Design Report

THE SHIPYARDS - COLLINGWOOD

Outfitting Dock and Launch Basin (West & South Sides)

FRAM Building Group / Slokker Canada Corp.

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SHOREPLAN

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1.0 INTRODUCTION

Shoreplan Engineering Limited (Shoreplan) was retained by FRAM Building Group to provide a detailed design for the shoreline improvements along the Outfitting Dock (also known as the Town Dock), and west and south sides of the former launch basin of the Collingwood Shipyards property. A plan of the site is presented in Figure 1.1. As part of this work a detailed diving inspection of the existing structures and testing of concrete samples from the existing walls were undertaken. This report and drawings present findings of these investigations and provide the details of the design of the proposed shoreline improvements in compliance with the requests of the "Shipyards" Master Document Agreement.

Previous reports prepared for this property and referred to in this report include:

- Preliminary Geotechnical Design CSL Lands Collingwood, Ontario (Terraprobe 2003);
- The Shipyards - Collingwood Preliminary Shoreline Review (Shoreplan, 2003);
- The Shipyards in Collingwood Design Wave Conditions (Shoreplan, 2004a); and
- The Shipyards - Collingwood Review and Assessment of Vertical Shore Structures (Shoreplan, 2004b).

Terraprobe 2003 is attached in Appendix A. The Shoreplan reports were provided previously and are not attached herein.

The shoreline of the subject lands was divided into fourteen reaches during the preliminary review work (Shoreplan, 2003). The locations of these reaches are presented in Figure 1.1. Each reach contains a section of shoreline with similar characteristics. The shoreline reaches are numbered starting at the west side of the site and progressing in an easterly direction around the site. The shoreline in reaches 1 to 3 was designed previously and shoreline works implemented in 2004 and 2005. Only reaches 4 to 6 are the subject of this report.

This report is divided into five chapters and seven appendices. Chapter one provides an introduction and a brief description of the history of the site use. Chapter two provides the condition of the existing structures, which includes the findings of the detailed diving inspection and concrete testings that were carried out. Chapter three describes the shoreline hazards at the site, provides the details for the design of the proposed structures. Chapter four describes the design life and remaining useful life of the existing and new structures. Chapter five describes the monitoring and maintenance program for the existing and new structures.

1.1 Brief History of Site Use

The site has been used for waterfront related industrial and commercial activities for many years. A detail review of the past site uses was not undertaken, but several drawings, charts and photographs provide glimpses of the past activities.

A navigational chart from 1888 shows the presence of two basins approximating the present dry dock and launch basin. The Outfitting Dock northwest of the launch basin is also shown on the chart. A Public Works Canada drawing of the Collingwood Harbour dated February 1928 shows two dry dock basins. Their locations appear to match the existing dry dock and launch basin. The existing dry dock basin is referred to as dry dock 1 and the existing launch basin is referred to as dry dock 2. Gates are shown at the north end of the both dry docks. An oblique aerial photograph presented on Figure 1.2 shows both dry docks and other mooring slips in the west part of the site. The date of the photograph is not known.

The west side of the launch basin was used as the construction area for ships built in the Collingwood Shipyards until its closing. The ships were built on “ways” and side-launched into the launch basin. This indicates that the structure was subjected to heavy and dynamic loads in the past. The Outfitting Dock (Town Dock) was used, as the name implies, to complete the outfitting of the ship after launching. Heavy equipment was operated on the dock and ships were tied off to the bollards on the deck.

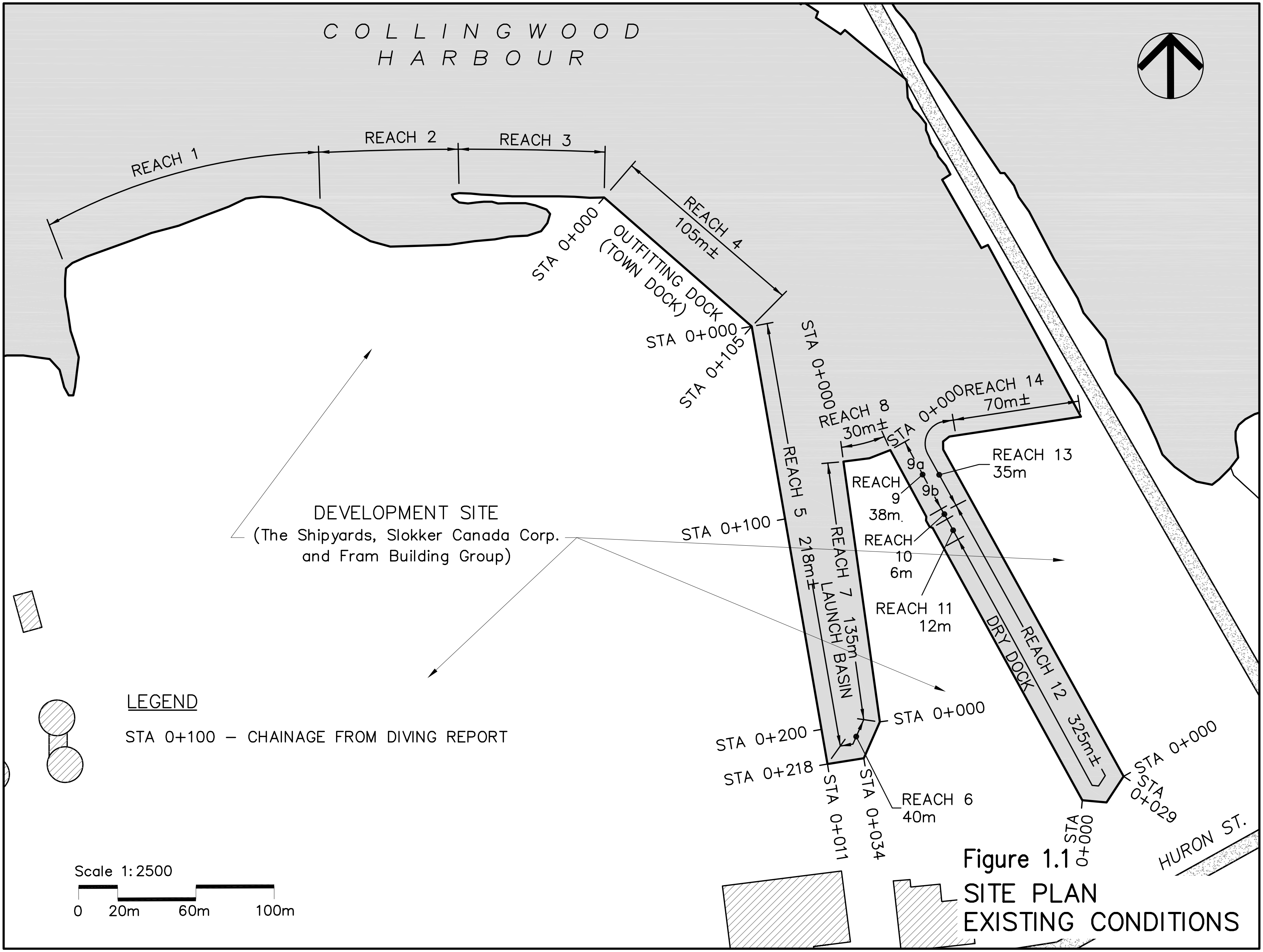


Figure 1.1
SITE PLAN
EXISTING CONDITIONS

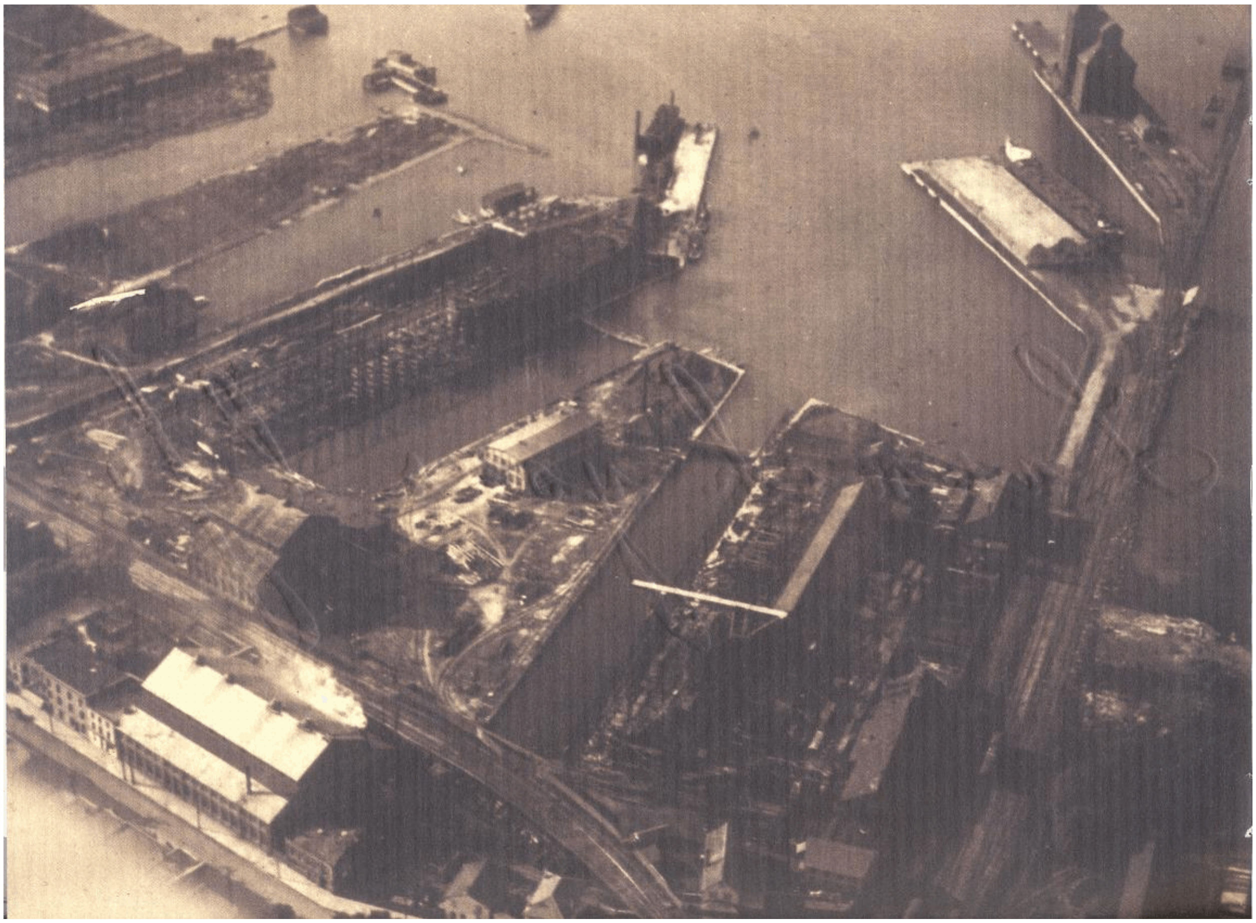


Figure 1.2 Aerial Photograph (date unknown)

2.0 CONDITION OF EXISTING SHORELINE STRUCTURES

2.1 Inspections and Testing

Description of the existing structures are provided by reach. Figure 1.1 presents the locations of these reaches at the project site. Inspection of the existing structures was carried out above and below the waterline. A description of the structures above and below water are provided in sections 2.1 to 2.3. Drawings that illustrate the existing condition of the structures were prepared and are included in Appendix B. Drawing WI-01 illustrates the details of the field wall inspection for the Outfitting Dock/ Town Dock, drawing WI-02 illustrates the west side of the Launch Basin and drawing WI-03 illustrates the south end of the Launch Basin. The drawings illustrate damage and/or deterioration such as cracking, spalling and broken concrete, and anomalies observed during the field inspections and are reported below. Photographs of the existing structures are included in Appendix B

Above water inspections of the structures for this phase of the work was carried out by Shoreplan Engineering Limited staff during various site visits in 2006 and 2007. Information collected during other site visits are also included in this report, where applicable. All of this work was carried out by professional and senior technical staff of the firm.

Description of the structures in sections 2.1 to 2.3 refer to sections L1, L2, M1 to M10 and N1 and N2. These sections are illustrated on detail design drawing 03-606LB 6. The locations of the sections are shown on detail design drawings 03-606LB 1 to 3. All of the design drawings for the proposed shoreline improvements are provided in Appendix C.

On October 24 and 25, 2006 a diving inspection of the existing structures in reaches 4 to 6 was carried out by a professional diving crew from Dundee Marine, under the guidance of Jane Graham, P. Eng of Shoreplan Engineering Limited. The water level at the time of the diving inspection was 176.0 metres IGLD, 1985. The diving inspection was recorded on three video tapes. A summary video was also taken and is presented in DVD format in Appendix D. A summary of the diving field notes is presented in Appendix E. Due to weather conditions at the time of the diving inspection, the inspection was carried out starting at reach 6 and proceeding north to reach 4, and the video and field notes are presented in the same sequence in the appendix.

In reach 4, the diving inspection was carried out to inspect the condition of the timber crib including verticality, soundness of the timbers, and chemical treatment, and assessment of the connecting bolts. In reaches 5 and 6, the diving inspection was carried out to inspect the condition of the concrete walls including areas of spalling, broken concrete and anomalies in construction.

