MAHONE HARBOUR FLOOD PREVENTION AND SHORELINE ENHANCEMENT PLAN

Mahone Bay, Nova Scotia

Final Report

CBCL LIMITED

Consulting Engineers

Prepared for

Mahone Bay

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CHAPTER 1 INTRODUCTION

1.1 Background

The Town of Mahone Bay is a charming historical town located on the South Shore of Nova Scotia in Lunenburg County. Combining the world famous "Three Churches" with a breathtaking view of the Bay attracts passersby's to the town.

The coastal zone in the town is a popular location for residential housing and a site of much commercial and recreational activity. However, the Town of Mahone Bay, like many waterfront towns, is highly vulnerable to the effects of climate change, including sea level rise and storm surge. The Town's valuable assets compound these risks significantly. While coastal hazard is the primary concern in terms of flooding in the town, flooding from the Ernst Brook is also considered in detail. The river normally drains into the bay but can back up with seawater during times of high water, which can result in flooding of streets and other low lying areas.

In 2015, the Town of Mahone Bay retained CBCL to conduct a flood prevention and shoreline enhancement plan involving the development of aesthetic flood mitigation solutions to address coastal and inland flooding.

1.2 Purpose of Study

The primary objectives of this project is to provide flood protection along Ernst Brook and enhance the shoreline to mitigate the impact of sea level rise and extreme weather events to protect businesses, private property, public spaces, retail and tourism in the Town of Mahone Bay. It is recognized that the whole bayfront is important to the character and appeal of the Town. The purpose of this study is not only making sure that the town is appropriately protected from flooding, but also making sure that those efforts to protect the area result in solutions that maintain or enhance the beauty and quality of life in the community.

This study included a comprehensive analysis of river and sea levels, as influenced by tides, wind and rainfall. Detailed modelling was conducted to provide a better understanding of the underlying causes of flooding, steering the assessment towards more efficient flood mitigation options. The detailed modelling also provided technical support for offering the most achievable and cost-effective overall flood mitigation plan to protect the Town from flooding.

The scope of the project spans Mahone Harbour from Mushamush River to Mader's Cove to the south. The northern portion between Ernst Book and the Mushamush River is dominated by public facilities, such as the walking trails, parking, open space, as well as the Three Churches. The southern portion from Ernst Brook to Mader's Cover is largely private property, with the exception of the Town wharf.

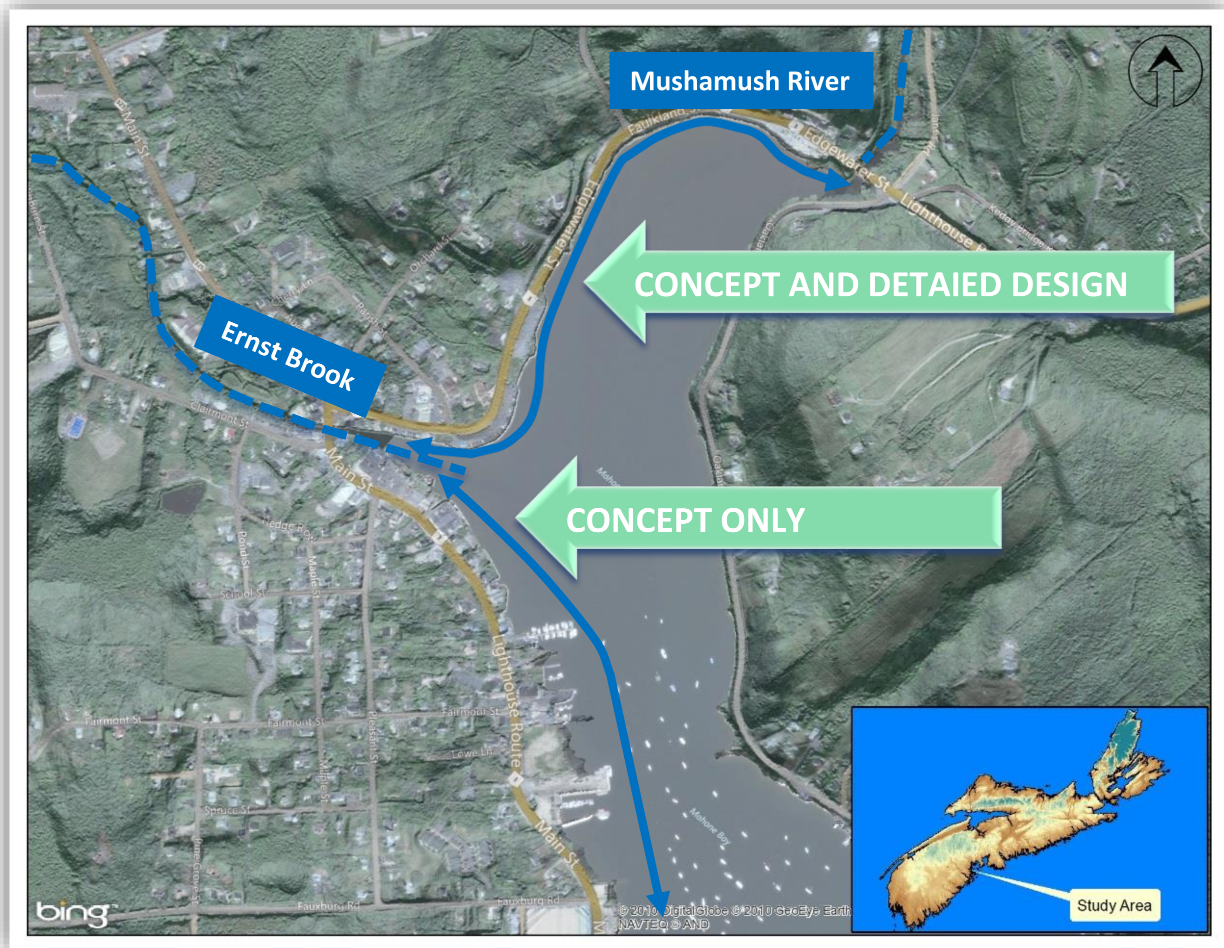


Figure 1.1: Study Location

1.3 Study Approach

The main parts of this study have included the following:

1. Screening of strategies for coastal adaption.
2. Conducting coastal flood mitigation assessment.
3. Performing stormwater and riverine flood mitigation analysis.
4. Evaluating and costing of multiple flood mitigation approaches and options.

These main parts have included the following steps:

- Meeting with the town's project team to start the project and review information requirements and project details;
- Obtaining and reviewing all available background material concerning the project, including previous reports, air photos, weather data, soil mapping and bedrock geology, etc.;

- Obtaining as much flooding extents information (including anecdotal) as possible to delineate the extents of previous flood events. This was used as a calibration data during this study;
- Obtaining bathymetric data from CHS navigation chart to build the numerical models;
- Obtaining information on all man-made changes to shoreline that may either be contributing to flooding, or may be providing a weak zone that would be at risk of collapsing, potentially aggravating the flooding risks;
- Coordinating the review and conversion of LiDAR data to a Digital Elevation Model;
- Conducting site reconnaissance to assess the environmentally sensitive features and identify habitat types;
- Carrying out topographic survey on shoreline construction, roadways, driveways, drainage structures, and wharfs to aid in the Concept Design;
- Updating the delineation of the watersheds tributary to the study area using the LiDAR and other available topographic data;
- Conducting a Radar-Rainfall analysis calibrated on ground rainfall gauges to obtain detailed information on storm patterns during calibration events;
- Investigating the shoreline evolution to identify potential erosion trends;
- Assembling computer models of the hydraulic system as well as a dynamic model (SWMM5 with the PCSWMM platform) and a tidal and wave model (MIKE 21). These models are complementary and each offers specific technical advantages over the other;
- Conducting comprehensive modelling to resolve the tidal peak water level characteristics of the site, as well as a tidal climate change impact analysis;
- Examining extreme water levels and wave heights to develop conceptual shoreline protection and enhancement options;
- Performing peak river water level calculations with the models, taking into account both tidal influence and rainfall;
- Identifying existing risks of flooding, for the 1 in 100 year event, by delineating and mapping floodlines throughout the study area;
- Identifying and evaluating temporary and permanent flood control options with the models, as well as non-structural measures. Conduct a cost analysis for each and rank them by cost- effectiveness;
- Preparation of Draft Concept Design Report for review by the town’s project team; and
- Preparation of the Final Report including any final adjustments.

CHAPTER 2 COASTAL SITE CHARACTERIZATION

The coastal protection component of the study examines coastal flooding and overtopping risks to determine appropriate design options to mitigate coastal flooding risk. The present chapter describes the current state of the shoreline and investigates water levels and wave conditions. Chapter 3 uses the information and development options for shoreline protection.

2.1 Exising Shoreline Condition

Kedy's Parking Lot - A narrow salt marsh fronts the parking lot. The marsh is wider to the east where it is supported by a mound of rocks. There are no signs of active erosion.

Edgewater Road - The shoreline is eroding, as evidenced by exposed tree roots and unstable road embankment. North of the Three Churches, Edgewater road was built by partly infilling the harbour, which would probably have led to the loss of salt marshes that existed at the time. The new embankment was too steep and/or made of unsuitable material to support new marsh growth.

Three Churches Parking Lot - The infilled parking lot is fronted by a gravel trail with a steep layer of large round armour stones that would typically be in the few 100's kg weight range, i.e. diameter 0.5 m ± 0.3 m. Some vegetation is growing between the stones in the upper slope, which indicates that storm events overtopping the structure are infrequent. However the round stones are not interlocking and there is evidence of (and ample room for) gravel washouts through the voids between the round stones. This seawall would probably suffer serious damage under a high storm surge and wave overtopping scenario. Engineered armour stone revetments are typically built using at least 2 layers of irregular, interlocking armour rock, which can be less aesthetically pleasing than the round stone used presently.

Ernst Brook Outlet – the brook outlet is constructed of concrete retaining walls on each side connected to a road bridge.

Town Waterfront – Private properties are mostly all fronted by round-stone seawalls as described above, which are unconnected and separated by wharves and/or boat ramps coming down from the road.

Lighthouse Route – The low-lying road is located along an exposed section of shoreline with partial rip rap or armour stone protection against erosion, and with occasional shore-perpendicular structures.

Figure 2.1: Shoreline Photos – North Section along Edgewater Street

Private seawalls along Town waterfront

Figure 2.2: Shoreline Photos – Southern Section along Town Waterfront

Figure 2.3: Shoreline Photo – Lighthouse Route Southeast of Town Waterfront Looking Northwest (Top) and Southeast (Bottom)

2.2 Bathymetry

Bathymetric data is presented in **Figure 2.4 and 2.5.** Bathymetry data for areas outside the harbour was obtained from CHS navigation chart and used to build the numerical models. The head of the harbour is shallow and muddy, and depths increase along the town waterfront going southward.

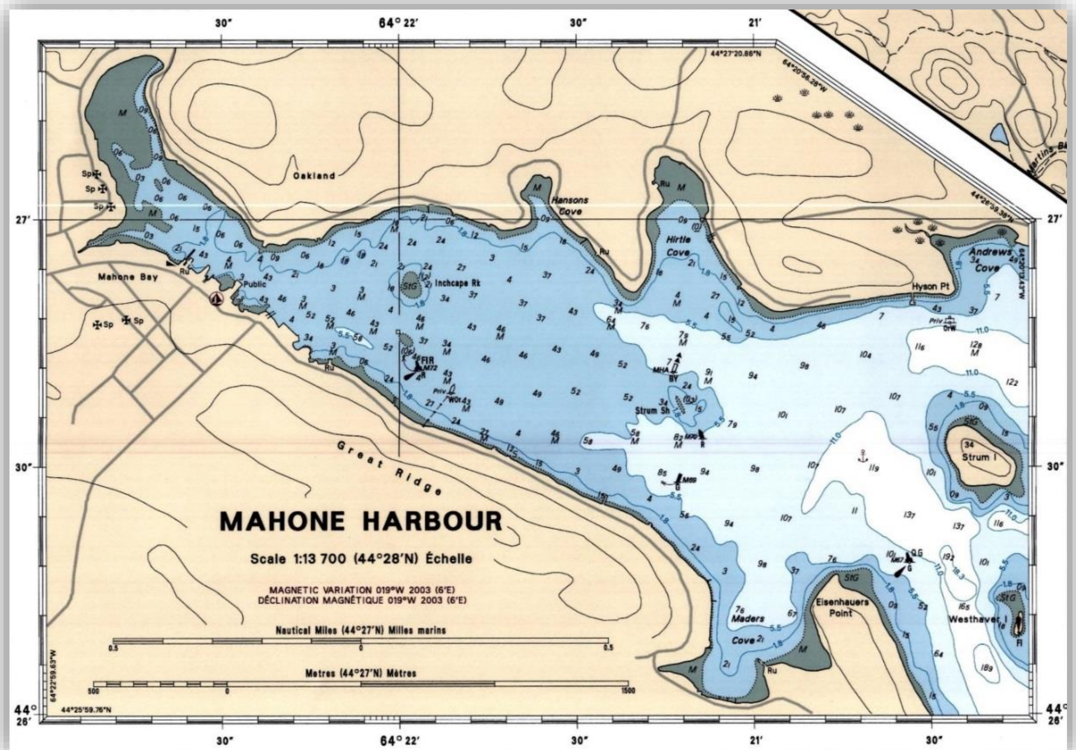


Figure 2.4: Bathymetry Reproduced from CHS Chart # 4381 (Depths in Metres CD)

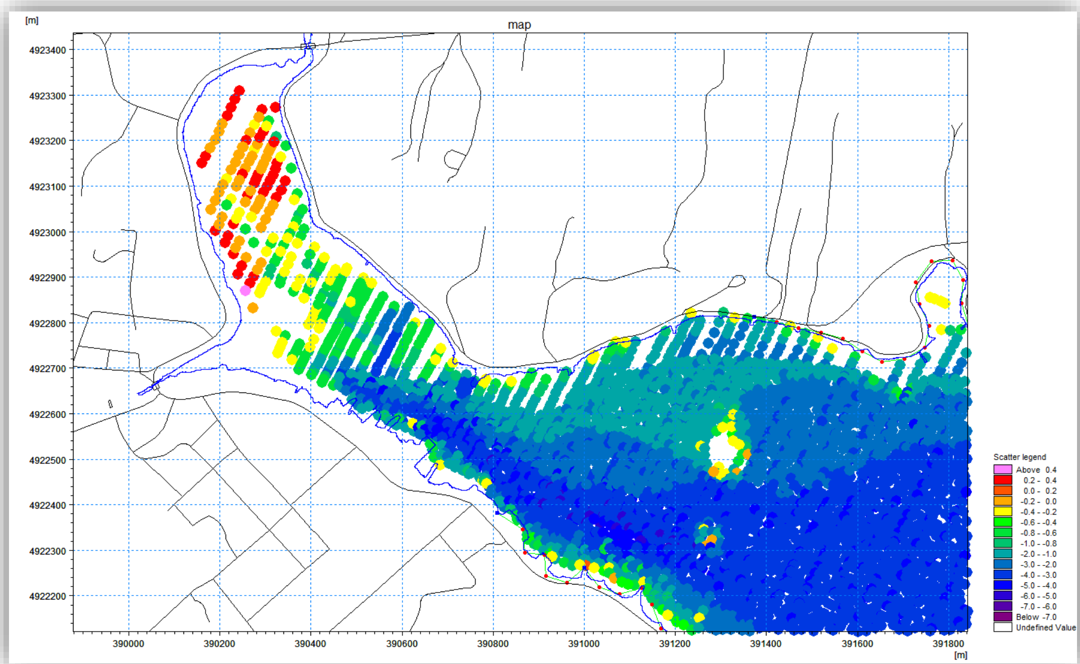


Figure 2.5: Soundings Provided by DFO (Depths in Metres CD)

2.3 Historical Shoreline Evolution

As part of the background coastal investigation, historical air photos were acquired to investigate the shoreline evolution and infer potential erosion trends (**Figure 2.6**). The shoreline has been historically infilled and hardened along Edgewater Street.

Discussion with stakeholders also indicated that some of the bottom deposits (including fine sediments and structural remnants) covering the head of the harbour may have originated from previous industrial uses of the areas, including ship building and lumber processing.

Some of the hard structures would have slowed down erosion, notably in front of the three churches. The infilling steepened the natural shoreline slopes likely over previous salt marshes. The steep slopes, and hard structures have significantly reduced the space available for estuarine and marsh lands.

The design options presented in the following sections seek to remediate these disadvantages while enhancing flood and erosion protection.

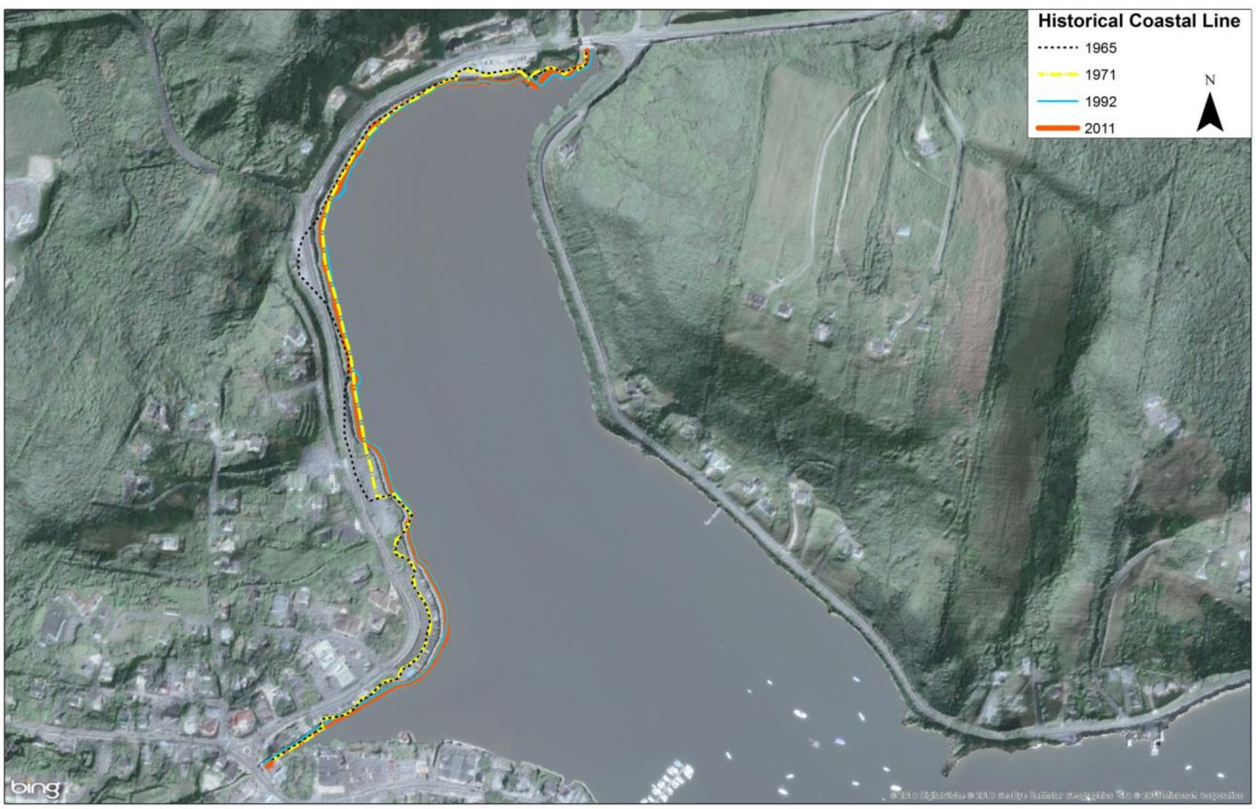


Figure 2.6: Mahone Bay Historical Shoreline Evolution

2.4 Water Levels

2.4.1 Tides

Local water levels are influenced by the combined effects of tides, storm surge and sea level rise. The local tide is semi-diurnal, with a maximum range of 2.2 m. Tidal elevations were derived from DFO values at Lunenburg. Upon consultation with DFO an extra 0.1 m factor was added for sea level rise that occurred since approximately year 2000 in Halifax when the tidal datums were last updated.

2.4.2 Storm Surge

Storm surges are created by meteorological effects on sea level, such as wind set-up[1] and low atmospheric pressure, and can be defined as the difference between the observed water level during a storm and the predicted astronomical tide. Regional storm surge trends can be inferred from large-scale models, such as Environment Canada's storm surge model (**Figure 2.7**).

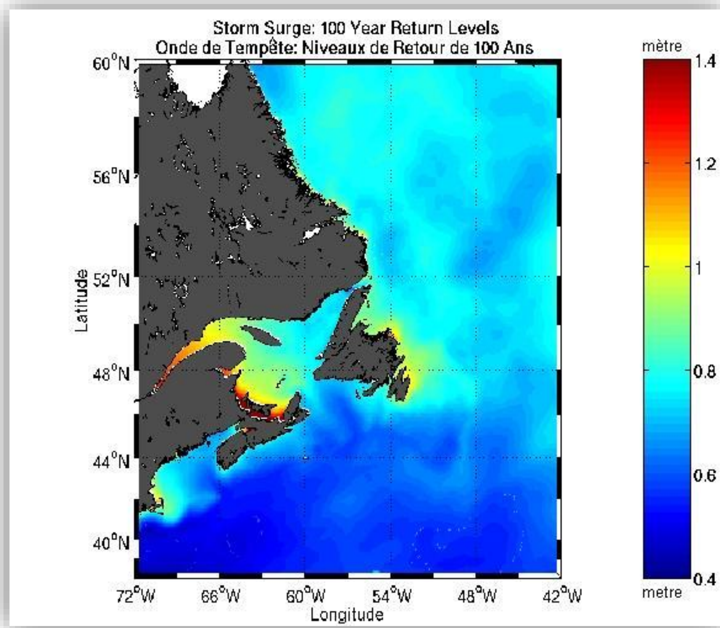


Figure 2.7: Modeled 100-year Return Storm Surge Levels in Atlantic Canada
Source: Environment Canada

On a local scale, extreme storm surge statistics are best derived from long-term tide gauge data, and the closest source is at Halifax. In order to investigate the applicability of storm surge statistics derived from the Halifax tide gauge to Mahone Harbour, we conducted additional hydrodynamic modelling work using the Danish hydraulic Institute's MIKE21 Cyclone Generation and Hydrodynamic models to simulate a known event, i.e. Hurricane Juan. The model domain covered Atlantic Canada with high resolution in Mahone Bay and Halifax Harbour. We input hurricane track, wind and atmospheric pressure data into a coupled wave and hydrodynamic model to simulate the storm surge. The modelled peak storm surge at the Halifax tide gauge was 1.5 to 1.6 m, which is consistent with the tide gauge record of a 1.5 m peak

[1] Wind set-up refers to the increase in mean water level along the coast due to shoreward wind stresses on the water surface.

for this event (**Figure 2.8**). The modelled peak storm surge in Mahone Bay was 0.8 m and occurred approximately 40 min before it peaked in Halifax Harbour.

An alternative simulation was then run with Hurricane Juan making landfall in Mahone Bay, after shifting its track 55 km to the West (**Figure 2.9**). The modelled peak storm surge in Mahone Harbour was then 1.5 m, i.e. comparable to what it actually was on that event in Halifax Harbour. We conclude that the storm surge statistics from the Halifax tide gauge can be applied for Mahone Harbour.

Storm surge statistics were derived from the long-term hourly tide gauge data at Halifax for the period 1919 to 2015. The extreme values are based on the time-series of water level peaks that was de-trended to the 2015 mean sea level. The resulting storm surge residuals (i.e. after removal of tide) were applied to Mahone Bay, after correcting for the difference in tidal range to obtain estimates of extreme water levels listed in **Table 2.1**. A safety factor of 0.1 m is added to account for the possibility of mean sea level fluctuations within 2-3 years, as shown by an analysis of mean sea level in Halifax.

Table 2.1: 2015 Tides and Extreme Still Water Levels (metres above 2015 Chart Datum)

Note: "Still Water Level" refers to water levels (tidal or extreme storm surge) without wave run-up.

MAHONE BAY YEAR 2015	m Chart Datum (CD) 2015	m CGVD28 2015	CD to CGVD28 conversion 1.079
HHWLT - Higher High Water Large Tide	2.60	1.52	
HHWMT - Higher High Water Mean Tide	2.20	1.12	
MWL - Mean Water Level	1.40	0.32	
LLWMT - Lower Low Water Mean Tide	0.70	-0.38	
LLWLT - Lower Low Water Large Tide	0.40	-0.68	

		Extreme water levels m CGVD28				
Year		**2015**	**2045**	**2065**	**2100**	**2115**
Upper-bound (95%) Sea Level Rise relative to 2015 (DFO 2014, based on IPCC RCP8.5)		0	0.29	0.53	1.08	1.35
HHWLT - Higher High Water Large Tide		1.52	1.81	2.05	2.60	2.87
Storm water levels for probabilistic coastal analyses	1-year	1.86	2.15	2.39	2.94	3.21
	10-year	2.14	2.43	2.67	3.22	3.49
	25-year	2.26	2.55	2.79	3.34	3.61
	50-year	2.35	2.64	2.88	3.43	3.70
	100-year	2.44	2.73	2.97	3.52	3.79
Plausible upper-bound levels if storm surge hits at HHWLT	100-year storm surge residual (1.15 m)	2.67	2.96	3.20	3.75	4.02
	Hurricane Juan Halifax storm surge residual (1.5 m)	3.02	3.31	3.55	4.10	4.37

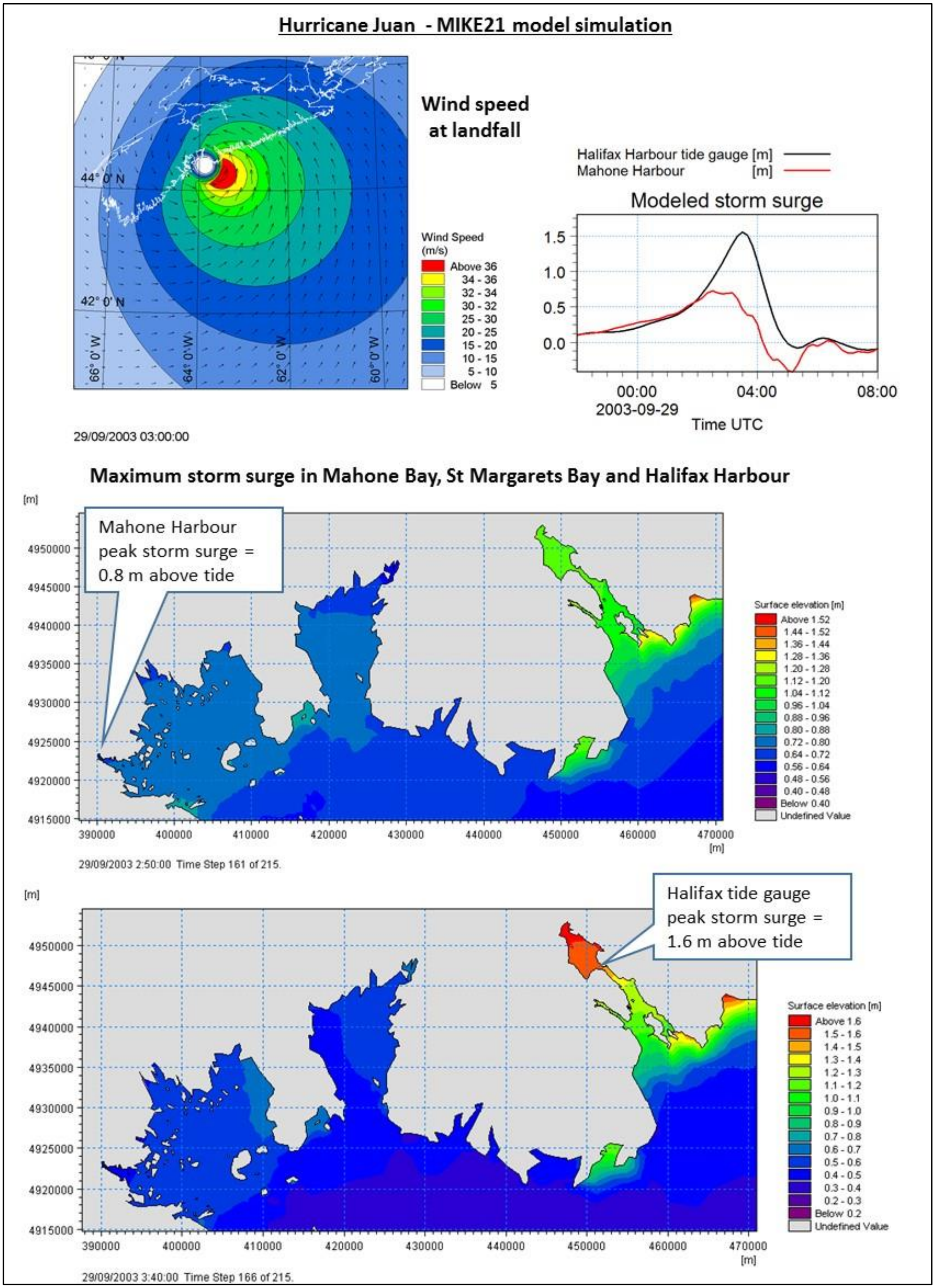


Figure 2.8: Hurricane Juan Storm Surge Model Simulation

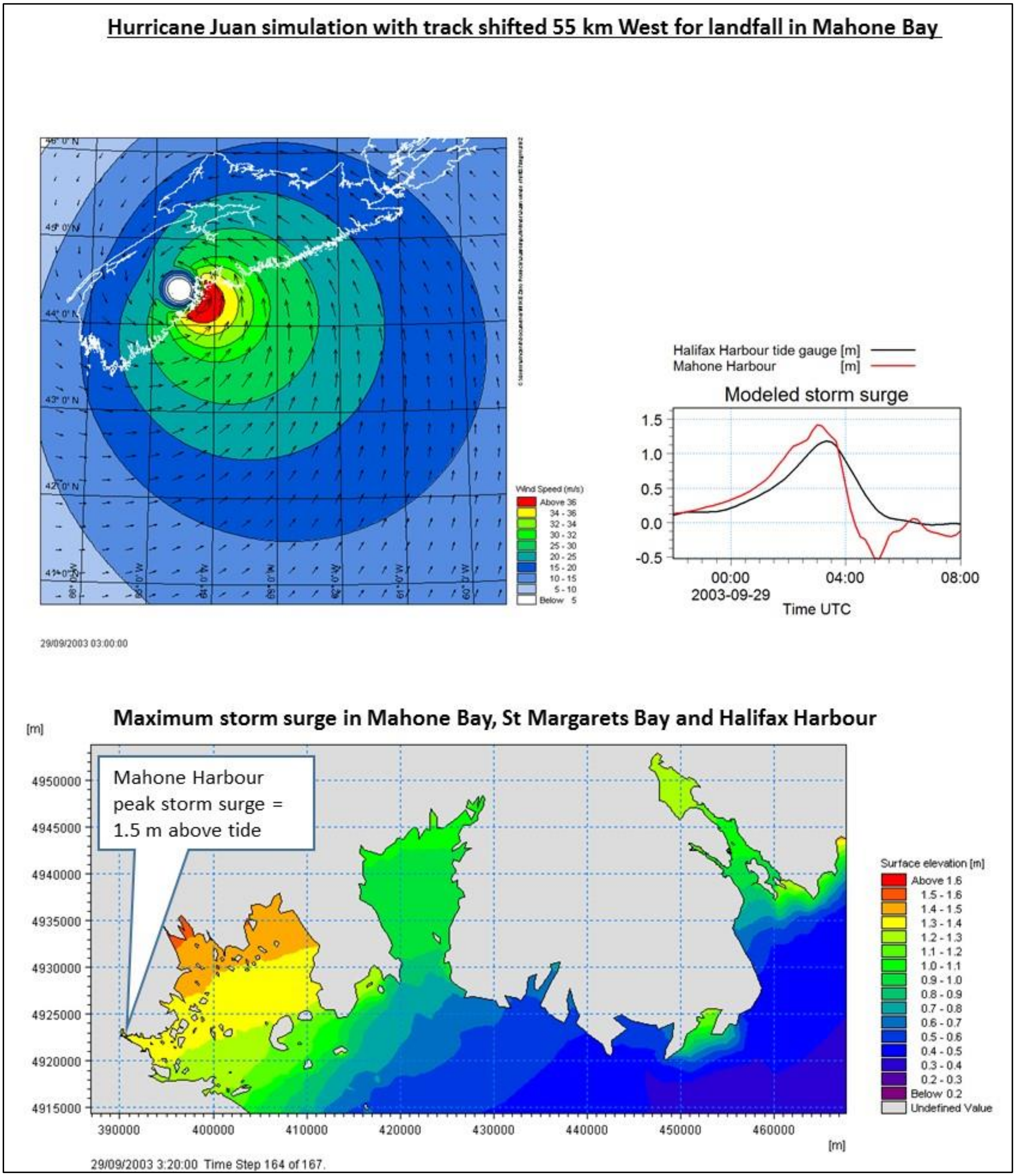


Figure 2.9: **Storm Surge Model Simulation Based on Hurricane Juan Making Landfall in Mahone Bay**

2.4.3 Sea Level Rise

Sea Level Rise (SLR) along eastern Canada's coast has been occurring since the end of the last ice age, about 10,000 years ago. The tide gauge observations in Halifax show a historical SLR rate of 3.2 mm/year in the last century, with a steeper rise of almost 100 mm in the last 15 to 20 years (**Figure 2.10**). The rate of global mean SLR is accelerating in the 21st century due to global warming impacts, notably the melting of polar ice caps. The Intergovernmental Panel on Climate Change (IPCC AR5, 2013) indicates that the current consensus is as follows:

- The likely range of global mean SLR for 2081-2100 relative to 1986-2005 was estimated from 0.26 m (lower bound value for low emission scenario) to 0.98 m (higher bound estimate for high emission scenario);
- There will be regional differences, with the north eastern coast of North America potentially experiencing a SLR rate higher than the global average; and
- There is currently insufficient evidence to evaluate the probability of specific levels above the assessed likely range. The probability cannot yet be reliably estimated because of difficulties in modelling ice sheet melting and associated feedback mechanisms on sea level rise. However recent research emphasizes that multi-metre sea level rise may happen faster than expected if greenhouse gas emissions are not curtailed – see Hansen et al, 2015.

