Appendix D

Reach 30 Detailed Analysis

W.F. Baird & Associates Ltd.

Office | 2924 Marketplace Drive, Suite 200, Madison, WI 53719, USA
Phone | +1 608 273 0592 Email | madison@baird.com

Ms. Christina Bright
Project Manager, Flood Risk Management
Toronto and Region Conservation Authority
101 Exchange Ave, Vaughan,
ON, L4K 5R6

via email to Christina.bright@trca.ca

Status: Correspondence
June 9, 2022

Dear Christina,

Reference # 13579.101.L05.Rev0

RE: HAZARD MAPPING REACH 30 DETAILED ASSESSMENT

Hazard mapping was completed in April 2022 for the full length of the TRCA waterfront using a reach-scale assessment of the shoreline conditions. Reach 30 in this assessment, extending along the 3.7 km length of shoreline from Meadowcliffe Drive to Guild Park (see image below) was identified as having a significant erosion allowance. The erosion allowance was defined as 100 years multiplied by the average annual recession rate (AARR) determined from a comparison of historical aerial photographs. An important factor in defining the erosion allowance is the manner in which existing structures are credited, or not credited, with providing erosion reduction benefits. Based on discussions with TRCA staff, major public structures that were built prior to 2005 were not granted any reduction in the 100 year recession estimate; this condition applied to most of Reach 30. Only in the southern 650 m of Reach 30, which was a major public structure constructed in 2013, was the erosion allowance reduced. The expected 50 years of life of this structure was reduced to 41 years from present (to account for 2013 construction date), and the AARR was applied over the remaining 59 years of the 100 year planning date.

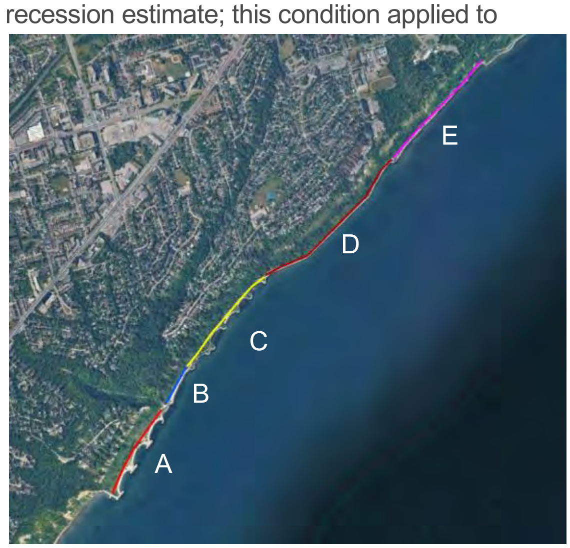

Figure 1: Study Area with Reaches A to E

This letter summarizes a more detailed investigation undertaken to assess the condition of shore protection works in Reach 30 and provides a recommended approach to assigning residual life to the existing structures.

The primary tasks that were completed for this study are:

- Review of technical reports and design drawings (where available) for the region of interest

- Assessment of the long term average annual recession rate (AARR). This is an extension, in greater detail, of what was already completed for the reach.
- Assessment of water depths and design wave height at the toe of the structures. From this, the stable stone size for each structure was determined.
- Review of inspection records and a site visit to document the existing conditions. Aerial drone imagery of the site was also collected.
- Review of typical profiles and damage progression of breakwaters and revetments.
- Estimation of likely progression of damage at Reach 30.

Average Annual Recession Rate (AARR) Assessment

A review of past studies in the area provided additional information on shoreline recession rates:

- Terraprobe (2001) provided calculations of slope crest erosion rates between 1991 and 2001 for the Meadowcliffe region and determined that localized crest loss ranged from 6 to 12 metres at the west part of the study area, to about 2 to 3.5 metres at the east part of the study area. This period is less than is typically used for assessing shoreline recession, but suggests an Average Annual Recession Rate (AARR) of 0.6 to 1.2 m/y in the western area, and 0.2 to 0.35 in the eastern area.
- Reinders (1989) reported the following AARR values for the Sylvan Ave region:
    - 0.25 to 0.50 m/y for the period of 1922 to 1952
    - 0.50 to 0.75 m/y for the period of 1950 to 1980
    - An approximate average value of 0.5 m/y
- Reinders et al (1994) also studied downcutting of the nearshore, which is closely tied to the long term erosion of the shoreline. Downcutting rates were estimated at four profiles and averaging their results they estimated that the current 2 metre contour would be lowered by 0.9 metres in 40 years, 1.4 metres in 75 years, and 1.8 metres in 110 years.

The rate of downcutting can be approximated by dividing the AARR by the nearshore slope. For example, an AARR of 1 m/year with a 100:1 slope suggests a nearshore downcutting rate of 1 cm per year. The Reinders downcutting values were compared to the nearshore slopes and the long term recession rates and found to be representative.

