Ausable Bayfield Conservation Authority

Shoreline Management Plan

CONSIDERATIONS FOR SHORE PROTECTION STRUCTURES

Final Report

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The following sections of this report present detailed discussions on the shoreline characteristics and processes within the jurisdiction of the ABCA, design considerations and criteria for erosion protection structures in this area, and a summary of conceptual alternatives which can be considered along the ABCA shoreline. Finally, a number of recommendations are made related to the design and implementation (permitting, construction and monitoring) of erosion protection structures in this area.

Finally, it is noted that **the information presented in this report is general in nature and intended for guidance purposes only. It is recommended that a qualified coastal engineer be retained to develop erosion protection designs for any specific site.**

2.0 SHORELINE CHARACTERISTICS

2.1 Introduction - Great Lakes Shorelines Versus Ocean Coastlines

Erosion and flooding hazards along the shorelines of the Great Lakes are similar to those encountered along the ocean coasts. However, a number of important differences exist with respect to shoreline characteristics and shoreline processes which make the Great Lakes unique in terms of shoreline management and protection alternatives.

The Great Lakes are, geologically speaking, very young, and have evolving shorelines. The erosion of the (generally) cohesive shorelines on the lower Great Lakes represents a natural, ongoing, and irreversible process as the shorelines respond to the wave and water level conditions which affect them. In contrast, the ocean coasts are much older in a geologic sense, and are approaching, if not having already achieved, an equilibrium condition with the wave and water level conditions to which they are exposed. Erosion observed along ocean coasts is generally the result of a deficit in sediment, rather than the continuing evolution of the shoreline.

A number of more specific differences can also be identified. First, Great Lakes shorelines are generally characterized by a limited supply of sediment. The vast majority of this material is supplied from the erosion of adjacent (updrift) shorelines, in particular the cohesive bluffs which typify the shorelines of the lower Great Lakes. In contrast, ocean coasts generally have a large supply of sediment which is transported from inland areas to the coast by fluvial (river) processes. The limited supply of sediment on the Great Lakes is transported along the shoreline by wave action, and generally forms narrow beaches and offshore bars. These features provide limited protection from erosion to the shoreline and bluffs. Significant areas of deposition, characterized by wide beaches and extensive dune systems, occur only where a headland (natural or man-made) interferes with the alongshore transport and traps this material. The stability of the beaches and bars, and their ability to protect the shoreline, is dependent on the supply of sand from updrift erosion. As a result, the construction of shoreline protection along updrift shorelines may not only impact alongshore transport processes, but will reduce the supply of sediment to downdrift shorelines. This is a critical point with respect to shoreline management on the Great Lakes.

Second, the wave climate of the Great Lakes is characterized by steep waves generated by local storms (referred to as "sea" waves), with very few occurrences of longer period "swell" waves generated by distant storms. The limited size of the Great Lakes does not allow for swell conditions. This is in contrast to ocean coasts where long period swells are a common occurrence. Sea and swell waves have a very different impact on sand beaches. Typically, sand is eroded from the beach face by the steep waves (seas) during the peak of a storm, and is moved a short distance offshore. On ocean coasts, swell waves, often occurring at the end of the storm (or at some later time), tend to move the sand back onshore. As these longer swell waves do not generally occur on the Great Lakes, it is hypothesized that the large depositional beaches on the Great Lakes may be subject to continuing and irrecoverable losses of sand offshore and, with time, will develop flatter profiles than ocean beaches. As such, these beaches will erode unless they are continually and sufficiently supplied with sand from an updrift source (such as shoreline erosion).

Finally, water levels on the Great Lakes are not affected by tides, but are subject to long term fluctuations in response to climatic variations in the Great Lakes basin area (in particular precipitation and evaporation), as well as to seasonal and short term (storm related) fluctuations (storm related fluctuations also occur on the oceans). The long term variations can not be predicted without a climatic forecast model for (at least) the Great Lakes basin; such a model is not currently available. Historical records indicate that the maximum long term fluctuation on Lake Huron is in the order of 1.6 m; typical seasonal fluctuations are in the order of 0.3 m, and maximum storm related fluctuations are in the order of 1 m. These fluctuations complicate the design of shore protection on the Great Lakes. For example, the design water level is generally the controlling factor in determining the crest elevation and armour stone size required for a shoreline revetment. A revetment designed for an extreme (infrequently occurring) high water level will be expensive, and obtrusive during lower water level conditions. On the other hand, a revetment designed for a lower water level (more frequently occurring) may not provide the required level of protection to the shoreline, and may be damaged or destroyed during storms at higher water levels. With respect to groynes, there is considerable debate in the scientific and engineering communities concerning their application along Great Lakes shorelines. For example, Kamphuis (1990) indicates that the nature of water level fluctuations on the Great Lakes (specifically the long term variations and the absence of tides) may intensify the impact of groynes on alongshore transport processes, and the potential for damage to downdrift properties. In addition, experience on the Great Lakes indicates that groynes provide only limited protection during high water level conditions, such that additional protection in the form of a revetment or seawall is required to provide full protection.

In summary, it is important to recognize that the Great Lakes shorelines are naturally evolving, and that future erosion in inevitable in the long term (hundreds of years). In the short term (tens of years), shoreline management along the Great Lakes must recognize the limited supply of sediment in the nearshore area. The construction of shoreline protection may not only interfere with the natural transport of this material along the shoreline, but will also reduce the supply of sediment to the system, with the potential of adverse, and far reaching, consequences. Finally, the design of shoreline protection must consider the long term fluctuations in water level which exist on the Great Lakes.

2.2 ABCA Shoreline Description

2.2.1 Introduction

The Ausable Bayfield Conservation Authority (ABCA) has jurisdiction over the 60 km length of Lake Huron shoreline between Concession Road 30 in Goderich Township, north of the Village of Bayfield, and Port Franks (approximately), as shown in Figure 2.1. This includes the shorelines of Goderich Township, the Village of Bayfield, Stanley Township, Hay Township, Stephen Township, the Village of Grand Bend, and Bosanquet Township (including the community of Port Franks).

2.2.2 General Background

As a result of the glacial history of this area, the entire region is covered by deep glacial deposits. A schematic cross-section through the eastern shoreline of Lake Huron is presented in Figure 2.2, and indicates the presence of bedrock overlain by Rannoch till, which is in turn overlain by St. Joseph till.

The tills contain differing proportions of sand and gravel in the soil matrix. The Rannoch till is very resistant to wave action as a result of its relatively high gravel content. The St. Joseph till

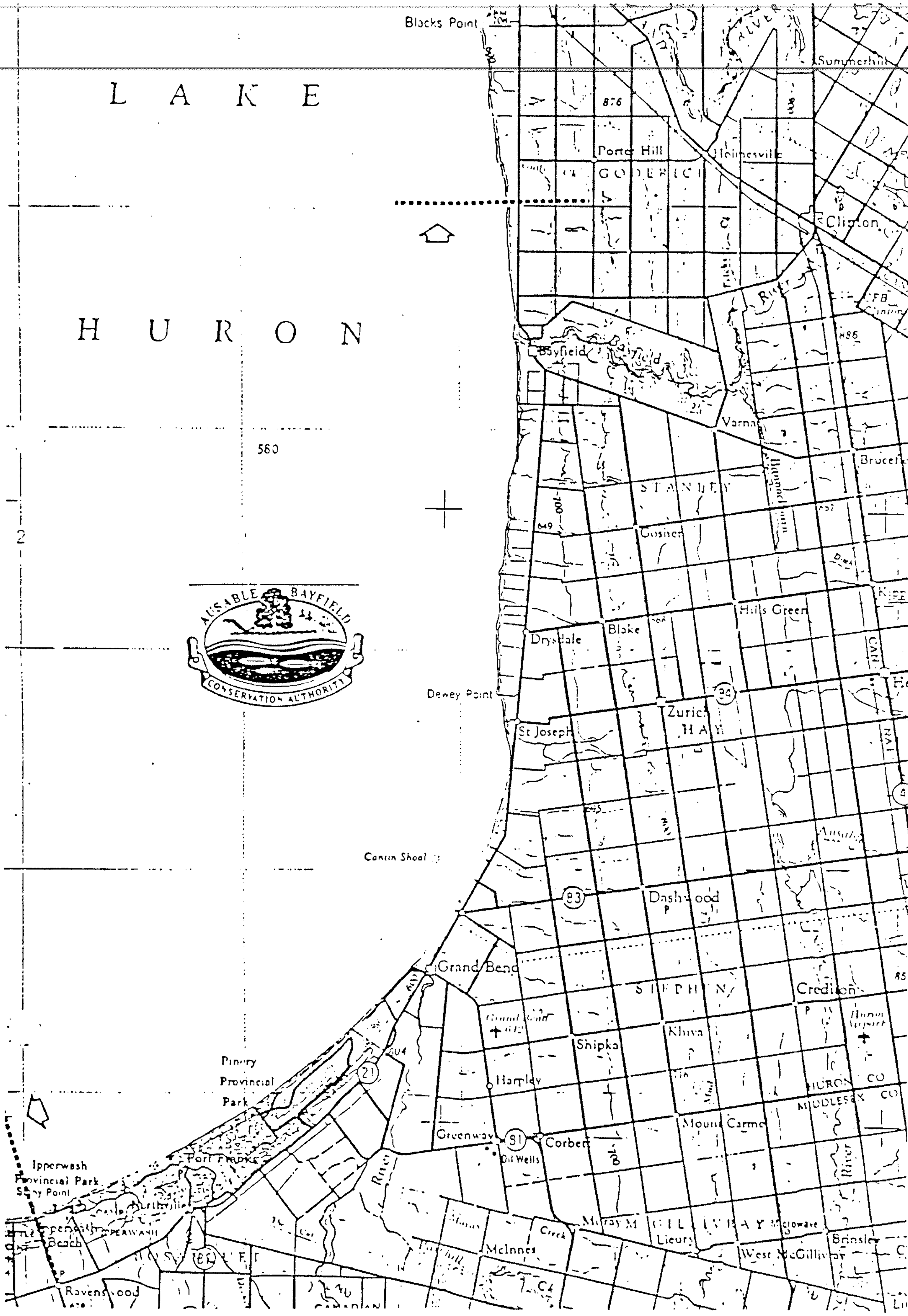

Figure 2.1
ABCA Shoreline Jurisdiction

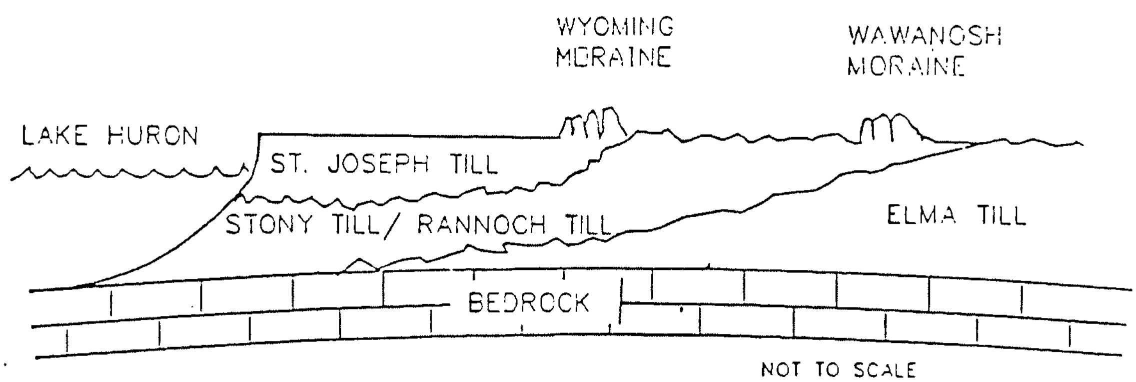

Figure 2.2
Schematic Cross-Section through East Shore of Lake Huron
(from Reinders, 1989)

contains a smaller proportion of gravel than the Rannoch till, and is thus significantly less durable than the Rannoch till. The majority of the exposed bluffs in this area consist of St. Joseph till, which is readily eroded by wave action.

Although wave action at the shore is the dominant force in the evolution of the shoreline, the response of the shoreline to wave action depends on the composition of the soil at the shoreline and on the nearshore lake bottom. Specifically, the presence of exposed Rannoch till on the nearshore lake bottom and at the base of the bluff results in a relatively stable (non-erodible) shoreline, while the presence of St. Joseph till on the nearshore lake bottom and at the base of the bluff results in an eroding shoreline (and nearshore lake bottom). It is believed that shore erosion is controlled and limited by the more resistant Rannoch till along much of the ABCA shoreline. Rocky Point and Dewey Point are examples of relatively stable headlands where the Rannoch till layer rises to an elevation close to the mean lake level.

Erosion of the bluffs and nearshore lake bottom supplies sediment (clay, silt, sand and gravel) to the shore zone. These materials are transported by wave action and currents. The finer sediments (clay and silt particles) are carried in suspension, and tend to deposit offshore in deep water, while the coarser sediments (sand and gravel) are transported along the shoreline and form beaches, dunes and nearshore bars. The extent of these beaches and bars is dependent on a number of factors, including the supply of sand and gravel to a particular location, and the nearshore wave climate and water depths.

2.2.3 Shoreline Characteristics

The ABCA shoreline can be generally classified into an "erosion zone" north of Maple Grove subdivision, and a historical "deposition zone" south of this subdivision. The different characteristics of these two areas are summarized below, followed by a brief description of development along the shoreline in general.

North of Maple Grove subdivision, the shoreline has a north-south orientation and consists of narrow sand beaches fronting till bluffs of moderate height (12 to 18 m). The bluff height tends to decrease to the south, and is in the order of 6 m high at Highway 83. Numerous gullies exist along this section of shoreline; these gullies have developed as a result of surface runoff, and may be stable or actively eroding. The bluffs have historically been eroding as a result of wave action undercutting the toe of the bluffs, which eventually leads to bluff instability and slumping. The extent of the erosion varies; between 1935 and 1988, the long term average erosion rate along the majority of the ABCA shoreline was less than 0.3 m/yr. However, severe erosion occurred in two areas, specifically in the vicinity of Melena Heights and Lakewood Gardens/Sunny Ridge/Poplar Beach, with long term average erosion rates in the order of 1 m/yr over this same 53 year period. Other locations were subject to moderate erosion, with long term average erosion rates in the order of 0.5 m/yr.

As discussed above, the erosion of the bluffs is preceded by, and controlled by, a slow but continuing erosion of the nearshore lake bottom. Although most of the visible erosion (i.e. bluff erosion above the water line) occurs during periods of high water levels, the controlling process of nearshore erosion continues during low water periods; however, the distribution of erosion across the nearshore zone may vary with fluctuating water levels.

The erosion of the bluffs and nearshore lake bottom along this section of shoreline, as well as gully erosion and creek and river sediment transport, provide materials to the nearshore area. Of particular interest is the coarser material, specifically sands and gravels, which can form beaches and bars along the shoreline and thus provide some protection to the shoreline, as well as recreational benefits. Along the ABCA shoreline north of Highway 83, it has been estimated (Reinders, 1989) that approximately 72% of the supply of sand and gravel to the nearshore area comes from bluff erosion, 10% from gully erosion, 17% from lake bed erosion, and 1% from creeks and rivers. This material is transported alongshore by waves and wave-induced currents. The magnitude of this transport is a function of the wave conditions (principally wave height and direction), water depth close to the shoreline and availability of sediments. Due to the wave climate and shoreline orientation in this area, the net transport is from north to south, although reversals do occur in response to individual storms.

To the south of Maple Grove subdivision, the shoreline orientation changes from north-south to northeast-southwest, and the shoreline characteristics change from cohesive till bluffs to sand dunes. As a result of the change in shoreline orientation, the sediment transport rate decreases significantly, and deposition of sand along the shoreline has, historically, occurred. Over thousands of years this deposition has resulted in the development of an extensive beach-dune system along the Grand Bend/Pinery/Ipperwash shoreline. The deposition of sand along this section of shoreline is offset to some extent by wind-blown (aeolian) losses from the beach to the dune and offshore loses. Of critical importance to shoreline management in this area is the fact that the stability of this beach-dune system is dependent on the supply of sand provided by updrift erosion processes, in particular bluff erosion between Grand Bend and Goderich.

