US Army Corps of Engineers

Geotechnical Feasibility Report

Craig Harbor Navigation Improvements

Craig, Alaska, Alaska District, Pacific Ocean Division

November 2014

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Status: FINAL

Department of the Army
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P.O. Box 6898
JBER, AK 99506-0898

CEPOA-EN-G-GM November 2014

MEMORANDUM FOR

Civil Works Project Management (CEPOA-PM-C-PL), Lorraine Cordova

SUBJECT: Geotechnical Feasibility Report for Craig Harbor Navigation Improvements, Craig, Alaska

1. This report was authorized and forwarded to Geotechnical and Materials Section by the Civil Works Project Management Branch via the Project Manager, Lorraine Cordova.

2. Enclosed is the Geotechnical Feasibility Report for the Craig Harbor Navigation Improvements, Craig, Alaska. Included with the report are the Project Location and Vicinity Map, the Potential Quarry Location Map, geologic conditions, a discussion of the quarry findings, and preliminary engineering design recommendation for the project with assumptions.

3. As a result of the site visit and knowledge of the area the project site is suitable for the construction of the proposed rubble-mound breakwater. There are several potential material sources in the area that are capable of providing rock for construction. This is only a feasibility level design and an in depth geotechnical investigation should be performed on the subsurface soils to verify assumptions made during the feasibility study.

4. Results and questions should be addressed to Robert Weakland at 907-753-2633 or John Rajek at 907-753-5695.

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1.0 Introduction

This report presents the findings of the Geotechnical Feasibility Study for the selected alternative from of the Navigation Improvements Feasibility Study for Craig, Alaska. This alternative involves dismantling an old dilapidated wood dock and constructing a rubble-mound breakwater in Wards Cove, Craig, Alaska.

The scope of the study was to perform a literature search, analyze the project site with available data, identify potential material sources, identify general surface and potential subsurface conditions, and address geotechnical concerns relevant to the project. There was no site specific investigation performed for this effort. This report presents a summary of the findings based on historical documents, site observations, and geotechnical assumptions to determine project feasibility with a geotechnical perspective. This report also includes a preliminary engineering analysis for assumed site conditions and preliminary geotechnical recommendations for the design and construction of the proposed rubble-mound breakwater system.

An extensive exploration program and a more detailed engineering analysis are needed before the final geotechnical recommendations for the design and construction of the proposed rubble-mound breakwater can be made.

2.0 Location and Project Description

The project site is on Prince of Wales Island in Southeast Alaska in the community of Craig located about 60 miles west of Ketchikan, Alaska. The location is shown on the enclosed Vicinity Map in Appendix A, Sheet A-1. A number of alternative were considered; the selected alternative would consist of a 10-acre basin protected by a 1,933-foot long rubble-mound breakwater configured in an “L-shape”. The proposed breakwater layout has no western opening except for a 10-foot gap between the stub breakwater and the main breakwater provide for fish passage. This design provides protection against waves from all westerly and northerly directions. This basin would be able to accommodate 145 vessels. The breakwater configuration is shown on the enclosed Project Layout Map in Appendix A, Sheet A-2.

2.1. Site History

The primary problem is current moorage demand at Craig, Alaska exceeds supply. The City of Craig and the surrounding area is heavily dependent upon access to protected moorage in order to safely and efficiently engage in commercial, recreational, and subsistence fishing activities. The Navigation Improvements Feasibility Study selected a project site that had been used by Wards Cove Packing Company. Many of the company's structures remain along the north shore of Wards Cove. The machinist shop, cannery, freezer house, seine net lofts and boilers are shown in Figure 1 and 2.

Figure 1. Old machinists shop at the old Wards Cove Packing Company Cannery site. Building next to it was the freezer house and the seine net lofts.

Figure 2. Photograph is to the northeast of Words Cove Packing Company Cannery old boilers.

The remains of a decaying timber dock and debris can be seen along the waterfront of the project site. Some steel rigging and machinery can be seen at low tide. Due to the sites past usage, marine related mechanical and structural debris is expected below the water in Wards Cove.

2.2.Literature Search

A literature search revealed mostly historical, environmental, and economical documentation, and very little geologic and geotechnical data.

2.2.1. Navigation and Harbor Improvement, 1979

A study was conducted in the late 1979 to improve the Craig Harbor and overall navigation following major damage to their floating dock system in Craig's south harbor. This report had a record of soil probing and test pits from September 1956. The probes and test pits indicated medium hard to hard blue clay, sand, gravel, cobbles, and boulders. These probes and test pits were in the vicinity of the south harbor. Following this study the city constructed two rubble-mound breakwaters to an elevation of 20 feet MLLW (mean lower low water) for south harbor moorage protection.

2.2.2. Craig Small Boat Harbor Entrance Channel Improvements, 1992

A study was conducted in January 1992 for new dredging in the south small boat harbor in Craig, Alaska. This study incorporated two test pits along the southern edge of the small boat harbor. The surface was reported to be loose clayey sand with gravel, cobbles and boulders (SC). The test pits indicated the subsurface consisted of dense clayey sand with gravel, cobbles and boulders (SC) and dense clayey gravel with sand, cobbles and boulders (GC). The clay matrix was reported to be very hard. These two test pits are very similar to the test pits performed in 1956.

These soils appear to be in a loose state near the surface, but become very hard at about three feet below the mudline. The loose soils are wet and became soupy when disturbed while the very hard soils are moist and have liquid limits ranging from 28 to 31 and plastic indices range from 14 to 17 for the clay matrix. Moisture contents of 9 and 11 percent were measured from samples procured in the very hard stratum. The very hard clayey stratum is reported to be underlain by bedrock. The bedrock was not confirmed during this exploration.

