Hamilton Conservation Authority

A Healthy Watershed for Everyone

Board of Directors To Sit As Hearing Board

Thursday, September 12, 2024 at 6:00 p.m.
Section 28 Hearing Re: 10 Lakeside Drive, Stoney Creek
Permit Application No. SC/F,C,A/24/40

Hamilton Conservation Authority is now conducting meetings in a hybrid format via an in-person and WebEx platform.

All hybrid meetings can be viewed live on HCA's You Tube Channel:
https://www.youtube.com/user/HamiltonConservation

6.4. Questions from applicant and/or applicant's counsel to HCA staff

6.5. Questions from HCA staff and/or staff counsel to applicant

6.6. Questions from Hearing Board to HCA staff and/or applicant

7. **Hearing Board to move In Camera**

8. **Hearing Board to reconvene in public forum**

9. **Chair to advise of Hearing Board's decision**

10. **Adjournment**

Hamilton Conservation Authority

A Healthy Watershed for Everyone

August 6, 2024 File: SC/F,C,A/24/40

BY EMAIL

Sayed Shakour
10 Lakeside Dr
Stoney Creek, ON L8E 5C2

Dear Mr. Shakour:

RE: NOTICE OF HEARING
Hearing under Section 28.1(5) of the *Conservation Authorities Act* for an Application by Sayed Shakour for Development in a Regulated Area of Lake Ontario at 10 Lakeside Drive, City of Hamilton (Stoney Creek)

This letter serves to inform you that the application by Sayed Shakour, received June 9, 2024, for development in a regulated area of Lake Ontario will be considered by the Board of Directors at the meeting scheduled for:

6:00 p.m. on September 12, 2024
Please note this Hearing will be held by Webex video conference.
Details on the video meeting link will be sent separately.

This is a Hearing under Section 28.1(5) of the *Conservation Authorities Act*. Please note that Authority staff is recommending **refusal** of the application on the basis that the development does not meet the requirements of the development Regulation under the *Conservation Authorities Act.* A copy of the staff report outlining staff's reasons for recommending refusal is included with this notice. Also attached is a copy of the HCA's Hearing Guidelines.

You are invited to speak in support of your application and submit supporting written material for the Hearing. You will be allotted approximately 20 minutes to speak at the Hearing. You may be represented by legal Counsel or have advisors present information to the Board of Directors. If you intend to appear, or if you believe that holding the hearing electronically is likely to cause significant prejudice, please contact Mike Stone, Acting Director, Watershed Management Services, to confirm attendees. You previously provided written material to present to the Board of Directors in advance of the Hearing date on July 11, 2024, which was cancelled due to technical difficulty. HCA staff understand you are not making changes to the materials that you intend to present; however, if you wish to make any further submissions, any additional material will be required to be submitted by August 28, 2024, to enable the Board members time to review the material along with the staff report.

This Hearing is governed by the provisions of the *Statutory Powers Procedure Act*. Under the Act, a witness is automatically afforded a protection that is similar to the protection of the *Ontario Evidence Act*. This means that the evidence that a witness gives may not be used in subsequent civil proceedings or in the prosecutions against the witness under a Provincial Statute. It does not relieve the witness of the obligation of this oath since matters of perjury are not affected by the automatic affording of the protection. The significance is that the legislation is Provincial and cannot affect Federal matters. If a witness requires protection of the *Canada Evidence Act*, that protection must be obtained in the usual manner.

The Ontario Statute requires the tribunal to draw this matter to the attention of the witness as this tribunal has no knowledge of the effect of any evidence that a witness may give.

If you do not attend at this Hearing, the Board of Directors of the Hamilton Conservation Authority may proceed in your absence, and you will not be entitled to any further notice of proceedings.

Please contact the undersigned at ext. 133 at this office if you have any questions regarding this matter.

Yours truly,

Mike Stone MCIP, RPP
Manager, Watershed Planning Services

Enclosures: Hamilton Conservation Authority Hearing Guidelines
Hamilton Conservation Authority Hearing Report

APPENDIX B

Hearing Procedures

1. Motion to sit as Hearing Board.
2. Roll Call followed by the Chairperson's opening remarks. For electronic hearings, the Chairperson shall ensure that all parties and the Hearing Board are able to clearly hear one another and any witnesses throughout the hearing.
3. Staff will introduce to the Hearing Board the applicant/owner, his/her agent and others wishing to speak.
4. Staff will indicate the nature and location of the subject application and the conclusions.
5. Staff will present the staff report included in the Authority/Executive Committee agenda.
6. The applicant and/or their agent will present their material
7. Staff and/or the conservation authority's agent may question the applicant and/or their agent if reasonably required for a full and fair disclosure of matters presented at the Hearing.[1]
8. The applicant and/or their agent may question the conservation authority staff and/or their agent if reasonably required for full and fair disclosure of matters presented at the Hearing.[2]
9. The Hearing Board will question, if necessary, both the staff and the applicant/agent.
10. The Hearing Board will move into closed session for deliberation. For electronic meetings, the Hearing Board will separate from other participants for deliberation.
11. Members of the Hearing Board will move and second a motion.
12. A motion will be carried which will culminate in the decision.
13. The Hearing Board will move out of closed session. For electronic meetings, the Hearing Board will reconvene with other hearing participants.
14. The Chairperson or Acting Chairperson will advise the owner/applicant of the Hearing Board decision, including providing the Board's reasons for the decision for approval or refusal.
15. If decision is "to refuse" or "approve with conditions", the Chairperson or Acting Chairperson shall notify the owner/applicant of his/her right to appeal the decision to the Ontario Land Tribunal within 30 days of receipt of the reasons for the decision.
16. Motion to move out of Hearing Board and sit as the Board of Directors.

[1, 2] As per the *Statutory Powers Procedure Act* a tribunal may reasonably limit further examination or cross-examination of a witness where it is satisfied that the examination or cross-examination has been sufficient to disclose fully and fairly all matters relevant to the issues in the proceeding.

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Hamilton Conservation Authority

A Healthy Watershed for Everyone

Hearing Report

TO:	**Board of Directors**
FROM:	**Lisa Burnside, Chief Administrative Officer (CAO)**
RECOMMENDED BY:	**T. Scott Peck, MCIP, RPP, Deputy Chief Administrative Officer/Director, Watershed Management Services**
PREPARED BY:	**Mike Stone, MCIP, RPP, Manager, Watershed Planning, Stewardship & Ecological Services** **Elizabeth Reimer, Conservation Planner, Watershed Planning, Stewardship & Ecological Services**
DATE:	**September 12, 2024**
RE:	**Hearing under Section 28.1(5) of the *Conservation Authorities Act* for an Application by Sayed Shakour for Development in a Regulated Area of Lake Ontario at 10 Lakeside Drive, City of Hamilton (Stoney Creek) – Permit Application No. SC/F,C,A/24/40**

STAFF RECOMMENDATION

THAT HCA staff recommends to the Board of Directors:

THAT the Board of Directors refuse the application made by Sayed Shakour for the construction of a second storey addition in a regulated area of Lake Ontario at 10 Lakeside Drive, City of Hamilton (Stoney Creek), as the development does meet the requirements of the *Conservation Authorities Act, R.S.O. 1990* for development activity in a regulated area.

BACKGROUND

Site Description

The property at 10 Lakeside Drive is a 0.05 ha (0.13 ac) property located on the north side of Lakeside Drive adjacent to the Lake Ontario shoreline (Attachment A). The lot is more or less rectangular, ±40 m deep, and ±12 m wide. The lot contains an existing residential dwelling, approximately 100 m^2 (1076 sq ft), plus a car port.

The property is regulated pursuant to *Ontario Regulation 41/24 (Prohibited Activities, Exemptions and Permits)* and the *Conservation Authorities Act, R.S.O. 1990* due to the proximity of Lake Ontario and its associated flooding and erosion hazards.

The Proposal

The subject application proposes to construct a second storey addition above the existing house, and to construct a new shore wall on the lot (see Site Plan in Attachment C).

Application Review to Date

The applicant first contacted HCA staff in July, 2022, inquiring about the applicable regulations, as the owner was proposing to either demolish the existing house and construct a new house, or construct a second floor on the existing house. Staff replied that the property is affected by the flood and erosion hazards associated with Lake Ontario, and that development is not permitted within the hazards, and that side yard access must be provided.

In February, 2023, HCA staff reviewed a coastal assessment prepared by Ahydtech, as well as plans for a proposed garage in the location of the carport. A second storey was proposed above the existing structure, as well as the proposed garage. HCA advised that this would not meet HCA policy, as it would further reduce the shoreline access for the property. The applicant advised that opportunities for access were already severely limited for the property, as elements of the septic system are within the current access. After discussion between the applicant, the coastal engineer, and HCA staff, the proposal was revised to maintain the existing access. The access is not sufficient for heavy equipment required for construction of a new shorewall, but may provide access for smaller routine repairs.

In September, 2023, a revised report was submitted. HCA responded in October, 2023, identifying several concerns with the proposal. The coastal assessment identified that the height of the wall could be reduced from 78.5 m to 78 m, because the presence of the groyne would reduce wave uprush. HCA responded that this plan should be approved by MNRF, and if MNRF was not supportive of reinforcing the groyne that the shorewall should be designed to protect the rear yard from flooding at an elevation of 78.5 m. In addition, HCA requested that construction access be confirmed. HCA requested that information be provided to address the potential for flank erosion from the adjacent property. The distance from the shore protection at 8 Lakeside Drive to the corner of the house at 10 Lakeside Drive is approximately 9 m, which poses a risk to the existing house.

In April, 2024, HCA received a copy of the authorization from MNRF for the work to the shorewall, including the reinforcement of the existing groyne. HCA staff reviewed the information provided, and advised that the reinforcement of the groyne should be rested below the scour depth to prevent the armoring from becoming undermined.

In May, 2024, revised drawings were submitted, and HCA staff advised that our technical comments relating to the shore protection had been satisfied. Staff further advised that the proposed development was within the shoreline hazard, and as such the application would not be supported by HCA policy. The applicant requested to have the proposal reviewed at a Hearing in front of the HCA Board of Directors, and accordingly submitted a completed permit application form and a final proposed site plan on June 9, 2024.

HCA staff provided information to the applicant that the permit application submission for the proposed addition was deemed complete but could not be supported by staff given the proposal did not conform to policy. In accordance with *Conservation Authority Act Hearing Guidelines* (MNRF October 2005, Amended 2021) and the *Hamilton Region Conservation Authority Administrative By-law* (HCA, Amended October 5, 2023), HCA provided the Notice of Hearing to the applicant, as well as a copy of this Hearing Report, which outlines HCA staff's analysis of the application and reasons for recommending refusal, on August 6, 2024.

STAFF COMMENT

<u>Applicable Policy</u>

HCA has a mandate to ensure that people and property are protected from impacts associated with natural hazards. The Province has delegated the authority for representing and implementing the provincial interest in natural hazards to Conservation Authorities. In evaluating the subject application, HCA staff must ensure that Provincial and HCA policies regarding development and hazardous lands are considered and met. The following outlines the key provincial and HCA hazard policies relevant to the subject application.

Provincial Policy

The Provincial Policy Statement (PPS) provides policy direction on matters of provincial interest related to land use planning and development. The PPS provides a policy framework for allowing appropriate development, while protecting resources of provincial interest, conserving the natural and built environment, and ensuring public health and safety.

With respect to hazards, the PPS states that development shall generally be directed to areas outside of hazardous lands, including hazardous lands adjacent to the shorelines of the Great Lakes, which are impacted by flooding and erosion hazards (PPS 3.1.1). Notwithstanding these restrictions, development may be permitted in those portions of hazardous lands where the effects and risks to public safety are minor and can be mitigated in accordance with provincial standards, and new hazards are not created or existing hazards aggravated (PPS 3.1.7).

HCA Policy

In accordance with *Ontario Regulation 41/24 (Prohibited Activities, Exemptions and Permits)* and the *Conservation Authorities Act, R.S.O. 1990*, no person shall undertake development in a regulated area without permission from the HCA. HCA may grant permission (issue a permit) for development in a regulated area if, in its opinion, the activity is not likely to affect the control of flooding, erosion, dynamic beaches or unstable soil or bedrock, and the activity is not likely to create conditions or circumstances that, in the event of a natural hazard, might jeopardize the health or safety of persons or result in the damage or destruction of property.

HCA's *Planning & Regulation Policies and Guidelines*, as approved by the HCA Board of Directors in October 2011, were developed to support the administration of HCA's Regulation (*Ontario Regulation 161/06)* and to implement provincial policy (PPS) direction, including provincial natural hazard policies. In addition, the HCA board recently approved the *Interim Policy Guidelines for the Administration and Implementation of Ontario Regulation 41/24 (Prohibited Activities, Exemptions and Permits)* to comply with the current legislation and regulations. HCA applies these policies to its review of planning and regulation proposals.

HCA policies generally do not permit development within the shoreline hazard limits associated with Lake Ontario. The shoreline hazard limit is the furthest landward extent of the combined flooding hazard, erosion hazard, and dynamic beach hazard. The following policies are particularly relevant to the subject application.

2.2.1.1. Flooding Hazard Limits

b. For the Lake Ontario shoreline, excluding Hamilton Harbour, the flooding hazard limit has been determined to be 78.5 m IGLD 1955 (International Great Lakes Datum). This elevation includes the 100-year flood level (76.0 m IGLD) plus the wave action and other water-related hazards (2.5 m) [Great Lakes-St. Lawrence River System and Large Inland Lakes Technical Guides (MNR & Watershed Science Centre, 2001) and Lake Ontario Waterfront Study, Stoney Creek (F.J. Reinders and Assoc. and Conroy Dowson Planning Consultants Inc., March 1980)].

2. 2. 1. 2 Erosion Hazard Limits

Where Authority staff consider development proposals and/or site alterations in or on the areas adjacent or close to the Lake Ontario shoreline the erosion hazard limit shall be applicable.

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.

2. 2. 2 Development

b. The Authority will generally direct development to occur outside of hazardous lands adjacent to the Lake Ontario shoreline that are impacted by flooding and/or erosion, unless the following conditions are met:
    ii. The hazards can be safely addressed, and the development and/or site alteration is carried out in accordance with floodproofing standards, protection works standards, and access standards;
    iii. Vehicles and people have a way of safely entering and exiting the area during times of flooding, erosion and other emergencies;
    iv. New hazards are not created and existing hazards are not aggravated; and
    v. No adverse environmental impacts will result.

2. 2. 2. 1 Shoreline Protection Works

a. Where shoreline protection works are proposed the applicant must meet the following requirements:

    i. The purpose of the proposed works must be clearly defined;
    ii. Shoreline works must be designed for the *100 year flood level*, *wave uprush*, and according to accepted scientific coastal engineering principles, where viable;
    iii. The works must be designed and/or approved by a professional engineer with experience and qualifications in coastal engineering;
    iv. Slope stability must be assessed by a professional engineer with experience and qualifications in coastal/geotechnical engineering;
    v. The ownership of land, where the protection works are proposed, must be clearly established by the applicant;
    vi. The design and installation of protection works must allow for access to and along the protection works for appropriate equipment and machinery for regular maintenance purposes and/or to repair the protection works should failure occur;
    vii. The works will not aggravate existing hazards and/or create new hazards at updrift/downdrift properties;

viii. In areas of existing *development*, protection works should be coordinated with adjacent properties, where possible; and
ix. The *Authority* requires that the protection works incorporate a minimum *erosion access allowance* of 6 m, where possible, and that the *erosion access allowance* permit access from a municipal roadway to and along the shoreline protection works for regular maintenance purposes and/or to repair the protection works, where possible. Side yard access allowances may be shared between adjacent landowners provided that the shared easement is registered on title.

b. The *Authority* will generally not support shoreline protection works that:

i. Do not consider natural coastal processes;
ii. Are not effective against long-term erosion;
iii. Do not preserve cobble/shingle beaches;
iv. Do not protect/regenerate aquatic and terrestrial habitat); and
v. Negatively impact neighbouring shorelines.

c. Where shoreline protection works exist, the *Authority* may request that the integrity of that protection works be assessed by a qualified coastal engineer, at the expense of the proponent, and any recommendations for improvement be incorporated into the *development* proposal.

<u>Application Assessment</u>

The property at 10 Lakeside drive is affected by shoreline hazards associated with Lake Ontario. The crest of the proposed shore protection structure is at 78.0 m. HCA policies recommend that properties be protected from flooding by construction shore protection to a height of 78.5m, which incorporates the 100-year lake level of 76.0 m, plus a wave uprush of 2.5 m. The report by Ahydtech identifies that the wave uprush will be reduced by the presence of the existing groyne (which will be reinforced as part of the proposed work), suggesting the property will be protected from the flooding hazard associated with Lake Ontario with the construction of the new shore wall. However, the rear of the property is subject to erosion hazards associated with the lake.

The erosion hazard was more specifically reviewed in erosion hazard assessments prepared by Ahydtech. The erosion hazard setback assumes that the property will be protected with a structure having a design life of 50 years (Attachment B). HCA staff have reviewed a proposed shore wall design prepared by Ahydtech (Attachment E), and are satisfied that the design is satisfies HCA polices and technical requirements, and accept the professional engineer’s opinion that the shore protection will have a 50-year design life.

The report prepared by Ahydtech identifies the erosion hazard extending 10 m from the stable top of lake bank (Attachment B). The coastal assessment indicates that a 0.2 m/yr recession rate may be applied to the property. Generally, HCA applies a recession rate of 0.3 m/yr to the Lake Ontario shoreline. If a 0.3 m/yr recession rate is assumed, the 10 m proposed erosion setback would not be sufficient, even if it is assumed that a shorewall with a 50-year design life is constructed.

In reviewing the Provincial technical guidance, staff note that the Technical Guide for Great Lakes – St Lawrence River Shorelines, Appendix A7.2 prepared by the Ontario Ministry of Natural Resources provides guidance for existing development within hazardous lands. More specifically, Table A7.2.1 indicates that major additions to structures on existing developed lots may be permitted, provided:

1) It meets requirements of the Protection Works Standard and the Access Standard to the maximum extent and level possible based on site-specific conditions; and,
2) It utilizes maximum lot depth and width; and,
3) As a minimum, uses the greater of a) erosion allowance based on planning horizon of not less than 50 years or, b) minimum setback from stable slope allowance of 15 m; and,
4) It does not increase the occupancy of existing structure; and,
5) It does not diminish maintenance access to any existing protection works.

Notwithstanding the coastal engineer's recommendation that a 10 m setback is appropriate, based on the above, the addition would not be permitted, as it does not meet the third criterion, as it is not a minimum of 15 m from the stable lake bank.

Concerns remain regarding the hazard setbacks related to the proposed second storey addition. Information provided in the HCA's new (draft) Shoreline Management Plan suggests that a recession rate of 0.3 m/yr may not be sufficiently conservative. In addition, the shorewall plans propose a tie-in to the existing shorewall at 8 Lakeside Dr, but if the shorewall on the adjacent property becomes damaged, there is still the potential for erosion from the flank. As described above, existing access to the property is constrained. Although there is no change to access relative to the existing conditions, future maintenance and construction on the shore will be a challenge.

HCA policies, as outlined above, do not permit development within the shoreline erosion hazard. HCA policy permits a reduction in the erosion hazard, in recognition that the hazards may be partially mitigated with the construction of adequate shoreline protection works. HCA staff are of the opinion that further reduction in the setback, as proposed by the applicant, would pose a hazard to the proposed development and therefore further reduction is not warranted. In considering the applicable policies, it is HCA staff's opinion the conditions under which a permit can be issued under *Ontario Regulation 41/24 (Prohibited Activities, Exemptions and Permits)* and the *Conservation Authorities Act, R.S.O. 1990)* are not met.

AGENCY COMMENTS

None

CONCLUSION

The subject application proposes development within the erosion hazard associated with Lake Ontario, and provides less side yard access than HCA policy recommends. Provincial and HCA policies take a preventative approach to addressing the potential risks and impacts associated with natural hazards by generally directing development to areas outside of hazardous lands. It is HCA staff's opinion the policy framework outlined in HCA's *Planning and Regulation Policies and Guidelines (October, 2011)* does not support the proposed development.

On this basis, the proposed development does not meet the conditions under which HCA may issue a permit under *Ontario Regulation 41/24 (Prohibited Activities, Exemptions and Permits)* and the *Conservation Authorities Act, R.S.O. 1990*. As such, it is the recommendation of HCA staff that the application be refused.

Attachment A – Site Location

10 Lakeside Drive, City of Hamilton (Stoney Creek)

Figure 1. Location of property (red rectangle)

Figure 2. Oblique view of shoreline at 10 Lakeside Dr

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Attachment B – Erosion Hazard Delineation
Prepared by Dr. Bahar SM, Ahydtech

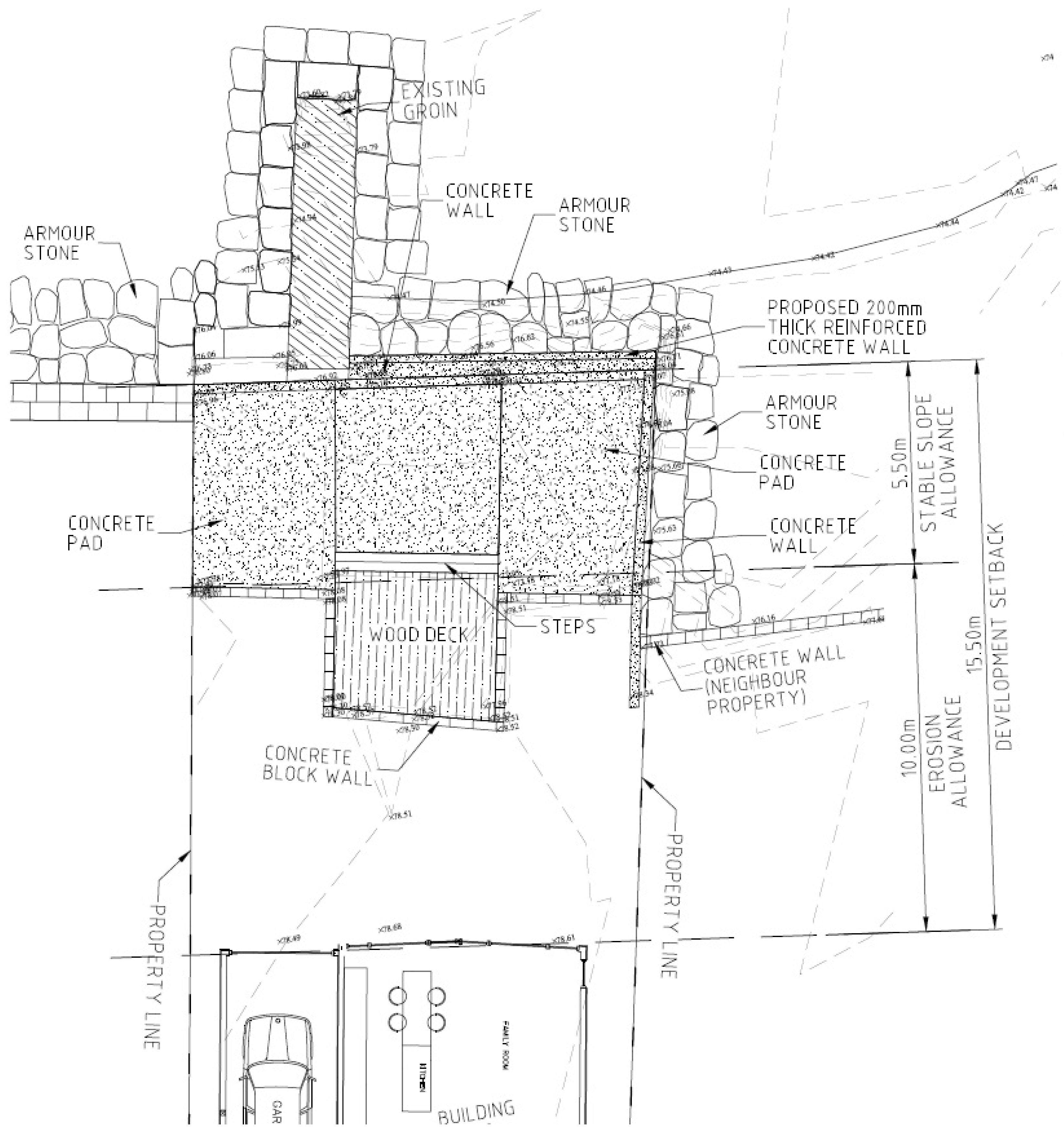

Figure 3. Erosion hazard associated with Lake Ontario. The 10.00 m erosion allowance presumes a recession rate of 0.2 m/yr.

Attachment C – Site Plan

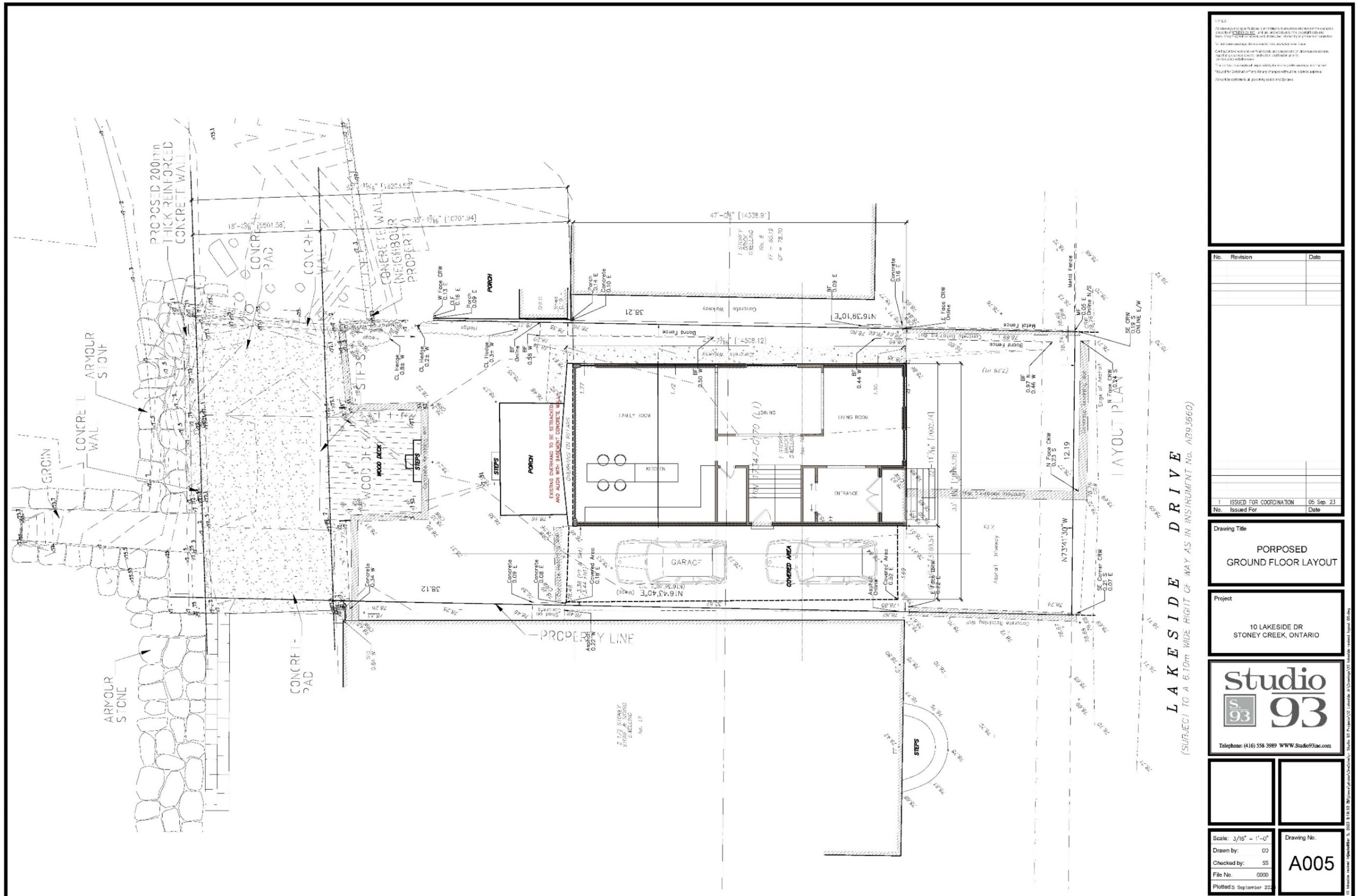

Figure 4. Proposed site plan.

11

Attachment D – Proposed Development

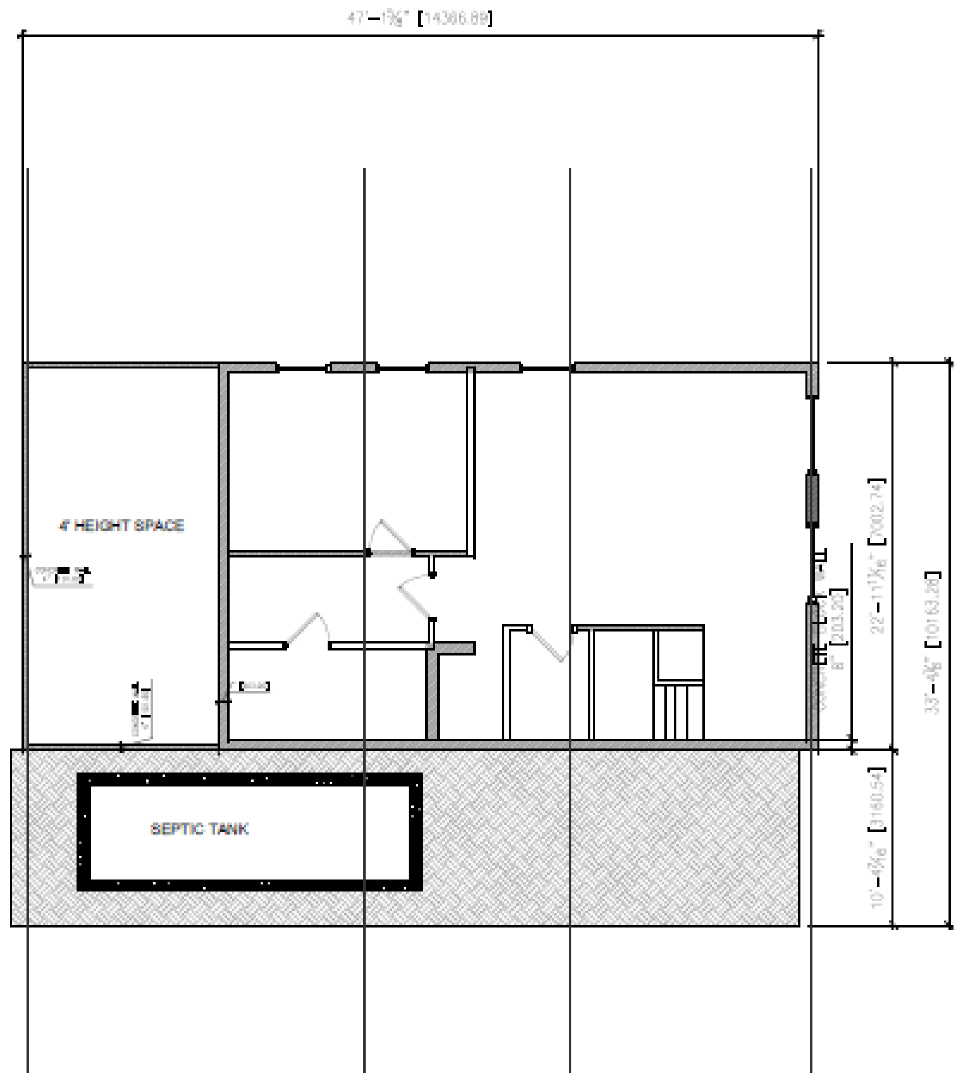

Figure 5. Existing basement.

