Technical Brief - Wave Uprush Analysis
Island Harbour Club, Gananoque, Ontario

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1 Introduction

The analyses presented herein relate to wave uprush (and associated hazard lands considerations) for potential redevelopment of property within the block bounded by Market St, Water St. Kate St. and St. Lawrence St. in the Town of Gananoque. The property is just inland of the shores of the St. Lawrence River and the Gananoque Municipal Marina.

Existing regulatory uprush elevations within the Cataraqui Region Conservation Authority (CRCA) area are derived from a regional planning level study, and therefore do not account for local site specific considerations with respect to wave uprush. Based on the regional study, the wave uprush elevation for the reach of shoreline which includes the property of interest is estimated 76.4 m (GSC). This elevation includes the 100 year static water level (75.9 m for Gananoque as per discussion with CRCA staff) and the wave uprush allowance.

The analyses presented herein have been completed in accordance with the Provincial Technical Guides for Flooding, Erosion and Dynamic Beaches in Support of Natural Hazards Policies 3.1 of the Provincial Policy Statement (MNR, 2001), herein referred to as the Provincial Technical Guides. Assumptions made as necessary to enable the computations are presented where relevant.

The analysis of wave uprush is performed in support of establishing required setbacks for the redevelopment of an existing site based on the flood hazard. The 100 year “Flooding Hazard Limit” is recommended as the superposition of a 10 – 20 year wave uprush condition on the 100 year water level (MNR, 2001). While CRCA policy is not specific in this regard, they typically consider a 25 year wave condition on a 100 year water level to be an appropriate combination of events; it is noted that there is typically a negligible difference between the 20 and 25 year wave conditions. This analysis considers a 100 year design wave condition approaching the Gananoque Municipal Marina as previously determined for the evaluation of the Gananoque Municipal Marina breakwater, with transformations based on local shoreline geometries and therefore, is expected to provide a conservative assessment of the wave uprush.

2 Study Area

The site is shown within the context of the local St. Lawrence River shoreline in Figure 2.1. The exposure of this site to wave runup is limited to a great degree by its setback from the water's edge and by the sheltering of the local shoreline area by Joel Stone Park headland.

A field investigation completed for this project included collection of limited depth soundings and topographic survey along the existing shorewall and adjacent boat launch at the west end of St. Lawrence Street. This information was used to supplement the available lidar data provided by CRCA. Elevations (referenced to International Great Lakes Datum, 1985) were determined based on relative heights from the local water levels as measured at Alexandria Bay at the time of the survey and to a geodetic bench mark on the old Customs House to the east of the proposed development property. Elevations surveyed with reference to this benchmark are Geodetic Survey of Canada (GSC) vertical datum. Where conversion between International Great Lakes Datum (IGLD) 1985 and GSC is required the conversion is based on a difference of 3 cm (IGLD(1985)-GSC = 0.03 m) given conversions of 0.04 m at Kingston and 0.02 m at Brockville (Provincial Technical Guides (Table A3.1.5), MNR, 2001).

A representative profile of the nearshore and upland area has been generated from the marina to an onshore elevation of approximately 77.0 m near the northwest corner of the proposed development property. The location of the profile is presented in Figure 2.2.

The shoreline configuration is somewhat complex with respect to the application of typical wave uprush formulae. Wave uprush formulae are generally developed for conditions of shore-normal wave approach, on relatively simple shoreline profiles. The existing shoreline is comprised of vertical shorewalls defining the boat launch area and the sloping concrete ramp surface.

As can be seen in Figure 2.1, the most direct exposure of the shoreline that would generate runup towards this site is from the southwest through the municipal marina. Waves considered in this analysis were estimated based on design wind speeds and transposed to the shoreline through numerical modelling techniques, as discussed further in Section 4.

Due to the generally smooth and moderately sloping shoreline (boat launch and street) profile, the wave uprush analysis has been completed based on wave runup on a beach and on a smooth sloping structure; this is discussed in Section 4. This approach requires the application of a typical sloping profile for the wave uprush development. The profile used in this analysis to assess simple slopes is based on the launch and upland areas as presented in Figure 2.3. The profile offshore of the boat launch is not considered relevant given the small wave periods and wave heights.

Typical photos of the local shoreline conditions are presented in Figures 2.4 and 2.5. It is important to note that the influence of local docks and breakwaters have not been accounted for in this analysis; these structures would serve to further reduce wave action at the shoreline.

