Technical Brief - Wave Uprush Analysis
129 South Street, Gananoque

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1 Introduction

The analyses presented herein relate to wave uprush (and associated hazard lands considerations) for potential redevelopment of property at 129 South Street in Gananoque (presently Gordon Marine). The property is located on the northern shore of the St. Lawrence River just east of the Gananoque River.

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 for the reach of shoreline which includes the property of interest is estimated to be 0.5 m above the static floodplain elevation for the Gananoque area as defined by the CRCA at 75.9 m elevation (IGLD 1985).

The difference between IGLD 1985 datum and Geodetic Survey of Canada (GSC) datum is reported to be 0.04 m in Kingston and 0.02 m in Brockville, and therefore, in Gananoque, is assumed to be approximately 0.03 m. This would result in a (GSC) static floodplain elevation of 75.87 m in Gananoque. Based on Lidar topography for the property, a significant portion of the property is within the static floodplain limits. The local wave uprush acts upon this static water level and is affected by the nearshore slope conditions and to a great degree by the shoreline structure geometry. This technical brief outlines the evaluation of the local wave uprush for this property.

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 presented herein 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. 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. Southwesterly through northeasterly wave conditions have been evaluated in this wave runup analysis; these wave conditoins have been estimated on the basis of regional hindcast equations.

2 Study Area

The property is situated at 129 South Street in Gananoque Ontario; the site is shown within the context of the local St. Lawrence River shoreline in Figure 2.1 with local shoreline orientation and wave fetches shown in Figure 2.2. The shoreline at Gananoque is protected to a significant degree by the numerous islands and shoals within the region. Relatively short local fetches contribute to the development of small to moderate local wave heights under storm conditions.

A field investigation completed for this project included collection of limited depth soundings along the existing shorewall to supplement the available Lidar data provided by CRCA and local topographic survey data collected by HCCSL (May 7, 2013). Depths were converted to elevations based on the local water levels measured at Kingston and Brockville at the time of the measurements. Elevations referenced herein are defined with respect to 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 0.03 m at Brockville (Provincial Technical Guides (Table A3.1.5)).

Representative profiles of the nearshore and upland areas have been generated from approximately 5 m depth to an onshore elevation of approximately 77 m for runup computations. These profiles have been developed for a number of locations along the shoreline in order to provide a representation of the variability, with profiles generally perpendicular to shore. Points of interest (profile locations) and profile plots (1 through 5) are presented in Figures 2.3 and 2.4.

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 a vertical shorewall which is submerged at the 100 year water level. Furthermore, the direction of wave attack is variable, with the largest fetches approaching at an angle to the shorewall structures. As can be seen in Figures 2.1 and 2.2, the largest fetches are from the west-southwest and from due east. The west-southwest fetch approaches generally parallel to the existing shoreline, and therefore is not considered in uprush calculations for the south shore, but is considered for wave uprush on the west wall of the wharf (location 6).

The direction of wave attack which would be most direct with respect to the property location is from the southeast. These waves are generated over a moderately short fetch of approximately 4.6 km. Waves considered in this analysis were estimated based on design wind speeds and adjusted according to angle of incidence, as discussed further in Section 4.

Figure 2.1: Study Site

Figure 2.2: Wave Approach to Local Shoreline (Base Source Google Earth)