2.2.3. Craig Coastal Management Program, 2006

This document is the amended 1984 Craig Coastal Management Plan, with the changes required to meet the standards of U.S. House Bill 191 and the resulting 11 AAC 110, 11 AAC 112 and 11 AAC 114. Along with inventories, conditions, policies, goals and objectives in this plan, a presentation of Craig's Geology and Soils is covered as well. The geology and soils remarks were prepared from seven test pits dotted about Craig Island. The test pits were conducted by Stragier Engineering Services for CH2M Hill for the 1984 Craig Coastal Management Plan. The soils Test Pit Logs report the subsurface was found to be predominately glacial till. The Stragier Engineering/CH2M Hill Report is unavailable. The Craig Coastal Management Plan Soils Map and Soils Test Pit Logs are included in Appendix A as Sheet A-3 and A-4.

2.2.4. Reconnaissance to Evaluate Material Sources Trip Report, 2014

Between 24 and 25 April 2014, Christine Morgan (Cost Engineering) and Coleman Chalup (Geotechnical and Materials Section) traveled to Craig Alaska to inspect the proposed harbor site and determine if any quarries in the area are capable of producing adequate rock for the proposed harbors breakwater. The team met with several local quarries owners during the two days to see if existing quarries are available and obtain information. This trip report is included in Appendix C.

3.0 Regional Geology

The Prince of Wales Mountains are underlain in part by well-consolidated slightly metamorphosed Paleozoic sedimentary and volcanic rocks and in part by crystalline schist and marble. Glaciations have been the most significant factor in modifying the land. Pleistocene glaciations by ice sheets and alpine glaciers had marked effects. Pre-glacial drainage lines were widened and deepened to form U-shaped valleys and deep fiords. Mountain peaks and ridges were rounded and probably only a fee1 of the highest peaks stood above the limits of glaciations. Also, as a result of the glaciations, much of the area is covered by glacial drift of varying thickness.

4.0 Site Conditions

No geotechnical information was available for the project site. Engineering assumptions, judgment, and visual observations were used to evaluate the probable site conditions. The assumptions and judgments within this report are based on past, nearby geotechnical related reports. Tidal data for the Craig local area is provided in Table 1. Saltwater density for calculations in this report is assumed to be 64.0 pounds per cubic foot. In order to obtain a more accurate understanding of the geotechnical site picture, a detailed site investigation will be needed.

Table 1. Tidal data for the Craig, Alaska referenced to Mean Lower Low Water (MLLW). Water level data is from the National Oceanic and Atmospheric Administration (NOAA) online database, 15 October 2013.

Description	Elevation (feet)
Highest Astronomical Tide	12.6
Mean Higher-High Water	10.2
Mean High Water	9.3
Mean Tide Level	5.4
Mean Low Water	1.4
Mean Lower-Low Water (Datum)	0.0
Lowest Astronomical Tide	-2.9

4.1. Surface Conditions

The project site has an approximate 13 percent grade sloping down from plus five feet to minus 20 feet MLLW to the north and shallows to an approximate six percent grade to minus 45 feet MLLW. The surface is composed of sand with some fines, gravel, and an abundance of cobbles and boulders; Figure 3 and 4. Remains of the Wards Cove Packing Company timber dock are littered over a large portion of the project site. This includes remnants of the dilapidated timber dock; scattered timber piles are protruding out of the water as well as lying on the sea floor, and large iron rigging, equipment and/or machinery; Figure 4 though 6.

Figure 3. Large boulders can be seen in the photograph near the eastern project limits.

Figure 4. Cobbles and boulders can be seen at the project site near the dilapidated timber dock.

Figure 5. Photograph is looking north; dilapidated wood dock of the Wards Cove Packing Company Cannery site.

Figure 6. Photograph shows a large unknown piece of iron equipment or machinery off the side of the old dock.

4.2. Subsurface Conditions

Subsurface conditions are expected to be similar to the subsurface conditions encountered at the Craig South Boat Harbor. Historical explorations performed at the south boat harbor indicated the subsurface consisted of dense clayey sand with gravel, cobbles and boulders. Bedrock was encountered during the 1956 soil probing from six to ten feet below mudline. The 1956 probing data was limited. There wasn't sufficient data to construct a bedrock profile; however, it appears the bedrock dips to the southwest. The current project site is west of this location, so the bedrock is suspected to be deeper; however, without further exploration, it is impossible to predict its true depth.

5.0 Rock Source

Two local quarries have the potential to produce the required size, quality, and quantity of rock needed for the project. Seven quarries were inspected during a site visit 24 and 25 April 2014. A trip report documents the inspection and is provided in Appendix C. Of the seven quarries, St. Johns Quarry and Southeast Road Builders Quarry are believed to have the best chance at producing the required rock. St. Johns Quarry is located five miles southwest of Craig on San Juan Bautista Island and the rock would have to be barged to the site. South east Road Builder Quarry is accessible by road and is approximately six miles to the north. The rock could be trucked or barged. Details, comments, photographs, and maps regarding the quarry inspection are in Appendix C.

6.0 Geotechnical Engineering Analysis

A section profile near the midpoint between the bend and end of the breakwater footprint was selected for analysis. This profile was selected because it had the maximum height, even though the crest elevation of the proposed breakwater is 18 feet MLLW. Based on weight, this point in the breakwater would have the lowest factor of safety; therefore, it was considered the critical section profile.

6.1. Design Factors of Safety

Appropriate factors of safety were utilized to ensure adequate performance of the project throughout its design life. Two of the most important considerations that determine appropriate magnitudes for factor of safety are uncertainties in the conditions being analyzed, including assumptions and consequences of failure and acceptable performance. Table 2 is a list of applicable Factors of Safety that were use for analysis.

