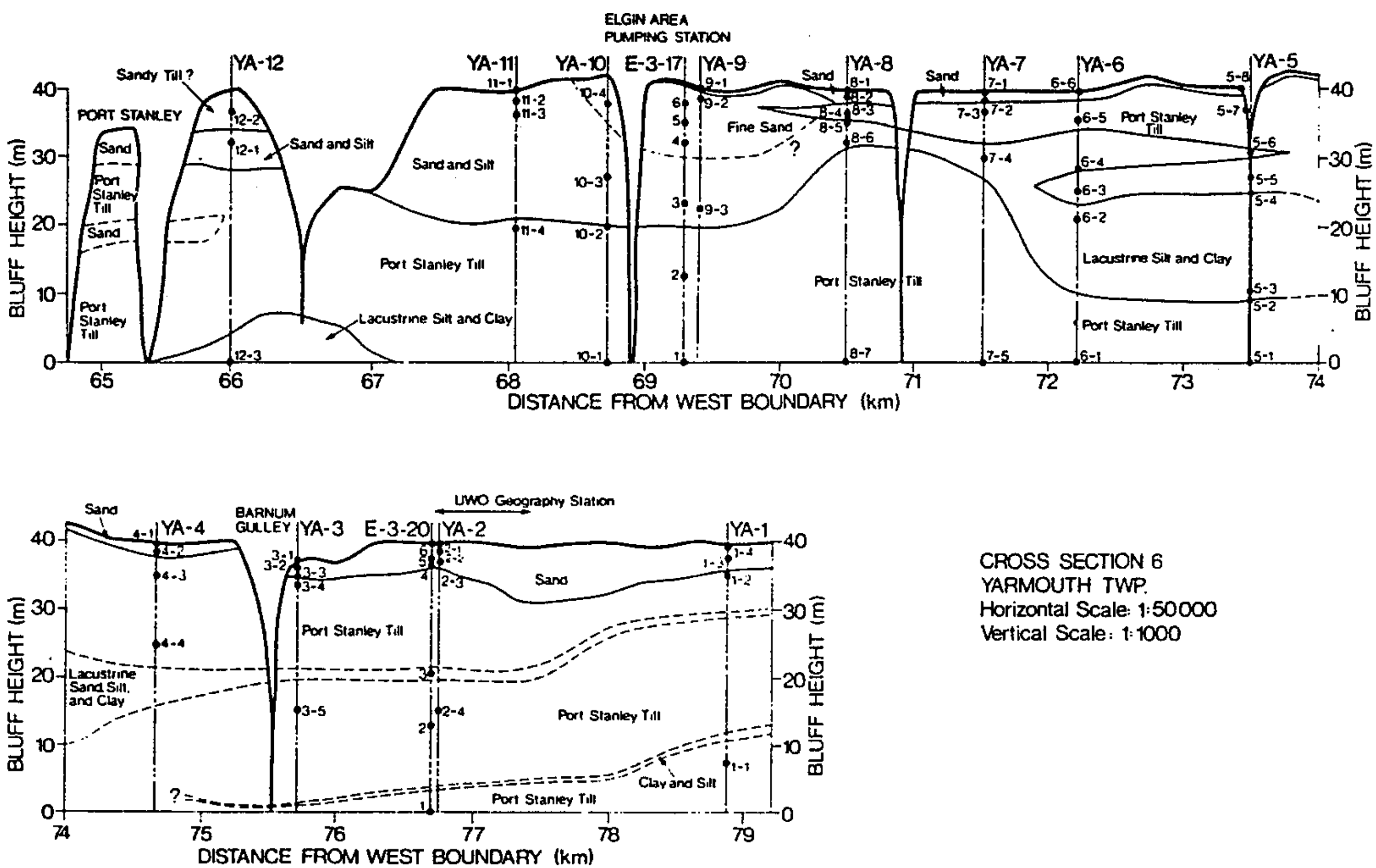

Fig. 2.6 Stratigraphic Cross Section 6, Yarmouth Township

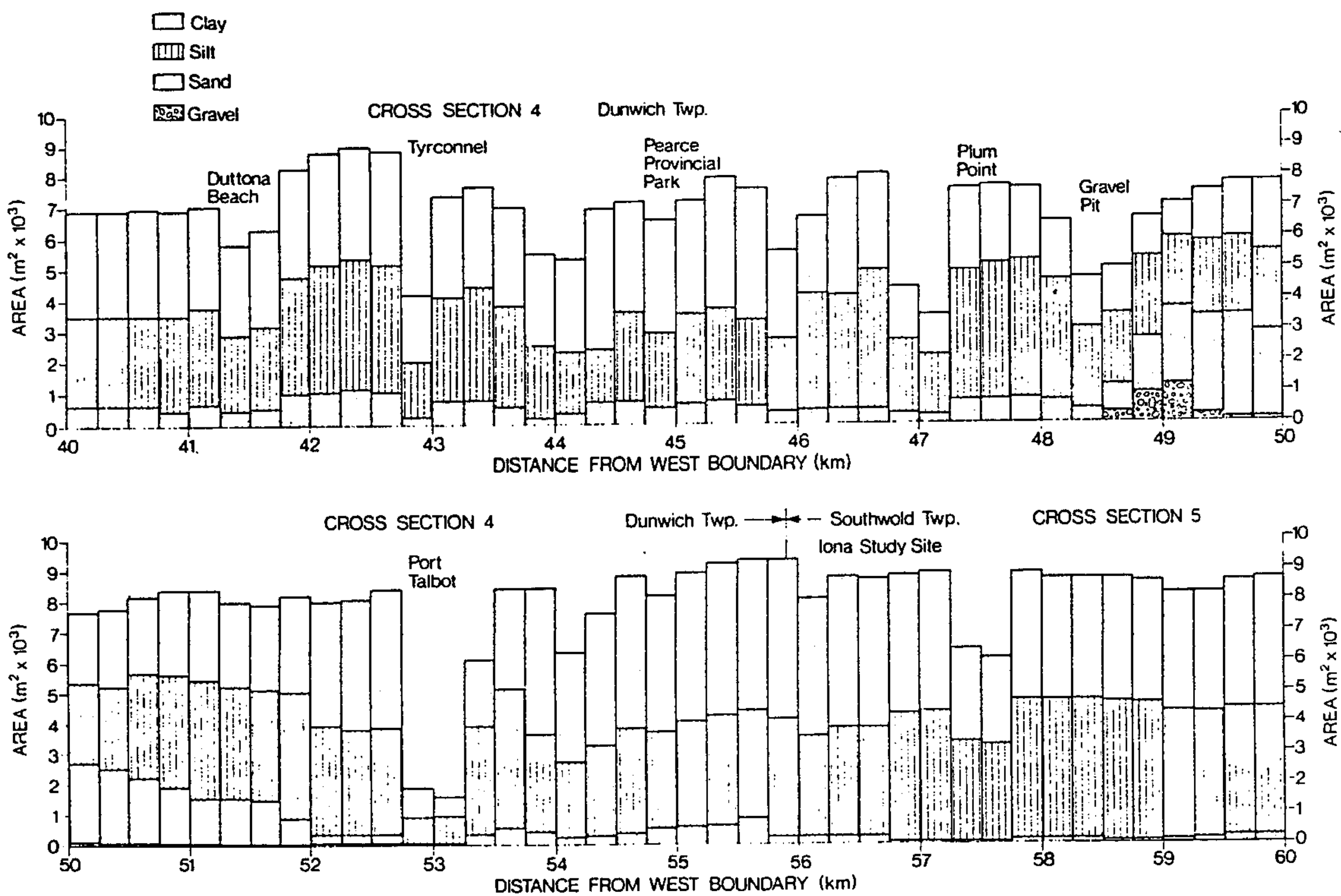

Fig. 3.3 Plot of Textural Composition, km 40.0-60.0

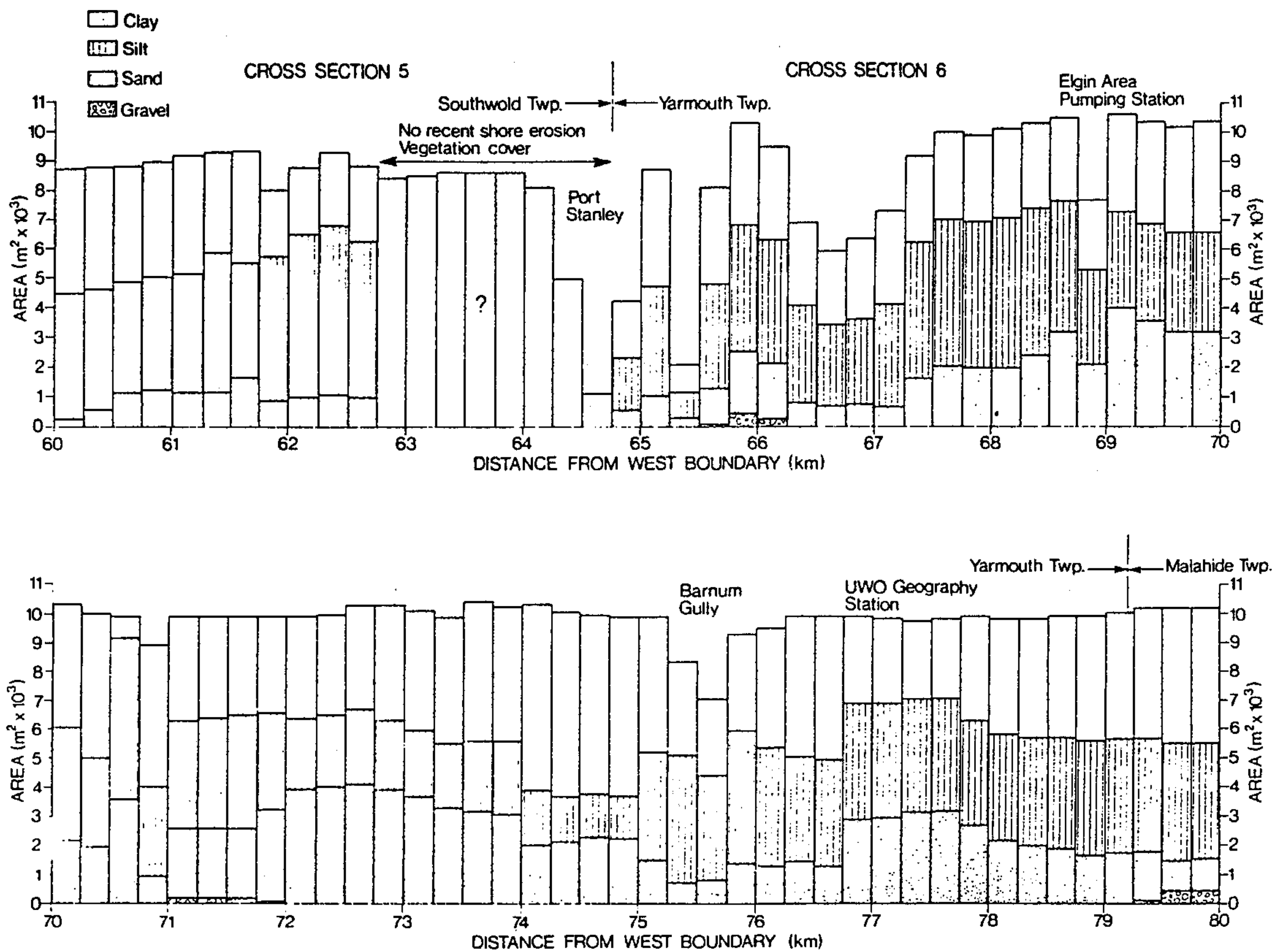

Fig. 3.4 Plot of Textural Composition, km 60.0-80.0

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Appendix G Bluff Recession Data

The following bluff recession data has been extracted from Fleming (1983a). The base chainage, column 1 of the following data, may be aproximately related to the baseline chainage used in this study by subtracting 64.40 from the base chainages in column 1. The exact relationship between the two baseline chainages is shown on the existing conditions photomosaics in Appendix A.

TABLE A.1 - RECESSION DATA 1896 TO 1975(continued)

