W.F. Baird & Associates Coastal Engineers Ltd.

Office | 1145 Hunt Club Road, Suite 500, Ottawa, Ontario K1V 0Y3, Canada
Phone | +1 613 731 8900 Email | ottawa@baird.com

Mr. Hector Carranco
Club Manager | Petrie Island Canoe Club
795 Tweddle Road,
Ottawa, ON

via email to petrieicc.manager@gmail.com

Status: Correspondence
February 13, 2023

Dear Hector,

Reference # 13909.101.L1.Rev0

RE: COASTAL ASSESSMENT FOR PETRIE ISLAND CANOE CLUB

Baird has been requested to undertake a coastal engineering assessment to support the construction of a storage building in the flood plain at Petrie Island. The structure, which will be used to store canoes and related equipment, will be subject to flooding during larger spring flood events. Therefore, the structure must withstand floodwater, wave forces, and current/impact loads.

This assessment builds upon the work that was completed for the installation of the "sea-cans" that are presently used for storage in the area. The proposed building will be located in a region with an elevation of about 43.5 m. The first floor elevation is set to 43.73 m.

Water Levels

Water levels are defined through the Rideau Valley Conservation Authority flood mapping, which is shown in Table 1; a plot of the water level exceedance is provided in Figure 1. This table and figure suggests that the 43.73 level is just below the 10 year return period and is exceeded about 0.2% of the time.

Table 1 – Summary of Modelled Water Levels from Ottawa River Flood Risk Mapping Study (RVCA, 2014)

	2-yr	5-yr	10-yr	20-yr	50-yr	100-yr	Regulatory Flood Level
Hull Marina	43.62	44.30	44.71	45.09	45.57	45.91	46.00
Petrie Island	42.82	43.43	43.80	44.15	44.60	44.92	45.00

