PORT GLASGOW MARINA & YACHT CLUB
Entrance Feasibility Study

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EXECUTIVE SUMMARY

Port Glasgow Marina is situated on the north shore of Lake Erie between Pointe aux Pins and Port Stanley, east of Sixteen Mile Creek. Shoreplan Engineering Limited (Shoreplan) was retained by the Municipality of West Elgin to carry out a review of the coastal conditions and provide a feasibility assessment of the entrance improvements to the existing marina at Port Glasgow. It is our understanding that boaters frequently experience rough wave conditions within the entrance to the marina. This makes the use of the harbour, both for local boaters and visitors, difficult and sometimes dangerous.

The marina entrance consists of two steel sheet pile piers that extend 35 metres from the shoreline. The marina accommodates approximately 80 boats and provides two public launch ramps.

A wave climate was analysed using a hindcast procedure based on forty years of wind data recorded at London Airport. The analysis shows two directions with concentrated deepwater wave energy, one from the east and the other from the southwest. The nearshore wave energy distribution showed a convergence of energy with the southwesterly peak moving the most.

Three options were considered to resolve the boat basin entrance issues. All of the options share some common design features. In all cases the west pier remains in its present location as relocation of the pier could impact the mouth of Sixteen Mile Creek. The channel is also widened to twenty metres within each option.

Option 1, East and West Pier Extension, includes placing a rip rap revetment along the existing west pier, removing the existing east pier and replacing it with a rip rap revetment 20 metres east of the existing pier, and extending both the relocated east pier and existing west pier 50 metres offshore. Wave agitation at the entrance is reduced by replacing the reflective steel sheet pile wall with a sloped stone revetment which absorbs wave energy. The channel is widened to improve boat manoeuverability. The increased length of the channel reduces the wave heights at the launch ramp to 0.3 metres during the boating season. The estimated cost of this option is $2,260,000 including an allowance for design and construction contingency.

Option 2, West Pier Extension, includes placing a rip rap revetment along the east side of the existing west pier, removing the existing east pier, replacing the east pier with a rip rap/ armour stone revetment 20 metres to the east and extending a 30 metre long, easterly oriented armour stone breakwater off the west pie. Changing the orientation of the entrance protects the entrance channel from the predominate southerly waves. However the entrance remains vulnerable to less frequent easterly storms. Wave agitation at the entrance is further reduced by replacing the reflective steel sheet pile wall with a sloped stone revetment. The channel is widened, improving boat manoeuverability. The lining of the channel with rip rap reduces the wave conditions at the launch ramp to less than 0.4 metre wave height during the boating season. The estimated cost of this option is $1,520,000 including an allowance for design and construction contingency.

Option 3, Offshore Breakwater includes building a breakwater approximately 50 metres offshore of the existing piers. The existing “L” extension of the east pier would be removed and an armour stone revetment would be placed around the end of each pier. The breakwater would be aligned such that it would reduce the wave heights from the south by fifty percent and from the east by seventy percent. Wave agitation at the entrance would be reduced by the armour stone revetment at the ends of the pier. The channel width would remain the same except at the entrance where it would be wider due to the removal of the “L” extension. The wave climate at the launch ramp is reduced due to a smaller wave entering the channel, not due to energy losses within the channel as in options 1 and 2. The estimated cost of this option is $1,850,000 including an allowance for design and construction contingency.

Our preliminary conclusion is that option 2 is the preferred option because the capital cost is relatively low and the concept plan can be implemented in three phases. The first phase would consist of construction of the west pier extension and removal of the “L” extension from the south end of the east pier at an estimated cost of $560,000. The second phase would be the relocation of the east pier, which is estimated to cost $680,000. The third phase would be the lining of the east side of the west pier with rip rap, which is estimated to cost $280,000.

Preliminary discussion with staff of the Ministry of Natural Resources (MNR), Lower Thames Conservation Authority and Department of Fisheries and Oceans, did not identify any site specific environmental issues. Therefore, we believe that impacts on the environment (coastal and aquatic) can be mitigated. A new land use permit from MNR would be required for any modified entrance configuration.

The possibility of installing a walkway along the crest of the east breakwater (to maintain the fishing opportunity that now exists from the east pier) was investigated. The crest of the east pier could be modified with a concrete walkway supported on piles at an additional cost of approximately $200,000 including an allowance for design and construction contingency.

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1.0 INTRODUCTION

Shoreplan Engineering Limited (Shoreplan) was retained by the Municipality of West Elgin to carry out a review of the coastal conditions and provide a feasibility assessment of entrance improvements to the existing marina at Port Glasgow. It is our understanding that boaters frequently experience rough wave conditions within the entrance to the marina. This makes the use of the harbour, both for local boaters and visitors, difficult and often dangerous. The nature of wave action in the entrance channel is illustrated in the photograph presented in Figure 1.1 The purpose of this work was to determine the general nature of the works required to provide a safe entrance and the associated capital costs.

The existing marina at Port Glasgow is operated by Port Glasgow Marina & Yacht Club (PGMYC). The facility accommodates approximately 80 boats and provides two public launch ramps. An aerial photo of the site is presented in Figure 1.2.

The report is divided into four chapters. Chapter 1 provides an introduction to the study. Chapter 2 provides a description of the shoreline, both regional and local, and describes the coastal analysis carried out. Chapter 3 describes the concepts developed to improve the harbour entrance. Chapter 4 provides recommendations.

1.1 Brief History of the Site

The site has been used for access to Lake Erie since early settlement days. Copies of newspaper articles documenting past activities are presented in Appendix A. These articles were obtained from PGMYC records. They indicate swimming and boating activities at the site in the late 1800s. Boating activities were always present and associated with the mouth of Sixteen Mile Creek. It is reported that erosion destroyed a natural cove near the mouth of the creek leading to the construction of a boating facility at the present location around 1960. Port Glasgow Yacht Club was incorporated on June 3, 1963.

The harbour was expanded to its present form in 1995 with the assistance of the Federal and Provincial governments. The Port Glasgow Marina Yacht Club covered one third of the cost of the expanded facilities.

Figure 1.1 Photograph of West Pier

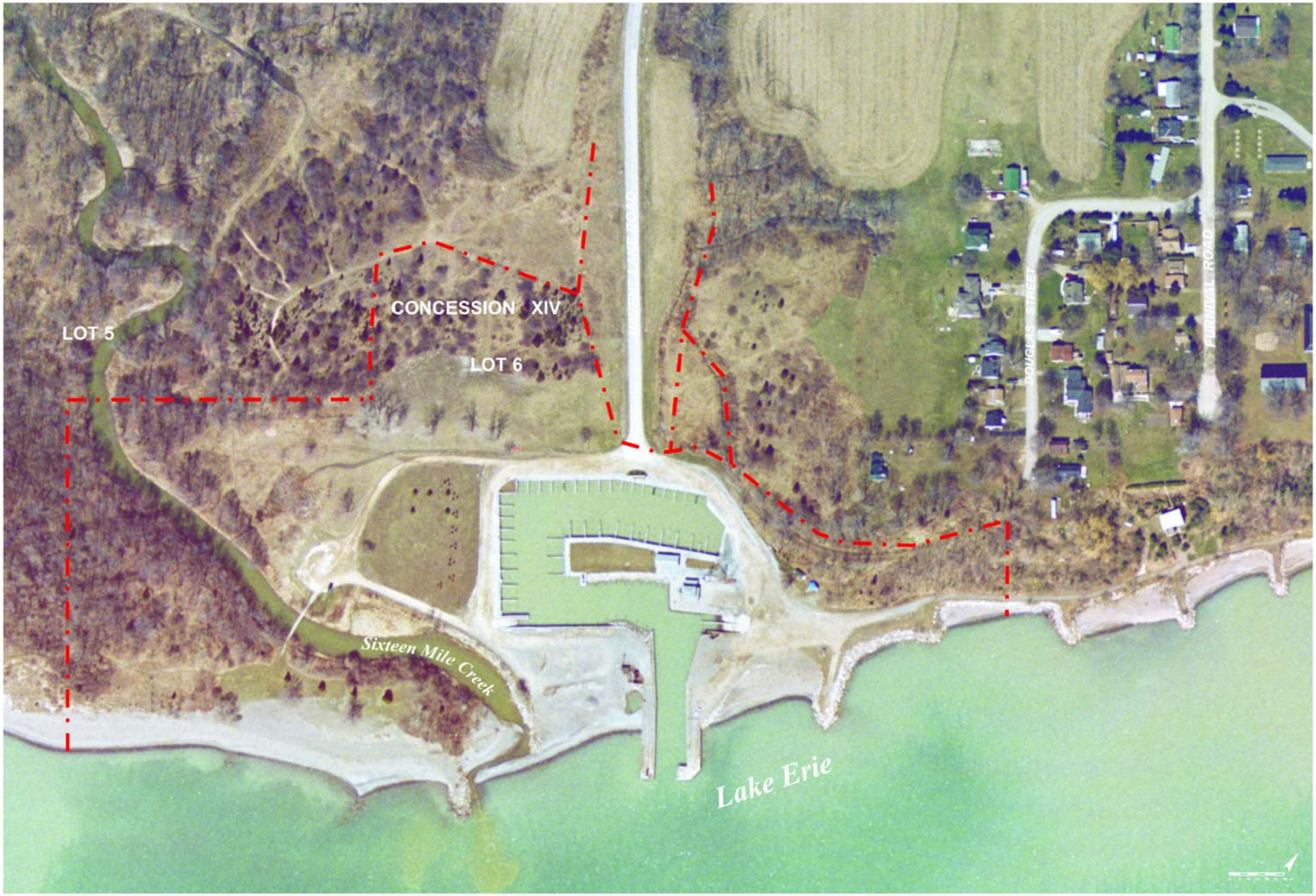

Figure 1.2
Aerial Photo of Site, Port Glasgow Marina & Yacht Club

2.0 SHORELINE DESCRIPTION

Port Glasgow is situated along the north shore of Lake Erie between Port Stanley and Pointe aux Pins. Figure 2.1 shows the location of Port Glasgow on Lake Erie. Based on our site observations and aerial photographs, the shoreline along this reach of Lake Erie could be characterized as a bluff and narrow beach. The Lower Thames Valley Conservation Authority (LTVCA) Shoreline Protection Concepts Study (Sandwell, 1993) describes the shoreline as beach/dune. Port Glasgow Marina is situated east of Sixteen Mile Creek.