Terraprobe (2003) summarizes the findings from borehole samples taken by Terraprobe and Bruce A. Brown Associates. Terraprobe and Bruce A. Brown Associates drilled 48 boreholes on the “Shipyards” property. The location of the boreholes are shown in Figure 2 of Terraprobe (2003) (Appendix A). Although, boreholes were not taken specifically for the design of the shoreline structures, the design parameters recommended in Terraprobe (2003) were considered suitable for design (confirmed in letter from Terraprobe dated June 10, 2008 included in Appendix A). The stratigraphy of the site is described as fill material overlaying a thin surficial deposit of fine sand or silty sands. Beneath the fine layer of sand is bedrock. Terraprobe (2003) provides a map of the bedrock elevation at the site which shows the bedrock surface ranging between elevation 173.0

metres and 176.8 metres. They also report that the launch basins were made by removal of rock. The top of the bedrock elevation drops significantly and abruptly in the vicinity of the basins were historic blasted rock faces were cut. The information in Terraprobe's report and the divers description of the bottom of the structures and basin were used to estimate the elevation of the bedrock for the purpose of design. Terraprobe (2003) also provided the properties of the soils and bedrock at the site. These properties are provided in Section 3, Design of Shoreline Improvements.

In November 2006, Terraprobe Testing Limited collected concrete core samples of the existing concrete cap and precast concrete blocks in reach 4, and the concrete walls in reaches 5 and 6. The samples were tested for unconfined compressive strength. The sample results are discussed in the following sections and a copy of their letter report is presented in Appendix F. The location of the samples are shown on the wall inspection drawings, WI-01 to WI-03, included in Appendix B.

The Master Development Agreement, section 4.10, required that the cores obtained shall be tested for unconfined compressive strength and alkaline aggregate reactivity. Terraprobe reported that "the concrete breakwall has been in service for many years and reveals no significant visual aggregate related problems", see Terraprobe letter dated July 20, 2007 in Appendix G. Therefore the Alkali Aggregate Reaction (AAR) testing was not carried out.

2.2 Description of Existing Conditions

2.2.1 Reach 4 - Outfitting Dock (Town Dock)

Reach 4 - Outfitting Dock, also known as the Town dock, is located at the north end of the subject property. The Town dock is approximately 105 metres long. A baseline was established along this reach starting at the north west corner of the structure heading southeast and was used as a reference line during the inspections. The following description is based on the above and below water review and observations by the diver and engineer. The text herein provides a general description. Further details are provided in Appendix C as previously noted.

The dock consists of a timber crib with a concrete cap. The crest elevation of this structure is approximately 177.8 metres. The timber crib and concrete deck are approximately 6.2 metres wide. The timber crib was constructed with 0.3 metres deep, and 0.3 metres high timbers. The exposed face of the crib parallel to the basin is a smooth face of timbers. The front face timbers have been notched out at regular intervals along the wall for the cross beam timbers which run perpendicular to the face back to the back of the crib. Steel rods running perpendicular to the face of the wall were also visible along the faces of the walls connecting the front and backsides of the cribs. The cross beam timbers and rods are visible on the back side of the crib where the crib is exposed. Where the concrete cap is missing, it could be seen that spikes driven vertically were used to secure the timbers in place.

The cribs appear to have been built in sections approximately 30 metres long and timbers were placed to bridge across the small gap, approximately 0.5 metres wide. Supported on the front and back edges of the timber crib are rows of precast concrete blocks. The concrete blocks are approximately 0.6 metres high, 0.8 metres deep and 1.2 metres long. The precast blocks are supported by the outside row of timbers and a second row of timbers set back 0.3 metres from the front timber. The bottom of the precast concrete block is shaped to key in between the space between the two timbers. The gap between the concrete blocks is filled with stone fill. On top of the concrete blocks is a cast-in-place concrete cap. The cast-in-place concrete cap is approximately 6.2 metres wide and 1.0 metres high at the front face. Section L1, shown on drawing 03-606LB-6, shows a typical section through the wall from chainage 0+000 to 0+098. Section L2, shown on drawing 03-606LB-6, shows a typical section through the wall between chainage 0+098 and 0+105 where the concrete cap is missing.

The diver found that the crib was founded on bedrock. No evidence of settlement of the crib was found. Overall the face of the wall below waterline was vertical. No distortion of the crib was noted by the diver. The elevation of the toe of the timber crib varied between 170.4 metres at the north west end to 171.7 at the south west end. There was only a minimal amount of silt at the toe of the structure, generally less than 100 mm.

The diver probed the timber cribs with a screw driver over the height of the crib at 10 metre intervals along the wall. The diver also probed randomly along the wall between the stations. The diving inspection found that, in general, the timber cribs below the water line are in good condition. No areas of deterioration were identified. There does not appear to be any timbers out of alignment and there were small to no gaps between the timbers. The diver did not find any loose soil or stone fill material along the toe of the wall. The cross beams were also tight within their notched out holes in the face of the wall. At only one location (0+010), a cross beam had broken and fallen back into the crib.

Above the waterline the timber crib showed some deterioration. Above the water line there was a row of horizontal spikes protruding out along the face of the timber crib. It was unclear if these spikes were part of the construction of the crib or holding a fender or equipment to the wall. In some places it appeared that the top timber had been set back 15 cm, or the precast concrete block had been placed out further on the timber crib for alignment. Although the outer timber was partially deteriorated, the precast concrete blocks appeared stable. At chainage 0+097 the concrete cap is missing and the top of the wooden crib is more deteriorated than the previous section of the wall.

Two locations were identified above water where surficial deterioration or damage of the timbers were noted. The first approximately between chainage 0+020 to 0+050 where the precast concrete block is out over the cap up to 150 mm. In this area there are spikes (approximately 17mm diameter) driven horizontally into the crib which are holding a 100mm by 300mm timber to the face of the crib. This timber is missing in parts and part is damaged. Where the timber is missing the spikes have been bent downward but are still securely embedded in the timber crib. The crib behind this timber is in good condition. The precast blocks are stable. At chainage 0+070 similar damage to the timber at the water line was noted.

The second location identified starts at chainage 0+097 and ends at the south end of the Outfitting Dock where the concrete cap is missing. The top row of timbers are damaged and/ or deteriorated. Where the concrete cap is missing the stone fill inside the crib was visible. The stone fill was above or at the top of the timber crib. The level of the stone fill within the timber crib under the concrete cap could not be confirmed.

The cast-in-place portion of the concrete cap is reinforced concrete. The reinforcing was visible at the south end concrete cap adjacent to where the cap is missing. There are layers of reinforcement each with 25.4 mm diameter bars spaced at 230 mm with bent ends. The spacing was confirmed at other locations where the bars could be observed due to local breakage or spalling. The concrete slab varied in thickness. The minimum thickness of the slab was 350 mm.

Where the concrete cap remains in place, it showed surficial damage consistent with the age of the concrete and the industrial use. There are construction joints visible at regular intervals of approximately 15 metres. There was minor spalling of the concrete along the interface between the timber crib and concrete block. The lakeside face of the concrete cap appeared to be spalled, cracked and pitted. None of the observed deterioration reduces the structural function of the concrete cap. At the south end of the Outfitting Dock, starting at chainage 0+097, the concrete cap is missing.

i) Detailed Visual Inspection and Testing in Reach 4

A one metre wide by the height of the timber crib area was power washed to remove the zebra mussels at stations 0+030, 0+070 and 0+105. The power washed area showed that the timbers were tight against one another and there did not appeared to be any deterioration of the timbers below the water line. Core samples of the timbers were also taken at these locations. A 25 mm diameter hole drill bit attached to an air drill was used to remove the samples from the timbers. Although the sample hole was clean the small diameter of the drill bit resulted in the sample being destroyed. The diver found that the timbers were hard. Inside the sample hole the wood was light brown in colour and there did not appear to be any discolouration of the wood which would indicate rot. The coring procedure was video taped and is shown on the video tapes.

Terraprobe tested five samples of the concrete cap along the Outfitting Dock, reach 4. Two samples were taken of the precast concrete blocks at chainage 0+064 and 0+090 (core samples 10 and 11). The compressive strength of the concrete was found to be 61.7 MPa and 53.7 MPa respectively. Three samples were taken of the cast in place concrete cap. Each sample was taken 3 metres south of the north (lakeside) face of the wall. The samples were taken at chainages 0+007, 0+024 and 0+086 (core samples 15, 14 and 13). The compressive strength of the concrete was found to be 46.3 MPa, 33.9 MPa, and 60.8 MPa respectively. Overall the concrete testing indicates that the existing concrete cap has a high compressive strength sufficient for its function.

The concrete tests and the observed structure's overall condition did not indicate structural failure or damage under its present loading. The minor timber surficial deterioration and damage does not appear to impact the overall stability and function of the structure under its present loading. The analysis of the stability and structural capacity of the existing structure are presented in Section 3.1.

2.2.2 Reach 5 - West side of Launch Basin

Reach 5 is the west side of the former launch basin. It is approximately 218 metres long and was the location at the shipyards where historically large lakers were constructed and launched over the side of the wall into the basin. The structures along the west side of the basin supported the hull of a laker and all the associated equipment for laker construction. The west shore of the launch basin is comprised of three different cross sections; a timber crib with concrete cap at the north end of the reach, a concrete “crib” with concrete cap south of the north end, and a concrete wall along the south end of the reach. The underwater inspection also revealed that at three locations along the wall there are the remnants of what appears to be old dry dock gate sills.

As for the Outfitting Dock, a base line was established along the length of the wall with the north end of the wall being chainage 0+000 and the south end of the wall being chainage 0+218 (see Figure 1.1). This baseline was used to reference our location along the wall during the diving inspection. Although there are only three types of wall along the west shore of the launch basin, the structure was subdivided into 10 sections which show the variation in the construction or foundation conditions. These 10 sections, M1 to M10 are shown on drawing 03-606LB-06. Description are provided below starting at the north end Section M1.

i) Section M1 (Approximately 20 metres long)

Section M1, located at the north end of the basin, is a timber crib with a concrete cap. This cross section extends from station 0+00 to 0+20. It is a similar in construction to the timber cribs along reach 4. There is a gap up to 0.5 metres wide in the timber cribs between the south end of the Outfitting Dock and the west shore of the launch basin. The timber crib was constructed with 0.3 x 0.3 metre timbers. The exposed face of the crib parallel to the basin is a smooth face of timbers. The front face timbers have been notched out for a timber which runs perpendicular to the face back into the wall. These cross beams are evenly spaced along the wall. Steel rods running perpendicular to the face of the wall were also visible along the face of the wall. Where the concrete cap is missing, it could be seen that spikes driven vertically were used to secure the timbers in place. The timber crib was founded on bedrock.

A cast-in-place concrete cap sits on top of the timber crib. The concrete cap is approximately 6.2 metres wide. A step is located at approximately 2.3 metres from the lakeside edge of the concrete cap and is approximately 2.3 metres wide. Adjacent the water, the top elevation of the concrete cap is approximately 176.6 metres. The wall steps up to an elevation of 176.8 metres. The wall then steps down to an elevation of approximately 176.6 metres at the back of the wall. The bottom of the concrete or top of the timber crib was at elevation 176.2 metres. Cross section M1 on drawing 03-606LB-6 shows a typical section of the wall with the timber crib at this location.

At the north end of wall, approximately 2 metres of the concrete cap adjacent the south end of the Outfitting Dock is missing. The timbers are exposed and the stone filled crib is visible. The top layer of timbers are deteriorated on the surface and damaged.

The diver probed with a screw driver over the height of the wall at 10 metre intervals along the wall and did not find any areas of deterioration. Below the water line the timbers are tight up against one another and are in good condition. There were no missing timbers. Below the water line there was no evidence of any loose spikes or steel rods and the cross beams are tight. The face of the wall is vertical. Overall the diver indicated that there was no distortions of the crib or damage to the crib in this area below the waterline. The diver also found that the timber crib was founded on bedrock. The elevation of the toe of the wall was found to be approximately 170.4 metres. The diver did not note any stone fill along the toe of the wall that would indicate loss of stone fill from the crib.

In summary, the concrete along two metres at the north end of this section of wall is missing and reinforcing bars are exposed The top layer of the timber crib is damaged in this area. The concrete cap along the waterline in the remainder of the reach showed deterioration with some exposed reinforcing bar. The top of the concrete wall was cracked and spalled but not structurally damaged. The timber crib, despite the deterioration at the waterline, does not show any indication of structural failure or damage under its present loading.

ii) Section M2 (Approximately 11 metres long)

The cross section of the wall changes to a concrete “crib” with concrete cap between chainages 0+020 to 0+031. The concrete cap is the same as the adjacent section to the north. The concrete cap has a bottom elevation 176.2 metres and has a top elevation of 176.6 metres at the water’s edge. The top of the concrete cap along this section of the wall has minor cracks and spalling. There is a horizontal steel channel along the bottom of the concrete cap. The concrete cap was spalled along the edge of the channel.

The concrete “crib” is made up a concrete wall supported between two vertical steel “H” piles at both ends of the section. The diver probed behind the wall at the joint between the adjacent concrete wall to the south and the steel H pile. There is hard material behind the concrete “crib” wall but he was unable to determine what it was. The diver also found that the wall is founded on bedrock. There was a small gap approximately 25 mm high between the bottom of the concrete “crib” wall and the bedrock floor. The elevation of the bedrock was approximately 173.0 m at this section of the wall. Cross section M2 on drawing 03-606LB-6 shows a typical section for this section of the wall.

iii) Section M3 to M10 (Approximately 187 metres long)

The third type of wall found along the west side of the launch basin is a concrete wall. It extends from station 0+031 to 0+218 at the south end. The wall extends from a top elevation of 176.6 metres at the lakeside edge, down to bedrock. The top of the concrete wall is approximately 4.6 metres wide. There is a step located at approximately 2.3 metres from the lakeside edge of the concrete and the upper deck is approximately 2.3 metres wide. The deck steps up approximately 0.2 metres to an elevation of 176.8 m. The top of the concrete wall has surficial spalling and some minor cracking. The concrete deck was repaired in the 1970s. However, the details of the repair are not known. Elevations taken along the wall showed that the top elevation of the wall was consistent along the wall.