Gardens/Sunny Ridge/Poplar Beach area. Bluff erosion supplies an average of approximately 32,600 m^3/yr of sand to the nearshore zone, while gully and lake bottom erosion supply approximately 4,200 and 7,400 m^3/yr respectively (Reinders, 1989).

Unique features along this section of shoreline include Rocky Point and Dewey Point, both of which are headlands projecting into the lake relative to the adjacent shorelines. As noted earlier, the long term stability of these points relative to the adjacent sections of shoreline is due to the presence of hard Rannoch till on the nearshore lake bottom rather than soft St. Joseph till.

Subcell 4 - Maple Grove Subdivision to Kettle Point
(Reach J in Reinders, 1989)

Between Maple Grove subdivision and Kettle Point, the shoreline consists of a relatively wide beach fronting sand dunes. This reach of shoreline represents the deposition zone for the material which has been eroded from the bluffs, gullies and lake bed along the "updrift" shoreline to the north. Over thousands of years, the deposition of sand along this reach of shoreline has resulted in the present day fully-developed beach-dune system. However, a comparison of shoreline conditions in 1935 and 1988 indicates that although the dune face has been relatively stable, the beach width has decreased substantially over this 53 year period. This apparent beach erosion may be due in part to the reduced supply of sand to this area caused by the construction of Goderich harbour in 1916, as well as possible losses to deep water caused by the harbour structures at Bayfield and Grand Bend. However, the observed beach erosion can not be fully explained by these factors. Losses from the beach also occur as a result of aeolian transport to the dunes and offshore transport to deep water during storms. However, there is no evidence to suggest that the magnitude of these losses has increased sufficiently to result in the observed beach erosion. Thus, at this time, it has not been possible to fully explain the erosion of the beach which has occurred along this reach of shoreline.

Similar to Bayfield, a fillet beach has developed to the north of the Grand Bend harbour structures (built in 1904). This beach extends to the Maple Grove area, and appears to have reached an equilibrium condition such that sand is now bypassing the harbour structures to be deposited further downdrift. Limited shoreline protection has been constructed to the north of the harbour, while extensive protection has been constructed to the south of the harbour, particularly within the Village limits. This protection consists of groynes, seawalls and revetments intended to limit erosion of the dune during periods of high water.

Figure 2.3 provides an overview of the shoreline processes within the Goderich to Kettle Point littoral cell, while a more detailed description of each of the four subcells is presented in Appendix A.

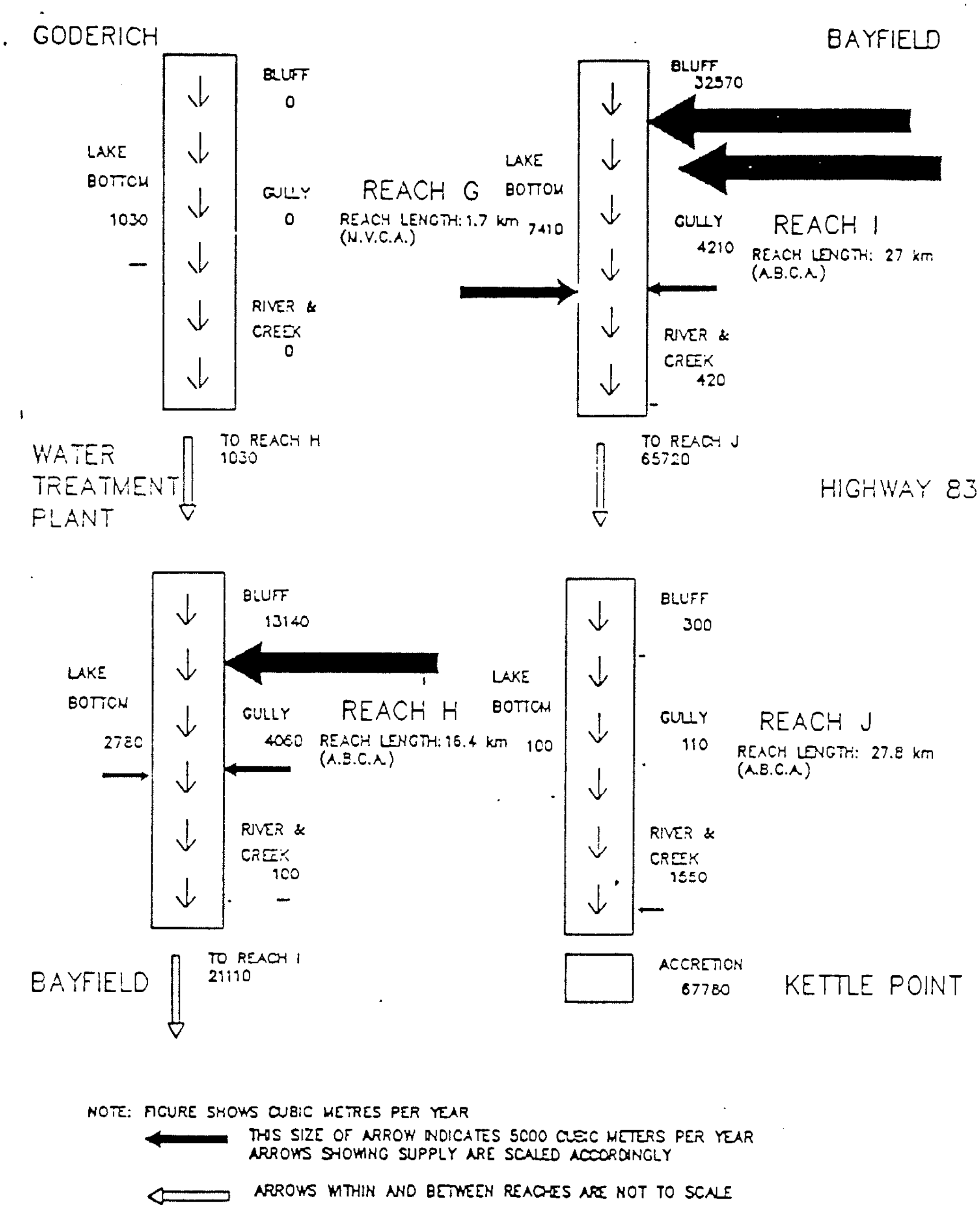

Figure 2.3

Summary of Sediment Supply between Goderich and Kettle Point

(from Reinders, 1989)

3.0 DESIGN CONDITIONS

3.1 Water Levels

Water levels on Lake Huron vary substantially in both the long and short term, as well as seasonally. Long term variations are the result of climatic changes, in particular precipitation and evaporation. The most recent period of high lake levels was 1985-86, while the most recent period of low lake levels was 1964-65. On Lake Huron, the difference between the maximum and minimum annual mean lake levels recorded since 1920 is 1.6 m (Environment Canada, 1988). It is important to note that due to the size of the Great Lakes and the limited discharge capacities of their outflow rivers, extreme high or low lake levels will generally persist for a period of years after the factors that caused them have changed. However, lake levels dropped from record highs to "normal" conditions more quickly than expected following the 1985-86 period of high water levels.

Seasonal fluctuations in the lake level are associated with the annual weather pattern. The lowest levels typically occur in the winter when most precipitation is snow and ice, while the highest lake levels typically occur in the summer following spring runoff. On Lake Huron, the average seasonal water level fluctuation is approximately 0.3 m. (Environment Canada, September 1991). Figure 3.1 shows the seasonal fluctuations in the average, maximum and minimum monthly mean water levels on Lake Huron between 1916 and 1991. This figure also indicates that the maximum recorded monthly mean levels all occurred during the most recent high water period (1985-86), while the minimum levels all occurred during the most recent low water period (1964-65). (Water levels in this figure, and throughout this report, are referenced to Lake Huron low water datum (LWD), which is equal to 175.8 m International Great Lakes Datum (IGLD 1955). In order to convert LWD elevations to IGLD elevations, one must add 175.8 to the LWD values.)

Finally, short term (hours or days) fluctuations in the water level occur due to the passage of weather systems, with wind stress on the water surface and atmospheric pressure changes causing localized setups referred to as storm surge, as shown in Figure 3.2. Storm surges along the ABCA shoreline may range from 0.4 to 0.8 m depending on the severity of a particular storm (Reinders, 1989) and the location along the shoreline.

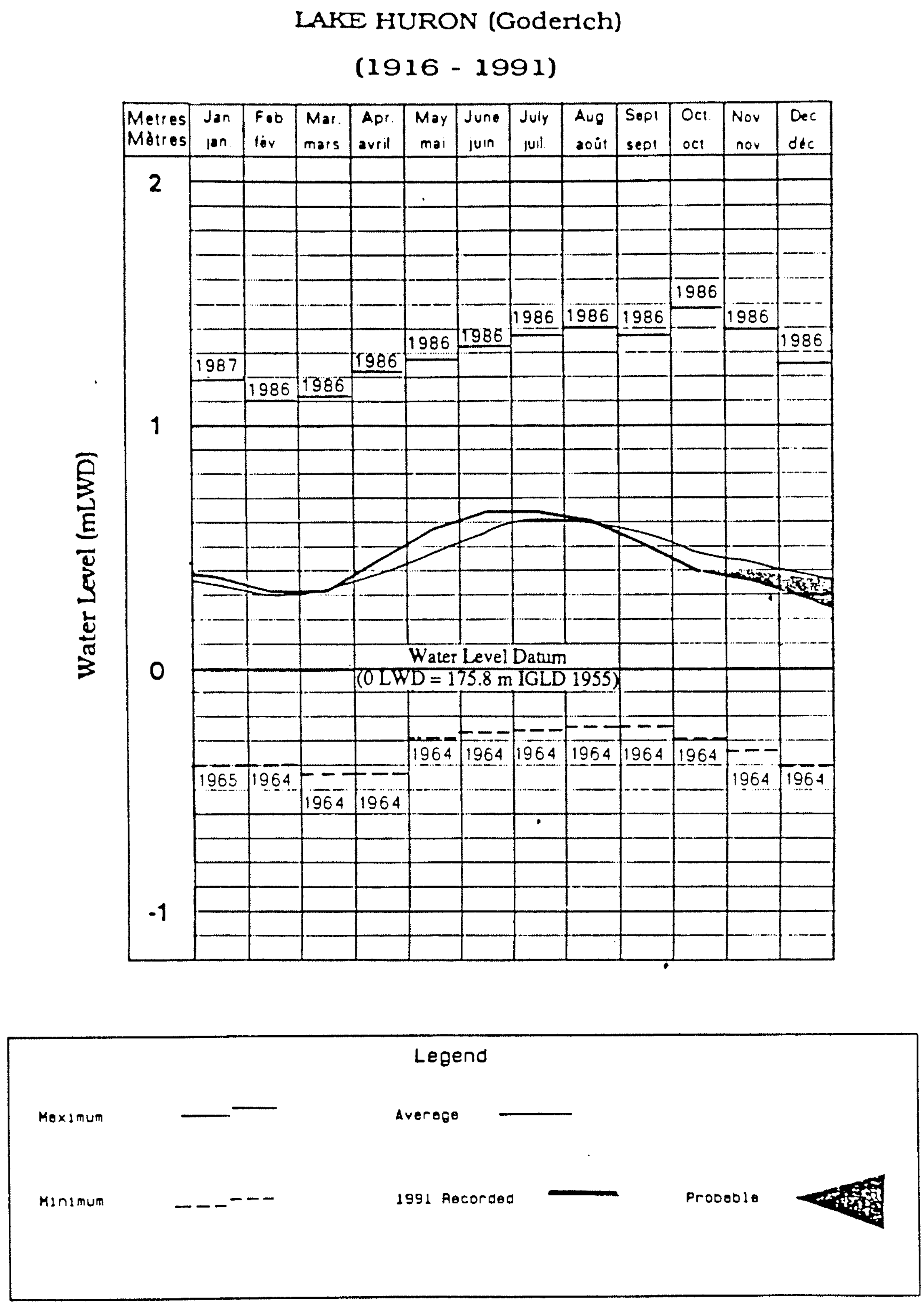

Figure 3.1

Seasonal Water Level Fluctuations on Lake Huron

There is considerable debate in the scientific and engineering communities concerning the selection of design water levels for coastal structures on the Great Lakes. Although the application of standard statistical techniques (such as frequency and extreme value analyses) is not strictly applicable to Great Lakes water levels, both MNR (1989) in Ontario and the U.S. Army Corps of Engineers (USACOE, 1988) in the U.S.A. have utilized such techniques to establish extreme water levels associated with selected return periods. The return period refers to the frequency of occurrence of a specified extreme event. For example, an event with a return period of 100 years will occur, on average, once every 100 years. The probability of this event occurring in any particular year is 1/100, or 1%. For the purposes of preliminary design of shoreline protection structures, the MNR (1989) results will be used to define the design water levels. A summary of these results for the ABCA shoreline is presented in Table 3.1.

Table 3.1

Design Water Levels for ABCA Shoreline
(from MNR, 1989)

Return Period (years)	Peak Instantaneous Water Level (m LWD)* Kettle Pt. - Dewey Pt.	Dewey Pt. - Goderich
5	+1.4	+1.3
10	+1.6	+1.5
25	+1.7	+1.6
100	+1.9	+1.8

*Note: 0 LWD = 175.8 m IGLD 1955

Although water levels are not systematically recorded anywhere within the jurisdiction of the ABCA, these estimated design water levels can be compared to the extreme water levels recorded by Environment Canada at Goderich. The maximum recorded daily mean and peak instantaneous levels at Goderich between 1910 and 1990 were +1.6 and +1.8 m LWD respectively; both were recorded on November 9, 1986. Thus, the peak instantaneous level recorded during this 80 year period is estimated to have a return period of 100 years.

The selection of a design water level is of critical importance to the design of a shoreline protection structure, as the wave height acting on a structure in shallow water adjacent to the shoreline will be limited by the depth of water. Higher water levels will allow larger waves to reach the structure, thus requiring more substantial structures. Similarly, erosion of the nearshore lake bottom will allow larger waves to reach structures adjacent to the shoreline, and must be considered for structures with a design life greater than approximately 5 to 10 years at locations where the nearshore lake bottom consists of the erodible St. Joseph till.

3.2 Nearshore Lake Bottom Erosion

As noted earlier, the nearshore area typically consists of a beach of varying width deposited over glacial till. The beach is very dynamic in nature, constantly changing in response to varying wave action and water levels. In addition, one or more sand bars may be present depending on the supply of sand. Clearly, the design of any shoreline protection structure must recognize the dynamic nature of the beach, and should not be dependent on the presence of the beach for its stability. An analysis of long term beach stability is relatively complicated, and such site specific investigations are beyond the scope of this study.

In addition, the design of shoreline protection structures must consider the slow, but ongoing, erosion of the nearshore lake bottom. This process is relatively independent of water level fluctuations, with erosion of the lake bottom continuing during periods of low water, as well as during periods of average and high water. The erosion may be insignificant over the short term, but may have significant implications to shoreline protection in the long term. Specifically, erosion of the nearshore lake bottom in front of a shoreline protection structure may result in undermining of the structure, leading to damage and perhaps failure of the structure. In addition, this process will result in deeper water in front of the structure, thus allowing larger waves to attack the structure. For shore protection to be effective over the long term (greater than 5 to 10 years), the design must consider the future erosion of the lake bottom, and the larger waves which will ultimately attack the structure.

No measurements of this process are available in the study area, and only limited measurements are available at other locations on the Great Lakes. For example, Davidson-Arnott (1986) undertook field measurements to monitor this process along the southern

Lake Ontario shoreline between Hamilton and Grimsby, and found that the rate of lake bottom erosion was in the order of 5 cm per year (vertical erosion) immediately adjacent to the shoreline, and decreased as one moved offshore into deeper water.

This topic is the subject of a number of on-going studies at different locations around the Great Lakes by various organizations. For example, Nairn and Baird & Associates (1992) recently completed a detailed investigation of Great Lakes erosion processes for the International Joint Commission. This study utilized a numerical model to estimate the long-term erosion of the shoreline and nearshore profile for typical Great Lakes shore types. The results of this investigation indicate that the erosion of a cohesive shoreline is controlled by nearshore lake bottom erosion. This process can be approximated as a landward shift of the nearshore profile at the same rate as bluff recession in the area, with the nearshore profile retaining its original shape, as illustrated in Figure 3.3.

