APPENDIX A – WAVE MODELING REPORT AND CALCULATIONS

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September 11, 2020

Re: Alcan Cove Final Wave Modeling Report by Richard Baker

Alcan Cove Wave Modeling Final Report

Objectives

The fuel pier facility at Alcan Cove, on Shemya Island, Alaska, experienced significant wave damage during a storm on January 31 and February 1, 2020. Appledore Marine Engineering is currently working on the design level inspection required to perform an assessment of existing conditions, and to produce conceptual designs for future repair and wave protection at these facilities. The purpose of the wave modeling in this proposal is to develop and apply a wave simulation tool that is capable of predicting wave conditions at the project site, during extreme storm events, under existing conditions and following construction of alternative wave protection facilities. This tool will subsequently be used to provide technical support to the Appledore project team, during development of the above conceptual designs.

Technical Approach

Extreme waves at the Alcan Cove wharf site are the result of storms passing over coastal and deep offshore waters of the Bering Sea, which is located approximately 10 kilometers north of Shemya Island. Large waves generated within these deep, open expanses of sea, can enter the shallower coastal waters around the island and eventually propagate into Alcan Cove, from various directions. Waves entering Alcan Cove are subjected to refraction, diffraction and shoaling, due to shallow depths and remnants of offshore and shoreline wave protection structures (such as former breakwaters and piles of dolos) placed during and after the Second World War. In order to better understand these wave generation and propagation processes, a 2-dimensional spectral wave model has been developed and applied during this project.

Task 1 – Develop Wave Modeling Tool

Implement Wave Modeling Software

Based on our current understanding of the wave generation, transport and dissipation processes which must be included in the wave model of Alcan Cove, SWAN (Simulating WAves Nearshore (Booij et al., 1999) was selected for use on this project. SWAN is a third-generation 2-dimensional, transient near-shore spectral wave model developed at Delft Technical University, in the Netherlands. It is widely used by universities, governments (US Navy) and engineering consultants for the design of shoreline wave protection facilities. The SWAN model Fortran source code was obtained from the public domain and successfully built into an application for use on this project. Model compilation was accomplished using the open-source GNU Fortran Compiler, running under OSX version 10.11.6, on a MacBook Pro. All wave modeling was also accomplished on this computer platform.

Develop SWAN Wave Modeling Domain

Land and open ocean boundaries of the wave model spatial domain were determined, in spherical coordinates (longitude and latitude), using Google Earth Pro (see Figure 1). It is important to note that the Semichi Island Chain centerline is oriented slightly clockwise from horizontal. However, for convenience in discussing directions, North is referred to as approximately on-offshore and east and west as approximately along-shore. The on-offshore extent of the domain starts with the northern shoreline boundaries of the Semichi Island Chain (Alaid, Nizki and Shemya) and extends out over 10 km, to deep water (2000 meters) in the Bering Sea. The two narrow passages between the Semichi islands were treated as shoreline, as a valid modeling simplification, since waves from the south do not likely impact the site significantly. The lateral (along-shore) extent of the domain starts at the western end of Alaid Island and extends eastward to the eastern end of Shemya Island. The western on-offshore open ocean boundary extends towards the northwest from the western tip of Alaid Island. Similarly, the eastern on-offshore open ocean boundary extends towards the northeast from the eastern tip of Shemya Island. The offshore ocean boundary extends between the offshore ends of these lateral boundaries, parallel to the edge of the coastal shelf, within kilometer deep Bering Sea waters.

Develop Wave Model Computational Grid

A high quality unstructured triangular finite element model grid was developed for the SWAN model, using Google Earth Pro (for shorelines), the Triangle utility (Schewchuk, 2002) and water depths mined from the following online NOAA Chart X,Y,Z database: https://www.ngdc.noaa.gov/mgg/coastal/. The following eight NOAA depth data sets were used to define depths throughout the model domain: H06873 (Alcan Cove-1945), H06987 (Offshore of Alcan Cove-1944), H06988 (Passage between Shemya and Nizki Islands-1944), H06937 (Semichi Island Chain-1944), H07994 (Alcan Cove Pier-1954), H06999 (Eastern Shore Shemya Island-1944), H07000 (Offshore Eastern Shoreline of Shemya Island-1944), and H07597 (Offshore of Shemya Island-1944). In order to better define present water depths in the vicinity of the Alcan Cove pier, Appledore Marine Engineering (Appledore) conducted a survey, during early August of 2020. This survey included lead-line soundings from a small craft (85 depth data points) and single-beam sonar sweeps of Alcan Cove and around the pier (1,989 depth data points). The SWAN model grid consists of a total of 3,932 triangular computational elements and 2,128 triangle corner nodes. Figures 2 and 3 show the entire grid and the portion of the grid in the vicinity of the pier in Alcan Cove, respectively. A total of 82,894 depth data points (longitude, latitude, depth) were extracted from the above NOAA and Appledore surveys. The model grid and depth data locations (dots), within the entire spatial domain and only within Alcan Cove, are shown in Figures 4 and 5, respectively. It is seen that the depth data coverage within the model grid extents is good. Figure 6 shows the location of depth data points collected during the 1944 and 1945 surveys within Alcan Cove and the recent Appledore lead-line and single-beam sonar surveys conducted during August 2020. The old 1944 Alcan Cove depth data was deleted in the vicinity of the pier, where the new single-beam depth data were collected. This was done so that only the new depth data were used to define model water depths in the vicinity of the pier. A Fortran utility was developed during this study to interpolate the horizontal array of depth data points onto each corner node of the triangular model elements.

Determine Wave Model Ocean Boundary Conditions

Another input required by the SWAN model are wave conditions at the three open ocean boundaries. All wave approach direction scenarios utilized a 100-year return period design storm wave specification at the offshore model boundaries. Historical hindcast wave and meteorological data is available at US Army Corps of Engineers Wave Information Study (WIS) Station 82431 (wis.usace.army.mil), which is located approximately 125 kilometers to the East of Shemya Island (See Figure 7). Although this WIS data is at some distance from the site, it is likely representative of historical wave conditions generally found within the Bering Sea and especially along the eastern and southern margins of its deeper western basin. The WIS hindcast-model generated dataset lists numerous wave and meteorological variables, at hourly intervals, for the period between 1954 and 2014. These data were analyzed to estimate 100-year return period wave characteristics, such as significant wave height (Hs), peak wave period (Tp) and wave train propagation direction (theta). The WIS data includes results of a statistical analysis of hourly wave hindcast data, using the top 61 discrete storm events contained within the 61-year predicted historical record. Results indicate a 61-year return period wave event of record with a height, Hmo, equal to 15.45 meters (51 feet), a period, Tp, equal to 16.25 seconds and an approach angle, theta, of 285 degrees (about from the west). This WIS wave event of record occurred at 11 AM on March 22, 1999. However, 2 of the top 10 WIS wave events, although 2-3 meters smaller in wave height, exhibit much different approach angles, at approximately 45 degrees (northeast). Further to the west along the Aleutian chain at Shemya Island, the more open expanse of the deep Bering Sea to the north will like result in historical conditions with extreme waves approaching from a range of directions, including from the west (parallel to the southern margin of the Bering Sea), north, from the middle of the Bering Sea, and east (again, parallel to the southern margin of the Bering Sea). An extrapolation of extreme wave characteristics predicted at WIS Station 82431 (see Figure 8) suggests that wave heights generated out in the deepest portion of the Bering Sea, during a 100-year event, could be as high as 17 meters. It is likely that these extreme waves could propagate towards Shemya from a range of angles, between the west and the east. As a result if this uncertainty in wave approach angle, over a range of 5 angles, including: from the west, northwest, north, northeast and east, were simulated with SWAN as a sensitivity analysis, using 100-year return period wave height (Hmo) predicted by extrapolation of wave height-frequency distributions developed previously at WIS Station 82431.

For computational purposes SWAN subdivides a wave spectrum, both within the domain and for wave inputs at its open ocean boundaries, into discrete frequency bands. Spectral wave energy density (E in m2/Hz) and wave propagation direction (alpha) are simulated, over each frequency band. For consistency, open ocean boundary wave inputs must also be defined, using E and alpha, over each frequency band. Unfortunately, the WIS hourly hindcast data includes only wave height (Hmo in meters), spectral peak period (Tp in seconds) and direction (alpha in degrees). The WIS data does not include the corresponding E and alpha values, over the full range of spectral frequencies likely to have occurred, during each historical storm event.

Fortunately, hourly spectral wave data is available for 12 separate years, between 2006 and 2019, at NOAA Buoy 46070 (ndbc.noaa.gov/station_history.php?station=46070). This buoy is located within deep waters of the Bering Sea, approximately 70 kilometers north of Shemya Island (see Figure 7). Available spectral wave data at this buoy include E and alpha, over 46 discrete frequency bands, spanning between 0.0325 Hz and 0.0485 Hz. This frequency range covers the entire wave spectrum monitored at the buoy. These historic hourly buoy data were analyzed using a Fortran utility developed during this project, to calculate corresponding significant wave heights (Hs) and approach angles (alpha), based on E and alpha values for each frequency band. All historical wave events with spectrally calculated Hs in excess of 10 meters (maximum Hs was 14.6 meters on January 23, 2008) were extracted from the data for closer examination. It was found that the extreme events had similar E distributions over the full spectrum, for a range of propagation (direction from) angles, between west and east. Accordingly, an extreme historical event with Hs of 11.5 meters (February 20, 2017) was selected for representing E and alpha spectral distributions of the design event. Table 1 shows the method used in this study to upscale E values measured at the NOAA buoy to a 100-year event, corresponding to the Hs of 17 meters estimated at the WIS station. In Table 1, buoy E values contained in column 1 were upscaled from an Hs (=4*SQRT(Sum Mo)) of 11.5 meters to E values (column 6) corresponding to an Hs of 17 meters. 100-year E and alpha values contained in Table 1 for the full spectrum were used to define the energy spectrum of incoming waves at all three open ocean boundaries of the SWAN model. During the SWAN wave direction sensitivity analysis of this study, the directions of incoming wave propagation at the three (3) model ocean boundary segments were specified for four (4) direction (wave from) scenarios: West, Northwest, North, Northeast and East. Incoming wave boundary conditions were held constant over each ocean boundary segment. All SWAN model runs made in this study were Static (constant in time), wherein a model convergence criterion of 99.5 % of steady-state conditions was imposed at all model nodes.

Task 2 - Apply Wave Modeling Tool

Subtask 1 – Model Existing Conditions

The SWAN wave model developed and tested during Task 1 for Existing Conditions was subsequently applied, during a sensitivity analysis for 5 boundary input wave angles and 3 still-water depth assumptions (a total of 15 runs). Still-water depth assumptions included: 1) worst-case maximum (2.9 meters), corresponding to a spring high tide (1.37 meters) plus a storm surge of (1.52 meters), 2) spring high tide only with no storm surge (1.37 meters), and 3) tides at MLLW datum. Worse-case storm surge was estimated as the sum of: 1) wind-induced storm surge (0.2 meters) using the method of Bretschneider (1966), 2) inverted barometer setup (0.7 meters) using the method of Herbich (1990), and 3) a moderate wave setup inside the surf zone (0.6 meters) using the method of FEMA (2005). The sensitivity analysis yielded existing condition waves within Alcan Cove and at the pier site, during a 100-year design wave event. Existing condition wave Hs values were generated with the SWAN model for all nodes within the model spatial domain. For the purpose of 2-d plotting, model predicted nodal Hs values for Alcan Cove nodes were interpolated onto a uniform grid, resulting in one (1) million data points for each plot. Figures 9 and 10 show 2-d color plots of SWAN input water depths (meters below MLLW), within the entire domain and just within Alcan Cove, respectively. Figure 10 also shows 4 locations in the vicinity of the Alcan Cove Pier, where detailed model results, including Hs, wave propagation angles and still-water depths were saved, during each of the 15 existing condition sensitivity runs. Existing Condition results for these 4 pier side locations are given in Table 2. It is seen that the worse-case ocean boundary wave approach angle is from the Northeast, at pier side location 2 (just northeast of north end pier), with an Hs value of 5.477 meters (18 feet).

SWAN model sensitivity analysis results were also used to develop 2-d wave height (Hs) color plots for the entire grid and for Alcan Cove (Figures 11 to 16). Corresponding 2-d plots of wave propagation angles are shown for the entire domain in Figure 17 (waves from Northeast only) and for 3 input wave directions (Northwest, North and Northeast) within Alcan Cove (Figures 18 to 20). Peak wave periods (Tp) were essentially constant over the entire model domain, at 14.85 seconds. Accordingly, 2-d plots of Tp were not generated.

SWAN results given in Table 2 were compared to predicted extreme waves at the pier, during 2 previous studies (Corps of Engineers and CH2MHill). The COE study predicted a worse-case Hs between of 5.5 meters near the pier, for a 50-year return period event. Their results, which utilized approximate methods, are found to be similar to those contained in Table 2, for a 100-year offshore wave input event. Even though the return periods were different, the resulting wave Hs at the pier were similar, due to the fact that very large waves entering Alcan Cove from offshore break due to shallow water and are further depth-limited near the pier location.

Subtask 2 – Model Future Conditions with Breakwater

The SWAN model developed in Subtask 1 was subsequently modified to investigate impacts of a breakwater on reducing extreme wave conditions within Alcan Cove and at the pier facilities. Figure 21 shows footprints of a 2 breakwater conceptual design developed by the Corps of Engineers (Seattle District) in 1944, for protection of Alcan Cove pier facilities from wave action. For purposes of the SWAN wave modeling of the present study, only the western 1944 breakwater footprint centerline was included, as a sub-grid obstacle, with zero wave reflection and transmission.

Based on the SWAN results given in Table 3, for conditions following construction of the 1944 conceptual design western breakwater, it is seen that the worse-case Hs at pier location 2 is 4.697 meters. This result corresponds to offshore ocean boundary waves approaching from the Northeast, a stillwater level 2.9 meters above MLLW. This result suggests that the breakwater will reduce Hs in the vicinity of the pier by as much as 0.78 meters, compared to Existing Conditions without a breakwater.

2-d color plots of worse-case wave Hs and wave Angle following construction of the breakwater, for offshore boundary waves approaching from the Northwest, North and Northeast, are given in Figures 22 to 24, respectively. Corresponding 2-d plots of wave Angles within Alcan Cove are shown in Figures 25 to 27.