The face of the concrete wall extends to bedrock. Cross sections M3 to M10 on drawing 03-606LB-6 show typical sections along the wall from chainage 0+031 to 0+218. These variations in the cross sections are due to the condition of the bedrock and the other features along the concrete wall. The variations are discussed below.

Between chainages 0+031 and 0+035, section M3, there were the remnants of what is appears to be an old sill. However, reviewed plans of the basin do not show this structure and there is no sill across the bottom of the basin for a gate to enclose. At the bottom of the wall, the concrete sill protrudes into the basin approximately 0.7 m at the south end and tapers towards the existing wall to the north. Vertically the top of the wall protrudes 0.2 m from the adjacent wall and tapers at approximate a 45 degree angle until it is 0.7 m out from the adjacent wall.

Between chainages 0+035 and 0+040, section M4, the concrete wall is founded on bedrock. The bedrock slopes down from the toe of the concrete wall at elevation 173.5 metres to an elevation 169.0 metres. The toe of the bedrock was approximately 2.0 metres lakeward from the face of the wall. The concrete wall along this section has areas that were spalled and has cracks. The diver did not note any significant deterioration of the wall along this section.

Between chainages 0+040 and 0+052, section M5, the concrete wall extends down to an elevation of approximately 171.5 metres. At the toe of the wall the bedrock extends out 0.2 to 1.0 m from the face of the wall and then drops off to the floor of the basin which is at an elevation of approximately 170.5 metres at this section of the wall. At the south end of this section concrete has been poured over the bedrock. The wall shows some signs of non structural, surface deterioration below the waterline to the bottom, such as spalling and cracks.

At two locations along the wall, between chainages 0+052 to 0+056 and 0+070 to 0+075, there are the remnants of the sill for the gate that was used for the dry dock. Section M6 shows a typical section through the sill. A historical photograph of the shipyards (Figure 1.2) shows a gate which corresponds with what the diver found at chainage 0+070 to 0+075. At the bottom of the wall the concrete protrudes approximately 1.0 m into the basin at the south end and tapers to the north end of the section to be in line with the adjacent section of wall. Vertically, starting approximately 0.75 m below datum the wall tapers out at approximately a 45 degree angle to be 1.0 m out from the face of the wall and continues vertically to the bedrock bottom.

Large pieces of concrete which appear to have been once part of the sill section of the wall were found on the bottom. The section of the wall, between chainage 0+070 and 0+075, has exposed reinforcing bars were the concrete had been broken off. The diver found it difficult to determine the actual length of the sill because of the missing concrete. There was no evidence of the sill across the bottom of the basin out from the wall. It is likely that the sill has been entirely removed across the bottom of the basin to accommodate the launching of ships into the basin when the basin ceased to be a dry dock.

Between chainages 0+056 to 0+070 and 0+075 to 0+100, the concrete wall extends down to the bottom of the basin floor. Section M7 shows a typical section through the wall along these sections of shoreline. The elevation of the bottom of the basin and of the toe of the wall was estimated to be 171.0 and 170.5 metres based on the divers observations. Approximately 1.1 to 1.5 metres above the soft sediment line there is a crevis that showed a separation in the concrete. The diver found that the separation is on average 10 cm wide and up to 30 cm wide and the diver could probe back on average 0.5 metres and up to 0.7 metres into the wall. The diver also found areas of spalling and cracking. At chainage 0+056 there is an area where concrete protrudes out approximately 0.1 metres from the face of the wall approximately 1.0 metre long and 0.5 to 0.6 metres high from the toe of the wall. At 0+065 there was a large piece of concrete at the toe of the wall. There was a crevice in the concrete wall 1.1 m high above the soft sediment line, 1.0 metre wide and 1.2 metres deep. The back of the crevice was hard like concrete or rock. The large piece of concrete most likely came from the crevice.

Between chainage 0+100 and 0+155 the concrete wall is similar to the adjacent section of wall to the north. The concrete wall extends to the bottom of the basin floor. The soft sediment line was found to vary between 4.7 and 5.2 metres below the top of the wall. The diver found a hard bottom approximately 1.0 m below the soft sediment line. As with the previous section, the diver found that there was a crevis in the wall approximately 1.0 to 1.2 metres above the soft sediment line. The crevis varied in width 8 to 10 cm and varied in depth between 8 to 6 cm. Section M8 shows a typical section of the wall at this location.

There was more surficial deterioration of the wall along this section than in the previous section to the north. At chainage 0+115, 10 cm below the water line is a horizontal crack approximately 8 cm deep and tapered down to 4 cm deep 2 metres along the wall. At approximately station 0+137 to 0+143 there is a crack in the wall approximately 0.5 metres below the waterline. The diver reported that it varies in height between 0 and 60 cm and is approximately 8 to 70 cm deep. At chainage 0+146, a crevice starts 2 metres below the water line and extends to the soft sediment line. The crevice is 20 cm deep at its deepest point . The stone in the concrete was visible and there was a rust coloured discolouration on the wall. As well, there is some spalling along the soft sediment line.

The concrete wall between chainage 0+155 to 0+170 has a step in the wall which is about 0.5 metres above the soft sediment line. The step is varies in width from 2 to 10 cm. Section M9 shows a typical section through the wall at this location. There are crevices in the concrete which the diver described as “pop outs” that are on average 2 cm and up to 8 cm deep. Approximately 2 metres below the water line there is a crack running along the length of this section of the wall. Over all this section the face of the wall has spalled concrete from the water line to the bottom. The diver was able to break off at the edges of the “pop outs” small surficial pieces of concrete off the face of the wall. Although it can not be confirmed, the “pop out” would be consistent with forceful removal of underwater formwork that may have been left in place after the repairs in the 1970s.

The wall between chainage 0+170 and 0+218 appeared to be similarly constructed as adjacent section to the north. Section M10 shows a typical section through the wall along this chainage. Approximately 2.0 m up from the soft sediment line the diver noted a “joint” in the wall. The concrete appeared to be bevelled around the joint. The diver broke off pieces of the concrete around the joint with his hand. The distance from the top of the wall to the soft sediment line varied between 4.3 m to 4.9 m. The silt was approximately 1.0 m deep. There were cracks along the wall. At chainage 0+173 approximately 1.0 metre above the soft sediment line there is a crevice that is 25 cm deep and 1 metre long. At chainage 0+180 there are two cracks that cross that are 5 metres long and vary in width between 1 cm to 10 cm.

At chainage 0+185 there is a crevices in the concrete starting 2 metres below the surface, 2 metres high, 2 metre wide and 30 cm deep at the top and midway down then cuts out to the toe of the wall. There is an exposed 1" diameter horizontal reinforcing bar in the crevice.

The wall ends at the concrete gravity wall at the south end of the basin. The interface between the south concrete wall and the west side concrete wall is in good condition.

iv) Detailed Visual Inspections and Testing in Reach 5

Four sections, one metre wide by the height of the wall, along reach 5 where cleared of zebra mussels with a power washer. At chainage 0+021, the south end of section M1, the wall was cleared. This revealed that the timbers were tight against one another and the face appeared smooth. A core sample was taken of the a timber half way down from the top of the crib. The coring was completed using a compressed air drill with a 25 mm diameter drill bit, in the same way described for the Outfitting Dock. The sample was destroyed due to the small diameter of the drill bit. The sample hole was clean and the wood felt hard when probed with a screw driver. Inside the sample hole, the timber was light reddish brown in colour with no evidence of rot.

At chainage 0+097, a section of the wall, that was considered to be typical of this portion of the wall, was power washed to clean off the zebra mussels. The separation of the concrete was more visible. It was approximately 8 cm deep and 1.0 m above the soft sediment line. This crevis extended from chainage 0+080 to 0+112. The crevis corresponds with the bedrock elevation along this section of the wall. The face has cracks and areas of spalled concrete. The height of the wall above the soft sediment line was found to be approximately 5.0 m. The diver found a hard bottom 1.3 m below the soft sediment line.

The wall was also cleared at chainage 0+150. The one metre wide by the height of the wall area was cleaned. Overall the face of the concrete wall is spalled. The spalling is on average 2 cm deep.

At chainage 0+190 a fourth one metre wide by the height of the wall was cleared. The face of the concrete wall is spalled and uneven. The spalling is on average 2 cm deep.

Terraprobe Testing Limited collected concrete core samples of the existing and concrete walls in reach 5. The samples were tested for unconfined compressive strength. A concrete sample was taken of the upper deck of the concrete cap at chainage 0+078 (core sample 5). It was found that the concrete had a compressive strength of 47.1 MPa. A sample was take of the lower deck but the sample was cracked so the compressive strength could not be tested. At chainage 0+153 samples were taken of the upper and lower deck of the concrete cap (core samples 3 and 4). The compressive strength of the samples were 44.5 MPa and 49.6 MPa respectively. At chainage 0+210 samples were taken of the concrete cap on both the upper and lower deck (core samples 1 and 2). It was found that the concrete had a compressive strength of 35.9 MPa and 68.1 MPa respectively.

2.2.3 Reach 6 - South Shore of Launch Basin

Reach 6 - South Shore of Launch Basin, is located at the south end of the subject property. The reach is 64 metres long. A baseline was established along this reach starting at the southwest corner of the basin proceeding east, see Figure 1.1.

The south shore of the Launch Basin is protected with a concrete gravity wall. The existing concrete gravity wall starts approximately eight metres west of the west side of the launch basin. The top of the wall gradually slopes up from an elevation of 177.8 metres at west end of the wall to 178.6 metres at west side of the launch basin. The wall has a top elevation of 178.6 metres for

23 metres east of the west side of the basin. Then the wall bends approximately 60 degrees to the north and the crest slopes down to an elevation of 178.4 metres where it meets the east side of the basin. Along the east side of the basin the wall continues to slope down to an elevation of 177.7 metres over 12 metres.

Previously, the wall was excavated to determine the back slope, depth and conditions of the structure, (Shoreplan, November 2004). The wall has a crest width of 350 mm. The front face of the wall has a 4.5 degree batter and the back face has a slope which was estimated to be 25 degrees. There was no damage or deterioration observed on the back side of the wall. Bedrock is encountered 4.5 metres below the crest of the wall. It was estimated that the bedrock elevation was 174.1 metres. This is consistent with the bedrock elevation in Terraprobe (2003).

The diving inspection of the south shore of the launch basin revealed that the concrete structure shows very little deterioration and only one area of damage. Overall the wall is smooth. The east side of the wall has surface cracks and spalling. At the south east corner there is a round hole which appears to be a drain hole 3 metres from the bottom of the wall. These drain holes were found along the entire length of the wall and they were approximately 2.5 metres apart. The wall surface is smooth and the face is straight. Further along the south section (approximately chainage 0+022) of the wall there was a crack at a joint in the middle of the wall. There is a triangular protrusion that started 0.5 metres from the bottom and is 1.5 metres high and 0.5 metres wide. The concrete protruded 50 to 75 mm from the face of the wall. Further along the wall to the west the wall appears to be good condition.

i) Detailed Visual Inspection and Testing in Reach 6

An area one metre wide by the height of the wall was power washed at two locations in Reach 6. The first location was at the joint with the triangular protrusion identified at chainage 0+022. The second location was at the intersection of the southeast wall and east wall (chainage 0+052). At both locations the concrete wall has minor spalling and cracking.

Core samples were taken along the concrete wall at three locations, 0+014, 0+035, and 0+048 (core samples 9, 8, and 7). It was found that the concrete had a compressive strength of 42.7 MPa, 33.5 MPa and 32.8 MPa respectively.

3.0 DESIGN OF SHORELINE IMPROVEMENTS

Overall the existing structures along the Outfitting Dock and west side of the launch basin, despite some deterioration along the waterline, do not show any signs of structural failure or damage under their present loading. The diving inspection did not identify any significant deterioration below the waterline and damage is minor and localized and can be associated with the industrial operations at the site in the past. However, these old structures were built for a specific industrial use. The appearance of these structures are not compatible with the proposed uses. The top elevations of the structures are also lower than required to address the flood hazard at this site. For these reasons and to extend the service life of the structure and account for any unknowns associated with the existing structures, it has been proposed that the existing shore protection works be structurally reinforced by adding an anchored steel sheet pile wall and associated elements.

Section 3.1 describes the shoreline hazards at this site and methods of addressing them at this part of the site. Sections 3.2 and 3.3 describe the overall design details for the proposed shoreline structures.

3.1 Shoreline Hazards

The Provincial Policy Statement (PPS) identifies three potential shoreline hazards along the shores of the Great Lakes. They are the flood, erosion and dynamic beach hazards. In order to develop this site these hazards must be addressed. The shoreline flood and erosion hazards at this site are discussed below. There is no dynamic beach hazard at this site.

3.1.1 Flood Hazard

The flood hazard is defined as the 100 year return period peak instantaneous water level plus an allowance for wave uprush. The wave uprush allowance for the Great Lakes is 15 metres, measured horizontally from the 100 year peak instantaneous water level, unless a site specific study is completed to determine a wave uprush elevation. A site specific wave analysis was undertaken for this site and the results were presented in Shoreplan (2004a).

The wave analysis looked at waves generated in the offshore that is on Georgian Bay and penetrating into the harbour and locally generated waves, that is wind generated waves over the smaller fetches inside Collingwood Harbour. It was found that locally generated waves created the design wave conditions for this site.