In order to estimate the long term erosion of the nearshore lake bottom, a methodology was developed (refer to Appendix B) to relate the lake bottom erosion (D) to the shape of the nearshore profile, the average annual bluff recession rate (R) and the time period of interest (t). Table 3.2 illustrates the deepening (erosion) of the nearshore lake bottom as a function of the quantity Rt and the offshore distance for a typical profile along the ABCA shoreline.

Table 3.2

Erosion of the Nearshore Lake Bottom
for Typical Nearshore Profile

Offshore Distance (m)	Existing Water Depth (m)		Future Water Depth (m) vs. Rt						
		Rt =	1	2	5	10	20	50	100
0	0.00		0.03	0.05	0.11	0.21	0.38	0.82	1.43
15	0.30		0.32	0.33	0.38	0.46	0.61	1.02	1.59
34	0.60		0.61	0.63	0.67	0.74	0.88	1.25	1.78
56	0.90		0.91	0.93	0.97	1.03	1.15	1.49	2.00
80	1.20		1.21	1.22	1.26	1.31	1.43	1.74	2.21
107	1.50		1.51	1.52	1.56	1.61	1.71	2.00	2.45

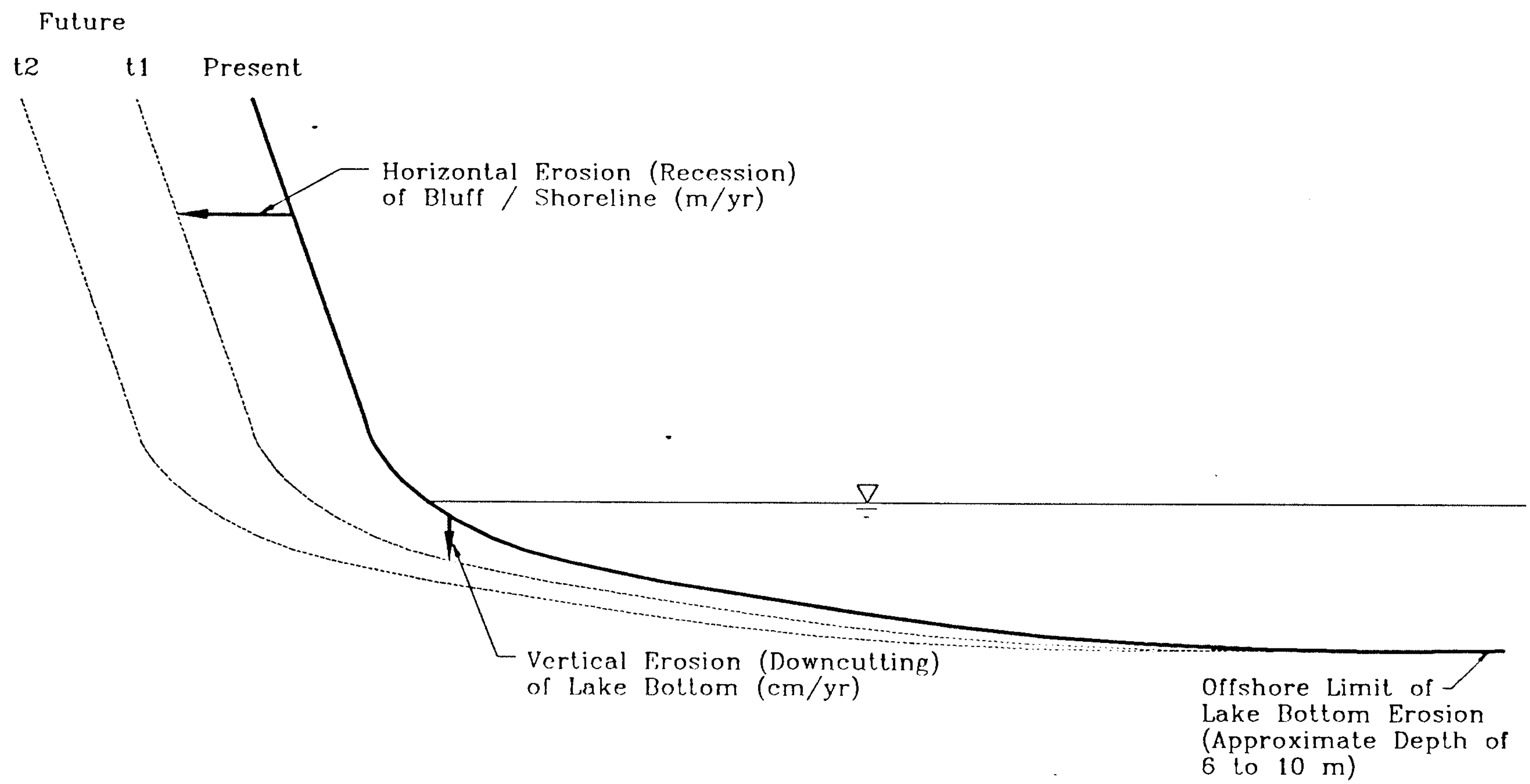

Figure 3.3

SCHEMATIC DIAGRAM OF NEARSHORE PROFILE EROSION

For example, assuming a bluff recession rate of 0.5 m/yr and a time span of 100 years (i.e. Rt = 50), the water depth at the present shoreline location will increase from 0 to 0.82 m over this period (refer to highlighted values in Table 3.2). A similar increase in depth would occur with a bluff recession rate of 1.0 m/yr over a period of 50 years (or any other combination of R and t yielding Rt = 50).

In the absence of reliable site specific information describing the erosion of the nearshore lake bottom, the preliminary approach described above should be utilized to estimate the future lake bottom elevation and water depth to be used in the design of any shoreline protection structure, in particular where a structure is intended to provide medium to long term protection in an area of moderate to severe erosion, as defined by an Rt value greater than 5 to 10. In these cases, overlooking the process of lake bed erosion may result in damage to or failure of the structure due to undermining and/or exposure to waves exceeding the design condition.

3.3 Waves

Deep water wave conditions offshore of the ABCA shoreline have been estimated at a number of locations using a wind-wave hindcast procedure (MNR, 1988). These long term (1953 - 1987) wave data are available in summary presentations, including scatterplots (which show the frequency of occurrence of different wave heights and period by direction) and wave roses, as well as hourly time series data in digital files. An estimate of nearshore wave conditions requires a site specific investigation of shallow water transformations, including refraction, shoaling, diffraction and breaking. These processes are discussed in detail in the Shore Protection Manual (USACOE, 1977, 1984).

The design wave height incident on a shoreline protection structure along this section of shoreline will be depth-limited. In other words, the magnitude of the largest wave which can impact the structure is controlled by the water depth in front of the structure. Although the nearshore slope will also affect the magnitude of the "breaking" waves, one can assume that the maximum wave height will be limited to approximately 80% of the water depth in front of the structure. An improved estimate of the design breaking wave height, which considers the slope of the nearshore lake bottom, can be developed using procedures presented in the Shore Protection Manual (USACOE, 1977, 1984) or in Goda (1970, 1985).

Clearly, water level variations and long term erosion of the nearshore lake bottom must be considered in establishing the design water depth and design wave height for a structure. Higher water levels and erosion of the lake bottom will both allow larger waves to reach the structure, and will have a significant impact on the design of shoreline protection structures. Thus, prior to determining the design wave height, one must establish the existing water depth in front of the proposed structure, and then add allowances for the design water level (considering both high lake levels and storm surges - refer to Section 3.1) and nearshore erosion (refer to Section 3.2) associated with the selected design life of the proposed structure. For preliminary design purposes, the design wave height can then be estimated as 80% of the total water depth. Table 3.3 summarizes the design water depth and preliminary design wave height for different design lives, assuming a typical nearshore profile and a shoreline/bluff recession rate of 1 m/yr. A more refined estimate of the design wave height (for example, using Goda (1970, 1985)) should be developed during the final design phase.

Table 3.3

Design Water Depths and Preliminary Design Wave Heights
(typical nearshore profile, R = 1.0 m/yr)

Design Life (t) (years)	Design Water Level (m LWD)	Rt (m)	Future Water Depth at Existing Shoreline Location (m LWD)	Total Water Depth (m)	Design Wave Height (m)
5	+1.4	5	-0.1	1.5	1.2
10	+1.6	10	-0.2	1.8	1.4
25	+1.7	25	-0.5	2.2	1.7
100	+1.9	100	-1.4	3.3	2.7

It is important to note that an increase in design wave height will result in a significant increase in the cost of a shoreline protection structure. For example, in the case of revetments, the geometric dimensions of the structure are proportional to the design wave height, while the stone sizes are proportional to the cube of the wave height. Thus, doubling the design wave height, as is more or less required to go from short term (5 to 10 years) to long term (100 years) protection will require a significantly larger structure (higher and wider crest, and deeper excavation for toe) protected by much larger stones. This would result in a significant increase in construction cost (perhaps by an order of magnitude), although maintenance, repair and replacement costs would be reduced or eliminated. Groynes and seawalls are also sensitive to the design wave height, although perhaps not as dramatically as revetments. However, groynes can not fully protect the shoreline under very severe conditions (extreme storms at high water levels), and would therefore require secondary protection in the form of a revetment or seawall buried behind the beach in order to prevent erosion under these conditions.

3.4 Ice Conditions

Ice forces must be considered in the design of any coastal structure on the Great Lakes. Horizontal ice forces may be caused by thermal expansion of the ice sheet or by moving ice flows. Vertical ice forces may be caused by variations in the water level if the ice sheet has affixed itself to a structure. In general, structures which extend into the lake (such as groynes) are more susceptible to ice damage than structures which extend along the shoreline (such as seawalls and groynes). Great Lakes experience suggests a horizontal design force in the order of 10,000 lb/ft for exposed structures with vertical faces. Sloping structures are generally subjected to lower ice forces, as the ice tends to fail in flexure as it encounters a sloping structure, rather than by crushing against a vertical face, which does not promote flexure of the ice sheet.

Piles are also susceptible to "ice jacking", which refers to the process in which the ice sheet freezes to the pile and may lift it when a rise in water level occurs. This process is generally irreversible, as a fall in water level generally causes fracture of the ice sheet adjacent to the pile rather than pushing the pile back into the ground. As a result, water level fluctuations during the winter, in particular the seasonal rise in water level which occurs each spring (March-April, see Figure 3.1) may progressively lift the pile, thereby reducing the pile penetration depth into the lake bottom and thus reducing its ability to resist loading conditions in the future. Thus, piles must be driven to a sufficient embedment depth to resist the forces associated with this process.

In general, the design of shore protection to resist ice forces is based on experience rather than analyses. Inspection of existing shoreline protection structures in this area demonstrates the susceptibility of the lakeward ends of steel sheet pile groynes to ice damage. As such, ice forces may be an important consideration in the design of such structures. Existing revetments and seawalls in the study area do not appear to have suffered any significant ice-related damage.

3.5 Geotechnical Considerations

An assessment of the foundation conditions should be undertaken prior to the design of any shoreline protection structure. Specifically, it is important to identify the presence of soft subsurface materials, which might result in excessive settlement and failure of the structure, and the presence of extremely hard subsurface materials, which might limit pile embedment depths. Along this shoreline, the nearshore area generally consists of a thin layer of unconsolidated beach deposits over glacial till. This till may be relatively soft and erodible (St. Joseph till), or relatively hard and non-erodible (Rannoch till). As noted earlier, the beach is very dynamic in nature, and any shoreline structure should be founded on the underlying glacial till. Further, the design should consider the erosion of the glacial till on the nearshore lake bottom if it is intended to provide long term protection to the shoreline. With respect to revetments, this will require excavation to the expected erosion depth or to the hard Rannoch till, whichever is reached first, in order to provide a stable foundation for the structure. With respect to sheet pile structures, this will require sufficient embedment depths and reinforcing or anchoring details to resist the applied loads under both existing and future conditions.

4.0 WAVE/SHORELINE ISSUES AND SHORE PROTECTION OBJECTIVES

4.1 Introduction

Prior to discussing shore protection alternatives, it is first necessary to clearly identify the shoreline problem at a particular site, and thus the objective(s) of the proposed structure. To assist with this analysis, different categories of wave/shoreline issues that exist along the ABCA shoreline are discussed below, and the corresponding objectives of shore protection are summarized. A detailed discussion of alternative shore protection concepts and their suitability to the ABCA shoreline is then presented in Section 5.0. Section 6.0 presents a discussion of issues related to the implementation of new shoreline protection structures. Finally, Section 7.0 presents specific recommendations for the selection, design and implementation of shore protection structures along the ABCA shoreline.

4.2 Discussion of Wave/Shoreline Issues

The primary issue of wave related damage along the ABCA shoreline can be split into two general categories, the first being storm (wave runup) damage which occurs during storms (typically during high water periods), and the second being long term erosion. Long term erosion can be further categorized as minor, moderate and severe. A secondary issue along the ABCA shoreline is the recreational aspects of the beaches in front of the bluffs. A discussion of these issues along the ABCA shoreline, and the corresponding objectives of shore protection structures, is presented below.

4.2.1 Severe Shoreline Erosion and Bluff Recession

There are two areas along the ABCA shoreline that experience severe shoreline erosion and bluff recession, specifically the Melena Heights subdivision and the Lakewood Gardens/Sunny Ridge/Poplar Beach area. The typical characteristics of these areas are as follows:

- Top of bluff is receding at an average long term rate in the order of 0.6 to 1.3 m/yr.

- Major slumps occur along the shoreline.

- Bluff face contains no vegetation.

- Undercutting of the base of the bluff is typical.

- Very little, if any, beach exists at the toe of the bluff.

- The nearshore lake bottom is also eroding. In fact, it is the erosion of the nearshore lake bottom that is controlling bluff erosion, as discussed earlier in this document (refer to Section 3.2).

A structure built along this shoreline would have the objective of stabilizing the shoreline at its current location. The nearshore lake bottom will continue to erode in front of the structure, resulting in deeper water and exposing the structure to larger waves in the future. This process must be considered in the design of the structure, and will result in a relatively large and costly structure if it is to stabilize the shoreline for a period of more than 5 to 10 years.

It is unlikely that a permanent beach could be developed adjacent to this shoreline without large groyne type structures and a significant quantity of coarse beach fill.

4.2.2 Moderate Shoreline Erosion and Bluff Recession

There are a number of areas along the ABCA shoreline that experience moderate shoreline erosion and bluff recession. The Salvation Army Camp and Vista Beach subdivisions are typical examples. The characteristics of these areas are as follows:

- Top of bluff is receding at an average long term rate in the order of 0.3 to 0.6 m/yr.

- Bluff experiences localized slumping.

- Bluff contains some vegetation. Typically, a steep unvegetated scarp of up to 3 m high exists at the base of the bluff.

- A small beach may exist at the base of the bluff.

- The nearshore lake bottom is eroding close to the shoreline. However, it is likely that in water depths exceeding approximately 2 m, the lake bottom will be covered and stabilized by lag deposits of gravel (including cobbles and boulders), indicating the presence of more resistant material (Rannoch till) below this elevation.

A structure built along this shoreline would have the primary objective of stabilizing the shoreline at its current location, and might have the secondary objective of preventing wave attack at the base of the bluff. Shore protection structures would need to be relatively substantial. Ideally, the base of the structure would extend to the depth of the more resistant Rannoch till, as the nearshore lake bottom will continue to erode in front of the structure until it reaches this level.

4.2.3 Minor Shoreline Erosion and Bluff Recession

The majority of the ABCA shoreline experiences minor shoreline erosion and bluff recession. Pope's Beach, Gammage and Durand-Huronview subdivisions are typical examples. The characteristics of these areas are as follows:

- Top of bluff is receding at an average long term rate in the order of 0.1 to 0.3 m/yr.

- Bluff may experience infrequent, localized slumping.

- Bluffs are largely vegetated with grasses, shrubs and small trees.

- A moderate sized beach exists at the base of the bluff.

- A "fillet" of sand may exist against the base of the bluff. This fillet of sand may be eroded by wave runup on the beach during periods of high water.

- Minor erosion of the nearshore lake bottom is occurring close to the shoreline. However, it is likely that in water depths exceeding approximately 1 m, the lake bottom will be covered and stabilized by deposits of gravel (including cobbles and boulders), indicating the presence of more resistant material (Rannoch till) below this elevation.

Structures built along this shoreline would have the objective of preventing wave runup from reaching the base of the bluff, and/or protecting walkways or patio areas built at the base of the bluff, particularly during periods of high water levels.