The marina entrance consists of two piers extending approximately 35 metres into Lake Erie. Figure 2.2 shows the site plan for Port Glasgow Marina & Yacht Club. Photographs of the site are presented in Appendix B. The piers are approximately 85 m long along the channel and 6 m wide and were constructed with steel sheet pile (see Photos B-1). The top elevation of the piers is approximately 176.0 m, International Great Lakes Datum (I.G.L.D.), 1985. The entrance channel is approximately 18 m wide with a depth of approximately 2.0 m below chart datum. The south end of the east pier is "L" shaped and extends west by approximately 5 metres. This narrows the very south end of the channel to approximately 13 metres. At the north end of the entrance, the channel turns towards the west and widens into the boat basin. A rip rap revetment is located along the north facing shore in this area, reducing wave reflection and wave penetration into the basin (see Photo B-2). To the east of the north end of the channel is a public launch ramp (see Photo B-3). It is a double wide ramp with a concrete travel surface approximately 11 metres wide. It is reported that sand and gravel deposited on the ramp makes use of the ramp difficult, necessitating regular clean up.

Sixteen Mile Creek enters Lake Erie west of the marina entrance between the west pier and a field stone groyne located approximately one hundred metres west. A second smaller groyne is located another fifty metres to the west and anchors a long sandy beach (see Photos B-4 and B-5). A sand bar extended across the mouth of the creek at the time of our site visit. It is expected that conditions at the mouth of the creek vary considerably depending on the water levels and wave conditions. We understand that the bay between the west pier and the large groyne is dredged on a regular basis under a permit issued by the Ministry of Natural Resources. The removal of the sand and gravel at this location minimizes the need for dredging of the entrance channel.

The shoreline east of the marina entrance is protected with a groyne field (see Photo B-6). Seven rubble groynes were constructed in the past. We understand that a concrete rubble road connects these groynes. The system has maintained a stable shoreline for several decades.

The LTVCA report states that the shoreline structures have resulted in beach accretion adjacent to the harbour and west of Sixteen Mile Creek. This report also states that installation of the harbour pier has resulted in the formation of a protective beach updrift. The report recommends maintenance of the pier structure and the two updrift rock groynes which protect the Creek entrance.

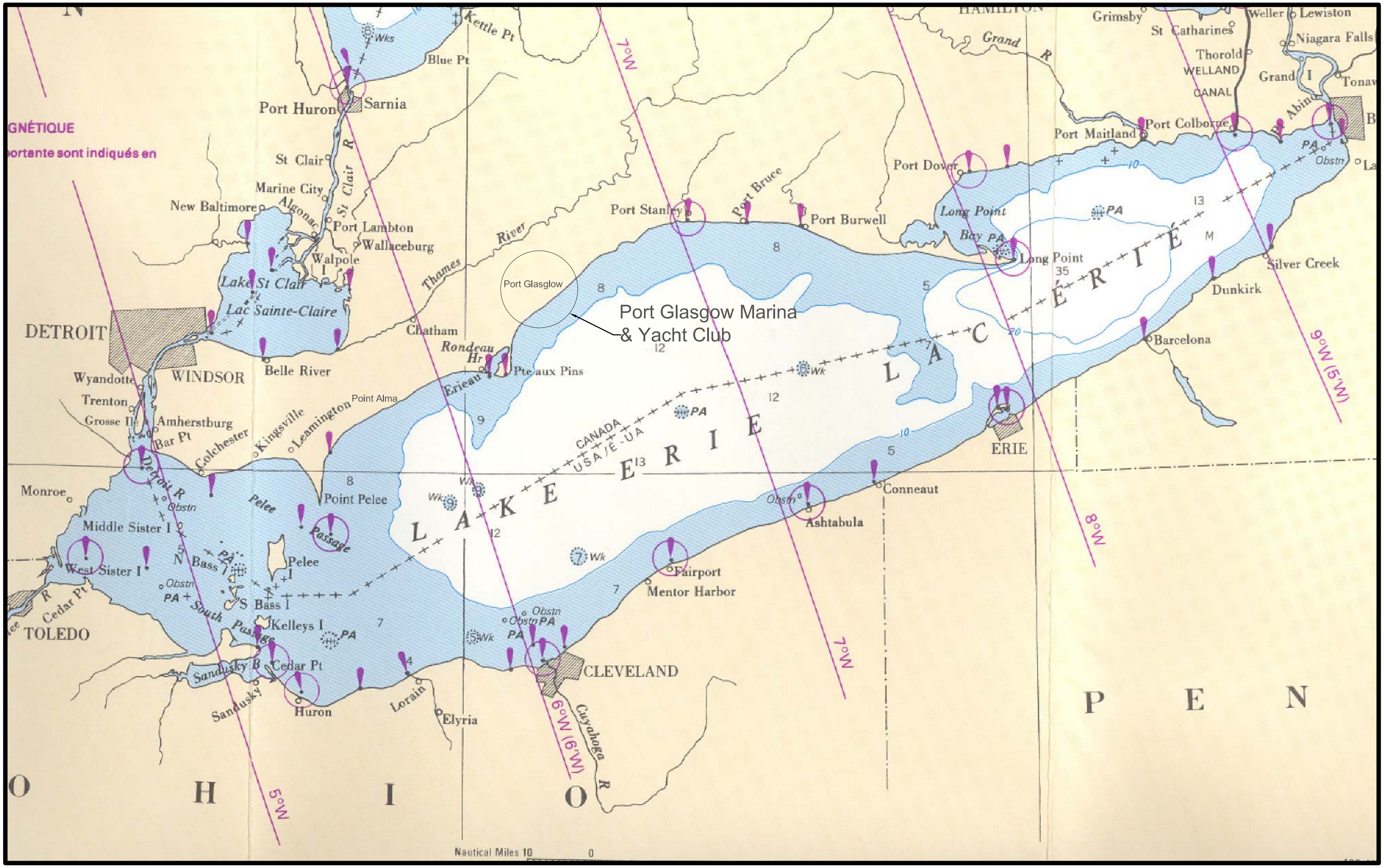

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Figure 2.1

Location Plan, Port Glasgow

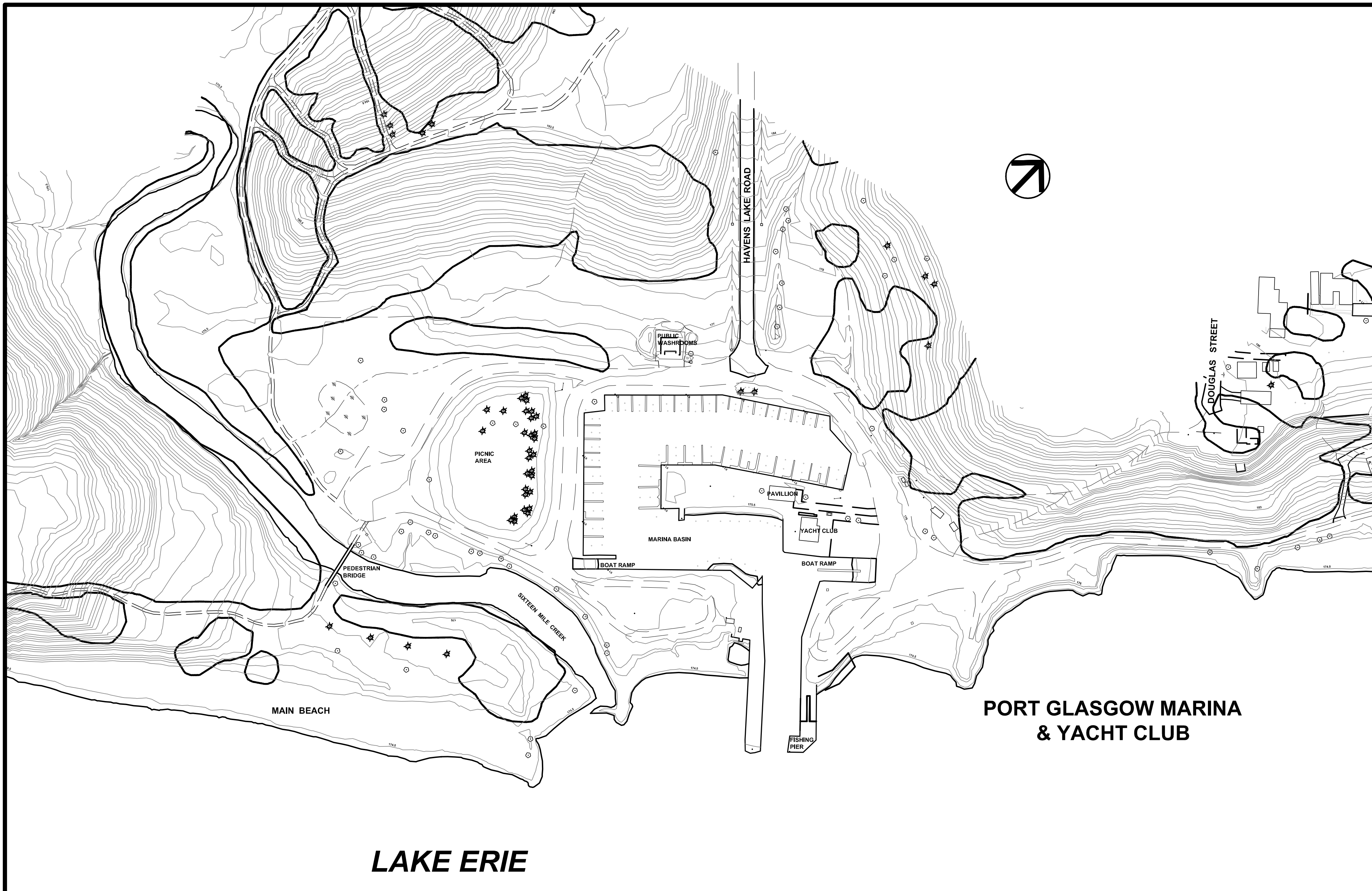


Figure 2.2
Site Plan
Port Glasgow Marina & Yacht Club

Scale 1:2500

3.0 COASTAL ANALYSIS

A review of coastal conditions in the area was undertaken. The review included bathymetry, water levels, wave climate and sediment transport. Understanding of coastal conditions is critical for the development of alternative concepts for entrance improvements.