BASE CHAINAGE COL(1)	STATION NO. COL(2)	YEAR COL(3)	REC(m.) 1896-(3) COL(4)	YEAR COL(5)	REC(m.) (3)-(5) COL(6)	ANNUAL RECESSION 1896-(3) COL(7)	ANNUAL RECESSION (3)-(5) COL(8)	ROLLING MEAN(51pt) (7) COL(9)	ROLLING MEAN(51pt) (8) COL(10)	RATIO (10)/(9) COL(11)	LOCATIONS
53.404	550	1936	58	1968	34	1.45	1.06	1.04	0.83	0.80	DUNWICH TWP
53.488	551	1936	54	1968	44	1.35	1.38	1.07	0.85	0.79	
53.572	552	1936	46	1968	42	1.15	1.31	1.10	0.86	0.79	
53.656	553	1936	44	1968	48	1.10	1.50	1.12	0.88	0.78	
53.739	554	1936	42	1968	47	1.05	1.47	1.14	0.88	0.77	
53.823	555	1936	38	1968	49	0.95	1.53	1.16	0.89	0.77	
53.907	556	1936	41	1968	48	1.02	1.50	1.18	0.91	0.77	
53.991	557	1936	43	1968	38	1.07	1.19	1.20	0.93	0.77	
54.074	558	1936	61	1968	40	1.52	1.25	1.22	0.94	0.77	
54.158	559	1936	64	1968	39	1.60	1.22	1.24	0.96	0.78	
54.242	560	1936	50	1968	43	1.25	1.34	1.26	0.98	0.78	
54.326	561	1936	56	1968	41	1.40	1.28	1.26	1.01	0.80	
54.422	562	1936	70	1968	-46	1.75	-1.44	1.27	1.04	0.82	
54.518	563	1936	86	1968	36	2.05	1.20	1.28	1.05	0.82	
54.614	564	1936	74	1968	42	1.85	1.31	1.29	1.06	0.82	
54.710	565	1936	54	1968	48	1.35	1.50	1.28	1.08	0.84	
54.806	566	1936	44	1968	45	1.10	1.41	1.30	1.10	0.84	
54.902	567	1936	37	1968	41	0.92	1.28	1.32	1.12	0.85	
54.998	568	1936	55	1968	26	1.37	0.81	1.35	1.13	0.83	
55.094	569	1936	55	1968	35	1.37	1.09	1.37	1.15	0.84	
55.190	570	1937	56	1968	38	1.37	1.23	1.39	1.16	0.83	- DUNWICH/SOUTHWOLD
55.286	571	1937	63	1968	37	1.54	1.19	1.40	1.19	0.85	TWP LIN
55.383	572	1937	66	1968	37	1.61	1.19	1.41	1.21	0.85	
55.479	573	1937	72	1968	36	1.76	1.16	1.42	1.23	0.86	
55.575	574	1937	71	1968	45	1.73	1.45	1.42	1.24	0.87	
55.671	575	1937	73	1968	44	1.78	1.42	1.43	1.24	0.87	
55.767	576	1937	73	1968	39	1.78	1.26	1.44	1.24	0.86	
55.863	577	1937	73	1968	41	1.78	1.32	1.45	1.25	0.86	
55.959	578	1937	63	1968	39	1.54	1.26	1.46	1.25	0.86	
56.055	579	1937	68	1968	20	1.66	0.65	1.47	1.24	0.85	
56.151	580	1937	53	1968	34	1.29	1.10	1.47	1.24	0.84	
56.247	581	1937	54@	1968	35@	1.32	1.15	1.48	1.23	0.83	
56.344	582	1937	55@	1968	32@	1.36	1.21	1.49	1.22	0.82	
56.441	583	1937	57	1968	39	1.39	1.26	1.50	1.21	0.81	
56.538	584	1937	46	1968	35	1.12	1.13	1.50	1.20	0.80	
56.635	585	1937	60	1968	24	1.46	0.77	1.49	1.20	0.80	
56.732	586	1937	43	1968	54	1.05	1.74	1.50	1.21	0.81	
56.829	587	1937	47	1968	46	1.15	1.48	1.50	1.21	0.81	
56.926	588	1937	58	1968	32	1.41	1.03	1.49	1.26	0.84	
57.023	589	1937	53	1968	42	1.29	1.35	1.49	1.25	0.84	
57.120	590	1937	59	1968	43	1.44	1.39	1.48	1.24	0.83	
57.217	591	1937	56	1968	52	1.37	1.68	1.48	1.23	0.83	
57.314	592	1937	59	1968	52	1.44	1.68	1.48	1.23	0.83	
57.412	593	1937	69	1968	34	1.68	1.10	1.49	1.24	0.83	
57.509	594	1937	50	1968	52	1.22	1.68	1.51	1.24	0.83	
57.606	595	1937	64	1968	42	1.56	1.35	1.52	1.25	0.82	
57.703	596	1937	57	1968	43	1.39	1.39	1.51	1.25	0.82	
57.800	597	1937	70	1968	49	1.71	1.58	1.51	1.25	0.82	
57.897	598	1937	67	1968	48	1.63	1.55	1.51	1.25	0.83	
57.994	599	1937	68	1968	40	1.66	1.29	1.51	1.26	0.83	
58.091	600	1937	77	1968	36	1.88	1.16	1.51	1.25	0.83	

NOTE: **** indicates where no data available , #### indicates where rolling mean<0.2m/year , ++++ indicates fillet beache

@ indicates interpolated data

TABLE A.1 - RECESSION DATA 1896 TO 1975(continued)

BASE CHAINAGE COL(1)	STATION NO. COL(2)	YEAR COL(3)	REC(m.) 1896-(3) COL(4)	YEAR COL(5)	REC(m.) (3)-(5) COL(6)	ANNUAL RECESSION 1896-(3) COL(7)	ANNUAL RECESSION (3)-(5) COL(8)	ROLLING MEAN(51pt) (7) COL(9)	ROLLING MEAN(51pt) (8) COL(10)	RATIO (10)/(9) COL(11)	LOCATIONS
58.188	601	1937	78	1968	36	1.90	1.16	1.51	1.25	0.83	SOUTHWOLD TWP
58.289	602	1937	65	1968	45	1.59	1.45	1.50	1.25	0.84	
58.390	603	1937	73	1968	43	1.78	1.39	1.49	1.25	0.84	
58.490	604	1937	65	1968	41	1.59	1.32	1.48	1.26	0.85	
58.591	605	1937	52	1968	32	1.27	1.03	1.48	1.27	0.86	
58.692	606	1937	59	1968	33	1.44	1.06	1.48	1.28	0.86	
58.793	607	1937	60	1968	32	1.46	1.03	1.48	1.28	0.86	
58.894	608	1937	65	1968	27	1.59	0.87	1.49	1.28	0.86	
58.994	609	1937	53	1968	24	1.29	0.77	1.49	1.28	0.86	
59.095	610	1937	58	1968	34	1.41	1.10	1.50	1.28	0.86	
59.196	611	1937	59	1968	51	1.44	1.65	1.50	1.30	0.87	
59.297	612	1937	64	1968	44	1.56	1.42	1.50	1.29	0.86	
59.398	613	1937	62	1968	32	1.51	1.03	1.50	1.30	0.86	
59.498	614	1937	65	1968	27	1.59	0.87	1.50	1.32	0.88	
59.599	615	1937	68	1968	20	1.66	0.65	1.49	1.34	0.90	
59.700	616	1937	53	1968	29	1.29	0.94	1.49	1.35	0.91	
59.801	617	1937	45	1968	49	1.10	1.58	1.47	1.35	0.92	
59.902	618	1937	64	1968	54	1.56	1.74	1.44	1.34	0.93	
60.002	619	1937	82	1968	35	2.00	1.13	1.44	1.34	0.93	
60.103	620	1937	77	1968	39	1.88	1.26	1.44	1.34	0.93	
60.204	621	1937	55	1968	38	1.34	1.23	1.43	1.33	0.93	
60.297	622	1937	63	1968	39	1.54	1.26	1.42	1.33	0.94	
60.390	623	1937	60	1968	45	1.46	1.45	1.40	1.33	0.95	
60.484	624	1937	65	1968	37	1.59	1.19	1.38	1.33	0.96	
60.577	625	1937	68	1968	38	1.66	1.23	1.39	1.35	0.97	
60.670	626	1937	69	1968	40	1.68	1.29	1.38	1.37	0.99	
60.763	627	1937	56	1968	47	1.37	1.52	1.38	1.40	1.01	
60.857	628	1937	52	1968	41	1.27	1.32	1.37	1.39	1.01	
60.950	629	1937	52	1968	47	1.27	1.52	1.36	1.42	1.04	
61.043	630	1937	62	1968	42	1.51	1.35	1.35	1.46	1.08	
61.136	631	1937	57	1968	45	1.39	1.45	1.35	1.45	1.07	
61.230	632	1937	62	1968	36	1.51	1.16	1.30	1.47	1.13	
61.323	633	1937	69	1968	37	1.68	1.19	1.29	1.49	1.16	
61.416	634	1937	60	1968	41	1.46	1.32	1.26	1.51	1.20	
61.509	635	1937	55	1968	42	1.34	1.35	1.22	1.54	1.27	
61.603	636	1937	57	1968	43	1.39	1.39	1.20	1.56	1.29	
61.696	637	1937	52	1968	48	1.27	1.55	1.17	1.58	1.34	
61.789	638	1937	50	1968	53	1.22	1.71	1.12	1.62	1.45	
61.882	639	1937	46	1968	65	1.12	2.10	1.02	1.71	1.67	
61.976	640	1937	51	1968	75	1.24	2.42	0.93	1.80	1.94	
62.069	641	1937	42	1968	62	1.03	2.02	0.80	1.87	2.34	
62.169	642	1937	25	1968	50	0.61	1.61	0.55	1.61	2.95	
62.269	643	1937	0	1968	37	0.00	1.21	0.00	1.21	####	
62.370	644	1937	0	1968	25	++++	++++	++++	++++	++++	
62.470	645	1937	0	1968	12	++++	++++	++++	++++	++++	
62.570	646	1937	0	1968	0	++++	++++	++++	++++	++++	
62.670	647	1937	0	1968	0	++++	++++	++++	++++	++++	
62.770	648	1937	0	1968	0	++++	++++	++++	++++	++++	
62.871	649	1937	0	1968	0	++++	++++	++++	++++	++++	
62.971	650	1937	0	1968	0	++++	++++	++++	++++	++++	
63.071	651	1937	0	1968	0	++++	++++	++++	++++	++++	