Groyne type structures may be considered to enlarge the existing beach to provide an improved recreational area, and to provide protection from wave runup reaching the bluff during periods of high water.

4.2.4 Stable Waterline and Bluff

There are some areas of the ABCA shoreline that have not experienced any noticeable erosion of the bluff during the last fifty years. These locations occur where bedrock or Rannoch till exists at the shoreline and across the nearshore area, such as at Dewey Point and Rocky Point, but also in areas such as Houston Heights, Vodden Beach and Ridgeway subdivisions. The characteristics of these areas are as follows:

- Top of bluff is relatively stable (average long term erosion rate < 0.1 m/yr).

- Bluff may experience very infrequent slumping as a result of ground water loading.

- Bluffs are well vegetated with mature trees.

- A moderate to large sized beach generally exists at the base of the bluff.

- A "fillet" of sand may exist against the base of the bluff. This fillet of sand may be eroded by wave runup on the beach during periods of high water.

- The nearshore lake bottom is relatively stable. The beach overlies the nearshore lake bottom, which consists of either Rannoch till (armoured by gravels, including cobbles and boulders) or bedrock.

Structures built along this shoreline would have the objective of preventing wave runup from reaching the base of the bluff, and/or protecting walkways or patio areas built on the base of bluff, particularly during periods of high water levels.

Groyne type structures might be considered to enlarge the existing beach to provide an improved recreational area, and to provide protection from wave runup reaching the bluff during periods of high water.

4.3 Summary

From the perspective of categorizing different types of shoreline protection structures, it is useful to consider the following three situations:

1) The nearshore lake bottom is eroding, and the shoreline and bluff are receding as a result of wave action.

2) The nearshore lake bottom is eroding, but only to a limited depth. The shoreline and bluff are receding as a result of wave action.

3) The nearshore lake bottom is stable. The shoreline and bluff are also stable (unless adverse landside influences exist).

These three situations are illustrated schematically in Figure 4.1.

In Situation 1, the primary objective of a shore protection structure would be to prevent continuing erosion of the shoreline. As erosion of the nearshore lake bottom will continue in the future, the design of the structure must have a footing at a sufficient depth to prevent undermining, and must be designed to resist the larger waves to which it will eventually be exposed .

Situation 2 is partially a problem of preventing continuing erosion of the shoreline (as in Situation 1) and partially a problem of eliminating wave runup from reaching the bluff during periods of high water.

In Situation 3, the issue is preventing wave runup on the beach from reaching the bluff during periods of high water levels.

The design considerations for these three situations are different. The choices for Situation 1 are limited. These structures must be carefully designed and will be relatively costly if effective long term shoreline stabilization is to be achieved.

Situation 2 is also complex, and requires detailed design analyses and well built structures if long term stabilization is to be achieved. However, short term protection for wave runup during a high water period may be achieved with relatively small and inexpensive structures.

With respect to Situation 3, many minor structures that do not require in-depth analysis can be used to prevent wave runup from reaching the bluff.

The following section of this report discusses the range of shore protection alternatives which will respond to Situations 1 and 2, where the principal objective of the structure is to prevent long term erosion and stabilize the shoreline at its existing location.

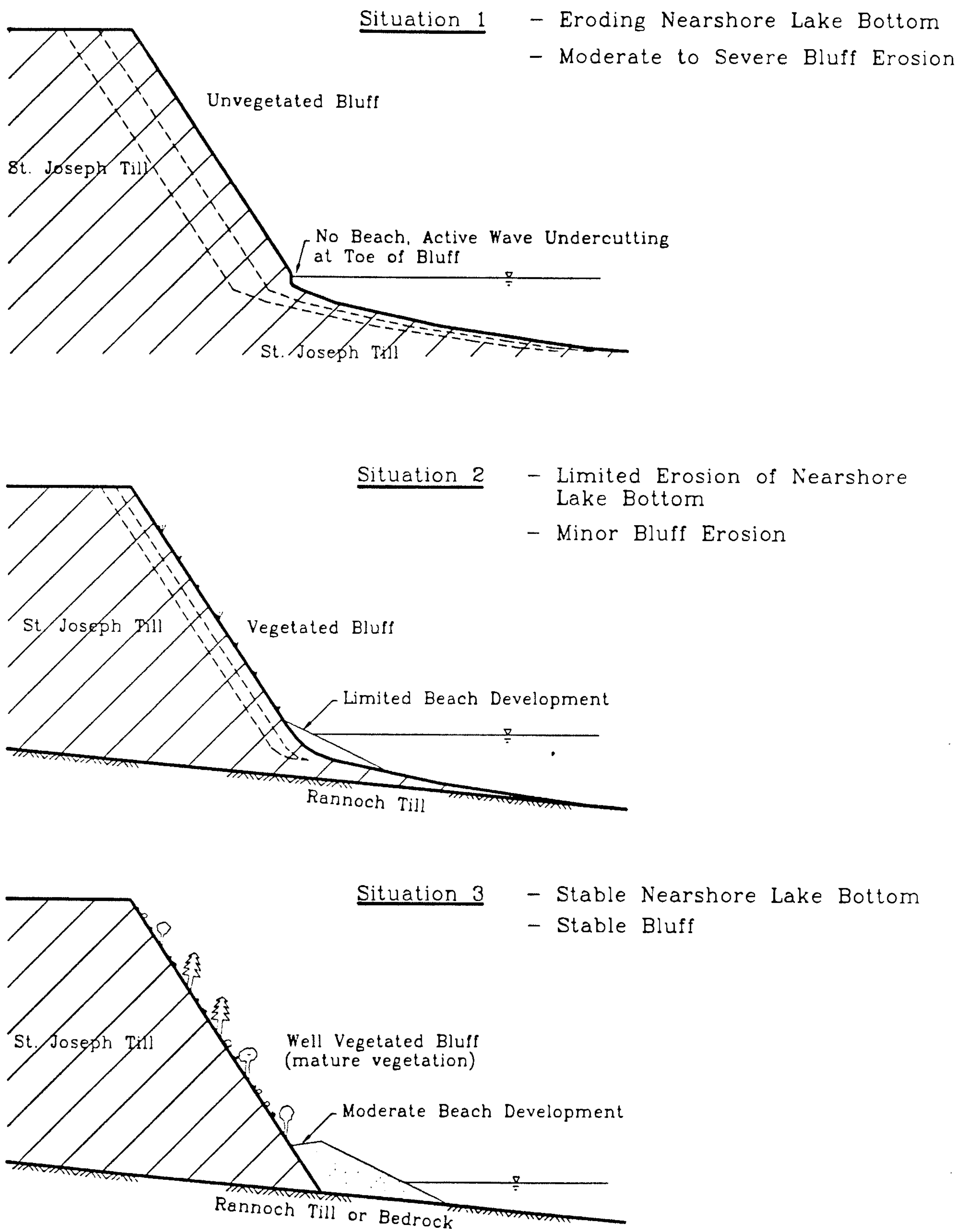

Figure 4.1

SHORE EROSION
THREE TYPICAL SITUATIONS ALONG ABCA SHORELINE

5.0 EROSION PROTECTION DESIGN CONCEPTS

5.1 Introduction

As discussed in Section 4.0, the following discussion of shore protection concepts focuses on the range of alternatives available where the principal objective of the structure is to prevent long term erosion of the shoreline. The alternatives considered include groynes, seawalls and revetments, which are currently in use along the ABCA shoreline, as well as beach nourishment and offshore breakwaters. This report does not address the many types of structures that may be used to prevent runup on the beach from reaching the base of the bluff, walkways or patio structures. These structures are addressed in detail in a number of other publications, including MNR (1986) and USACOE (1978, 1981).

The selection of a particular approach, including the type of structure and an appropriate design life, is a complicated decision which must consider many factors, including cost (capital and maintenance), performance (protection to the shoreline), aesthetics (principally the structure elevation), access (to the water), and impacts on the nearshore environment and neighbouring shoreline properties. These impacts may extend beyond the immediately adjacent areas and could affect the entire downdrift shoreline as a result of reduced sediment supply to the nearshore system caused by reduced erosion of the backshore. Finally, it is important to note that shoreline protection can reduce or eliminate erosion of the backshore, but the long term erosion of the nearshore lake bottom will continue. Thus shore protection designs must consider this future deepening of the nearshore, or suffer the consequences, which will ultimately lead to a requirement for costly maintenance/repair/replacement works or alternatively retreat from the shoreline; such "prevention" alternatives should also be considered at this time.

It is emphasized that designs presented in this report are preliminary in nature. **Final designs should be developed on a site specific basis, within the overall framework of the Shoreline Management Plan (SMP), by a qualified coastal engineer.** Issues associated with implementation of these structures are discussed in Section 6.0, including final design, permits and approvals, financing, construction, monitoring and maintenance. Finally, specific recommendations with respect to the selection, design and implementation of shore protection structures along the ABCA shoreline are presented in Section 7.0.

5.2 Existing Shore Protection Structures

Various forms of shoreline protection have been constructed along the ABCA shoreline. A detailed inventory of these structures is presented in ABCA (1990). The design of individual structures, and the extent of these structures along the shoreline, varies considerably within the jurisdiction of the ABCA. In general, the level of protection is limited, due to the relatively limited development (cottage subdivisions) along the majority of the shoreline.

There are large unprotected areas (for example, the shoreline between Soper's Beach and Elliot's Grove), as well as areas with significant protection (for example, the shorelines between Houston Heights North and Homestead Heights, and to the south of Grand Bend harbour). Groynes and seawalls are the predominant structures, and are generally constructed of steel sheet piling, although gabions, concrete (precast and cast in place) and timber have also been used. A limited number of rubblemound revetments have also been constructed.

In general, the existing structures represent efforts to protect the shoreline from storm related wave runup damage, and have been constructed during or after periods of high water levels. In areas subject to moderate or severe long term erosion, such as Melena Heights and Poplar Beach, the existing structures will have no significant impact on long term erosion of the shoreline and bluff.

More substantial shore protection has recently been constructed north of Grand Bend harbour to protect the Beachplace condominium development (Sandwell Inc., 1990), and also to the south of the harbour to protect the Southcott Pines subdivision (Butler & Associates Ltd., 1986). The Beachplace structure, which consists of a rubblemound revetment and a concrete retaining wall, was designed considering an extreme (100 year return period) erosion event on the beach fronting it. The design of the Southcott Pines revetment does not appear to have considered the potential for erosion of the nearshore lake bottom in this area.

In summary, existing shore protection structures may be effective in reducing or preventing wave runup damage during storms, but will generally not have a significant impact on long term erosion of the shoreline.

5.3 Groynes

5.3.1 Discussion

Groynes are structures built perpendicular (more or less) to the shore to encourage the development, or prevent the erosion, of a beach. They accomplish this by trapping coarse sediments and reorienting the beach such that the alongshore transport of these coarse materials, which is partially dependent on the angle of incidence of the waves relative to the shoreline, is reduced or eliminated. Groynes generally extend across the normal breaker zone, thus reducing or eliminating the alongshore transport of coarser sediment fractions close to the shore (on the beach and inner bar(s)), but not significantly affecting the alongshore transport of finer material on the outer bar(s).

Groynes are a popular form of shore protection that may increase beach stability and size, and provide effective shoreline protection at a relatively low cost compared to other alternatives. In addition, larger beaches will provide increased recreational potential along the shoreline. However, groyne design is relatively complex, and the concept is not applicable to all situations. For example, **groynes are dependent on a sufficient supply of littoral drift to "feed" the beaches.** Also, in general, **groynes can not, on their own, provide full protection to the backshore under extreme conditions** (severe storms at high water levels). Thus, artificial beach nourishment and/or supplementary shore parallel protection (revetments or seawalls) may be required in conjunction with groynes to provide effective shoreline protection.

There is considerable debate in the scientific and engineering communities concerning the use of groynes as shoreline protection, particularly on the Great Lakes, where their application is complicated by long term water level fluctuations and where poor design and implementation have often resulted in relatively ineffective shore protection and significant downdrift impacts. For example, Kamphuis (1990) notes that the adverse impacts of groynes on shoreline processes are intensified by the nature of Great Lakes water level fluctuations, notably the long term variations and the absence of tides. The recent experience with groynes along the shorelines of Cedar Banks, Shadeview and Ridgeway subdivisions highlights these comments.

An **effective groyne system** along the ABCA shoreline would have the following characteristics:

1) located in an area with a stable (non-eroding) lake bottom (i.e.: erosion resistant Rannoch till);

2) a continuous, and consistent, series of groynes (i.e.: uniform spacing and lengths);

3) elevation and length of groynes sufficient to retain beaches during periods of high water levels (alternatively, shore parallel structures, such as revetments, could provide the additional protection required during extreme conditions);

4) a sufficient supply of sand to maintain the beaches;

5) a sediment grain size sufficiently coarse to provide stable beaches during periods of high water (alternatively, beach nourishment could be placed following severe erosion events).

Point 1 clearly limits the application of groynes to areas which are not subject to significant long term erosion. Point 2 would require a community approach rather than an individual approach. Point 3 must consider the questions of aesthetics and risk. Higher and longer groynes would allow increased beach development (assuming a sufficient supply of suitable granular material), but would be major obstacles along the beach, and would cause increased downdrift impacts. Points 4 and 5 must be significant concerns for the future, as increasing shoreline protection will further restrict the already limited supply of littoral material along this shoreline, thus suggesting the need for artificial beach nourishment in the future.

5.3.2 Application Along ABCA Shoreline

Based on the preceding discussion, it can be concluded that the application of groynes along the ABCA shoreline should be restricted to areas which are stable or subject to only minor shoreline and bluff erosion (long term average recession rate < 0.3 m/yr). In addition, due to the limited supply of sediment along the ABCA shoreline, and in order to minimize downdrift impacts, any groyne construction must be accompanied by prefilling with a suitable granular material (clean sand and gravel, D50 > 0.3 mm). However, it is important to note that **MNR currently has a "no groyne" policy on the Great Lakes,** at least in part due to recent litigation related to groyne construction and downdrift impacts in the Cedar Banks/Shadeview/Ridgeway area.

5.3.3 Design Features

The design of a groyne system is relatively complex, and is perhaps closer to an art than a science. Detailed design is beyond the scope of this report; however, a number of general recommendations can be made, as presented in Figures 5.1 and 5.2 and summarized below:

• groyne length	30 m maximum
• groyne spacing	2 times groyne length
• groyne elevation	+ 2 m LWD maximum
• groyne construction	armour stone recommended steel sheet pile acceptable
• beach fill	clean granular fill (D50 > 0.3 mm)
• supplemental protection (if required)	rubblemound revetment recommended retaining wall acceptable

The design of a groyne field should be undertaken on a site specific basis by a qualified coastal engineer. Additional details which may require attention include the potential for outflanking of the groynes, the potential for damage to the groynes due to wave forces, ice forces and soil loading conditions, and the potential for downdrift impacts, which may lead to permitting difficulties and mitigation requirements.

