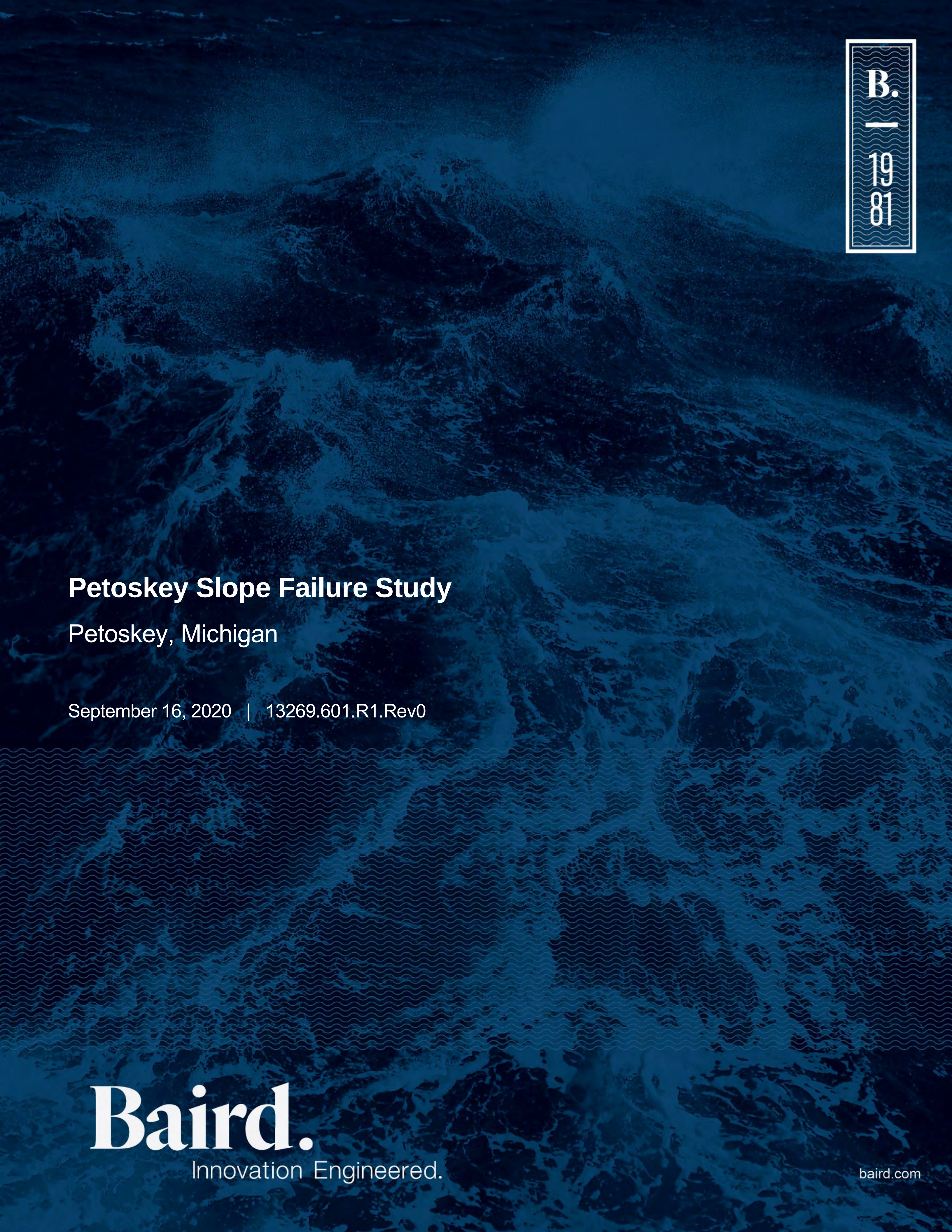

Petoskey Slope Failure Study

Petoskey, Michigan

September 16, 2020 | 13269.601.R1.Rev0

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Petoskey Slope Failure Study

Petoskey, Michigan

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Revision	Date	Status	Comments	Prepared	Reviewed	Approved
RevA	8/26/2020	Draft for Review		RPA/MM	EAL	EAL

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1. Introduction

1.1 Project Description

In late 2019 City of Petoskey (City) staff identified numerous minor slope failures along an elevated portion of the Little Traverse Wheelway (trailway), located between Magnus Park and East Park. This stretch of trailway is approximately one mile in length and founded on a historic railbed that is terraced into the mid-slope of a natural Lake Michigan coastal bluff on the south shore of Little Traverse Bay. A significant portion of the coastal bluff is vegetated from the shoreline to the crest along this reach, with U.S. Highway 31 running parallel to the trailway atop the bluff. There are also several residential properties located near the bluff crest along Arrowhead Drive near East Park, which are in Resort Township. The City of Petoskey is responsible for maintaining this portion of the trailway; however, the trailway cuts through multiple parcels of lands owned by others (i.e. Emmet County and Resort Township residential properties) through an easement agreement.

The combination of observed/ ongoing erosion and recent increase in Lake Michigan water levels raised concerns regarding the overall stability and safety of this reach of shoreline. To better understand and quantify the risks associated with the stability of the shoreline/ bluff between Magnus Park and East Park a group of key stakeholders, the City of Petoskey, Emmet County, and Resort Township (herein all referred to as Owner), retained W.F. Baird & Associates Ltd. (Baird) and OHM Advisors (OHM) to perform a preliminary investigation and analysis of the shoreline/ bluff, and develop conceptual design alternatives to potentially mitigation the ongoing issue(s).

Unfortunately, near the onset of this study (on April 13, 2020) a large section of coastal bluff slope collapsed during a Lake Michigan storm event; destroying approximately 150 lineal feet (LF) of trailway. Fortunately, there were no injuries and although the extent of the failure nearly reached the U.S. Highway 31, it did not cause damage to or require closure of this roadway, which is a main arterial route between Petoskey and neighboring municipalities.

The proximity of the slope failure and potential instability of the collapsed bluff slope in relation to the roadway is concerning. Immediately following this event, the Michigan Department of Transportation (MDOT) installed inclinometers to actively monitor for latent movement/ recession of the slope failure. At present, no activity has been reported since the installation of said devices. In addition, MDOT installed ground water monitoring wells near the recent bluff collapse (on July 28, 2020), the results of which are discussed and utilized in this study.

A project location map, highlighting the battery limits of this study, and the location of this recent major slope failure as well as additional minor slope failures along trailway (i.e. sloughing) is shown in Figure 1.1. Images of the April 13, 2020 slope failure are shown in Figure 1.2. An image of a separate sloughing failure (west of the coastal collapse) that was recently observed is shown in Figure 1.3.

1.2 Report Purpose

The purpose of this report is to provide a summary of the existing site conditions and preliminary engineering analysis (methodologies and results) pertaining to the stability of the bluff and shoreline, as well as provide conceptual mitigation design alternatives for the Owner’s consideration. This report shall serve as a basis to help the Owners make informed decisions regarding this issue.

Figure 1.1: Project Location Map

Figure 1.2: Photo 1 - UAV Image of April 13, 2020 Coastal Bluff Slope Collapse (Baird, 2020)

Figure 1.3: Photo 2 – UAV Image of Minor Sloughing Failures (Baird, 2020)

2. Existing Conditions

Documenting the historic and current site conditions was completed to develop an understanding of the ongoing shoreline and bluff erosion issues. This section of the report provides a detailed summary of the existing conditions data review and field data collection effort. Existing conditions information is subsequently used in this study to support analysis and modeling efforts (i.e. GIS analysis, cross-shore sediment transport shoreline modeling, and slope stability modeling) – the methodologies and results of which are discussed in the following section of this report.

In addition, gaps, limitations, or other deficiencies in the available information that may impact the level of accuracy in the results of this study are identified, along with recommendations to address these deficiencies.

2.1 Background Data Review

Table 2.1 provides a summary description of existing conditions information collated and reviewed for this project.

Table 2.1: Existing Data Review

Data Type	Item Description
Property Ownership	An interactive property ownership map for the site was reviewed via Emmet County online viewer (see Figure 2.1).
Historic Slope Stability Study	NDG slope stability study (dated circa June, 2005).
Topographic Data	Topographic LiDAR data (USACE, 2012 and FEMA, 2015) was obtained via NOAA.
Bathymetric Data	Bathymetric LiDAR data (USACE, 2012) was obtained via NOAA.
Aerial Imagery	Historic USGS aerial imagery of the site was obtained from the following sources/ dates: • USGS - 1954 • USGS - 1956 • USGS - 1968 • USGS - 1974 • USGS - 1978 • USGS - 1993 • USGS - 1998 • USDA - 2005 • USDA - 2009 • NOAA - 2011 • USACE - 2012 • USDA - 2014 • USDA - 2016 • USDA - 2018

Data Type	Item Description
Critical Dune Mapping	Michigan Department of Natural Resources Great Lakes Information System: Department of Natural Resources: Land and Water Management Division Critical Dune Mapping. Source: https://www.michigan.gov/documents/egle/wrd-dune-cda-all_687912_7.pdf. Note, this property is not currently listed as a Critical Dune Area by the MDNR.
Historic Construction Documents	Trailway Construction Documents (NDG, 2007). U.S. 31 Roadway Construction Documents (MDOT, 2014).
Historic Geotechnical Data	(5) Soil Borings (NDG, 2005) (4) Ground Water Monitoring Wells (NDG, 2005).
MDOT	Two ground water monitoring wells (MDOT, 2020).

The following is a list of additional data Baird recommends be acquired for further review and analysis prior to future detailed design and engineering, or construction of a mitigation measure to address the ongoing issue.

Table 2.2: Additional Data Recommended for Review

Data Type	Item Description
Utility Locations	Existing utility location surveys (i.e. buried municipal/ private utilities, municipal water vs. private wells, etc.).
Geotechnical	Historic soil borings were reviewed for this study; however, additional/ deeper soil borings may be required to identify/ further define the geotechnical variability and slope stability for the site.
Natural Resources	Regional hydrogeology investigation to better assess/ understand groundwater properties/ implications.
Bathymetry	Acquire detailed bathymetric survey information extending from the existing shoreline to a depth of approximately 30 ft to capture any recent changes in the lakebed.

Figure 2.1: Property Ownership Map (Emmet County Equalization/GIS Department)

2.2 Field Data Collection

Baird and OHM performed multiple site visits to visually assess the site and perform field data collection tasks. Site visit dates and a brief description of the tasks completed is provided in Table 2.3.

Table 2.3: Field Data Collection Summary

Date	Description
April 22 - 23, 2020	Unmanned aerial vehicle (UAV) site mapping, survey control point collection, lakebed sediment and subsurface assessment (jet probes), and nearshore survey profiles – Baird/ OHM
March 27, 2020	Initial site visit to walk the project and assess the condition of the previously observed erosion and identify any new areas of concern – OHM.
April 13, 2020	General site observations following the coastal bluff collapse – OHM.
June 4, 2020	Site visit to assess the ground water conditions at the location of the coastal bluff collapse – OHM.

A detailed description of the project area and site-specific details (i.e. topography, bathymetry, geotechnical and coastal conditions) are summarized in the following sections of this report.

2.3 Project Area Description

The study area consists of approximately 5,500 lineal feet of continuous Lake Michigan shoreline, between Magnus Park and East Park. The area of focus (battery limits) for this study area is concentrated on the bluff and trailway corridor, extending from the shoreline to U.S. Highway 31 (see Figure 1.1). The coastal analysis aspects of the study extend lakeward to assess the general characteristics of the nearshore bathymetry, adjacent shorelines, the natural movement of sediment along the shoreline (littoral processes), lake level records, and wave data. In addition, the overall extent of the assessed site topography and drainage patterns extend inland (beyond U.S. Highway 31).

2.3.1 Overall Geological and Geomorphological Considerations

A natural slope—such as along the stretch of Little Traverse Bay under consideration—is highly dynamic. From a geological point of view the changes which occur on natural slopes and bluffs happen very quickly; in a matter of years, days, or minutes at times. For example, some geological phenomena (e.g., formation and subsequent erosion of mountain ranges, substantial movement of the Earth's plates, etc.) occur over the course of millions of years. By contrast, the changes we observe in how the waves change the toe of a bluff, or how the groundwater may change over the years, happen on a much faster scale. For example, the shoreline may change rapidly depending on a storm event or change in wind direction. Trees and brush, of course, grow in a matter of years or decades. Surface erosion may become visible after a single major storm. And, at times, a dramatic slope failure may occur as it did on this section of trail on April 13, 2020. These rapid changes are in stark contrast to the geological changes mentioned above. In short, these rapid changes (which occur on a "human timescale") as observed in the environment of a natural slope, may then also directly impact humans much more than those phenomena related to the "geologic timeframe."

2.3.2 Site Conditions and Background

A railroad once operated along what is now as the Little Traverse Wheelway (trailway) for 100 plus years. The trailway exists along a bench that is situated approximately midway up the bluff, with Little Traverse Bay (connected to Lake Michigan) to the north. The slope in the area along the section of shoreline under consideration varies in grade, with the steepest portions having approximately 1.3 horizontal to 1 vertical (1.3H:1V), and the average being around 1.9H:1V.

Some evidence of shallow surface slides (not occurring in the recent past) are evident. We know of one past significant failure circa 1913 (as shown Figure 2.2); however, the location and exact date of this event could not be confirmed and this appears to be related to a landside "washout" as opposed to failure of the slope. More recently, several relatively shallow surface slides have occurred (first reported to the project team in late 2019). Finally, as noted, one other more significant slide occurred on April 13, 2020. This slide, as opposed to the recent shallow slides—was a rather deep-seated coastal bluff collapse. This particular slide was utilized to "tune" the soil parameters for the slope stability analyses aspects of this study (described in Section 4.2).

Figure 2.2: Historic Coastal Bluff Washout (1913)

The vegetation on the slope is highly variable with certain parts containing sparsely populated trees and brush. Other areas of the slope are heavily vegetated. Vegetation in the area of the bluff under consideration most often includes pines, aspen, cedar, and various shrubs and brush. Seepage exiting on the face of the slope at various elevations is sometimes visible. For example, some seepage was observed near the toe of the slope during the initial design work for the new trailway completed in 2009. More recently, we observed seepage at elevations higher than that of the trail in the area that recently failed.

Along this portion of shoreline there are several built features of note, including: the trailway, a group of condominiums situated along the top of the bluff near the western end of the battery limits (Pine Shores), several residential homes near the eastern end of the battery limits (Arrowhead Shores), a 170 LF section of steel sheet pile wall located along the lakeward edge of the trailway (located at the eastern portion of the battery limits), stormwater drainage infrastructure (i.e. two rip-rap drainage channels running perpendicular to/ down the bluff slope, which are located near the recent bluff collapse, and several drainage inlets/ outlets along the trailway), pedestrian shoreline access stairways, trailway bridging, and a pile supported public lakefront overlook structure (located immediately west of the recent coastal bluff collapse).

The toe region of the slope at the shoreline, generally contains visible gravel, cobbles, and boulders.

The observed/ ongoing slope-related issues (i.e. shallow failures) were located east of the recent coastal bluff collapse, with the exception of minor failures to the immediate west of this recent, large failure.

2.4 Site Topography

Our team acquired and reviewed publicly available topographic LiDAR data (USACE, 2012 and FEMA, 2015) for the study area. In addition, a high-resolution, digital terrain model was processed from the UAV mapping conducted by Baird on April 22 and 23, 2020. The extents of the digital terrain model focused on the areas with observed slope failure issues (i.e. from the coastal bluff collapse area to East Park). Note, LiDAR data was utilized to assess topographic elevations for areas that the UAV survey was not able to generate representative data (i.e. areas with significant tree canopy cover).

An interactive 3D map of the post-processed UAV mapping can be viewed here: https://www.arcgis.com/home/webscene/viewer.html?webscene=88df78ec021c4708898c0dc4606beac0&viewpoint=cam:293.36456919,698.99409548,650.886,102689;141.599,59.625.

An overview map with topographic contours (extracted from the UAV digital terrain model) is show in Figure 2.3. A series of profiles (comparing the various bluff topography data sets) is show in Figure 2.4 and Figure 2.6. The comparison of topographic data (i.e. 2012 – 2020) portrays the extent of the recent coastal bluff collapse, as well as minor sloughing failures.

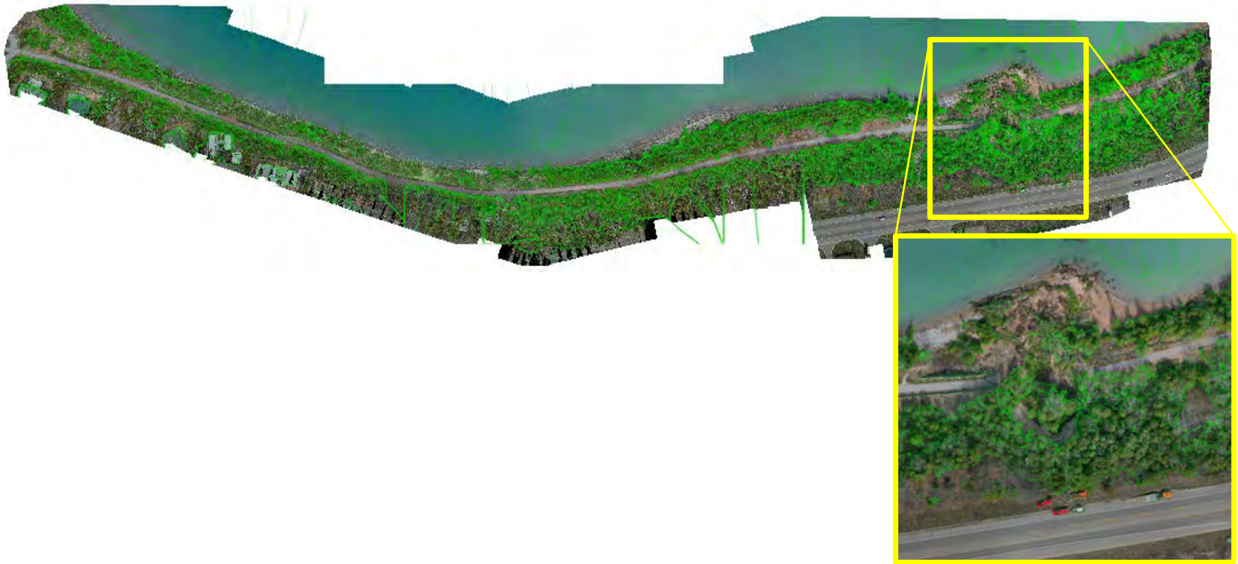

Figure 2.3: Site Topography (Baird, 4/22/2020)

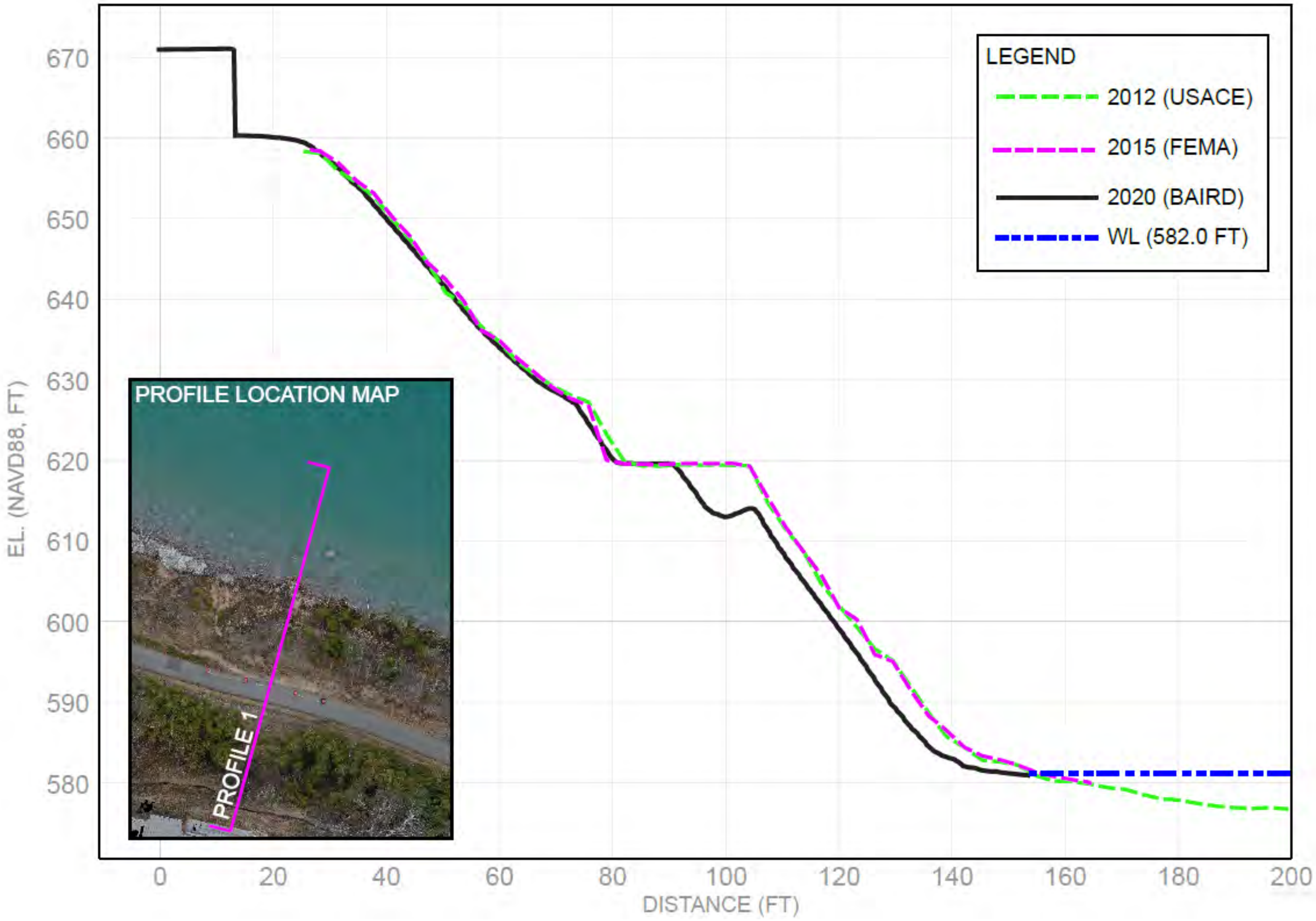

Figure 2.4: Bluff Profile Comparison (2012 to 2020) – Slough Failure

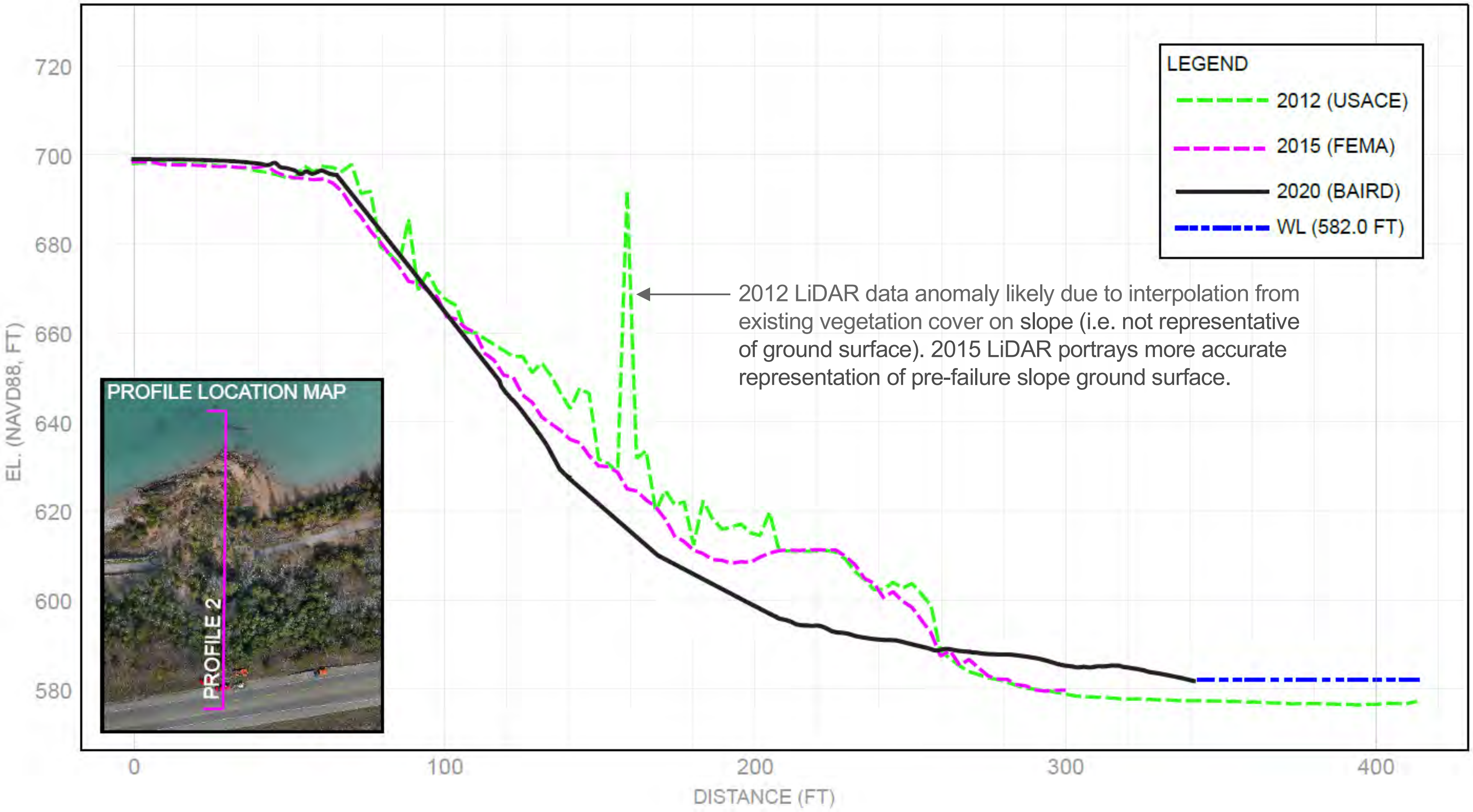

Figure 2.5: Bluff Profile Comparison (2012 to 2020) – Deep-seated Failure

2.5 Lakebed Bathymetry

Bathymetric LiDAR (USACE,2012) was obtained from NOAA's online data repository.[1] An overview map with USACE 2012 LiDAR data is provided in Figure 2.6. Baird also collected multiple survey profiles along the shoreline to verify the current elevation of the nearshore area. The location of Baird's nearshore survey profiles is shown in Figure 2.7. Note, these were collected by wading into the nearshore with survey equipment, therefore the depth is limited to approximately 3 feet for the profiles.

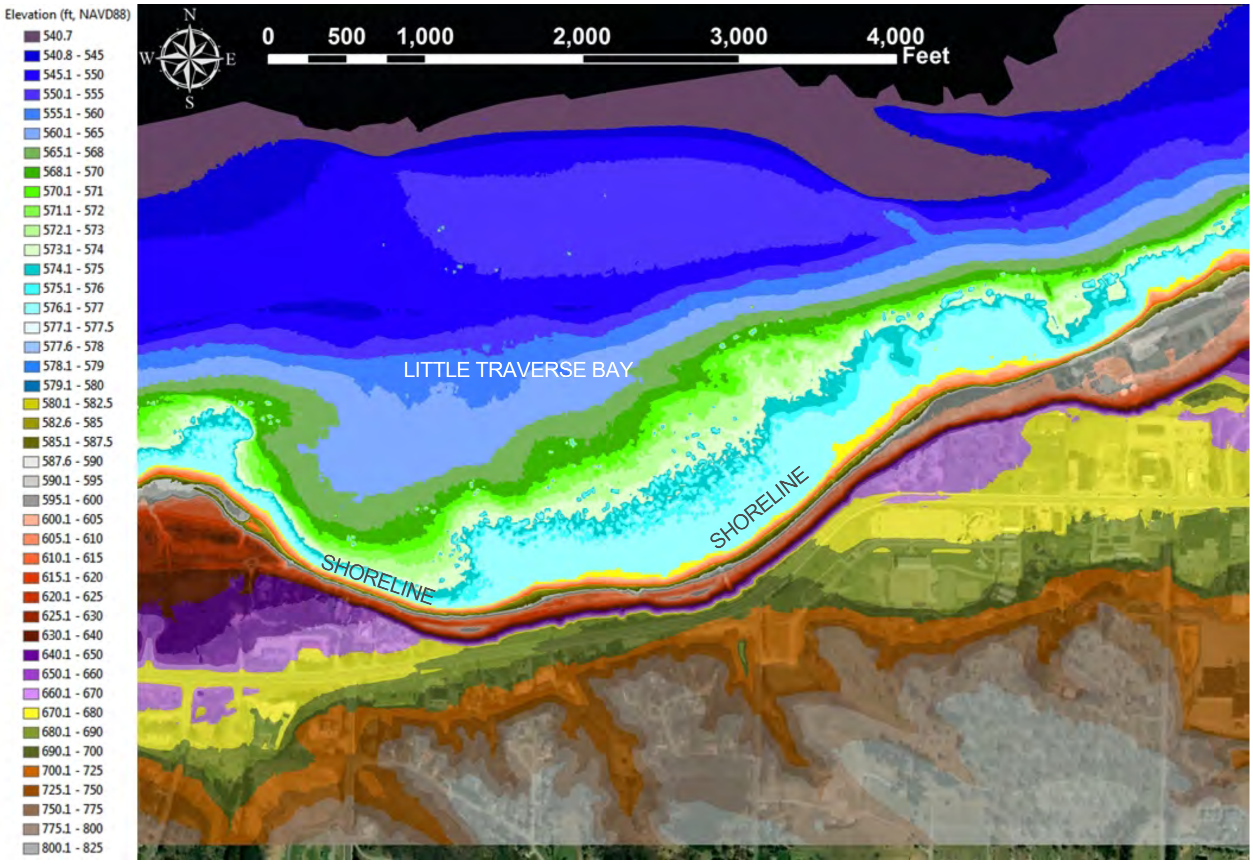

Figure 2.6: Site Bathymetry (USACE, 2012)

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[1] Source: https://coast.noaa.gov/dataviewer/#/

Figure 2.7: Baird Nearshore Survey Profile Locations

2.6 Geotechnical Characterizes

2.6.1 Bluff

The soils making up the bluff are mainly lacustrine sand and gravel according to the *1982 Quaternary Geology of Michigan* map (MDNR, 1999 after W.R. Farrand, 1982). These ground conditions came about as a result of sediments accumulation during and after the latest (Wisconsinan) glaciation. Some of the ground conditions to the south of the trailway, and on the south side of US-31, are characterized as coarse textured glacial till.