Figure 2.3 : Location of Profiles for Analysis

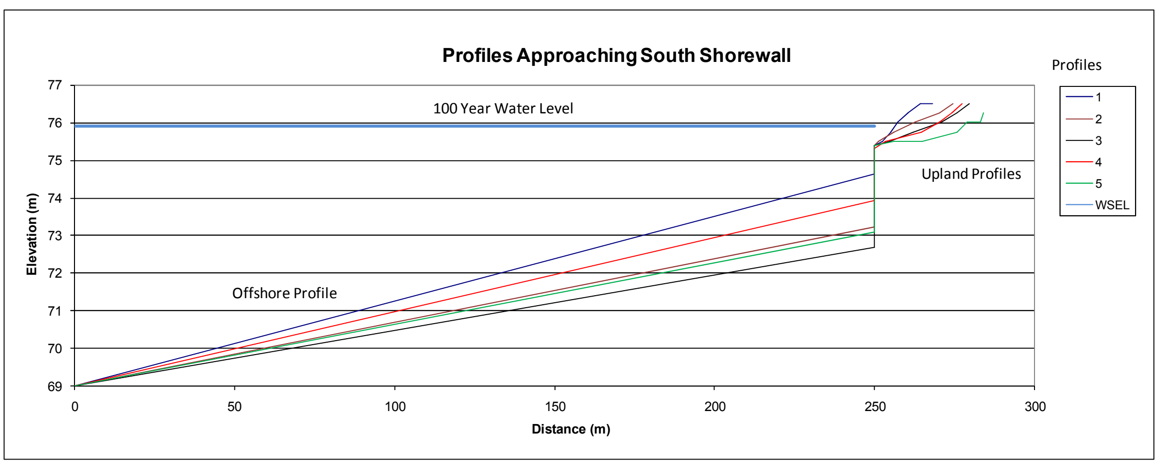

Figure 2.4 : Nearshore and Onshore Profiles

Due to the vertical nature of the shoreline, combined with the fact that it is submerged at the design (100 year) water level, the wave uprush analysis has been completed based on two different approaches as further discussed in Section 4. These approaches require the application of a typical slope for the wave uprush development. Simple slope profiles were estimated for the upland portion of profiles 1 - 5, as was a typical in-water slope representative of a "beach" condition for the shoreline in general (ignoring the vertical wall/upland portion of the profile) The approximate relevant slope characteristics and water depths at the top of wall (profile 1 - 5) are summarized in Table 2.1.

Typical photos of the local shoreline conditions are presented in Figures 2.5 and 2.6. It is important to note that the influence of local docks and breakwaters have not been accounted for in this analysis, and therefore, the wave conditions should generally be conservative in nature.

Table 2.1 : Characteristics of Typical Profiles

Profile	Cotan Slope	Depth at Top of Wall (m)[1]
1 (upland)	16	0.48
2 (upland)	22	0.50
3 (upland)	27	0.48
4 (upland)	23	0.58
5 (upland)	39	0.50
Typical Submerged	50	n/a

Notes: 1. Depths from 100 year water level to top of wall.

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Figure 2.5: Eastern portion of Existing Shoreline

Figure 2.5 : Western Portion of Existing Shoreline

3 Analysis of Environmental Variables

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 (for year 2000) were compared to determine the most conservative data for use. The results show that Kingston windspeeds are generally higher than Grenadier Island, and therefore were used in this analysis. Samples of wind persistence for SE and SW wind conditions for 2000 are presented in Figure 3.1.

Wind conditions at Kingston are represented by the wind rose in Figure 3.2, showing the relative occurrence by magnitude and direction for hourly wind conditions. 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.

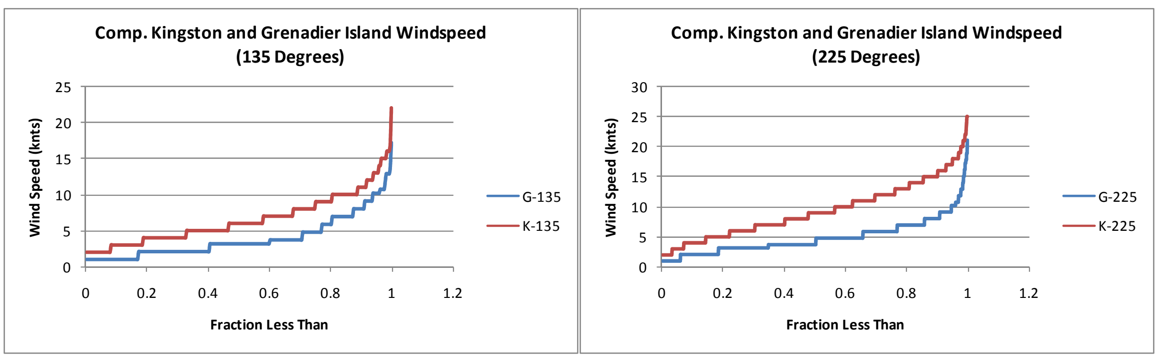

Figure 3.1: Comparison of Select Wind Data (Year 2000) at Kingston (K) and Grenadier Island (G)

Wave conditions at the site were determined by parametric hindcast equations, relating fetch and windspeed to wave height and period. It has been assumed that the waves are developed on the local fetches as shown in Figure 2.2, with a depth of 10 m. These assumptions are again relatively conservative, as the multitude of small islands will serve to limit well defined wave growth and the real depths are less than 10 m over the majority of the fetch lengths.