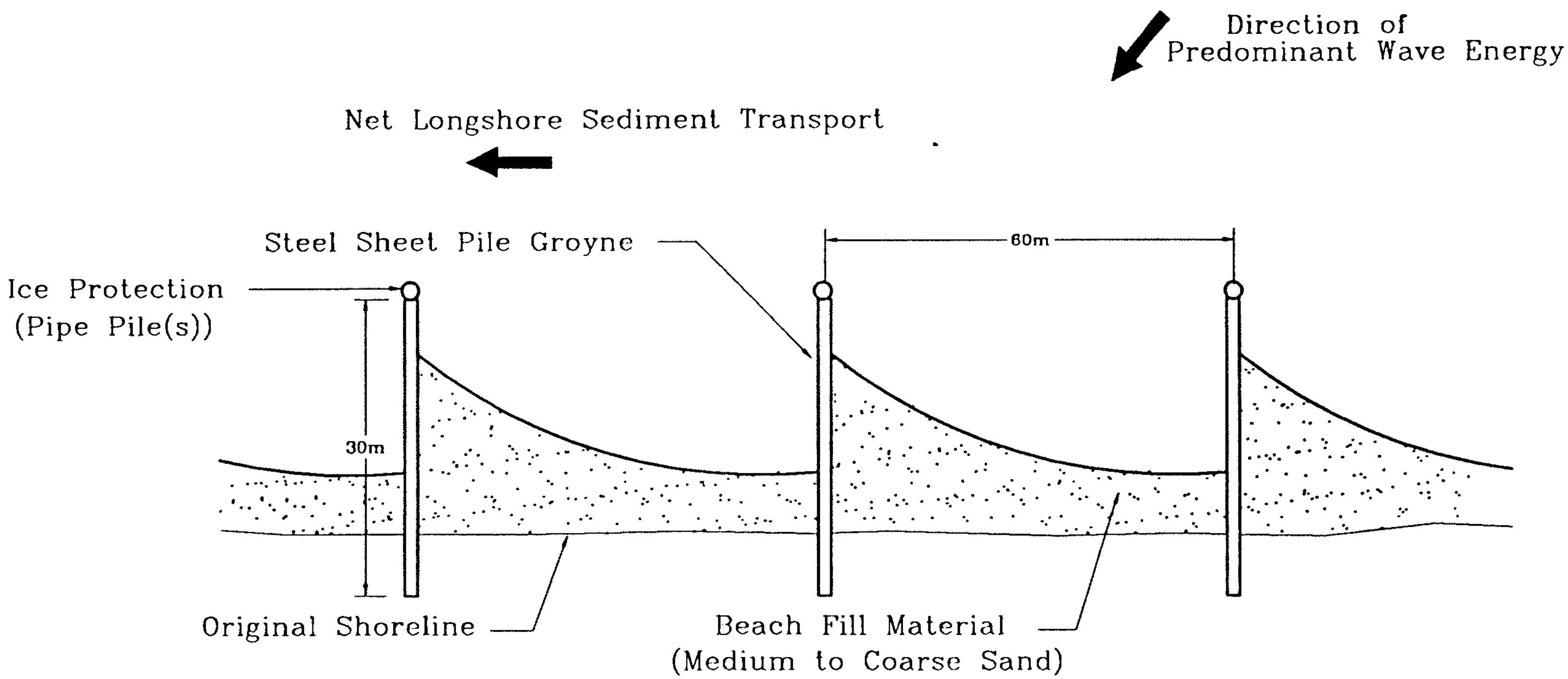

Figure 5.1

PLAN VIEW OF TYPICAL GROYNE FIELD

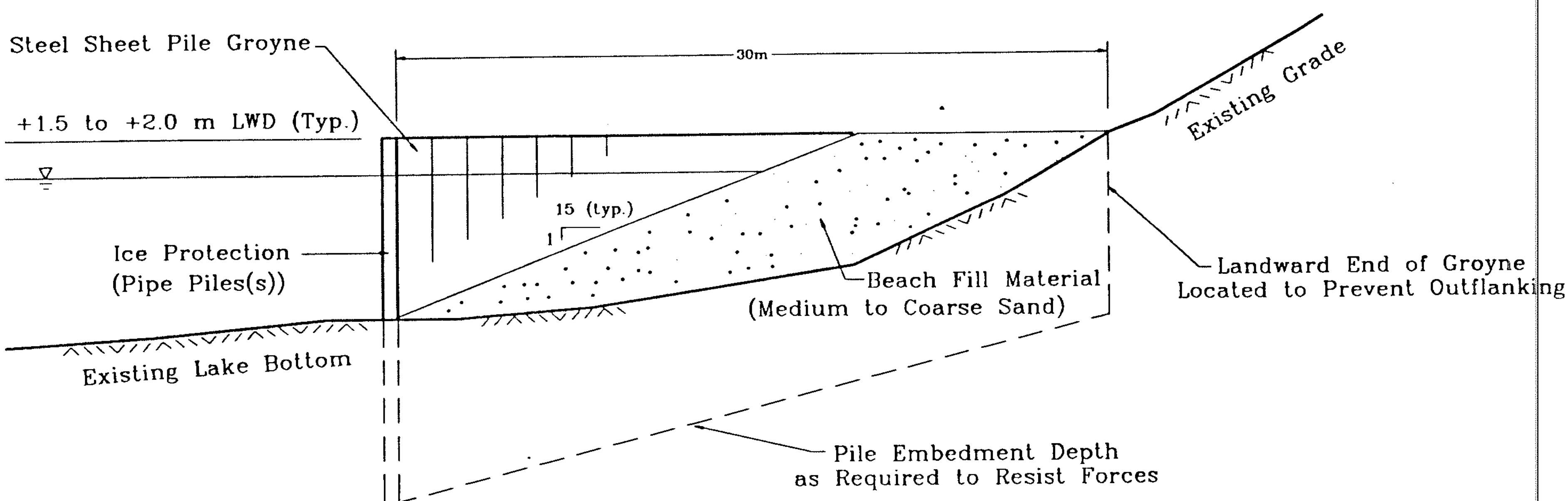

Figure 5.2

CROSS-SECTION OF TYPICAL GROYNE

More detailed information on the use of groynes for shore protection is presented in Philpott (1986) and CIRIA (1990). The Philpott reference deals specifically with Great Lakes shorelines, while the CIRIA reference provides an excellent overview of the subject and design guidance for the preparation of detailed designs.

5.4 Revetments

5.4.1 Discussion

Revetments are sloped shore parallel structures with a protective layer of large "armour" stones that are built to prevent the direct attack of waves on the toe of a bluff or a sand dune. These structures rely on the mass of the armour stones to withstand the forces of the waves. As waves impact the structure, energy is dissipated as the water moves over the rough, permeable sloped face of the structure, and through the voids between the armour stones. The land behind the structure is thus protected from the erosional stress that results from wave attack.

Armour stone revetments have advantages over many other forms of shore protection, because they can be designed to provide full protection to the bluff under any conditions encountered on Lake Huron. The use of larger armour stones and/or a higher crest elevation will provide a stable structure which protects the backshore under more severe conditions. This type of structure can also be designed to accommodate the ongoing erosion of the lake bottom, thus providing long term protection to the backshore. However, this will have a significant impact on the capital construction cost, although annual maintenance costs will be reduced. Finally, revetments are MNR's preferred method of shore protection, as they are considered to have a reduced impact on the shoreline environment relative to other alternatives, and may actually enhance fish habitat in some areas.

However, revetments, like any other shore protection structure, have a number of disadvantages that make them inappropriate for some conditions. Unlike groynes, revetments may severely limit access to the beach and water, and do nothing to increase the amount of recreation space. Beach or water access must often be provided by staircases or ramps located intermittently along the shoreline. Another disadvantage of revetments is that the structure does not encourage beach development, and may in fact increase the rate of erosion in front of the structure. This results from wave energy that is reflected from the structure, which increases the erosional stress and causes scour in front of the structure. If the lake bottom erodes, higher waves may be able to reach the structure, further eroding the bottom and possibly undermining the structure.

Finally, armour stone revetments may be relatively expensive compared to other shore protection structures, depending on the exposure of the site, the selected design life of the structure, and the availability of suitable quarried stone material. In this area, there is no local quarry to supply large armour stone for shoreline protection projects (the closest suitable quarries are in Ingersoll and Owen Sound), so the material must be trucked a considerable distance, which results in higher costs. In addition, access to the shoreline for large construction equipment is limited and difficult over much of this area.

5.4.2 Application Along ABCA Shoreline

Based on the preceding discussion, it can be concluded that a revetment structure can be used to protect the shoreline from long term erosion, even in areas where the nearshore lake bottom is eroding and the shoreline and backshore is subject to moderate to severe erosion. Application of this approach in such areas requires careful consideration of the lake bottom erosion, as discussed in the following section. A revetment can also be used to protect shorelines from storm wave runup damage, and can be designed to provide this protection even under extreme conditions. However, a revetment will not provide any recreational benefit to the shoreline, and may in fact reduce access and result in a reduction of existing beach deposits in front of it. Finally, it is noted that revetments are MNR's preferred method of shore protection due to their reduced impact on the shoreline environment compared to other alternatives.

5.4.3 Design Features

The key design features of a revetment are the armour stone size (which must be sufficient to resist the depth limited waves which reach the structure), the crest elevation (which controls the level of runup and overtopping, and thus the potential for damage to the backshore), the toe elevation (which must consider scour of loose sediments in front of the structure as well as the long term erosion of the nearshore lake bottom), and the filter layer (which prevents the loss of fine materials behind the revetment through the armour layer).

Revetments built along the ABCA shoreline may use different sizes of armour stones, depending on the design life of the structure and the value of the property being protected. For example, the revetment structure recently constructed along the Southcott Pines subdivision is protected by 3 to 4 tonne armour stones (estimated weight). The design of this structure does not appear to have considered the potential for future erosion of the nearshore lake bottom in front of the structure; this is likely common practice along this shoreline.

The crest elevation of a revetment structure will greatly affect its performance in high water and/or severe wave conditions. A higher structure is less prone to overtopping by waves, meaning that the area behind the structure is more protected. If excessive overtopping occurs, damage to the structure may result as the back of the structure is eroded, or damage to the protected property may result. Wave runup and overtopping levels on a sloping structure may be estimated using a number of approaches, as summarized by Atria (1991). Selecting the appropriate crest elevation is generally undertaken by comparing the cost of different crest heights with the associated risk. If the need for a high crest is established but is not desirable, other alternatives may be possible, such as increasing the armour thickness or providing a splash berm or apron.

Revetments must be designed such that scour (erosion) of loose material which may exist directly in front of the structure will not undermine the structure. Scour is eliminated as a potential failure mechanism through the use of "toe protection" or digging the structure deep enough into the sand to provide the necessary support after scour has occurred. The design of scour protection should be considered carefully and carried out by a qualified coastal engineer. A related issue is the long term erosion of the nearshore lake bottom; the impact of this process on the revetment design is discussed later in this section.

Another important consideration in the design of a revetment is the design of the filter layer between the armour stone and the natural material or backfill over which the structure will be constructed. The filter layer must ensure that any fine material beneath the structure is not washed out through the large voids that exist in the armour layer. This is done through the use of various layers of smaller rock and possibly a geotextile filter fabric.

Finally, as noted earlier, a revetment structure can be designed to accommodate the effects of erosion of the nearshore lake bottom. To illustrate the impact of this process on the magnitude and cost of revetment structures, preliminary designs have been prepared for revetments with design lives of 5, 25 and 100 years assuming construction at the location of the existing shoreline. Nearshore downcutting was estimated assuming a typical nearshore profile and a bluff recession rate of 1 m/yr, as discussed in Section 3.2. Cross-sections for the three structures are shown in Figures 5.3, 5.4 and 5.5, while design details and cost estimates are summarized in Table 5.1.

Table 5.1

Revetment Design Details and Cost Estimates
(typical nearshore profile, bluff recession rate = 1 m/yr)

	Design Life (years)		
	5	25	100
Existing Water Depth (m LWD)	0.0	0.0	0.0
Design Water Level (m LWD)	+1.4	+1.7	+1.9
Nearshore Erosion (m)	0.1	0.5	1.4
Total Design Depth (m)	1.5	2.2	3.3
Design Wave Height (m)	1.2	1.7	2.7
Armour Stone Size (t)	0.15 to 0.25	0.4 to 0.7	1.7 to 2.8
Crest Elevation (m LWD)	+2.6	+3.4	+4.6
Toe Elevation (m LWD)	-0.5	-1.1	-2.3
Estimated Cost per metre ($/m)	$700	$1,400	$3,700

Note: 0 LWD = 175.8 m IGLD

Clearly, the impact of nearshore erosion on the design of a revetment is significant if one intends to provide long term protection to a shoreline subject to significant erosion.

The preliminary designs presented above are based on standard procedures presented in the Shore Protection Manual (USACOE, 1977), and do not consider site specific details nor the availability of suitable quarried stone materials. The cost estimates, in 1992 dollars, are based on recent experience with similar structures in this area. Numerous design alternatives do exist which could lead to significant cost savings. However, these are

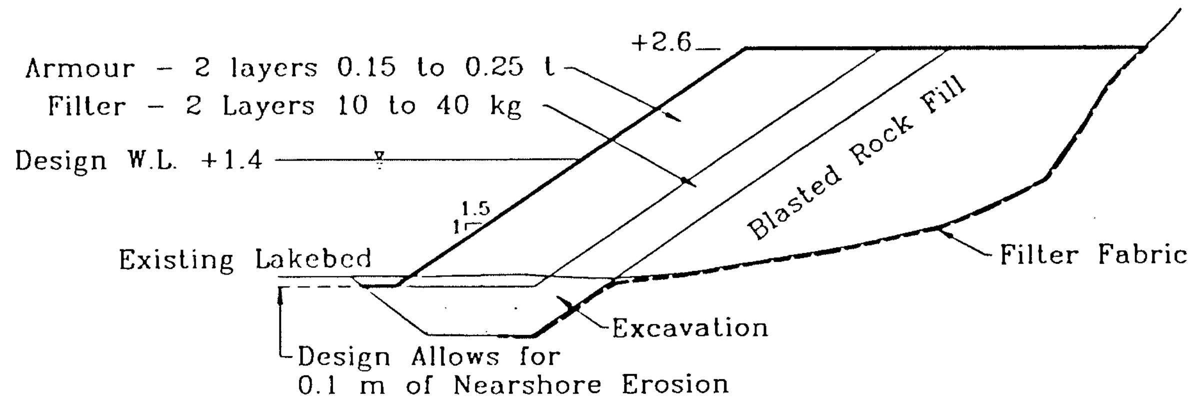

Figure 5.3

PRELIMINARY DESIGN FOR SHORELINE REVETMENT
(5 Year Design Life)

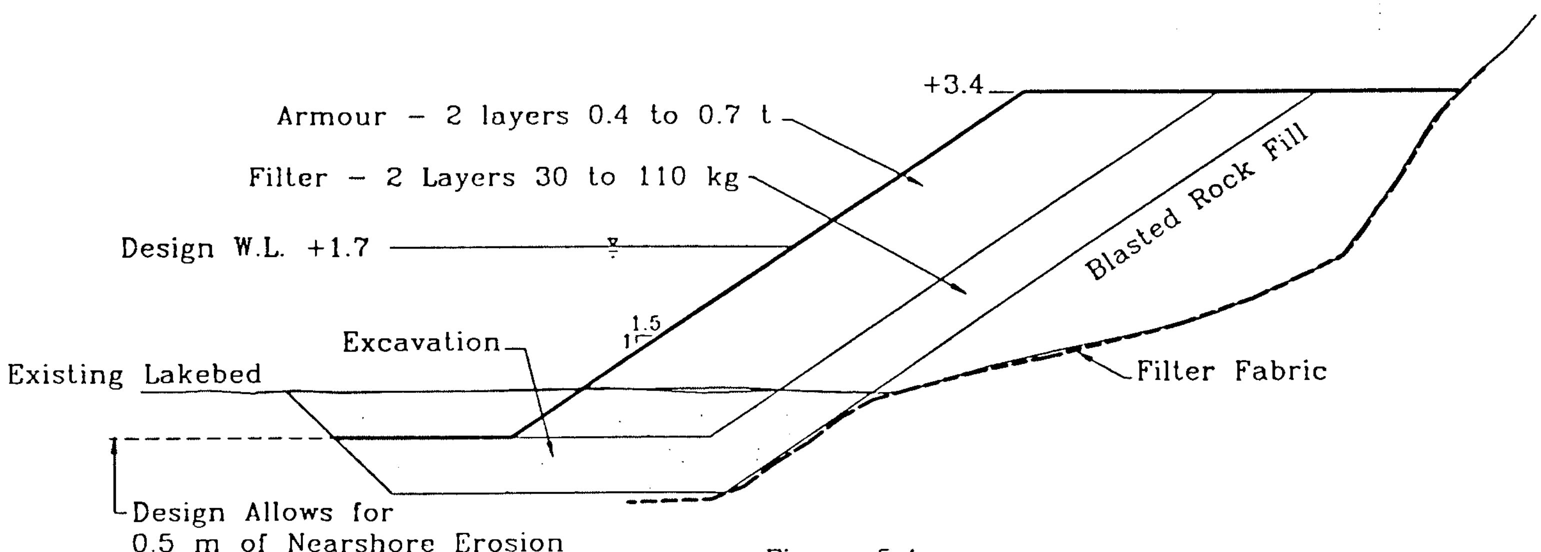

Figure 5.4

PRELIMINARY DESIGN FOR SHORELINE REVETMENT
(25 Year Design Life)