List of Figures and Tables

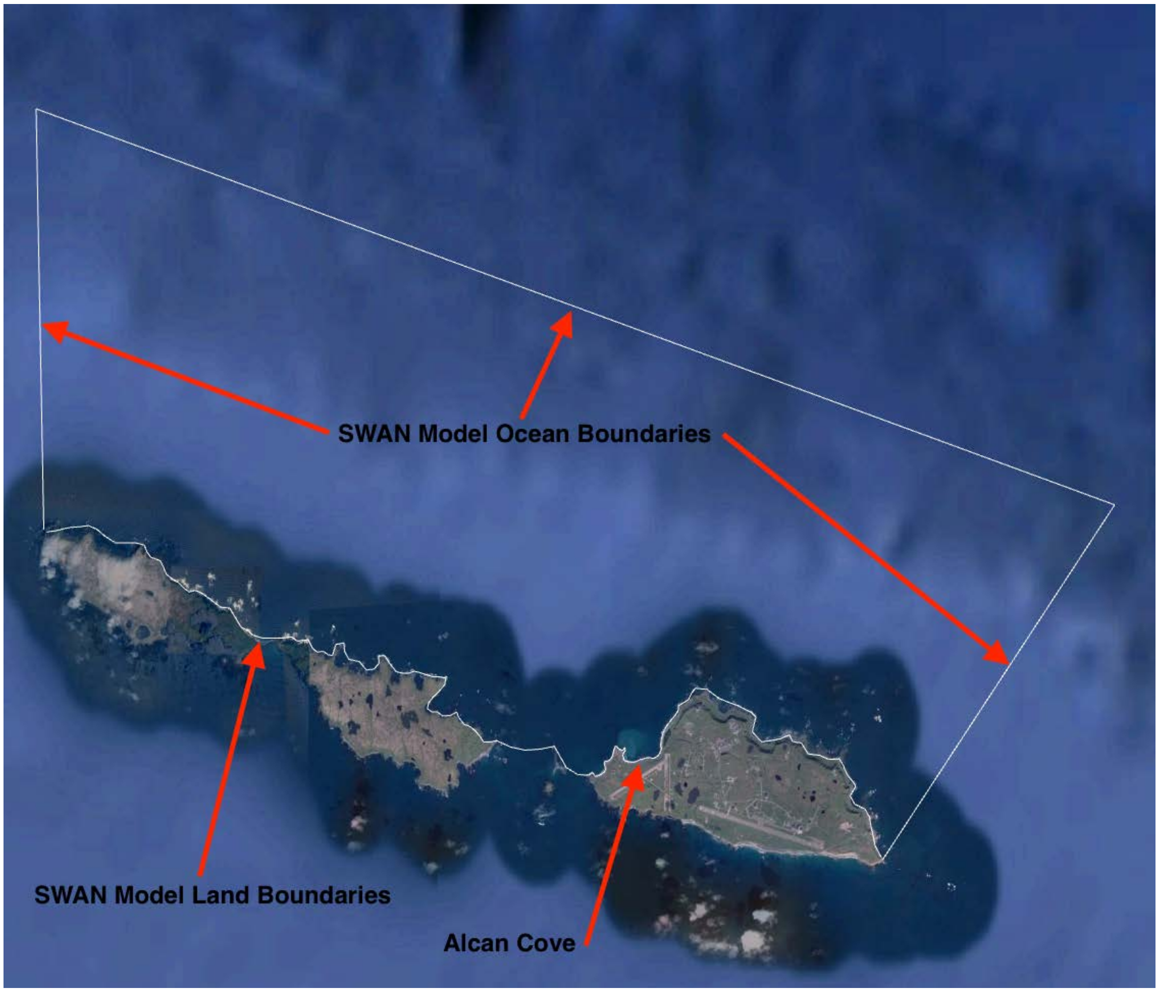


Figure 1: SWAN Model Boundaries

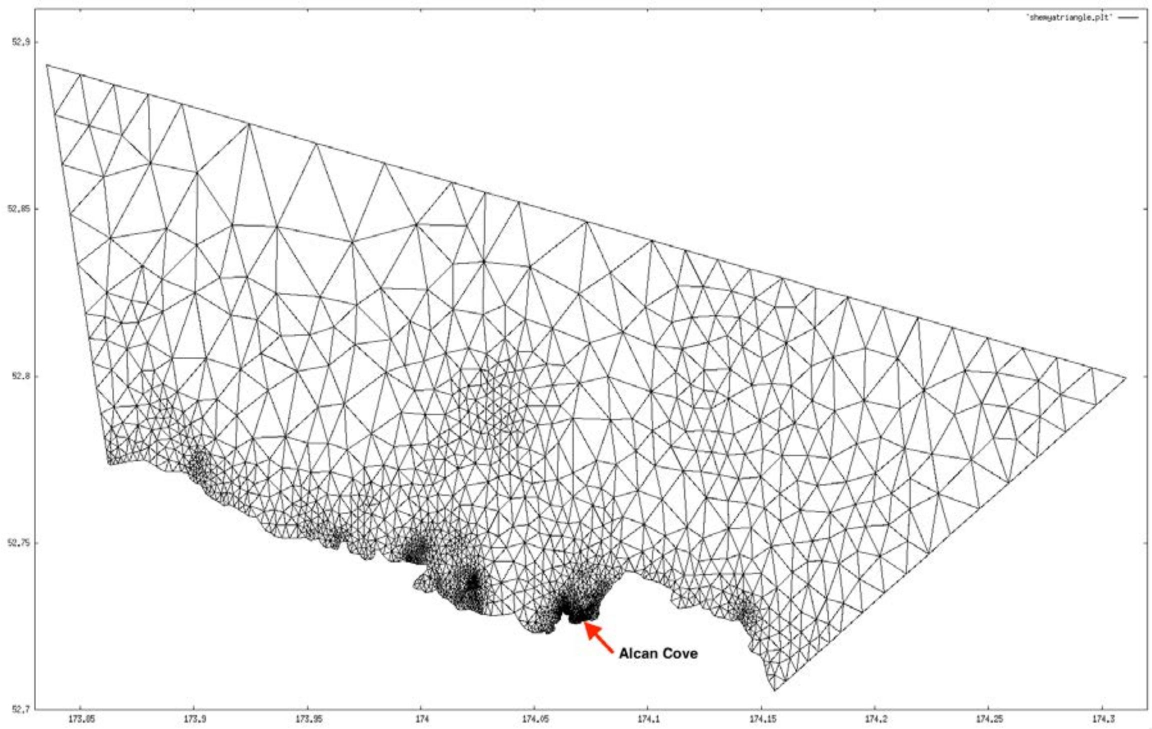


Figure 2: SWAN Finite Element Unstructured Grid for Entire Domain

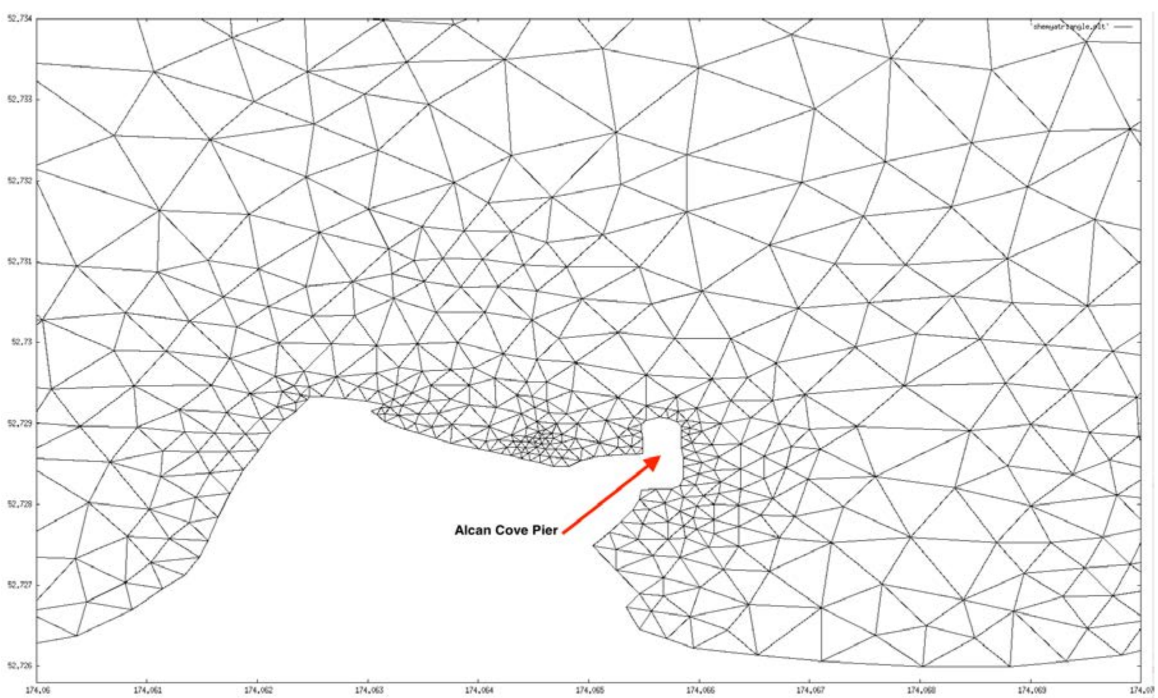


Figure 3: SWAN Finite Element Unstructured Grid in Vicinity of Alcan Cove Pier

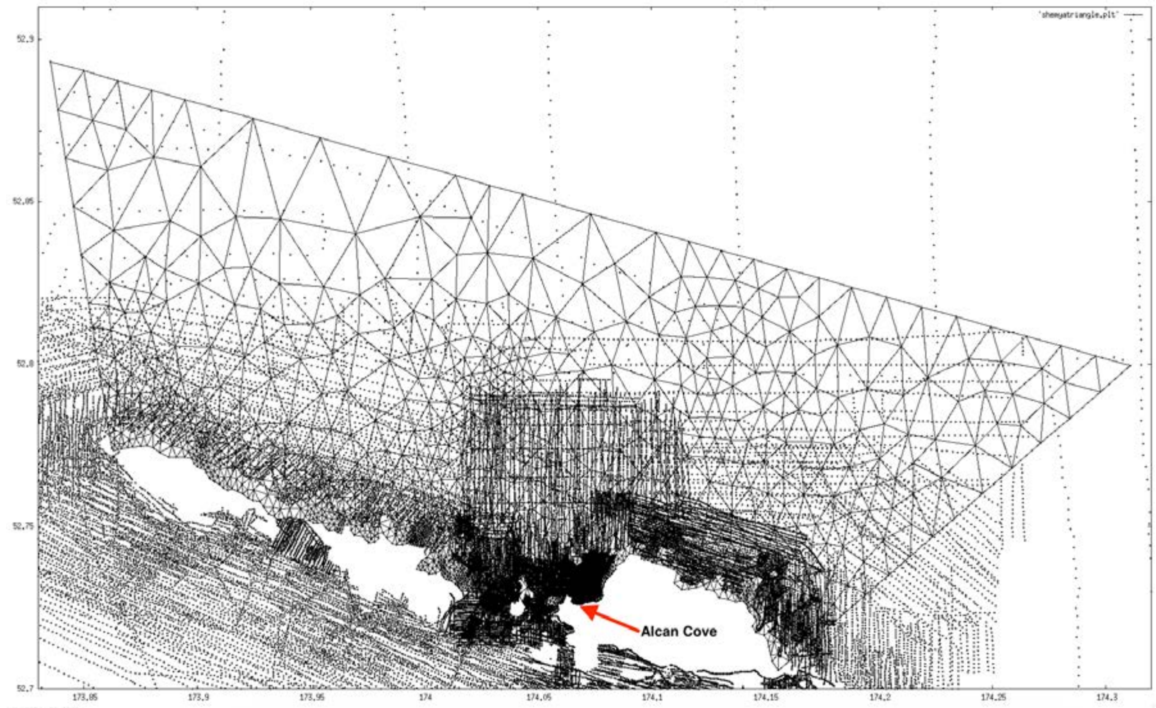


Figure 4: SWAN Grid and Available Depth Data Locations (dots)

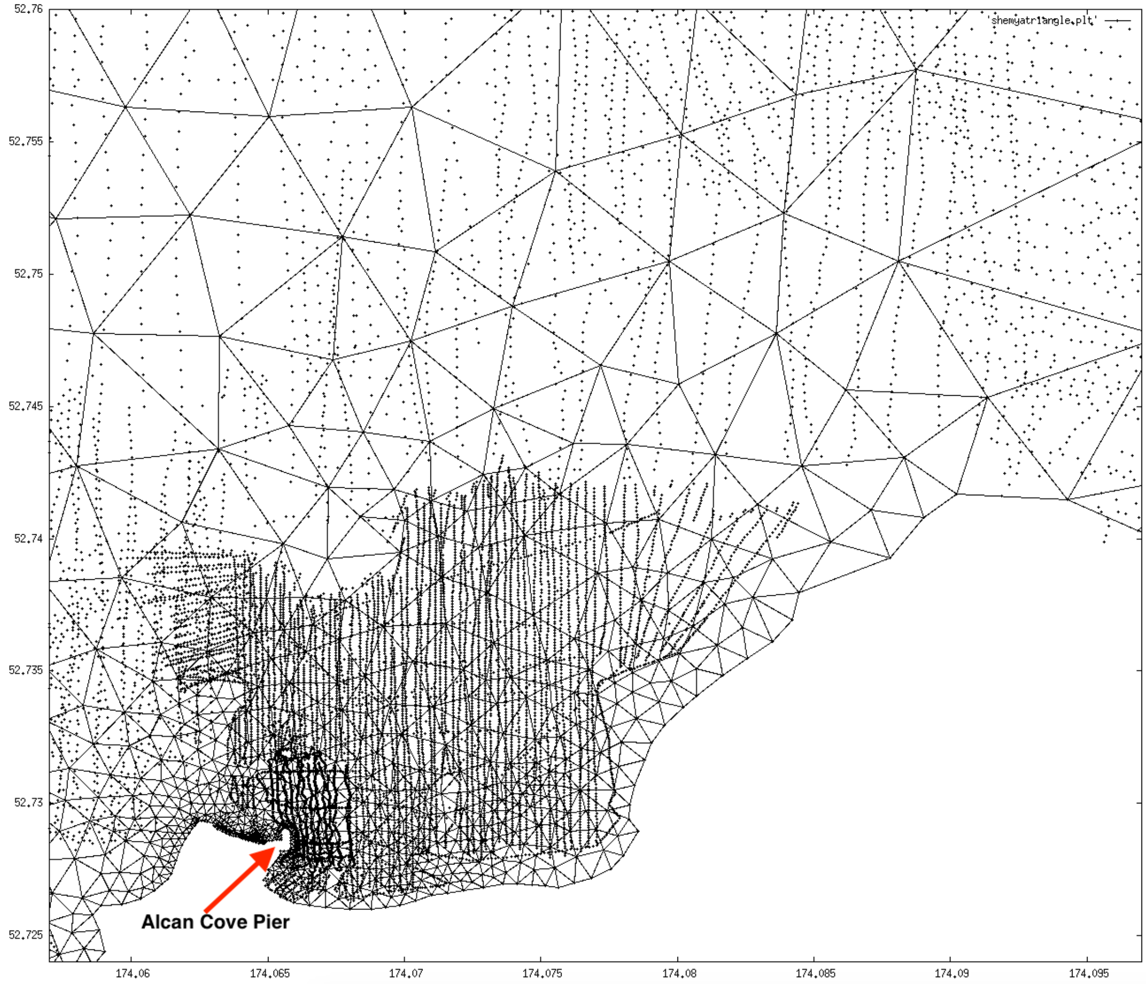


Figure 5: SWAN Grid Within Alcan Cove and Available Depth Data Locations (dots)

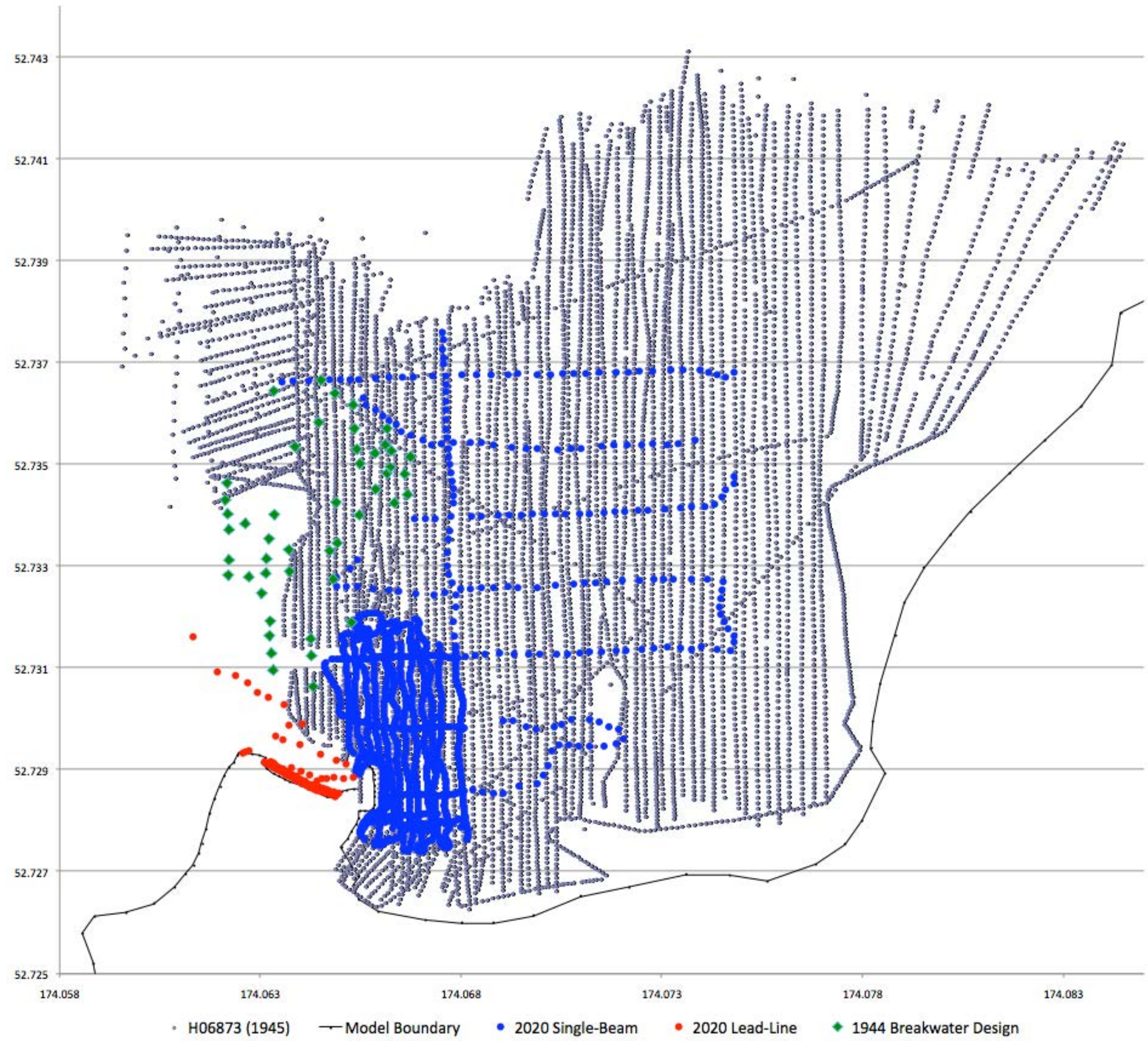


Figure 6: Hydrographic Surveys within Alcan Cove

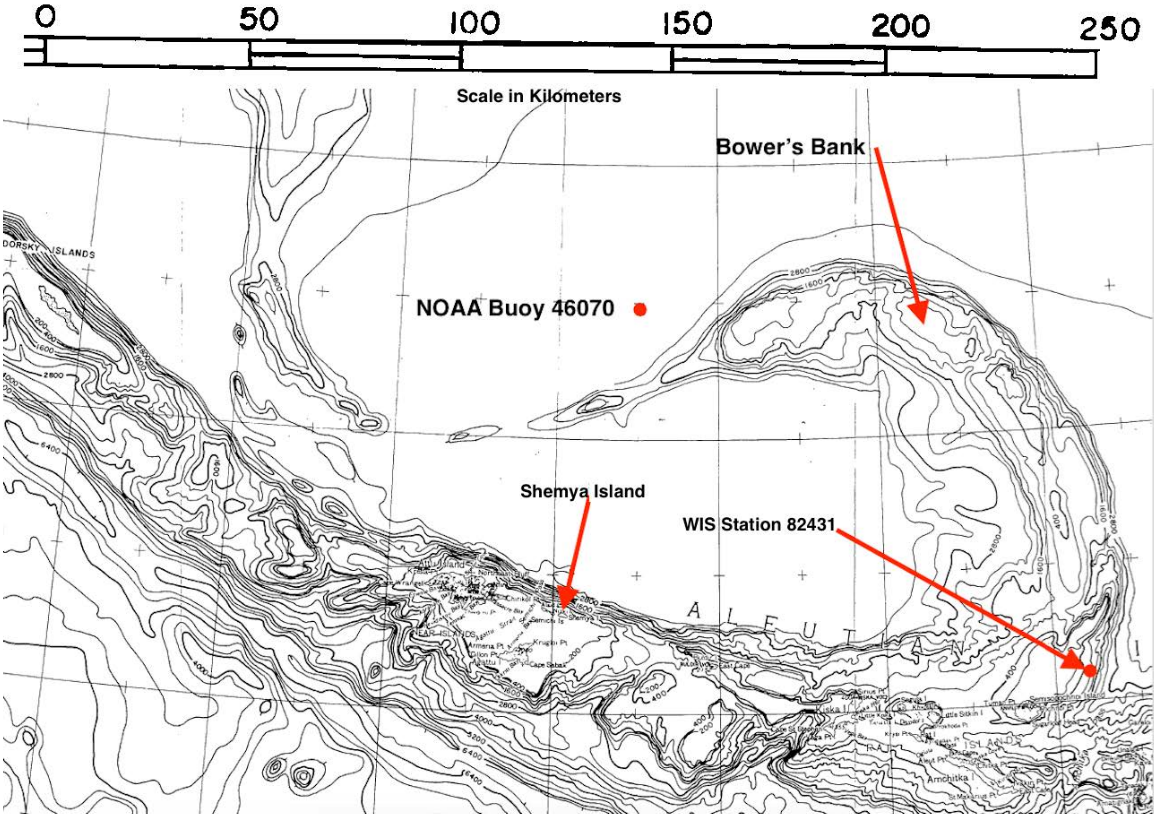
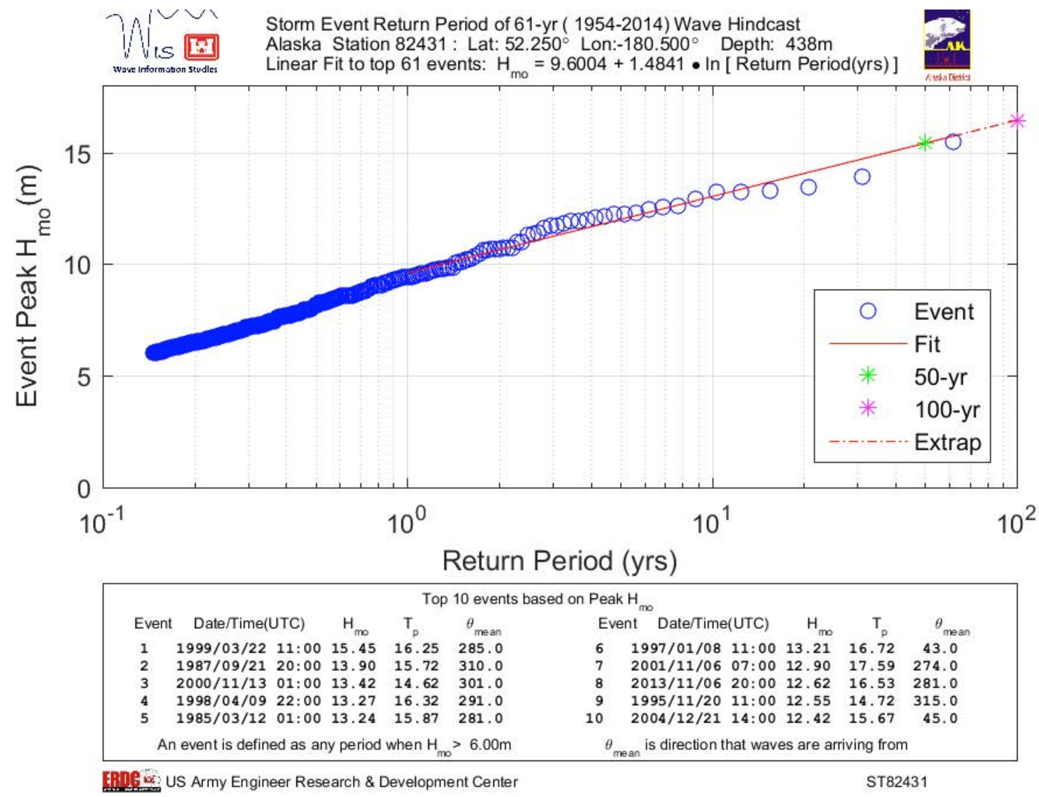


Figure 7: Locations of WIS Station and NOAA Buoy 46070



Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}	Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}
1	1999/03/22 11:00	15.45	16.25	285.0	6	1997/01/08 11:00	13.21	16.72	43.0
2	1987/09/21 20:00	13.90	15.72	310.0	7	2001/11/06 07:00	12.90	17.59	274.0
3	2000/11/13 01:00	13.42	14.62	301.0	8	2013/11/06 20:00	12.62	16.53	281.0
4	1998/04/09 22:00	13.27	16.32	291.0	9	1995/11/20 11:00	12.55	14.72	315.0
5	1985/03/12 01:00	13.24	15.87	281.0	10	2004/12/21 14:00	12.42	15.67	45.0

Figure 8: WIS Station 82431 Hs Frequency Plot of Extreme Wave Events

2/20/2017>>]				100-year E scale factor = 2.17	
E (m2/Hz)	freq bin (Hz)	bin width(Hz)	Mo (E*width)	Mo (E*width)	Upscaled E (m2/Hz)
0	0.0325	0.005	0	0	0.00
0	0.0375	0.005	0	0	0.00
0	0.0425	0.005	0	0	0.00
0	0.0475	0.005	0	0	0.00
8.81	0.0525	0.005	0.04405	0.0955885	19.12
83.35	0.0575	0.005	0.41675	0.9043475	180.87
254.68	0.0625	0.005	1.2734	2.763278	552.66
322.49	0.0675	0.005	1.61245	3.4990165	699.80
251.26	0.0725	0.005	1.2563	2.726171	545.23
138.85	0.0775	0.005	0.69425	1.5065225	301.30
112.12	0.0825	0.005	0.5606	1.216502	243.30
106.22	0.0875	0.005	0.5311	1.152487	230.50
44.28	0.0925	0.0075	0.3321	0.720657	96.09
44.93	0.1	0.01	0.4493	0.974981	97.50
38.14	0.11	0.01	0.3814	0.827638	82.76
17.43	0.12	0.01	0.1743	0.378231	37.82
13.61	0.13	0.01	0.1361	0.295337	29.53
12.13	0.14	0.01	0.1213	0.263221	26.32
9.89	0.15	0.01	0.0989	0.214613	21.46
3.98	0.16	0.01	0.0398	0.086366	8.64
4.65	0.17	0.01	0.0465	0.100905	10.09
2.23	0.18	0.01	0.0223	0.048391	4.84
4.61	0.19	0.01	0.0461	0.100037	10.00
1.88	0.2	0.01	0.0188	0.040796	4.08
1.16	0.21	0.01	0.0116	0.025172	2.52
1.07	0.22	0.01	0.0107	0.023219	2.32
1.2	0.23	0.01	0.012	0.02604	2.60
1.05	0.24	0.01	0.0105	0.022785	2.28
0.46	0.25	0.01	0.0046	0.009982	1.00
0.31	0.26	0.01	0.0031	0.006727	0.67
0.44	0.27	0.01	0.0044	0.009548	0.95
0.38	0.28	0.01	0.0038	0.008246	0.82
0.36	0.29	0.01	0.0036	0.007812	0.78
0.2	0.3	0.01	0.002	0.00434	0.43
0.22	0.31	0.01	0.0022	0.004774	0.48
0.18	0.32	0.01	0.0018	0.003906	0.39
0.09	0.33	0.01	0.0009	0.001953	0.20
0.16	0.34	0.01	0.0016	0.003472	0.35
0.16	0.35	0.015	0.0024	0.005208	0.35
0.08	0.365	0.02	0.0016	0.003472	0.17
0.08	0.385	0.02	0.0016	0.003472	0.17
0.05	0.405	0.02	0.001	0.00217	0.11
0.03	0.425	0.02	0.0006	0.001302	0.07
0.02	0.445	0.02	0.0004	0.000868	0.04
0.02	0.465	0.02	0.0004	0.000868	0.04
0.01	0.485	0.02	0.0002	0.000434	0.02
		Hs (m) >>>	11.54940691	17.01333877	