Northwest Design Group (NDG) originally completed five (5) soil borings throughout this area in June of 2005. The location of these historic soil borings is shown in Figure 2.8.

Figure 2.8: Historic Soil Boring Location Map (NDG, 2005)

The results of these borings generally indicated that the geotechnical conditions consist of fine to medium sands, with coarse sand, gravel, and cobbles occasionally encountered. The soil boring logs for these five (5) borings are included in Appendix A. For all of the borings, the soil near the surface was typically found to be loose to medium dense, while the deeper soils were dense to very dense. A 4-inch layer of hard clay was observed at a depth of 4 feet in soil boring 6, which was located near the middle of the trailway. In addition, limestone bedrock was encountered at a depth of 35 feet at the soil boring 2 location.

2.6.1.1 Groundwater

At the time of the drilling in June 2005, groundwater was observed in the borings at depths 10 to 16 feet higher than the lake levels at that time, which were approximately 578.2 feet IGLD 1985. Groundwater was not encountered in the borings at the top of the bluff (soil boring 3 and 7) due to the higher surface elevations at those borings.

As previously noted, MDOT installed two new ground watering wells (on July 28, 2020). In addition, while onsite MDOT surveyed the location where groundwater was observed discharging from the face of the bluff slope near the recent collapse.

The location/ results of MDOT's monitoring well installations and observations are summarized in Table 2.4. The horizontal coordinate system/ vertical datum for this data is as follows:

- Horizontal: NAD 1983 2011 State Plane Michigan Central FIPS 2112 Ft Intl.
- Vertical: IGLD 1985 Ft.[2]

Table 2.4: MDOT Groundwater Monitoring Well Installations/ Observations

Monitoring Well	Monitoring Well Location			Groundwater El.
	Easting	*Northing*	*Ground Elevation*	
1	19525429.2	748441.2	696.03	606.03
2	19525442.7	748369.4	698.33	610.13
Observed Groundwater Discharge	19525407.7	748578.3	597.85	597.85

---

[2] Note, two vertical datums are presented in this report (IGLD 1985, Ft and NAVD 88, Ft). IGLD 1985 is approximately 0.17 feet (~2 inches) below NAVD 88. Elevation conversion: IGLD 1985, Ft + 0.17 Ft = NAVD 88, Ft.

Figure 2.9: MDOT Groundwater Monitoring Well Map

2.6.1.2 Former (2008) Slope Stability Analyses

A relatively recent trailway improvement project (in 2008) was undertaken, and these project documents were reviewed for this study. As part of the trailway improvements of 2008, Northwest Design Group (NDG, now part of OHM Advisors) prepared a geotechnical report. This geotechnical report contained information about the background of the site along with initial slope stability analyses.

The results of the 2008 analyses indicated factors of safety of near, yet a bit greater, than unity for shallow surface slides. The factor of safety (herein referred to as FOS) is a ratio of the resistance of the soil to that of the forces attempting to pull the slope downward, toward the shoreline. NDG estimated that the factors of safety for deeper-seated failure surfaces (that could potentially undermine or damage the trailway) ranged from about 1.1 to 1.3. NDG further noted that typical acceptable safety factors for this type of installation are between 1.3 to 1.5.

2.6.2 Shoreline

Information regarding the depth of erosive lakebed sediment (i.e. sand) in the nearshore was analyzed during the field data collection effort. Jet probes, which involve driving a steel pipe (attached to a hose and water pump) into the lakebed to a depth of refusal (or hardpan) to inform sediment layer thickness and subsequently erosion potential, were attempted at several locations along this shoreline where erosive conditions were identified (see Figure 2.10). However, the nearshore lakebed was not penetrable with the jet probe equipment as the lakebed in this area generally consists of stone material (cobble and boulders), as opposed to finer grain, sandy material. Based on these preliminary observations, it is assumed that minimal deepening or downcutting of the nearshore likely occurs due to energy associated with wind/ wave processes and shoreline transport. However, during periods of high lake levels waves reaching the toe of the bluff are able to erode material immediately adjacent to the shoreline. The erosion of the bluff toe leads to an over steepened/ undermined slope, causing sloughing and recession between the trailway and the shoreline, as shown in Figure 2.11. Additional analysis regarding erosion potential of the bluff toe is discussed in Section 4.

Figure 2.10: Jet Probe Location Map (4/23/2020, Baird)

Figure 2.11: Existing Slope Failure and Nearshore Bluff Recession

3. Coastal Analysis

Analyses were performed to characterize water level, wave, wind, ice, and sediment transport conditions at the project site and to determine water levels and wave conditions suitable for preliminary design. This section outlines the methodology and results of the analyses, as well as recommendations for more refined analyses.

3.1 Water Levels

Long term (monthly mean) lake levels were extracted from USACE records (USACE, 2020), which is determined from a coordinated set of gages on Lake Michigan. Monthly mean water levels for Lake Michigan-Huron are shown in Figure 3.1 from 1918 to present. As evident from the records, long term lake levels have fluctuated considerably in past decades.

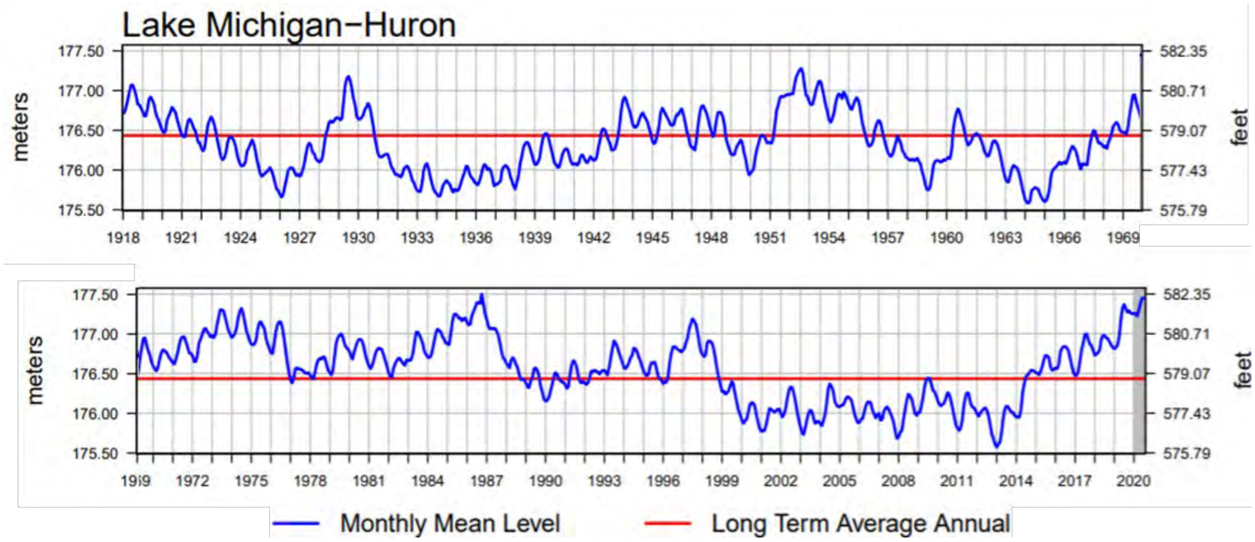


Figure 3.1: Monthly mean water levels (IGLD85) for Lake Michigan-Huron from 1918 to 2019 (adopted from USACE, 2019)

Great Lakes water levels tend to fluctuate on various time scales and are dependent on many factors. Interannual fluctuations are caused by changes in climatic conditions over the Great Lakes drainage basin (in particular, precipitation and evaporation). Seasonal fluctuations are caused by seasonal weather patterns in the region (i.e. precipitation and runoff), while short term localized variations are caused by the influence of individual storm events. A particular point of emphasis is the recent rise in lake levels on Lake Michigan-Huron following an extended period of relatively low lake levels from approximately 2000 to 2014. Long term trends show that water levels on Lake Michigan are currently the highest levels they've been in several decades.

The USACE does provide 12-month water level forecasts (USACE, 2020) for each of the Great Lakes, though they are subject to considerable uncertainty (as shown in Figure 3.2) and do not provide the information required for long-term design.

In addition to the long-term water level records, historical hourly water level observations were collected from a nearby tidal gage (NOAA-9087096) located in Port Inland, MI. Together, the long-term and short-term water level records provide the input required for a joint-probability analysis to determine extreme water levels, surge levels, and combined water levels.

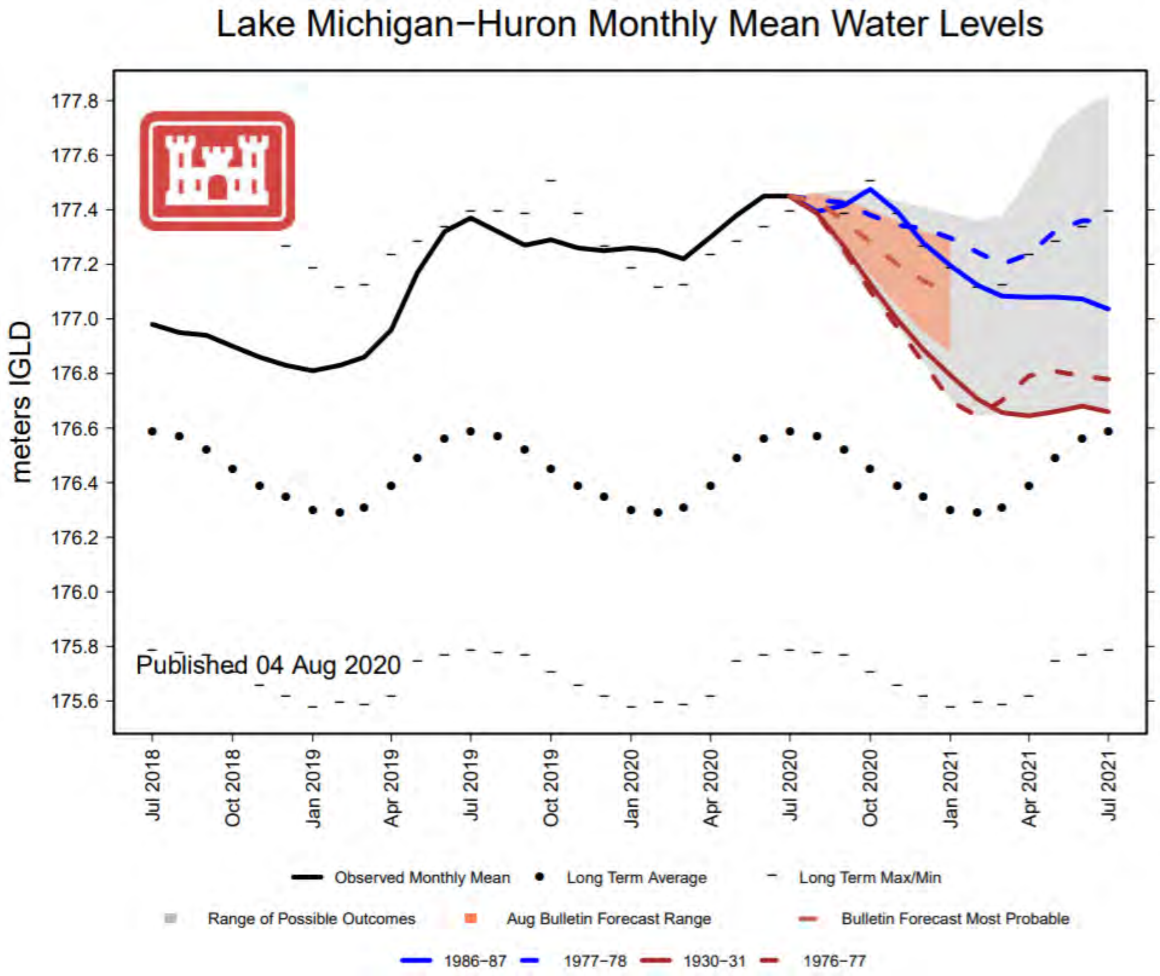


Figure 3.2: USACE 12-month water level forecast for Lake Michigan-Huron (USACE, 2012)

The joint-probability analysis was conducted according to *FEMA Great Lakes Coastal Guidelines* (FEMA, 2014), which prescribes the methodology outlined in Melby et al., 2012. The joint-probability analysis was performed for the period over which both long term and short term records overlapped (1970 to present). Annual maximums of monthly mean water levels were extracted from the data. Surge levels were extracted from the hourly time series using a 30-day Gaussian smoothing technique. Then, extreme surge levels were extracted using a peak-over-threshold (POT) method. A probability distribution was then fit to both datasets (according to Melby et al., 2012) to determine extreme values and combinations of each parameter.

The results are presented using terminology outlined in FEMA, 2014:

- Lake level – water level that includes the long-term water level changes in the Great Lakes plus seasonal water level changes.

- Storm surge/ Seiche – rise of the lake surface that occurs in response to barometric pressure variations (the inverse barometer effect) and to the stress of the wind acting over the water surface (wind setup component).
- Still water level (SWL) – water level defined by lake level plus storm surge/ seiche.

The results of the joint-probability analysis for Port Inland, MI (NOAA-9087096) are listed in Table 3.1.

Table 3.1: Port Inland, MI (NOAA-9087096) return period water levels

Water Level	Return Period Water Levels (ft. and ft. IGLD85)					
	2 year	5 year	10 year	25 year	50 year	100 year
Lake level	579.75	580.90	581.47	582.02	582.35	582.61
Storm surge	1.41	1.74	2.03	2.52	2.98	3.53
Still water level	581.29	582.43	583.03	583.66	584.06	584.43

Conversions of water levels on the Great Lakes from IGLD85 to LWD datums can be completed using conversion values provided by NOAA (https://tidesandcurrents.noaa.gov/gldatums.html). For Lake Michigan-Huron, 0 ft. LWD is equal to 577.5 ft. IGLD85.

It is important to note that the Port Inland gage is not located at Petoskey, and there are differences between the extreme water levels at these two sites. For the purposes of preliminary design of coastal and shoreline structures at Petoskey, the extreme water levels presented herein are considered generally representative and suitable for application.

3.2 Wave Climate

Due to the location within Little Traverse Bay, the project site generally experiences W–NW waves approaching from Lake Michigan and considerably smaller N–NE waves generated locally from wind forcing within Little Traverse Bay.

This section outlines the methodology used to characterize the wave conditions and to determine extreme wave conditions at the project site.

3.2.1 Deepwater Wave Hindcast

A wave buoy exists nearby (NOAA-45022, approximately 8km NW), with wave climate observations dating back to mid-2010. Because of the limited duration of observations available at this buoy, the data may not capture the full range of expected deepwater wave conditions (from Lake Michigan) in this area and is not sufficient to perform a statistical analysis to determine extreme wave conditions.

In order to overcome this limitation, deepwater wave information was extracted from Baird's existing 32-year (1979-2011) offshore wave hindcast for Lake Michigan. This hindcast was developed utilizing a 2D wave model for the entire lake, and Climate Forecast System Reanalysis (CFSR) wind conditions from NOAA (NOAA, 2018). Figure 3.3 shows the positions of the existing wave buoy as well as the nearest hindcast output point that was used to represent deepwater wave conditions for this analysis. A comparison of the overlapping periods of record for both the wave buoy and hindcast was conducted; the quantile-quantile plot is shown in Figure 3.4 along with a wave rose summarizing the wave conditions at the hindcast output point.

Figure 3.3: NOAA-45022 wave buoy and Baird CFSR hindcast output point

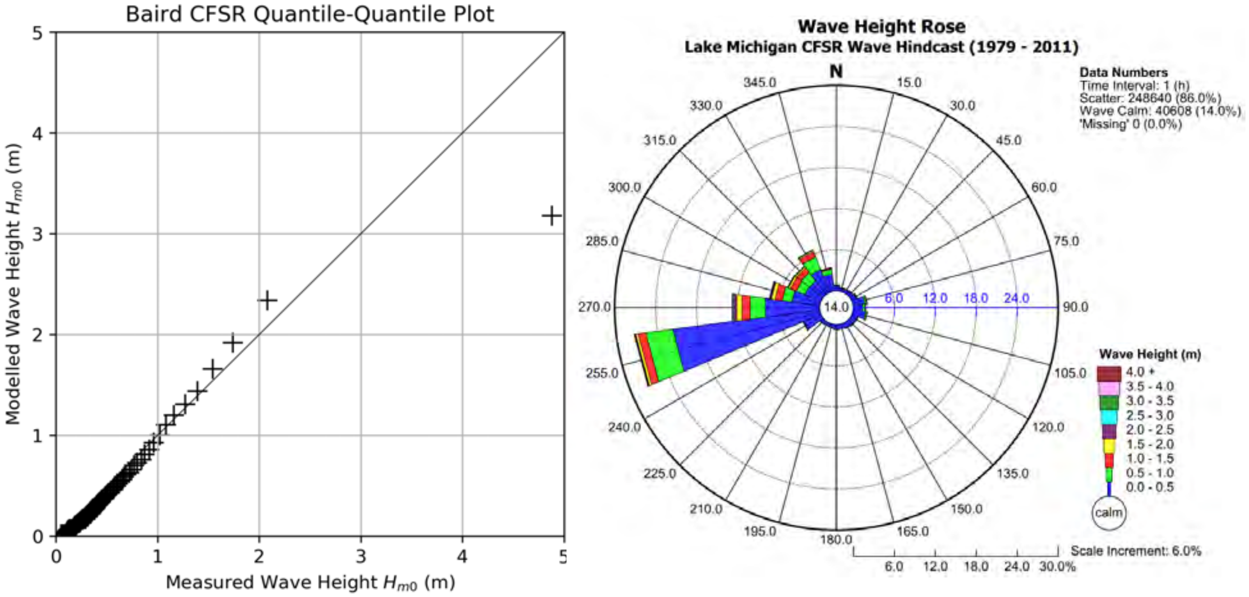


Figure 3.4: Quantile-quantile comparison of NOAA-45022 buoy and Baird's CFSR hindcast significant wave heights (left), Baird CFSR hindcast significant wave height rose (right)

The quantile-quantile plot shows that, in general, the Baird CFSR hindcast is in good agreement with the observed wave conditions near Petoskey but may tend to slightly over-estimate wave heights in the 1 m to 2.5 m range. An outlying data point with approximately 5 m significant wave height was recorded at the buoy that the hindcast under-estimates, but it is unclear whether this was an erroneous measurement or not.

3.2.2 Nearshore Waves

As deepwater waves propagate into shallow water, they begin to transform due to processes such as refraction, diffraction, shoaling, and breaking. These processes are dependent on the wave characteristics, local bathymetric conditions, and existing structures at areas of interest.

To determine nearshore wave conditions at the project site, Baird utilized the MIKE21 Spectral Wave (M21SW) model to transform deepwater waves. Various combinations of deepwater wave heights, periods, and directions were simulated using M21SW to develop a transfer function that defines the relationship for the change in wave characteristics from offshore to nearshore. The transfer function was then used to transfer the full 32-year hindcast to the project site, at a water depth of -10 m LWD.

Figure 3.5 and Figure 3.6 shows the extents of the M21SW model, the bathymetry, and the extraction points used for the project site.

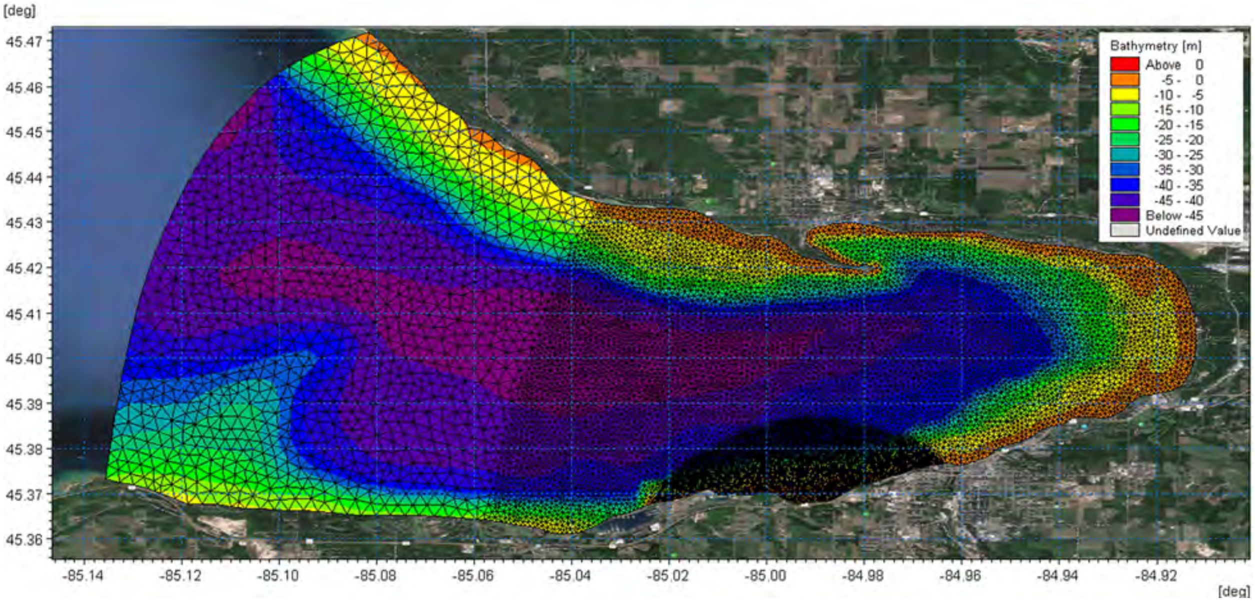


Figure 3.5: M21SW model extents used for deepwater wave transformation simulations (m LWD)

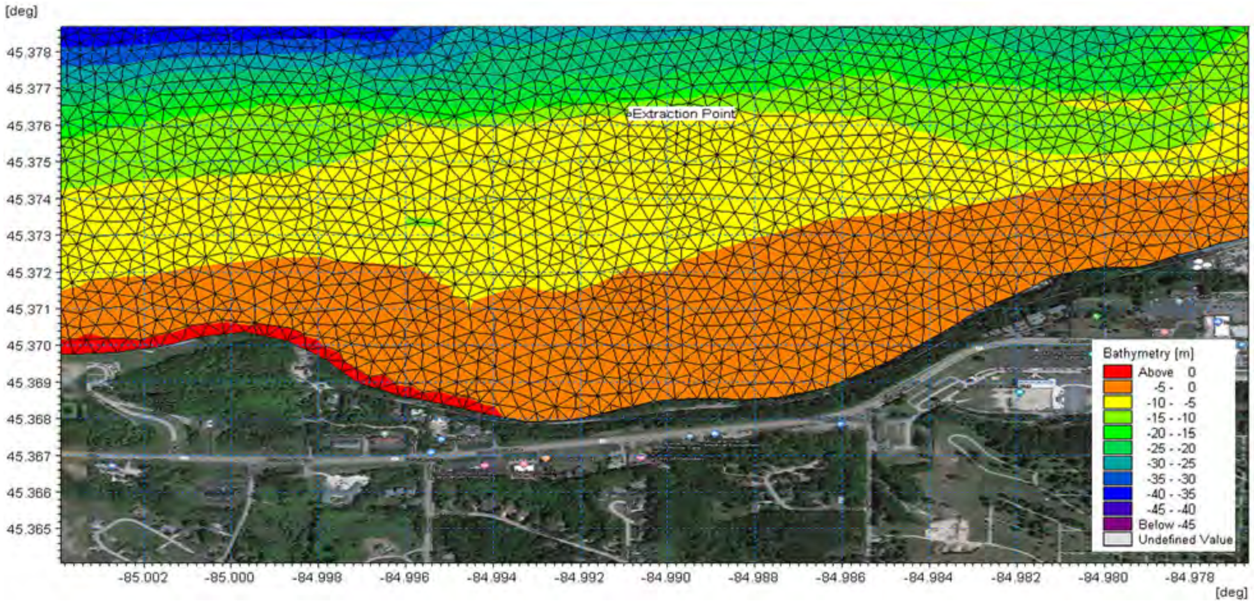


Figure 3.6: M21SW model grid and output points used for deepwater wave transformation simulations (m LWD)

The westerly deepwater waves tend to refract over the nearshore bathymetry and approach the site from the WNW direction.

Figure 3.7 summarizes the nearshore wave conditions as a result of applying the transformation. Figure 3.8 shows the general refraction pattern observed in the M21SW simulations.

The majority of waves at the project site occur within the 0 m to 0.25 m height range, with some occurring in the 0.25 m to 1 m range, and less frequent larger waves. Westerly approaching offshore waves tend to refract due to nearshore bathymetry.

There is a topographical feature (at East Park) that leaves an area of sheltering on the west side of the project area, approximately near Arrowhead Dr. As shown in Figure 2.6 (and Figure 3.6), the east side of the project area tends to have shallower bathymetry that extends further from the shoreline. This localized difference appears to cause more wave transformation, leading to higher wave heights from shoaling and subsequent breaking closer to shore.

There also appears to be sections of shoreline along the project site where concentration of wave energy is occurring. Notably, these include both the locations of minor sloughing and the coastal bluff collapse on (outlined in Section 1, Figure 1.2 and Figure 1.3).

3.2.3 Locally Generated Waves

The locally generated wave climate within Little Traverse Bay was determined by applying CFSR winds over the M21SW model domain. Due to the location of the hindcast point and model boundary, waves from approximately 0° to 180° will not propagate to the project sites. This modeling approach accounts for these waves, as well as the wave growth that would occur over the fetch from the model boundary to the project site.

Similar to the deepwater wave transformation, a range of wind speeds and directions were simulated to generate a transfer function. This transfer function was then used to transfer the Baird 32-year hindcast wind records to locally generated waves at the same project site extraction point (at -10 m LWD).

In general, the majority of the winds approached from the WSW-NW directions. This resulted in waves from those directions, mostly in the 0 m to 0.5 m height range. A smaller proportion of waves approached from the NW-NE directions, mainly in the 0 m to 0.5 m height range.