NOTE: **** indicates where no data available , #### indicates where rolling mean<0.2m/year , ++++ indicates fillet beaches

TABLE A.1 - RECESSION DATA 1896 TO 1975(continued)

BASE CHAINAGE COL(1)	STATION NO. COL(2)	YEAR COL(3)	REC(m.) 1896-(3) COL(4)	YEAR COL(5)	REC(m.) (3)-(5) COL(6)	ANNUAL RECESSION 1896-(3) COL(7)	ANNUAL RECESSION (3)-(5) COL(8)	ROLLING MEAN(51pt) (7) COL(9)	ROLLING MEAN(51pt) (8) COL(10)	RATIO (10)/(9) COL(11)	LOCATIONS
63.171	652	1937	0	1968	0	++++	++++	++++	++++	++++	SOUTHWOLD TWP
63.271	653	1937	0	1968	0	++++	++++	++++	++++	++++	
63.372	654	1937	0	1968	0	++++	++++	++++	++++	++++	
63.472	655	1937	0	1968	0	++++	++++	++++	++++	++++	
63.572	656	1937	0	1968	0	++++	++++	++++	++++	++++	
63.672	657	1937	0	1968	0	++++	++++	++++	++++	++++	
63.772	658	1937	0	1968	0	++++	++++	++++	++++	++++	
63.873	659	1937	0	1968	0	++++	++++	++++	++++	++++	
63.973	660	1937	0	1968	0	++++	++++	++++	++++	++++	
64.073	661	1937	0	1968	0	++++	++++	++++	++++	++++	
64.167	662	1937	0	1968	0	++++	++++	++++	++++	++++	
64.262	663	1937	0	1968	0	++++	++++	++++	++++	++++	
64.356	664	1937	0	1968	0	++++	++++	++++	++++	++++	
64.451	665	1937	****	1968	****	****	****	****	****	****	
64.545	666	1937	****	1968	****	****	****	****	****	****	
64.640	667	1937	****	1968	****	****	****	****	****	****	- PORT STANLEY/KETTLE C
64.734	668	1937	****	1968	****	****	****	****	****	****	& SOUTHWOLD/YARMOUTH
64.828	669	1937	****	1968	****	****	****	****	****	****	TWP LINE
64.923	670	1937	****	1968	****	****	****	****	****	****	
65.017	671	1937	****	1968	****	****	****	****	****	****	
65.112	672	1937	****	1968	****	****	****	****	****	****	
65.206	673	1937	****	1968	****	****	****	****	****	****	
65.301	674	1937	****	1968	****	****	****	****	****	****	
65.395	675	1937	****	1968	****	****	****	****	****	****	
65.490	676	1937	****	1968	****	****	****	****	****	****	
65.584	677	1937	****	1968	****	****	****	****	****	****	
65.678	678	1937	****	1968	****	****	****	****	****	****	
65.773	679	1937	****	1968	****	****	****	****	****	****	
65.867	680	1937	****	1968	****	****	****	****	****	****	
65.962	681	1937	****	1968	****	****	****	****	****	****	
66.061	682	1937	****	1968	****	****	****	****	****	****	- ORCHARD BEACH LIMIT
66.160	683	1937	75	1968	130	1.83	4.19	1.83	4.19	2.29	
66.259	684	1937	87	1968	138	2.12	4.45	2.17	4.17	1.92	
66.358	685	1937	105	1968	120	2.56	3.87	2.27	4.14	1.83	
66.358	685	1937	105	1968	120	2.56	3.87	2.32	3.93	1.69	
66.457	686	1937	109	1968	126	2.66	4.06	2.41	3.64	1.51	
66.556	687	1937	89	1968	128	2.17	4.13	2.41	3.47	1.44	
66.655	688	1937	97	1968	111	2.37	3.58	2.35	3.40	1.45	
66.754	689	1937	105	1968	100	2.56	3.23	2.31	3.31	1.43	
66.853	690	1937	113	1968	87	2.76	2.81	2.29	3.22	1.41	
66.952	691	1937	110	1968	76	2.68	2.45	2.24	3.16	1.41	
67.051	692	1937	99	1968	87	2.41	2.81	2.23	3.06	1.37	
67.150	693	1937	98	1968	81	2.39	2.61	2.20	2.94	1.33	
67.249	694	1937	81	1968	95	1.98	3.06	2.18	2.86	1.31	
67.348	695	1937	83	1968	90	2.02	2.90	2.15	2.75	1.28	
67.447	696	1937	85	1968	85	2.07	2.74	2.12	2.68	1.26	
67.546	697	1937	85	1968	85	2.07	2.74	2.07	2.63	1.27	
67.645	698	1937	89	1968	78	2.17	2.52	2.01	2.57	1.28	
67.744	699	1937	84	1968	80	2.05	2.58	1.91	2.54	1.33	
67.843	700	1937	79	1968	80	1.93	2.58	1.85	2.46	1.33	
67.942	701	1937	76	1968	86	1.95	2.77	1.82	2.43	1.34	