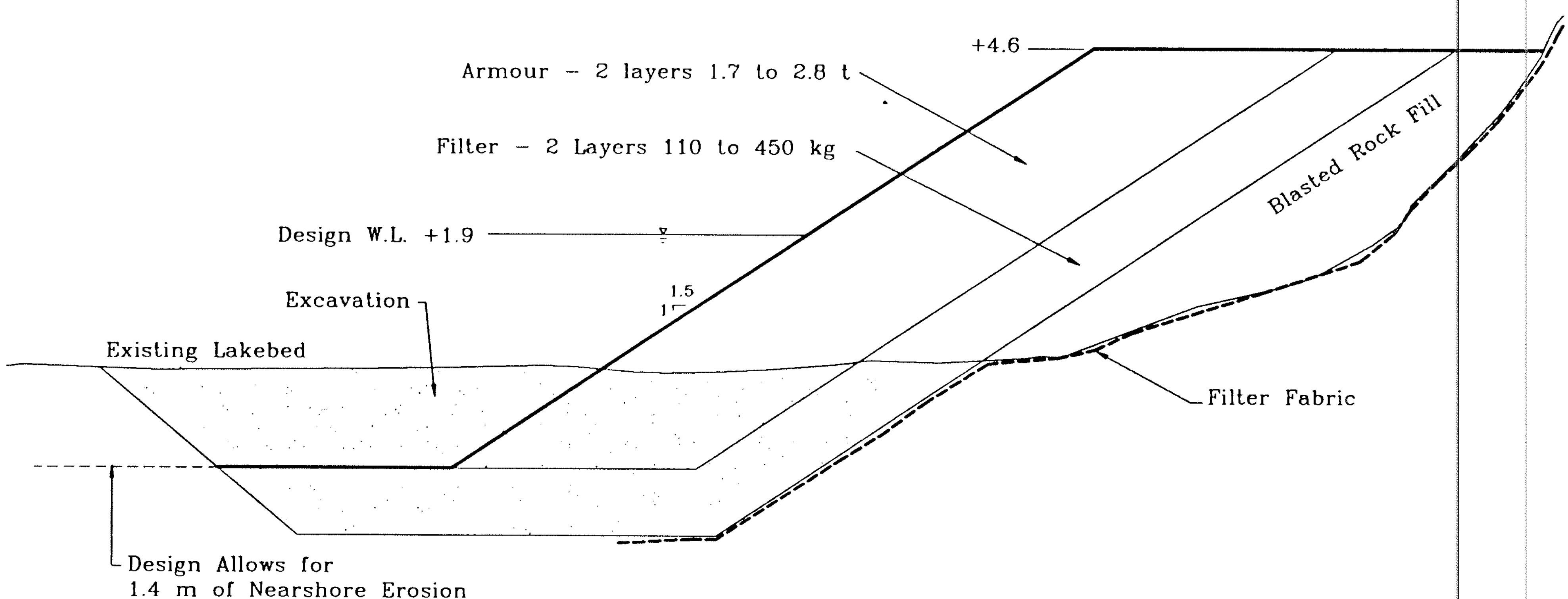

Figure 5.5

PRELIMINARY DESIGN FOR SHORELINE REVETMENT
(100 Year Design Life)

beyond the scope of the present study, but should be considered by a qualified coastal engineer during final design development for shoreline protection at any specific site.

5.5 Seawalls

5.5.1 Discussion

Seawalls are vertical, sloped, curved or stepped shore parallel walls that function in a very similar manner to a revetment. They are typically made of steel sheet piles or concrete (precast or cast in place), and are placed to protect the toe of a bluff or dune from wave attack. Wave energy is primarily reflected back into the lake, as opposed to revetments which typically dissipate a large percentage of the wave energy on their porous slope.

Most property owners consider seawalls to be more aesthetically pleasing than revetments for a number of reasons. For example, seawalls allow people to be closer to the water and/or beach than a stone slope. It is also easier to incorporate stairs or ramps for access to the water. Seawalls also require less width than a revetment, possibly making construction feasible in some areas with a steep backshore where a sloped structure might require large amounts of earth moving.

However, seawalls are generally less stable than revetments, and have a shorter life. In addition, seawalls, due to their steep (often vertical), impermeable and generally smooth face, cause more wave reflection, resulting in increased erosion in front of the structure and more problems with scour and undermining at the toe of the structure. Because of this, seawalls may fail catastrophically if proper design is not used. Seawalls also require higher crests than revetments if overtopping is to be prevented. In addition, it is noted that MNR prefers revetments over seawalls, as seawalls are considered to create a sterile nearshore environment, while revetments may actually enhance the shoreline environment (with respect to fish habitat).

Finally, the cost of a seawall may be less than or greater than that for a revetment depending on the site conditions, design conditions and required design life of the structure. Large seawalls can be very complicated to build, requiring anchoring of the walls to prevent overturning and/or very deep penetration depths for pile structures.

5.5.2 Application to ABCA Shoreline

Based on the preceding discussion, in particular the possibility of increased nearshore erosion due to wave reflections and MNR's preference for revetments, it is recommended that revetments be constructed rather than seawalls. This recommendation applies to both long term erosion protection, as well as to storm wave runup protection, throughout the length of the ABCA shoreline.

Retaining walls (which are similar to seawalls, but are located on land rather than in the water) may be utilized for protection against storm wave runup as long as they are located sufficiently landward of the shoreline such that they are not exposed to direct wave attack, and only to wave runup during extreme conditions. The design of such retaining walls is beyond the scope of this study; detailed information on these structures may be found in MNR (1986) and USACOE (1977, 1981).

5.6 Beach Nourishment

5.6.1 Discussion

Beach nourishment refers to supplementing the naturally occurring supply of sand to the shoreline by importing suitable material from other sources. This approach may be applied locally if suitable containment structures (groynes or breakwaters) exist, or regionally (within a littoral cell or subcell). This section focuses on regional beach nourishment, as local beach nourishment is discussed in sections which deal specifically with groynes and offshore breakwaters.

The primary advantage of regional beach nourishment is that it enhances the naturally occurring shoreline processes by increasing the supply of sand to a "sand starved" environment. Beach widths would increase, with corresponding benefits in terms of shoreline protection and recreational aspects.

However, regional beach nourishment has a number of practical disadvantages. It requires an extensive supply of suitable granular material; this may be difficult to secure, particularly over the long term. Nourishment must be conducted on a regular basis (every few years), and may be required at numerous locations along an extended length of shoreline in order to obtain its benefits within a reasonable length of time, as the natural shoreline processes would take many years to transport the sand over tens of kilometres. Finally, the level of protection provided to the shoreline may not be sufficient to fully protect some sections of the shoreline, specifically where deep water exists in the nearshore area, which is typical of shorelines subject to moderate to severe erosion.

5.6.2 Application to ABCA Shoreline

Two alternatives are available for regional beach nourishment along the ABCA shoreline. The first would be to bypass sand across Goderich harbour. This would restore the supply of sand along the ABCA shoreline to that which naturally occurred prior to the construction of Goderich harbour in 1916. Bypassing at Goderich could be provided by a permanent sand bypassing system (such as a sand fluidization and pumping process) or by mechanical excavation and transport by barge or truck. In the first case, the sand would be deposited immediately downdrift of the harbour, and it would take many years for the benefits to be achieved along the full ABCA shoreline. In the second case, the excavated material could be distributed at selected locations along the shoreline, thus reducing the time required for the benefits to spread along the shoreline.

The second alternative would be to import suitable granular material (D50 >0.3 mm) from an inland source such as a sand and gravel pit, or perhaps from the extensive dune deposits in the Pinery/Ipperwash area. This latter approach could be considered as a unique "recycling" program, as material originally deposited in the Pinery/Ipperwash "sink" area would be put back into the system at the updrift "source" area. It might be difficult to secure an inland source of suitable material, particularly over the long term, and it seems unlikely that excavation in the Pinery/Ipperwash area would be permitted.

Clearly, regional beach nourishment would require the cooperation of the local, provincial and federal governments, and it seems unlikely that such a scheme could occur in the near future. However, as discussed elsewhere in Section 5, local beach nourishment should be considered at locations where containment structures (such as groynes and offshore breakwaters) are present or proposed in order to provide improved shore protection and recreational beaches.

5.7 Offshore Breakwaters

5.7.1 Discussion

Offshore breakwaters may be used to provide protection to an eroding shoreline. These structures are generally of rubblemound construction, with armour stone placed over a rock filter layer and blasted rock core. As such, they are similar to a revetment, but are typically constructed in 2 to 4 m of water, and thus require at least limited armour protection on the rear slope. Offshore breakwaters protect the shoreline from direct wave attack (although, depending on the design requirements, they may allow some wave overtopping and transmission through the structure), thus reducing the erosional stress on the shoreline. On shorelines where there is a sufficient supply of alongshore transport, deposition of sand in the lee of the breakwater(s) (the wave "shadow" zone behind the structure) may result in a wider beach in this area. Alternatively, a series of offshore breakwaters may be utilized to contain imported beach fill, thus providing shoreline protection with significant recreational benefits. Both approaches have been utilized on the Great Lakes. An example of a beach fill contained by offshore breakwaters on Lake Michigan is presented in Figure 5.6.

One of the advantages of a series of offshore breakwaters is that they can be designed to protect shorelines which are subject to significant erosion. As the structures are located a certain distance offshore, they will actually protect a portion of the nearshore lake bottom from further erosion, although their design must consider erosion of the lake bottom which may occur lakeward of the breakwater. However, the magnitude of lake bottom erosion is lower in this area than in the immediate vicinity of the shoreline, so it may be easier to incorporate in the design than for a shoreline revetment. Another advantage of an offshore breakwater system is that it can be used to retain beach fill, thus limiting sand losses from the beach area (both alongshore and offshore) and providing a beach of improved stability.

The primary disadvantages of offshore breakwater systems are high cost, the requirement for detailed design investigations, and the relatively difficult construction, which may require large marine-based equipment. In addition, these structures may result in adverse downdrift impacts due to interference with the natural alongshore transport processes.

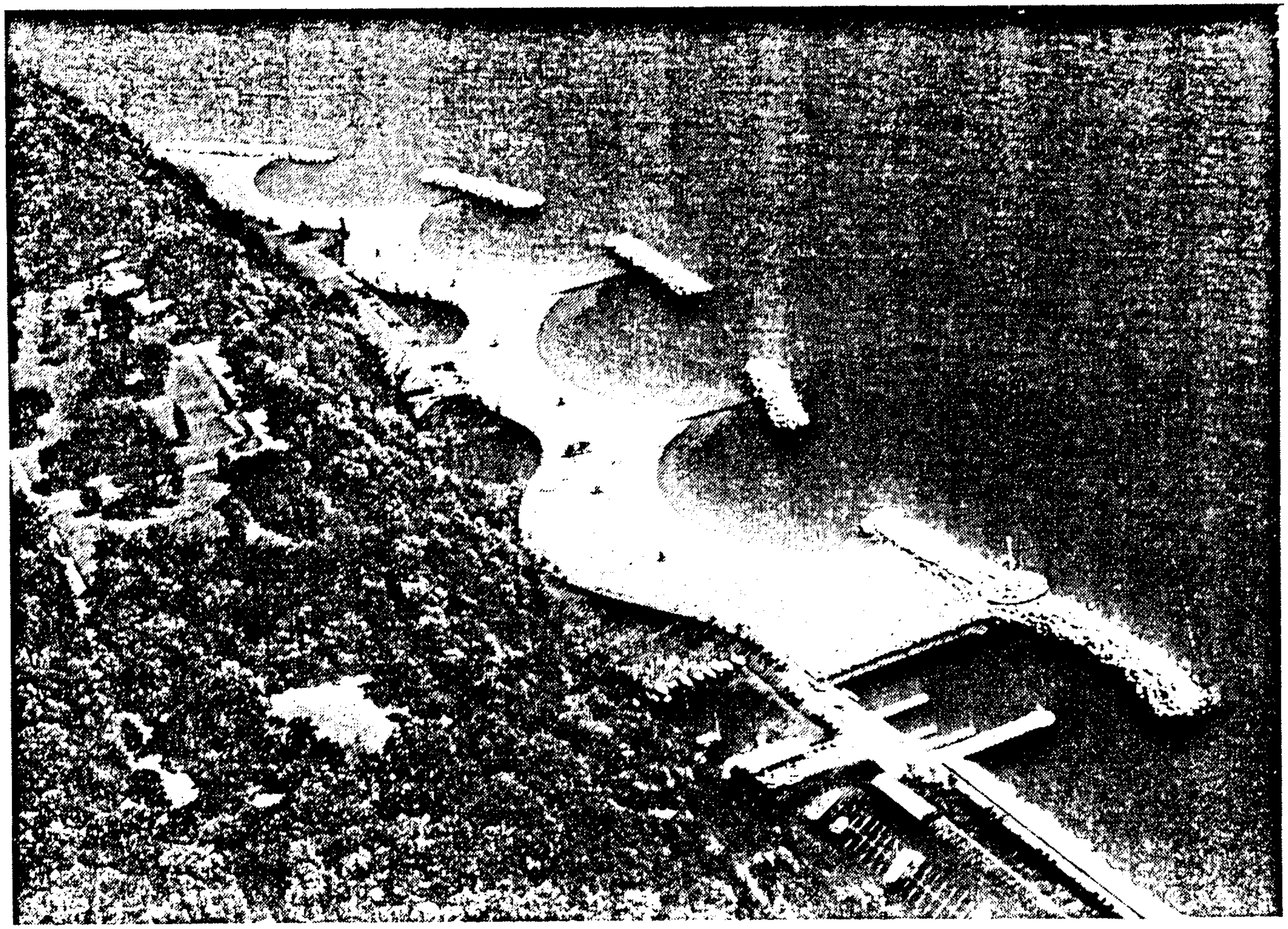

Figure 5.6

Offshore Breakwater/Beach Fill Project on Lake Michigan

5.7.2 Application To ABCA Shoreline

An offshore breakwater system could be considered as an alternative to a revetment structure to protect shorelines which are subject to moderate to severe erosion. This approach has the added benefit of permitting the development of a stable recreational beach, which is not possible with a revetment. However, the high cost of such an approach necessitates a community approach. For example, it might be in the public interest for the Village of Bayfield to acquire riparian rights to the shoreline to the south of the harbour in order to construct an offshore breakwater system in this area. This would protect the eroding shoreline to the south of the harbour, and beach fill could be placed to provide a large public beach. Downdrift impacts associated with such a development at this particular site would be limited due to the location of the site immediately south of the existing harbour and within the zone of influence of the existing harbour structures.

5.8 Improvements to Existing Structures

As discussed in Section 5.2, existing shore protection within the jurisdiction of the ABCA shoreline generally consists of groynes and seawalls which provide some protection against damage due to storm wave runup, but only limited protection (if any) against long term erosion. Methods to improve the performance of the existing structures in reducing shore damage are discussed briefly below.

Artificial beach nourishment should be considered in areas where groynes already exist and are in good repair. This would provide improved recreational beaches, increased protection to the backshore area, and reduced downdrift impacts. The beach fill should consist of a medium to coarse sand ($D_{50} > 0.3$ mm).

In areas where artificial beach nourishment is not feasible, for example where deep water exists immediately adjacent to a seawall, it is suggested that an armour stone revetment should be considered. Replacement of existing seawalls with armour stone revetments will reduce wave runup and overtopping onto the backshore, as well as reducing wave reflections and the associated erosional stress on the nearshore lake bottom.

6.0 IMPLEMENTATION

Previous sections of this report have discussed shoreline issues and the related objectives of shore protection, as well as design considerations and criteria, and alternative methods of shore protection for the ABCA shoreline. The following section discusses the various issues associated with the implementation of a shore protection project along the ABCA shoreline.

6.1 Community Approach

As noted earlier, a coordinated approach to shoreline protection by a community or subdivision, as opposed to an individual property by property approach, has a number of important advantages. For example, works planned and constructed along an extended section of shoreline will provide more effective protection than shorter individual works. In addition, overall construction (and design) costs are reduced through a coordinated approach, and maintenance work will be easier to undertake and less expensive than for a series of isolated projects. Finally, a coordinated effort may improve the opportunities for government financing, and may also assist during the permit and approval phase. For these reasons, a community approach to shoreline protection is strongly recommended wherever possible along the ABCA shoreline.