DFO's Canadian Technical Report of Hydrography and Ocean Sciences 300 (Zhai et. al., 2014) presents sea level rise projections at different sites along the coasts of Canada. The local estimates are determined based on the Representative Concentration Pathway (RCP) scenarios of the projections of regional sea-level rise from the IPCC Fifth Assessment Report (AR5, IPCC 2013). For each scenario, DFO's estimates are given within a 5% to 95% confidence bracket. Given the requirement for long-term protection, the permanent nature of the infrastructure considered for this study, and the aforementioned notes of caution regarding IPCC projections, the future water levels values are based on the high emission scenario (RCP 8.5) and 95% upper bound sea level rise projections for Halifax. This translates into 0.53 m of sea level rise in the next 50 years (2015 to 2065).

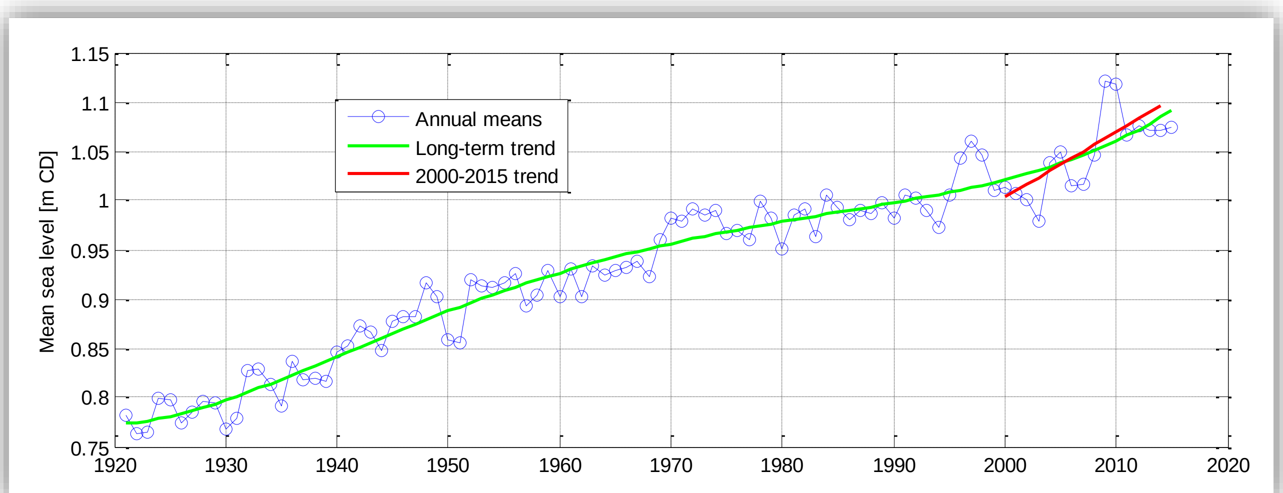


Figure 2.10: Observed Sea Level Rise at Halifax

2.4.4 Future Extreme Water Levels

Given the SLR projections, extreme water levels with a low return period today will be very common in a few decades, therefore increasing the potential damage frequency. Calculations were made for return periods (**Table 2.1**) and probabilities of exceedance (**Figure 2.11**) of extreme water levels into the future accounting for sea level rise, for using the results in the design process. Based on the following assumptions:

- Sea level rise projections from DFO derived from IPCC AR5 2013 using the upper-bound 95 percentile estimates of the RCP8.5 scenario; and
- Storm surge statistics from the Halifax tide gauge, the recommended minimum elevation for waterfront structures with a 50-year lifetime is rounded up to **2.8 m CGVD28** (2.75 m rounded to the nearest 1/10 m) which does not account for wave overtopping in exposed areas. The required elevation for a 100-year lifetime would be 3.6 m.

The calculated coastal flood lines due to SLR and storm surges are mapped on **Figure 2.12** for the study area, based on LiDAR elevation data provided by the Town. The map identifies vulnerable areas for which mitigation options were developed.

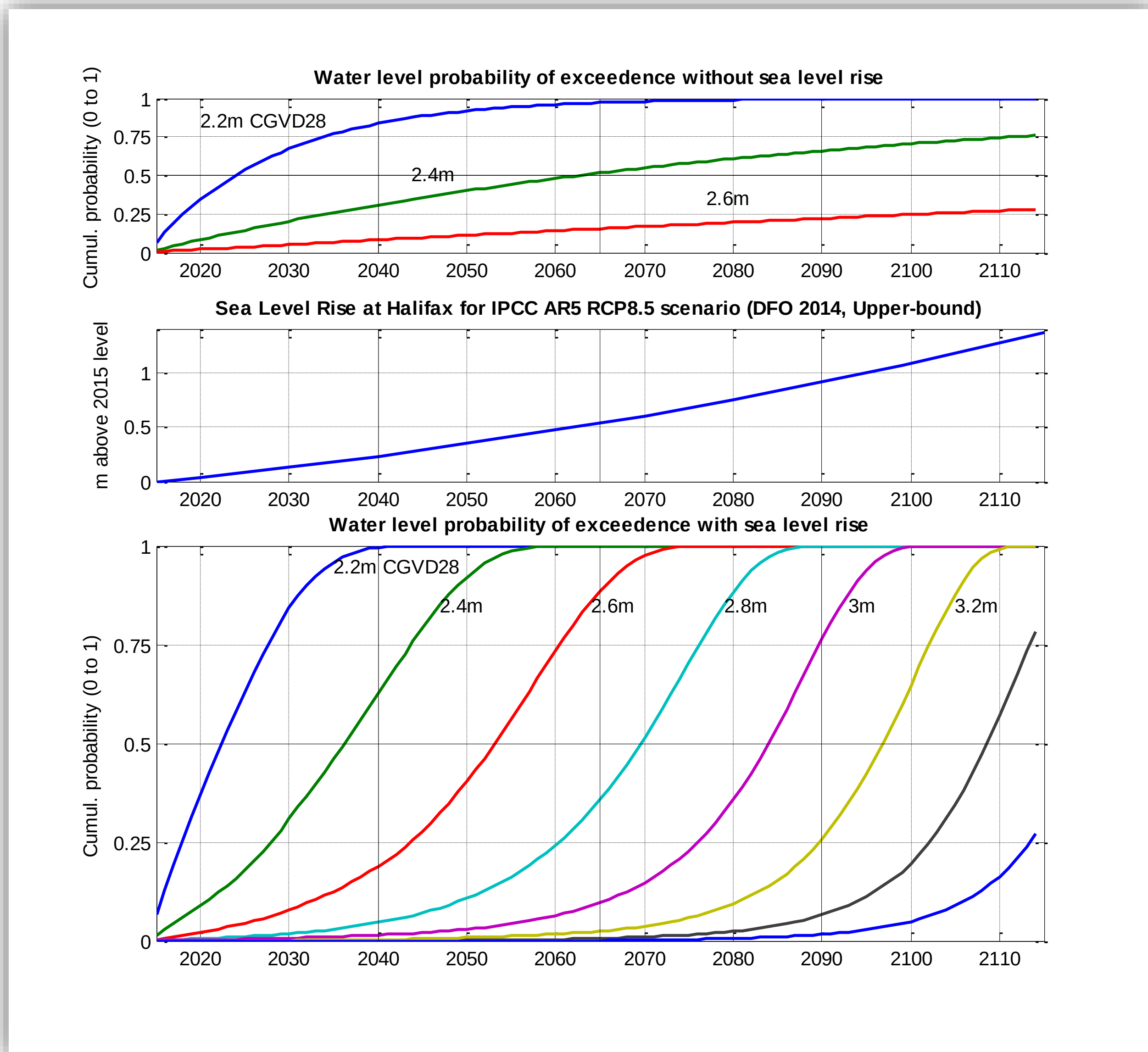
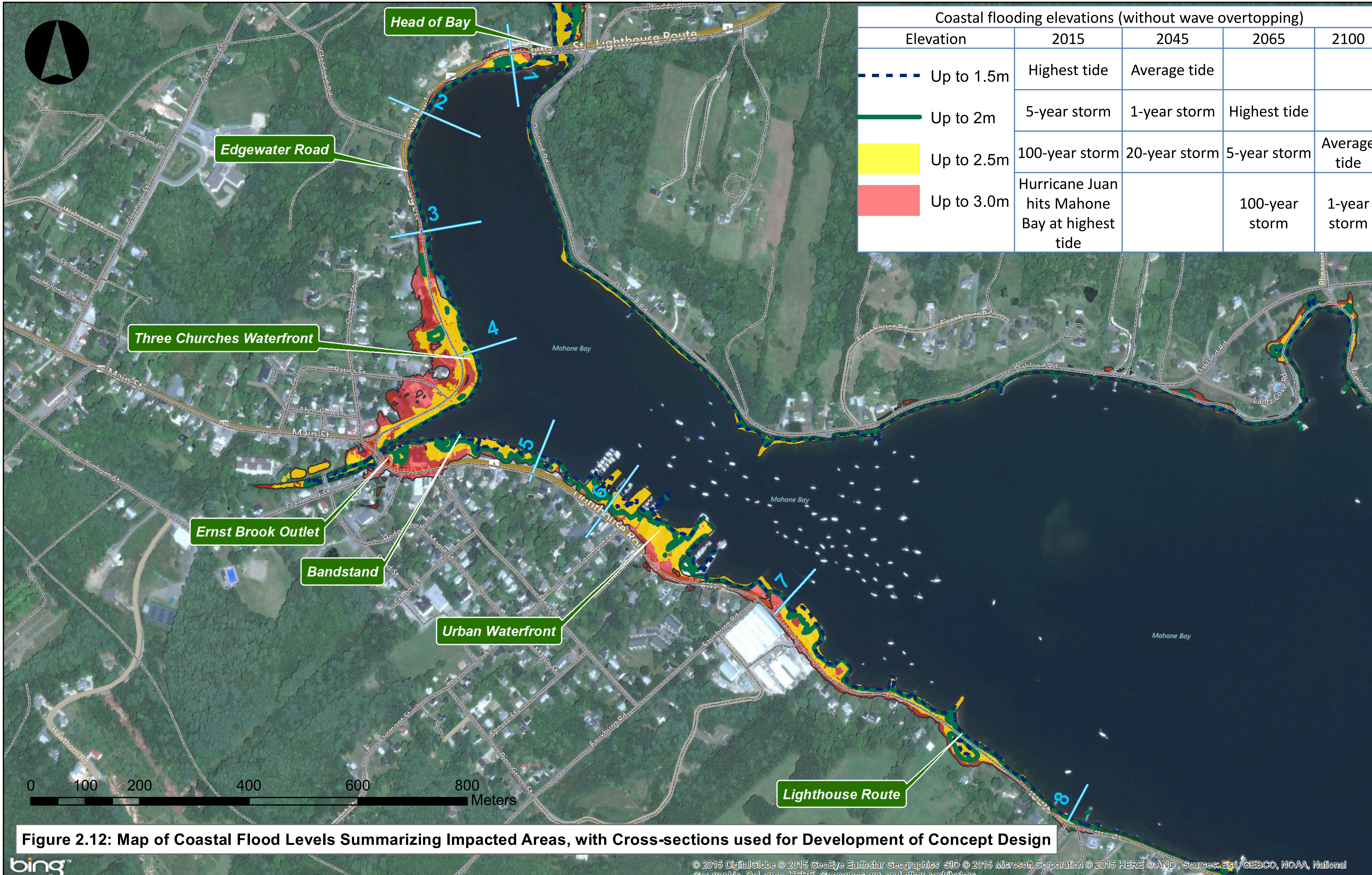


Figure 2.11: Impact of Projected Sea Level Rise on Water Level Statistics over the Next 100 Years

For example, a typical coastal or waterfront structure with a 50-year lifetime (i.e. to year 2065) may be designed such that the cumulative probability of flooding does not exceed 50% over its lifetime. The required elevation is rounded to **2.8 m CGVD28** (it would have been 2.4 m without sea level rise). This does not yet account for wave overtopping, which is addressed next. The required elevation for a 100-year lifetime would be 3.55 m.



Coastal flooding elevations (without wave overtopping)

Elevation	2015	2045	2065	2100
Up to 1.5m	Highest tide	Average tide		
Up to 2m	5-year storm	1-year storm	Highest tide	
Up to 2.5m	100-year storm	20-year storm	5-year storm	Average tide
Up to 3.0m	Hurricane Juan hits Mahone Bay at highest tide		100-year storm	1-year storm

Figure 2.12: Map of Coastal Flood Levels Summarizing Impacted Areas, with Cross-sections used for Development of Concept Design

2.5 Wave Climate

2.5.1 Offshore Wave Data

The MSC50 offshore wind and wave model hindcast from January 1954 to December 2013 contains hourly time series of wind and wave parameters at a location offshore Mahone Bay (44.3°N, 64.1°W, 70 m depth). The dataset is a state-of-the art hindcast, i.e., data computed from all existing wind and wave measurements that were re-analysed and input to a 0.1-degree resolution ocean wave growth model that includes the effect of depth and ice cover. The MSC50 hindcast was developed by Oceanweather Inc. and is distributed by Environment Canada (Swail et al., 2006). Extreme value analyses were conducted on the offshore wave data and the results were input into a numerical wave model.

2.5.2 Nearshore Wave Transformation Model

A wave model of the area was developed using the industry-standard modeling package MIKE21 SW, available from the Danish Hydraulic Institute (DHI). The model domain features higher resolution within Mahone Harbour (**Figure 2.13**). The model run to evaluate operational and extreme conditions under various scenarios of waves and water levels. The model simulates the following physical phenomena:

- Refraction and shoaling due to depth variations;
- Dissipation due to depth-induced wave breaking - A typical breaking coefficient of 0.8 was assumed (i.e. the ration of breaking wave height / water depth);
- Dissipation due to bottom friction (a typical bottom roughness of 0.04 m was assumed);
- Dissipation due to white-capping;
- Diffraction;
- Non-linear wave-wave interaction; and
- Wind-wave growth (based on uncoupled formulation recommended by DHI, which has produced satisfactory results calibrated to wave observations by CBCL at Lower Sandy Point and Wedgeport Harbours, South of Mahone Bay in NS).

Sample model runs for the 100-year return storm are shown in **Figure 2.14** (entire Bay) and **Figure 2.15** (Mahone Harbour). Extreme wave parameters along the harbour shoreline (**Table 2.2**) were used for design purposes as presented in the next Chapter.

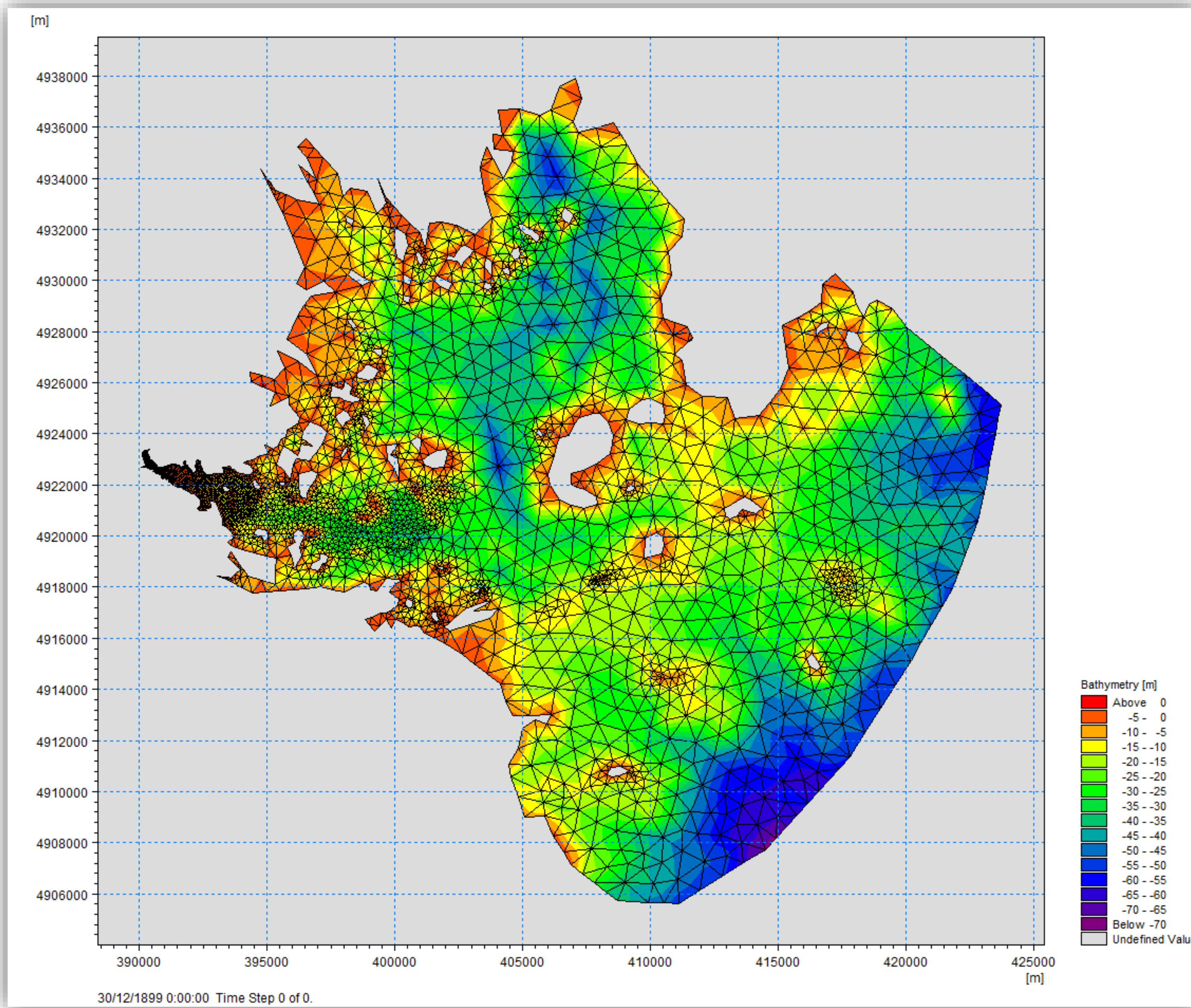


Figure 2.13: MIKE21 Model Domain of Mahone Bay

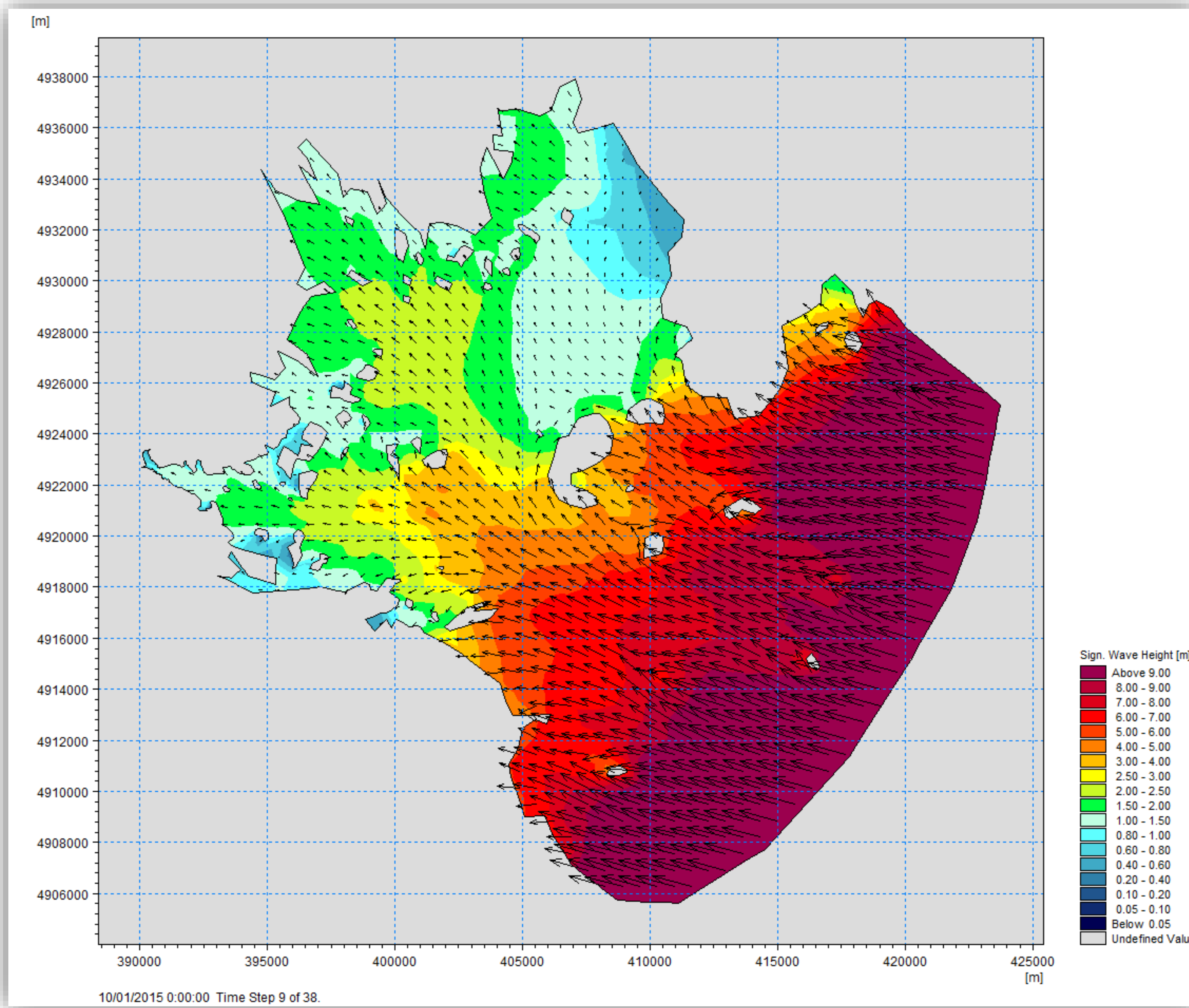


Figure 2.14: Modelled 100-year Return Significant Wave Heights Over Mahone Bay

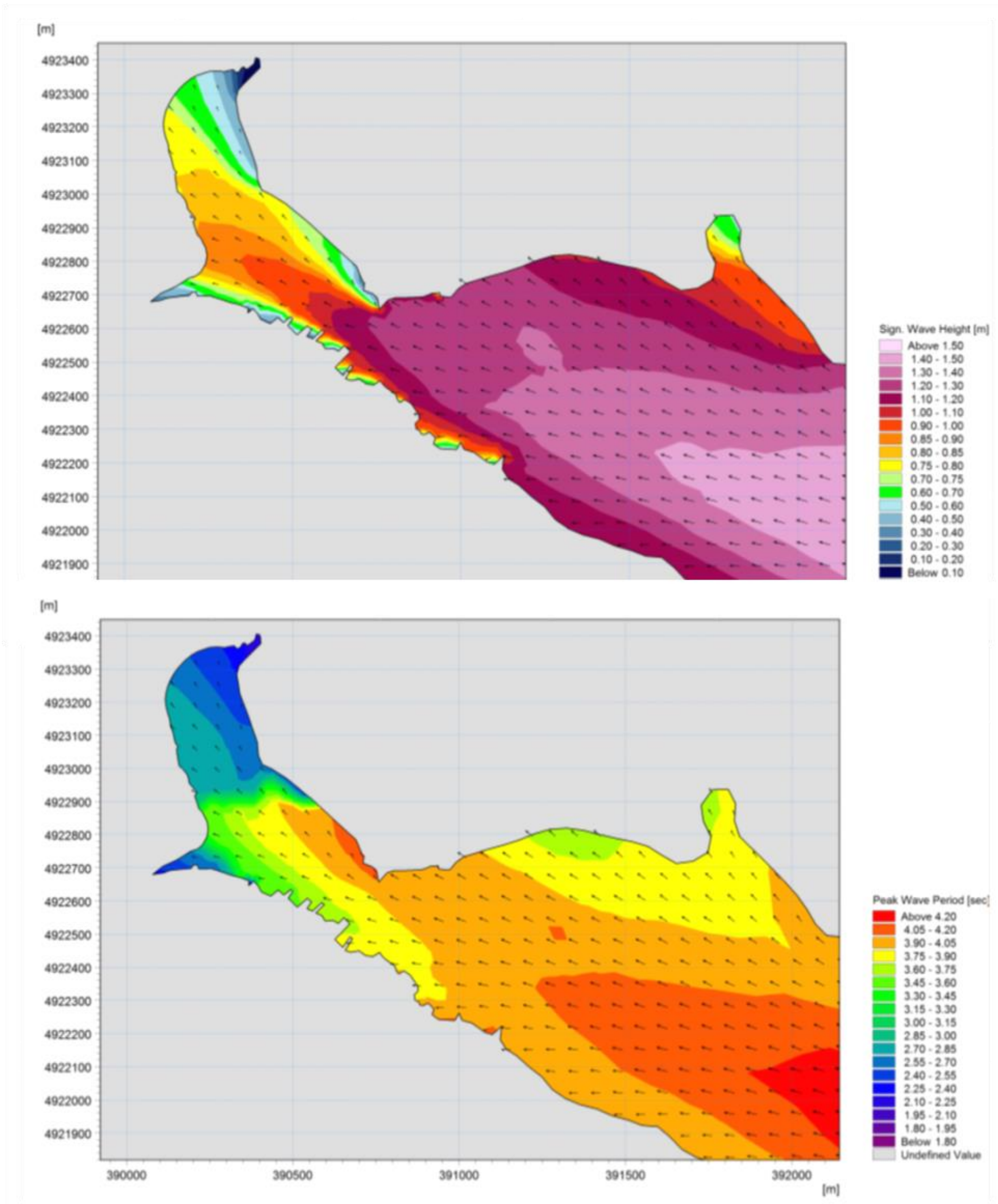


Figure 2.15: Modelled 100-year Return Significant Wave Heights (Top) and Peak Period (Bottom) Over Mahone Harbour

Table 2.2: Wave Model Inputs and Results at 7 Output Locations

Output locations distributed from head of Bay (site 1) to harbour mouth (7) - see Fig. 2.12

Return period [years]	Offshore conditions			Significant wave height [m]							Peak wave period [s]						
	Hsig [m]	Tp [s]	Wind speed [m/s]	1	2	3	4	5	6	7	1	2	3	4	5	6	7
1	5.7	11.5	16.5	0.08	0.22	0.33	0.43	0.40	0.51	0.55	1.8	1.8	1.8	2.9	2.9	3.0	3.0
2	6.3	12.0	17.9	0.13	0.31	0.41	0.49	0.45	0.57	0.62	1.8	1.8	2.0	3.0	3.0	3.1	3.2
5	7.3	12.6	19.8	0.21	0.39	0.50	0.57	0.52	0.65	0.70	2.0	2.1	2.2	3.2	3.1	3.2	3.3
10	8.1	13.1	21.2	0.23	0.42	0.54	0.62	0.57	0.72	0.77	2.0	2.2	2.3	3.2	3.2	3.3	3.4
50	9.9	13.9	24.3	0.29	0.52	0.66	0.76	0.70	0.87	0.94	2.2	2.4	2.5	3.3	3.3	3.5	3.6
100	10.8	14.3	25.7	0.32	0.57	0.72	0.82	0.76	0.94	1.01	2.3	2.5	2.6	3.4	3.4	3.6	3.6

2.6 Water Quality

Information was obtained the Canadian Shellfish Sanitation Program is jointly administered by the Department of Fisheries and Oceans (DFO), Canadian Food Inspection Agency (CFIA) and Environment Canada (EC). The programs primary objective is to protect public health by controlling the recreational and commercial harvesting of bivalve shellfish within Canada. Mahone Harbour is a relatively populated area with homes lining the shoreline. There is a wastewater treatment plant which serves part of the subsector, a two cell lagoon with disinfection. There is a large area in Mahone Harbour closed to shellfish harvesting. The mean fecal coliform bacterial count observed in the middle of the Harbour from 2006 to 2010 was 21 MPN/100ml (Source: EC). This exceeds the 14 MPN guideline for shellfish harvesting, but meets the 200 MPN/100 ml guideline for swimming. However, it is possible that localized bacterial counts close to the shore near the head of the harbour be occasionally higher due to limited flushing, the presence of storm sewers and waterfowl.

CHAPTER 3 DEVELOPMENT OF COASTAL PROTECTION OPTIONS

3.1 Design Basis

3.1.1 Design Life and Level of Protection

The coastal engineering component of the design study examines coastal flooding and overtopping risks to recommend appropriate design options. The design options must carefully consider risk, intended design purposes and design life. The design life of a structure is the period for which it can be fully operational and used for its intended purposes, given a planned maintenance program. Its level of protection is traditionally defined using a given failure probability over the lifetime. As per **Table 3.1**, for the majority of coastal protection works, a failure probability of 0.40 to 0.64 over the lifetime is chosen (which would correspond to a 100-year to 50-year return storm over a 50 year lifetime using a deterministic approach).

Table 3.1: Design Life vs. Hazard Type and Level of Protection

Design Life	Risk of Human Life or Environmental Damage in Case of Failure	Hazard Type and Reason	Typical Level of Protection (Return Period, Years)	Encounter Probability of the Design Event Over the Lifetime (0 to 1)
25	Small	Temporary or short term measures, e.g. pavements	25	0.64
			50	0.40
50	Moderate	Majority of shoreline protection works, general use infrastructure, e.g. sea dikes in rural areas	50	0.64
			100	0.40
100	High	Flood defences protecting large areas at risk, e.g. dykes in urban areas	100 to 10,000	0.64 down to 0.01
200	Very high	Special structures with very high cost (e.g. some European storm surge barriers)	Up to 10,000	Down to 0.02

3.1.2 Adaptive Management Approach

For Mahone Bay, while the planning horizon is understood to be 100 years, individual coastal structures should be designed for a shorter lifespan. Repair or rebuild cycles for basic municipal infrastructure such as roads, parks etc. (with small to moderate risk of human life or environmental damage in case of failure) can be as short as 20 to 30 years. In cases where conditions change over time, particularly with sea level rise, it generally makes good economic sense to adopt an 'Adaptive Management' approach, i.e. avoid over-design and use flexible designs that can accommodate the typical repair/rebuild funding cycle. We propose to use a 50-year lifetime as a design basis. While the 50-year basis exceeds the aforementioned 20-30 year cycle, it provides some allowance for the possibility that sea level rise (SLR) rates may exceed projections within the 20-30 year period post-construction.

3.1.3 Probabilistic Considerations

Extreme water levels with a low return period today will be very common in a few decades due to SLR. Therefore, the traditional design parameter of return period becomes a moving target and the common deterministic engineering practice of designing for the N-year storm and expecting a given probability of occurrence within the design life time is rendered invalid by SLR. As the sea level rises throughout the life cycle of the structure, the probability of damage will increase. Therefore, a probabilistic approach is warranted, and we propose to keep the cumulative flooding probability below 50% over the lifetime of the structure, i.e. typically 50 years for the majority of shore protection works. The following sections describe the input data and analysis procedures conducted.

3.2 Range of Options

Coastal protection options were developed with the objective to balance the following requirements:

- Flood and erosion mitigation;
- Preservation and, wherever possible, enhancement of natural shoreline habitat (i.e. the intention goes beyond simply raising the existing waterfront);
- Public access to the shoreline; for the North shoreline, a waterfront trail is proposed along Edgewater Street; and
- Aesthetics.

Potential options based on wave exposure and shoreline type are summarized in the **Table 3.2**. The ranking of options reflects the analyses presented in the following sections.

Options for flood mitigation include:

- Raising the road;
- Raising buildings;
- Floodproofing buildings, i.e. ensuring basements and underground parking garages can sustain temporary flooding (which is referred to as 'wet floodproofing') or sealing out the exterior of the building up to the anticipated flood level (which is referred to as 'dry floodproofing');
- Raising retaining walls;
- Building an elevated berm that could support a waterfront trail; and
- Building a seawall in front of the infrastructure at risk.

Table 3.2: Coastal Protection Options

Shoreline Area (going seaward)		Length m	Relative wave exposure and potential for erosion	Shoreline type	Applicability of General Options						
					Coastal flood mitigation					Coastal erosion mitigation	
					Raise road	Adapt buildings		Raise retaining walls	Elevated waterfront trail/berm	Armour stone seawall / slope	Living shoreline with rock sills/breakwaters
						Raise	Floodproof				
Area for detailed design	Head of bay (at Kedy's restaurant)	80	Low	Low sloped tidal flats							
	Edgewater rd	350	Medium	Eroding bluff and tidal flats							
	Three Churches waterfront	360	High	Steep armoured slope							
	Ernst brook outlet	150 / 120	Low	Built along river edge							
Area for concept design	Southwest Town Waterfront (bandstand to boat docks)	260	High	Semi-built							
	Southeast Town Waterfront	1300	High	Built							
	Lighthouse route	800	High	Road embankement with rip-rap and tidal flats							

Legend
- Recommended (detailed design area) / Applicable (concept design area)
- Potential or long-term option
- Not applicable or assessed at this stage

Engineering options for shoreline erosion mitigation typically range from hard structures to softer approaches, as per the local example in **Figure 3.1**. In this study, options considered include:

- Hard protection in the form of a traditional armoured seawall at a relatively steep slope; and
- Soft protection in the form of a 'living shorelines' approach to mitigate wave overtopping and erosion before it actually impacts the infrastructure.