Design (25 year) wave conditions developed based on the hindcast equations (from 25 year windspeeds) are presented in Table 3.2. These results represent the wave conditions that would exist in 10 m of water at the site. Local influence of wave shoaling and refraction have not been explicitly assessed for these design wave conditions, due to the depth limited conditions at the existing shorewall as discussed further below. It is also worth noting that wave setup is not computed here as the waves to not break offshore of the structure given the relatively deep waters and the small periods. Wind setup potential is already accounted for in the design water level conditions as computed by Environment Canada.

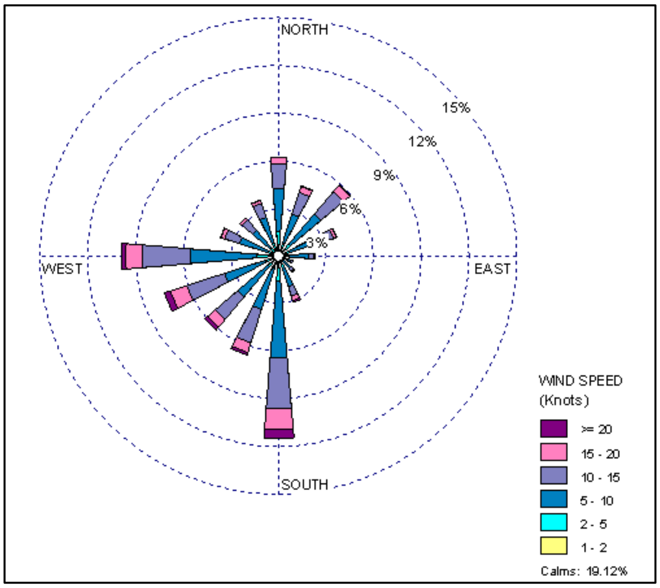

Figure 3.2 : Kingston Airport Wind Conditions

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

T (yrs)	N	NE	E	SE	S	SW	W	NW
2	45.3	40.2	35.3	39.7	55.0	55.5	56.0	40.8
5	51.2	43.6	39.2	45.5	59.2	61.8	62.4	45.0
10	56.0	46.2	42.2	49.9	62.3	66.5	67.2	48.2
25	62.6	49.6	46.2	55.6	66.1	72.7	73.5	52.4
50	67.8	52.2	49.2	60.0	68.9	77.5	78.3	55.6
100	73.2	54.8	52.1	64.4	71.6	82.2	83.1	58.8

Table 3.2: Extreme Wave Heights (m) by Direction

Parameter	**WSW**	**SW**	**S**	**SE**	**E**	**ENE**
Fetch (m)	4300	1450	900	4600	7900	7400
Hs (m)	0.8	0.46	0.36	0.58	0.43	0.48
Tp (s)	2.9	2.0	1.7	2.7	2.4	2.6

As previously noted, the WSW waves approach parallel to the south shore of the property, and are not considered for runup on this shoreline. They are used for assessing potential runup/overtopping along the westerly wall of the waterfront, and are reduced according to the potential for wave diffraction around the small headland immediately west of the site. All other directions are considered in the analysis of runup on the south shorewall.

As previously noted, the waves approaching the south shorewall will be influenced significantly by the vertical step at the wall and the significant reduction in depth, inducing breaking of the approaching wave. The local depth-limited wave height was estimated for each profile location as summarized in Table 3.3, assuming a typical depth limiting factor of 0.78 times the local depth.

Table 3.3: Depth Limited Wave Height on top of Shorewall

Site (Profile)	1	2	3	4	5
Top of Wall Depth (m)	0.48	0.50	0.48	0.58	0.50
Local Wave Height (m)	0.37	0.39	0.37	0.45	0.39

The wave periods are assumed to remain unchanged under the depth limited conditions, and as presented in Table 3.2.

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 estimate 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 simple sloping approach to the local depth at the toe of the structure.

The local shoreline site is not representative of a natural beach due to the large step imposed by the local shoreline structure, but is also not entirely representative of a formal shoreline structure due to the submergence of the wall. For the purpose of this analysis, the south shoreline has been considered in two fashions:

- represented by a mildly sloping shoreline structure (assuming upland profile slopes from Table 2.1) with a wave height at the toe of structure defined by the depth-limiting condition on the top of wall, and
- represented by a plane sloping beach with a slope consistent with the steepest riverbed approach slope (~ 1:50) approaching the wall, ignoring the abrupt step to the upland area.