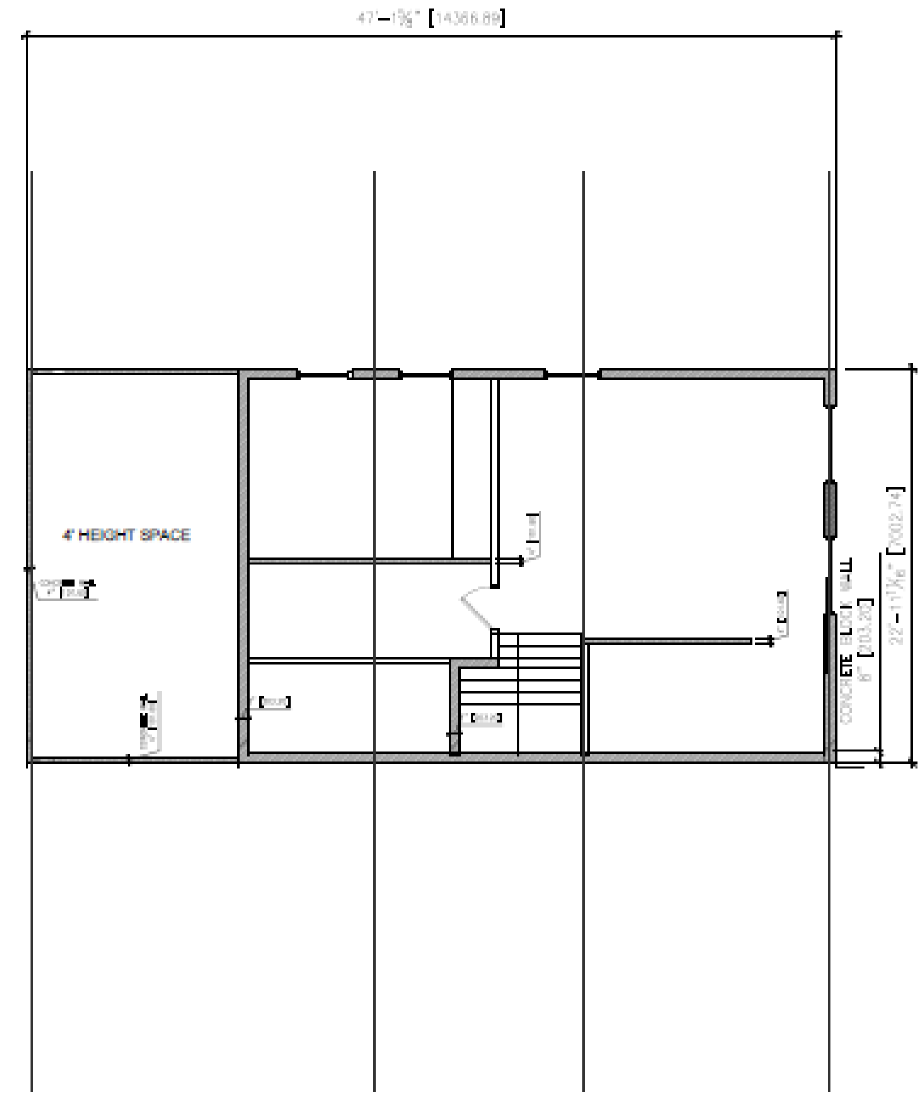

Figure 6. Proposed basement

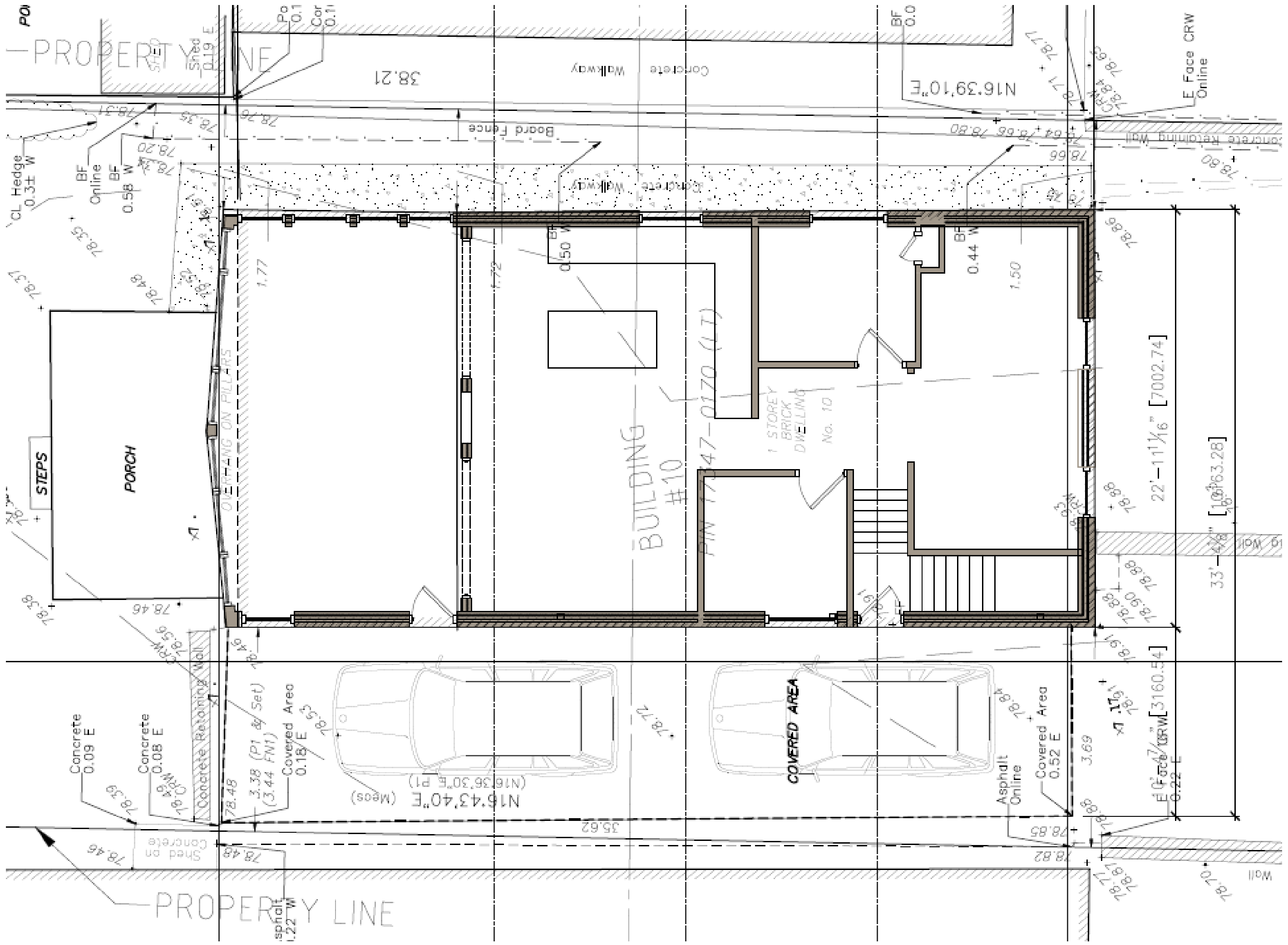

Figure 7. Existing main floor.

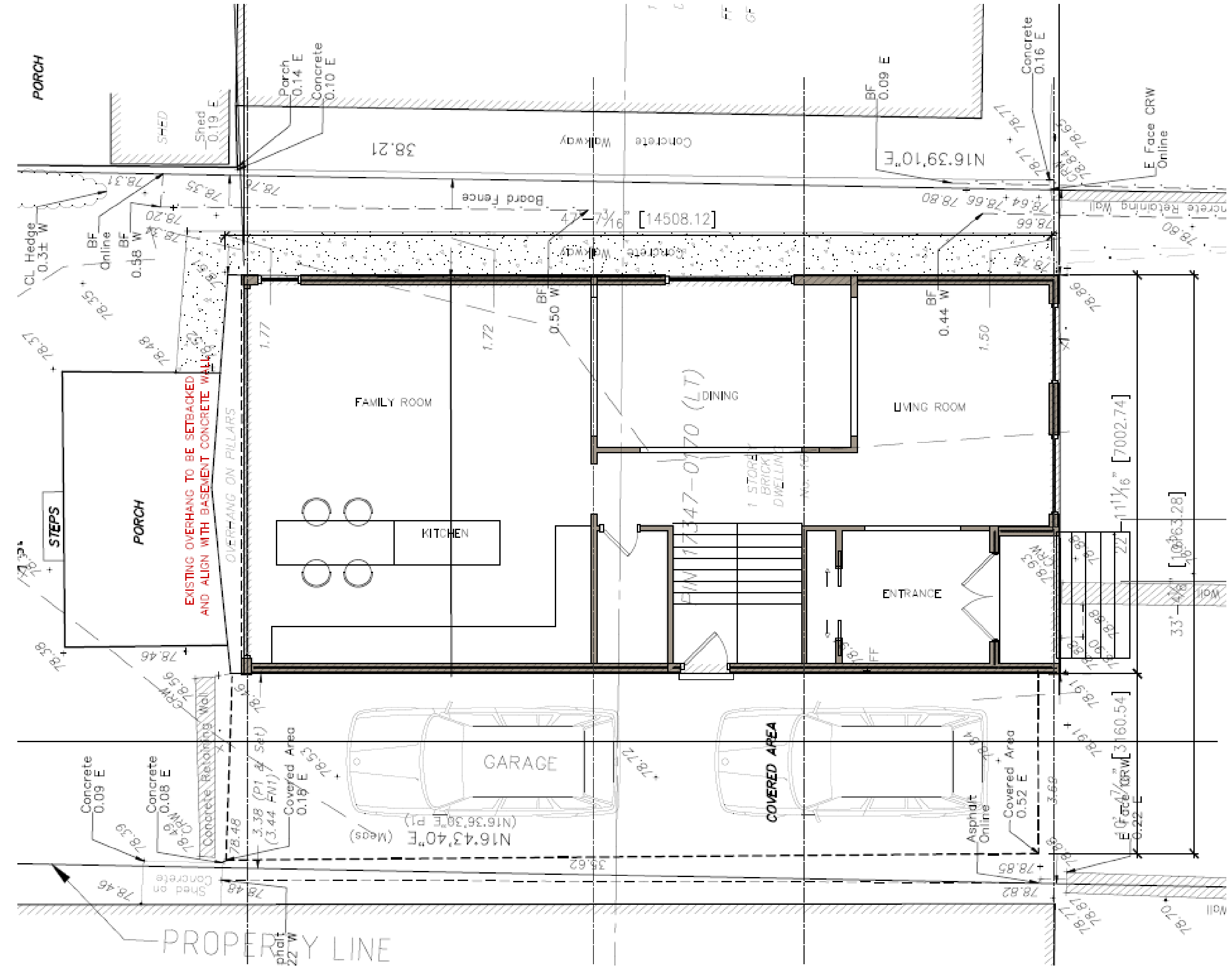

Figure 8. Proposed main floor.

13

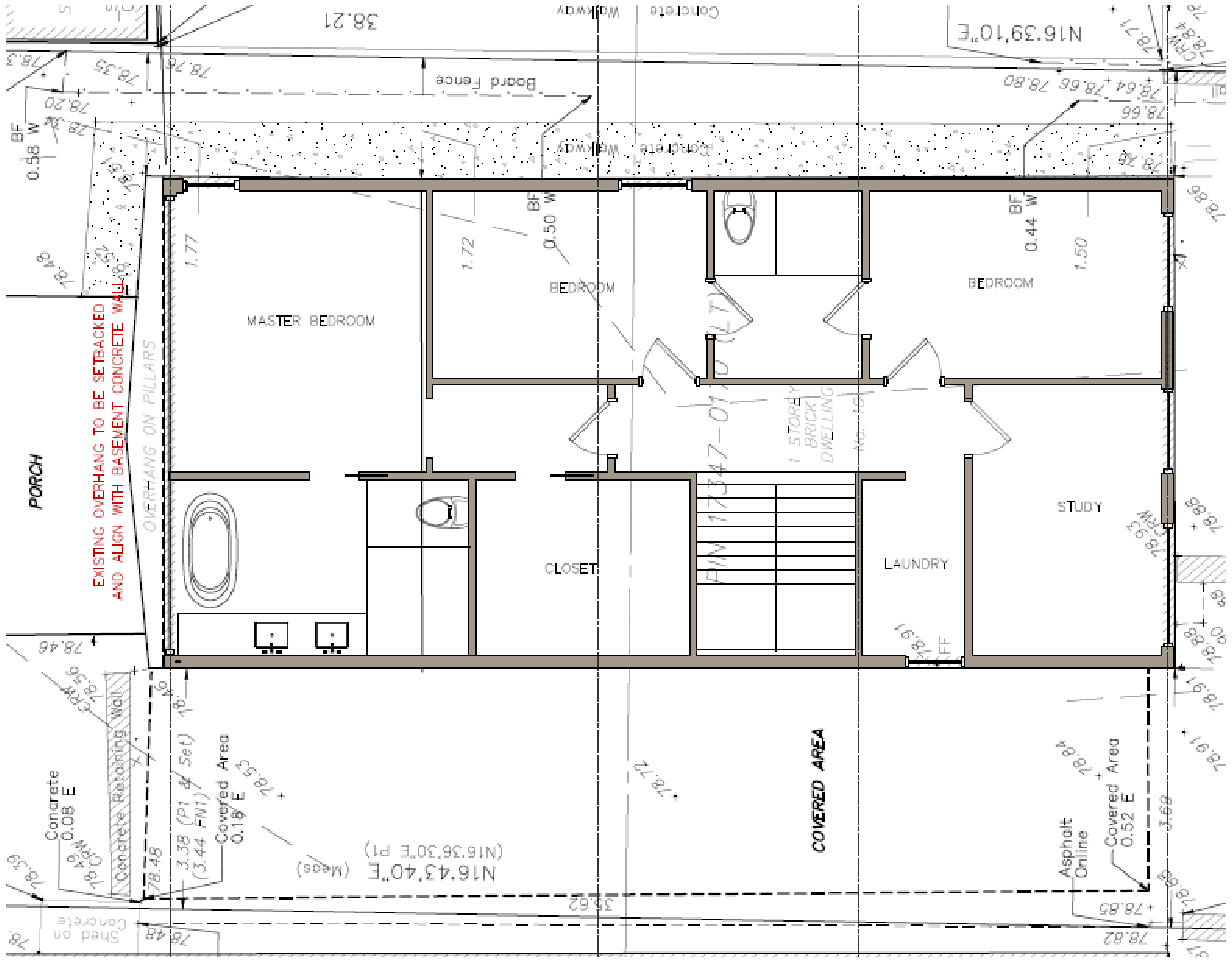

Figure 9. Proposed new second floor.

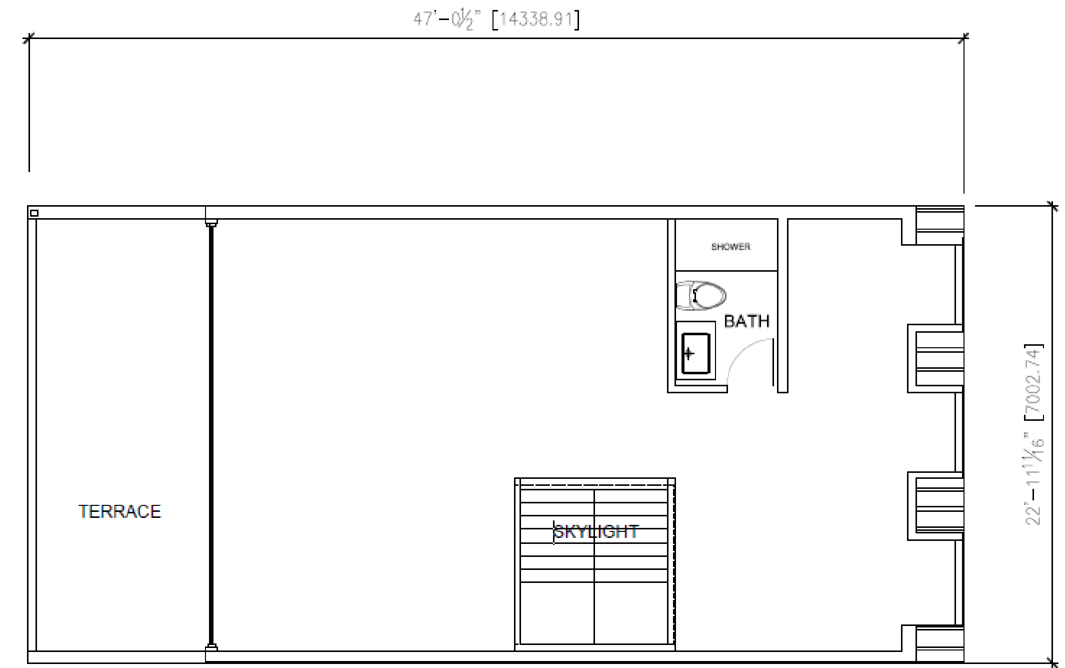

Figure 10. Proposed new attic level.

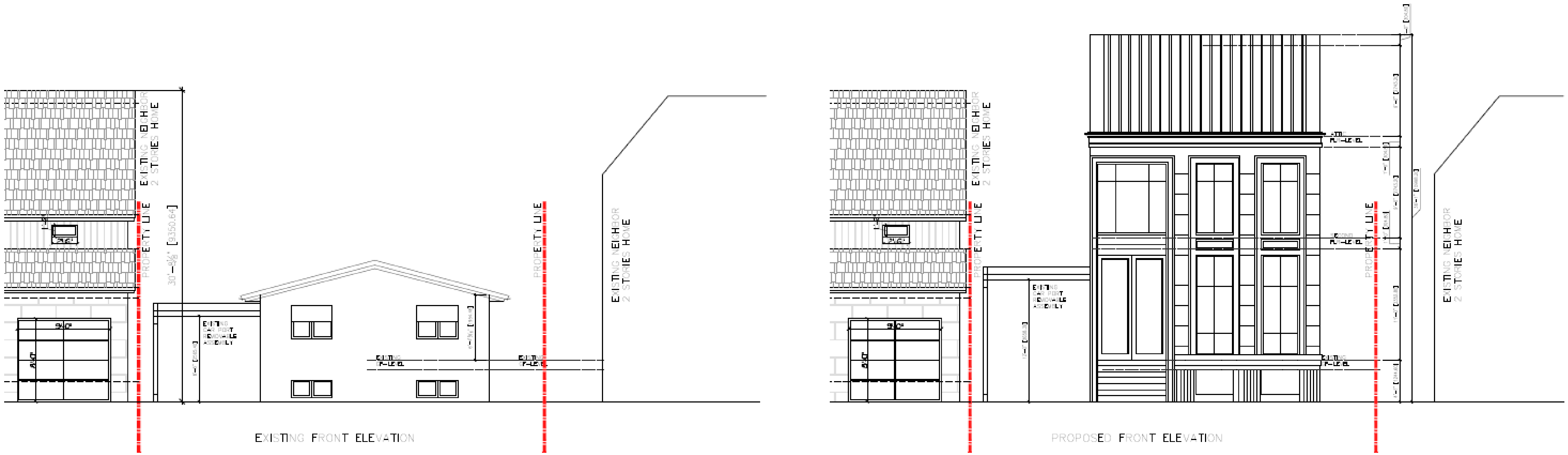

Figure 11. Existing and proposed elevations.

Attachment E Shore Protection Design

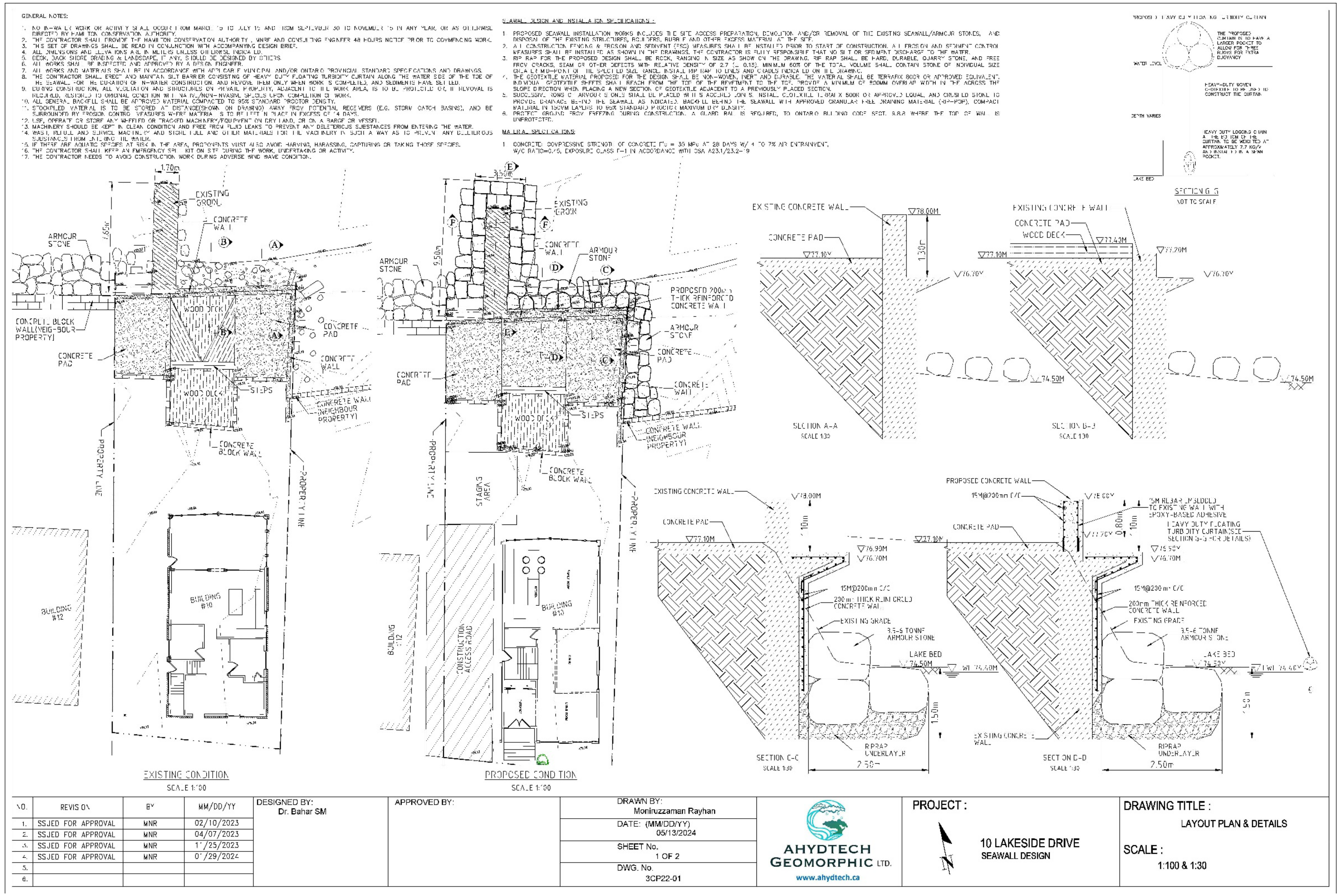

Figure 12. Proposed shorewall design.

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Hamilton Conservation Authority

6.3.1

Coastal Engineering Analysis, Hazard Limit Delineation & Seawall Design

at

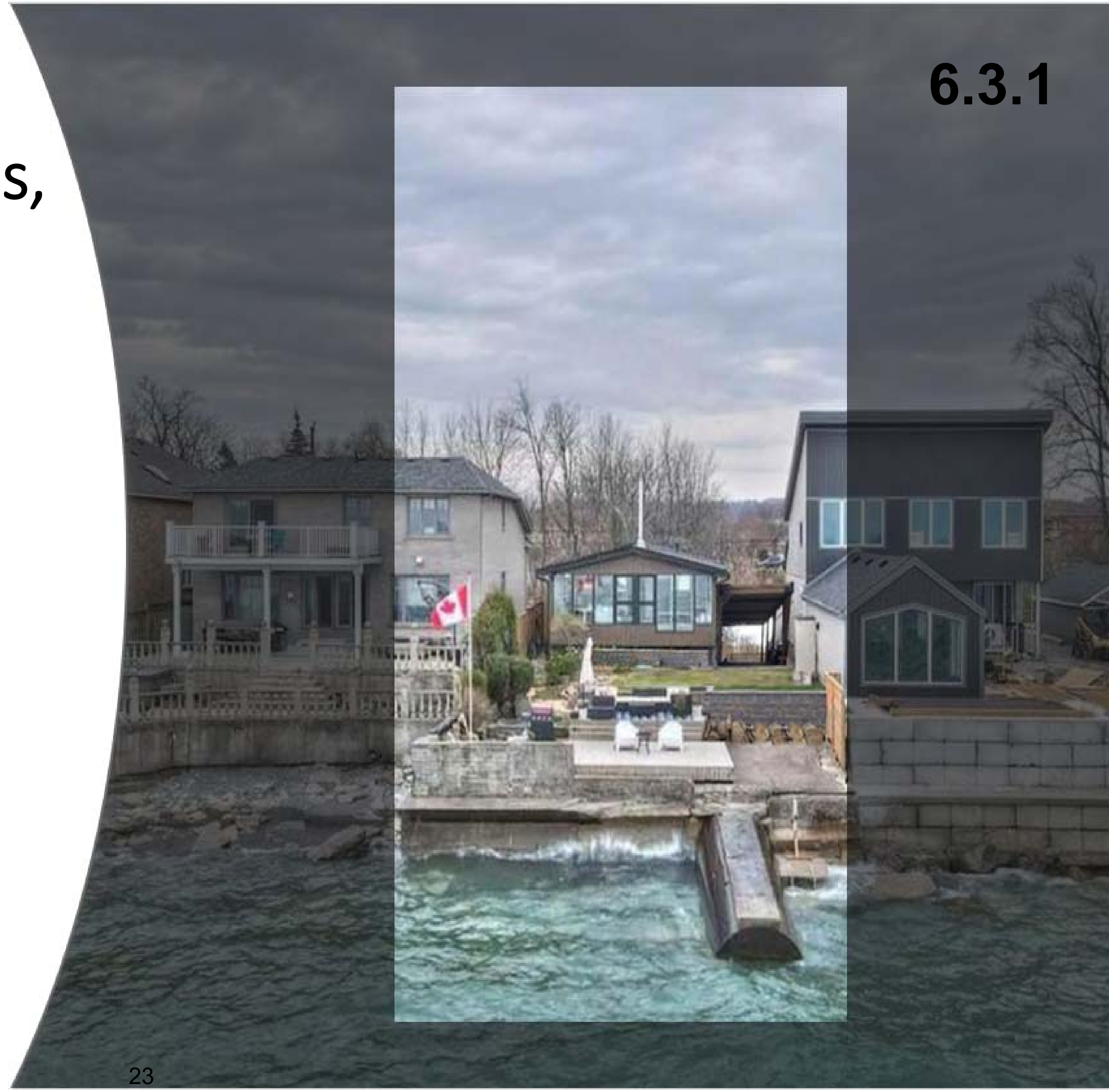
10 Lakeside Drive, Stoney Creek, Ontario

AHYDTECH GEOMORPHIC

July 11, 2024

Hearing under Section 28.1(5)
of the
Conservation Authorities Act
Development in a Regulated Area of Lake Ontario

Application by Sayed Shakour
Permit Application No. SC/F,C,A/24/40



BACKGROUND OF THE STUDY

Existing Condition Observed During Field Investigation:

Artificial Shoreline

Existing Concrete Seawall	Existing Steel Barrel Groin

- Exposed Reinforcements
- Scour at Toe
- Fails preventing flood water from overtopping

Cobble Stone Beach at the Toe

AHYDTECH GEOMORPHIC
ADVANCED HYDROLOGY HYDRAULIC GEOMORPHOLOGY

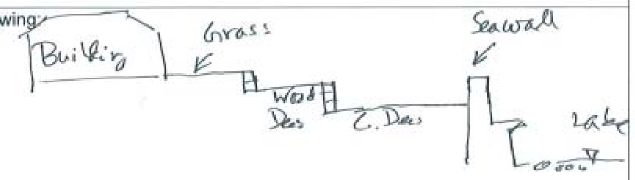
Shoreline Characteristics/Assessment

Date: Nov. 25 2022 Weather: Sunny Crew: Bahar
Reach: Water Body: Lake Ontario
Location: 10 Lakeside Drive Stoney Creek Project Code/Phase:

Shoreline Planform Drawing;

See Next Page

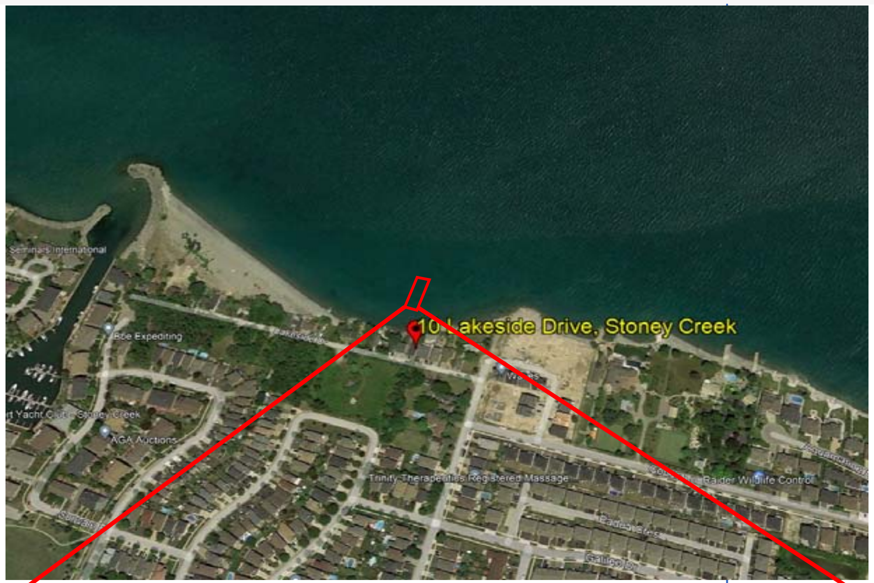
Shoreline Cross-shore Drawing:



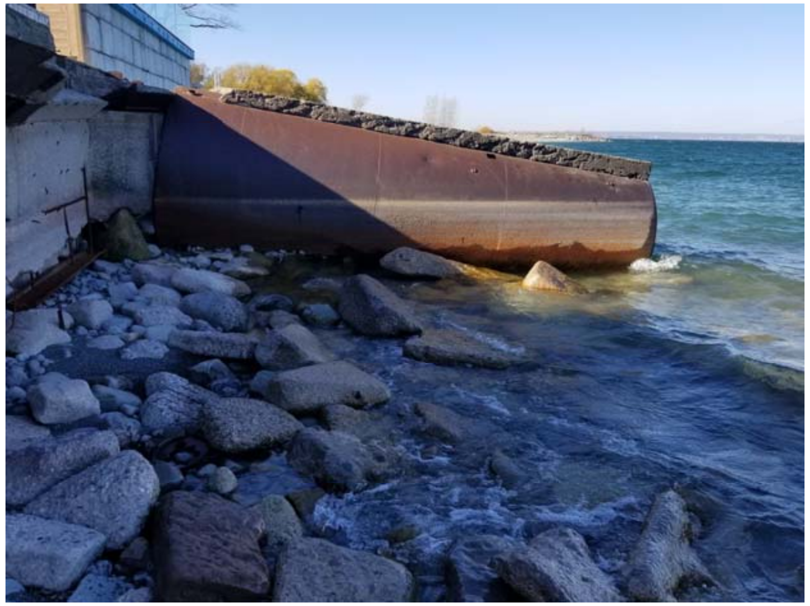
Composition	Vegetation:
Bluff/cliff:	Armouring:
Beach: Cobble, stone	Bluff/cliff height:
Nearshore: Seawall / broken	Bluff Strength
Exposure and Planform	Totvane
Headland Bay Partial Headland Exposed	Penetrometer

General Shoreline Character	Nearshore Controlling Surface	Surficial Substrate
Artificial Existing Seawall	Sand	Sand
Natural	Silt/clay	Silt/organic
Bedrock Cliff	Bedrock	Bedrock
Bedrock Low Plain	Cobble/boulder Till	Cobble/boulder
Cohesive/Non-cohesive Bluff	Fine Grained Cohesive	Gravel/cobble/boulder
Cohesive/Non-cohesive Low Bluff	Gravel/cobble/boulder stone	
Dynamic Beach Backed by Cliff/Bluff	Concrete Seawall &	
Dynamic Beach Low Plain	Steel barrel groin	
Dynamic Beach Barrier		

Cliff/bluff: steeper than 1:3 and >2m high
Low plain: landward slope flatter than 1:3 or, 2m high
Dynamic beach



FIELD PHOTOS



Condition of Existing Steel Barrel Groin in front of 10 Lakeside Drive Property

Condition of Existing Concrete Seawall in front of 10 Lakeside Drive Property

Evidence of Exposed Reinforcements and Scour at Toe

Condition of Neighbouring Shore with an Existing Shoreline Structure at 8 Lakeside Drive

Condition of Neighbouring Seawall at 12 Lakeside Drive

Hamilton Conservation Authority

AHYDTECH GEOMORPHIC

SCOPES
OF THE STUDY

Field Investigation

- Topographic Surveying
- Shoreline Characterization

Coastal Analysis

- Analysis of Wind-Wave Environment
- Seawall Design

Shoreline Hazard Limit Delineation

- Average Annual Recession and Erosion Allowance
- Stable Slope Allowance
- Development Setback

Hamilton Conservation Authority

AHYDTECH GEOMORPHIC

1.