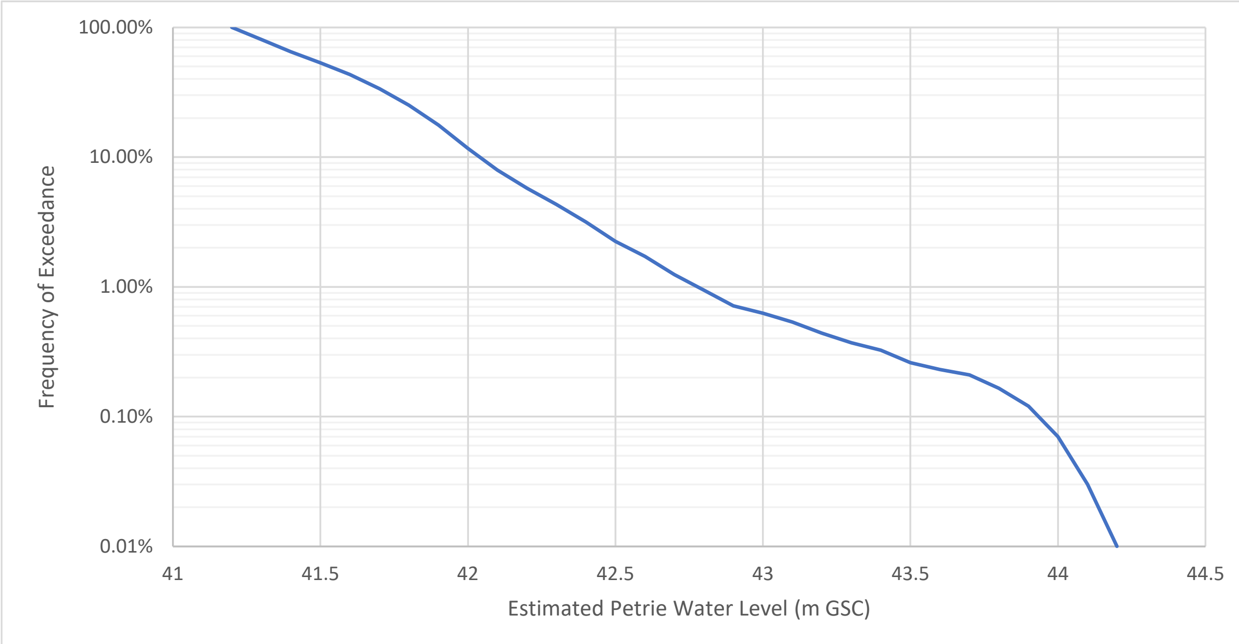

Figure 1: Water Level Exceedance at Petrie Island

Flood duration was also assessed based on the measured water levels at Hull, which were converted to Petrie Island based on a relationship between the gauges at Hull and Cumberland. The duration of the flood above three selected levels is presented in Table 2. In the period from 1965 to 2022, the 43.75 building level was only exceeded on four years. The longest duration of flooding was 24 days in 2019, while other years were significantly less.

Table 2: Flood Durations at Petrie Island

Year	Consecutive Days Above Specified Water Level		
	43.25 m	43.50 m	43.75 m
1974	13	9	5
1976	5	3	2
1979	4	0	0
1981	3	0	0
2017	13	11	7
2019	41	29	24

Flood duration and water levels at Petrie Island are estimated from adjacent measured levels and modeling results. Ice jams, sub-day effects, and other unusual variations are not considered in this analysis. It was this uncertainty that led to the assessment of the 43.25 and 43.50 m levels. Flood at the 43.5 level (the surrounding land) showed only slightly longer duration flooding.

Wave Forces

One of the design considerations at the site is the potential for waves to impact the building during a flood condition. The largest waves at the site will be generated from an ENE direction, where the effective fetch is about 4 km. Waves could also approach from the north, but with a much smaller fetch and smaller wave height from this direction.

Waves are determined based on an assessment of wind speeds at the Ottawa Airport, and wave hindcasting equations to determine the wave height and period over the appropriate wave generation fetch. Using a 20 year wind speed of 70 km/h, the resulting wave condition over a 4 km fetch is 0.86 m (Hm0) with a wave period of 3 seconds. For waves approaching from the north, the fetch is about 1 km, and the resulting wave height is 0.53 m. Lower wind speeds will result in less severe wave conditions

Wave transformation from the deeper sections of the river to the building site is completed using Goda's formula for depth limited wave breaking. With a 1.1 m water depth (approximately the 50 year water level) above the 43.5 m surrounding grade, the significant wave height impacting the easterly wall of the building would be about 0.62 m. For the north wall of the building, the wave height would be about 0.46 m in 1.1 m of water. However this assumes large waves are occurring during the flood.

Wave forces impacting the wall are determined based on the maximum single wave height that is expected in these depth limited conditions. The force calculations are not very sensitive to small changes in the offshore wave height as the wave height that can impact the wall is limited by the available water depth. The wave force is defined by a peak force at the waterline, a force at the ground level, and an elevation for the crest of the impacting wave. The results of these force calculations for a wall exposed to the ENE is shown in Figure 2 for a water depth of 1.1 m (44.6 m water level) and for a water depth of 0.75 m (44.25 m water level). Similar data are presented in Figure 3 for a wall that is exposed to shorter fetches to the north and therefore a lesser wave condition. A 1.1 m water depth is approximately the 50 year water level, while a 0.75 m water depth is about a 25 year water level.