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

The special case of wave runup on the west wall of the property has been approached using the method of Cox and Machemehl (1986) as presented in MNR (2001) which estimates an extent of wave "excursion" onto an upland area that is elevated above the design high water level. The physical wave uprush will typically run up the face of the structure, with possible overtopping where the wave crest exceeds the structure crest elevation. At this point, there is a sharp break in the slope supporting the runup, and the wave would be tripped at the break in slope, ultimately projecting onto the relatively flat upland slope. The typical scenario for a low sloping bluff is presented in Figure 4.1. This approach is applied to the vertical wall in the present case.

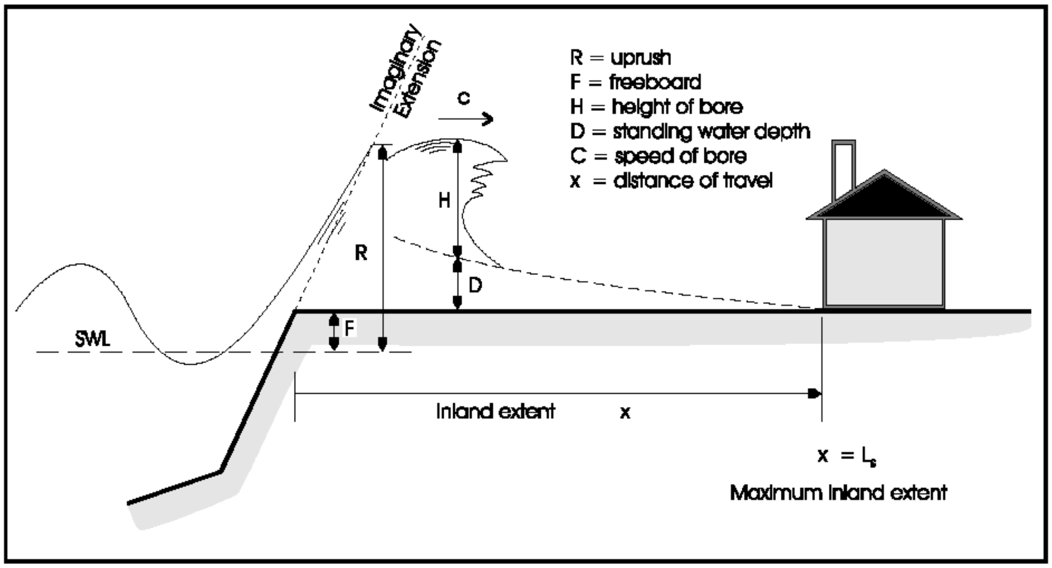

Figure 4.1 : Conceptual Representation of Runup over Low Beach Berm
Cox and Machemehl (1986) : Source MNR, 2001

The results of the analyses for the various profiles (points of interest) are presented in Table 4.1. As indicated by the highlighted results in Table 4.1, southeast waves result in the largest runup in general for each profile, but the runup estimate based on a plane beach wave uprush approach provides the most conservative estimate of uprush overall and will be assumed for this property.

The west wall of the site is exposed to southwesterly waves. The most direct wave attack is that from the west-southwest fetch, with increasing exposure as one moves south along the wall. Wave runup at the south end of the west wall will be within the runup region defined for the south wall, and therefore has not been computed. Moving north along this wall beyond the limit of the runup from the south wall, the exposure is reduced due to protection from the small headland immediately west of the site. Overtopping of the wall in this location was estimated for west-southwest through southery wave exposure. The method of Cox and Machemehl (1986) was used to estimate the extent of wave action due to overtopping at Site #6, with the worst case scenario defined for southerly wave attack. Due to the reflective nature of the wall, the runup was assumed to be twice the incident wave height.