The past reports demonstrate a wide range of AARR values. It is notable that none of the time periods that were assessed exceeded 30 years. The variability may be related to the dates that were chosen, spatial variations in the processes, and also temporal variation.

With significant shore protection in the area of interest, the analysis that was completed for the broader study had challenges with obtained adequate data using only unprotected areas. Comparisons were made based on shoreline positions from 1942 and 1972, although these comparisons were limited in their spatial extent to locations between installed shore protection totaling only 170 m of shoreline. There were also challenges related to the visibility of the shoreline and bluff from these photographs. From this assessment an initial AARR value of 1.57 m/yr (1.26 + 0.31 representing the mean plus one standard deviation) was derived for Reach 30.

In subsequent analysis, additional photographs were purchased from the National Air Photo Library from 1931 and 1946, and additional comparisons were made. Reach 30 was broken into four subreaches for the more detailed aerial photo analysis, using different photos/dates for different areas. An overview of the results is provided in Figure 2.

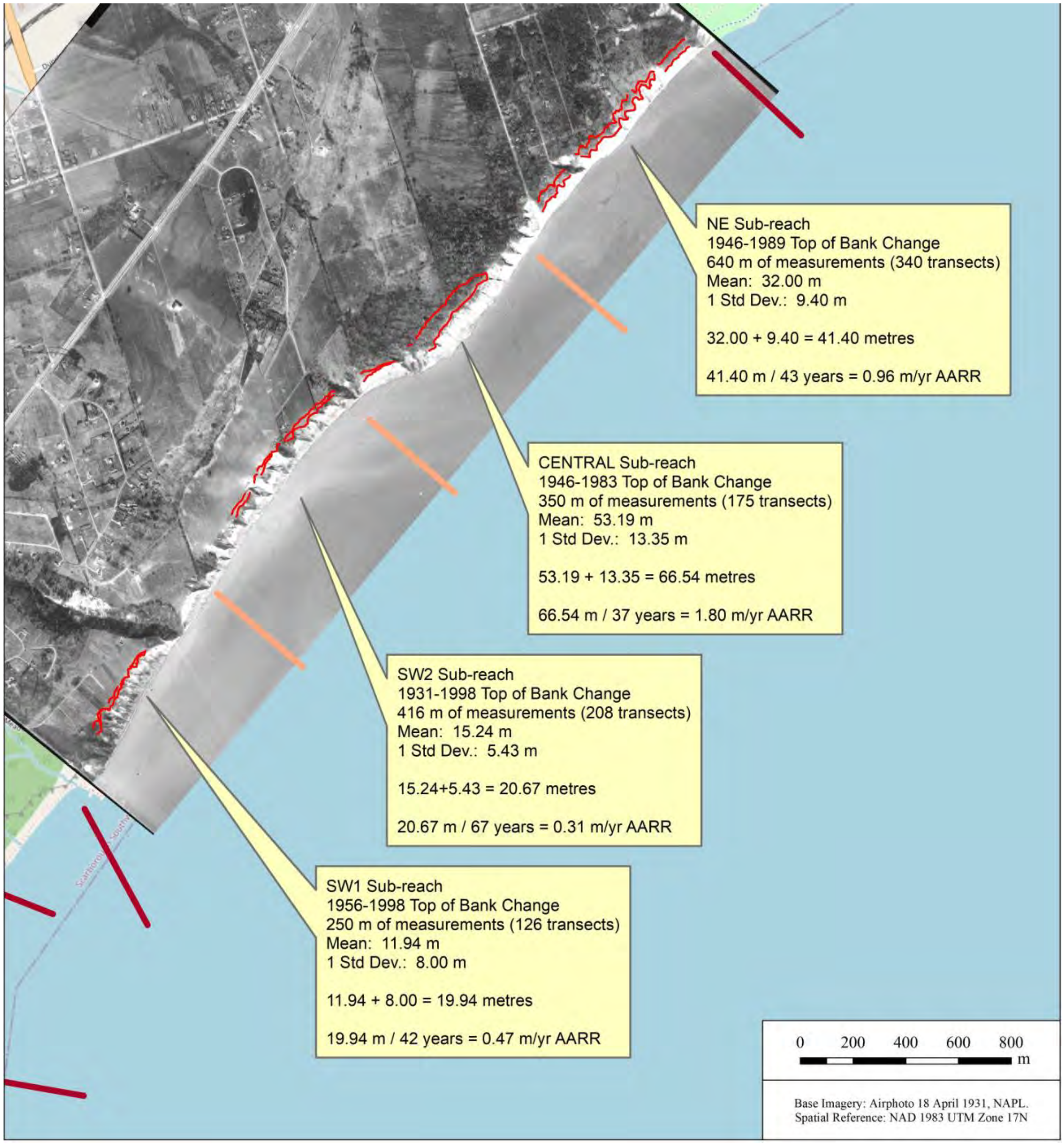

Figure 2: Revised Aerial Photo Analysis for Reach 30

The assessment also made use of some photographs that were taken just after construction of shore protection, based on the assumption that erosion would have mostly stopped on the date when construction started. From this assessment, we determined that the two southwesterly reaches had similar AARR values and could be combined with an AARR of 0.47 m (the higher of the two sub-reaches). The central and NE sub-reaches would use revised values of 1.80 m/yr and 0.96 m/yr. This assessment showed that AARR values vary significantly within the reach.