Shorelines in Reach 4 - Outfitting Dock and Reach 5 - West Side of Launch Basin are oriented in a northwesterly direction. The design wave generated by northwesterly winds (Shoreplan, 2004a) runs parallel to the structures and will not produce maximum wave uprush. In order to determine the wave uprush elevation along these structures, locally generated waves perpendicular to the structures were also considered. The wave uprush elevations for reaches 4, 5 and 6 were assessed using ACES (1992) for waves perpendicular to the structures. Wave uprush elevation for the waves running parallel to the structure were approximated by using 60% of the maximum wave height parallel to the structure above the design high water level. This is based on an approximation of a first order wave profile above water level with a 20% increase in height . It was found that the maximum wave uprush elevation for reach 4 and the north part of reach 5 (approximately north 30 metres) is 178.5 metres. The remainder of reach 5 is subjected to waves with a maximum uprush elevation of approximately 178.3 metres. The wave uprush elevation for Reach 6 is 178.7 metres. The structures were designed for no overtopping except as described below. Overtopping rates were calculated using the methods outlined in the Technical Guide (MNR, 2001) for the waves perpendicular to the shore. Table 3.1 provides the design wave height and run up elevation and overtopping rate if the wave overtops the structure.

Table 3.1 - Design Wave Height, Runup and Overtopping Rate

Reach	Crest Elevation (m)	Wave Travel Direction Relative to Shore	Wave Height (m)	Run Up Elevation (m)	Overtopping Rate (l/m/s)
4	178.4 to 179.0	Parallel to Shore	0.61	178.4	n/a
4 (0+000 - 0+096)	178.5	Perpendicular to Shore	0.24	178.5	n/a
4 (0+096 - 0+104)	178.4	Perpendicular to Shore	0.24	178.5	4
5 (0+000-0+199)	178.3 to 178.4	Parallel to Shore	0.32	178.2	n/a
5 (0+199-0+220)	178.3 to 178.0	Parallel to Shore	0.32	178.2	See Note 1
5 (0+000-0+030)	178.4	Perpendicular to Shore	0.24	178.5	4
5 (0+030-0+199)	178.3	Perpendicular to Shore	0.15	178.3	n/a
5 (0+197-0+220)	178.0 to 178.3	Perpendicular to Shore	0.15	178.3	See Note 1
6	178.0	Perpendicular to Shore	0.32	178.6	See Note 1

Note 1) Backshore is designed by others as the Regional Storm flood way.

In reach 4, the proposed top elevation of the steel sheet pile wall with concrete cap slopes from 179.0 metres at the northwest end to 178.5 metres to chainage 0+040. From 0+040 to 0+096 the top elevation of the wall remains constant at 178.5 metres. From 0+096 to the southeast corner the top elevations slopes from 178.5 metres to 178.4 metres. Based on the coastal analysis of the site discussed above, the wave uprush elevation under design conditions along reach 4 is 178.5 metres. Overtopping occurs in reach 4 southwest of chainage 0+096 to the end of the reach (approximately 8 metres) onto the top of the structure. The backshore is designed for overtopping. The land adjacent to the crest of the wall is intended to be a promenade along the waterfront.

Landward of the promenade the grade is proposed to be approximately 179.2 metres, which is above the design wave uprush elevation. The promenade will be treated with a non erodible surface and there will be positive drainage towards the lake. Drawing L2 by MBTW provides details of the grading and landscaping proposed for in this area.

In reach 5, the proposed top elevation of the steel sheet pile wall with concrete cap varies between 178.4 metres at the north end to 178.0 metres at the south end with most of the structure having a top elevation of 178.3 metres. Based on the coastal analysis of the site discussed above, the wave uprush elevation was found to be 178.5 metres at the north 30 metres of the structure and 178.3 metres for the remaining 190 metres of wall. It is expected that the north end of the structure (north 30 metres) will be overtopped during the design storm conditions. The south end of the basin will be a flood way for the Regional Storm. The area will also be overtopped during the design storm. It is designed to withstand flooding and overtopping. As discussed in reach 4, the land immediately adjacent wall is intended to be a promenade along the waterfront. The area will be a treated with a non erodible surface and there will be positive drainage towards the lake. The surface will withstand wave overtopping. Landward of the walkway the elevation of the table land will be approximately 178.9, which is higher than the wave uprush elevation. Drawing L2 by MBTW provides details of the grading and landscaping proposed for in this area.

In reach 6, the proposed top elevation of the wall at the south end is 178.0 metres and 178.3 metres along the southeast of the basin. These elevations are below the wave uprush elevation for this end of the basin. The structure's crest elevation was set to 178.0 metres at the south end of Hurontario Street to allow for drainage of the Regional Storm. As a result, the structure will be overtopped during the design storm in this area. The area landward of the wall will also be part of the promenade. It will be finished similar to the other reaches with a non erodible surface. This backshore area will be designed to weather both the runoff from the Regional Storm and overtopping during the 100-year return period design storm by others. Details of the backshore finishing and grading are provided in drawings prepared by MBTW and drawings prepared by C.F. Crozier and Associates. The overtopping rate is such that access to the shoreline should be controlled by the town during the design storm on the lake and during the Regional Storm.

3.1.2 Erosion Hazard

An erosion hazard is normally defined by a stable slope allowance plus 100 times the average annual recession rate. Where recession rate data is unavailable, the provincial policy defines the erosion rate as 0.3 metres per year. The stable slope allowance is 3 times the height of the bank unless defined by a site specific geotechnical investigation.

It is our view that a standard approach to an assessment of the erosion hazard, as described in the provincial policy, does not strictly apply. This site condition are artificially created and rely on shore structures for stability. The harbour was created in the 1800's and has been protected by structures since that time. The Provincial Policy and guidelines allow development within the less hazardous portion of the hazard land if the hazard can be overcome. The erosion hazard at this site will be addressed by upgrading the existing shoreline protection structures such that they have a minimum design life of 50 years and providing maintenance access for any future repairs to the wall and their maintenance in perpetuity. Details of the structural design are discussed in the following sections.

3.2 Design Standards and Guidelines

The standards and guidelines used to design the shoreline protection are provided in Section 5.2 of Schedule “G” of The “Shipyards” Master Development Agreement, which is attached in Appendix H. The following is a list of standards and guidelines used specifically for this design.

- National Building Code of Canada
- Canadian Foundation and Engineering Manual (4^{th} edition)
- CAN/CSA S6-06 Canadian Highway and Bridge Design Code
- CSA A23.1-04 Concrete Materials and Methods of Concrete Construction and CSA A23.2-04 Methods of Test and Standard Practices for Concrete
- CSA A23.3 Design of Concrete Structures for Buildings
- Handbook of Steel Construction
- Coastal Engineering Manual (USACE, 2002)
- Technical Guide for Flooding, Erosion and Dynamic Beach (MNR, XX)
- Great Lakes Small-Craft Harbour and Structure Design for Ice Conditions: An Engineering Manual (Wortley, 1984).

3.3 General Description of the Design of Structures

3.3.1 Design Parameters and Loads

The proposed steel sheet pile wall will be pinned along the toe of the wall to bedrock in front of the existing structures and anchored at the top of the wall to either a concrete anchor wall or the existing concrete cap, along the Outfitting Dock. The earth pressures acting on the proposed wall are based on the design soil parameters reported in Terraprobe 2003 (confirmed in the letter from Terraprobe dated June 10,2008 and included in Appendix A). Appendix A provides the boreholes information and maps of the bedrock elevation. Table 3.1 summarizes the earth pressure design parameters used for the design of the steel sheet pile wall and analysis of the existing walls.

Historical photographs show that the area behind the outfitting dock and launch basin were once underwater and subsequently filled. Terraprobe 2003, (confirmed in letter from Terraprobe dated June 10, 2008 included in appendix A) describe the soil in the vicinity of the existing structures as either loose sand fill (γ = 19 kN/m^3, Ka = 0.36) or compact native sand (γ = 21 kN/m^3, Ka = 0.31). It was determined through the structural analysis of the wall that the worse case condition occurred when the design parameters for the compact native sand were used.

Table 3.2
Earth Pressure Design Parameters (Terraprobe, 2003)

Soil	Internal Angle of Friction, ϕ (degrees)	Bulk Unit Weight of Soil, γ (kN/m^3)	Active Earth Pressure Coefficient (Rankin), K_a	At-rest Earth Pressure Coefficient (Rankin), K_o	Passive Earth Pressure Coefficient (Rankin), K_p
Compacted Granular Fill Granular ‘B’, (OPSS 1010)	32	20	0.31	0.47	3.25
Earth Fill	28	19	0.36	0.53	2.77
Undisturbed sand and silt	32	21	0.31	0.47	3.25
Bedrock	28	24	-	-	-

A surcharge load of 10 kPa was included in the design loads to reflect pedestrian traffic and light maintenance vehicles along the wall. A moored boat load of 1 kN per metre of wall was also included in the design loads.

The wall cross section along the west side of the basin, reach 5, could not be ascertained during our investigations without destroying the wall. Consequently, the design of the shore protection works assumes that the existing concrete wall along the west side of the launch basin is concrete rubble. The steel sheet pile wall was designed for the concrete rubble having a unit weight (γ) of 20 kN/m^3, an active earth pressure coefficient (K_a) of 0.31, and an internal angle of friction (ϕ) 32°.

The design high water level (DHWL) used for the design of the steel sheet pile wall, timber crib and concrete wall was the 100 year peak instantaneous water level, 178.0 m. The design water level used for the concrete anchor was the maximum monthly mean lake level, 177.5 m. The design low water level (DLWL) was based on the minimum monthly mean lake level of 175.6 m.

Ice forces acting both horizontally and vertically (uplift) on the wall were also considered in the design of the wall. The horizontal ice force acting on the wall was not included in the design condition as it did not contribute to the worse case condition. The bottom of the concrete cap is situated above the ice. Therefore the cap was not designed for uplift force due to ice. The tie rods were designed to withstand the shear force the ice acting on the exposed ends of the tie rods.

Terraprobe, 2003 describes the bedrock “predominantly of limestone/dolostone of the Simcoe Group. The rock is of medium strength and it is generally competent.” The unconfined compressive strength of the limestone bedrock was estimated to be between 20 to 50 MPa. A compressive strength of 20 MPa was used for the design. The design bearing pressure is noted to be 5000 kPa.

The bedrock elevation used in the design is based on the observations of the diver while inspecting the existing structures and the mapping shown in Figure 3 of Terraprobe, 2003.

Figures 3.1 to 3.6 show typical sections of the existing wall and the proposed structure. Each figure summarizes the geotechnical parameters used for the analysis of the existing and proposed wall and the design loads applied to the proposed wall. Four design cases were analysed for each typical section of wall. The cases are as follows;

- Case 1 - DLWL, no live load on existing structure and 10 kPa live load behind existing structure;
- Case 2 - DLWL and 10 kPa live load behind and on entire structure;
- Case 3 - DHWL, no live load on existing structure, and 10 kPa live load behind existing structure;and
- Case 4 - DHWL and 10 kPa live load behind and on entire structure.

Table 3.3 summarizes the existing structure stability with the proposed new steel sheet pile wall for sections L1, L2 and M1 where the existing timber crib and concrete cap are components of the new structure. The stability analysis of the existing gravity wall, section N1, are also included in the table. Section N1 was also analysed for the Regional Storm (Case 5) which is also included in Table 3.3. Table 3.4 provides the loads on the anchors for sections L1, L2, M1, M3 and M5 for each of the four cases. Table 3.4 also provides the design loads for the lookout. Table 3.5 provides the loads resisted by and applied to the major components of the structure after construction. Table 3.6 provides the loads resisted by and applied to the major structural components after a design life of 50 years.

The proposed steel sheet pile wall and associated components (e.g. tie rods, wale and bolts) were selected based on their ability to resist the applied force at the end of their design life. A corrosion rate of steel structures was applied based on its environment. Pile Buck, 2007 reports that for the immersed/ semi- immersed zones an average rate of corrosion is 2.5 mils of a inch per year (0.06mm/year). It is also noted that, “Others describe rates of from 2 to 5 mils per year for the first several years, after which the rate decreases to an insignificant amount”. For the purpose of this design a corrosion rate of 0.06 mm per year was used for steel products. It is estimated that the steel sheet pile wall will have a minimum design life of 50 years.

3.3.2 Reach 4 - Outfitting Dock (Town Dock)

The proposed work in this reach includes; the replacement of deteriorated timbers, grouting of voids between the existing concrete cap and crib fill, and construction of a steel sheet pile wall with an anchor system and a concrete cap. Granular ‘B’ backfill will be placed behind the steel sheet pile wall to the grades specified on the drawings.

The steel sheet pile wall will be anchored at the top to the existing concrete cap. The anchor system will consist of a tie rod anchored to the steel sheet pile wall at elevation 177.8 metres and to a steel plate anchored into the landward side of the existing concrete cap with two anchor bolts. The toe of the wall will be pinned to bedrock. The pin was sized to resist both the force and moment applied if there is a gap of 200 mm between the bedrock and the bottom of the steel sheet pile. The pin will be grouted into the bedrock. The gap between the existing structure and sheet pile will be filled with tremie concrete and clear stone. Figure 3.1 Section L1 shows a typical section with the existing concrete cap and proposed steel sheet pile wall and the design parameters and loads used for the design of the wall at this location. Drawing 03-606LB-3 shows a detailed site plan for the Outfitting Dock and drawings 03-606LB-6, 7 and 9 show typical sections and details of the wall.

Where the concrete cap is missing at the south end of the reach (Section L2), the top of the steel sheet pile wall will be anchored to a cast-in-place continuous concrete anchor. The toe of the wall will be pinned as described above. The gap between the existing structure and steel sheet pile wall will be filled with tremie concrete and clear stone. Granular 'B' backfill will be placed and compacted behind the wall to the grades specified on the drawings. Figure 3.2 Section L2 shows a typical section where the concrete cap is missing and the design parameters and loads used for the design of the wall at this location.