Figure 2.1: Study Site

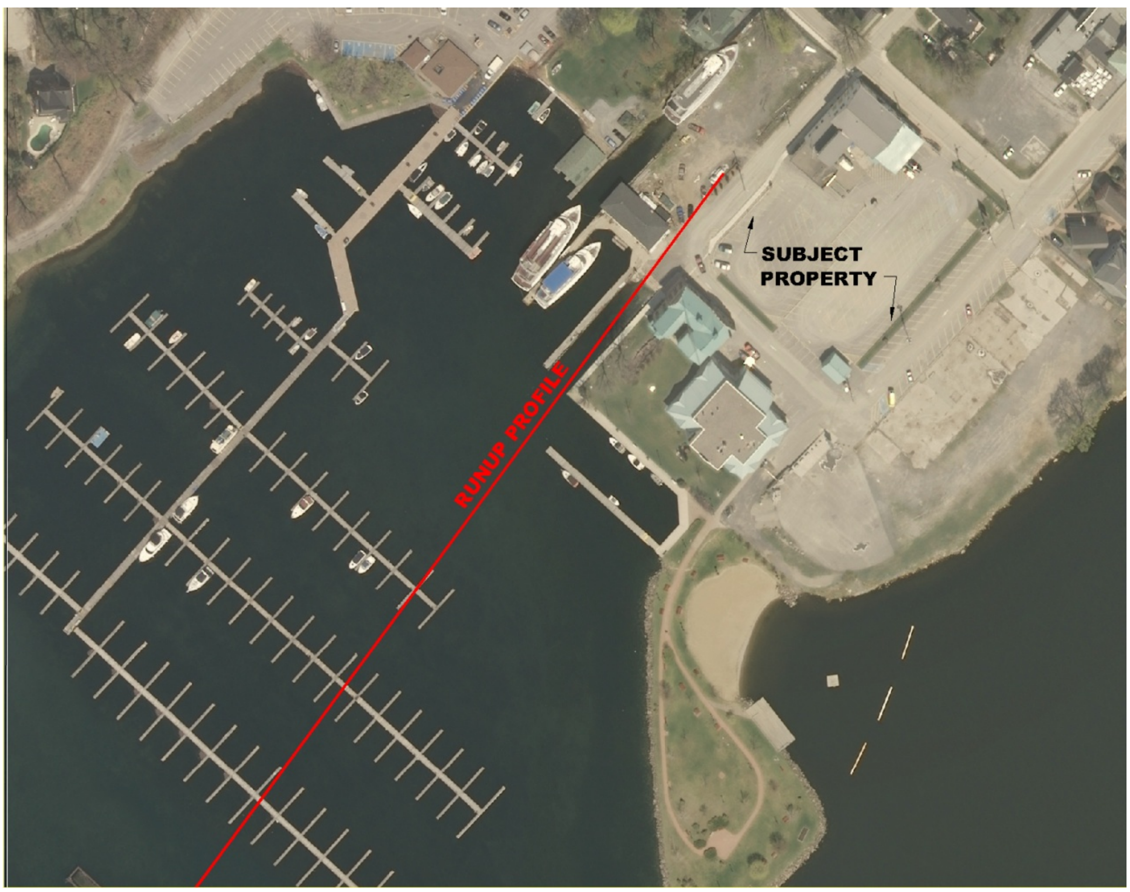


Figure 2.2 : Location of Profile for Uprush Analysis
Image Source CRCA (Groupe Alta (C) 2008)

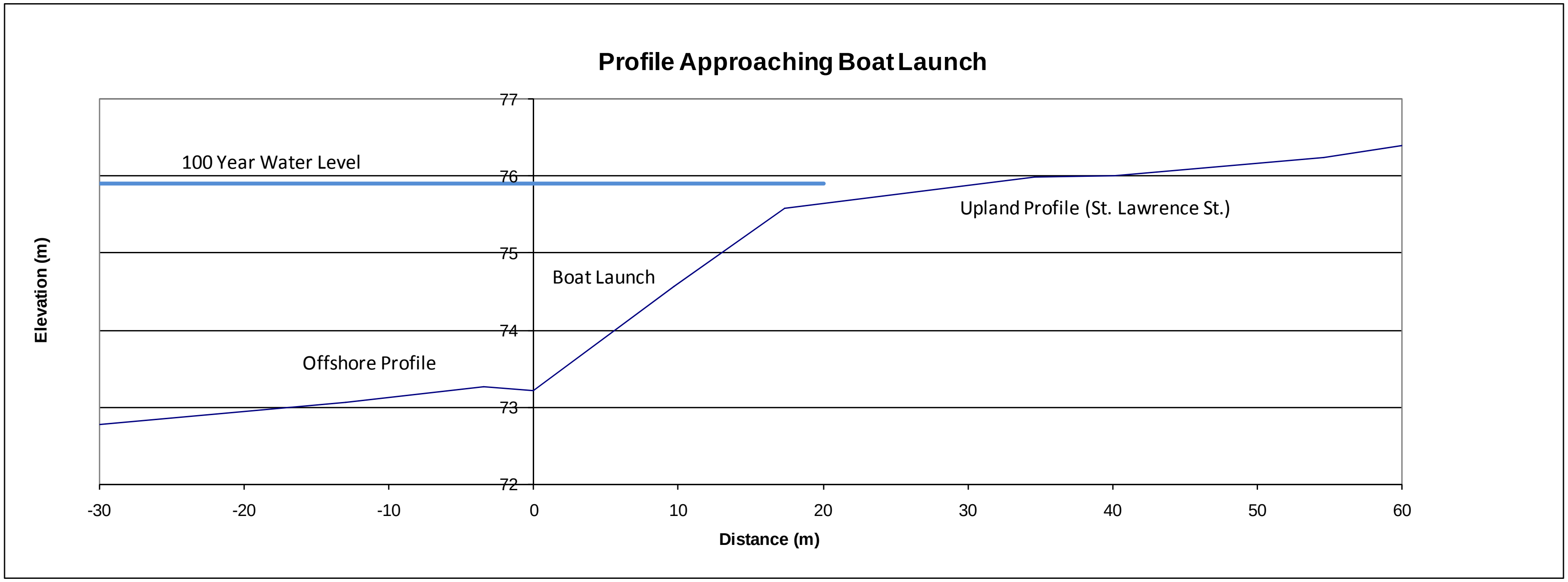


Figure 2.3 : Nearshore and Onshore Profile

Figure 2.4: Looking Southwest from Launch Slip Area

Figure 2.5 : Looking Northeast Along Launch Slip towards St. Lawrence St.

