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34 West Avenue (Hamilton) Inc.
c/o LiUNA Local 837
44 Hughson Street S.,
Hamilton, ON L8N 2A7
Attention: Mr. Shawn Marr

via email to smarr@thehi-risegroup.com

Status: Final
July 31, 2020

Dear Mr. Marr,

Reference # 13267.101.L1.Rev1_Shore hazard assessment

RE: 526 WINONA ROAD LiUNA GARDENS SHORELINE HAZARD ASSESSMENT AND SETBACK REQUIREMENTS

Executive Summary

The shoreline hazard limit for redevelopment at LiUNA Gardens is 32 metres measured back from the shoreside edge of the top of the existing concrete block seawall. The hazard limit includes allowances for the flooding hazard, new shoreline protection with a design life of fifty years, an erosion allowance based on the natural average annual recession rate and a stable slope based on a site-specific geotechnical engineering investigation. A new armour stone revetment is proposed; the lakeward extent of the protection will be at the position of the existing concrete block wall. The existing concrete block seawall will be removed. Maintenance access, 6 m wide, will be provided along the shoreline and is contained within the 32 m hazard limit. Maintenance access to the shoreline from a municipal road will also be incorporated in the plan.

Introduction

W.F. Baird & Associates Coastal Engineers Ltd. (Baird) was retained to complete a shoreline hazard assessment for LiUNA Gardens in Stoney Creek for the purpose of determining the shoreline hazard limit to establish an appropriate setback for redevelopment of the site.

The site is located on the shore of Lake Ontario at 526 Winona Road, Hamilton. The shoreline frontage is approximately 225 m, extending from Winona Road to East Street (Figure 1). The existing shoreline protection consists of a stacked concrete block seawall (Figure 2). The westerly portion of the site is occupied by a banquet hall and landscaped grounds (Figure 3) and the easterly portion by a work area and storage yard (Figure 4). It is proposed that the site be redeveloped into a residential area.

Figure 1: LiUNA Gardens shoreline, 526 Winona Road, Hamilton

Regulatory Requirements for Shoreline Hazards

Hamilton Conservation Authority (HCA) regulates the shoreline under Ontario Regulation 161/06[1] and any redevelopment at the shoreline is subject to review and permitting by HCA. HCA has regulations and guidelines[2] governing the shoreline regulation limit, the shoreline hazard limit and development setbacks, and shoreline protection. The HCA regulation limit is the furthest landward extent of the aggregate of the flooding hazard limit plus the erosion hazard limit plus the dynamic beach hazard limit plus 15 m inland. The regulation limit defines the jurisdiction of the HCA. At the site, the shoreline hazard limit and development setback are governed by a combination of shoreline protection, an erosion allowance, and a stable slope allowance. The flooding hazard limit is contained within the erosion hazard limit. The dynamic beach hazard allowance does not apply at the site.

Other agencies such as the federal Department of Fisheries and Oceans (DFO) and Ontario Ministry of Natural Resources (MNRF) may also be involved, depending on the type of protection and the location of the protection with respect to the water's edge and potential encroachment onto Crown Land. The City of Hamilton will also have requirements if the shoreline is being dedicated to the City as part of the project.

[1] Ont. Reg. 161/06, Hamilton Region Conservation Authority: Regulation of development, interference with wetlands and alterations to shorelines and watercourses
[2] Hamilton Conservation Authority, 2011. Planning & Regulation Policies and Guidelines, October

Figure 2: Existing stacked concrete block seawall

Figure 3: Backshore landscaping at banquet hall

Figure 4: Backshore at easterly portion of property

Existing Shoreline Protection

The site is located on the shoreline of Lake Ontario and is subject to erosion and flooding hazards due to fluctuating water levels, wave action and ice action. The hazards are mitigated to some extent by the existing shoreline protection. The existing shoreline protection structure at the site is described in this section.

Baird completed a reconnaissance of the existing shoreline on June 21, 2019; the water level recorded at the Canadian Hydrographic Services gauge in Burlington was 75.95 m International Great Lakes Datum (IGLD) 1985. At Burlington, the difference between IGLD1985 and Canadian Geodetic Vertical Datum (CGVD)-1928:1978 is zero. Generally, lake level data is referenced to IGLD1985 and topographic data to CGVD.

It is understood that construction rubble material was placed at the shoreline circa 1988 – 1989 and that the existing concrete block seawall was subsequently added to contain the perimeter of the lakefilling. The evolution of the shoreline is discussed in more detail in a later section.

The existing seawall is comprised of precast concrete blocks stacked up vertically, with each block slightly setback from the block below it, creating a slight batter to the wall face (Figure 2). The concrete blocks are typically 3 m long by 1 m high by 1 m deep. The wall appears to be four blocks high above the lakebed. It is not known if there are more blocks buried further below the lakebed, but this is considered unlikely; the condition and elevation of the base of the seawall has not been confirmed. The westerly backshore area behind the seawall consists of grass turf and landscaped grounds (Figure 3); the backshore at the easterly part is mostly unimproved with rubble and gravel immediately behind the wall (Figure 4). Presently, the seawall is generally intact and in fair condition. There are irregularities and curves in the alongshore alignment of the seawall (Figure 5); it is not known if the overall alignment has shifted or changed since the original construction of the wall.

A topographic survey was completed by A.T. McLaren Limited (Dwg. No. 35476; survey was completed on November 27, 2017; referenced to CGVD-1928:1978). The top elevation of the seawall is approximately 77.8 m to 78.0 m CGVD. The shoreline grade gradually rises from the seawall to about elevation 79.2 m CGVD, approximately 35 m back from the seawall. Spot measurements at five locations along the seawall indicate the lakebed elevation at the base of the wall is about 74.0 m. Further discussion of the bathymetry is provided in a later section.

Figure 5: Wall alignment irregularities

Figure 6: Immediate backshore at easterly part of site

A steel railing extends along the length of the block seawall.

At the west end of the site, the concrete block seawall transitions to a row of armour stone, approximately 10 m long. At the crest of the armour stone are some gabion baskets, loose stone rubble and an upper retaining wall of small concrete blocks (Figure 7). There is evidence of wave overtopping damage in this area.

The site is bounded to the west by Winona Road. A concrete headwall with a storm water drainage pipe and grate is located at the end of Winona Road (Figure 8). The storm drain discharges into an outlet channel that is lined with armour stone on both sides and extends to the shoreline at the Lake (Figure 9).

The site is bounded to the east by East Street. At the east end of the site, the concrete block seawall abuts against a concrete headwall at the shore end of East Street (Figure 10). Two drainpipes outlet through the concrete headwall. Armour stone has been placed along the base of the East Street headwall.

The adjacent properties to the east and west have shoreline protection structures. To the east, there are also multiple docks and piers extending from the shore (see left hand side of Figure 1).