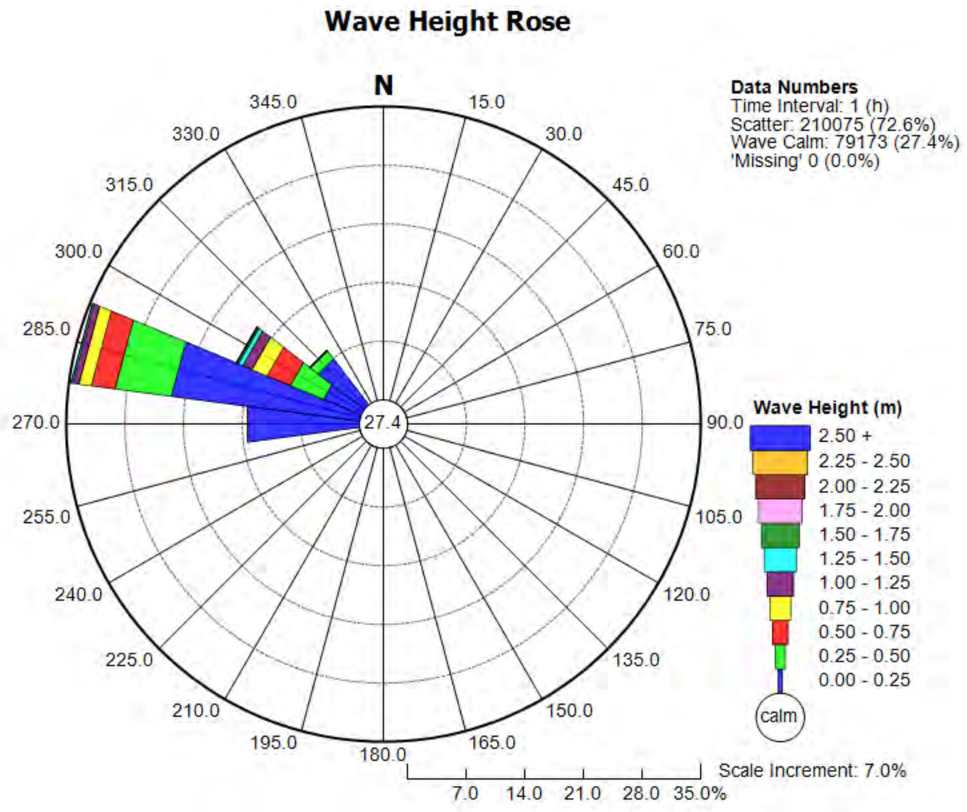


Figure 3.7: Significant wave height rose at each project site (deepwater transformation only)

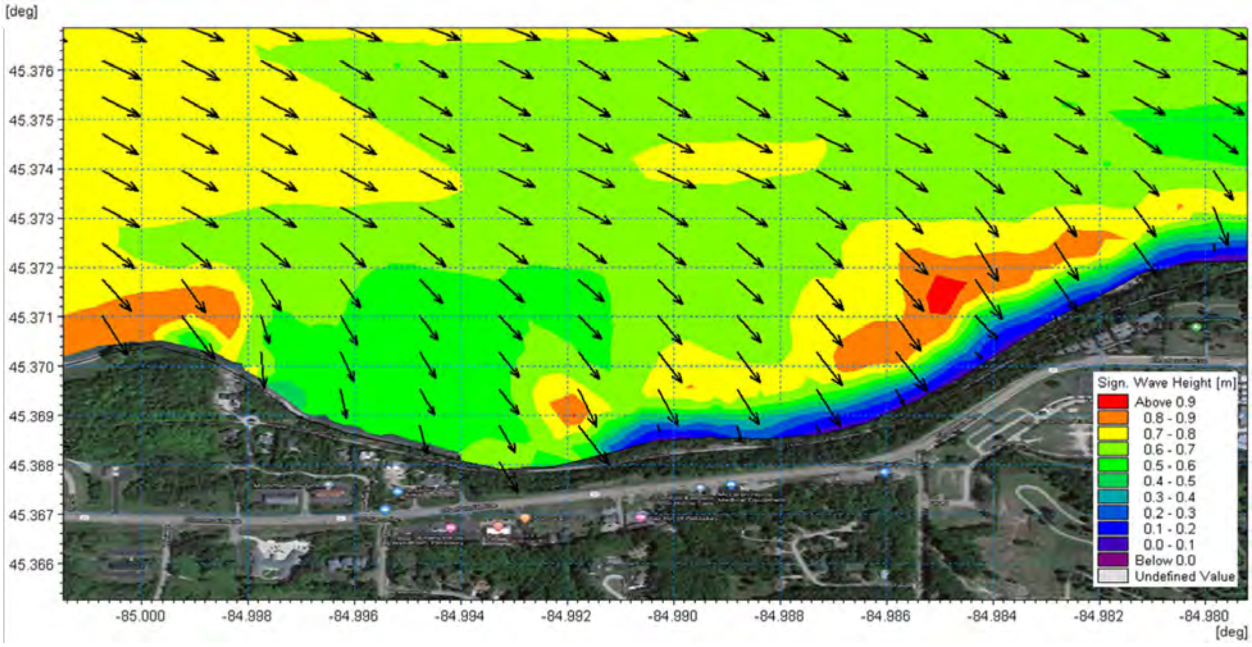


Figure 3.8: General refraction pattern observed in M21SW simulations

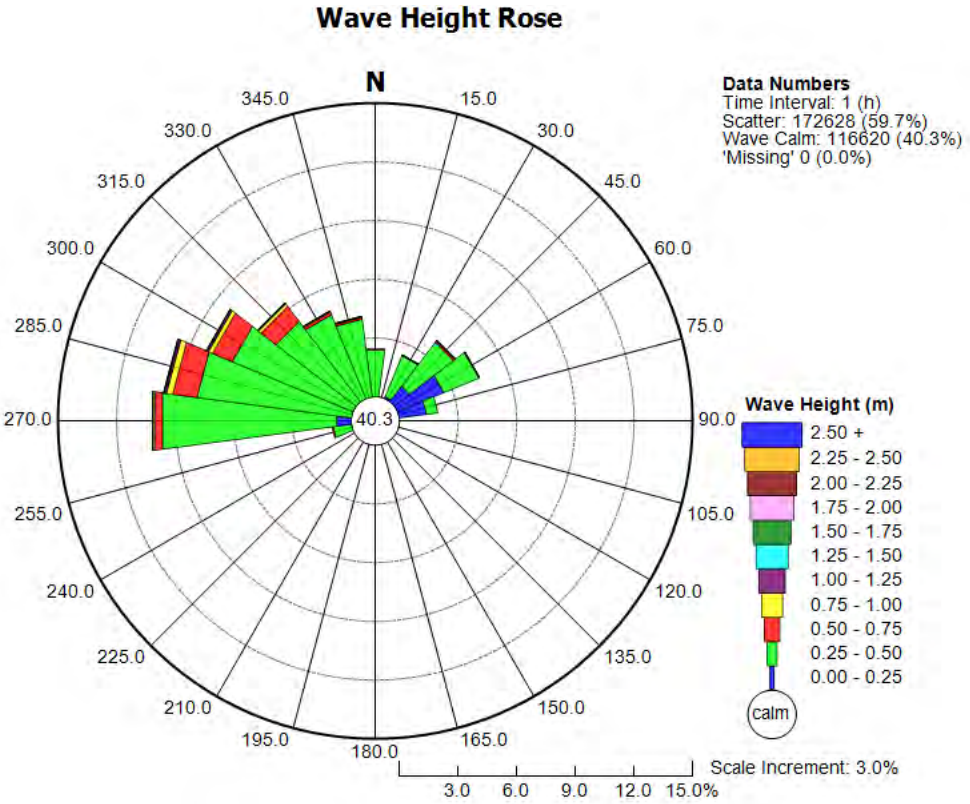


Figure 3.9: Significant wave height rose at Bayfront Park Central (local transformation only)

3.2.4 Combined Wave Climate

The transformed deepwater waves (approaching from Lake Michigan) were combined with the locally generated waves to develop an estimate of the overall wave climate at the project site. The resulting wave rose presented in Figure 3.10 summarizes the overall wave climate. The overall wave climate is generally similar to the transformed deepwater wave conditions, albeit with some slight height and directional changes. The majority of waves tends to approach the project site from approximately the WNW direction. A small portion of waves (locally generated) also approaches from approximately the NE direction. Most waves at the project site occur within the 0 m to 0.50 m height range, with some occurring in the 0.50 m to 1 m range, and less frequent larger waves.

A tabular summary of the wave climate (frequency of occurrence of by wave height and direction) is provided for the project site, refer to Table 3.2). The values in each cell of the tables show the frequency of wave conditions occurring within that specific significant wave height bin and wave direction bin (i.e. 43.40% of waves are ≥ 0.00 m and < 1.75 m in height, and come from directions ≥ 285° and < 300°).

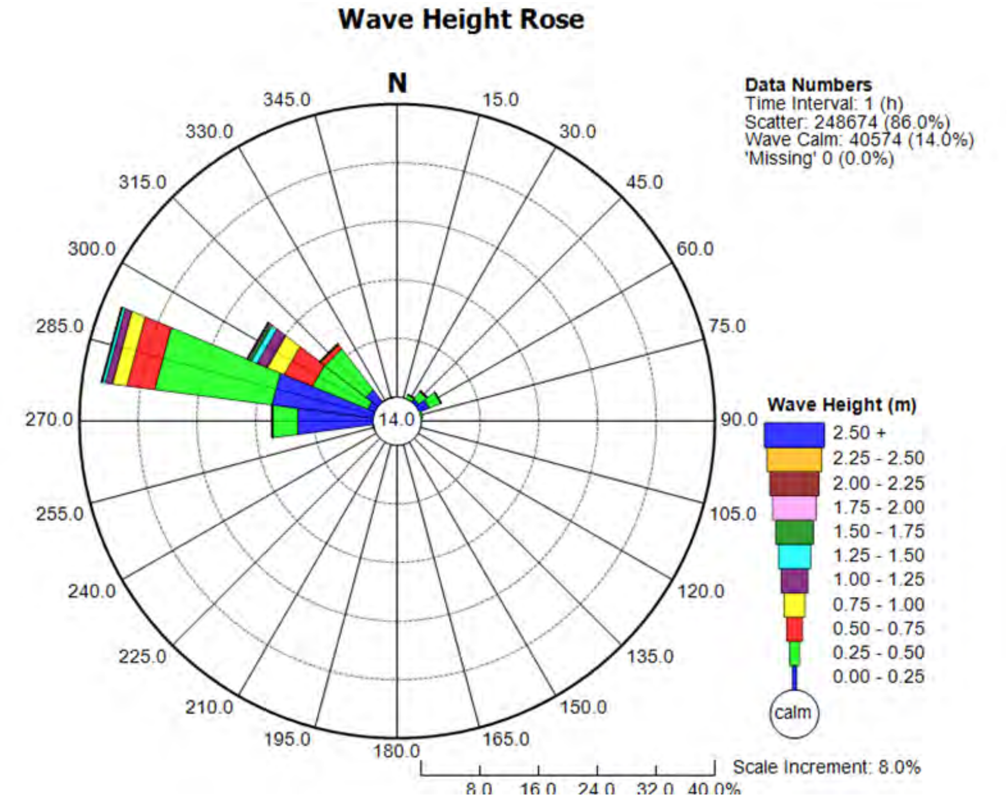


Figure 3.10: Significant wave height rose at each project site (combined wave climate)

Table 3.2: Combined significant wave height and mean wave direction joint occurrence table

Significant Wave Height (m)	Mean Wave Direction (degrees)																							
	0	**15**	**30**	**45**	**60**	**75**	**90**	**105**	**120**	**135**	**150**	**165**	**180**	**195**	**210**	**225**	**240**	**255**	**270**	**285**	**300**	**315**	**330**	**345**	**Total**
0.00	0.00	0.00	0.10	0.60	1.60	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	12.10	16.20	1.20	2.40	0.00	0.00	34.20
0.25	0.00	0.00	0.60	1.50	2.00	0.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.00	18.70	9.50	8.20	0.00	0.00	44.90
0.50	0.00	0.00	0.00	0.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	4.50	4.90	0.80	0.00	0.00	10.30
0.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.30	2.80	0.10	0.00	0.00	5.30
1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.20	1.80	0.00	0.00	0.00	3.00
1.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.50	0.90	0.00	0.00	0.00	1.40
1.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.40	0.00	0.00	0.00	0.50
1.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.20	0.00	0.00	0.00	0.20
2.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.10	0.00	0.00	0.00	0.10
2.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total	0.00	0.00	0.70	2.20	3.70	0.30	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	16.10	43.40	21.90	11.60	0.00	0.00	100.00

3.2.5 Extreme Waves

To determine the characteristics and recurrence intervals of extreme wave conditions at the project sites, a statistical analysis was performed on the overall wave climate at the project site determined from M21SW modelling.

Table 3.3 summarizes the extreme wave conditions for various return periods at the project, extracted at the same extraction point mentioned previously.

Table 3.3: Extreme wave conditions at the project site

Site	Return Period (yr)	H_s (ft.)	T_p (s)	Mean Wave Direction (deg.)
Project Site	2	7.46	6.0 - 9.0	280 - 300
	5	7.90	6.5 - 9.0	280 - 300
	10	8.19	7.0 - 10.0	280 - 300
	25	8.51	7.0 -10.0	280 - 300
	50	8.73	7.0 - 10.0	280 - 300
	100	8.94	7.5 - 10.5	280 - 300

*Note: H_s = significant wave height, is defined as the average of the largest one third of the waves in a wave train; the maximum wave height (H_{max}) may be 1.5 to 2 times this value.

3.3 Preliminary Ice Analysis

A preliminary analysis was conducted to characterize ice conditions at the project site during winter months, floe patterns during spring break-up, and to identify potential issues concerning the design of shoreline structures or shoreline stability. This section summarizes the data that was collected and results from the ice preliminary analysis.

From analysis of satellite imagery in the area, Little Traverse Bay tends to completely freeze over during most winters. Figure 3.11 shows this area frozen over during February 2017. Ice in the bay typically forms from floes that are generated within the bay and ice that is pushed into the bay from westerly winds; these then freeze together to form large sheets. During extremely cold temperatures, the bay will completely freeze up and remain static until spring break-up. However, local accounts suggest that ice can shift during winter months due to wind and water conditions within Lake Michigan (Sherburne, 2013).

Figure 3.11: Completely frozen Little Traverse Bay in February, 2017 (from Google Earth Engine)

Historical observations of ice coverage and thickness do exist near the NOAA-45022 wave buoy, located approximately 8 km from the project site. Ice concentration charts (derived from satellite imagery) and gridded forms of the same data are available for download from NOAA (https://www.glerl.noaa.gov/data/ice/), along with a wealth of additional information regarding historical and forecasted ice conditions on the Great Lakes. Figure 3.12 shows the maximum extent of ice coverage and concentration on Lake Michigan for winter 2018-2019. As shown in Figure 3.12, Little Traverse Bay tends to experience total or nearly total ice coverage.

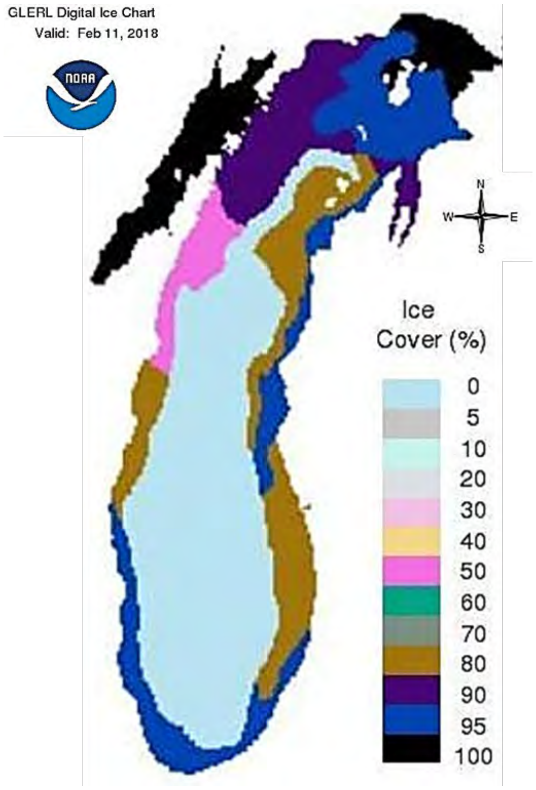


Figure 3.12: Maximum extent of ice coverage and concentration on Lake Michigan for winter 2018-2019

This same information (extracted from a gridded format) is shown in Figure 3.13 near the NOAA-45022 wave buoy, superimposed with historical ice thickness observations collected by NOAA (https://nsidc.org/data/g00803). The ice thickness records only span the winters of 1965-1977 but provide an indication of the range of ice thicknesses that may be present at the project sites during a typical winter.

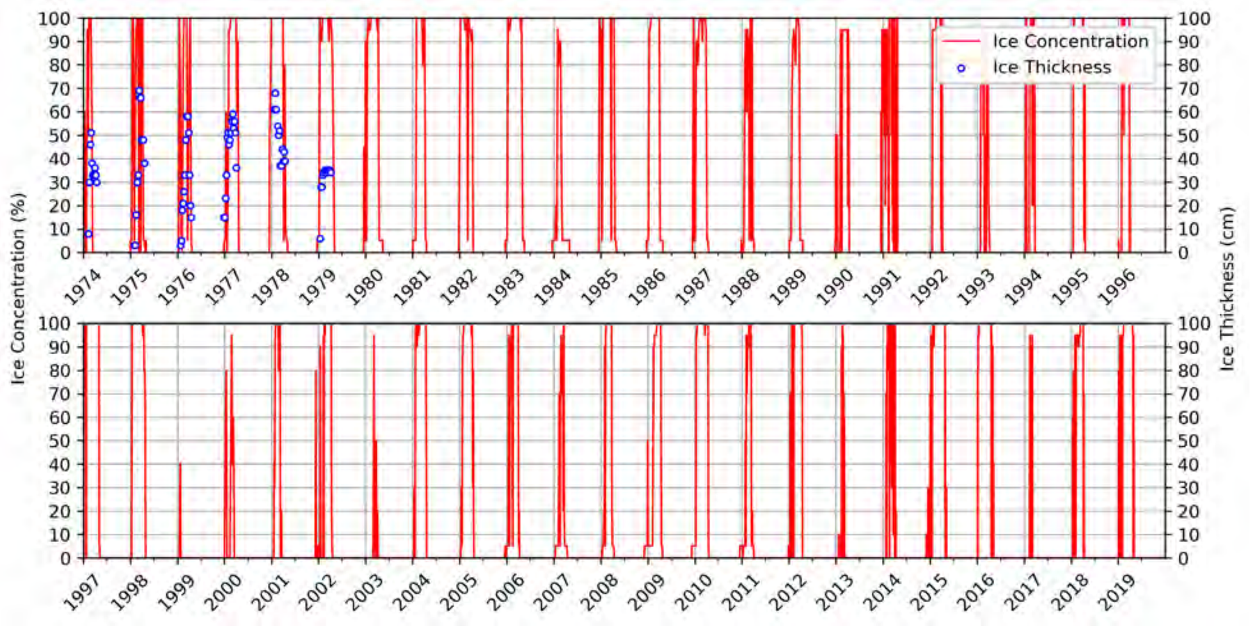


Figure 3.13: Time history of ice concentration and thickness near NOAA-45022 wave buoy

Analysis of this data indicates that, near the project site, ice begins to develop around mid to late December, with maximum coverage for approximately 3 months, and begins to melt and break up around late March to early April. Variations from this general trend have occurred and should be taken into account. Ice thicknesses have been observed in the area up to 70 cm. Figure 3.14 shows the transport patterns that can occur during the spring melt and break-up process along the Petoskey shoreline and at project site.

Figure 3.14: Spring melt and break-up transport patterns in March, 2010 (from Google Earth)

Due to the nature of dominant wind and wave conditions along the shoreline, ice floes tend to move eastward, leaving the project site relatively ice free following the spring break-up process. Due to the interruption of this longshore transport from the breakwater at Petoskey City Marina, ice can build up on the west side of the breakwater and along the shoreline east of the project site; this can reduce the amount of longshore ice transport that occurs during spring.

Ice is a key consideration for the design of shoreline structures and shoreline stability. Freeze and thaw cycles can affect the integrity of revetment stone and induce movement or entrapment of filter material. Interaction of shoreline structures with moving ice floes can produce effects that may not occur due to forces typically considered in revetment or shoreline design. During the spring break-up process, ice debris may increase loading experienced by shoreline structures during wave action.

Baird recommends that a detailed assessment of ice conditions at the project sites be undertaken prior to final design development, including the following items:

- Further analysis of historical data available from NOAA and other sources;
- Typical extreme ice thicknesses and material properties near the project sites;
- Local experience of spring break-up processes; and
- Local experience and/or literature review of ice-related damage to coastal structures in the area.

3.4 Preliminary Sediment Transport Analysis

A preliminary sediment transport investigation was undertaken to better understand longshore transport patterns, and to identify potential issues for the design of structures and shoreline stability at the project site.

A field investigation was conducted on April 22 – 23, 2020, which included the collection and analysis of bed material at the project site using jet probes. This is discussed in more detail in Section 2.4. The results of this investigation showed that the bed material along the shoreline and nearshore area generally consists of coarse material ranging up to cobbles and boulders in size, which is common for the area. It is expected that the lakebed in the area is generally stable.

Historical boreholes have shown that the geotechnical conditions in the bluff material are of fine to medium sands, with coarse sand, gravel, and cobbles occasionally encountered. The bluff material is generally non-cohesive and finer than the bed material. The bluff is therefore more susceptible to erosion/wash-out from coastal forces. Undercutting of the bluff toe or damage to the slope face can occur during storms with both low/high water levels and extreme wave conditions. Material can also be eroded from the bluff over time due to general coastal processes in the area.

It appears that there is limited supply of sediment in general, resulting mainly in narrow cobble and gravel beaches throughout the area. Based on the bed and bluff material, it is expected that the main source of sediment is the finer grained material eroded from the bluff along the shoreline, and from similar processes and sources further west along the shoreline. This finer grained bluff material is sorted and transported away from the larger bed material by easterly littoral currents resulting from the dominant wave/refraction patterns along the shoreline near the project site.

It is likely that the project site shoreline experiences most of its longshore transport during extreme wave conditions and is generally otherwise stable. Natural headlands exist at the east and west extents of the project site, and a smaller, less pronounced headland exists near the middle (immediately west of the April 13, 2020 coastal bluff collapse location). These features may contribute to localized build-up of sediment on their west sides, though the amount of sediment supply in the area suggests this may only occur temporarily during storms when sediment gets washed into the lake.

4. Shoreline and Slope Stability Analysis

Analyses were completed to understand the stability of the shoreline/ bluff throughout the project battery limits. This included numerical modeling of the bluff toe erosion, and bluff slope stability modeling. The results and conclusions of the analyses are provided below.

4.1 Numerical Modeling of Potential Future Bluff Toe Erosion

While the nearshore lakebed conditions are assumed to be relatively stable due to the observation of cobble and stone material (as opposed to sand), erosion at the toe of the shoreline bluff caused by wave action may result in oversteepening of the bluff slope and trigger eventual bluff failure/ recession. Baird utilized the COSMOS model to estimate the extents of bluff toe erosion under a variety of wave and water level conditions. COSMOS is a two-dimensional (2D) profile change model that consists of several predictive modules for simulation of nearshore processes. The COSMOS model requires lakebed profile, sediment size/type, waves, and water levels as input. A cross-shore profile was developed from the bathymetry and topographic data as described in Section 2. The profile extends from the top of the bluff, offshore to a depth of approximately 33 ft (10 m) as shown in Figure 4.1. It was assumed that the lower portion of the shoreline bluff contains more than 85% sand with median grain size of 0.21 mm for the model calculations. The model was run for a total of two storm events (see Figure 4.2), which represent the top two storms in the 35-year hindcast, at three water levels (2, 10, and 100-year return period). This combination resulted in six modeled scenarios.

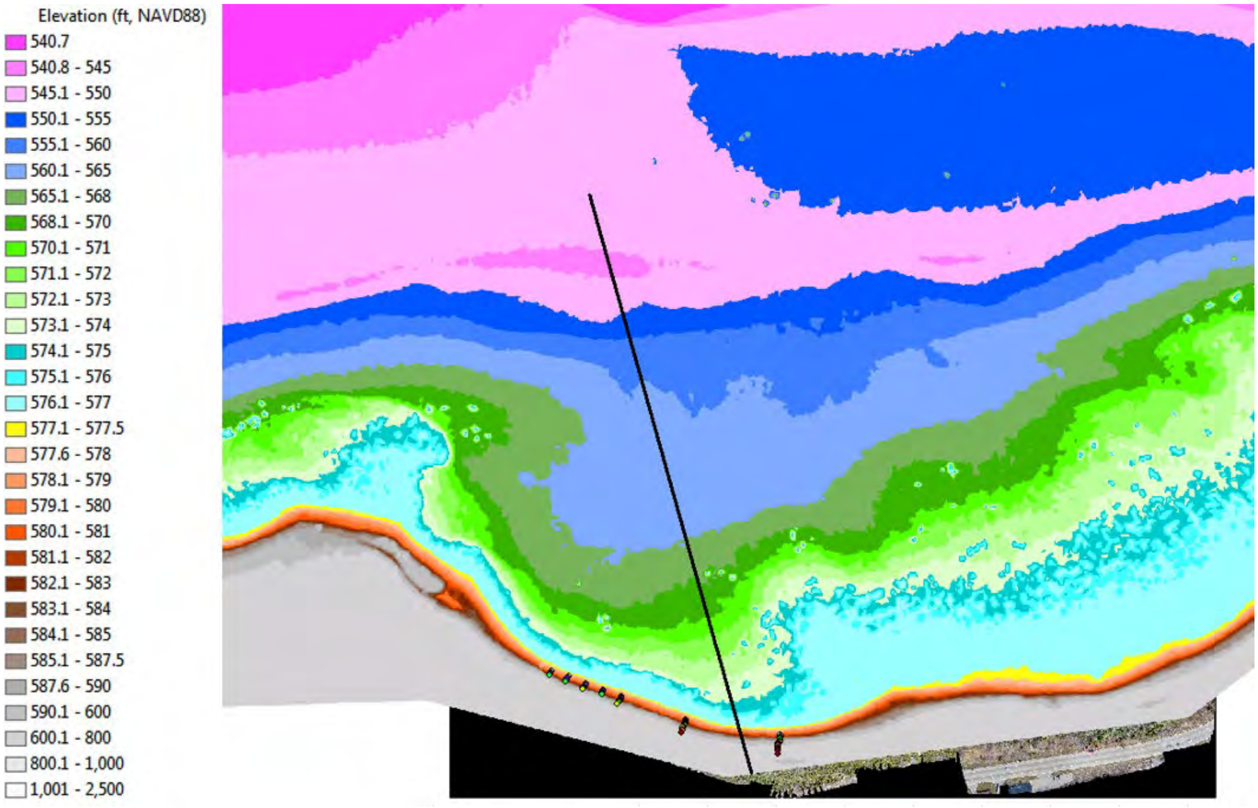


Figure 4.1: COSMOS Profile Location

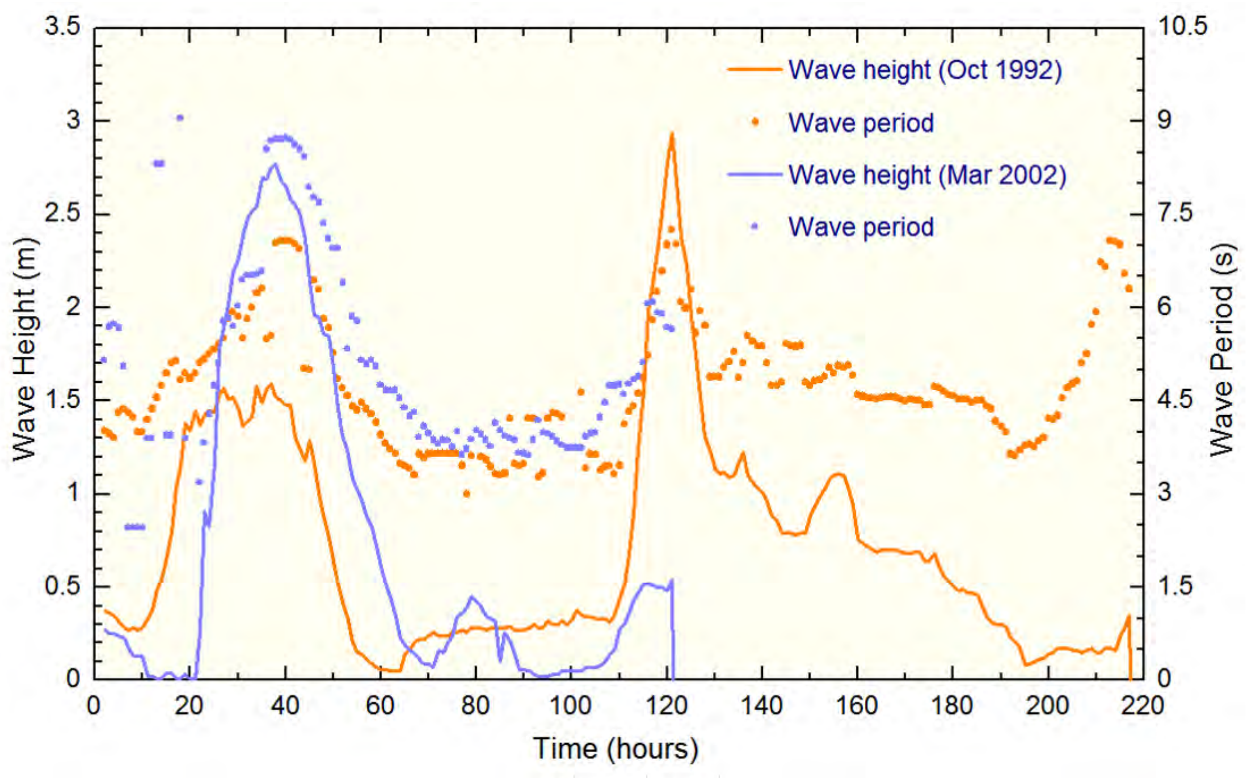


Figure 4.2: Hourly Time Series of Selected Three Storms

The approximate return period for the three selected storm events used in the COSMOS model is shown below.

- October 1992 (100-year event)
- March 2020 (100-year event)

Predicted toe erosion for all cases are summarized in Figure 4.3. COSMOS predicted up to 6 m (~20 ft) of toe erosion (or recession) under the 100-year lake level in combination with the October 1992 storm, which is the least likely modeled event based on return period. COSMOS also predicted up to 4 m (~13 ft) of toe erosion under the 2-year lake level in combination with the March 2020 storm. It should be noted that COSMOS assumes the lakebed material is non-erodible substrate (i.e. cobble and stone) and the material at the toe of the bluff is sand material with no cohesion. These results will be considered in slope stability analysis discussed in the next section.

