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PRELIMINARY GEOTECHNICAL DESIGN REPORT

Proposed North Wharf Expansion
Goderich, Ontario

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1.0 INTRODUCTION

This report presents an overview of existing geotechnical data and past construction experience, along with preliminary geotechnical recommendations to aid with the planning and design of the new construction work associated with the proposed North Wharf Expansion of Goderich Harbour. This report consolidates and summarizes the results of previous geotechnical investigations at the Goderich Harbour conducted by Golder Associates Ltd. (Golder) and others. A recommended program of investigation is also included to define lakebed conditions in areas not explored by the existing boreholes. The approximate location of the site is shown on the Key Plan on Figure 1, which also shows the location of the previous investigations.

It is understood that this preliminary geotechnical report will serve to supplement the ongoing Environmental Assessment (EA) for the expansion project.

This report was prepared in general accordance with the Golder Proposal No. P2-1132-0059-P01 dated April 16, 2012, and the subsequent addendum P2-1132-0059-P02 dated May 31, 2012. Written authorization to proceed with the work was received from Mr. Andrew Ross of B.M. Ross and Associates Ltd. on June 15, 2012.

2.0 SITE DESCRIPTION

The Town of Goderich, situated at the mouth of the Maitland River on the east shore of Lake Huron, is the county seat of Huron County. It is the location of the only deep water commercial port along Lake Huron, a salt mining centre, and a manufacturing centre.

At Goderich, the Maitland River has entrenched itself in a valley about 50 metres (m) deep. The banks of the river valley and the Lake Huron bluffs in the area are generally quite steep.

The Goderich Harbour facilities presently consist of protective northeast, northwest and south breakwaters surrounding the outer harbour, north and south piers, and an inner harbour. The main channel of the harbour extends from the opening between the northwest and south breakwaters some 950 m eastward, between the north and south piers to the inner harbour. The north pier and adjacent Sifto Salt Mine facilities form a peninsula separating the eastern portion of the harbour channel and the inner harbour from the mouth of the Maitland River to the north.

Water depths in the area of the proposed expansion in the outer harbour, as indicated by the current bathymetry data on Figure 1, are about 3 to 4 m adjacent to the northeast breakwater, gradually increasing westward to about 5 to 6 m adjacent to the northwest breakwater. The elevation of the lakebed drops somewhat sharply in a southerly direction resulting in water depths of about 8 to 9 m adjacent to the proposed retaining structures and the existing north pier.
3.0 PROJECT DESCRIPTION

The North Wharf Expansion of Goderich Harbour includes the construction of a new wharf facility as well as land reclamation in the outer harbour to provide new berthing facilities and loading/unloading space for ships and storage space for salt and other commodities. It is understood that three alternatives are being considered for the construction of the proposed wharf:

i) No Slip Option: The proposed conceptual wharf design for this option consists of six new, 12.2 m diameter, circular steel sheet pile cells at 50 m centre-to-centre spacing filled with 100 to 200 mm stones and capped with a concrete pad. This option requires the greatest volume of harbour filling.

ii) Single Slip Option: Two different construction styles are under consideration for the new wharf. The east end of the new wharf consists of a new dockwall formed by continuous steel sheet piles supported by deadman anchors and backfilled with selected material. The west end of the new wharf consists of five new, 12.2 m diameter, circular steel sheet pile cells at 50 m centre-to-centre spacing filled with 100 to 200 mm stones and capped with a concrete pad. This option includes a single slip between the new wharf and the north pier peninsula.

iii) Twin Slip Option: Two different construction styles are under consideration for the new wharf. The east end of the new wharf consists of a new dockwall formed by continuous steel sheet piles supported by deadman anchors and backfilled with selected material. The west end of the new wharf consists of five new, 12.2 m diameter, circular steel sheet pile cells at 50 m centre-to-centre spacing filled with 100 to 200 mm stones and capped with a concrete pad. This option includes two slips between the new wharf and the north pier peninsula, and requires the least volume of harbour filling.

For all of the options, the proposed elevation of the top of the concrete pad/coping wall is 179.5 m. The area of fill behind the cells is planned to have a final elevation of 179.3 m to the top of asphalt. The sheet pile cells are intended to be used as mooring dolphins. The sloping faces of the fill are to be covered by rock revetment to protect against wave-induced erosion.

This report considers the twin slip option as the base example and has used the base plan for the preparation of Figures 1, 2 and 3. The recommendations included in this report generally also apply to the other options. However, the recommended investigation program may need to be modified to address the greater extent of fill associated with the no slip option if this is eventually selected for final design and construction.

4.0 SOURCES OF INFORMATION

The subsurface data and past construction experience described in this report were gathered from the following sources:


5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS OVERVIEW

5.1 Regional Geology

The study area is located in the physiographic region of Southwestern Ontario known as the Huron Slope which is considered to be a clay plain of glacial Lake Warren overlying a fine-grained (i.e., primarily silt and clay sized particles) basal glacial till. Till is sometimes exposed at the surface and thin surficial layers of sand are found in the immediate area of Goderich.

The bedrock in the study area consists of light brown to grey, fine to medium grained, dolomite and limestone. The rock, which is of Middle Devonian age, belongs to the Dundee Formation of the Hamilton Group or the underlying Lucas Formation of the Detroit River Group. The Dundee Formation (also known as the Norfolk Formation) contains light brown and tan limestone containing quartz sand grains and chert. The Maitland River has cut through the Dundee Formation to expose rock of the Lucas Formation, Detroit River Group. These rocks are light tan dolomite and brown, tan and grey limestone. The core obtained in the previous geotechnical investigations appeared to originate from the Lucas Formation.