NOTE: **** indicates where no data available , #### indicates where rolling mean<0.2m/year , ++++ indicates fillet beaches

TABLE A.1 - RECESSION DATA 1896 TO 1975(continued)

BASE CHAINAGE COL(1)	STATION NO. COL(2)	YEAR COL(3)	REC(m.) 1896-(3) COL(4)	YEAR COL(5)	REC(m.) (3)-(5) COL(6)	ANNUAL RECESSION 1896-(3) COL(7)	(3)-(5) COL(8)	ROLLING MEAN(5pt) (7) COL(9)	(8) COL(10)	RATIO (10)/(9) COL(11)	LOCATIONS
68.041	702	1937	84	1968	67	2.05	2.16	1.79	2.42	1.35	YARMOUTH TWP
68.140	703	1937	84	1968	61	2.05	1.97	1.79	2.41	1.35	
68.240	704	1937	78	1968	54	1.90	1.74	1.79	2.41	1.34	
68.339	705	1937	83	1968	49	2.02	1.58	1.79	2.40	1.31	
68.438	706	1937	76	1968	65	1.85	2.10	1.82	2.39	1.29	
68.537	707	1937	80	1968	59	1.95	1.90	1.85	2.38	1.28	
68.637	708	1937	78	1968	43	1.90	1.39	1.86	2.37	1.25	
68.736	709	1937	72	1968	43	1.76	1.39	1.87	2.33	1.23	
68.835	710	1937	68	1968	50	1.66	1.61	1.86	2.29	1.24	- ELGIN AREA PUMPING
68.934	711	1937	72	1968	55	1.76	1.77	1.83	2.26	1.24	STATION
69.034	712	1937	63	1968	61	1.54	1.97	1.81	2.24	1.23	
69.133	713	1937	46	1968	54	1.12	1.74	1.80	2.21	1.23	
69.232	714	1937	53	1968	52	1.29	1.68	1.79	2.20	1.21	
69.331	715	1937	28	1968	53	0.68	1.71	1.79	2.18	1.22	
69.431	716	1937	17	1968	66	0.41	2.13	1.79	2.17	1.23	
69.530	717	1937	12	1968	61	0.29	1.97	1.78	2.18	1.23	
69.629	718	1937	34	1968	31	0.83	1.00	1.77	2.18	1.23	
69.728	719	1937	26	1968	40	0.63	1.29	1.76	2.17	1.24	
69.828	720	1937	52	1968	50	1.27	1.61	1.74	2.15	1.23	
69.927	721	1937	50	1968	63	1.22	2.03	1.74	2.14	1.23	
70.027	722	1937	52	1968	68	1.27	2.19	1.73	2.12	1.22	
70.127	723	1937	57	1968	64	1.39	2.06	1.72	2.10	1.22	
70.228	724	1937	69	1968	75	1.68	2.42	1.71	2.08	1.22	
70.328	725	1937	65	1968	71	1.59	2.29	1.71	2.06	1.21	
70.428	726	1937	66	1968	68	1.61	2.19	1.70	2.04	1.20	
70.528	727	1937	84	1968	60	2.05	1.94	1.70	2.01	1.18	
70.628	728	1937	107	1968	66	2.61	2.13	1.68	1.99	1.18	
70.729	729	1937	111	1968	67	2.71	2.16	1.67	1.98	1.18	
70.829	730	1937	102	1968	65	2.49	2.10	1.67	1.96	1.17	
70.929	731	1937	98	1968	74	2.39	2.39	1.67	1.96	1.17	
71.029	732	1937	86	1968	67	2.10	2.16	1.66	1.96	1.18	
71.129	733	1937	92	1968	69	2.24	2.23	1.64	1.95	1.19	
71.229	734	1937	88	1968	62	2.15	2.00	1.63	1.95	1.20	
71.330	735	1937	78	1968	71	1.90	2.29	1.61	1.95	1.21	
71.430	736	1937	31	1968	80	0.76	2.58	1.60	1.95	1.22	
71.530	737	1937	63	1968	83	1.54	2.68	1.58	1.94	1.22	
71.630	738	1937	76	1968	85	1.85	2.74	1.58	1.93	1.22	
71.730	739	1937	75	1968	91	1.83	2.94	1.58	1.92	1.21	
71.831	740	1937	114	1968	70	2.78	2.26	1.58	1.91	1.22	
71.931	741	1937	99	1968	79	2.41	2.55	1.58	1.91	1.20	
72.031	742	1937	87	1968	89	2.12	2.87	1.60	1.89	1.18	
72.130	743	1937	81	1968	90	1.98	2.90	1.62	1.88	1.16	
72.230	744	1937	79	1975	75	1.93	1.97	1.64	1.89	1.15	
72.329	745	1937	51	1975	87	1.24	2.29	1.66	1.88	1.14	
72.429	746	1937	68	1975	79	1.66	2.08	1.66	1.88	1.13	
72.528	747	1937	63	1975	75	1.54	1.97	1.67	1.87	1.12	
72.628	748	1937	68	1975	61	1.66	1.61	1.67	1.85	1.11	
72.727	749	1937	65	1975	62	1.59	1.63	1.67	1.83	1.10	
72.827	750	1937	79	1975	59	1.93	1.55	1.66	1.81	1.09	
72.926	751	1937	76	1975	55	1.85	1.45	1.67	1.78	1.07	
73.026	752	1937	62	1975	44	1.51	1.16	1.67	1.76	1.05	

NOTE: **** indicates where no data available , #### indicates where rolling mean<0.2m/year , ++++ indicates fillet beaches

TABLE A.1 - RECESSION DATA 1896 TO 1975(continued)