3.1 Field Data

A sounding survey was carried out by Shoreplan personnel along the entrance channel to the marina, and updrift and downdrift of the site. This work was completed May 10, 2006. Figure 3.1 shows the locations of the sounding survey. Data from the sounding survey was combined with field sheet data to produce a bathymetric plan of the area.

3.2 Water Levels

A summary of the water level variations and wind set up at Port Glasgow on Lake Erie is presented in Table 3.1. The summary is based on a water level analysis completed by the Ontario Ministry of Natural Resources (MNR, 1989).

Table 3.1 Water Levels and Wind Set Up for Lake Erie at Port Glasgow

Return Period (years)	2	5	10	25	50	100
Peak Instantaneous Water Level, (m, IGLD 1985)	174.68	174.94	175.08	175.22	175.31	175.39
Wind Set Up, Wind Surges (m)	0.30	0.40	0.47	0.58	0.67	0.77

Water level fluctuations on Lake Erie, as described by MNR (1989), are the result of several natural factors. There are three types of water level fluctuations that effect Lake Erie: long term (over years), seasonal (within one year), and short period (less than an hour to several days).

Long term fluctuations are a result of persistent low or high net basin supplies. They can produce extremely low levels such as were recorded on Lake Erie in 1926, the mid-1930s and mid-1960s, or extremely high levels such as in 1952, 1973, 1985-86 and 1997-98. Figure 3.2 shows the historical monthly mean water levels from 1918 to present (Canadian Hydrographic Survey Canada, 2006).

Seasonal fluctuations are a result of higher net basin supplies during the spring and early summer and lower net basin supplies during the remainder of the year. The maximum lake level usually occurs in June on Lake Erie and the minimum lake level occurs in February. The magnitude of the seasonal fluctuation is usually in the order of 0.5 m. The average mean monthly water level (1996-2005) in June was 174.42 m and in February was 174.08 m, IGLD 1985.

Short-period fluctuations, lasting from less than an hour to several days, are caused by meteorological conditions. Wind and differences in barometric pressure over the lake surface create temporary imbalances in the water level at various locations. Storm surges are largest at the ends of an elongated basin, particularly when the long axis of the basin is aligned with the wind. On Lake Erie, water-level differences from one end of the lake to the other of more than 5 metres have been observed.

3.3 Wave Analysis

Wave conditions at the site were analysed using numerical models. A hindcast was used to determine the offshore wave conditions and a wave transformation model was used to bring the offshore waves into the site, considering the effect of the nearshore bathymetry. Each of these procedures is described separately below.

3.3.1 Offshore Wave Conditions

Wave hindcasting was used to estimate the wave climate at an offshore location where changes in water depths do not effect wave generation and propagation. Wind data recorded at London airport was used to predict the wave conditions that would have been generated by those winds. The recorded wind speeds were factored to account for the differences in over-land and over-water boundary layer friction. A detailed wind calibration analysis was undertaken during a recent study examining sediment transport at Wheatley Harbour (Shoreplan, 2006).

The wave hindcast provides hourly estimates of the wave conditions for a 40 year period from January 1965 to December 2004. This is a sufficiently large database to be considered representative of the long term wave conditions.

The results of the wave hindcast are best presented with wave tables, as shown in Appendix C, and with wave statistics plots such as those shown in Figures 3.3 and 3.4. The wave heights and periods shown on these plots and tables are the significant wave height and the peak wave period. These are single values used to statistically represent all of the individual waves which occurred during one hour. These statistics were calculated for the average annual open water season defined by assuming ice was present (and therefore no waves were generated) from January 1 to March 8 each year. These dates were selected after reviewing the weekly median ice concentration charts presented in the NOAA Great Lakes Ice Atlas (Assel, 2003).

Figure 3.3 shows the highest hindcast wave heights and total wave energy distribution by direction for the 40 year hindcast. It can be seen from Figure 3.3 that the wave energy distribution shows two distinct peaks; one from the east and one from the southwest. The maximum wave heights from the east are slightly higher than those from the southwest. This is mainly due to the longer over-water fetch lengths to the east. There is more wave energy coming from the southwest, however, due to the greater frequency of higher wind speeds in that direction. The maximum wave height from the east was 3.6 metres with a wave period of 8.5 seconds and the maximum wave height from the southwest was 3.5 metres with a wave period of 8.5 seconds.

Figure 3.4 shows both the wave height and wave period exceedance plots for the 40-year hindcast. Exceedance plots show the percentage of time that any given wave height or period is exceeded.

An extreme value analysis found the offshore wave condition with a 25 year return period to have a wave height of 4.5 m and wave period of 9.7 s. The 100 year return period wave condition was found to have a wave height of 4.7 m and a wave period of 10 seconds.

3.3.2 Nearshore Wave Conditions

A nearshore wave climate was produced by transferring the 40 years of hourly hindcast wave data from deeper water into the site using a wave transformation model. Wave transformation models are required to account for the effects that changing bathymetry has on the waves as they propagate into the site. For this analysis we used a wave ray based spectral transfer model that traces wave rays from a nearshore point of interest out to deep water. The wave ray paths are then used to transfer the offshore wave energy back into the point of interest. The nearshore point of interest is located just outside the surf zone where significant wave breaking occurs. The lake bed bathymetry considered by this model was based on surveys carried out by the Canadian Hydrographic Service. Figure 3.5 shows the nearshore bathymetry covered by the wave transformation model, including the layout of the model grids. Figure 3.6 shows the maximum wave height and period for each direction. The maximum wave height from both the south and east was 2.1 metres and wave period was 9 seconds.

Figure 3.7 shows a comparison of the directional distribution of the wave energy offshore and in the nearshore at the site. It can be seen that there has been a convergence of the two energy peaks with the southwesterly peak moving the most. At the site, the wave energy is now predominantly from the south. This shows that the southwesterly waves undergo significantly more refraction then the easterly waves. The southwesterly wave energy peak is still dominant, leading to a west to east net transport of nearshore sediments.

3.3.3 Operational Wave Conditions

Nearshore waves for the boating season (May 1 to September 30) were also examined and the nearshore wave tables are included in Appendix C. During the boating season, over half the time the wave height exceeds a threshold wave height of 0.3 m at the entrance to Port Glasgow Yacht Club. Figure 3.8 shows the nearshore wave height and wave energy distribution by direction during the boating season. The maximum wave height from the east was 1.8 m with a wave period of 7.6 seconds and from the south the wave height was 1.4 metres with a wave period of 6.7 seconds.

Presently, the marina entrance is oriented in a south easterly direction which allows waves from this direction to travel directly into the harbour. Waves from both the east and south diffract around the entrance piers and also travel into the harbour. Once inside the entrance channel, the waves travel along the existing vertical steel sheet pile wall to the basin. The waves reflect off the structure therefore the wave height is either the same or higher as it travels down the entrance channel towards the east side launch ramp. The increase in wave height along the sheet pile walls is due to mach wave reflection.

3.4 Sediment Transport

Sediment is mobilized in the nearshore by waves and currents. Sediment can move parallel to the shoreline (alongshore sediment transport) or perpendicular to the shoreline (cross shore sediment transport).

Alongshore transport rates are determined by investigating the sources and sinks of material (sediment budget), and the wave energy available to transport the sediment along the shore. Littoral cells, which are defined as sections of shoreline where there is no flow of sediment in or out at the boundaries, are established along a shoreline. As part of the work carried out for the LTVCA (Sandwell,1993), littoral cells were established along the shoreline from west of Port Alma to Port Burwell. Port Glasgow is located at the west end of littoral cell E4 which extends from Port Glasgow to Long Point. The report states that the littoral transport rate was estimated to be 32,000 cu. m. per year towards Long Point. Port Glasgow is considered to be the western boundary of the littoral cell. However, observations while at the site showed that the western boundary of the cell is most likely located west of Port Glasgow.

3.5 Summary of Design Conditions

Design water level and wave conditions for the improvements were selected to be consistent with the technical guidelines prepared in support of the Provincial Policy Statement. The design high water level at Port Glasgow is the 1:100 year return period peak instantaneous water level which is 175.4 m, IGLD 1985. The design wave condition is the 1:25 year offshore wave height of 4.5 metres with a 9.7 second wave period. Based on the results from the nearshore wave transformation the wave height would be 2.5 metres approximately 500 metres offshore of the structures, in a depth of 5 metres below datum.

The design wave condition at the site was determined using the PMS model from the Danish Hydraulic Institute. This is a complex numerical wave transformation model capable of considering refraction, diffraction and wave breaking. Figure 3.9 shows the wave height decay of various wave heights and periods including the design wave. The design wave height at the end of the existing piers is 1.7 metres. The design wave height for the offshore breakwater is 2.1 metres.