The interesting pattern from this assessment is that the greatest shoreline erosion appears to be occurring in the region of shoreline that is more protruding into the lake. This suggests that erosion may have been slower in this region in the past, and is more recently accelerating to seek a more linear shoreline alignment. This could relate to wave focusing or changes in nearshore bathymetry.

Structures Condition Assessment

The present condition of the structures in Reach 30 was documented through a site visit by Fiona Duckett on May 12, 2022 and drone aerial photographs on May 4, 2022. The drone photographs were taken during ideal conditions with clear water and calm winds, allowing for a clear view of the structures' above and below-water characteristics. An example of the drone imagery is shown in Figure 3.

For the purpose of this study, the shoreline was divided into five distinct sub-reaches with similar shore protection and present conditions. From southwest to northeast these sub-reaches are described below.

Sub-reach A - Meadowcliffe Drive

This sub-reach is 630 m in length and shore protection was built in 2013; it is referred to as WF30.xx in TRCA's structures review. The shoreline is protected with four shore-connected, (mostly) shore-parallel structures with three cobble beaches between them. The structures appear to be in good condition and were built with a single layer of large blocky armourstone of 4 to 6 tonnes over a second layer of 0.5 to 1 tonne stones. These structures are expected to remain stable for a longer period of time than the beach cells. Based on the construction drawings, the beaches consist of cobbles with a D_{50} of 100 mm over concrete rubble (of unknown size). The cobble beaches are offset from the toe of the bluff by about 40 m.

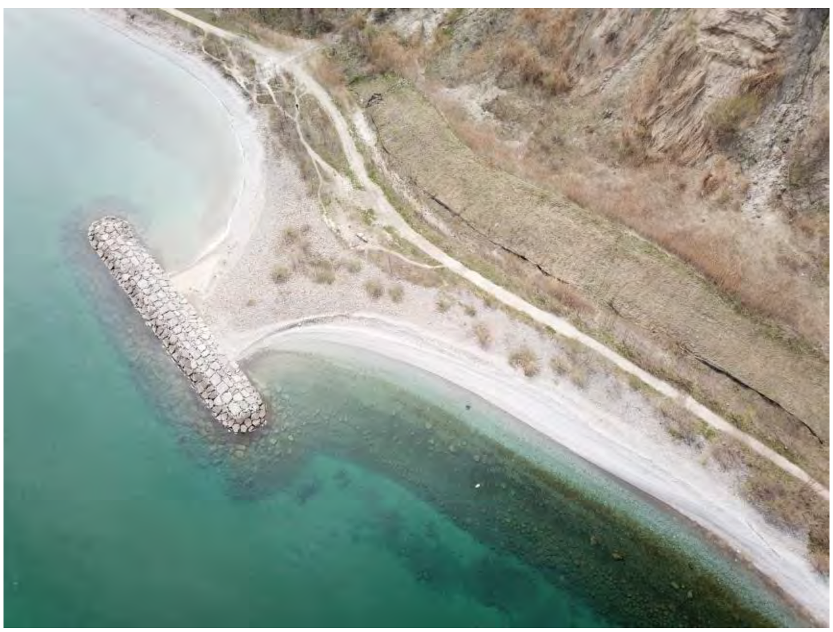

Figure 3: Headland at Sub-reach A, Meadowcliffe Drive

Inspection of these headland structures shows stone sizes that align with the design documents. Toe depths in front of the structures will limit the wave height that will impact the structures. Wave heights and required stone sizes after downcutting are in alignment with the observed stone sizes and design. Therefore, we are not concerned about the size of the stone relative to the anticipated design wave conditions.

The bigger concerns with these structures are the toe scour protection and general degradation of the stones over time. The toe of the structure has two armour stones excavated into the lakebed. These would likely move (slump or roll lakeward) if, over the next 100 years, 2 m of downcutting was experienced. On the armour stone face, the method of packing smaller chinking between larger (loosely packed) armour stone is not a common practice in typical coastal works and gradual loss of this chinking is expected. Overall, we expect some slumping of these structures over the next 100 years; however, they are expected to remain mostly in place albeit at a lower elevation. This could impact the extent to which they protect the tombolo formation and the stability of the cobble beach.

The longevity of the cobble beach is dependent on the integrity of the stone, the anchoring of the position with the headlands, and the overall stable shape of the bays. The shape of each of the bays is “flatter” than a typical beach-bay planform and is defined mostly from its construction alignment. Some realignment of these bays is expected. The constructed cross section is shown below.