Table 2. List of required factors of safety.

Reference	Analysis Condition	Required Minimum Factor of Safety
Engineering and Design Manual, EM 1110-1-1904	Settlement Analysis	Conducted to design crest elevation
Engineering and Design Manual, EM 1110-1-1905	Bearing Capacity	2.5
Engineering and Design Manual, EM 1110-2-1902	Slope Stability, End-of-Construction	1.3
Engineering and Design Manual, EM 1110-2-1902	Slope Stability, Long Term	1.5
Engineering and Design Manual, EM 1110-2-1902	Slope Stability, Earthquake Loading	>1.0

6.2. Assumed Soil Properties

6.2.1. Insitu

Geotechnical data available from existing reports, surveys, test pits, and test probes are insufficient to accurately describe the engineering properties of the insitu soil within the project vicinity. Therefore, the properties of the insitu soils were assumed based on available geotechnical data. The proposed rubble-mound breakwater will be constructed directly on the insitu soil with no excavation or foundation preparation. The insitu soils were assumed to be one homogenous unit of poorly graded clayey gravel with sand, cobbles and boulders with a unit weight of 130 pcf (pound per cubic foot) and an internal friction angle of 32 degrees (Cornell University, 1990). Soil cohesion and bedrock were neglected.

6.2.2. Rubble-mound Breakwater Material

The breakwater is composed of three different rock materials; armor rock (A Rock), intermediate rock (B Rock), and core rock. For estimating purposes we have assumed a porosity (n) value of 37 percent for all of the in-place large stone products for all three sites. A conservative estimate of specific gravity of the stone to be used for the project is 2.65. To calculate the estimated dry unit weight of in-place large stone the following relationship was used between specific gravity, porosity, and the unit weight of water:

$$\gamma_d = G_s(1 - n)\gamma_w$$

$$\gamma_d = \text{Estimated Dry Unit Weight (lbs/ft}^3\text{)}$$

$$G_s = \text{Relative Density (Specific Gravity), (BSSD)}$$

$$n = \text{Porosity (assumed 37 percent) (USACE Shore Protection Manual)}$$

$$\gamma_w = \text{Unit Weight of water} \quad \left(64.0\,\frac{\text{lb}}{\text{ft}^3}\right)$$

$$\gamma_d = 2.65(1 - 0.37)64.0\,\frac{lb}{ft^3} = 107\,\frac{lb}{ft^3}$$

This value was used for all rock material used for the design of the breakwater system.

6.1. Bearing Capacity

The allowable bearing capacity "Q_a" is the ultimate bearing capacity "Q_u" divided by an appropriate factor of safety "FS". A reasonable factor of safety is based on the available subsurface and surface information, variability of the soil, soil layering and strengths, type and importance of the structure and past experience with like structures. The FS range for embankment is typically 2 to 4; however for marine structures 2.5 is recommended.

The Meyerhoff's general bearing capacity equation was used to check the subgrade. For this analysis, the insitu soil is assumed to be in a drained condition; its unit weight, internal friction angle, and width are assumed to be 130 pcf, 32 degrees, and 240 feet respectively. The embedment depth is zero and assuming there is no cohesion, two of the bearing capacity factors drop out; N_q and N_c, and the factor for unit weight "N_γ" is 22.0. The ultimate bearing capacity is as follows:

$$Q_u = \frac{1}{2}\gamma' B N_\gamma = \frac{1}{2}\cdot(130 - 64)\cdot 240 \cdot 22.0 = 174.2ksf$$

The loaded area is essentially flat with very little relief; therefore, no eccentric loading is assumed. For the most conservative factor of safety, the lowest estimated water level (-2.9 feet MLLW) was used when calculating the embankment loading. The waterline relative to the mudline is 42.1 feet, the embankment height relative to mudline is 63.0 feet and the embankment material is assumed to be 107 pcf. Calculation of the embankment loading is:

$$Q_a = 107\cdot(63.0 - 42.1) + 42.1\cdot(107 - 64) = 4.1ksf$$

The allowable bearing capacity to check the factor of safety yields:

$$FS_{bearing\ capacity} = \frac{Q_u}{Q_a} = \frac{174.2}{4.1} = 42.5$$

Ultimate shear failures are seldom a controlling factor in design because few structures are able to tolerate the rather large deformations that occur in soil prior to failure. It is expected slope stability failure will occur as a partial bearing capacity failure well before a full failure. For this reason, slope stability will control the design.

6.2. Slope Stability

Analytically there are three different conditions that must be evaluated for slope stability analyses; static, pseudo-static, and post earthquake. For this feasibility report, a post earthquake analysis was not performed.

The critical section identified in beginning of Section 6.0 was modeled and analyzed in Slope/W®; part of the GeoStudio 2012® software package. The conventional limit-equilibrium methodology (Spencer Method) was used for the primary analysis. The pore water pressure was modeled with a static piezometric line set at the lowest astronomical tide. This method identified the most critical slip surface with the lowest factor of safety from numerous slip surfaces. To enhance the search process for the critical slip surface and because the slip surface typically is not circular, a Monte-Carlo type optimization technique, which is also known as the "random walking" method (Greco, 1996), was employed.

The required minimum factor of safety for "long term" slope stability is greater than the "end-of -construction" slope stability. The results concluded the end-of-construction factor of safety meets the minimum factor of safety for long term stability; therefore, long term slope stability was not analyzed. The results of this end-of-construction analysis are shown on Table 4 and the analysis can be found in Appendix B.