For marina operations, the design wave height was based on the maximum boating season wave height from the 40-year hindcast. The maximum wave height from the east was 1.8 metres with a wave period of 7.6 seconds, and from the south the wave height was 1.4 metres with a wave period of 6.7 seconds. Using the wave decay model, these waves became 1.4 metres and 1.3 metres respectively at the south end of the existing piers.

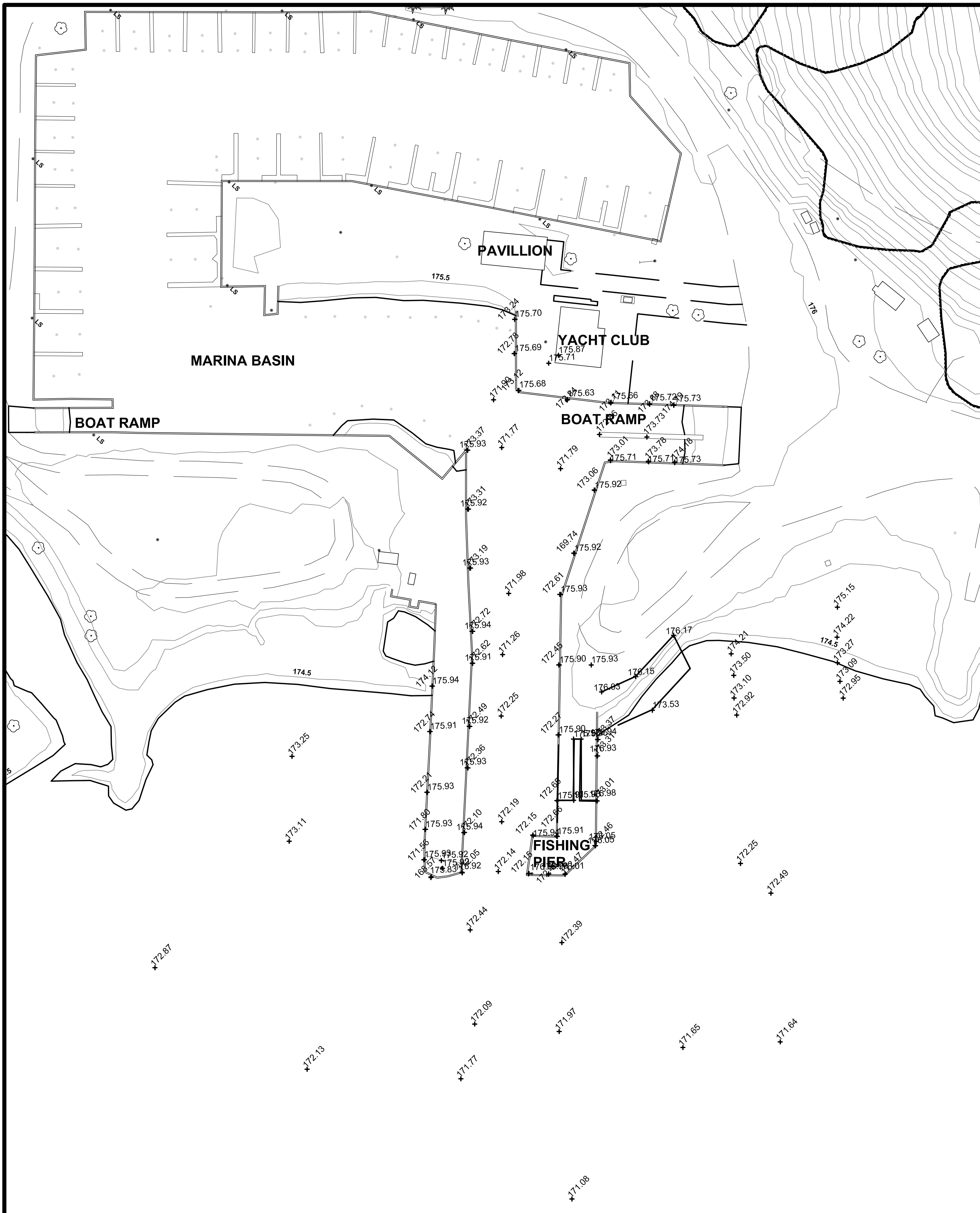


Figure 3.1
Bathymetry
Port Glasgow Marina & Yacht Club

Scale 1:1000

SHOREPLAN

Figure 3.2
Historical Monthly Mean Water Level, Lake Erie

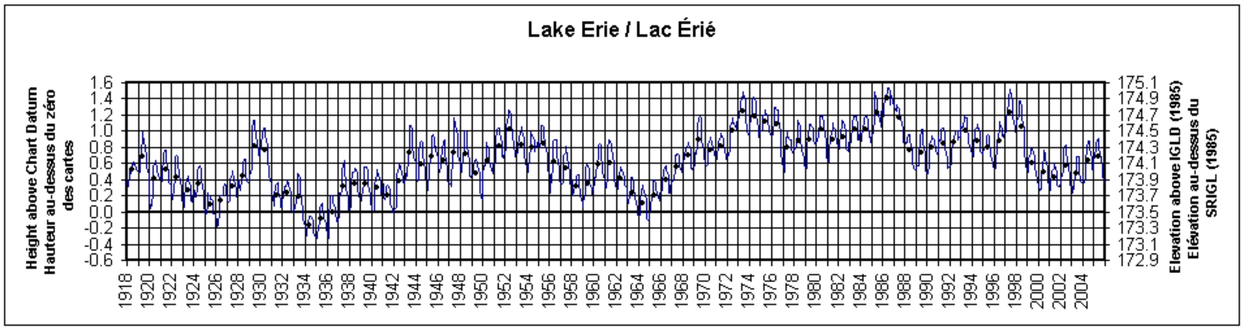


Figure 3.3 Directional Distribution of Offshore Wave Heights and Wave Energy

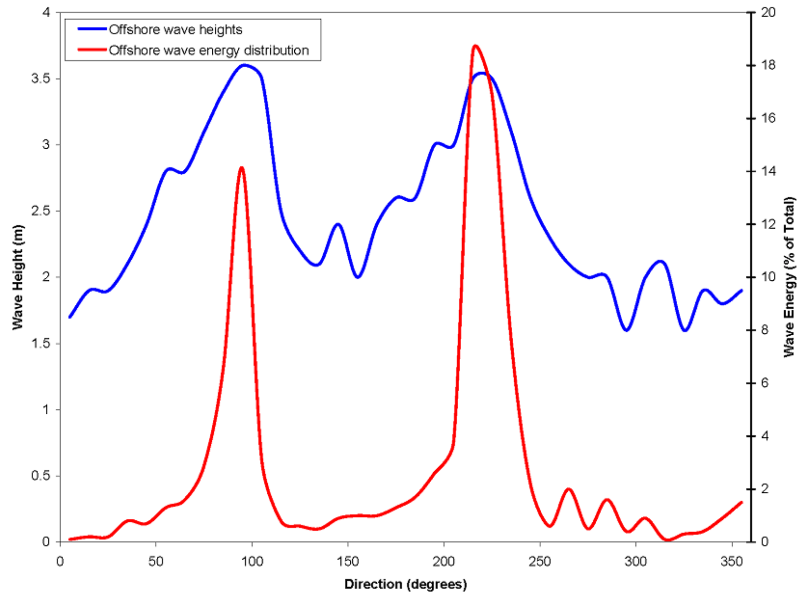


Figure 3.4 Wave Height and Wave Period Exceedance Diagrams

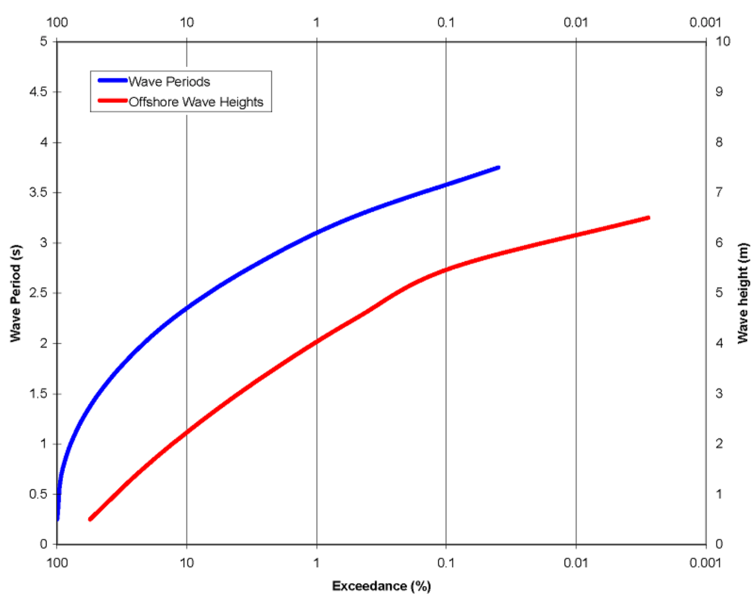


Figure 3.5
Nearshore Bathymetry and Modelling Grid Layout

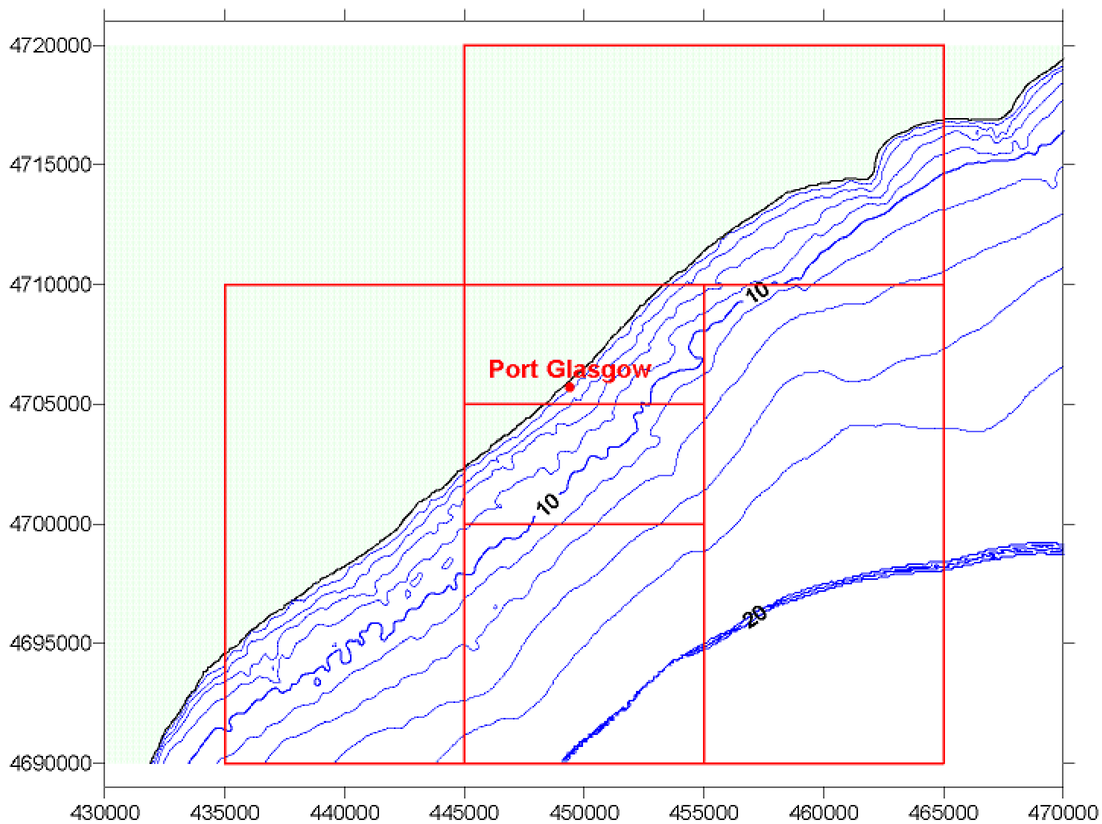


Figure 3.6 Directional Distribution of Nearshore Wave Height and Period

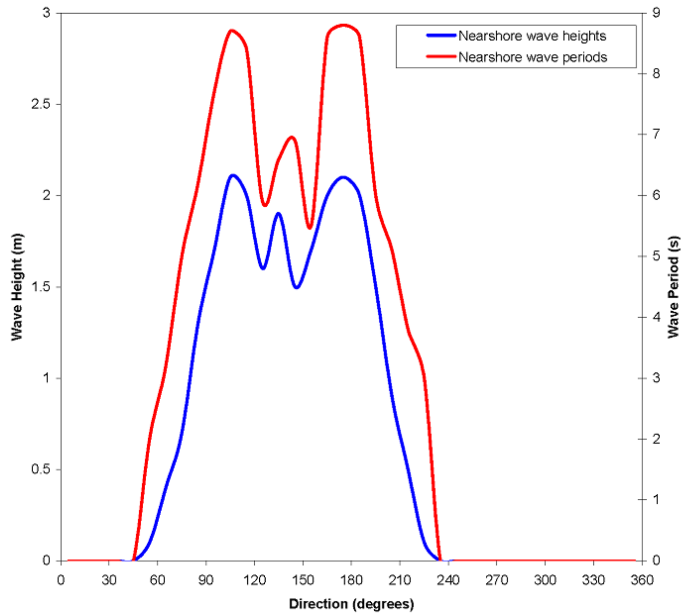


Figure 3.7 Comparison of Offshore and Nearshore Wave Energy Distributions

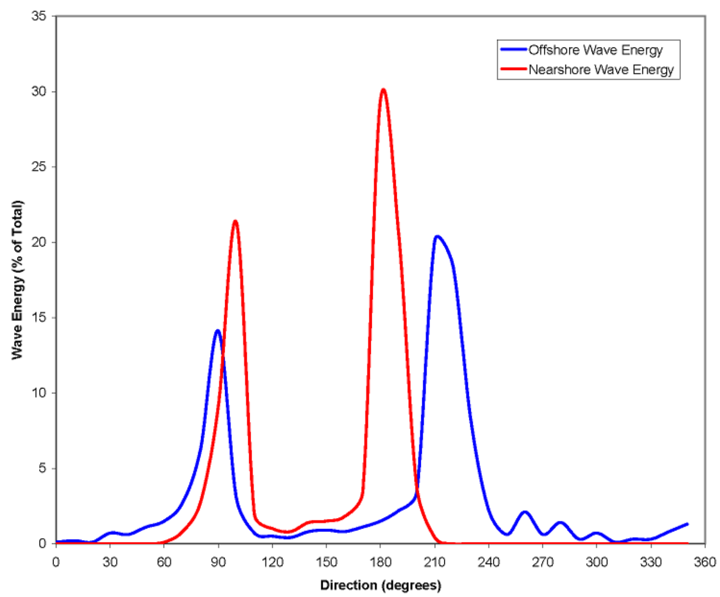


Figure 3.8 Directional Distribution of Nearshore Wave Height and Wave Energy (Boating Season)

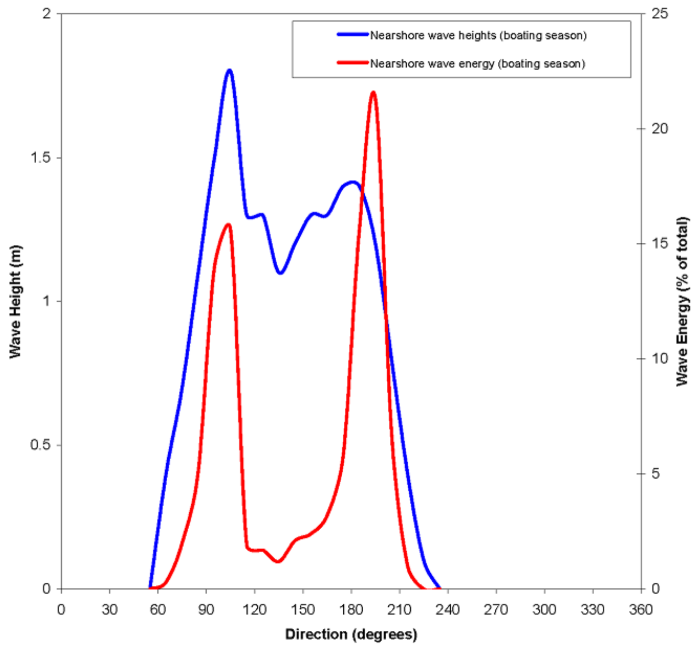
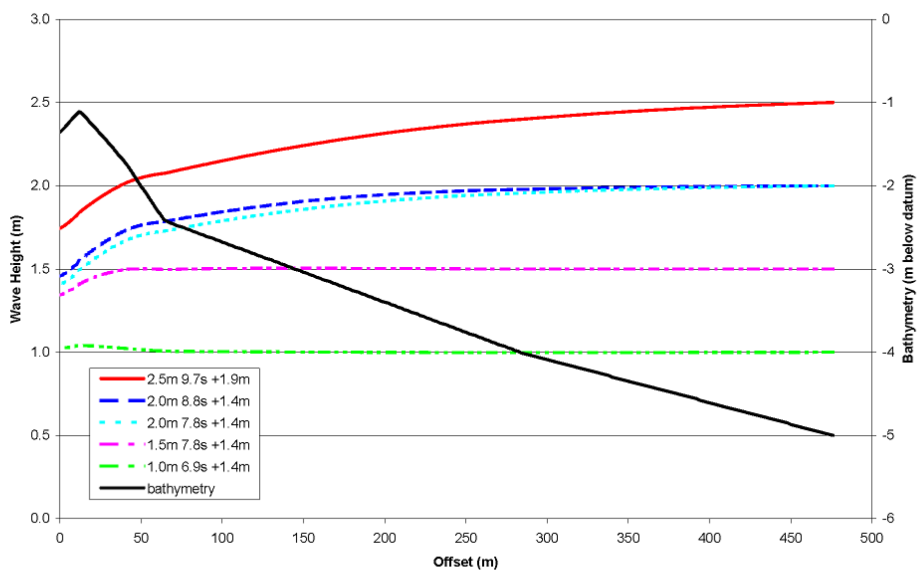


Figure 3.9 Wave Height Decay



4.0 Discussion of Alternative Concepts