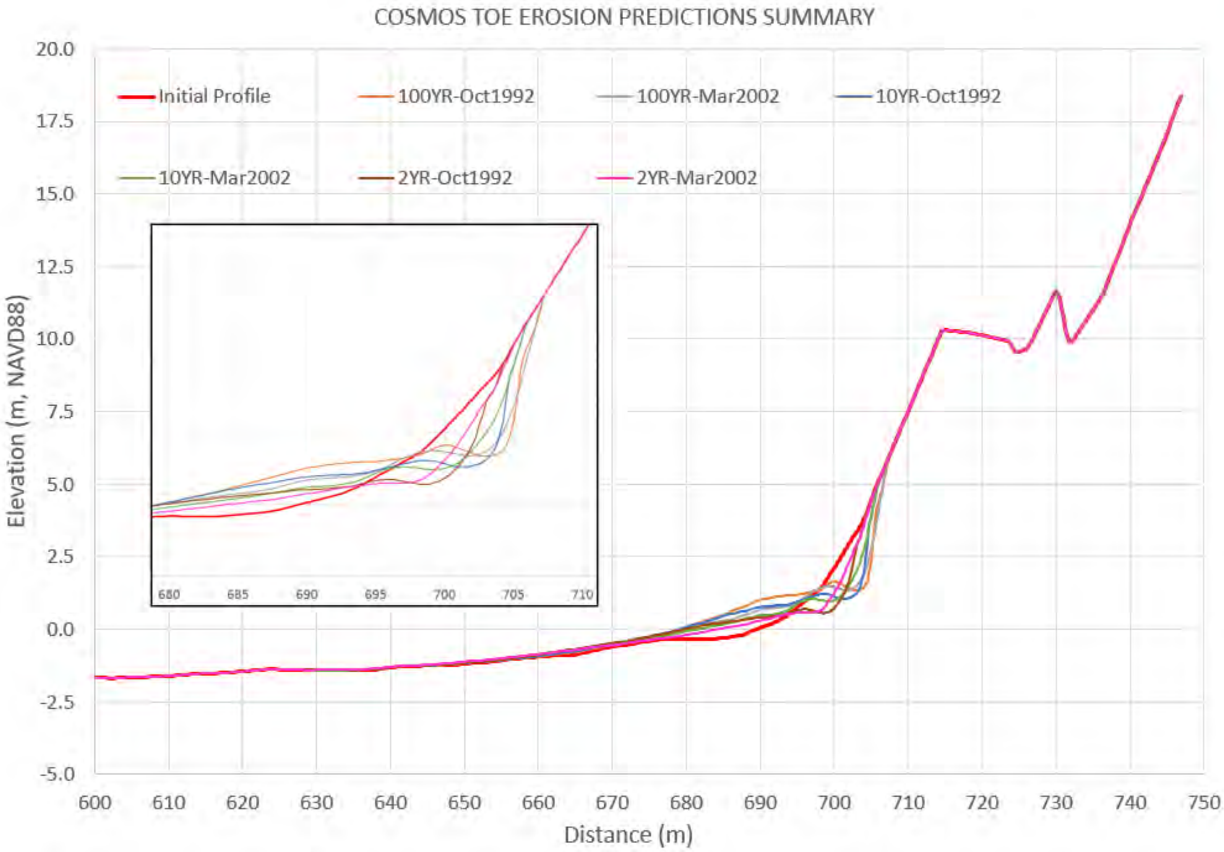


Figure 4.3: Predicted Toe Erosion for Nine different Scenarios (a close up of the toe area is shown in the inset figure)

A profile through the same location that was modeled (comparing the 2015 FEMA LiDAR with the 2020 Baird UAV survey data) shows approximately 12 m (~40 ft) of recession at the toe of the bluff, with patterns similar to that portrayed in the COSMOS results (i.e. minimal erosion/ change in lakebed elevation/ recession of the nearshore bluff toe), as shown in Figure 4.4.

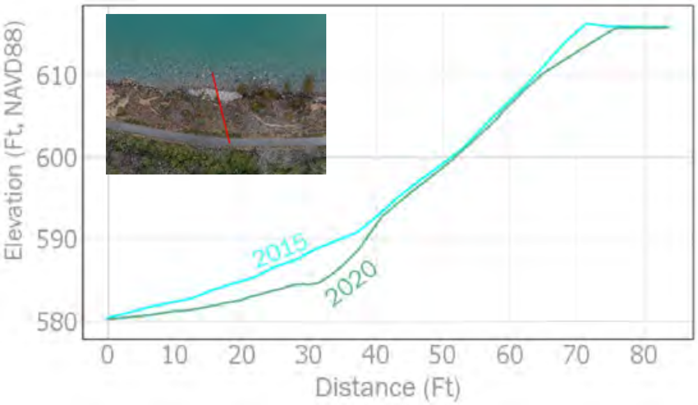


Figure 4.4: Survey Profile Comparison (2015 FEMA LiDAR and 2020 Baird UAV)

4.2 Slope Stability Evaluation

Two main types of slope movement were considered in our analyses:

1. Shallow surface slides; and
2. Deep-seated failures/ slides.

Shallow slides typically consist of relatively thin “veneers” of earth which move downslope. The movement may range from rather high velocity to a creep over several weeks, months, or years. The failure surface, or sliding surface, is typically parallel to the surface of the slope. Since failures of this type are commonly rather thin compared with their length, they are often treated as “infinite slopes” for the purpose of analysis.

The shallow slides are also often referred to as “nuisance” slides since they often do not do substantial damage at the time, and appear to be more a nuisance than anything. This colloquial phrase (nuisance slide) may be a rather dangerous misnomer at times, as even these relatively shallow slides may negatively affect the trail, an adjacent retaining wall, or other structural element in or near their paths. Furthermore, if shallow slides occur several times over the course of years or decades, the cumulative effects have greater potential to negatively affect a structure of even greater importance yet (ie. a residence, the trailway, stairways, etc.).

The deeper-seated slides have an obvious danger recognized by most observers in that a larger volume of earth/ soil may slide downslope. Much like shallow failures, this movement may occur slowly over the course of weeks, months, or years, or rather suddenly, as in the April 2020 slide. The deep-seated failures may be predicted using instrumentation (such as that recently installed by MDOT near the April slide site). It is MODT’s hope, for example, that the monitor wells and the inclinometers may serve to warn them (and the public) of subsequent, retrogressive failures; possibly jeopardizing that area of US-31.

Analyses pertaining to both types of failures (deep-seated and shallow slides) are addressed in Section 4.2.1 and 4.2.2, respectively.

4.2.1 Deep-seated slope failures

OHM used the computer software STABLPro by Ensoft for the slope stability analyses for the deep-seated mode of failure. Most slope stability approaches (there are many) use certain common parameters and ideas. Figure 4.5 shows a schematic of a typical “method by slices” type of slope stability analysis. In this figure, one example slice is shown in an expanded fashion to illustrate a typical ‘freebody’ diagram forces on that slice. Note that two types of forces (shear and normal, and/or side forces aligned at some angle off of perpendicular from the adjacent surface) are typically included on each surface, representing the stress conditions on each slice. The shear forces act parallel to a surface and the side forces typically act perpendicular to each surface, although these forces vary depending on the analysis approach used. The Bishop method has been shown to yield reasonable results, consistent with observed behavior and many of the more computationally rigorous methods, all while retaining the benefit of the operator being able to solve for the factor by safety by hand, when required.

The Bishop approach—like a number of commonly applied slope stability approaches used in practice—is a *limit equilibrium* approach. A limit equilibrium approach generally includes examining driving forces (those forces trying to pull a slope down) and resisting forces (those forces trying to keep the slope up, in its existing configuration). A profile under consideration is typically divided into discrete slices or sections, where each slice has a given area, and therefore weight. Additionally, each slice has a calculated friction force present at the base of that slice, acting at a given angle depending on the location of the slice bottom along the assumed circular failure surface. As a matter of the analysis method, we note that ascertaining displacements of a slope are not possible with this approach. Instead, a limit equilibrium analysis speaks more directly to a failure involving shearing the ground on some failure surface below grade. Therefore, the limit equilibrium method is not well-suited to estimating conditions necessary for failure types of very slow rate (e.g., slope creep over long periods of time). On the other hand, it does give an indication of a slope's general stability (with the actual rate of failure remaining unknown).

The minimum FOS desired for this application is normally between 1.3 and 1.5. Note that this FOS range does not guarantee that slope instability will be precluded. Rather, the FOS may be thought of as corresponding to a general risk assessment figure. For example, the greater the FOS, the less likely it should be for the FOS to drop below unity (i.e. 1.0), which represents the FOS suggesting a "failure". Part of the challenge of estimating the actual FOS relates to how well we are able to estimate soil parameters, the ground profiles, groundwater conditions, and other factors.

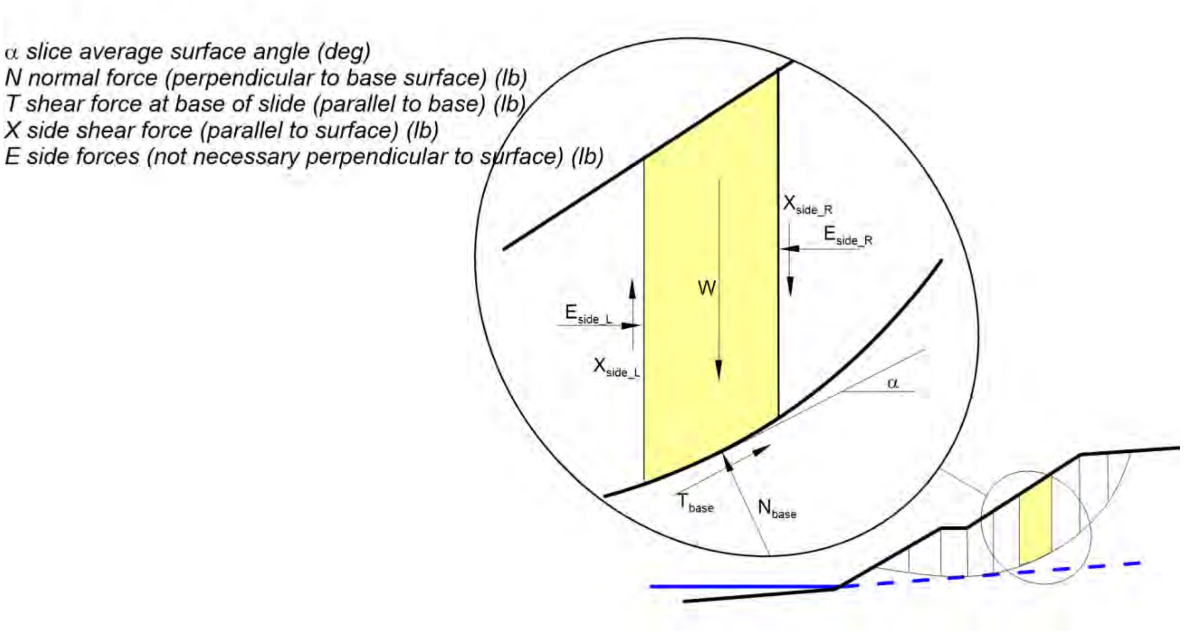


Figure 4.5: "Method by Slices" General Approach to Slope Stability Analysis

Using the April 2020 deep-seated failure surface as a guide (observed/measured failure geometry, current lake levels, etc.), OHM calibrated the soil strength parameters for use in the deep-seated slope stability analyses. It should be noted that this overall analysis also required judgements in terms of actual soil properties and parameters, and groundwater levels within the slope to define that surface which was believed to be present at the time of the failure.

Figure 4.6 was used as a means of calibrating the soil parameters including unit weight, internal angle of friction, and cohesion intercept (γ, ϕ', and c', respectively). For this trial, the FOS was forced to be near that of unity, as this analysis represents the pre-failure and post-failure slope geometry of the recent April 2020 slope failure. The ground topography and profile for the representative section (pre and post failure) were obtained using a combination of current UAV topographic information recently collected for this study, along with existing topographic information from the previous 2008 project for the area. For this initial trial, we used an assumed toe erosion of approximately 20 horizontal feet into the slope. This distance and geometry of the probable toe erosion is consistent with the erosion modeling described earlier in this report. We note that this particular parameter (toe erosion) was not directly observed by anyone leading up to the failure, and it represents an assumed condition-yet one that is probable at the time of the failure. We then assumed groundwater conditions based on MDOT's recent monitoring well readings in this area of the bluff. This parameter, too, is assumed on some level as the groundwater elevations recorded in July by MDOT may not be entirely representative of the groundwater conditions at the time of the failure in April 2020. A moist unit weight, γ, of 108 pcf; a saturated unit weight, γ_{sat}, of 120 pcf; a friction angle, ϕ', of 29 degrees; and a cohesion intercept, c', of 24 psf were used in subsequent analyses of potential deep seated failures for three representative profiles along the section of interest in this study. The profile locations are shown in Figure 4.7.

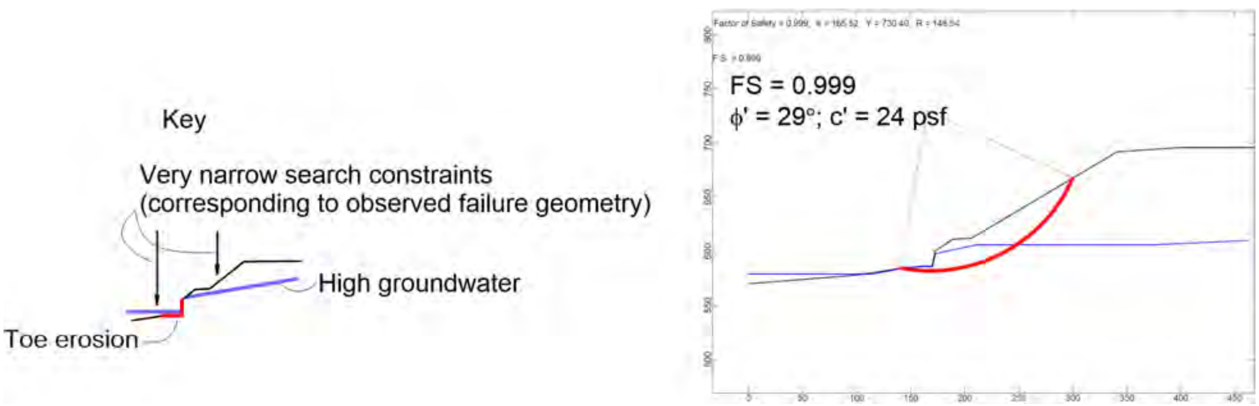


Figure 4.6: Slope Stability Model As Used for Calibration of Soil Parameters

Figure 4.7: Slope Stability Analysis Profile Locations

After the calibration of the ground parameters (results shown in Figure 4.6) using the recent failure as a back analysis, limit equilibrium analyses were conducted in forward analysis applications on the same original profile (Profile 1) and two others (Profile 2 and 3), but with each analysis featuring different conditions. Once again the purpose of this exercise was to compare the relative contributions to the FOS with each varying condition for the purpose of studying the main factors leading to this—and future—failures. An average lake elevation of 579 feet IGLD 1985 was used for all subsequent analyses. The results for each forward analysis (Profile 1 through 3) are summarized below.

Profile 1 Summary

Figure 4.8 (A) includes the Profile 1 slope geometry with an elevated groundwater level consistent with a wet season (i.e. during the spring or sometime shorter after, depending on lag time related to groundwater appearance, etc.). Toe erosion is included in this trial as well. In essence, this particular trial is intended to model the slope under rather extreme conditions to observe the FOS under these conditions. The FOS was 0.975; below unity, or the failure conditions (FOS=1).

Figure 4.8 (B) had similar conditions to that of Figure 4.8 (A), except that the ground water elevation was lowered. The FOS with this reduction in ground water elevation was 1.179; approximately 20% improvement in FOS from the analysis in Figure 4.8 (A).

Figure 4.8 (C) included the higher groundwater table, but without the 20 feet of toe erosion. From this trial, we see the FOS decreases to 1.121, yet still an approximate 15% improvement over the conditions in Figure 4.8 (A).

Figure 4.8 (D) did not include toe erosion, and the groundwater table was lowered. These conditions represent the most favorable in terms of slope stability. The FOS in this case was 1.193; about 22% greater than the condition in Figure 4.8 (A) where there was toe erosion combined with a high groundwater table.

We see that the elevated groundwater appears to represent a change in FOS (slope stability) by 6% to 20% (as compared with equal toe conditions), while the addition of the toe erosion seems to reduce or alter the FOS by between 1% and 15%, with the groundwater conditions remaining constant. It is clear that both groundwater position/ elevation and the presence of toe erosion play important roles in the stability of a slope. Notably, when the two occur simultaneously (elevated groundwater and toe erosion), the effect on the FOS is reduced further yet; near or below that of the failure condition (FOS=1).

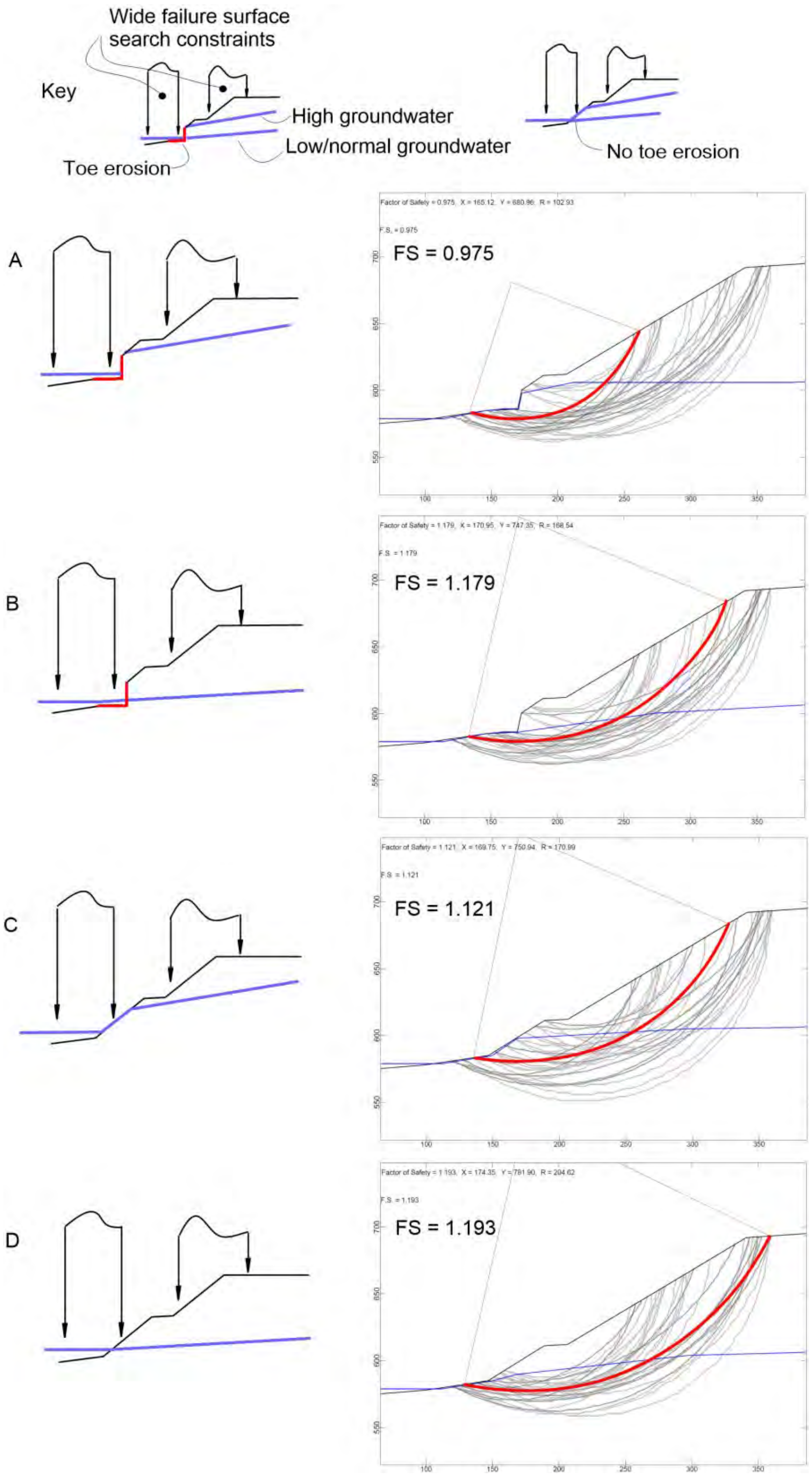


Figure 4.8: Profile 1 Slope Stability Analysis

We performed a similar suite of slope stability analyses on a different profile (Profile 2) as shown in Figure 4.9 (A through D).

Profile 2 Summary

Using the same soil parameters, groundwater condition, and toe erosion assumptions, this location also appeared to be relatively unstable. We note that even though many of the following factors of safety results indicate failure, this may or may not exactly match the observed condition of the slope. There is a possibility that the parameters, toe erosion, and/or groundwater conditions are different at that site. The main idea of this exercise is to take the same parameters and conditions and apply them to a slightly different slope profile geometry to observe the results.

In this case, the effects of the toe erosion individually, holding the groundwater constant (i.e. comparing Figure 4.9 (A with C) and Figure 4.9 (B with C)) yields a difference of about 10%. The effects of examining groundwater individually (i.e. comparing Figure 4.9 (A with B) and Figure 4.9 (C with D)) suggests a variation of around 30%. Once again, both of these contributing factors are important when considering the stability of a slope.

Finally, we examined Profile 3 in a similar way (see Figure 4.10 A through D).

Profile 3 Summary

This profile was rather unique in that it featured a rather prominent swale on the roadside of the existing trail. It is important to note that the critical failure surface (shown in red) was found to be on the upper part of the slope, likely due to the geometry and steepness of the ground in that area (see Figure 4.10 (B) and (D)). Figure 4.10 (A) and (C), on the other hand, exhibited a deeper-seated failure surface, extending from near the crest of the bluff to the beach area. This is a reminder that localized deep-seated failures should be considered along with the failures extending throughout the bluff face.

A comparison between Figure 4.10 (A) and Figure 4.10 (C), along with Figure 4.10 (B) and Figure 4.10 (D), to examine the effect of toe erosion only, suggested a difference between 4 and 7%. A comparison between Figure 4.10 (A) and Figure 4.10 (B), and Figure 4.10 (C) with Figure 4.10 (D), showed a difference between 2 and 5%. Interestingly, for this particular slope geometry, the toe erosion played a greater role than the groundwater elevation, although both features led to measurable differences in the stability of the slope.

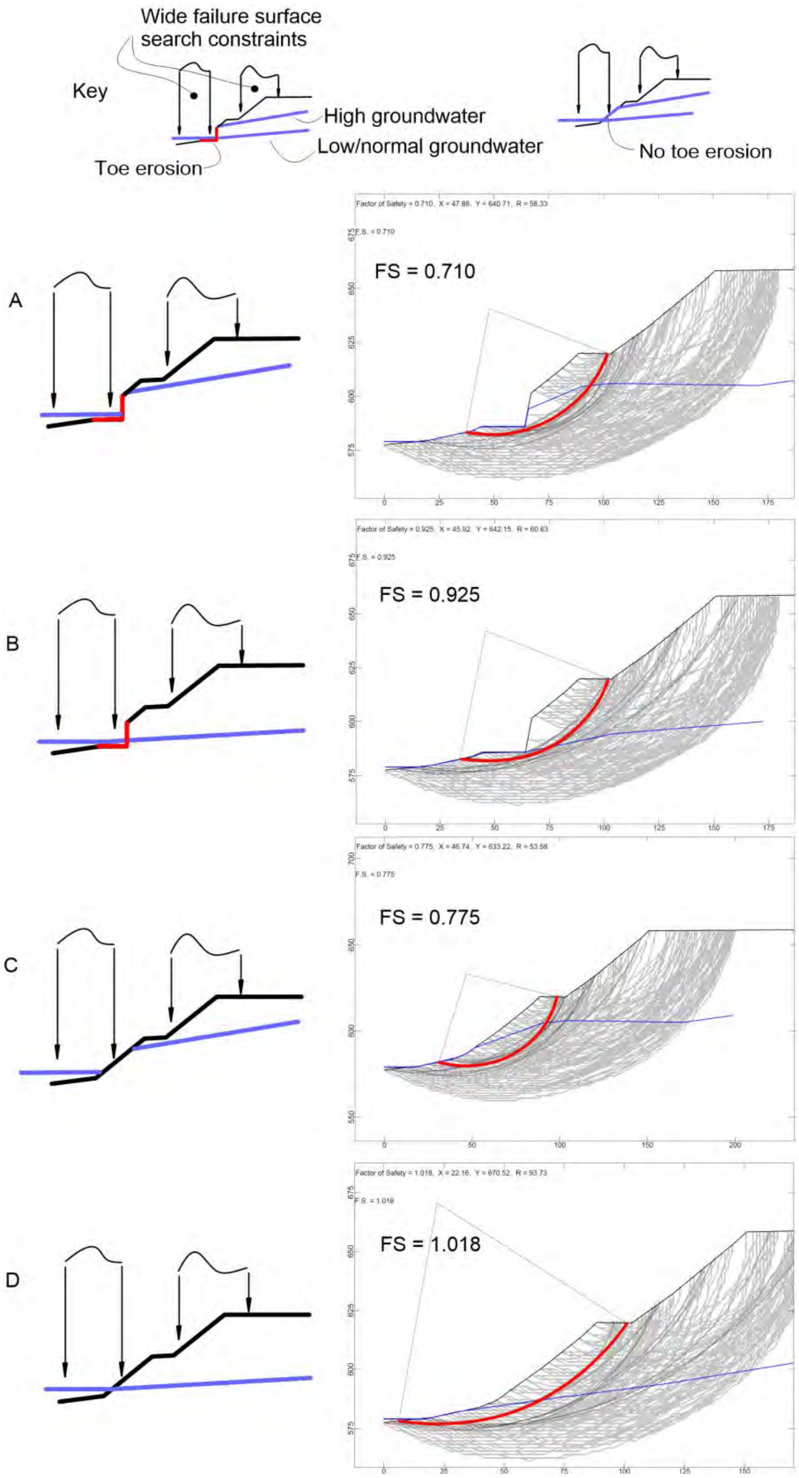


Figure 4.9: Profile 2 Slope Stability Analysis

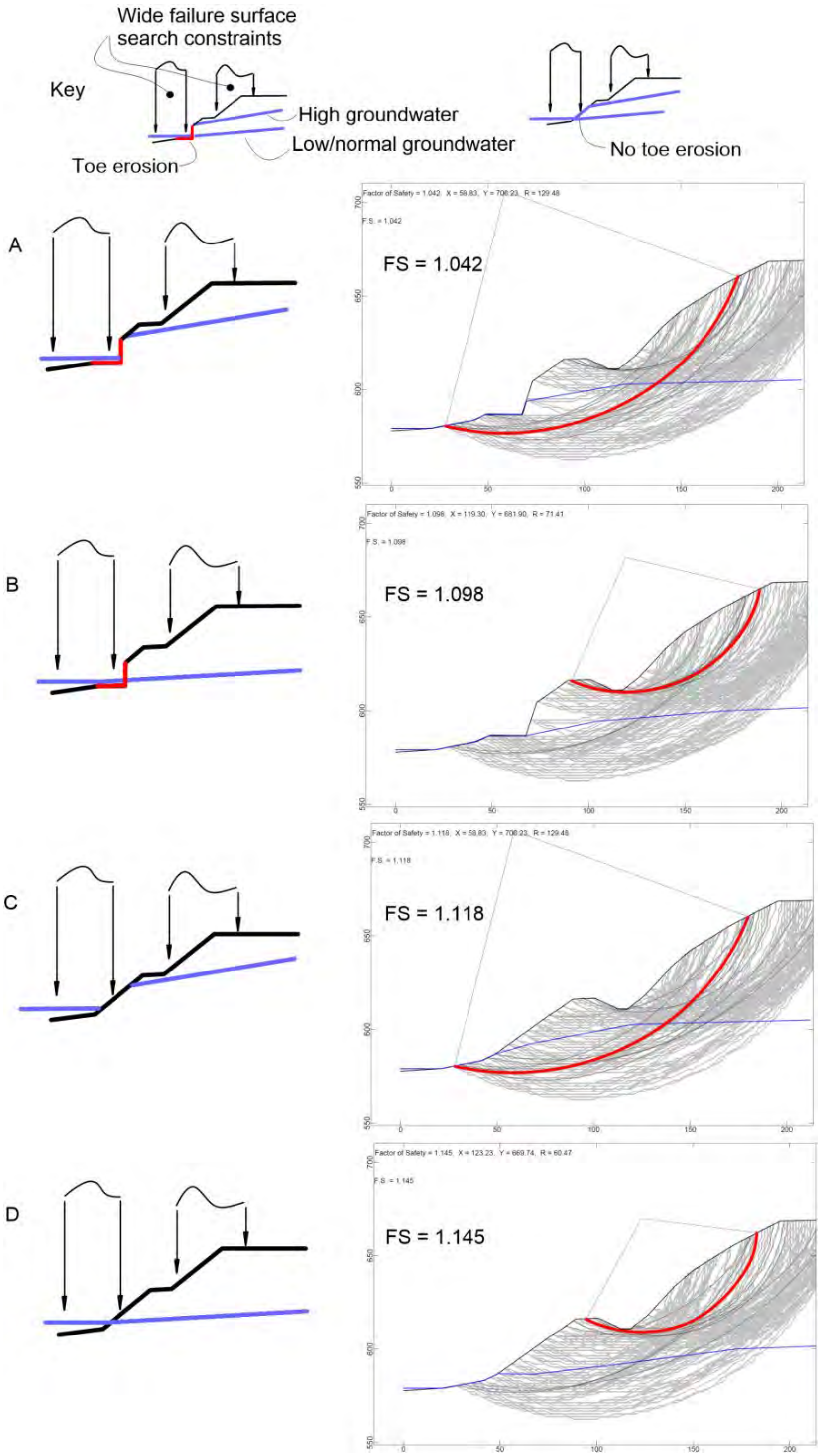


Figure 4.10: Profile 3 Slope Stability Analysis

4.2.2 Shallow Slope Failures

Although STABLPro may be adapted for use in evaluating both shallow and deep slides, it is often more convenient to utilize this software for slides of the deeper variety. The analysis approaches in STABLPro are better suited for circular failure surfaces. Shallow slides, on the other hand, may be evaluated using relatively simple equations as the surfaces of these slides may often be represented as a planer surface. In fact, the shallow slide analysis may often be evaluated using what is sometimes referred to as “infinite slope” analyses, with determining the tendency for sliding in a manner similar to calculating the angle at which a wooden block begins to slide down a flat table surface when elevated.