5.2 General Site Stratigraphy

The following discussion is based on the subsurface conditions encountered in the previously drilled boreholes in the area of the proposed North Wharf Expansion as well as adjacent areas. Information from a total of 37 boreholes was reviewed to interpret the subsurface conditions. The following paragraphs are intended to provide a summary of the subsurface conditions simplified for the purposes of preliminary geotechnical design. The interpreted boundaries are inferred from non-continuous soil sampling and continuous rock coring together with observations of drilling resistance and typically represent a transition from one soil or rock type to another. They should not be interpreted to represent exact planes of geologic change.
The boreholes generally encountered fill or ice and water overlying sand, sand and gravel, glacial till and/or bedrock. Summary descriptions of the fill, soil and rock materials are provided below.

5.3 Fill

Fill materials consisting of broken rock fill and gravel were encountered in Borehole (BH) 1. The thickness of the fill was about 5.8 m. The fill exhibited Standard Penetration Test (SPT) N values between 7 and 23 blows per 0.3 m of penetration. The natural water contents varied from about 6 to 15 per cent.

5.4 Sand

An upper deposit of very loose to very dense sand containing occasional pieces of wood was found on the lake bottom in BHs 2, 3, 12, 101 to 105, 112, 115, 302, 306, 307, 308 and 309 to a maximum thickness of about 3.7 m. Borehole 301 encountered sand beneath an upper layer of sand and gravel. Standard Penetration Test N values ranged from no measurable value in BH 104 to 77 blows per 0.3 m in BH 3. In the area of the proposed work, the SPT N values in this upper deposit of sand were typically less than or equal to 10 blows/0.3 m. This deposit is likely the result of natural littoral drift deposition of sand. The average natural water content for the sand was about 25 per cent, with one high value (60 per cent) recorded in BH 309.

5.5 Silty Sand

A layer of very dense silty sand, about 1.7 m thick, was encountered in BH 309 beneath the sand layer. Standard Penetration Test N values for the deposit were over 100 blows per 0.3 m. The natural water content varied from 3 to 13 per cent.

5.6 Upper Sand and Gravel

Boreholes 1, 4, 7, 10, 12, 13, 105, 112 to 116, 301, 304 and 305 encountered a stratum of sand and gravel varying in thickness from about 1.2 to 4.9 m. The sand and gravel was encountered from surface in BH 7, formed the lake bottom in BHs 4, 10, 13, 113, 114, 116, 301, 304 and 305, beneath the fill in BH 1, and beneath a thin layer of sand in BHs 12, 105, 112 and 115.

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1 The SPT N value is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimeters to drive a split spoon sampler a distance of 300 millimeters.
The sand and gravel varied from loose to very dense with measured SPT N values ranging from 7 to greater than 100 blows per 0.3 m. The natural water content of the sand and gravel samples ranged from about 3 to 22 per cent. Based on observations of drilling resistance, visual observations, and as noted on the borehole logs, the stratum should be expected to contain numerous cobbles and boulders.

### 5.7 Glacial Till

The majority of the boreholes encountered strata of glacial till varying in gradation from clayey silt to sandy silt till. In some areas, the glacial till included interbedded layers or zones of very dense sand and gravel, hard clayey silt and silt. All of these soil types are expected to contain cobbles and boulders. These materials are described in greater detail below.

#### 5.7.1 Clayey Silt Till

When fully penetrated, the clayey silt till strata varied in thickness from about 0.3 to 6.9 m. Clayey silt till was encountered beneath the surficial sand or upper sand and gravel in BHs 1 to 4, 7, 12, 104, 105, 112, 113, 114 to 116, and underlying the sandy silt till in BHs 13 and 113. Further, based on a dynamic cone penetration test in BH 10, clayey silt till is inferred to underlie the sand and gravel in that borehole.

The clayey silt till was generally hard with SPT N values ranging from 38 to greater than 100 blows per 0.3 m; the higher value of 100 blows per 0.3 m being typical of the deposit. The natural water content ranged from 6 to 13 per cent. The average liquid and plastic limits were 23 and 14 per cent, respectively, based on the results of three Atterberg limits determinations. The results of a single pressuremeter test carried out in the clayey silt till in BH 12 indicated a pressuremeter modulus of about 85 MPa.

#### 5.7.2 Sandy Silt Till

Layers of sandy silt till some 0.8 to 3.4 m thick were encountered beneath the sand, clayey silt till, or sand and gravel in BHs 7, 12, 13, 103, 112, 113, 304, 305 and 308. The sandy silt till exhibited SPT N values from 61 to greater than 100 blows per 0.3 m and were typically in excess of 100 blows per 0.3 m. The natural water content ranged from 3 to 15 per cent.

The results of a single pressuremeter test carried out in the sandy silt till in BH 12 indicated a pressuremeter modulus of about 270 MPa.

Distinct layers of sand and gravel and silty sand some 0.2 to 0.9 m thick were encountered within the glacial till in BHs 12, 13 and 104. These granular layers had SPT N values of 59 to greater than 100 blows per 0.3 m, and natural water contents of about 11 to 13 per cent.
Based on observations of drilling resistance and sample recovery, as well as examination of cored samples, the presence of cobbles and boulders within the till is widespread and should be anticipated.

### 5.7.3 Interbedded Soil Layers within Till

Layers of very dense silt were encountered beneath the sandy silt till in BH 7 and overlying the bedrock in BH 13. The silt layers were about 0.3 to 1.2 m thick. The natural water content of the deposit was about 20 per cent.

Clayey silt layers, varying in thickness from about 0.5 to 2.9 m and containing silt partings and sand layers, were encountered within the clayey silt till in BHs 13 and 112, and underlying a thin layer of sand at the lake bottom in BH 308. The clayey silt had SPT N values from 30 to greater than 100 blows per 0.3 m and an average natural water content of about 14 per cent. A single pressuremeter test carried out in the clayey silt in BH 13 indicates a pressuremeter modulus of about 51 MPa.