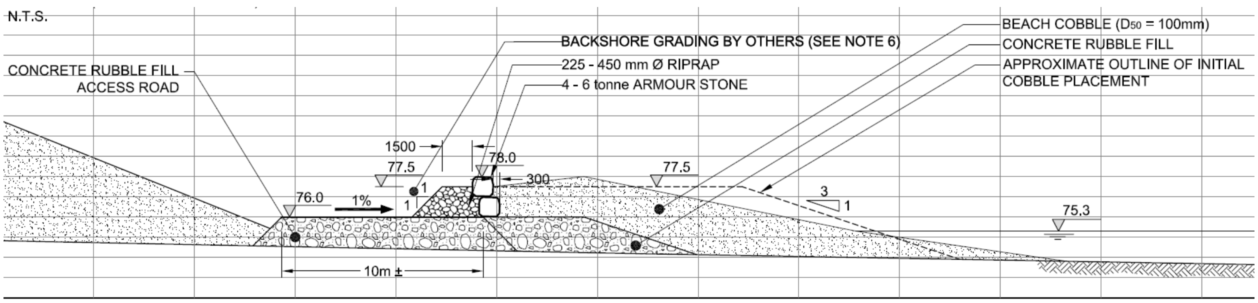

Figure 4: Cross Section of Cobble Beach in Sub-reach A

Bricks and concrete were visible in some areas of the beach, suggesting that the profile has already been reshaped enough to reach into the lower layer below the cobbles. Any significant downcutting of the shoreline would likely result in loss of cobbles into the nearshore scoured area and a reduction in the height and width of the crest.

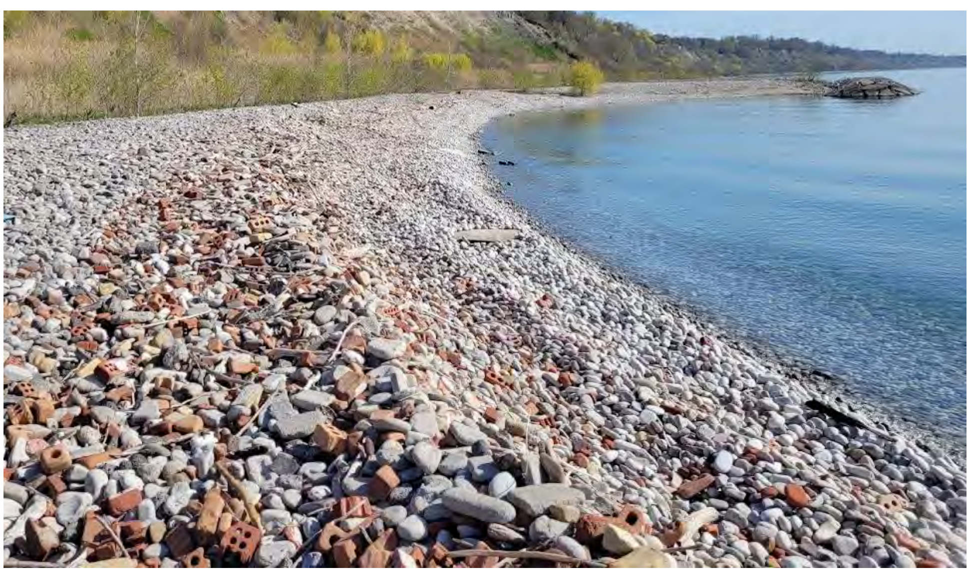

Figure 5: Cobble / Concrete / Brick Beach in Sub-reach A

The conclusion for Sub-reach A is that some reshaping of the shore protection is expected over the next 100 years, especially if significant downcutting occurs. However, there is a significant volume of material along the shoreline and at the very least, these cobbles and armour would act as a reef if it was significantly damaged. This would reduce the wave energy reaching inland of the beach position towards the bluff toe.

Sub-reach B - North of Bellamy Creek

This sub-reach is 270 m in length and consists of a poorly constructed rubble shoreline built from construction rubble. The TRCA inspection records show construction in 1999, although this was likely a narrower structure (lakefill) than what is present today. Significant filling and widening of the structure took place between 2009 and 2013. We are not aware of any drawings or reports for this section. The elevation at the toe is approximately 73 m, with some evidence of local scour. The slope of this structure is about 6:1 (H:V) and it has probably been shaped by waves to achieve this slope. Inspection reports from TRCA suggest that the structure needs major repairs and in 2019 TRCA noted that erosion had reached the path. An overhead view (UAV imagery), of one section of this shoreline is shown in Figure 6.

During storm events when the structure is exposed to significant wave energy, the cobbles and concrete pieces would experience significant movement and would be expected to degrade with time. In comparison, a well built revetment has limited mobility and there are less concerns with the abrasion and breaking down of material. This suggests that the longevity of the material on this reach would be significantly reduced compared to armour on a robust revetment.