COMMENTS ON APPLICATION ASSESSMENT

By HCA Staff Provided in *Hearing Report* on June 14, 2024

HCA policies recommend that properties be protected from flooding by construction shore protection to a height of 78.5m, which incorporates the 100-year lake level of 76.0 m, plus a wave uprush of 2.5 m. The report by Ahydtech identifies that the wave uprush will be reduced by the presence of the existing groyne (which will be reinforced as part of the proposed work), suggesting the property will be protected from the flooding hazard associated with Lake Ontario with the construction of the new shore wall. However, the rear of the property is subject to erosion hazards associated with the lake

Hamilton Conservation Authority

AHYDTECH GEOMORPHIC

PREVIOUS DISCUSSION ON HEIGHT OF SHORE PROTECTION STRUCTURE

Comments on Technical Report of 10 Lakeside Drive Regarding Hazard Assessment by HCA Staff - Elizabeth Reimer on March 2, 2023	**Responses Made by AHYDTECH Geomorphic Ltd.** on March 21, 2023
• It is not clear why a 78.00 m flood hazard limit is recommended, when the assessment for 12 Lakeside Drive recommends using the elevation of 78.50 m as adopted by HCA for the entire Stoney Creek Shoreline. The shorewall should protect the rear yard from flooding up to an elevation of 78.5m. - Email on "10 Lakeside Drive Seawall Repair and Hazard Limit Delineation" Sent by Elizabeth Reimer (Conservation Planner, Hamilton Conservation Authority) on Thursday, March 2, 2023 2:12 PM ***(Annex – II)***	• AHYDTECH followed the MNR Provincial Guidelines and found the **100-year flood level** for the property site to be **76.00 m in IGLD 1985** Datum. There is an existing steel barrel groin at the west of the shoreline of 10 Lakeside Drive property. Due to the presence of an additional shore protection structure (Steel Barrel Groin) at the west of the shoreline, the waves will overtop at a lower level. Based on the present shoreline condition of the property site, the total crest elevation required for the proposed seawall has been recommended to be 78.0m.

Condition of Existing Steel Barrel Groin and Seawall at Neighbouring Property

Hamilton Conservation Authority

AHYDTECH GEOMORPHIC

2.

COMMENTS ON APPLICATION ASSESSMENT

By HCA Staff Provided in *Hearing Report* on June 14, 2024

HCA staff have reviewed a proposed shore wall design prepared by Ahydtech, and are satisfied that **the design is satisfies HCA polices and technical requirements, and accept the professional engineer's opinion that the shore protection will have a 50-year design life.**

The report prepared by Ahydtech identifies the erosion hazard extending 10 m from the stable top of lake bank. **The coastal assessment indicates that a 0.2 m/yr recession rate may be applied to the property**. **Generally, HCA applies a recession rate of 0.3 m/yr to the Lake Ontario shoreline.** If a 0.3 m/yr recession rate is assumed, the 10 m proposed erosion setback would not be sufficient, even if it is assumed that a shorewall with a 50-year design life is constructed

Hamilton Conservation Authority

AHYDTECH GEOMORPHIC

2.

COMMENTS ON APPLICATION ASSESSMENT

By HCA Staff Provided in *Hearing Report* on June 14, 2024

In reviewing the Provincial technical guidance, staff note that the Technical Guide for Great Lakes – St Lawrence River Shorelines, Appendix A7.2 prepared by the Ontario Ministry of Natural Resources provides guidance for existing development within hazardous lands. More specifically, Table A7.2.1 indicates that **major additions to structures on existing developed lots may be permitted, provided**:

1) It meets requirements of the Protection Works Standard and the Access Standard to the maximum extent and level possible based on site-specific conditions; and,
2) It utilizes maximum lot depth and width; and,
3) As a minimum, uses the greater of a) erosion allowance based on planning horizon of not less than 50 years or, b) minimum setback from stable slope allowance of 15 m; and,
4) It does not increase the occupancy of existing structure; and,
5) It does not diminish maintenance access to any existing protection works.

Notwithstanding the coastal engineer's recommendation that a 10 m setback is appropriate, based on the above, the addition would not be permitted, as it does not meet the third criterion, as it is not a minimum of 15 m from the stable lake bank.

Hamilton Conservation Authority

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Shoreline Hazard Limit Delineation

According to the ***MNRF Provincial Policy Statement*** (i.e., Policy 3.1) and the ***Hamilton Conservation Authority (HCA) Policies and Guidelines***,

Erosion Hazard Limit = Stable Slope Allowance + Erosion Allowance

STABLE SLOPE ALLOWANCE:

- According to the Provincial Standard, the ***3 (Horizontal) : 1 (vertical)*** slope method is required to determine the stable slope allowance, if there is no geotechnical report on slope stability.
- Geotechnical and slope stability investigation of the shoreline has been performed by **SOIL-MAT ENGINEERS & CONSULTANTS LTD.** The study recommends ***2 (Horizontal) : 1 (vertical)*** for the stable slope.

- "Geotechnical Consultation Report for Proposed Seawall Reconstruction at 12 Lakeside Drive, Stoney Creek, Ontario" by SOIL-MAT Engineers and Consultants Ltd. (December 13, 2019) ***(Annex – III)***

- After applying 2 (Horizontal): 1 (vertical) slope, the stable slope allowance will be ***5.5 meters*** from the toe of the lake bed. The vertical distance is measured from the toe of the natural shoreline to the top of the first landward break. Then the horizontal distance is just two times of the vertical distance.

- "Technical Report on Coastal Engineering Analysis, Hazard Limit Delineation and Seawall Design at 10 Lakeside Drive, Stoney Creek, Ontario" by AHYDTECH Geomorphic Ltd. (April 9, 2023) ***(Annex – IV)***

- "Layout Plan & Details Drawing of Development Limit Analysis for 10 Lakeside Drive, Stoney Creek, Ontario by AHYDTECH Geomorphic Ltd. (April 9, 2023) ***(Annex – V, Sheet 2)***

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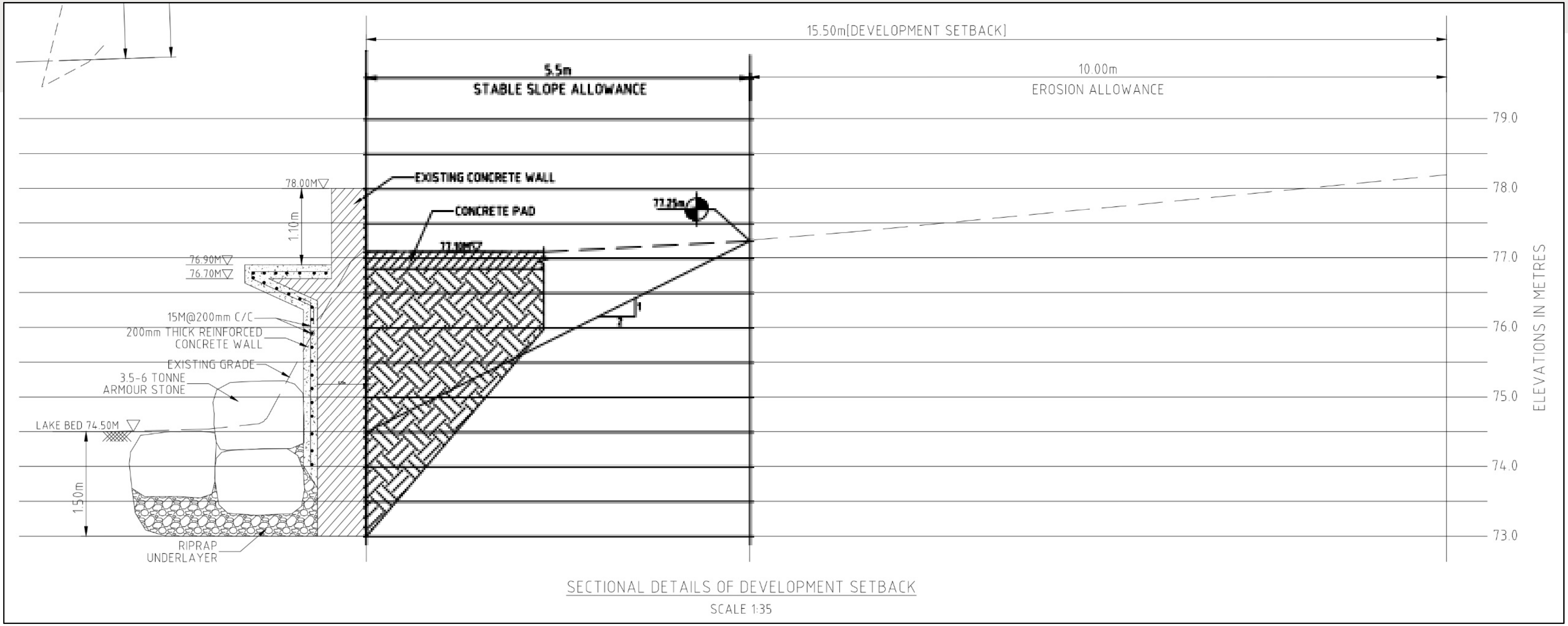
Hamilton Conservation Authority

STABLE SLOPE ALLOWANCE



Section Details of Development Setback at 10 Lakeside Drive Property ***(Annex – V, Sheet 2)***

Hamilton Conservation Authority

AHYDTECH GEOMORPHIC

RECESSION RATE ANALYSIS

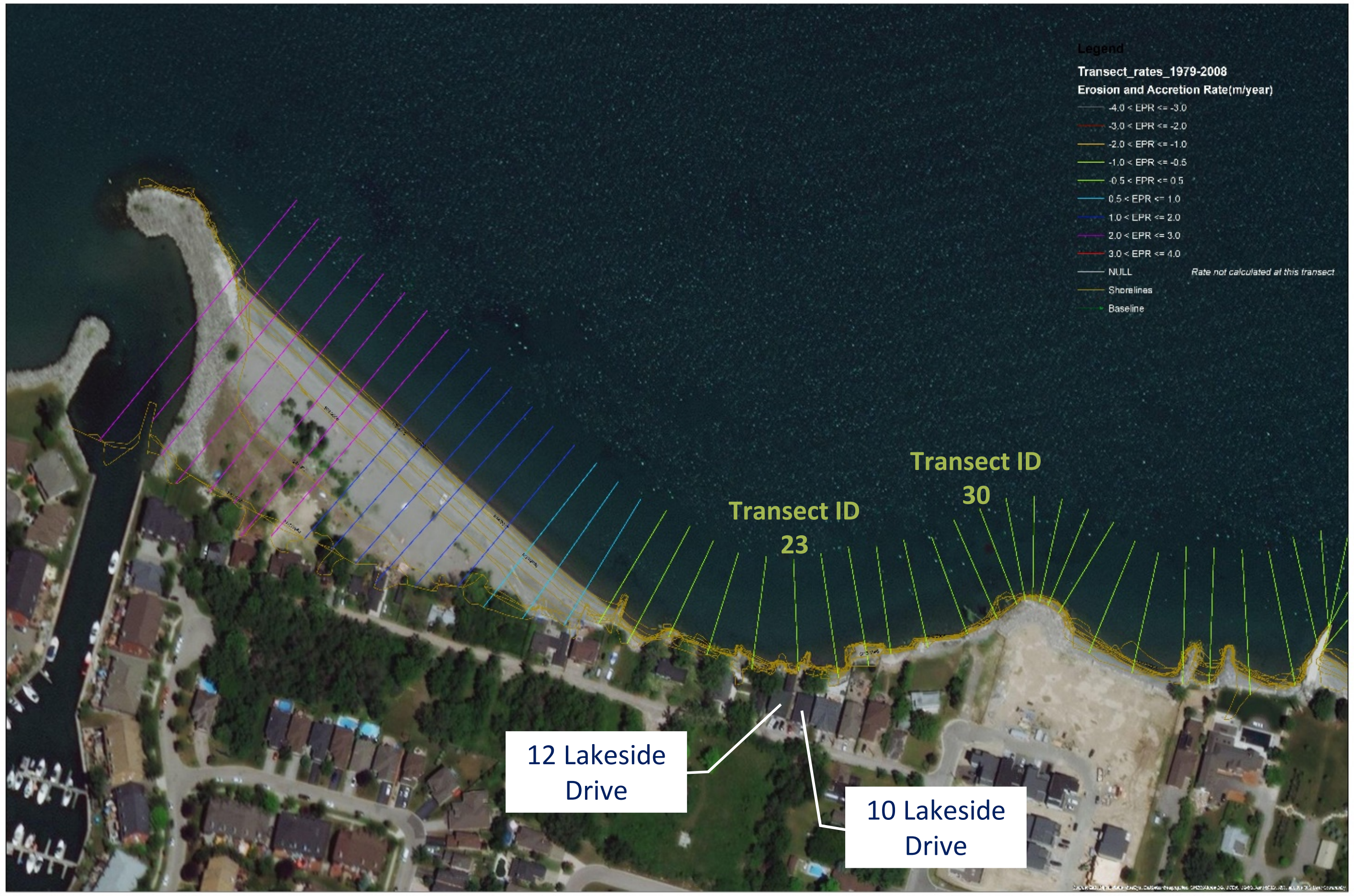
Performed for 12 Lakeside Drive:

- Erosion hazard analysis was performed using the ***Digital Shoreline Analysis System (DSAS)*** in ArcGIS over a 40-year period.
- The analysis indicated an average accretion rate of **+0.55 m/year** along the 2 km shoreline **from 1979 to 2019.**
- Transect ID 23, in front of 12 Lakeside Drive, showed an accretion rate of, suggesting no significant erosion of **+0.04 m/year** over the 40-year period.
- The Maximum Recession Rate observed along the 2 km shoreline was **0.09 m/year.**
- The finally accepted recession rate for 12 Lakeside Drive was **0.2 m/year.**

- Technical Report on Coastal Engineering Analysis and Seawall Design for 12 Lakeside Drive, Stoney Creek, Ontario by AHYDTECH Geomorphic Ltd. (2020) ***(Annex – VI)***

- Plan Drawing on Shoreline Hazard Assessment for 12 Lakeside Drive, Stoney Creek by AHYDTECH Geomorphic Ltd. (2020) ***(Annex – VII)***

12 Lakeside_Shoreline Recession



Transect ID	Recession/Accretion Rate (1979-2019)
23	0.04
24	0.02
25	0.21
26	-0.01
27	-0.02
28	0
29	-0.05
30	-0.09 (Maximum Recession)
31	-0.02

Section 3.3 of *Annex - VI*

Hamilton Conservation Authority

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DETAILS OF RECESSION RATE ANALYSIS

Performed for 12 Lakeside Drive

Transect ID	Recession/Accretion Rate (1979-2019)	Transect ID	Recession/Accretion Rate (1979-2019)
1	2.81	29	-0.05
2	2.15	30	-0.09
3	2.32	31	-0.02
4	2.36	32	0.04
5	2.26	33	0.05
6	2.17	34	0.07
7	2.1	35	0.07
8	2.06	36	0.08
9	1.96	37	0.01
10	1.84	38	0.27
11	1.76	39	-0.02
12	1.48	40	-0.03
13	1.47	41	0
14	1.09	42	-0.02
15	0.85	43	0
16	0.66	44	-0.01
17	0.56	45	-0.01
18	0.24	46	0
19	0.03	47	0.08
20	0.1	48	-0.05
21	0.07	49	0
22	0.06	50	0.04
23	**0.04 (Accretion)**	51	0.01
24	0.02	52	-0.01
25	0.21	53	0
26	-0.01	54	-0.01
27	-0.02	55	-0.02
28	0	56	0
	LONG-TERM AVERAGE		**0.55**

Hamilton Conservation Authority

Section 3.3 of *Annex - VI*

AHYDTECH GEOMORPHIC

PREVIOUS DISCUSSION ON RECESSION RATE ANALYSIS

Applied for 10 Lakeside Drive

Comments on Technical Report of 10 Lakeside Drive Regarding Hazard Assessment by HCA Staff - Elizabeth Reimer on March 2, 2023	**Responses Made by AHYDTECH Geomorphic Ltd.** on March 21, 2023
• HCA staff noted that the accepted recession rate for 12 Lakeside Dr was 0.2 m/yr, which was based on identical input. It is not clear how the 0.09 m/yr value was determined. In order to ensure the most conservative assessment is used, ***either the 0.2m/yr should be applied***, or high-resolution imagery should be provided to confirm that the recession rate of 0.09 m/yr is accurate. - Email on "10 Lakeside Drive Seawall Repair and Hazard Limit delineation" Sent by Elizabeth Reimer (Conservation Planner, Hamilton Conservation Authority) on Thursday, March 2, 2023 2:12 PM ***(Annex – II)***	• AHYDTECH agreed with ensuring the most conservative assessment and applied recession rate of 0.2m/yr while delineating the erosion hazard limit for the property site. The Technical Report and Layout Plan Drawing have been updated accordingly. - Technical Report on Coastal Engineering Analysis, Hazard Limit Delineation and Seawall Design at 10 Lakeside Drive, Stoney Creek, Ontario by AHYDTECH Geomorphic Ltd. (April 9, 2023) ***(Annex – IV)*** - Layout Plan & Details Drawing of Development Limit Analysis for 10 Lakeside Drive, Stoney Creek, Ontario by AHYDTECH Geomorphic Ltd. (April 9, 2023) ***(Annex – V, Sheet 2)***

Applying Annual Recession Rate = 0.2 meters per year

100 Years Erosion Allowance = 0.2 x 100
= 20 meters

Credit After Reconstruction of the Structure = 50%
= 20 x 0.5
= ***10 meters***

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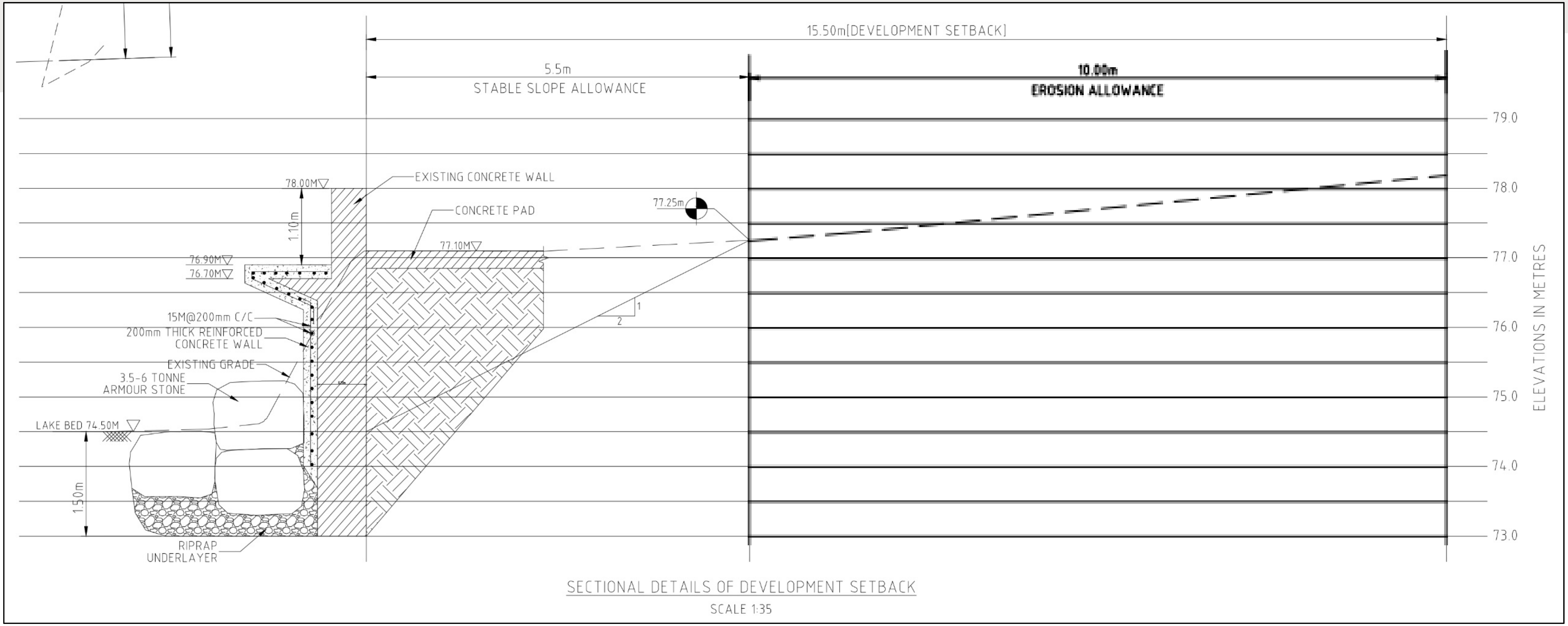
EROSION ALLOWANCE



Section Details of Development Setback at 10 Lakeside Drive Property ***(Annex – V, Sheet 2)***

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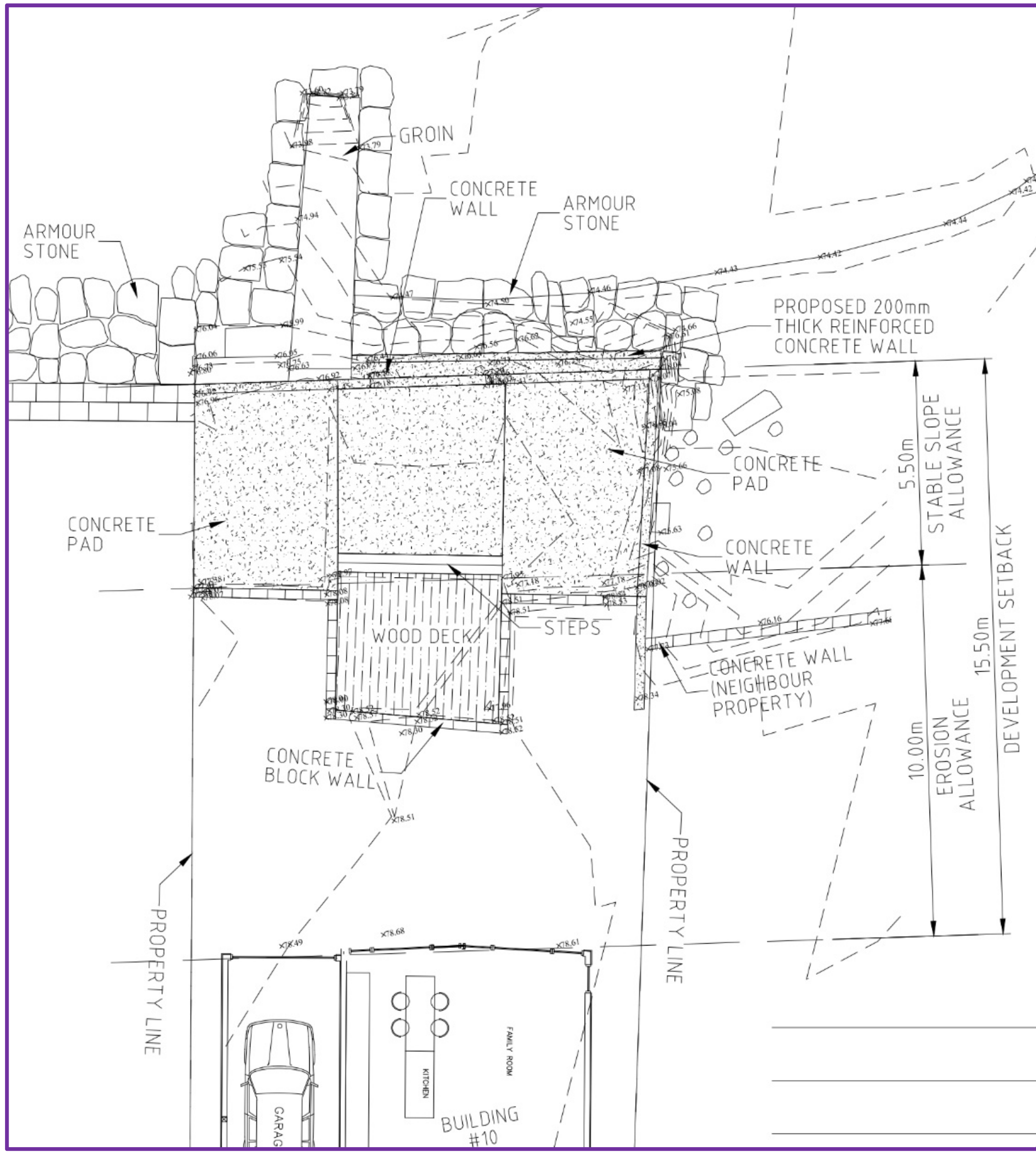
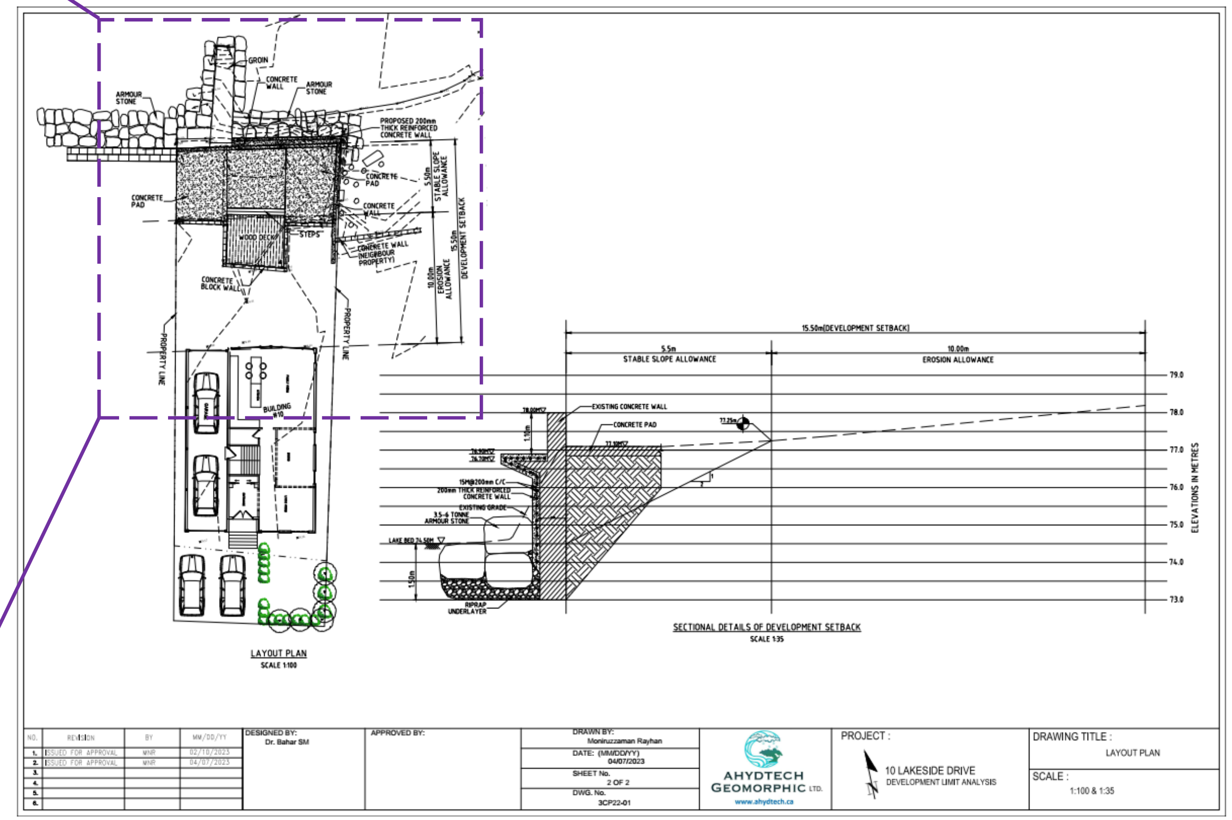
AHYDTECH GEOMORPHIC

EROSION HAZARD LIMIT



Extent of Erosion Hazard Limit in the 10 Lakeside Drive Property



Layout Plan and Section Details of Erosion Hazard Limit/Development Setback at 10 Lakeside Drive Property ***(Annex – V, Sheet 2)***

Hamilton Conservation Authority

AHYDTECH GEOMORPHIC

3.