Figure 7: Transition from concrete block seawall to armour stone, gabion baskets, and stone rubble splash pad at west end of site

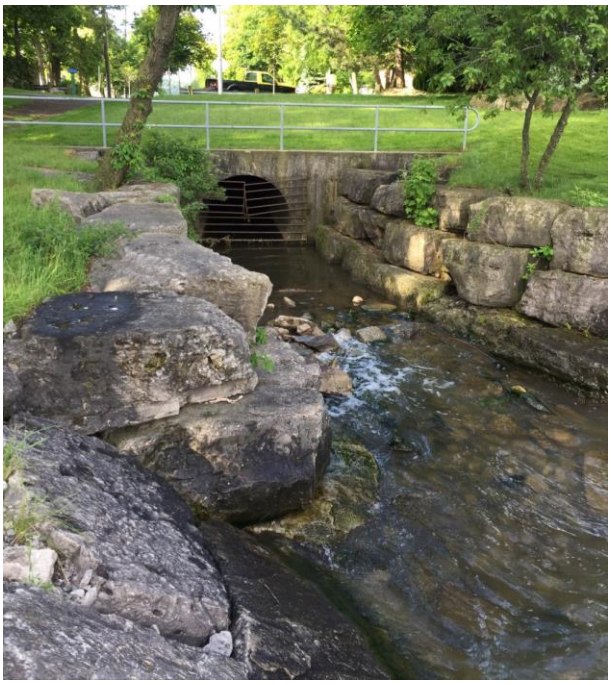

Figure 8: Concrete headwall with storm water pipe and grate at end of Winona Road

Figure 9: Armour stone-lined storm water discharge channel at Winona Road

Figure 10: East end of concrete block seawall abutting concrete headwall at East Street

Along the length of the existing seawall at the site, several of the blocks were observed to have shifted and are now displaced various amounts towards the lake and some blocks have settled (Figure 11, Figure 12). Washed out areas behind the crest of the seawall, with loss of grass cover and backfill, were observed indicating damage due to wave overtopping (Figure 13 and Figure 14). The condition of the wall underwater was not observed.

Figure 11: Shifted and settled concrete blocks

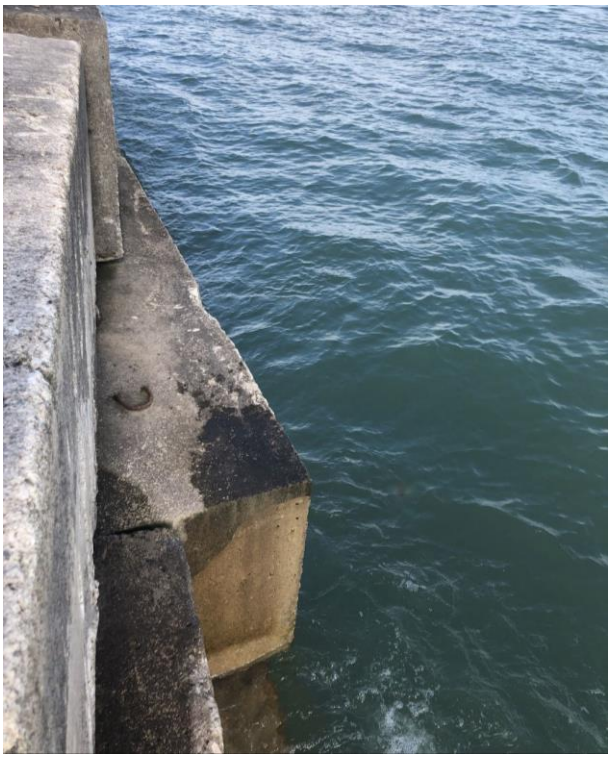

Figure 12: Displaced concrete block

Figure 13: Evidence of wave overtopping

Figure 14: Example of washout and loss of backfill behind concrete block shorewall

Bathymetry

The bathymetry[3] in the vicinity of the site is presented in (Figure 15). Approximately 80 m offshore the depth is 2 m below chart datum (CD); CD is 74.2 m IGLD1985 and is an elevation that the water level rarely goes below. The bathymetric survey does not include the area close to the seawall. From the initial site visit the lakebed elevation at the base of the seawall is estimated to be elevation 74.0 m IGLD, or about 0.2 m below CD. A nearshore survey extending from the shoreline seawall out to about 2 m to 3 m below chart datum (74.2 m IGLD1985) is proposed for the spring of 2020 to confirm the conditions for final design.

[3] Canadian Hydrographic Services LiDAR bathymetry survey completed in 2017

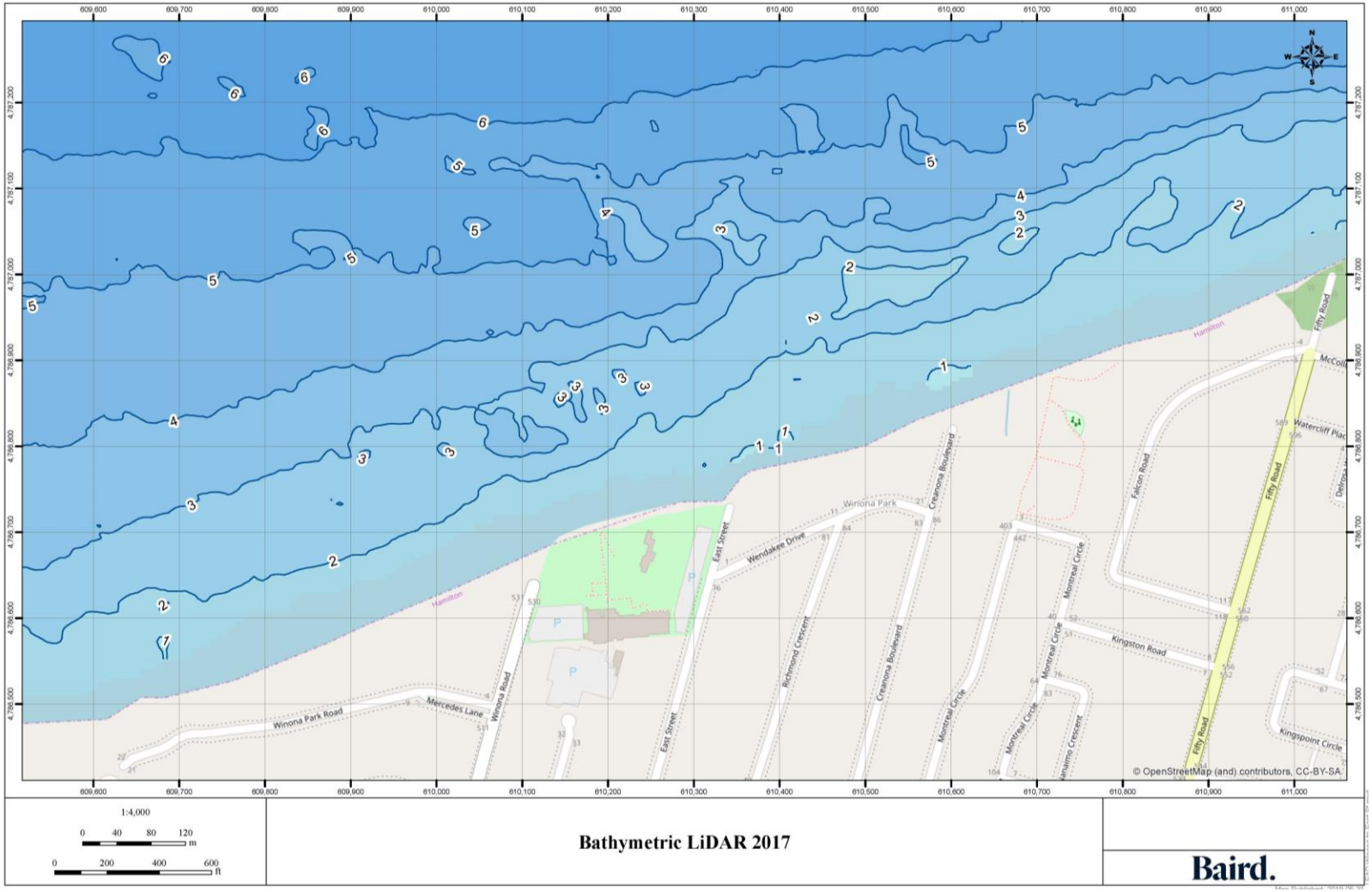

Figure 15: Bathymetry (in metres below chart datum 74.2 m IGLD1985; ref. CHS LiDAR 2017)