BASE CHAINAGE COL(1)	STATION NO. COL(2)	YEAR COL(3)	REC(m.) 1896-(3) COL(4)	YEAR COL(5)	REC(m.) (3)-(5) COL(6)	ANNUAL RECESSION 1896-(3) COL(7)	ANNUAL RECESSION (3)-(5) COL(8)	ROLLING MEAN(51pt) (7) COL(9)	ROLLING MEAN(51pt) (8) COL(10)	RATIO (10)/(9) COL(11)	LOCATIONS
73.126	753	1937	57	1975	46	1.39	1.21	1.66	1.76	1.06	YARMOUTH TWP
73.225	754	1937	57	1975	48	1.39	1.26	1.63	1.75	1.08	
73.325	755	1937	84	1975	40	2.05	1.05	1.60	1.74	1.09	
73.424	756	1937	76	1975	51	1.85	1.34	1.58	1.73	1.10	
73.524	757	1937	52	1975	75	1.27	1.97	1.56	1.71	1.10	
73.623	758	1937	50	1975	62	1.22	1.63	1.53	1.70	1.11	
73.723	759	1937	39	1975	52	0.95	1.37	1.51	1.68	1.12	
73.822	760	1937	38	1975	59	0.93	1.55	1.49	1.67	1.12	
73.922	761	1937	39	1975	54	0.95	1.42	1.47	1.65	1.12	
74.022	762	1937	47	1975	42	1.15	1.11	1.47	1.63	1.11	
74.123	763	1937	51	1975	62	1.24	1.63	1.46	1.60	1.10	
74.223	764	1937	50	1975	39	1.22	1.03	1.44	1.58	1.10	
74.324	765	1937	43	1975	59	1.05	1.55	1.42	1.55	1.09	
74.424	766	1937	45	1975	53	1.10	1.39	1.39	1.53	1.10	
74.525	767	1937	58	1975	49	1.41	1.29	1.36	1.50	1.10	
74.625	768	1937	57	1975	47	1.39	1.24	1.34	1.46	1.09	
74.726	769	1937	62	1975	59	1.51	1.55	1.31	1.43	1.09	
74.826	770	1937	66	1975	42	1.61	1.11	1.30	1.42	1.10	
74.926	771	1937	66	1975	51	1.61	1.34	1.29	1.40	1.09	
75.027	772	1937	53	1975	59	1.29	1.55	1.27	1.39	1.09	
75.127	773	1937	53	1975	44	1.29	1.16	1.25	1.38	1.10	
75.228	774	1937	58	1975	45	1.41	1.18	1.24	1.37	1.11	
75.328	775	1937	62	1975	39	1.51	1.03	1.22	1.37	1.12	
75.429	776	1937	73	1975	41	1.78	1.08	1.20	1.36	1.14	
75.529	777	1937	78	1975	48	1.90	1.26	1.18	1.36	1.16	
75.630	778	1937	51	1975	75	1.24	1.97	1.16	1.37	1.18	- BARNUM GULLEY
75.730	779	1937	42	1975	63	1.02	1.66	1.14	1.38	1.21	
75.831	780	1937	59	1975	57	1.44	1.50	1.12	1.38	1.23	
75.931	781	1937	53	1975	63	1.29	1.65	1.10	1.38	1.26	
76.031	782	1937	53	1975	49	1.29	1.29	1.09	1.39	1.28	
76.130	783	1937	36	1975	60	0.88	1.58	1.08	1.39	1.28	
76.230	784	1937	42	1975	50	1.02	1.32	1.07	1.39	1.30	
76.330	785	1937	43	1975	49	1.05	1.29	1.07	1.40	1.31	
76.430	786	1937	37	1975	45	0.90	1.18	1.07	1.41	1.32	
76.529	787	1937	42	1975	58	1.02	1.53	1.07	1.42	1.33	
76.629	788	1937	38	1975	60	0.93	1.58	1.06	1.44	1.35	
76.729	789	1937	38	1975	57	0.93	1.50	1.06	1.44	1.37	
76.829	790	1937	40	1975	50	0.98	1.32	1.05	1.46	1.38	
76.928	791	1937	38	1975	43	0.93	1.13	1.06	1.47	1.39	
77.028	792	1937	39	1975	43	0.95	1.13	1.06	1.47	1.40	
77.128	793	1937	41	1975	42	1.00	1.11	1.05	1.49	1.41	
77.228	794	1937	32	1975	51	0.78	1.34	1.05	1.50	1.43	
77.327	795	1937	39	1975	47	0.95	1.24	1.04	1.50	1.44	
77.427	796	1937	31	1975	55	0.76	1.45	1.02	1.51	1.48	
77.527	797	1937	35	1975	54	0.85	1.42	1.00	1.52	1.52	
77.627	798	1937	32	1975	52	0.78	1.37	0.98	1.53	1.56	
77.726	799	1937	31	1975	54	0.76	1.42	0.97	1.54	1.59	
77.826	800	1937	33	1975	52	0.80	1.37	0.94	1.54	1.64	
77.926	801	1937	32	1975	49	0.78	1.29	0.91	1.53	1.68	
78.027	802	1937	30	1975	54	0.73	1.42	0.89	1.51	1.68	
78.128	803	1937	29	1975	63	0.71	1.66	0.88	1.50	1.72	

NOTE: **** indicates where no data available , #### indicates where rolling mean<0.2m/year , ++++ indicates fillet beaches

TABLE A.1 - RECESSION DATA 1896 TO 1975(continued)