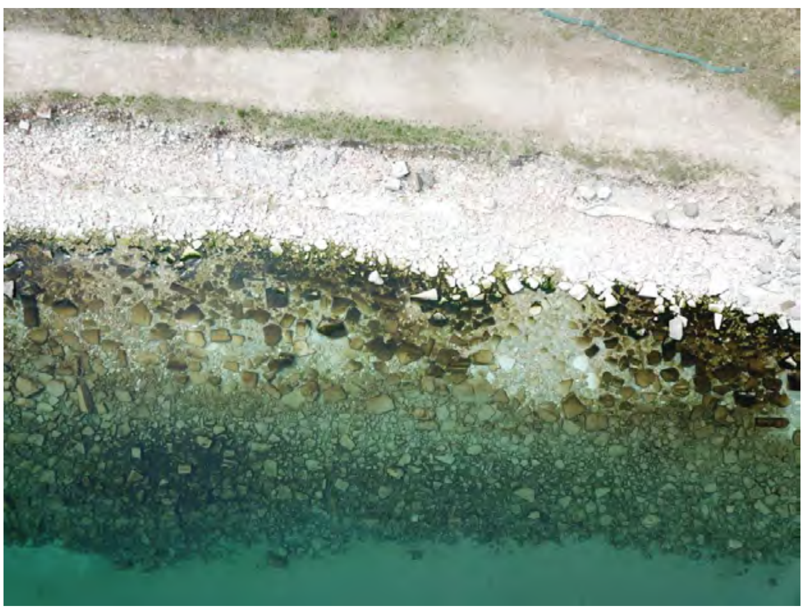

Figure 6: Cobble / Rubble Shoreline in Sub-reach B

This is a structure with a short design life. There is evidence of repairs in the past and the need to repeat them after the high water periods of 2017 and 2019.

Sub-reach C - Sylvan Ave

Shore protection along this sub-reach consists of headlands and pocket beaches over a shoreline length of about 670 m; the shoreline length around the structures exceeds that. These structures are described as WF24.xx in TRCA documentation and were built in 2002. The headlands are armourstone, while the pocket beaches are typically concrete rubble with a cobble surface layer that reshapes during larger wave events. At the back of the beach is a line of two (high) stacked armour stone units.

The headland sections are built with armour stone that appears to be consistent with the required stone size at present and during expected downcutting over the next 100 years. Stones in the range of 4 to 6 tonnes were measured in this area. The greater concern with the headlands is the chinking with smaller stone between the armour, which appears to be gradually becoming displaced with time and allowing some settlement of the structure. It is also not clear that the toe has been designed with adequate protection for significant lakebed downcutting. Despite these issues, we believe that the headlands will remain largely in place and continue to act as headlands over the next 100 years.

The pocket beaches are a greater concern in terms of the durability of the shore protection. An example is seen in Figure 7, where irregular rubble provides limited protection to the back wall. The back wall exhibits evidence of overtopping and some movement of the armour stone. The displacement is minimal to date but would be expected to progress more quickly if any deepening in front of the wall were to occur (we consider this likely).

Figure 7: Rubble Beach and Armour Stone Wall in Sub-reach C

Following displacement and toppling of the wall at some point in the future, there would be large armour stone in the water, close to the water level, which would act somewhat like a submerged breakwater. Erosion would continue behind this wall, but it would be at a much slower rate than the natural condition.

Sub-reach D - South Marine Drive to Livingston Road

This sub-reach is protected with an 1100 m long revetment that was built in 1992 and reaches from the northeast headland of Sub-reach C to Livingston Rd. The revetment appears to be more robust than the revetment along Sub-reach E (Figure 8). The notes in the TRCA inspection records (WF25) indicate some gradual deterioration along the toe of the structure. This is consistent with our concerns about many of these structures in that they appear to have limited toe stone to accommodate downcutting of the lakebed. There are also notes about some of the smaller material being lost, which is another general comment that we have on the approach of using chinking to fill gaps between loosely packed armour stones. It appears (from aerial imagery) that the stone in the southwestern 490 m of the section has a different placement (possibly more robust) than the northeastern length, although there is no indication of a difference in design.

Figure 8: Revetment Face in Sub-reach D

The central and northeastern part of this section of shoreline shows slightly steeper nearshore slopes (40:1 compared to about 70:1 further south). This steeper slope introduces the possibility of more downcutting of the lakebed and a deeper toe adjacent to the structure. We estimate that after 100 years, the wave height will require armour stone larger than the 4 to 6 tonnes that seems to be common in this area. Our observations were of more widely varied armour stone sizes, ranging from about 2 to 8 tonnes.

This area may see some gradual slumping of the armour stone if toe erosion continues and if some of the chinking is lost. However, while some damage may occur, we do not anticipate any significant erosion of the lakefill behind the revetment (about 40 m width of fill) to an extent that would start to erode the toe of the bluff until significant lowering of the section had occurred.