The existing reinforced concrete cap was analysed to determine if it could support the proposed load. The existing cross section of the concrete cap is as shown in Figure 3.1, Section L1. The slab varied in thickness. Therefore the slab was analysed with a minimum thickness of approximately 350 mm. Our observations at the south end of the slab indicate that there are two layers of reinforcement each with 25.4 mm diameter spaced at approximately 230 mm with bent ends. The same spacing was confirmed at other locations where the bars could be observed due to local breakage or spalling. Details of the design loads acting on the slab and reinforcement in the cap are shown in the Figure 3.1. The analysis indicates that the slab will support the applied load as shown in Table 3.3. However, the subsequent decision to grout the void will result in greater support to the concrete slab.

The concrete cap is resting on precast concrete blocks which are supported on the timber crib. The proposed steel sheet pile wall will be anchored to the back of the existing concrete cap and the area between the concrete cap and stone filled cribs will be grouted. The sliding of the concrete cap on the stone and grout filled crib due to the horizontal loads, that is the anchor load and lateral earth pressure were checked and it was found to be stable. The results are presented in Table 3.4.

The stability analysis of section L1, stone filled timber crib and concrete cap, and section L2, stone filled timber crib, are capable of withstanding the additional loads with the factors of safety greater than 1.5 for sliding and two for overturning. It was also calculated that the resultant force from all applied forces fell within the middle third of the base of the structure. The base bearing pressure was found to be less than the design bearing pressure, 5000 kPa, for the limestone bedrock at this site.

3.3.3 Reach 5 - West side of Launch Basin

There are three different structures along the shoreline of this reach. Section 2.1.2 describe these structures and their existing condition. The top elevation of the proposed structure along this reach varies from 178.4 metres at the north end to 178.0 metres at the south end, with approximately 193 metres of wall having a constant elevation of 178.3 metres. The existing concrete cap has a top elevation of 176.6 metres. Therefore, approximately a 1.7 metre thick layer of backfill will be placed on top of the existing structure. A steel sheet pile wall will be constructed in front of the existing structure and will retain the proposed backfill. The steel sheet pile wall will be pinned to bedrock at the toe of the wall and anchored at the top of the wall to a cast-in-place continuous concrete anchor.

The wall will be anchored into the bedrock floor of the basin using steel pins. Steel pipe will be welded to the steel sheet pile inpans which will act as a guide for the pins. Holes will be drilled into the bedrock and the steel pin inserted and grouted.

During the diving inspection it was found that at some locations at the north end of the launch basin the bedrock sloped down from the base of the wall to the basin floor. At these locations it is proposed that the bedrock be removed to form a vertical face to the basin floor. The bedrock removal will be carried out by line drilling in front of the existing wall and breaking away the bedrock with a hydraulic hammer to minimize potential for fracturing the rock. The bottom of the cavity between the new steel sheet pile wall and the existing wall will be filled with tremie concrete and clear stone above that. The steel sheet pile wall will be finished with a concrete cap. Drawing 03-606LB-2 shows a detailed site plan of the west side of the launch basin. Drawings 03-606LB-6, 7 and 9 show typical sections and details.

The most northerly part of the existing structure along this reach is a timber crib with concrete cap similar to reach 4, see Figure 3.3 Section M1. A stability analysis of the existing timber crib found that it was capable of withstanding the additional loads. The factor of safety is greater than 1.5 for sliding and two for overturning, as shown in Table 3.3. It was also calculated that the resultant force from all applied forces fell within the middle third of the base of the structure. The base bearing pressure was found to be less than the design bearing pressure, 5000KPa, for the limestone bedrock at this site.

The other structures along this reach were concrete. Although the diving inspection and the concrete core samples of the concrete wall did not indicate any structural instability, details of construction of the existing structures could not be ascertained through these investigations. Therefore the steel sheet pile wall was designed to retain concrete rubble and backfill above the elevation of the bedrock. The bedrock elevation was based on the mapping provided in Terraprobe, 2003 and observations during the diving inspection. Figures 3.4 and 3.5 show typical sections M3 and M5 along the wall, the geotechnical parameters and design loads.

3.3.4 Reach 6 - South Shore of Launch Basin

The diving and previous inspections of the structure along the south shore showed that the existing concrete gravity wall does not show and sign of structural failure or damage with its present loading. The proposed improvements will increase the overall stability of the wall, accommodate site flood conditions, and improve the appearance of the structure.

The portion of the wall that is at the end of Hurontario Street will become a flood way for the regional storm. As such, the top elevation of the wall will be reduced to 178.0 metres. Where the wall bends to the north the top elevation will gradually rise from 178.0 metres at the southwest end 178.3 metres at the northeast end.

It is proposed that the top of the existing concrete gravity wall be cut off to allow the construction of a new concrete cap consistent with the concrete cap proposed along the Outfitting Dock and west side of the launch basin. The top of the existing wall will be cut at an elevation of 177.5 metres along the south end of the basin, at 177.5 to 177.8 metres along the southeast side.

Reinforcing bars will be grouted into the existing wall and the new concrete cap will be formed to the elevations described in the above paragraph.

Rock anchors will be installed along the back of the wall. A ledge will be cut into the existing wall and a rectangular hollow structural steel beam will be installed along the ledge. The rock anchors will be installed through the beam and existing concrete wall into bedrock.

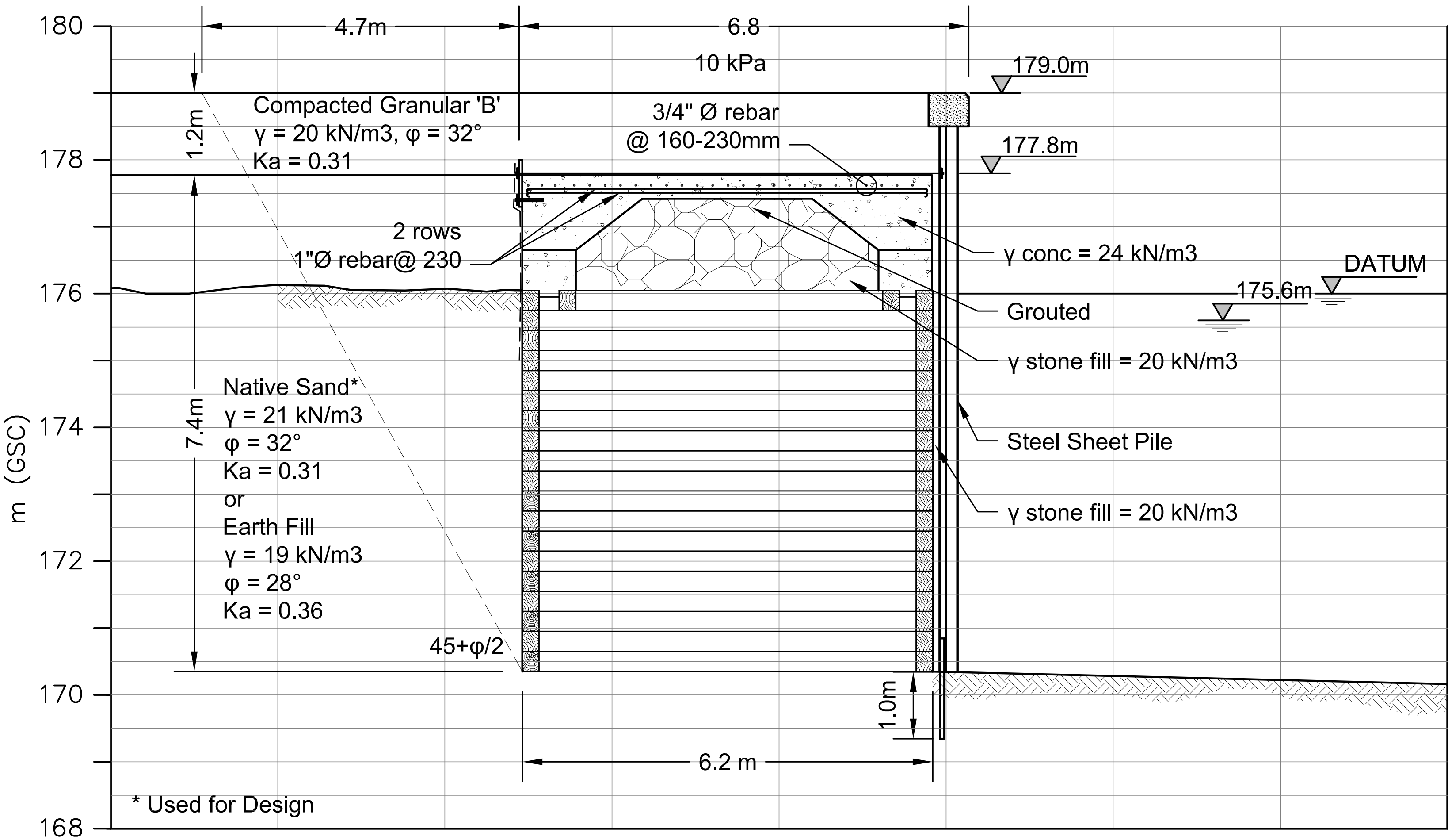
The limit of construction for this project is the east end of the south wall along the end of the launch basin. Although the existing wall continues along the east side of the basin, this wall has not been included in this work. Drawing 03-606LB-3 shows a detailed site plan of the south end of the launch basin. Drawings 03-606LB-6, 7, 8 and 9 show typical sections and details.

Figure 3.6, Section N1, shows the geotechnical parameters used to analysis the wall and the design loads. Section N1 will be the flood way for the regional storm. Analysis of the wall was carried out for the worse case condition of the regional flood and a low water level of 175.6 m in the basin. Analysis of the new wall conditions showed that the gravity wall was capable of withstanding the additional load . The factor of safety sliding was 1.7 and 3.7 for overturning, as shown in Table 3.3. It was also calculated that the resultant force from all applied forces fell within the middle third of the base of the structure. The base bearing pressure was found to be less than 5000 kPa.

3.3.5 Lookout

It is proposed that a lookout be installed along reach 5 between chainage 0+117 and 0+127. The look out will be 9.6 m long and extend 2.4 metres lakeward of the front face of the proposed steel sheet pile wall. The lookout will consist of a permeable steel deck supported on joists which are supported on four beams. The beams are supported on two columns which are anchored to bedrock. Drawing 03-606LB-2 shows a detailed site plan of the west side of the launch basin. Drawings 03-606LB-8 shows details of the Lookout.

The lookout was designed for a live load of 5 kPa and a wave uplift force of 28 kPa for a short duration and 3.5 kPa wave uplift force for a sustained condition. Table 3.5 and 3.6 provide the resisted and applied loads to the major structural components of the lookout after construction and after 50 years.