3 Analysis of Environmental Variables

Offshore wave conditions for this analysis have been estimated on the basis of regional wave generation modelling with design wind conditions used to generated wave conditions near the site.

Typical wind conditions for this area are not well documented locally. Wind recording stations are present at Kingston and at Grenadier Island. While a detailed wind study is beyond the scope of this analysis, typical data from Kingston and Grenadier Island have been compared for previous investigations in this region. The results show that Kingston windspeeds are generally higher than Grenadier Island and show a greater distribution of wind directions. Given the uncertainty in the local wind conditions and the potentially significant influence of regional land physiography, the Kingston data which is considered to provide a more conservative estimate of wave conditions was used in this analysis. Samples of wind conditions at Kingston and Grenadier Island are presented in Figure 3.1.

Statistical analysis of hourly wind conditions at Kingston have also been completed by peak over threshold analysis of the recorded winds to define discrete “events”, for statistical analysis. Assuming events defined by a minimum threshold of 20 km/hr and grouping the events into 45 degree sectors, the results of statistical analysis of wind conditions are presented (by direction) in Table 3.1.

Critical wave conditions developed on the basis of the regional wave generation modelling (from 100 year windspeeds) are presented in Table 3.2. Local influence of wave shoaling and refraction have not been explicitly assessed for these design wave conditions, but are considered in the subsequent analysis discussed further below. It is also worth noting that wave setup is not computed here as the waves to not break intensely in the nearshore region and therefore do not generate significant local wave setup.

Table 3.1: Kingston Airport Extreme Winds by Direction (km/hr)

T (yrs)	SE	S	SW
2	39.7	55.0	55.5
5	45.5	59.2	61.8
10	49.9	62.3	66.5
25	55.6	66.1	72.7
50	60.0	68.9	77.5
100	64.4	71.6	82.2

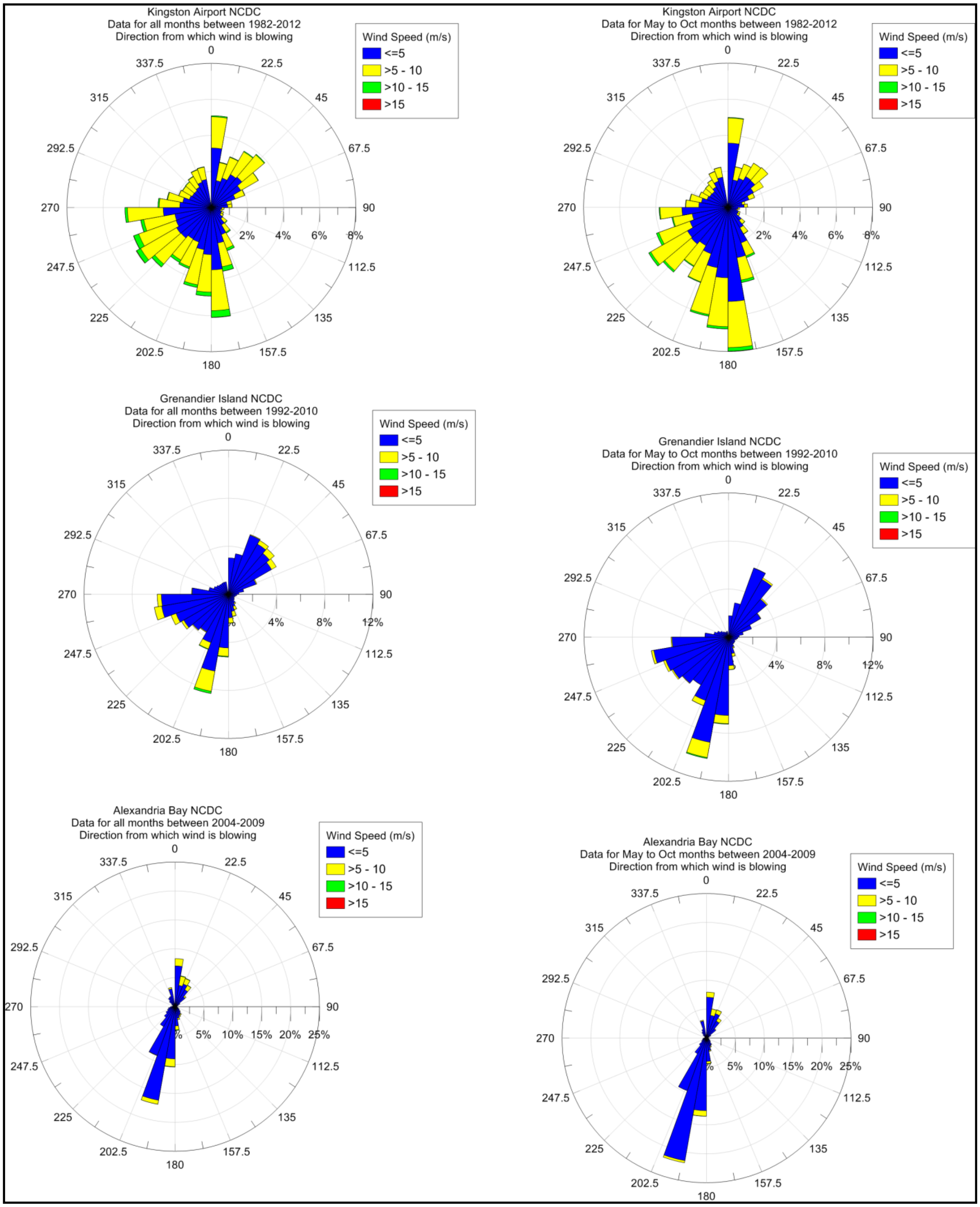


Figure 3.1: Comparison of Wind Data at Kingston Alexandria Bay and Grenadier Island