The causes and variables involved in the shallow slope failures were examined analytically for general conditions using a modified version of the infinite slope analysis. The infinite slope analysis used was modified to help take into account the effects of root systems within the ground. Root systems tend to add strength to the ground. Soil is mainly frictional material meaning that it derives its strength from the weight of the overlying soil at a point, multiplied by a friction coefficient, coming in the form of a “friction angle” parameter, ϕ’, similar in concept to the classical physics experiment of a block of wood sliding on a table surface. In that case, the more force you place vertically on the block, the greater the resistance of the block to sliding. A second strength parameter, seemingly independent of the overlying soil overburden weight is the cohesion intercept, c’. This parameter helps to provide a more complete soil strength “envelope” for a given analysis type such as these.

4.2.2.1 Soil Parameters

Table 4.1 includes two soil strength conditions: upper row with ϕ’ = 29 degrees and c’ = 10 psf; and the lower row with ϕ’ = 33 degrees and c’ = 10 psf. We note that ϕ’ is the internal friction angle of the soil, while c’ is the cohesion intercept. These parameters are commonly used in soil mechanics to describe the rate of increase of shear strength with increasing vertical effective stress. The greater the values, the greater the soil strength expected.

As mentioned previously, the strength parameters selected for the deep-seated failure models were initially based on reasonable parameters for the type of ground conditions observed in the soil borings along the bluff area (i.e. ϕ’ = 29 degrees and c’ = 24 psf). However, additional adjustments were made to the strength parameters (ϕ’, c’) for the soil type in the lower bluff (i.e. sand) according to several considerations, as described further next. The parameter ϕ’ defines the angle of the line, while the c’ parameter defines the value over zero along the y/vertical axis. Figure 4.11 (A) shows an example of the typical interpreted τ - σ’ relationship for a sand, with ϕ’>0° and c’=0 psf.

The strength parameters for a soil are intended to mathematically model the relationship between shear stress, τ, and effective normal stress, σ’; both in units of pressure (e.g., psf). The line—or strength envelope—is defined by the equation $\tau = \sigma' \tan\phi' + c'$, which takes on the form of an equation of a straight line. This particular equation, in the case of soil strength and slope stability, is used internally in slope stability analyses in order to define the anticipated shear stress, τ, for a given normal stress, σ’, for each of the “slices” defined for a particular failure geometry. This allows us to determine the resistance available, and ultimately the FOS for a slope failure condition.

However, in reality, the strength envelope is often slightly curved for any soil (see Figure 4.11 (B)). With this slight curvature, it becomes possible to interpret the ϕ’ and c’ parameters in different ways. As an example, for a σ’ value defined at Point X in Figure 4.11 (C), the ϕ’ and c’ parameters may be defined according to either Line 1, 2, or 3 (different parameters to define a strength envelope to arrive at the same shear stress, τ, for the same normal stress, σ’). Therefore, for a soil (sand or clay), the strength envelope may be defined in such a way that the c’>0 psf. That is, it is possible to arrive at the expected shear stress for a given σ’ with different ϕ’ and c’ parameter combinations. In this way, the presence of a c’>0 psf—whether it be assigned to a clayey soil or a sandy soil—becomes a mathematical convenience of sorts where the goal is to define the τ for a given σ’ that represents the behavior of ground undergoing a slope failure.

Also apparent (Figure 4.11 (D)), is the fact that with a slightly curved strength envelope, a different ϕ’ and c’ may also be interpreted at different normal stresses, σ’ (or overburden pressure in the ground at different places along the slope failure; some areas have little overburden pressure (Y) perhaps near the exit and entrance of an possible slope failure, while others near the center of the slope failure have greater overburden pressure, Z). A combination of ϕ’ with a c’ slightly greater than zero may often be a good pair to represent the available average shear strength along all points of the failure geometry. Additionally, including a slightly elevated c’ parameter may possibly model the shape of the failure surface closer to reality, as it is believed that the shallow overburden pressures may mimic real soil behavior a bit more closely.

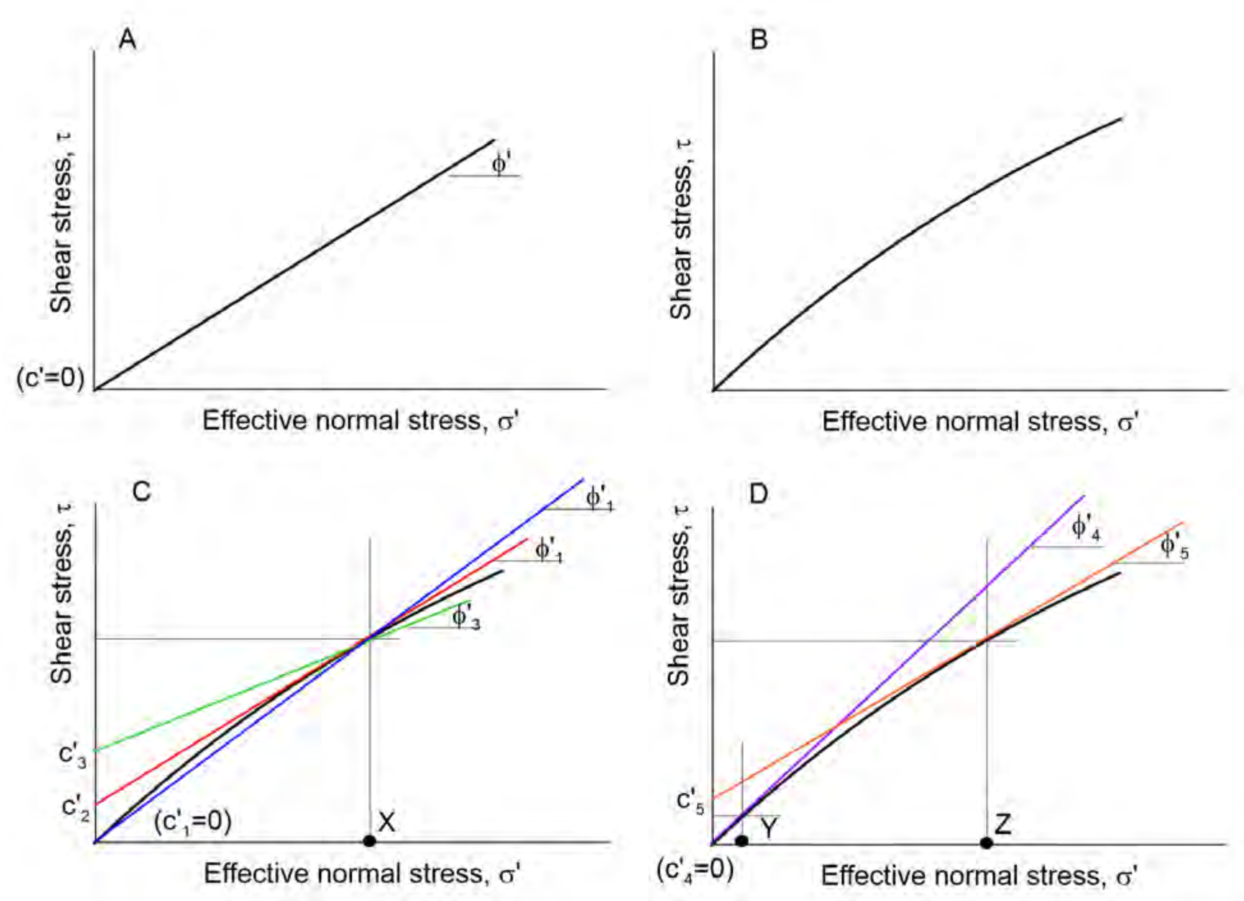


Figure 4.11: Soil Strength Parameters for Shallow Slope Analysis

4.2.2.2 Shallow Slope Failure Analysis

The US Forest Service infinite slope equation (1994) was used as a means to investigate sliding on the face of the bluff. The Cr parameter—tree/plant root strength—was one of the primary parameters of interest. In the case of an infinite slope analysis, a slice of soil parallel to the slope face may be stable at one time. Then, with some amount of toe erosion (perhaps near the beach due to wave action), that slice of soil would lose some of its ability to maintain equilibrium. The question that then comes in is whether the friction of the soil along the potential shearing plane has enough strength to stay in place. The plant/tree root “cohesion” contribution also becomes important, among other parameters. Figure 4.12 shows a depiction of the US Forest Service approach for reference.

$$FS = \frac{C_r + c' + \cos^2\alpha[q_o + \gamma(D-D_w) + (\gamma_{sat} - \gamma_w)D_w]\tan\phi'}{\sin\alpha\cos\alpha[q_o + \gamma(D-D_w) + (\gamma_{sat} \times D_w)]}$$

α slope & sliding surface angle (deg)
q_o surcharge due to trees/brush (psf)
D total soil thickness above the sliding surface (ft)
D_w saturated soil thickness (ft)
C_r tree/plant root strength (psf)
c' soil cohesion (psf)
ϕ' effective internal friction angle (deg)
γ moist soil unit weight (pcf)
γ_{sat} saturated soil unit weight (pcf)
γ_w unit weight of water (pcf)
FS factor of safety with respect to sliding

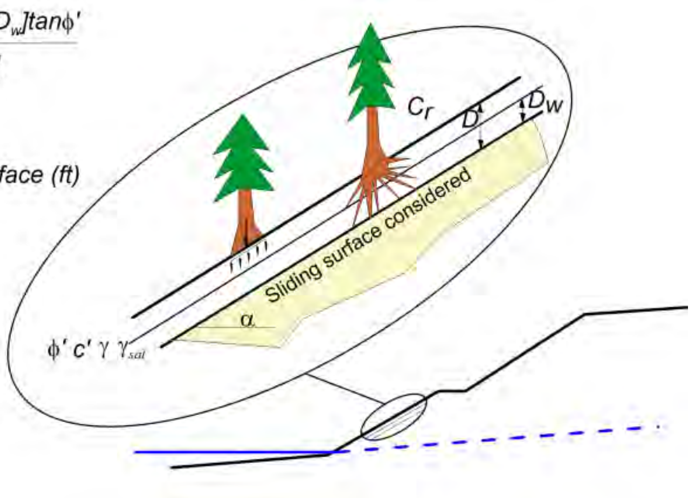


Figure 4.12: US Forest Service Infinite Slope Equation and Schematic

In reviewing existing literature on the subject, a reasonable C_r has been found to be on the order of 100 psf added to the contribution due to friction. We considered this strength contribution, along with a reduced root cohesion parameter of 50 psf to investigate the general effects on the FOS with respect to sliding.

The role of surface water (precipitation and run off from a variety of areas on the site to a point of interest) and groundwater add to the level of complexity of the stability of slope in an infinite slope mode of potential failure. For example, if the zone of potential sliding becomes saturated, with water filling a portion of the potential failure zone (and in particular, if the water seeps downward, parallel to the ground), the FOS with respect to sliding may be reduced dramatically.

Table 4.1 shows a series of trial analyses for shallow failures. There are two main rows; the top row of results correspond to a friction angle, ϕ’, of 29 degree and cohesion intercept, c’, of 10 psf for the soil, while the bottom row examines the effects on the stability if the ground strength parameters are 33 degrees and 10 psf for ϕ’ and c’, respectively. It should be noted that any number of trials and conditions may be run for future discussions. For now, the purpose of showing and briefly describing Table 4.1 is to demonstrate the most important factors in slope instability under this mode of failure, and to obtain a feel for the relatively contributions.

Note that the slope angle (also an important factor) for each of the example analyses is 33 degrees; equivalent to an approximate 1.5h:1v slope. This slope angle was observed at various areas along the bluff face during our site review.

We that note for the first row, when Cr is set to 100 psf and the tree surcharge is 10 psf for a 2 ft thick potential failure section, with no groundwater within that zone of potential failure, the FOS is 1.44. When the tree root cohesion vanishes, along with the tree surcharge pressure, the FOS falls to 0.91; indicating failure conditions. If the friction angle of the soil (as in the second row) is increased to 33 degrees (perhaps a better estimate of near-surface friction angle for these soils), the initial FOS is 1.69, dropping to 1.07 under the same conditions as described above; certainly a dramatic decrease, yet still slightly above sliding conditions. However, we must point out that even if the FOS is slightly above unity, the slope may experience a different type of movement, often called “creep.” Creep may be observed primarily by trees and/or power poles leaning over. In the case of trees, the creep often happens so slowly that the trunks of the trees may appear curved, while the tree continually tries to grow straight up. This creates a bowing effect of the tree. For this reason, it is beneficial to aim for factors of safety of about 1.5 or greater, where possible.

Table 4.1: Shallow Slope Failure Analyses Results

		Condition of slope, grounwater, roots system, etc.											
Soil strength parameters		"Base line" conditions as a benchmark		If slope becomes saturated with seepage parallel to the slope		With no tree roots		With lack of roots and seepage parallel to the slope		With lack of roots and 1/2 depth seepage parallel to slope		With partial roots and 1/2 depth seepage parallel to slope	
ϕ' = 29, c'=10psf	Root cohesion	C_r	100 psf	C_r	100 psf	C_r	0 psf	C_r	0 psf	C_r	0 psf	C_r	75 psf
	Soil cohesion	C'_s	10 psf	C'_s	10 psf	C'_s	10 psf	C'_s	10 psf	C'_s	10 psf	C'_s	10 psf
	slope angle	α	33 deg	α	33 deg	α	33 deg	α	33 deg	α	33 deg	α	33 deg
	tree surcharge	q_o	10 psf	q_o	10 psf	q_o	0 psf	q_o	0 psf	q_o	0 psf	q_o	5 psf
	moist unit weight	γ	108 pcf	γ	108 pcf	γ	108 pcf	γ	108 pcf	γ	108 pcf	γ	108 pcf
	saturated unit weight	γ_{sat}	125 pcf	γ_{sat}	125 pcf	γ_{sat}	125 pcf	γ_{sat}	125 pcf	γ_{sat}	125 pcf	γ_{sat}	125 pcf
	water unit weight	γ_w	62.4 pcf	γ_w	62.4 pcf	γ_w	62.4 pcf	γ_w	62.4 pcf	γ_w	62.4 pcf	γ_w	62.4 pcf
	vadose thickness	D	2 ft	D	2 ft	D	2 ft	D	2 ft	D	2 ft	D	2 ft
	saturated thickness	D_w	0 ft	D_w	2 ft	D_w	0 ft	D_w	2 ft	D_w	1 ft	D_w	1 ft
	friction angle	ϕ'	29 deg	ϕ'	29 deg	ϕ'	29 deg	ϕ'	29 deg	ϕ'	29 deg	ϕ'	29 deg
	Factor of safety		1.44		0.96		0.91		0.48		0.68		1.06
ϕ' = 33, c'=10psf	Root cohesion	C_r	100 psf	C_r	100 psf	C_r	0 psf	C_r	0 psf	C_r	0 psf	C_r	75 psf
	Soil cohesion	C'_s	10 psf	C'_s	10 psf	C'_s	10 psf	C'_s	10 psf	C'_s	10 psf	C'_s	10 psf
	slope angle	α	33 deg	α	33 deg	α	33 deg	α	33 deg	α	33 deg	α	33 deg
	tree surcharge	q_o	10 psf	q_o	10 psf	q_o	0 psf	q_o	0 psf	q_o	0 psf	q_o	5 psf
	moist unit weight	γ	108 pcf	γ	108 pcf	γ	108 pcf	γ	108 pcf	γ	108 pcf	γ	108 pcf
	saturated unit weight	γ_{sat}	125 pcf	γ_{sat}	125 pcf	γ_{sat}	125 pcf	γ_{sat}	125 pcf	γ_{sat}	125 pcf	γ_{sat}	125 pcf
	water unit weight	γ_w	62.4 pcf	γ_w	62.4 pcf	γ_w	62.4 pcf	γ_w	62.4 pcf	γ_w	62.4 pcf	γ_w	62.4 pcf
	vadose thickness	D	2 ft	D	2 ft	D	2 ft	D	2 ft	D	2 ft	D	2 ft
	saturated thickness	D_w	0 ft	D_w	2 ft	D_w	0 ft	D_w	2 ft	D_w	1 ft	D_w	1 ft
	friction angle	ϕ'	33 deg	ϕ'	33 deg	ϕ'	33 deg	ϕ'	33 deg	ϕ'	33 deg	ϕ'	33 deg
	Factor of safety		1.69		1.12		1.07		0.56		0.79		1.25

4.3 Shoreline and Slope Stability Analysis Conclusion

A series of slope stability analyses were completed. Some of these analyses were intended to model specific on-site conditions (i.e. deep-seated failures for Profiles 1 through 3), while other analyses were not intended for any specific area along the shoreline, but rather intended to be more general for comparison purposes (i.e. infinite slope analysis). Regardless of the type of analysis approach, the results helped to inform and guide our judgement about the mitigation measures developed (which are presented in Section 5). The main conclusions developed with regard to these analyses and research conducted are summarized below:

- Wave action in the Little Traverse Bay area is highly dynamic and is capable of quickly changing the configuration of the bluff toe. COSMOS modeling indicates that rather extensive toe erosion is possible along this section of shoreline (i.e. ~20 ft during 100-year lake level in combination with 100-year storm), which has also been validated along some areas of the shoreline to date by direct observation (see Figure 4.4).
- Toe erosion and groundwater conditions (elevation) are important factors in the stability of the bluff with respect to deep-seated failures along this section of shoreline. Under some modeled slope profile geometries, groundwater appears to play a larger role in the stability, while in other cases, toe erosion may be the biggest contributing factor regarding slope stability.
- High groundwater within the slope combined with the presence of toe erosion leads to the greatest amount of instability and chances of a deep-seated failure (worst case FOS). Conversely, where groundwater is maintained at a low elevation and the toe is protected, the stability of the slope is at its greatest for a given profile (best case FOS). The STABLPro modeled FOS results for these two conditions (worst/ best case) is summarized below for the three assessed profiles.
    - Profile 1 (0.975/ 1.193)
    - Profile 2 (0.710/ 1.018)
    - Profile 3 (1.042/ 1.145)
    - All modeled FOS results for deep-seated failures are either below or nearing unity (i.e. 1.0). The minimum desired FOS for deep-seated failures is 1.3 to 1.5.
- Shallow slope failures are also influenced by the condition/ presence and geometry of the toe. If the toe is compromised, the section of earth will depend on the internal strength of the ground, along with the presence and extent of tree/ plant/ bush root systems, and whether water is present. If surface or groundwater collects in a potential sliding zone and begins seeping downslope internally, the stability of the slope falls dramatically.
- The infinite slope analysis conducted for shallow slope failures considered two conditions for a 1.5H to 1V slope, the FOS results of which are summarized below.
    - Condition 1 (ϕ' = 29 degrees and c' = 10 psf):
        - Best Case (vegetated/ unsaturated topsoil): 1.44 FOS
        - Worst Case (unvegetated/ saturated topsoil) = 0.48 FOS
    - Condition 2 (ϕ' = 33 degrees and c' = 10 psf):
        - Best Case (vegetated/ unsaturated topsoil): 1.69 FOS
        - Worst Case (unvegetated/ saturated topsoil) = 0.56 FOS
    - For Condition 1 and 2, lack of vegetation root systems and saturated topsoil conditions reduce the FOS well below the minimum desired FOS for shallow slope failures (1.5).

In understanding shoreline and bluff geomorphology from elsewhere along the Great Lakes, we have observed, once again, that these shorelines are generally dynamic, prone to erosion, and ever-changing systems. This bluff has been eroding and changing since its formation through wave and wind action, and gravity constantly pulling the earth downward to a lower energy configuration (continually attempting to make the area flatter, overall). Although these processes will continue, certain mitigation approaches may help control these phenomena to allow for continued use of the shoreline for the enjoyment of the residents of the City of Petoskey, Resort Township, and Emmet County, along with their visitors to the area.

The results and conclusions presented in this report supersede any previous project correspondence. Potential mitigation measures are discussed in the next section of this report. These measures have been developed specifically to counteract the detrimental effects of toe erosion and elevated groundwater levels, along with measures promoting slope stability and safety of the area by additional means.

5. Potential Mitigation Measures

5.1 Overview

During the course of the various analyses, potential shortcomings of the existing bluff in terms of its stability were identified (i.e. toe erosion potential, ground water elevation, slope saturation, and vegetation impacts). These observations led to the development of several preliminary remedial work options, each of which may help improve the stability of the bluff in different ways.

There are two main modes of slope instability that were presented in Section 4.2: 1) relatively shallow “surface” slides, and 2) potential deeper-seated slope failures. A high-level summary of potential mitigation measures for the Owner’s consideration for addressing the slope stability for either of these two main modes of failures is provided in Table 5.1. The items in this table have to do with either:

1. Remedial repair of affected areas to date,
2. Mitigation measures to promote greater stability on areas that have not yet experienced apparent instability, and/ or
3. Monitoring approaches.

5.1.1 Remedial Repair of Affected Areas to Date

As previously discussed, numerous areas of the slope have experienced on-going shallow surface sliding and movement. Other areas have experienced deeper-seated failures, including the most recent in April 2020, along with another past failure in 1913.

For shallow failures (those that have experienced sliding or are actively sliding at a slow rate), a number of mitigation techniques are available to consider in arresting the downslope movement (refer to Table 5.1).

5.1.2 Mitigation Measures to Promote Greater Stability

Here, again, two major modes of earth movement—shallow and deep slides—are of interest. The following sections discuss each type of slide in more detail.

The areas of the slope along the shoreline not currently experiencing signs of shallow or deep slides may be strengthened with a number of mitigation techniques. It must be cautioned that it is often challenging to know, with certainty, where such slides will occur. However, two approaches may be taken in addressing instability in these areas: treat/strengthen the entire slope shoreline and face, and/or identify discrete sections of concern along the bluff face. One consideration may be the relatively importance of an area, or the level of potential damage should a failure or slope movement occur. Note, the schematic designs developed for this project area focus on addressing specific areas within the project battery limits where ongoing issues were identified during this study.

5.1.2.1 Shallow slides

Areas of the bluff face not currently experiencing sliding may be subject to earth movement in the future. It is often difficult to identify these areas. On the other hand, a survey with the intent of identifying relatively steep areas may be helpful in prioritizing areas for mitigation.

During our study, we observed that all of the shallow slides observed occurred on the lakeward edge of the trail; near the shoreline. This suggests to us that wave action and erosion near the beach elevation and area may be an important contributing factor. Conversely, we did not observe obvious signs of shallow slides on the roadside of the trail; well upslope of the beach. We note that it is certainly possible for shallow slides to occur anywhere on the slope, but when wave-related erosion is not a factor, the slope generally appears to be more stable.

Certain areas of localized steepness (i.e. in the areas of the existing swales adjacent to the trails) are believed to be areas of interest in the context of likely shallow slide areas (see Section 5.4 for more discussion).

Table 5.1 provides potential measures for shallow slides mitigation and/or prevention.

5.1.2.2 Deep slides

Primary factors related to whether a deep-seated failure is imminent along the bluff face include: 1) groundwater location/elevation, 2) condition of the toe near the beach, and 3) geometry/steepness of the slope. Several secondary factors contribute as well (i.e. surface water management, existing and condition of the swales, surcharge loadings on and near the bluff crest and/or on the face of the bluff).

The mitigation measures developed very often are intended to address one of the primary factors, but also may be introduced into the bluff face to strengthen the existing ground (i.e. soil nailing).

Table 5.1 provides potential measures for deep slides mitigation and/or prevention. Figure 5.1 shows a number of example mitigation approaches for a given section along the bluff (Profile 2). The purpose of Figure 5.1 is to help illustrate relative improvements for various mitigation approaches. Below is a description of the various options evaluated:

- Figure 5.1 A shows a “baseline” of sorts in that this slope includes no toe erosion and a rather high (internal) groundwater table. The level of the bay in this example is also elevated to the approximate current levels. The FOS with respect to a relatively deeper, circular failure is 0.904, indicating failure conditions (FOS<1).
- Figure 5.1 B shows the change in the FOS when a buttressed zone is added near the beach area. In this case, the FOS increases to 1.079; a value that is now greater than unity. This is an approximate 20% increase over the baseline conditions (Figure 5.1 A).
- Figure 5.1 C features the same conditions as in in the baseline figure (Figure 5.1 A), with the exception of some grading applied to the upper part of the slope. In this case, that area of the slope has been flattened to some degree. The FOS become 0.912; only about 1% improvement over the baseline condition (Figure 5.1 A).
- Figure 5.1 D includes the installation of soil nail elements, driven or installed to about 30 to 40 feet at the angle shown. Notice that the buttress and the slope flattening measures have both been cleared, as to compare this latest mitigation measure (soil nails). The FOS under these conditions was 1.022; an approximate 13% improvement over the baseline conditions (Figure 5.1 A).
- Figure 5.1 E includes a combination of a toe buttress and upper slope flattening/re-grading. In this scenario the FOS is 1.167; nearly a 30% increase over the baseline condition (Figure 5.1 A).
- Figure 5.1 F includes the original slope geometry, with no other mitigations measures other than horizontal drains to lower the internal groundwater elevation. With this adjustment, the FOS increases to 1.169 (about 30%) over the baseline condition (Figure 5.1 A).
- Finally, Figure 5.1 G includes all four of the mitigation measures discussed at various times above (toe buttress, upper slope flattening, soil nails, and horizontal drains near the toe of the slope) as a means to observe the effects of all measures existing together at once. In this case, the FOS increases to 1.544; approximately 70% more than the baseline conditions (Figure 5.1 A).

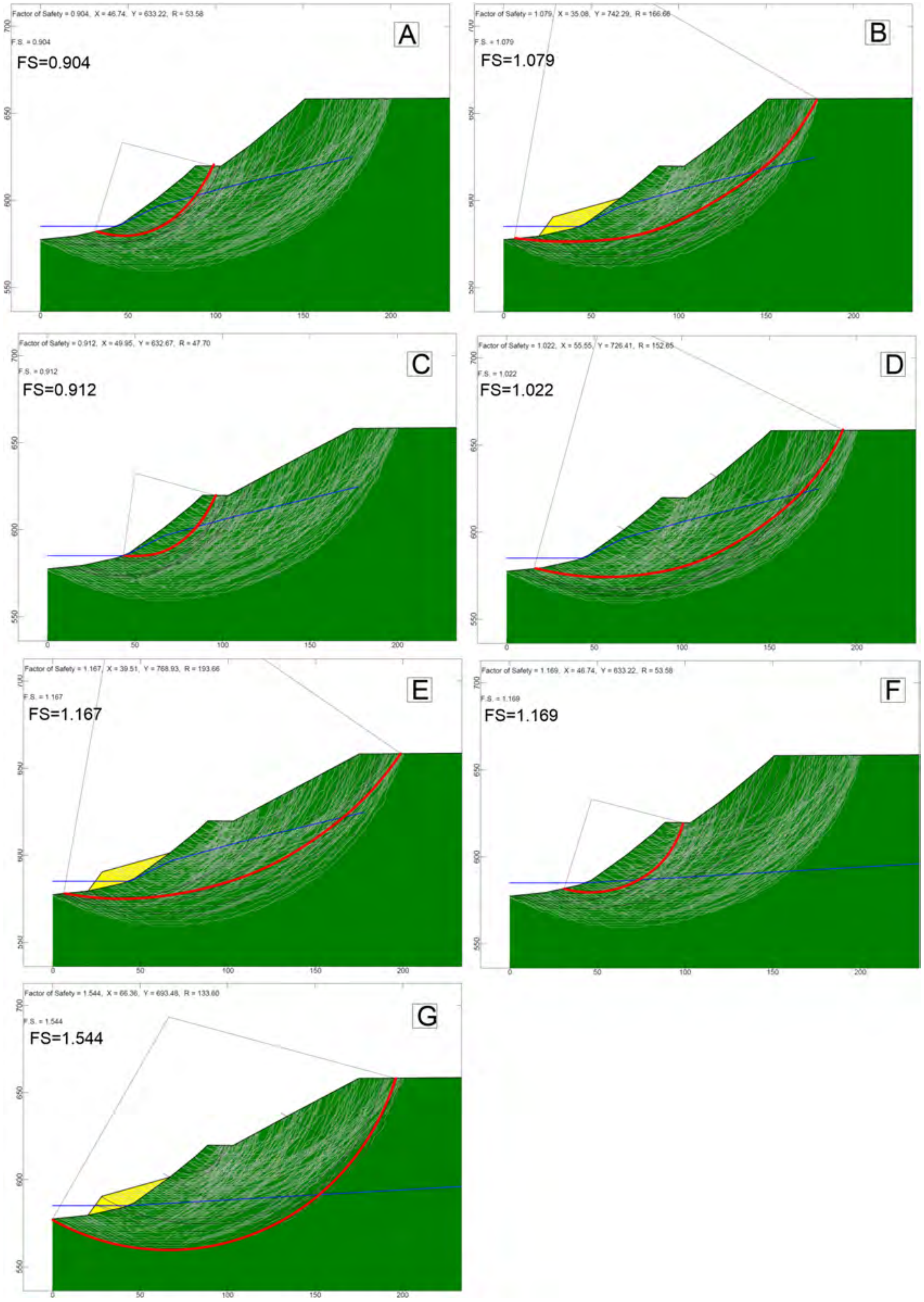


Figure 5.1: Potential Mitigation Measures for Deep-seated Failures

A second round of analyses was undertaken to investigate the efficacy of some more specific mitigation measures that with the failure that occurred.