Boreholes 2, 7, 12, 14, 101, 102, 105, 112, 114, 115, 301, 302, 305, 306, 307 and 309 all encountered a lower stratum of sand and gravel. The lower sand and gravel formed the lake bottom in BH 14, was encountered beneath the glacial till in BHs 2, 7, 105, 114, 115 and 305, beneath the sand in BHs 101, 102, 301, 306 and 307, beneath the silty sand in BH 309, beneath the silt in BH 302, and beneath the clayey silt in BH 112. While not fully penetrated, the lower sand and gravel was measured to a maximum thickness of about 2.9 m in BH 301.

The lower sand and gravel exhibited SPT N values varying from 13 to greater than 100 blows per 0.3 m. Natural water contents ranged from 3 to 22 per cent.

### 5.8 Bedrock

Bedrock was cored in BHs 1, 7, 12 to 14, 105A, 112 and 113. Rock Quality Designation (RQD) values ranged from 0 to 50 indicating “very poor” to “poor” rock quality. To some extent, the drilling operations conducted from barges may have resulted in lower core recoveries and RQD values due to wind and wave conditions. However, the fact that many of the land based boreholes show identical characteristics suggests that rock quality is relatively poor.

Based on the recovered core, two principal rock types were identified. These were a sandy dolomite and a fine-grained limestone. Correlation between the boreholes suggests that the strata comprise thin limestone bands interbedded within a more extensive sandy dolomite.

The bedrock surface appears to be uneven with distinct ridges and hollows, some of which could be preferentially oriented with respect to glacial ice directions.

As indicated on the borehole logs, the RQD values and the spacing and orientation of fractures suggest that the rock at surface exhibits numerous discontinuities. In addition, associated with the top of the limestone layer cored in BH 13, a seam of calcareous clay was encountered. Several other holes exhibited low core recoveries.
and RQD values. This suggests that a distinct, weak, possibly clay infilled, bedding plane may occur at this level forming the top of a discrete limestone unit.

Two uniaxial compression tests carried out on typical cores of the sandy dolomite exhibited strengths of the order of 30 to 40 MPa. Such values are more typical for shales and siltstones than well well-cemented dolomites. Although not tested, it would be expected that compressive strengths of the better quality portion of the limestone would range from 100 to 150 MPa, while specimens of the more vuggy and open structured units would typically show strengths of 50 to 80 MPa.

Pressuremeter testing in the sandy dolomites tends to support the data from the RQD logging that the rock is extensively broken by jointing and bedding planes. For instance, the fact that the single pressuremeter test carried out in sandy dolomite in BH 13 indicated a pressuremeter modulus of about 200 MPa in rock with an RQD of zero per cent, while the modulus values calculated from the tests on core specimens suggested values of the order of 5 to 20 GPa (some two orders of magnitude higher) is not considered inconsistent with an explanation of an extremely broken upper zone of the rock mass.

The results from the pressuremeter tests in the limestone in BH 12 clearly show the effects of both increased depth into the bedrock (hence tighter fractures) and the greater competence of the rock material. The pressuremeter results suggest in situ moduli only somewhat less than that which would be expected for intact core samples. Typical modulus values for intact core samples of the limestones would tend to range between 10 and 80 GPa dependent on grain size and porosity. Although weaker than the limestone, the sandy dolomite at depth, if devoid of fractures, would also be expected to exhibit higher moduli, again tending towards that of the intact specimens.

5.9 Groundwater

The groundwater level, one or two weeks following drilling, was measured in the standpipe installed in BH 1 at elevation 177.3 m. The corresponding lake level was at about elevation 176.7 m. Artesian water levels in the sand and gravel or bedrock underlying the till were measured in the standpipe sealed in BH 7 and in the drill casing following the completion of drilling in BHs 2, 12 and 105. The artesian water levels varied from elevation 178.3 to 179.5 m compared to a lake level between elevation 176.5 and 177.0 m during the investigation. The artesian water level, as well as the elevation of the encountered bedrock surface increased to the east.

5.10 Summary of Subsurface Conditions

The preceding sections described the individual soil types encountered in different boreholes. For the purposes of planning and design of the proposed retaining structures and berthing facilities, as well as dredging operations, the subsurface conditions along the edge of the proposed wharf can be simplified in terms of two major soil groups: i) soils exhibiting SPT N values of less than 50 blows per 0.3 m (recent deposits), and ii) soils
with SPT N values equal to or greater than 50 blows per 0.3 m (glacial till or soils with till-like density/consistency).

An SPT N value of 50 blows per 0.3 m was considered an appropriate threshold signifying a very dense or hard state. Figures 2 and 3 present simplified stratigraphic conditions along cross-sections A-A and B-B, respectively (see Figure 1 for location of the cross-sections). A dotted line is shown on both figures indicating the upper surface of soils with N values equal to or greater than 50 blows per 0.3 m. A brief description of the two soil groups is provided below:

i) Recent deposits (N value less than 50 blows/0.3 m): Very loose to compact sandy deposits varying in thickness from about 1.5 to 2.5 m were encountered overlying the very dense or hard soils at the edge of the proposed west wharf. Occasional deposits of compact sand and gravel were also found in this area. These deposits exhibited SPT N values between “weight-of-hammer” and 17 blows/0.3 m with the majority of the blow counts being less than 10 blows/0.3 m. East of BH 115 (inclusive), i.e., at the edge of the proposed east wharf, the composition of the recent deposits changed to loose to dense upper sand and gravel layers, about 1.5 to 3.0 m thick and occasionally overlain by thin layers of loose sand. SPT N values for these deposits ranged between 9 and 47 blows/0.3 m.

ii) Glacial till or till-like interbedded deposits (SPT N value equal to or greater than 50 blows/0.3 m): Soils consisting typically of clayey silt till or sandy silt till with interbeds of sand and gravel were encountered underneath the recent deposits. These deposits were characterized by SPT N values generally well in excess of 50 blows per 0.3 m. Cobbles and boulders were also frequently present in these deposits. For the ease of reference, all of these deposits with N values equal to or greater than 50 blows/0.3 m will hereafter be referred to as only “Till”. Previous Golder reports have identified the glacial till at Goderich Harbour to be extremely dense or hard even compared to other tills of known high density or hardness.