COMMENTS ON APPLICATION ASSESSMENT

By HCA Staff Provided in *Hearing Report* on June 14, 2024

In addition, the shorewall plans propose a tie-in to the existing shorewall at 8 Lakeside Dr, but if the shorewall on the adjacent property becomes damaged, there is still the potential for erosion from the flank.

Hamilton Conservation Authority

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ANALYSIS OF WIND-WAVE ENVIRONMENT

Wave Height Distribution

- Most waves within the 0-1 meter range
- Some within 2-3 meters
- From North-East Direction
- Moderate wave activity with occasional higher waves

Wind Speed Distribution

- Majority of wind speeds within the 0-10 m/s range
- Higher speeds less frequently occurring

Design Criteria Followed

- Breaking Wave Height of Conservative Design Case, (2.34 meters), following *"Coastal Engineering Manual USCAE, 2006, page II-4-3"*
- Transitional Wall at Both Neighbouring Properties to ensure comprehensive protection and structural integrity

- Technical Report on Coastal Engineering Analysis, Hazard Limit Delineation and Seawall Design at 10 Lakeside Drive, Stoney Creek, Ontario by AHYDTECH Geomorphic Ltd. (April 9, 2023) ***(Annex – IV)***

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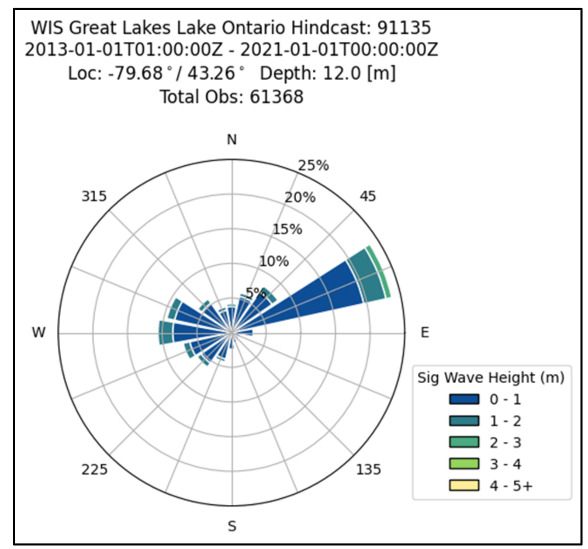
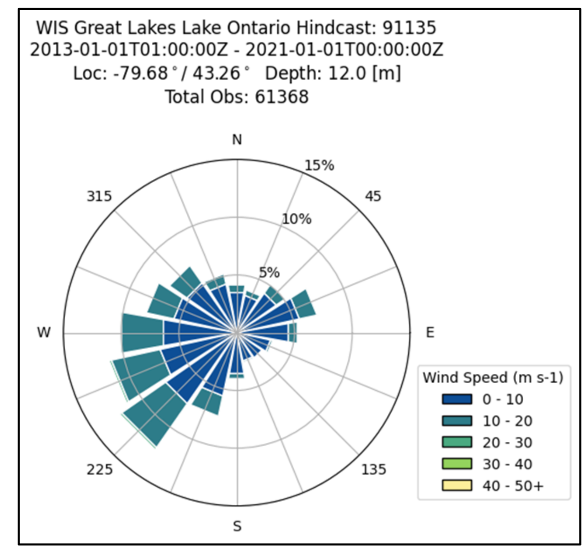
Location of Nearest WIS Station



Wave Rose Graph for Significant Wave Height



Wind Rose Graph for Wind speed

PRESENCE OF GROINS IN THE VICINITY OF THE PROPERTY SHORELINE

Presence of Shoreline Protection Structures in the Vicinity of 10 Lakeside Drive Property
(Google Earth Imagery Date: 04/25/2024)

4.

COMMENTS ON APPLICATION ASSESSMENT

By HCA Staff Provided in *Hearing Report* on June 14, 2024

Existing access to the property is constrained. Although there is no change to access relative to the existing conditions, future maintenance and construction on the shore will be a challenge.

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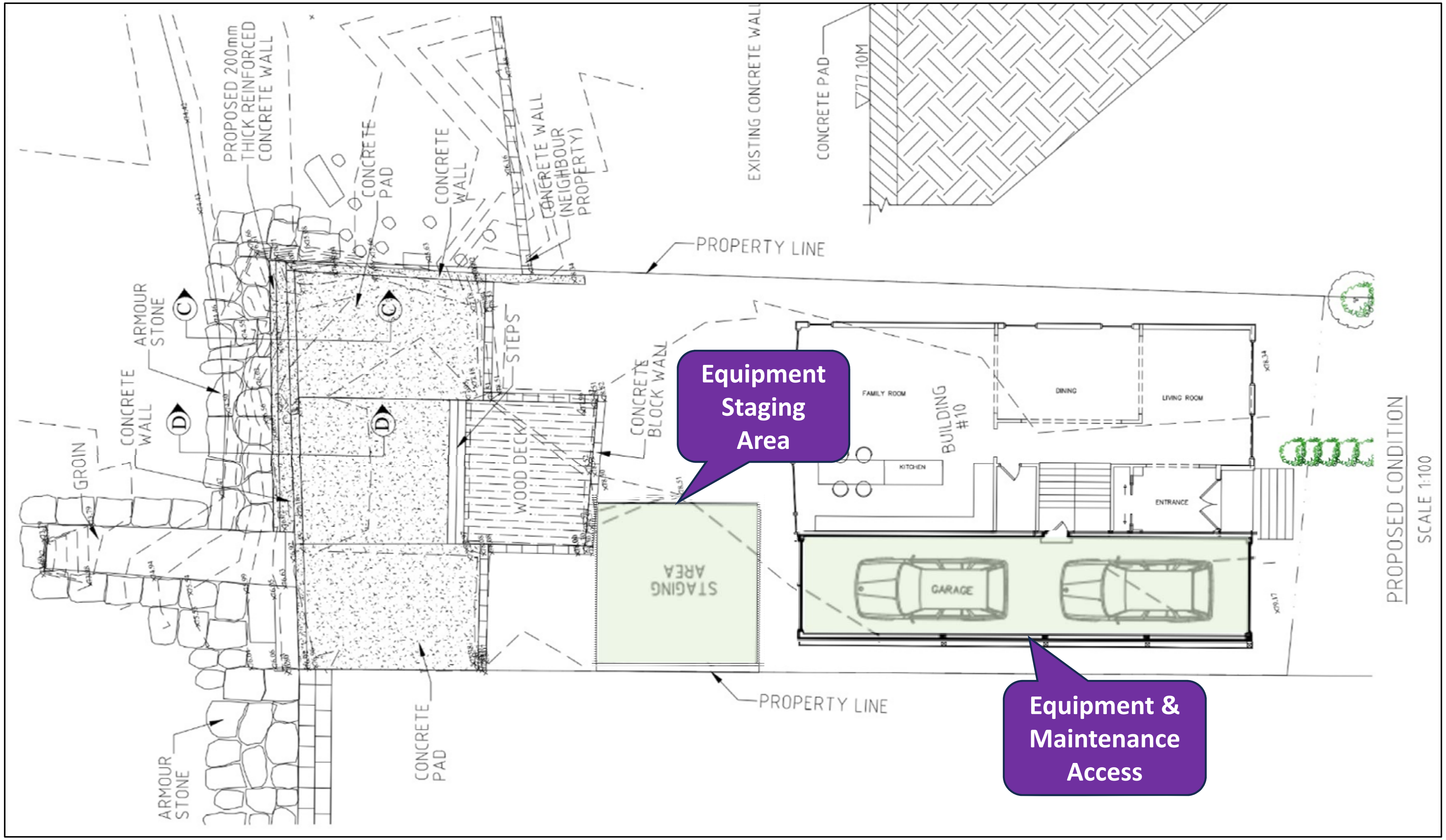
EQUIPMENT & MAINTENANCE ACCESS TO THE PROPERTY



Layout Plan & Details Drawing of Seawall Design for 10 Lakeside Drive Property ***(Annex – V*, *Sheet 1)***

Hamilton Conservation Authority

AHYDTECH GEOMORPHIC

Hamilton Conservation Authority

THANK YOU

Dr. Bahar SM ***P. Geo, P. Eng***

Phone: +1519-400-0264
Email: bahar@ahydtech.ca

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ADVANCED HYDROLOGY HYDRAULIC GEOMORPHOLOGY

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ADVANCED HYDROLOGY HYDRAULIC GEOMORPHOLOGY

Shoreline Characteristics/Assessment

Date: Nov. 25, 2022 Weather: Sunny Crew: Bahar

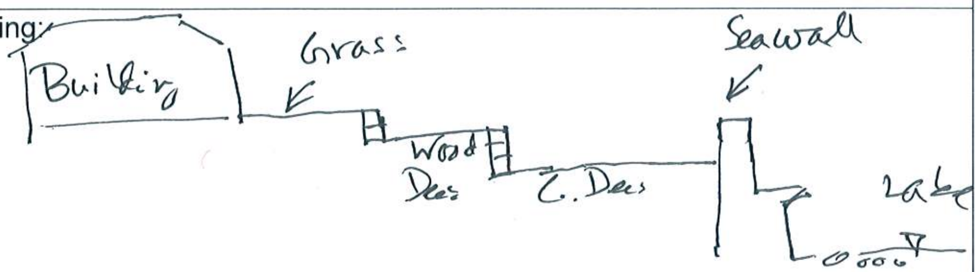
Reach: ______ Water Body: Lake Ontario

Location: 10 Lakeside Drive Stoney Creek Project Code/Phase: ______

Shoreline Planform Drawing:

See Next Page

Shoreline Cross-shore Drawing:



Composition	Vegetation:
Bluff/cliff:	Armouring:
Beach: Cobble, stone	Bluff/cliff height:
Nearshore: Seawall / broken	Bluff Strength
Exposure and Planform	Totvane
Headland Bay Partial Headland Exposed	Penetrometer

General Shoreline Character	Nearshore Controlling Surface	Surficial Substrate
Artificial Existing Seawall	Sand	Sand
Natural	Silt/clay	Silt/organic
Bedrock Cliff	Bedrock	Bedrock
Bedrock Low Plain	Cobble/boulder Till	Cobble/boulder
Cohesive/Non-cohesive Bluff	Fine Grained Cohesive	Gravel/cobble/boulder
Cohesive/Non-cohesive Low Bluff	Gravel/cobble/boulder stone	
Dynamic Beach Backed by Cliff/Bluff	Concrete Seawall & steel barrel groin	
Dynamic Beach Low Plain		
Dynamic Beach Barrier		

Cliff/bluff: steeper than 1:3 and >2m high
Low plain: landward slope flatter than 1:3 or, 2m high
Dynamic beach

Checked by: ______

Additional Notes Continued: The property shoreline has a concrete seawall and a groin made of steel barrel. The seawall toe has scour and its reinforcement is exposed. Flood water overtops the seawall. The seawall ~~and groin~~ needs repair for its long-term stability. The shoreline structure including the groin needs armourstone toe protection for its long-term stability.

Checked by: ______________________

From: Elizabeth Reimer <ereimer@conservationhamilton.ca>
Sent: Thursday, March 2, 2023 2:12 PM
To: steve@studio93inc.com
Cc: bahar@ahydtech.com
Subject: RE: 10 Lakeside Drive Seawall Repair and Hazard Limit delineation

Hello Steve and Bahar,
HCA staff have reviewed the application and hazard assessment and offer the following comments:

1) HCA staff note that the accepted recession rate for 12 Lakeside Dr was 0.2 m/yr, which was based on identical input. It is not clear how the 0.09 m/yr value was determined. In order to ensure the most conservative assessment is used, either the 0.2m/yr should be applied, or high-resolution imagery should be provided to confirm that the recession rate of 0.09 m/yr is accurate.
2) It is not clear why a 78.00 m flood hazard limit is recommended, when the assessment for 12 Lakeside Drive recommends using the elelvation of 78.50 m as adopted by HCA for the entire Stoney Creek Shoreline. The shorewall should protect the rear yard from flooding up to an elevation of 78.5m.
3) The existing wooden deck in two levels should not be part of the shorewall design as the wall is not intended for recreational purposes.
4) The life span of the existing shorewall should be inspected by a qualified structural engineer experienced in concrete properties assessment to ensure that the wall can safety withstand applicable loads over the next 50 years.
5) The erosion hazard setback should be measured from the rear of the proposed wall and illustrated on a section drawing supporting the shoreline hazard assessment.
6) The shorewall tie-in with the shorewall to the east should be clarified.
7) Additional toe protection may required to provide adequate tie-in with the westerly shorewall.
8) Machinery and equipment access to the shorewall and material staging locations should be identified and labeled on the site grading plan.
9) A heavy-duty curtain should be installed in-lake for adequate sedimentation control during the entire construction period.
10) Side yard access from the road to the shoreline should be demonstrated. HCA policy indicates a 6 m access should be maintained where possible, but given the existing constraints to access, a reduced allowance may be accepted if an access plan is provided. Access may be shared with adjacent properties, but the shared access must be registered on title in the form of an easement.

The proposed work would be classified as an Intermediate Alteration to Shorelands, and the associated fee is $4,294.00 (including HST). Payment may be made by credit card or e-transfer.

Regards,

Elizabeth Reimer
Conservation Planner
Hamilton Conservation Authority
838 Mineral Springs Road, P.O. Box 81067
Ancaster, ON L9G 4X1
Phone: 905-525-2181 Ext. 165
Email: ereimer@conservationhamilton.ca
www.conservationhamilton.ca

Hamilton Conservation Authority

A Healthy Watershed for Everyone

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

www.soil-mat.ca info@soil-mat.ca TF: 800.243.1922

Hamilton: 130 Lancing Drive L8W 3A1 T: 905.318.7440 F: 905.318.7455
Milton: PO Box 40012 Derry Heights PO L9T 7W4 T: 800.243.1922

PROJECT NO.: SM 190695-G December 13, 2019

ORION SLOLEY
12 Lakeside Drive
Stoney Creek, Ontario
L8E 5C2

GEOTECHNICAL CONSULTATIONS
PROPOSED SEAWALL RECONSTRUCTION
12 LAKESIDE DRIVE
STONEY CREEK, ONTARIO

Dear Mr. Sloley,

Further to your request, and our correspondence and discussion with Dr. Bahar SM of Ahydtech Geomorphic, SOIL-MAT ENGINEERS is pleased to offer the following geotechnical comments with respect to the proposed reconstruction of the seawall along shoreline of the above noted address.

It is understood that it is proposed to construct new shoreline protection works in order to protect the long-term stability of the existing dwelling on the subject property. Our office was provided with a copy of the proposed sea wall design drawings by Ahydtech Geomorphic.

A senior representative of our office attended the subject site on November 22, 2019 to observe the existing conditions. The existing sea wall was noted to consist of a cast in place concrete wall structure, evidently of considerable age. The concrete seawall appeared to be in fair condition, given its apparent age, with evidence of deterioration of the exposed concrete face as a function of freeze-thaw, etc. From the top of the existing sea wall the grade is relatively flat and evening, rising slightly to the private roadway at the front [south] of the lot.

A review of available published information [Quaternary Geology of Ontario, Southern Sheet Map 2556] indicates the overburden soils to consist of clay to silt textured till [derived from glaciolacustrine deposits or shale], transitioning to Queenston Shale bedrock at depth. This is consistent with our experience in the area, including investigations for a number of nearby developments, which have typically encountered very stiff silty clay/clayey silt overburden soils to depths on the order of 10 to 15 metres. The very stiff cohesive overburden soils are considered to be competent from a geotechnical point of view.

Table 4.3 of the Ministry of Natural Resources publication "Geotechnical Principles for Stable Slopes" indicates stable slope inclinations in glacial till, consistent with the established overburden soils in the area, to range from 1.5 horizontal to 2 horizontal to 1 vertical. It is our opinion, based on our local experience and available information, that a stable slope inclination of 2 horizontal to 1 vertical would be considered appropriate for use in the design of the shoreline protection works.

The proposed new sea wall is noted to be constructed in front of the existing concrete wall, and consist of a large armour stone and pre-cast concrete unit system, also incorporating reinforced cast in place concrete elements, with large 'rip rap' stone and compacted granular backfill. The design drawings note the provision of a heavy geofabric material around granular backfill deposits, which will serve to prevent the intrusion of fines or 'wash out'. The proposed system with allow for good drainage from behind the sea wall, limiting the effect of frost action, contributing to the long-term stability.

It is our opinion that, with the shore protection works implemented as per Ahydtech Geomorphic recommendations and design, that the slope would remain sufficiently stable in the long-term, from a geotechnical point of view.

We trust that these geotechnical comments are sufficient for your present requirements. Should you require any additional information or clarification as to the contents of this document, please do not hesitate to contact the undersigned.

Yours very truly,
SOIL-MAT ENGINEERS & CONSULTANTS LTD.

LICENSED PROFESSIONAL ENGINEER
Dec 13/19
I. D. SHAW
100042315
PROVINCE OF ONTARIO

Ian Shaw, P.Eng., QP_{ESA}
Senior Engineer

Distribution: Mr. Orion Sloley [1, plus pdf by email]
Ahydtech Geomorphic [1 pdf by email]

Coastal Engineering Analysis, Hazard Limit Delineation and Seawall Design

10 Lakeside Drive, Stoney Creek, Ontario

Prepared By

AHYDTECH Geomorphic Ltd.

April 9, 2023

AHYDTECH GEOMORPHIC
ADVANCED HYDROLOGY HYDRAULIC GEOMORPHOLOGY

Table of Contents

List of Figures

List of Tables

1.0 INTRODUCTION

AHYDTECH Geomorphic Ltd. is retained by the property owner of 10 Lakeside Drive, Stoney Creek, Ontario to provide coastal engineering consulting services and analysis for the project site, as shown in **Figure 1** and **Figure 2**.

The project site is located on the shoreline north of Hwy QEW, and Waterbeach Drive about 475 m northeast of Waterford Park and 480 m to the east of the New Port Yacht Club in Stoney Creek. The property is 4.8 Km northwest of Fifty Point Conservation Area. The shoreline at the property has an existing steel barrel groyne and a concrete seawall but it is fractured at many parts and the rebars at the toe are exposed due to wave actions and scour. The property owner is planning to repair/rehabilitate the existing seawall for the protection of the property.

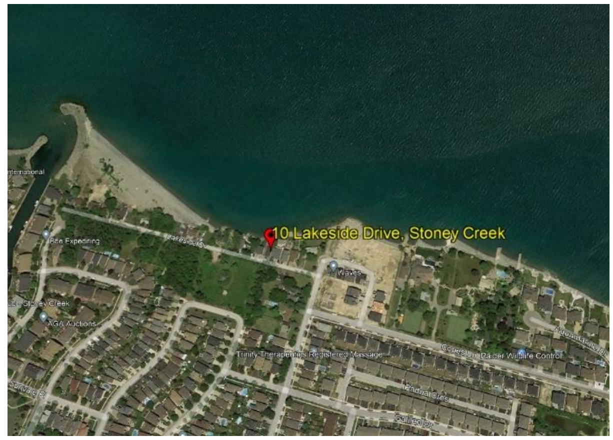


Figure 1: Site Location

In 2006, the Ontario Government passed *Ontario Regulation 161/06 (Development, Interference with Wetlands and Alterations to Shorelines and Watercourses Regulation)* that would require approvals from local Conservation Authorities for developments near a lake, river or wetland. Since the project site is on the shoreline of Stoney Creek, within the jurisdiction of Hamilton Conservation Authority (HCA), it will require the establishing development to get approval from HCA under its Regulation, made under the authority of Section 27 of the Conservation Authorities Act (Ontario Regulation 161/06).

Figure 2: Closest WIS Station Location

For this project, AHYDTECH is retained by the property owner to propose repairment and installation of a properly designed seawall. The seawall design followed the MNR Technical Guidelines (2001) for the Great Lakes - St. Lawrence River System and the Coastal Engineering Manual USCAE (1996). This study has followed the MNR Technical Guidelines, the Natural Hazard Policy (3.1) of the Provincial Policy Statement of the Planning Act and the available engineering practices to calculate the wave height and wave energy for the design.

1.1 STUDY AREA

The property site is located at 10 Lakeside Drive, Stoney Creek, within the jurisdiction of Hamilton Conservation Authority (HCA). It spans between Lakeside Drive and the southwest bank of Lake Ontario. It is on the shoreline north of Hwy QEW. about 475 m to the northeast of the Waterford Park and 480 m to the east of the New Port Yacht Club. The surrounding areas are used for residential and commercial purposes with Hallex Engineering and granite companies, such as Stonehaven Granite Works, around project site.

The shoreline faces to the north-northeastern direction specifically at the site location. AHYDTECH members conducted several site visits for coastal & topographic data collection and assessment purpose. A visual shoreline characteristics assessment was performed during the site visits and the assessment form is attached in ***Appendix A.***

There is an existing groyne made of steel barrel and a concrete seawall at the shoreline with cobble stones at the toe. The seawall is fractured and worn out with time. There are visible scour and exposed rebars at the toe of the seawall. The land between the seawall and the property is covered with a two-step wooden deck and grass lawn. The wooden deck and the grass lawn is separated with a stonewall and a brick wall.

2.0 FIELD INVESTIGATION

AHYDTECH performed field visits to the site on November 7th and 8th, 2022. The field data collection and investigation included limited depth soundings, topographic elevations along the shoreline and structural stability of the shoreline protection structure. The onshore parts of the profiles were created through topographic surveying using a GNSS RTK surveying equipment. Fixed site features and shoreline protection structures were also measured during topographic survey. The shoreline of the project site was walked by the field crew to document the shoreline characteristics, protection structure description, evidence of scouring and undercutting of structure, and note for any other concerns in words and graphical representations on field assessment forms and photos. These field data and profiles are used for the coastal analysis. Field assessment form and site photos are attached in ***Appendix A and B*** respectively.

2.1 TOPOGRAPHIC SURVEYING

AHYDTECH used the RTK/GPS to determine UTM coordinates (Zone 17, NAD83 horizontal datum projection), for vertical correction. AHYDTECH staff carefully measured the water level. The correction was then completed using Environment Canada water level data from the the Burlington Station water level above 74.2m IGLD chart datum. AHYDTECH used the X, Y, Z coordinates of the benchmark to determine the reference coordinates for our topographic survey of the site.

Lead by the Senior Engineer, Dr. Bahar SM (P. Eng.), AHYDTECH's staff used an RTK/GPS station and followed proper industry and equipment guidelines to perform a topographic survey. Measurements were also taken along the top and bottom of the concrete wall at the property site. Water level measurements were taken as the reference datum to calculate near shore shallow water depths. The RTK/GPS unit used to record the relative location of site features. The survey provided complete topographic data of ground surface and all site features including the water level, shoreline boundary and other site feature locations. The collected data was in the format of the Zone 17, NAD83 horizontal datum projection with X, Y, Z coordinates.

2.2 SHORELINE CHARECTERIZATION

Observed from the site visits, the property shoreline has an artificial shoreline with a concrete seawall at the front and a groyne made of steel barrel at the western side. The existing concrete wall is not in a fair condition and has collapsed in the bottom. It shows scour at the toe and its rebars are exposed due to wave actions over time. The length of the existing steel barrel groyne is approximately 6m and the diameter is 2m. Due to the presence of the groyne, the shoreline at the eastern side of the property has formed a cobble beach. The neighbouring property at the east side has another groyne at the rightmost boundary of the property. Due to these groynes, effect of wave action at the shoreline of the property site is less significant.

Figure 3: Existing building, grass lawn and wood deck in the property site

Figure 4: Shoreline of the property with existing concrete seawall

Figure 5: Condition of the toe of the existing seawall

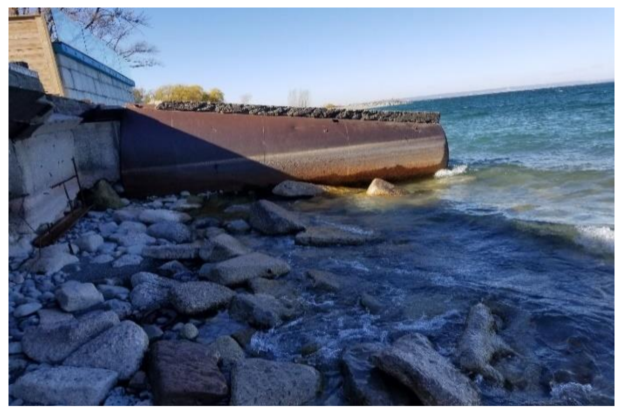


Figure 6: Existing steel barrel groyne at the western side of the property

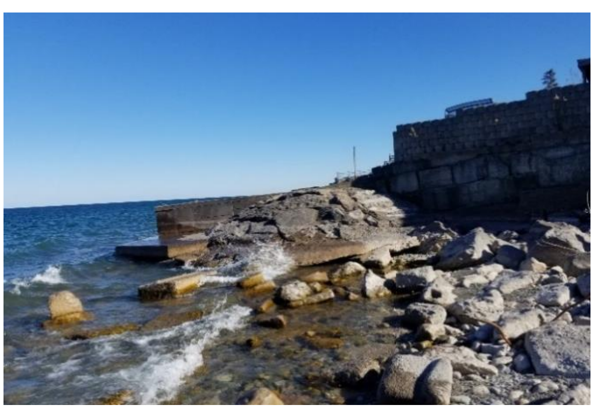


Figure 7: Shoreline of the neighbouring property at right with a groyne

Figure 8: Shoreline of the neighbouring property at left with a newly built seawall and toe protection

Details of the shoreline characterization were documented on the field assessment form in ***Appendix A***. In front of the residential building the property has a grass lawn, a concrete block wall, a stepped down wooden deck and again a concrete wall. There are concrete pads on either side of the wooden deck and concrete wall only in front of the eastern side concrete pad. There are some cobble stones at the tow of the concrete wall and the steel barrel groyne. The property is facing Lake Ontario to the northeast direction.

3.0 COASTAL ANALYSIS

According to the MNR Technical Guidelines (2001), the regulated 100-year flood level for the Western Lake Ontario is 76.0m GSC (Geodetic Survey of Canada). As the project site is located on the Western Lake Ontario shoreline, we have also analyzed wind-wave data of the Western Lake Ontario for confirmation of the proposed seawall design.

3.1 ANALYSIS OF WIND-WAVE ENVIRONMENT

The Wave Information Studies (WIS) data collected by the United States Army Corps of Engineers (USACE) were used for the wind and wave frequency analysis at the project site. The Project site is located on the shoreline of Lake Ontario, north of Hwy QEW. This location is nearest to the WIS station number ST91135 (shown in **Figure 9** and **Figure 10**). The data record period is from the years 1979 to 2014. **Figure 9** illustrates the wave rose graph generated by the USACE WIS for the significant wave height from all directions. It is observed from the graph that the majority of the waves are coming from the east and north-east directions. As mentioned earlier the project site faces the north-northeast. **Figure 10** shows the wind speeds are higher from the southwest and west directions.

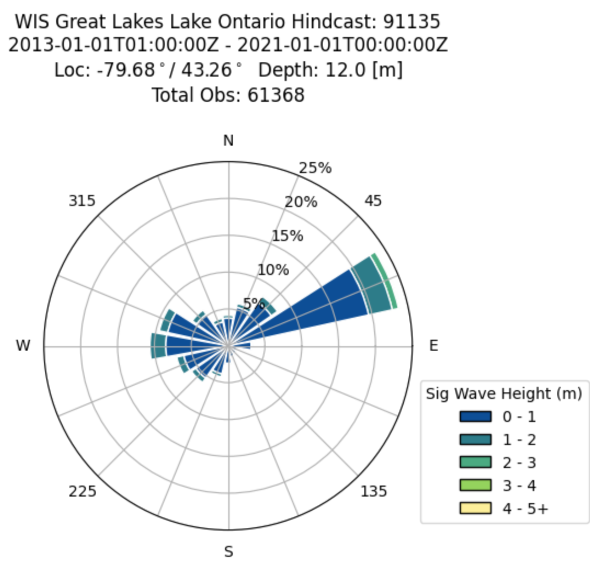


Figure 9: Wave Rose Graph for Significant Wave Height

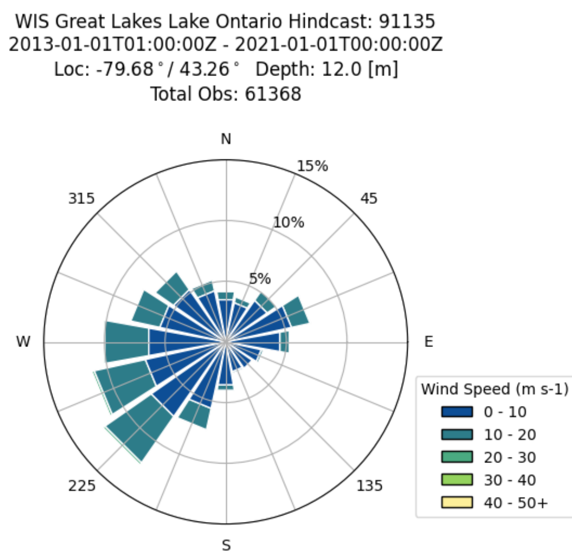


Figure 10: Wind Rose Graph for Wind speed

A frequency analysis for the wind speed and significant wave height was conducted and analyzed using the WIS data. It can be seen in **Table 1** that there are 9 direction categories, one all directions category and 8 individual direction categories. The raw data from WIS has specific degree angles measured from true north rather than just stating the direction range. However, for the analysis in this study, 8 direction categories were adopted. The 8 direction categories, formed by dividing the 360-degree angle into 8 equal angles by 8 lines from the center, starting from the true north. Then the degree angles within the ±22.5 degree range from the true north were considered to be the north direction. All other directions were categorized in a similar way. The maximum annual wind and wave data categorized by 8 different directions and all directions was obtained by inputting the raw data into a programming code developed by Dr. Bahar SM. There are 36 years of data available. Each of the processed data sets for all the 9 direction categories were ranked from smallest to largest then distributed and extended to 100 years using lognormal, linear, or exponential distribution where appropriate. Then the values for the 10, 20, and 25 years return periods were estimated from the data trend line calculated by the distribution methods.