- Figure 5.2 (D) shows the inclusion of a cobble beach along the shoreline, regrading of the nearshore bluff slope, rebuilding the trailway at a lower elevation, and regrading the area of upland bluff that failed. Said improvements increase the FOS to 1.278.

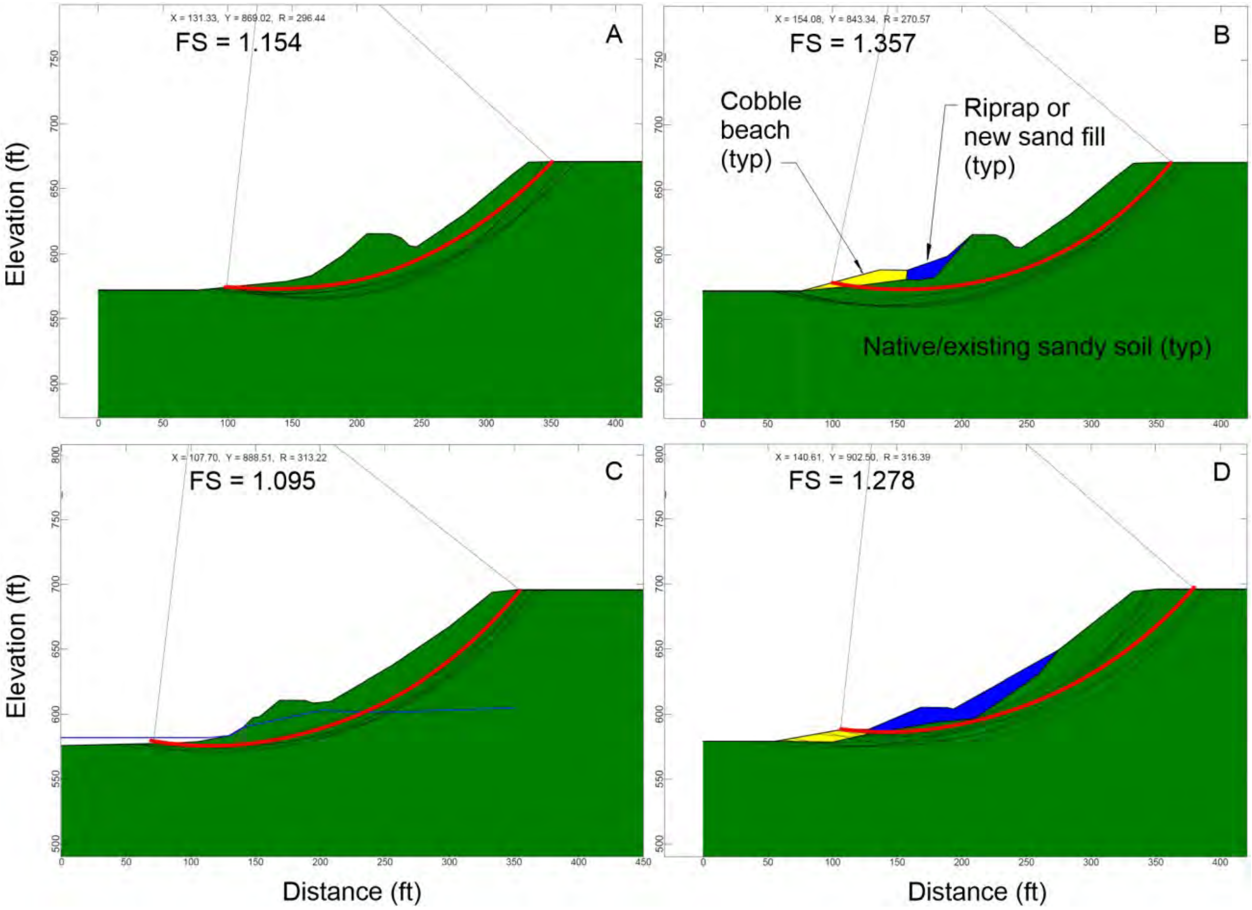


Figure 5.2: Mitigation Measures for Preliminary Site-Specific Purposes

5.1.3 Monitoring Approaches

Monitoring approaches generally include one or more of the following: 1) visual observation and logging of the ground, and 2) groundwater monitor wells, and 3) inclinometers installed at various locations on the bluff face. This list is not exhaustive and there may be others to consider.

5.2 Current Surface Drainage Observations

In further developing our analyses, certain areas where additional information may be useful were identified. One such area is that of the regional and/or perched groundwater tables possibly present in the vicinity of the trail. We used limited existing information concerning the regional ground water with respect to how those levels may affect the slope stability in the area. However, it is widely known that groundwater table elevation(s) may vary with time and season; both on an annual basis as well as cycles over the course of decades. For this reason, we suggest that a hydrogeologic study may be of benefit as long term solutions to stabilizing the slope are explored.

In the context of this site and needs for the study, we anticipate that a hydrogeological study would consist of the installation of a number of groundwater monitor wells throughout the area (not necessarily within the area of the slope or the slope movement). In a comprehensive hydrogeological study, as an example, monitoring wells (perhaps similar to those recently installed by MDOT) would be installed at areas including near the crest of the slope, near the existing elevation of the trail, and possibly in areas on the south side of US-31.

The goal of a detailed hydrological study would be to understand if there is a relationship between regional hydrology (including precipitation) and local ground water levels. This would be very important when interpreting the well monitoring data recently collected by MDOT. It is also possible that local ground water elevations are sensitive to swale features (located in the mid-slope of the bluff along the landward edge of the trailway), as shown in Figure 5.3. Installing additional ground water monitoring wells in the swale areas is recommended so that this data can be assessed and compared with other non-swale areas. The goal of this study is to better understand the issues causing high ground water levels in the slope (i.e. whether it is a regional or local phenomenon) as our slope stability modeling indicates the overall stability of the slope is very sensitive to ground water elevation.

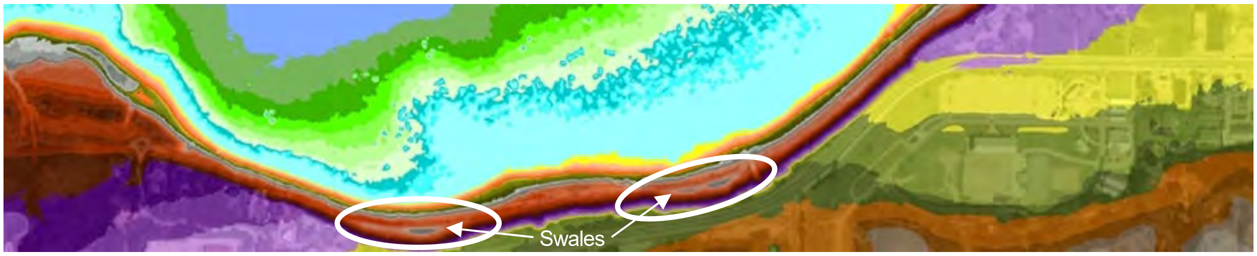


Figure 5.3: Mid-slope Swale Location

As indicated in the table below, there are numerous potential mitigation measures that could be explored to address the ongoing issue, with varying costs. Our team developed initial design alternatives based on our findings to address areas experiencing ongoing issues related to shoreline instability and erosion. It is important to note that the alternative developed for the study are schematic in nature and additional data collection and study (as previously discussed) is recommended to advance these initial design alternatives.

Table 5.1: Summary of Potential Mitigation Measures

Option	Category[3]	Description	Advantages	Disadvantages	Cost[4]
Toe revetment	RR/ MM	Stone revetment installed near the toe of the slope to provide both protection from wave action and/or to provide a heavy mass of earth materials to help further stabilize the slope.	Commonly used approach/widely accepted as a solid practice for stabilizing shoreline position. Helpful for both shallow and deep-seated failure concerns.	May not necessarily help solve localized/ ongoing failures well upslope of the toe, and construction access may be challenging.	H
Cobble Beach	RR/MM	Develop an expansive cobble beach along the shoreline to dissipate incoming wave energy and buttress the slope toe.	Maintains/ improves public shoreline access and common to this area. Helpful for both shallow and deep-seated failure concerns.	Requires large volume of stone material to provide adequate beach width and crest elevation.	H
Re-grading slope	RR/ MM	Slope flattening either on the bayside of the trail and/or on the roadside of the trail intended to improve overall stability of the slope overall or in localized zones.	Relatively common, construction equipment needed is readily available in the area. Helpful for both shallow and deep-seated failure concerns.	May involve a large amount of earthmoving, and construction access may be challenging.	H - VH
Trail elevation	MM	Alter (raise/lower) existing trail elevation in various areas along alignment to arrive at a slope geometry with greater stability.	Requires relatively common equipment available in the area.	May involve a large amount of earthmoving, and construction access may be challenging.	M-H
Surface drainage	MM	Construct controlled drainage elements (riprap-lined drainage ways, French drains, etc.) to control surface water/ precipitation as it seeks to reach the level of the lake.	Relatively common best management practice for storm water runoff. Helpful for both shallow and deep-seated failure concerns.	May require numerous drainage paths/areas along the trail adding to cost. Additional study required to determine locations needed.	M - H
Under drainage by gravity	MM	Construct/install horizontal drains near the toe of the slope to permanently draw groundwater down in the area.	High groundwater may be prevented, and water may be drained by non-mechanical means (by gravity).	Site access near the toe (for the equipment anticipated) may be limited, rather costly, there is a chance the groundwater will be lower than the	H-VH

---

[3] Category reference: RR = Remedial Repair; MM = Mitigation Measure; M = Monitoring.
[4] Comparative CAPEX cost reference: L = Low; M = Medium; H = High; VH = Very High.

Option	Category[3]	Description	Advantages	Disadvantages	Cost[4]
				drainage elements. Addresses mainly deep-seated failures.	
Under drainage by mechanical means	MM	Install vertical groundwater wells with submersible pump intended to permanently lower groundwater in the vicinity of the face of the slope.	Wells may be installed to very deep levels; well below the level of the bay, which may help lower the groundwater table further yet than the gravity system described above.	Requires electricity during the times of operation. May negatively affect existing groundwater, existing wells in the area. Addresses mainly deep-seated failures.	H-VH
Soil nails/ ground reinforcement	RR/ MM	Install steel elements in the ground to provide additional strength along a would-be failure surface to add stability to the slope.	These types of elements have been shown to add significant strength to a slope. May be applied to both shallow and deep failures.	Specialized equipment/contractors typically necessary, often relatively costly. More costly for application to mitigation of deep-seated failures.	H-VH
Vegetation	RR/ MM	Plants specifically intended to have exceptionally long root systems, native to the area, to help repair current slide areas and/ or other others area of concern (either before construction activity or soon after over bare soil).	Relatively easy to plant, may possibly be accomplished with groups of volunteers, roots for some plants may extend to five feet and greater.	May only be beneficial for relatively shallow slides, large area of planting potentially required. Helpful only for relatively shallow failures.	L-M
Groundwater Monitoring wells	M	Install additional groundwater monitoring wells to serve as a warning of high groundwater, and impending slope instability.	Monitoring wells offer reliable groundwater depths/ elevations. Groundwater has an important effect on slope stability (i.e. solid correlation to slope stability).	Although only moderately costly to install, they may foul up/fail over time. Some maintenance may be required. If remote measurements are taken, up-front costs (CAPEX) may be high. Helpful primarily for deep-seated failure mitigation.	M-H
Inclinometers	M	Install additional inclinometers in various area either believed to be prone to shallow or deep-seated failures and/or areas of special concern near structures.	May help warn of an impending failure well before the failure occurs, allows for real-time information about slope displacement.	Relatively costly to install and obtain measurements. If remote measurements are taken, upfront cost may be high. Often only used for deep-seated failures.	M-H
Visual inspection	M	Regular visual inspection along trail area, slope face, and crest noting evidence of tension cracks in the ground.	Relatively simple and cost effective to carry out with some minimal training for those involved. Beneficial for shallow and deep-seated failures.	Logging observations and maintaining records requires some level of time and managing to be effective. Surface features may not be an indication of actual internal stability.	L

5.3 Schematic Design Alternatives

Two initial design alternatives were developed for this study, as shown in Figure 5.4 and Figure 5.5.

A description of each is provided in Table 5.2 below:

Table 5.2: Initial Schematic Design Alternatives

Design Alternative	Description
Option1	This design alternative incorporates a continuous stone revetment structure along the shoreline to protect the toe from ongoing erosion associated with coastal processes. Regrading/ revegetating of the lower portion of the bluff (from the trailway to the revetment) is proposed for areas experiencing ongoing shallow failures. Similar improvements are proposed near the recent trailway damage and coastal bluff collapse (i.e. stone revetment and regrading of the nearshore slope), as well as the addition of cobble beach material along the shoreline (near the termination points of the revetment fronting the recent collapse). The elevation of the reconstructed trailway is lowered to reduce fill associated with the proposed repairs. Drainage infrastructure is also proposed within two existing swale areas located on the mid-slope of the bluff.
Option 2	This concept incorporates similar improvements along the shoreline for the trailway reconstruction and drainage infrastructure. The main difference between the two options is that Option 2 proposes constructing a continuous cobble beach along the shoreline (as opposed to a shoreline revetment). The cobble beach would provide the added benefit of maintaining/ improving pedestrian shoreline access throughout this reach of shoreline.

A complete preliminary drawing set for each option is provided in Appendix B.

Additional slope stability infrastructural improvements (i.e. under drainage by mechanical and/ or gravity means, and soil nails/ ground reinforcement, discussed in Table 5.1) – which are less commonly applied for this particular type of slope stabilization challenge – were not proposed in either option because the slope stability modeling results for shore-based measures alone (as shown in Figure 5.2) increased the FOS for deep-seated failures to (1.357 and 1.278), which is within/ nearing the approximate recommended FOS for a stable slope (i.e. 1.3 to 1.5). Regrading, drainage infrastructure, and establishing vegetation is also proposed for these design alternatives to resolve ongoing sloughing of the bluff. Additional infrastructural improvements (in addition to the currently proposed mitigation measures) could be considered to further increase the FOS for specific location, as needed. However, additional study is required to determine specific location/ extent of said infrastructure.

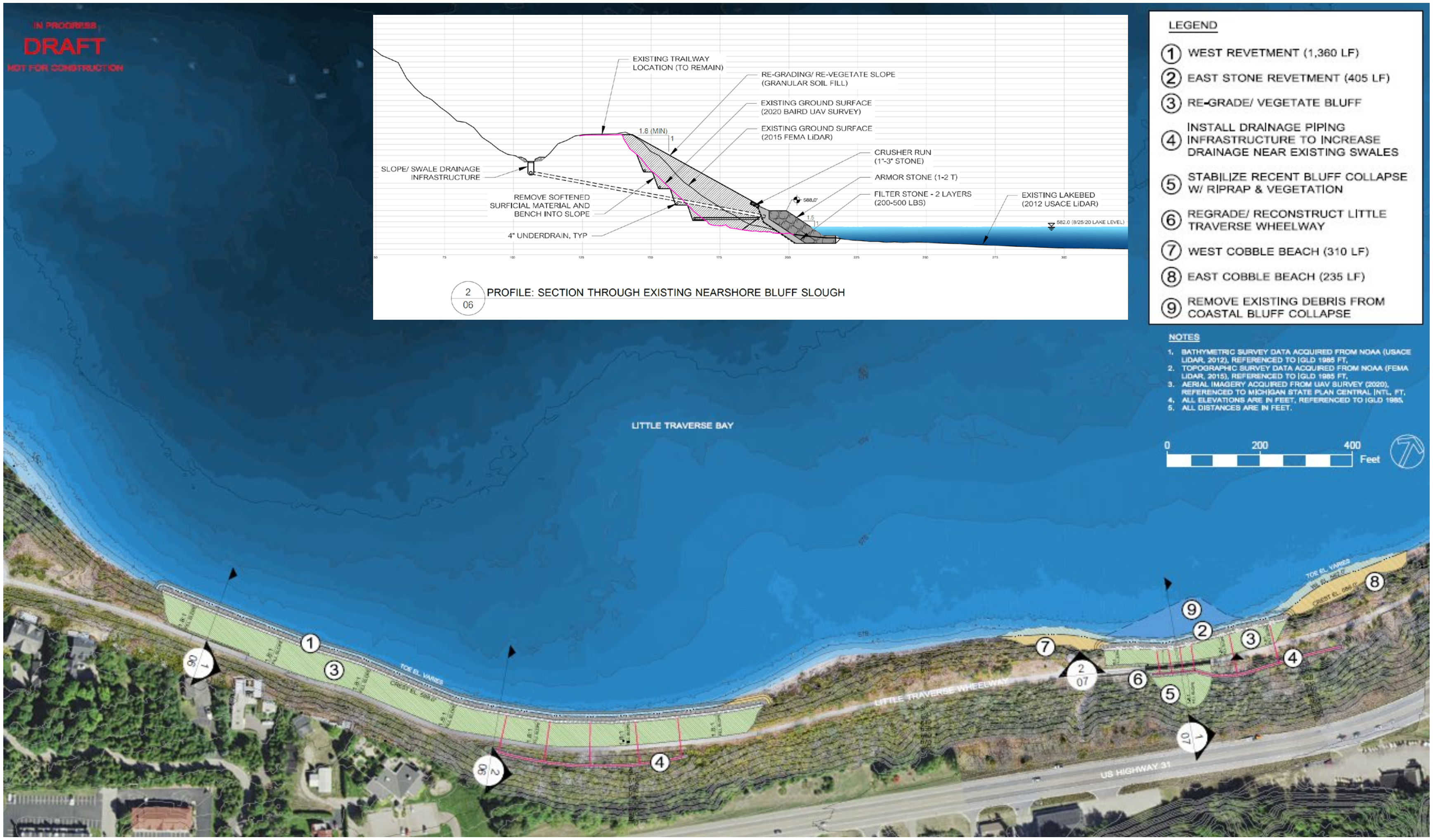


Figure 5.4: Initial Schematic Design (Option 1)

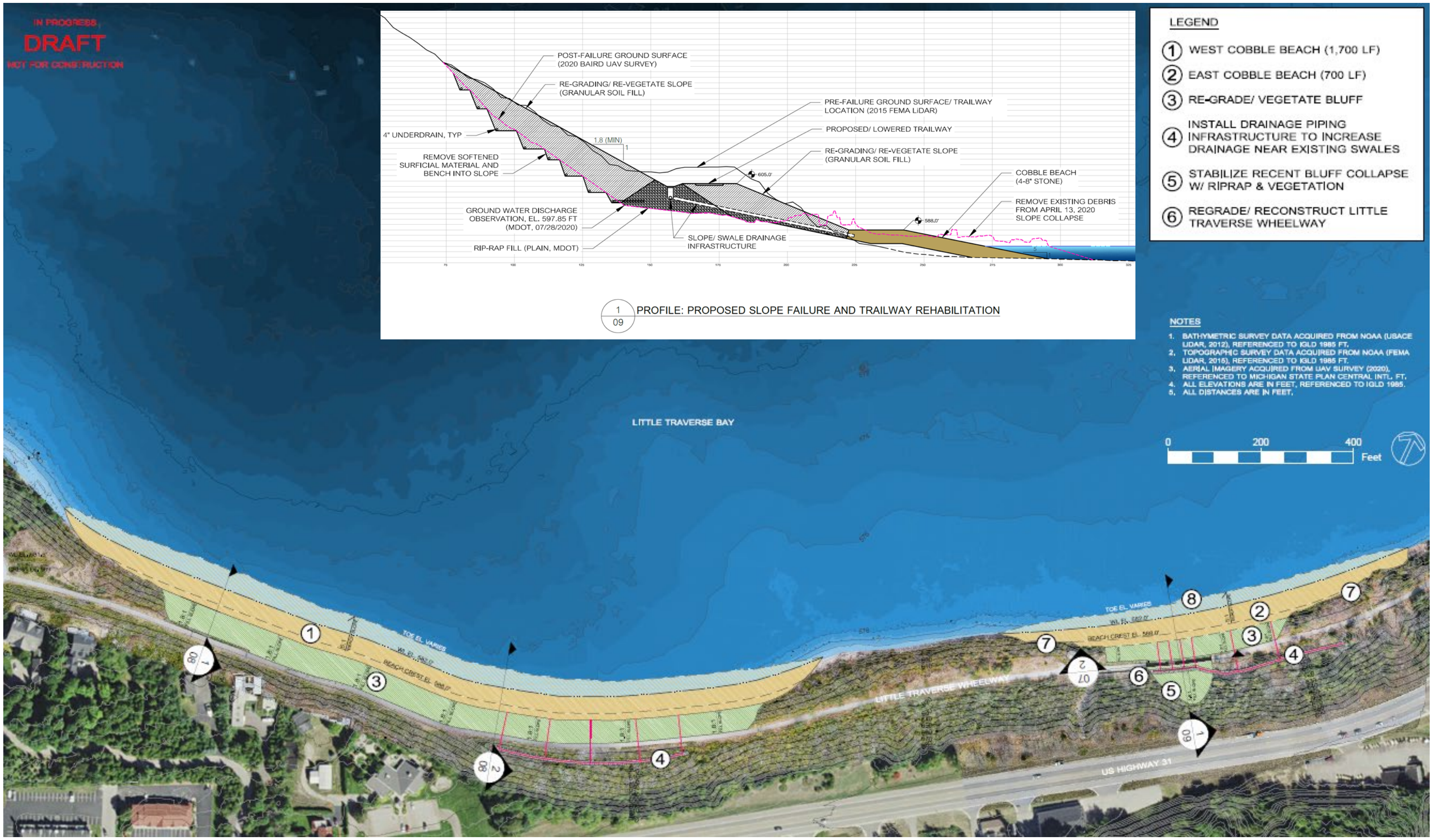


Figure 5.5: Initial Schematic Design (Option 2)

5.4 Opinion of Probable Construction Costs

An Engineer's Opinion of Probable Construction Cost (OPCC) was developed for each schematic design alternative (Options 1-2) to conduct a high-level order of magnitude, comparative feasibility assessment. The OPCC's are deemed Class 5 estimates, per the AACE International Cost Estimation Classification System based on the schematic nature of the design concepts, thus upper range (+30%) and lower range (-20%) variations have been provided for each OPCC. Itemized unit rates were developed based on coordination with local contractors and material suppliers, construction crew-based cost estimation software (MCASES MII), and Baird's in-house cost database. Construction material volumes were developed utilizing 3D CAD software (Autodesk Civil 3D).

OPCC summaries for Option 1 and Option 2 are provided in Table 5.3. Itemized OPCC's for each concept is provided in Appendix C.

Table 5.3: Initial Schematic Design Alternative OPCC

Site	OPCC	Lower range Estimate (-20%)	Upper range Estimate (+30%)
Option 1	$6.6M	$5.3M	$8.6M
Option 2	$7.0M	$5.6M	$9.1M

5.5 Construction Access and Contractor/ Material Availability

Site access to construct the improvements presented in this study will be challenging due to the topography, shallow depths, and coastal conditions. In addition, contractors and stone material for shoreline work are in high demand due to high lake levels. These factors may contribute to inflation/ uncertainty of costs associated with future shoreline protection projects.

5.6 Permitting Process

The permitting process for the project will involve extensive communication with regulatory agencies and project stakeholders. This is necessary for the applicant to be fully compliant and for the regulatory agencies to obtain the information they need to make an informed decision. It is anticipated that the approach will involve the following steps:

1. Pre-Application Meeting – A meeting is needed with regulatory authorities for the purposes of introducing them to the project, confirming the type of permits needed, and early stage identification of potential issues.
2. Permit Application - To proceed with a shoreline revetment, a Joint Permit Application will be needed. This permit satisfies regulatory requirements from both the Michigan Department of Environment, Great Lakes and Energy (EGLE) and the USACE. Salient Points include:
    - The Michigan Natural Resources and Environmental Protection Act of 1994, Part 323 "Shorelands Protection and Management" is the applicable law pertaining to shoreline rehabilitation.
    - While this property is not located in a designated High Risk Erosion Area (per EGLE), the designations for said areas carry particularities that may need to be addressed in the permitting process. Per Part 323 Administrative Rules, R 281.22 High-risk erosion areas, (8), the permit application shall contain all of the following information:

(a) A legal description of the property.

(b) A description of the proposed permanent structure.

(c) A sketch of the proposed site which shows the location of the proposed permanent structure in relation to the location of the property lines and prominent features.

(d) The signature and address of the applicant.

Furthermore, per Part 323 Administrative Rules, R 281.22 High-risk erosion areas, (11), A special exception shall be granted, and a portion of the required setback distance waived, for the installation of an approved shore protection project if all of the following conditions are met:

(a) A local agency is contractually responsible for the perpetual care of the shore protection structure. The responsibility will be defined in a written agreement between the department and the local agency. The local agency shall agree to perform maintenance or repairs to maintain the integrity of the shore protection. The local agency shall submit to the department a financial plan for maintaining the structure.

(b) The shore protection structure is designed and constructed to meet or exceed a 50- year storm standard. The design and construction shall be certified by a professional engineer. If the structure is constructed in the waters of the Great Lakes or lies below the ordinary high watermark, a permit pursuant to the provisions of Act No. 247 of the Public Acts of 1955, as amended, being S322.701 et seq. of the Michigan Compiled Laws, shall be obtained for the shore protection structure.

(c) A favorable finding is made by the local agency, with input by the department, that a greater public good exists to support the use of a shore protection structure rather than a natural shoreline in terms of all of the following: (i) The preservation of fish and wildlife habitat. (ii) The value to the entire community of a natural shoreline as opposed to the value to the entire community of additional development that is made possible by the shore protection. (iii) The impact of the loss of sand movement along the shoreline. (iv) The impact on erosion of land in the immediate area of the shore protection structure. Before making the finding, the local agency shall hold a public hearing. Notice shall be sent to all riparians within 300 feet of the proposed shore protection structure and to the department.

(d) A favorable finding is made by the department that a greater public good exists to support the use of a shore protection structure rather than a natural shoreline in terms of all of the following: (i) The preservation of fish and wildlife habitat. (ii) Protection of the public trust. (iii) The impact of the loss of sand movement along the shoreline. (iv) The impact on the erosion of land in the immediate area of the shore protection structure.

(e) There is a minimum of 30 feet from the shore protection to any permanent structure. If the bluff or dune is unstable due to height, slope, wind erosion, or groundwater seepage, the department may require a setback of more than 30 feet or an engineered bluff or dune stabilization plan, or both. In areas of steep slopes, a greater setback may be necessary to provide access for maintenance equipment and a safe building site. If the parcel has existing permanent structures which are less than 30 feet from the proposed shore protection, there shall be sufficient access to permit the maintenance and repair of the shore protection.

(f) Shore protection is already a common feature of the shoreline lying within 1,000 feet of the proposed shore protection structure.

3. Permit Review - According to the EGLE, generally it will take from 30 to 90 days from the time the application is submitted until a decision is made. During this time, it may be required to respond to additional queries from the agencies, including an official response to public comments. These are usually requested in writing and could delay the permit determination decision.

Appendix A

Historic Soil Borings

SUBSURFACE EXPLORATION LOG

Boring No.: 2
Sheet: 1 of 2

Project Name: Resort Bluff Trail
Client / Owner: Top of Michigan Trails Council
Site Location: Section 1 Emmet County
Boring Location: STA 118+00 Center Line of Trail

Job No.: 1030110
Architect/Engineer: Lucas C. Porath P.E.
Date of Drilling: 6/14/2005 0:00

Unconfined Compression Strength (tons/sft) (X) I 1 I 2 I 3 I 4 I 5
Plastic Limit (%) Water Content (%) Liguid Limit (%) I 10 I 20 I 30 I 40 I 50
Standard Penetration (N) (blows/foot) (O) I 10 I 20 I 30 I 40 I 50

Surface Elevation: 616.3 ft.

Depth (feet)	Stand.Penetration Blows/ 6 inch	Recovery (%)	Sample Number	Sample Type	Description of Material	Remarks
					Dark brown medium SAND (SP) with gravel/cobble and rock - dry	
	2 2 2	75	1	SS	Light brown medium SAND (SP) trace gravel - dry - loose	
5	3 3 3	0	N/A	SS		
10	1 1 1	50	2	SS	Light brown medium-coarse SAND (SP) trace gravel - very loose	
15	2 2 2	0	N/A	SS	(SAND in tip)	
20	6 6 6	10	3	SS	Light brown medium SAND (SP) with gravel/cobble dry - medium dense	
25						

The stratification lines represent approximate boundry lines between soil types. The material descriptions and strata lines are based on split spoon sample intervals and cuttings from the auger flights.