Table 1: Wave Boundary Condition Energy Density Scaling Methodology

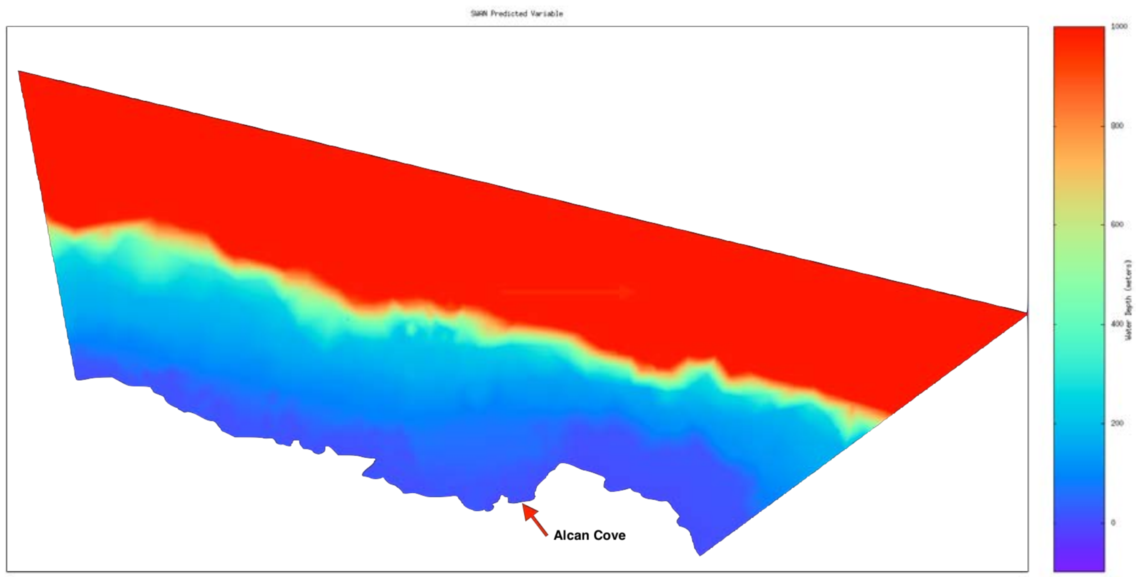


Figure 9: SWAM Input Depths for Entire Domain (meters below MLLW)

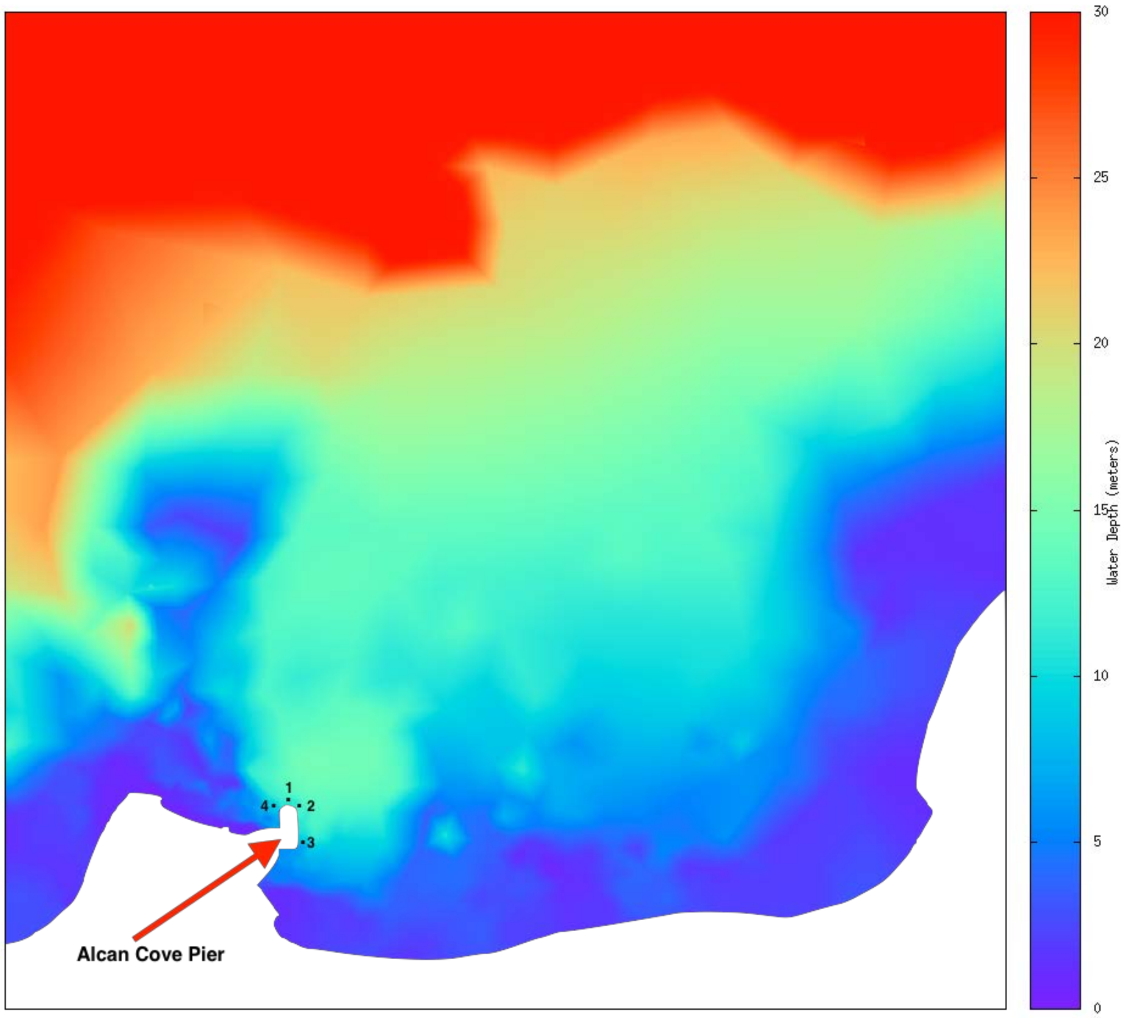


Figure 10: SWAN Input Depths for Alcan Cove (m below MLLW)

SWAN Wave Model Results Matrix

EXISTING CONDITIONS
Stillwater Elevation at 2.9 m (High Tide=1.37 m MLLW)+(Surge=1.52 m)
Tp from all runs = 14.84 seconds

		Significant Wave Height (m) worst-case Stillwater Level					Wave Angle Towards in Cartesian (deg)				
Depth (m)	Location	W	NW	N	NE	E	W	NW	N	NE	E
15.0	1	4.120	4.698	5.161	5.275	4.807	265	263	259	258	253
14.4	2	4.156	4.787	5.346	5.477	4.990	268	267	263	261	257
13.1	3	3.497	4.096	4.693	4.836	4.437	262	261	259	258	255
9.0	4	3.448	3.901	4.254	4.325	3.967	258	256	254	253	251

EXISTING CONDITIONS
Stillwater Elevation at 1.37 m (High Tide=1.37 m MLLW)

		Significant Wave Height (m)					Wave Angle Towards in Cartesian (deg)				
Depth (m)	Location	W	NW	N	NE	E	W	NW	N	NE	E
13.5	1	3.652	4.126	4.573	4.675	4.371	263	261	258	256	253
12.9	2	3.746	4.255	4.802	4.914	4.589	267	265	262	261	257
11.6	3	3.196	3.677	4.234	4.346	4.068	261	260	258	257	255
7.5	4	3.010	3.369	3.717	3.784	3.551	255	254	251	251	249

EXISTING CONDITIONS
Stillwater Elevation at 0.0 m (MLLW)

		Significant Wave Height (m)					Wave Angle Towards in Cartesian (deg)				
Depth (m)	Location	W	NW	N	NE	E	W	NW	N	NE	E
12.1	1	3.231	3.627	4.038	4.151	3.953	260	258	256	255	252
11.5	2	3.357	3.796	4.308	4.437	4.206	265	263	261	259	256
10.2	3	2.900	3.304	3.805	3.927	3.714	259	259	257	257	254
6.1	4	2.622	2.925	3.229	3.301	3.141	252	251	250	249	248

Table 2: SWAN Existing Conditions Results Matrix for 4 Pier Locations (15 runs)

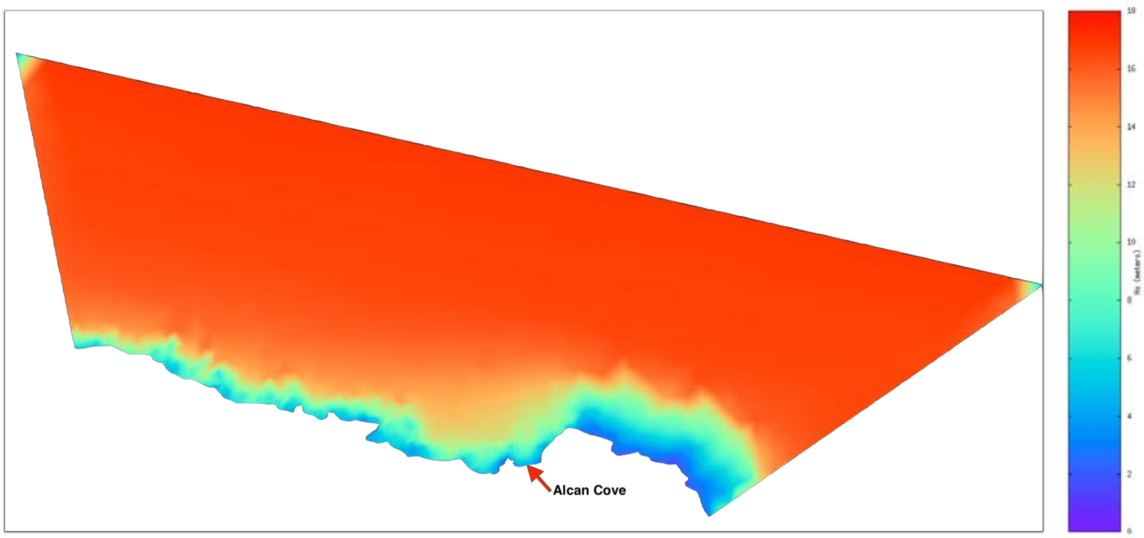


Figure 11: SWAN Predicted Hs for Entire Domain, Waves from Northeast

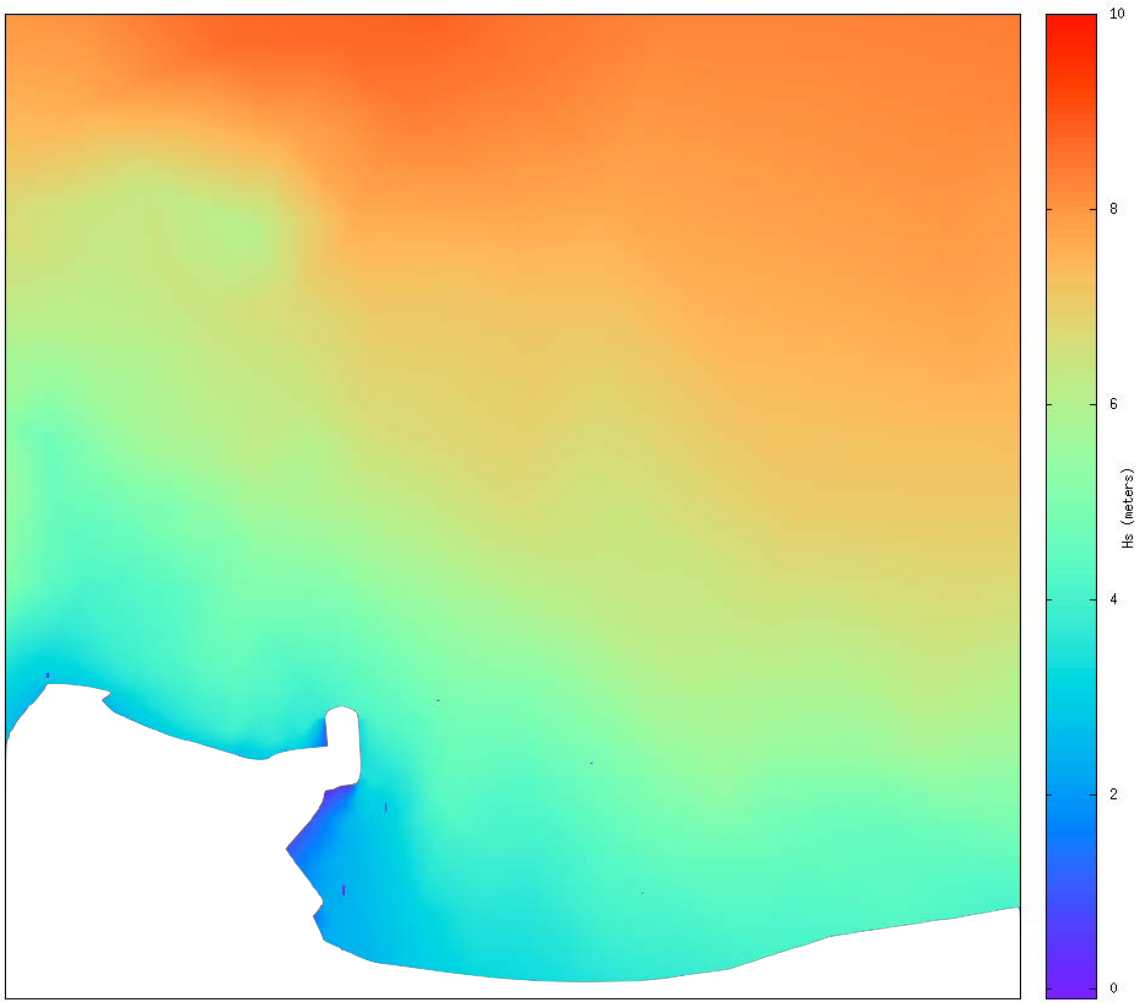


Figure 12: SWAN Predicted Hs for Alcan Cove, Waves from West (meters)

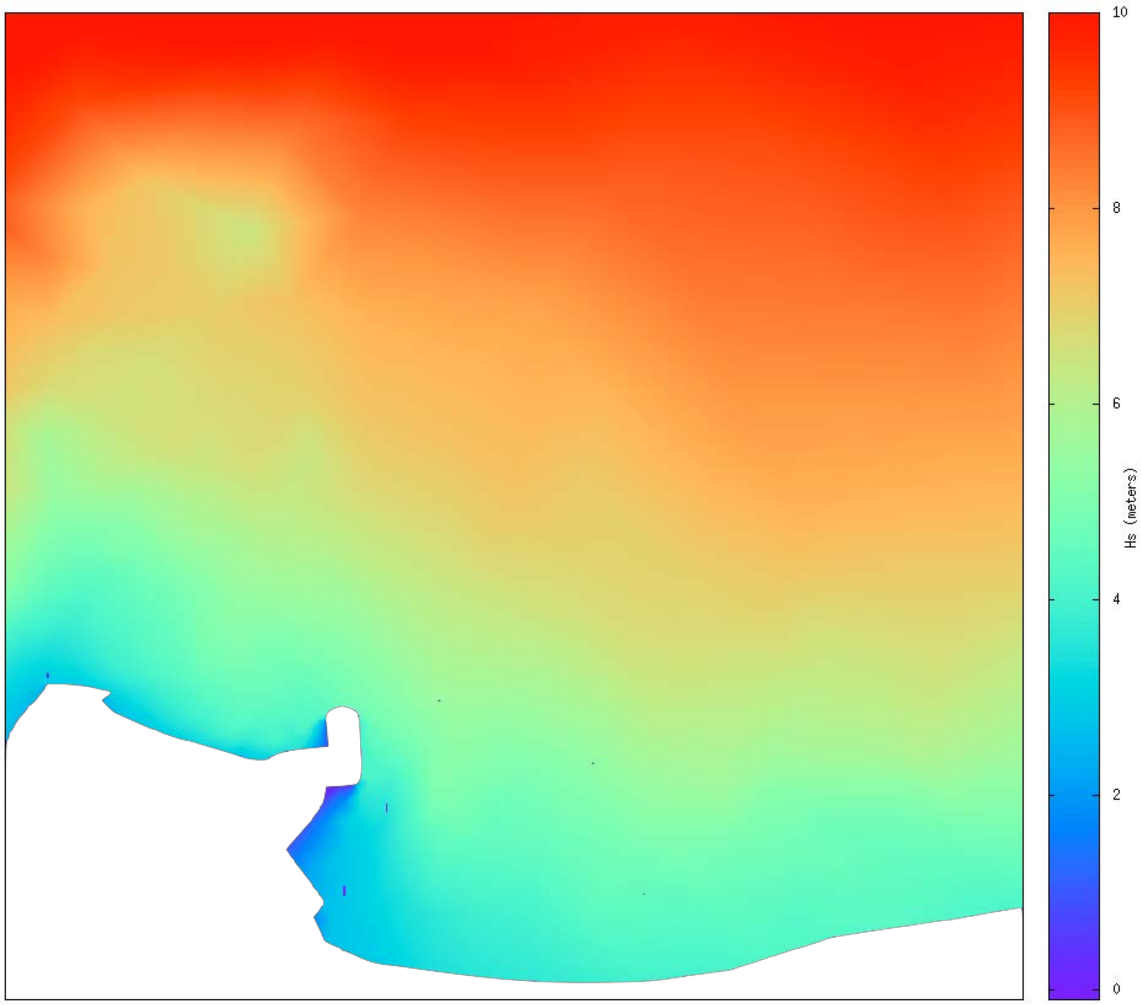


Figure 13: SWAN Predicted Hs for Alcan Cove, Waves from Northwest (meters)

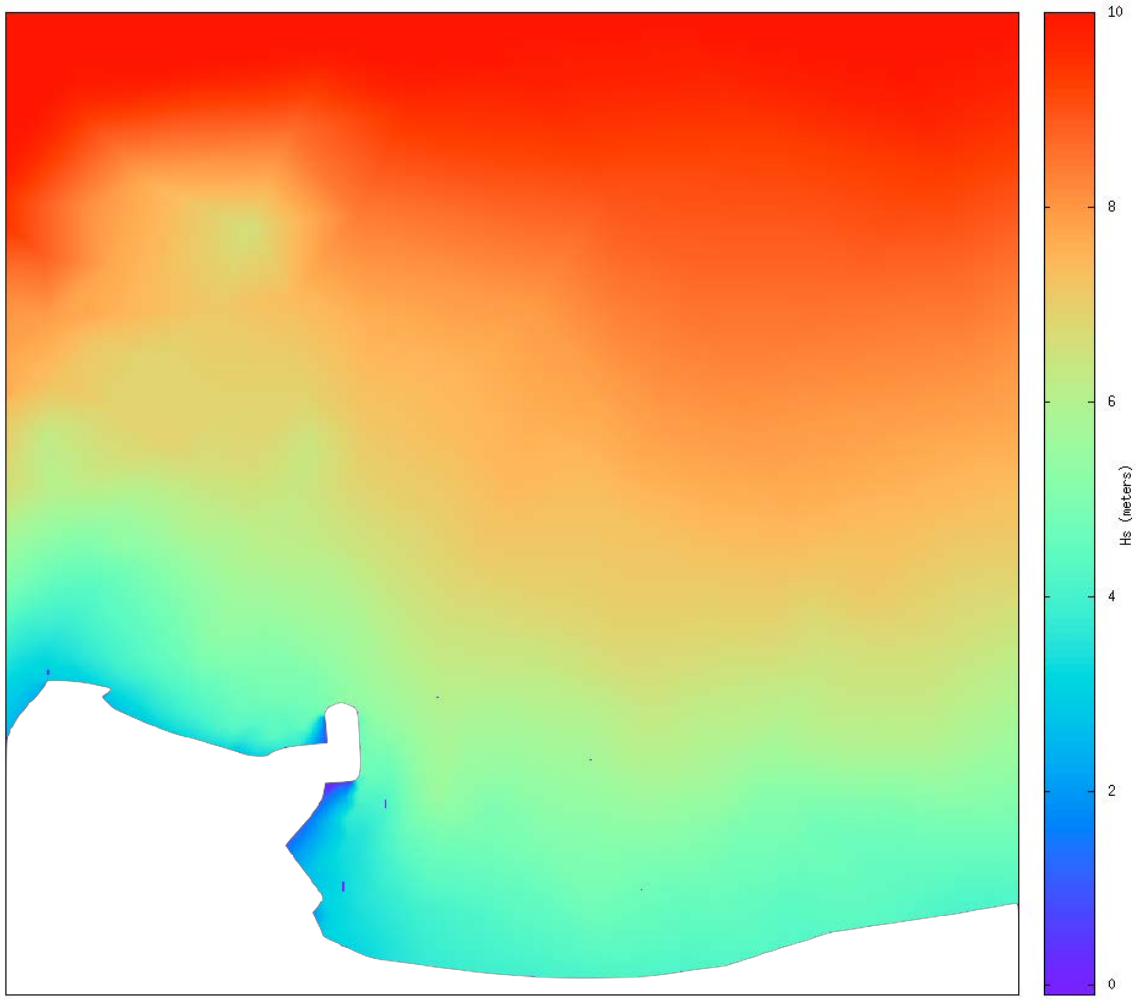


Figure 14: SWAN Predicted Hs for Alcan Cove, Waves from North (meters)