6.3. Earthquake Ground Motions Slope Stability

The project is in a seismic area where major earthquakes can and have occurred. Per ASCE/SEI 7-10, this breakwater system and location classifies as Seismic Design Category E, corresponds to Occupancy Groups I, II, and III; an area near major active faults and is a non-essential structure. Due to it classification and non-life safety status, seismic analysis is not required; however, some analysis was conducted to show this project will perform during a design level earthquake.

Two earthquake events were investigated to determine performance of the proposed rubble-mound breakwater based on general criteria provided in engineering regulation 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects and IBC 2012. The two levels of earthquakes evaluated were defined as the Maximum Design Basis Earthquake (MDE) defined by ground motions having a 10 percent probability of being exceeded in 50 years; the Maximum Credible Earthquake (MCE) defined by ground motions having a two percent probability of being exceeded in 50 years. The MDE is used to evaluate design level performance; whereas, the MCE illustrates the small safety margin for higher design earthquakes.

The peak horizontal ground acceleration for each design earthquake event was determined using time-independent probabilistic seismic hazard maps of Alaska provided by the U.S. Geological Survey (USGS). The USGS probabilistic seismic hazard deaggregation for peak ground acceleration for ten percent in 50 years is shown in Figure 7.

The horizontal seismic coefficient, Kh, used as a pseudostatic loading condition was assumed to be half of the peak horizontal ground acceleration based on recommendations provided in the AASHTO LRFD Bridge Design Manual (2012). The seismic coefficient values and the peak horizontal ground accelerations for the MDE and the MCE are provided in Table 3.

Table 3. Seismic Coefficients for Design Earthquake Events

Design Earthquake Event	Probability of Exceedance	PHGA (g)	Kh (g)
Maximum Design Earthquake (MDE)	10% in 50 years	0.10	0.05
Maximum Credible Earthquake (MCE	2% in 50 years	0.18	0.09

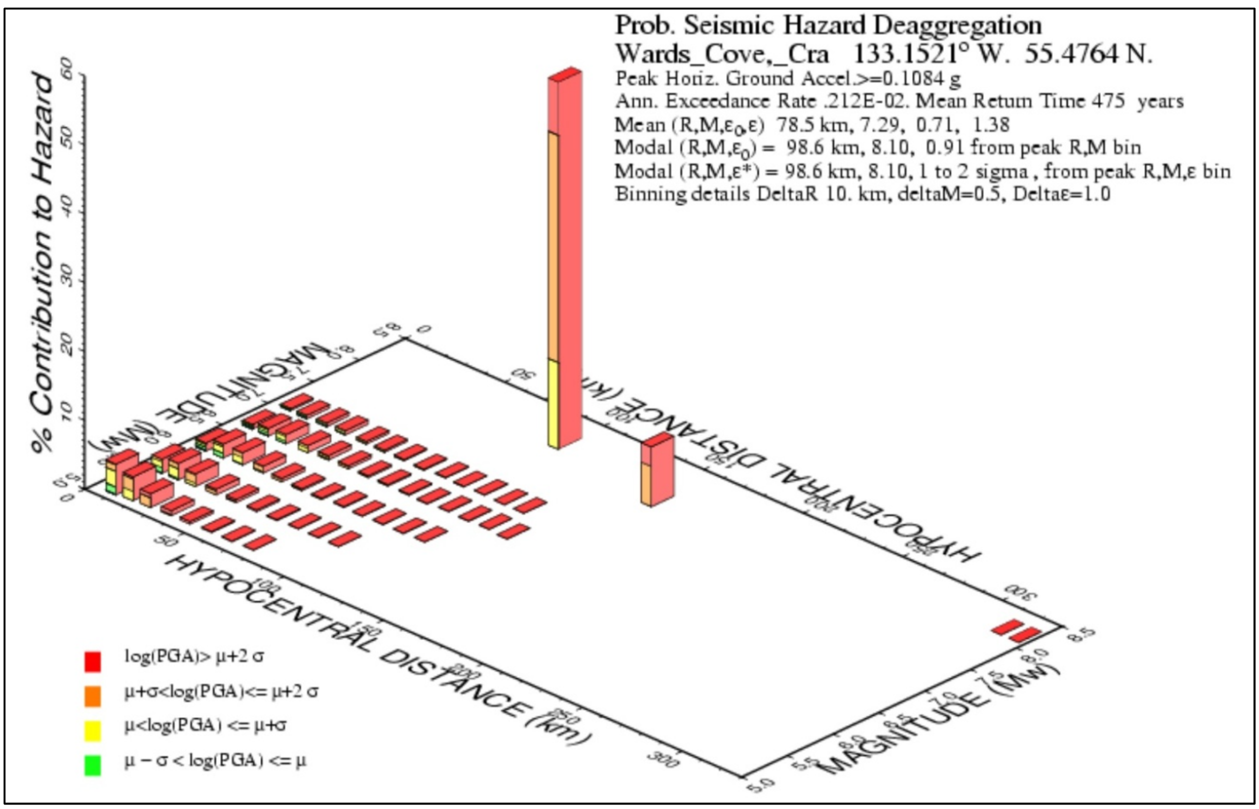

Figure 7. USGS Probabilistic Seismic Hazard Deaggregation for Peak Ground Acceleration two percent in 50 years.

The end-of-construction condition was evaluated for slope stability during earthquake loading for the critical profile for the maximum design earthquake and the maximum credible earthquake. The results are provided in Table 4 for both slip surfaces of the critical profile section.

Table 4. Comparison of calculated and required minimum factors of safety based on EM 1110-2-1902.