BASE CHAINAGE COL(1)	STATION NO. COL(2)	YEAR COL(3)	REC(m.) 1896-(3) COL(4)	YEAR COL(5)	REC(m.) (3)-(5) COL(6)	ANNUAL RECESSION 1896-(3) COL(7)	ANNUAL RECESSION (3)-(5) COL(8)	ROLLING MEAN(5pt) (7) COL(9)	ROLLING MEAN(5pt) (8) COL(10)	RATIO (10)/(9) COL(11)	LOCATIONS
78.229	804	1937	15	1975	65	0.37	1.71	0.87	1.50	1.72	YARMOUTH TWP
78.330	805	1937	22	1975	47	0.54	1.24	0.87	1.50	1.73	
78.430	806	1937	39	1975	44	0.95	1.16	0.86	1.50	1.75	
78.531	807	1937	39	1975	61	0.95	1.61	0.85	1.51	1.77	
78.632	808	1937	40	1975	66	0.98	1.74	0.85	1.51	1.78	
78.733	809	1937	36	1975	73	0.88	1.92	0.85	1.52	1.80	
78.834	810	1937	31	1975	75	0.76	1.97	0.84	1.54	1.82	
78.935	811	1937	33	1975	77	0.80	2.03	0.85	1.56	1.84	
79.036	812	1937	43	1975	72	1.05	1.89	0.85	1.57	1.84	
79.137	813	1937	38	1975	74	0.93	1.95	0.86	1.59	1.84	
79.238	814	1937	34	1975	71	0.83	1.87	0.90	1.60	1.79	- YARMOUTH/MALAHIDE
79.338	815	1936	44	1975	72	1.10	1.85	0.91	1.62	1.77	TWP LINE
79.439	816	1936	45	1975	74	1.12	1.90	0.91	1.61	1.78	
79.540	817	1936	45	1975	72	1.12	1.85	0.92	1.65	1.80	
79.641	818	1936	49	1975	74	1.22	1.90	0.92	1.65	1.80	
79.742	819	1936	48	1975	71	1.20	1.82	0.93	1.62	1.74	
79.843	820	1936	42	1975	64	1.05	1.64	0.88	1.57	1.79	
79.944	821	1936	33	1975	69	0.82	1.77	0.76	1.52	2.00	
80.036	822	1936	22	1975	70	0.55	1.79	0.51	1.45	2.83	
80.127	823	1936	7	1975	71	0.17	1.82	0.17	1.36	####	
80.219	824	1936	0	1975	66	++++	1.69	++++	1.25	++++	
80.311	825	1936	0	1975	44	++++	1.13	++++	1.04	++++	
80.402	826	1936	0	1975	22	++++	0.56	++++	0.56	++++	
80.494	827	1936	0	1975	0	++++	+++	++++	0.00	++++	
80.586	828	1936	0	1975	0	++++	++++	++++	++++	++++	
80.678	829	1936	0	1975	0	++++	++++	++++	++++	++++	
80.769	830	1936	0	1975	0	++++	++++	++++	++++	++++	
80.861	831	1936	0	1975	0	++++	++++	++++	++++	++++	
80.953	832	1936	0	1975	0	++++	++++	++++	++++	++++	
81.044	833	1936	0	1975	0	++++	++++	++++	++++	++++	
81.136	834	1936	0	1975	0	++++	++++	++++	++++	++++	
81.228	835	1936	0	1975	0	++++	++++	++++	++++	++++	
81.319	836	1936	0	1975	0	++++	++++	++++	++++	++++	
81.411	837	1936	0	1975	0	++++	++++	++++	++++	++++	
81.503	838	1936	0	1975	0	++++	++++	++++	++++	++++	
81.594	839	1936	0	1975	0	0.00	0.00	0.00	0.00	####	- PORT BRUCE &
81.686	840	1936	14	1975	25	0.35	0.64	0.22	0.39	1.76	CATFISH CREEK
81.778	841	1936	13	1975	21	0.32	0.54	0.47	0.52	1.12	
81.872	842	1936	29	1975	27	0.72	0.69	0.59	0.70	1.18	
81.965	843	1936	38	1975	29	0.95	0.74	0.68	0.83	1.21	
82.059	844	1936	29	1975	41	0.72	1.05	0.69	0.97	1.41	
82.152	845	1936	42	1975	47	1.05	1.21	0.74	1.14	1.55	
82.246	846	1936	48	1975	45	1.20	1.15	0.80	1.27	1.58	
82.340	847	1936	34	1975	57	0.85	1.46	0.88	1.32	1.51	
82.433	848	1936	32	1975	57	0.80	1.46	0.94	1.37	1.46	
82.527	849	1936	25	1975	67	0.62	1.72	0.99	1.40	1.42	
82.620	850	1936	45	1975	76	1.12	1.95	1.02	1.43	1.39	
82.714	851	1936	36	1975	88	0.90	2.26	1.06	1.46	1.37	
82.808	852	1936	50	1975	82	1.25	2.10	1.10	1.47	1.34	
82.901	853	1936	49	1975	80	1.22	2.05	1.10	1.50	1.36	
82.995	854	1936	56	1975	64	1.40	1.64	1.10	1.52	1.38	

NOTE: **** indicates where no data available , #### indicates where rolling mean<0.2m/year , ++++ indicates fillet beaches

Appendix H Official Plan and Zoning Regulations

The Township of Yarmouth and Township of Southwold have separate Official Plans but the sections dealing with the Lake Erie shoreline are identical. Both Townships have defined the shoreline as Lake Erie Hazard Prone Area. The explicit policies dealing with this area are found in Section 9 of the Official Plans and have been extracted and presented following:

9.4 Lake Erie Shoreline

9.4.1 The areas along the Lake Erie shoreline , which are subject to flooding and erosion as a result of wind and wave action are considered as Hazard Prone Areas. The areas susceptible to flood and erosion are delineated on Schedule "A" as the Lake Erie Hazard Prone Area. This designation has been established on the basis of protection from a 1 in 100 year probability flood calculated so as to include possible wave uprush and a 100 year erosion limit based on degree of slope, the eroding agent and the type of material involved. The setback limit established by the Lake Erie Hazard Prone Designation may be increased subject to Municipal Council and Conservation Authority requirements, if erosion conditions warrant said increase.

9.4.2 The Lake Erie Hazard Prone Area is not a specific land use category, but shall be interpreted as a performance category in which the policies of this section and section 9.3 are to apply in conjunction with the policies of the underlying land use designation.

9.4.3 Notwithstanding other provisions of this Plan there shall be as a general principle no structural development or major landscape alterations within this designation, with the exception of flood and erosion control structures and devices.

Subsection 9.2 provides for general goals for hazard prone areas as follows:

- To delineate land having inherent physical hazards such as flood susceptibility, erosion, steep slopes or other physical conditions which act as a constraint to development.

- To encourage land owners to retain and maintain the natural environment.

- To encourage open air, recreational use of Hazard prone areas consistent with their environmental limitations and the need to conserve significant areas of natural vegetation.

- To reduce the potential for hazard to life and property and social disruption from the physical hazards of flooding and erosion.

Subsection 9.3, Hazard Prone Area, provides some indirect policies in that the shoreline is included within this category. They are as follows:

9.3.4 Where development or construction is proposed on lands within the Hazard Prone Area, the limits of this designation shall be determined by site inspection and discussion with the Conservation Authority having jurisdiction.

9.3.5 Within the Hazard Prone Areas no buildings or structures or additions thereto shall be permitted, with the exception of buildings or structures required for flood control, erosion control, or other conservation purposes, without the approval of the Municipality, the Conservation Authority having jurisdiction over the area. Such approval shall take into consideration:

- the existing or potential physical hazards;

- the potential impact of these hazards on the proposed building or structure; and on adjacent properties;

- the proposed methods by which the impact will be overcome in a manner consistent with accepted engineering techniques and sound resource management practices;

- adequate building setbacks in relation to the kind, extent and severity of both the existing or potential hazards;

- the costs and benefits in monetary, social and environmental value in terms of any engineering works or resource management practices needed to overcome the impact of the existing or potential hazard.

9.3.8 The Township Zoning Bylaw shall contain general provisions with respect to minimum setback distances for buildings and structures from the top of bank of watercourses within the Hazard Prone Area. If after a review of specific site conditions the Conservation Authority having jurisdiction determines that the established setback distances can be altered then the Committee of Adjustment may consider the granting of a minor variance.

9.3.9 No placing or removal of any fill whether originating on the site or elsewhere, shall be permitted within the Hazard Prone Area except in accordance with the regulations of the Conservation Authority having jurisdiction in the area.

The existing Official Plan for the Village of Port Stanley was also reviewed. The Village of Port Stanley shoreline is included in the Hazard Land classification and falls within the Lake Erie Erosion Area. Policies directly pertaining to this area are as follows:

Section 3.7.1. Floodplain Policies

Port Stanley has a major portion of its urban area within the Hazard Land classification, namely the Kettle Creek floodplain or the Lake Erie Erosion Area. Hence, the following special policy considerations will apply to the Village of Port Stanley.