Geotechnical Investigation (Landtek Limited)

A geotechnical investigation report was prepared by Landtek Limited (November 29, 2019; File 19318) and was provided to Baird. The Landtek report and subsequent correspondence was relied upon to provide factual information regarding the geotechnical conditions at the site and the slope stability recommendations.

The geotechnical site investigation included drilling multiple boreholes and monitoring wells at the site; four of these are in proximity to the seawall and were considered for this study: Borehole (BH) 1, Borehole/Monitoring Well (BH/MW) 2, BH/MW 3 and BH4.

- BH1 is located near the westerly end of the site, approximately 30 m back from the seawall. Soils encountered were very stiff to hard clayey silt till to elevation 66.5 m where the borehole was terminated.
- BH/MW2 is in the easterly part of the site, near the existing seawall. Soils encountered were approximately 3 m of sand, gravel and concrete rubble fill down to elevation 75.6 m, over very stiff clayey silt till to elevation 56.6 m where the hole was terminated.
- BH/MW3 is located approximately 30 m back from the seawall at BH/MW2. Soils encountered were 0.7 m of sand and gravel fill over very stiff to hard clayey silt till to elevation 55.3 m where the hole was terminated on presumed bedrock. Landtek reported the bedrock is red shale.
- BH4 is located near the easterly end of the site, approximately 45 m back from the seawall. Materials encountered were approximately 2.3 m of fill (clayey silt, some gravel, some concrete, limestone fragments (gravel sizes)) over hard clayey silt till to elevation 76.0 m where the hole was terminated.

Landtek reported that the fill materials encountered in boreholes BH/MW2 and BH4 are associated with the historical filling works that are known to have been undertaken at the shoreline.

The stable slope inclination of the native soil (clayey silt till) was estimated by Landtek (email James Dann, Landtek, Dec. 12, 2019) to be 2.1:1 (horizontal:vertical). Landtek will be submitting a slope stability assessment report under separate cover.

General Shoreline Processes

Overall, the shoreline is classified as a “cohesive shoreline”. Cohesive shorelines are comprised of consolidated mixture of silts, clays, sand and gravel. This is consistent with the material reported in the site borehole data (stiff to hard clayey silt till; bedrock was not encountered until about elevation 55.3 m).

The controlling process for the recession of a cohesive shoreline bluff is the downcutting, or downwards erosion of the nearshore cohesive lakebed profile by the wave induced forces and water pressures. Figure 16 presents a schematic of cohesive shoreline nearshore downcutting and bluff recession processes. The ongoing downcutting of the nearshore profile eventually results in the toe of the shoreline bluff being undercut. Continued undercutting of the bluff will over-steepen the bluff face and it will collapse, resulting in recession of the crest. If the backshore composition of the shoreline is the same, the cohesive shoreline recedes over time without a change in the shape of its profile. The erosion or downcutting of the cohesive material in the nearshore profile is irreversible and ongoing. Even though the process is governed by the downward cutting of the nearshore bottom by wave action, the visible effect is a horizontal translation of the entire shoreline profile at the long-term average bluff recession rate.

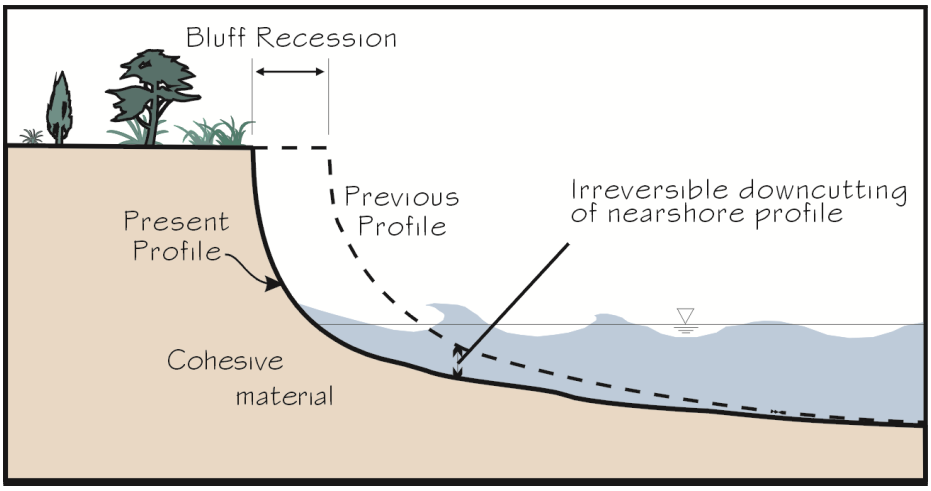

Figure 16: Cohesive shoreline nearshore downcutting and bluff recession

Cohesive shoreline nearshore profiles in predominantly fine-grained material (i.e., relatively high percentage of silts and clays with relatively lower percentages of sand and gravel or cobbles) are characterized as distinctively concave. The constant concave profile shape over time indicates that the downcutting rate is the greatest at the shoreline and gets less towards the offshore. A nearshore profile at Winona Road (Erosion Profile O-2-13) is presented in Figure 17 and demonstrates this concave shape, characteristic of a cohesive shoreline profile, with a nearshore slope of approximately 1:30 (vertical:horizontal) in the first 80 m.

Cohesive shoreline material is typically comprised of a relatively large percentage by volume of fine particles, such as silt and clay, and a smaller percentage of coarser materials, such as sand, gravel, and cobbles suitable for beaches. The fine particles in the eroded material are not stable at the shoreline edge and are carried offshore by wave action to the deep water where they settle to the bottom and are lost from the littoral system. The coarser beach materials, which are large enough to remain in the breaking wave zone near to the shore, are moved alongshore and on- and offshore by the wave action. However, typically along cohesive shorelines, the wave energy available to move these coarser beach materials exceeds the volume of beach materials available from erosion of the shoreline, meaning that, the beach material cannot accumulate in a large enough volume to form a protective cover for the underlying erodible cohesive substrate during storms. Narrow beaches may form during the calmer summer months or extended periods of low water, but they are generally transient and insufficient to provide protection. There is no beach at the site, even at average water levels.