WL	Rig/Foreman:
WL	Logged by: Anthony Prantera
WL	Approved by:

SUBSURFACE EXPLORATION LOG

Boring No.: 3
Sheet: 1 of 2

Project Name: Resort Bluff Trail
Client / Owner: Top of Michigan Trails Council
Site Location: Section 1 Emmet County
Boring Location: Top of Bluff Offset from STA 118+00

Job No.: 1030110
Architect/Engineer: Lucas C. Porath P.E.
Date of Drilling: 6/9/2005 0:00

Description of Material

Surface Elevation: 676.3 ft.

Unconfined Compression Strength (tons/sft) (X)
I 1 I 2 I 3 I 4 I 5

Plastic Limit (%) Water Content (% Liguid Limit (%)
I 10 I 20 I 30 I 40 I 50

Standard Penetration (N) (blows/foot) (O)
I 10 I 20 I 30 I 40 I 50

Depth (feet)	Stand.Penetration Blows/ 6 inch	Recovery (%)	Sample Number	Sample Type	Description of Material	Standard Penetration (N)	Remarks
					6" Topsoil		
					Light brown medium SAND (SP) with gravel/cobble - dry Dense		
	15 25 25	100	1	SS		O	
5	10 20 12	100	2	SS	Rock in spoon tip		4" Penetration 1" Penetration
	10 11 12	5	3	SS	Rock in spoon tip		
10	5 8 8	100	4	SS	Light brown medium SAND (SP) - dry - medium dense	O	
15	15 25 29	100	5	SS	Light brown medium SAND (SP) - dry - very dense	N>50	N=54
20	10 15 30	70	6	SS		O	
					Light brown medium-coarse SAND (SP) with gravel - dry (Rocky drill) - dense		
25	15 30 33	100	7	SS	Light brown medium SAND (SP) trace gravel - very dense	N>50	N=63

The stratification lines represent approximate boundry lines between soil types. The material descriptions and strata lines are based on split spoon sample intervals and cuttings from the auger flights.

WL	Rig/Foreman:
WL	Logged by:
WL	Approved by:

SUBSURFACE EXPLORATION LOG

Boring No.: 4
Sheet: 1 of 1

Project Name: Resort Bluff Trail
Client / Owner: Top of Michigan Trails Council
Site Location: Section 1 Emmet County
Boring Location: STA 115+00 Center Line of Trail

Job No.: 1030110
Architect/Engineer: Lucas C. Porath P.E.
Date of Drilling: 6/14/2005 0:00

Unconfined Compression Strength (tons/sft) (X)
I 1 I 2 I 3 I 4 I 5

Plastic Limit (%) Water Content (%) Liguid Limit (%)
I 10 I 20 I 30 I 40 I 50

Standard Penetration (N) (blows/foot) (O)
I 10 I 20 I 30 I 40 I 50

Surface Elevation: 619 ft.

Depth (feet)	Stand.Penetration Blows/ 6 inch	Recovery (%)	Sample Number	Sample Type	Description of Material	Standard Penetration (N)	Remarks
					Brown medium SAND (SP) trace gravel - dry		
	4 2 2	90	1	SS	Light brown fine SAND (SP) - dry - loose	o	
5	2 2 4	100	2	SS		o	
	5 2 2	100	3	SS		o	
10	8 7 10	100	4	SS	Light brown fine SAND (SP) - dry - Medium dense	o	
					Very Rocky - large cobble		
15					No Sample - driving direct on rock		
20					No Sample - driving direct on rock		
					Rock Refusal @ 22 ft.		
25							

The stratification lines represent approximate boundry lines between soil types. The material descriptions and strata lines are based on split spoon sample intervals and cuttings from the auger flights.

WL	Rig/Foreman:
WL	Logged by:
WL	Approved by:

SUBSURFACE EXPLORATION LOG

Boring No.: 6
Sheet: 1 of 3

Project Name: Resort Bluff Trail
Client / Owner: Top of Michigan Trails Council
Site Location: Section 1 Emmet County
Boring Location: STA 129+50 Center Line of Trail

Job No.: 1030110
Architect/Engineer: Lucas C. Porath P.E.
Date of Drilling: 6/13/2005 0:00

Depth (feet)	Stand.Penetration Blows/ 6 inch	Recovery (%)	Sample Number	Sample Type	Description of Material Surface Elevation: 614 ft.	Unconfined Compression Strength (tons/sft) (X) I 1 I 2 I 3 I 4 I 5 Plastic Limit (%) Water Content (%) Liguid Limit (%) I 10 I 20 I 30 I 40 I 50 Standard Penetration (N) (blows/foot) (O) I 10 I 20 I 30 I 40 I 50	Remarks
					Light brown fine SAND (SP) - dry		
	7						
	11	100	1	SS	4" Brown Clay (CH) - dry	x>4.5	
	16		2		Light brown fine SAND (SP) - dry - Medium dense	O	
5							
	7						
	13	100	3	SS		O	
	16						
	8						
	13	100	4	SS		O	
	15						
10							
	7						
	13	100	5	SS		O	
	14						
15							
	14				Light brown fine SAND (SP) - dry - Dense		
	18	100	6	SS		O	
	23						
20							
	12					O	
	14	100	7	SS			
	15				Light brown fine SAND (SP) - saturated - Medium Dense		
25							

The stratification lines represent approximate boundry lines between soil types. The material descriptions and strata lines are based on split spoon sample intervals and cuttings from the auger flights.

WL	Rig/Foreman:			
WL	Logged by:			
WL	Approved by:			

SUBSURFACE EXPLORATION LOG

Boring No.: 7
Sheet: 1 of 3

Project Name: Resort Bluff Trail
Client / Owner: Top of Michigan Trails Council
Site Location: Section 1 Emmet County
Boring Location: Top of Bluff Offset from STA 129+50

Job No.: 1030110
Architect/Engineer: Lucas C. Porath P.E.
Date of Drilling: 6/10/2005 0:00

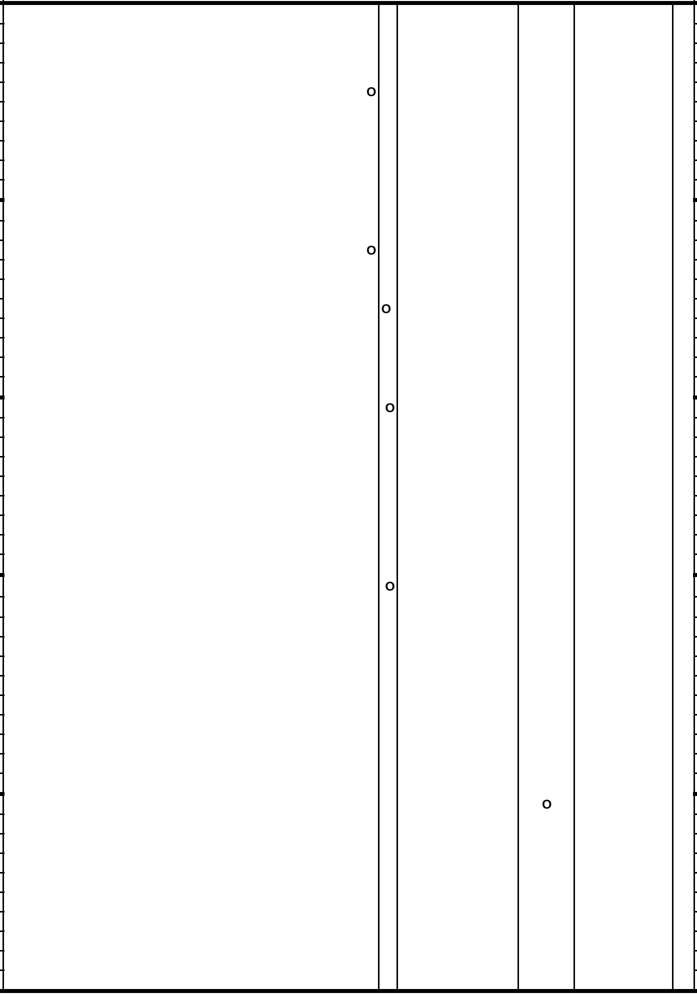
Surface Elevation: 691.6 ft.

Unconfined Compression Strength (tons/sft) (X) I 1 I 2 I 3 I 4 I 5

Plastic Limit (%) Water Content (%) Liguid Limit (%) I 10 I 20 I 30 I 40 I 50

Standard Penetration (N) (blows/foot) (O) I 10 I 20 I 30 I 40 I 50

Depth (feet)	Stand.Penetration Blows/ 6 inch	Recovery (%)	Sample Number	Sample Type	Description of Material	Remarks
					Brown fine-medium SAND (SP) - dry - loose	
	3 3 4	100	1	SS		
5	4 4 4	100	2	SS		
					Light brown meduim SAND (SP) trace gravel - dry - loose	
	6 6 7	100	3	SS	Light brown fine SAND (SP) - dry - medium dense	
10	7 8 11	0	N/A	SS		
15	7 13 14	100	4	SS		
20	10 14 17	100	5	SS	Light brown fine SAND (SP) - dry - Dense	
25	10					



The stratification lines represent approximate boundry lines between soil types. The material descriptions and strata lines are based on split spoon sample intervals and cuttings from the auger flights.

WL	Rig/Foreman:
WL	Logged by:
WL	Approved by:

Appendix B

Schematic Design Drawings

PHOTO 1
APRIL 13, 2020 COASTAL BLUFF COLLAPSE

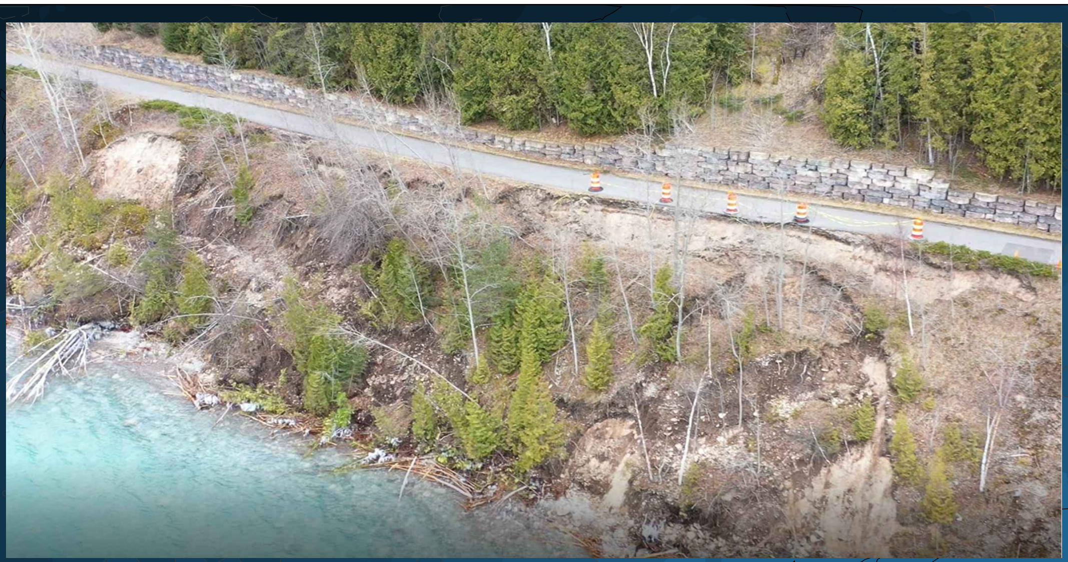


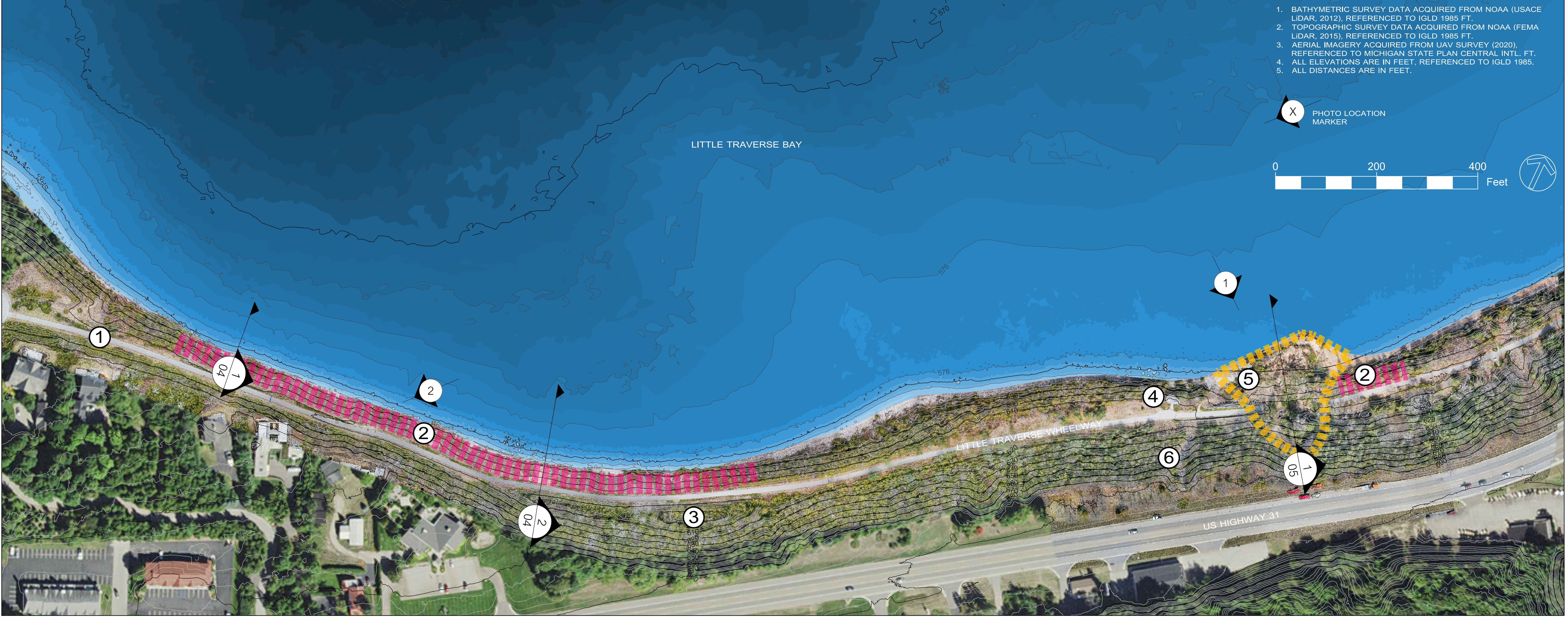
PHOTO 2
ONGOING BLUFF EROSION (SLOUGHING)

LEGEND

①	LITTLE TRAVERSE WHEELWAY
②	ONGOING BLUFF EROSION (SLOUGHING)
③	EXISTING BLUFF SWALE
④	SHORELINE OVERLOOK STRUCTURE
⑤	APRIL 13, 2020 COASTAL BLUFF COLLAPSE EXTENTS
⑥	EXISTING DRAINAGE INFRASTRUCTURE

NOTES

1. BATHYMETRIC SURVEY DATA ACQUIRED FROM NOAA (USACE LiDAR, 2012), REFERENCED TO IGLD 1985 FT.
2. TOPOGRAPHIC SURVEY DATA ACQUIRED FROM NOAA (FEMA LiDAR, 2015), REFERENCED TO IGLD 1985 FT.
3. AERIAL IMAGERY ACQUIRED FROM UAV SURVEY (2020), REFERENCED TO MICHIGAN STATE PLAN CENTRAL INTL. FT.
4. ALL ELEVATIONS ARE IN FEET, REFERENCED TO IGLD 1985.
5. ALL DISTANCES ARE IN FEET.



EXISTING CONDITIONS

PETOSKEY SLOPE FAILURE STUDY

Project Number:
(Project#-SKT-Figure#)
13269.601-SKT-01

Date:
(YYYY-MM-DD)
2020-08-26

Baird.

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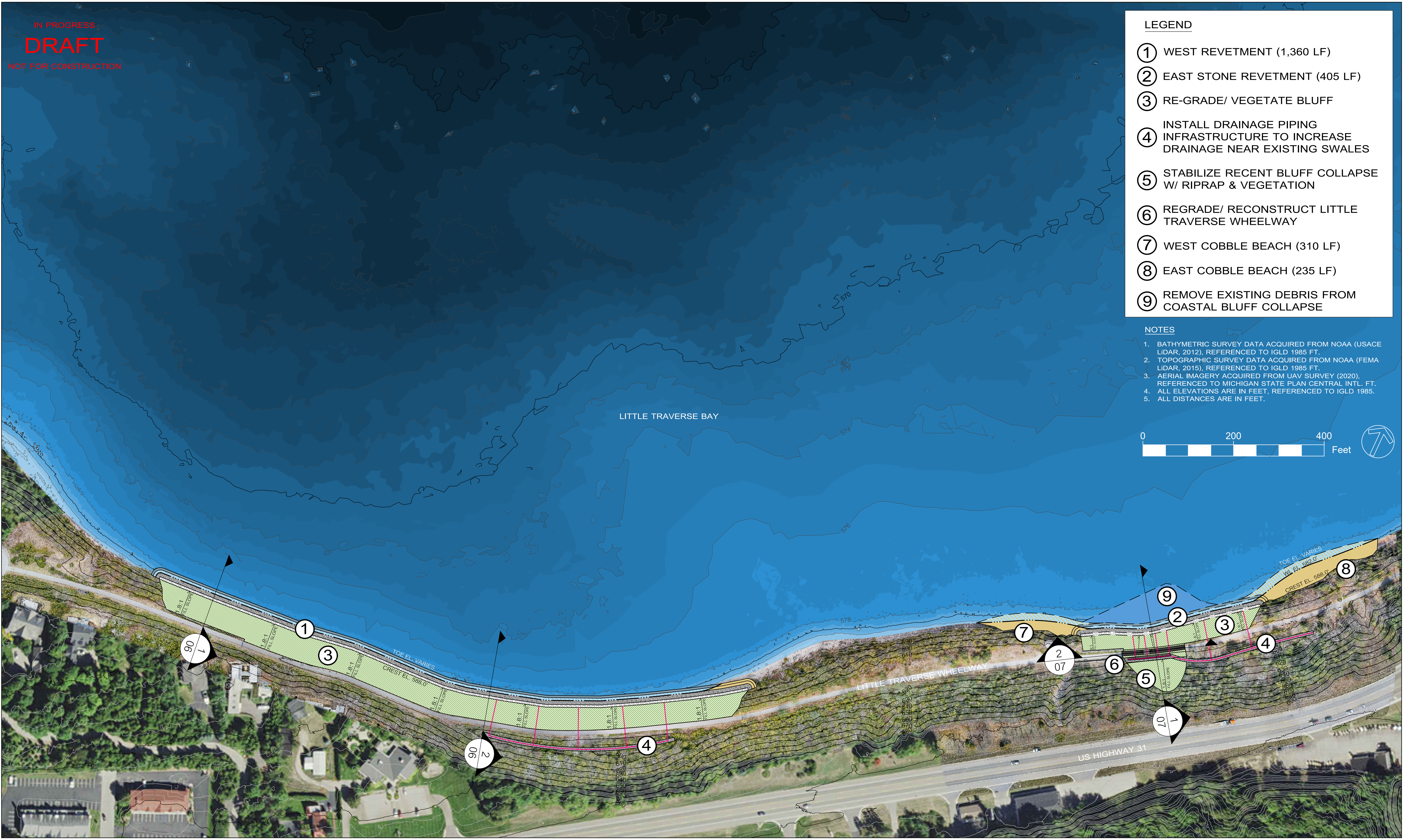

LEGEND

1. WEST REVETMENT (1,360 LF)
2. EAST STONE REVETMENT (405 LF)
3. RE-GRADE/ VEGETATE BLUFF
4. INSTALL DRAINAGE PIPING INFRASTRUCTURE TO INCREASE DRAINAGE NEAR EXISTING SWALES
5. STABILIZE RECENT BLUFF COLLAPSE W/ RIPRAP & VEGETATION
6. REGRADE/ RECONSTRUCT LITTLE TRAVERSE WHEELWAY
7. WEST COBBLE BEACH (310 LF)
8. EAST COBBLE BEACH (235 LF)
9. REMOVE EXISTING DEBRIS FROM COASTAL BLUFF COLLAPSE

NOTES

1. BATHYMETRIC SURVEY DATA ACQUIRED FROM NOAA (USACE LiDAR, 2012), REFERENCED TO IGLD 1985 FT.
2. TOPOGRAPHIC SURVEY DATA ACQUIRED FROM NOAA (FEMA LiDAR, 2015), REFERENCED TO IGLD 1985 FT.
3. AERIAL IMAGERY ACQUIRED FROM UAV SURVEY (2020), REFERENCED TO MICHIGAN STATE PLAN CENTRAL INTL. FT.
4. ALL ELEVATIONS ARE IN FEET, REFERENCED TO IGLD 1985.
5. ALL DISTANCES ARE IN FEET.

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PROPOSED PLAN - STONE REVETMENT (OPTION 1)

PETOSKEY SLOPE FAILURE STUDY

Project Number:
(Project#-SKT-Figure#)
13269.601-SKT-02

Date:
(YYYY-MM-DD)
2020-08-26

Baird.

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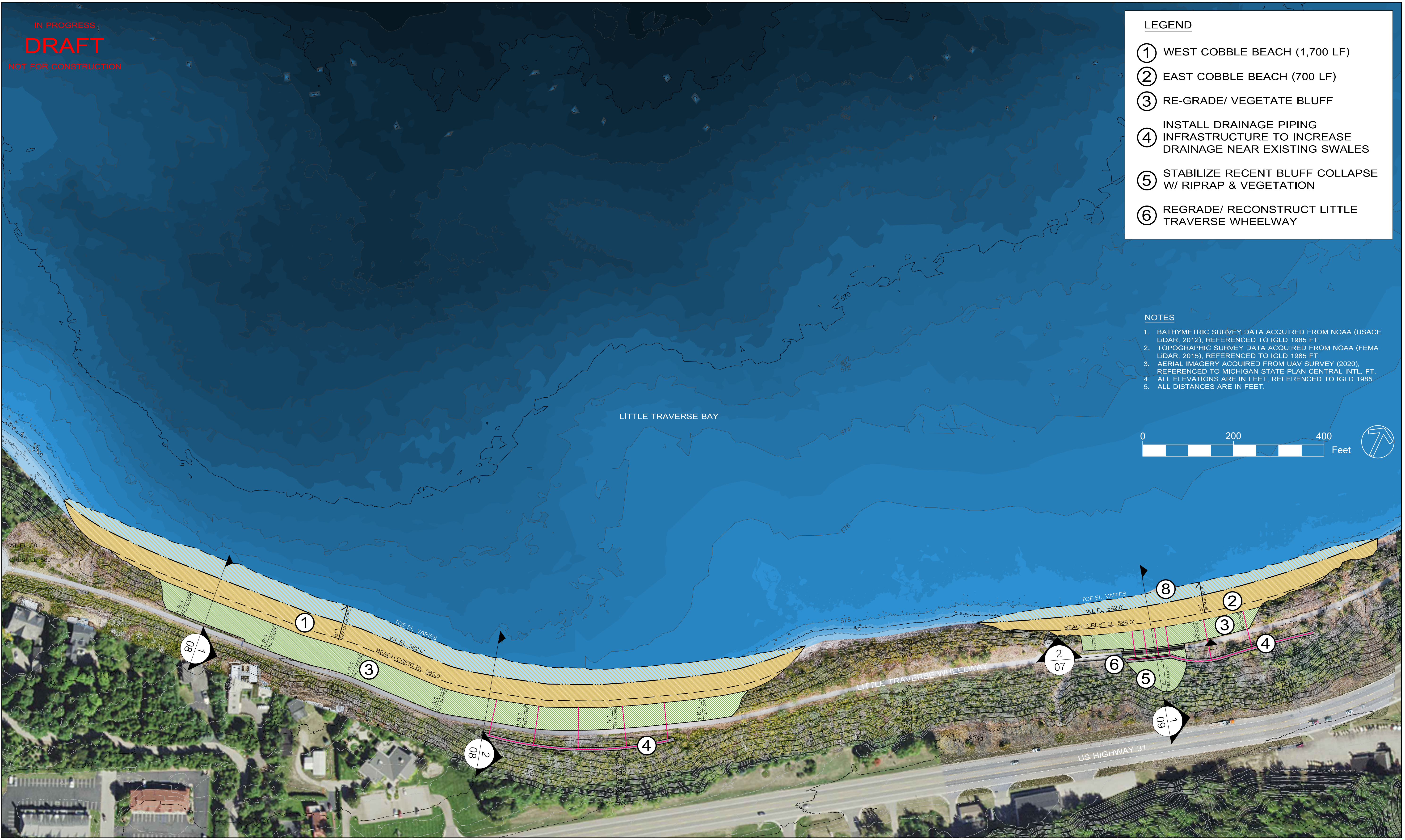

LEGEND

① WEST COBBLE BEACH (1,700 LF)
② EAST COBBLE BEACH (700 LF)
③ RE-GRADE/ VEGETATE BLUFF
④ INSTALL DRAINAGE PIPING INFRASTRUCTURE TO INCREASE DRAINAGE NEAR EXISTING SWALES
⑤ STABILIZE RECENT BLUFF COLLAPSE W/ RIPRAP & VEGETATION
⑥ REGRADE/ RECONSTRUCT LITTLE TRAVERSE WHEELWAY

NOTES

1. BATHYMETRIC SURVEY DATA ACQUIRED FROM NOAA (USACE LiDAR, 2012), REFERENCED TO IGLD 1985 FT.
2. TOPOGRAPHIC SURVEY DATA ACQUIRED FROM NOAA (FEMA LiDAR, 2015), REFERENCED TO IGLD 1985 FT.
3. AERIAL IMAGERY ACQUIRED FROM UAV SURVEY (2020), REFERENCED TO MICHIGAN STATE PLAN CENTRAL INTL. FT.
4. ALL ELEVATIONS ARE IN FEET, REFERENCED TO IGLD 1985.
5. ALL DISTANCES ARE IN FEET.

PROPOSED PLAN - COBBLE BEACH (OPTION 2)

PETOSKEY SLOPE FAILURE STUDY

Project Number:
(Project#-SKT-Figure#)
13269.601-SKT-03

Date:
(YYYY-MM-DD)
2020-08-26

Baird.

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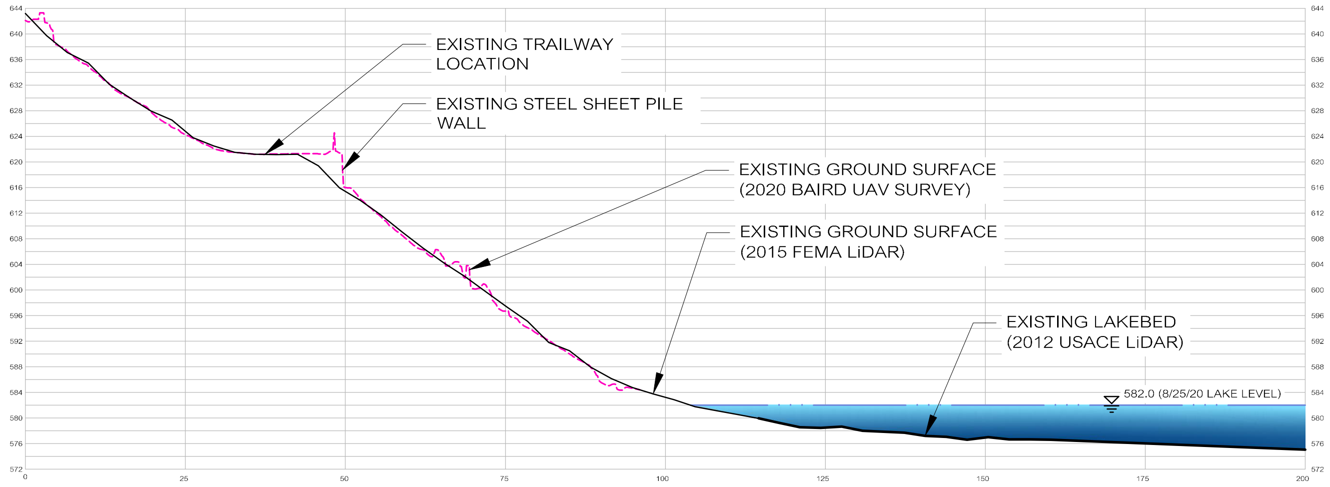

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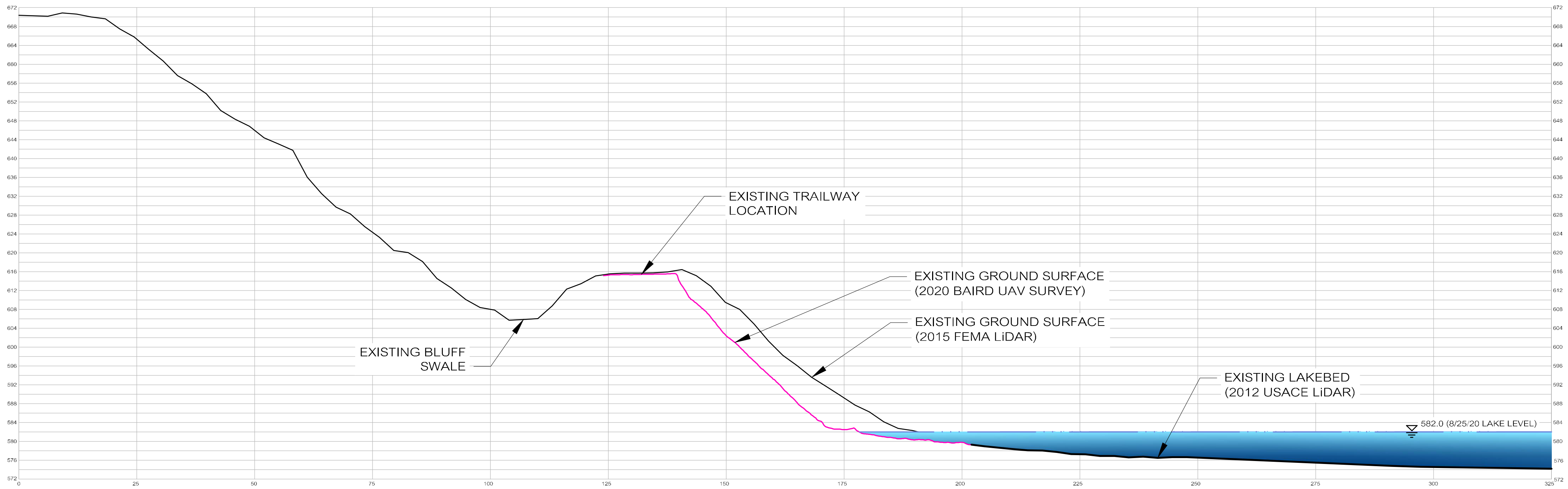
04
PROFILE: SECTION THROUGH EXISTING STEEL SHEET PILE WALL



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04
PROFILE: SECTION THROUGH EXISTING NEARSHORE BLUFF SLOUGH

NOTES:
1. ALL ELEVATIONS ARE REFERENCE TO FT. IGLD85
2. ALL DISTANCES ARE IN FEET.