We have identified two issues that need to be addressed at Port Glasgow Marina. The first issue relates to wave conditions at the harbour entrance caused by the combination of large incident waves and waves that reflect off the existing steel sheet pile wall. The second issue relates to the difficulty of manoeuvring boats within the narrow entrance channel during these wave conditions.

Three alternative concepts were considered to address these issues. These alternatives are referred to as Option 1 - East and West Pier Extension, Option 2 - West Pier Extension, and Option 3 - Offshore Breakwater. These alternative concepts were designed to provide an appropriately wide navigational channel, to reduce wave reflections within the channel and to reduce wave energy in front of and/or within the entrance channel. The concepts are designed to reduce the incoming wave heights to between approximately 0.3 and 0.6 metres at the north end of the entrance channel, which is the location of the existing launch ramp. We have used concept level analyses for our designs and further refinement will be needed for the detailed design of the solution to be implemented

There are two design elements common to the three concepts. The first is that the width of the harbour entrance and the entrance navigation channel is increased to a minimum of 20 metres. That width is measured at the minimum navigational depth, which is two metres below chart datum. This increases the width of the existing channel by two metres and increases the width of the tip of the entrance by seven metres, when measured at the minimum navigational depth. Because the sides of the proposed channel are sloped rather than vertical, as exists now, the effective increase in the usable channel width will be more than 20 metres.