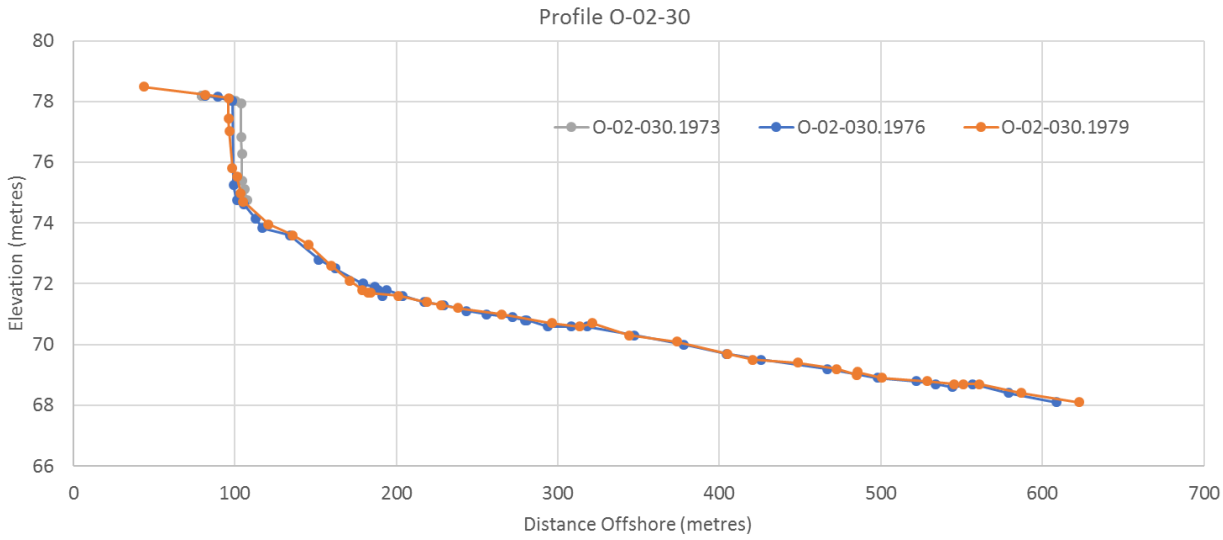

Figure 17: Nearshore profile at Station O-02-30 (at Winona Road), 1973 to 1979 (ref. MNR shoreline profile database)

Average Annual Recession Rate

The erosion allowance component of the erosion hazard limit is based on the natural shoreline recession rate (i.e., without shoreline protection). Baird completed a site-specific analysis of the natural shoreline average annual recession rate (AARR) using historic data, aerial photographs and land surveys and accepted, best practices, including modern digital GIS technology. The AARR used for this study is 0.3 metres/year.

Historical aerial photos from 1931 (April 18), 1965 (July 26), 1986 (August 19), 1989 (April 21) and orthophoto imagery from 2018 (spring) were obtained and used to estimate the average annual recession rate (AARR) at the site. The aerial photos were georegistered and the top of bank in each image was digitized in GIS. Figure 18 presents the 1965 digitized top of bank superimposed on the 1965, 1986, 1989 and 2018 images. The following historical surveys were also obtained and georegistered in GIS: MacKay MacKay & Peters Limited Plan 62R-11395, January 1989; and Plan of Lakeside Park, RP-1032, John T. Peters, 1955 (surveyed 1953 to 1955). Relevant lines from these surveys are shown in Figure 19.

Circa 1988 – 1989 the shoreline at the site was filled with construction rubble to protect the eroding shoreline; the lakefilling can be seen in Figure 18 (April 21, 1989 photo). The concrete block seawall was added to contain the perimeter of the lakefilling. Because of the addition of the shoreline protection, only air photos and surveys prior to 1988 - 1989 were used in the recession rate analysis for this study.

Figure 19 shows a summary of shoreline position data considered for the recession rate analysis. It was concluded that over the 34-year period from 1931 to 1965, the average annual recession rate of the top of bank ranged from 0.12 m/year to 0.21 m/year. For the erosion hazard limit for this study, an average annual recession rate (AARR) of 0.3 m/year has been used (i.e., 1.4 times the maximum measured value).

For comparison, another study indicated a lower AARR of 0.03 to 0.04 m/year between 1928 and 1966[4] for this area.

[4] Geomatics International, 1992. Great Lakes Shoreline Classification and Mapping Study: Canadian Side, Final Report, July

We are aware of the 1980 Stoney Creek Waterfront Study prepared by Reinders and Associates. The recession rate information in the 40-year old Reinders' study is based on Rutka's master's degree thesis completed five years prior in 1975[5]. Rutka's analysis was based on 45-year old technology and was limited by the resources available at the time. Since then, there have been many advancements in the technology and methods of shoreline recession analysis. As described above, for the recession rate analysis completed for the site, we used modern, best practices including state-of-the-art digital technology and GIS-based tools allowing for georegistration of photos to correct for distortion. When working with air photos, digital processing provides a better, more accurate dataset for measurement because it is possible to correct for issues like lens distortion and then digitally scan and magnify the imagery at details beyond what the human eye can see and measure manually from contact prints, as was done by Rutka (1975). The present study included a georegistration process to remove the radial lens distortions and match the historic photos to the 2018 orthophotos. Rutka did not undertake georegistration of his photos; he manually measured directly from prints of the of the photos. Rutka noted that "the photos themselves have inherent errors due to photographic distortion as well as to expansion and contraction of the photographic paper with climatic changes".

Both the present study and Rutka (1975) used the 1:15,000-scale air photo from 18 April 1931 as the historic base image. We obtained a digitally scanned version of the 1931 photo from the National Air Photo Library. For the present analysis, for the second image we used a higher resolution photo at a better scale (i.e., 1965 image at 1:20,000-scale) than was used by Rutka (i.e., 1969 image at 1;30,000-scale according to National Air Photo Library, although Rutka identified it as 1:26,500). Use of the better 1965 photo, although it provided four years less of comparison than the 1969 photo, provides more reliable results.

The present study also involved multiple measurements at locations along the 220 m length of the site, resulting in a range of average recession rate values from 0.12 m/year to 0.21 m/year; for the present study we adopted a recession rate of 0.3 m/year. Near the site, Rutka (1975) reported only two discrete measurement locations approximately 750 m apart, one each on either side of the site, with average annual recession rates of 0.6 m/year and 0.8 m/year, respectively.