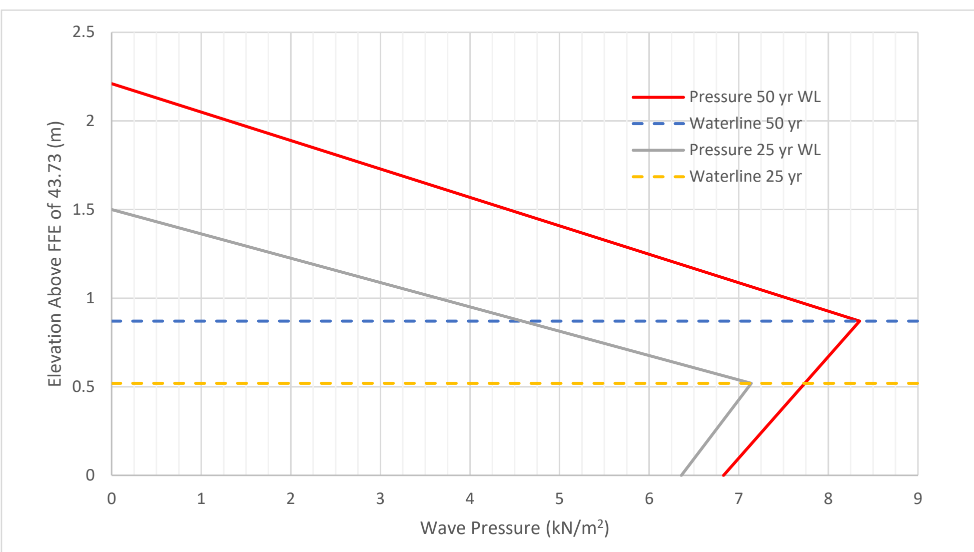

Figure 2: Wave Pressures Along a Wall for Easterly Wave Exposure

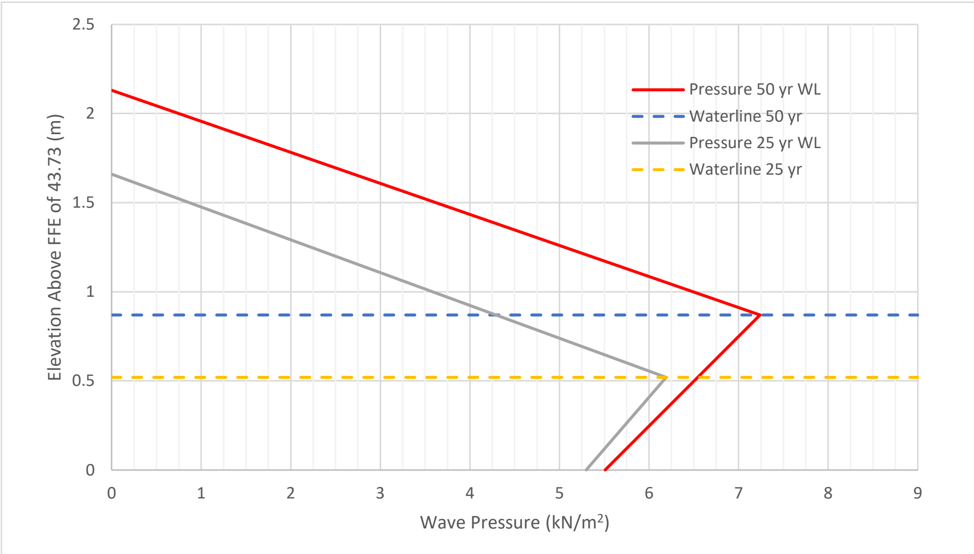

Figure 3: Wave Pressures Along a Wall for Northerly Wave Exposure

The final selection of a design wave load should consider that a flood may occur, but wave forces require that higher water levels and higher winds occur concurrently. A water depth at 1.1 m above a ground level of 43.5 m is about a 50 year water level and has been assessed with a wind speed of 70 km/h, which is about a 20 year wind speed. This has a very high return period (arguably 1000 years); however, a lesser wind (e.g. 50 km/h) would produce smaller offshore waves. These waves would also break before impacting the building and would result in a similar wave height at the building. Therefore, a 50 year flood level, with a strong (but not extreme) wind during the flood might be something more like a 200 year return period, if we assume there is a 25% chance of a strong wind during the flood. Similarly, a 25 year water level with a strong wind would be more representative of about a 100 year event.