6.2 Ownership

The first step in the design of any shoreline structure should be to establish the ownership of the land on which the structure is to be built. The owner should not assume, without supporting evidence, that the lot extends to the waterline or into the lake. The legal definition of the lakeward limit of waterfront lots varies within the jurisdiction of the ABCA. There are examples along the ABCA shoreline where lots extend to a defined line landward of the top of the bluff, or to the top of the bluff, or to the waterline, or finally, to a defined line lakeward of the top of the bluff.

Thus, prior to any design effort, the property owner should obtain a copy of the registered survey/deed for the property. If the lot limits are unclear, this matter should be discussed with a lawyer having experience with lakefront ownership issues.

6.3 Preparation of a Final Design

As noted earlier, the designs presented in this report are preliminary designs and should not be used for construction. The designs are based on limited information, and assume typical site and design conditions for the ABCA shoreline. In addition, the cost estimates are approximate only, and have been based on recent experience with similar projects in southwestern Ontario.

The design of structures located above the 100 year flood level (+1.9 m LWD south of Dewey Point, and +1.8 m north of Dewey Point) that are intended to provide protection from wave run-up may be prepared with the assistance of the various publications available, for example, MNR (1986), USACOE (1978, 1981). Useful assistance may be obtained from a professional engineer with experience in coastal engineering.

The design of structures extending below the 100 year flood level and/or that are intended to stabilize the shoreline against continuing erosion, particularly if the structures extend into the water, should be prepared by a professional engineer with experience and qualifications in coastal engineering.

The development of shore protection designs should be compatible with the ABCA shoreline management plan. The development of specific designs will, typically, include the following activities:

(i) Discussions between the owner and engineer concerning:

- historical changes to the beach, shoreline and bluff,

- primary objectives of the structure (stabilize shoreline, protection from wave run-up),

- secondary objectives of the structure (walkways, access, boating, beach recreation).

(ii) Site inspection. This may include:

- characteristics of the bluff,
- characteristics of the shoreline,
- depth of sand in the beach,
- type of material underneath the beach (till, gravel, bed rock),
- offshore extent of beach,
- characteristics of lake bottom where beach finishes,
- surveyed profiles across the beach face and into 1 to 2 m of water, or further for larger projects,
- soil borings (larger projects).

(iii) Desk studies. These may include:

- establish historical bluff recession rates (from ABCA files),
- estimate historical downcutting of nearshore lake bottom (refer to Section 3.2 of this report),
- establish design conditions for water levels and waves (refer to Section 3.0 of this report),
- prepare conceptual designs (refer to Section 5.0 of this report),

- assessment of impacts on coastal processes and adjacent shoreline,

- bluff slope stability analysis, and drainage issues.

(iv) Discussion of conceptual designs between owner and engineer.

(v) Prepare final design, considering the following:
- availability and cost of materials,
- access to the site,
- construction methodology,
- impacts on shoreline and coastal processes.

(vi) Apply for permits.

(vii) Obtain bids from contractors and select most suitable contractor. Prepare an agreement.

(viii) Supervise construction.

(ix) Monitor performance of completed structure.

6.4 Permits and Approvals

It is recommended that the approval of the ABCA be required prior to constructing any structures within the Regulatory Shoreline, as defined in the draft MNR policy. This includes any form of development (agricultural, seasonal or permanent residential, commercial, or industrial) as well as both shoreline protection and bluff stabilization works. A review and approval by other government agencies may also be required, as discussed later.

assessment of the impacts of the proposed structure on the adjacent shoreline. This submission should address the following issues:

- site location,
- site description, including environmentally significant features,
- coastal conditions, design parameters, and sand transport,
- description of the need for and details of the proposed works,
- design calculations,
- construction schedule,
- access and maintenance requirements,
- impact on sand transport, the nearshore environment and adjacent properties,
- monitoring program.

Further, the impact assessment should demonstrate the following key points:

- the proposed works will not increase the long term shoreline recession rate at adjacent properties,
- the proposed works will not adversely affect alongshore sand transport rates,
- the proposed works will not adversely affect adjacent structures,
- the proposed works will not adversely affect the environment.

Upon receipt of the impact assessment, the ABCA will circulate it to all relevant approval agencies, as well as to updrift and downdrift property owners within 150 m of the property in question, in order to solicit their comments, concerns or participation. The ABCA would then develop a coordinated response to the application, specifically allowing the work to proceed as proposed or with specified modifications, or not at all.

As noted earlier, approvals by other agencies may be required depending on the nature and magnitude of the proposed works. These are summarized in Table 6.1, as reproduced from MNR (1986).

Table 6.1

Potential Approvals Required

Activity	Agency	Legislation	Who Needs to Apply	Description
• Construction on Crown Land	MNR	Public Lands Act	Municipalities and private landowners	- no structure or other matter may be situated on crown lands without approval.
• Construction in Lakes and Rivers	MNR	Lakes and Rivers Improvement Act	Municipalities and private landowners	- permit is required for construction of any structure in or along any stream, river or lake. - this includes protection works on the beach and in the water.
Removing sand and gravel	MNR	The Aggregate Resources Act	Cons. Auths. municipalities and private landowners	- regulates the removal of sand and gravel from beaches and under the waters of any lake, river or stream. - intended to prevent and minimize erosion of beach property.
• Fill in Floodplain	Cons. Auth.	The Conservation Authorities Act	Municipalities and private landowners	- controls placement of fill in regulated floodplains.

Table 6.1 cont'd

Potential Approvals Required

Activity	Agency	Legislation	Who Needs to Apply	Description
• Construction in Floodplain	Cons. Auth.	The Conservation Authorities Act	Municipalities and private landowners	- controls construction in regulated flood-plains to prevent loss of life or property.
• Construction in a Navigable Water	Trans-port Canada	Navigable Waters Protection Act	Province, Cons. Auth. municipalities, and private landowners	- controls construction in navigable waters. - exemptions are usually obtained for protection works.
• Construction in Lakes and Rivers	DFO	Fisheries Act	Province, C.A.'s, municipalities and private landowners	- "no net loss" policy for fish habitat. - construction may require compensation or mitigation.
Placement of materials in lakes and rivers	MOE	Water Resources Act	Cons. Auth., municipalities and private landowners	- no permit required prior to construction but MOE can stop work if they judge the work to adversely affect water quality.
Environmental Assessment (Class EA)	MOE	Environmental Assessment Act	Cons. Auth., MNR and municipalities	- environmental screening of projects dealing with shore protection.
Environmental Assessment (Individual EA)	MOE	Environmental Assessment Act	Cons. Auth., MNR and municipalities	- environmental impact assessment for projects of larger size (i.e. over $2 million in Dec. 1977 dollars) and of potential significant impact.
Construction over any public shore, bay, harbour, river or water	Munic-pality	Municipal Act	Private Landowners	- approval for construction over public shores and water, if municipality passes by-law.
Building Permit	Munic-pality	Municipal Act	Private Landowners	- required where retaining walls are constructed.

• Normal approvals required by individual Landowners

Of particular relevance is the Public Lands Act (MNR), which requires approvals for all works extending lakeward of the normal shoreline. The following is quoted from MNR's policy on water lots:

"9. Authorization for new or existing works which extend beyond the normal shoreline (e.g., groynes, off-shore breakwaters, beaches, sills, etc.) shall be subject to the alternative requirements listed below. This is because such works usually have a significant effect on shore processes - causing littoral drift for example - to the detriment of neighbouring landowners. Tenure for such works may issue only if:

(a) The applicant obtains and submits written concurrence from all landowners within 500 feet (150 m) along the shore.

or (b) the applicant provides, at his expense, an engineer's report and/or a biologist's report which indicates that the works will cause no adverse effects;

or (c) The District Manager Holds a hearing, to which the applicant and all potentially affected landowners are invited, and the hearing results in a favourable consensus;

or (d) The applicant, where a series of works would achieve the desired result with minimum adverse effects, organizes the neighbours to undertake simultaneous construction of the requisite number of shoreline works. (MNR would deal with the proposal as a "package" but tenure would be granted to the individual owners in front of whose property each work was being built.);

or (e) The municipality becomes involved and takes responsibility for co-ordinating the installation and control of protection works along a given stretch of shoreline. In such case, it would be advisable to have the municipality enter into a Beach Management Agreement with MNR.

10. Where an existing occupancy cannot be authorized because it fails to substantially comply with the requirements of this policy and the occupant

refuses or neglects to take reasonable corrective action, or the occupant, being not entitled to "free use", refuses or neglects to take out authority, removal of the improvement or structure may be undertaken, in accordance with Policy & Procedure LM7.06.01, "Control of Unauthorized Improvements".

Removal with support of local municipality, should be considered where the improvement or structure:

(i) is located in Crown land in front of someone else's property and it is concluded that the normal use and activities of the other owner(s) are adversely affected;

(ii) is of a size substantially larger than that required for the current purpose of use;

(iii) has an adverse impact on the programs of this Ministry;

(iv) is in conflict with the current land use pattern of the area;

(v) is detrimental to the normal pursuits of other users of the waterway;

(vi) other valid reasons."

6.5 Financing

There are very few sources of funding for either private landowners or the municipality to complete shore protection projects. Private landowners can apply to the Shoreline Property Assistance Act (administered by the Ministry of Municipal Affairs and Housing and the local municipality) and the Local Improvement Act (administered by the local municipality), while the municipality can apply to the Parks Assistance Act (administered by the Ministry of Tourism and Recreation) and the Conservation Authorities Act, (administered by the local Conservation Authority).

6.6 Construction

Although construction can, in some cases, be undertaken by the landowner, in general it should be completed by a contractor with related experience in shoreline construction. Landowners would be well-advised to meet and discuss the project with several qualified contractors, and to obtain written quotes from each of them based on the final designs, plans and specifications for the work. Prior to selecting a contractor, it would also be beneficial to examine past performance on similar projects, identified by a list of references provided by the contractor. Based on all of this information, the landowner can make an informed selection of the best contractor for the job. It is advisable that a formal, signed agreement be completed with the contractor prior to undertaking any construction.

Depending on the nature and magnitude of the project, it may also be advisable to provide on-site inspection of the work as it proceeds. This might involve part or full-time observation by the landowner, and/or specific site visits by a qualified engineer, preferably the project designer. Quality control during construction is an essential component of a successful project, and should not be overlooked. Construction which does not meet the project specifications may not achieve the level of performance intended by the original design, and could result in costly damages and maintenance /repair requirements.

6.7 Monitoring and Maintenance

An essential component of any shoreline protection project is an on-going monitoring and maintenance program. A visual inspection of structures should be completed by a qualified individual on an annual basis, and following severe storms, such that potential problems can be identified and addressed before excessive and unrepairable damage occurs. In order to maintain the performance of the structure according to its original design intent, maintenance and repairs should be undertaken as soon as possible after a potential problem area is identified.

It is also recommended that property owners monitor the bluff and shoreline on a regular basis. The resulting information will be of great value when a structure is to be designed. Surveys may consist of measurements of the top of the bluff, bottom of the bluff and beach relative to fixed features. A photographic record with photographs taken from a similar position and including fixed features in the field of view would also be useful. Surveys and photographs should be taken on a regular basis, possibly in the spring and fall of each year and following severe storms.

Specific conclusions for the selection, design and implementation of shore protection structures along the ABCA shoreline are presented in the following section.

7.0 RECOMMENDATIONS

Recommendations addressing shore protection along the ABCA shoreline have been developed. One objective of these recommendations has been to balance the desire to maintain (and enhance if possible) the existing sand beaches along the shoreline (which requires maintaining the source of sand from eroding bluffs, the longshore transport of sand to the south, and the deposition of sand in the Grand Bend/Pinery/Ipperwash beach system) with the increasing pressure for shoreline protection. A second objective has been to develop specific recommendations with respect to the selection, design and implementation of shore protection structures along the ABCA shoreline. These recommendations are summarized below. The recommendations address structures that are intended to stabilize the shoreline in areas that are eroding (i.e. erosion protection), as opposed to structures built along a relatively stable shoreline that are intended to protect a building or the bluff from wave run-up during periods of high water levels (i.e. wave damage protection).

7.1 Prevention versus Protection

- Wherever possible along the ABCA shoreline, the use of development setbacks, the relocation of existing buildings, and the acquisition of shoreline property by public organizations (i.e. the townships, municipalities and ABCA) should be utilized rather than the construction of shore protection structures. For new development, the application of this concept is relatively simple, and requires that no new development be constructed within the 100 year erosion hazard zone. For existing development, the application of this concept is more complicated (refer to SMP Section 3.3 - Policy).

- Eliminating shore protection structures allows the bluffs to continue to erode and provides sand to the shoreline.

7.2 Protection Alternatives

- From a theoretical perspective, regional beach nourishment would be a desirable protection alternative with respect to maintaining/enhancing coastal processes. However, from a practical perspective, it is unlikely that a regional beach nourishment scheme could be implemented in the foreseeable future. A nourishment scheme would involve placing in the order of 30,000 to 60,000 cubic metres of sand on the shoreline each year.

- In areas subject to moderate to severe long term erosion (average erosion rate > 0.3 m/yr), an engineered rubblemound revetment is the recommended erosion protection structure. The design of any revetment should consider the erosion (or downcutting) of the nearshore lake bottom. Groynes are not recommended in areas subject to moderate to severe long term erosion.

- In areas subject to minor long term erosion (< 0.3 m/yr), revetments are recommended for erosion protection, but groynes may also be considered. However, groynes must be prefilled with suitable beach fill (clean sand and gravel, $D_{50} > 0.3$ mm) in order to minimize downdrift impacts. It should be noted that groynes will not provide full protection to the shoreline during extreme conditions (severe storms at high water levels). Also, it must be noted that MNR currently has a "no groyne" policy on the Great Lakes, and is unlikely to issue a permit for new groyne structures in the foreseeable future.

- Offshore breakwaters containing imported beach fill should be considered by the Village of Bayfield for the area to the south of the harbour. This type of approach is relatively expensive, but can provide significant recreational benefits as well as effective erosion protection. This approach is not recommended elsewhere along the ABCA shoreline due to potential adverse impacts on the longshore transport of sand. The potential impacts of such a project located immediately south of Bayfield harbour would be limited due to the presence of the harbour structures.

- Reflective seawalls, such as steel sheet pile walls, are not recommended for erosion protection anywhere along the ABCA shoreline.

- Any number of structures may be considered for wave damage protection. These include revetments and groynes, as discussed above, and retaining walls of various construction (gabion baskets, steel sheet piling, concrete). Retaining walls should be constructed behind the active beach zone (i.e. not exposed to direct wave action, and only exposed to wave runup during extreme conditions (storms at high water levels) in order to minimize impacts on the beach and shoreline processes. Information on the many different types of structures designed to resist wave runup are available in other reports and are not discussed in this document (refer to MNR (1986) and USACOE (1978, 1981).

- With respect to improving the performance of existing shore protection structures, beach nourishment should be considered in areas where there are groynes which are in good repair but are not full with sand. Consideration should also be given to replacing reflective seawalls with rubblemound revetments.

7.3 Implementation

- A co-ordinated approach (by community or subdivision) is recommended.

- Prior to design, the ownership of the land on which the structure is to be built should be clearly established.

- The design of structures located above the 100 year flood level that are intended to provide protection from wave runup and storm damage should follow guidance presented in MNR (1986) and/or USACOE (1978, 1981).

- The design of structures which extend below the 100 year flood level and/or that are intended to stabilize the shoreline against continuing erosion should be prepared by a professional engineer with experience and qualifications in coastal engineering.

- Any application to construct shore protection structures must be accompanied by a detailed description of the site and proposed work, and an impact assessment which demonstrates the following points:

- the proposed works will not increase the long term shoreline erosion rate at adjacent properties,

- the proposed works will not adversely affect longshore sand transport rates,

- the proposed works will not adversely affect adjacent structures, and

- the proposed works will not adversely affect the environment.

• Any application for shore protection, including the impact assessment, should be circulated to all property owners within 150 m of the property boundaries in question to solicit their written comments prior to the ABCA responding to the applicant.

• Quality control during construction is an essential component of a successful project, and suitable construction observation services should be provided.

• Monitoring of completed projects should be completed annually, and following severe storms, such that potential problems can be identified before excessive and unrepairable damage occurs.