It must be noted, however, that the new entrance channel is still considered to be narrow. Current standards use a navigational depth entrance width of 25 to 30 metres for this type of marina facility but there are two restrictions that prevent us from achieving this greater width. Relocating the west pier in a westerly direction would reduce the length of the shoreline between the pier and the mouth of Sixteen Mile Creek. That could impact the mouth of the creek and the effectiveness of the existing dredging operations. That is not an acceptable impact so the location of the steel sheet pile wall of the west pier has not been altered in any of our concepts. Increasing the entrance channel width further to the east than we have proposed would require relocating the existing east launch ramp. That would in turn require land-side changes which would significantly increase the capital costs.

The second common design element is the crest width of the proposed new entrance structures. The structure crests must be wide enough to allow access by heavy construction equipment, such as a large backhoe. This access is necessary for future maintenance of the structures and for potential maintenance dredging of the channels. A crest width of 4.0 metres (13 ft) is generally preferred but a minimum crest width of 3.0 metres (10 ft) can be used. We have used the minimum crest width of 3.0 metres in order to minimize construction costs.

4.1 Option 1 - East and West Pier Extension

Option 1 consists of widening the existing channel and extending the piers 50 metres offshore. The existing west pier will remain but a 2:1 (h:v) sloped rip rap revetment will be placed against the steel sheet piles. A 50 metre long extension will be constructed as an armour stone breakwater with 2:1 (h:v) front and back slopes.

The existing east pier will be removed and a new east pier will be constructed further to the east to create a 20 metre wide entrance channel. The new east pier will also be constructed at a 2:1 (h:v) slope. Where there is existing land base the slope will be constructed as a rip-rap revetment. Where the pier extends out into Lake Erie it will be constructed as an armour stone breakwater. This new breakwater will be approximately 85 metres long in order to extend as far offshore as the new west pier breakwater.

Figure 4.1 shows the proposed layout of this option. Figure 4.2 shows typical sections through the east and west piers.

This concept addresses wave agitation within the entrance channel by changing the smooth, vertical sheet pile wall to a rough, sloped stone wall. That changes a highly reflective surface to one which absorbs energy. The wave heights will gradually reduce as they travel within the channel. The wider channel provides more room for boat manoeuvring. The ends of the piers are also sloped and therefore reduce wave reflection in front of the entrance.

With this option the orientation of the channel has not changed. Wave conditions outside the channel entrance will be similar to existing conditions. The channel is lengthened to reduce wave heights at the launch ramp. The length of the channel is based on the relationship between wave height reduction along a rip rap channel and the channel side slopes, channel width and wave period, as determined by Bishop (1987). The length of the channel was selected such that the wave conditions at the north end of the channel are less than or equal to 0.3 m during the boating season.

When compared to the other options, Option 1 occupies the largest lakebed area. This may be of concern to the Department of Fisheries and Oceans. The extension of the piers 50 metres further to the south will act as a barrier to alongshore sediment transport. However, the ongoing dredging activities will continue to remove a majority of the sediment reaching the site. Therefore, no further impact on sediment transport is expected.

The estimated cost of this option is approximately $2,260,000. This estimate includes a contingency allowance for design and construction.

4.2 Option 2 - West Pier Extension

Option 2 consists of widening the existing channel and re-orienting the channel entrance. As with Option 1, the existing west pier will remain but a 2:1 (h:v) sloped rip rap revetment will be placed against the steel sheet piles. A 40 metre long extension will be constructed as an armour stone breakwater with 2:1 (h:v) front and back slopes. The extension will extend approximately 10 metres to the south then turn eastward for approximately 30 metres. The west pier extension will have a crest width of 3 metres and a crest elevation of 176.5 metres. That elevation is approximately 0.5 metres higher than the existing west pier.

The existing east pier will be removed and a new east pier will be constructed further to the east to create a 20 metre wide entrance channel. The new east pier will also be constructed at a 2:1 (h:v) slope. Where there is existing land base the slope will be constructed as a rip-rap revetment. Where the pier extends out into Lake Erie it will be constructed as an armour stone breakwater. This new breakwater will be approximately 40 metres long to extend as far offshore as the existing east pier. Figure 4.3 shows the layout for Option 2 and Figure 4.2 shows typical sections through the structures.

This option addresses the wave conditions at the south end of the entrance channel by turning the entrance to face an easterly direction. By rotating the entrance to the east, the entrance is protected from the predominate southerly and south-westerly waves which occur approximately 67% of the time during the boating season. The less frequent easterly waves, which occur approximately 33% of the time during the boating season, will enter the channel and hit the west pier breakwater/revetment slope where some of the wave energy will be absorbed. Based on Bishop (1987), the design boating season wave from the east will be reduced to a 1.1 metre high wave after the west pier extension and then further reduced to a 0.4 metre high wave near the launch ramp. The south and south-westerly waves will be less than 0.3 metres at the same location during the boating season.

This option occupies a smaller area of the lake bed than Option 1 but this would still be of concern to the Department of Fisheries and Oceans. Lining the entrance channel with rip rap provides a better environment for fish as opposed to the existing steel sheet pile walls.

Although the proposed structure does extend further into the lake, it is still in line with the adjacent shoreline. Therefore, this new structure will be less of a barrier to sediment transport compared with Option 1.

The estimated cost of this option is $1,520,000. This estimate includes a contingency allowance for design and construction.

4.3 Option 3 - Offshore Breakwater

Option 3 consists of building a detached breakwater approximately 50 metres offshore of the existing piers and modifying the ends of the existing east and west piers. The breakwater will have a crest elevation of 176.5 metres and will be located in a depth of 2.5 metres below chart datum. The crest will be approximately 3 metres wide and have side slopes of 2:1 (h:v). The breakwater will be protected by two layers of armour stone covering a rip rap core. Figure 4.4 shows the layout for this option. Section A from Option 1, shown in Figure 4.2, also applies as a typical section through the breakwater.

It is also proposed that the “L” extension of the east pier be removed and the ends of both piers be reinforced with sloping armour stone revetments. This will minimize wave reflection at the entrance. However, it is critical that the entrance width not be reduced by the placement of the armour stone.

The breakwater in this option is aligned such that it will protect the harbour from direct wave attack. Therefore, waves hitting the existing steel sheet pile piers will be substantially reduced in height. Smaller diffracted waves will be able to penetrate down the existing channel. However, since the east and west piers will remain steel sheet pile, these diffracted waves will reflect off the walls to the north end of the channel. The wave heights at the north end of the channel will either be the same or bigger than at the south end of the channel. Based on diffraction tables from the Shore Protection Manual 1977, wave heights from the east will be reduced by 30% and wave heights from the south will be reduced by 50% where they reach the existing channel entrance.

This option occupies a large area of the lakebed. Therefore it will be of concern to the Department of Fisheries and Oceans.

The breakwater will act as a partial barrier to sediment transport. The reduced wave climate behind the structure may cause suspended sediment to fall out behind the structure. Periodic maintenance dredging will be required.

The estimated cost of this option is $1,850,000. This option will require more detailed engineering analysis than the other options. Detailed numerical modelling of the wave climate behind the breakwater will need to be undertaken to optimize the design of this structure. This option will also require that the breakwater be built by marine based construction or by building construction access from land. This estimate includes an allowance for design and construction.