The encountered till surface was found to generally rise from west to east, being at about elevation 168.0 m at the location of BH 103 and gradually rising to about elevation 172.0 m at BH 105, as shown on Section A-A in Figure 2. This represents an upward gradient of about 0.7 per cent in the easterly direction. Although somewhat erratic between BH 112 and BH 1, the top of the till surface derived from Section B-B shown on Figure 3 generally follows the same trend from west to east.

The harbour bottom at the time of previous geotechnical investigations was at about elevation 171.0 m at the west end, gradually rising to about elevation 175 m at the east end. Current bathymetry data indicates past dredging activity showing that the harbour bottom has been deepened since then with an approximate elevation 166.5 m at the west end, rising to about elevation 172.0 m at the east end. Compared with the existing borehole data, this indicates that the harbour bottom is now formed by the till, possibly underlying a relatively thin cover of looser sediments.
6.0  PAST CONSTRUCTION EXPERIENCE

6.1  Sheet Piling and Other Retaining Structures

6.1.1  Construction of the Existing South Pier

Various forms of construction were employed for the south pier as indicated by Public Works Canada drawing No. 9028M and also summarized on Figures 8 and 27 of Golder Report No. 831-3218-1. The main components of the south pier construction can be summarized as follows:

i)  Deck elevation – 177.72 to 178.09 m

ii)  Concrete or asphalt deck width – 3.23 m to 15.43 m

iii)  Tie rod diameter – 44.5 mm to 70 mm

iv)  Deadmen – concrete block, concrete wall or steel sheet pile

All sections of the dock have a steel sheet pile facing. Section 1 is located at the west limit of the south pier with the section numbering increasing to the east. With the exception of Section 2, all sections are indicated to have a stone filled timber crib core. The bottom elevation of the timber crib is understood to be at about 170.8 m. It is further understood that the timber cribbing was originally constructed between 1872 and 1877, followed by the installation of the sheet pile facing much later, in 1953. Section 2 is understood to have originally had a stone filled timber crib core that later settled and collapsed. In addition, the Section 1 cribs apparently settled as much as 2.5 m and collapsed in 1948 and the Section 6 and 7 cribs collapsed in 1935.

All of the timber cribs had timber piles driven in front of them in the 1920s but their founding elevations or bearing conditions are not known. It is doubtful whether they extend below the existing sheet piling or provide any support for the south pier.

Figure 8 of Golder Report No. 831-3218-1 illustrates tip elevations for sheet piling inferred from documents received from PWC. The information is summarized below:

<table>
<thead>
<tr>
<th>Section #</th>
<th>Section Type</th>
<th>Length (m)</th>
<th>Tip Elevation of Sheet Piling (m)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SSP BZ II N</td>
<td>70.5</td>
<td>166.1 to 165.4</td>
<td>Piles terminating at or above rock surface. Timber crib collapsed.</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
<td>54.6</td>
<td>164.2 to 163.6</td>
<td>Piles terminating at or slightly below rock surface. Timber crib collapsed.</td>
</tr>
<tr>
<td>3</td>
<td>-</td>
<td>151.4</td>
<td>166.1 to 163.7</td>
<td>Piles to or set in rock.</td>
</tr>
</tbody>
</table>
### 6.1.2 Pile Driving Study Report, 1955

This section relates to a report titled “Report on Driving of Test Piles-North Harbour Wall”, dated April 11-15, 1955, provided by Public Works Canada to Golder. The test driving was carried out by Intrusion-Prepakt Ltd. of Toronto. The following facts from the report are noted:

1. The purpose of the study was to determine the feasibility of driving steel sheet piles into the dolomitic limestone. The testing was carried out at three locations along the north edge of the inner harbour, just south of North Harbour Road West.

2. The steel sheet pile section used was A-7 produced by Algoma Steel, weighing 36 pounds per foot with a web thickness of 7/16 inch. The hammer used was McKiernan-Terry double acting air hammer, size 9-B-3 with a ram weight of 1600 pounds (727 kg-force).

3. Damage to the pile tops was noted. Penetration into rock of up to 5 ft was reported at one of the testing locations. However, based on Golder’s review of the geotechnical investigation data and Golder’s opinion that the inferred rock surface elevation in the study area may actually have been lower than that reported, Golder concluded that penetration of the rock by steel sheet piling for a depth of more than several inches was highly unlikely.

### 6.1.3 Domtar Inc. Letter, 1983

This section relates to Golder’s review of construction information provided by Domtar Inc. (Domtar) in a letter dated November 23, 1983 to Transport Canada who loaned the information to Golder. Steel sheet piling was installed by Domtar on the north side of the existing north pier in order to reclaim land for salt storage. Design of the work was by Domtar, Montreal and installation was by Birmingham Construction from Hamilton. The northerly portion of the wall measured approximately 180 m and the westerly portion was 55 m. The wall was constructed using Arbed BZ-250 sheet piles, 28 m long anchors and BZ-250 sheet pile deadman anchors. Based on a review of the pile tip elevation information, the sheet piles were understood to have penetrated to but
not into the clayey silt till. In general, sheeting could not be driven below elevation 170.5 m or into soils with SPT N values on the order of 100 blows per 0.3 m.

After initial construction, a portion of the sheet piling at the northwest corner failed during a storm, possibly due to inadequate toe penetration in the failure area as indicated by pre-failure bottom soundings. Repairs were carried out by removing and replacing 27 m of wall. The section of the wall removed indicated considerable tip damage due to hard driving (an example of damaged piles is provided on Figure 4). The replacement sheet piling consisted of Arbed BZ-350 sheet piles which were subjected to hard driving with a Birmingham B-400 hammer (46,000 ft-lbs/63,000 joules). Although piles were driven to a set of 5 or 6 blows per 25 mm and boreholes were used to aid penetration, piles could not be driven beyond elevation 170.4 m. This elevation corresponded to the top of till encountered in the nearby borehole.