Table 4.1: Wave Runup Estimates - South Shorewall

Incident Wave Direction	SW	S	SE	E	ENE
Site 1 (Smooth Sloping Structure)					
Wave Approach Angle (deg.) [1]	45	30	10	60	80
Wave Period (s)	2.1	1.6	2.7	2.4	2.5
Depth Limited Wave Height (m)	0.37	0.37	0.37	0.37	0.37
Cotan Slope[3]	16	16	16	16	16
Runup (m)	**0.09**	**0.07**	**0.13**	**0.10**	**0.10**
Site 2 (Smooth Sloping Structure)					
Wave Approach Angle (deg.) [1]	45	30	10	60	80
Wave Period (s)	2.1	1.6	2.7	2.4	2.5
Depth Limited Wave Height (m)	0.39	0.39	0.39	0.39	0.39
Cotan Slope[3]	22	22	22	22	22
Runup (m)	**0.06**	**0.05**	**0.10**	**0.08**	**0.07**
Site 3 (Smooth Sloping Structure)					
Wave Approach Angle (deg.) [1]	45	30	10	60	80
Wave Period (s)	2.1	1.6	2.7	2.4	2.5
Depth Limited Wave Height (m)	0.37	0.37	0.37	0.37	0.37
Cotan Slope[3]	27	27	27	27	27
Runup (m)	**0.05**	**0.04**	**0.08**	**0.06**	**0.06**
Site 4 (Smooth Sloping Structure)					
Wave Approach Angle (deg.) [1]	45	30	10	60	80
Wave Period (s)	2.1	1.6	2.7	2.4	2.5
Depth Limited Wave Height (m)	0.45	0.45	0.45	0.45	0.45
Cotan Slope	23	23	23	23	23
Runup (m)	**0.07**	**0.05**	**0.10**	**0.08**	**0.08**
Site 5 (Smooth Sloping Structure)					
Wave Approach Angle (deg.) [1]	45	30	10	60	80
Wave Period (s)	2.1	1.6	2.7	2.4	2.5
Depth Limited Wave Height (m)	0.39	0.39	0.39	0.39	0.39
Cotan Slope[3]	39	39	39	39	39
Runup (m)	**0.04**	**0.03**	**0.06**	**0.04**	**0.05**
All Sites (Plane Beach Runup)					
Wave Approach Angle (deg.) [1]	45	30	10	60	80
Wave Period (s)	2.1	1.6	2.7	2.4	2.5
Deepwater Wave Height (m)	0.46	0.30	0.58	0.43	0.50
Cotan Slope[3]	50	50	50	50	50
Runup (m)	**0.12**	**0.08**	**0.19**	**0.14**	**0.14**

Notes:
1. Approach angle defined based on fetch delineation (Figure 2.2)
2. Runup elevation has been adjusted for approach angle
3. Slopes as per Table 2.4 and discussion above.

The results of the analysis are presented in Table 4.2, which predicts a wave excursion of 2.2 m onto the upland area.

Table 4.2 : Projection of Estimated Runup (Wave Excursion) on Upland Area at Point #6

Condition	2% Wave Height
Artificial Runup Height (m)[1]	0.92
Freeboard[2]	0.36
Upland Slope (m/m)	0.10±
Wave Excursion (m)	2.2

Notes: 1. Runup assumed to be 2 X local (diffracted) wave height at Point #6
2. Freeboard to top of wall from 100 year water level.

5 Conclusions

In summary, the nature of the existing shoreline is such that the southerly shorewall is submerged under the 100 year water level to a depth of approximately 0.5 m to 0.6 m. This 100 year water level extends onto the southern-most portion of the site and permits some propagation of waves onto the upland area. The approaching waves will however experience a significant and abrupt change in depth at the shorewall which will reflect a portion of the wave's energy at depth and cause wave breaking at the surface as the wave passes over the wall. The wave that remains after breaking will continue to propagate on the flooded area, and runup above the CRCA 100 year water level of 75.9 m (IGLD 1985).

A number of assumptions have been made in the analysis, and generally, these assumptions are expected to result in a conservative 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 is more consistent with the physical setting at Gananoque.
- wind fetches have been estimated in a relatively conservative manner, ignoring the local influence of small islands, and assuming deep open water conditions for wave growth,
- the influence of local breakwaters and docks has not been accounted for in the analysis, and
- uprush has been computed assuming runup on a smooth structure-type profile after breaking, and also assuming runup on a beach-type profile based on deepwater wave conditions with the most conservative result assumed in the analysis.

Based on this analysis, the expected extents of the wave uprush are presented in Figure 5.1. These lines are derived from the projected 2% wave runup to elevation 76.06 m calculated based on the 0.19 m runup (beach-type shoreline) above the 100 year water level of 75.87 m GSC (75.9 m IGLD 1985) along the southern portion of the site, and the predicted 2.2 m overtopping excursion near the northern portion of the west wall of the property (Point #6). The estimated uprush limit line is estimated through interpolation of the lidar data for the site.