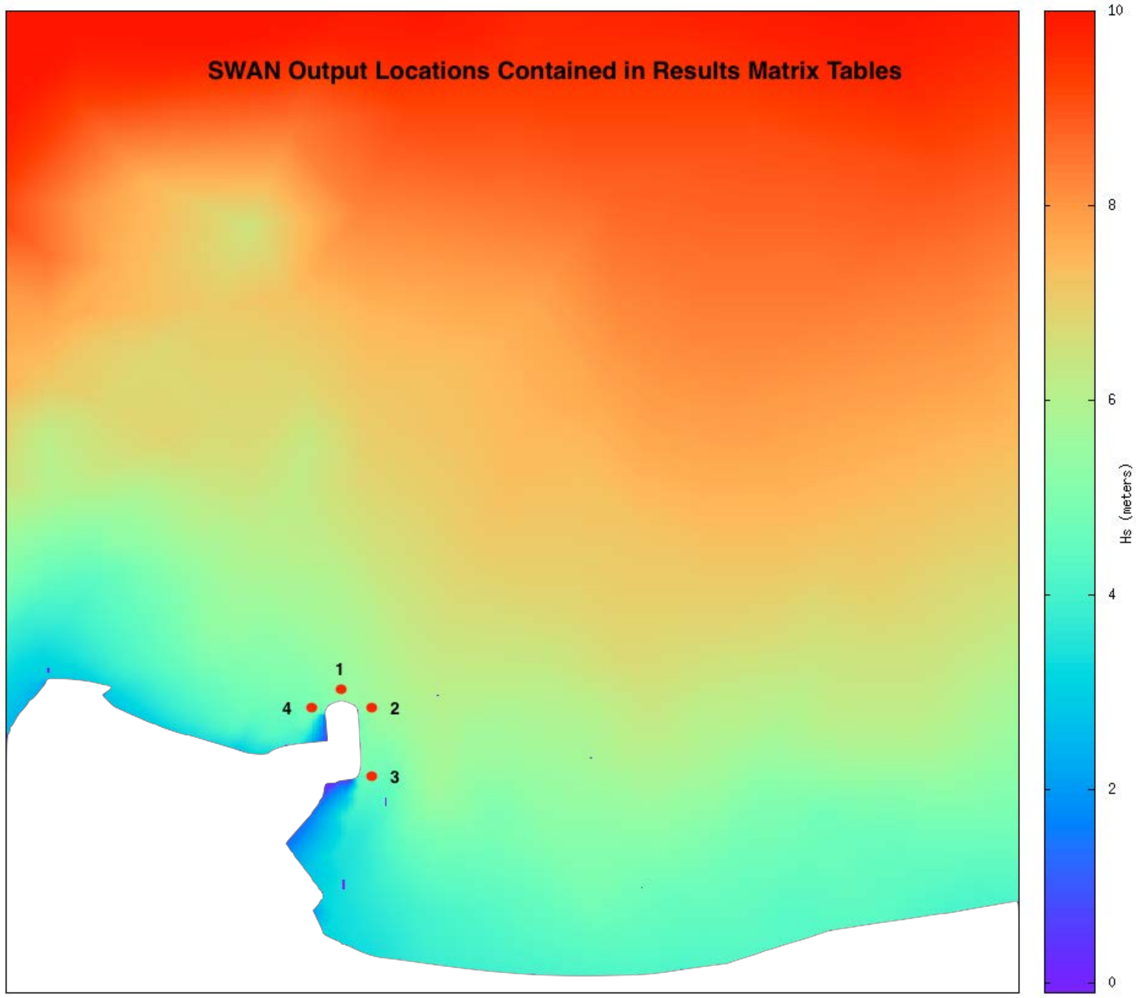


Figure 15: SWAN Predicted Hs for Alcan Cove, Waves from Northeast

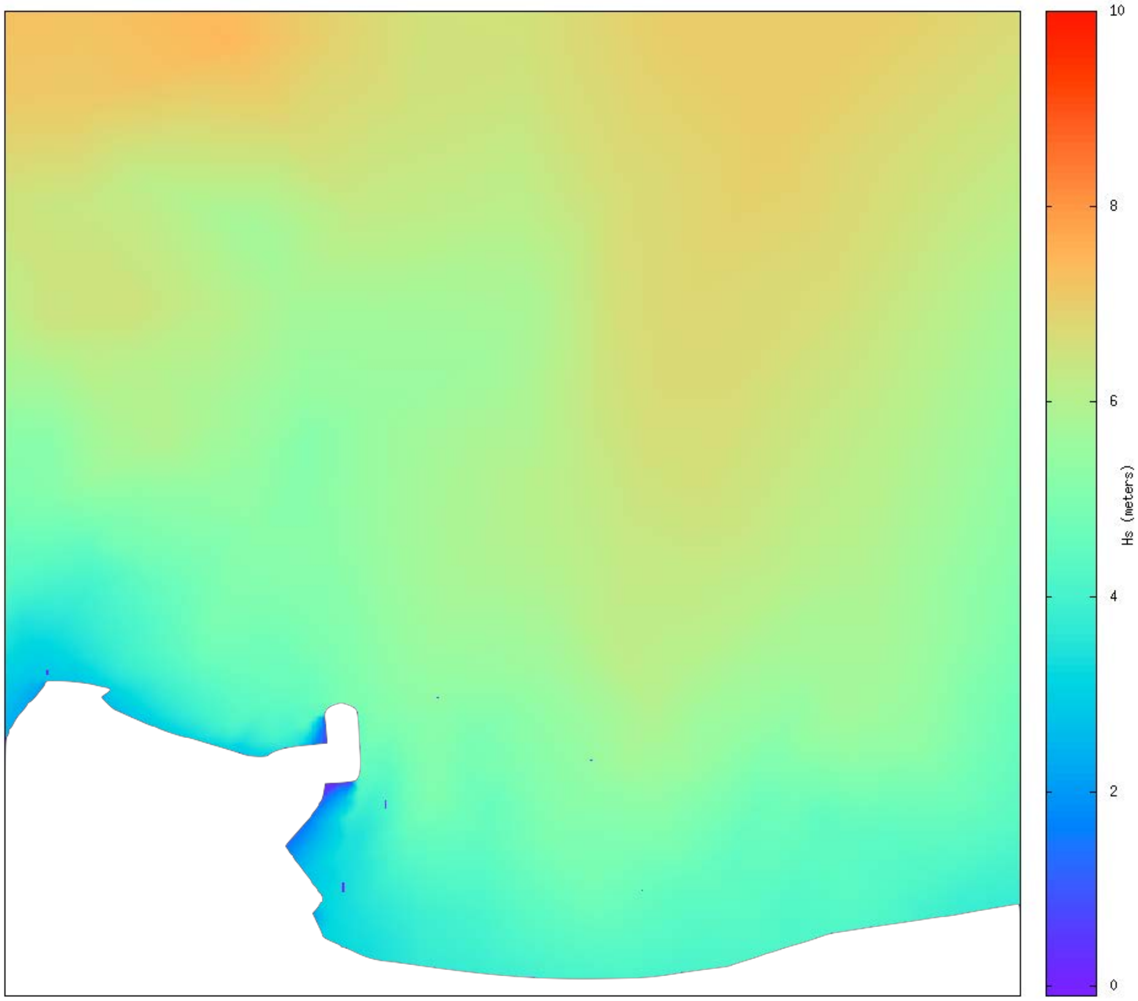


Figure 16: SWAN Predicted Hs for Alcan Cove, Waves from East

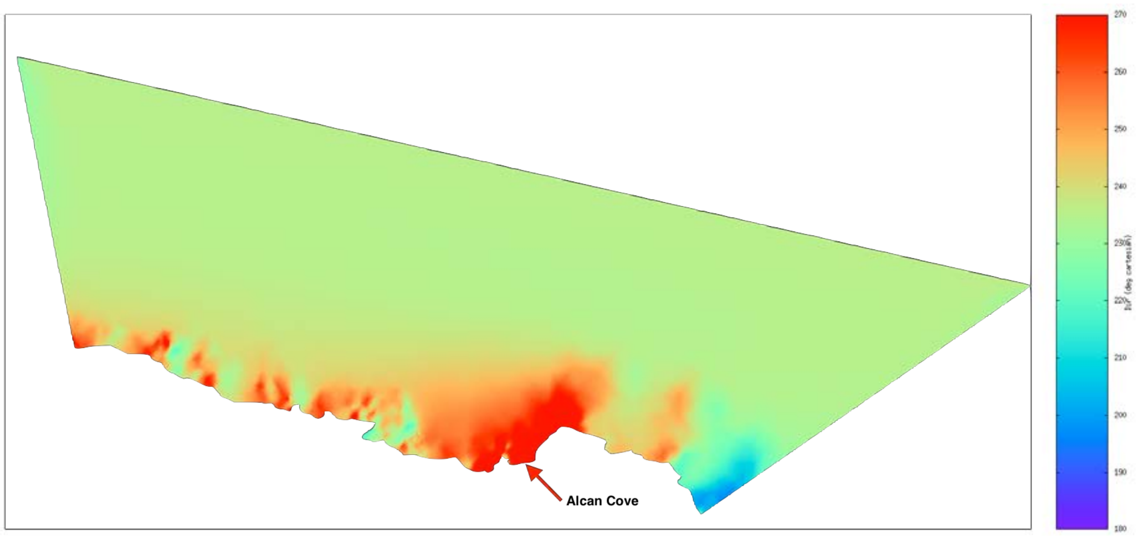


Figure 17: SWAN Predicted Angles for Entire Domain, Waves from Northeast

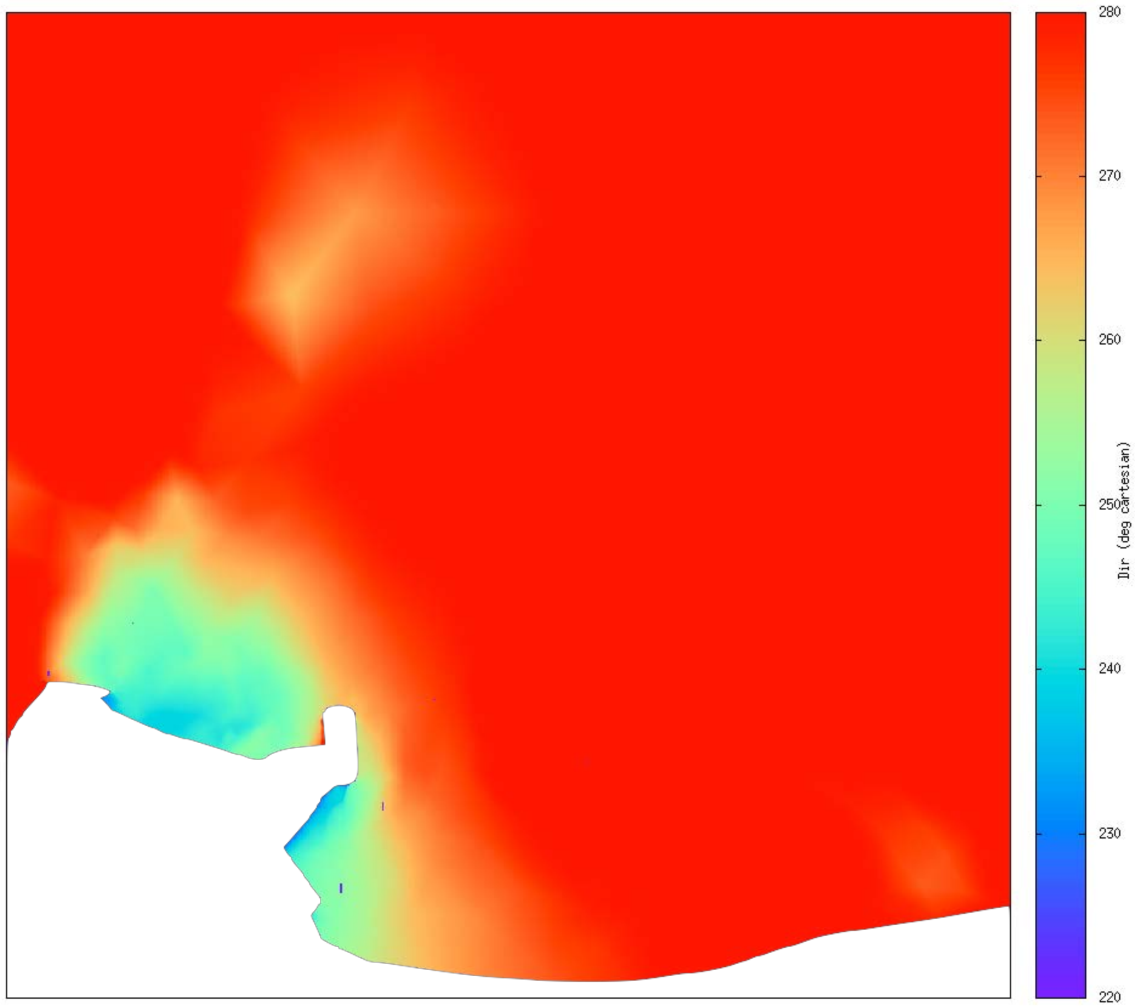


Figure 18: SWAN Predicted Angles for Alcan Cove, Waves from Northwest

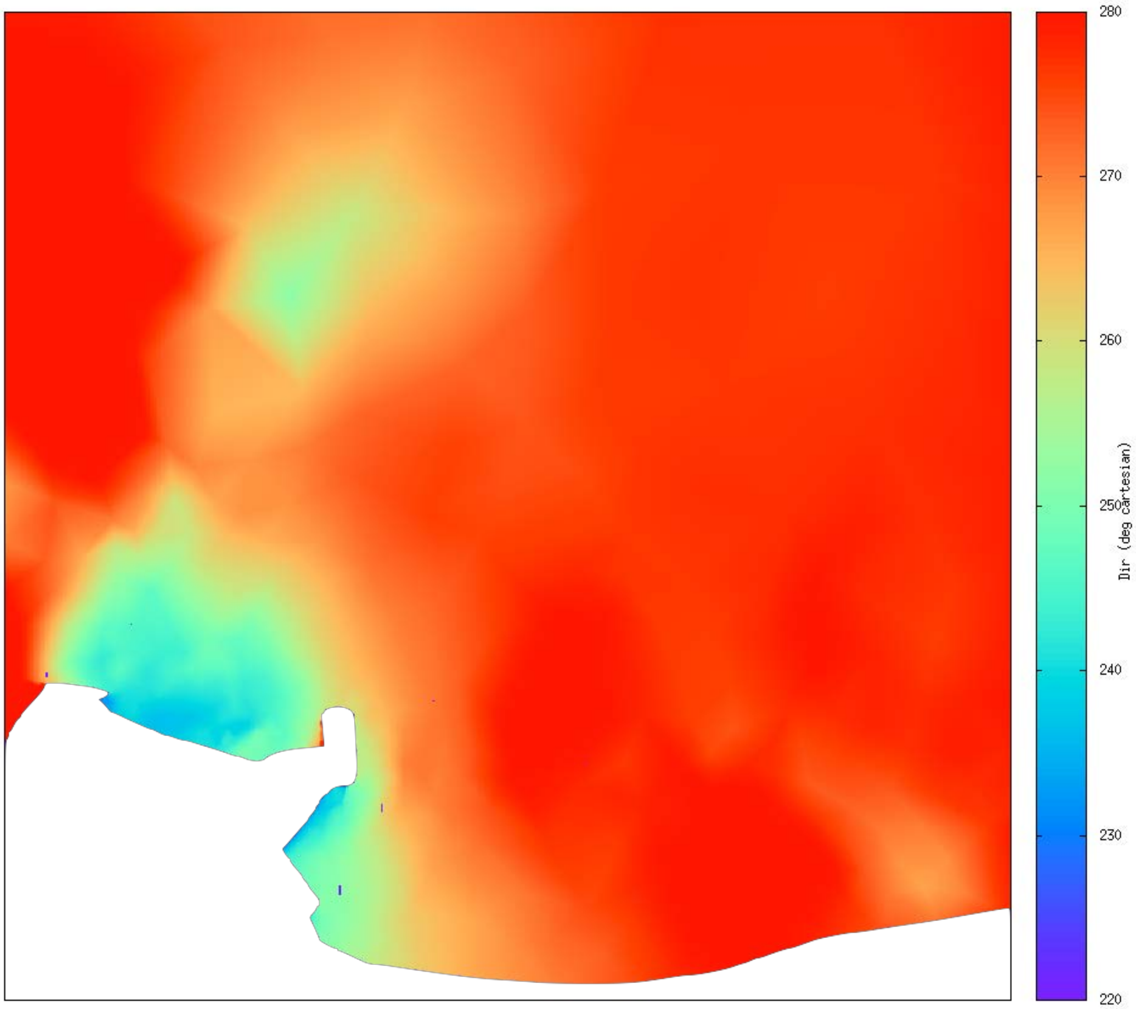


Figure 19: SWAN Predicted Angles for Alcan Cove, Waves from North

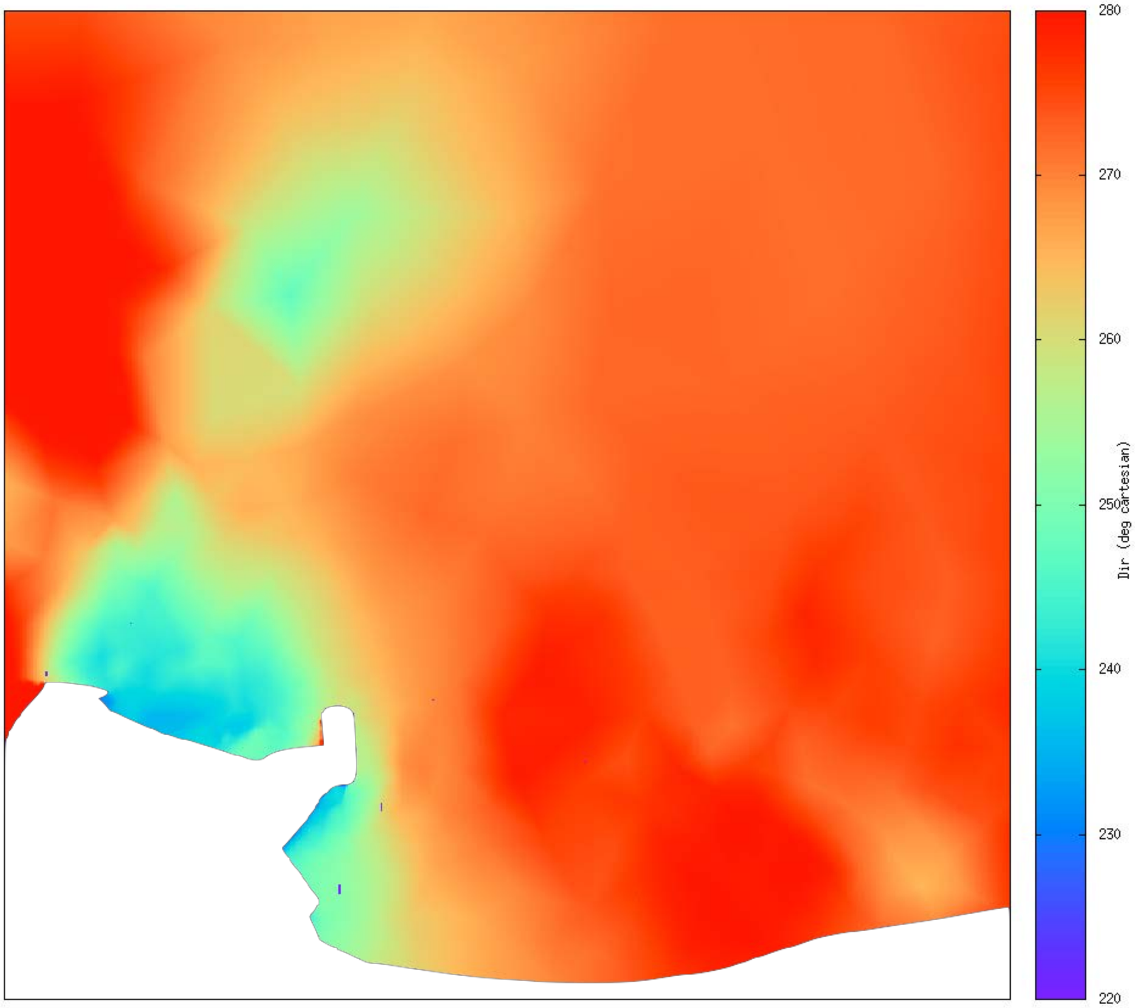


Figure 20: SWAN Predicted Angles for Alcan Cove, Waves from Northeast

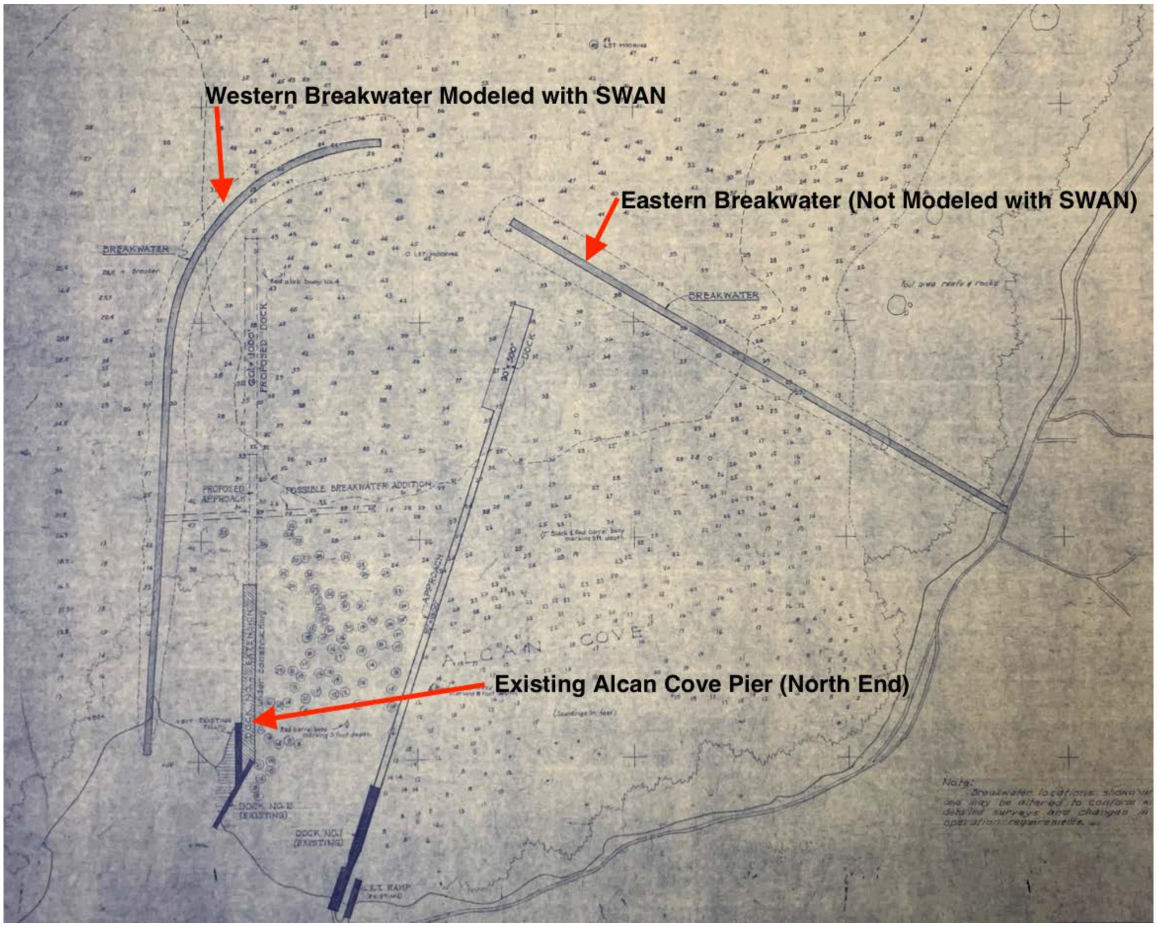


Figure 21: 1944 Western Breakwater Conceptual Design Plan

SWAN Wave Model Results Matrix

WITH 1944 BREAKWATER
Stillwater Elevation at 2.9 m (High Tide=1.37 m MLLW)+(Surge=1.52 m)