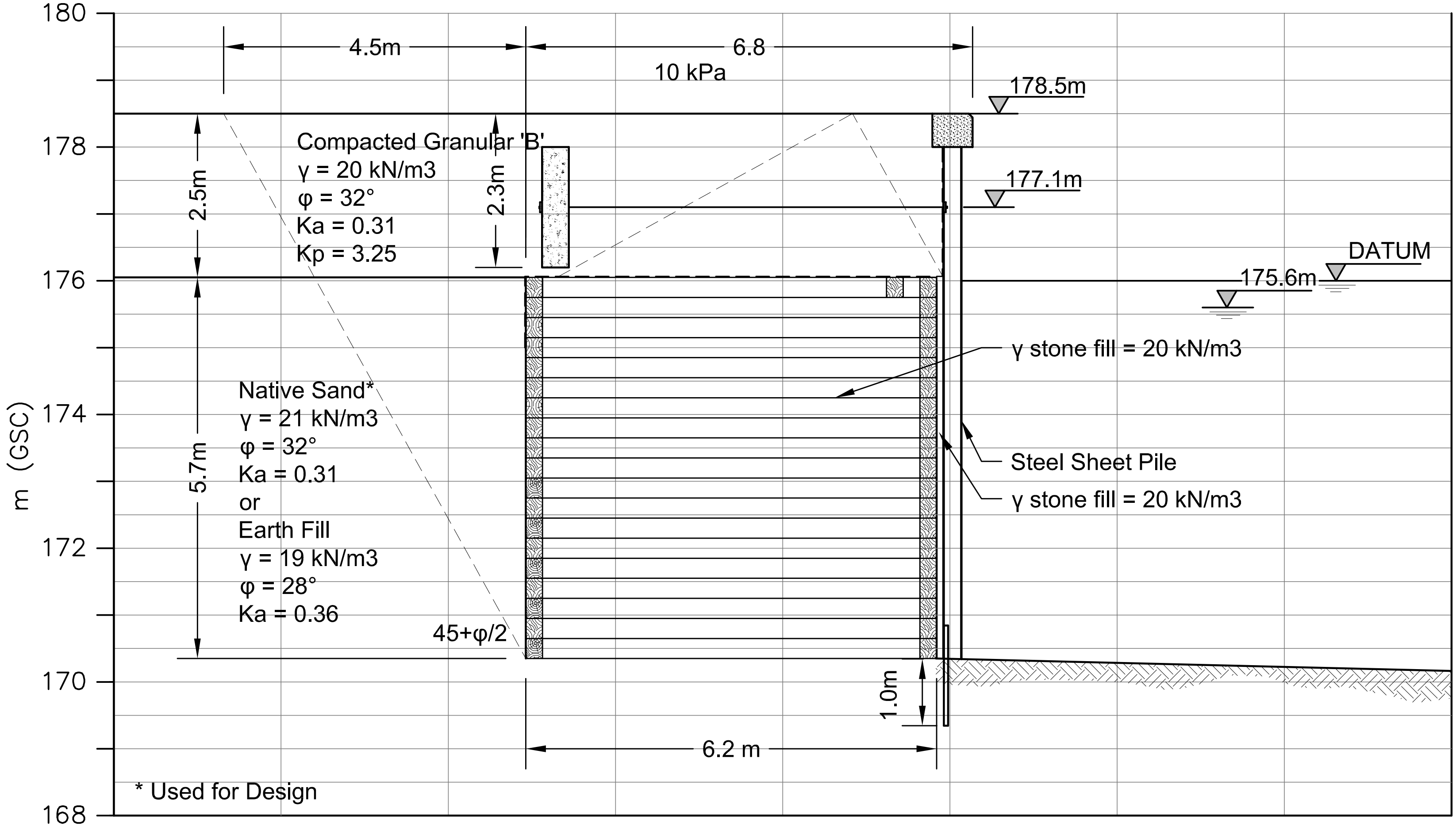


SECTION L1
TIMBER CRIB ALONG OUTFITTING DOCK

Scale 1:100

SHOREPLAN

Figure 3.1
Design Loads Outfitting Dock
Collingwood Shipyards Launch Basin

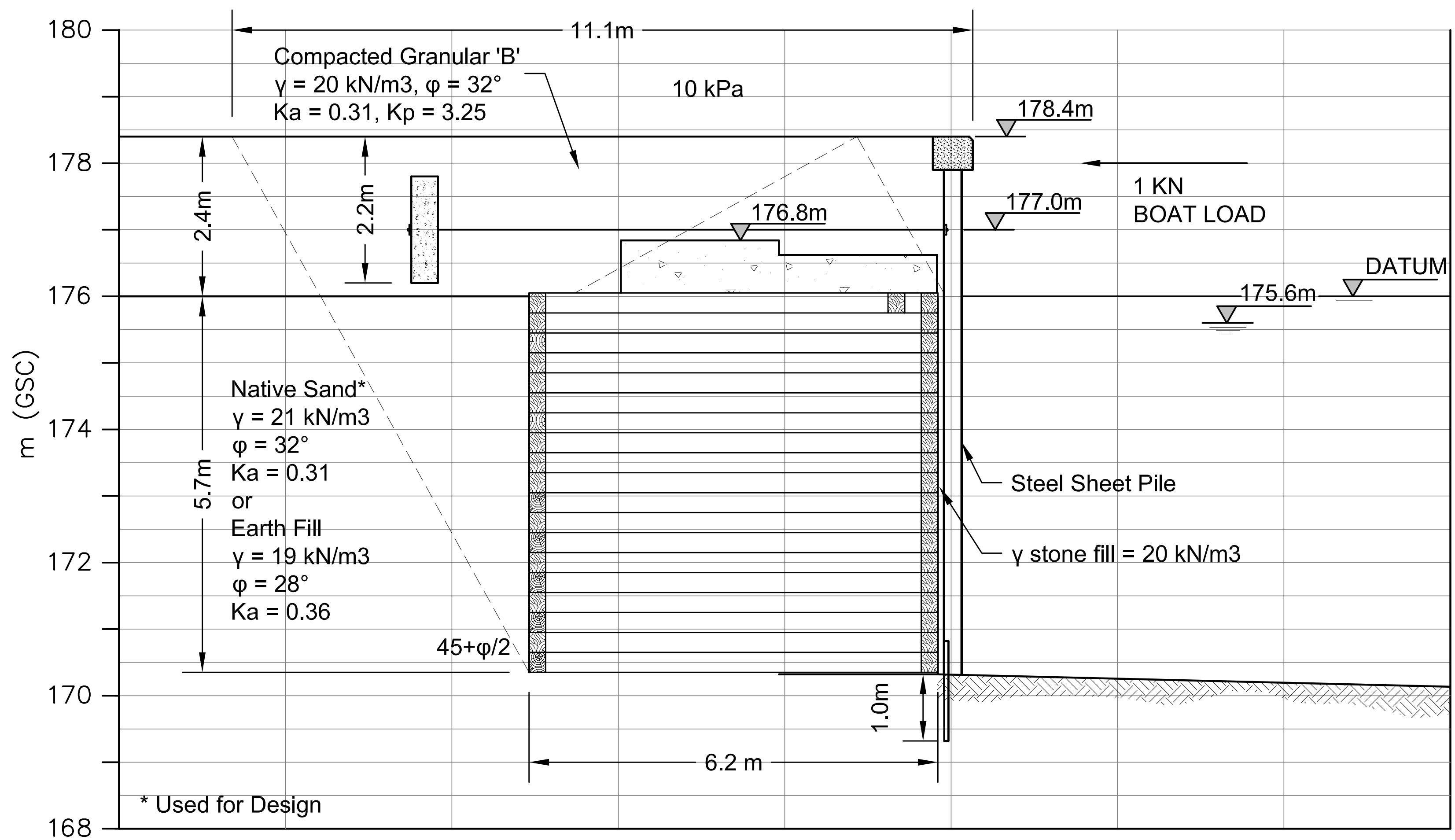


SECTION L2
TIMBER CRIB ALONG OUTFITTING DOCK

Scale 1:100

SHOREPLAN

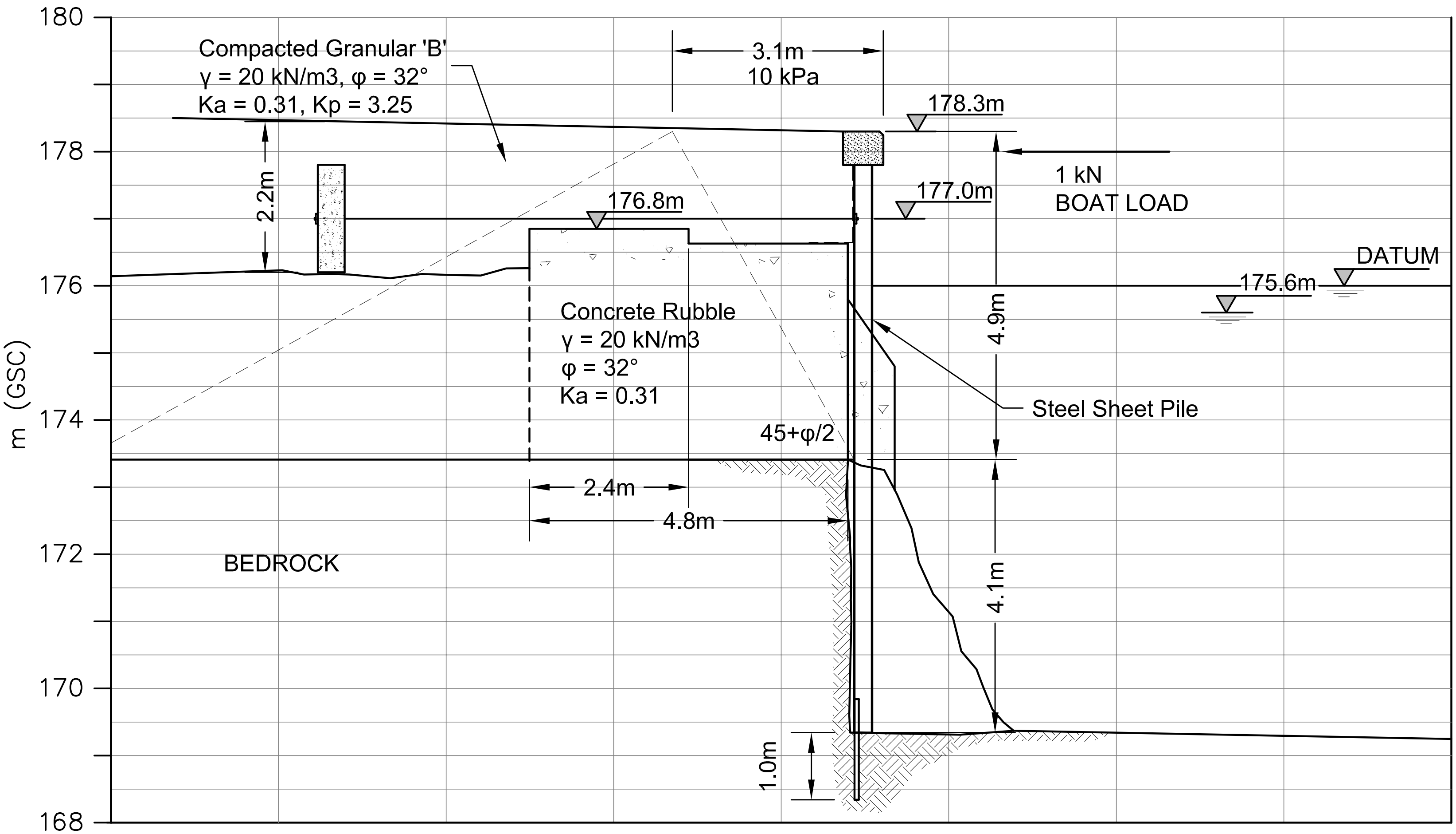
Figure 3.2
Design Loads Outfitting Dock
Collingwood Shipyards Launch Basin



SECTION M1 (0 to 20m)
CONCRETE WALL ALONG LAUNCH BASIN, WEST SIDE

Scale 1:100
SHOREPLAN

Figure 3.3
Design Loads West Side of Launch Basin
Collingwood Shipyards Launch Basin

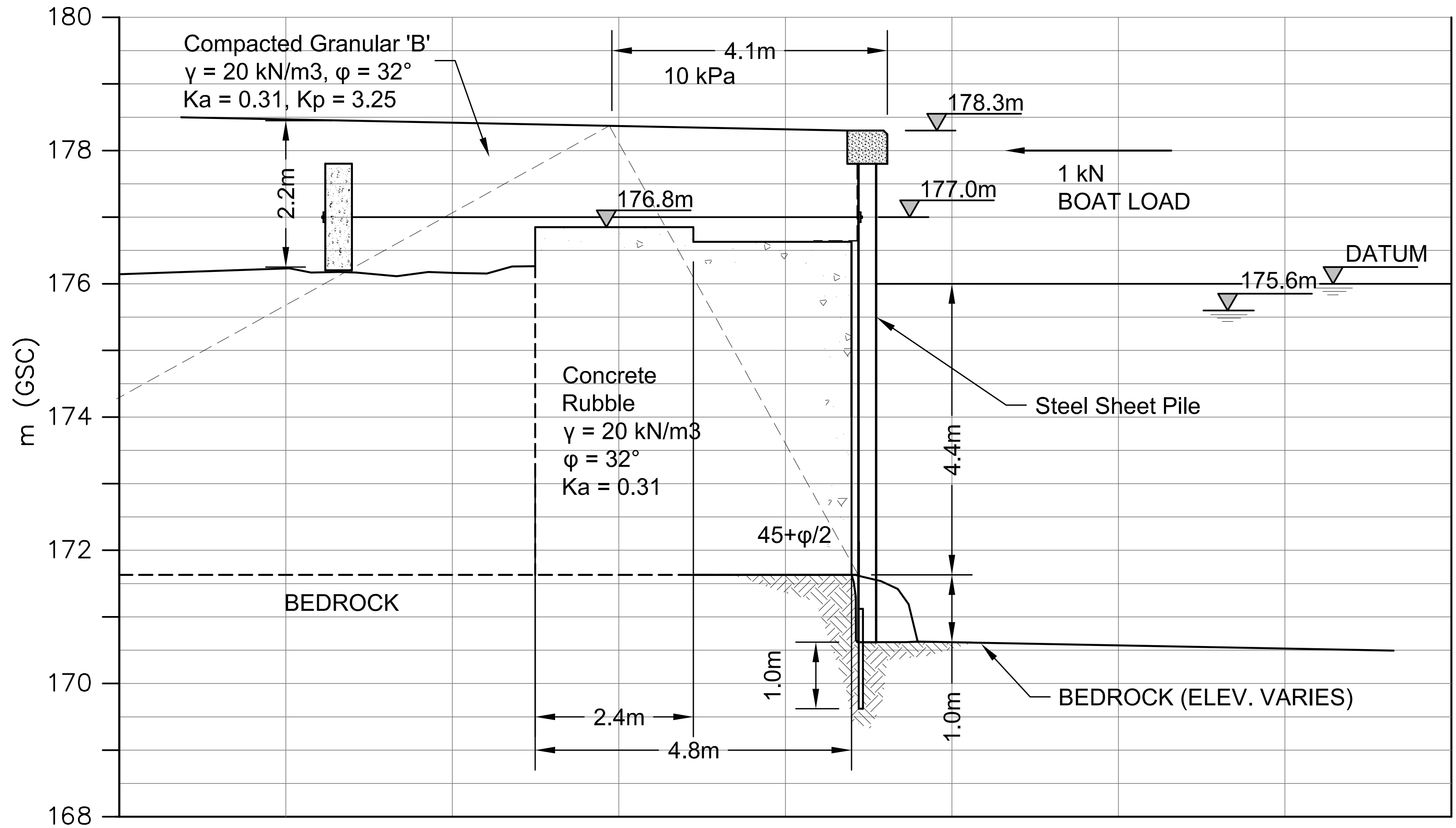


SECTION M3 (31 to 35m)
CONCRETE WALL ALONG LAUNCH BASIN, WEST SIDE

Scale 1:100

SHOREPLAN

Figure 3.4
Design Loads West Side of Launch Basin
Collingwood Shipyards Launch Basin