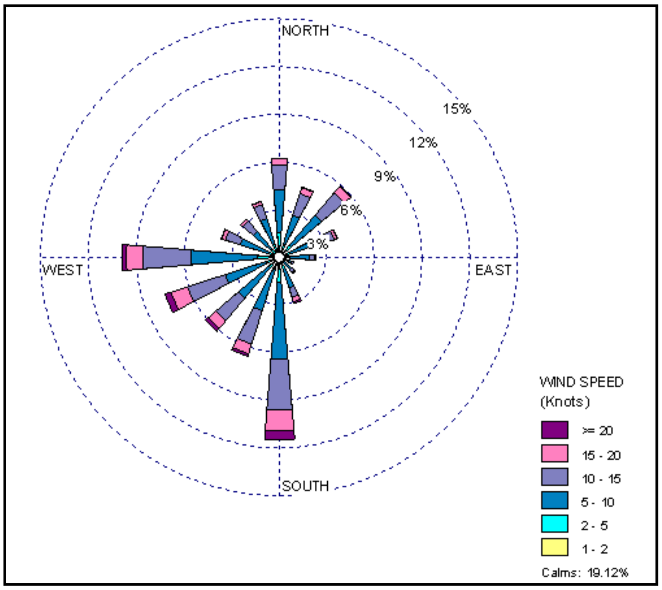


Figure 3.2 : Kingston Airport Wind Conditions

Table 3.2: Extreme Wave Heights (m) Just offshore of Marina

Parameter	ESE	SSE	SW
Hs (m)	0.70	0.78	0.76
Tp (s)	3.1	2.9	2.9

Due to the complex shoreline configuration approaching the boat launch area where wave runup may occur, a refinement of the wave conditions presented in Table 3.2 was undertaken using the CGWAVE model. As previously noted, the influence of the floating breakwater was not considered in this evaluation as it is not a permanent shoreline feature. The local wave height was estimated at the boat launch area for each of the critical design wave approach conditions. The results are presented in Table 3.3, and used in the wave uprush analysis; local graphical representation of wave heights for each of these directions is included in Appendix A. Typical analyses results within the CGWAVE domain for the SW wave direction which is critical for the uprush analysis are presented graphically in Figure 3.3.