Sub-reach E - Guild Park

This sub-reach extends over about 900 m and is referred to as WF26 in the TRCA inspection records. The construction date for this revetment and path is unclear; however, it appears that significant repair of slumps and erosion areas were repaired in 2009. This revetment appears to be less robust than the revetment that protects Sub-reach D to the southwest. It was noted in the 2017 inspection that six areas of the revetment were experiencing major erosion (about 50 linear metres), which were subsequently repaired with concrete rubble.

This revetment is in much poorer condition than revetments in other sub-reaches. There are some larger pieces of armour stone, and other areas where there is smaller riprap and scrap concrete / bricks (Figure 9). There is still the potential for lakebed erosion and deepening of the profile. With nearshore slopes of about 38:1 (steeper than other areas) the potential for downcutting is greater than in areas with flatter slopes. This would result in greater wave heights and severely undersized stone in this area. The profile would then be expected to flatten and get lower with more highly mobile and deteriorating shoreline fill.

Figure 9: Examples of Shore Protection in Sub-reach E

Without maintenance, the shoreline revetment would incur further damage, the crest would be eroded and lowered and the structure would behave more like a reef. This flatter and lower position would have more stable stone (much smaller stone is required in a reef structure) limiting wave heights passing over it. Eventually waves could reach back to the bluff face but would be greatly reduced in height compared to a fully unprotected scenario.

Estimating Changes to the Average Annual Recession Rate (AARR) Over Time

If it is assumed the shore protection works are not maintained, but are allowed to deteriorate and fail over time, the lakefill behind the protection works will be exposed to wave action and will erode. Eventually the bluff will be exposed to wave action and it will also erode.

Estimating the remaining useful life of shore protection works along these sub-reaches is based on typical damage progression. This is a highly variable process and must include some engineering judgment and also includes conservative assumptions that can be supported with coastal engineering methods. It has also been demonstrated through the structure inspection reports that some regular maintenance of these structures occurs. This means that any assessment that ignores this fact (as we do in our assessment) will therefore have an additional level of conservatism.

A review of water depths and stone sizes suggests that for the revetment sections (Sub-reaches C and D and the breakwaters of Sub-reach A), the armour stone is expected to remain mostly stable, even with long term downcutting of the lakebed. Some minor shifting and damage are possible; however, the primary mode of failure will be from slumping of the front face due to toe scour and loss of smaller material that is packed between the loosely placed armour. Toe scour is also a likely mode of failure for the flatter cobble sections, in addition to the deterioration of the more mobile cobbles and rubble on the shore.

A review of the past breakwater and revetment failures around the Great Lakes was completed based on available data from some of Baird's other recent projects. Many profiles were available from breakwaters in the Milwaukee, Chicago and Port Washington area of Lake Michigan. The general mode of failure for these breakwaters was to have the profile reshaped so that the crest level was just above the high water level. This deterioration would be expected to further gradually decrease; it is reasonable to assume that the crest would progress to somewhere close to the water level during higher than normal lake levels. Many of these structures were over 100 years old. After having slower damage progression over the past decades, it was the higher water levels in the past few years and some higher wave events that caused significant damage. This suggests that damage will be significantly related to water levels and storm events; it is not a linear process, although we may treat it that way.

The process of eroding the shoreline and potentially the bluff has multiple phases:

- Failure of the revetment and/or cobble beach to a position where it starts to act more as a reef. This is highly dependent on water levels and estimates of the rate at which this might occur are largely based on observing the performance of the structures thus far. If we assume that the structure crest is reduced to a 5 year water level (75.68) then a 100 year water level (76.17) would result in about 0.5 m water depth over the crest of the eroded structure. If we assume that after failure 0.5 m of water exists over the eroded structure much of the time, this would imply a very eroded structure crest.
- Progression of the erosion through the fill behind the structure to the toe of the bluff. These distances are typically 20 to 40 m. Wave heights reaching this fill would be significantly limited by the height of the structure (reduced to a reef). Because we are uncertain about the type of fill that is present (smaller stone sizes may be in the upper layers close to the path surface), and since the elevation of the fill is much lower than the bluff (less volume to erode) we have selected an erosion rate of 1 m/yr. This would have been even larger were it not for the partial protection from the shoreline structures. More rapid erosion would occur initially, followed by slower erosion further inland.
- Erosion into the base of the bluff and recession of the bluff crest. The erosion of the bluff would be at a lower rate than the pre-protection rate since the offshore reef protection (the eroded revetment) would reduce the wave energy reaching the bluff. The depth of the water from the structure crest (reef) and the width of the eroded fill will control the height of the wave that can reach the bluff. Seabrooke (1997) undertook laboratory investigations of the wave height as a function of water depth and distance over a shallow shelf. Based on a this work we estimate that with a shelf of 0.5 m depth, a wave height of 1 m approaching the structure will be reduced to 30% after 20 m, 20% after 30 m, and 15% after 40 m. Sediment transport (and erosion rates) are typically proportional to wave power, which is related to the square of the wave height. It is therefore logical to assume that erosion rates would be related to the square of the values listed above. To remain conservative, we will use a direct relationship (rather than squared) since there are many unknowns including water depth.