Wave forces on the south and west sides of the building should be very limited due to trees and the limited fetch for wave growth. It is also unlikely that a single maximum design wave would impact the full length of the building concurrently. The wave would be at a slightly oblique angle and/or would be higher in one area and reduced elsewhere.

Floodwater Velocity and Debris Impact

Floodwater velocity was addressed in the assessment that was completed for the sea-cans; the results are the same for the proposed structure in the same location.

The water velocity at the project site was estimated using the results from RVCA's HEC-RAS model of the Ottawa River. Predicted water levels for the 2-year and 100-year flood at the Petrie Island cross-section (#1015) are shown in Figure 4. The proposed building is located approximately 1650 m from the left river bank at an elevation of 43.5 m CGVD28. RVCA used a Manning's roughness coefficient of 0.0288 for the main river channel and 0.07 for the floodplain.

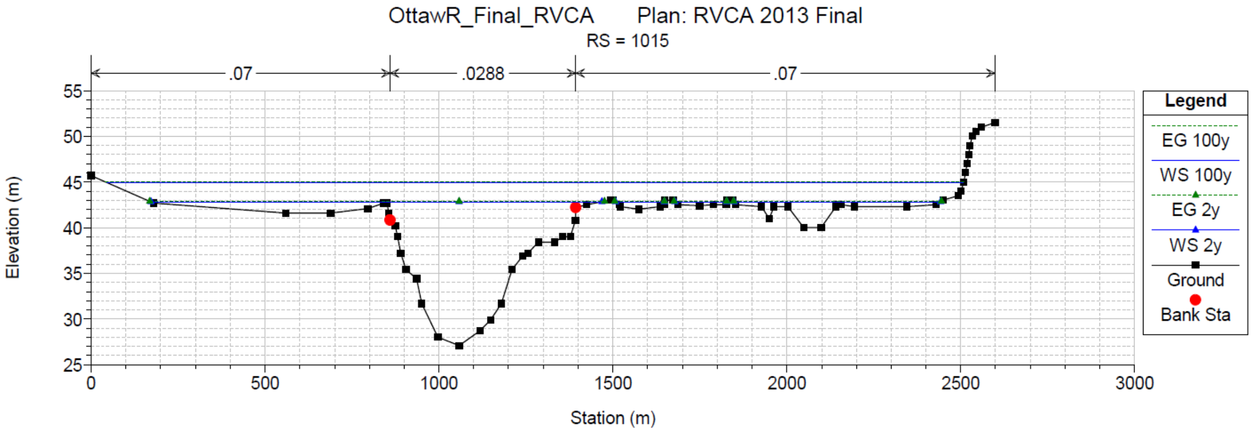

Figure 4 – Predicted 2-year and 100-year Water Levels at Petrie Island from RVCA's HEC-RAS Model

In order to estimate the water velocity at the building location, Baird subdivided the river cross-section into 10 m segments and calculated the volume of water (and velocity) flowing in each section. The Manning's roughness coefficient for the floodplain between the main river channel (~1400 m) and the parking lot (~1700 m) was reviewed and reduced to 0.0288 to account for the lack of heavy vegetation in this region. Reducing the roughness coefficient results in higher predicted water velocities.

For the 100-year flood, a water velocity of approximately 0.5 m/s is predicted at the proposed building location (see Figure 5). This compares with PICC's estimates of water velocity (0.2 to 1.0 m/s) from the site visit. Note that the predicted water velocities decrease abruptly at station 1700 m due to the higher Manning's roughness in the heavily vegetated region.