Table 3.3: Wave Conditions at Boat Launch

Parameter	ESE	SSE	SW
Hs (m)	0.10	0.25	0.30
Tp (s)	3.1	2.9	2.9

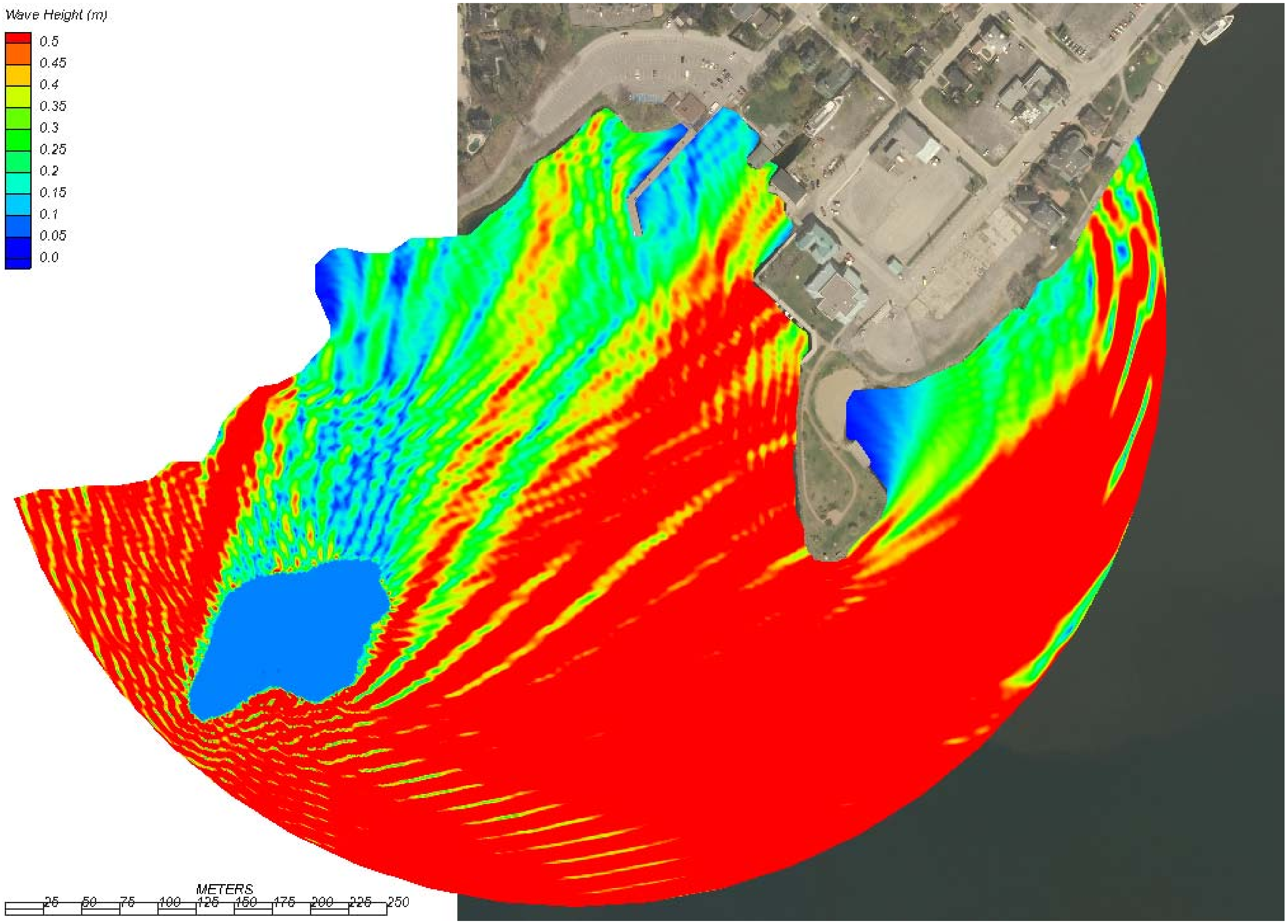


Figure 3.3 : Wave Analysis for Local Wave Conditions - SW Wave Attack
Image Source CRCA (Groupe Alta (C) 2008)

4 Wave Uprush Analysis

Potential wave uprush on structures and natural shorelines is an active area of research and due to the large number of potential influences relating to approach wave and shoreline characteristics, there is no single recommended and accepted method of analysis. Generally, analysis techniques are based on results of laboratory and field investigations, and have been presented in the form of empirical equations relating the potential runup height to a characteristic incident wave condition and a representative shoreline profile.

Typically, wave uprush computations are performed for natural beaches (plane slopes with normal wave incidence) with uprush estimated based on deepwater wave conditions or formalized shoreline structures of simple geometries with runup based on local wave conditions at the toe of structure. A wave runup equation based on deepwater conditions inherently accounts for the shoaling of the wave and the local wave setup as the wave gradually transforms over the uniform slope to its maximum runup extent. Runup estimates based on wave conditions at a structure would typically involve a local wave form that has developed on a sloping approach to the local depth at the toe of the structure.

The local shoreline is not entirely representative of a natural beach with unobstructed wave approach due to the relatively complex shoreline configuration. Furthermore, the relatively steep slope of the launch ramp leading to the relatively flat upland area is not typical of natural beach slopes. The launch ramp is however flatter than typical shoreline structures. Another factor that complicates the runup estimate is that the launch ramp slope does not extend to the full extent of the wave runup, and therefore there is a significant change in wave form at the top of the ramp where the depth is very shallow and the slope changes abruptly.

Given the complicating factors, the runup estimate has been established through comparison of results assuming runup on both a beach and a structure. The characteristics of each configuration for the purpose of the estimate is as follows:

- Beach : the estimate of runup on a beach-type slope assumes that the beach includes the ramp and the upland area, with a combined slope of approximately 5%, and
- Structure: the estimate of runup on a structure assumes that the launch ramp acts as a submerged shoal, which limits wave heights at the crest of the ramp, and the runup beyond the top of ramp is generated by a depth limited wave height acting on an upland slope of approximately 2%.

Runup has been computed assuming standard accepted methodologies (as employed in the USACE ACES approach) for wave runup due to irregular waves on smooth structures and wave runup on a beach.

The results of the analyses are presented in Table 4.1 which provides the 2% wave runup estimate (i.e. the wave runup that is exceeded by only 2% of wave heights in a typical Rayleigh distribution of storm waves). As indicated by the results in Table 4.1, the runup estimate for a beach-type profile results in the largest runup in general for the site. This runup is predicted to be 0.25 m above the static water level.

Table 4.1: 2% Wave Runup Estimates

Beach-Type Profile	
Wave Height (Hs)[1]	0.40
Wave Period (s)	2.9
Cotan Beach Slope[2]	20
Significant Wave Runup	0.19
2% Runup (m)[3]	**0.25**
Submerged Structure	
Submergence of Top of Ramp (m)	0.31
Depth Limited Wave Height (m)[4]	0.24
Wave period (s)	2.9
Cotan Upland Slope	50
Significant Wave Runup	0.06
2% Runup (m)[3]	**0.08**

Notes:
1. Design wave condition based on CGWAVE local approach wave
2. Runup is computed assuming average slope of launch ramp and upland (St. Lawrence St.)
3. Where not explicitly defined in the analysis, the 2% runup is estimated by multiplying Hs runup by 1.4 (Rayleigh Dist.)
4. Wave height for runup on upland area defined by depth limited condition at top of launch ramp

5 Conclusions

In summary, the nature of the existing shoreline is such that the existing boat launch area is protected to a significant degree by the local shoreline configuration. The marina breakwater would provide additional protection to this shoreline, but has not been considered in this analysis as it is a floating structure and its long-term configuration cannot be guaranteed.

The 100 year static water level at this site as provided by Cataraqui Region Conservation Authority as per their Regulatory Guidelines is 75.90 m. This water level submerges the top of the existing boat launch to a depth of approximately 0.3 m, extending onto St. Lawrence Street. Wave runup above this static water level has been estimated assuming:

- the boat launch and Market Street profile reflect a single average slope with runup processes similar to that of a beach profile with incident wave defined by that modelled within the launch area, and
- wave runup on only the St. Lawrence Street profile with the incident wave defined by the depth limited wave condition at the crest of the boat launch.