During the preparation of conceptual options, stakeholders inquired about the possibility of a large breakwater at the entrance of the harbour. This option would be effective for a local reduction in wave heights in the shadow of the structure. However, it would not reduce the elevation of the storm surge or provide any benefits for long-term coastal flooding risks. Therefore it was not investigated further.

Finally, we note that the options must not only consider coastal flooding, but also river flooding. This means any berm structures designed to block the storm surge must include one-way drainage structures to accommodate potential rainfall runoff.

Figure 3.1: Engineering Options for Shoreline Erosion Mitigation Showing Local Example of Hard vs. Soft Options

3.3 Overtopping Analyses to Determine Crest Elevations

Following the probabilistic approach outlined in the design basis, extreme water levels and wave heights were used to estimate overtopping and required crest elevations. In areas protected from wave overtopping (i.e. upstream of Ernst Brook bridge), the recommended static coastal flood levels are 2.8 m CGVD28 for a 50-year infrastructure lifetime (and 3.6 m for a 100-year lifespan). In more exposed areas, the crest elevation of coastal structures is generally determined based on a wave overtopping limit. **Table 3.3** presents overtopping limits typically used in the design of sea defenses and coastal structures.

Table 3.3: Limits for Overtopping

Source: Pullen et al 2007, USACE CEM 2012.

Hazard Type and Reason	Mean Discharge q [l/s/m]	Applicability to Proposed Mahone Bay Coastal Protection Options
Damage to grassed or lightly protected promenade on seawall	> 50	Edgewater Street waterfront trail – **paved** option
Hazard to vehicle driving at low speed or damage to small boat in marina behind breakwater	> 10	Edgewater Street and Lighthouse route road elevation
Damage to unprotected (gravel) promenade on seawall	> 2	Edgewater street waterfront trail – **gravel** option
Building structure elements Trained staff expecting to get wet	> 1	Town waterfront seawall

A probabilistic overtopping assessment was conducted to determine the optimum crest elevation for waterfront structures. The calculations are based on the probabilistic overtopping equations presented in Pullen et al 2007[2]. Overtopping discharge is a function of the crest elevation of the structure above still water level ("freeboard"), and the wave height. Therefore, as the sea level rises throughout the life cycle of the structure, the probability of discharges above the design damage threshold will increase. In order to evaluate this variability, the cumulative probability of a damage event was calculated for a range of crest elevations considering sea level rise. Results for a 50-year lifetime are provided in **Table 3.4**.

These recommended crest elevations were further confirmed by a preliminary life-cycle cost benefit analysis. The capital and maintenance costs for waterfront trail fill and surfacing were calculated for a range of crest elevations. For each crest elevation, the occurrence of damage events (i.e. overtopping exceeding the limit) and resulting resurfacing costs were computed over the 50-year lifetime, taking into account sea level rise.

[2] Using a roughness factor $\gamma f = 0.55$ for impermeable structures.

Table 3.4: Overtopping Analysis Results

Shoreline section (see Fig 2.12)	Significant wave height [m] by return period			Recommended overtopping limit [l/s/m]	Required crest elevation [m CGVD28] to limit cumulative overtopping probability to <50% to year 2065	
	10-year	50-year	100-year		Armourstone seawall / slope	Living shorelines with rock sills / breakwaters
1 - Head of Bay	0.23	0.29	0.32	2 (gravel trail) to 50 (paved trail)	N/A	3 - 2.9
2 - Edgewater str	0.42	0.52	0.57		3.4 - 3.1	3.1 - 3.0
3 - Edgewater str	0.54	0.66	0.72		3.6 - 3.2	3.2 - 3.0
4 - Three Churches Waterfront	0.62	0.76	0.82		3.7 - 3.2	3.2 - 3.0
5 - Southwest Town Waterfront	0.57	0.70	0.76	1 (building structure elements, staff expected to get wet)	3.7	3.2
6 - Town Waterfront - Marina	0.72	0.87	0.94		4	3.4
7 - Southeast Town Waterfront	0.77	0.94	1.01		4.1	3.4
8 - Lighthouse road	0.89	1.08	1.16	10 (Hazard to vehicle driving at low speed)	3.8	3.2

The conclusions of the overtopping assessment are as follows:

- The salt marsh option is recommended over the armoured seawall, because it offers wave energy attenuation and allows a lower crest elevation, therefore lower construction cost. Once armour rock costs are factored in (i.e. continuous armouring for seawall vs. split system of nearshore breakwaters using less rock), the cost advantage of the salt marsh option is even greater; and
- The asphalt surfacing would allow a lower crest elevation than the gravel option, which would tend to even out the higher surfacing cost. Asphalt surfacing is therefore recommended.

3.4 Option for North Shoreline: Living Shorelines Approach

The type of shoreline at the head of Mahone Harbour (from the Three Churches and up) is moderately exposed to waves and has a wide intertidal flat. These are ideal conditions for the development of a living shoreline approach, which is more effective at dissipating wave energy than a traditional armoured seawall approach. It is also more cost effective, as presented in the following sections.

3.4.1 Rationale and Benefits

The head of Mahone Harbour along Edgewater Street has moderate wave exposure and a relatively mild seafloor slope. These conditions are well suited to the application of living shorelines techniques for flooding and erosion mitigation.

As per the definition by the Maryland Department of Natural Resources: “Living shorelines are the result of applying erosion control measures that include a suite of techniques which can be used to minimize coastal erosion and maintain coastal process. Techniques may include the use of fibre coir logs, sills, groins, breakwaters or other natural components used in combination with sand, other natural materials and/or marsh plantings. These techniques are used to protect, restore, enhance or create natural shoreline habitat”.

Living shorelines aim to combine traditional rock-based erosion mitigation with vegetation where space allows. In practical terms, the rock protection is moved seaward of the infrastructure to protect, and the space in between is vegetated to create a marsh. The vegetation buffer then reduces the wave energy well before it reaches the infrastructure to protect. The wider the vegetation buffer, the more wave energy is reduced. In fact, recent research on the performance of existing living shorelines projects (Gittmana et al, 2014, 2015) indicates that living shoreline approaches have the following advantages:

- Salt marshes with and without rock sills are more durable and may protect shorelines from erosion better than hard structures (e.g. seawalls) in a Category 1 storm; and
- Rock sills fronting salt marshes support a higher abundance of species and greater diversity than unvegetated habitat next to seawalls, and even more than natural marshes used in the comparison.

3.4.2 Examples

Local Example: Natural Marsh

There is a local example of a salt marsh fronting a road near Mahone Harbour (**Figure 3.2**). The 10 to 20 m wide marsh fronts a vegetated road embankment. It is located in a relatively sheltered cove with exposure similar to the head of Mahone Harbour. The marsh elevation at the edge was estimated at typically 0.1 – 0.2 m above mean water level.

International Examples: Engineered Living Shorelines

Living shorelines approaches have been successfully used in the last 20 years at many sites, notably in Chesapeake Bay (**Figure 3.3**) along eroding tidal shorelines with moderate wave exposure comparable to Mahone Harbour. A database for these projects was compiled by the Virginia Institute of Marine Sciences (VIMS) and is available at:
http://www.vims.edu/research/departments/physical/programs/ssp/shoreline_management/breakwaters/ge_map/index.php

Two examples are shown in the following pictures. Design elements typically include rock sills fronting salt marshes growing on sand fill, and detached nearshore breakwaters with pocket beaches for more exposed locations. Openings in the rock sills allow tidal circulation and marsh development. Multiple and wider openings require a greater footprint, as the breakwaters have to be located further off the shoreline to allow for a stable pocket beach to form behind the opening. Wave exposure for these two projects was comparable to that of Mahone Harbour.

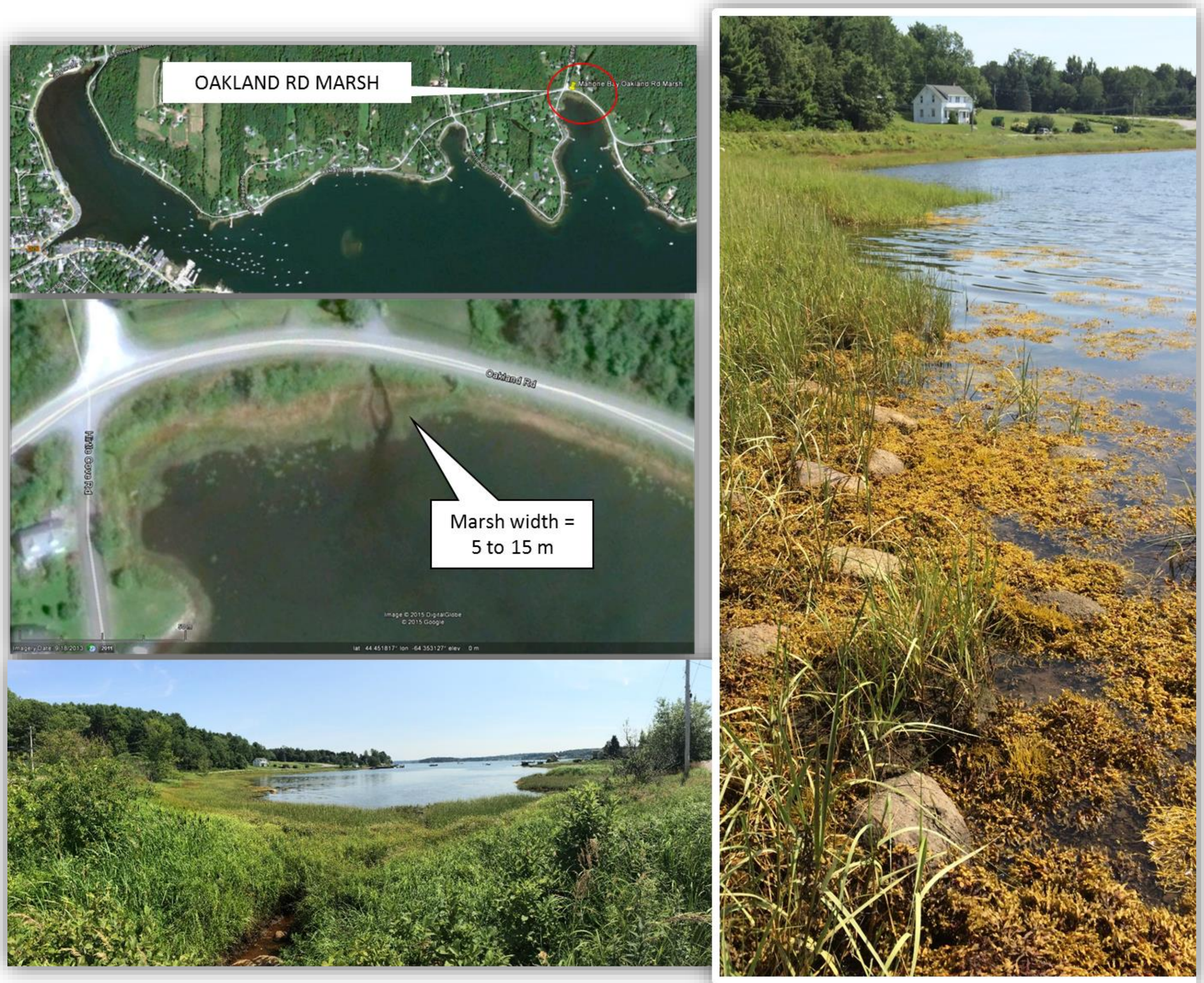


Figure 3.2: Local Salt Marsh Fronting Oakland Road in Mahone Bay

Figure 3.3: Examples of Engineered Living Shoreline Protection using Combinations of Rock Sills and Breakwaters in Chesapeake Bay

Note: these examples were built on nearshore slopes that are steeper than Mahone Bay's along Edgewater road. In Mahone Bay, the low tide contour would come close to breakwaters built 15-20 m from the shore. As tidal elevations increase with sea level rise, this solution applied to Mahone Bay would eventually resemble the examples above, with the low tide contour located landwards of the gap between the breakwaters.

3.4.3 Design Guidelines

To determine the correct approach within the living shorelines framework, the decision tree developed by the VIMS was applied, using inputs from the wave study. Results are presented in the following table.

Table 3.5: Options Selection Process for Living Shorelines Approach

Site	Kedy's Parking Lot	Edgewater Road Embankment	Three Churches Parking Lot	Southwest Town Waterfront (Bandstand to Boat Docks)
Potential for coastal flooding	High	High	High	High
Bank erosion	Low	High	High if unprotected	High if unprotected
Forested shoreline	No	No	No	No
Marsh present	Yes < 5m wide	No	No	No
Beach present	No	No	No	No
Fetch	Low < 800 m	Moderate 1-3 km	High > 3km	High > 3km
Nearshore water depth	Shallow (< 1 m deep 10 m away from mean low tide mark)	Shallow	Shallow	Shallow
Recommendation 1	**Vegetation management**	**Grade bank and vegetate AND**	**Grade bank and vegetate AND**	**Grade bank and vegetate AND**
Recommendation 2	**Marsh with fiber log at toe**	**Marsh with rock sill**	**Breakwaters with sand fill**	**Breakwaters with sand fill**

* The applicability to the south shoreline is constrained by limited space, and would depend on local bathymetry which shall be confirmed in future stages.

The length of the rock sills/breakwaters is a function of the desired along-shore marsh width behind. Long breakwaters with short gaps will promote wider marsh growth and can be located closer to shore. Short breakwaters with larger gaps must be located further away from the shore, and can be used to stabilize pocket beaches.

Engineering design guidelines for these shoreline protection techniques were recently compiled by Miller et al. for the State of New Jersey (2015). Key parameters are presented in **Figure 3.4**. Based on the local conditions at Mahone Harbour, the appropriate living shorelines approach would include a combination of shore-parallel breakwaters and rock sills. The breakwater crest is typically at mean high water. The structure gets overtopped during storm surge events, and the wave energy is dissipated by the marsh behind. This allows for more economical rock use than shoreline armouring.

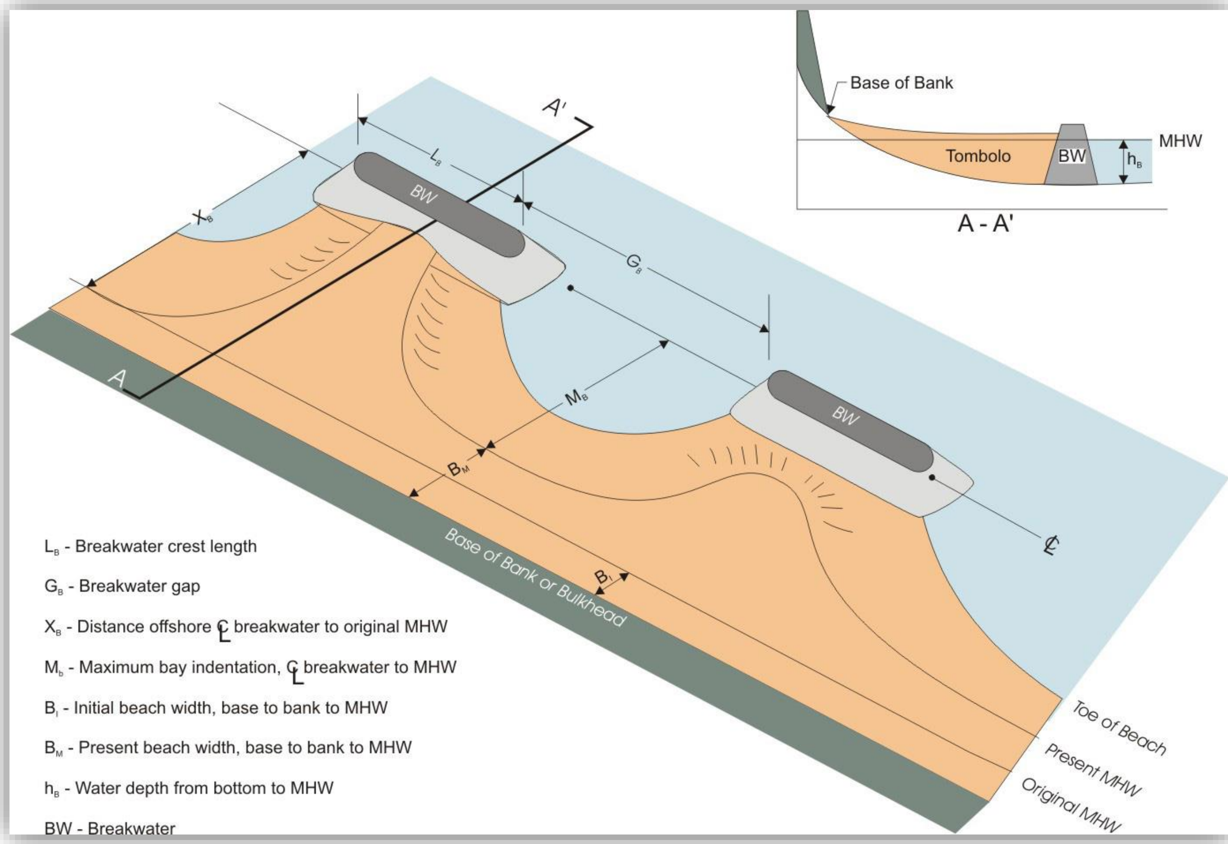


Figure 3.4: Design Parameters for Nearshore Breakwaters
Reproduced from: Hardaway & Gunn, 2010

The design parameters were established based on Miller et al, 2015 and Hardaway et Gunn 2010 (figure above) and Bodge 1998:

- Marsh width 10 to 20 m for protected to moderately exposed shoreline;
- Marsh slope 1:10 or flatter;
- Typical breakwater length Lb should be at least twice the wavelength. Based on the local wave climate, this translates into Lb > 20 m;
- Include gaps in the sill for tidal circulation at least every 30 to 35 m; and
- Typical breakwater length ratio Gb/Mb = 1.4 to 1.9 (bi-modal and unidirectional wave climate, respectively), with typical value 1.65; the ratio Gb/Mb becomes 3 when using the mean low water contour (Bodge 1998). This means that wider gaps between the breakwaters, to accommodate a greater beach opening, require to locate the breakwater further offshore.

3.4.4 Conceptual Design Parameters

Based on the above design guidelines applied to the local context, the recommended conceptual dimensions are presented in **Table 3.6**.

Table 3.6: Conceptual Design Parameters for Living Shorelines Option

		Edgewater Road Embankment	Three Churches Parking Lot	Southwest Town Waterfront (bandstand to boat docks)
Shoreline length		350 m	400 m	260 m
Waterfront trail width		3 m		N/A
Waterfront berm crest elevation m CGVD28		2.9 m	3 m	3.4 m
Embankment slope from trail crest to HHWLT (1.5 m CGVD28)		3:1		
Marsh dimensions behind sill/breakwater	Width	10 m	15 m	10 to 15 m
	Average slope from HHWLT (1.5 m) down to down 0.5 m (i.e. 0.2 m above MWL)	10 : 1	15 : 1	10:1 to 15:1
Rock structures	Length	30 m	35 m	30 m
	Crest elevation	1.1 m CGVD28		
	Gap length G	5 m	15 m	8 m
	Gap width (cross-shore distance up the beach) to HHWMT =G/1.65	3 m	9 m	5.1 m

3.4.5 Conceptual Cross-Sections and Plan View

Conceptual sketches for the living shorelines option along the North shoreline are shown in **Figure 3.5, Figure 3.6, Figure 3.7** (cross-sections) and **Figure 3.8** (plan view).

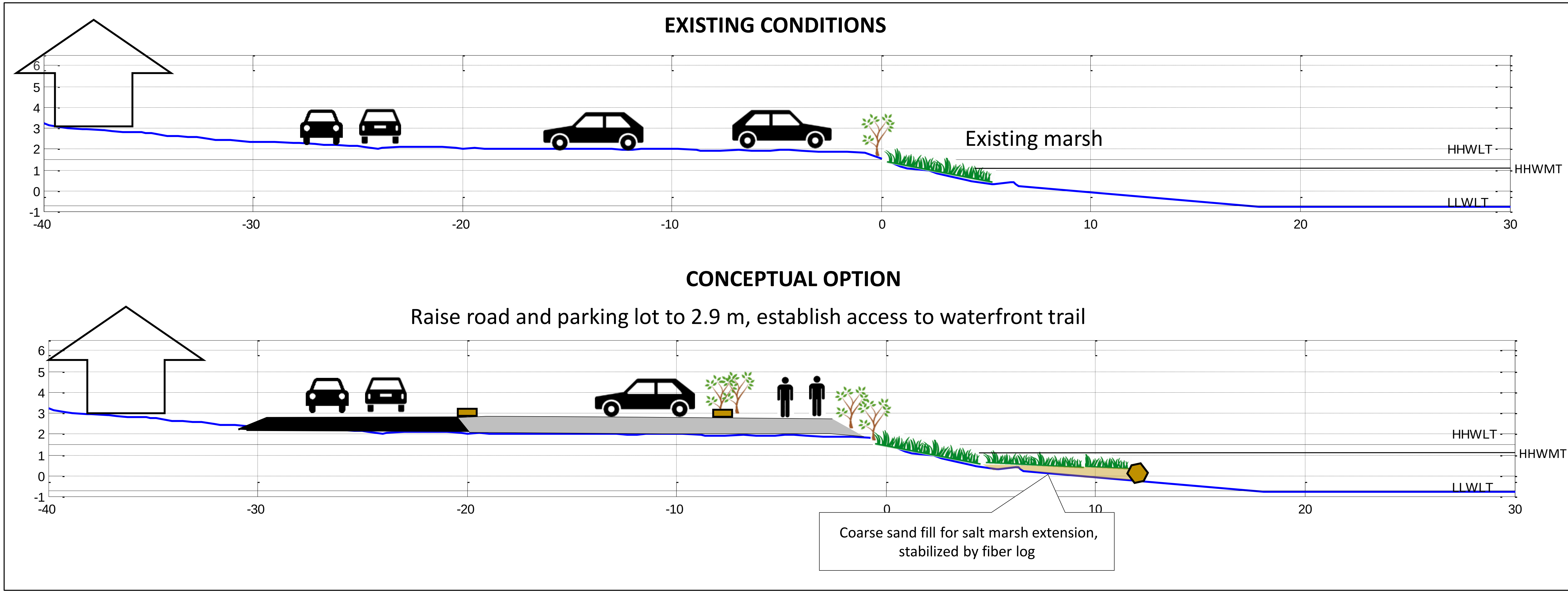


Figure 3.5: Cross-Section 1 – Edgewater Street at Kedys Restaurant – Options for Consideration in Detailed Design

This section of shoreline is at risk of flooding while wave action is too low to pose erosion risks. The road and/or parking lot should be raised, and provide access to the waterfront trail to be built along Edgewater road. The salt marsh may be extended by adding sand fill landward of the existing marsh, to be stabilized by geosynthetic fiber log or rock sill.

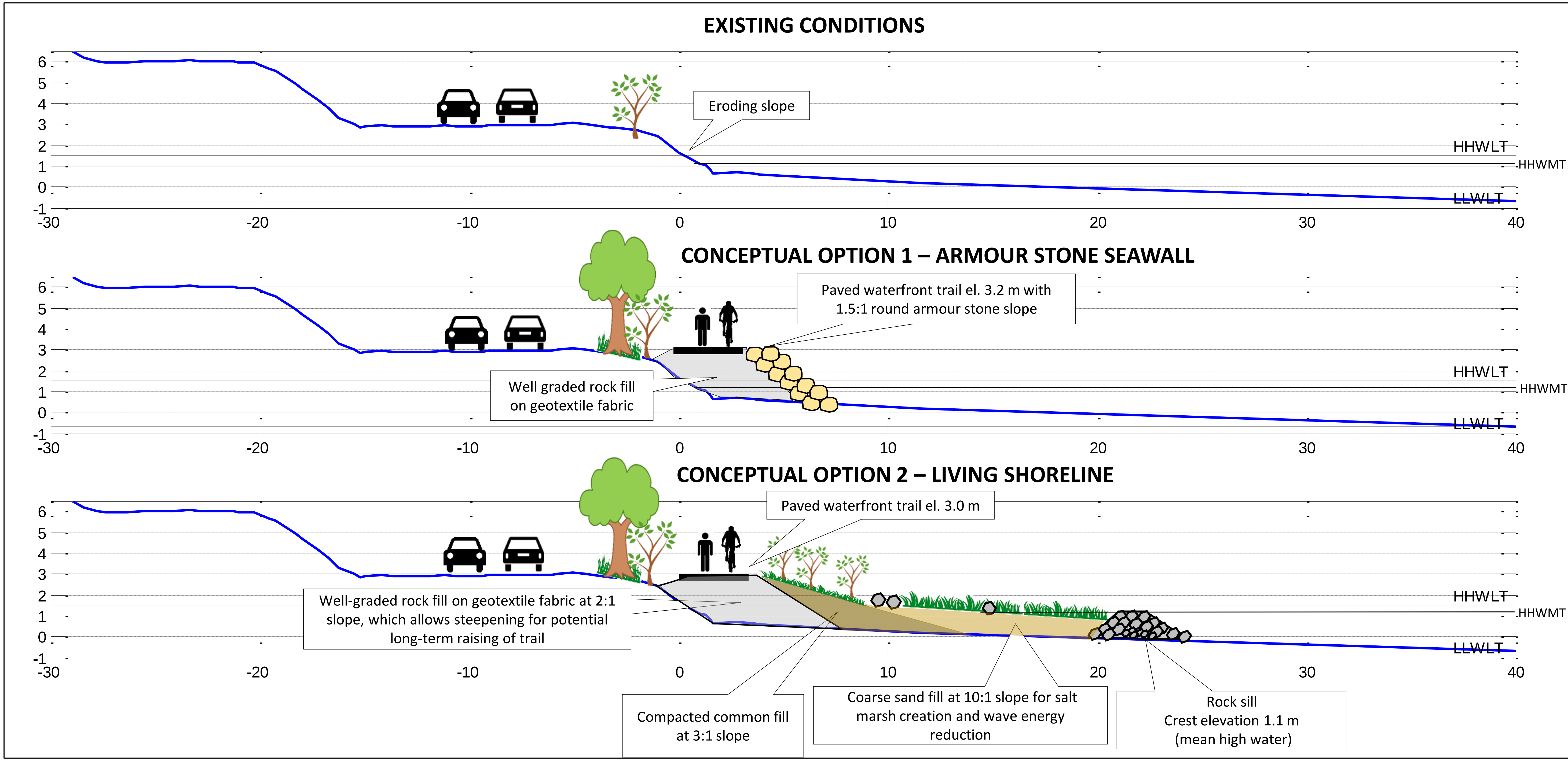


Figure 3.6: Cross-Section 2 and 3– Edgewater Street Waterfront Trail - Options for Consideration in Detailed Design

This section of shoreline is currently eroding due to the unstable road embankment. The proposed concepts include a multi-use waterfront trail, to be protected against wave action by either:

- Traditional seawall made with place round armour stone that fits into local aesthetics; and
- Living shorelines concept that allows dissipation of wave energy through rock sills fronting an area of sand fill to support the growth of an intertidal salt marsh.

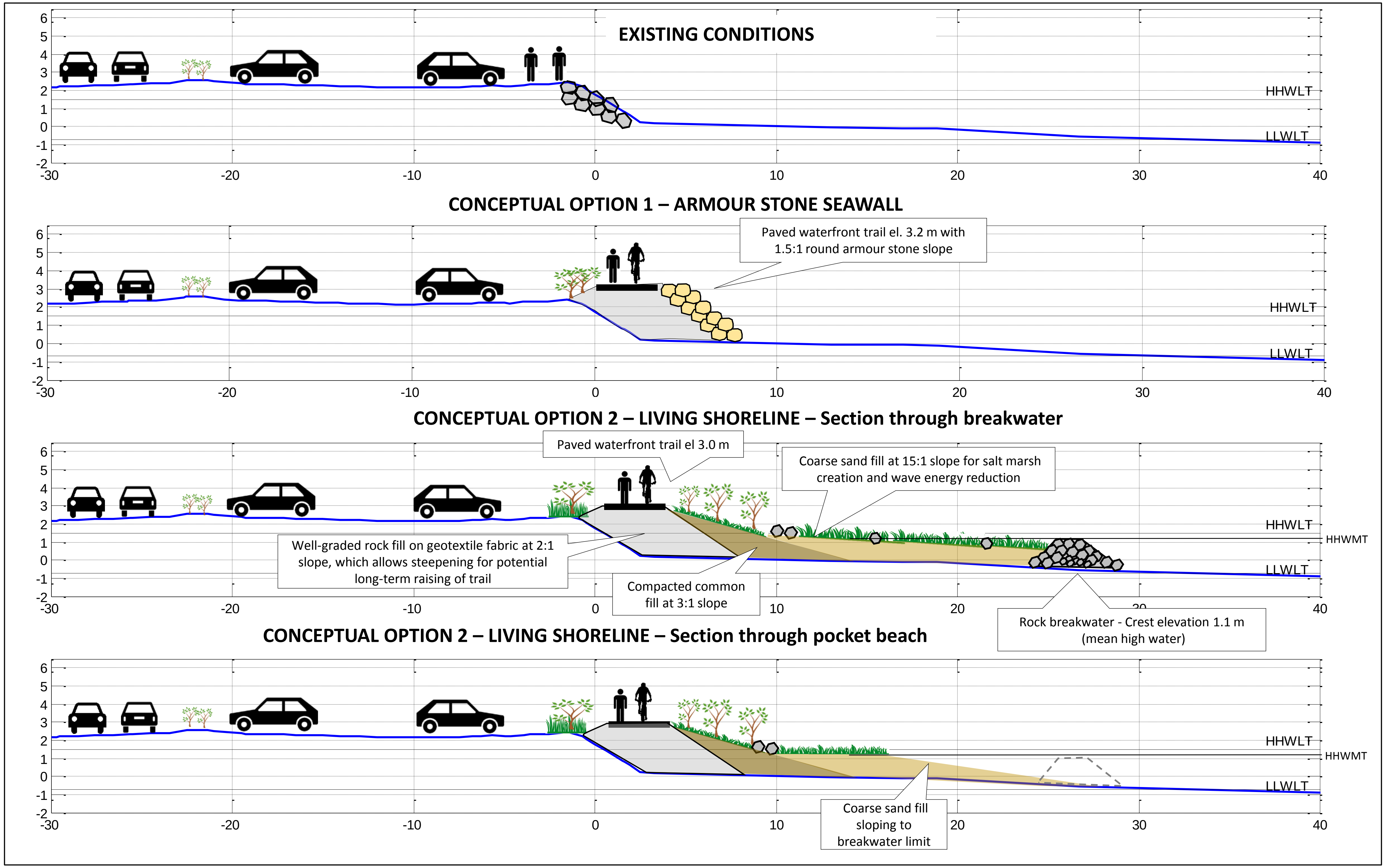
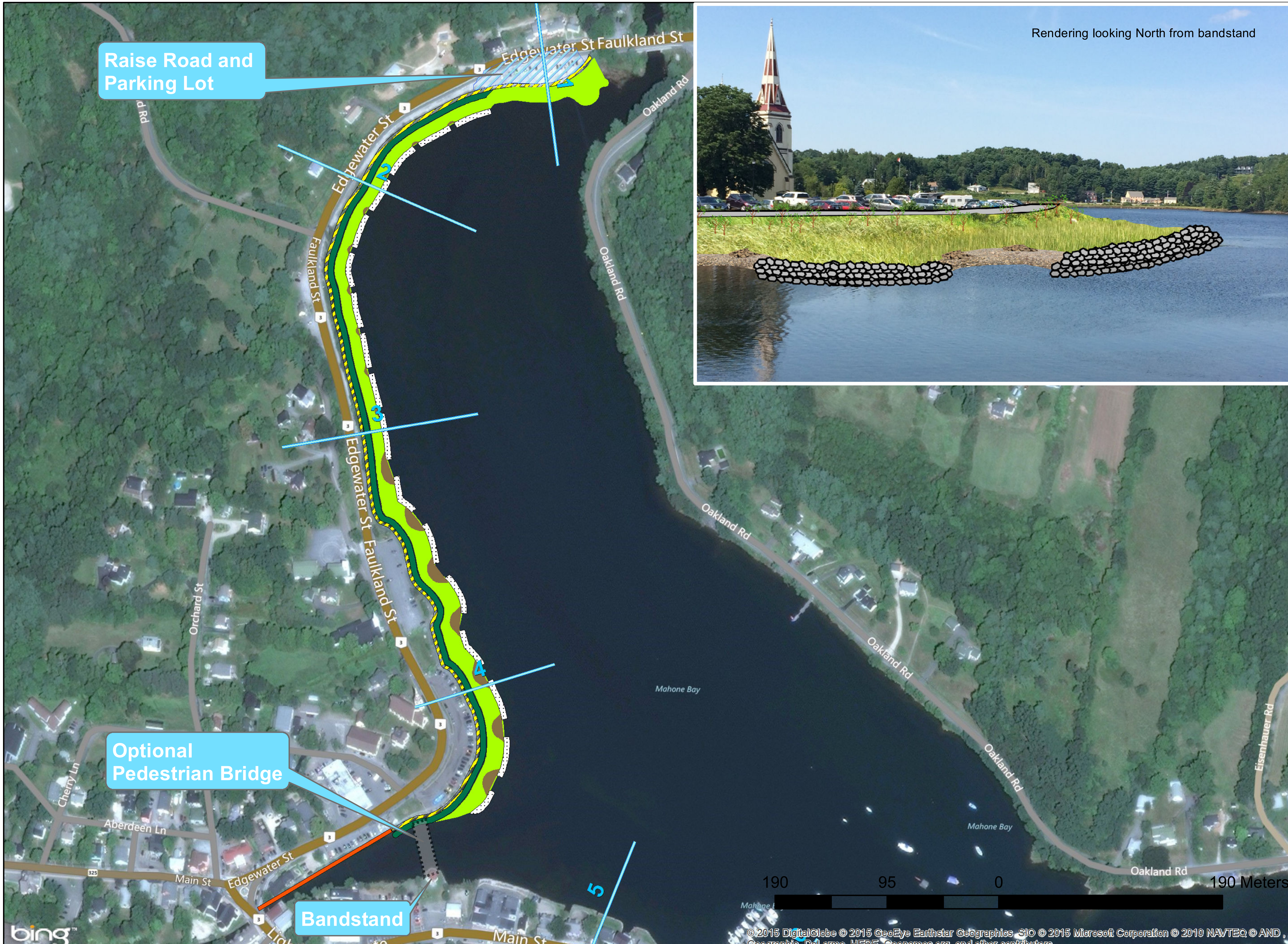


Figure 3.7 **Cross-Section 4 – Edgewater Street along Three Churches Parking Lot - Options for Consideration in Detailed Design**

The proposed concepts include a multi-use waterfront trail, to be protected against wave action by either:

- Traditional seawall made with place round armour stone that fits into local aesthetics; and
- Living shorelines concept that allows dissipation of wave energy through rock breakwaters fronting areas of sand fill to support the growth of an intertidal salt marsh. Public access to the shoreline will be enhanced by pocket beaches between the rock breakwaters.