		Significant Wave Height (m) worst-case Stillwater Level					Wave Angle Towards in Cartesian (deg)				
Depth (m)	Location	W	NW	N	NE	E	W	NW	N	NE	E
15.0	1	1.977	2.787	3.812	4.149	4.178	249	253	252	251	248
14.4	2	2.203	3.252	4.377	4.697	4.562	255	258	257	257	253
13.1	3	2.105	3.065	4.079	4.349	4.178	254	257	256	256	253
9.0	4	1.560	2.300	3.196	3.472	3.502	253	251	249	249	248

WITH 1944 BREAKWATER
Stillwater Elevation at 1.37 m (High Tide=1.37 m MLLW)

		Significant Wave Height (m)					Wave Angle Towards in Cartesian (deg)				
Depth (m)	Location	W	NW	N	NE	E	W	NW	N	NE	E
13.5	1	1.914	2.603	3.504	3.780	3.877	250	253	252	251	248
12.9	2	2.154	3.052	4.060	4.317	4.271	256	258	257	257	254
11.6	3	2.039	2.852	3.752	3.965	3.876	254	257	256	256	253
7.5	4	1.522	2.154	2.933	3.142	3.237	252	250	249	248	247

WITH 1944 BREAKWATER
Stillwater Elevation at 0.0 m (MLLW)

		Significant Wave Height (m)					Wave Angle Towards in Cartesian (deg)				
Depth (m)	Location	W	NW	N	NE	E	W	NW	N	NE	E
12.1	1	1.840	2.429	3.211	3.455	3.574	250	252	252	251	249
11.5	2	2.093	2.862	3.748	3.084	3.973	257	258	257	257	254
10.2	3	1.960	2.647	3.428	3.625	3.574	255	257	256	256	254
6.1	4	1.486	2.022	2.650	2.834	2.933	251	249	248	248	247

Table 3: With Breakwater - SWAN Results Matrix (15 runs)

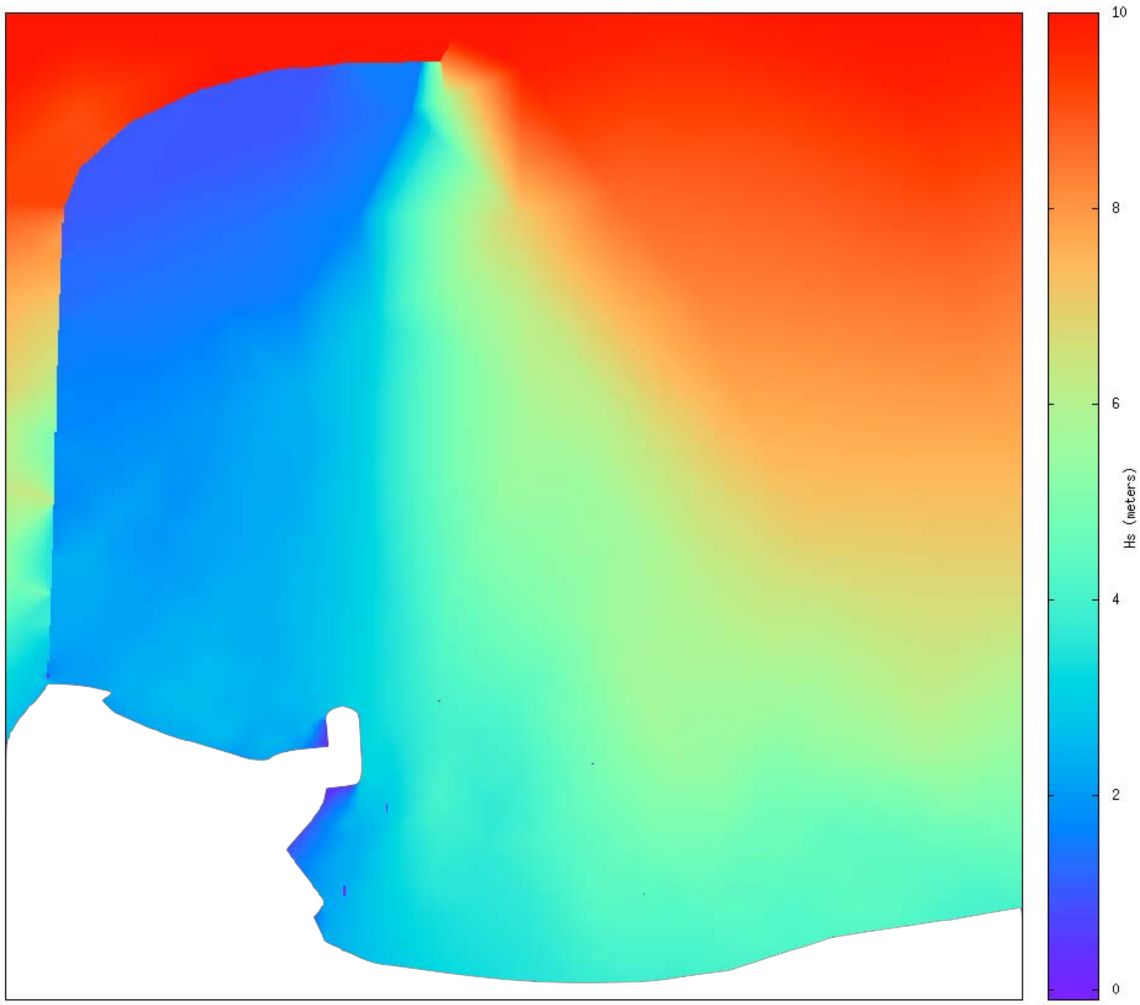


Figure 22: With Breakwater - SWAN Predicted Hs for Alcan Cove, Waves from Northwest

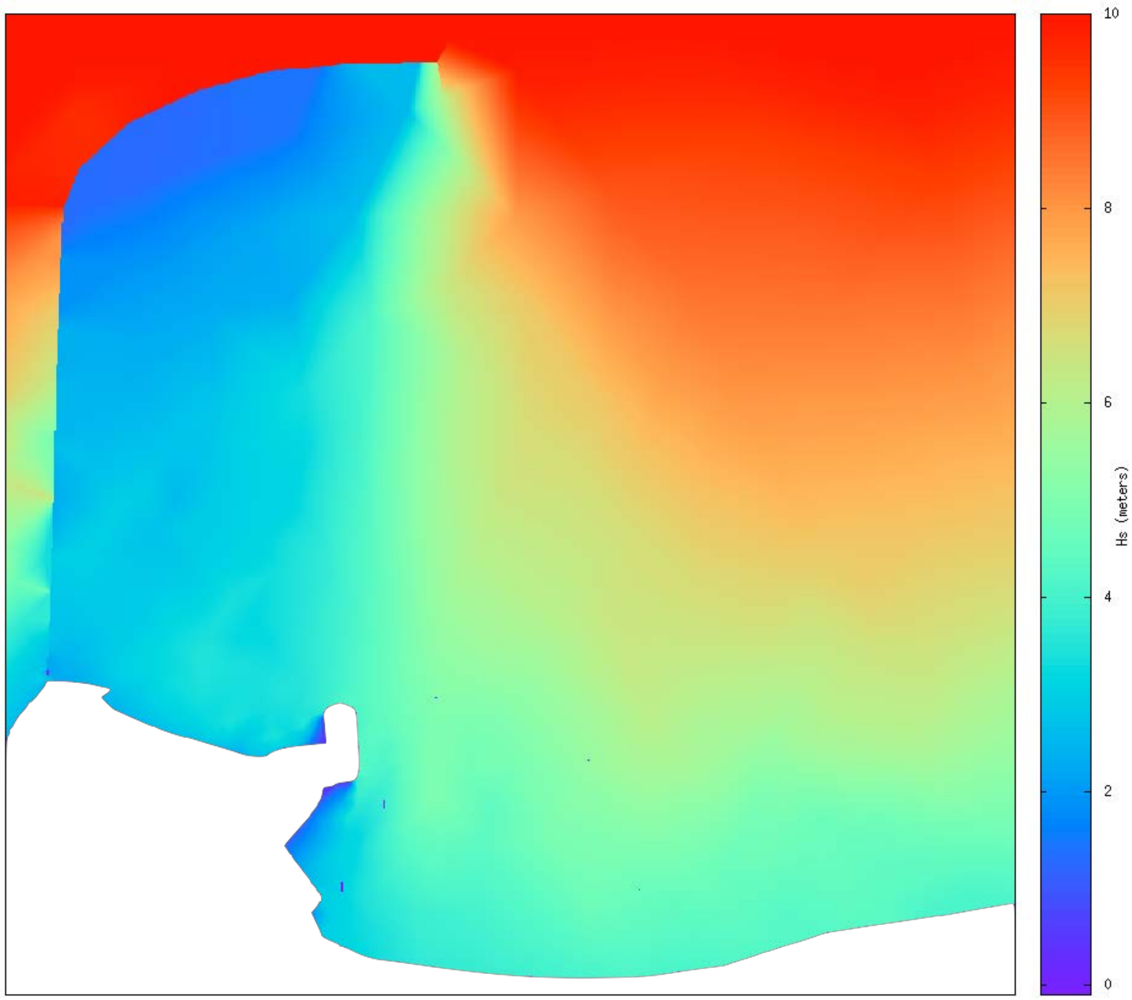


Figure 23: With Breakwater - SWAN Predicted Hs for Alcan Cove, Waves from North

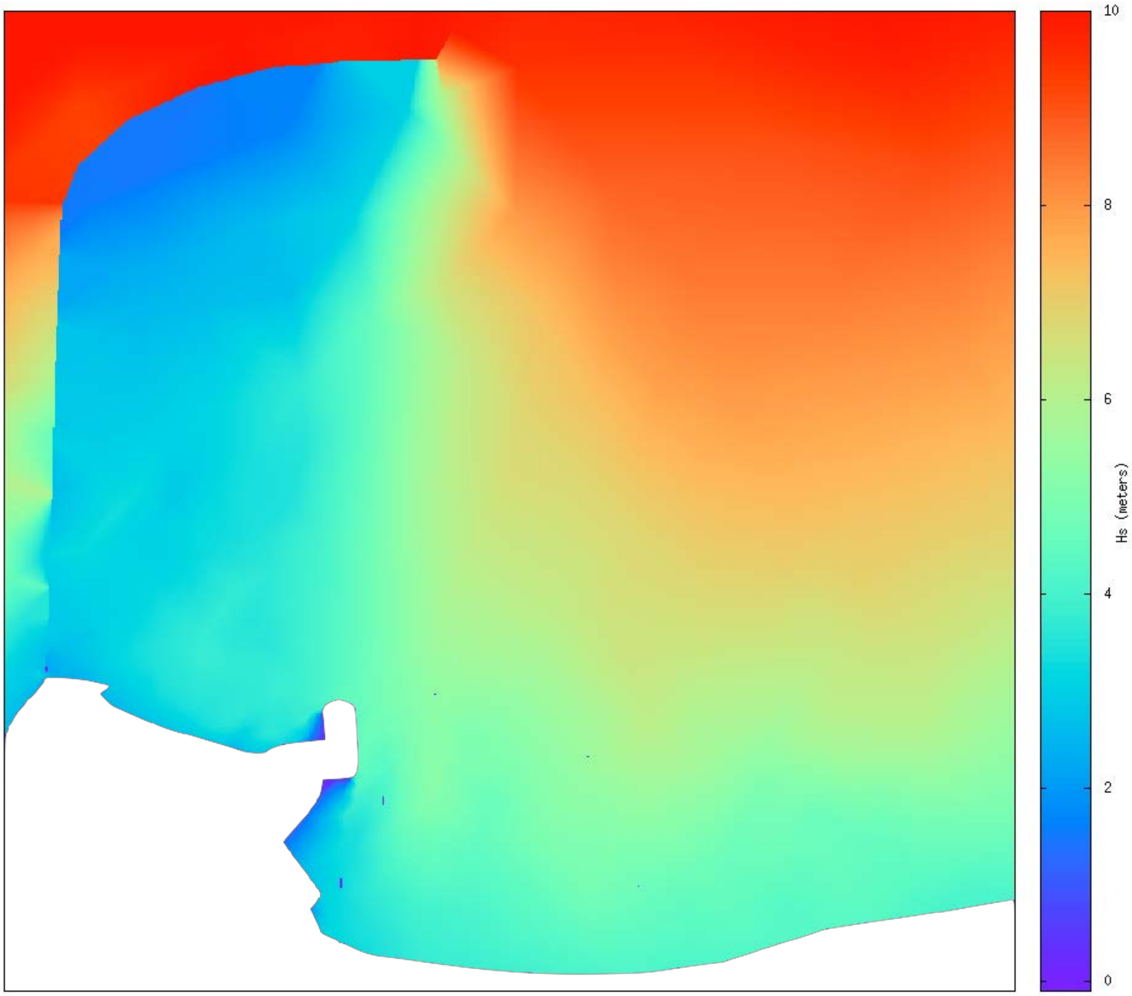


Figure 24: With Breakwater - SWAN Predicted Hs for Alcan Cove, Waves from Northeast

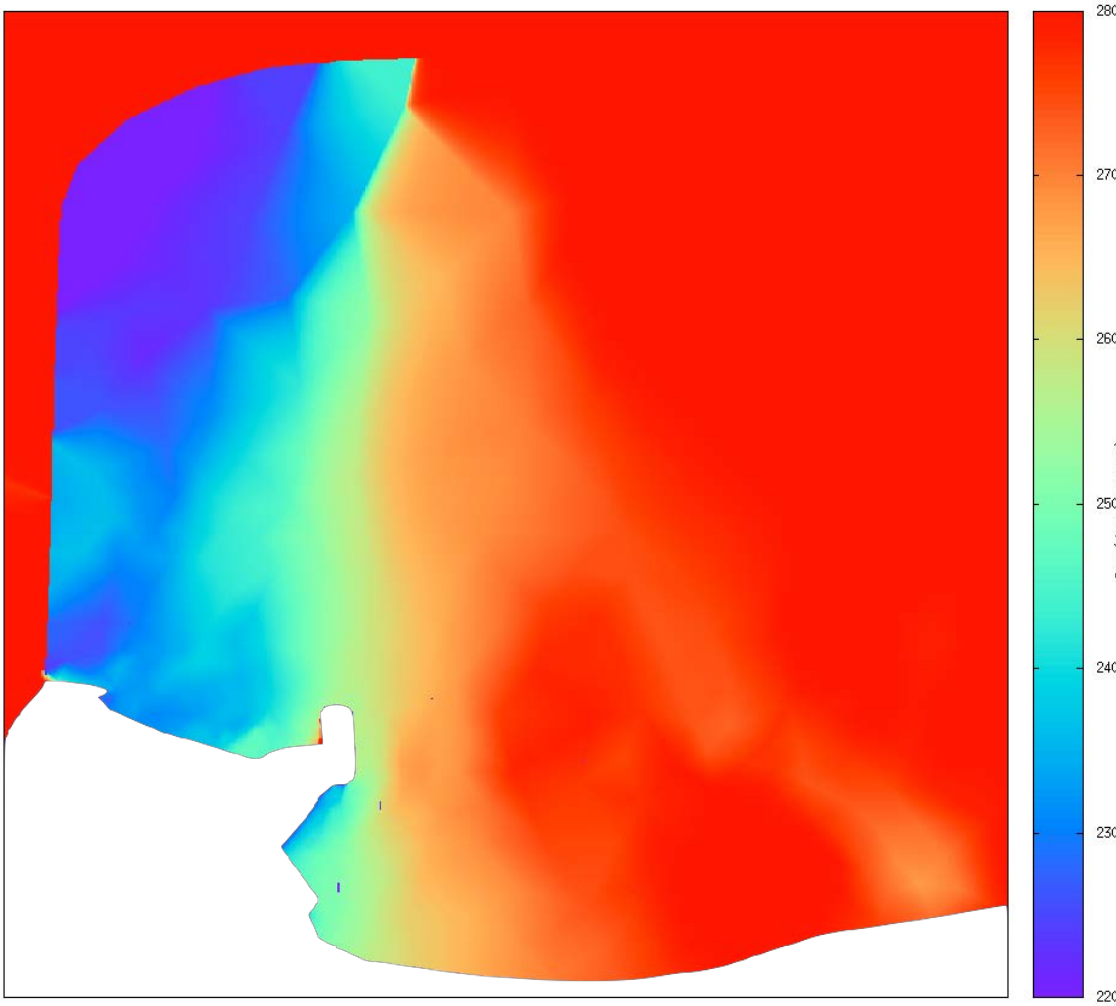


Figure 25: With Breakwater - SWAN Predicted Angles for Alcan Cove, Waves from Northwest

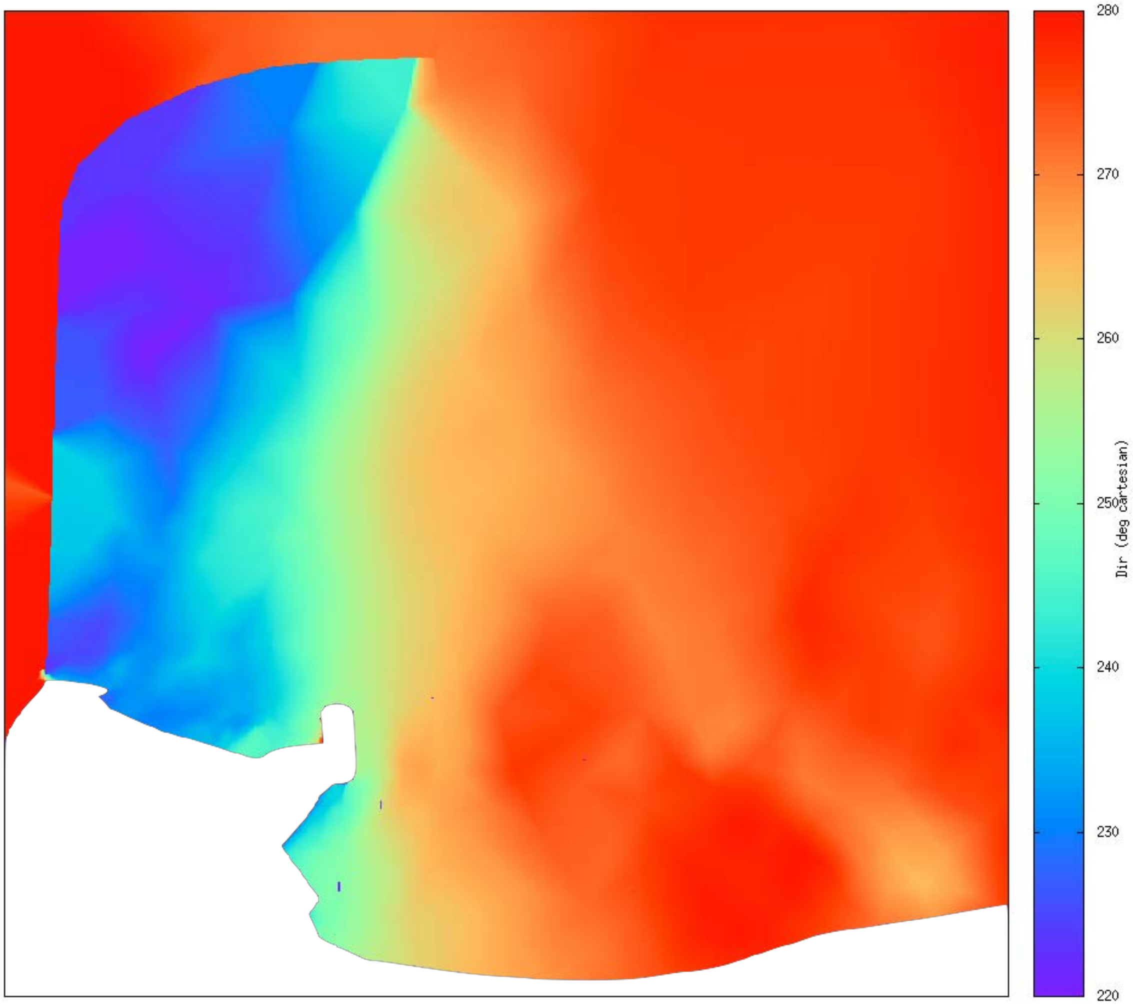


Figure 26: With Breakwater - SWAN Predicted Angles for Alcan Cove, Waves from North