Profile Section		Factor of Safety					
		*EOC		**MDE		***MCE	
		Minimum	Calculated	Minimum	Calculated	Minimum	Calculated
Slip Surface	Harborside	1.3	1.71	>1.0	1.37	NA	1.15
	Seaside	1.3	1.77	>1.0	1.42	NA	1.21

* End of Construction

** Maximum Design Earthquake

*** Maximum Credible Earthquake

6.4. Liquefaction Analysis

Loose sandy coarse grained soil or soft fine soil are not expected at the project site. As the project requirements developed, the primary purpose of our slope stability and liquefaction analyses has been to assess the likelihood of slope movements that could result in slope failure of the rubble mound breakwaters. A more detailed site specific investigation is required to conduct a complete liquefaction analysis.

6.5. Settlement Analysis

Placement of an embankment load on the surface of a soil mass introduces stress in the soil that causes the soil to deform and leads to settlement. Total deformation is significant relative to the crest minimum height specifications, and materials consumed by the construction of the embankment. Any calculation using assumed soil properties would lead to gross approximations of settlement values. Two breakwaters have been constructed for the Craig South Boat Harbor and experience little to no apparent settlement and current surveys show no long term differential settlement. It is assumed, most to all the settlement occurred during construction. Due to the performance of these two breakwaters, it is feasible to assume little to no settlement or differential settlement will occur after the construction of the proposed rubble-mound breakwater.

7.0 Engineering Recommendations

As a result of this geotechnical feasibility study, the project site is suitable for the construction of the proposed rubble-mound breakwater system. All calculate geotechnical engineering factors of safety within this report meet or exceed the USACE's minimums. Rock sources are available in the local area that can produce the required size, quality, and quantity for the proposed project. Any quarry selected must produce rock that meets rock quality specification outline in the USACE's Shore Protection Manual.

Recommendations presented in this section are meant as preliminary engineering recommendations for the proposed rubble-mound breakwater system for the selected alternative. A more in-depth geotechnical evaluation should be prepared in the event that this project is brought to 100 percent design. Preliminary recommendations presented in this section can generally be applied to the all segments of breakwaters. No recommendations are given for dredging; whereas dredging was not included in the selected alternative.

8.0 References

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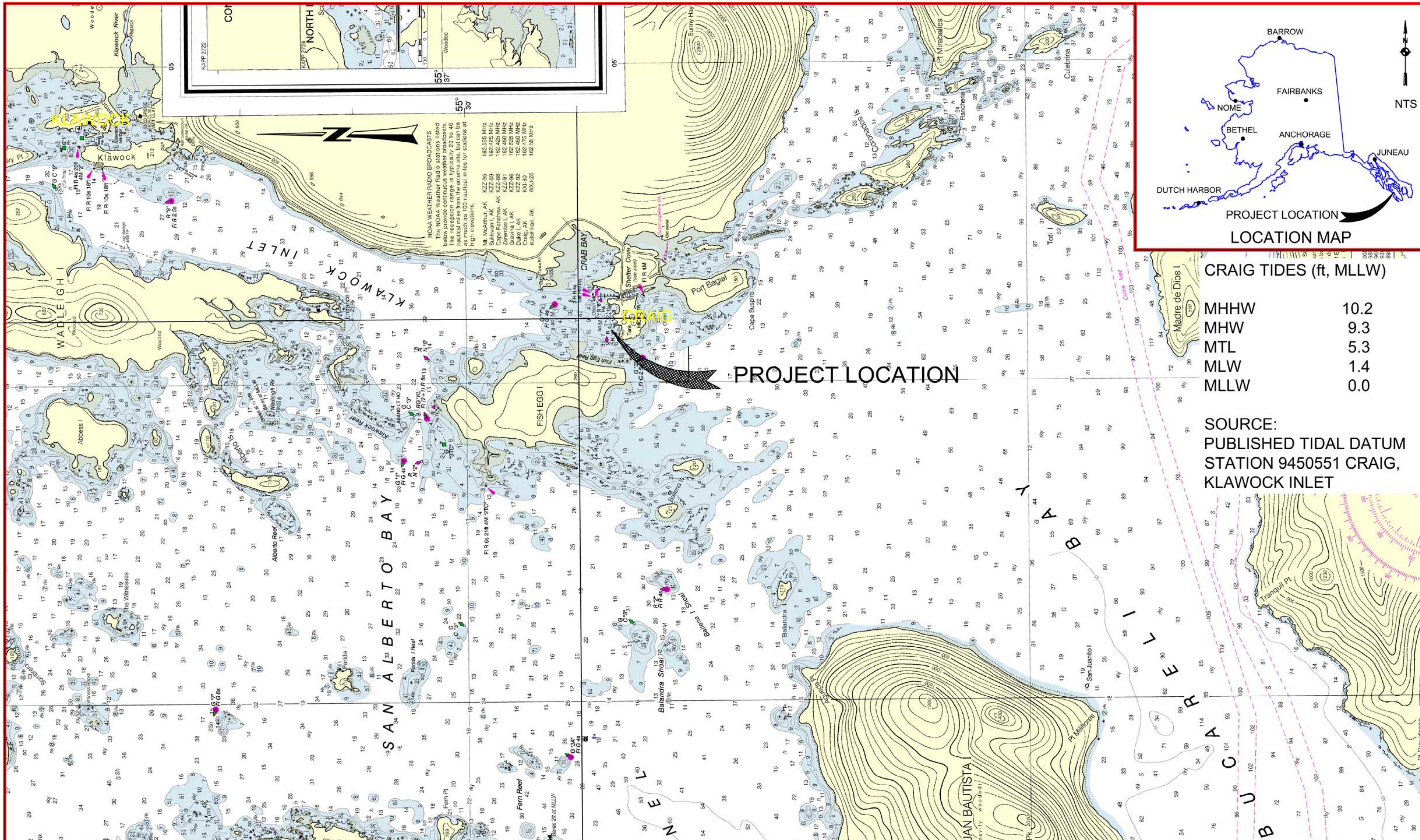
APPENDIX A
Project Maps