Table 1 and **Table 2** represent the frequency analysis for wind speed and significant wave height for all directions and each individual direction for return periods 10, 20, and 25 years. As shown in **Table 1**, the wind speeds are the largest coming from the southwest direction among all the direction categories other than the all directions category. It also should be noted that the wind speeds from the southwest direction is much greater than the wind speeds from the southeast direction. **Table 2** shows that the significant wave heights are the largest from the east direction among all direction categories other than the all directions category. The shoreline at the project site is facing northeast, and the significant wave height in that direction for 10, 20 and 25 years return period are 2.36, 2.93 and 3.11 meters respectively.

Table 1: Wind Speed Frequency Analysis

	Wind Speed (m/s)		
Return Period	10	20	25
all directions	21.79	23.78	24.42
N	14.82	17.02	17.73
NE	15.59	17.81	18.52
E	16.38	18.66	19.40
SE	11.85	13.22	13.66
S	15.64	18.46	19.37
SW	20.33	22.99	23.85
W	19.62	21.38	21.94
NW	17.08	18.68	19.19

Table 2: Significant Wave Height Frequency Analysis

Return Period	10		20		25	
	HMO (m)	TP (s)	HMO (m)	TP (s)	HMO (m)	TP (s)
all directions	2.88	6.93	3.46	7.63	3.64	7.63
N	1.26	4.31	1.49	3.91	1.56	3.91
NE	2.36	5.73	2.93	6.93	3.11	6.93
E	2.67	6.93	3.29	6.93	3.48	7.63
SE	1.05	3.56	1.36	6.93	1.45	4.31
S	1.28	3.91	1.56	3.91	1.65	4.74
SW	1.34	3.91	1.54	3.91	1.60	3.91
W	1.20	3.23	1.33	3.56	1.37	3.56
NW	1.06	3.23	1.21	3.23	1.26	3.56

Note:

HMO is the significant wave height in metres
TP is the associated wave period for the HMO in seconds

3.2 SEAWALL DESIGN

The design of the seawall followed Coastal Engineering Manual USCAE, 1996, Table VI-5-53, 54 & 55. The 100-year flood level for Lake Ontario was obtained as 76.0 m in IGLD'85 datum which is applied as the seawall design water level.

The design followed the Coastal Engineering Manual USCAE (1996) and the Goda formula was modified to include impulsive forces from head-on breaking waves (Takahashi, Tanimoto and Shimosaka 1994a). The seawall design was checked for both low water level and design water level and wave height. Its stability was checked for wave, earth, hydrostatic, resultant normal and frictions forces. It can be seen that the proposed seawall design has a significant amount of factor of safety against stability for both the low water and design water levels. The detailed design calculations are shown in ***Appendix C***.

The major component of the proposed seawall design is the rehabilitation of the existing vertical concrete wall which is of 0.7-meter width and 4.2 meter height with a cantilever concrete beam at the height of 3.7 meter. A 200mm thick reinforced concrete wall will be layered at the outer face of the existing concrete wall covering from the toe to crest. An extension of 0.8-meter concrete wall will be constructed upon the crest of the existing seawall in the sections where currently enough crest height is not present. The resulting elevation of the seawall crest will be 78.0 meter. Rebars will be embedded to the existing wall with epoxy-based adhesive. The existing structure has a concrete pad and a wood deck of 300mm thickness (each) behind the crest of the seawall at an elevation of 76.8 meter. Proper pipes and materials will be applied behind these three portions for drainage.

The seawall toe was designed to protect against scour by wave and wave-induced forces. The design has applied the USACE manual (1995) to calculate scour depth at the seawall toe. According to the manual, the scour depth will be equal or 1.5 times greater than breaking wave height at the shore. Therefore, the design toe protection scour depth can vary from 1.00 to 1.5m (See ***Appendix C***). The toe of the structure will be embedded 1.5 meter into the lakebed. Three staggered layers of armour stones weighing 3.5-6 tonnes will be provided on an adequate layer (150-200 mm) of riprap underlayer at the toe below the lakebed.

More details of the proposed seawall design can be found in the drawing in ***Appendix D***.

3.3 SHORELINE HAZARD LIMIT DELINEATION

The flooding hazard limit in Lake Ontario is determined based on the combination of the 100 year regulated flood level, the maximum wave uprush, and other water-related hazards. In the western Lake Ontario, wave uprush height is about 2.0m. Accordingly, the flooding hazard limit adopted for the 10 Lakeside Drive property is 78.0 m. Top elevation of the proposed shoreline protection structure is 78.0 m, which is equal to the flooding hazard elevation. Therefore, the property will be flooding hazard free after reconstruction of the proposed seawall. The hazard limit of the Great Lakes - St. Lawrence River system is defined by the combination of flooding hazards, erosion hazards, and dynamic beach hazards along a shoreline. As the property will have the artificial shoreline protection structure, there will not be any dynamic beach hazard, and thus, the assessment of dynamic beach hazard component is not necessary. The erosion hazard analysis performed by AHYDTECH is presented in the following sections.

AHYDTECH performed a desktop analysis following both the MNR and HCA guidelines and regulations. According to the “Understanding Natural Hazards Great Lakes – St. Lawrence River System and Large Inland Lakes, River and Stream Systems Hazardous Sites” introductory guideline (MNR, 2001), the erosion hazard is determined by the stable slope allowance plus the erosion allowance. The erosion allowance can be calculated as the product of the average annual recession rate times the 100 year time span or simply 20m as the erosion allowance if the average annual recession rate is not available. AHYDTECH has followed the erosion threatened area calculation method stated in MNR guideline (MNR, 2001). The HCA Planning and Regulation Policies and Guidelines (HCA, 2011) stated that the adopted hazard limit should be the furthest landward extent among flooding hazard, erosion hazards, and dynamic beach hazard, plus another 10m inland. The details of the erosion hazard determination are presented below.

Average Annual Recession and Erosion Allowance

AHYDTECH followed Ministry of Natural Resources (MNR) standards for the Great Lakes - St. Lawrence River system (MNR, 2001) for the property at 10 lakeside drive and applied the recession rate of 0.2m/year. After applying the 0.2 meters per year recession rate, the 100 years erosion allowance without shoreline structure will be 20m. However, AHYDTECH recommends reconstructing the existing seawall with properly designed configurations as provided in the design drawings (Appendix-D). After reconstruction of the structure, it can be ensured that the wall will have a safe life span and withstand applicable loads over the next 50 years of life span. According to the HCA Planning and Regulation Policies and Guidelines (HCA, 2011), the property can get 50% credit of the Erosion Hazard Limit for the proposed sea wall. So, the erosion allowance for the property will be 10m.

Stable Slope Allowance

The Provincial Policy Statement (i.e., Policy 3.1) applies a two-step method for calculating the hazard limit. This method suggests estimating the stable slope allowance first and then account for the average annual rate of recession. According to the Provincial Standard, the 3 (Horizontal):1 (vertical) slope method is required apply to determine the stable slope allowance, if there is no geotechnical report on slope stability. In that case, stable slope profile is projected from the toe of the lake bed. The owner of the property retained SOIL-MAT ENGINEERS & CONSULTANTS LTD. for geotechnical and slope stability investigation of the property shoreline. SOIL-MAT recommends 2 (Horizontal): 1 (vertical) for the stable slope. After applying 2 (Horizontal): 1 (vertical) slope, the stable slope allowance will be 5.5m from the toe of the lake bed. The vertical distance is measured from the toe of the natural shoreline to the top of the first landward break. Then the horizontal distance is just two times of the vertical distance.

Development Setback

This study has applied both the MNRF Provincial Policy Statement (i.e., Policy 3.1) and the Hamilton Conservation Authority (HCA) Policies and Guidelines.

The erosion hazard limit is based on measurement of the stable slope allowance recommended by SOIL-MAT and add to it the average annual rate of recession, as shown below:

Erosion Hazard Limit = Stable Slope Allowance + Erosion Allowance

15.5 m = 5.5 m + 10m

With addition of the slope allowance (5.5m) and the erosion hazard allowance (10m), the total hazard limit from the toe of the natural shoreline will be 15.5m.

This existing wall is 15.5m from the main house. The total erosion hazard limit (Stable Slope Allowance + Erosion Allowance) would be 15.5 m. Therefore, the existing main building structure of the property is located outside the erosion hazard limit of 15.5m. (See attached design drawings in ***Appendix D***).

Sincerely,

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Coastal Engineer, Fluvial Geomorphologist, Water Resources Engineer
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22 Zecca Drive,
Guelph, ON, N1L 1T1
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4.0 REFERENCES

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.*

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APPENDIX A: FIELD ASSESSMENT FORM

APPENDIX B: SITE PHOTOS

Figure 1: Existing Building at 10 Lakeside Drive

Figure 2: Driveway to the Property

Figure 3: Grass Lawn, Concrete Wall, Brick Wall and Wooden Deck in front of the Building

Figure 4: Wooden Fence at the Left Boundary of the Property

Figure 5: Concrete Pad and Brick Wall Beside the Wooden Deck

Figure 6: Shoreline in front of the Property with Existing Seawall and Steel Barrel Groyne

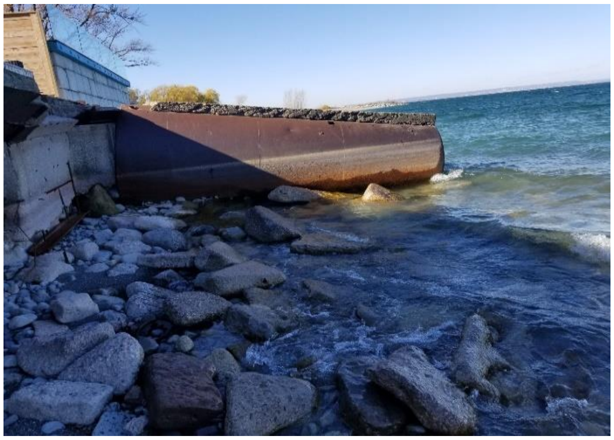


Figure 7: Existing Steel Barrel Groyne

Figure 8: Condition of the Seawall at the Shoreline in front of the Wooden Deck

Figure 9: Exposed Reinforcement of the Concrete Seawall at the Shoreline with Washed out Small and Medium Stones at the Toe

Figure 10: Seawall of Neighbouring Property at Left with Large Armourstones used as Toe Protection

Figure 11: Neighbouring Property Building at Right with Boundary Wall in front of the Building

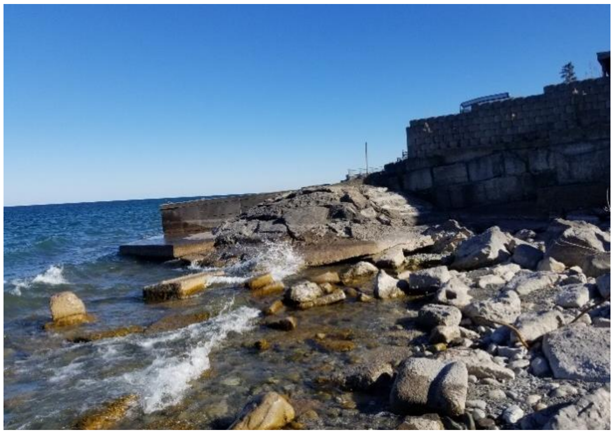


Figure 12: Shoreline of the Neighbouring Property at Right with a Groyne at the Rightmost Boundary of the Wall

APPENDIX C: DESIGN CALCULATIONS

Seawall Design Calculation

SEAWALL DESIGN: 10 Lakeside Drive, Stoney Creek, ON

Coastal Engineering Manual USCAE, 1996, Table VI-5-53, 54 & 55

GODA Formula Modified to Include Impulsive forces from Head-on Breaking Waves (Takahashi, Tanimoto and Shimosaka 1994a)

A. DESIGN WATER LEVEL

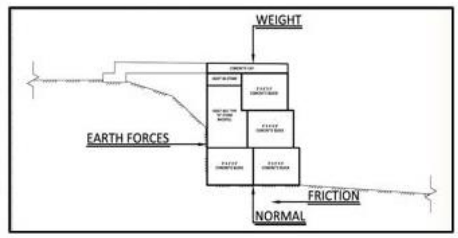
	Meter	
100 Year Design Water Level =	76	**Reference :** "Regulatory Flood Levels, March 1993, Table 4.1, page 4.1"

B. DESIGN WAVE HEIGHT

INITIAL DESIGN CASE		
Lake Bottom Elevation =	74.5	
Structure Depth = d_s =	1.5	
Breaking Wave Height = Hb = 0.78 x ds =	1.17	**Reference :** "Coastal Engineering Manual USCAE, 2006, page II-4-3"
CONSERVATIVE DESIGN CASE		
Toe of Structure =	73	
Structure Depth = d_s =	3	
Breaking Wave Height = H_b = 0.78 x d_s =	2.34	**Reference :** "Coastal Engineering Manual USCAE, 2006, page II-4-3"

C. SEAWALL CREST ELEVATION

INITIAL DESIGN CASE	
Wave Crest Elevation = DWL + 0.7 Hb =	76.819
CONSERVATIVE DESIGN CASE, IF TOE STRUCTURE IS SCOURED	
Wave Crest Elevation = DWL + 0.7 H_b =	77.638
CONSERVATIVE DESIGN CASE: SET SEAWALL CREST ELEVATION AT	77.638
Design Crest Elevation	78



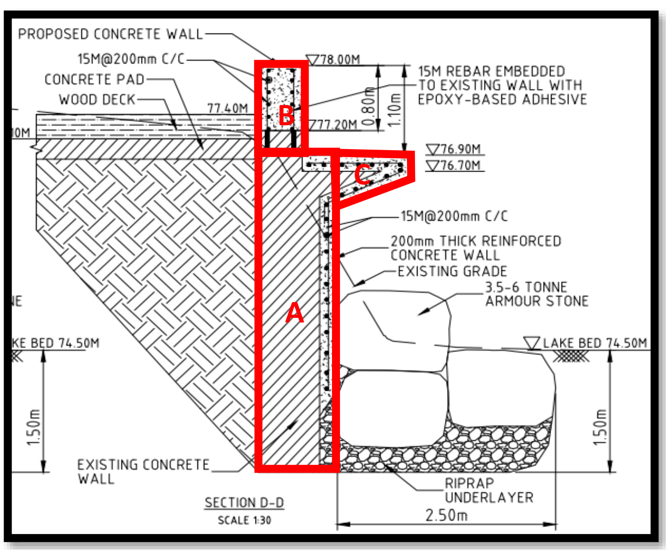
D. EXTERNAL STABILITY

CONSIDER 2 DESIGN CASES

2. DESIGN WATER LEVEL & WAVE HEIGHT

CASE 1: LOW WATER LEVEL (AT 74.95M)

DETERMINE FORCES: I. STRUCTURE CROSS SECTION WEIGHT, II. NORMAL FORCE, II. FRICTION, IV. EARTH FORCES



AHYDTECH GEOMORPHIC
ADVANCED HYDROLOGY HYDRAULIC GEOMORPHOLOGY

I. STRUCTURE CROSS SECTION WEIGHT

	COMPONENT	B Width / Diameter	Height / Diameter (m)	Area (m^2)	Density/Unit WT (kg/m3)	NO.	Mass/m (Kg/m)	Mass/m (ton/m)	Bearing Pressure (KPa)
1	Toe Protection Structure (Armour Stone Block)	1.25	1.15	1.4375	2650	3	11428.125	11.428125	89.687925
2	Seawall (RCC Rectangular Section A)	0.9	3.9	3.51	2500	1	8775	8.775	95.6475
3	Seawall (RCC Rectangular Section B)	0.5	1.1	0.55	2500	1	1375	1.375	26.9775
4	Seawall (RCC Trapezoidal Section C)	1	1	0.5	2500	1	1250	1.25	12.2625
5	Riprap Underlayer	2.5	0.2	0.5	1602	1	801	0.801	3.143124
	II. NORMAL FORCE = WEIGHT =		**23.63**		Weight/m (Ton/m)		23629.125	23.629125	

III. FRICTION

Angle of Internal Friction, α = 35 assumed	35.00	
Coefficient of Static Friction, μ =TAN α =	0.70	
Friction = N μ =	16.55	Ton/m

IV. EARTH FORCES

h_w = Overall Height of Structure	5.00	m
β = Slope of Backfill =	0.00	
Υ = Unit Weight of Backfill	1600.00	kg/m3
ϕ = Angle of Internal Friction of Backfill =	40.00	
K_a = Active Earth Coefficient = $TAN^2(45-\phi/2)$	0.22	
F_E = Earth Forces = $1/2\ \Upsilon\ hw^2\ Ka\ COS\ \beta$ =	4348.86	Kg/m
	4.35	Ton/m

SLIDING STABILITY		
Stability Forces = Friction =	16.55	Ton/m
Anti-Stability Forces = Earth Forces =	4.35	Ton/m
Factor of Safety = Stability Forces/AntiStability Forces =	3.80	

Overturning Stability - Calculate Moments about Structure Toe STABILIZING MOMENT		
Structure Weight =	23.63	Ton/m
Moment Arm =	0.90	
Stabilizing Moment =	21.27	m-Tom/m

ANTI-STABILIZING MOMENT		
Earth Forces =	4.35	
Moment Arm =	1.67	
Anti-Stabilizing Moment =	7.25	m-Tom/m
Factor of Safety = Stabilizing/Anti-Stabilizing =	2.93	

TOE SCOUR DEPTH		
Toe Scour Depth, ($H_b \leq a_s$ $1.5H_b$), m	1.17	**Reference:**
Design Toe Scour Depth, m	1.46	"Design of Revetments, Seawalls and Bulkheads, USACE, 1995"

CASE 2: DESIGN WATER LEVEL & WAVE HEIGHT

DETERMINE FORCES: I. STRUCTURE CROSS SECTION WEIGHT, II. WAVE FORCES, III. EARTH FORCES, IV. HYDROSTATIC (BOUYANT) FORCES, V. NORMAL FORCES (RESULTANT), VI. FRICTION

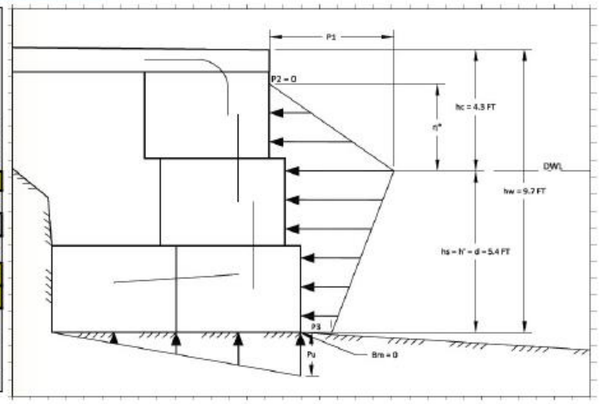
DESIGN ASSUMPTIONS: For Conservative Design Assume High Lake Water Level & Low Ground Water Level (Seperated by Seawall). Hydrostatic Forces Must be Considered

I. STRUCTURE CROSS SECTION WEIGHT	22.828125	(Ton/m)

II. WAVE FORCES

WAVE FORCE CALCULATION BASED ON GODA FORMULA FOR IRREGULAR WAVES MODIFIED TO INCLUDE IMPULSIVE FORCES FROM HEAD ON BREAKING WAVES AND ADAPTED TO THE GEOMETRY OF THE PROPOSED SEAWALL.
REFERENCE: "COASTAL ENGINEERING MANUAL" USACE, 1996, TABLES VI-5-53, 54 & 55.

P_1 = Wave Pressure at Design Water Level	
P_2 = Wave Pressure at Wave/Seawall Crest Height	
P_3 = Wave Pressure at Base of Seawall	
P_u = Wave Uplift Pressure at Base of Seawall	
η^* = Wave Crest Height	
h_s = Depth of Structure at Toe	1.50
d = Water Depth of Structure at Toe	1.50
h' = Total Depth of Structure	1.50
β = Angle of Incedence of Design Waves = 0 Degree	0.00
H_{design} = Design Wave Height = H_b =	2.34
h_w = Overall Height of Structure =	5.00
h_c = Height of Structure Above DWL =	2.00
B_m = Width of Rubble Foundation =	2.50
T_s = Wave Period (assumed for Hb)	3.56
h_b = Water Depth at a Distance 5 X Hdesign from Seawall	
= h_s + 5 x H_{design} X TAN φ =	1.70
L= Wave Length at Water Depth h_s = T_s (g * h_s)^0.5	14.56



DETERMIINE MODIFICATIONS TO GODA FORMULA

$$\delta_2 = \begin{cases} 4.9 \cdot \delta_{22} & \text{for} \quad \delta_{2\,2} \leq 0 \\ 3 \cdot \delta_{22} & \text{for} \quad \delta_{22} > 0 \end{cases}$$

$$\delta_{22} = -0.36\left(\frac{B_m}{L} - 0.12\right) + 0.93\left(\frac{h_s - d}{h_s} - 0.6\right)$$

δ_{22} =	-0.58
δ_2 =	-2.83

$$\delta_1 = \begin{cases} 20 \cdot \delta_{11} & \text{for} \quad \delta_{11} \leq 0 \\ 15 \cdot \delta_{11} & \text{for} \quad \delta_{11} > 0 \end{cases}$$

$$\delta_{11} = 0.93\left(\frac{B_m}{L} - 0.12\right) + 0.36\left(\frac{h_s - d}{h_s} - 0.6\right)$$

δ_{11} =	-0.17
δ_1 =	-3.36

$$\alpha_* = \text{largest of } \alpha_2 \text{ and } \alpha_I$$

$$\alpha_2 = \text{the smallest of } \frac{h_b - d}{3\,h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}}$$

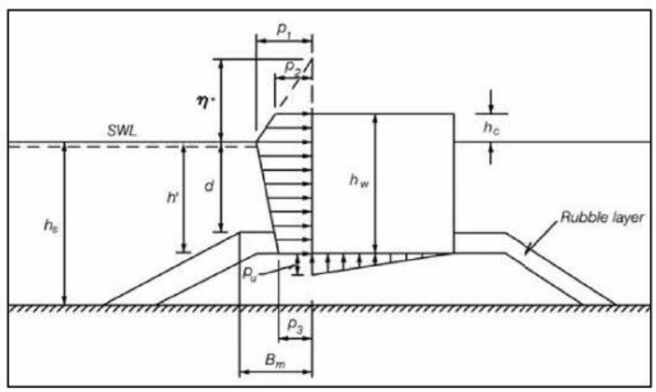
$$\alpha_I = \alpha_{I0} \cdot \alpha_{I1}$$

$$\alpha_{I0} = \begin{cases} H_{design}/d & \text{for} \quad H_{design}/d \leq 2 \\ 2.0 & \text{for} \quad H_{design}/d > 2 \end{cases}$$

$$\alpha_{I1} = \begin{cases} \frac{\cos\delta_2}{\cosh\delta_1} & \delta_2 \leq 0 \\ \frac{1}{\cosh\delta_1 \cdot (\cosh\delta_2)^{\frac{1}{2}}} & \delta_2 > 0 \end{cases}$$

$$\delta_1 = \begin{cases} 20 \cdot \delta_{11} & \text{for} \quad \delta_{11} \leq 0 \\ 15 \cdot \delta_{11} & \text{for} \quad \delta_{11} > 0 \end{cases}$$

α_{I0} =	1.56
α_{I1} =	0.00
$\alpha_I = \alpha_{I0} \cdot \alpha_{I1}$	0.00
α_2 =	0.10
α_2 =	1.28
Smaller α2 =	0.10
α_* =	0.10



STRUCTURE MODIFICATION FACTORS

$\lambda_1 = \lambda_2 = \lambda_3$ = 1.0 for Conventional Vertical Wall Structures	1.00
DETERMINE PRESSURE COEFFICIENT FOR GODA FORMULA	
α_* = 0.014 (Modified α^* for Impulse Forces)	0.01

$$\begin{aligned} \alpha_* &= \alpha_2 \\ \alpha_1 &= 0.6 + 0.5\left[\frac{4\pi h_s/L}{sinh\,(4\pi h_s/L)}\right]^2 \\ \alpha_2 &= \text{the smallest of } \frac{h_b - d}{3h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}} \\ \alpha_3 &= 1 - \frac{h_w - h_c}{h_s}\left[1 - \frac{1}{cosh\,(2\pi h_s/L)}\right] \end{aligned}$$

α_1 =	0.89	
α_2 =	0.01	
α_3 =	0.64	

$$\begin{aligned} \eta^* &= 0.75(1+cos\beta)\,\lambda_1 H_{design} \\ p_1 &= 0.5(1+cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_* cos^2\beta)\,\rho_w\, g\, H_{design} \\ p_2 &= \begin{cases} \left(1 - \frac{h_c}{\eta^*}\right) p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \le h_c \end{cases} \\ p_3 &= \alpha_3\, p_1 \\ p_u &= 0.5(1+cos\beta)\lambda_3\alpha_1\alpha_3\rho_w g\, H_{design} \end{aligned}$$

η^* =	3.51	
P_1 =	22736.22	Pa
P_2 =	9781.11	Pa
P_3 =	14633.97	Pa
P_u =	13199.16	Pa

DETERMINE LEVELS OF UNCERTAINITY

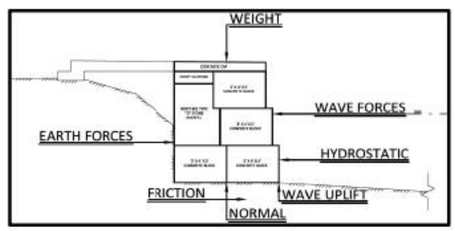
DETERMINE LEVELS OF UNCERTAINTY	REFERENCE: "COASTAL ENGINEERING MANUAL" USACE, 2006, TABLE VI-5-55.
FOR HORIZONTAL FORCE, UFH = 0.90	
FOR UPLIFT FORCE, UFU = 0.77	
FOR HORIZONTAL MOMENT, UMH = 0.81	
FOR UPLIFT MOMENT, UMU = 0.72	

CALCULATE WAVE FORCES PER UNIT LENGTH OF STRUCTURE

Horizontal Wave force, F_H = UFH (1/2 $(P_1+P_2)h_c$ + 1/2$(P_1+P_3)h'$ =	54490.48	Kg/m
Wave Uplift Force, F_u = UFU x 0.5 P_u x B =	5081.68	Kg/m
B = 1 meter		

III. EARTH FORCES

$FE = \frac{1}{2}\gamma\, hw^2\, Kp\, COS\,\beta$



β = Slope of Backfill = 0 degree	0.00	
h_w = Overall Height of Structure =	5.00	m
Υ = Unit Weight of Backfill Soil =	1600.00	kg/m3
ϕ = Angle of Internal Friction of Backfill =	40.00	
Kp = Passive Earth Coefficient = $TAN^2(45+\phi/2)$	4.60	
FE = 1/2 $\Upsilon\, h_w^2\, K_p$ COS β =	91978.20	Kg/m
	91.98	Ton/m

IV. HYDROSTATIC FORCES

Υ_w = Density of Water	1000.00	kg/m3
h = Water Depth at the Toe of the Structure =	1.50	
F_{hydro} = 1/2 $\Upsilon_w\, h^2$ =	1125.00	Kg/m
	1.13	Ton/m

V. RESULTANT NORMAL FORCES

Resultant Vertical force, N = WEIGHT - WAVE UPLIFT =	18547.45	Kg/m
	18.55	Ton/m

VI. FRICTION

Angle of Internal Friction, α = 35 degrees	35.00	
Coefficient of Static Friction, μ = TAN α =	0.70	
FRICTION = N μ =	12987.06	Kg/m
SLIDING STABILITY		
Stabilizing Forces = FRICTION + EARTH FORCES	104965.26	Kg/m
Anti-Stabilizing Forces = WAVE FORCES + HYDROSTATIC FORCES	55615.48	Kg/m
FACTOR OF SAFETY = STABILIZING FORCES/ ANTI-STABILIZING FORCES	1.89	

OVERTURNING STABILITY - CALCULATE MOMENTS ABOUT STRUCTURE HEEL

A) STABILIZING MOMENTS

Structure Weight =	23629.13	Kg/m
Moment Arm = width/2 = 0.5	2.50	
STABILIZIING MOMENT =	59072.81	Kg-m/m
Earth Forces =	91978.20	Kg/m
Moment Arm = Height of Structure/3	1.67	
STABILIZIING MOMENT =	153297.00	Kg-m/m
TOTAL STABILIZING MOMENT =	212369.81	Kg-m/m

B) ANTI-STABILIZING MOMENTS

Horizontal Wave Force = F_H =	54490.48	
Moment Arm = UMH x h_c =	1.62	
ANTI-STABILITY MOMENT =	88274.57	Kg-m/m
Uplift Wave Force = F_U =	5081.68	
Moment Arm = UMU x h' =	1.08	
ANTI-STABILITY MOMENT =	5488.21	Kg-m/m
Hydrostatic Force =	1125.00	
Moment Arm = $h_s/3$ =	0.50	
ANTI-STABILITY MOMENT =	562.50	Kg-m/m
TOTAL ANTI-STABILIZING MOMENT =	94325.29	Kg-m/m