EXISTING PROFILES

PETOSKEY SLOPE FAILURE STUDY

Project Number: 13269.601-SKT-04
Date: 2020-08-26

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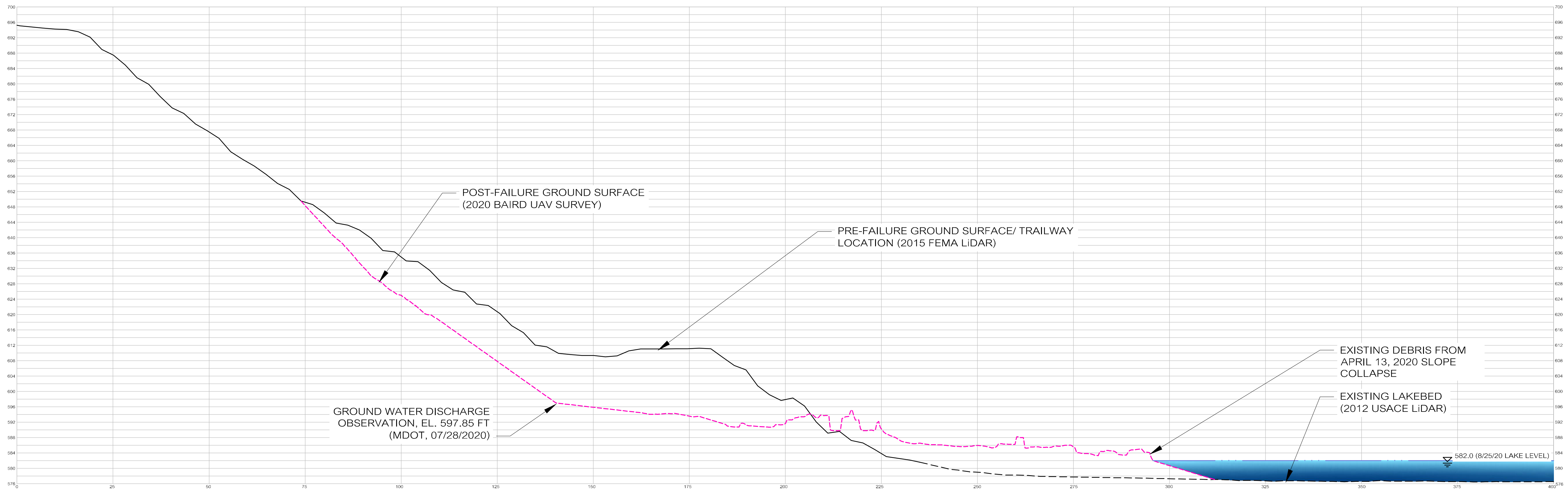

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05
PROFILE: EXISTING SLOPE FAILURE AREA

NOTES:
1. ALL ELEVATIONS ARE REFERENCE TO FT. IGLD85
2. ALL DISTANCES ARE IN FEET.

EXISTING PROFILE - SLOPE FAILURE AREA

PETOSKEY SLOPE FAILURE STUDY

Project Number:
(Project#-SKT-Figure#)
13269.601-SKT-05

Date:
(YYYY-MM-DD)
2020-08-26

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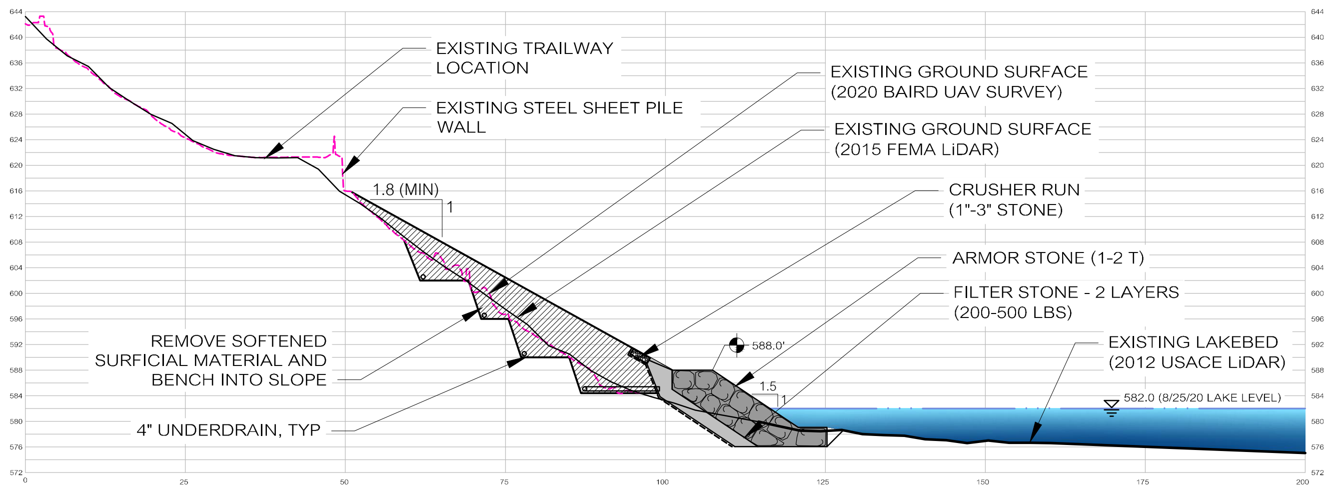

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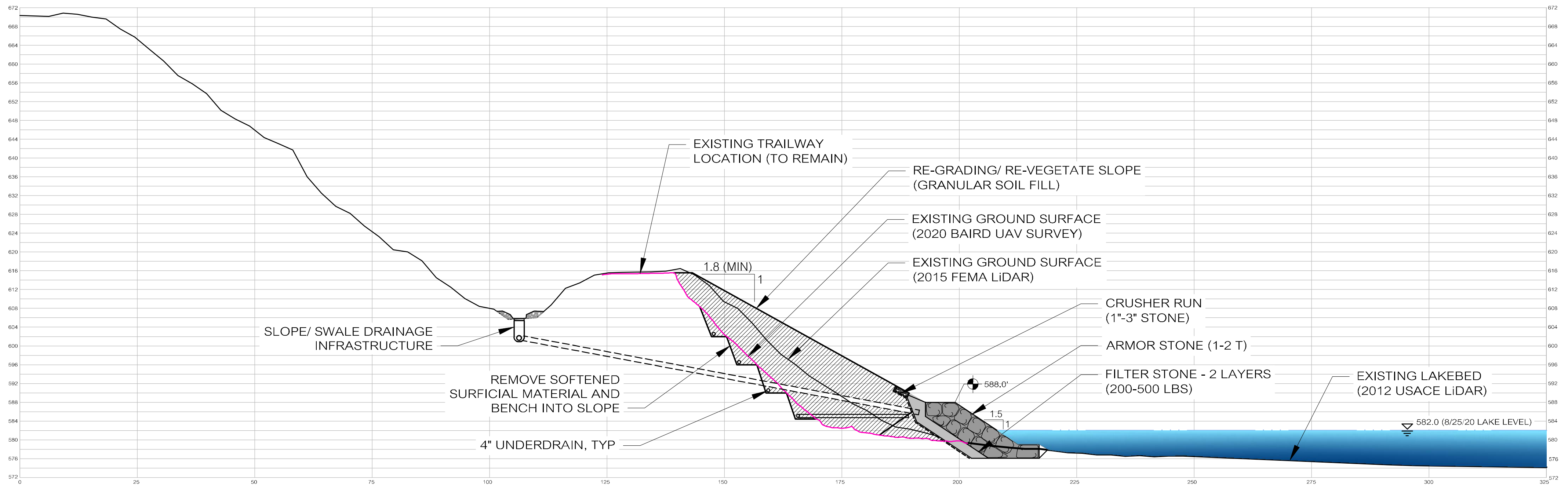
PROFILE: SECTION THROUGH EXISTING STEEL SHEET PILE WALL



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06
PROFILE: SECTION THROUGH EXISTING NEARSHORE BLUFF SLOUGH

NOTES:
1. ALL ELEVATIONS ARE REFERENCE TO FT. IGLD85
2. ALL DISTANCES ARE IN FEET.

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PROPOSED PROFILES - STONE REVETMENT (OPTION 1)

PETOSKEY SLOPE FAILURE STUDY

Project Number:
13269.601-SKT-06

Date:
2020-08-26

Baird.

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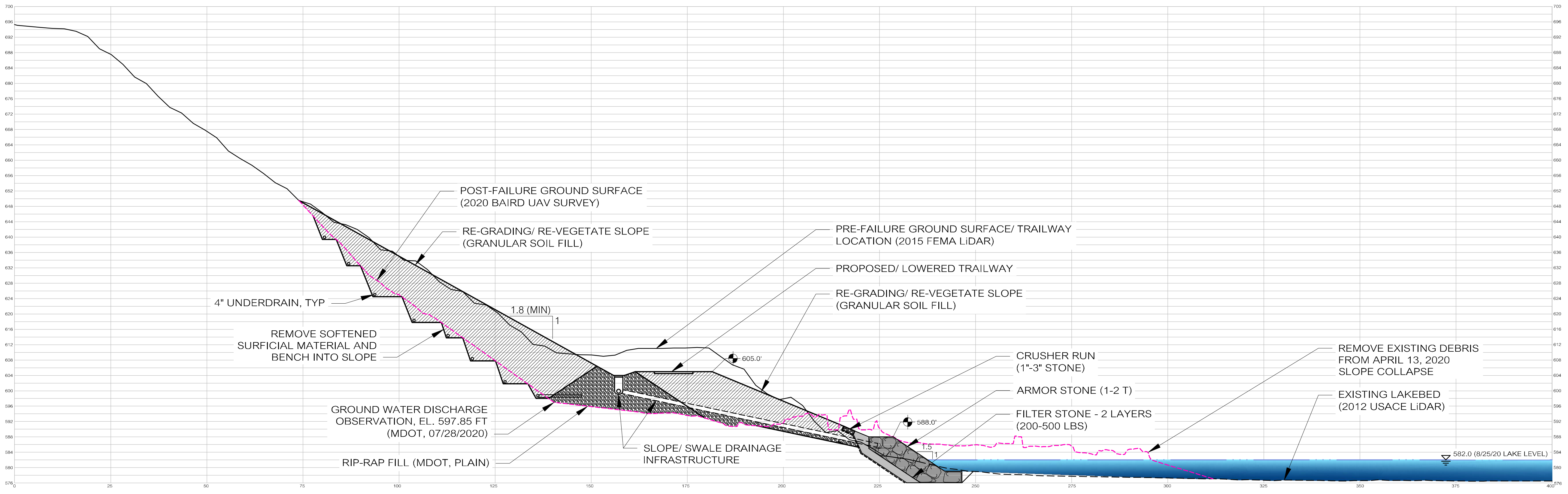

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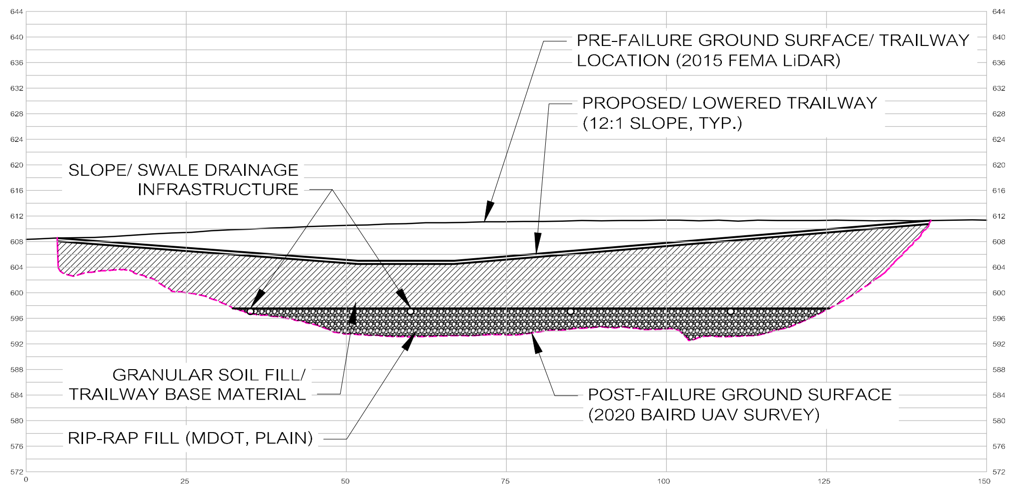
07
PROFILE: PROPOSED SLOPE FAILURE AND TRAILWAY REHABILITATION



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07
PROFILE: PROPOSED LOWERED TRAILWAY

NOTES:
1. ALL ELEVATIONS ARE REFERENCE TO FT. IGLD85
2. ALL DISTANCES ARE IN FEET.

PROPOSED PROFILE - SLOPE FAILURE AREA (OPTION 1)

PETOSKEY SLOPE FAILURE STUDY

Project Number:
(Project#-SKT-Figure#)
13269.601-SKT-07

Date:
(YYYY-MM-DD)
2020-08-26

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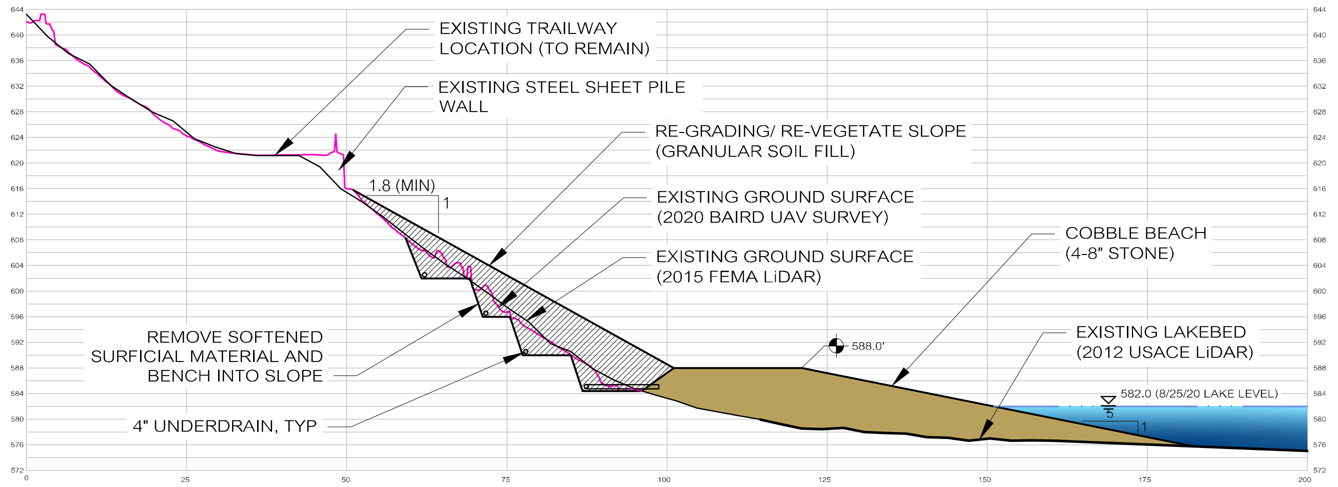

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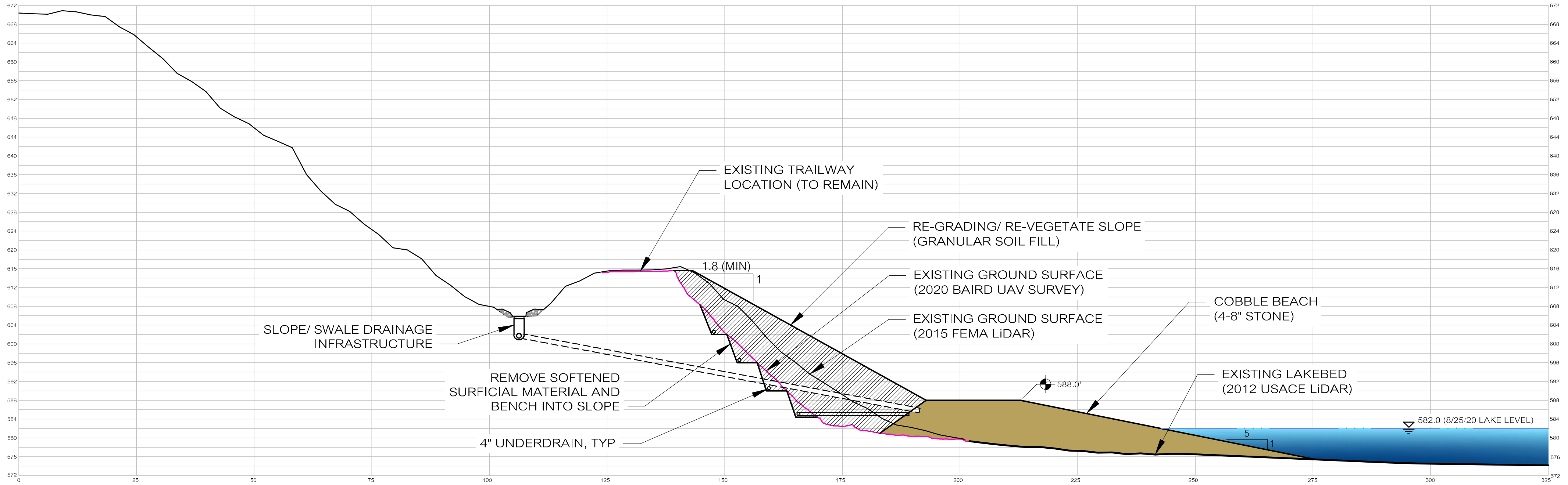
08
PROFILE: SECTION THROUGH EXISTING STEEL SHEET PILE WALL



2
08
PROFILE: SECTION THROUGH EXISTING NEARSHORE BLUFF SLOUGH

NOTES:
1. ALL ELEVATIONS ARE REFERENCE TO FT. IGLD85
2. ALL DISTANCES ARE IN FEET.

PROPOSED PROFILES - COBBLE BEACH (OPTION 2)

PETOSKEY SLOPE FAILURE STUDY

Project Number:
13269.601-SKT-08

Date:
2020-08-26

Baird.

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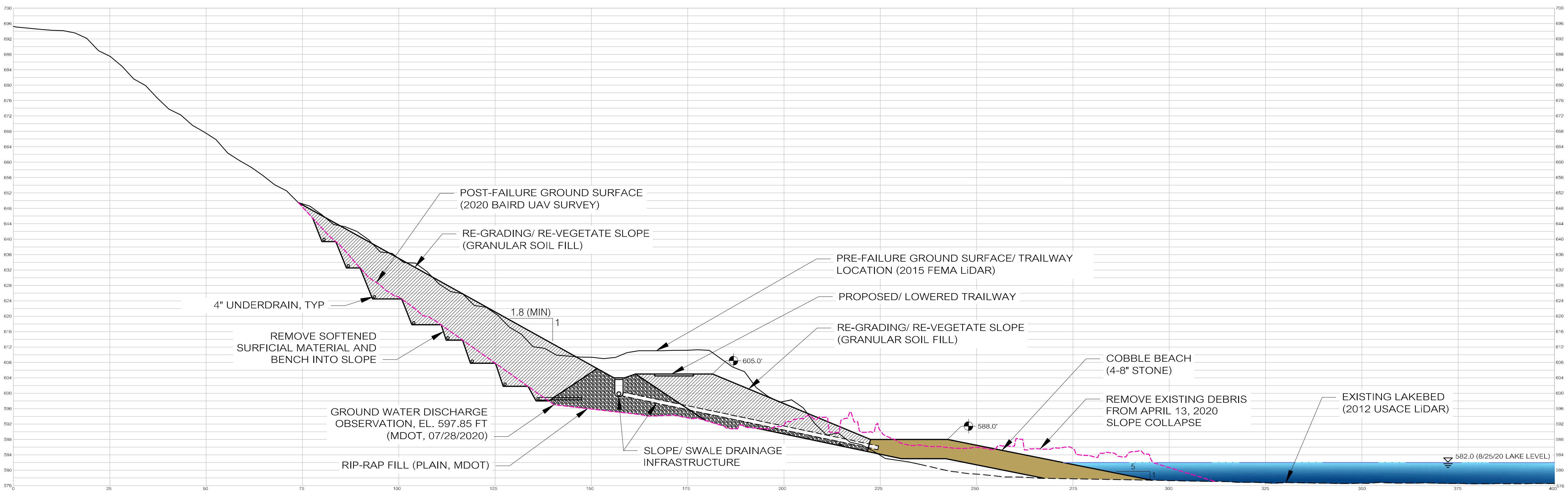

1
09
PROFILE: PROPOSED SLOPE FAILURE AND TRAILWAY REHABILITATION

NOTES:
1. ALL ELEVATIONS ARE REFERENCE TO FT. IGLD85
2. ALL DISTANCES ARE IN FEET.

PROPOSED PROFILE - SLOPE FAILURE AREA (OPTION 2)

PETOSKEY SLOPE FAILURE STUDY

Project Number:
(Project#-SKT-Figure#)
13269.601-SKT-09

Date:
(YYYY-MM-DD)
2020-08-26

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Appendix C

Engineer's Opinion of Probable Construction Costs

Petoskey Slope Stability

Baird.

Opinion of Probable Construction Cost (OPCC)

Project No 13269.601

Option 1 - Schematic Design

Date: 09/16/2020

Rev0

Item	Unit	Quantity	Unit Cost	Extension	Sub Total
Mobilization/Demobilization (Assumes Land-Based Construction)	ALLOW	1	$400,000	$400,000	$400,000
Site Preparation					
A SITE ACCESS & LAYDOWN AREA DEVELOPMENT	ALLOW	1	$75,000	$75,000	$75,000
B CLEARING GRUBBING & DISPOSAL OF EXISTING VEGETATION	ACRE	3	$10,000	$30,000	$30,000
West Revetment					
A NEW ARMOR STONE (1-2T)	TON	10,132	$125	$1,266,458	
B FILTER STONE (200-500 LBS)	TON	3,703	$100	$370,260	
D EXCAVATION, REGRADING & DISPOSAL	CY	5,556	$30	$166,667	
E GEOTEXTILE	SQ FT	57,120	$1.25	$71,400	
F CRUSHER RUN	CY	151	$35	$5,289	
G GRANULAR SOIL FILL	CY	15,400	$25	$385,000	
H NEARSHORE SLOPE DRAINAGE INFRASTRUCTURE	LS	1	$85,000	$85,000	
I REPAVE TRAILWAY	SQ YD	1,440	$35	$50,400	
J LANDSCAPING/ NATIVE PLANTINGS (SEEDED)	SQ YD	7,796	$10	$77,960	
K SWALE DRAINAGE INFRASTRUCTURE	LS	1	$120,000	$120,000	
L EROSION CONTROL	SQ YD	7,796	$2	$15,592	$2,614,025
East Revetment & Landscape Improvements					
A NEW ARMOR STONE (1-2T)	TON	3,017	$125	$377,144	
B FILTER STONE (200-500 LBS)	TON	1,103	$100	$110,261	
C EXCAVATION, REGRADING & DISPOSAL	CY	4,167	$30	$125,009	
D GEOTEXTILE	SQ FT	17,010	$1.25	$21,263	
E CRUSHER RUN	CY	70	$35	$2,456	
F COBBLE BEACH STONE (4-8" STONE)	TON	2,965	$50	$148,251	
G GRANULAR SOIL FILL	CY	8,055	$25	$201,374	
H RIPRAP FILL	TON	2,312	$75	$173,423	
I NEARSHORE SLOPE DRAINAGE INFRASTRUCTURE	LS	1	$25,000	$25,000	
J REPAVE TRAILWAY	SQ YD	415	$35	$14,525	
K LANDSCAPING/ NATIVE PLANTINGS (SEEDED)	SQ YD	2,398	$10	$23,977	
L SWALE DRAINAGE INFRASTRUCTURE	LS	1	$100,000	$100,000	
L EROSION CONTROL	SQ YD	2,398	$2	$4,795	$1,327,478

Sub Total	$4,446,503
Site Overhead 8%	$355,720
Office Overhead & Profit 15%	$666,975
Bond 1%	$44,465
Contingency 20%	$1,102,733
Total	$6,616,396
Lower Range Estimate (-20%)	$5.3M
Upper Range Estimate (+30%)	$8.6M

ITEMS NOT INCLUDED:

- ADDITIONAL PERMITTING, FINAL DESIGN AND ENGINEERING.
- TURBIDITY CURTAIN (PER REGULATORY AGENCY REQUIREMENTS)
- ADDITIONAL BATHYMETRIC SURVEYS
- ADDITIONAL SLOPE STABILITY INFRASTRUCTURE (I.E. SOIL NAILS)

Petoskey Slope Stability

Opinion of Probable Construction Cost (OPCC)

Option 2 - Schematic Design

Project No 13269.601
Date: 09/16/2020
Rev0

Item	Unit	Quantity	Unit Cost	Extension	Sub Total
Mobilization/Demobilization (Assumes Land-Based Construction)	ALLOW	1	$500,000	$500,000	$500,000
Site Preparation					
A SITE ACCESS & LAYDOWN AREA DEVELOPMENT	ALLOW	1	$75,000	$75,000	$75,000
B CLEARING GRUBBING & DISPOSAL OF EXISTING VEGETATION	ACRE	3	$10,000	$30,000	$30,000
West Cobble Beach					
A COBBLE BEACH STONE (4-8" STONE)	TON	38,788	$50	$1,939,389	
B EXCAVATION, REGRADING & DISPOSAL	CY	5,556	$30	$166,667	
C GRANULAR SOIL FILL	CY	15,400	$25	$385,000	
D NEARSHORE SLOPE DRAINAGE INFRASTRUCTURE	LS	1	$85,000	$85,000	
E REPAVE TRAILWAY	SQ YD	1,440	$35	$50,400	
F LANDSCAPING/ NATIVE PLANTINGS (SEEDED)	SQ YD	7,796	$10	$77,960	
G SWALE DRAINAGE INFRASTRUCTURE	LS	1	$120,000	$120,000	
L EROSION CONTROL	SQ YD	7,796	$2	$15,592	$2,840,008
Eastern Cobble Beach & Landscape Improvements					
A COBBLE BEACH STONE (4-8" STONE)	TON	11,887	$50	$594,343	
B EXCAVATION, REGRADING & DISPOSAL	CY	4,167	$30	$125,009	
C RIPRAP FILL	TON	2,312	$75	$173,423	
D GRANULAR SOIL FILL	CY	8,055	$25	$201,374	
E NEARSHORE SLOPE DRAINAGE INFRASTRUCTURE	LS	1	$25,000	$25,000	
F LANDSCAPING/ NATIVE PLANTINGS (SEEDED)	SQ YD	2,571	$10	$25,706	
G REPAVE TRAILWAY	SQ YD	415	$35	$14,525	
H SWALE DRAINAGE INFRASTRUCTURE	LS	1	$100,000	$100,000	
L EROSION CONTROL	SQ YD	2,571	$2	$5,141	$1,264,521

Sub Total	$4,709,529
Site Overhead 8%	$376,762
Overhead & Profit 15%	$706,429
Bond 1%	$47,095
Contingency 20%	$1,167,963
Total	$7,007,779

Lower Range Estimate (-20%)	$5.6M
Upper Range Estimate (+30%)	$9.1M

ITEMS NOT INCLUDED:

ADDITIONAL PERMITTING, FINAL DESIGN AND ENGINEERING.
TURBIDITY CURTAIN (PER REGULATORY AGENCY REQUIREMENTS)
ADDITIONAL BATHYMETRIC SURVEYS
ADDITIONAL SLOPE STABILITY INFRASTRUCTURE (I.E. SOIL NAILS)