CRAIG TIDES (ft, MLLW)

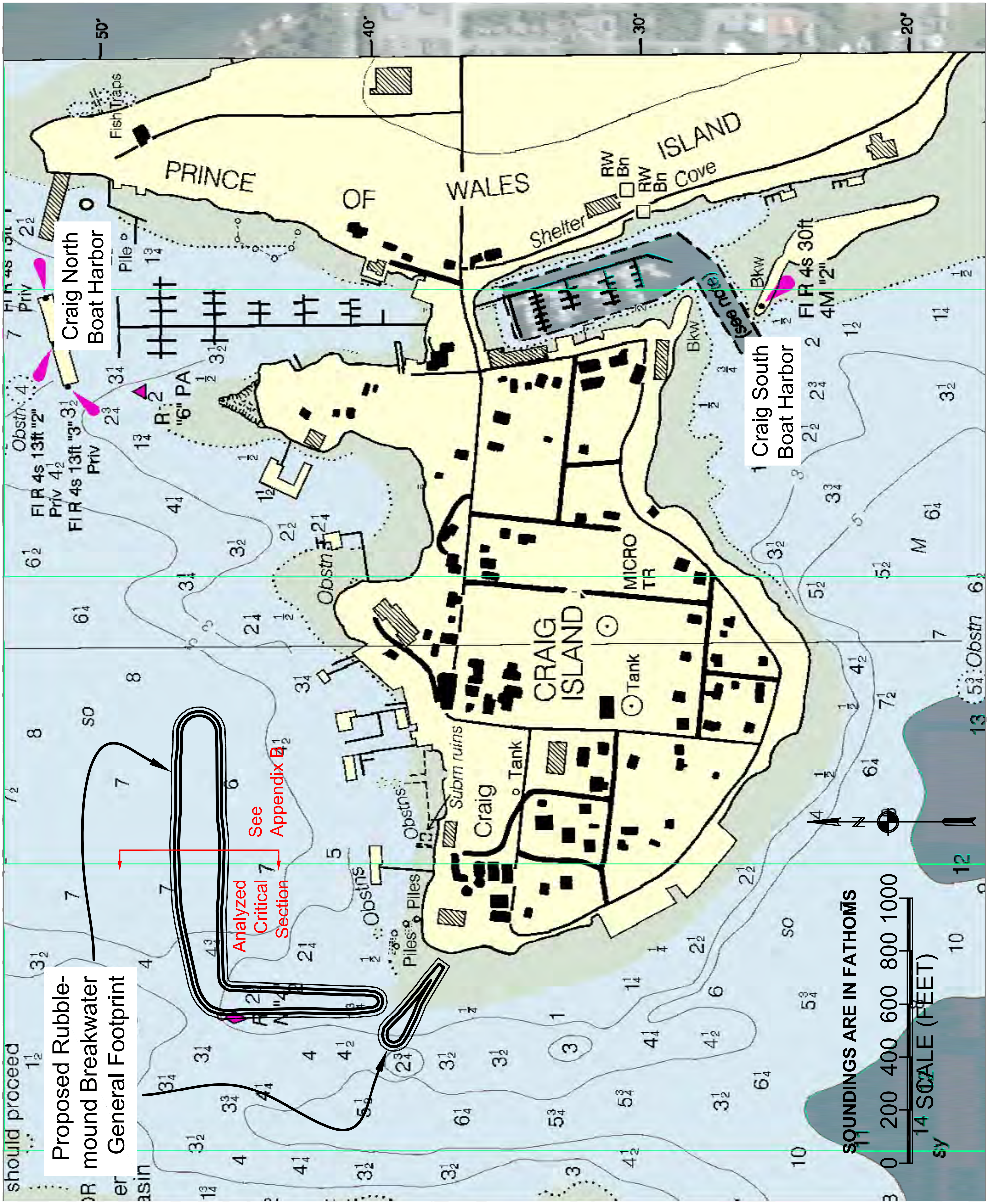
MHHW	10.2
MHW	9.3
MTL	5.3
MLW	1.4
MLLW	0.0