There will be some overlap between each of these steps; there will not be a clear transition from one stage to the next. The table below outlines what we believe to be a conservative estimate of the erosion rates through different parts of the shoreline. Notes related to each line in the table are present below the table.

Table 1: Estimation of Erosion Allowance for Reach 30

	Parameter	Sub-reach A	Sub-reach B	Sub-reach C	Sub-reach D	Sub-reach E	Units
A	Unprotected AARR	0.47	0.47	0.47	1.80	0.96	m/yr
B	Protection Condition	Medium	Poor	Medium	Good	Poor	
C	Years to substantial failure	40	20	30	40	20	years
D	Lowering to take on reef effect	10	5	10	10	5	years
E	AARR through fill	0.75	1	1	1	1	m/yr
F	Width of fill	40	20	30	30	20	m
G	Years to erode through fill	53	20	30	30	20	years
H	Total years to reach bluff	103	45	70	80	45	years
I	Factored bluff AARR with degraded protection	15%	30%	20%	15%	30%	%
J	100 year erosion into toe of bluff	-1	7	2	5	15	m
K	Erosion allowance from structure	39	27	32	35	35	m

A. This is the revised unprotected AARR calculated from comparison of pre-construction aerial photos
B. This is a qualitative assessment that aligns with the projected values in rows C and D.
C. This is estimated based on the age of the structure, the size of the material, the crest elevation, etc.
D. A structure that is made from much smaller stone/cobble/rubble will lower more quickly than a structure constructed with large armour stone.
E. The AARR through the fill is difficult to determine. It would not be linear, with the region close to the structure (where the waves are larger) eroding more rapidly. Further inland the waves will be smaller and the material needs to be moved further offshore (past the existing structure) before its protection characteristics are lost, making the erosion rate much slower. It is more likely that the erosion involves pushing up the rubble into a “cobble beach” and quite a slow erosion process. We estimate this value at 1 m/yr, which is likely conservative. Note that during periods of low water, this process may be completely absent, and would only be significant during high water periods.
F. This is an estimate of the width from the back of the primary armour to the toe of the bluff. Note that the toe of the bluff is not cleanly defined as there is significant talus at the base of the bluff, landward of the path.
G. This is the width divided by the AARR.
H. This is the sum of the estimated years until the shore protection structure fails, plus the years to lower the shore protection structure crest, plus the years to erode the backshore to the bluff.
I. The rate of the bluff erosion with a submerged structure in front, expressed as a percentage of the pre-construction erosion rate (A). This is dependent on wave attenuation from the structure to the bluff (the width of fill (F)). We make the conservative assumption that wave height and erosion are linearly related, when in fact a wave height of only 20% of full wave height results in probably much less than 20% of the erosion rate.

J. This is the erosion allowance into the toe of the bluff. This is computed as (100-H)*I*A, where H, I, and A are the rows described above.

K. The total erosion allowance as the sum of (F) plus (J), measured from the back of the revetment or curb wall. The stable slope allowance will subsequently be added to this value.

Variation in long term lake levels will introduce significant variability into the erosion rates, making the accuracy of erosion predictions very approximate. Some qualitative considerations pertaining to the individual sub-reaches are outlined below.

Sub-reach A: The offshore structures are in better condition and appear to be more robust than the cobble beaches between them. It is not clear that the beaches are in a natural planform and there may be some re-shaping of the bays with more erosion in the central parts of the bays. The two stone high curb at the back of beach could topple fairly easily if there was any downcutting or erosion at the toe and reshaping of the profile. Despite this, this sub-reach still has substantial protection, and the offshore structures will slow the reach-averaged erosion through the backshore. It is because of these offshore structures that we have estimated a slower rate of erosion through the fill.

Sub-reach B: This structure is in poor condition and would degrade much more quickly than adjacent sub-reaches. The smaller stone size also means that further degradation and lowering of the profile will be more rapid than in other sub-reaches. Figure 10 shows how cobble and rubble have been pushed inland at the west end of Sub-reach B. The width of fill behind the path is also relatively narrow here.

Figure 10: Shoreward Progression of Cobble/Rubble at West end of Sub-reach B

Sub-reach C: The headlands at Sylvan Ave appear fairly robust above the water, although small chinking stone is gradually being lost. The view from the drone shows that the armour has some very large gaps filled with chinking (the armour is widely spaced in some areas) and the armour extends only about one stone below the water level, below which it is a mixture of smaller material. We have not seen any drawings that suggest the armour extends below the visible limit from the drone images. For this reason, the section is described as moderate, rather than good.

Figure 11: Minimal Below-Water Armour in Sub-reach C (Sylvan Ave.)