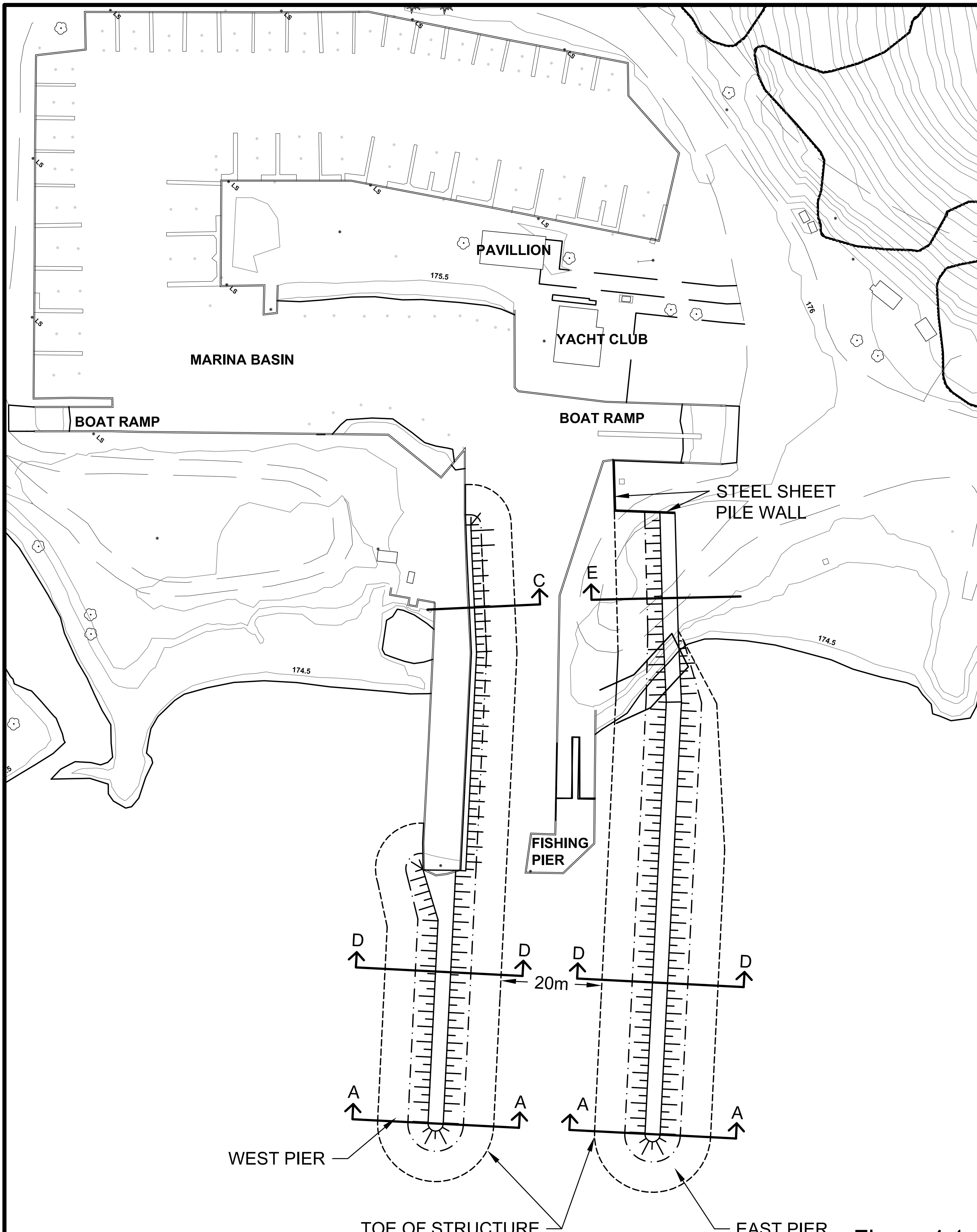


Figure 4.1
Option 1 - East and West Pier Extension
Port Glasgow Marina & Yacht Club

Scale 1:1000

SHOREPLAN

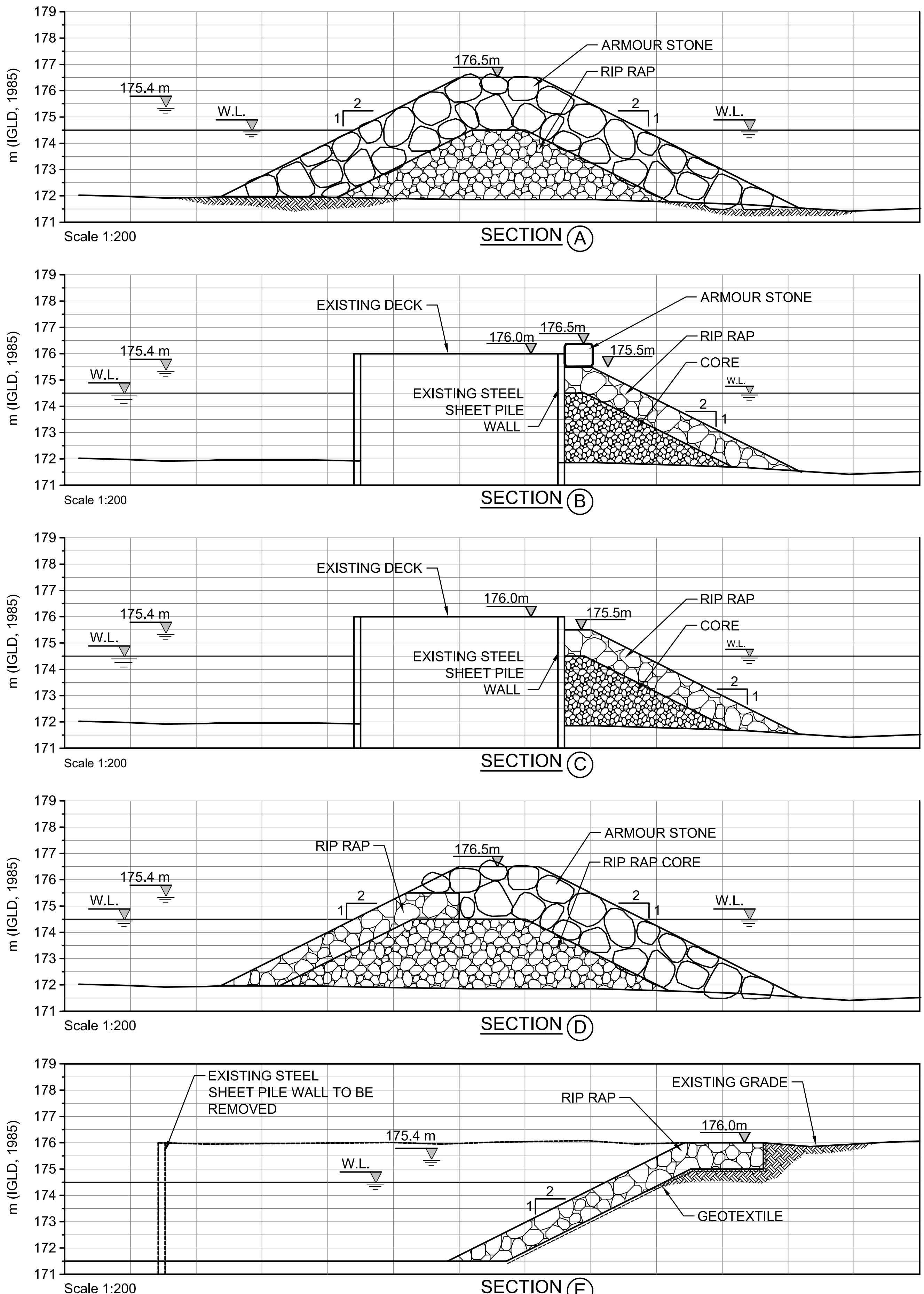


Figure 4.2
Typical Sections

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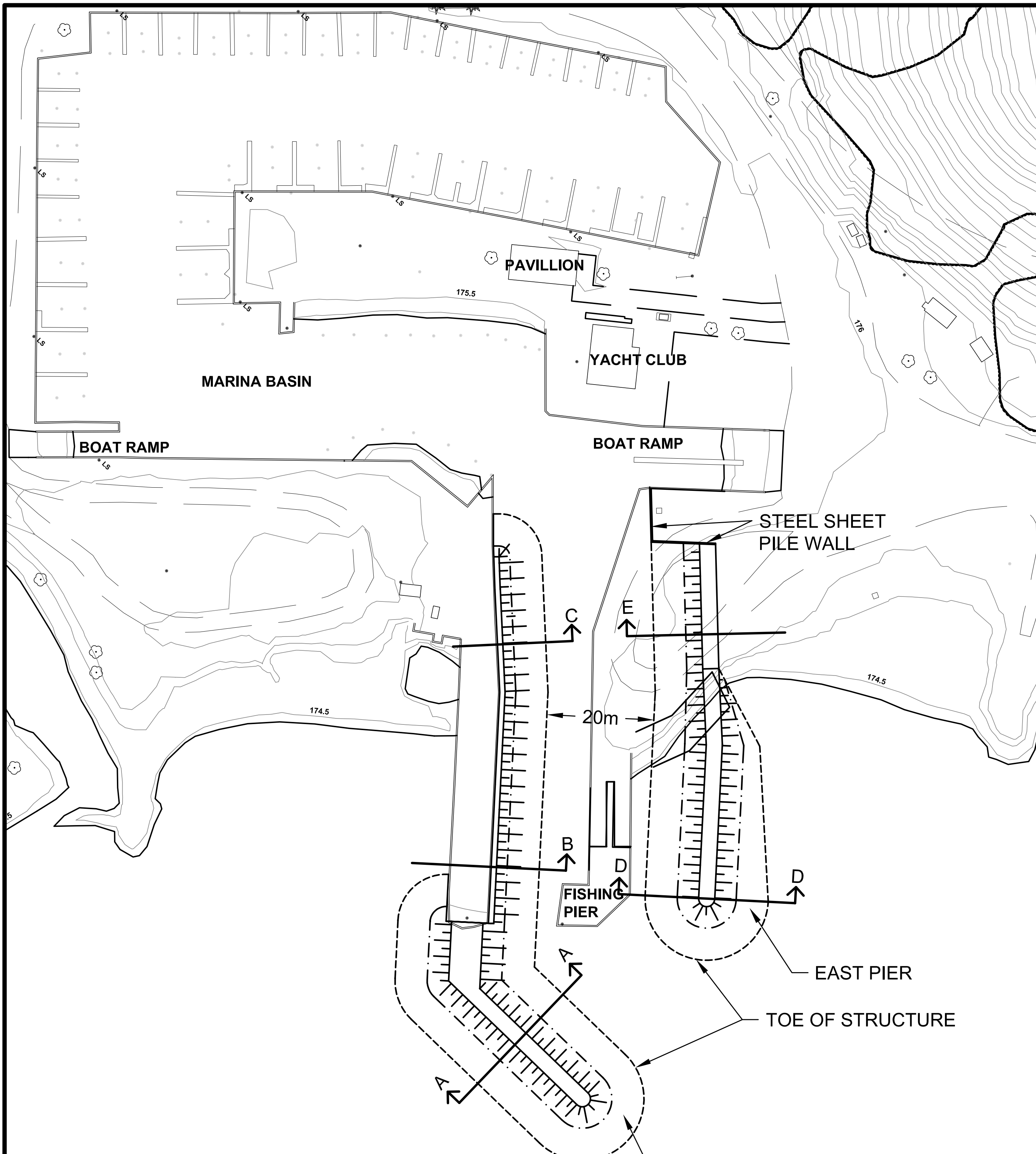