FACTOR OF SAFETY = STABILIZING MOMENT/ ANTI-STABILIZING MOMENT	2.25

APPENDIX D: DESIGN DRAWINGS

6.3.2 Annex - V

GENERAL NOTES:

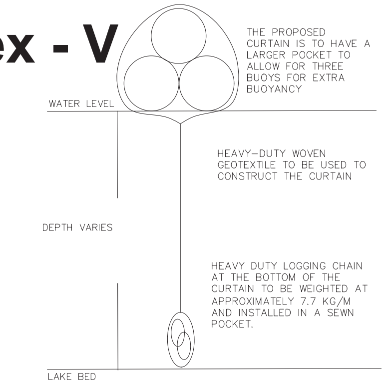
1. NO IN-WATER WORK OR ACTIVITY SHALL OCCUR FROM MARCH 15 TO JULY 15 AND FROM SEPTEMBER 30 TO NOVEMBER 15 IN ANY YEAR, OR AS OTHERWISE DIRECTED BY HAMILTON CONSERVATION AUTHORITY.
2. THE CONTRACTOR SHALL PROVIDE THE HAMILTON CONSERVATION AUTHORITY , MNRF AND CONSULTING ENGINEER 48 HOURS NOTICE PRIOR TO COMMENCING WORK.
3. THIS SET OF DRAWINGS SHALL BE READ IN CONJUNCTION WITH ACCOMPANYING DESIGN BRIEF.
4. ALL DIMENSIONS AND ELEVATIONS ARE IN METERS UNLESS OTHERWISE INDICATED.
5. DECK, BACK SHORE GRADING & LANDSCAPE, IF ANY, SHOULD BE DESIGNED BY OTHERS.
6. ALL WORKS SHALL BE INSPECTED AND APPROVED BY A DESIGN ENGINEER.
7. ALL WORKS AND MATERIALS SHALL BE IN ACCORDANCE WITH APPLICABLE MUNICIPAL AND/OR ONTARIO PROVINCIAL STANDARD SPECIFICATIONS AND DRAWINGS.
8. THE CONTRACTOR SHALL ERECT AND MAINTAIN SILT BARRIER CONSISTING OF HEAVY-DUTY FLOATING TURBIDITY CURTAIN ALONG THE WATER SIDE OF THE TOE OF THE SEAWALL FOR THE DURATION OF IN-WATER CONSTRUCTION. AND REMOVE THEM ONLY WHEN WORK IS COMPLETED, AND SEDIMENTS HAVE SETTLED.
9. DURING CONSTRUCTION, ALL VEGETATION AND STRUCTURES ON PRIVATE PROPERTY, ADJACENT TO THE WORK AREA, IS TO BE PROTECTED OR, IF REMOVAL IS REQUIRED, RESTORED TO ORIGINAL CONDITION WITH NATIVE/NON-INVASIVE SPECIES UPON COMPLETION OF WORK.
10. ALL GENERAL BACKFILL SHALL BE APPROVED MATERIAL COMPACTED TO 95% STANDARD PROCTOR DENSITY.
11. STOCKPILED MATERIAL IS TO BE STORED AT DISTANCE(SHOWN ON DRAWING) AWAY FROM POTENTIAL RECEIVERS (E.G. STORM CATCH BASINS), AND BE SURROUNDED BY EROSION CONTROL MEASURES WHERE MATERIAL IS TO BE LEFT IN PLACE IN EXCESS OF 14 DAYS.
12. USE, OPERATE OR STORE ANY WHEELED OR TRACKED MACHINERY/EQUIPMENT ON DRY LAND, OR ON A BARGE OR VESSEL.
13. MACHINERY SHOULD BE KEPT IN CLEAN CONDITION AND FREE FROM FLUID LEAKS TO PREVENT ANY DELETERIOUS SUBSTANCES FROM ENTERING THE WATER.
14. WASH, REFUEL AND SERVICE MACHINERY AND STORE FUEL AND OTHER MATERIALS FOR THE MACHINERY IN SUCH A WAY AS TO PREVENT ANY DELETERIOUS SUBSTANCES FROM ENTERING THE WATER.
15. IF THERE ARE AQUATIC SPECIES AT RISK IN THE AREA, PROPONENTS MUST ALSO AVOID HARMING, HARASSING, CAPTURING OR TAKING THOSE SPECIES.
16. THE CONTRACTOR SHALL KEEP AN EMERGENCY SPILL KIT ON SITE DURING THE WORK, UNDERTAKING OR ACTIVITY.
17. THE CONTRACTOR NEEDS TO AVOID CONSTRUCTION WORK DURING ADVERSE WIND WAVE CONDITION.

SEAWALL DESIGN AND INSTALLATION SPECIFICATIONS :

1. PROPOSED SEAWALL INSTALLATION WORKS INCLUDES THE SITE ACCESS PREPARATION, DEMOLITION AND/OR REMOVAL OF THE EXISTING SEAWALL/ARMOUR STONES, AND DISPOSAL OF THE EXISTING STRUCTURES, BOULDERS, RUBBLE AND OTHER EXCESS MATERIAL AT THE SITE.
2. ALL CONSTRUCTION FENCING & EROSION AND SEDIMENT (ESC) MEASURES SHALL BE INSTALLED PRIOR TO START OF CONSTRUCTION. ALL EROSION AND SEDIMENT CONTROL MEASURES SHALL BE INSTALLED AS SHOWN IN THE DRAWINGS. THE CONTRACTOR IS FULLY RESPONSIBLE THAT NO SILT OR SEDIMENT DISCHARGE TO THE WATER.
3. RIP RAP FOR THE PROPOSED DESIGN SHALL BE ROCK, RANGING IN SIZE AS SHOW ON THE DRAWING. RIP RAP SHALL BE HARD, DURABLE, QUARRY STONE, AND FREE FROM CRACKS, SEAM OR OTHER DEFECTS WITH RELATIVE DENSITY OF 2.7 (± 0.15). MINIMUM 60% OF THE TOTAL VOLUME SHALL CONTAIN STONE OF INDIVIDUAL SIZE GREATER MID-POINT OF THE SPECIFIED SIZE RANGE. INSTALL RIP RAP TO LINES AND GRADES INDICATED ON THE DRAWING.
4. THE GEOTEXTILE MATERIAL PROPOSED FOR THE DESIGN SHALL BE NON-WOVEN, INERT AND DURABLE. THE MATERIAL SHALL BE TERRAFIX 800R OR APPROVED EQUIVALENT. INDIVIDUAL GEOTEXTILE SHEETS SHALL REACH FROM THE TOP OF THE REVETMENT TO THE TOE. PROVIDE A MINIMUM OF 800MM OVERLAP WIDTH IN THE ACROSS THE SLOPE DIRECTION WHEN PLACING A NEW SECTION OF GEOTEXTILE ADJACENT TO A PREVIOUSLY PLACED SECTION.
5. SUCCESSIVE ROWS OF ARMOUR STONES SHALL BE PLACED WITH STAGGERED JOINTS. INSTALL GEOTEXTILE TERRAFIX 800R OR APPROVED EQUAL, AND CRUSHED STONE TO PROVIDE DRAINAGE BEHIND THE SEAWALL AS INDICATED. BACKFILL BEHIND THE SEAWALL WITH APPROVED GRANULAR FREE DRAINING MATERIAL (RIP-POP), COMPACT MATERIAL IN 150MM LAYERS TO 95% STANDARD PROCTOR MAXIMUM DRY DENSITY.
6. PROTECT GROUND FROM FREEZING DURING CONSTRUCTION. A GUARD RAIL IS REQUIRED, TO ONTARIO BUILDING CODE SECT. 9.8.8 WHERE THE TOP OF WALL IS UNPROTECTED.

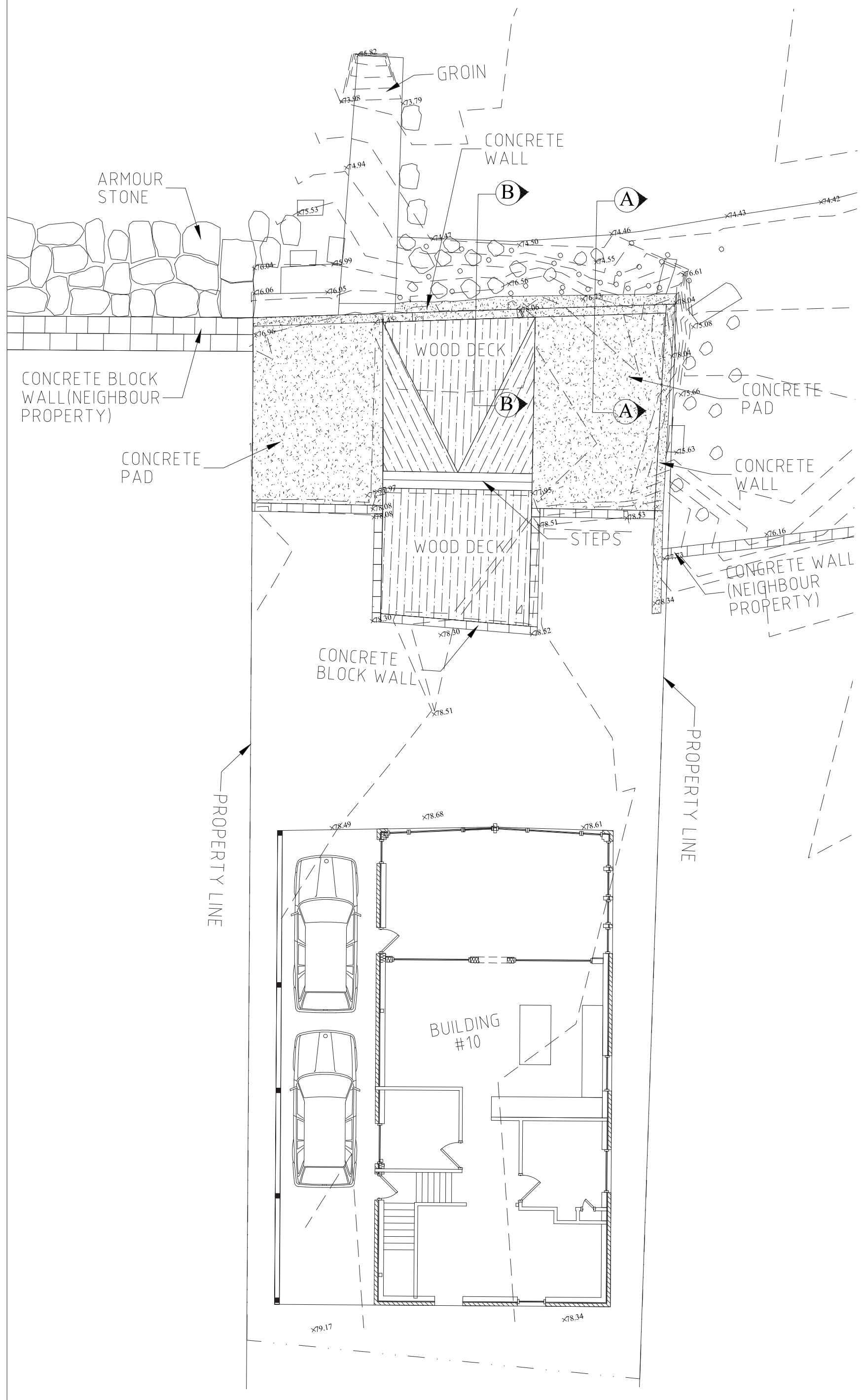
MATERIAL SPECIFICATIONS:

1. CONCRETE: COMPRESSIVE STRENGTH OF CONCRETE F'c = 35 MPa AT 28 DAYS W/ 4 TO 7% AIR ENTRAINMENT, W/C RATIO=0.45, EXPOSURE CLASS F-1 IN ACCORDANCE WITH CSA A23.1/23.2-19



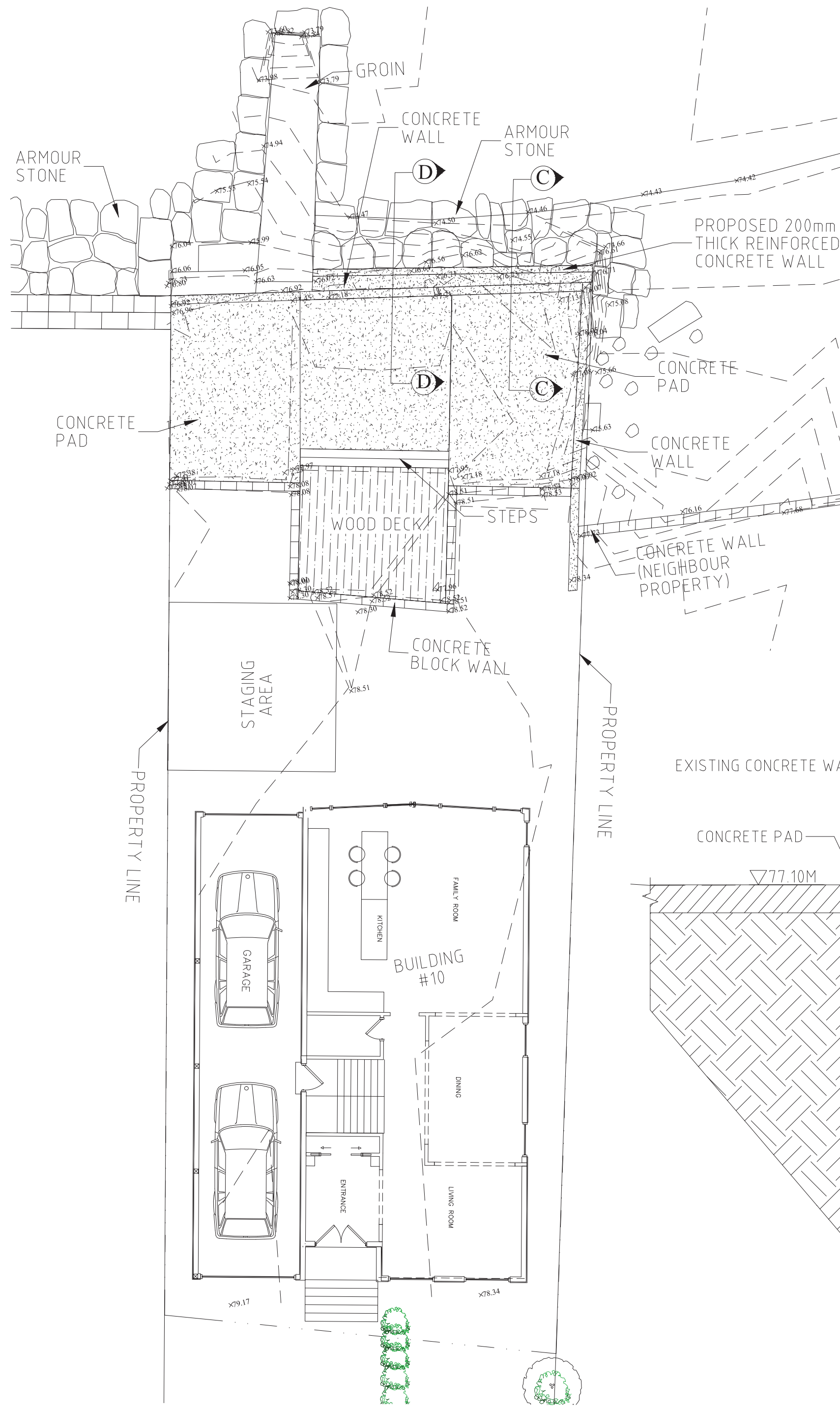
SECTION E-E

NOT TO SCALE



EXISTING CONDITION

SCALE 1:100



PROPOSED CONDITION

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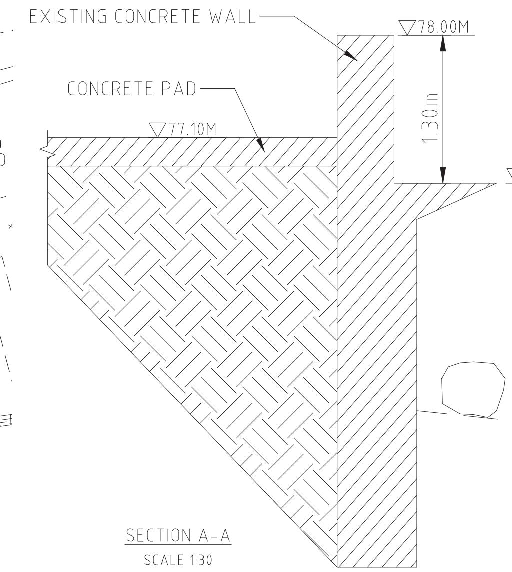
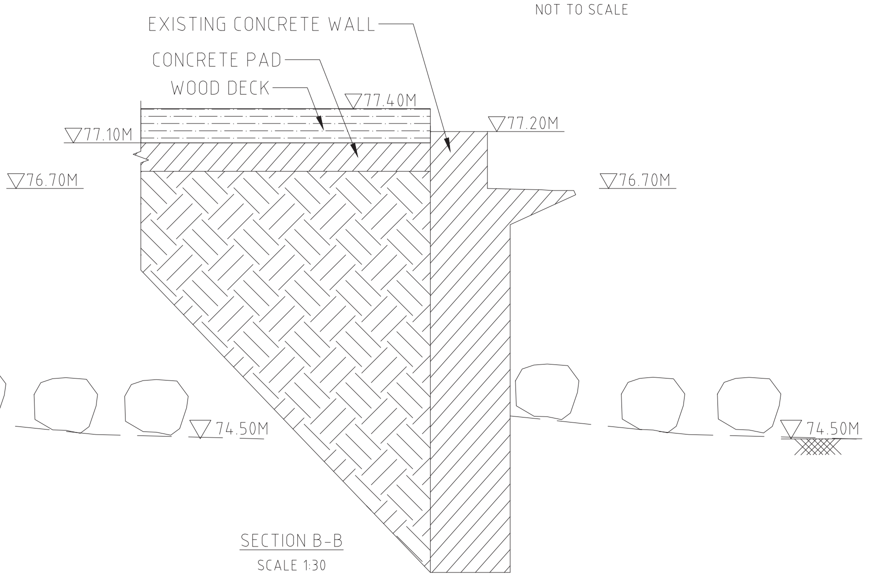


SECTION A-A

SCALE 1:30



SECTION B-B

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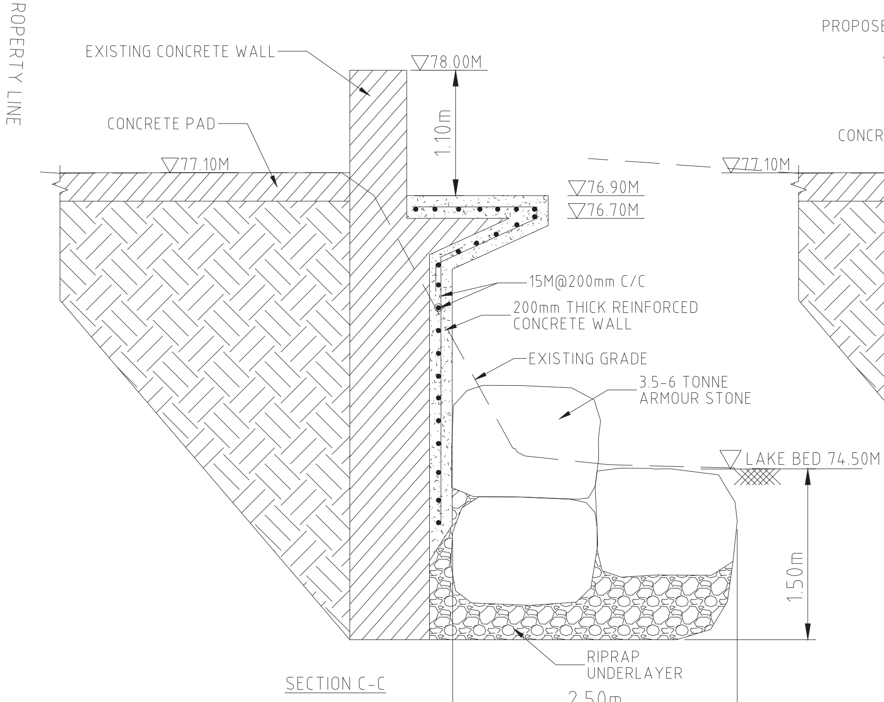
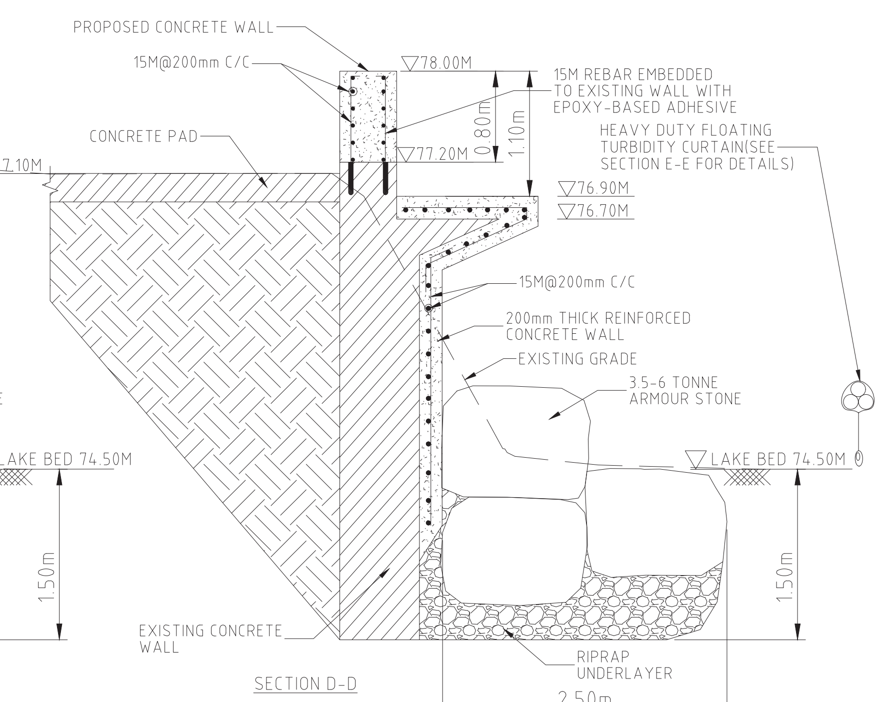


SECTION C-C

SCALE 1:30



SECTION D-D

SCALE 1:30

NO.	REVISION	BY	MM/DD/YY
1.	ISSUED FOR APPROVAL	MNR	02/10/2023
2.	ISSUED FOR APPROVAL	MNR	04/07/2023
3.			
4.			
5.			
6.			

DESIGNED BY:
Dr. Bahar SM

APPROVED BY:

DRAWN BY:
Moniruzzaman Rayhan

DATE: (MM/DD/YY)
04/07/2023

SHEET No.
1 OF 2

DWG. No.
3CP22-01

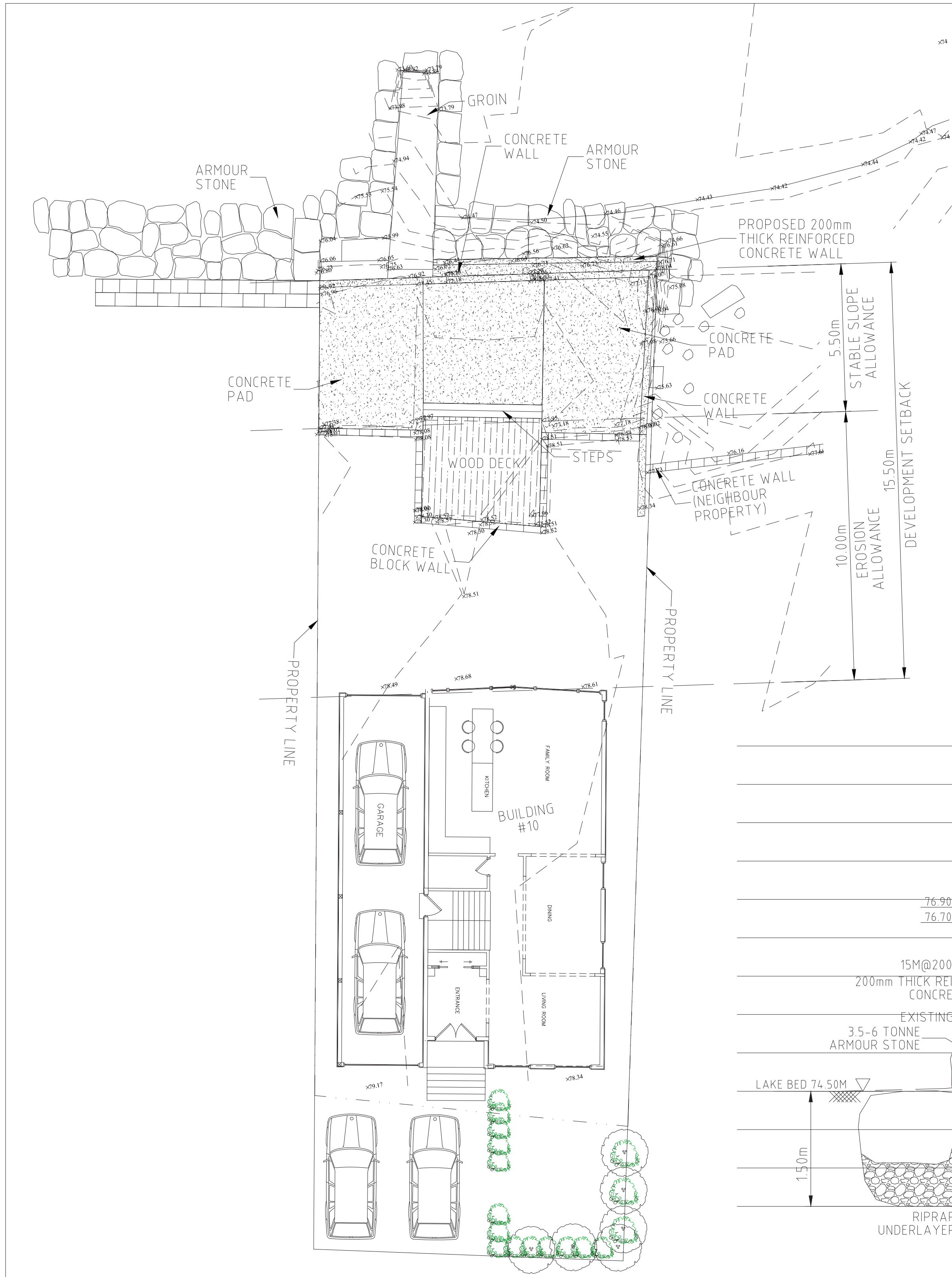
AHYDTECH GEOMORPHIC LTD.
www.ahydtech.ca

PROJECT :
10 LAKESIDE DRIVE
SEAWALL DESIGN

N

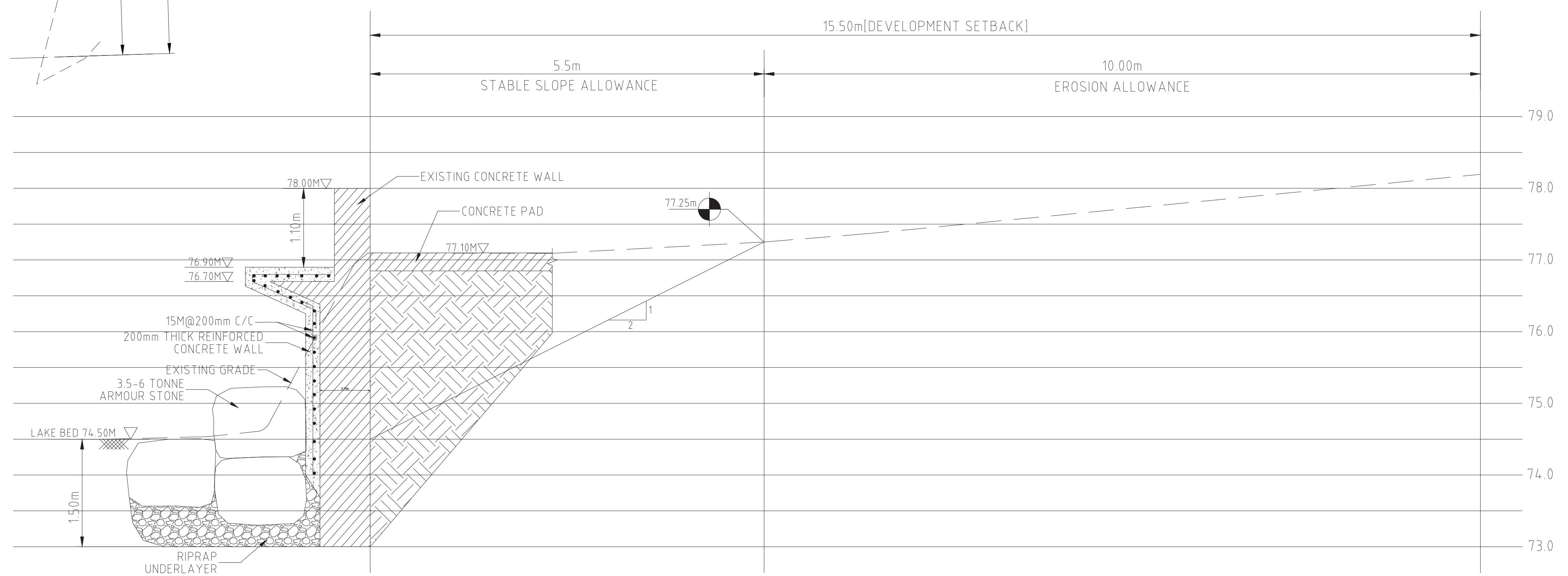
DRAWING TITLE :
LAYOUT PLAN & DETAILS

SCALE :
1:100 & 1:30



LAYOUT PLAN

SCALE 1:100



SECTIONAL DETAILS OF DEVELOPMENT SETBACK

SCALE 1:35

NO.	REVISION	BY	MM/DD/YY
1.	ISSUED FOR APPROVAL	MNR	02/10/2023
2.	ISSUED FOR APPROVAL	MNR	04/07/2023
3.			
4.			
5.			
6.			