[5] Rutka, R., 1975. Bluff Recession on Western Lake Ontario, Dissertation submitted to University of Waterloo for M.A.Sc. Degree

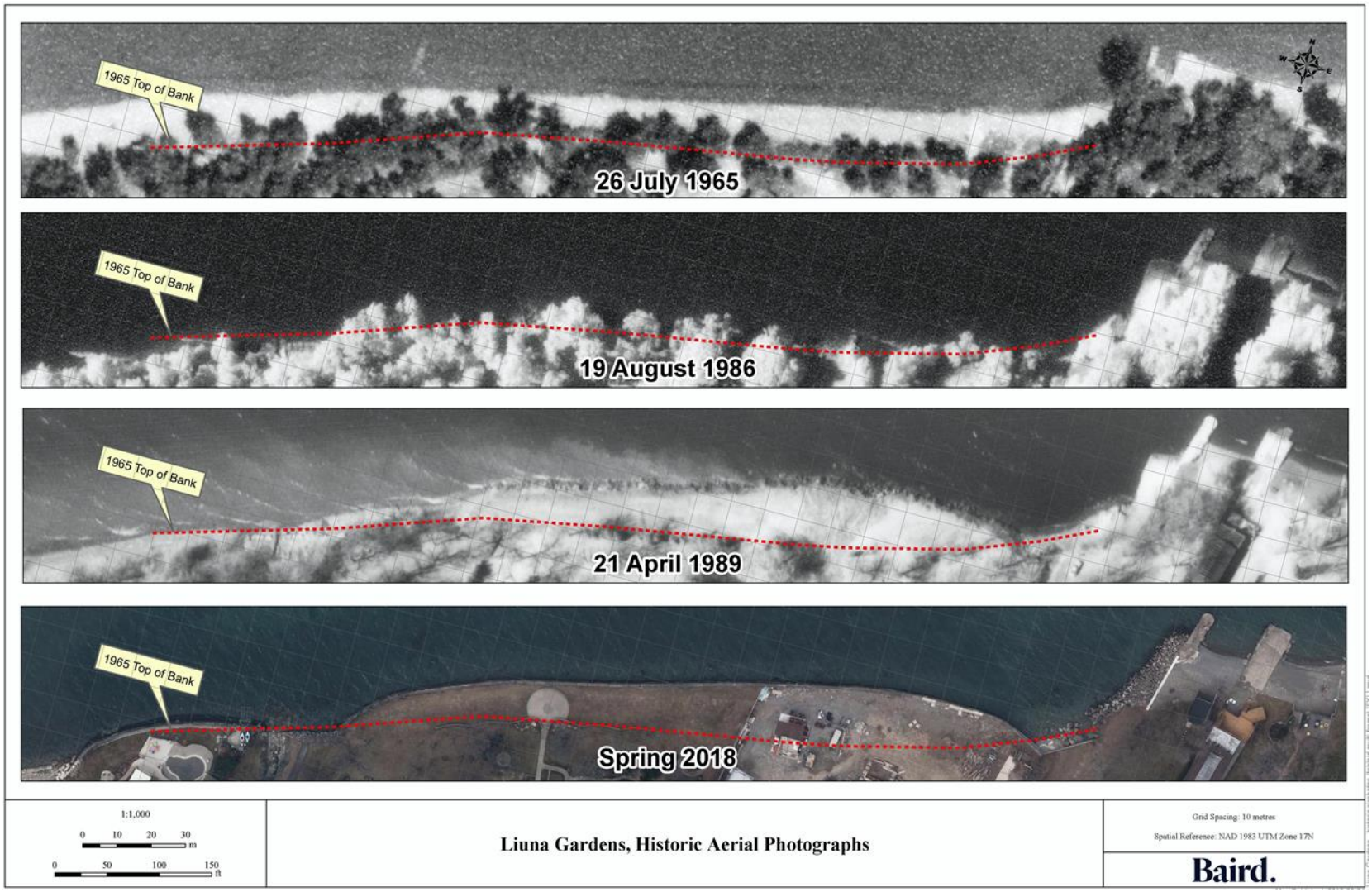

Figure 18: 1965 top of bank superimposed on aerial images from 1965, 1986, 1989 and 2018

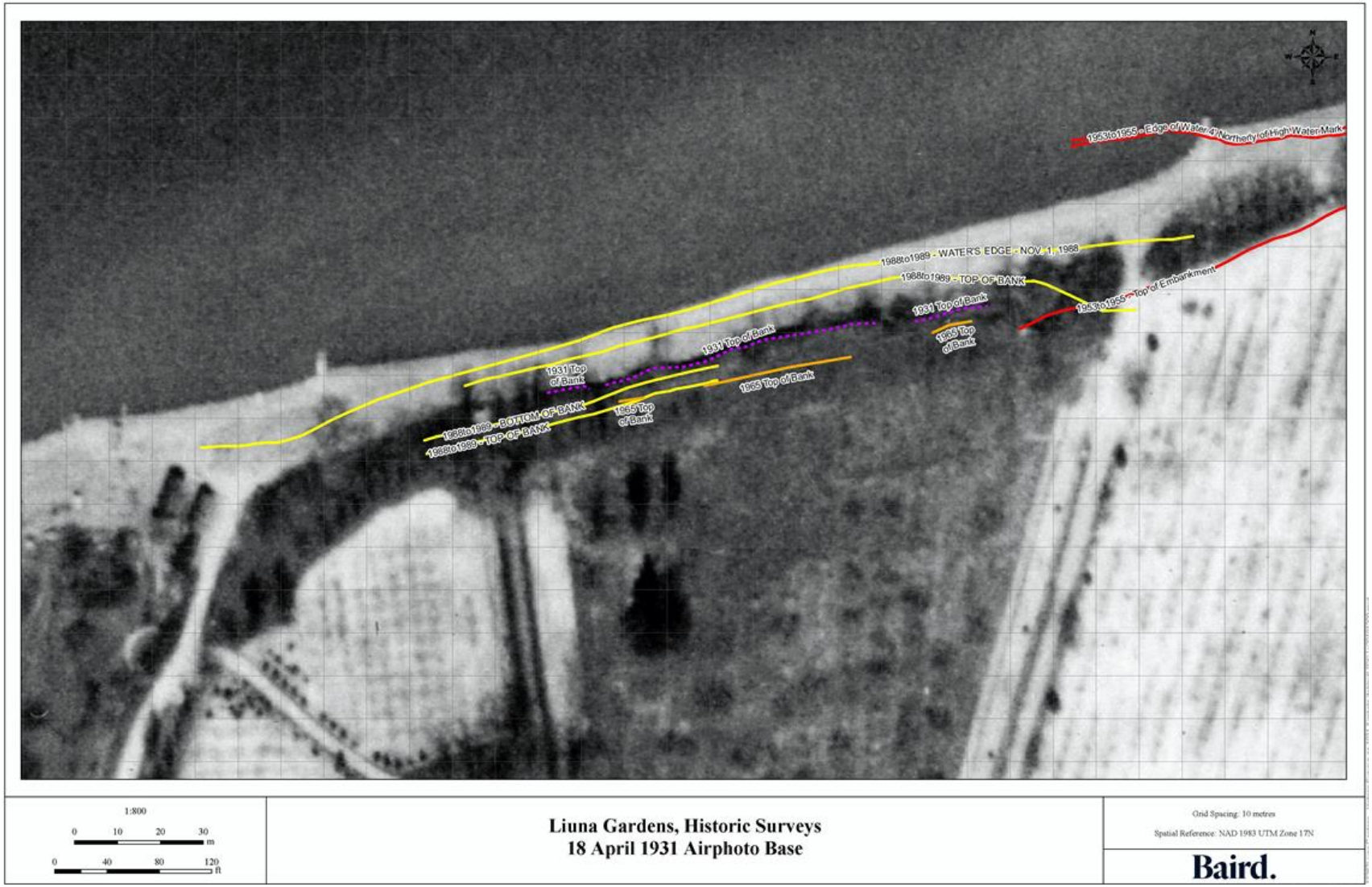

Figure 19: Top of bank shoreline position comparison, 1931 (dashed magenta line) to 1965 (orange line)

Overview of Shoreline Hazard Assessment and Development Setback

Baird completed an assessment of the shoreline hazards based on our experience and understanding of the methods established by Hamilton Conservation Authority (HCA), Ministry of Natural Resources and Forestry and the City of Hamilton. The shoreline hazard limit for redevelopment at LiUNA Gardens is 32 metres measured back from the shoreside edge of the top of the existing concrete block seawall. The hazard limit includes an allowance for new shoreline protection with a design life of fifty years, an erosion allowance based on the natural average annual recession rate and a stable slope allowance based on a geotechnical engineering investigation at the site. A new armour stone revetment is proposed to replace the existing concrete block shorewall; the lakeward extent of the new revetment protection will be at the position of the existing concrete block wall. Maintenance access is provided along the shoreline and to the shoreline from a municipal road.