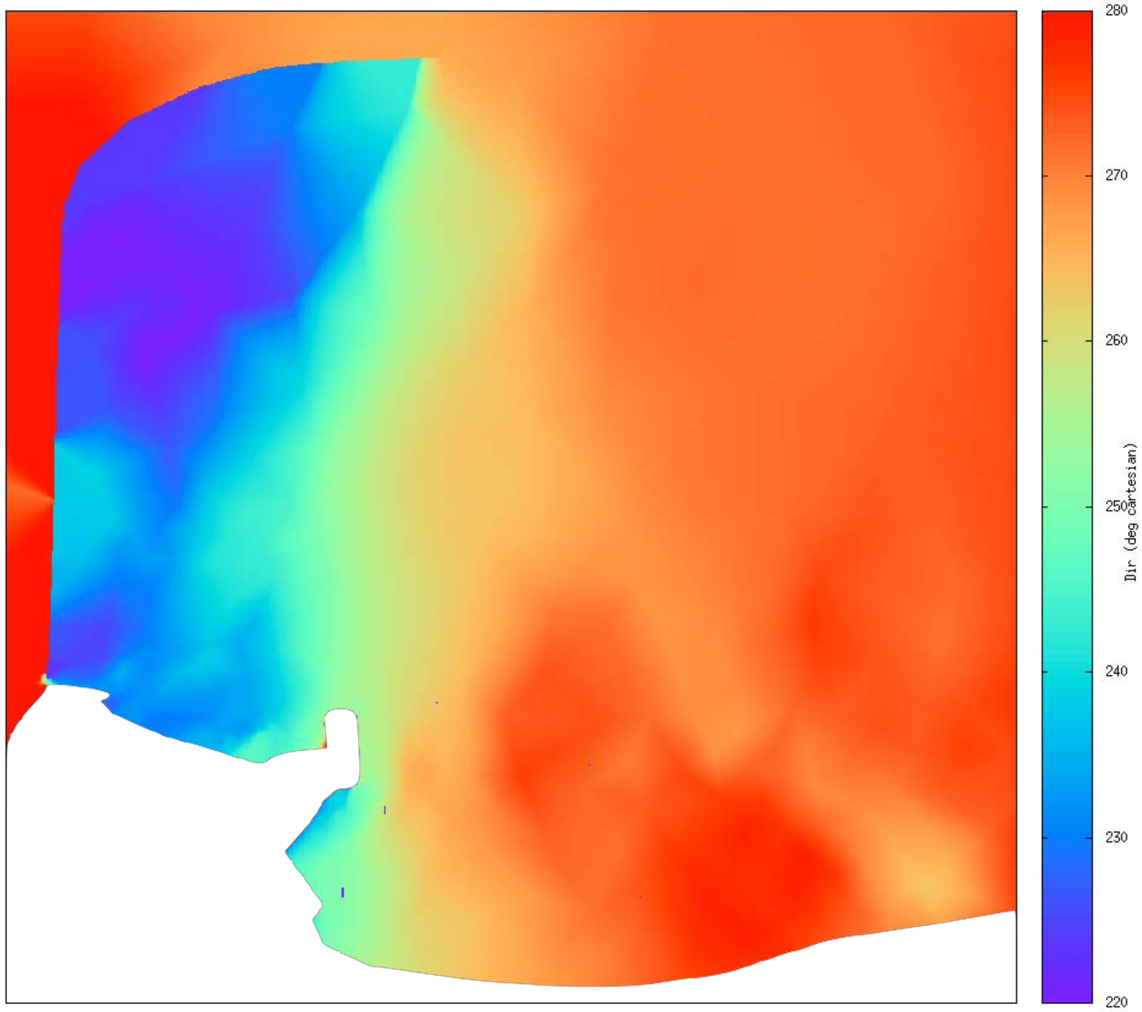


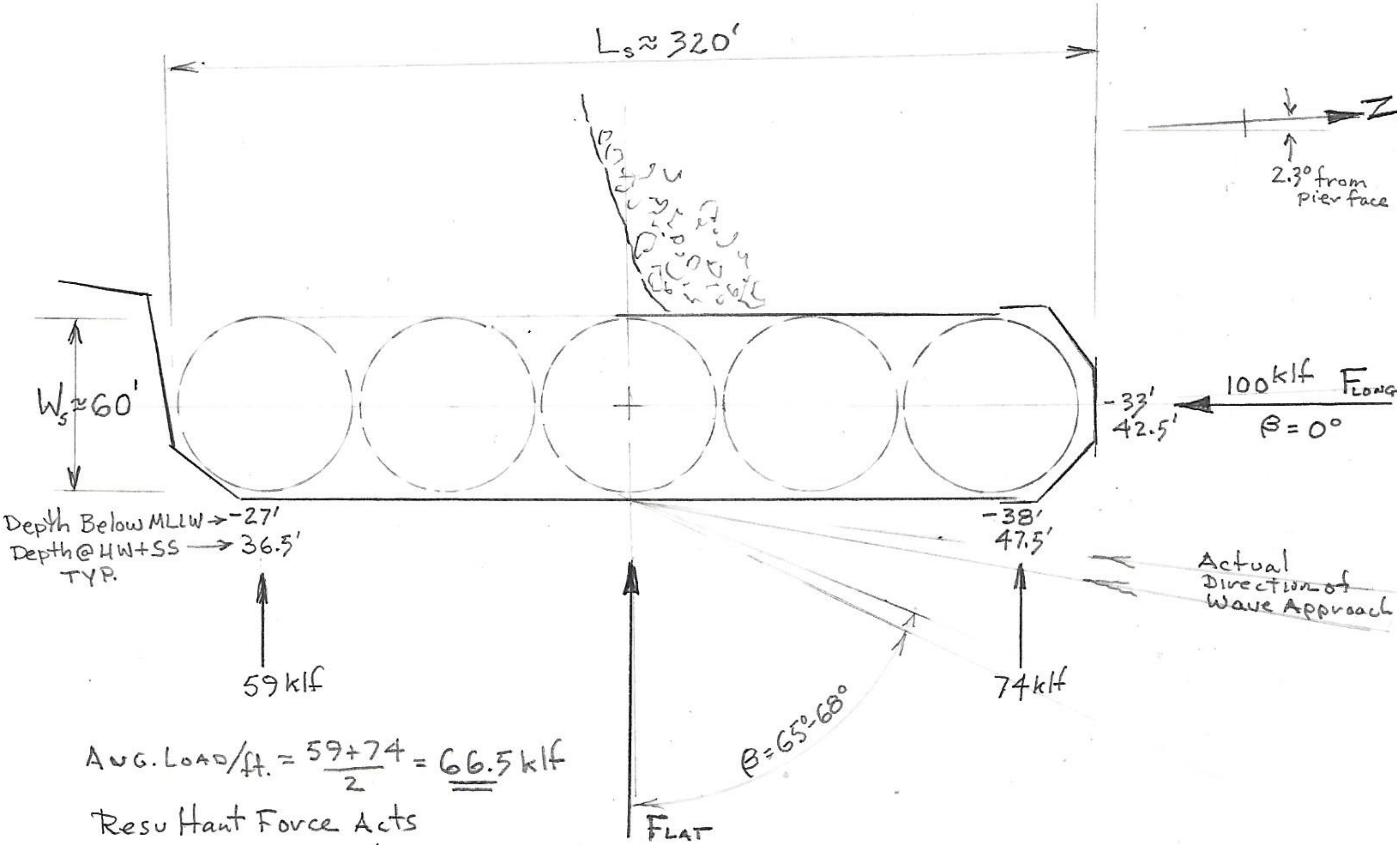
Figure 27: With Breakwater - SWAN Predicted Angles for Alcan Cove, Waves from Northeast

SHEMYA IS. FUEL PIER
AME#10430

9/10/20

WAVE FORCE CALCULATIONS SUMMARY OF RESULTS

FOR CONTROLLING CONDITIONS WIND/WAVES FROM NE
AND HIGH TIDE + STORM SURGE WATER LEVEL (+9.5' MLLW)*



AVG. LOAD/ft. = $\frac{59+74}{2}$ = 66.5 klf

Resultant Force Acts at approx. 0.7d Above Mudline

MAX. TOTAL WAVE FORCE ON WHARF OVERALL (Do Not Occur Simultaneously)

$F_{LAT.} \approx 66.5\text{ klf} \times 320' \times 0.6 = 12{,}800$ kips (Rounded) TYP.

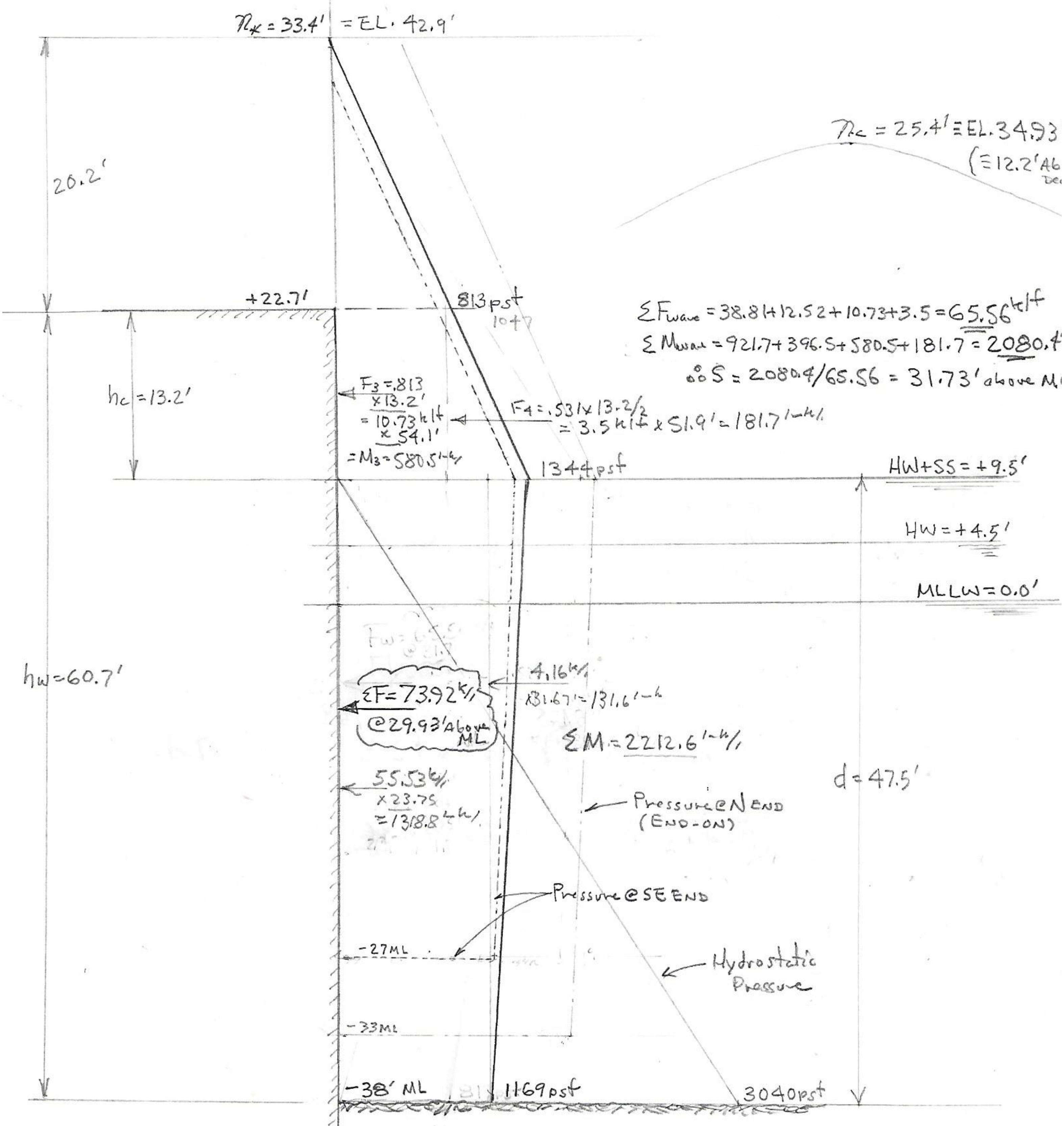
$F_{LONG.} \approx 100\text{ klf} \times 60' \times 0.6 = 3{,}600$ kips

Note: Wave Pressures are Additive to Normal Hydrostatic Pressures

* Wave Conditions from SWAN Model Results
Wave Forces Calculated Using GODA's Method per USACE, CEM, PART VI
(EM 1110-2-1100), 2006

9/10/20

MAX. (100YR) WAVE FORCE/PRESSURE DIAGRAM
FOR High Tide + Storm Surge (=+9.5') W.L. @ N.E. END FACE OF SHEMYA WHARF

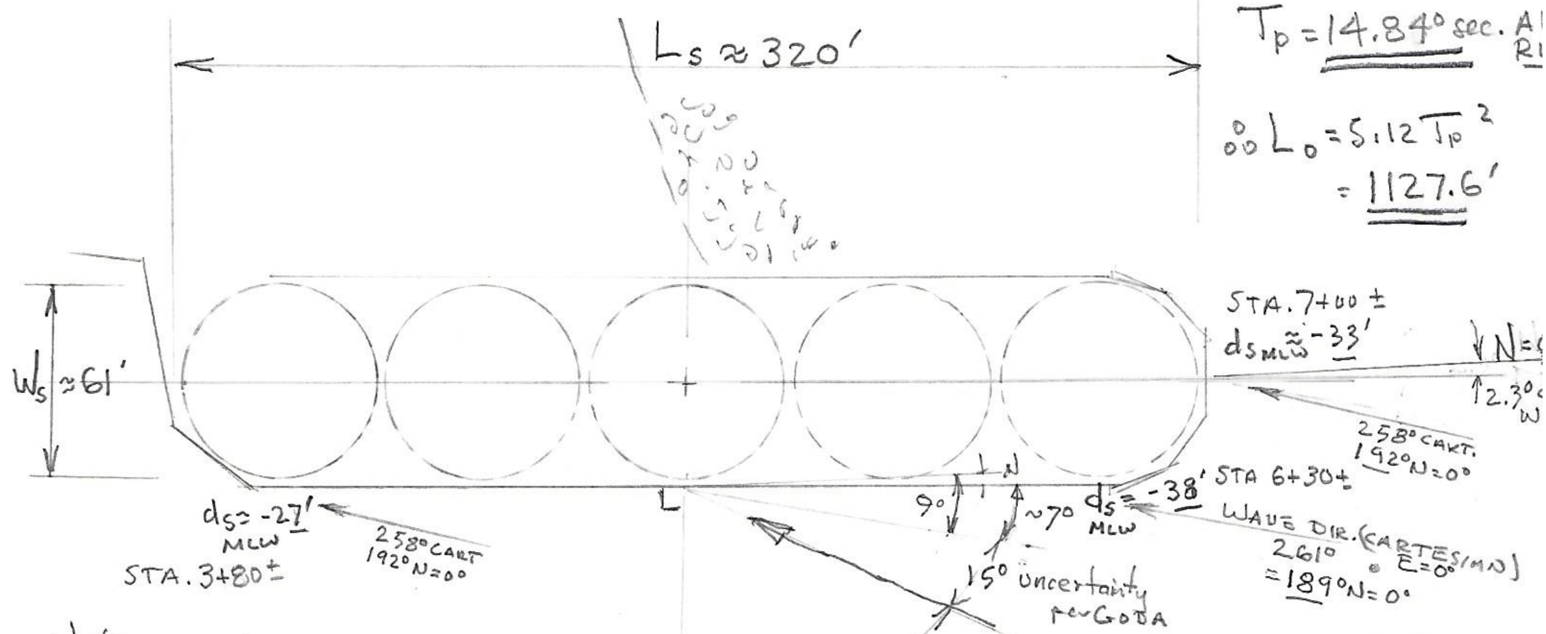


SECTION AT NE END FACE OF WHARF

JA 9/8/2

SWAN WAVE MODEL RESULTS SUMMARY AT SHEMYA WHARF

Note Wind/Waves from NE controls ALL per SWAN Model Results



Note: Take; h_b = water depth @ $5 \times H_s$ from structure = depth from SWAN model (Round to Nearest 1')

AT NE END Wharf Face

Take $d = h_b$

High Tide + Surge = +9.5' MLLW; $d = 47'$, $H_s = 18'$, $\measuredangle = 261^\circ$ (CART.) $\therefore d/L_o = \frac{47'}{1128'} = .0417$ $\rightarrow d/L = .08520$, $L = \underline{551.6}$

High Tide = +4.5 MLLW; $d = 42'$, $H_s = 16'$ $\measuredangle = 261^\circ$

$d/L_o = \frac{42'}{1128'} = .0372 \rightarrow d/L = .08007$ $\therefore L = \underline{524.5'}$

AT SE END Wharf Face

HW + Surge: $d = 43'$, $H_s = 16'$, $\measuredangle = 258^\circ$ $d/L_o = .0381 \rightarrow d/L = .08112$

$L = \underline{530'}$

HW: $d = 38'$, $H_s = 14'$, $\measuredangle = 257^\circ$

$d/L_o = .0337 \rightarrow d/L = .07593$ $L = \underline{500.5'}$

AT N Facing End

HW + Surge: $d = 49'$, $H_s = 17'$, $\measuredangle = 258^\circ$ $d/L_o = .0434 \rightarrow d/L = .08708$

$L = \underline{562.7'}$

HW: $d = 44'$, $H_s = 15'$, $\measuredangle = 256^\circ$ $d/L_o = .0390 \rightarrow d/L = .08215$ $L = \underline{535.}$

DJA 9/10/20

CALCULATE WAVE FORCES ON WHARF PER GODA Method (1985) & CEM PART VI, 2006

See sht. # for GODA Definitions & Equations & sht # For site conditions

For Wall like structure w/o Rubble Fdn.

Take; $h_s = h' = d$, Take; $\lambda_1 = \lambda_2 = \lambda_3 = 1.0$ All cases

See sht. # for SWAN Model Results, Wave Properties

Take: $H_{design} = 1.8 H_s$ & $\rho_w g = 64$ pcf ALL cases

For Waves at NE Face of Wharf & High Tide + Storm Surge = +9.5' MLLW

$$\eta^* = 0.75(1+\cos\beta)\lambda_1 H_{des.} = .75(1+\cos 68°)1.0\times 1.8\times 18' = \underline{33.4'}$$

($H_{des} = 32.4'$)

$$P_1 = 0.5(1+\cos\beta)(\lambda_1\alpha_1 + \lambda_2\alpha_2^{\to 0}\cos^2\beta)\rho g H_{des}$$

$$= .5(1.3746)(.943+0)\,64\text{pcf}\times 32.4' = \underline{\underline{1344}}\text{ psf}$$

$$P_2 = \left(1-\frac{h_c}{\eta^*}\right)P_1 = .6048\times 1344 = \underline{\underline{812.8}}\text{ psf}$$

$$P_3 = \alpha_3 P_1 = 0.87\times 1344 = \underline{\underline{1169}}\text{ psf}$$

where; $\alpha_1 = 0.6 + 0.5\left[\frac{4\pi d/L}{\sinh 4\pi d/L}\right]^2$

$= .6 + 0.343 = 0.\underline{943}$

where; $d = 38' + 9.5' = 47.5'$

& $d/L = \frac{47.5'}{551.6'} = .0861$ ← see sht #

$\therefore 4\pi d/L = 1.0821$

$$\alpha_2 = \min.\left\{\frac{h_b - d}{3h_b}\left(\frac{H_{des}}{d}\right)^2, \frac{2d}{H_{des}}\right\} \equiv \frac{47'-47.5}{3\times 47'} \to 0 \therefore \underline{\alpha_2 = 0}$$

$$\alpha_3 = 1 - \left(\frac{h_w - h_c}{h_s}\right)\left[1 - \frac{1}{\cosh 2\pi d/L}\right]$$

where; $h_w = 22.7' + 38' = 60.7'$

$h_c = 22.7' - 9.5' = 13.2$

.13×

$$= 1 - \frac{60.7' - 13.2'}{47.5'}\left[1 - .867\right] = 1 - 0.13 = 0.\underline{87}$$

Check Total Force over Length of Wharf for: $L_s/L \simeq \frac{320'}{552'} = 0.58'$ & $\beta = 68°$

→ $\gamma_F \cong 0.6$ → $F_{TOTAL} \cong .6\times 320'\times 73.93$ klf = $\underline{\underline{14,195 \text{ kips}}}$ ← Revise for Avg. Force over Length of Wharf see Sht. # ____

↑ Per Burcharth & Liu, 1968 CEM Part VI

Check Wave Crest EL., η_c from Non-Dim. Graph (CEM, P-VI)

Input period Parameters: $d/gT^2 = \frac{47'}{32.2\text{fps}^2\times(14.84)^2} = \underline{.0066}$

$H/gT^2 = \frac{32.4'}{32.2\times(14.84)^2} = .0046$

$\frac{\eta_c}{H} \cong .785 \to \eta_c = \underline{\underline{25.4'}}$

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Portsmouth, NH 0380
(603) 766-1870

Calculated By: J. Gaythwaite
Date: 09/08/2020
Checked By: E. Levesque
Date: 09/09/2020

Description:

Determine wave forces on Shemya Wharf. Load Case NE End Wharf Face at High Tide & Surge.