Mahone Bay

Town of Mahone Bay

Mahone Harbour Flood Prevention and Shoreline Enhancement Plan

Legend:

- Raised Retaining Wall
- Rock Sills
- Embankment
- Marsh
- Waterfront Trail
- Pocket Beach

Figure 3.8

North Shoreline Schematic Plan View Including Living Shorelines Concept

Date: December 2015
CBCL Project #: 151016.00

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Consulting Engineers

3.4.6 *Environmental and Regulatory Considerations*

In terms of environmental benefits, the low-crested breakwaters and sills are to reduce nearshore wave energy to support marsh habitat in their lee. Local species are already adapted to a low-to-medium wave energy environment, and are expected to colonize the area behind the breakwater. Intertidal marshes provide habitat for a wide variety of organisms, including fish, invertebrates, birds and also mammals using the marsh for foraging, breeding and refuge. The hard rock substrate will provide habitat for organisms, and shelter for fish and invertebrates. The exposed crests of the structures may be used by seabirds. In this area of Mahone Harbour, these structures will represent the only form of hard bottom habitat available, resulting in increased local biodiversity.

Table 3.7: Regulatory Considerations for Shoreline Protection Options along Edgewater Street

Options and Infill Elements	Length m	Width m	Environmental and Regulatory Implications
Living Shoreline with rock sills fronting sand fill to support salt marsh and waterfront trail			
Waterfront trail and vegetated embankment	700	8	• Municipal Development Approval.
Sand fill for salt marsh	700	10-15	• Fisheries Act authorization for footprint below high-water; • Beaches Act and Crown Land Act (NSDNR Approval); • Navigation Protection Act approval; • Canadian Environmental Assessment Act Section 67; and • Municipal Development Approval.
Rock sills and breakwaters	550	5	• Fisheries Act authorization for footprint below high-water; • Beaches Act and Crown Land Act (NSDNR Approval); • Navigation Protection Act approval; • Canadian Environmental Assessment Act Section 67; and • Municipal Development Approval.
Armour Stone Seawall			
Armoured waterfront trail	700	10	• Fisheries Act authorization for footprint below high-water; • Beaches Act and Crown Land Act (NSDNR Approval); • Navigation Protection Act approval; • Canadian Environmental Assessment Act Section 67; and • Municipal Development Approval.

The options considered to support a waterfront trail would entail infilling into tidal waters, which means habitat alteration under the Fisheries Act. The habitat alteration associated with the armour stone seawall option (roughly 7,000 m^2) would be less than that from the living shorelines option (15,000 to 20,000 m^2). The same general regulatory requirements exist for both options. However, the living shorelines option would likely be viewed more favourably by the regulators because it is in essence a self-offsetting project, i.e. the habitat created (rock reefs and salt marsh) would be more productive for the local fishery than the current habitat (uniform muddy bottom).

3.5 Options for Town Waterfront

The town waterfront is protected from erosion by numerous seawalls made of boulders, however none are high enough to deal with long-term sea level rise and overtopping. Flood mitigation options for the Town Waterfront are constrained by the following factors:

- Waterfront divided between multiple privately owned properties;
- Space limitations due to existing infrastructure; and
- Requirement for boat access along the southeast section.

From a planning point of view, since the land is divided across multiple private property owners, an individual adaptation approach on a property-by-property basis would be easier to implement.

Feasible options for the next 50 years include a mix of hard protection, such as higher seawalls, infilled shoreline with flood berms, and where feasible, adaptation of infrastructure to more frequent flooding, such as flood-proofing or raising buildings and road. These options are explored in the sections below and illustrated in **Figure 3.9**.

3.5.1 Seawall Built on Existing Shoreline

Some properties have already built seawalls. However the waterfront is only as flood-resistant as its weakest link, and total protection can only be achieved by a common seawall. A seawall concept built on existing land would require buy-in from all property holders. Otherwise the seawall would have to be built out into the harbour, which would be making boat access more difficult. Difficulties around shoreline indentations such as boat ramps and waterfront parking would have to be resolved. One could decommission secondary boat ramps, and raise one main boat ramps to be tied back to the street over a berm. In the long-term (50+ years), backfilling behind the wall would be required and eventually raising all infrastructures to avoid infiltration issues under permanent sea level rise scenarios. The total shoreline length accounting for all indentations is estimated at 1600 m.

3.5.2 Floodproof Buildings

Adapting the buildings to sea level rise means flood-proofing them. This can be done incrementally, on a property-by-property basis, as follows.

1. **Raising the building** involves elevating the critical use area of a building (or other infrastructure) above flood levels. A building's elevation can be increased through the use of stilts or raised foundations. Stilts create non-living space under the building such as a garage or patio area. Another way to increase a building's elevation is to increase the height of the land with fill before the building is constructed. It is usually easier to build a brand new raised building than to raise an existing building. The principle can also be used to adapt vital infrastructure such as utilities and roads. It is typically feasible to raise buildings on a crawlspace or basement foundation, as for most residential waterfront properties, with the main level a few feet above ground. Raising a building is less practical if it is built on a slab-on-grade, as would likely be the case for larger industrial or retail buildings.

2. **Wet flood proofing** accommodates the possibility of flooding into the structure. This type of building technique is only applicable for building levels that are not used for residential space. It is best used for parking structures and storage of goods that would not be damaged by water. This

technique allows water to flow in and out of the lower level of the buildings. Significant cleanup will often still be necessary after a flood.

3. **Dry floodproofing** with an exterior floodwall - Floodwalls are used primarily in high value built up areas where other coastal protection or management options are limited, or when individual property owners want to protect their assets beyond whatever measures are already in place. The flood walls are usually made of concrete or are earth mounds. Their purpose is to enclose a property to prevent floodwater or storm surge from impacting the more valuable structures within. Dry flood proofing involves applying protective (waterproof) coatings to the structures that prevent water from penetrating the structure. These are not primary protection strategies and should only be considered as back up for emergency events.

The above options are described with practical details in a 2010 online document by FEMA for homeowners (see FEMA 2010).

3.5.3 Raise Main Street

Raising the road would protect properties on the landward side. However this would reduce room for property access and likely create some integration issues with driveways and adjacent infrastructure. Main Street could be raised to protect infrastructure on the landward side. Infrastructure on the seaward side would still need individual floodproofing. This alternative is severely constrained by the minimal space between the road and the buildings. It would create some integration issues with driveways and adjacent infrastructure. For all these reasons, it is not considered the most practical or desired option for the short term.

In summary, under existing conditions the ground elevation for waterfront properties can be as low as 0.5 m above high tide for some sections. In this context, a seawall and/or floodproofing options are realistic and practical for the short- to medium-term in the next few decades, when flooding will remain occasional due to temporary storm surges. However, for permanent sea level rise in the long term (50 years and beyond) when existing infrastructure have reached the end of their useful life, the two following approaches will need to be considered as well:

- Complete rebuilding of the downtown area to a higher elevation; and/or
- Planned withdrawal and relocation when and where opportunities to do so arise (i.e. property ownership transfers), which may eventually leave more space to raise the road that could then be engineered as a sea defense.

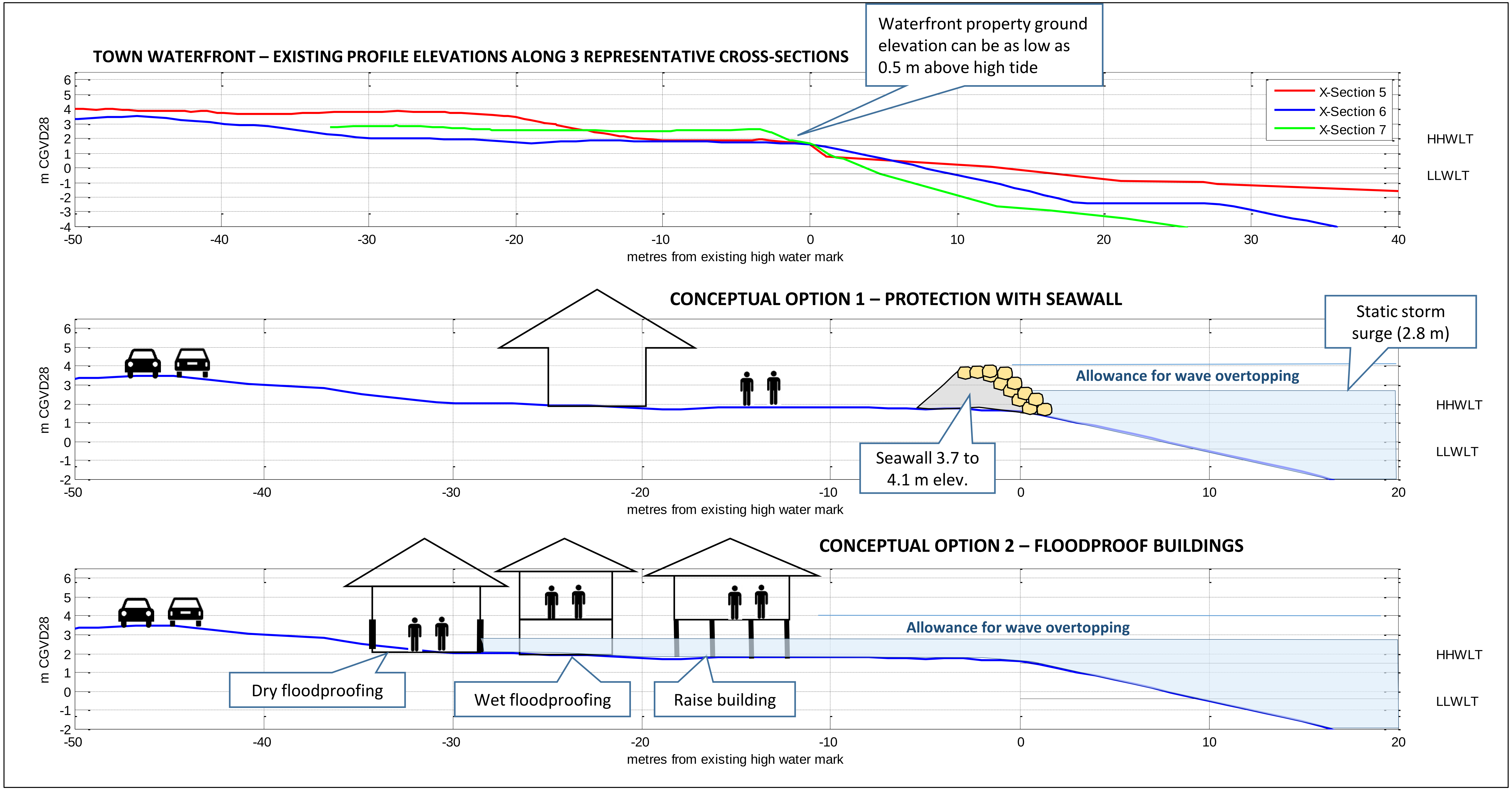


Figure 3.9: Cross-sections 5, 6, 7 – Town Waterfront (Conceptual Design Options)

3.5.4 *Flood Dyke and Living Shoreline (Southwest Section)*

This alternative 'seawall' option would be best suited to the shallowest northwest section of shoreline, a 320 m long distance between Ernst Brook Outlet and the first boat dock (**Figure 3.10.a**). Deeper water and the requirement for boat access make it more difficult to implement elsewhere. Where space allows, a living shoreline could be incorporated (**Figure 3.10.b**), comparable to that proposed for the North shoreline. This option could be implemented by the Town, as an alternative to individual property owners upgrading their own seawalls.

Figure 3.10.a: Cross-sections 5 – Flood Dyke and Living Shoreline Alternative for Southwest Town Waterfront (Conceptual Design)

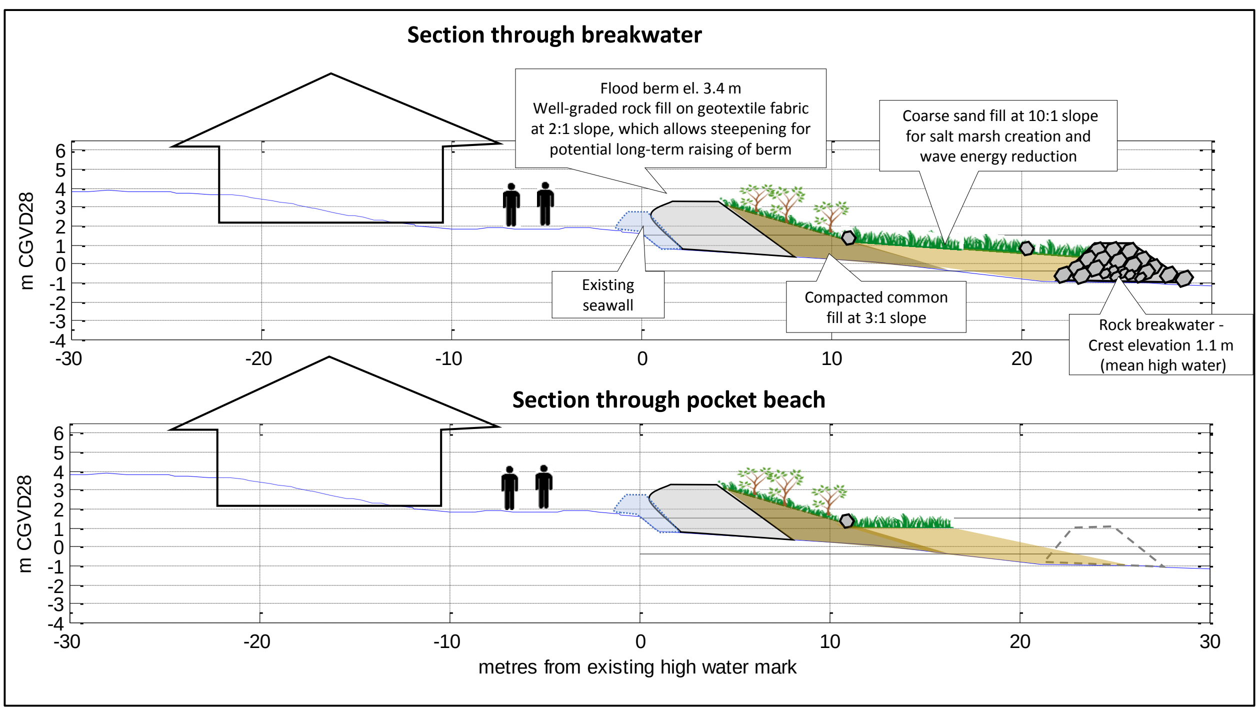


Figure 3.10.b: Cross-sections 5 – Flood Dyke and Living Shoreline Alternative for Southwest Town Waterfront (Conceptual Design)

3.6 Options for Lighthouse Route

The average road elevation south of the town waterfront is 2.5 m. The length considered is approximately 800 m, as shown in Figure 2.12. It is very exposed to wave action and overtopping. Options are shown in **Figure 3.11** and include:

- Raising the road to at least 2.8 m over an approximate length of 700 m (recommended);
- Traditional armour stone revetment approach, with crest at 3.8 m over an approximate length of 800 m; and
- Living shorelines approach with crest at 3.2 m, using nearshore breakwaters and pocket beaches, which would offer recreational opportunities, over an approximate length of 800 m.

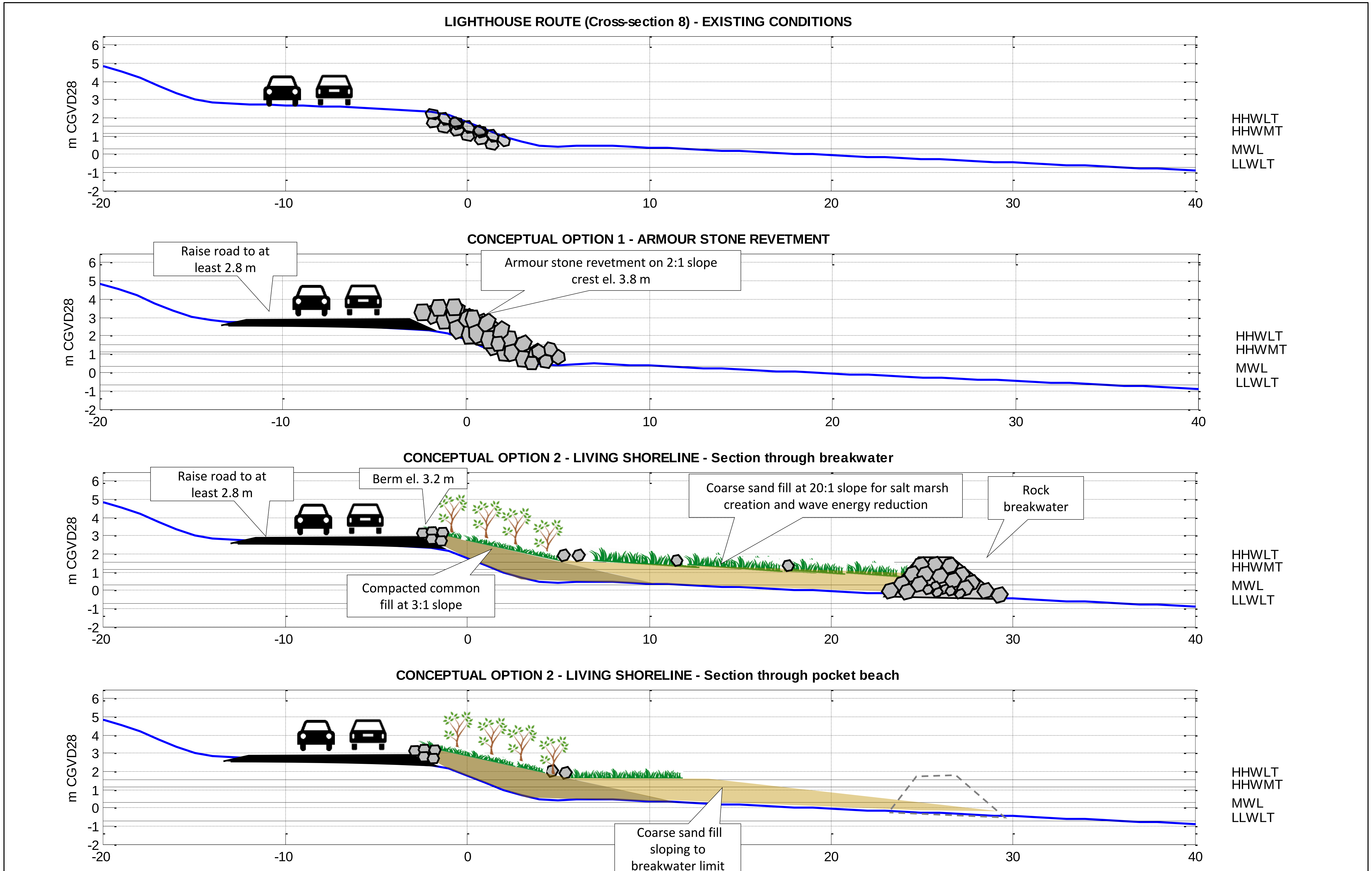


Figure 3.11: **Cross-section 8 – Lighthouse Route (Conceptual Design Options)**

CHAPTER 4 STORMWATER SYSTEM CHARACTERIZATION

While sea level rise and storm surges are the primary concern in terms of flooding in the Town of Mahone Bay, flooding from the Ernst Brook was also considered. This study conducted a comprehensive hydrologic and hydraulic analysis of the Ernst Brook water levels as influenced by tides and rainfall to provide a better understanding of the underlying causes of flooding, steering the assessment towards more efficient flood mitigation options.

4.1 Data Collection

4.1.1 Existing Data Collection

The following existing data was obtained and reviewed for the Mahone Bay Flood Study:

- Provincial topographic data (5m contours) from the Government of Nova Scotia;
- LiDAR data (1m) from the Town;
- Soil and geology mapping from Agriculture and Agri-Foods Canada;
- Intensity-Duration-Frequency (IDF) curves for Halifax International Airport climate station from Environment Canada;
- Historical rainfall radar data for the Town of Mahone Bay from Environment Canada;
- Historical tide data and tide predictions for Lunenburg from Fisheries and Oceans Canada;
- Historical tide data and tide predictions for Halifax from Fisheries and Oceans Canada; and
- Published reports on climate change by Environment Canada and Fisheries and Oceans Canada.

4.1.2 Field Data Collection

Measurements and photos were taken for four bridges along the Ernst River so that they could be included in the hydraulic model. This ensures any restrictions along the river will be accurately represented. The location s of the bridges for the study are presented in **Figure 4.1**.

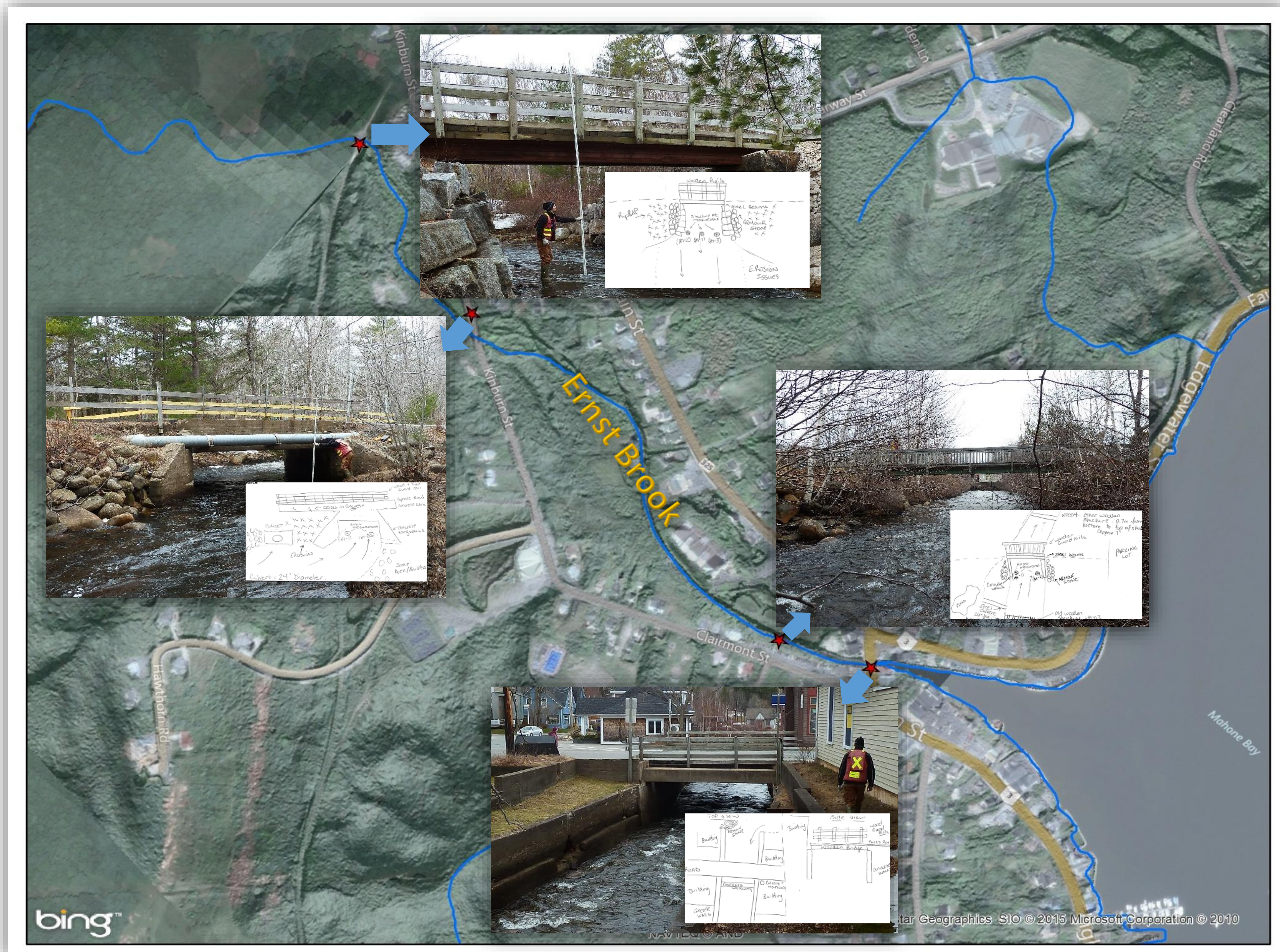


Figure 4.1: Hydraulic Structure Survey

4.2 Watershed Delineation and Watershe Characteristics

Watersheds were delineated using a combination of the LiDAR data and the 5m contours for the Ernst Brook. The major and minor watershed delineations are presented in **Figure 4.2**.

Watershed characteristics were estimated based on the LiDAR data, aerial photography, land use mapping and soil mapping using GIS techniques. Imperviousness and roughness coefficients were estimated for each land use and applied to the watersheds using area-weighted averages. The capillary suction head and saturated hydraulic conductivity of the soil were estimated for each soil class from the soil mapping provided by Agriculture and Agri-Foods Canada and then applied to the watersheds using area-weighted averages. The estimated watershed characteristics for each watershed are presented in **Table 4.1**.

Table 4.1: Watershed Characteristics

Subwatersheds	Area (ha)	Slope (%)	Maximum Overland Flow Length(m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)
S1	4.7671	38.4	222.457	62.228	0.023	0.075	169.926	0.066
S2	20.586	9.12	229.583	16.608	0.022	0.215	158.473	0.061
S3	58.457	9.03	814.993	39.736	0.012	0.132	125.903	0.048
S5	1426.6	1.73	2547.488	18.296	0.156	0.207	117.574	0.01
S4	13.232	7.06	263.428	7.834	0.012	0.23	156.952	0.061
S6	10.739	6.6	237.143	2.772	0.044	0.168	156.952	0.061
S7	16.793	4.58	401.144	28.106	0.031	0.186	156.952	0.061

4.3 Land Use Mapping

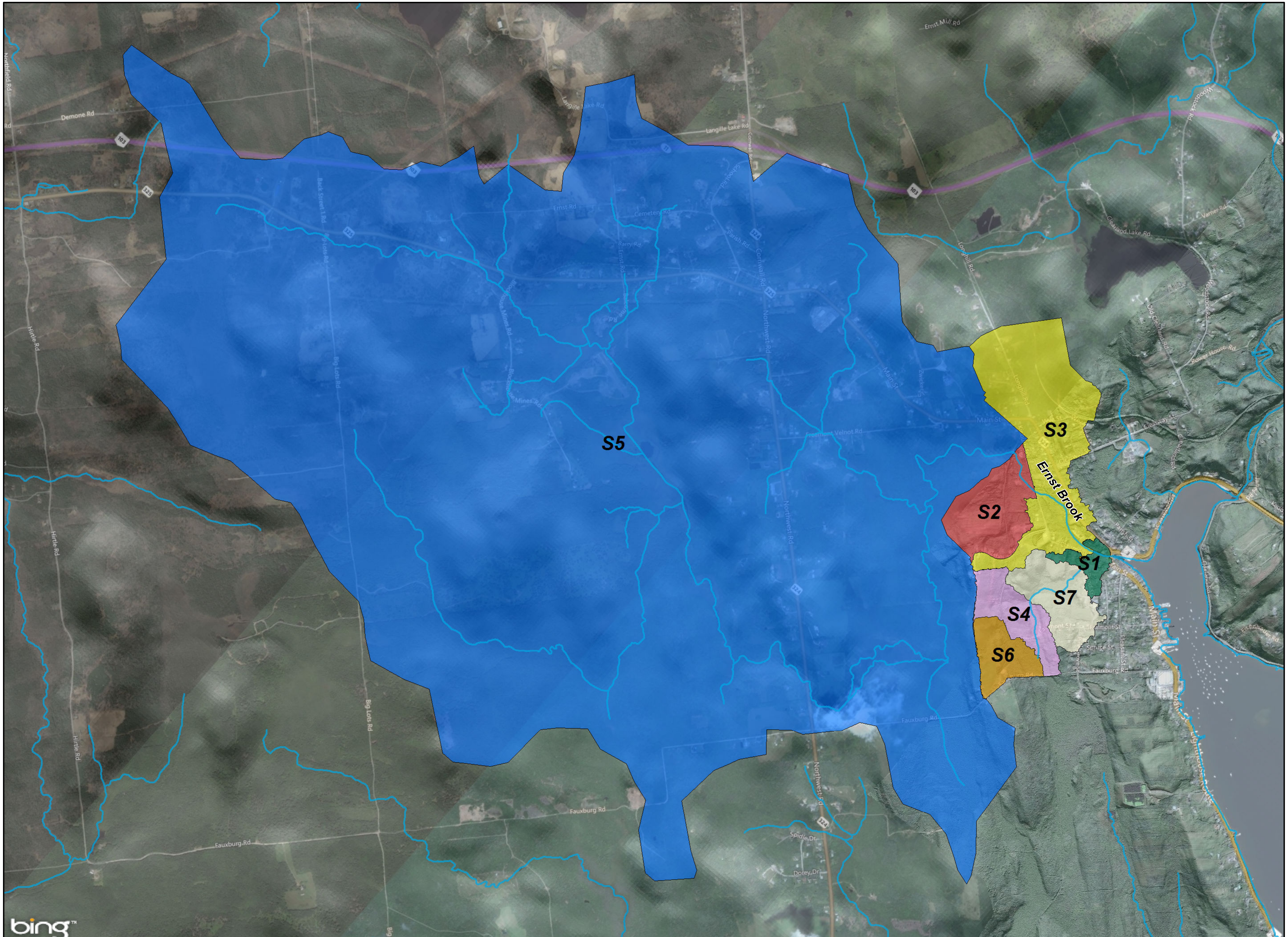
Land use areas were delineated within the watersheds based on aerial photography and the Nova Scotia Department of Natural Resources GIS database for the following five land use types: Forest, Brush, Developed, Waterbody and Wetland. The resultant land use mapping is presented in **Figure 4.3**.

4.4 Rainfall Analysis

Multiple flood events have occurred within the Town of Mahone Bay over the past decade that have resulted in significant damage to private and public properties.The December 2014 rainfall event was selected for hydraulic model calibration since it was one of the largest rainfall event that had occurred in recent years with a total rainfall amount of 107 mm recorded at Halifax International Airport.

4.4.1 Radar Rainfall Analysis

Rainfall data is often one of the largest uncertainties during the calibration process. This uncertainty is amplified when the watershed is large and the rain gauges are very sparse. The rainfall intensity and total rainfall volume can vary significantly according to the location within the watershed. As the watershed size becomes larger, using a single point to estimate the rainfall across the entire watershed becomes less and less representative. The other uncertainty with using an isolated rainfall gauge is that the peak flow can be significantly affected by how the storm travels over the watershed.