The changing water levels, severe wave action and ice forces of Lake Ontario result in significant erosion and flooding hazards to shoreline properties. To address the erosion hazard and the flooding hazard, Hamilton Conservation Authority (HCA) requires development to be setback from the shoreline in accordance with their regulations and guidelines[6]. Other agencies such as the federal Department of Fisheries and Oceans (DFO) and Ontario Ministry of Natural Resources and Forestry (MNRF) may also be involved, depending on the type of protection and the location of the protection with respect to the water's edge and Crown Land.

Where Hamilton Conservation Authority (HCA) considers development proposals and/or site alterations in or on the areas within their jurisdiction adjacent or close to the Lake Ontario shoreline, they specify that the erosion hazard limit shall be applicable as follows[7]:

a. *Erosion hazards are based on a combined influence of:*
    i. *Stable slope allowance of 3(H):1(V);*
    ii. *A 30 m toe erosion allowance (measured from stable slope allowance); and*
    iii. *The existence or absence of shoreline protection works.*
b. *A valid engineering study, undertaken by a qualified coastal engineer and at the expense of the proponent, may be undertaken or may be required to be undertaken, in areas where the exact extent of the erosion hazard limit needs to be verified. The need for greater hazard land limits may be demonstrated through the completion of this study*.

Additionally, in accordance with the Ontario Ministry of Natural Resources (OMNR) Technical Guide[8], the erosion allowance is based on a combination of shoreline protection and a further allowance for continued recession of the shoreline beyond the expected design life of the shoreline protection.

As a qualified coastal engineering firm, Baird completed an appropriate valid engineering study of the shoreline hazard limit and development setback requirements based on our professional engineering understanding and interpretation of HCA Planning & Regulation Policies and Guidelines[9], the OMNR Technical Guide, and the actual conditions encountered at the site. This study supports a site-specific stable slope allowance of 2.1:1 and a 15 m erosion allowance based on the proposed shoreline protection works with a design life of 50 years. A detailed stable slope analysis will be submitted by Landtek under separate cover.

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[6] Hamilton Conservation Authority, 2011. Planning & Regulation Policies and Guidelines, October
[7] *Ibid.*
[8] Ministry of Natural Resources (MNR), 2001. Technical Guide for Flooding, Erosion and Dynamic Beaches, Great Lakes – St. Lawrence River System and Large Inland Lakes. Ontario Ministry of Natural Resources.
[9] *Ibid.*

Shoreline Hazard Limit

The shoreline hazard limit at the site is governed by the erosion hazard limit. New development should be setback from the shoreline, at a minimum, to the erosion hazard limit.

The proposed shoreline hazard limit, as governed by the erosion hazard limit, is located 32 m measured from the back edge of the top row of concrete blocks in the existing seawall. The 32 m dimension is comprised of an allowance of 8 m required for the construction of new shoreline protection (with a design life of 50 years), measured to the back of the proposed protection at elevation 75.0 m, plus a 15 m erosion allowance (50 years @ 0.3 m/year)' plus a 9 m stable slope allowance (from elevation 75.0 m to elevation 79.2 m @ 2.1:1). Figure 20 shows the location of the 32 m development setback limit in plan view with new shoreline protection with a minimum design life of 50 years. Figure 21 shows the cross-section of the 32 m shoreline hazard limit. The site plan should provide for a 6 m wide access to the shoreline from either of the municipal roads adjacent to the site and a 6 m wide access along the shore for future maintenance, repairs and upgrading of the protection.

Details of the various components of the shoreline hazard limit are outlined in the following sections.

Erosion Hazard Limit

The erosion hazard limit is determined based on the design life and effectiveness of the shoreline protection, an additional erosion allowance, based on the recession rate of the shoreline and the residual time between the end of the design life of the protection structure and the end of the planning horizon for the project, and a stable slope allowance. The recession of the shoreline is based on the ambient, natural recession of the shoreline in the absence of shoreline protection. An average annual recession rate of 0.3 m/year has been used for the site, as discussed in an earlier section. The erosion hazard limit is measured along elevation 75.0 m (see Figure 21), which matches the approximate elevation of the toe of a natural shoreline bluff.

Shoreline Protection

Protection works can mitigate the shoreline erosion, wave action and ice forces, but the protection works do not eliminate the hazards entirely. Based on the cohesive shoreline type at the site, as described earlier, protection works will only control the visible erosion at the water's edge for a limited period; the protection will not stop ongoing erosion of the lakebed in front of the structure. Figure 22 illustrates how erosion of the lakebed in front of a protection structure will eventually lead to undermining of the structure. Due to the nearshore recession, undermining of any shore protection structure is a concern and erosion of the lakebed will be addressed in the final design.

Based on our experience, it is expected that the City of Hamilton will require a design life of 50 years for the shoreline protection structure at the site. Structure design life is the length of time that a structure, with maintenance, can safely and adequately perform its function. The shoreline protection structure must have an adequate crest elevation to protect against overtopping waves and be of durable and robust design. It is our opinion that the existing stacked concrete block wall does not meet the requirement for a 50-year design life; the foundation is unknown, and likely not sufficiently embedded into the lakebed; the crest elevation is inadequate; and there is evidence of shifting and settlement of the concrete blocks.

It is recommended that a new sloping armour stone revetment, like the structure west of Millen Road (Figure 23), be implemented at the site. A double layer armour stone revetment concept is proposed and is shown in Figure 21. An allowance of 8 m has been provided for the new revetment structure, measured from a theoretical line extended downward from the position of the shoreside edge of the top concrete block of the existing seawall to the back of the new protection at elevation 75.0 m. A stone revetment structure provides a

robust level of protection. It is a flexible structure with a good reserve capacity if wave conditions exceed design conditions and damage that occurs is generally progressive, allowing time for repairs if required. A stone revetment can be repaired by adding additional stone if required in the future.

To facilitate permitting by the regulatory agencies, it is proposed that the toe of the new revetment structure will not encroach further into the water than the toe of existing concrete block wall structure; once the revetment is installed the concrete block wall will be removed. HCA generally does not support protection works that encroach on fish habitat. Similarly, DFO has regulations against impacts to fisheries, and MNRF would be concerned with any encroachment onto Crown Land. Also, by maintaining the same toe location as the existing wall and by replacing the vertical, impermeable face of the existing concrete seawall with a sloping, more permeable stone revetment, the proposed shoreline protection will not negatively impact neighbouring shorelines.