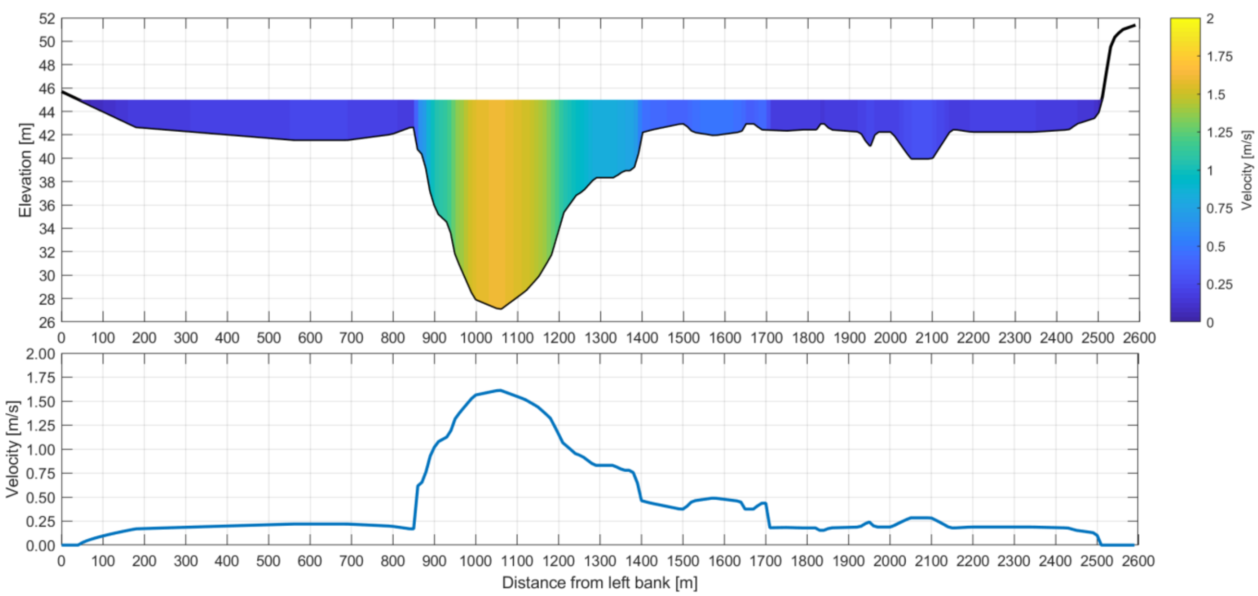

Figure 5 – Predicted Depth-Averaged Water Velocity at Petrie Island for the 100-year Flood Condition

Debris impact is typically considered for logs, ice and in some instances vessels that may break loose in a flood. At the project location, there is adequate tree cover upstream such that large ice floes or vessels are not expected to impact the site. The most likely type of debris that may impact the site would be logs or sections of a tree.

When a tree/log is moving with floodwater, it has a tendency to move with the water and will often avoid large obstacles as the water flows around them. In some cases, there can be a direct impact, but these would be rare. A direct impact from the log aligned with the direction of flow is the worst case and the force can be approximated based on the mass of the log, and the speed using a momentum approach. The publication "Engineering Principles and Practices for Retrofitting Flood-prone Residential Buildings" (FEMA 1995) provides guidance on this and suggests a 1 s time frame for the log to stop on impact. If we assume a 10 m, 0.37 m diameter log, this is approximately one tonne of mass (assuming well saturated). Figure 6 shows the resulting force for different stopping times. This suggests a load of about 1 kN; however, this assumption of a one second duration to stop does not seem like a conservative assumption. A shorter duration higher load should also be considered as possible, although perhaps in combination with a smaller log mass.