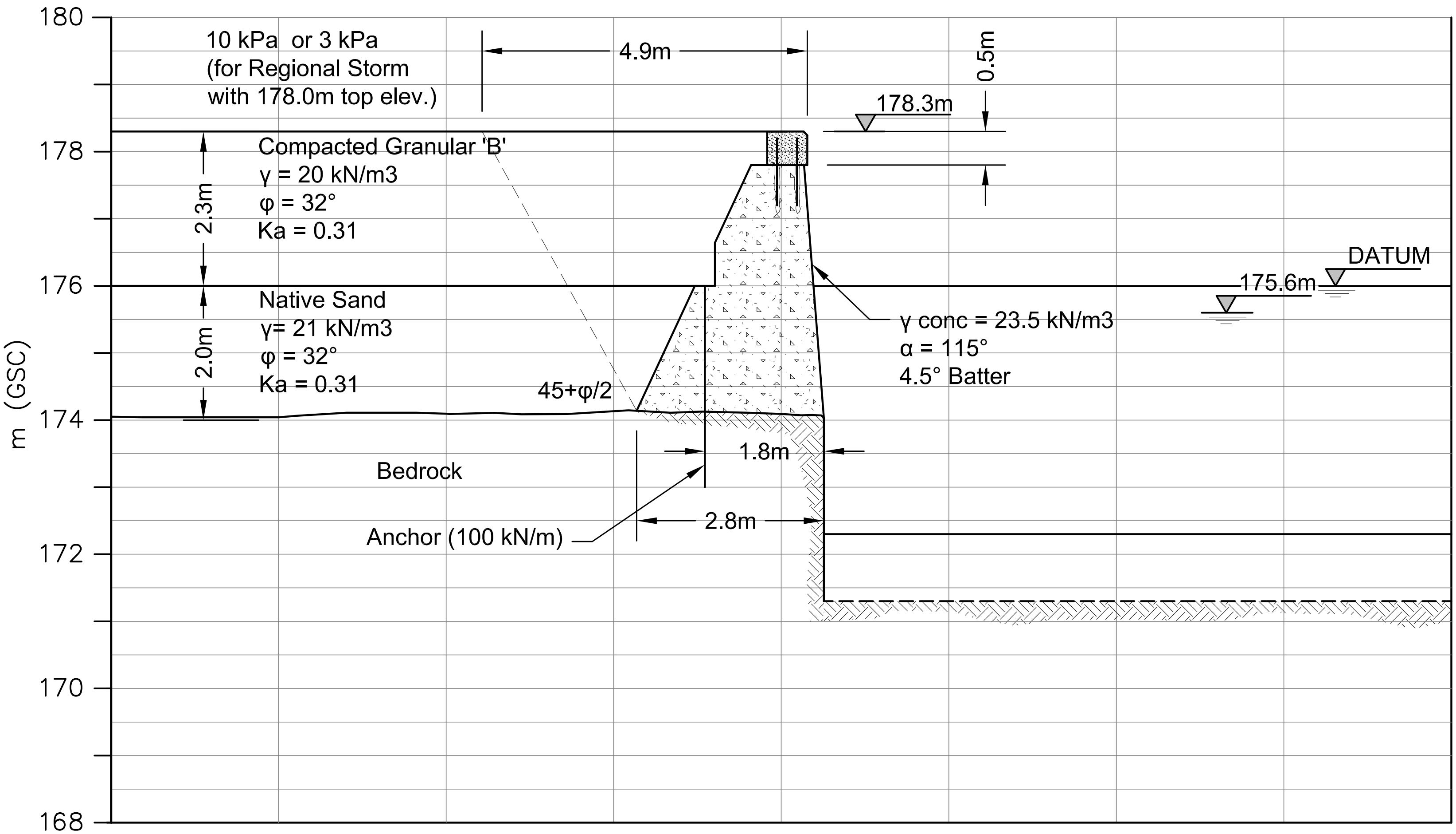


SECTION M5 (52 to 40m)
CONCRETE WALL ALONG LAUNCH BASIN, WEST SIDE

Scale 1:100

SHOREPLAN

Figure 3.5
Design Loads West Side of Launch Basin
Collingwood Shipyards Launch Basin



SECTION N1
CONCRETE WALL ALONG LAUNCH BASIN, SOUTH SIDE

Scale 1:100

SHOREPLAN

Figure 3.6
Design Loads South Side of Launch Basin
Collingwood Shipyards Launch Basin

Table 3.3 Existing Structure Stability with Steel Sheet Pile Wall

Legend	
F.S	Factor of Safety
Rv	Vertical Force
Rh	Horizontal Force
Rv	Vertical Reaction Force
Mv	Moment from Vertical Forces
Mh	Moment from Horizontal Forces

Case 1

Surcharge on Structure 0.0 kPa
Surcharge behind Structure (Crib) 10.0 kPa
Design Water Level 175.6 DLWL

Section	Elevation (m)	Rv (kN/m)	Rh (kN/m)	Mv (kNm/m)	Mh (kNm/m)	F.S. Sliding	F.S. Overturning
L1*	179.0	792.6	233.3	2457.1	810.4	1.9	3.0
L2**	178.5	688.4	209.6	2134.2	637.3	1.9	3.3
M1**	178.4	676.7	207.8	2056.3	624.5	1.9	3.3
N1	178.3	312.2	69.6	486.7	128.3	2.6	3.8
N1	178.0	305.4	102.8	471.3	127.1	1.7	3.7

* anchored to concrete cap
** anchored to new anchor wall

Case 2

Surcharge on Structure 10.0 kPa
Surcharge behind Structure (Crib) 10.0 kPa
Design Water Level 175.6 DLWL

Section		Rv (kN/m)	Rh (kN/m)	Mv (kNm/m)	Mh (kNm/m)	F.S. Sliding	F.S. Overturning
L1*	179.0	854.6	237.3	2649.3	840.0	2.1	3.2
L2**	178.5	750.4	209.6	2326.4	637.3	2.0	3.7
M1**	178.4	737.7	207.8	2242.4	624.5	2.0	3.6

* anchored to concrete cap
** anchored to new anchor wall

Case 3

Surcharge on Structure 0.0 kPa
Surcharge behind Structure (Crib) 10.0 kPa
Design Water Level 178.0 DHWL

Section		Rv (kN/m)	Rh (kN/m)	Mv (kNm/m)	Mh (kNm/m)	F.S. Sliding	F.S. Overturning
L1*	179.0	649.3	184.6	2012.7	643.8	2.0	3.1
L2**	178.5	542.6	145.6	1682.1	437.7	2.1	3.8
M1**	178.4	533.2	140.3	1618.7	415.1	2.2	3.9
N1	178.3	260.1	49.2	397.2	82.5	3.0	4.8
N1	178.0	235.7	40.2	376.4	61.7	3.3	6.1

* anchored to concrete cap
** anchored to new anchor wall

Case 4

Surcharge on Structure 10.0 kPa
Surcharge behind Structure (Crib) 10.0 kPa
Design Water Level 178.0 DHWL

Section		Rv (kN/m)	Rh (kN/m)	Mv (kNm/m)	Mh (kNm/m)	F.S. Sliding	F.S. Overturning
L1*	179.0	708.8	188.7	2197.2	674.1	2.1	3.3
L2**	178.5	604.6	145.6	1874.3	437.7	2.4	4.3
M1**	178.4	594.2	140.3	1804.8	415.1	2.4	4.3

* anchored to concrete cap
** anchored to new anchor wall

Case 5 Regional Storm

Surcharge Regional Storm 3.0 kPa
Surcharge behind Structure 0.0 kPa

Section		Rv (kN/m)	Rh (kN/m)	Mv (kNm/m)	Mh (kNm/m)	F.S. Sliding	F.S. Overturning
N1 (175.6, DLWL)	178.0	265.1	59.7	398.6	112.1	2.5	3.6
N1 (178.0, DHWL)	178.0	235.7	31.5	376.4	44.4	4.3	8.5

Existing Concrete Structures

	Mr (kNm/m)	Mf (kNm/m)	Vr (kN/m)	Vf (kN/m)
Existing Concrete Cap (350mm slab)	286	279	217	107
Section N1 at anchor			1358	536

Table 3.4 Anchor and Lookout Loads

Anchor Design Loads

Case 1

Surcharge on Structure	0.0	kPa
Surcharge behind Anchor	10.0	kPa
Design Water Level	175.6	DLWL

Section	Elevation (m)	Anchor Pull from wall (kN/m)	Toe (kN/m)	Anchor Pull on anchor (kN/m)	Anchor Resistance (kN/m)	Factor of Safety
L1*	179.0	10.8	4.2	10.8	179.2	16.6
L1*	178.5	7.7	4.4	7.7	156.7	20.3
L2	178.5	21.6	4.5	19.8	76.6	3.9
M1	178.4	14.5	3.5	10.5	76.6	7.3
M3	178.3	50.8	16.3	50.8	153.2	3.0
M5	178.3	61.0	51.0	61.0	153.2	2.5
M6	178.3	55.0	32.1	55.0	153.2	2.8

* Anchored to cap (anchor resistance = 0.5(Weight of Cap and Fill)- lateral earth pressure)

Case 2

Surcharge on Structure	10.0	kPa
Surcharge behind Anchor	10.0	kPa
Design Water Level	175.6	DLWL

Design Case for Steel Sheet Pile

Section	Elevation (m)	Anchor Pull from wall (kN/m)	Toe (kN/m)	Anchor Pull on anchor (kN/m)	Anchor Resistance (kN/m)	Factor of Safety	Maximum Moment (kN m/m)
L1*	179.0	17.6	3.7	17.6	232.5	13.2	5.2
L1*	178.5	10.4	4.3	10.4	211.6	20.3	4.5
L2	178.5	29.5	4.3	27.0	76.6	2.8	10.8
M1	178.4	22.2	3.3	16.0	76.6	4.8	26.6
M3	178.3	68.5	13.7	68.5	153.2	2.2	96.8
M5	178.3	75.0	57.4	75.0	153.2	2.0	117.7
M6	178.3	68.4	36.4	68.4	153.2	2.2	96.4

* Anchored to cap (anchor resistance = 0.5(Weight of Cap and Fill)- lateral earth pressure)

Case 3

Surcharge on Structure	0.0	kPa
Surcharge behind Anchor	10.0	kPa
Design Water Level	177.5	DHWL

Section	Elevation (m)	Anchor Pull from wall (kN/m)	Toe (kN/m)	Anchor Pull on anchor (kN/m)	Anchor Resistance (kN/m)	Factor of Safety
L1*	179.0	8.7	3.9	8.7	97.6	11.2
L1*	178.5	5.7	4.1	5.7	76.7	13.6
L2	178.5	15.6	4.0	14.2	63.0	4.4
M1	178.4	10.6	3.4	7.6	63.0	8.3
M3	178.3	31.9	10.4	31.9	126.0	3.9
M5	178.3	39.5	35.3	39.5	126.0	3.2
M6	178.3	35.1	21.3	35.1	126.0	3.6

* Anchored to cap (anchor resistance = 0.5(Weight of Cap and Fill)- lateral earth pressure)

Case 4

Surcharge on Structure	10.0	kPa
Surcharge behind Anchor	10.0	kPa
Design Water Level	177.5	DHWL

Section	Elevation (m)	Anchor Pull from wall (kN/m)	Toe (kN/m)	Anchor Pull on anchor (kN/m)	Anchor Resistance (kN/m)	Factor of Safety
L1*	179.0	12.8	3.6	12.8	128.6	10.1
L1*	178.5	7.9	4.0	7.9	107.7	13.6
L2	178.5	23.5	3.9	21.5	63.0	2.9
M1	178.4	16.3	3.2	11.8	63.0	5.3
M3	178.3	44.8	12.7	44.8	126.0	2.8
M5	178.3	53.5	41.7	53.5	126.0	2.4
M6	178.3	48.5	25.6	48.5	126.0	2.6

* Anchored to cap (anchor resistance = 0.5(Weight of Cap and Fill)- lateral earth pressure)

Lookout Design Loads

Live Load	5 kPa	
Wave Uplift 178.0 DHWL	28 kPa	(short duration)
Wave Uplift 178.0 DHWL	3.5 kPa	(quasi sustained)

Table 3.5 Structural Components (After Construction)

	Resisted						Unfactored				Factor of Safety			
Component	Mr (kNm)	Vr (kN)	Tr (kN)	Mrx (kNm)	Mry (kNm)	Cr (kN)	M (kNm)	V (kN)	T (kN)	C (kN)	Mr/M	Vr/V	Tr/T	Cr/C
Steel Sheet Pile														
EZ80*	297						118				2.5			
Anchor Rod														
#14 Grade 75			563						286				1.8	
Toe Pin														
63.5 mm	7.5	658					3.6	18.2			2.1	36.2		
76.2 mm (Section M6, M7 & M8) Fy=400MPa	15.7	1083					9.2	46.2			1.7	23.4		
88.9 mm (Section M5) Fy = 400MPa	24.8	1474					14.6	73			1.7	20.2		
Wale														
2-C250X37		1297		177.9			109	286			1.6	4.1		
1-C250x37 & 200x450x15mm Plate		1094		119			33	124			3.6	8.8		
M24			188						95				2.0	
8-M16		536						76				7.1		
Section L1 Anchor														
700x475x50mm Plate	59	3741					30	67			2.0	50.6		
975x475x50mm Plate	59	3741					35	67			1.7	50.6		
2-M32x406 Hilti Anchors (L1 elev. 179.0)**			459						67				6.9	
2-M32x406 Hilti Anchors (L1 elev. 178.5, 700mm anchor)**			459						40				11.5	
2-M32x406 Hilti Anchors (L1 elev. 178.5, 975mm anchor)**			459						95				4.8	
Reinforced Concrete Anchor A1														
Horizontal	194	1275					109	314			1.8	4.1		
Vertical	118						54				2.2			
Reinforced Concrete Anchor A2														
Horizontal	212.5	1275					92	267			2.3	4.8		
Vertical	118						54				2.2			
Section N1														
HSS 254x254	280	1070					106	325			2.6	3.3		
R1H16C28 Spin Lock Anchor		490	974						325				3.0	
Lookout														
W200X100	362	691					13	45			27.8	15.4		
C230x30	60.2	542					13	45			4.6	12.0		
W310x202	1068	1418		1068	490	3430	85	85		85	5.8	16.7		40.4
R1H16C28 Spin Lock Anchor	7	490	974				0.8	4	24		8.8	122.5	40.6	