6.1.4 Summary

In summary, construction experience at Goderich Harbour suggests that steel sheet piling was able to penetrate relatively dense sand and gravel but did not significantly penetrate the very dense or hard glacial till or the bedrock. Based on a review of past construction performance achieved by different contractors and different types of driving equipments, the following general conclusions can be made:

1. It is unlikely that sheet piles penetrated hard or dense till (SPT N >100 blows per 0.3 m) by more than 0.5 m.
2. Where piles are indicated to have been driven into or through one or more metres of very dense sand and gravel or hard till, each containing cobbles and boulders, it is likely that some of the piles were heavily damaged.
3. It is unlikely that piles penetrated the dolomite or limestone bedrock by more than 150 mm if the bedrock was encountered at all.

6.2 Dredging

Previous dredging operations at Goderich Harbour have experienced difficult dredging conditions and low production rates due to the very dense or hard nature of the till, leading to a claim by a contractor (Canadian Dredge and Dock Inc.) related to the volume and density/consistency of the dredged till during Phase-1 dredging operations in 1985 as part of the Goderich Harbour expansion. The average production rates achieved by using different dredging equipment during the 1985 dredging operations are summarized below:

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Average Production Rate (Cubic Metres per Hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Looser/Softer Material Overlying Till</td>
<td>Glacial Till</td>
</tr>
</tbody>
</table>
### Equipment Average Production Rate (Cubic Metres per Hour)

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Average Production Rate (Cubic Metres per Hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Looser/Softer Material Overlying Till</td>
</tr>
<tr>
<td>4600 Manitowoc Clamshell</td>
<td>55.9</td>
</tr>
<tr>
<td>4500 Manitowoc Dipper</td>
<td>-</td>
</tr>
<tr>
<td>981 Liebherr Backhoe</td>
<td>37.6</td>
</tr>
</tbody>
</table>

Based on observations during a Golder site visit on November 15, 1985, a short duration comparison (involving several cycles) was made between excavation rates achieved by two different dredges. It was noted that the Dredge 79 (Cartier McNamara) was experiencing better production than the Andrew B in spite of a smaller bucket, longer cycle and thicker till. The approximate comparison is as follows:

<table>
<thead>
<tr>
<th>Dredge</th>
<th>Bucket Size (m³)</th>
<th>Quantity Excavated per Cycle (m³)</th>
<th>Cycle Time (sec)</th>
<th>Excavation Rate (m³/hour)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Andrew B</td>
<td>4.6</td>
<td>1.5</td>
<td>84</td>
<td>64.8</td>
</tr>
<tr>
<td>Dredge 79</td>
<td>3.1</td>
<td>3.1</td>
<td>117</td>
<td>93.6</td>
</tr>
</tbody>
</table>

The comparison implies that the productivity could have been higher on a mixed face, i.e., had the materials overlying the till not already been removed.

A marine project in Collingwood related to construction of a sewer out fall required dredging of a hard till similar to that described for Goderich. It was found that this material could be excavated, with considerable difficulty, by a large barge-mounted backhoe working in shallow water. In deeper water, excavation was attempted using a 1.9 m³ clam mounted on a 150 ton LS518 Link-Belt Crane. Successful use of this equipment required that the till be first drilled and blasted.

An example of damage to dredging equipment caused by the difficult soil conditions encountered in past dredging operations at Goderich Harbour is provided on Figure 4.

### 7.0 RECOMMENDATIONS FOR CONCEPTUAL DESIGN

#### 7.1 Retaining Structures

##### 7.1.1 Anchored Sheet Pile Bulkhead/Wall

The use of steel sheet pile bulkhead is being considered for the east wharf according to the “single slip” and “twin slip” options. The retained height of fill is understood to be 12.5 m and a concrete cap or coping wall is
planned to be constructed along the top of the piles. The proposed elevations for the top of coping wall and the top of fill are 179.5 m and 179.3 m, respectively, with the proposed dredge line at about elevation 167.0 m.

As discussed previously, there is considerable evidence that steel sheet piles installed previously in the harbour were able to penetrate the relatively dense sand and gravel in some cases, with damage, but did not significantly penetrate the glacial till or the bedrock. Based on this information, it is considered that achieving toe penetration of the sheet piling would be difficult and likely insufficient by conventional pile driving and the penetration required for stability would have to be achieved by means of excavating a trench in these materials or by installing soldier piles (“shear keys”) in front of the sheet piles. Depending on the trench geometry and construction procedures, the sheet piles would need to be installed in the trenches and the trenches then backfilled with concrete placed by tremie methods around the pile toe depth.

Conceptual design of sheet pile bulkheads may be carried out using the following values for soil parameters:

- Total unit weight of wall backfill: 21.5 kN/m³
- Submerged unit weight of wall backfill: 11.7 kN/m³
- Coefficient of active earth pressure, \( K_a \): 0.3
- Total unit weight of native soil at and below the dredge line (native till): 22.5 kN/m³
- Submerged unit weight of native till at toe: 12.7 kN/m³
- Coefficient of passive earth pressure for native till, \( K_p \): 3.8

A conventional factor of safety of 1.3 should be applied to the calculated depth of toe embedment.

Anchors for the sheet pile wall will be located in the fill behind the wall. Sheet pile anchors for wharf construction are typically placed above the low water level. The deadman anchors can be discrete precast or cast-in-place concrete blocks or continuous sheet pile walls. Anchor locations would be established by determining the zone of active earth pressure zone in the retained fill and the passive earth pressure zone in front of the anchorage and ensuring that the two zones do not intersect below the surface of the backfill. This is to allow for the full development of the passive resistance for the ground anchorage without interruption from the active pressure zone behind the sheet pile wall. An angle of internal friction, \( \phi' \), of 32 degrees may be assumed for the backfill in determining the active and passive zones until later stages of design when the backfill materials are more clearly defined.