Mahone Bay

Town of Mahone Bay

Mahone Harbour
Flood Prevention and
Shoreline Enhancement Plan

Legend:

Watersheds

- S1
- S2
- S3
- S4
- S5
- S6
- S7

Figure 4.2

Watershed Delineation

Scale: 1:20,000

Date: July 2015
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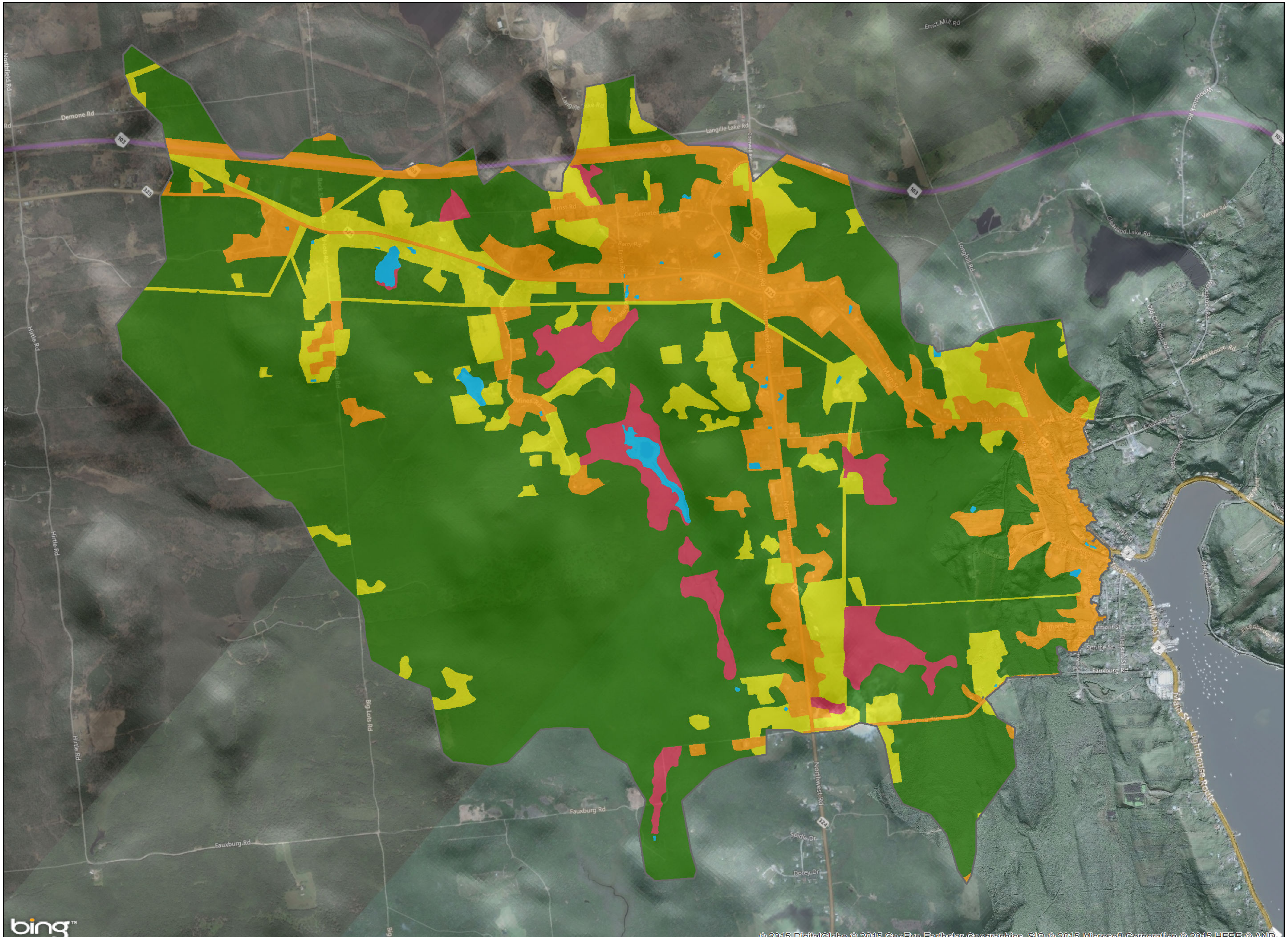

Mahone Bay

Town of Mahone Bay

Mahone Harbour Flood Prevention and Shoreline Enhancement Plan

Legend:

Land Use

- Clear Cut
- Forest
- Lake
- Urban
- Wetland

Figure 4.3

Land Use Mapping

Scale: 1:20,000

Date: July 2015
CBCL Project #: 151016.00

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Radar data must be calibrated before it is used for modelling. Radar data is very useful for determining the spatial distribution of a storm event, however, areal measurements and ground measurements (from point rain gauges) are often different and can be in error of a factor of 2 or more. This error is due to the vertical and horizontal air motions, the measurement of radar reflectivity factor, evaporation and advection of the precipitation prior to reaching the ground and variations in drop-size distribution. One of the most common calibration methods is to use rain gauge data to “ground-truth” the radar data. Calibration results will improve based on the amount of rain gauges within the study area and the spatial distribution of the rain gauges. Calibration results will also improve based on how close the rain gauges are to the study area itself.

For the December 10th 2014 storm event, there are no Environment Canada operated rainfall gauges within the study area. However, one Environment Canada rain gauging station and three private rainfall gauges are within relatively close proximity to the study area were used to calibrate the radar data. The proximity of these rainfall gauges are shown on **Figure 4.4** and include:

- Shearwater RCS station (Environment Canada);
- INOVASCO139 (Private);
- INOVASCO58 (Private); and
- INSFOXCR2 (Private).

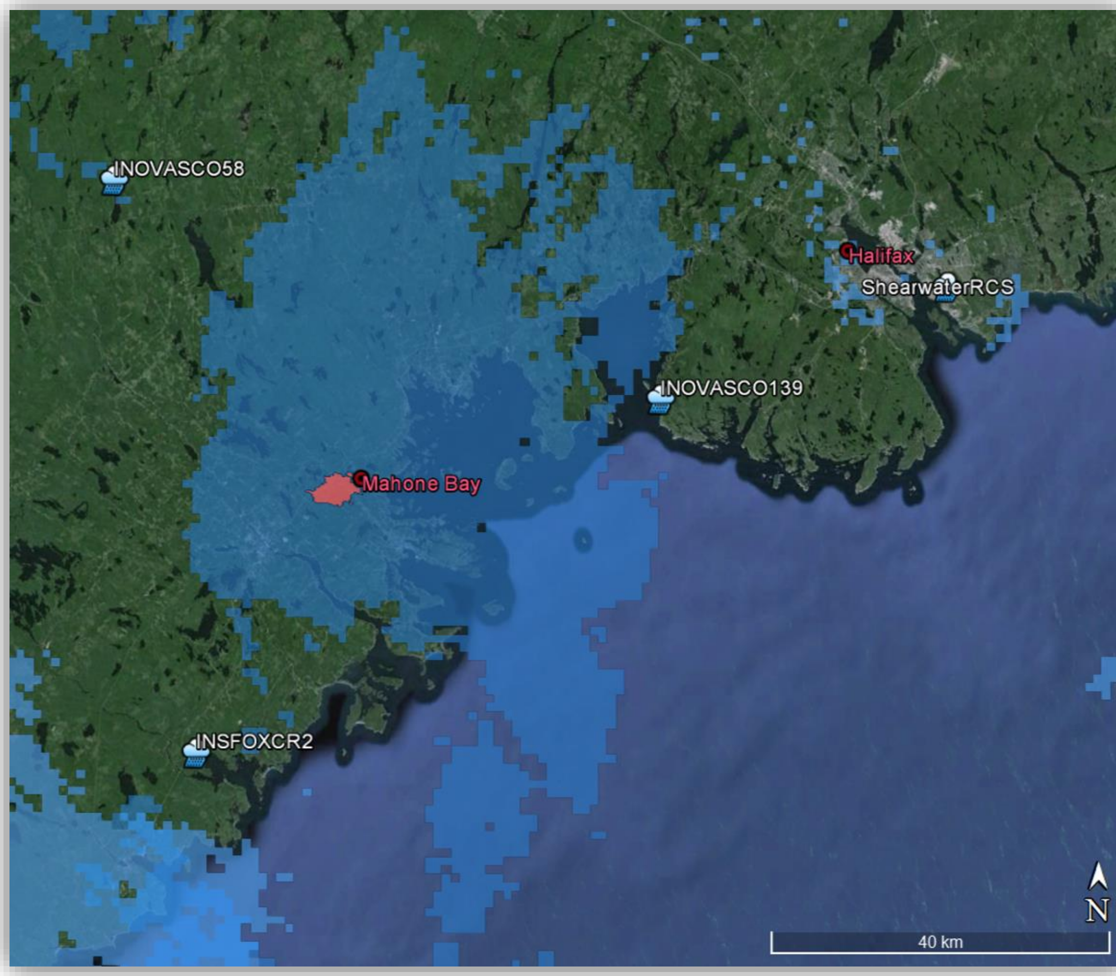

Figure 4.4: Spatial Variation of Rainfall Intensity and Proximity of Rainfall Gauges to Mahone Bay

Calibration and the radar analysis was completed using PCSWMM's Radar Acquisition and Processing (RAP) Toolkit. PCSWMM's RAP toolkit is an innovative tool that, in Canada, is currently limited to the province of Nova Scotia. The calibration method chosen was the average method, which compares the average rainfall of each 1 km^2 radar grid element to the average rainfall measured by nearby rain gauges over a specified duration of time. The average measured rainfalls of the nearby gauges are averaged between the applicable gauges. Based on this comparison, a calibration factor for each 1 km^2 grid element is calculated and then applied to the radar measured rainfall. These calibrated rainfall intensities (for each 1 km^2 grid element) are then averaged across each watershed in the study area, providing each watershed with a unique and representative rainfall intensity distribution that can then be used for a more representative hydrologic model calibration. Although the watershed area is relatively small, the results, shown on **Figure 4.5**, clearly show the spatial variation of rainfall intensity over the watersheds.

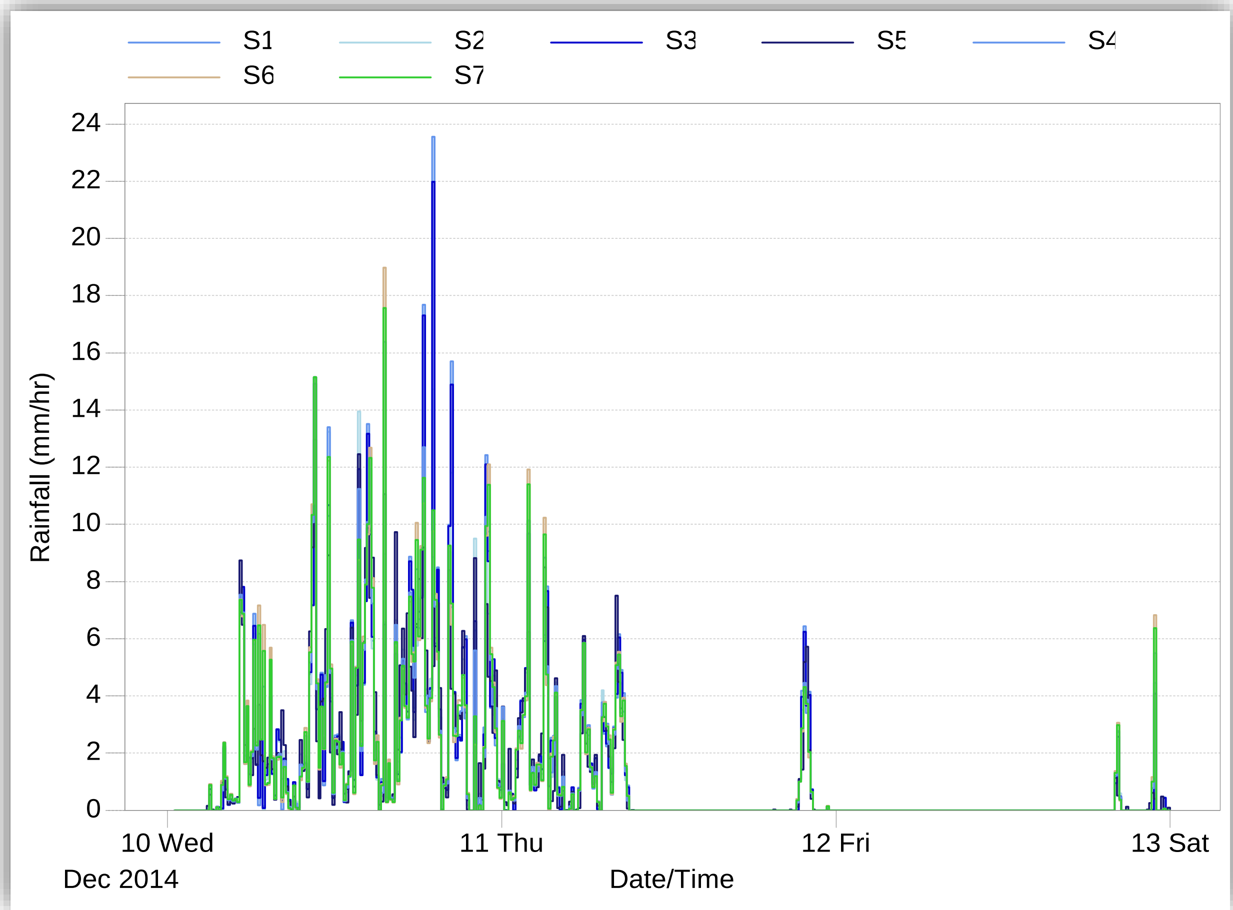

Figure 4.5: Spatial Variation of Rainfall Intensity and Proximity of Rainfall Gauges to Mahone Bay

4.4.2 Impact of Climate Change on Rainfall

The impact of climate change on extreme rainfall amounts was analysed for this study to model future climate change conditions for the year 2115. Environment Canada (EC) has used the results from several Global Climate Change models to estimate changes in the 24 hour rainfall amount for various return periods at several major climate stations in the region located at Greenwood, Shearwater and Sydney. These results are outlined in the 2009 EC report “Climate Change Scenarios for Atlantic Canada Utilizing a Statistical Downscaling Model”. According to the report, rainfall intensity in the Halifax area may increase by 30% within the next 100 years. Due to the proximity of the Town of Mahone Bay to the Halifax area we can use these results to estimate the potential effect of climate change in the study area. Therefore, climate change scenarios for future condition assessment were based on a 30% increase to the calculated design storms and to the projected extreme flows.

CHAPTER 5 DESIGN OF STORMWATER MANAGEMENT INFRASTRUCTURE

5.1 Modelling Approach

Hydrologic and hydraulic modelling was carried out for this study to quantify and analyse the flood risks in the community. Flood simulations were performed using the hydrologic and hydraulic modelling program PCSWMM.

PCSWMM integrates Version 5 of the Storm Water Management Model (SWMM) with a GIS engine and is capable of performing 2D hydrodynamic simulations. SWMM is a hydrologic and hydraulic model produced by the United Stated Environmental protection Agency to study urban drainage systems. It conducts unsteady flow calculations to simulate water backup, pooling and culvert hydraulics by dynamically solving the continuity and momentum equations with a finite difference scheme.

PCSWMM-2D was used for this study to estimate watershed flows and to simulate flooding in the Town of Mahone Bay.

5.2 Hydrologic and Hydraulic Model Analysis

A hydrologic model was developed using PCSWMM to estimate the flows from each watershed. The model was developed by inputting the estimated watershed characteristics (**Table 4.1)**. The flows estimated by the hydrologic model were then used as inputs for the hydraulic model.

An integrated 1D and 2D hydrodynamic model of the watercourses that pass through the Town of Mahone Bay was developed using the LiDAR mapping and field measurements to estimate water levels along the floodplain and coastline. The 1D model components included the bridges and culverts as well as the component of the river below the LiDAR surface. The river and coastal floodplain was then modelled by assembling a 2D mesh based on the LiDAR surface with a mesh resolution that varied between 2.5 m to 10 m. Break lines were included in the mesh at roadways to ensure that locations with sharp changes in elevation are modelled with sufficient detail.

5.3 Model Calibration

No historical water level or flow data was available within the Town of Mahone Bay. The hydrologic and hydraulic model was therefore calibrated based on anecdotal flooding information gathered from meetings with Town Staff and residents by simulating a recent flood event and comparing the results to what was reported.

The December 2014 rainfall event was selected for hydraulic model calibration since it was one of the largest rainfall events that had occurred in recent years with a total rainfall amount of 107 mm recorded at the Halifax International Airport. Both hydrologic and hydraulic parameters were modified in the model until the flooded areas identified by records were able to be simulated by the model during the December 2014 event.

Floodlines developed for the December 2014 event are presented in **Figure 5.1** comparing the estimated flood extents with the identified flood areas. The floodlines were produced by using GIS tools to interpolate the model results to the resolution of the LiDAR data, which is a 1 m by 1 m horizontal square grid. The model and resulting floodplain mapping does not include the local flooding issues in these areas likely caused by groundwater and local drainage issues.

5.4 Extreme Watershed Flow Estimation

The hydrologic model calculated extreme runoff flows from each watershed for the 1 in 2 and 100 year storm event with impacts of climate change, which would be used as input for the hydraulic model for the flood protection design. Rainfall hyetographs that follow the Chicago Distribution with 5-minute discretization intervals and 24-hour durations were developed for each storm event based on the Intensity-Duration-Frequency (IDF) curves (upper bound 95% confidence interval) from Environment Canada for the Halifax International Airport climate station. Based on Rainfall Frequency Altas for Canada (1985), Mahone Bay is in the same range as Halifax International Airport station which has IDF curves calculated from 18 years of rainfall data (1977-2013).

Rainfall hyetographs for future climate change conditions were then developed for the 1 in 2 and 1 in 100 year storm events by scaling the hyetographs to 30% predicted increases in 24-hour rainfall amounts for the year 2115.

All rainfall hyetographs for future (2115) climate change conditions are presented in **Figure 5.2**. Peak watershed flows estimated by the hydrologic model for each storm event are presented in **Table 5.1**.

Mahone Bay

Town of Mahone Bay

Mahone Harbour Flood Prevention and Shoreline Enhancement Plan

Legend:

Floodplain 2014 Dec 10th

Coastal Flooding 2014 Dec 10th

Figure 5.1

December 2014 Event

Scale: 1:3,000

Date: July 2015
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Table 5.1: Estimated Peak Flows and Runoff Coefficients

Subwatershed	1in 2 Year Strom (2115)		1in 100yr Year Strom (2115)	
	Peak Runoff (m^3/s)	Runoff Coefficient	Peak Runoff (m^3/s)	Runoff Coefficient
S1	1.17	0.98	2.75	0.99
S2	2.47	0.95	6.08	0.96
S3	7.64	0.96	19.01	0.97
S5	42.2	0.79	94.56	0.80
S4	1.18	0.89	2.94	0.91
S6	0.98	0.89	2.5	0.91
S7	1.78	0.91	4.5	0.93

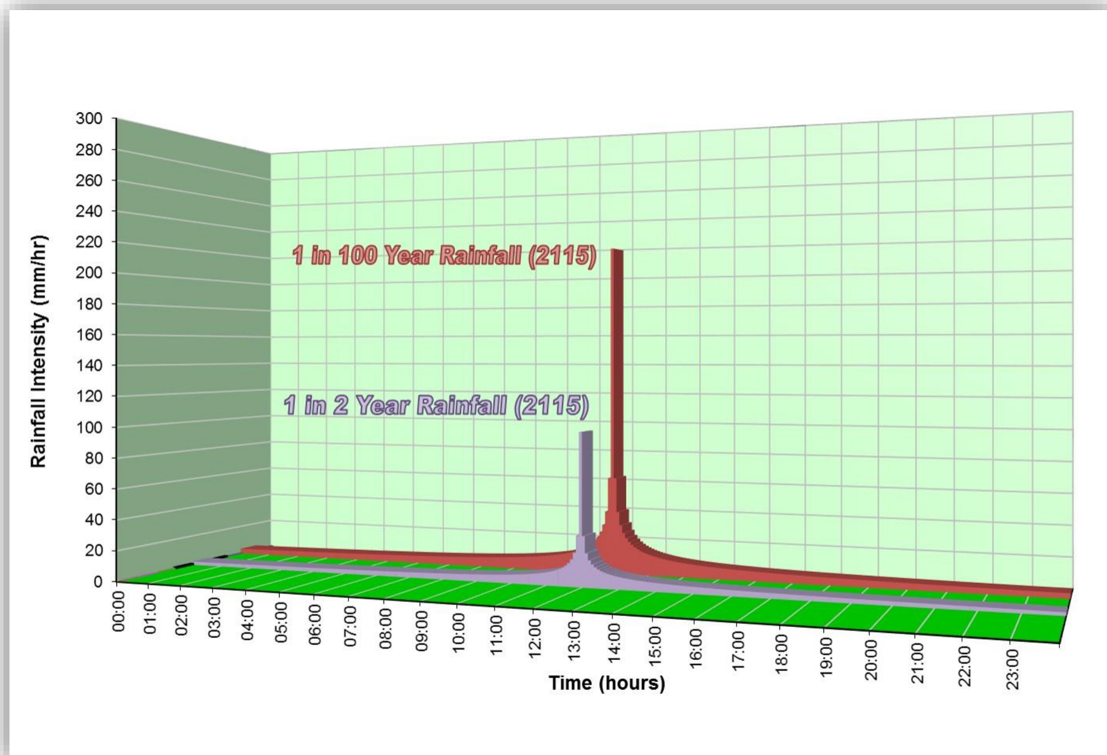

Figure 5.2: Rainfall Hyetographs for Future (2115) Climate Change Conditions

5.5 Calculation of the Ernst Brook Extreme Water Levels:

Once compiled and calibrated, the model was used to assess the 1 in 100 year flood scenarios for the Ernst Brook. Since the rainfall and tides have their own individual influences on the river's extreme water levels, each event was modelled with a dominating influence from the rainfall and tides. Two scenarios consisting of combinations of rainfall, tide, storm surge, sea level rise conditions were simulated for floodlines. Since extreme rainfall events often bring storm surges, the 1 in 2 year storm surge occurring during the HHWLT was selected as representative of the sea level conditions during the 1 in 100 year rainfall events, and vice versa. Thus, each floodline delineation consists of the maximum of the extreme rainfall flood event, extreme sea level flood event. The resulting floodline therefore does not represent the flood extents for a single event that can occur, but rather the combined flood extents for two different types of events, each having the same return period.

Climate change will likely bring more precipitation to Atlantic Canada. In this study future (2115) flood risks are assessed. Assessing future flood risk will allow for better management of surface and storm water resources through land use planning and infrastructure design specifications. Floodlines were delineated based on the selected event combinations, which is presented in **Table 5.2.** The detailed information of tides and extreme still water levels can be found in **Table 2.1**. The 1 in 2 year storm surge value was interpolated from the upper bound estimates presented by Richards & Daigle using a log-normal distribution. Floodplains for these two scenarios were presented in **Figure 5.3.**

Table 5.2: Selected Event Combinations for Floodline Delineations

Scenario	Rainfall (Return Period)	Tide Scenario	Storm Surge (Return Period)	Sea Level Rise (Year)	Peak Sea Level [geodetic] (m)
Scenario-1	100yr (2115)	HHWLT	2yr	2115	3.45
Scenario-2	2yr (2115)	HHWLT	100yr	2115	**4.02**

5.6 Analysis of Flooded Areas

The floodline delineations shown in **Figure 5.3** demonstrates that there are widespread flooding risks estimated in the Town of Mahone Bay. The inland flooding in Mahone Bay is caused by the complex interaction between rainfall, river flows, waves, tides and storm surge. Storms cause coastal flooding when water from the ocean is driven onto Mahone Bay by wind, tides, waves and storm surge. The severity of these floods can increase when intense rain falls upstream on the Ernst Brook is influenced by tides and surge.

Flood risks were found to be located along Edgewater Street and Main Street. These locations would be especially disastrous due to its high population density, its role as a central commercial area, and the significant amount of vulnerable infrastructure located inside the 1 in 100 year floodplain. The flooding occurring in the downtown of Mahone Bay during 1 in 100 year storm event is the combined effects of coastal and inland floods.

Flood risks were also noticed along Ernst Brook, which including residents and commercial buildings. Additional flooding occurred in the area adjacent to a pond to the west of Pond Street. The pond is connected to Ernst Brook by an outlet pipe with the diameter of 0.6m. When the brook floods in severe weather, the 0.6m outlet pipe reportedly backs up causing overflow from the pond. The identified flood risks around this area also indicated that the pond's outlet pipe was under sized.

Mahone Bay

Town of Mahone Bay

Mahone Harbour Flood Prevention and Shoreline Enhancement Plan

Legend:

Rain Floodplai Scenario-1

Tide Floodplain Scenario-2

Figure 5.3

1 in 100 Year Floodplains under Climate Change (2115)

Scale: 1:3,000

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5.7 Analysis of Flood Mitigation Options

The purpose of assembling a hydraulic model is not only to understand better the processes that lead to flooding, but also to allow for testing of options for flood mitigation. Flooding can be caused by a multitude of factors, including not only high flows and inadequate infrastructure, but also high surface roughness, low slope, or lack of sufficient room in the floodplain for water storage. Flood mitigation options explored in this study were mainly focused on the sites which are showed in **Figure 5.4.** There are nine options included in this analysis, and each of these options was tested against extreme rainfall and tide using the various hydraulic models. Main types of flood mitigation techniques including:

- Constructing sea walls to contain the extreme tides;
- Constructing berms to contain river floods;
- Upgrading the existing main street bridge to protect specific area at risk;
- Upgrading hydraulic structures to reduce specific areas at risk; and
- Implementing Best Management Practice to reduce flows.

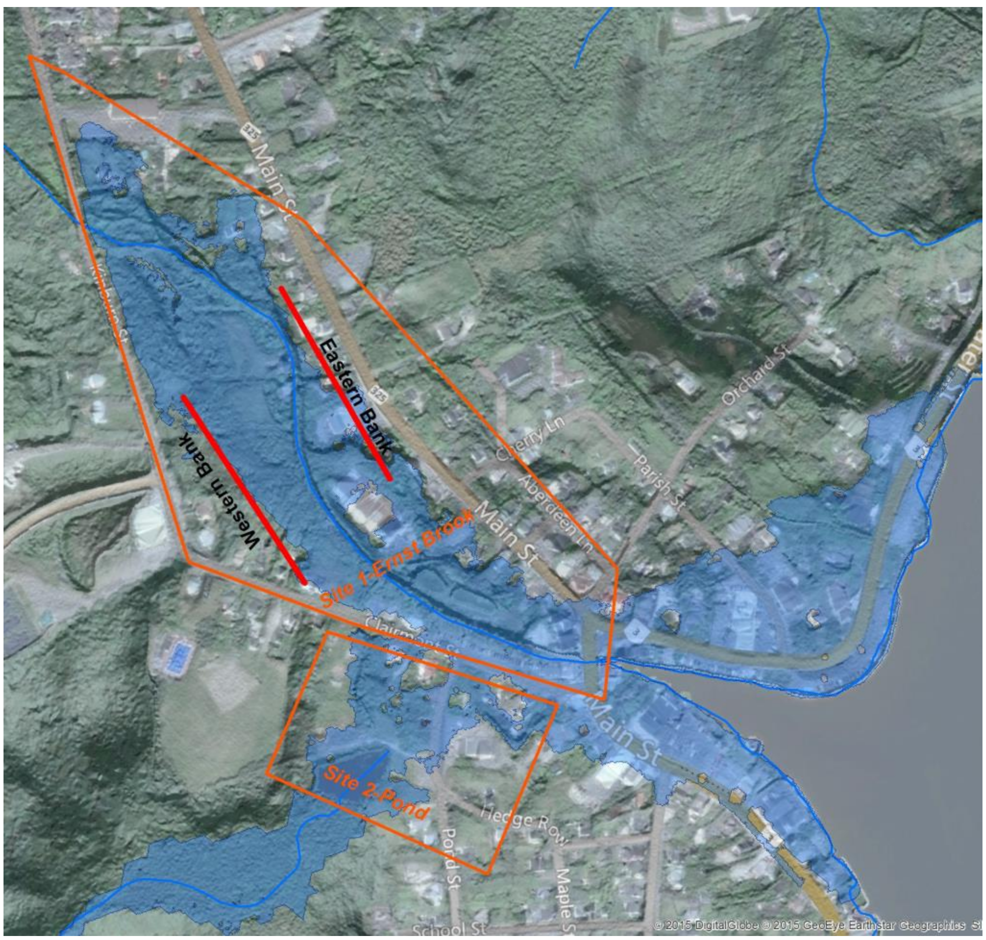


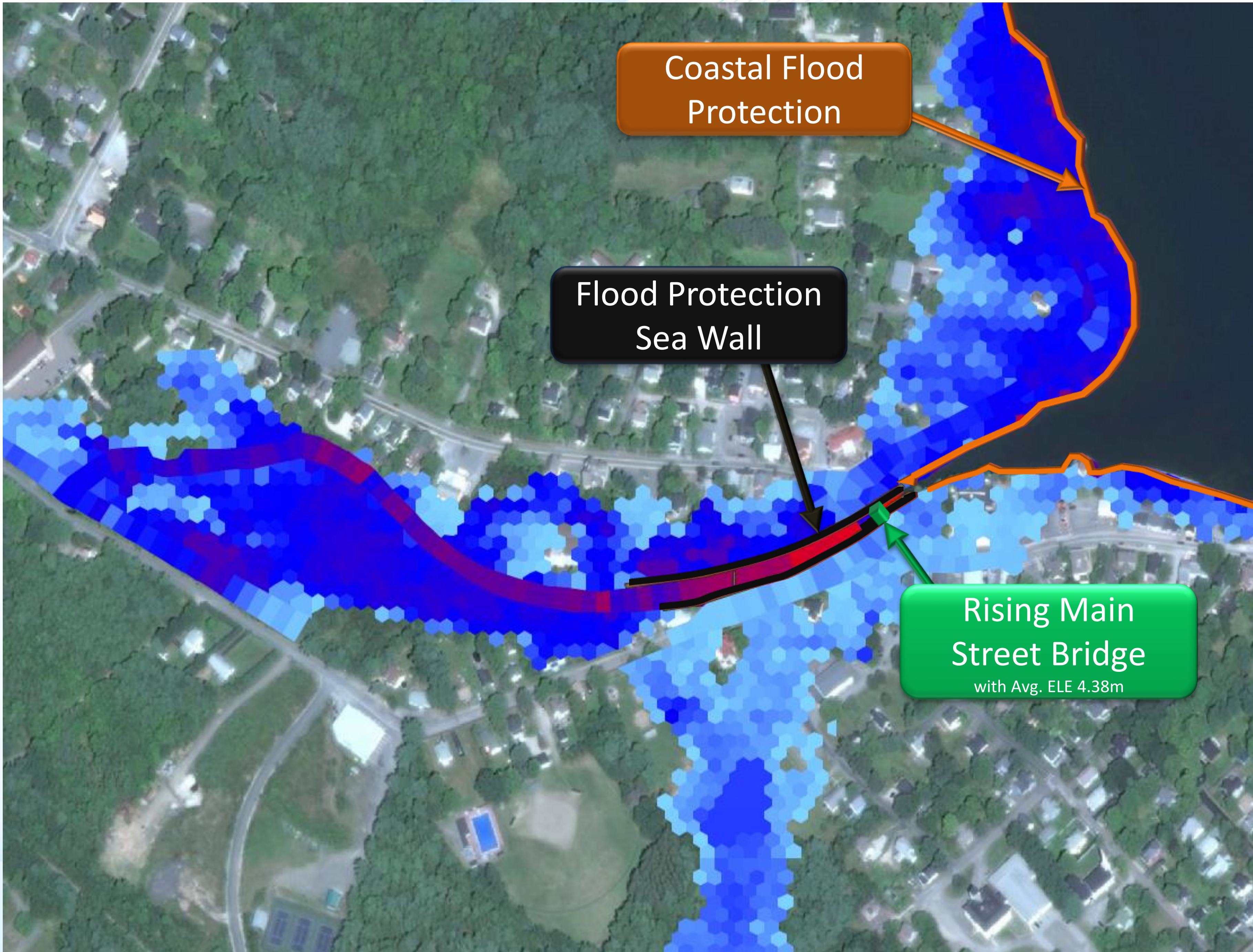
Figure 5.4: Site Plan for Flood Mitigation Options

The implementation of these flood mitigation measures are summarized in **Table 5.3**. Each flood protection technique was represented by a unique colours.

Table 5.3: Flood Mitigation Options

Ernst Brook Area	Applicability of General Flood Mitigation Options							
	Vertical Sea Wall	Berm			Hydraulic Structure Upgrades			BMP
		Berm at Eastern Bank	Berm at Western Bank	Berm along the Pond	Main Street Bridge Upgrade	Pond Outlet Pipe Upgrade (Drains Water to the Ernst Brook)	Pond Outlet Pipe Upgrade (Drains Water to the Sea)	
Option1	X				X			
Option2	X	X						
Option3	X		X					
Option4	X	X	X					
Option5	X	X	X					
Option6	X							X
Option7	X					X		
Option8	X						X	
Option9	X			X				

Figure 5.5 to 5.22 show the mitigation options presented in Table 5.3 for Scenario 1 and Scenario 2.