REFERENCES

ABCA, "Inventory of Erosion Control Structures on Lake Huron", October 1990.

Atria Engineering Hydraulics Inc., "Wave Uprush - Great Lakes and St. Lawrence River Shorelines: Guidelines for Acceptable Practice", report prepared for Conservation Authorities and Water Management Branch, Ontario MNR, March 1991.

Butler & Associates (1985) Ltd., "Shoreline Erosion Control Programme", report prepared for Southcott Pines Rate Payers Association, June 1986.

CIRIA (Construction Industry Research and Information Association), "Guide on the Use of Groynes in Coastal Engineering", Report RP119, 1990.

Davidson-Arnott R.G.D., "Erosion of the Nearshore Profile In Till: Rates, Controls and Implications for Shoreline Protection", Proceedings of Symposium on Cohesive Shores, Burlington, Ontario NRC ACROSES, May 1986.

Environment Canada, Inland Waters Directorate, Water Resources Branch, "Historical Water Levels Summary, Ontario, to 1987", 1988.

Environment Canada, Inland Waters Directorate, "Monthly Water Level Bulletin - Great Lakes and Montreal Harbour", September 1991.

F.J. Reinders & Associates, "Lake Huron Shoreline Processes Study", report prepared for ABCA, MVCA, SCRCA and SVCA, December 1989.

Goda, Y., Random Seas and Design of Maritime Structures, University of Tokyo Press, 1985.

Goda, Y., "A Synthesis of Breaker Indices", Transactions of the Japanese Society of Civil Engineers, Vol. 2, Part 2, pp 227-230, 1970.

Goda, Y. "Irregular Wave Deformation in the Surf Zone", Coastal Engineering Japan, Vol. 18, pp 13-26, 1975.

Golder Associates, "Preliminary Study and Evaluation - Lake Huron Erosion", report prepared for Southcott Pines Ltd., December 1972.

Great Lakes Environmental Research Laboratory, "Great Lakes Storm Surge Planning Program (SSPP)", NOAA Technical Memorandum ERL GLERL-65, August 1987.

Great Lakes Basin Commission, "The Role of Vegetation in Shoreline Management", undated.

Hubertz, J., "A Wind-Wave Hindcast for the Great Lakes, 1956-87", Proceedings of 2nd International Workshop on Wave Hindcasting and Forecasting, Vancouver, B.C., April 1989.

Kamphuis, J.W., "Improving Shore Protection Design", Canadian Journal of Civil Engineering, Vol. 17, No.2. pp. 142-147, April 1990.

MNR, "How to Protect Your Shore Property", 1986.

MNR, "Wave Climate Database Catalogue", May 1988.

MNR, "Great Lakes Flood Levels and Water Related Hazards", February 1989.

MNR, "Great Lakes System - Flood Levels and Water Related Hazards", February 1989.

MNR, "Great Lakes - St. Lawrence River, Flood and Erosion Policy Statement" (draft), September 1990.

Nairn, R.B. and Baird & Associates, Erosion Processes Evaluation Paper, final report prepared for the International Joint Commission, June 1992.

Peever, D. Personal Communication, 1991.

Philpott Consulting Ltd., "Guidelines for Groyne Installations in the Great Lakes:, report prepared for MNR, November 1986.

Sandwell Inc., "Report on Coastal Processes and the Impact of the Beachplace Development/Trillium Beach Estates Development on the Beach at Grand Bend", report prepared for Chatham District MNR, March 1990.

U.S. Army Corps of Engineers Waterways Experiment Station, Shore Protection Manual, Third Edition, 1977.

U.S. Army Corps of Engineers Waterways Experiment Station, Shore Protection Manual, Fourth Edition, 1984.

U.S. Army Corps of Engineers North Central Division, "Help Yourself", September 1978.

U.S. Army Corps of Engineers, "Low Cost Shore Protection", 1981.

U.S. Army Corps of Engineers, "Revised Report on Great Lakes Open-Coast Flood Levels", report prepared for the Federal Emergency Management Agency, February 1988.

U.S. Army Corps of Engineers, ACES (Automated Coastal Engineering System), Version 1.05, April 1991.

APPENDIX A

Summary of Shoreline Reaches Between Goderich and Kettle Point

(from Reinders, 1989)

Reach: **G. Goderich (Maitland River) to Goderich Water Treatment Plant**

Length: 1.7 km

Description:

Nearshore: Bedrock controls wave action at the shoreline.

Shoreline: Sand beach at the north in the lee of the harbour structures. A sand/cobble beach has been built at Christopher Beach using dredge spoil. The shoreline has been protected at the water treatment plant.

Bluff:
- Bluffs are now protected and slope stability is the principal consideration.
- Recession rate is 0.00 m/year.

Source of Sand:

Bluff: None

Lake Bottom: 1,030 cubic metres/year

Creeks & Rivers: None

Gullies: None

Sand Losses: None

Sand Transport: Minor due to control imposed by man-made structure.

Structures and Shore Protection:
- Shoreline completely protected.

Shoreline Management Recommendations:
- Study to be undertaken of feasibility of nourishing the shoreline with volume of sand equivalent to that supplied to the shoreline if the bluffs were allowed to erode.
- Study bypassing sand from north side of harbour past this reach (see notes for previous reach).

References: Bishop (1987), Boyd et al (1986), Etmanski and Scroth (1979), Etmanski and Scroth (19890), Golder Associates (1979), Golder Associates (1984), MacLaren (1979), Reinders (1984).

Reach: **H. Goderich Water Treatment Plant to Bayfield**

Length: 16.4 km

Description:

Nearshore: Relatively deep water with some shallower shelves.

Shoreline: Some small beaches.

Bluff:
- Eroding where not protected by structures or beach.
- Recession rates range from 0 - 0.6 m/year.

Source of Sand:

Bluff: 13,140 cubic metres/year

Lake Bottom: 2,780 cubic metres/year

Creeks & Rivers: 100 cubic metres/year (Bayfield River)

Gullies: 4,060 cubic metres/year

Sand Losses: Minor wind blown losses at beach north of Bayfield Harbour. Sand blown into marina and channel.

Sand Transport: North to south, controlled by supply of sand, depth of water and shoreline orientation. Net transport into north end of reach 1,030 cubic metres/yr. Net transport from south end of reach 21,110 cubic metres/yr. Sand transport forced into offshore sand bar by Bayfield harbour structures.

Structures and Shore Protection:
- 10% of shoreline protected.
- Bayfield harbour structures extend approximately 100 m into the lake and have produced a beach, sand now bypasses this beach with no further lakeward accretion occurring.

Shoreline Management Recommendations:
- Erosion of bluff provides sand to shoreline to the south.
- In general, shoreline development or shore protection not recommended. Establish setbacks based on shoreline recession and slope stability.
- Some development adjacent to headlands and nearshore shelf areas can be considered and will be defined by recession setback.

References: Boyd et al (1986), Etmanski and Scroth (1979), Etmanski and Scroth (1980), Golder Associates (1979), MacLaren (1979), Ross (1976).

Reach:	**I. Bayfield to Highway #83**
Length:	27 km
Description:	
Nearshore:	Relatively deep water with some shallower shelves.
Shoreline:	Small, frequently persistent beaches, depending on depth of water.
Bluff:	- Eroding when not protected by structures or beach. - Recession rates range from 0 - 0.7 m/year.
Sources of Sand:	
Bluff:	32,570 cubic metres/year.
Lake Bottom:	7,410 cubic metres/year
Creeks & Rivers:	420 cubic metres/year
Gullies:	4,210 cubic metres/year
Sand Losses:	Minor
Sand Transport:	North to south, controlled by supply of sand, depth of water and shoreline orientation. Net transport into north end of reach, 21,110 cubic metres/yr. Net transport from south end of reach 65,714 cubic metres/yr.
Structures and Shore Protection:	- 22% of shoreline protected. Approximately half of groynes within this reach are short groynes (short groynes do not extend significantly past the end of the beach). - Bayfield harbour structures have detrimental effect on shoreline immediately to the south. Sand is forced to the offshore sand bar before returning to the shoreline.
Shoreline Management Recommendations:	- Erosion of bluffs provides sand to shoreline to the south. - Shoreline development or protection not recommended. - Establish setbacks based on recession. - Consider shore protection south of Bayfield structures where sand transport has been forced offshore.
References:	Quigley et al., (1974)

Reach:	**J. Highway #83 to Kettle Point Lighthouse**
Length:	27.8 km
Nearshore:	Sand, extension of beach.
Shoreline:	Fully developed sand beach.
Bluff:	- None, backshore is extensive sand dunes to the south. - Where bluff erosion occurs, recession rate is 0.15 m/year.
Sources of Sand:	
Bluff:	300 cubic metres/year
Lake Bottom:	100 cubic metres/year
Creeks & Rivers:	1,550 cubic metres/year (Ausable River)
Gullies:	110 cubic metres/year
Sand Losses:	Gross potential for wind blown losses to backshore dunes is estimated to be in the order of 90,000 cubic metres/year (see Appendix C). Depending on vegetation on foredunes, actual sand loss from beach system may be significantly less.
Sand Transport:	Potential sand transport becomes very low because of shoreline orientation. Net transport becomes zero at some point along the beach system. Beach may be stable with supply of sand from north equal to wind blown losses to backshore dunes.
Structures and Shore Protection:	Where protection exists, seawalls are quite common. - Jetty at mouth of Ausable River at Grand Bend extends approximately 100 m into the lake and has created a wide fillet beach. - 12% of shoreline protected from high water erosion of dune.
Shoreline Management Recommendations:	- This is an active shoreline with low net transport. - Shoreline development should consider setback based on highwater erosion and flooding and wind blown movement of sand into the system of dunes. - Sustainable development may be considered. - Particular attention should be given to protecting the dune system and associated vegetation adjacent to the beaches.
References:	Alexander (1982), Baird and MacIntosh (1983), Fisher et al. (1987), Hall et al. (1983), Hall et al. (1983a).

APPENDIX B

Nearshore Lake Bottom Erosion

Summary of Methodology

The process of nearshore lake bottom erosion involves a landward shift of the nearshore profile at the same rate as bluff recession in the area, with the nearshore profile retaining its original shape (Nairn and Baird & Associates, 1992). Thus, in order to estimate the long term erosion of the nearshore lake bottom, a methodology was developed to relate the lake bottom erosion (D) to the shape of the nearshore profile, the average annual bluff recession rate (R) and the time period of interest (t), as illustrated in Figure B.1.

Initially, a nearshore profile with a general shape defined by the equation $y = ax^m + bx + c$ was assumed, where x is the distance offshore from the shoreline and y is the water depth below an assumed datum. The constants a, b, c and m must be evaluated for a particular site using information on water depths and lake bottom slopes at different distances offshore. For example, a typical nearshore profile along the ABCA shoreline has zero depth and a 1:20 slope at the shoreline, and a 6 m depth and 1:500 slope at 1000 m offshore. Using this information (obtained from CHS chart 2260 and CHS field sheet 8089), the site specific profile equation was found to be $y = -0.0235\ x^{1.091} + 0.05x$.

This equation represents the existing profile at time t = 0. In order to account for the future erosion of this profile, it is assumed that the profile shifts landward at the bluff recession rate, R. Thus, after t years, the horizontal shift would be Rt. The future profile after any time, t, can be estimated by the transformed equation $y = -0.0235\ (x - Rt)^{1.091} + 0.05\ (x - Rt)$. The lowering of the lake bottom at any location, x, can now be estimated by the difference in depths, y, at present (t = 0) and any time, t, in the future for any specified bluff recession rate R, For example, Table B.1 illustrates the deepening (erosion) of the nearshore lake bottom as a function of the quantity Rt and the offshore distance x for the profile described above.

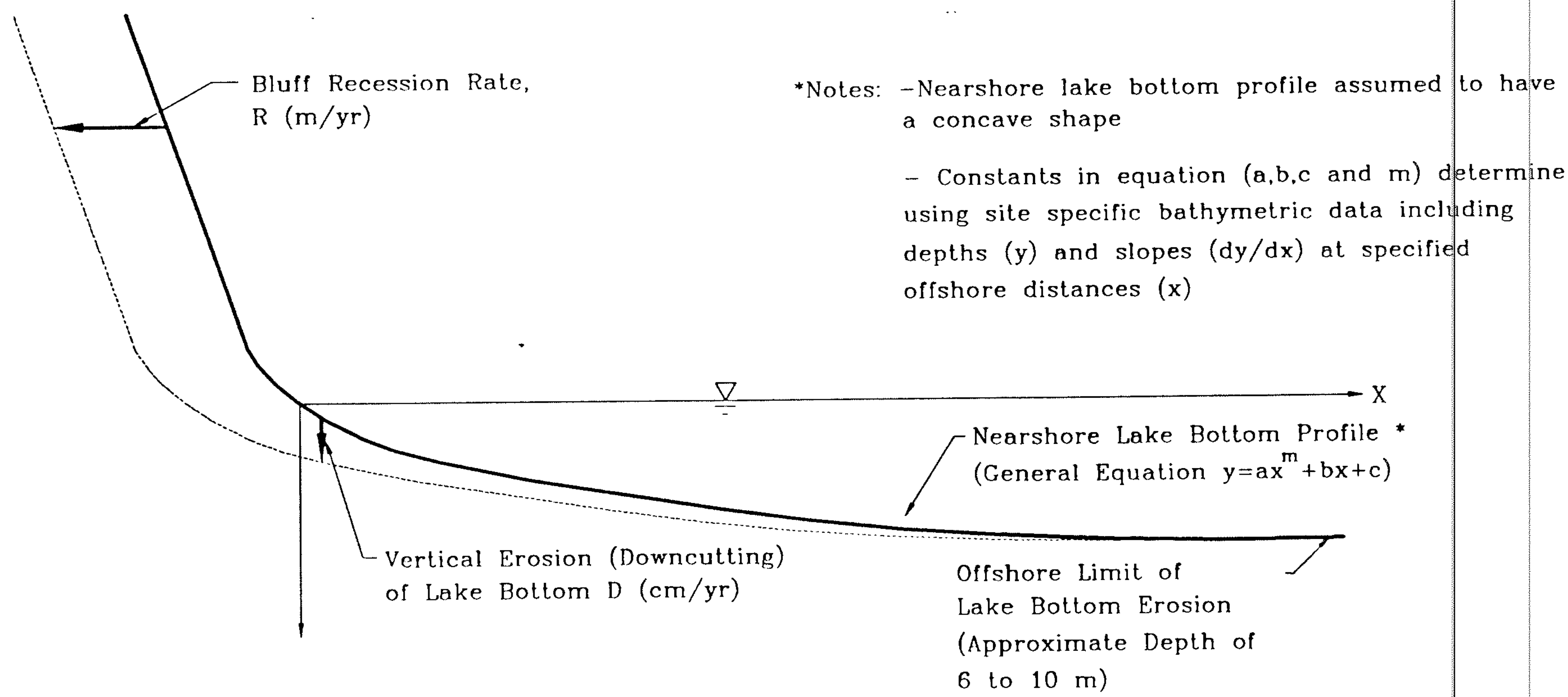

Figure B.1

SCHEMATIC DIAGRAM OF BLUFF
AND NEARSHORE LAKE BOTTOM EROSION

Table B.1

Erosion of the Nearshore Lake Bottom
for Typical Nearshore Profile

Offshore Distance x (m)	Existing Water Depth (m)	Future Water Depth (m) vs. Rt						
		Rt = 1	2	5	10	20	**50**	100
0	**0.00**	0.03	0.05	0.11	0.21	0.38	**0.82**	1.43
15	0.30	0.32	0.33	0.38	0.46	0.61	1.02	1.59
34	0.60	0.61	0.63	0.67	0.74	0.88	1.25	1.78
56	0.90	0.91	0.93	0.97	1.03	1.15	1.49	2.00
80	1.20	1.21	1.22	1.26	1.31	1.43	1.74	2.21
107	1.50	1.51	1.52	1.56	1.61	1.71	2.00	2.45

For example, assuming a bluff recession rate of 0.5 m/yr and a time span of 100 years (i.e. Rt = 50), the water depth at the present shoreline location will increase from 0 to 0.82 m over this period (refer to highlighted values in Table B.1). A similar increase in depth would occur with a bluff recession rate of 1.0 m/yr over a period of 50 years (or any other combination of R and t yielding Rt = 50).