It should be noted that this analysis is specific to potential future development at 129 South Street in Gananoque, and should not be assumed relevant to adjacent structures due to variability in shoreline orientation, exposure and elevation.

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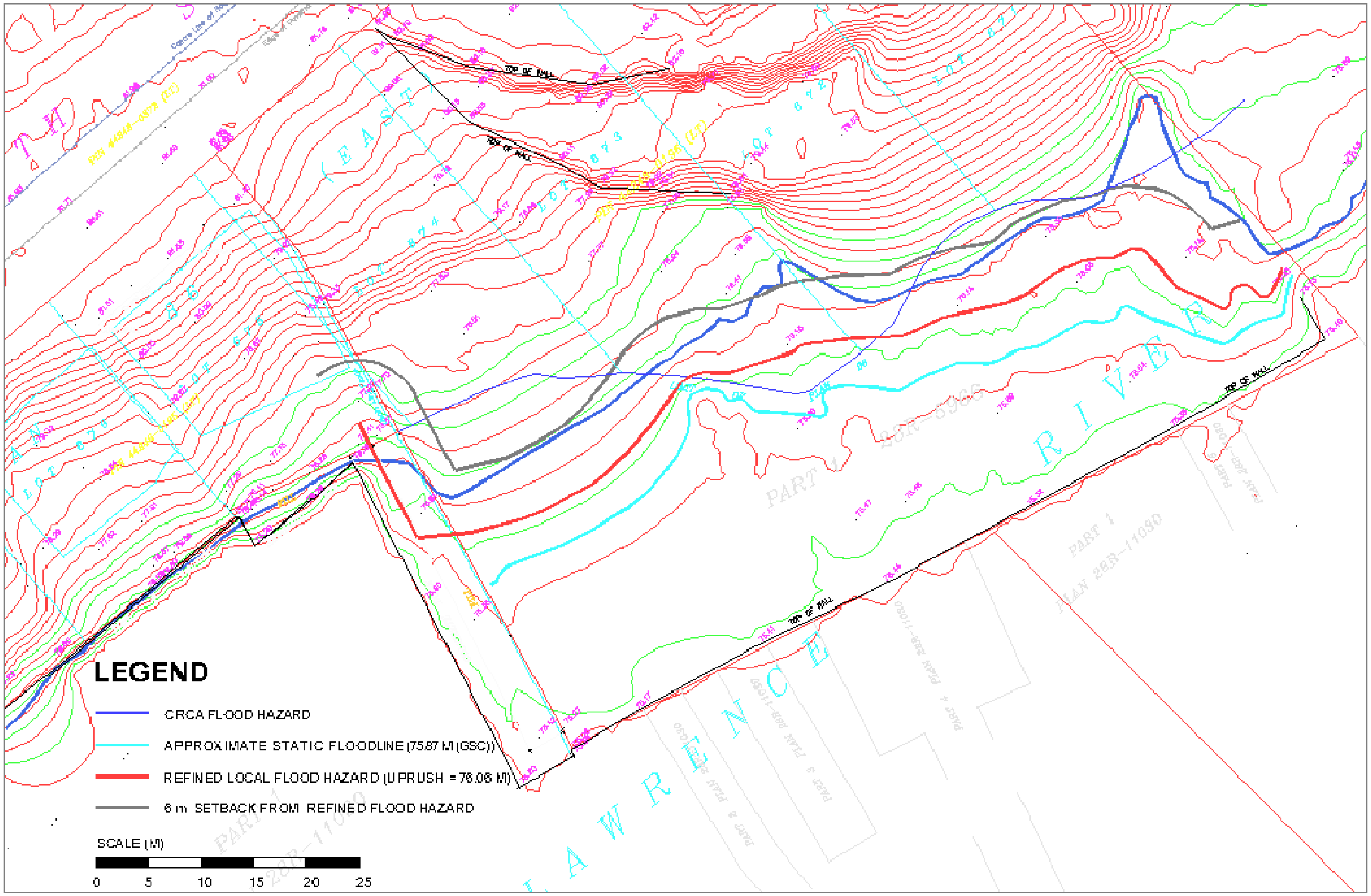

Figure 5.1: Calculated Extent of Regulatory Wave Uprush
(Base Plan source includes CRCA Lidar and HCCSL Topographic Survey, May 13, 2013)

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*.