DESIGNED BY: Dr. Bahar SM

APPROVED BY:

DRAWN BY: Moniruzzaman Rayhan

DATE: (MM/DD/YY) 04/07/2023

SHEET No. 2 OF 2

74

DWG. No. 3CP22-01

AHYDTECH GEOMORPHIC LTD.
www.ahydtech.ca

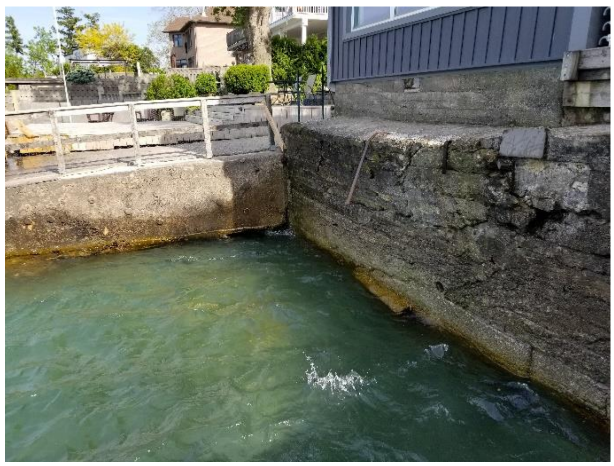
PROJECT :
N
10 LAKESIDE DRIVE
DEVELOPMENT LIMIT ANALYSIS

DRAWING TITLE : LAYOUT PLAN

SCALE : 1:100 & 1:35

6.3.2 Annex - VI

Coastal Engineering Analysis, Seawall Design for 12 Lakeside Drive, Stoney Creek



AHYDTECH Geomorphic Ltd.
22 Zecca Drive, Guelph, Ontario
N1L 1T1
(519) 400-0264
www.ahydtech.ca

Date: March 23, 2020

AHYDTECH GEOMORPHIC
ADVANCED HYDROLOGY HYDRAULIC GEOMORPHOLOGY

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

AHYDTECH Geomorphic Ltd. is retained by the property owner at 12 Lakeside Drive, Stoney Creek, Ontario to provide coastal engineering consulting service and analysis for the project site, as shown on Figure 1 and Figure 2. The project site is located on the shoreline north of Hwy QEW, and Waterbeach Drive about 470 m east of Waterford Park, between Waterford Crescent and Jones Road in Stoney Creek. The shoreline at the property has seawall but it is fractured at many parts and the foundation sheet piles are exposed due to wave actions and the structure was undercut from beach erosion. The property owner is planning to build a new seawall for the protection of the property.

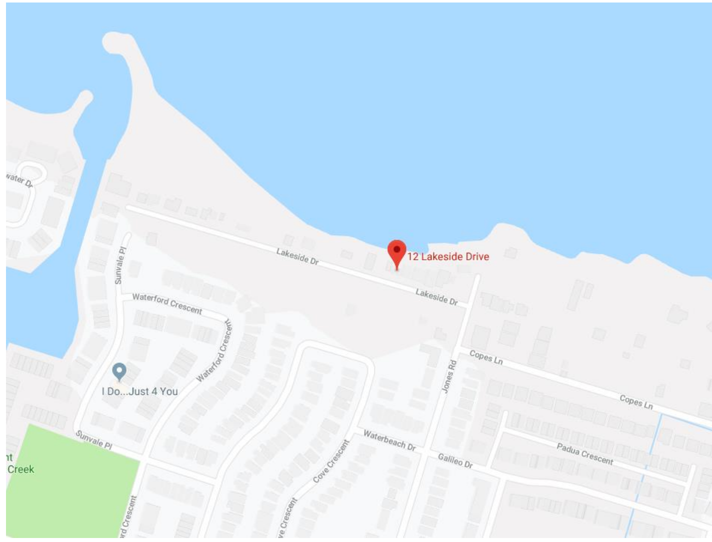


Figure 1: Site Location

In 2006, the Ontario Government passed Ontario Regulation 161/06 (Development, Interference with Wetlands and Alterations to Shorelines and Watercourses Regulation) that would require approvals from local Conservation Authorities for developments near a lake, river or wetland. Since the project site is on the shoreline of Stoney Creek, within the jurisdiction of Hamilton Conservation Authority (HCA), it will require the establishing development to get approval from HCA under its Regulation, made under the authority of Section 28 of the Conservation Authorities Act (Ontario Regulation 161/06).

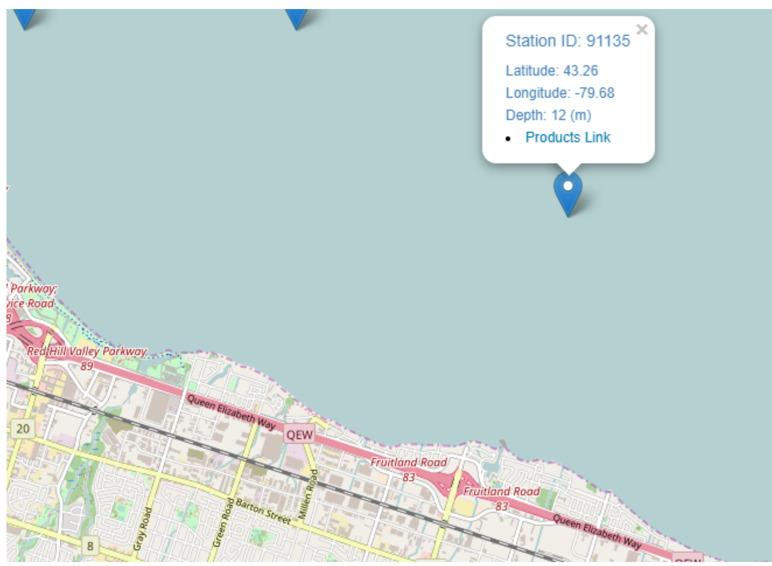


Figure 2: Site & Closest WIS Station Location

For this project, AHYDTECH is retained by the property owner to propose the installation of a properly designed seawall. The seawall design followed the MNR Technical Guidelines (2001) for the Great Lakes - St. Lawrence River System and the Coastal Engineering Manual USCAE (1996). This study has followed the MNR Technical Guidelines, the Natural Hazard Policy (3.1) of the Provincial Policy Statement of the Planning Act and the available engineering practices to calculate the wave height and wave energy for the design.

1.1 STUDY AREA

The project site is located at 12 Lakeside Drive, Stoney Creek, within the jurisdiction of the Hamilton Conservation Authority (HCA) on the shoreline north of Hwy QEW, between Waterford Crescent and Jones Road, as shown in Figure 1 and Figure 2. The property is located about 470 m east of Waterford Park. The surrounding land use is mainly residential and commercial with Hallex Engineering and granite companies, such as Stonehaven Granite Works, around project site. The shoreline, specifically at the site location, is facing northwest. AHYDTECH members conducted several site visits for coastal data collection and assessment purpose. A visual shoreline characteristics assessment was performed during the site visits and the assessment form is attached in Appendix A.

There is an existing seawall protecting the property as shown in Appendix A. However, the seawall is fractured, especially the northern portion. The ground surface is majorly covered by concrete, bricks and sparse grass. A few small trees were observed at both edges of the fence of the property owner. Steps constructed by concrete blocks were detected at the site. It provides an access to the water body.

2.0 FIELD INVESTIGATION

AHYDTECH performed a field visit for data collection and field investigation purposes on June 12, 2019. The field data collection and investigation included limited depth soundings, topographic elevations along the shoreline and structural stability of the shoreline protection structure. The onshore parts of the profiles were created through topographic surveying using a GNSS RTK surveying equipment. Fixed site features and shoreline protection structures were also measured during topographic survey. The shoreline of the project site was walked by the field crew to document the shoreline characteristics, protection structure description, evidence of scouring and undercutting of structure, and note for any other concerns in words and graphical representations on field assessment forms and photos. These field data and profiles are used for the coastal analysis. Field assessment form and site photos are attached in Appendix A and C respectively.

2.1 TOPOGRAPHIC SURVEYING

A bench mark for the topographic survey was determined by the combination of a reference elevation from the MNRF Control Survey Information Exchange (COSINE) control database benchmark station located at Glover Road bridge over QEW in Stoney Creek and AHYDTECH's RTK/GPS unit. Then AHYDTECH used the RTK/GPS to determine UTM coordinates (Zone 17, NAD83 horizontal datum projection) of the bench mark. AHYDTECH used the X, Y, Z coordinates of the bench mark to determine the reference coordinates for our topographic and bathymetry survey of the site.

Measurements were also taken along the top of the existing fractured seawall at the property. A water level measurement was taken at the beginning of the topographic survey as the reference datum to calculate near shore shallow water depths. The RTK/GPS used to record the relative location of site features including trees fence, eroded shoreline, steps and fractured seawall. The survey provided complete topographic data of ground surface and all site features including the water level, shoreline boundary and other site feature locations. The collected data was in the format of the Zone 17, NAD83 horizontal datum projection with X, Y, Z coordinates.

2.2 CROSS SHORE BATHYMETRIC PROFILE

During the field investigation, a single point water surface elevation was measured using the RTK/GPS close to the shoreline. The measured lake water surface elevation was compared and corrected with the Burlington Station water level above 74.2m IGLD chart datum. The Lake Ontario Water Level at the Burlington station on June 12, 2019 was 75.8945m. Using the single point water surface elevation as the reference point, the sounding depths were subtracted to get the elevation of the lake bed at the associated points.[Reference: 18 Lakeside Project] Figure-3 illustrates the 2 cross shore profiles for a total length of about 200m from offshore to the onshore seawall.

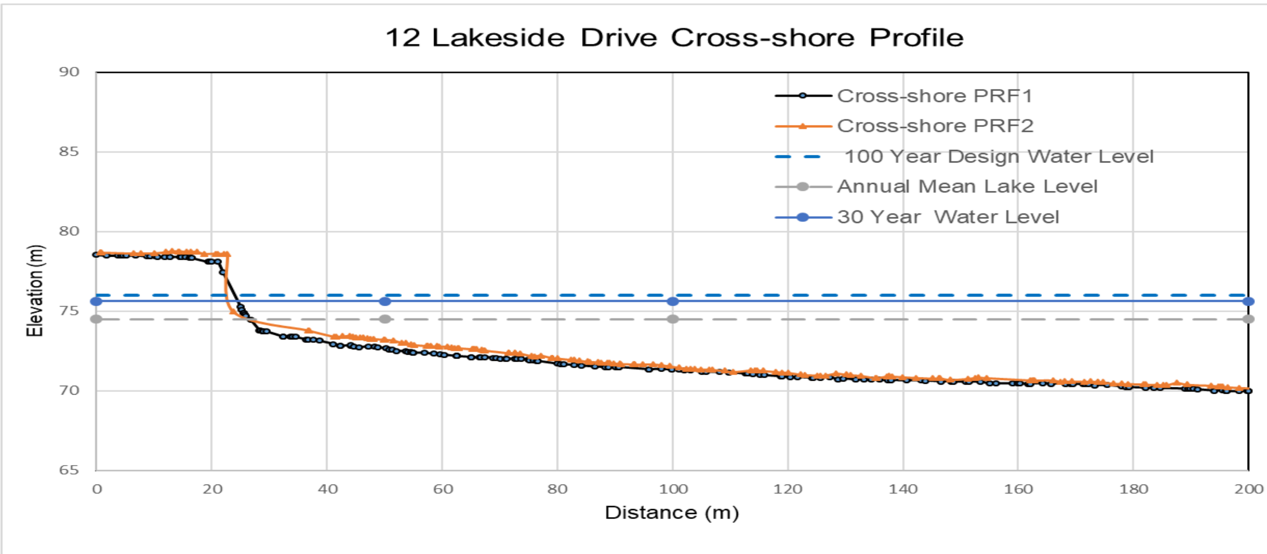


Figure 3: Cross-Shore Profile with Lake Bed Elevations

2.3 SHORELINE CHARACTERIZATION

Observed from the site visits, the shoreline can be categorized as an artificial shore backed by a steep vertical wall composed of cast concrete and steel sheet piles. Details of the shoreline characterization were documented on the field assessment form in Appendix A.

It was observed during the site visit that the existing seawall was fractured and undercut from erosion due to wave action, especially the northern portion. The shoreline type was categorized as headland bay and artificial concrete seawall with about 4m height, which was also the controlling structure for the shoreline. At the northern portion of the toe protection of structure, some concrete blocks have fallen (Figure 4 and Appendix C). There are steps constructed by concrete blocks at

Figure 4: Existing fractured Seawall of the Property (Facing North)

the shore, which provides an access to the water body. Along the sea wall, there is a groyne of 6.5m long from the existing seawall. The existing seawall cannot provide enough scour protection. Concrete wall near house is undercut from beach erosion. The seawall is composed of cast concrete and steal sheet piles which are in poor condition and exposed. Some fractures have developed near stairs and poured concrete were placed for repairs at the deck. The ground surface is majorly covered by concrete, bricks and sparse grass. A few small trees ware observed at both edges of the fence of the property owner. More details can be found from the site pictures in Appendix C.

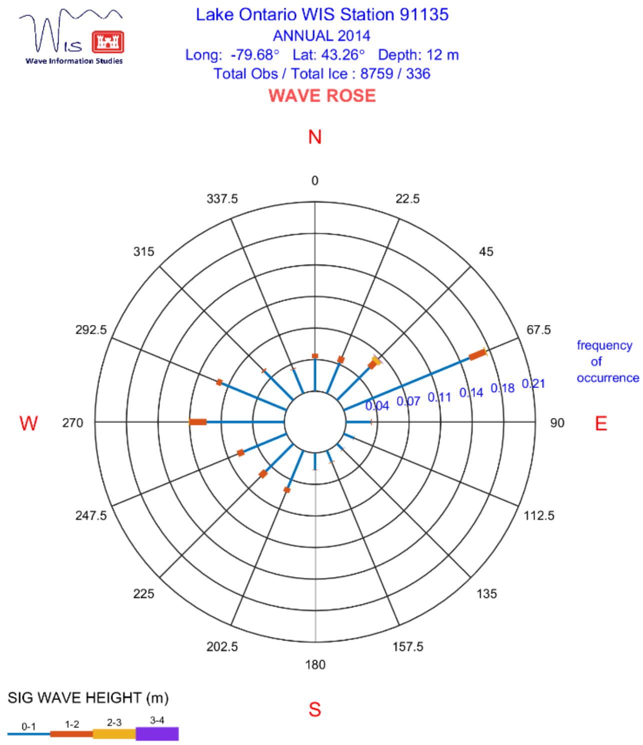
Figure 5: Existing Groin in Neighbor Property

3.0 COASTAL ANALYSIS

According to the MNR Technical Guidelines (2001), the regulated 100-year flood level for the Western Lake Ontario is 76m GSC (Geodetic Survey of Canada). As the project site is located on the Western Lake Ontario shoreline, we have also analyzed wind-wave data of the Western Lake Ontario for confirmation of the proposed seawall design.

3.1 ANALYSIS OF WIND-WAVE ENVIRONMENT

The Wave Information Studies (WIS) data collected by the United States Army Corps of Engineers (USACE) were used for the wind and wave frequency analysis at the project site. The project site is located on the shore north of Hwy QEW and Waterford Park in Stoney Creek. The site is closed to Lake Ontario WIS station 91135 (shown in Figure 2). Therefore, the data with record period from 1979 to 2014 of this station were used for this project. Figure 7 illustrates the wave rose graph generated by the USACE WIS for the significant wave height from all directions. It is observed from Figure 7 that the majority of the waves are coming from the west, northwest and northeast directions and a minority of waves come from all the other directions. As mentioned earlier, the shoreline at the project site is facing northeast. Figure 8 shows the wind speed from all the directions at the WIS station. The majority of winds and higher wind speed are coming from the west, southwest, and northwest directions. Any wind-wave coming from the northeast direction will have the greatest influence on the property shoreline.



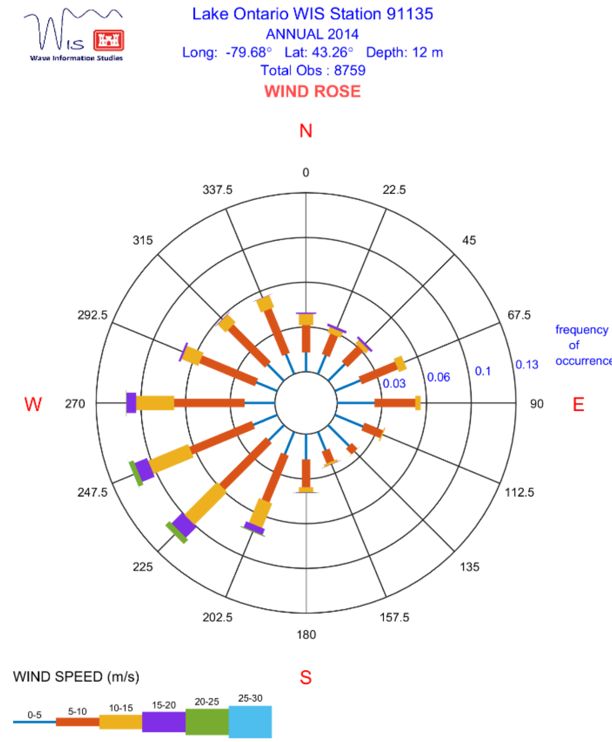


Figure 6: Wave Rose Graph for Significant Wave Height (US ACE, 2014)

Figure 7: Wind Rose Graph for Wind Speed (US ACE, 2014)

A frequency analysis for the wind speed and significant wave height was conducted and analyzed using the WIS data. It can be seen in Table 1 that there are 9 direction categories, one all directions category and 8 individual direction categories. The raw data from WIS has specific degree angles measured from true north rather than just stating the direction range. However, for the analysis in this study, 8 direction categories were adopted. The 8 direction categories, formed by dividing the 360-degree angle into 8 equal angles by 8 lines from the center, starting from the true north. Then the degree angles within the ±22.5 degree range from the true north were considered to be the north direction. All other directions were categorized in a similar way. The maximum annual wind and wave data categorized by 8 different directions and all directions was obtained by inputting the raw data into a programming code developed by Dr. Bahar SM. There are 36 years of data available. Each of the processed data sets for all the 9 direction categories were ranked from smallest to largest then distributed and extended to 100 years using lognormal, linear, or exponential distribution where appropriate. Then the values for the 10, 20, and 25 years return periods were estimated from the data trend line calculated by the distribution methods.

Table 1 and Table 2 represent the frequency analysis for wind speed and significant wave height for all directions and each individual direction for return periods 10, 20, and 25 years. As shown in Table 1, the wind speeds are the largest coming from the southwest direction among all the direction categories other than the all directions category. It also should be noted that the wind speeds from the southwest direction is much greater than the wind speeds from the southeast direction. Table 2 shows that the significant wave heights are the largest from the east direction among all direction categories other than the all directions category. The shoreline at the project site is facing northeast, and the significant wave height in that direction for 10, 20 and 25 years return period are 2.36, 2.93 and 3.11 respectively.

Table 1: Wind Speed Frequency Analysis

Wind Speed (m/s)			
Return Period	10	20	25
all directions	21.79	23.78	24.42
N	14.82	17.02	17.73
NE	15.59	17.81	18.52
E	16.38	18.66	19.40
SE	11.85	13.22	13.66
S	15.64	18.46	19.37
SW	20.33	22.99	23.85
W	19.62	21.38	21.94
NW	17.08	18.68	19.19

Table 2: Significant Wave Height Frequency Analysis

Return Period	10		20		25	
	HMO (m)	TP (s)	HMO (m)	TP (s)	HMO (m)	TP (s)
all directions	2.88	6.93	3.46	7.63	3.64	7.63
N	1.26	4.31	1.49	3.91	1.56	3.91
NE	2.36	5.73	2.93	6.93	3.11	6.93
E	2.67	6.93	3.29	6.93	3.48	7.63
SE	1.05	3.56	1.36	6.93	1.45	4.31
S	1.28	3.91	1.56	3.91	1.65	4.74
SW	1.34	3.91	1.54	3.91	1.60	3.91
W	1.20	3.23	1.33	3.56	1.37	3.56
NW	1.06	3.23	1.21	3.23	1.26	3.56

Note:

HMO is the significant wave height in metres

TP is the associated wave period for the HMO in seconds

3.2 SEAWALL DESIGN

The design of the seawall followed Coastal Engineering Manual USCAE, 1996, Table VI-5-53, 54 & 55. The manual suggested to use 30-year lake water level as seawall design level. This study has done frequency analysis of Water Survey Canada (WSC) station data (02HB017) at Burlington. The frequency analysis used 32 years of data records. The 30-year return period water level at the station is 75.63m, which is applied as the seawall design water level. Applying a free board of 1.07m into the design, it requires the minimum top of the seawall to reach an elevation of 76.70m.

In order to define safe or acceptable design height for coastal structures, traditional theories and methods are being applied for estimating size of the proposed seawall at the property shoreline. The seawall design applied D M Herbert method and the MNR Technical Guideline (see detailed design sheet in Appendix D) to estimate its stone size. The estimated D_{50} stone size will be 460.7mm, and the design stone size will be 762mm, after applying 1.65 factor of safety. The stone/concrete block size calculation has taken the design lake water level (75.63m) and toe elevation of the proposed structure (74.5m) into consideration. The calculation also considered the 2.67m off-shore significant wave height (Table 2) coming from the east direction.

The design followed the Coastal Engineering Manual USCAE (1996) and the Goda formula was modified to include impulsive forces from head-on breaking waves (Takahashi, Tanimoto and Shimosaka 1994a). The seawall design was checked for both low water level and design water level and wave height. Its stability was checked for wave, earth, hydrostatic, resultant normal and frictions forces. The detailed design calculations are shown in Appendix D. It can be seen that the proposed seawall design has a significant amount of factor of safety against stability for both the low water and design water levels. The proposed seawall design can generally be considered to have three major portions, starting from its foundation to its top. The first portion is the foundation of the proposed design. It will be constructed using two layers of existing seawall concrete blocks with dimensions of 1800mm X 1200mm X 600mm, a compact base with a thickness of 150mm at the bottom and a concrete pad with a thickness of 254mm at the top. The second portion lays above the first portion, including two layers of concrete blocks and a 2.5m wide concrete floor. The concrete blocks have the same dimensions of 1524mm X 762mm X 762mm and will be placed in a vertical order. A concrete beam will be constructed behind the top layer of concrete blocks to tie the two layers of concrete blocks, with rebars drilled into the side and the bottom. The concrete pad has a width of 2.5m and will be constructed at the top of the concrete blocks. Another three layers of smaller concrete blocks with dimensions of 1219mm X 600mm X 600mm will be placed at the end of the concrete floor, as the third portion of this design. Proper pipes and materials will be applied behind these three portions for drainage.

The seawall toe was designed to protect against scour by wave and wave-induced forces. The design has applied the USACE manual (1995) to calculate scour depth at the seawall toe. According to the manual, the scour depth will be equal or 1.5 times greater than breaking wave height at the shore. Therefore, the design toe protection scour depth can vary from 1.00 to 1.5m (See Appendix D).

More details of the proposed seawall design can be found in the drawing in Appendix B.

3.3 SHORELINE HAZARD LIMIT DELINEATION

The flooding hazard limit in Lake Ontario is determined based on the combination of the 100 year regulated flood level, the maximum wave uprush, and other water-related hazards. In the western Lake Ontario, wave uprush height is about 2.5m. Accordingly, the flooding hazard limit adopted for the 12 Lakeside Drive property is 78.5 m. Top elevation of the proposed shoreline protection structure is 78.5m, which is equal to the flooding hazard elevation. Therefore, the property will be flooding hazard free after construction of the proposed seawall. The hazard limit of the Great Lakes - St. Lawrence River system is defined by the combination of flooding hazards, erosion hazards, and dynamic beach hazards along a shoreline. As the property will have the artificial shoreline protection structure, there will not be any dynamic beach hazard, and thus, the assessment of dynamic beach hazard component is not necessary. The erosion hazard analysis performed by AHYDTECH is presented in the following sections.

AHYDTECH performed a desktop analysis following both the MNR and HCA guidelines and regulations. According to the “Understanding Natural Hazards Great Lakes – St. Lawrence River System and Large Inland Lakes, River and Stream Systems Hazardous Sites” introductory guideline (MNR, 2001), the erosion hazard is determined by the stable slope allowance plus the erosion allowance. The erosion allowance can be calculated as the product of the average annual recession rate times the 100 year time span or simply 20m as the erosion allowance if the average annual recession rate is not available. AHYDTECH has followed the erosion threatened area calculation method stated in MNR guideline (MNR, 2001). The HCA Planning and Regulation Policies and Guidelines (HCA, 2011) stated that the adopted hazard limit should be the furthest landward extent among flooding hazard, erosion hazards, and dynamic beach hazard, plus another 10m inland. The details of the erosion hazard determination are presented below.

Average Annual Recession and Erosion Allowance

AHYDTECH performed a desktop analysis using Digital Shoreline Analysis System (DSAS) in ArcGIS to determine the shoreline recession rate over a 40-year period. The historical changes in shoreline was analyzed using a Digital Shoreline Analysis System (DSAS 5) computer software which is an extension of ArcGIS. The Digital Shoreline Analysis System (DSAS) computes rate-of-change statistics from multiple historic Shoreline positions delineated in GIS. Three statistical methods were used to calculate the change in rates of shoreline from 1979-2019. The Methods Were **End Point Rate (EPR), Weighted Linear Regression (WLR), and Linear Weighted Regression (LWR)**. In DSAS work flow the EPR is calculated by dividing the displacement of the shoreline by the time (in years) elapsed between the oldest and the latest shoreline available.

12 Lakeside_Shoreline Recession

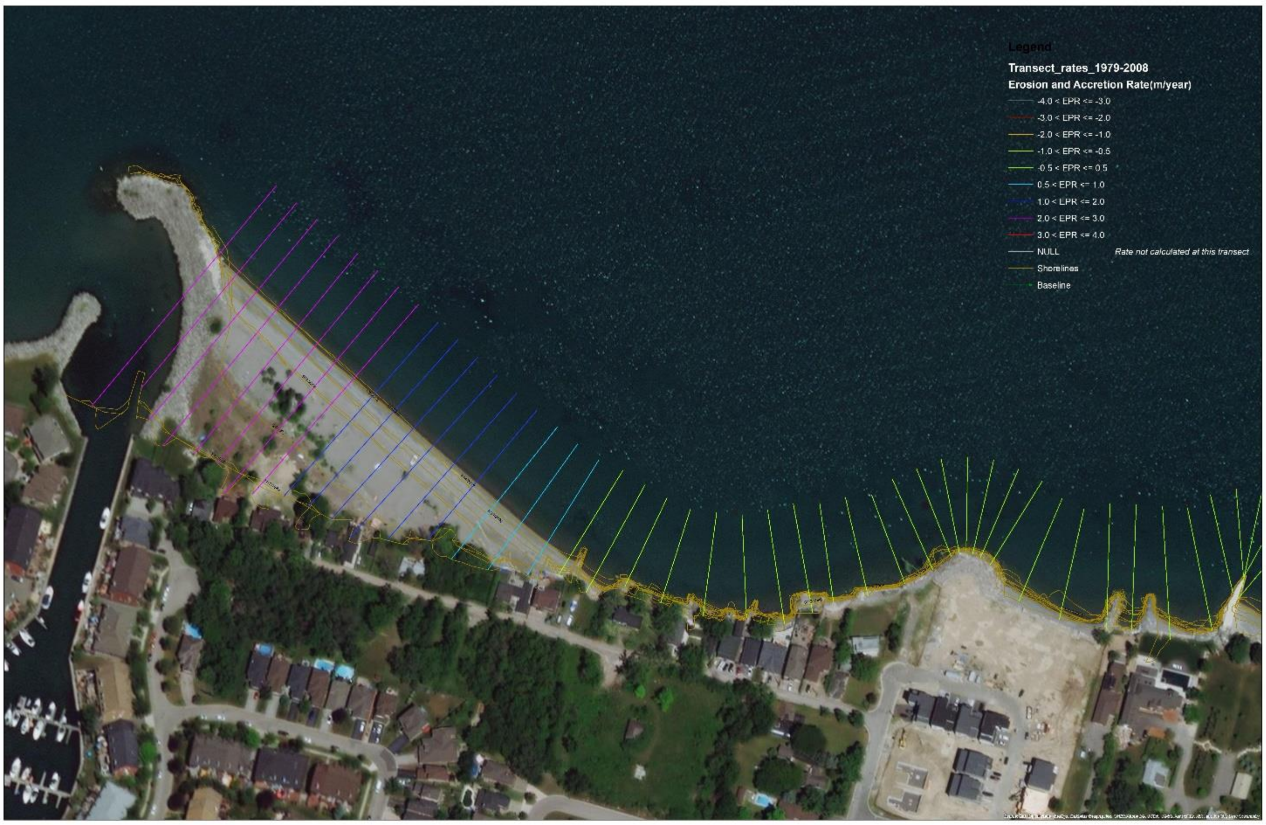


Figure 8: Rate of shoreline change (EPR m/year) along the shore from 1979-2019

The study area for the desktop analysis using DSAS in ArcGIS has been chosen from the mouth of the marina at Newport Yacht Club to 16 lakeshore drive. It covers approximately 2 KM shoreline along with south western Lake Ontario. The shoreline considered for this study is both natural dymanic beach and artificial beach in nature. There are several shoreline protection structures including seawall, revetment and groin. The shoreline was digitized from 1979, 1980, 1986, 1992, 2005, 2006, 2008, 2009, 2009, 2015, 2016, 2017 and 2018. A total of 56 transects along this were generated perpendicular to the shoreline with 15 m spacing and an average change rate was calculated from 1979 to 2019.

AHYDTECH performed shoreline recession/accretion analysis for the period 1979-2019 which revealed that most of the beach front underwent accretion with erosion observed in small patches. Summary of recession rates along the shores are given at the table below:

Transect ID	Recession/Accretion Rate 1979-2019	Transect ID	Recession/Accretion Rate 1979-2019
1	2.81	29	-0.05
2	2.15	30	-0.09
3	2.32	31	-0.02
4	2.36	32	0.04
5	2.26	33	0.05
6	2.17	34	0.07
7	2.1	35	0.07
8	2.06	36	0.08
9	1.96	37	0.01
10	1.84	38	0.27
11	1.76	39	-0.02
12	1.48	40	-0.03
13	1.47	41	0
14	1.09	42	-0.02
15	0.85	43	0
16	0.66	44	-0.01
17	0.56	45	-0.01
18	0.24	46	0
19	0.03	47	0.08
20	0.1	48	-0.05
21	0.07	49	0
22	0.06	50	0.04
23	0.04	51	0.01
24	0.02	52	-0.01
25	0.21	53	0
26	-0.01	54	-0.01
27	-0.02	55	-0.02
28	0	56	0

LONG-TERM AVERAGE	0.55

A comprehensive analysis was done to understand the recession and accretion pattern along 2KM shoreline which covers adjacent property shoreline of 12 Lakeside over 40-year span. From the analysis it was found that average rate of recession/accretion rate is +0.55m/year. It indicates most beaches front underwent accretion rather than erosion over 40-year period. Also, number of numbers of accretional transects is 37 out of 56. From here we can say most of the beaches face accretion and maximum value of accretion is 2.81m/year which is along transect ID 1. Transect ID 1 is located in front of 50 Lakeside drive property. Shoreline at that property can be characterized as natural dynamic beach. A large groin is situated adjacent to the beach. As a result of this large intervention the beach was deposited and formed as a natural dynamic beach over 40 year period. On the other hand, erosion is observed in small patches and 10.71% of all transects have faced statistically significant erosion. Erosional transects are situated more eastward than our 12 Lakeside. The erosional shoreline faces mostly northwest and as a result, these are facing erosion mostly due to the combined directional wave-wind effect and lack of intervention like Groyne. Maximum value of erosion among the erosional transects is only -0.09m/year which is very less compared to average rate of accretion/recession of the study area.