SOURCE:
PUBLISHED TIDAL DATUM STATION 9450551 CRAIG, KLAWOCK INLET

ALASKA DISTRICT
U.S. ARMY
CORPS OF ENGINEERS

LOCATION & VICINITY
CRAIG NAVIGATION IMPROVEMENTS FEASIBILITY STUDY

A-1



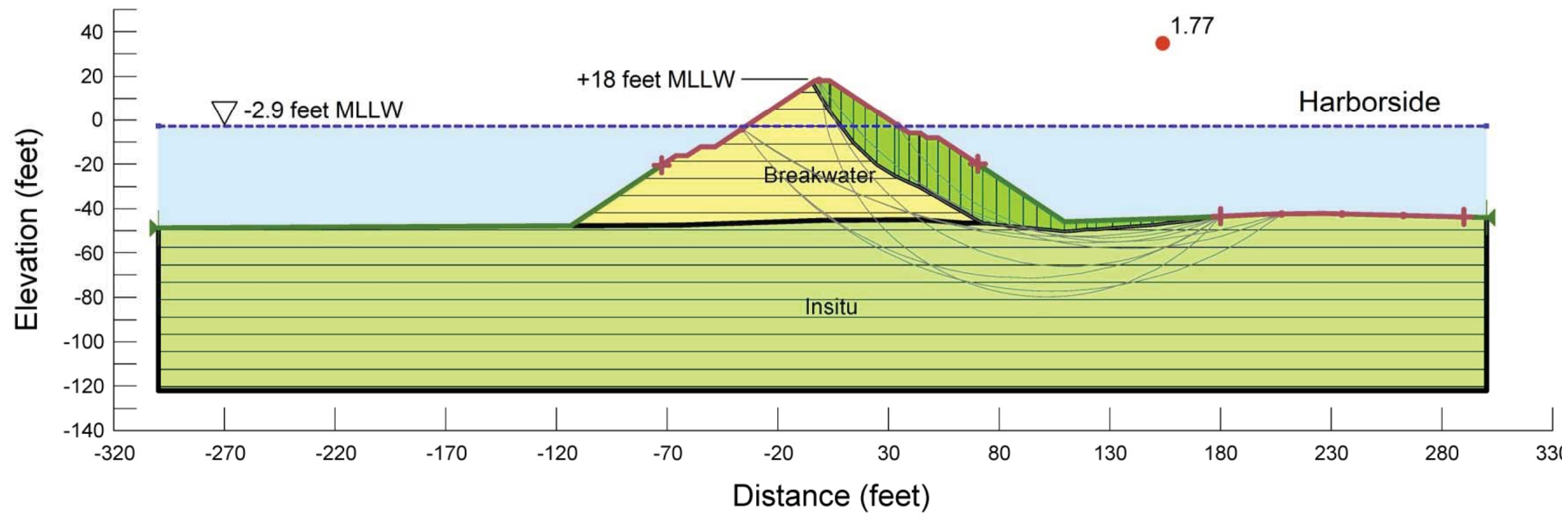
ALASKA DISTRICT
CORPS OF ENGINEERS
Geotechnical and Materials

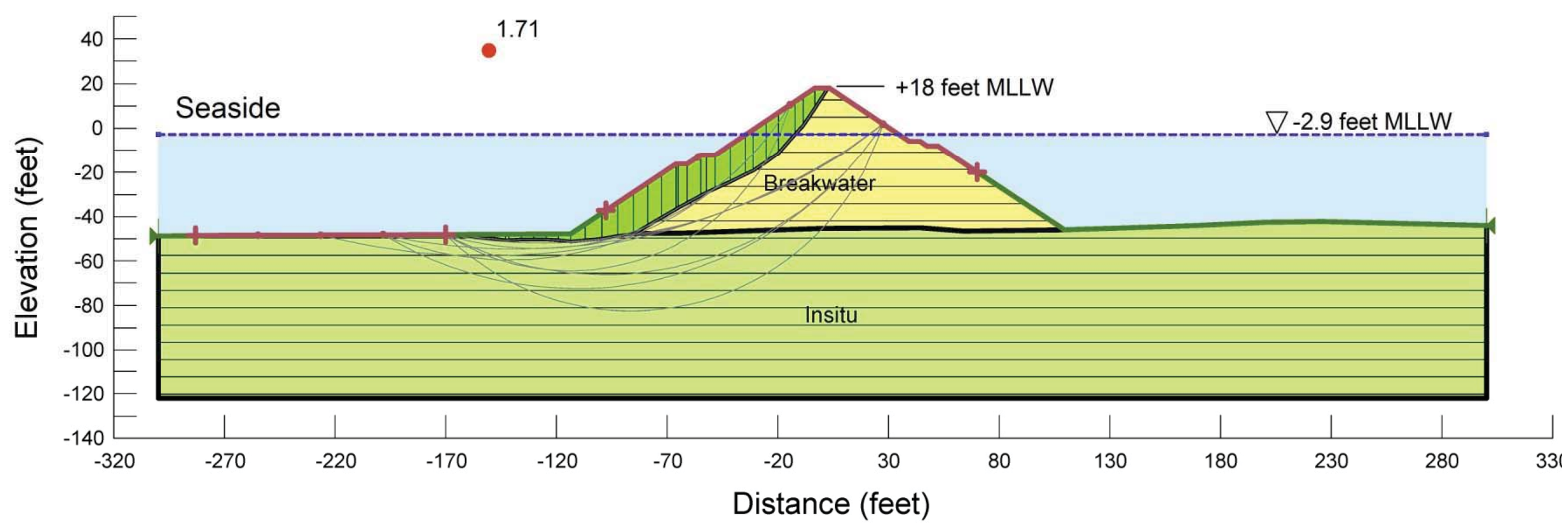
PROJECT LAYOUT MAP CRAIG NAVIGATION IMPROVEMENTS FEASIBILITY STUDY
CRAIG, ALASAK

SCALE:	GRAPHIC
DATE:	NOV 2014
DRAWN/RVW:	RW/JR
SHEET A-2	

APPENDIX B

Slope Stability





BUILDING STRONG®

SLOPE/W Analysis

Harborside

File Information

Created By: Weakland, Robert T POA
Last Edited By: Weakland, Robert T POA
Revision Number: 21
File Version: 8.2
Tool Version: 8.12.3.7901
Date: 11/20/2014
Time: 12:54:17 PM
File Name: SlopeStability 1500 Harborside.gsz
Directory: O:\EN\Private\ES\ES-SG\1 MAIN PROJECT FILES\3 Civil Works\Craig\Craig harbor Navigation Improvements\Project Data\Analysis\
Last Solved Date: 11/20/2014
Last Solved Time: 12:54:22 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 64 pcf
View: 2D
Element Thickness: 1