Figure 4.3
Option 2 - West Pier Extension
Port Glasgow Marina & Yacht Club

Scale 1:1000

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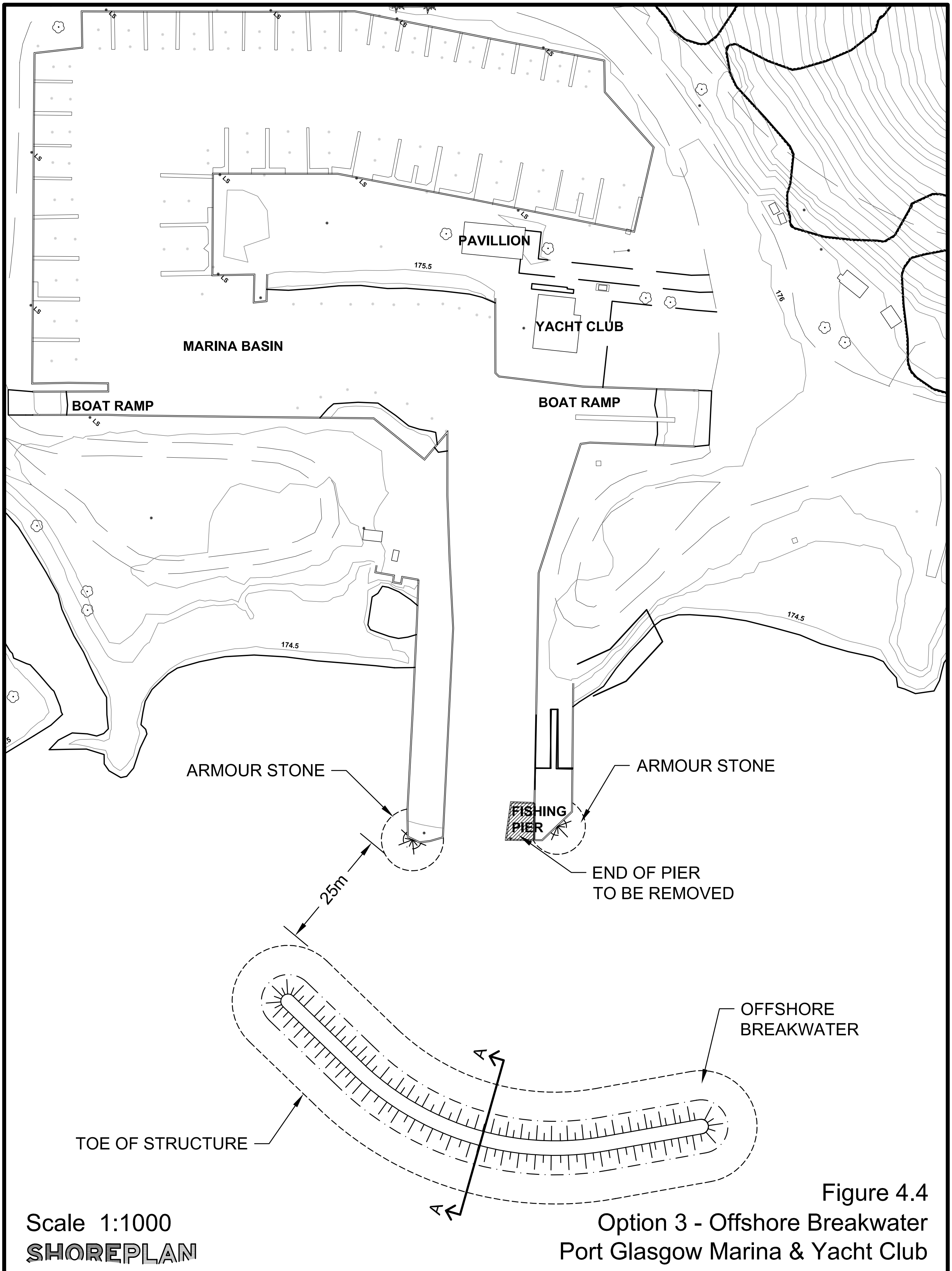

Figure 4.4
Option 3 - Offshore Breakwater
Port Glasgow Marina & Yacht Club

5.0 Preliminary Conclusions and Recommendations

Wave conditions in front of and within the entrance channel of the Port Glasgow Marina & Yacht Club were found to be severe on a regular and frequent basis. The present orientation of the entrance and the type of construction materials used in the channel lining cause very limited reduction in wave height in the channel. Under certain conditions, waves can increase in height in close proximity to the steel sheet pile wall due to mach wave reflections. These wave conditions, combined with a very narrow entrance channel, make the navigation of the channel frequently very challenging and sometimes dangerous.

All of the options described in Section 4 will substantially improve the wave problems at the entrance channel to Port Glasgow Marina. Although no cost benefit analysis was completed as part of this project, it would appear that construction cost estimates are relatively high for all reviewed options. Table 5.1 summarizes the costs for each of the proposed options.

Table 5.1 Summary of Options

Option	Description	Cost
1	East and West Breakwater Extension	$2,260,000
2	West Breakwater Extension	$1,520,000
3	Offshore Breakwater	$1,850,000

Option 1 provides the most effective protection for storms from all directions. Its effectiveness is due to the length of the channel rather than the orientation. However, the high effectiveness comes at a high cost.

Option 2 reduces the wave conditions at the entrance substantially and is particularly designed to be effective under the most frequent waves from the southwest quadrant. However, the less frequent easterly waves will still cause some agitation within the channel, although wave conditions will be much improved from existing conditions.

Option 3 reduces the wave climate at the entrance, but not as effectively as the other options.

Our preliminary conclusion is that option 2 is the preferred option based on the following considerations:

- The total capital cost is the lowest of the three concepts developed

- The concept plan can be implemented in stages working towards the ultimate solution; an improvement can be obtained with relatively low investment

- Impact on environment (coastal and aquatic) can be mitigated

The staging of implementation could be as follows:

Phase 1: Construction of the west pier extension and removal of the “L” extension from the south tip of the east pier. The cost of this phase is estimated to be $560,000.

Phase 2: Relocation of east pier at a cost of $680,0000

Phase 3: Lining of the east side of the west pier with rip rap at a cost of $280,000

There may some additional mobilization/ demobilization costs associated with doing the work in phases. However, these costs are relatively small in context of the overall project.

The proposed breakwaters are designed primarily to provide improved navigation in the entrance channel and wave protection for the launch ramp and harbour. They are not specifically designed to provide public access along the crest of the breakwaters. Existing fishing opportunities provide by the existing piers will be eliminated. Although fishing off of armour stone breakwaters is quite common, it is not as convenient or safe as the existing conditions. Should a fishing opportunity be a desired activity in the area, the proposed entrance structures can be modified.

It is suggested that modification to accommodate fishing be limited to the east pier. The west pier will be exposed to more frequent wave activity and public access to the west breakwater should not be encouraged. The east pier section can be redesigned to provide a concrete walkway along the crest. The walkway will need to be supported on piles driven through the core of the breakwater and into the lake bottom. Due to the potential for wave and ice uplift on the walkway, the pile supports have to be substantial and the concrete deck heavily reinforced. A typical section of the of the modified east pier is presented on Figure 5.1. The sloping sides of the breakwaters must be maintained to prevent wave reflection and penetration into the basin. The construction cost of the concrete walkway and supporting piles is estimated to be $200,000. This amount includes a design and contingency allowance. A photo of a concrete walkway on top of an armour stone breakwater is presented on Figure 5.2.

Discussions with a Ministry of Natural Resources (MNR) biologist report that the fowler toad, an endangered species, has been seen in the general area. This may be a potential restriction for expanding or relocating the piers, but MNR is checking into this in more detail. Staff of MNR has subsequently confirmed that the sightings were not at the proposed entrance location.

Discussions with the staff of the Lower Thames Conservation Authority and the Department of Fisheries and Oceans did not identify any site specific concerns dealing with the natural environment. Protection of the mouth of Sixteen Mile Creek is important and compensation measures will need to be negotiated for any loss or alteration of habitat. Expanded or relocated breakwater(s) will also require a new land use permit from the Ministry of Natural Resources.
This has been confirmed in a discussion with MNR staff. Approvals will be required from the above noted agencies and Transport Canada under the Navigable Waters Protection Act.

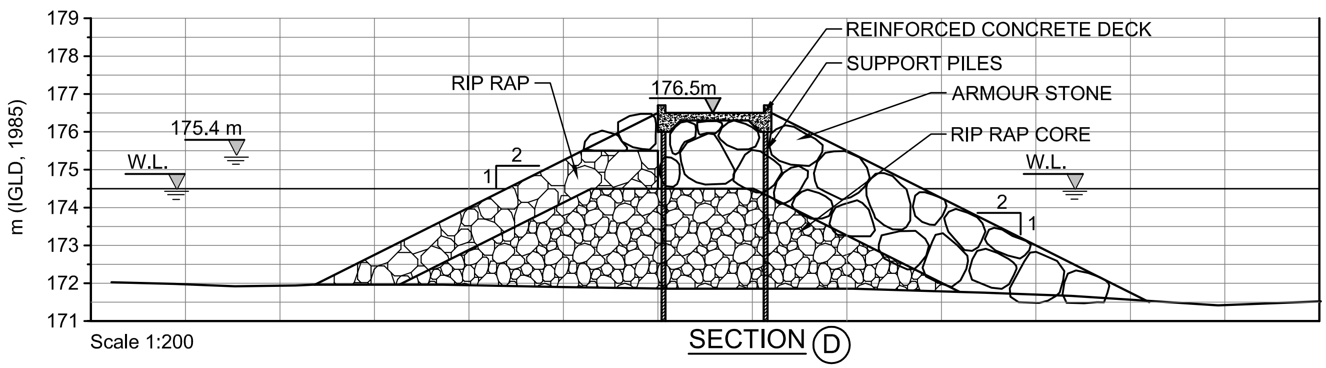

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Figure 5.1
Modified East Pier with Walkway


Figure 5.2 Photograph of Typical Concrete Walkway

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