Water Resources

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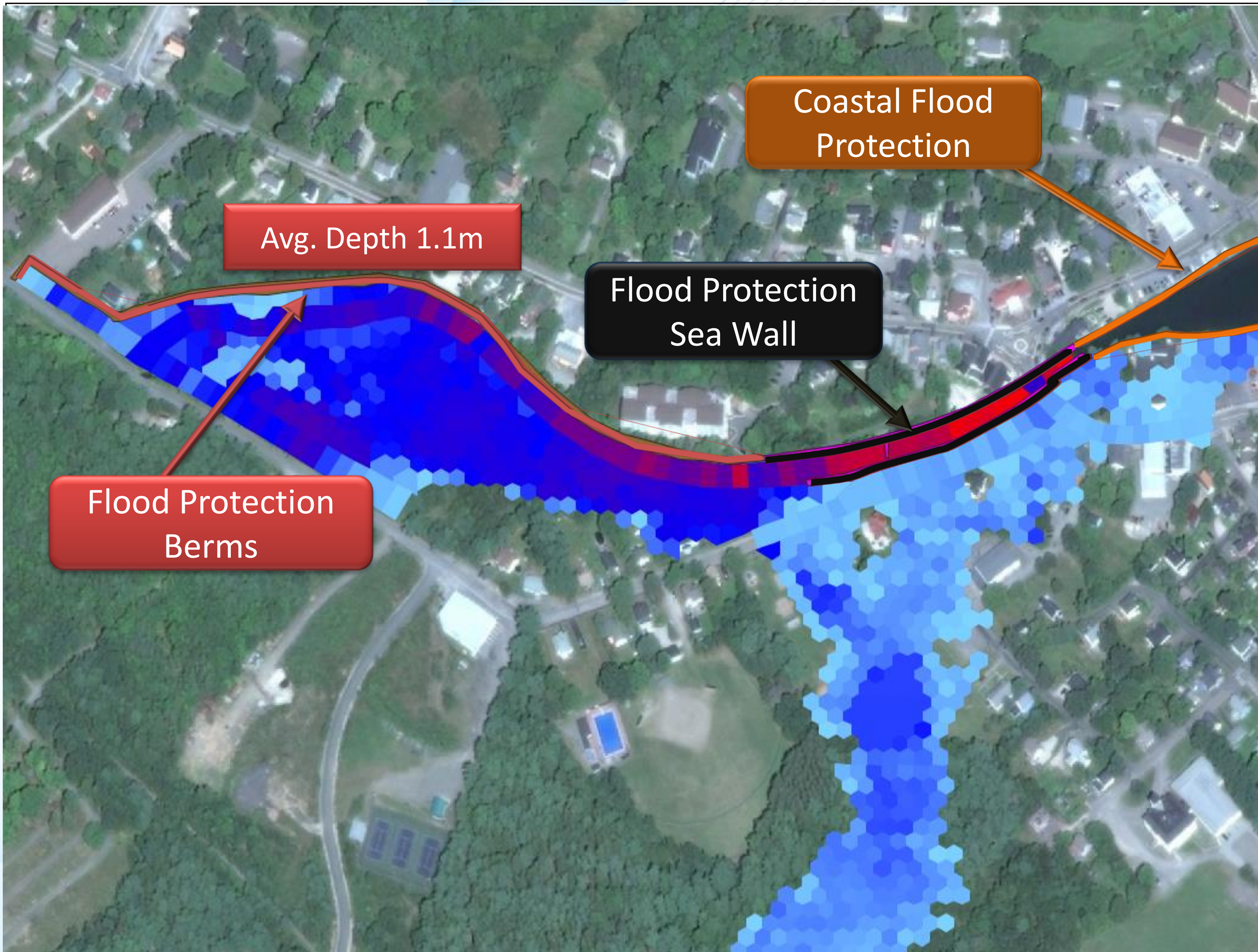
Sketch of Flood Mitigation Options for Scenario-1 (Option1)

Figure 5.5



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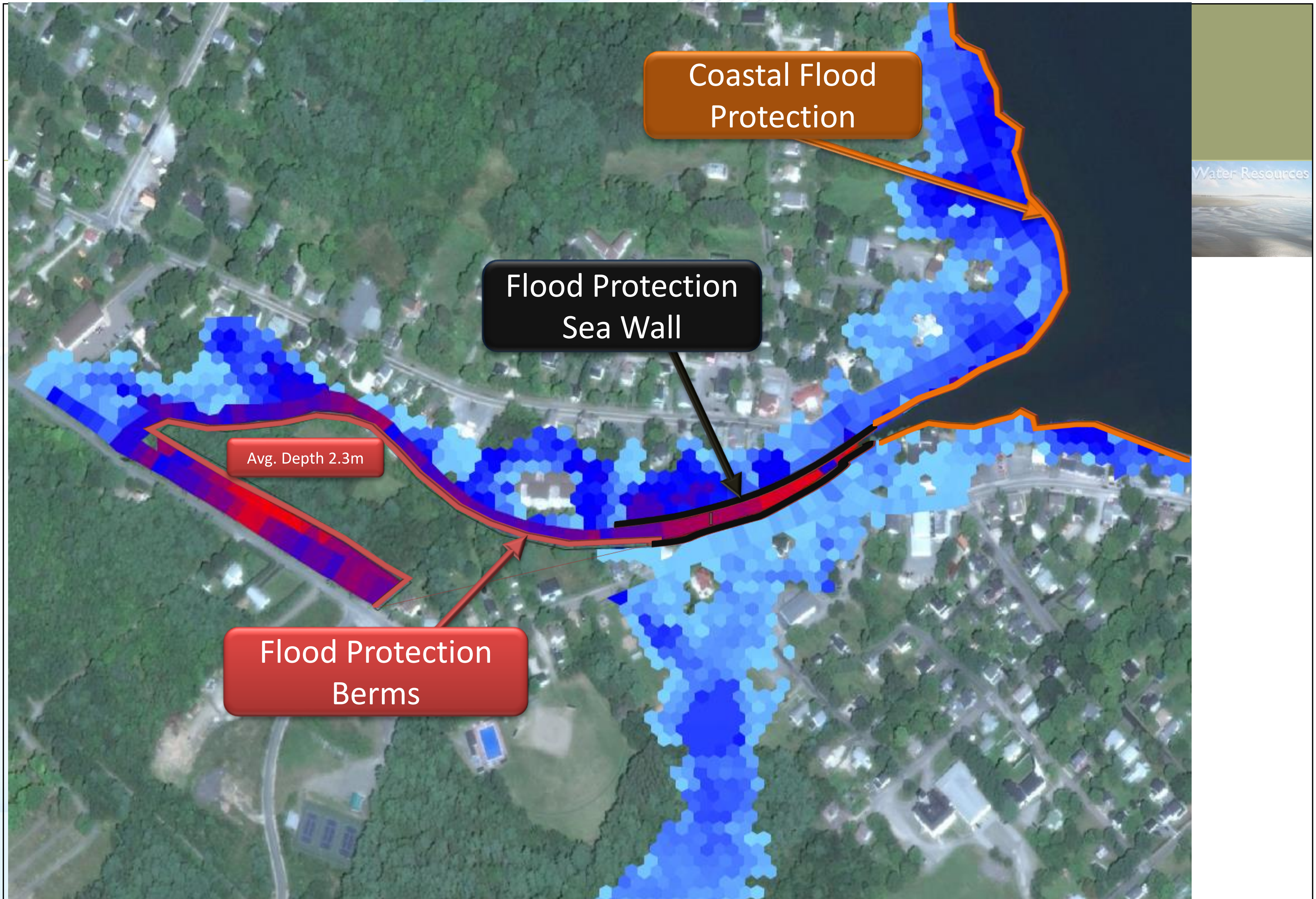
Sketch of Flood Mitigation Options for Scenario-1 (Option2)

Figure 5.6



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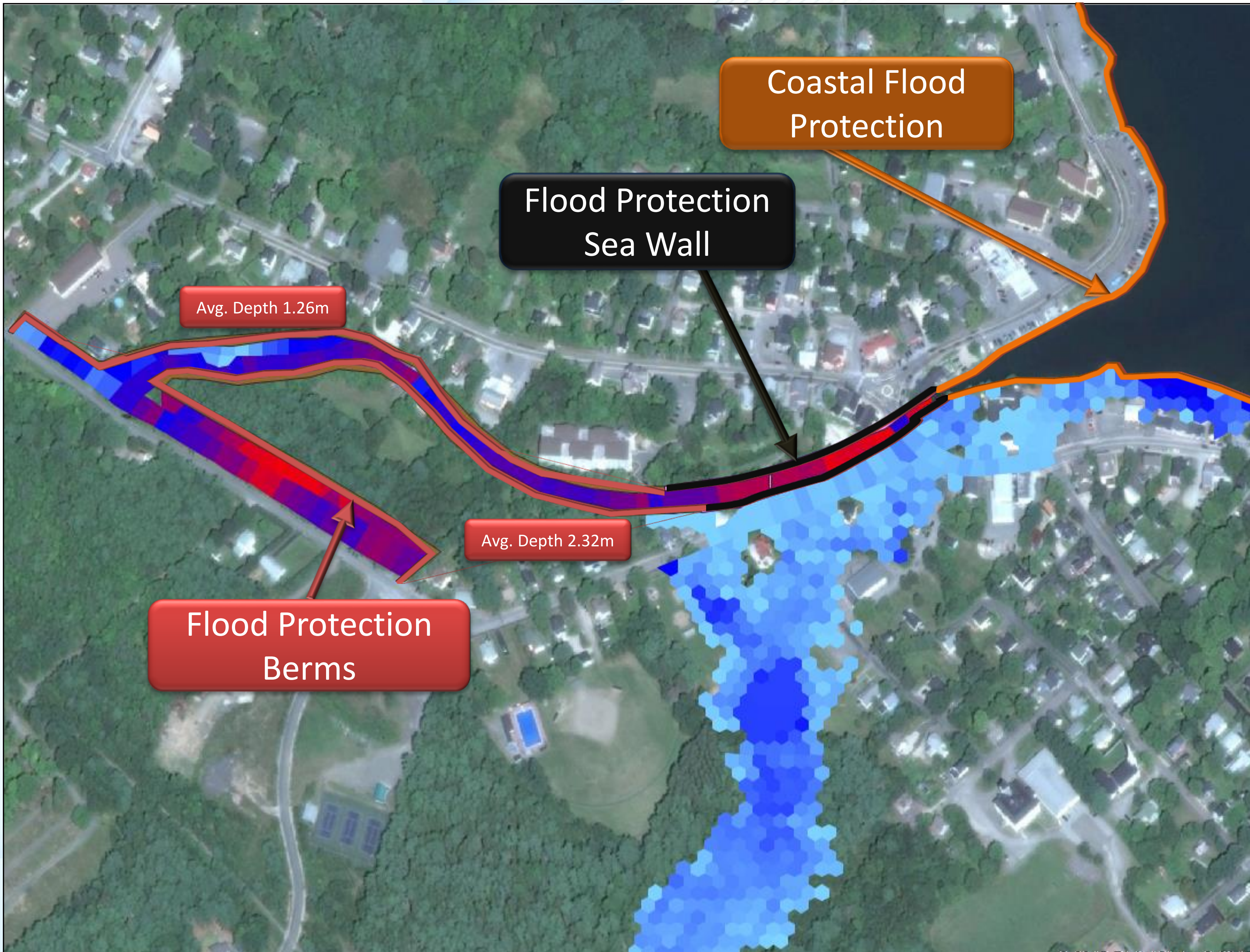
Sketch of Flood Mitigation Options for Scenario-1 (Option3)

Figure 5.7

Water Resources



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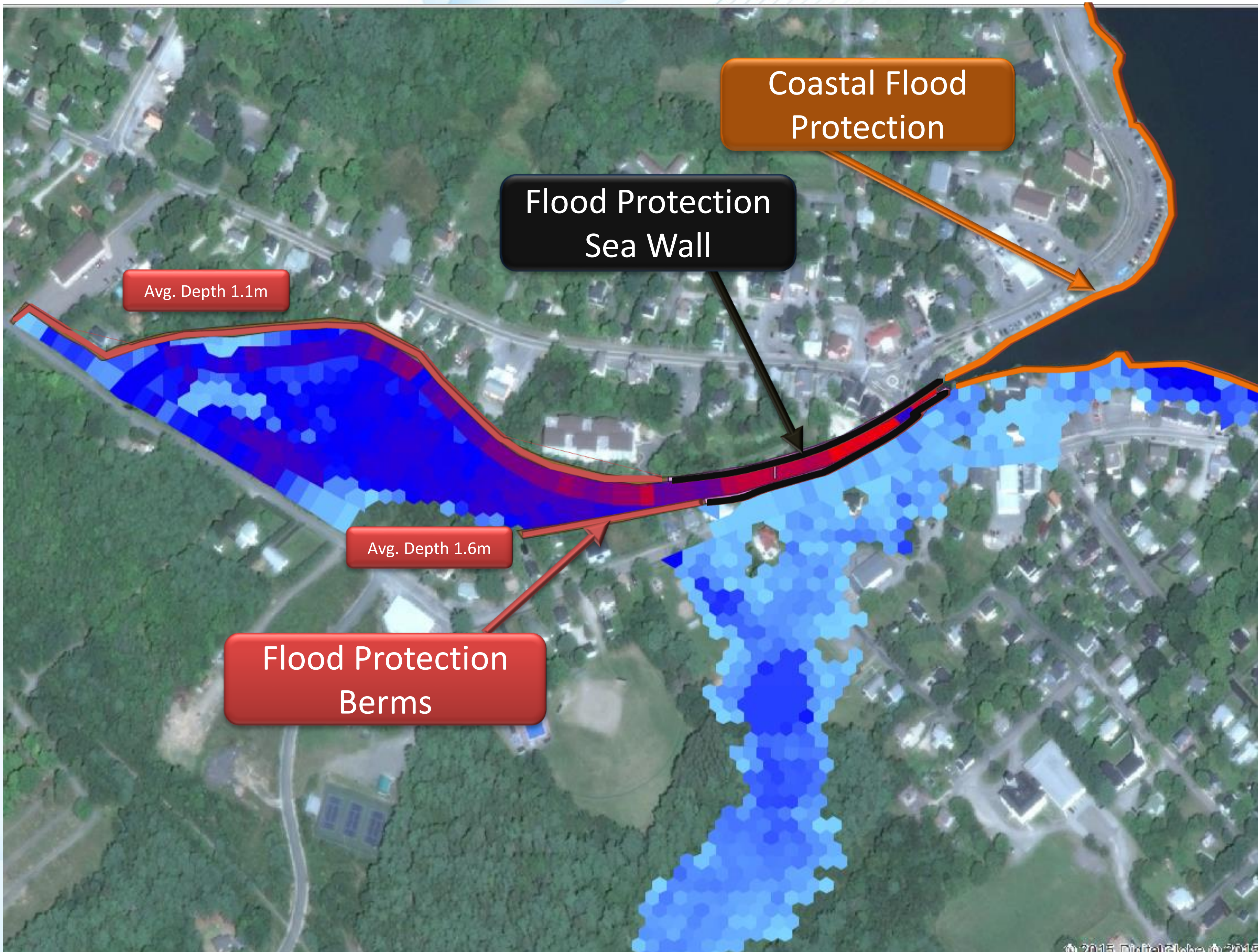
Sketch of Flood Mitigation Options for Scenario-1 (Option4)

Figure 5.8



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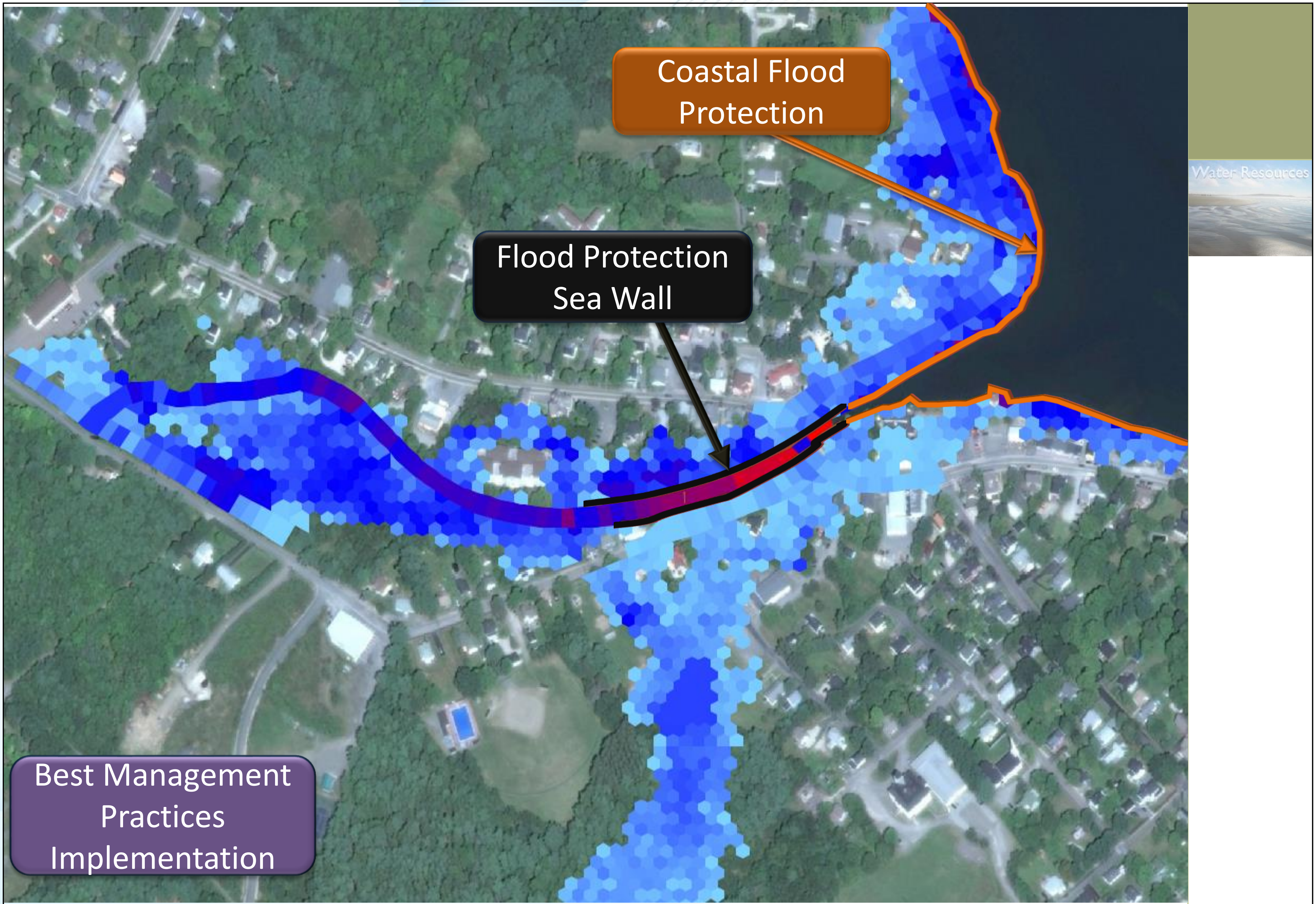
Sketch of Flood Mitigation Options for Scenario-1 (Option5)

Figure 5.9



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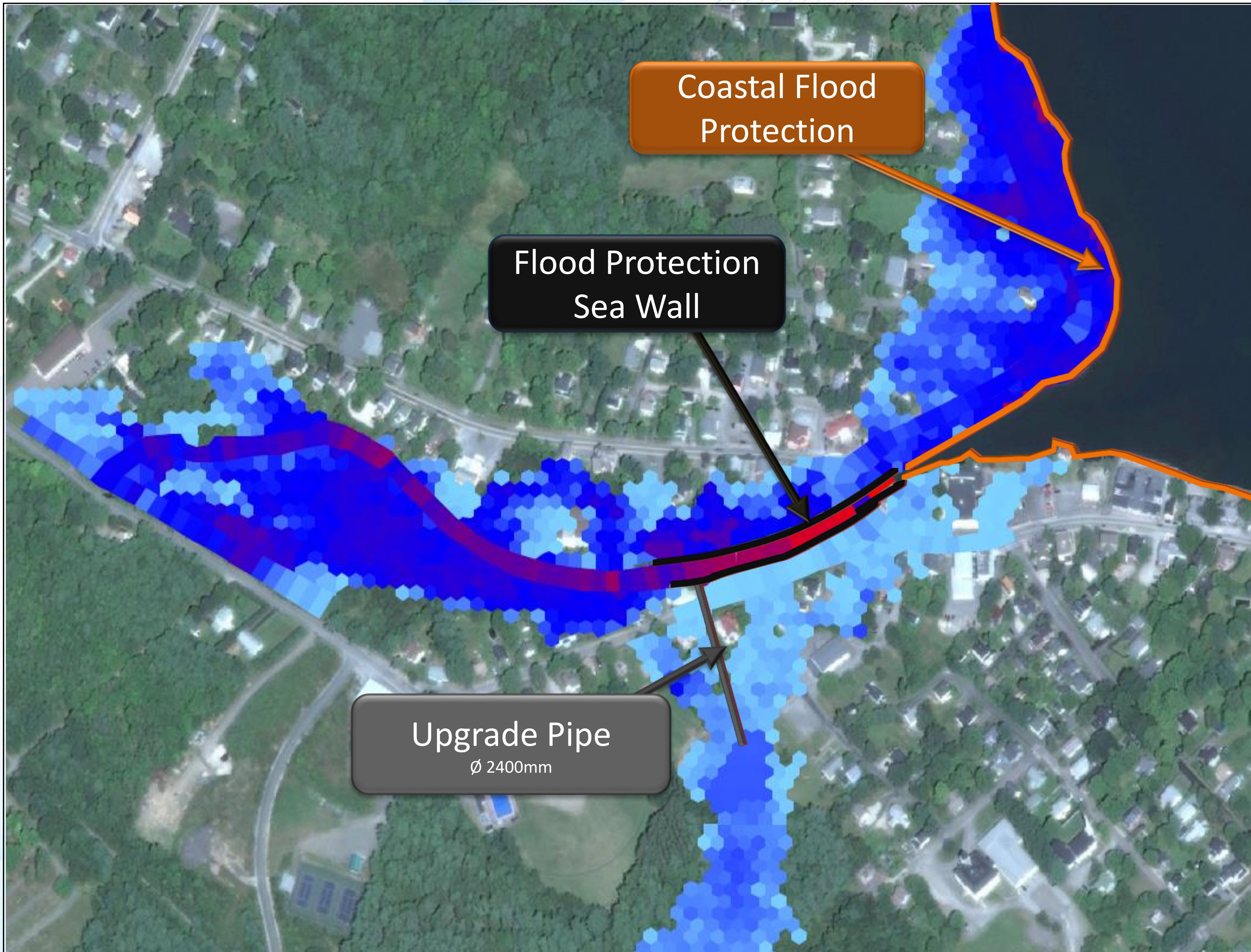
Sketch of Flood Mitigation Options for Scenario-1 (Option6)

Figure 5.10



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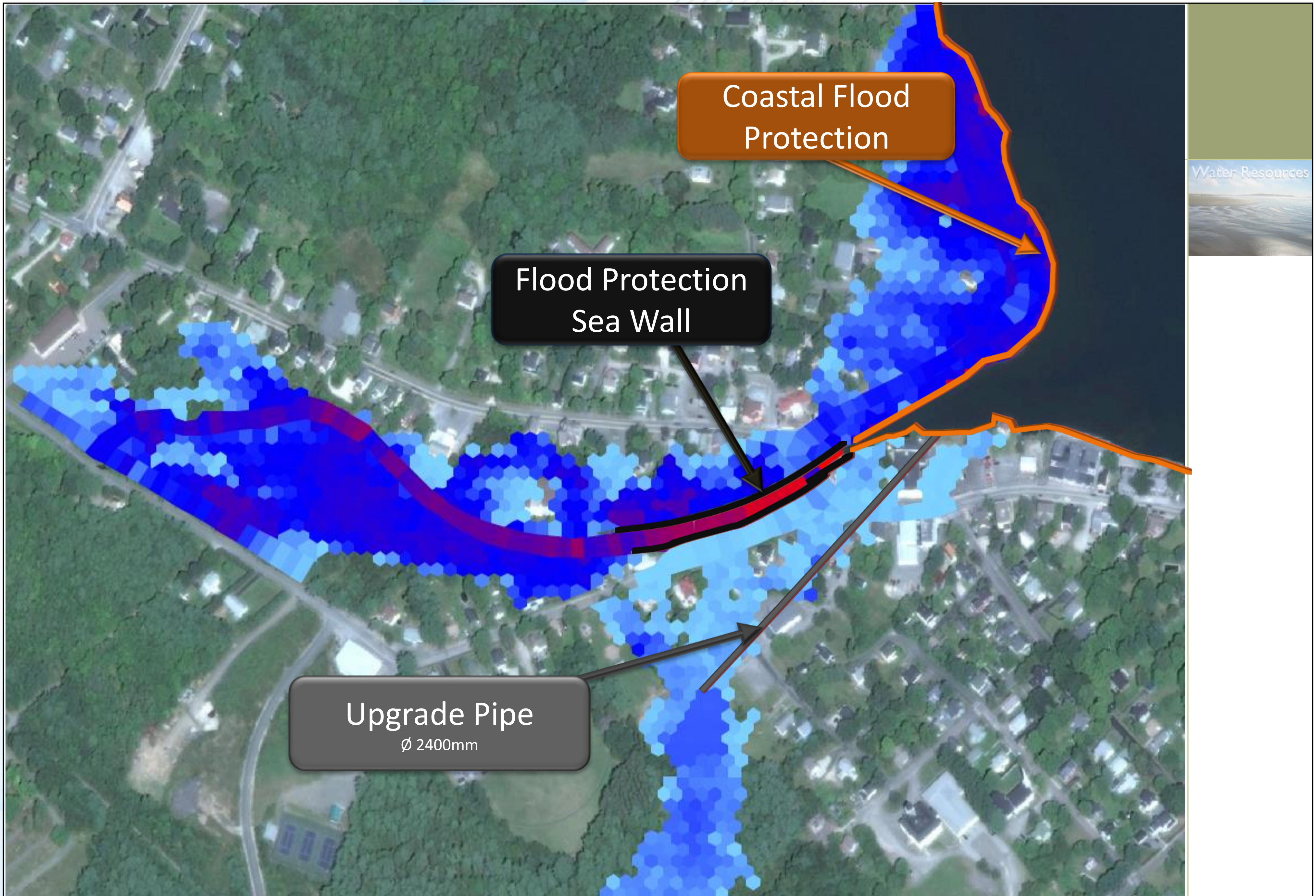
Sketch of Flood Mitigation Options for Scenario-1 (Option7)

Figure 5.11



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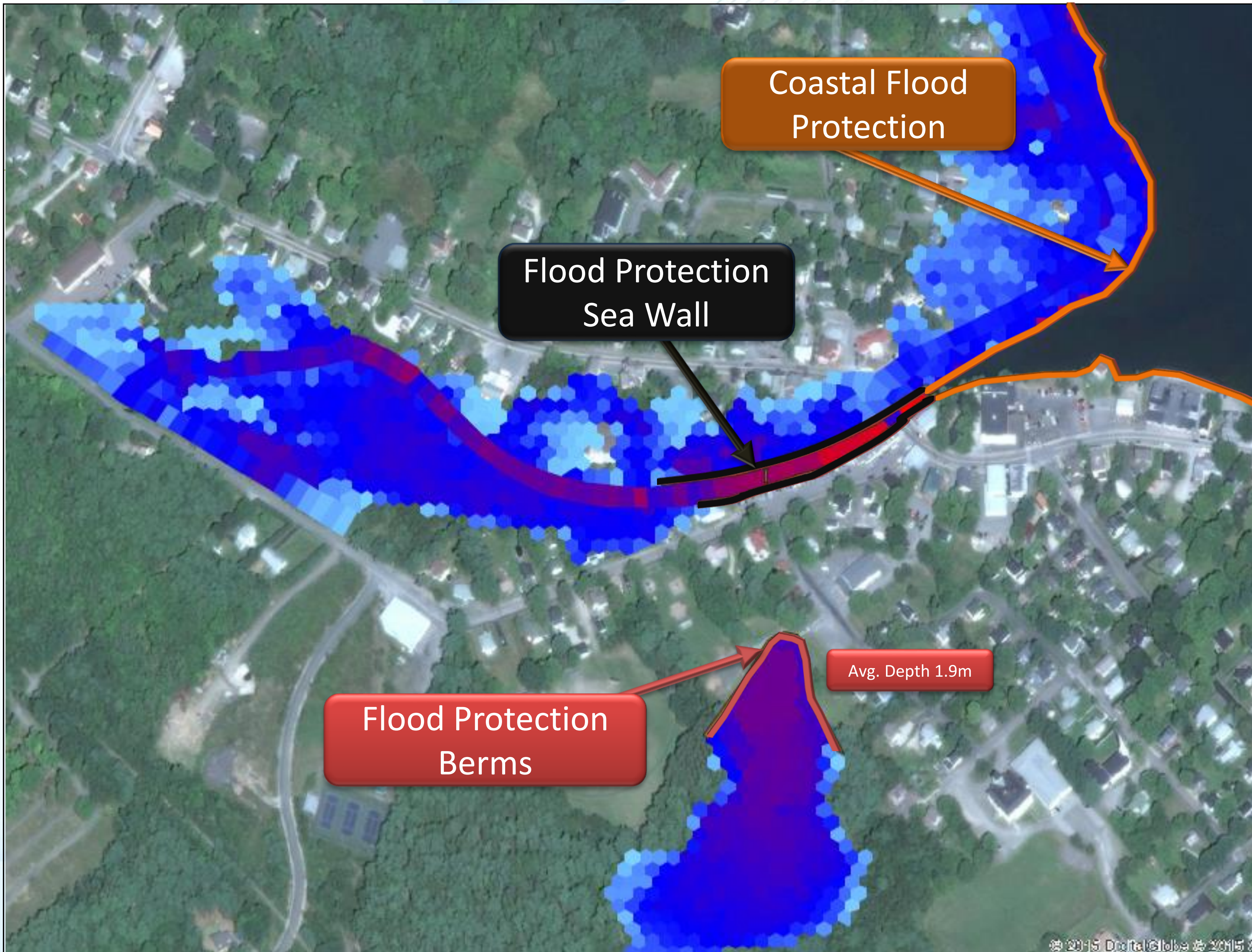
Sketch of Flood Mitigation Options for Scenario-1 (Option8)

Figure 5.12



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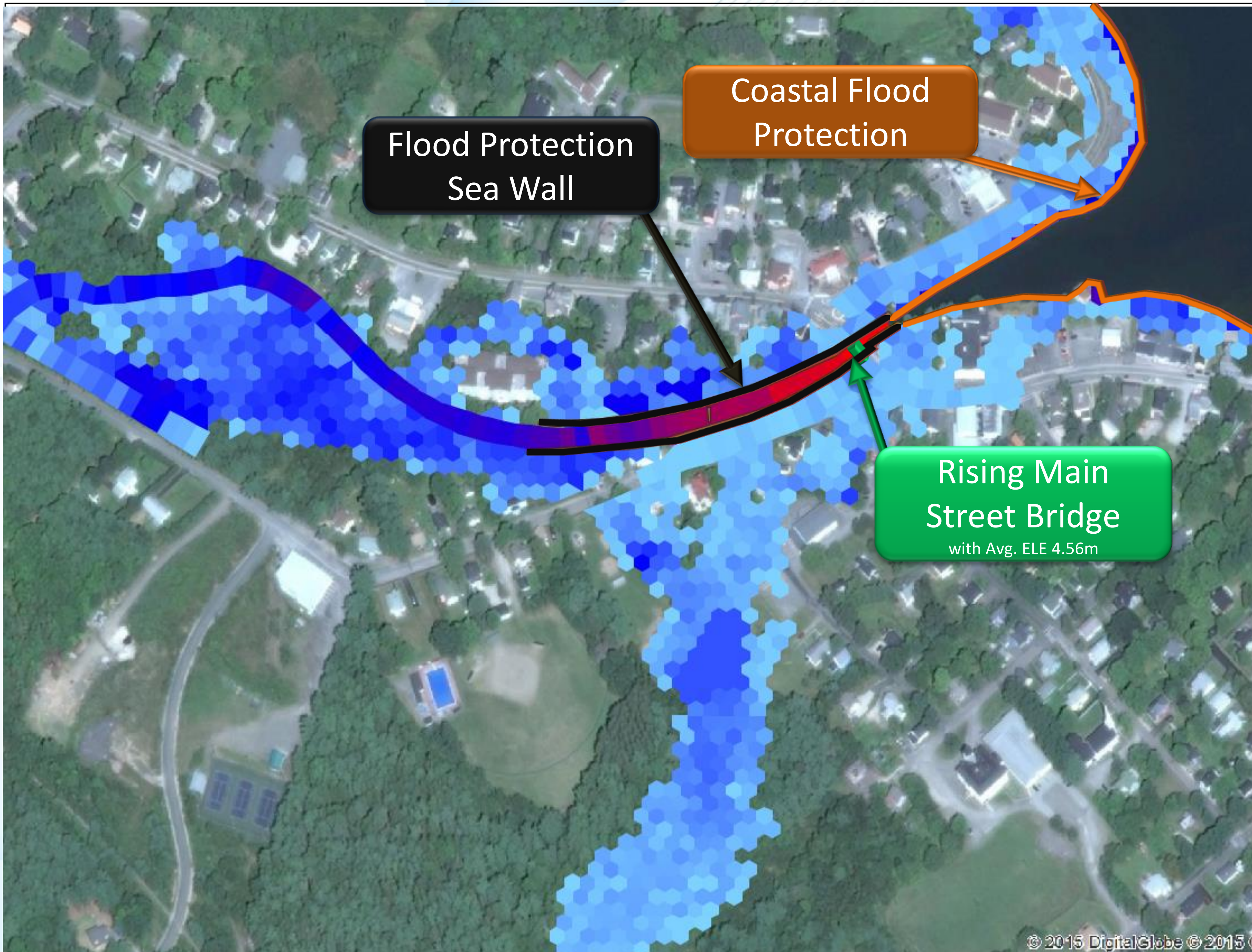
Sketch of Flood Mitigation Options for Scenario-1 (Option9)

Figure 5.13



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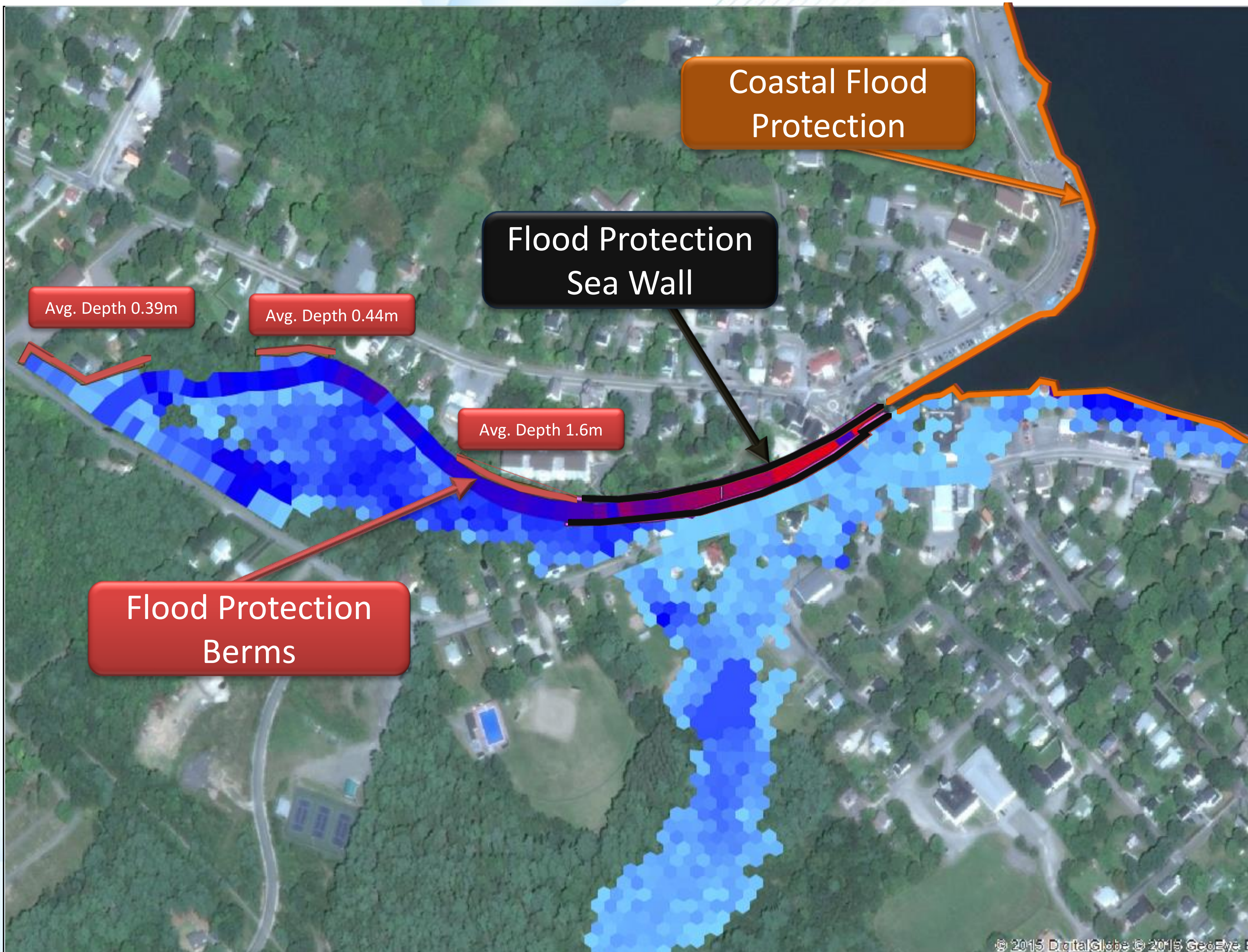
Sketch of Flood Mitigation Options for Scenario-2 (Option1)