The runup defined by the average slope of the launch and St. Lawrence St. profile is the more conservative result, and is presented here as the recommended wave uprush elevation. Based on a 100 year static water level of 75.9 m (IGLD 1985), the Regulatory flood elevation including wave uprush would be 76.15 m (IGLD 1985) at this site. This elevation would be estimated to be 76.12 m GSC based on the difference between IGLD 1985 and GCS at Kingston and Brockville (MNR, 2001).

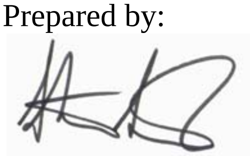
A number of assumptions have been made in the analysis, and generally these assumptions are made such that conservative factors are used in the estimate of the local runup:

- Kingston wind data has been used in the wave prediction calculations where this data appears to provide a more conservative condition than that which would be predicted using Grenadier Island data even though the riverine setting of Grenadier Island may be more consistent with the physical setting at Gananoque.
- a 100 year windspeed has been used to generate the wave conditions at the boundary to the numerical wave model,
- the influence of local breakwaters and docks has not been accounted for in the analysis,
- uprush has been computed assuming the largest wave height modelled within the boat launch area and assuming this wave approaches perpendicular to the local shoreline, and
- the wave uprush on-land is based on an extension of a plane slope, whereas the true geometry of the constrained boat launch width transitioning to the grading at the intersection of Kate and St. Lawrence St. would favour a spreading of the uprush volume, and associated reduction in wave momentum as the uprush spreads.

Based on this analysis, the Regulatory flood line including wave uprush is interpolated based on the LIDAR contours in Figure 5.1. As the LIDAR elevations are with respect to IGLD 1985, the 76.15 m elevation is plotted.

It should be noted that this analysis is specific to the site of interest, and should not be assumed relevant to adjacent structures due to variability in shoreline orientation, exposure and elevations.

Prepared by:



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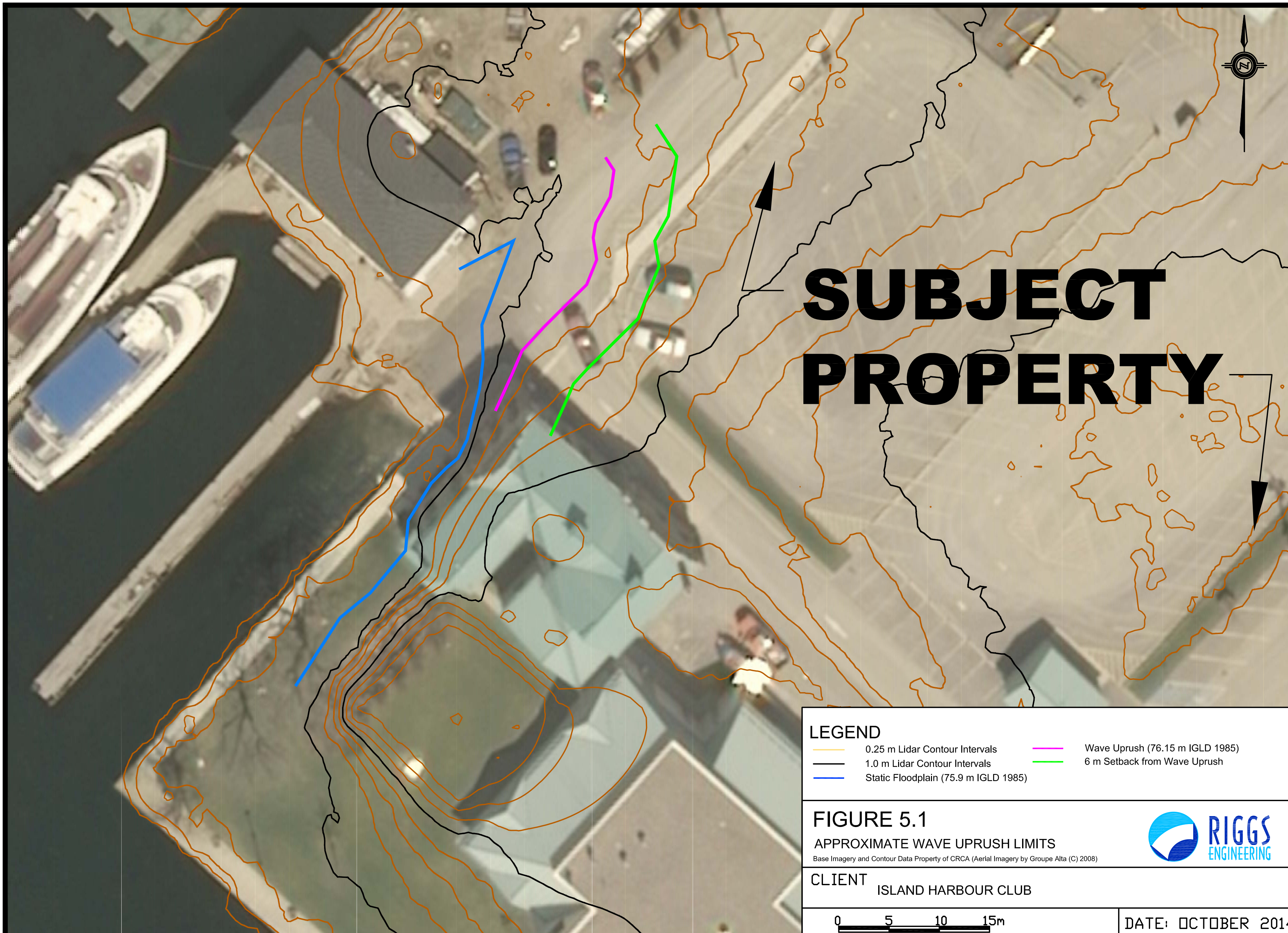