References:

1. FEMA, Guidance for Flood Risk Analysis and Mapping, Coastal Wave Runnup and Overtopping, February 2018.
2. USACE, Coastal Engineering Manual, EM 1110-2-1100 (Part VI), Change 3 - Sept 2011.
3. USACE, Shore Protection Manual, Volume II, 1984.

Calculations:

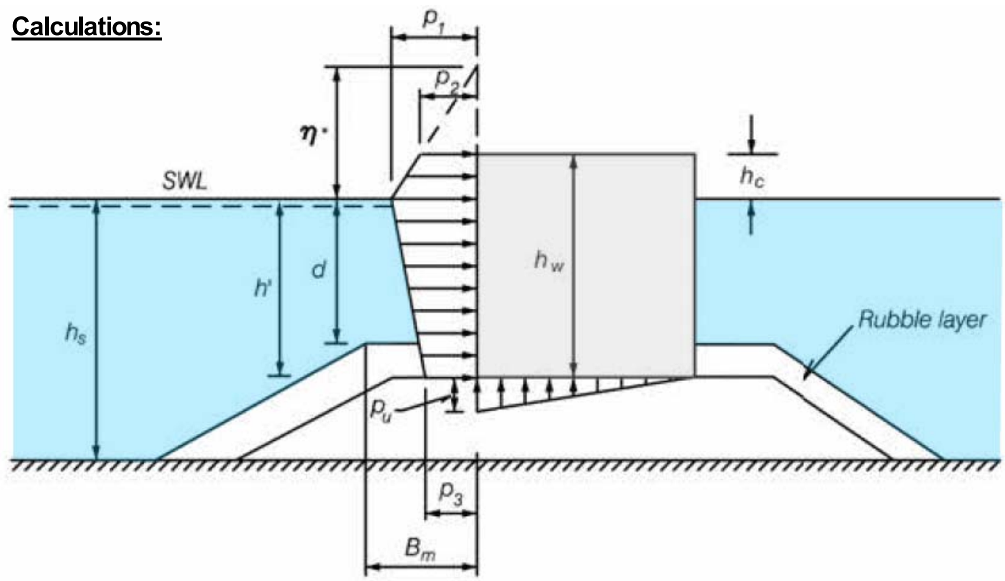

Figure 1: Table VI-5-53 Goda Formula for Irregular Waves (Excerpt from Ref 2)

Step 1: Determine design parameters and heights

$H_s := 18\text{ft}$ — Significant Wave Height

$H_{design} := 1.8 \cdot H_s$ — Design Wave Height — $H_{design} = 32.4\text{ ft}$

$EL_{top} := 22.7\text{ft}$ — Top of wall elevation

$EL_{bot} := -38\text{ft}$ — Mudline Elevation

$EL_{WL} := 9.5\text{ft}$ — Water level elevation

$h_s := EL_{WL} - EL_{bot}$ $\quad h' := h_s \quad d := h_s$ — $h_s = 47.5\text{ ft}$

$h_c := EL_{top} - EL_{WL}$ — $h_c = 13.2\text{ ft}$

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$h_b := 47ft$ Water Depth at distance of 5*Hs

$h_w := EL_{top} - EL_{bot}$ Height of structure from deck to mudline $h_w = 60.7\ ft$

$T_p := 14.84 sec$ Period of significant wave

$L := 551.6ft$ Wave Length (at structure - Per Table C1, Ref 3)

$g := 32.2\ \frac{ft}{s^2}$ Acceleration due to gravity

$\gamma_w := 64pcf$ Density of seawater

Step 2: Determine p_1, p_2, p_3

$\beta := 68deg$ Angle of incidence of waves

$\lambda_1 := 1 \qquad \lambda_2 := 1$ Modification factors (Assumed for conventional vertical wall structure)

$$\eta' := 0.75 \cdot \left[(1 + \cos(\beta)) \cdot \lambda_1 \cdot H_{design}\right] \qquad \eta' = 33.4\ ft$$

$$\alpha_1 := 0.6 + 0.5 \cdot \left(\frac{4 \cdot \pi \cdot \frac{d}{L}}{\sinh\left(4 \cdot \pi \cdot \frac{d}{L}\right)} \right)^2 \qquad \alpha_1 = 0.943$$

$$\alpha_2 := \min\left[\frac{h_b - d}{3 \cdot h_b} \cdot \left(\frac{H_{design}}{d} \right)^2, \frac{2 \cdot d}{H_{design}} \right] \qquad \alpha_2 = -0.002$$

$$\alpha_3 := 1 - \left(\frac{h_w - h_c}{h_s} \right) \cdot \left(1 - \frac{1}{\cosh\left(\frac{2 \cdot \pi \cdot d}{L} \right)} \right) \qquad \alpha_3 = 0.87$$

$$p_1 := 0.5 \cdot (1 + \cos(\beta)) \cdot \left[\lambda_1 \cdot \alpha_1 + \lambda_2 \cdot \left[\alpha_2 \cdot (\cos(\beta))^2 \right] \right] \cdot \gamma_w \cdot H_{design} \qquad p_1 = 1343.99 \cdot psf$$

$$p_2 := \begin{cases} \left(1 - \frac{h_c}{\eta'} \right) \cdot p_1 & \text{if } \eta' > h_c \\ 0ksf & \text{otherwise} \end{cases} \qquad p_2 = 812.88 \cdot psf$$

$$p_3 := \alpha_3 \cdot p_1 \qquad p_3 = 1168.7 \cdot psf$$

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Step 3: Determine Total Wave Force & Location on the wall

$P_1 := \left(\frac{p_1 + p_2}{2}\right) \cdot h_c$ — Wave force above water level — $P_1 = 14.24 \cdot klf$

$y_{P1} := \left[\frac{p_1 + 2 \cdot p_2}{3 \cdot (p_1 + p_2)}\right] \cdot h_c$ — Centroid of P1 — $y_{P1} = 6.06\ ft$

$H_{P1} := h_b + y_{P1}$ — Height of P1, from mudline — $H_{P1} = 53.06\ ft$

$M_{P1} := P_1 \cdot H_{P1}$ — Moment due to P1, from mudline — $M_{P1} = 755.3 \cdot \frac{kip \cdot ft}{ft}$

$P_2 := \left(\frac{p_1 + p_3}{2}\right) \cdot d$ — Wave force below water level — $P_2 = 59.68 \cdot klf$

$y_{P2} := \left[\frac{p_3 + 2 \cdot p_1}{3 \cdot (p_1 + p_3)}\right] \cdot d$ — Centroid of P2 — $y_{P2} = 24.3\ ft$

$H_{P2} := y_{P2}$ — Height of P2, from mudline — $H_{P2} = 24.3\ ft$

$M_{P2} := P_2 \cdot H_{P2}$ — Moment due to P2, from mudline — $M_{P2} = 1450.27 \cdot \frac{kip \cdot ft}{ft}$

$P_T := P_1 + P_2$ — Total wave force — $P_T = 73.91 \cdot klf$

$H_T := \frac{M_{P1} + M_{P2}}{P_T}$ — Height of total wave force, from mudline — $H_T = 29.84\ ft$

![Appledore Marine Engineering, LLC]

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Description:

Determine wave forces on Shemya Wharf. Load Case NE End Wharf Face at High Tide.

References:

1. FEMA, Guidance for Flood Risk Analysis and Mapping, Coastal Wave Runnup and Overtopping, February 2018.
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Calculations:

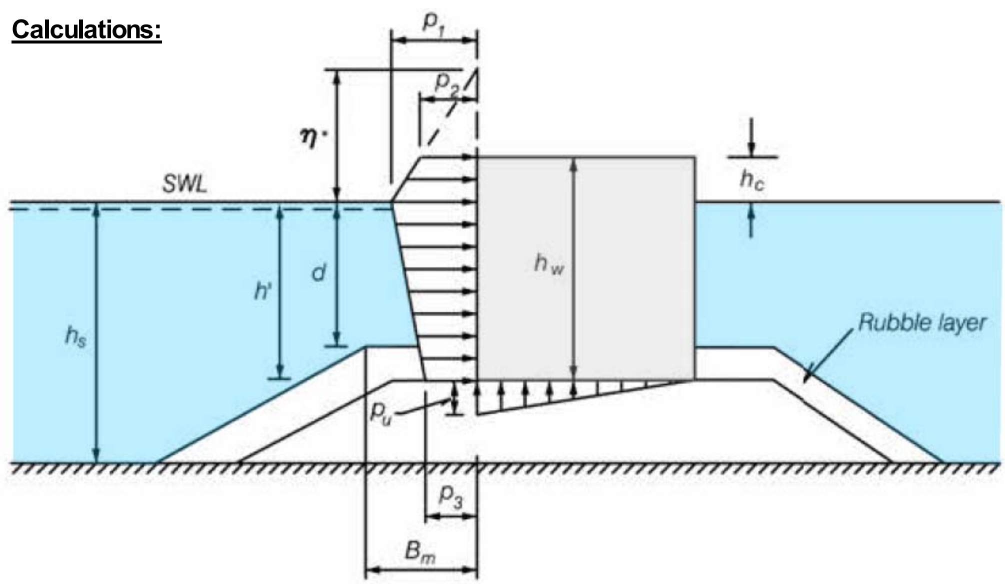

Figure 1: Table VI-5-53 Goda Formula for Irregular Waves (Excerpt from Ref 2)

Step 1: Determine design parameters and heights

$H_s := 16ft$ Significant Wave Height

$H_{design} := 1.8 \cdot H_s$ Design Wave Height $H_{design} = 28.8\ ft$

$EL_{top} := 22.7ft$ Top of wall elevation

$EL_{bot} := -38ft$ Mudline Elevation

$EL_{WL} := 4.5ft$ Water level elevation

$h_s := EL_{WL} - EL_{bot}$ $\quad h' := h_s \quad d := h_s$ $\quad h_s = 42.5\ ft$

$h_c := EL_{top} - EL_{WL}$ $\quad h_c = 18.2\ ft$

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$h_b := 42ft$ Water Depth at distance of 5*Hs

$h_w := EL_{top} - EL_{bot}$ Height of structure from deck to mudline $h_w = 60.7\ ft$

$T_p := 14.84sec$ Period of significant wave

$L := 524.5ft$ Wave Length (at structure - Per Table C1, Ref 3)

$g := 32.2\ \frac{ft}{s^2}$ Acceleration due to gravity

$\gamma_w := 64pcf$ Density of seawater

Step 2: Determine p_1, p_2, p_3

$\beta := 68deg$ Angle of incidence of waves

$\lambda_1 := 1 \qquad \lambda_2 := 1$ Modification factors (Assumed for conventional vertical wall structure)

$$\eta' := 0.75\cdot\left[(1+\cos(\beta))\cdot\lambda_1\cdot H_{design}\right] \qquad \eta' = 29.69\ ft$$

$$\alpha_1 := 0.6 + 0.5\cdot\left(\frac{4\cdot\pi\cdot\frac{d}{L}}{\sinh\left(4\cdot\pi\cdot\frac{d}{L}\right)}\right)^2 \qquad \alpha_1 = 0.958$$

$$\alpha_2 := \min\left[\frac{h_b - d}{3\cdot h_b}\cdot\left(\frac{H_{design}}{d}\right)^2, \frac{2\cdot d}{H_{design}}\right] \qquad \alpha_2 = -0.002$$

$$\alpha_3 := 1 - \left(\frac{h_w - h_c}{h_s}\right)\cdot\left(1 - \frac{1}{\cosh\left(\frac{2\cdot\pi\cdot d}{L}\right)}\right) \qquad \alpha_3 = 0.883$$

$$p_1 := 0.5\cdot(1+\cos(\beta))\cdot\left[\lambda_1\cdot\alpha_1 + \lambda_2\cdot\left[\alpha_2\cdot(\cos(\beta))^2\right]\right]\cdot\gamma_w\cdot H_{design} \qquad p_1 = 1213.16\cdot psf$$

$$p_2 := \begin{cases}\left(1 - \frac{h_c}{\eta'}\right)\cdot p_1 & \text{if } \eta' > h_c \\ 0ksf & \text{otherwise}\end{cases} \qquad p_2 = 469.53\cdot psf$$

$$p_3 := \alpha_3\cdot p_1 \qquad p_3 = 1071.29\cdot psf$$

Step 3: Determine Total Wave Force & Location on the wall

$P_1 := \left(\frac{p_1 + p_2}{2}\right)\cdot h_c$	Wave force above water level	$P_1 = 15.31\cdot\text{klf}$
$y_{P1} := \left[\frac{p_1 + 2\cdot p_2}{3\cdot(p_1 + p_2)}\right]\cdot h_c$	Centroid of P1	$y_{P1} = 7.76\,\text{ft}$
$H_{P1} := h_b + y_{P1}$	Height of P1, from mudline	$H_{P1} = 49.76\,\text{ft}$
$M_{P1} := P_1\cdot H_{P1}$	Moment due to P1, from mudline	$M_{P1} = 761.94\cdot\frac{\text{kip}\cdot\text{ft}}{\text{ft}}$
$P_2 := \left(\frac{p_1 + p_3}{2}\right)\cdot d$	Wave force below water level	$P_2 = 48.54\cdot\text{klf}$
$y_{P2} := \left[\frac{p_3 + 2\cdot p_1}{3\cdot(p_1 + p_3)}\right]\cdot d$	Centroid of P2	$y_{P2} = 21.69\,\text{ft}$
$H_{P2} := y_{P2}$	Height of P2, from mudline	$H_{P2} = 21.69\,\text{ft}$
$M_{P2} := P_2\cdot H_{P2}$	Moment due to P2, from mudline	$M_{P2} = 1052.93\cdot\frac{\text{kip}\cdot\text{ft}}{\text{ft}}$
$P_T := P_1 + P_2$	Total wave force	$P_T = 63.86\cdot\text{klf}$
$H_T := \frac{M_{P1} + M_{P2}}{P_T}$	Height of total wave force, from mudline	$H_T = 28.42\,\text{ft}$

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Description:

Determine wave forces on Shemya Wharf. Load Case SE End Wharf Face at High Tide & Surge.

References:

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3. USACE, Shore Protection Manual, Volume II, 1984.

Calculations:

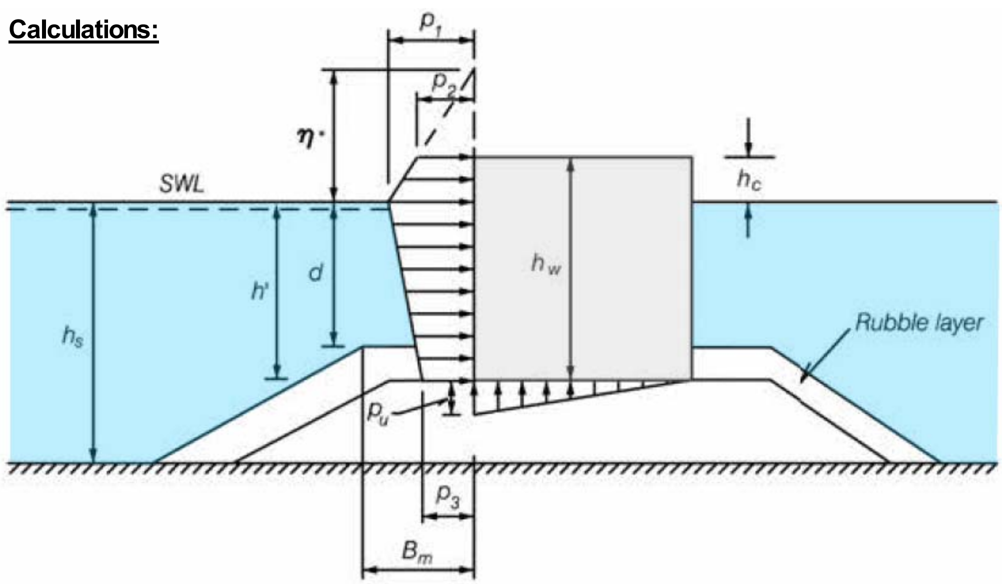

Figure 1: Table VI-5-53 Goda Formula for Irregular Waves (Excerpt from Ref 2)

Step 1: Determine design parameters and heights

$H_s := 16ft$ Significant Wave Height

$H_{design} := 1.8 \cdot H_s$ Design Wave Height $H_{design} = 28.8\ ft$

$EL_{top} := 22.7ft$ Top of wall elevation

$EL_{bot} := -27ft$ Mudline Elevation

$EL_{WL} := 9.5ft$ Water level elevation

$h_s := EL_{WL} - EL_{bot}$ $h' := h_s$ $d := h_s$ $h_s = 36.5\ ft$

$h_c := EL_{top} - EL_{WL}$ $h_c = 13.2\ ft$

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$h_b := 43\text{ft}$ Water Depth at distance of 5*Hs

$h_w := EL_{top} - EL_{bot}$ Height of structure from deck to mudline $h_w = 49.7\text{ ft}$

$T_p := 14.84\text{sec}$ Period of significant wave

$L := 530\text{ft}$ Wave Length (at structure - Per Table C1, Ref 3)

$g := 32.2\ \frac{\text{ft}}{\text{s}^2}$ Acceleration due to gravity

$\gamma_w := 64\text{pcf}$ Density of seawater

Step 2: Determine p_1, p_2, p_3

$\beta := 65\text{deg}$ Angle of incidence of waves

$\lambda_1 := 1 \quad \lambda_2 := 1$ Modification factors (Assumed for conventional vertical wall structure)

$$\eta' := 0.75\cdot\left[(1 + \cos(\beta))\cdot\lambda_1\cdot H_{design}\right] \qquad \eta' = 30.73\text{ ft}$$

$$\alpha_1 := 0.6 + 0.5\cdot\left(\frac{4\cdot\pi\cdot\frac{d}{L}}{\sinh\left(4\cdot\pi\cdot\frac{d}{L}\right)}\right)^2 \qquad \alpha_1 = 0.992$$

$$\alpha_2 := \min\left[\frac{h_b - d}{3\cdot h_b}\cdot\left(\frac{H_{design}}{d}\right)^2, \frac{2\cdot d}{H_{design}}\right] \qquad \alpha_2 = 0.031$$

$$\alpha_3 := 1 - \left(\frac{h_w - h_c}{h_s}\right)\cdot\left(1 - \frac{1}{\cosh\left(\frac{2\cdot\pi\cdot d}{L}\right)}\right) \qquad \alpha_3 = 0.913$$

$$p_1 := 0.5\cdot(1 + \cos(\beta))\cdot\left[\lambda_1\cdot\alpha_1 + \lambda_2\cdot\left[\alpha_2\cdot(\cos(\beta))^2\right]\right]\cdot\gamma_w\cdot H_{design} \qquad p_1 = 1307.76\cdot\text{psf}$$

$$p_2 := \begin{cases}\left(1 - \frac{h_c}{\eta'}\right)\cdot p_1 & \text{if } \eta' > h_c \\ 0\text{ksf} & \text{otherwise}\end{cases} \qquad p_2 = 745.99\cdot\text{psf}$$

$$p_3 := \alpha_3\cdot p_1 \qquad p_3 = 1194.21\cdot\text{psf}$$

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Step 3: Determine Total Wave Force & Location on the wall

$P_1 := \left(\frac{p_1 + p_2}{2}\right)\cdot h_c$	Wave force above water level	$P_1 = 13.55\cdot\text{klf}$
$y_{P1} := \left[\frac{p_1 + 2\cdot p_2}{3\cdot(p_1 + p_2)}\right]\cdot h_c$	Centroid of P1	$y_{P1} = 6\ \text{ft}$
$H_{P1} := h_b + y_{P1}$	Height of P1, from mudline	$H_{P1} = 49\ \text{ft}$
$M_{P1} := P_1\cdot H_{P1}$	Moment due to P1, from mudline	$M_{P1} = 664.16\cdot\frac{\text{kip}\cdot\text{ft}}{\text{ft}}$
$P_2 := \left(\frac{p_1 + p_3}{2}\right)\cdot d$	Wave force below water level	$P_2 = 45.66\cdot\text{klf}$
$y_{P2} := \left[\frac{p_3 + 2\cdot p_1}{3\cdot(p_1 + p_3)}\right]\cdot d$	Centroid of P2	$y_{P2} = 18.53\ \text{ft}$
$H_{P2} := y_{P2}$	Height of P2, from mudline	$H_{P2} = 18.53\ \text{ft}$
$M_{P2} := P_2\cdot H_{P2}$	Moment due to P2, from mudline	$M_{P2} = 845.92\cdot\frac{\text{kip}\cdot\text{ft}}{\text{ft}}$
$P_T := P_1 + P_2$	Total wave force	$P_T = 59.22\cdot\text{klf}$
$H_T := \frac{M_{P1} + M_{P2}}{P_T}$	Height of total wave force, from mudline	$H_T = 25.5\ \text{ft}$

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Calculated By: J. Gaythwaite
Date: 09/08/2020
Checked By: E. Levesque
Date: 09/09/2020

Description:

Determine wave forces on Shemya Wharf. Load Case SE End Wharf Face at High Tide.