Sub-reach D: This is the most robust looking section of revetment in Reach 30, although there are still some concerns about the excessive use of chinking with small stones between the large stones (Figure 12). When downcutting occurs, there is expected to be some shifting of the armour layers down the slope. There is evidence of more loss of chinking stone lower in the profile and this is expected to continue and get worse as the armour shifts more. This will result in a lowering of the crest of the structure. Despite this, the existing armour stones are large and will provide significant protection even after some settlement.

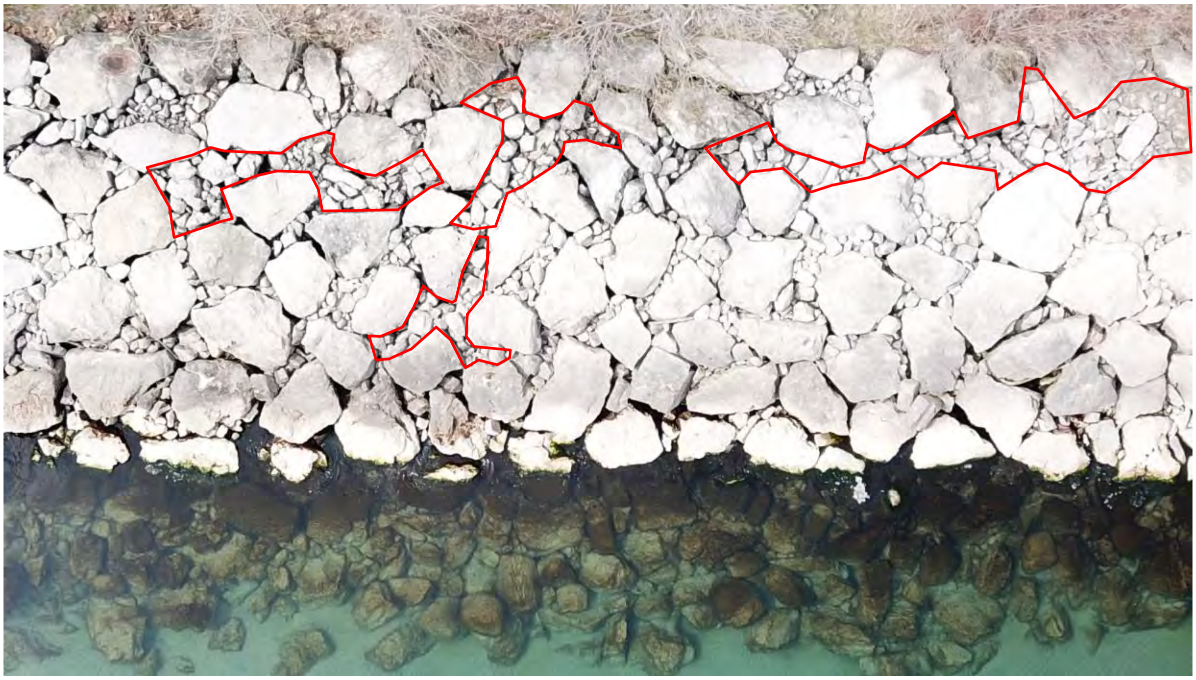

Figure 12: Excessive Chinking in Sub-reach D (selected examples in red)

Sub-reach E: This is a revetment of poorer construction than Sub-reach D. The stones are smaller, and we estimate that the time to failure of the this structure is much less than for Sub-reach D. The subsequent time for lowering of the structure is also less due to the smaller stone size.

Mapping Revisions

Based on the values outlined in Table 1, the erosion allowance has been updated on the hazard mapping sheets for Reach 30. The stable slope allowance was then applied at a slope of 1.8:1. This has resulted in a substantial reduction in the erosion allowance in Reach 30.

Conclusions

The shoreline erosion rates described in this letter are a significant change from those that were initially determined for this study. We recognize that older and more poorly constructed structures will not last as long as more robust structures, yet some level of protection is still offered even from a deteriorating structure. Compared to the standard approach that was applied throughout the remainder of the TRCA shoreline, there are three primary differences for this change in the erosion allowance:

- A time frame is required for failure of the structure, typically through a gradual lowering of the cross section associated with downcutting of the lakebed.
- Time is required to erode through 20 to 40 m of fill towards the bluffs. This will be faster at first but will slow further inland as there will be reduced wave action.
- The erosion rate of the bluff will be reduced (compared to the no protection case) based on a reduction in the wave height reaching the toe of the bluff.

The revised mapping significantly reduces the extent to which residential developments fall within the regulated hazard limit. The TRCA has demonstrated a pattern and willingness to undertake repairs to these structures and it is unlikely that this erosion, which would take decades to progress to the bluff, would not be halted before reaching the bluff. We therefore believe that these erosion rates will likely turn out to be conservative in the long-term.

Sincerely,

Derek Williamson, P.Eng | Principal
Baird & Associates
E: dwilliamson@baird.com
M: 6138686213
S: dcwilliamson