The use of a reinforced concrete relieving platform can be considered for reducing the lateral earth pressures on the sheet pile wall. Relieving platforms are often used in harbour construction which typically involve heavy surcharge loads (e.g., stockpiled bulk materials, heavy wharf cranes). The platforms are constructed such that the surface surcharge loads and the load from the retained soil above the platform are transferred to a lower stratum via bearing piles, thereby reducing the active pressure on the sheet pile wall. In addition, the bearing piles that support the platform may also serve as an anchorage for the tie rod. In this particular case, pile construction for such a relieving platform would likely require use of drilled shafts (i.e., bored piles, “caissons”) constructed using temporary casings and rock drilling methods to adequately penetrate into the glacial till materials. In general, driven bearing piles should not be considered for this project since lateral forces and toe depths may be inadequate due to difficult driving conditions.
A concrete capping beam should be provided along the top of the sheet piles to stiffen the bulkhead top. Well-graded backfill behind the sheet pile wall generally assists in limiting the loss of fines through the interlocks and consequent subsidence of the surface of the retained fill. In addition, the use of backfill materials selected to form graded filter zones will be necessary to minimize the potential for loss of sand and silt particles from any backfill materials through sheet pile interlocks or into the void spaces of other fill materials (e.g., rock fill). Recommendations related to fill materials and their placement are provided in a subsequent section of this report. Properly filtered weep holes should be provided for the sheet piles above the mean low water level to promote full drainage of the backfill with changing lake levels.

On account of the insufficient penetration likely to be achieved by driving sheet piles, the use of “shear keys” may be considered to provide toe restraint of the sheet piles if excavated and concrete-filled trenches are not used to facilitate toe penetration and passive resistance. Such shear keys often include the use of driven or drilled piles immediately against the dredged side of the sheet pile toe that penetrate well below the sheet pile toe where the sheet piles cannot be driven sufficiently deep without sustaining significant damage. Shear keys have been used in portions the existing south pier to provide toe restraint. However, considering the constructability issues associated with the difficulty of installing shear keys into the very dense and/or hard glacial till and sand and gravel and the poor quality of the rock, shear keys may not provide a technical or cost-effective option. Subsequent design phases should examine the potential advantages and disadvantages of using shear keys in greater detail to ascertain their economic and technical viability for toe support.

### 7.1.2 Sheet Pile Cells

The use of circular steel sheet pile cells filled with 100 to 200 mm maximum dimension stone material is being considered for the west wharves of the “single slip” and “twin slip” option, and also for the “no slip” option. The cells are planned to be 12.2 m in diameter, spaced at 50 mm centre to centre each capped with a concrete pad with a top elevation of 179.5 m. The proposed fill area for the new wharf behind the cells (top elevation of 179.3 m) will slope towards the dredge line at a 2H:1V slope and will be interrupted by the cells at approximately mid-height of the slope. It is understood that the cells will serve as berthing or mooring dolphins for vessels and allow for loading or unloading of cargo. Access to the dolphins from the infill area will be provided by bridges.

The conceptual cellular marine structures are designed to be self-supporting gravity retaining structures constructed using straight-web sheet piles. If provided with a solid foundation, they require only nominal penetration to be stable. Pile penetration will mainly assist in resisting any lateral loading during the construction of the cofferdam before the internal fill is placed and the cell becomes inherently stable. Penetration depth is also selected to address potential future dredging needs. Usually, the driving of sheet piles for such cellular structures should be performed with extreme care to prevent rupture along the pile interlocks whose function is critical in preventing cofferdam failures under hoop tension. Similar to the recommendations for sheet pile bulkhead construction at Goderich Harbour, the penetration will likely have to be achieved by means of excavating a trench in the till or bedrock, placing the piles and backfilling the trench with tremie-placed concrete, or by installing rock pins or rock-socketed soldier piles.

In general, free draining granular fill with negligible amount of fines (< 5% passing N0. 200 sieve) and a high angle of internal friction should be used as cell fills. This will result in the least amount of internal lateral pressure.
on the cell walls that would need to be resisted by the hoop tension in the sheet pile interlocks, and allow for a more economical design in terms of sheet piling. Cell fill materials are generally deposited under water once the sheet piles are in place and hence may be relatively loose. The relative density of the cell fill may be increased by careful compaction with vibratory equipment. Use of 100 to 200 mm maximum dimension stone materials should be suitable for conceptual design and costing for the cellular sheet pile structures. Recommendations related to selection and placement of fill materials are provided in a subsequent section of this report.

For the landward side of the cells, estimates of lateral earth pressures induced by the sloping fill and/or any surcharge load may be based on design parameters provided for the steel sheet pile wall in Section 7.1.1 provided the fill consists of similar free draining granular material. Sliding resistance against the unbalanced loading from the sloping fill may be estimated using a base friction coefficient of 0.4 for the angular stone or selected granular cell fill materials and the native glacial till. Sliding resistance can be improved by increasing the depth of penetration of the cell below the planned dredge line, as for the toe penetration of sheet pile bulkheads.

During subsequent phases of design, the cell structure design should also be checked for adequate factors of safety against global slope stability, cell shear along the centreline and cell bursting due to interlock failure. Heavy surcharge loads such as cranes should be accounted for in the cell design. Further, uncontrolled dredging below the design dredge line and/or scouring of the lake bed at the sheet pile toe on the lakeward side during floods or propeller turbulence could result in cell displacement and/or damage.

The conceptual design indicates that the cells are to be capped with a concrete pad. However, the internal fill in the cells will settle (possibly differentially) and could cause cracking of the overlying slab. The use of an asphalt mix may be better suited to control cracking, surface water infiltration, and facilitate relatively cost-effective repairs. Alternatively, a combination of cap and sheet pile cell coping design that permits relative movement between the coping and cap may alleviate damage associated with settlement of the cell infill. Design and placement of the cell fill and cap should be examined in detail during subsequent phases of design.