The existing seawall materials (concrete blocks) may be incorporated into the new shoreline protection design where possible. It is acknowledged that there may be concerns with the potential quality of the existing concrete blocks; during the final design process these concerns will be taken into consideration and only blocks of good quality will be reused and only in the second (lower) armour layer where they would be less exposed to deterioration. If any blocks are not suitable for reuse as described, consideration will be given to breaking the concrete blocks down to rip rap-sized material and reused in the underlayers or core of the new structure. Unsuitable material will not be used in the revetment.

The potential downcutting will be estimated during the detail design and will be incorporated into the final design drawings, including an additional 0.3 m embedment for the toe elevation, as recommended by Conservation Hamilton. The downcutting elevation will be determined following completion of the site specific nearshore bathymetric survey that is being undertaken at the site.

The final design of the revetment will be designed for the 100-year flood level, wave uprush, and in accordance with accepted scientific and coastal engineering principles and practice.

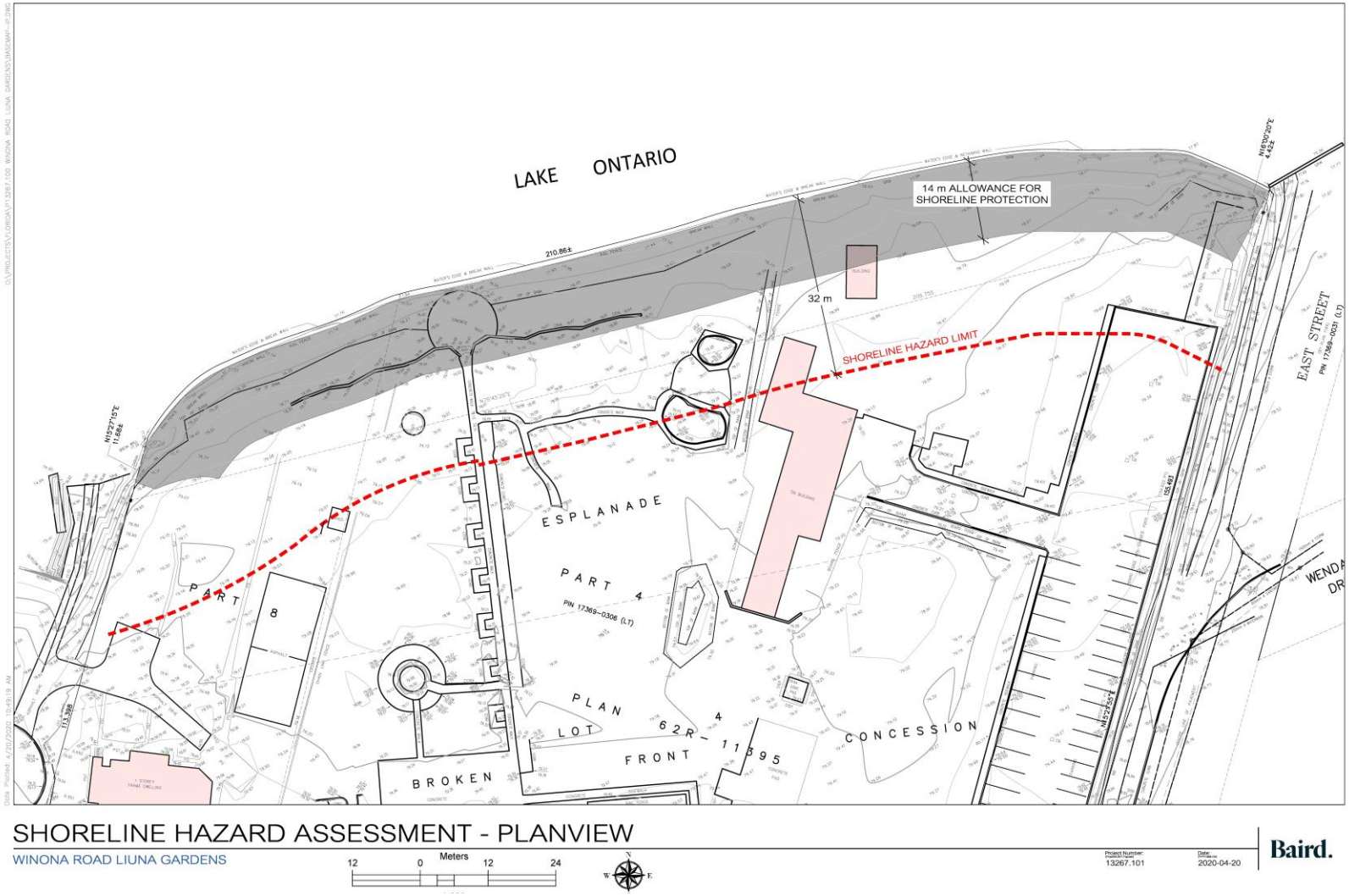

Figure 20: Shoreline hazard development setbacks (32 m from shore side of existing concrete seawall) and allowance for new protection

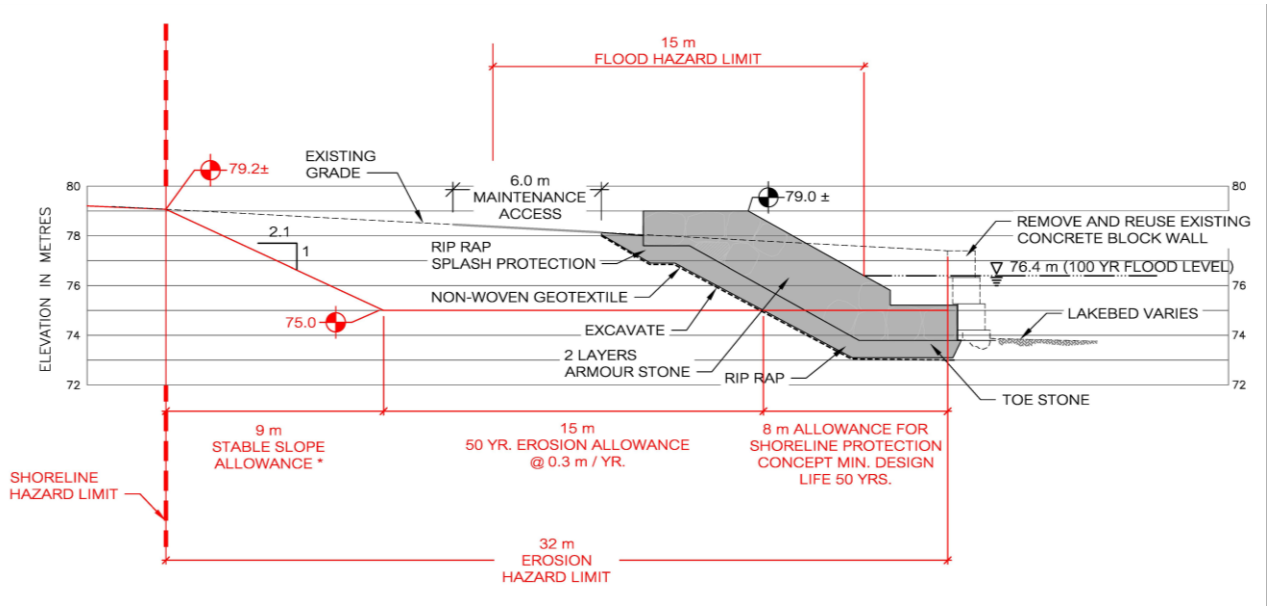

Figure 21: Shoreline hazard limit with shoreline protection concept, erosion allowance and stable slope allowance and incorporating 6 m wide maintenance access