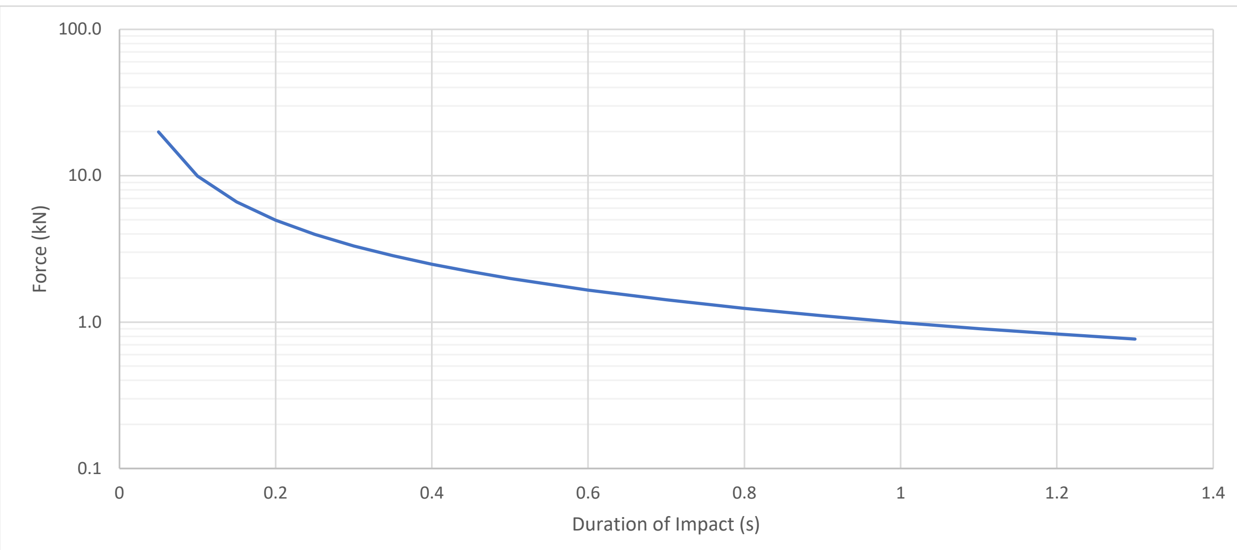

Figure 6: Force Exerted by One Tonne Log Moving at 1 m/s

Overall, there are many assumptions about the potential impact load on the building. A one tonne log is certainly larger than most that would be encountered in the river. If it has branches it could be slowed down due to dragging in the shallow water. The duration of the load would also be variable depending on the integrity and shape of the log, with a flat, strong, even end being the worst case for a direct impact.

With the range in potential object sizes that could be considered, and the influence of impact time, an impact force could arguably be selected over a wide range. A value of 2 kN could be considered as appropriate on the upstream side of the building; however, this should be considered in context with the other building design parameters.

Hydrodynamic Loads

Hydrodynamic loads are loads caused by moving, rather than still water. The magnitude of the loads depends on the water depth, velocity, direction of flow, and shape of the building (see Figure 7). Provisions in ASCE 7 allow for hydrodynamic loads to be converted to an equivalent hydrostatic load for velocities less than 3 m/s.

This is done by increasing the design flood elevation by an equivalent surcharge depth on the upstream side of the building (see Figure 8). The surcharge depth considers the drag caused by the shape of the building and water velocity. Water velocities will vary over a region due to differences in terrain, vegetation, flow paths and obstructions.

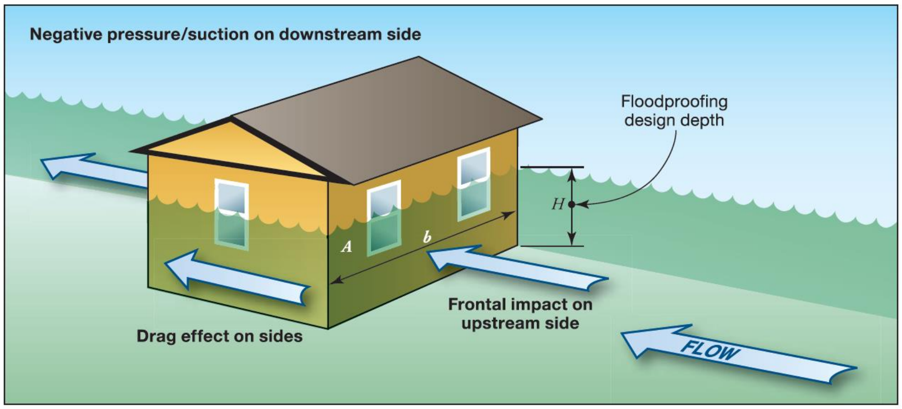

Figure 7: Hydrodynamic Loads on a Building (FEMA P-259).