LEGEND

- 0.25 m Lidar Contour Intervals
- 1.0 m Lidar Contour Intervals
- Static Floodplain (75.9 m IGLD 1985)
- Wave Uprush (76.15 m IGLD 1985)
- 6 m Setback from Wave Uprush

FIGURE 5.1

APPROXIMATE WAVE UPRUSH LIMITS

Base Imagery and Contour Data Property of CRCA (Aerial Imagery by Groupe Alta (C) 2008)

RIGGS ENGINEERING

CLIENT

ISLAND HARBOUR CLUB

0 5 10 15m

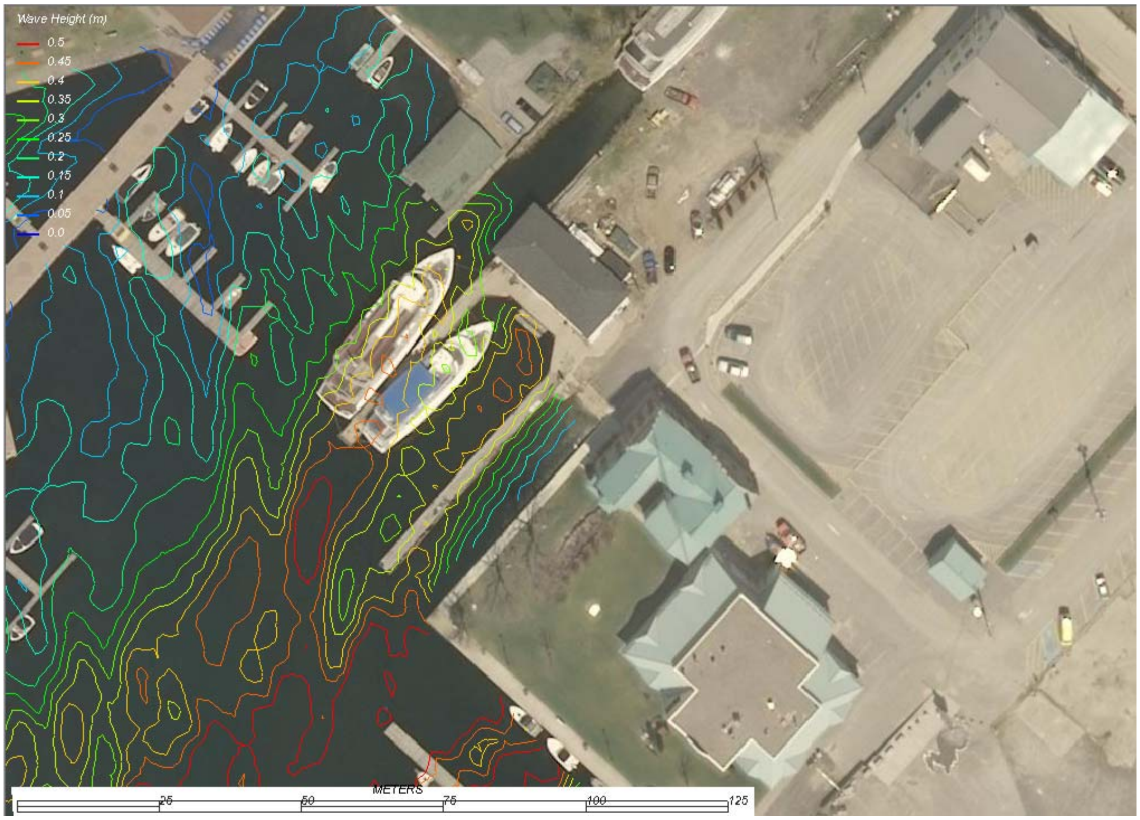
DATE: OCTOBER 2014

References:

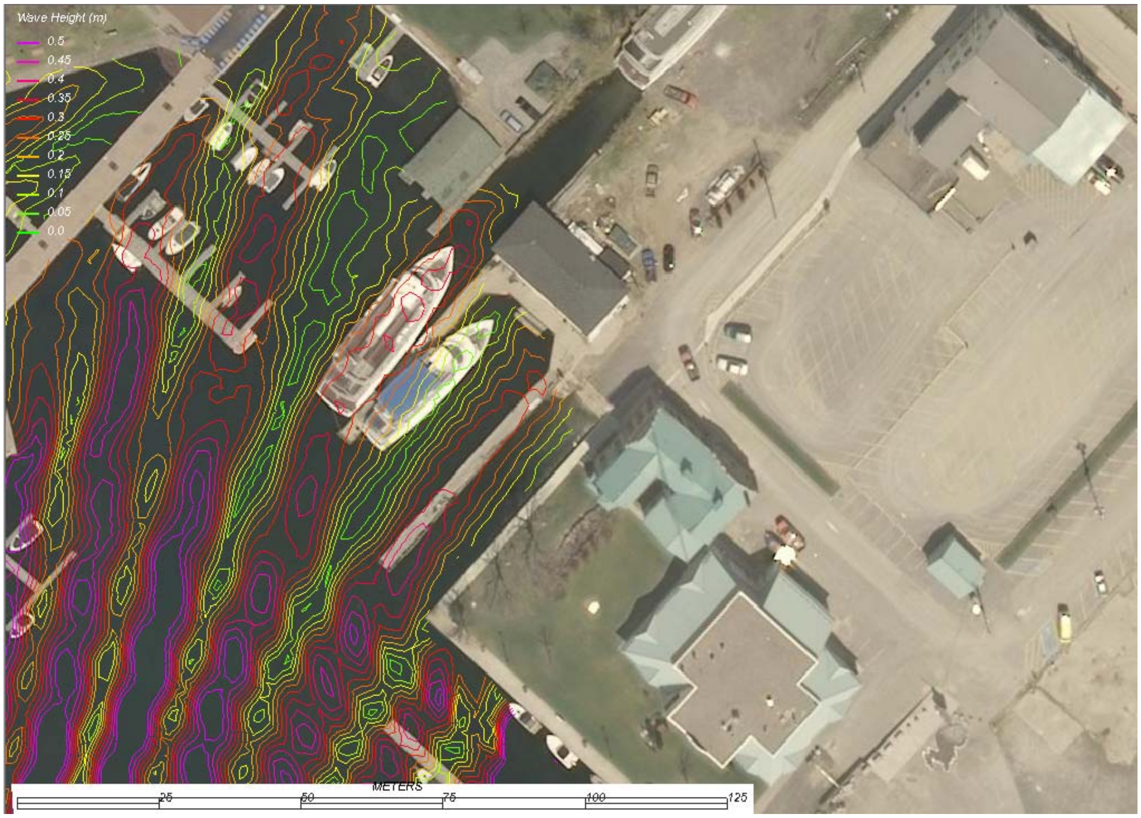
ACES, USACE, 1992. Automated Coastal Engineering System Technical Reference

MNR (Ontario Ministry of Natural Resources), 2001. *Great Lakes – St. Lawrence River System and Large Inland Lakes Technical Guides for Flooding, Erosion and Dynamic Beaches in Support of Natural Hazards Policies 3.1 of the Provincial Policy Statement*.

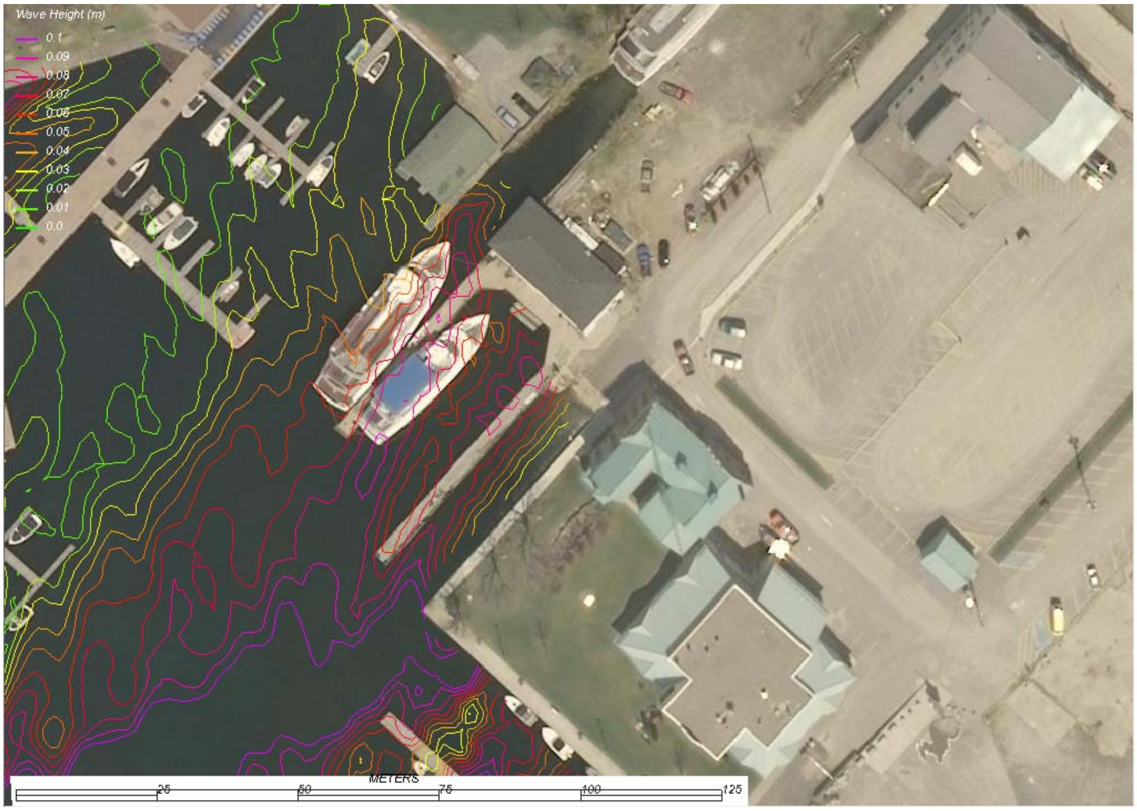
APPENDIX A
LOCAL WAVE HEIGHTS



A.1 - Local Wave Heights : SW Wave Attack
Image Source CRCA (Groupe Alta (C) 2008)



A.2 - Local Wave Heights : S Wave Attack
Image Source CRCA (Groupe Alta (C) 2008)



A.3 - Local Wave Heights : SE Wave Attack
Image Source CRCA (Groupe Alta (C) 2008)