- new development which will initiate or increase existing erosion rates of the floodplain, valley walls and Lake Erie Erosion Area, will not be permitted.
- The Village Council will review within the Kettle Creek Conservation Authority, all such applications for new development which may aggravate these areas before the issuance of a building permit.
- ... Within the Lake Erie Erosion Area, the advice and guidance of the Ministry of Natural Resources will be secured before Council approves any shoreline related development.
- no new development including buildings and other structures will be permitted on the toe-of-slope and side-slopes adjacent to the beach area, and in any other locations where the toe-of-slope and side slopes are unstable and susceptible to erosion. Slope stabilization measures such as reforestation shall be undertaken.

Indirect policies applying to the Hazards Lands category include:

> The Hazard Land Designation shown on Schedule "A" includes all lands having inherent environmental hazards, such as organic soils, flood susceptibility, erosion, steep slopes, or any other physical condition which makes the land unsuitable for development or building purposes. Lands so designated are intended primarily for preservation and conservation of the natural land and/or environment, and are to be managed in such a fashion as to complement adjacent land uses and to protect such uses from any physical hazards or their effects.
>
> The uses permitted in the Hazard areas shall be limited to agriculture, conservation, horticultural nurseries, forestry, wildlife areas, public or private parks, golf courses and other recreational activities.
>
> Where land designated as Hazard Land is under private ownership, this Plan does not indicate that the land will necessarily remain as such indefinitely, nor shall it be construed as implying that such areas are free and open to the general public or will be purchased by the Municipality or other public agency. An application for the redesignation of Hazard Lands for other purposes may be given due consideration by the Municipality after taking into account:
>
> - the existing environmental and/or physical hazards;
> - the potential impacts of these hazards; and
> - the proposed methods by which these impacts may be overcome in a manner consistent with accepted engineering techniques and renewable resource management practices.
> - the costs and benefits in monetary, social and biological value terms of any engineering works and/or resource management practices needed to overcome these impacts.

Also contained in the Official Plan for Port Stanley (but not Southwold and Yarmouth) are policies covering future development. Subsection 3.5.2.1 (Public Space) Main Beach provides that:

> Public ownership of lands, abutting Lake Erie, west of the lighthouse pier in the Village of Port Stanley for recreational purposes, should be the long range goal of government.

The Kettle Creek Conservation Authority owns approximately 244 metres of water frontage west of Lake Street.

Village Council, with assistance from senior levels of government, should attempt to acquire for pubic open space purposes, the Lake Erie frontage bounded on the west by Lake Street, on the north by the toe-of-slope of a hill approximately 30 metres in height, and to the east by the west limit of lands known locally as the Stork Club Property.

From the Stork Club Property easterly to the lighthouse pier, commercial development, oriented towards recreational and service needs of the visiting public, should be concentrated. The recreational potential of the beach area should be developed.

.....Council should investigate ways and means to acquire cottage properties along the Lakefront and seek assistance of senior levels of government, not only for the purchase of land, but also for the future development of the area.

Subsection 3.5.2.2. Inner Harbour provides that:

Recreational use of the Port Stanley Harbour shall be encouraged. Council shall negotiate with the Federal Government

1. towards the establishment of a public ramp for boat unloading on the west side of the Harbour and
2. the feasibility of reinforcing the east pier and the creation of a small Craft harbour on the east side of the Inner Harbour.

Subsection 3.5.2.3. Eastern Beach provides that:

Council shall preserve and maintain the Eastern Beach. Every attempt should be made to obtain more public beach in Port Stanley to enhance development of the recreational potential of the Harbour.

As of the preparation of this plan a new Official Plan for the Village of Port Stanley had been drafted but had not yet been adopted. A copy of the draft plan is enclosed.

Draft Terms of Reference
FOCALerie

September 6, 1995

Background/Rationale

Conservation Authorities are community based partnerships between the municipalities and the province working to improve the health of Ontario river systems and their watersheds. Each Authority's area of jurisdiction is based on a drainage area or watershed.

Conservation Authorities are structured to enable funds to be generated from a variety of sources. This can be through fundraising/donations or fees charged for services such as entry fees to conservation areas. Many projects are also cost shared with the province and/or municipalities. Generally, for every dollar the province provides in funding, the Authorities raise two dollars locally.

Conservation Authorities in the Lake Erie Basin were formed between 1947 and 1973. Each of the nine Authorities (Figure 1) has since developed programs and initiatives, with all levels of government and ministries, which are in response to local community concerns. Each Authority's program is directed by members from the participating municipalities. Programs are "grassroots" – they originate from the needs and demands of the community. The result has been a strong, community-driven environmental program which has involved many partners through consultation, input, implementation and financial support.

The strength of these individual community efforts has, at times, created challenges in working closely with provincial and federal governments. Conservation Authorities of the Lake Erie Basin formed a cooperative agreement in 1994 to combine the strengths of their individual, long-term community partnerships into "one voice" for the Lake Erie Basin.

Membership/Geographic Scope

FOCALerie consists of nine Conservation Authorities that encompass the drainage basin of Lake Erie including the Detroit River and Lake St. Clair. The nine Authorities in the association are: the Catfish Creek Conservation Authority, the Essex Region Conservation Authority, the Grand River Conservation Authority, the Kettle Creek Conservation Authority, the Long Point Region Conservation Authority, the Lower Thames Valley Conservation Authority, the Niagara Peninsula Conservation Authority, the St. Clair Region Conservation Authority and the Upper Thames River Conservation Authority.

These Authorities are members of the Association of Conservation Authorities of Ontario (ACAO) and operate under the Conservation Authorities Act, RSO 1990.

Purpose

The purpose of FOCALerie is to facilitate working relationships among Conservation Authorities and between the Conservation Authorities and their respective stakeholders. These stakeholders include groups such as those that have special interests, municipal bodies and provincial and federal agencies. The primary purpose is to concentrate these combined efforts towards the betterment of Lake Erie and the Basin.

Goal

The goal of FOCALerie is to bring together the Conservation Authorities within the Lake Erie Basin, to communicate, share expertise, knowledge and form an association that facilitates participation in large scale projects.

To pursue opportunities for efficient program delivery promoting and encouraging common standards and consistency where possible

Objectives

To provide a mechanism (forums and practices) for the Authorities involved to communicate, share expertise and experience and provide shared services to the public.

To provide opportunities for the sharing of resources and staff among the Authorities involved.

To provide federal and provincial governments with direct and easy access to the partner networks, expertise and communication efforts developed by the Conservation Authorities.

To act as a facilitation mechanism for discussions among agencies and government in the development of programs and initiatives.

To provide the means for government to receive valuable public feedback in order to tailor programs to local needs, concerns and/or impacts.

To promote the system of natural and recreational lands owned/managed by the participating Conservation Authorities.

To promote awareness and publicize education materials which further the environmental goals both locally and for the benefit of the Lake Erie basin amongst Conservation Authorities and other public agencies.