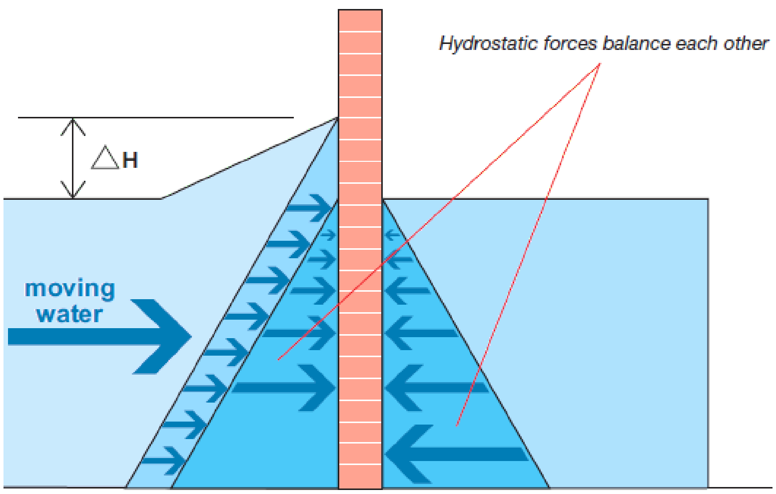

Figure 8: Conversion of Hydrodynamic Loads to Equivalent Hydrostatic Loads (Hawkesbury-Nepean Floodplain Management Steering Committee, 2007)

The 100 year flood event (rather than a lower return period value) is selected for hydrostatic loads as there is no additional circumstance required (like a concurrent wave); this is a stand-alone loading event. For the Petrie building site, we calculated a water depth of 1.2 m and an average water velocity of 0.5 m/s for the 100-year flood event. This produces a building-width-to-water-depth ratio of approximately 15, resulting in a low drag coefficient. Due to uncertainties in the estimation of the water velocity at the site caused by two-dimensional effects (variations in topography, vegetation, obstructions, etc.), we have selected a design velocity to 1.5 m/s. This compares with the current speeds estimated by PICC on May 8, 2017 (0.2 to 1.0 m/s).

The resultant hydrodynamic force on the upstream (western) face of the building is estimated to be 1.9 kN/m (34 kN applied over the 60 ft building length). Note that this current force would not need to be combined with wave loads, as waves approaching from the west would be minimal due to the forest cover.

Scour

Scour is localized erosion caused by the acceleration of water around an obstruction such as the proposed building at Petrie Island. Stone protection is often required around the base of in-water structures to prevent scour and undermining of the structure.

To determine the required stone size for scour protection at bridge piers, NCHRP (2006) recommends that the Isbash equation be used to estimate the median stone diameter. The velocity used in the Isbash equation should be representative of conditions in the immediate vicinity of the structure, which for square-faced structures is 1.7 times the average velocity. For the Petrie Island site, this yields a design water velocity of 2.5 m/s in the vicinity of the structure and a median stone diameter of 0.13 m for the scour protection. Ontario Provincial Standard Specification (OPSS) 1004, Class G-10 gabion stone (0.1 to 0.18 m) would meet the scour protection requirements for this building. With the proposed concrete pad around the building, only the unprotected west (upstream) side of the building would need scour protection. The stone protection should be installed to a depth of 0.4 m and extend 3 m out from the building wall.

Sincerely,

Derek Williamson, P.Eng | Principal
Baird & Associates
E: dwilliamson@baird.com
M: 613 868 6213