Figure 5.14



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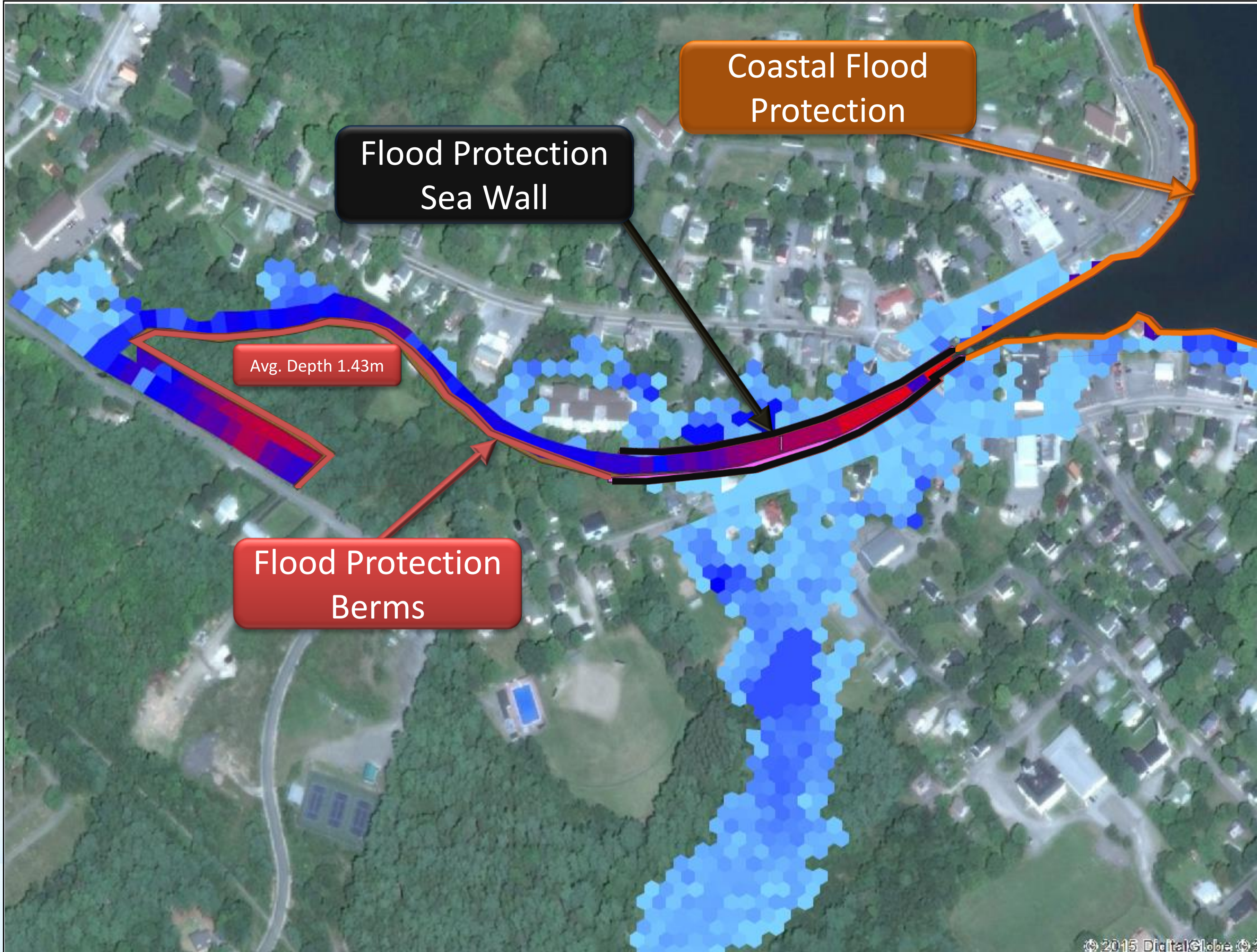
Sketch of Flood Mitigation Options for Scenario-2 (Option2)

Figure 5.15



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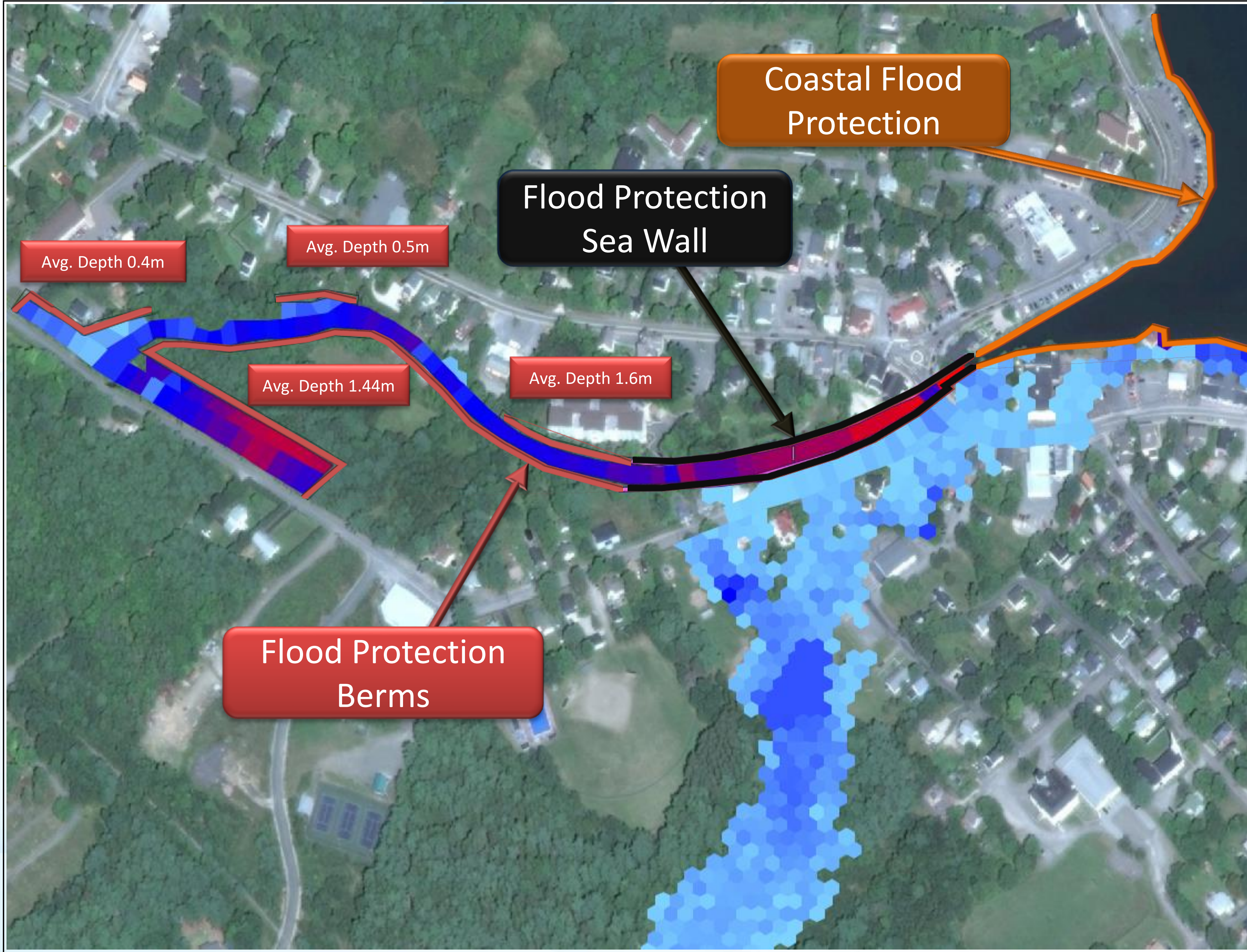
Sketch of Flood Mitigation Options for Scenario-2 (Option3)

Figure 5.16



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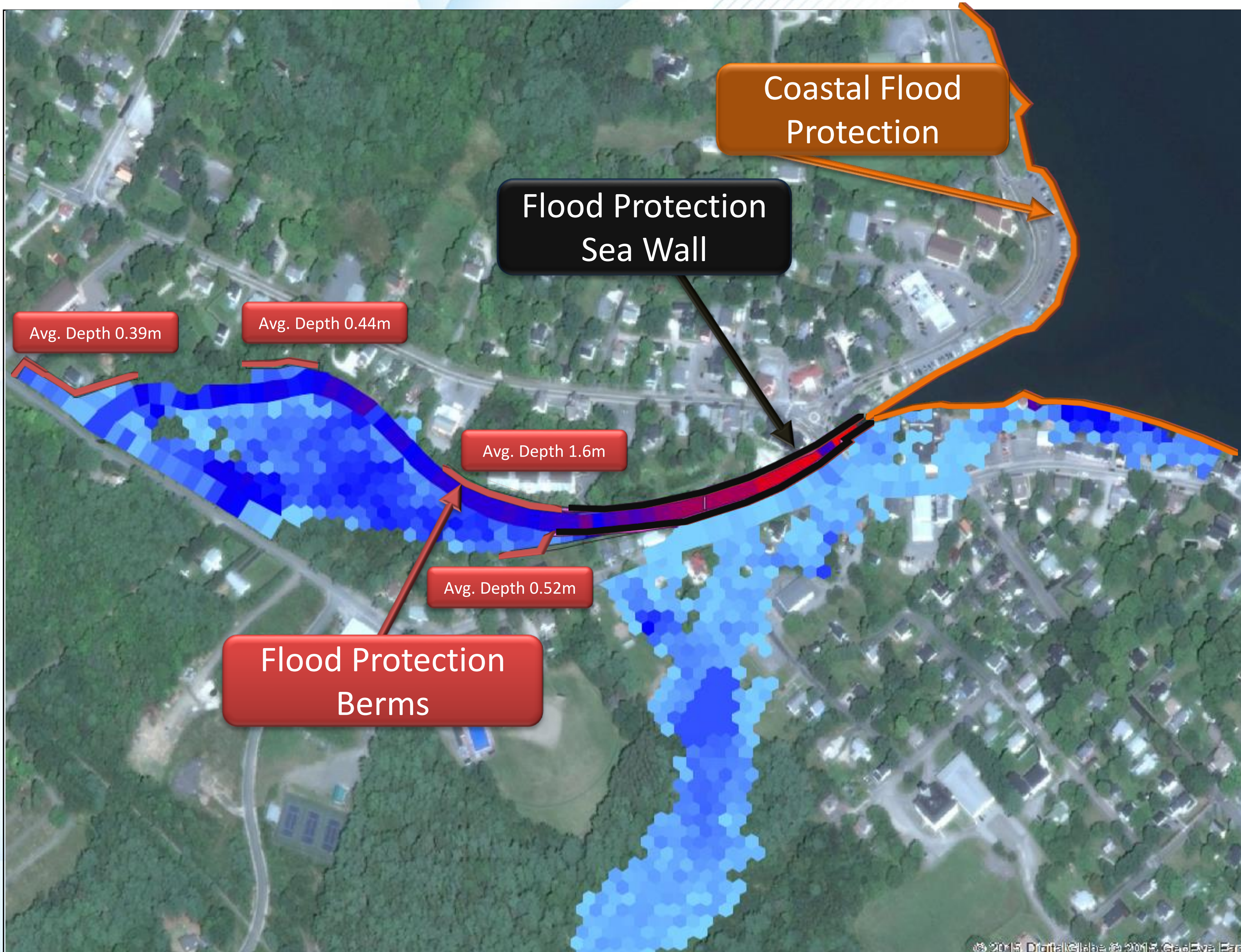
Sketch of Flood Mitigation Options for Scenario-2 (Option4)

Figure 5.17



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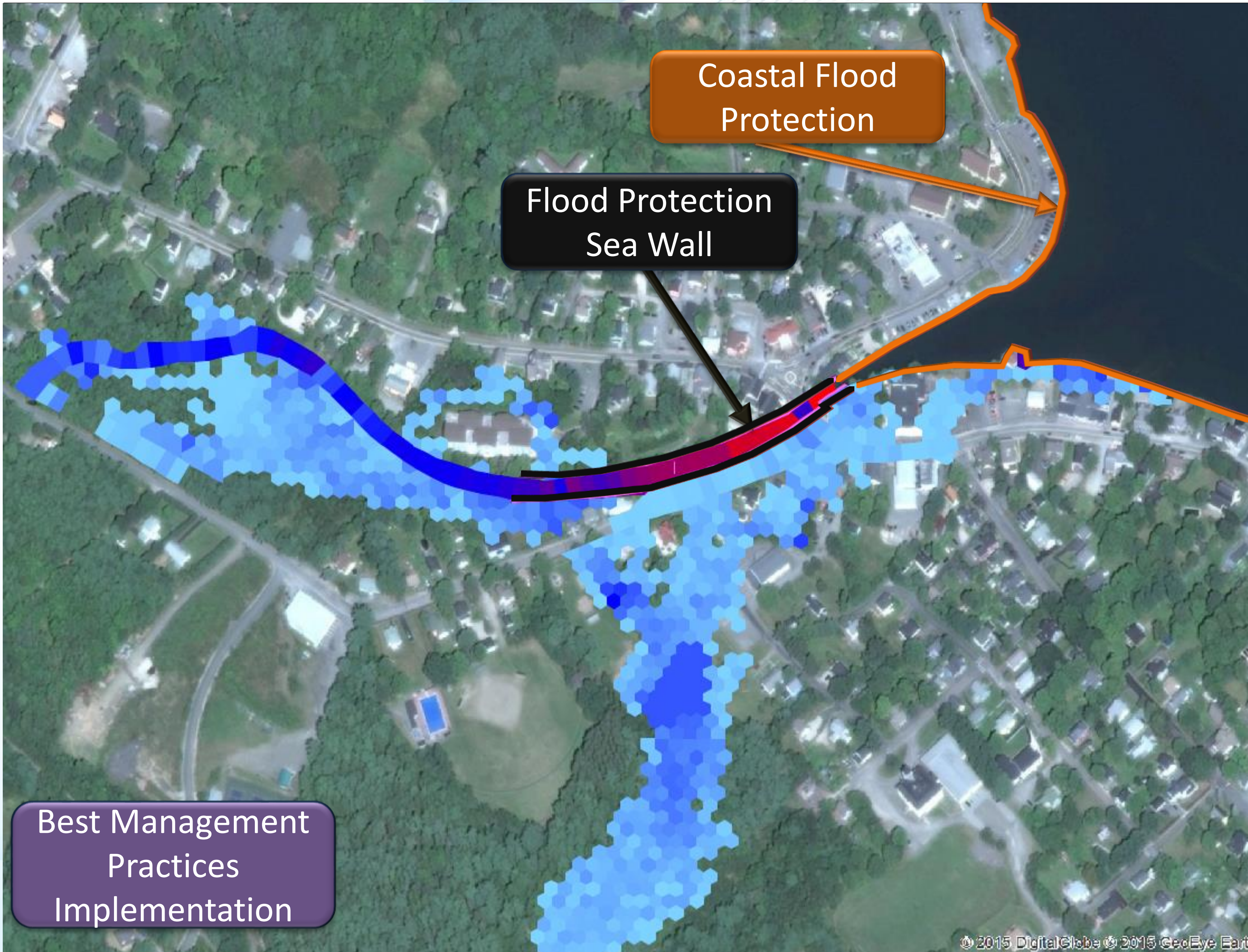
Sketch of Flood Mitigation Options for Scenario-2 (Option5)

Figure 5.18



Water Resources

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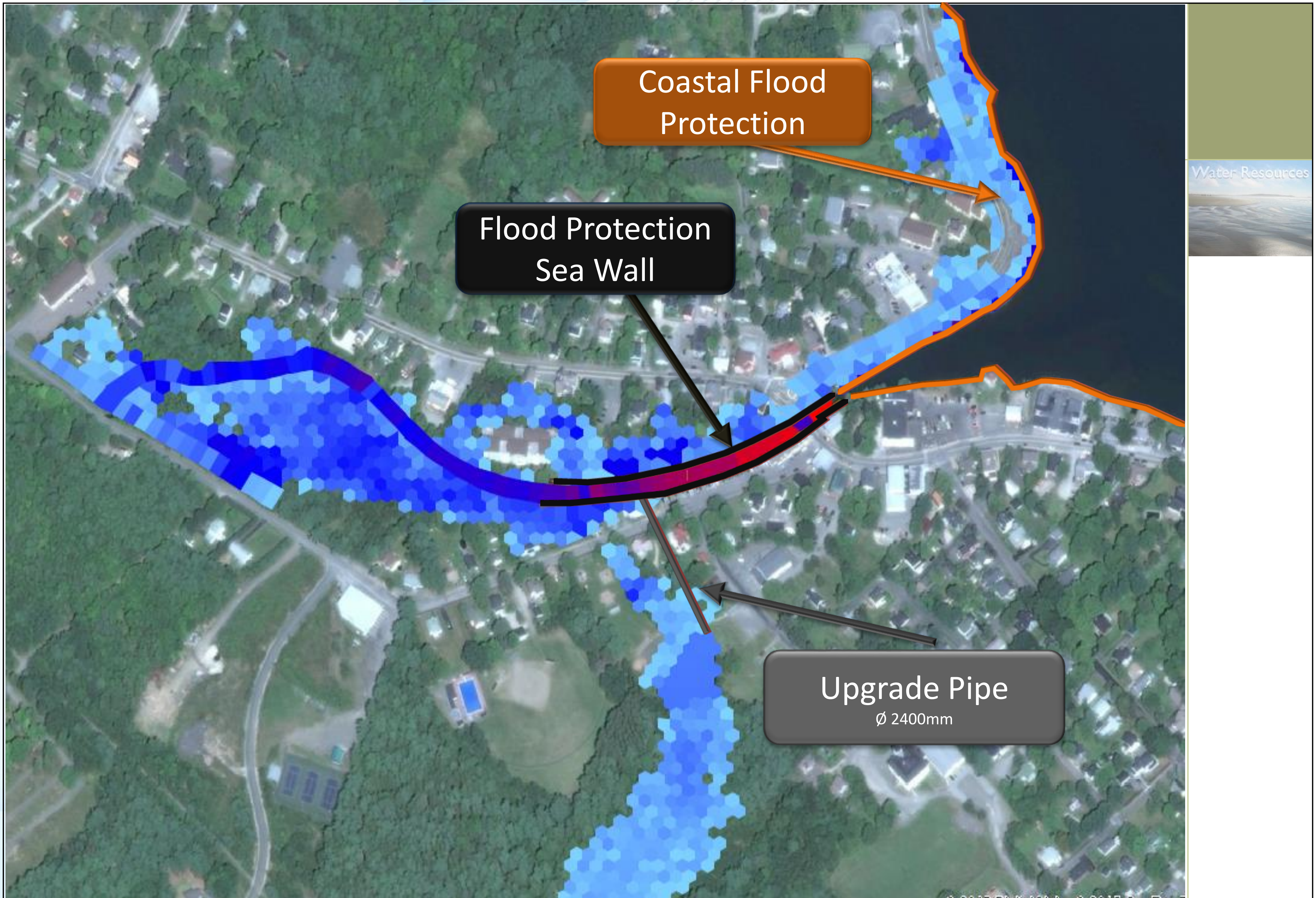
Sketch of Flood Mitigation Options for Scenario-2 (Option6)

Figure 5.19



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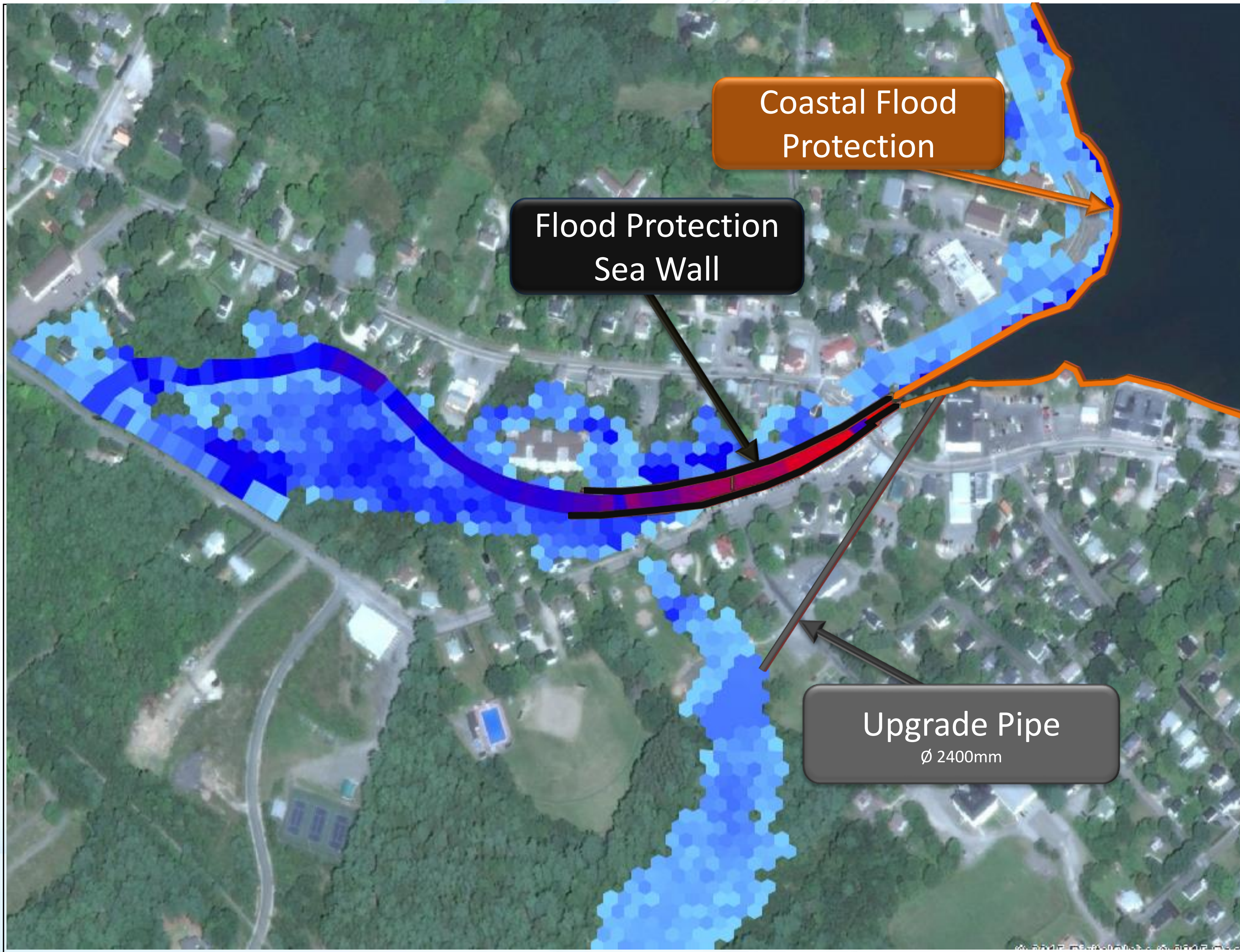
Sketch of Flood Mitigation Options for Scenario-2 (Option7)

Figure 5.20



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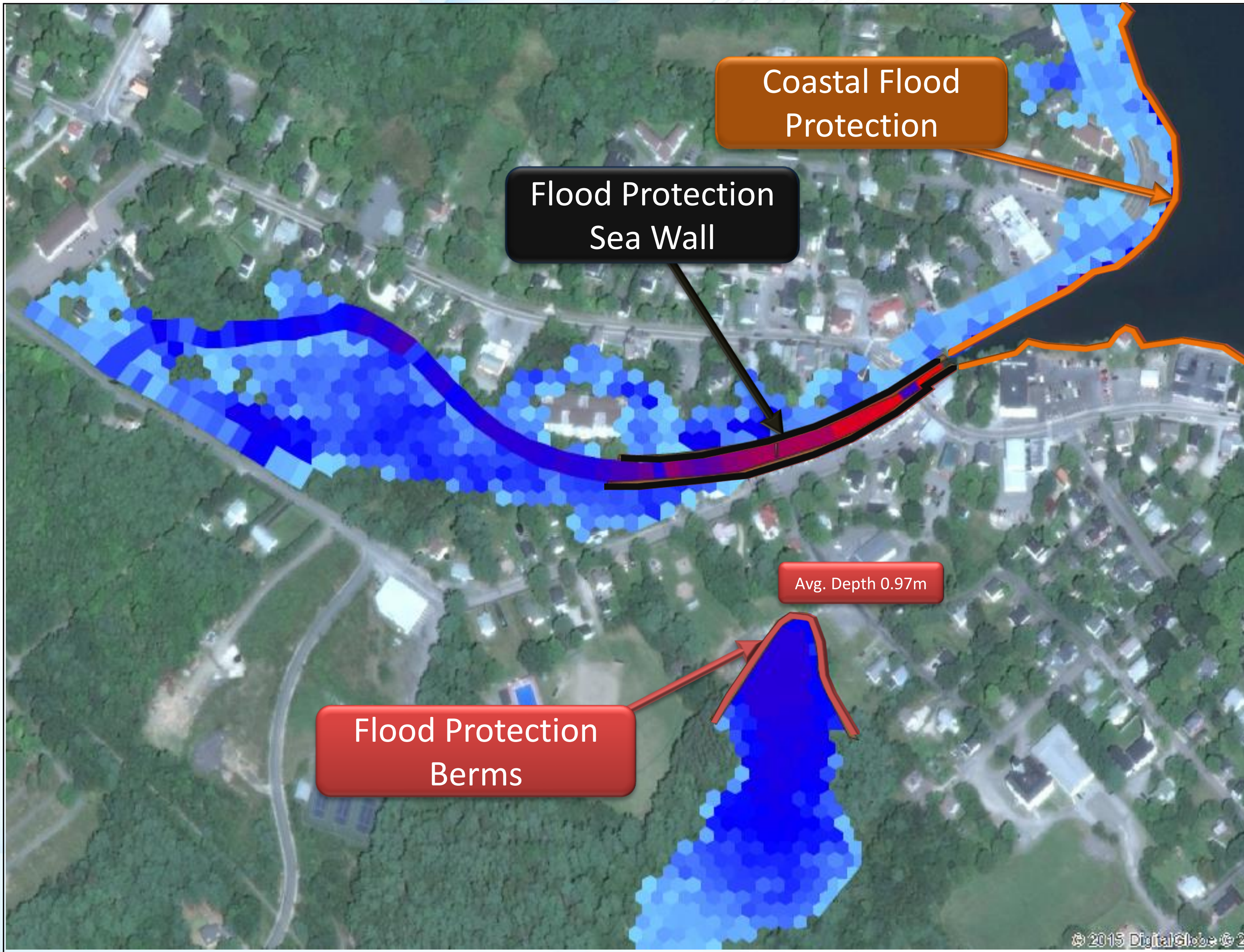
Sketch of Flood Mitigation Options for Scenario-2 (Option8)

Figure 5.21



Water Resources

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Sketch of Flood Mitigation Options for Scenario-2 (Option9)

Figure 5.22

CHAPTER 6 CONCEPTUAL COST ESTIMATES

6.1 Approach

In order to provide a clear and defendable recommendation for the plan of action to put forward, this study also included a cost-benefit analysis to assess the benefits of flood risk management.

The approach to determining the most suitable option is based on factors such as protection of life, infrastructure to support protection of life, life cycle cost, or cost effectiveness. Because the ranking is based on a combination of factors, the most appropriate option, or combination of options, may not be the option that costs the least, nor that reduces the flooded area the most, nor the option that is the most cost-effective. There are many aspects to take into account, which renders this task challenging, and by no means final. Nevertheless, in an effort to be fair to each aspect, various ideas were considered. The main sources of information that have been taken into account include:

- The protection level provided by each option during each type of event - extreme rainfall or tide obtained through the modelling effort;
- Both the initial cost of each option, and more importantly the "life cycle cost" of each option, which is the total cost needed to construct, operate and maintain a system of protection over the expected lifetime of the system, in today's dollar value.
- The value of the land protected. An obvious question is: "does it make sense to spend more money to protect land than the land is worth?”
- Environmental and permitting requirements: some options may have significant negative impacts on the environment. If so, they may have unsurmountable permitting challenges that would render the option unfeasible.

As mentioned above, "life cycle costs" are used as the basis for comparison of the various options. This is the total cost needed to construct, operate and maintain a system of protection over the expected lifetime of the system, in today's dollar value. This is a fairer approach to comparing options, as it would be unfortunate to recommend options that are relatively inexpensive to construct in the first few years, but which required expensive repairs and maintenance through the years, and ended up carrying more of a burden to the taxpayers than other options which may perhaps be more expensive at first, but which would then prove to be less expensive over time.

6.2 Conceptual Cost Estimates for Coastal Protection

The costs were developed based on concepts and cross-section presented in Chapter 3.

6.2.1 North Shoreline

Conceptual cost estimates for the North Shoreline options are presented below. We note that the armour stone seawall option was priced assuming an aesthetically pleasing seawall face made of placed, round armour stone (as opposed to less expensive angular stone or rip rap, which would be less fitting for the local context). The living shorelines option would be more economical.

Table 6.1: Concept-Level Budget Estimates of Probable Construction Cost for North Shoreline

				Armour stone seawall				Living shorelines			
No.	DESCRIPTION	UNIT	UNIT COST	EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL	EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL
	Mobilization & Demobilization	LS	$ 50,000			1	$ 50,000			1	$ 50,000
	Environmental Protection	LS	$ 50,000			1	$ 50,000			1	$ 50,000
	Structural backfill	m^3	$ 30	12	700	8,400	$ 252,000	12	700	8,400	$ 252,000
	Geotextile fabric	m^2	$ 3	8	700	5,600	$ 16,800	8	700	5,600	$ 16,800
	Ashphalt path	m^2	$ 35	3	700	2,100	$ 72,576	3	700	2,100	$ 72,576
	Common Fill	m^3	$ 15			-	$ -	5	700	3,500	$ 52,500
	Sand fill	m^3	$ 25			-	$ -	14	700	9,725	$ 243,125
	Plantings for trail embankement	m^2	$ 20			-	$ -	5	700	3,500	$ 70,000
	Round armour stone seawall face	m^2	$ 350	4.5	700	3,150	$ 1,102,500				
	R1 Armour rock	m^3	$ 100					5	545	2,725	$ 272,500
	Geosynthetic composite	m^2	$ 10			-	$ -	6	545	3,270	$ 32,700
	Culverts	Ea	$ 5,000			5	$ 25,000			5	$ 25,000
							$ -			-	$ -
	Raise and pave Parking lot & Road at Kedys by 0.9 m	m^2	$ 62	45	45	2,025	$ 124,659	10	70	700	$ 43,092
	Pedestrian bridge (35 m span) to bandstand	m	$ 5,000	1	35	35	$ 175,000	1	35	35	$ 175,000
	Raise retaining wall at Ernst brook outlet	m	$ 720	1	150	150	$ 108,000	1	150	150	$ 108,000
	Miscellaneous						$ -			-	$ -
	.1 (Provisionals)	LS	$ 30,000			1	$ 30,000			1	$ 30,000
						-	$ -			-	$ -
TOTAL CONSTRUCTION COST without CONTINGENCY & ENGINEERING DESIGN							**$ 2,007,000**				**$ 1,493,000**
	CONTINGENCIES and ALLOWANCES										
	Design Development Contingency		15%				$ 300,000				$ 225,000
	Construction Contingency		10%				$ 200,000				$ 150,000
	Escalation / Inflation (Assuming 2016 Tender Call)		4%				$ 80,000				$ 60,000
	Location Factor		0%				$ -				$ -
TOTAL CONSTRUCTION with CONTINGENCIES without HST							**$2,587,000**				**$1,928,000**
	ENGINEERING DESIGN & ENVIRONMENTAL PERMITTING										
	Engineering Services During Construction		7%				$ 181,000				$ 135,000
	Environmental Permitting & Regulatory Process						$ 100,000				$ 100,000
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES witho							**$2,868,000**				**$2,163,000**
		HST - NS	15%				$ 430,200				$ 324,500
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES with I							**$3,298,000**				**$2,488,000**
						$/m	$ 4,700				$ 3,600

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6.2.2 Town Waterfront

Seawall Built on Existing Shoreline

The shoreline length along the waterfront is estimated at approximately 1,600 m, accounting for all indentations (which is conservative). The lineal cost for a local seawall (round placed stones on a near-vertical slope) is estimated at approximately $2,400 per m for a 2.5 m average height wall. The conceptual order of magnitude cost for a seawall may be in the $2.2 to 4 million range (planning level estimate) depending on shoreline length. It is cautioned that the actual lineal cost will depend on the local elevation of the shoreline at each individual property. If this option was to be adopted for only a section of shoreline, remaining sections would have to consider flood proofing buildings, or an infilled berm as presented in the next paragraphs.