### 7.1.3 Other Gravity Structures

Due to the expected difficulty with driving sheet piles in the construction of sheet pile bulkheads or cellular sheet pile structures, the use of other types of retaining structures that rely on their geometry and mass to resist lateral and overturning loads from backfill may also be considered as alternatives to structures composed of sheet piles. Two possible options are briefly described below:

i) **Reinforced concrete, ballasted caissons**: These are precast reinforced concrete structures that are floated to the appropriate location and sunk on a prepared foundation consisting of free-draining granular materials. The caissons are filled with gravel and rock and capped with cast-in-place concrete and, depending on the final design, may include a relatively low-height retaining wall at the top to be integrated with the general harbour fill behind the structure. Select granular materials are usually placed as backfill to the small retaining structures cast at the top of the caissons. It is understood that the existing concrete mooring dolphins along the west end of the existing salt storage and distribution wharf were constructed using precast concrete units. In general, design of precast concrete cells will require consideration of
foundation preparation, tension forces, infill material selection, and capping as discussed for the sheet pile cell structures, above.

ii) Precast Concrete Blocks: Large precast concrete blocks resting on a prepared foundation consisting of free-draining granular materials may also be used for forming the mass of the wharf. The blocks are commonly placed in an offset manner such that the centre of gravity is shifted landward to reduce the eccentricity of base reaction and the toe pressure at the foundation level.

7.2 Protection of Slope Face

It is understood that the faces of land reclamation fill slopes will be protected by armour stone designed by others to address dissipation of wave energy. Once the armour stone is selected, the particle or grain size distribution of materials beneath the armour stone will need to be carefully selected to achieve appropriate filtration characteristics to minimize the potential for migration of fill materials that are smaller than the armour stone (e.g., silt, sand, and gravel) into or through the void space in the armour stone layer. Discussions related to preliminary selection of fill materials is provided below.

7.3 Fill for Land Reclamation

Significant infilling of the lake is planned for all three design options, the least and the greatest volume of infilling being for the “twin slip” and the “no slip” options, respectively. The lake area to be reclaimed is bounded by the northeast and northwest breakwaters, the retaining structures along the southern edge, and the existing north pier along the eastern edge.

Land reclamation for marine facilities is commonly performed using hydraulic fill derived from concurrent dredging operations because of availability and low cost. Material other than hydraulic fill may be considered depending on the availability of onshore borrow areas and cost considerations. The possible sources for fill at this site may be categorized as follows:

i) Granular fill from nearby quarries: These fill materials may provide the best control in terms of achieving the desired grain size distribution characteristics for underwater fills.

ii) Hydraulic fill obtained by suction dredging: These fill materials will comprise primarily of the loose sandy deposits overlying the glacial till.

iii) Fill consisting of mixed materials: Fill materials obtained by mechanical dredging will be a mixture of the loose sediments and the glacial till materials, and are expected to contain the greatest amount of fine particles (silts and clays, i.e., soils passing the #200 sieve (0.075 mm)), and the greatest potential for adverse segregation during subaqueous placement among the three options. Fill materials from uncontrolled sources (e.g., local construction projects) should not be used without stringent control of grain size distribution and environmental chemistry characteristics.

Design and selection of the land reclamation materials will need to consider the following:
Coarse-grained fills with less than about 15% fines are typically the best type of fill materials for underwater placement as they would drain quickly, minimizing long-term settlement problems and cause the least turbidity during placement. The use of vibro-compaction methods may be considered for densifying such fills, if needed.

Fill materials containing a significant amount of fine grained soils (silts and clays) could initially be at high water content following placement, will drain slowly due to low permeability of the soils, and will cause long-term consolidation settlement problems. The use of sand drains coupled with surcharging may be contemplated to accelerate consolidation of such fills that may otherwise take years to achieve full consolidation. The magnitude of such settlements is considered unquantifiable and will likely be differential across the reclaimed area owing to the potentially uncontrolled nature of the fill materials, variable thickness dictated by the contours of the lake bottom, and the thickness of the native loose sediments underlying the fill area.

Monitoring of settlement and excess pore water pressure dissipation in the fill area may be required to understand the settlement rates and ascertain the time of any planned construction supported by the fill.

7.4 Dredging

Based on the information available, the primary soils to be dredged are sand, sand and gravel and till. The till, because of its very dense or hard nature, will be difficult to excavate. Depending on the type of dredging equipment employed by contractors, drilling and blasting may be required to loosen the deposits. The presence of boulders should also be anticipated during dredging and drilling for marine blasting operations.

If marine blasting techniques are to be used, active bubble curtains and other measures will be necessary to minimize the effects of such dredging methods on aquatic life. Such work has been carried out for marine construction projects in Ontario and should be considered in more detail as the design progresses.

8.0 FINAL INVESTIGATIONS AND CLOSURE

This preliminary geotechnical design report has been prepared based on the existing information to aid with the planning and design of the new construction work associated with the proposed North Wharf Expansion of Goderich Harbour. Provided that dredging methods are appropriately selected and a cleaned and dredged surface within the native glacial till or sound rock is achieved, subsurface conditions at the proposed dredge level are favourable for gravity structures for all of the three design options currently under consideration. In general, it is preferable to support all structures on the native glacial till rather than penetrating to near the interface of the underlying granular soils or bedrock because of the artesian water pressures. During final design, artesian water pressures will require further consideration. It is unlikely that sufficient toe penetration would be achieved by driving steel sheet piling or bearing piles. Bored piles are a potential alternative for wharf construction but difficult installation conditions are anticipated because of both difficult drilling and artesian water pressures. The use of
gravity retaining structures, such as ballasted reinforced concrete caissons or precast concrete blocks that do not rely on toe penetration for stability, may be the most suitable alternatives.