Transect ID 23 has been drawn perpendicular to the shoreline in front of 12 Lakeside drive property where the proposed seawall would be constructed. Accretion rate is found +0.04 m/year here, which means there is no erosion observed over the year (1979-2019). Its shoreline hardly has changed over the last 40 years. The maximum recession is 0.09m/year in Transect ID 30 (see Table below).

Transect ID	Recession/Accretion Rate
	1979-2019
23	0.04
24	0.02
25	0.21
26	-0.01
27	-0.02
28	0
29	-0.05
30	**-0.09**
31	-0.02

For the property at 12 lakeside drive, this recession rate (0.09m/year) is modest compared to the rate defined by the Ministry of Natural Resources (MNR) standards for the Great Lakes - St. Lawrence River system (MNR, 2001). After applying the 0.09 meters per year recession rate, the 100 years erosion allowance without shoreline structure will be 9m. However, this study is proposing a new seawall, which will have more than 50 years of life span. According to the HCA Planning and Regulation Policies and Guidelines (HCA, 2011), the property can get 50% credit of the Erosion Hazard Limit for the proposed sea wall. So, the erosion allowance for the property will be 4.5m.

Stable Slope Allowance

The Provincial Policy Statement (i.e., Policy 3.1) applies a two-step method for calculating the hazard limit. This method suggests estimating the stable slope allowance first and then account for the average annual rate of recession. According to the Provincial Standard, the 3 (Horizontal):1 (vertical) slope method is required apply to determine the stable slope allowance, if there is no geotechnical report on slope stability. In that case, stable slope profile is projected from the toe of the lake bed. The owner of the property retained SOIL-MAT ENGINEERS & CONSULTANTS LTD. for geotechnical and slope stability investigation of the property shoreline. SOIL-MAT recommends 2 (Horizontal): 1 (vertical) for the stable slope. After applying 2 (Horizontal): 1 (vertical) slope, the stable slope allowance will be 6.8m from the toe of the lake bed. The vertical distance is measured from the toe of the natural shoreline to the top of the first landward break. Then the horizontal distance is just two times of the vertical distance.

Development Setback

This study has applied both the MNRF Provincial Policy Statement (i.e., Policy 3.1) and the Hamilton Conservation Authority (HCA) Policies and Guidelines.

The erosion hazard limit is based on measurement of the stable slope allowance recommended by SOIL-MAT and add to it the average annual rate of recession, as shown below:

Erosion Hazard Limit = Stable Slope Allowance + Erosion Allowance

11.3 m = 6.8 m + 4.5m

With addition of the slope allowance (6.8m) and the erosion hazard allowance (4.5m), the total hazard limit from the toe of the natural shoreline will be 11.3 m.

This first existing wall is 17m from the main house, and the second existing wall distance is 13m. The total erosion hazard limit (Stable Slope Allowance + Erosion Allowance) would be 11.3m, which is within the second wall and the main house.

Therefore, the existing main building structure of the property is located outside, but the lake room is within the erosion hazard limit of 11.3m (See attached design drawing).

Sincerely,

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Managing Director
Coastal Engineer, Fluvial Geomorphologist
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4.0 REFERENCES

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APPENDIX A: FIELD ASSESSMENT FORM

APPENDIX B: SITE PHOTOS

Figure 1: Owner's House on shoreline seawall

Figure 2: Existing Seawall from deck

Figure 3: Fractured Seawall at right side of deck

Figure 4: Exposed steal sheet piles

Figure 5: Concrete Steps to the Lake

Figure 6: Scour under existing seawall

Figure 7: Fallen Concrete blocks underwater

Figure 8: Toe of the seawall

Figure 9: West side of existing sea

Figure 10: Toe of the seawall

Figure 11: Existing groin at neighbour property

Figure 12: Design crest elevation at neighbour deck

APPENDIX C: DESIGN CALCULATIONS

Stone/Concrete Block Size Calculation

Design Water Level	75.63
Toe of Structure =	74.00
h is the water depth at the toe of the structure	1.63
Hso is the offshore significant wave height	2.67
Rc is the height of the crest above still water level	0.47
h/Hso	0.610
Rc/Hso	0.176
Dimensionless Discharge $Q / (2gH^3_{so})^{1/2}$	0.01
Q, Discharge cms	0.193
Design Discharge, Qdesign cms	0.260885632
Design Discharge, Qdesign cfs	9.213097841
Embankment Slope	1
D_{50} in inch	18.13714276
D_{50} in meter	0.4607

1140 COASTAL ENGINEERING 1994

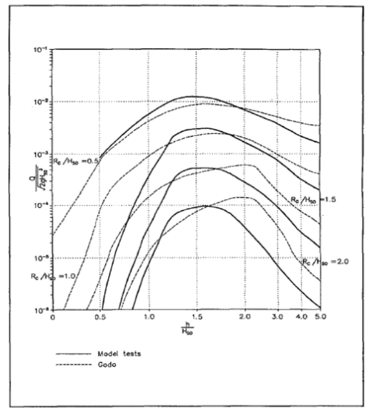


Figure 7 **Vertical wall discharges, 1:30 slope, s_{om} = 0.036**

The model data gave good agreement with the work of Goda. This is illustrated in Figure 7 where a dimensionless discharge, Q^*, is plotted against a dimensionless water depth, h/H_{so}, where:-

$$Q^* = Q / (2gH^3_{so})^{1/2} \quad (10)$$

Reference: Technical Guideline Page 113

Referenece: "Overtopping of sea walls under random waves" by D M HERBERT1, N W H ALLSOP1 and M W OWEN2, Page 1140

Wave Uprush and Overtopping: Methodologies and Applications
Great Lakes - St. Lawrence River System
Ontario Ministry of Natural Resources

Page 113
April 1997

In an attempt to determine the rip-rap layer stability for angular shaped stones when subjected to overtopping flow, the rip-rap layer median stone size D_{50} was correlated to the overtopping unit discharge at failure, q_f (q_f should be a momentary discharge per characteristic wave and not the time-averaged discharge, Q) and found to be:

$$D_{50} = 5.23\, S^{0.43}\, q_f^{0.56}$$

where S is the embankment slope (this function assumes a rip-rap specific gravity of 2.65).

Incipient stone movement occurred at approximately 74% of the rip-rap layer failure unit discharge. It is imperative that the rip-rap layer be designed to prevent failure therefore the median stone size should be sized to resist stone movement. To account for this Abt and Johnson recommend sizing the stone based on a design flow rate which is 1.35 times that of the overtopping flow rate:

$$q_{design} = 1.35\, q_f$$

In this way, the median stone size is designed to resist stone movement using the design unit discharge as follows:

$$D_{50} = 5.23\, S^{0.43}\, (q_{design})^{0.56}$$

where: D_{n50} in inches; and q in cubic feet per second.

It was determined that rounded stones should be oversized approximately 40% to provide comparable protection of angular stone. Also, flows can concentrate and form subchannels in the riprap layer. A flow concentration factor may be incorporated into the stone size analysis by multiplying q_f by a factor of approximately 1.0 to 3.0. The factor selected will depend upon the hazard level of the protected area.

Seawall Design Calculation

SEAWALL DESIGN: 12 Lakeside Drive, Stoney Creek

Coastal Engineering Manual USCAE, 1996, Table VI-5-53, 54 & 55
GODA Formula Modified to Include Impulsive forces from Head-on Breaking Waves (Takahashi, Tanimoto and Shimosaka 1994a)

A. DESIGN WATER LEVEL

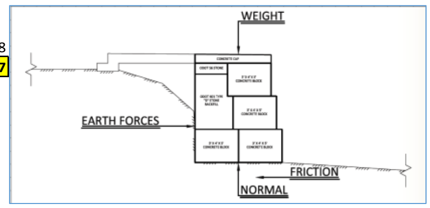
	Meter	
30 Year Design Water Level =	75.63	Reference : "Regulatory Flood Levels, March 1993, Table 4.1, page 4.1"

B. DESIGN WAVE HEIGHT

INITIAL DESIGN CASE		
Lake Bottom Elevation =	74.5	
Structure Depth = ds =	1.13	
Breaking Wave Height = Hb = 0.78 x ds =	0.8814	Reference : "Coastal Engineering Manual USCAE, 2006, page II-4-3"
CONSERVATIVE DESIGN CASE		
Toe of Structure =	74.5	
Structure Depth = ds =	1.13	
Breaking Wave Height = Hb = 0.78 x ds =	0.8814	Reference : "Coastal Engineering Manual USCAE, 2006, page II-4-3"

C. SEAWALL CREST ELEVATION

INITIAL DESIGN CASE	
Wave Crest Elevation = DWL + 0.7 Hb =	76.24698
CONSERVATIVE DESIGN CASE, IF TOE STRUCTURE IS SCOURED	
Wave Crest Elevation = DWL + 0.7 Hb =	76.24698
CONSERVATIVE DESIGN CASE: SET SEAWALL CREST ELEVATION AT	76.24698
Design Crest Elevation	76.7



D. EXTERNAL STABILITY

CONSIDER 2 DESIGN CASES

1. WATER AT MEAN LOW LEVEL
2. DESIGN WATER LEVEL & WAVE HEIGHT

CASE 1: LOW WATER LEVEL

DETERMINE FORCES: I. STRUCTURE CROSS SECTION WEIGHT, II. NORMAL FORCE, II. FRICTION, IV. EARTH FORCES

I. STRUCTURE CROSS SECTION WEIGHT

COMPONENT	B Width / Diameter	Height / Diameter (m)	Area (m2)	Density/Unit WT (kg/m3)	NO.	Mass/m (Kg/m)	Weight/m (ton/m)
Armour Stone Block	0.762	0.762	0.580644	2400	3	4180.6368	4.1806368

II. NORMAL FORCE = WEIGHT =		4.18	Weight/m (Ton/m)

III. FRICTION

Angle of Internal Friction, α = 35 assumed	35.00	
Coefficient of Static Friction, μ =TAN α =	0.70	
Friction = N μ =	2.93	Ton/m

IV. EARTH FORCES

hw = Overall Height of Structure	1.75	m
β = Slope of Backfill =	0.00	
ϒ = Unit Weight of Backfill	2600.00	kg/m3
ϕ = Angle of Internal Friction of Backfill =	40.00	

Ka = Active Earth Coefficient = $TAN^2(45-\phi/2)$	0.22	
FE = 1/2 ϒ hw^2 Ka COS β =	862.71	Kg/m
	0.86	Ton/m

SLIDING STABILITY

Stability Forces = Friction =	2.93	Ton/m
Anti-Stability Forces = Earth Forces =	0.86	Ton/m
Factor of Safety = Stability Forces/AntiStability Forces	3.39	

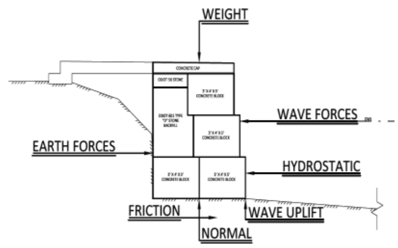
Overturning Stability - Calculate Moments about Structure Toe

STABILIZING MOMENT

Structure Weight =	4.18	Ton/m
Moment Arm =	0.76	
Stabilizing Moment =	3.19	m-Tom/m

ANTI-STABILIZING MOMENT

Earth Forces =	0.86	
Moment Arm =	0.76	
Anti-Stabilizing Moment =	0.66	m-Tom/m
Factor of Safety = Stabilizing/Anti-Stabilizing =	**4.86**	

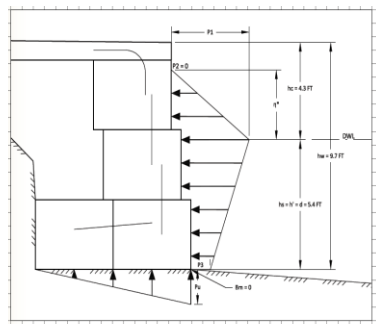


CASE 2: DESIGN WATER LEVEL & WAVE HEIGHT

DETERMINE FORCES: I. STRUCTURE CROSS SECTION WEIGHT, II. WAVE FORCES, III. EARTH FORCES, IV. HYDROSTATIC (BOUYANT) FORCES, V. NORMAL FORCES (RESULTANT), VI. FRICTION

DESIGN ASSUMPTIONS For Conservative Design Assume High Lake Water Level & Low Ground Water Level (Seperated by Seawall). Hydrostatic Forces Must be Considered

I. STRUCTURE CROSS SECTION WEIGHT	4.1806368	(Ton/m)



II. WAVE FORCES

WAVE FORCE CALCULATION BASED ON GODA FORMULA FOR IRREGULAR WAVES MODIFIED TO INCLUDE IMPULSIVE FORCES FROM HEAD ON BREAKING WAVES AND ADAPTED TO THE GEOMETRY OF THE PROPOSED SEAWALL.
REFERENCE: "COASTAL ENGINEERING MANUAL" USACE, 1996, TABLES VI-5-53, 54 & 55.

P1 = Wave Pressure at Design Water Level	
P2 = Wave Pressure at Wave/Seawall Crest Height	
P3 = Wave Pressure at Base of Seawall	
PU = Wave Uplift Pressure at Base of Seawall	
η* = Wave Crest Height	
hs = Depth of Structure at Toe	1.13
d = Water Depth of Structure at Toe	1.13
h' = Total Depth of Structure	1.13
β = Angle of Incedence of Design Waves = 0 Degree	0.00
H design = Design Wave Height = Hb =	0.88
hw = Overall Height of Structure =	2.20
hc = Height of Structure Above DWL =	1.07
Bm = Width of Rubble Foundation =	0.00
Ts = Wave Period (assumed for Hb)	3.56
hb = Water Depth at a Distance 5 X Hdesign from Seawall = hs + 5 x Hdesign X TAN φ =	1.21
L = Wave Length at Water Depth hb = Ts (g hb)**0.5	12.25

Assuming Design Wall Has 45 degree angle (1:1 slope)

DETERMIINE MODIFICATIONS TO GODA FORMULA

$$\delta_2 = \begin{cases} 4.9 \cdot \delta_{22} & \text{for} \quad \delta_{2\,2} \le 0 \\ 3 \cdot \delta_{22} & \text{for} \quad \delta_{22} > 0 \end{cases}$$

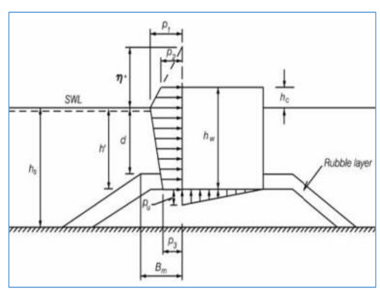
$$\delta_{22} = -0.36\left(\frac{B_m}{L} - 0.12\right) + 0.93\left(\frac{h_s - d}{h_s} - 0.6\right)$$

δ_{22} =	-0.51
δ_2 =	-2.52

$$\delta_1 = \begin{cases} 20 \cdot \delta_{11} & \text{for} \quad \delta_{11} \le 0 \\ 15 \cdot \delta_{11} & \text{for} \quad \delta_{11} > 0 \end{cases}$$

$$\delta_{11} = 0.93\left(\frac{B_m}{L} - 0.12\right) + 0.36\left(\frac{h_s - d}{h_s} - 0.6\right)$$

δ_{11} =	-0.33
δ_1 =	-6.55



$$\alpha_* = \text{largest of } \alpha_2 \text{ and } \alpha_I$$

$$\alpha_2 = \text{the smallest of } \frac{h_b - d}{3\,h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}}$$

$$\alpha_I = \alpha_{I0} \cdot \alpha_{I1}$$

$$\alpha_{I0} = \begin{cases} H_{design}/d & \text{for} \quad H_{design}/d \le 2 \\ 2.0 & \text{for} \quad H_{design}/d > 2 \end{cases}$$

$$\alpha_{I1} = \begin{cases} \dfrac{\cos\delta_2}{\cosh\delta_1} & \delta_2 \le 0 \\ \dfrac{1}{\cosh\delta_1 \cdot (\cosh\delta_2)^{\frac{1}{2}}} & \delta_2 > 0 \end{cases}$$

$$\delta_1 = \begin{cases} 20 \cdot \delta_{11} & \text{for} \quad \delta_{11} \le 0 \\ 15 \cdot \delta_{11} & \text{for} \quad \delta_{11} > 0 \end{cases}$$

α_{I0} =	0.78
α_{I1} =	0.00
$\alpha_I = \alpha_{I0} \cdot \alpha_{I1}$	0.00
α_2 =	0.01
α_2 =	2.56
Smaller α2 =	0.01
α_* =	0.01

STRUCTURE MODIFICATION FACTORS

λ1 = λ2 = λ3 = 1.0 for Conventional Vertical Wall Structures	1.00

DETERMINE PRESSURE COEFFICIENT FOR GODA FORMULA

α* = 0.014 (Modified α* for Impulse Forces)	0.01

$$\alpha_* = \alpha_2$$
$$\alpha_1 = 0.6 + 0.5\left[\frac{4\pi h_s/L}{sinh\,(4\pi h_s/L)}\right]^2$$
$$\alpha_2 = \text{the smallest of } \frac{h_b - d}{3\,h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}}$$
$$\alpha_3 = 1 - \frac{h_w - h_c}{h_s}\left[1 - \frac{1}{cosh\,(2\pi h_s/L)}\right]$$

α_1 =	0.93
α_2 =	0.01
α_3 =	0.85

$$\eta^* = 0.75(1 + cos\beta)\,\lambda_1 H_{design}$$
$$p_1 = 0.5(1 + cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_* cos^2\beta)\,\rho_w\, g\, H_{design}$$
$$p_2 = \begin{cases} \left(1 - \frac{h_c}{\eta^*}\right)p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \leq h_c \end{cases}$$
$$p_3 = \alpha_3\, p_1$$
$$p_u = 0.5(1 + cos\beta)\lambda_3\alpha_1\alpha_3\rho_w g\, H_{design}$$

η* =	1.32	
P1 =	827.20	Pa
P2 =	157.73	
P3 =	705.47	
PU =	6818.40	

DETERMINE LEVELS OF UNCERTAINITY

DETERMINE LEVELS OF UNCERTAINTY
FOR HORIZONTAL FORCE, UFH = 0.90
FOR UPLIFT FORCE, UFU = 0.77
FOR HORIZONTAL MOMENT, UMH = 0.81
FOR UPLIFT MOMENT, UMU = 0.72

REFERENCE: "COASTAL ENGINEERING MANUAL" USACE, 2006, TABLE VI-5-55.

CALCULATE WAVE FORCES PER LINEAR METER OF STRUCTURE

Horizontal Wave force, FH = UFH (1/2 (P1+P2)hc + 1/2(P1+P3)h' =	1253.61	Kg/m
Wave Uplift Force, FU = UFU x 0.5 PU x B =	2625.08	Kg/m
B = 1 meter		

III. EARTH FORCES

$$FE = \tfrac{1}{2}\,\gamma\, hw^2\, Kp\, COS\,\beta$$

β = Slope of Backfill = 0 degree	0.00	
hw = Overall Height of Structure =	2.20	m
ϒ = Unit Weight of Backfill Rock =	2400.00	kg/m3
ϕ = Angle of Internal Friction of Backfill =	40.00	
Kp = Passive Earth Coefficient = $TAN^2(45+\phi/2)$	4.60	

FE = 1/2 ϒ hw^2 Kp COS β =	26710.47	Kg/m
	26.71	Ton/m

IV. HYDROSTATIC FORCES

ϒw = Density of Water	1000.00	kg/m3
h = Water Depth at the Toe of the Structure =	1.13	

Fhydro = 1/2 ϒw h^2 =	638.45	Kg/m
	0.64	Ton/m

V. RESULTANT NORMAL FORCES

Resultant Vertical force, N = WEIGHT - WAVE UPLIFT =	1555.55	Kg/m
	1.56	Ton/m

V. FRICTION

Angle of Internal Friction, α = 35 degrees	35.00	
Coefficient of Static Friction, μ = TAN α =	0.70	
FRICTION = N μ =	1089.21	Kg/m
SLIDING STABILITY		
Stabilizing Forces = FRICTION + EARTH FORCES	27799.68	Kg/m
Anti-Stabilizing Forces = WAVE FORCES + HYDROSTATIC FORCES	1892.06	Kg/m

FACTOR OF SAFETY = STABILIZING FORCES/ ANTI-STABILIZING FORCES	14.69

STRUCTURE MODIFICATION FACTORS

λ1 = λ2 = λ3 = 1.0 for Conventional Vertical Wall Structures	1.00

DETERMINE PRESSURE COEFFICIENT FOR GODA FORMULA

α* = 0.014 (Modified α* for Impulse Forces)	0.01

$$\begin{aligned}
\alpha_* &= \alpha_2 \\
\alpha_1 &= 0.6 + 0.5\left[\frac{4\pi h_s/L}{sinh\,(4\pi h_s/L)}\right]^2 \\
\alpha_2 &= \text{the smallest of } \frac{h_b - d}{3h_b}\left(\frac{H_{design}}{d}\right)^2 \text{ and } \frac{2d}{H_{design}} \\
\alpha_3 &= 1 - \frac{h_w - h_c}{h_s}\left[1 - \frac{1}{cosh\,(2\pi h_s/L)}\right]
\end{aligned}$$

α_1 =	0.93
α_2 =	0.01
α_3 =	0.85

$$\begin{aligned}
\eta^* &= 0.75(1+cos\beta)\,\lambda_1 H_{design} \\
p_1 &= 0.5(1+cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_* cos^2\beta)\,\rho_w\, g\, H_{design} \\
p_2 &= \begin{cases} \left(1 - \frac{h_c}{\eta^*}\right) p_1 & \text{for } \eta^* > h_c \\ 0 & \text{for } \eta^* \le h_c \end{cases} \\
p_3 &= \alpha_3\, p_1 \\
p_u &= 0.5(1+cos\beta)\lambda_3\alpha_1\alpha_3\rho_w g\, H_{design}
\end{aligned}$$

η* =	1.32	
P1 =	827.20	Pa
P2 =	157.73	
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DETERMINE LEVELS OF UNCERTAINITY

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III. EARTH FORCES

$$FE = \frac{1}{2}\gamma\, hw^2\, Kp\, COS\, \beta$$

β = Slope of Backfill = 0 degree	0.00	
hw = Overall Height of Structure =	2.20	m
ϒ = Unit Weight of Backfill Rock =	2400.00	kg/m3
ϕ = Angle of Internal Friction of Backfill =	40.00	
Kp = Passive Earth Coefficient = $TAN^2(45+\phi/2)$	4.60	

FE = 1/2 ϒ hw^2 Kp COS β =	26710.47	Kg/m
	26.71	Ton/m

IV. HYDROSTATIC FORCES

ϒw = Density of Water	1000.00	kg/m3
h = Water Depth at the Toe of the Structure =	1.13	

Fhydro = 1/2 ϒw h^2 =	638.45	Kg/m
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V. RESULTANT NORMAL FORCES

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V. FRICTION

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Coefficient of Static Friction, μ = TAN α =	0.70	
FRICTION = N μ =	1089.21	Kg/m
SLIDING STABILITY		
Stabilizing Forces = FRICTION + EARTH FORCES	27799.68	Kg/m
Anti-Stabilizing Forces = WAVE FORCES + HYDROSTATIC FORCES	1892.06	Kg/m
FACTOR OF SAFETY = STABILIZING FORCES/ ANTI-STABILIZING FORCES	**14.69**	

OVERTURNING STABILITY - CALCULATE MOMENTS ABOUT STRUCTURE HEEL

STABILIZING MOMENTS

Structure Weight =	4180.64	Kg/m
Moment Arm = width/2 = 0.5	0.38	
STABILIZIING MOMENT =	1592.82	Kg-m/m
Earth Forces =	26710.47	Kg/m
Moment Arm = Height of Structure/3	0.73	
STABILIZIING MOMENT =	19587.68	Kg-m/m
TOTAL STABILIZING MOMENT =	21180.50	Kg-m/m

ANI-STABILIZING MOMENTS

Horizontal Wave Force = FH =	1253.61	
Moment Arm = UMH x hc =	0.87	
ANTI-STABILITY MOMENT =	1086.50	Kg-m/m
Uplift Wave Force = FU =	2625.08	
Moment Arm = UMU x h' =	0.81	
ANTI-STABILITY MOMENT =	2135.77	Kg-m/m
Hydrostatic Force =	638.45	
Moment Arm = hs/3 =	0.38	
ANTI-STABILITY MOMENT =	240.48	Kg-m/m
TOTAL ANTI-STABILIZING MOMENT =	3462.75	Kg-m/m

FACTOR OF SAFETY = STABILIZING MOMENT/ ANTI-STABILIZING MOMENT 6.12

TOE SCOUR DEPTH

Design of Revetments, Seawalls and Bulkheads, USCAE, 1995

Hb, Breaking Wave Height

Toe Scour Depth, (Hb≤a≤1.5Hb), m	0.88
Design Toe Scour Depth, m	1.32

APPENDIX D: DESIGN DRAWINGS

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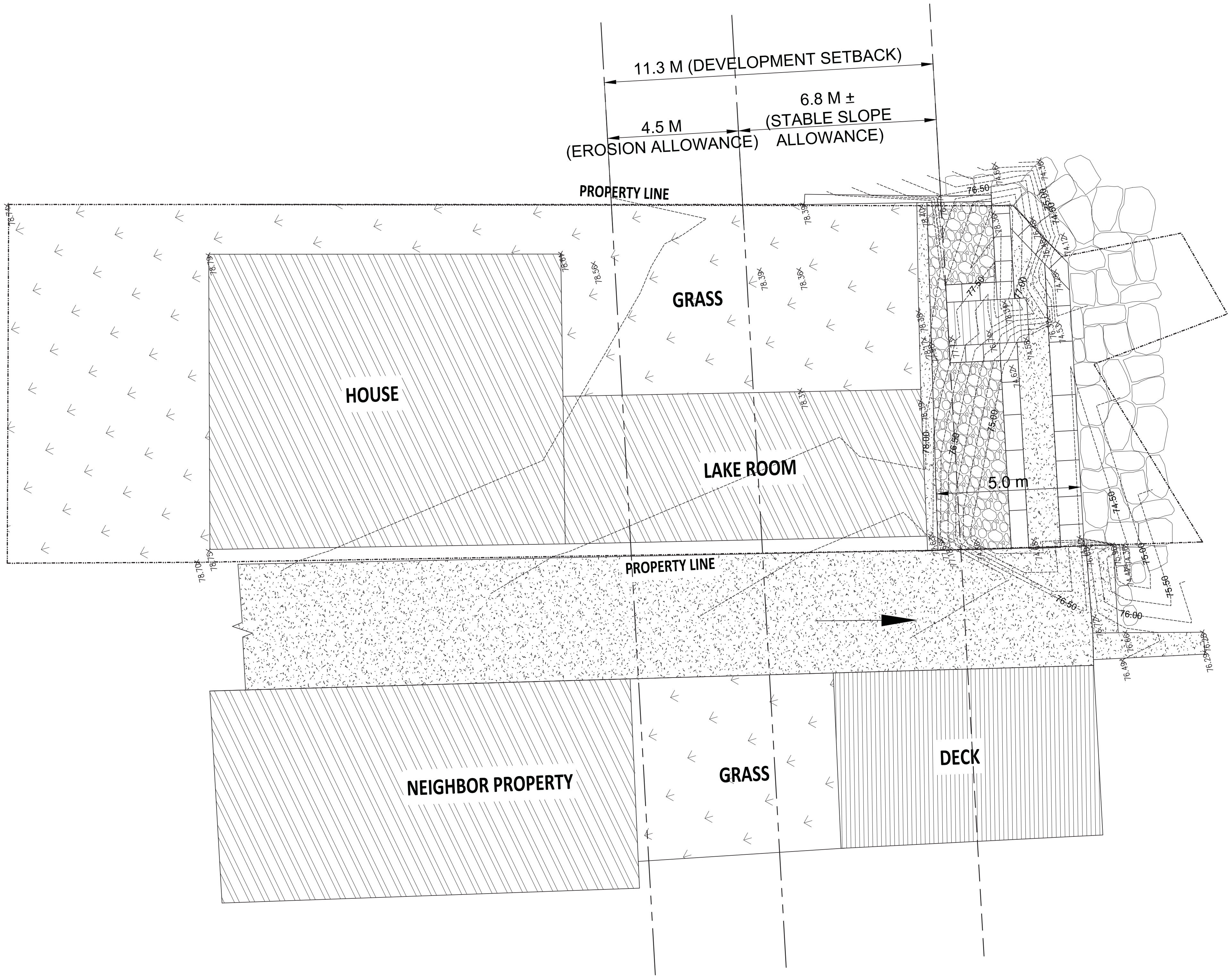
6.3.2 Annex - VII



12 LAKESIDE DRIVE
SHORELINE HAZARD ASSESSMENT
SCALE-1:75

NO.	REVISION	BY	MM/DD/YY
1.	1ST SUBMISSION	MONIRUZZAMAN	06/25/19
2.	2ND SUBMISSION	MONIRUZZAMAN	06/30/19
3.	3RD SUBMISSION	MONIRUZZAMAN	04/15/20
4.			
5.			
6.			

DESIGNED BY:
Dr. Bahar SM

APPROVED BY:

DRAWN BY:
Moniruzzaman Rayhan
DATE: (MM.DD.YYYY)
05.14.2020
SHEET No.
2 of 2
DWG. No.
2

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SHORELINE HAZARD ASSESSMENT
FOR
12 LAKESIDE DRIVE