Notes

All Steel Shapes, Rods & Plates Fy = 350 MPa unless otherwise noted
All Bolts A325
All Reinforcing Bar Fy= 400 MPa
All Concrete F'c = 35 MPa
*Steel Sheet Pile Fy=300 MPa, Limit States Design
** Ultimate Bond/ Concrete Capacity F.S.>=4

Legend

Mr	Moment Resisted	M	Moment
Vr	Shear Force Resisted	V	Shear Force
Tr	Tension Resisted	T	Tension
Cr	Compression Resisted	C	Compression

Table 3.6 Structural Components (After 50 years)

Component	Resisted (After 50y)						Unfactored				Factor of Safety (After 50y)			
	Mr	Vr	Tr	Mrx	Mry	Cr	M	V	T	C	Mr/M	Vr/V	Tr/T	Cr/C
	(kNm)	(kN)	(kN)	(kNm)	(kNm)	(kN)	(kNm)	(kN)	(kN)	(kN)				
Steel Sheet Pile														
EZ80 *	186						118				1.6			
Anchor Rod														
#14 Grade 75			487						286				1.6	
Toe Pin														
63.5 mm	7	598					3.6	18.2			1.9	32.9		
76.2 mm (Section M6, M7 & M8) Fy=400MPa	14	1000					9.2	46.2			1.5	21.6		
88.9 mm (Section M5) Fy = 400MPa	23	1377					14.6	73			1.6	18.9		
Wale														
2-C250X37**		1085		140			109	286			1.3	3.5		
1-C250x37 & 200x450x15mm Plate**		915		95			33	124			2.9	7.4		
M24			144						95				1.5	
8-M16		354						76				4.7		
Section L1 Anchor														
700x475x50mm Plate	52	3494					30	67			1.7	47.2		
975x475x50mm Plate	52	3494					35	67			1.5	47.2		
2-M32x406 Hilti Anchors (L1 elev. 179.0)***			459						67				6.9	
2-M32x406 Hilti Anchors (L1 elev. 178.5, 700mm anchor)***			459						40				11.5	
2-M32x406 Hilti Anchors (L1 elev. 178.5, 975mm anchor)***			459						95				4.8	
Section N1														
HSS 254x254	228	858					106	325			2.2	2.6		
R1H16C28 Spin Lock Anchor		490	974						325				3.0	
Lookout														
W200X100	253	459					13	45			19.5	10.2		
C230x30	36	438					13	45			2.8	9.7		
W310x202	928	1195		928	426	2998	85	85		85	5.0	14.1		35.3
R1H16C28 Spin Lock Anchor	6	490	974				0.8	4	24		7.5	122.5	40.6	

Notes
All Steel Shapes, Rods & Plates Fy = 350 MPa unless otherwise noted
All Bolts A325
All Reinforcing Bar Fy= 400 MPa
All Concrete F'c = 35 MPa
*Steel Sheet Pile Fy=300 MPa
** After 50years if not Galvanized
*** Ultimate Bond/ Concrete Capacity F.S.>=4

Legend

Mr	Moment Resisted	M	Moment
Vr	Shear Force Resisted	V	Shear Force
Tr	Tension Resisted	T	Tension
Cr	Compression Resisted	C	Compression

4.0 DESIGN LIFE AND REMAINING USEFUL LIFE

The structures along the shoreline of the Collingwood Shipyards are required under the Master Development Agreement to have a design life of 50 years. The proposed shoreline protection incorporates both the existing and new structures. For the new components, allowances have been made for rusting of the steel components to provide the required 50 years life, as described in section 3.3.1. The reinforced concrete is of the appropriate Class of Exposure (C-1) to ensure the specified design life with minimal maintenance.

4.1 Reach 4 - Outfitting Dock (Town Dock)

The exact date of construction of the Outfitting Dock is not known. However, a structure matching the location and dimension of this dock is illustrated on a site plan produced by Public Works Canada in 1928. Given the type of construction observed in the dock, we are of the opinion that the dock shown in the plan is the existing dock and the dock is more than 80 years old. Timber cribs have a very long life, as long as the timber component of the structure remains under water. Factors which impact the life of timber cribs include exposure to ice, vessel impact abrasion, cycles of wetting and drying and exposure to wind, waves and currents. The existing stone filled timber crib with concrete cap will be rehabilitated. The design includes the replacement of damaged timbers at the waterline and grouting of the void under the concrete cap. Once the timbers at the water crib is enclosed with steel sheet pile all of the factors except cycles of wetting and drying are removed. Based on our review of the dock, it is our opinion that the timber crib and the concrete will provide substantially more than 50 years life. All of the new components have been designed to provide a minimum 50 year life and with appropriate factors of safety remaining at 50 years.

4.2 Reach 5 - West side of Launch Basin

Reach 5 consists of retaining walls illustrated in sections M1 to M10. The exact age of these structures is not known, but charts dating into the late 1800s show components of the dry dock gates that were observed by the divers. There are three types of existing structures in reach 5, one of which, section M1, relies on the existing timber crib as a structural component of new structure. The timber crib section in section M1 did not show any signs of structural failure or damage below water. For the reasons stated in section 4.1, it is our opinion that the timber crib will provide substantially more than 50 years of life.

The remaining life of the concrete structure in reach 5, sections M2 to M10 is not required because the proposed structure does not rely on the existing structure. Therefore, only the design life of the new components need to be considered. All of the new components have been designed to provide a minimum 50 year life and with appropriate factors of safety remaining at 50 years.

4.3 Reach 6 - South Shore of Launch Basin

The concrete in the gravity retaining wall south end of the Launch Basin (section N1) shows minimal amount of deterioration and tests of concrete strength show that high strength concrete was used. This gravity structure will be rehabilitated to include an anchor to bedrock so that it will have the appropriate factors of safety against overturing and sliding with the proposed new loads. It is our professional opinion that the concrete will provide an additional 50 years. All of the new components have been designed to provide a minimum 50 year life and with appropriate factors of safety remaining at 50 years.

5.0 MONITORING AND MAINTENANCE PROGRAM

5.1 Monitoring Program

The recommended monitoring program for the completed works follows the basic requirements outlined in “Guidelines for Inspection and Maintenance of Marine Facilities”, Public Works Canada/ Transport Canada, 1984, hereafter referred to as Guidelines. Appendix I contains Part A3 - Inspection Guidelines. This chapter provides details of the monitoring and inspections procedures. This is the standard reference for monitoring of shoreline structures in Canada. There are three types of monitoring programs that should be carried out after the implementation of the shoreline structures. These include visual inspections, detailed routine inspections and special inspections. Description of activities under each program are provided below.

We recommend that the visual inspections are carried out by the staff of the Town of Collingwood during their routine site reviews. The reviews should be carried out in the spring and again in the fall. We recommend that digital photographs are used to document conditions during the visual inspection. This photographic documentation is more appropriate for the Town’s staff than the manual diagrams suggested by the Guidelines. The photographs should be taken from the same location during each visit. The photographs should provide a view along crest of each structure and a general view onto the structure, where access allows. Detailed views of areas should be taken only where specific local conditions need to be document, i.e. crack in concrete, a gap in a structure or similar. The staff must develop a formal filing system for the information collected during these reviews.

The Routine Detailed Inspections should be carried out every five years for the first twenty five years. Adjustment in the frequency can be made after that time based on the findings, if required. Table A3.3 of the Guidelines can be used to guide decisions regarding scheduling of the inspections beyond 25 years. The first Routine Detailed Inspection should be carried out just prior to acceptance of the works by the Town of Collingwood at the expiry of maintenance period of two years, as referred to in clause 3.7 of the Master Development Agreement.

The inspection should follow the format outlined in the above noted Guidelines. The manual provides detailed outlined of inspection procedures and reports in sections A3.3.4 to A3.3.6.

The specific elements of the structures in Phase 2 of the Shipyards development to be inspected include:

- joints between individual sheet piles
- verticality of the piles
- uniformity of alignment
- separation between the pile cap support and concrete cap
- spalling and/or excessive cracking of the concrete pile cap
- visible underwater deposits of soil material
- depressions in walkways areas

Based on our analysis, the minimum measurements of the main components that are required to provide the appropriate factors of safety are outlined in Table 5.1 below. Most of these components, wale anchor rod and toe pins, will not be visible during routine detailed inspections.

However, this information is provided for special inspection of individual components when required.

Table 5.1
Minimum Allowable Measurements of Main Structural Components

Component	Required Capacity	Supplied Dimension	Minimum Thickness
Steel Sheet Pile (Fy = 300 MPa)	Mr = 118 kN-m (unfactored)	8 mm	5mm
Anchor Rod (Fy = 517 MPa)	Tr = 286 (unfactored)	43 mm diam.	40 mm diam.
Wale (Fy = 350 MPa)	Mr = 109 kN-m (unfactored)	C250x37 t web = 13.4 mm t flange = 11.1 mm	C250x37 t web = 11.9 mm t flange = 9.4 mm
Toe Pins, 63.5 mm diam. (Fy = 350 MPa)	Mr = 3.6 kN-m (unfactored)	63.5 mm diam.	56.0 mm diam.
Toe Pins, 76.2 mm diam. (Fy = 400 MPa)	Mr = 9.2 kN-m (unfactored)	76.2 mm diam.	73.0 mm diam.
Toe Pins, 88.9 mm diam. (Fy = 400 MPa)	Mr = 14.6 kN-m (unfactored)	88.9 mm diam.	85.2 mm diam.
Timber Crib	2 to 4 MPa Allow. 2 used	200 mm bearing width	115 mm bearing width

The Routine Detailed Inspections should be carried out under the supervision of a coastal and marine structure specialist professional engineer registered or licenced in the Province of Ontario. The cost of Routine Detailed Inspection of the shoreline components in Phase 2 (Outfitting Dock and west and south side of the Launch basin) is estimated to be in the order of $15,000 to $20,000 (2008 $).

Special Inspections should be carried out any time a Visual or Routine Detailed Inspections discover notable deterioration or damage to the shoreline structures. Special Inspections should be carried out under the supervision of a coastal and marine structure specialist professional engineer registered or licenced in the Province of Ontario. A full diving team shall be employed to review the underwater portion of the structure. Section A3.4 of the Guidelines provides details of inspection procedures for various types of deterioration. The cost of a special inspection will vary depending on the nature of the work required.

5.2 Maintenance Program

It is difficult to establish an accurate maintenance cost for the shore protection works. Technical literature suggests that 0.5 to 1.0 % of the capital cost is a reasonable range of annual maintenance cost for marine structures. Maintenance budget for marine installations in Federal Govenment departments of Canada are usually based on the following:

- 0.5% of the construction cost during the first 15 years in the life of a structure;
- 1% of the construction cost during the next 15 years in the life of the structure; and
- An expenditure of 10% to 15% of the construction cost at midlife to repair major damage and or deterioration that has occurred due to operation of the facilities.

The frequency of maintenance for seawall structures constructed of steel sheet pile, timber cribs or concrete is difficult to quantify. Steel sheet piles generally do not require maintenance during their design life unless damaged by external impacts. The design life of this sheet pile wall, as noted previously, is 50 years. No specific maintenance of the steel sheet pile wall is expected to be required during its design life.

Timber cribs also have a very long life, as long as the timber component of the structure remains under water. There are many examples of timber structures built in the mid to late 1800s on the Great Lakes in an exposed wave climate conditions that are still standing. There are many timber crib structures along the Toronto waterfront that have been constructed in the early 1900s that are still in fully functional condition. Repairs of crib structures are undertaken more frequently due to settlement of the structure caused by poor foundation conditions than due to the timber crib failure itself. Settlement is not a possible mode of failure here since the cribs are founded on bedrock. Deteriorated timbers at the low water line will be replaced during this project. These new timbers and those remaining will have a minimum design life of 50 years based on the estimated rate of deterioration since the construction of the structure. Given that the timbers will be covered by a steel sheet pile wall, the future rate of deterioration is expected to be much lower. This will provide a design life in excess of 50 years.

Repairs to concrete walls are generally limited to concrete damage due to freeze/frost action or settlement. Settlement is not expected to be a problem at The Shipyards due to the presence of bedrock. All of the concrete wall structures within the this phase are founded on or very close to bedrock.

Maintenance is not expected to be required on an annual basis. Maintenance should be scheduled only as required based on the findings of the monitoring program.

Based on our construction cost estimate for the proposed work, a preliminary budget suggests an annual maintenance cost in the order of $9,000 to $18,000 (2008$). The estimated annual maintenance cost breakdown by structure is $2,250 to $4,500 for the Outfitting Dock/ Town Dock, $6,500 to $13,000 for the west side of Launch Basin, and $250 to $500 for the south end of Launch Basin. The actual amount required in a given year may be substantially different. For example when the structure is new, there will be no maintenance cost involved. After 30 years of service expenditures could be much higher. The values listed above provide a guide for the amount that will be required for maintenance over the service life of the structures.

All of which is respectfully submitted.

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