Once design concepts for the harbour modification have been developed further and founding elevations proposed for the retaining structures, a final lakebed investigation program should be developed. The goal of the final geotechnical investigation would be to:

- confirm the conditions used for design that were based on the existing data;
- optimize and potentially reduce the on-water field work; and
- delineate the conditions only at the locations of critical construction components and also in the planned land reclamation area not explored by the existing boreholes.

During subsequent design stages of the project, Golder should be afforded the opportunity to provide design consultation related to the retaining structures, placement and staging of fill materials, marine blasting, etc.
We trust that this report provides all of the geotechnical information presently required. Should any point require clarification, or should you have any comments on this report, please contact this office.

GOLDER ASSOCIATES LTD.

ORIGINAL SIGNED
Mrinmoy Kanungo, M.E.Sc., P.Eng.

ORIGINAL SIGNED
Storer J. Boone, Ph.D., P.Eng.
Associate

MK/SJB/cr/ly

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

**Soil, Rock and Groundwater Conditions:** Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.
IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont’d)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client’s expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder’s report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder’s report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder’s report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder’s report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder’s responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.
NOTES
This drawing is schematic only and is to be read in conjunction with accompanying text.
For cross-sections, refer to Figure 2 and 3. All locations are approximate.

LEGEND
- EOREHCLE (Golder Investigation 831-3118-2)
- EOREHCLE (Golder Investigation 831-3118-1)
- EOREHCLE (Golder Investigation 831-3118)
- EOREHCLE (Donion Soils Investigation 85-6-4 Enc. 1)

REFERENCE

GODERICH HAREBOUR, ONTARIO
LOCATION PLAN
FIGURE 1
CROSS-SECTION A-A

LEGEND

SIMPLIFIED STRATIGRAPHY

- WATER
- SAND
- SANDY SILT TILL
- CLAYEY SILT TILL
- SAND
- SAND AND GRAVEL
- SILTY CLAY
- WATER
- COBBLES
- BOULDERS
- DOLOMITE
- LIMESTONE

REFERENCE


NOTES

THIS DRAWING IS SCHEMATIC ONLY AND IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

ALL LOCATIONS ARE APPROXIMATE.

FIGURE 2

PROPOSED NORTH WHARF EXPANSION
GODERICH HARBOUR, ONTARIO

BOREHOLE (Golder Investigation 831-3218-1)
BOREHOLE (Golder Investigation 831-3218)
BOREHOLE (Dominion Soils Investigation 85-6-5 Enc. 1)

SIMPLIFIED STRATIGRAPHY

SANDY SILT TILL
CLAYEY SILT TILL
SAND
SAND AND GRAVEL
SILTY CLAY
WATER
COBBLES
BOULDERS
DOLOMITE
LIMESTONE

BOREHOLE (Golder Investigation 831-3218-3)
Photograph 1: Damaged sheet pile toes shown after extraction

Photograph 2: Damaged and worn dredging equipment

NOTES
FIGURE TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.
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November 2, 2012

Mr. Andrew Ross, P.Eng.
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PRELIMINARY GEOTECHNICAL DESIGN REPORT
PROPOSED NORTH WHARF EXPANSION
GODERICHER, ONTARIO
REPORT NUMBER 12-1132-0059-1000-R01, AUGUST 2012

Dear Mr. Ross:

As discussed in our October 25, 2012 meeting, it is understood that the preferred option for the harbour expansion will be the two-slip option. Further, based on the lakebed subsurface conditions described in the Golder Associates Ltd. (Golder) report (referenced above), the main wharf bulkhead will likely be formed using interlocking steel sheet piles to form circular cellular structures (cofferdams) that are then topped and fronted by a cast-in-place concrete parapet wall. One of the benefits of this construction approach is that it should not be necessary to embed the sheet piles significantly into the glacial till, pending final decisions related to future dredging levels. Given the preferred two-slip option, this letter summarizes our recommendations related to further investigations for detail design of the structures.

In the report referenced above, it was noted that final design geotechnical investigations should be based on the following goals:

- confirm the conditions used for design that were based on the existing data;
- optimize and potentially reduce the on-water field work; and
- delineate the conditions only at the locations of critical construction components and also in the planned land reclamation area not explored by the existing boreholes.

For the two-slip, cellular cofferdam option, the information that will be needed for detail design and contracting will largely relate to defining the depth from the present harbour bottom elevations to the top of the glacial till. Since installation of shear pins or sheet pile toe trenches penetrating into the glacial till are considered higher risk options for this project and will likely not be constructed, additional drilling into the glacial till and coring of bedrock is not considered necessary at this time. The existing borehole data provides sufficient information to characterize the subsurface materials for design of cellular cofferdam structures and additional investigation to better define the depth to the glacial till will assist with estimating sheet pile tip depths and, thus, construction costing.
For final investigations, Golder recommends completion of a marine geophysical survey for the following reasons:

a) Data collection can be carried out from conventional watercraft and therefore jack-up or spud barges that would otherwise be necessary to maintain a stable platform for drilling and coring would not be needed, thus significantly reducing the investigation costs.

b) Given the hard and very dense character of the glacial till, the marine geophysics should be suitable for defining the interface between the relatively recent sediments and glacial till with sub-metre resolution. It is recommended that the results from the geophysical survey work be confirmed with a limited program of jet probing. This would help to calibrate the geophysical survey and improve the accuracy of the result.

c) A relatively large area of the harbour, on the order of 300 m by 70 m, can be surveyed within approximately one day of on-water work.

d) If the results of the geophysical survey are indeterminate or inconclusive, this will be apparent within the first hour of the field work. The geophysical survey could then be stopped and an expanded jet probe survey could be completed within the allotted on-water work time.

A cost estimate for completing the marine geophysics survey will be provided in a separate letter.

GOLDER ASSOCIATES LTD.

Storer J. Boone, Ph.D., P.Eng.
Associate

Mark Monier-Williams, M.Sc.
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SJB/MMW/cr