References:

1. FEMA, Guidance for Flood Risk Analysis and Mapping, Coastal Wave Runnup and Overtopping, February 2018.
2. USACE, Coastal Engineering Manual, EM 1110-2-1100 (Part VI), Change 3 - Sept 2011.
3. USACE, Shore Protection Manual, Volume II, 1984.

Calculations:

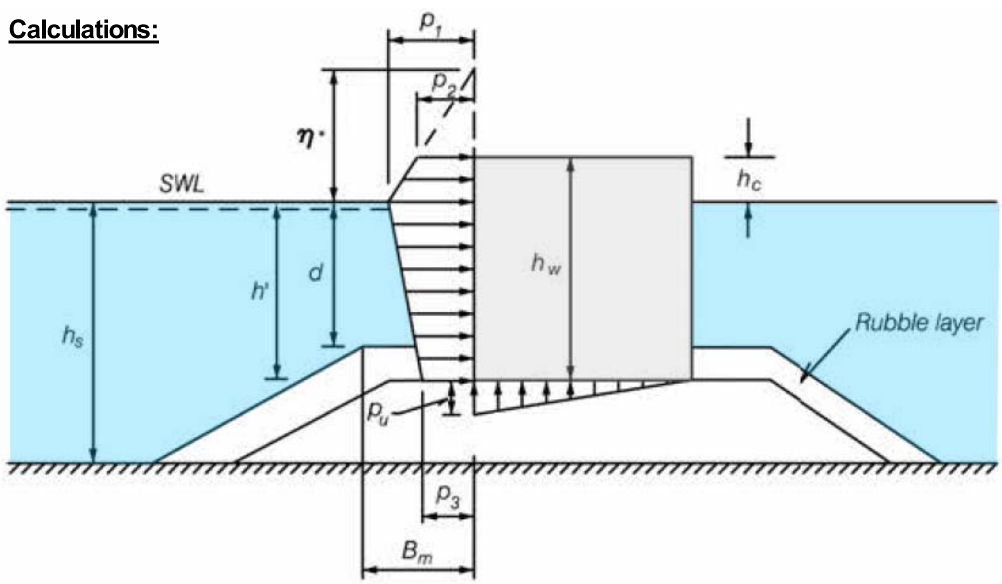

Figure 1: Table VI-5-53 Goda Formula for Irregular Waves (Excerpt from Ref 2)

Step 1: Determine design parameters and heights

$H_s := 14ft$	Significant Wave Height	
$H_{design} := 1.8 \cdot H_s$	Design Wave Height	$H_{design} = 25.2\ ft$
$EL_{top} := 22.7ft$	Top of wall elevation	
$EL_{bot} := -27ft$	Mudline Elevation	
$EL_{WL} := 4.5ft$	Water level elevation	
$h_s := EL_{WL} - EL_{bot}$	$h' := h_s$ $\quad$ $d := h_s$	$h_s = 31.5\ ft$
$h_c := EL_{top} - EL_{WL}$		$h_c = 18.2\ ft$

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$h_b := 38\text{ft}$	Water Depth at distance of 5*Hs	
$h_w := EL_{top} - EL_{bot}$	Height of structure from deck to mudline	$h_w = 49.7\text{ ft}$
$T_p := 14.84\text{sec}$	Period of significant wave	
$L := 500.5\text{ft}$	Wave Length (at structure - Per Table C1, Ref 3)	
$g := 32.2\ \dfrac{\text{ft}}{\text{s}^2}$	Acceleration due to gravity	
$\gamma_w := 64\text{pcf}$	Density of seawater	

Step 2: Determine p_1, p_2, p_3

$\beta := 65\text{deg}$	Angle of incidence of waves	
$\lambda_1 := 1 \qquad \lambda_2 := 1$	Modification factors (Assumed for conventional vertical wall structure)	

$$\eta' := 0.75\cdot\left[(1 + \cos(\beta))\cdot\lambda_1\cdot H_{design}\right] \qquad \eta' = 26.89\text{ ft}$$

$$\alpha_1 := 0.6 + 0.5\cdot\left(\frac{4\cdot\pi\cdot\frac{d}{L}}{\sinh\left(4\cdot\pi\cdot\frac{d}{L}\right)}\right)^2 \qquad \alpha_1 = 1.008$$

$$\alpha_2 := \min\left[\frac{h_b - d}{3\cdot h_b}\cdot\left(\frac{H_{design}}{d}\right)^2, \frac{2\cdot d}{H_{design}}\right] \qquad \alpha_2 = 0.036$$

$$\alpha_3 := 1 - \left(\frac{h_w - h_c}{h_s}\right)\cdot\left(1 - \frac{1}{\cosh\left(\frac{2\cdot\pi\cdot d}{L}\right)}\right) \qquad \alpha_3 = 0.927$$

$$p_1 := 0.5\cdot(1 + \cos(\beta))\cdot\left[\lambda_1\cdot\alpha_1 + \lambda_2\cdot\left[\alpha_2\cdot(\cos(\beta))^2\right]\right]\cdot\gamma_w\cdot H_{design} \qquad p_1 = 1163.4\cdot\text{psf}$$

$$p_2 := \begin{cases}\left(1 - \frac{h_c}{\eta'}\right)\cdot p_1 & \text{if } \eta' > h_c \\ 0\text{ksf} & \text{otherwise}\end{cases} \qquad p_2 = 375.9\cdot\text{psf}$$

$$p_3 := \alpha_3\cdot p_1 \qquad p_3 = 1078\cdot\text{psf}$$

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Step 3: Determine Total Wave Force & Location on the wall

$P_1 := \left(\dfrac{p_1 + p_2}{2}\right) \cdot h_c$	Wave force above water level	$P_1 = 14.01 \cdot \text{klf}$
$y_{P1} := \left[\dfrac{p_1 + 2 \cdot p_2}{3 \cdot (p_1 + p_2)}\right] \cdot h_c$	Centroid of P1	$y_{P1} = 7.55\ \text{ft}$
$H_{P1} := h_b + y_{P1}$	Height of P1, from mudline	$H_{P1} = 45.55\ \text{ft}$
$M_{P1} := P_1 \cdot H_{P1}$	Moment due to P1, from mudline	$M_{P1} = 638.02 \cdot \dfrac{\text{kip} \cdot \text{ft}}{\text{ft}}$
$P_2 := \left(\dfrac{p_1 + p_3}{2}\right) \cdot d$	Wave force below water level	$P_2 = 35.3 \cdot \text{klf}$
$y_{P2} := \left[\dfrac{p_3 + 2 \cdot p_1}{3 \cdot (p_1 + p_3)}\right] \cdot d$	Centroid of P2	$y_{P2} = 15.95\ \text{ft}$
$H_{P2} := y_{P2}$	Height of P2, from mudline	$H_{P2} = 15.95\ \text{ft}$
$M_{P2} := P_2 \cdot H_{P2}$	Moment due to P2, from mudline	$M_{P2} = 563.07 \cdot \dfrac{\text{kip} \cdot \text{ft}}{\text{ft}}$
$P_T := P_1 + P_2$	Total wave force	$P_T = 49.31 \cdot \text{klf}$
$H_T := \dfrac{M_{P1} + M_{P2}}{P_T}$	Height of total wave force, from mudline	$H_T = 24.36\ \text{ft}$

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Description:

Determine wave forces on Shemya Wharf. Load Case N End Wharf Face at High Tide & Surge.

References:

1. FEMA, Guidance for Flood Risk Analysis and Mapping, Coastal Wave Runnup and Overtopping, February 2018.
2. USACE, Coastal Engineering Manual, EM 1110-2-1100 (Part VI), Change 3 - Sept 2011.
3. USACE, Shore Protection Manual, Volume II, 1984.

Calculations:

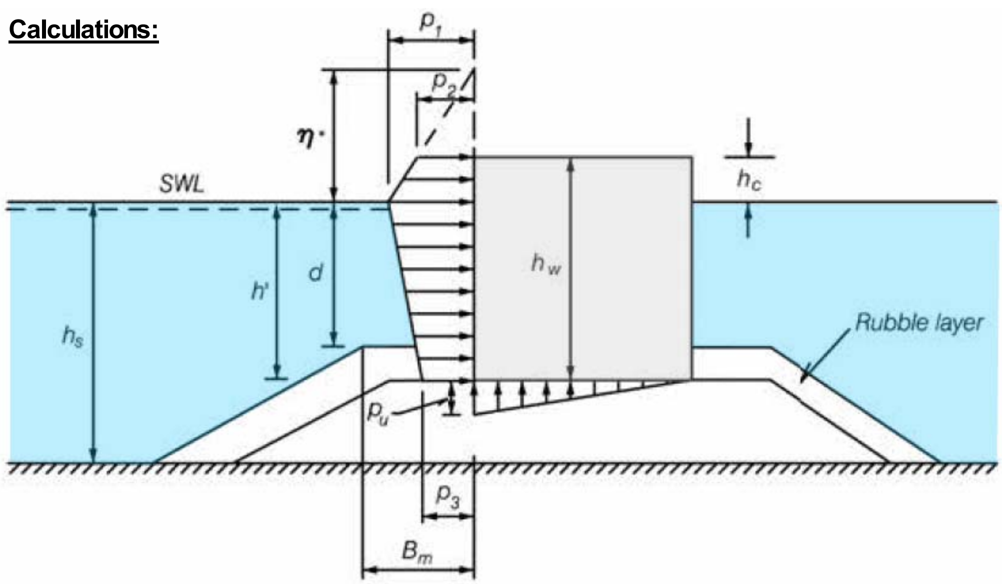

Figure 1: Table VI-5-53 Goda Formula for Irregular Waves (Excerpt from Ref 2)

Step 1: Determine design parameters and heights

$H_s := 17ft$ Significant Wave Height

$H_{design} := 1.8 \cdot H_s$ Design Wave Height $H_{design} = 30.6\ ft$

$EL_{top} := 22.7ft$ Top of wall elevation

$EL_{bot} := -33ft$ Mudline Elevation

$EL_{WL} := 9.5ft$ Water level elevation

$h_s := EL_{WL} - EL_{bot}$ $h' := h_s$ $d := h_s$ $h_s = 42.5\ ft$

$h_c := EL_{top} - EL_{WL}$ $h_c = 13.2\ ft$

$h_b := 49ft$ Water Depth at distance of 5*Hs

$h_w := EL_{top} - EL_{bot}$ Height of structure from deck to mudline $h_w = 55.7\,ft$

$T_p := 14.84sec$ Period of significant wave

$L := 562.7ft$ Wave Length (at structure - Per Table C1, Ref 3)

$g := 32.2\,\frac{ft}{s^2}$ Acceleration due to gravity

$\gamma_w := 64pcf$ Density of seawater

Step 2: Determine p_1, p_2, p_3

$\beta := 0deg$ Angle of incidence of waves

$\lambda_1 := 1 \quad \lambda_2 := 1$ Modification factors (Assumed for conventional vertical wall structure)

$$\eta' := 0.75\cdot\left[(1+\cos(\beta))\cdot\lambda_1\cdot H_{design}\right] \qquad \eta' = 45.9\,ft$$

$$\alpha_1 := 0.6 + 0.5\cdot\left(\frac{4\cdot\pi\cdot\frac{d}{L}}{\sinh\left(4\cdot\pi\cdot\frac{d}{L}\right)}\right)^2 \qquad \alpha_1 = 0.973$$

$$\alpha_2 := \min\left[\frac{h_b - d}{3\cdot h_b}\cdot\left(\frac{H_{design}}{d}\right)^2, \frac{2\cdot d}{H_{design}}\right] \qquad \alpha_2 = 0.023$$

$$\alpha_3 := 1 - \left(\frac{h_w - h_c}{h_s}\right)\cdot\left(1 - \frac{1}{\cosh\left(\frac{2\cdot\pi\cdot d}{L}\right)}\right) \qquad \alpha_3 = 0.897$$

$$p_1 := 0.5\cdot(1+\cos(\beta))\cdot\left[\lambda_1\cdot\alpha_1 + \lambda_2\cdot\left[\alpha_2\cdot(\cos(\beta))^2\right]\right]\cdot\gamma_w\cdot H_{design} \qquad p_1 = 1951.35\cdot psf$$

$$p_2 := \begin{cases}\left(1 - \frac{h_c}{\eta'}\right)\cdot p_1 & \text{if } \eta' > h_c \\ 0ksf & \text{otherwise}\end{cases} \qquad p_2 = 1390.18\cdot psf$$

$$p_3 := \alpha_3\cdot p_1 \qquad p_3 = 1750.51\cdot psf$$

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Step 3: Determine Total Wave Force & Location on the wall

$P_1 := \left(\frac{p_1 + p_2}{2}\right) \cdot h_c$	Wave force above water level	$P_1 = 22.05 \cdot \text{klf}$
$y_{P1} := \left[\frac{p_1 + 2 \cdot p_2}{3 \cdot (p_1 + p_2)}\right] \cdot h_c$	Centroid of P1	$y_{P1} = 6.23\ \text{ft}$
$H_{P1} := h_b + y_{P1}$	Height of P1, from mudline	$H_{P1} = 55.23\ \text{ft}$
$M_{P1} := P_1 \cdot H_{P1}$	Moment due to P1, from mudline	$M_{P1} = 1218.06 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$
$P_2 := \left(\frac{p_1 + p_3}{2}\right) \cdot d$	Wave force below water level	$P_2 = 78.66 \cdot \text{klf}$
$y_{P2} := \left[\frac{p_3 + 2 \cdot p_1}{3 \cdot (p_1 + p_3)}\right] \cdot d$	Centroid of P2	$y_{P2} = 21.63\ \text{ft}$
$H_{P2} := y_{P2}$	Height of P2, from mudline	$H_{P2} = 21.63\ \text{ft}$
$M_{P2} := P_2 \cdot H_{P2}$	Moment due to P2, from mudline	$M_{P2} = 1701.86 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$
$P_T := P_1 + P_2$	Total wave force	$P_T = 100.72 \cdot \text{klf}$
$H_T := \frac{M_{P1} + M_{P2}}{P_T}$	Height of total wave force, from mudline	$H_T = 28.99\ \text{ft}$

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Description:

Determine wave forces on Shemya Wharf. Load Case N End Wharf Face at High Tide.

References:

1. FEMA, Guidance for Flood Risk Analysis and Mapping, Coastal Wave Runnup and Overtopping, February 2018.
2. USACE, Coastal Engineering Manual, EM 1110-2-1100 (Part VI), Change 3 - Sept 2011.
3. USACE, Shore Protection Manual, Volume II, 1984.

Calculations:

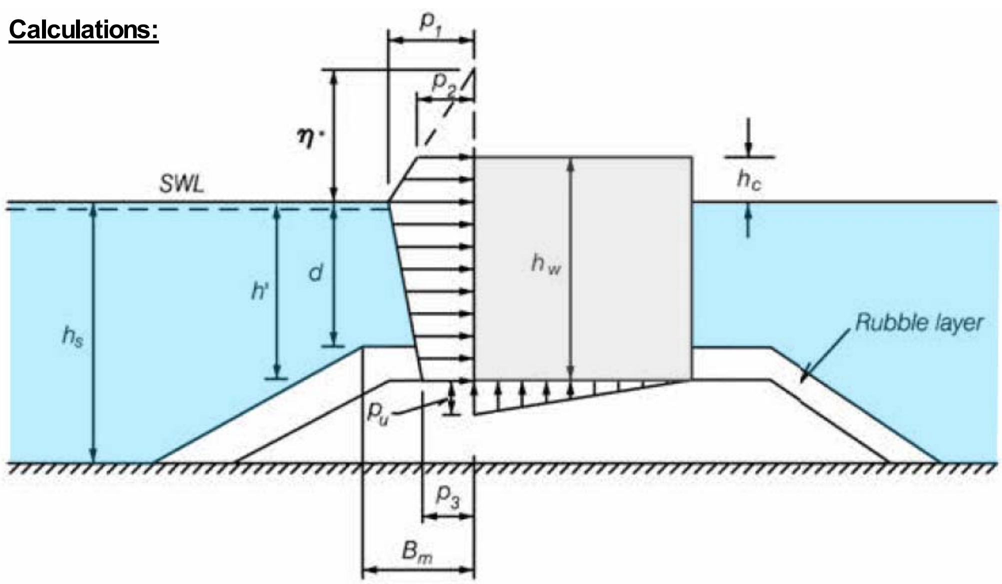

Figure 1: Table VI-5-53 Goda Formula for Irregular Waves (Excerpt from Ref 2)

Step 1: Determine design parameters and heights

$H_s := 15ft$ Significant Wave Height

$H_{design} := 1.8 \cdot H_s$ Design Wave Height $H_{design} = 27\ ft$

$EL_{top} := 22.7ft$ Top of wall elevation

$EL_{bot} := -33ft$ Mudline Elevation

$EL_{WL} := 4.5ft$ Water level elevation

$h_s := EL_{WL} - EL_{bot}$ $h' := h_s$ $d := h_s$ $h_s = 37.5\ ft$

$h_c := EL_{top} - EL_{WL}$ $h_c = 18.2\ ft$

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$h_b := 44\text{ft}$ Water Depth at distance of 5*Hs

$h_w := EL_{top} - EL_{bot}$ Height of structure from deck to mudline $h_w = 55.7\text{ ft}$

$T_p := 14.84\text{sec}$ Period of significant wave

$L := 535.6\text{ft}$ Wave Length (at structure - Per Table C1, Ref 3)

$g := 32.2 \frac{\text{ft}}{s^2}$ Acceleration due to gravity

$\gamma_w := 64\text{pcf}$ Density of seawater

Step 2: Determine p_1, p_2, p_3

$\beta := 0\text{deg}$ Angle of incidence of waves

$\lambda_1 := 1 \qquad \lambda_2 := 1$ Modification factors (Assumed for conventional vertical wall structure)

$$\eta' := 0.75 \cdot \left[(1 + \cos(\beta)) \cdot \lambda_1 \cdot H_{design}\right] \qquad \eta' = 40.5\text{ ft}$$

$$\alpha_1 := 0.6 + 0.5 \cdot \left(\frac{4 \cdot \pi \cdot \frac{d}{L}}{\sinh\left(4 \cdot \pi \cdot \frac{d}{L}\right)}\right)^2 \qquad \alpha_1 = 0.989$$

$$\alpha_2 := \min\left[\frac{h_b - d}{3 \cdot h_b} \cdot \left(\frac{H_{design}}{d}\right)^2, \frac{2 \cdot d}{H_{design}}\right] \qquad \alpha_2 = 0.026$$

$$\alpha_3 := 1 - \left(\frac{h_w - h_c}{h_s}\right) \cdot \left(1 - \frac{1}{\cosh\left(\frac{2 \cdot \pi \cdot d}{L}\right)}\right) \qquad \alpha_3 = 0.91$$

$$p_1 := 0.5 \cdot (1 + \cos(\beta)) \cdot \left[\lambda_1 \cdot \alpha_1 + \lambda_2 \cdot \left[\alpha_2 \cdot (\cos(\beta))^2\right]\right] \cdot \gamma_w \cdot H_{design} \qquad p_1 = 1752.66 \cdot \text{psf}$$

$$p_2 := \begin{cases} \left(1 - \frac{h_c}{\eta'}\right) \cdot p_1 & \text{if } \eta' > h_c \\ 0\text{ksf} & \text{otherwise} \end{cases} \qquad p_2 = 965.05 \cdot \text{psf}$$

$$p_3 := \alpha_3 \cdot p_1 \qquad p_3 = 1595.74 \cdot \text{psf}$$

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Calculated By: J. Gaythwaite
Date: 09/08/2020
Checked By: E. Levesque
Date: 09/09/2020

Step 3: Determine Total Wave Force & Location on the wall

$P_1 := \left(\frac{p_1 + p_2}{2}\right) \cdot h_c$	Wave force above water level	$P_1 = 24.73 \cdot klf$
$y_{P1} := \left[\frac{p_1 + 2 \cdot p_2}{3 \cdot (p_1 + p_2)}\right] \cdot h_c$	Centroid of P1	$y_{P1} = 8.22\ ft$
$H_{P1} := h_b + y_{P1}$	Height of P1, from mudline	$H_{P1} = 52.22\ ft$
$M_{P1} := P_1 \cdot H_{P1}$	Moment due to P1, from mudline	$M_{P1} = 1291.48 \cdot \frac{kip \cdot ft}{ft}$
$P_2 := \left(\frac{p_1 + p_3}{2}\right) \cdot d$	Wave force below water level	$P_2 = 62.78 \cdot klf$
$y_{P2} := \left[\frac{p_3 + 2 \cdot p_1}{3 \cdot (p_1 + p_3)}\right] \cdot d$	Centroid of P2	$y_{P2} = 19.04\ ft$
$H_{P2} := y_{P2}$	Height of P2, from mudline	$H_{P2} = 19.04\ ft$
$M_{P2} := P_2 \cdot H_{P2}$	Moment due to P2, from mudline	$M_{P2} = 1195.56 \cdot \frac{kip \cdot ft}{ft}$
$P_T := P_1 + P_2$	Total wave force	$P_T = 87.51 \cdot klf$
$H_T := \frac{M_{P1} + M_{P2}}{P_T}$	Height of total wave force, from mudline	$H_T = 28.42\ ft$