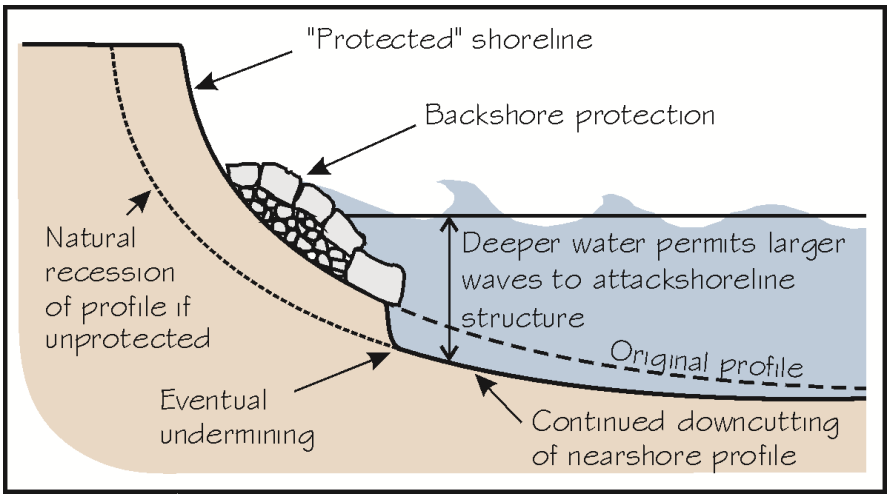

Figure 22: Limited design life of shoreline protection due to continued downcutting of nearshore

Figure 23: Example of double layer armour stone revetment (west of Millen Road)

Erosion Allowance

An erosion allowance is required in addition to the new shoreline protection revetment. The erosion allowance was determined in accordance with accepted engineering and scientific practice, as outlined by the OMNR Technical Guide and HCA Policies and Guidelines. After establishing the expected design life of the shoreline protection works (i.e., 50 years as discussed earlier), the additional erosion allowance required to the end of the development planning horizon is determined based upon the natural average annual recession rate at the site. The development planning horizon is 100 years. The balance of time required for the additional erosion allowance to the end of the development planning horizon will therefore be 50 years (i.e., 100-year planning horizon minus 50-year design life for the new protection structure). The additional erosion allowance (horizontal distance measured in metres) is thus determined as 15 m, based on 50 years of residual time between the end of the design life of the protection structure and the end of the planning horizon for the project multiplied by the average annual recession rate of 0.3 m/year established for the shoreline. For purposes of determining the development setback, the 15 m erosion allowance is measured horizontally from the location of the back of the proposed protection structure at elevation 75.0 m to the toe of the stable slope allowance, as shown in Figure 21.

Stable Slope Allowance

The erosion hazard limit also includes a stable slope allowance of 9 m (Figure 21). The stable slope allowance was determined by extending up from elevation 75.0 m at a stable slope inclination of 2.1:1 (reference Landtek geotechnical investigation) to the existing elevation of 79.2 m in the backshore. A detailed stable slope analysis will be submitted by Landtek under separate cover.

Maintenance Access

Damage and deterioration of any shoreline protection structure should be expected, and access for future repairs and upgrades is a requirement of HCA. The site plan will provide for a 6 m wide access to the shoreline from at least one of the municipal roads adjacent to the site; the final design of the maintenance access to the shore will be incorporated into the landscape and public access plans. A 6 m wide access along the top of the protection works, connecting to the access from the road, will also be provided. The maintenance access must remain reasonably unobstructed. The 6 m wide access along the shoreline is included within the development setback (see Figure 21); this is consistent with *HRCA Planning & Regulation Policies and Guidelines* (October 2011) and the Ministry of Natural Resources Technical Guide for Great Lakes – St. Lawrence River Shorelines (2001).

Flooding Hazard Limit

The flooding hazard limit is contained within the shoreline hazard limit (Figure 21). The flooding hazard is defined by the 100-year flood level and a 15 m horizontal allowance for wave uprush and other water related hazards.

Present HCA guidelines state that the flooding hazard limit for the shoreline is elevation 78.5 m IGLD1955; this is equivalent to elevation 78.58 m CGVD. This level includes the 100-year flood level plus 2.5 m for wave action and other water-related hazards. HCA notes that this flooding limit is based on a study from 1980.

The 100-year flood level established by OMNR in 1989[10] for the shoreline in the area is 76.0 m CGVD28. Since that time, there are 30 years of additional water level records, the lake regulation plan has changed, and climate change effects are being considered. Recent analysis by Baird suggest that the 100-year level should be increased by 0.4 m and it is recommended that the 100-year flood level be increased to 76.4 m CGVD28 for the purposes of this study. The tableland elevation at the shoreline hazard limit is approximately 79.2 m, or about 2.8 m above the revised 100-year flood level and is safely above the flood and wave uprush level established in HCA policy.

Additionally, the flooding hazard limit as established by the standard 15 m horizontal offset for wave uprush measured from the contour of the 100-year flood has been delineated and is contained within the erosion hazard limit (see Figure 21).

The primary development is located landward of the flood hazard limit, therefore floodproofing standards are not applicable. This is evident because the elevation of the backshore (approximately 79.0 m and higher) is above the floodproofing standard elevation provided by the OMNR Technical Guide, which is an elevation equal to, as a minimum, the sum of the 100-year monthly mean lake level plus the 100-year wind setup plus an allowance for wave uprush and other water-related hazards. For the project site, the 100-year monthly mean lake level is 75.94 m CGVD and the 100-year wind setup is 0.94 m. Therefore, the floodproofing standard elevation is 76.9 m CGVD plus an allowance for wave uprush and other water-related hazards. Assuming even a 1.0 m allowance for wave action, the minimum floodproofing standard elevation would be 77.9 m CGVD. The tableland elevation at the shoreline hazard limit is approximately 79.2 m and is safely above the floodproofing standard elevation.

[10] Ministry of Natural Resources (MNR), 2001. Technical Guide for Flooding, Erosion and Dynamic Beaches, Great Lakes – St. Lawrence River System and Large Inland Lakes. Ontario Ministry of Natural Resources

Summary

The shoreline hazard limit for redevelopment at LiUNA Gardens is 32 metres measured from the shoreside edge of the top of the existing concrete block seawall. The hazard limit includes allowances for new stone revetment shoreline protection with a design life of fifty years, a fifty-year erosion allowance based on the natural average annual recession rate of 0.3 m/year determined by a site-specific study, and a stable slope based on a site-specific geotechnical engineering investigation. A new armour stone revetment is proposed; the lakeward extent of the new protection will be at the position of the existing concrete block wall and the existing concrete block wall will be removed when the new revetment is in place. The flooding hazard limit is contained within the shoreline hazard limit. A minimum 6 m wide maintenance access along the shoreline is contained within the shoreline hazard limit and a 6 m wide maintenance access to the shoreline from a municipal road will be provided.

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