Table 6.2: Concept-Level Budget Estimates of Probable Construction Cost for Town Seawall Built on Existing Shoreline

No.	DESCRIPTION	UNIT	UNIT COST	EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL
	Mobilization & Demobilization	LS	$ 50,000			1	$ 50,000
	Environmental Protection	LS	$ 50,000			1	$ 50,000
	Common backfill	m^3	$ 16	10	1600	16,000	$ 256,000
	Geotextile fabric	m^2	$ 3			-	Included
	Round armour stone seawall face	m^2	$ 430	2.5	1600	4,000	$ 1,720,000
	Culverts	Ea	$ 5,000			10	$ 50,000
	Raise Retaining wall at Town Side of Ernst Brook	m	$ 720	1	80	80	$ 57,600
	Miscellaneous						$ -
	.1 (Provisionals)	LS	$ 30,000			1	$ 30,000
						-	$ -
TOTAL CONSTRUCTION COST without CONTINGENCY & ENGINEERING DESIGN							**$ 2,214,000**
	CONTINGENCIES and ALLOWANCES						
	Design Development Contingency		15%				$ 330,000
	Construction Contingency		10%				$ 220,000
	Escalation / Inflation (Assuming 2016 Tender Call)		4%				$ 90,000
	Location Factor		0%				$ -
TOTAL CONSTRUCTION with CONTINGENCIES without HST							**$ 2,854,000**
	ENGINEERING DESIGN & ENVIRONMENTAL PERMITTING						
	Engineering and Design		5%				$ 143,000
	Engineering Services During Construction		7%				$ 200,000
	Environmental Permitting & Regulatory Process						$ 100,000
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES without HST							**$ 3,297,000**
		HST - NS	15%				$ 494,600
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES with HST							**$ 3,792,000**
						$/m	$ 2,400

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Floodproof Buildings

Table 6.3 provides an overview of the number of buildings that area. The cost to raise one average house with a crawlspace/basement and framed floor system is estimated at $50,000, which will vary depending on the complexity and age of the building**.** Planning level costs for floodproofing are also included. The total cost to adapt buildings along the Town waterfront may be in the $1 to 3 million range (planning level estimate), depending on the number of houses considered. This would be somewhat more economical than the seawall option ($2.2 to 4 million range) priced in the previous section. From a planning point of view, since the land is divided across multiple private property owners, an individual adaptation approach on a property-by-property basis would be easier to implement.

Table 6.3: Number of Buildings below 4.0 m Elevation and Associated Conceptual Costs to Adapt Them

	Between Water and Road	Landward of Road	Total
Number of houses	27	22	49
Raising *Assuming $60,000 for new foundation and jacking, for a typical house with a crawlspace/basement and framed floor system*	$ 1.6 M	$ 1.3 M	$ 2.9M
Wet flood-proofing (i.e. making provisions for occasional flooding of basement or parking garage) *Assuming $20,000 per house*	$ 540 k	$ 440 k	$ 1 M
Dry floodproofing *Assuming $50,000 per house for new or waterproofed foundation, or floodwalls*	$ 1.3 M	$ 1.1 M	$ 2.4 M

Notes:

- *The 4.0 m elevation is conservative and represents the upper range of wave overtopping allowance for the next 50 years. Buildings between the water and the road are at higher risk for wave overtopping; and*
- *Costs may considerably vary from building to building. Local market trends, non-competitive bidding situations, unforeseen labour and material adjustments, and other factors are beyond the control of CBCL Limited and as such we cannot warrant or guarantee that actual costs will not vary from the opinions provided.*

The above options (seawall and/or floodproofing existing infrastructure) are realistic and practical for the short- to medium-term in the next few decades, when flooding will remain occasional due to temporary storm surges. However, for permanent sea level rise in the long term (50 years and beyond) when existing infrastructure have reached the end of their useful life, the two following approaches will need to be considered:

- Complete rebuilding of the downtown area to a higher elevation, a large effort which is estimated to be well upwards of $10 million in an order-of-magnitude sense; and

- Planned withdrawal and relocation when and where opportunities to do so arise (i.e. property ownership transfers), which may eventually leave more space to raise the road that could then be engineered as a sea defense.

Flood Dyke and Living Shoreline Alternative for Southwest Town Waterfront

The costs shown in the following table refer to Figures 3.10.a and 3.10.b, for the South shoreline section only. This does not include shoreline past 260 m to the east of the bandstand, as the water becomes deeper and boat access is assumed to be required. For the remainder of the shoreline to the southeast, other options would be necessary.

Table 6.4.a: Concept-Level Budget Estimates of Probable Construction Cost for: Flood Dyke and Living Shoreline Alternative for Southwest Town Waterfront

No.	DESCRIPTION	UNIT	UNIT COST	EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL
	Mobilization & Demobilization	LS	$ 50,000			1	$ 50,000
	Environmental Protection	LS	$ 50,000			1	$ 50,000
	Structural backfill for flood berm	m^3	$ 35	13	320	4,160	$ 145,600
	Geotextile fabric	m^2	$ 3	8	320	2,560	$ 7,680
	Common Fill	m^3	$ 15	5.5	260	1,430	$ 21,450
	Sand fill	m^3	$ 25	15	260	3,900	$ 97,500
	Plantings for berm embankement	m^2	$ 20	5	260	1,300	$ 26,000
	R1 Armour rock	m^3	$ 100	5	210	1,050	$ 105,000
	Geosynthetic composite	m^2	$ 10	6	210	1,260	$ 12,600
	Culverts	Ea	$ 5,000			2	$ 10,000
						-	$ -
	Raise retaining wall at Ernst brook outlet	m	$ 720	1	60	60	$ 43,200
	Miscellaneous					-	$ -
	.1 (Provisionals)	LS	$ 30,000			1	$ 30,000
						-	$ -
TOTAL CONSTRUCTION COST without CONTINGENCY & ENGINEERING DESIGN							**$ 599,000**
	CONTINGENCIES and ALLOWANCES						
	[1] Design Development Contingency Allowance		25%				$ 150,000
	[2] Construction Contingency		10%				$ 60,000
	Escalation / Inflation (Assuming 2016 Tender Call)		4%				$ 25,000
	Location Factor		0%				$ -
TOTAL CONSTRUCTION with CONTINGENCIES without HST							**$ 834,000**
	ENGINEERING DESIGN & ENVIRONMENTAL PERMITTING						
	Engineering and Design		5%				$ 42,000
	Engineering Services During Construction		7%				$ 58,000
	Environmental Permitting & Regulatory Process						$ 100,000
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES without HST							**$1,034,000**
		HST - NS	15%				$ 155,100
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES with HST							**$1,189,000**
						Cost per m	$ 4,000

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6.2.3 Lighthouse Route

The average road elevation south of the town waterfront is 2.5 m. The length considered is approximately 800 m (see Figure 2.12). Options include:

- Raising the road to at least 2.8 m (recommended);
- Traditional armour stone revetment approach, with crest at 3.8 m; and
- Living shorelines approach with crest at 3.2 m, using nearshore breakwaters and pocket beaches, which would offer recreational opportunities.

Conceptual costs are presented in the following table.

Table 6.4.b: Concept-Level Budget Estimates of Probable Construction Cost for Lighthouse Route

No.	DESCRIPTION	UNIT	UNIT COST	EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL	EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL
	Mobilization & Demobilization	LS	$ 50,000			1	$ 50,000			1	$ 50,000
	Environmental Protection	LS	$ 50,000			1	$ 50,000			1	$ 50,000
	Structural backfill	m^3	$ 30	3.6	800	2,880	$ 86,400	3.6	800	2,880	$ 86,400
	Geotextile fabric	m^2	$ 3	10	800	8,000	$ 24,000	10	800	8,000	$ 24,000
	Ashphalt Road	m^2	$ 90	10	700	7,000	$ 630,000	10	700	7,000	$ 630,000
	Common Fill	m^3	$ 15			-	$ -	8	800	6,400	$ 96,000
	Sand fill	m^3	$ 25			-	$ -	19	800	15,000	$ 375,000
	Plantings for Road Embankment	m^2	$ 20			-	$ -	5	800	4,000	$ 80,000
	R1 Armour Rock	m^3	$ 100	6	800	4,800	$ 480,000	6	600	3,600	$ 360,000
	Geosynthetic composite	m^2	$ 10			-	$ -	6	600	3,600	$ 36,000
	Culverts	Ea	$ 5,000			4	$ 20,000			4	$ 20,000
							$ -			-	$ -
	Miscellaneous						$ -			-	$ -
	.1 (Provisionals)	LS	$ 30,000			1	$ 30,000			1	$ 30,000
						-	$ -			-	$ -
TOTAL CONSTRUCTION COST without CONTINGENCY & ENGINEERING DESIGN							**$ 1,370,000**				**$ 1,837,000**
	CONTINGENCIES and ALLOWANCES										
	Design Development Contingency		10%				$ 135,000				$ 185,000
	Construction Contingency		10%				$ 135,000				$ 185,000
	Escalation / Inflation (Assuming 2016 Tender Call)		4%				$ 55,000				$ 75,000
	Location Factor		0%				$ -				$ -
TOTAL CONSTRUCTION with CONTINGENCIES without HST							**$ 1,695,000**				**$ 2,282,000**
	ENGINEERING DESIGN & ENVIRONMENTAL PERMITTING										
	Engineering and Design		5%				$ 85,000				$ 114,000
	Engineering Services During Construction		7%				$ 119,000				$ 160,000
	Environmental Permitting & Regulatory Process						$ 100,000				$ 100,000
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES without HS							**$ 1,999,000**				**$ 2,656,000**
		HST - NS	15%				$ 299,900				$ 398,400
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES with HST							**$ 2,299,000**				**$ 3,054,000**
						$/m	$ 2,900			$/m	$ 3,800

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6.3 Conceptual Cost Estimates for Stormwater and Riverine Flood Mitigation

The flood mitigation options for stormwater and riverine flood mitigation discussed in Chapter 5 were evaluated by the models. Recommended flood mitigation options were identified based on a cost benefit analysis of each flood mitigation approach. The various flood mitigation approaches are presented in **Figures 5.4 to 5.22,** to protect against the Scenario 1 and 2 design events (extreme rainfall or extreme tidal event). All these options included constructing sea walls at the outlet of Ernst Brook.

The opinions of probable costs for the various approaches are presented in **Table 6.5.** As shown in this table, upgrading the existing Main Street Bridge was found to have a negligible impact on flooding. The bridge upgrade would slightly reduce water levels upstream of the structure, but would have no significant improvement to the flood risk within the Town. The cost estimate breakdown for stormwater flooding protection approaches can be found in **Appendix A**. From **Figure 5.9** and **Figure 5.13** it would seem that the most efficient options would include constructing shallow berms on the eastern bank of the brook, along with a berm on the downstream side of the pond. It was found that upgrading the Pond's outlet pipe would result in a very minor reduction of the flood levels during a 1 in 100 year storm event. The recommended cost effective approach to address the flood risk from the pond is to construct an engineered berm to contain stormwater and release it at a controlled rate to prevent downstream flooding.

It is noted that those events are for the year 2115 scenario, which indicates that the lifetime of the stormwater protection structures is 100 years. Designs for structures that would last 50 years would then be made by scaling back the designs presented, and lowering the height to 50% of the values shown.

Table 6.6 shows the recommended approach to protection within the Town, with costing information. The recommended approach includes a combination of Options 5 and 9, which combine the berm and flow control at the pond, with the shallow berms on the eastern and western banks of the brook. It is recommended to construct structures that will protect the Town against existing flooding risks, as opposed to future (2115) flooding risks. The berms should be designed to withstand risks that will exist up to the year 2065. At this time (2065), the structures can be enhanced, raised and lengthened to withstand risks beyond this timeline. The importance of providing designs that can work up to the year 2115 is that the initial structures have the ability to be expanded to provide the protection that will be needed in the future. There will be no need to abandon structures and design completely new structures at other locations, and therefore losing some cost-efficiencies.

Table 6.5(a): Cost Comparion of Various Approaches to Flood Mitigation

Scenarios	Options	Total Capital Cost	Number of Houses Flooded	Other Significant Structures
Scenario-1 1 in 100 Year Rainfall +1 in 2 Year Sea Level with Climate Change (2115)	Future Conditions (2115) Without Flood Mitigation Options	$0	60	Three Churches, Irving Gas Station, Museum, Condominium, Canada Post-Postal Outlet, Calvary Temple, Atlantic Save Easy, and 985m Road
	Option1 (raising Main Street bridge)	$3,010,000	49	Three Churches, Irving Gas Station, Condominium, Canada Post-Postal Outlet, Calvary Temple, Atlantic Save Easy, and 814m Road
	Option2 (berms along the eastern bank)	$1,190,000	21	Canada Post-Postal Outlet, Atlantic Save Easy, and 323m Road
	Option3 (berms along the western bank of the brook)	$1,530,000	44	Three Churches, Irving Gas Station, Condominium, Canada Post-Postal Outlet, Calvary Temple, Atlantic Save Easy, and 804m Road
	Option4(berms along the eastern and western banks of the brook)	$1,930,000	17	Canada Post-Postal Outlet, Atlantic Save Easy, and 283m Road
	Option5 (berms along the eastern bank and the downstream side of the western bank)	$1,300,000	18	Canada Post-Postal Outlet, Atlantic Save Easy, and 283m Road
	Option6(Best Management Practice)	$178,470,000	37	Three Churches, Irving Gas Station, Condominium, Canada Post-Postal Outlet, Calvary Temple, Atlantic Save Easy, and 804m Road
	Option7 (upgrading Pond's outlet pipe to drain into the brook)	$1,820,000	45	Three Churches, Irving Gas Station, Condominium, Canada Post-Postal Outlet, Calvary Temple, Atlantic Save Easy, and 814m Road
	Option8 (upgrading Pond's outlet pipe and to drain into the sea)	$2,640,000	45	Three Churches, Irving Gas Station, Condominium, Canada Post-Postal Outlet, Calvary Temple, Atlantic Save Easy, and 814m Road
	Option9 (berms along the Pond)	$990,000	34	Three Churches, Irving Gas Station, Condominium, Calvary Temple, and 538m Road

Table 6.5(b): Cost Comparion of Various Approaches to Flood Mitigation

Scenarios	Options	Total Capital Cost	Number of Houses Flooded	Other Significant Structures
Scenario-2 1 in 2 Year Rainfall +1 in 100 Year Sea Level with Climate Change(2115)	Future Conditions (2115) Without Flood Mitigation Options	$0	53	Three Churches, Irving Gas Station, Museum, Condominium, Canada Post-Postal Outlet, Calvary Temple, Atlantic Save Easy, and 994m Road
	Option1 (raising Main Street bridge)	$3,520,000	24	Condominium, Canada Post-Postal Outlet, Atlantic Save Easy, Calvary Temple, and 581m Road
	Option2 (berms along the eastern bank)	$1,470,000	17	Canada Post-Postal Outlet, Atlantic Save Easy, and 283m Road
	Option3 (berms along the western bank of the brook)	$1,700,000	19	Condominium, Canada Post-Postal Outlet, Calvary Temple, Atlantic Save Easy, and 411m Road
	Option4(berms along the eastern and western banks of the brook)	$1,840,000	11	Canada Post-Postal Outlet, Atlantic Save Easy, and 283m Road
	Option5 (berms along the eastern bank and the downstream side of the western bank)	$1,490,000	13	Canada Post-Postal Outlet, Atlantic Save Easy, and 283m Road
	Option6(Best Management Practice)	$55,255,000	11	Condominium, Canada Post-Postal Outlet, Atlantic Save Easy, and 283m Road
	Option7 (upgrading Pond’s outlet pipe to drain into the brook)	$2,270,000	15	Condominium, Calvary Temple, and 306m Road
	Option8 (upgrading Pond’s outlet pipe and to drain into the sea)	$3,080,000	15	Condominium, Calvary Temple, and 306m Road
	Option9 (berms along the Pond)	$1,470,000	15	Condominium, Calvary Temple, and 306m Road

Table 6.6: Cost Estimate for Recommended Approach to Flood Mitigation

Scenarios	Description	Design Life	Capital Cost
Scenario-1 1 in 100 Year Rainfall + 1 in 2 Year Sea Level with Climate Change (Combination of Options 5 and 9)	• Building berms along the eastern bank (av. **Height** of 0.55m); • Building berms at the downstream side of the western bank (av. **Height** of 0.8m); • Constructing berms along the Pond (av. **Height** of 0.95m); • Constructing sea wall, 410m in length in tidal area; and	50 (2065)	$1,330,000
	• Building berms along the eastern bank (av. **Height** of 1.1m); • Building berms on the downstream side of the western bank (av. **Height** of 1.6m); • Constructing berms along the Pond (av. **Height** of 1.9m); and • Constructing sea wall, 410m in length in tidal area.	100 (2115)	$1,430,000
Scenario-2 1 in 2 Year Rainfall + 1 in 100 Year Sea Level with Climate Change (Combination of Options 5 and 9)	• Building berms along the eastern bank (av. **Height** of 0.4m); • Building berms at the downstream side of the western bank (av. **Height** of 0.3m); • Constructing berms along the Pond (av. **Height** of 0.95m); and • Constructing sea wall, 508m in length in tidal area.	50 (2065)	$1,580,000
	• Building berms along the eastern bank (av. **Height** of 0.8m); • Building berms on the downstream side of the western bank (av. **Height** of 0.5m); • Constructing berms along the Pond (av. **Height** of 1.9m); and • Constructing sea wall, 508m in length in tidal area.	100 (2115)	$1,620,000

CHAPTER 7 CONCLUSIONS & RECOMMENDATIONS

This study has placed significant emphasis on the following aspects for the Town of Mahone Bay:

- Prioritization of vulnerable areas;
- In-depth modelling;
- Assessment of many different options to identify the most cost effective and achievable solutions; and
- Conducting the overall assessment in a holistic approach to make sure that recommendations make sense for the Town of Mahone Bay and are sustainable in the long term.

Conceptual shoreline protection and enhancement options were developed based on a detailed study of local shoreline, wave and water level conditions. The objectives are to balance the following requirements:

- **Flood and erosion mitigation** – The design is based on a desired cumulative probability of coastal flooding of 50% over the next 50 years, based on worst-case sea level rise estimates from the IPCC adapted to the site;
- **Preservation and, wherever possible, enhancement of natural shoreline habitat** - The intention goes beyond simply raising the existing waterfront. Notably, we incorporated living shorelines design approaches which would be very fitting to the Mahone Harbour context;
- **Public access to the shoreline**; and
- **Aesthetics**.

Based on the relatively protected nature of Mahone Harbour, a ‘living shoreline’ concept was found to be a viable alternative to traditional armour rock for the northern section. This concept makes use of partial infilling into the harbour to create salt marsh habitat fronted by rock sills, to protect a new waterfront trail along Edgewater Street. Our opinion of probable construction costs for this option is $2.5million.

For the South shoreline along the Town Waterfront, options for the next 50 years include a mix of the following alternatives:

- Incrementally upgrading existing seawalls would require buy-in from all property holders for effective flood defense. Our opinion of probable cost for this option is $2,400 per m of shoreline (average). Overall cost would depend on the total shoreline length;

- For incremental flood-proofing and/or raising buildings, opinion of probable cost is in the $1 million to $3 million range, depending on the number of buildings considered.
- Alternatively, a flood dyke and living shoreline could be considered for the first 320 m-long section between Ernst Brook Outlet and the first boat dock (deeper water and the requirement for boat access are impracticalities elsewhere). Opinion of probable construction costs for this option is $1.2 million.

We note that the options must not only consider coastal flooding, but also river flooding. The inland flooding in Mahone Bay is caused by the complex interaction between rainfall, river flows, waves, tides and storm surge. The most efficient flood mitigation option for the Ernst brook watershed would include the following:

- Building shallow berms along the eastern bank;
- Building shallow berms at the downstream side of the western bank;
- Constructing berms along the downstream side of the Pond; and
- Constructing sea wall, 410m in length in tidal area.

Similarly to the coastal protection system, it can be constructed incrementally, with a final opinion probable construction cost of $1.4 million.

Should you have any questions, please do not hesitate to contact the undersigned.

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APPENDIX A

Cost Estimates

CONCEPT-LEVEL BUDGET ESTIMATE OF PROBABLE CONSTRUCTION COST

Mahone Bay North Shoreline (Options for Consideration in Detailed Design)

Date: 15 Sep 2015

CBCL # 151016.00

Prepared by: VL / AT

No.	DESCRIPTION	UNIT	UNIT COST	Armour Stone Seawall EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL	Living Shorelines EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL
	Mobilization & Demobilization	LS	$ 50,000			1	$ 50,000			1	$ 50,000
	Environmental Protection	LS	$ 50,000			1	$ 50,000			1	$ 50,000
	Structural backfill	m^3	$ 30	12	700	8,400	$ 252,000	12	700	8,400	$ 252,000
	Geotextile fabric	m^2	$ 3	8	700	5,600	$ 16,800	8	700	5,600	$ 16,800
	Ashphalt path	m^2	$ 35	3	700	2,100	$ 72,576	3	700	2,100	$ 72,576
	Common Fill	m^3	$ 15			-	$ -	5	700	3,500	$ 52,500
	Sand fill	m^3	$ 25			-	$ -	14	700	9,725	$ 243,125
	Plantings for trail embankement	m^2	$ 20			-	$ -	5	700	3,500	$ 70,000
	Round armour stone seawall face	m^2	$ 350	4.5	700	3,150	$ 1,102,500				
	R1 Armour rock	m^3	$ 100					5	545	2,725	$ 272,500
	Geosynthetic composite	m^2	$ 10			-	$ -	6	545	3,270	$ 32,700
	Culverts	Ea	$ 5,000			5	$ 25,000			5	$ 25,000
							$ -			-	$ -
	Raise and pave Parking lot & Road at Kedys by 0.9 m	m^2	$ 62	45	45	2,025	$ 124,659	10	70	700	$ 43,092
	Pedestrian bridge (35 m span) to bandstand	m	$ 5,000	1	35	35	$ 175,000	1	35	35	$ 175,000
	Raise retaining wall at Ernst brook outlet	m	$ 720	1	150	150	$ 108,000	1	150	150	$ 108,000
	Miscellaneous						$ -			-	$ -
	.1 (Provisionals)	LS	$ 30,000			1	$ 30,000			1	$ 30,000
						-	$ -			-	$ -
TOTAL CONSTRUCTION COST without CONTINGENCY & ENGINEERING DESIGN							**$ 2,007,000**				**$ 1,493,000**
	CONTINGENCIES and ALLOWANCES										
	Design Development Contingency		15%				$ 300,000				$ 225,000
	Construction Contingency		10%				$ 200,000				$ 150,000
	Escalation / Inflation (Assuming 2016 Tender Call)		4%				$ 80,000				$ 60,000
	Location Factor		0%				$ -				$ -
TOTAL CONSTRUCTION with CONTINGENCIES without HST							**$ 2,587,000**				**$ 1,928,000**
	ENGINEERING DESIGN & ENVIRONMENTAL PERMITTING										
	Engineering Services During Construction		7%				$ 181,000				$ 135,000
	Environmental Permitting & Regulatory Process						$ 100,000				$ 100,000
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES without HS							**$ 2,868,000**				**$ 2,163,000**
		HST - NS	15%				$ 430,200				$ 324,500
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES with HST							**$ 3,298,000**				**$ 2,488,000**
						$/m	$ 4,700				$ 3,600

CBCL LIMITED
Consulting Engineers

CONCEPT-LEVEL BUDGET ESTIMATE OF PROBABLE CONSTRUCTION COST

Town Waterfront Seawall (Conceptual Design)
Date: 15 Sep 2015
CBCL # 151016.00
Prepared by: VL / AT

Town Seawall

No.	DESCRIPTION	UNIT	UNIT COST	EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL
	Mobilization & Demobilization	LS	$ 50,000			1	$ 50,000
	Environmental Protection	LS	$ 50,000			1	$ 50,000
	Common backfill	m^3	$ 16	10	1600	16,000	$ 256,000
	Geotextile fabric	m^2	$ 3			-	Included
	Round armour stone seawall face	m^2	$ 430	2.5	1600	4,000	$ 1,720,000
	Culverts	Ea	$ 5,000			10	$ 50,000
	Raise Retaining wall at Town Side of Ernst Brook	m	$ 720	1	80	80	$ 57,600
	Miscellaneous						$ -
	.1 (Provisionals)	LS	$ 30,000			1	$ 30,000
						-	$ -
TOTAL CONSTRUCTION COST without CONTINGENCY & ENGINEERING DESIGN							**$ 2,214,000**
	CONTINGENCIES and ALLOWANCES						
	Design Development Contingency		15%				$ 330,000
	Construction Contingency		10%				$ 220,000
	Escalation / Inflation (Assuming 2016 Tender Call)		4%				$ 90,000
	Location Factor		0%				$ -
TOTAL CONSTRUCTION with CONTINGENCIES without HST							**$ 2,854,000**
	ENGINEERING DESIGN & ENVIRONMENTAL PERMITTING						
	Engineering and Design		5%				$ 143,000
	Engineering Services During Construction		7%				$ 200,000
	Environmental Permitting & Regulatory Process						$ 100,000
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES without HST							**$ 3,297,000**
		HST - NS	15%				$ 494,600
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES with HST							**$ 3,792,000**
						$/m	$ 2,400

THIS OPINION OF PROBABLE COSTS IS PRESENTED ON THE BASIS OF EXPERIENCE, QUALIFICATIONS AND BEST JUDGEMENT. IT HAS BEEN PREPARED IN ACCORDANCE WITH ACCEPTABLE PRINCIPLES AND PRACTICES. MARKET TRENDS, NON-COMPETITIVE BIDDING SITUATIONS, UNFORSEEN LABOUR AND MATERIAL ADJUSTMENTS AND THE LIKE ARE BEYOND THE CONTROL OF CBCL LIMITED AND AS SUCH WE CANNOT WARRANT OR GUARANTEE THAT ACTUAL COSTS WILL NOT VARY FROM THE OPINION PROVIDED.

Note 1 A Design Development Contingency is intended to allow for growth scope; quantities; and material costs as the work is better defined in the future.

Note 2 A Construction Contingency is intended for the potential cost of additional work over and above the original contract price.

CBCL LIMITED
Consulting Engineers

CONCEPT-LEVEL BUDGET ESTIMATE OF PROBABLE CONSTRUCTION COST

Mahone Bay Southwest Town Waterfront Shoreline (Conceptual Design)
Date: 14 Dec 2015
CBCL # 151016.00
Prepared by: VL

CBCL No:	151016.00
PREPARED BY:	VL/AT
EST. DESCRIPTION:	Class D

No.	DESCRIPTION	UNIT	UNIT COST	EST. QTY. PER M	LENGTH M	TOTAL EST. QTY.	TOTAL
	Mobilization & Demobilization	LS	$ 50,000			1	$ 50,000
	Environmental Protection	LS	$ 50,000			1	$ 50,000
	Structural backfill for flood berm	m^3	$ 35	13	320	4,160	$ 145,600
	Geotextile fabric	m^2	$ 3	8	320	2,560	$ 7,680
	Common Fill	m^3	$ 15	5.5	260	1,430	$ 21,450
	Sand fill	m^3	$ 25	15	260	3,900	$ 97,500
	Plantings for berm embankement	m^2	$ 20	5	260	1,300	$ 26,000
	R1 Armour rock	m^3	$ 100	5	210	1,050	$ 105,000
	Geosynthetic composite	m^2	$ 10	6	210	1,260	$ 12,600
	Culverts	Ea	$ 5,000			2	$ 10,000
						-	$ -
	Raise retaining wall at Ernst brook outlet	m	$ 720	1	60	60	$ 43,200
	Miscellaneous					-	$ -
	.1 (Provisionals)	LS	$ 30,000			1	$ 30,000
						-	$ -
TOTAL CONSTRUCTION COST without CONTINGENCY & ENGINEERING DESIGN							**$ 599,000**
	CONTINGENCIES and ALLOWANCES						
	[1] Design Development Contingency Allowance		25%				$ 150,000
	[2] Construction Contingency		10%				$ 60,000
	Escalation / Inflation (Assuming 2016 Tender Call)		4%				$ 25,000
	Location Factor		0%				$ -
TOTAL CONSTRUCTION with CONTINGENCIES without HST							**$ 834,000**
	ENGINEERING DESIGN & ENVIRONMENTAL PERMITTING						
	Engineering and Design		5%				$ 42,000
	Engineering Services During Construction		7%				$ 58,000
	Environmental Permitting & Regulatory Process						$ 100,000
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES without HST							**$ 1,034,000**
		HST - NS	15%				$ 155,100
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES with HST							**$ 1,189,000**
						Cost per m	$ 4,000

THIS OPINION OF PROBABLE COSTS IS PRESENTED ON THE BASIS OF EXPERIENCE, QUALIFICATIONS AND BEST JUDGEMENT. IT HAS BEEN PREPARED IN ACCORDANCE WITH ACCEPTABLE PRINCIPLES AND PRACTICES. MARKET TRENDS, NON-COMPETITIVE BIDDING SITUATIONS, UNFORSEEN LABOUR AND MATERIAL ADJUSTMENTS AND THE LIKE ARE BEYOND THE CONTROL OF CBCL LIMITED AND AS SUCH WE CANNOT WARRANT OR GUARANTEE THAT ACTUAL COSTS WILL NOT VARY FROM THE OPINION PROVIDED.

Note 1 A Design Development Contingency is intended to allow for growth scope; quantities; and material costs as the work is better defined in the future.

Note 2 A Construction Contingency is intended for the potential cost of additional work over and above the original contract price.

CONCEPT-LEVEL BUDGET ESTIMATE OF PROBABLE CONSTRUCTION COST

Lightouse Route (conceptual design)
Date: 15 Sep 2015
CBCL # 151016.00
Prepared by: VL / AT

				Armour Stone Revetment				Living Shorelines			
No.	**DESCRIPTION**	**UNIT**	**UNIT COST**	**EST. QTY. PER M**	**LENGTH M**	**TOTAL EST. QTY.**	**TOTAL**	**EST. QTY. PER M**	**LENGTH M**	**TOTAL EST. QTY.**	**TOTAL**
	Mobilization & Demobilization	LS	$ 50,000			1	$ 50,000			1	$ 50,000
	Environmental Protection	LS	$ 50,000			1	$ 50,000			1	$ 50,000
	Structural backfill	m^3	$ 30	3.6	800	2,880	$ 86,400	3.6	800	2,880	$ 86,400
	Geotextile fabric	m^2	$ 3	10	800	8,000	$ 24,000	10	800	8,000	$ 24,000
	Ashphalt Road	m^2	$ 90	10	700	7,000	$ 630,000	10	700	7,000	$ 630,000
	Common Fill	m^3	$ 15			-	$ -	8	800	6,400	$ 96,000
	Sand fill	m^3	$ 25			-	$ -	19	800	15,000	$ 375,000
	Plantings for Road Embankment	m^2	$ 20			-	$ -	5	800	4,000	$ 80,000
	R1 Armour Rock	m^3	$ 100	6	800	4,800	$ 480,000	6	600	3,600	$ 360,000
	Geosynthetic composite	m^2	$ 10			-	$ -	6	600	3,600	$ 36,000
	Culverts	Ea	$ 5,000			4	$ 20,000			4	$ 20,000
							$ -			-	$ -
	Miscellaneous						$ -			-	$ -
	.1 (Provisionals)	LS	$ 30,000			1	$ 30,000			1	$ 30,000
						-	$ -			-	$ -
TOTAL CONSTRUCTION COST without CONTINGENCY & ENGINEERING DESIGN							**$ 1,370,000**				**$ 1,837,000**
	CONTINGENCIES and ALLOWANCES										
	Design Development Contingency		10%				$ 135,000				$ 185,000
	Construction Contingency		10%				$ 135,000				$ 185,000
	Escalation / Inflation (Assuming 2016 Tender Call)		4%				$ 55,000				$ 75,000
	Location Factor		0%				$ -				$ -
TOTAL CONSTRUCTION with CONTINGENCIES without HST							**$ 1,695,000**				**$ 2,282,000**
	ENGINEERING DESIGN & ENVIRONMENTAL PERMITTING										
	Engineering and Design		5%				$ 85,000				$ 114,000
	Engineering Services During Construction		7%				$ 119,000				$ 160,000
	Environmental Permitting & Regulatory Process						$ 100,000				$ 100,000
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES without HS							**$ 1,999,000**				**$ 2,656,000**
		HST - NS	15%				$ 299,900				$ 398,400
TOTAL CONSTRUCTION and ENGINEERING DESIGN with CONTINGENCIES with HST							**$ 2,299,000**				**$ 3,054,000**
						$/m	$ 2,900			$/m	$ 3,800

THIS OPINION OF PROBABLE COSTS IS PRESENTED ON THE BASIS OF EXPERIENCE, QUALIFICATIONS AND BEST JUDGEMENT. IT HAS BEEN PREPARED IN ACCORDANCE WITH ACCEPTABLE PRINCIPLES AND PRACTICES. MARKET TRENDS, NON-COMPETITIVE BIDDING SITUATIONS, UNFORSEEN LABOUR AND MATERIAL ADJUSTMENTS AND THE LIKE ARE BEYOND THE CONTROL OF CBCL LIMITED AND AS SUCH WE CANNOT WARRANT OR GUARANTEE THAT ACTUAL COSTS WILL NOT VARY FROM THE OPINION PROVIDED.

Note 1 A Design Development Contingency is intended to allow for growth scope; quantities; and material costs as the work is better defined in the future.

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