Analysis Settings

SLOPE/W Analysis

Kind: SLOPE/W
Method: Spencer
Settings
  Lambda
    Lambda 1: -1
    Lambda 2: -0.8
    Lambda 3: -0.6
    Lambda 4: -0.4
    Lambda 5: -0.2
    Lambda 6: 0
    Lambda 7: 0.2
    Lambda 8: 0.4
    Lambda 9: 0.6
    Lambda 10: 0.8
    Lambda 11: 1

PWP Conditions Source: Piezometric Line
Apply Phreatic Correction: No
Use Staged Rapid Drawdown: No
Slip Surface
Direction of movement: Left to Right
Use Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: Yes
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Breakwater

Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion': 0 psf
Phi': 45 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Insitu

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion': 0 psf
Phi': 32 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (-72.60831, -20.5722) ft

Left-Zone Right Coordinate: (70.25, -20) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (180, -43.5682) ft
Right-Zone Right Coordinate: (290, -43.75502) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (-300, -48.60337) ft
Right Coordinate: (300, -43.9825) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
Coordinate 1	-300	-2.9
Coordinate 2	300	-2.9

Points

	X (ft)	Y (ft)
Point 1	-300	-48.60337
Point 2	-113.56868	-47.87912
Point 3	-64.12049	-47.45269
Point 4	0	-45.40455
Point 5	44.55991	-45.11672
Point 6	64.12932	-46.59365
Point 7	109.38485	-46.0899
Point 8	168.76117	-44.09303
Point 9	200.88919	-42.59272
Point 10	225.79381	-42.29448
Point 11	300	-43.9825
Point 12	300	-122
Point 13	-300	-122
Point 14	52.25	-8
Point 15	47.25	-8
Point 16	44.25	-6
Point 17	39.25	-6
Point 18	3.25	18
Point 19	-3.25	18

Point 20	-48.25	-12
Point 21	-54.75	-12
Point 22	-61.125	-16.25
Point 23	-66.125	-16.25

Regions

	Material	Points	Area (ft^2)
Region 1	Breakwater	5,6,7,14,15,16,17,18,19,20,21,22,23,2,3,4	7,406.6
Region 2	Insitu	1,13,12,11,10,9,8,7,6,5,4,3,2	45,533

Current Slip Surface

Slip Surface: 126
F of S: 1.765
Volume: 2,388.0923 ft^3
Weight: 261,363.68 lbs
Resisting Moment: 16,122,797 lbs-ft
Activating Moment: 9,134,052.8 lbs-ft
Resisting Force: 87,360.523 lbs
Activating Force: 49,653.494 lbs
F of S Rank: 1
Exit: (180, -43.568201) ft
Entry: (-4.4874419, 17.175039) ft
Radius: 99.475387 ft
Center: (126.82069, 97.484705) ft

Slip Slices

	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
Slice 1	-3.8687209	16.137395	-1,218.3933	73.950875	73.950875	0
Slice 2	0	9.6492465	-803.15178	425.85699	425.85699	0
Slice 3	3.50907	3.7642607	-426.51269	717.16139	717.16139	0
Slice 4	6.0992665	0.21489	-199.35296	914.06009	914.06009	0
Slice 5	11.225901	-6.635405	239.06592	1,199.0339	959.96802	0
Slice 6	16.669442	-12.821025	634.9456	1,669.9289	1,034.9833	0
Slice 7	21.965508	-17.721455	948.57312	1,852.5069	903.93378	0
Slice 8	28.73864	-22.703095	1,267.3981	2,128.2859	860.88779	0
Slice	33.73187	-	1,455.0429	2,216.4842	761.44133	0

9		25.635046				
Slice 10	36.925	- 27.108247	1,549.3278	2,260.8936	711.56578	0
Slice 11	41.359125	- 29.154002	1,680.2561	2,395.3063	715.05015	0
Slice 12	43.859125	- 30.349358	1,756.7589	2,447.3369	690.578	0
Slice 13	45.75	-31.42464	1,825.5769	2,532.3801	706.80319	0
Slice 14	49.75	- 33.699314	1,971.1561	2,693.2706	722.1145	0
Slice 15	54.842908	- 36.595492	2,156.5115	2,935.464	778.95252	0
Slice 16	60.028725	- 39.544503	2,345.2482	3,111.7484	766.50021	0
Slice 17	65.214542	- 42.493514	2,533.9849	3,288.0328	754.0479	0
Slice 18	70.11245	-45.23397	2,709.3741	3,463.484	754.10988	0
Slice 19	75.503533	- 46.812029	2,810.3699	3,898.5361	679.96169	0
Slice 20	81.66914	- 47.435585	2,850.2774	3,827.586	610.69019	0
Slice 21	87.828186	- 48.058477	2,890.1425	3,756.7283	541.50293	0
Slice 22	93.987233	- 48.681369	2,930.0076	3,685.8707	472.31568	0
Slice 23	100.14628	-49.30426	2,969.8727	3,615.013	403.12843	0
Slice 24	106.30533	- 49.927152	3,009.7378	3,544.1554	333.94117	0
Slice 25	109.40409	- 50.240544	3,029.7948	3,249.8316	137.49422	0
Slice 26	112.01754	- 50.058267	3,018.1291	3,327.6618	193.4175	0
Slice 27	117.20596	- 49.689822	2,994.5486	3,289.9425	184.58262	0
Slice 28	121.79286	-49.39563	2,975.7203	3,251.6974	172.44961	0
Slice 29	127.55674	-48.99101	2,949.8246	3,223.748	171.16631	0
Slice 30	135.23651	-48.41793	2,913.1475	3,160.1487	154.34349	0
Slice 31	141.61152	- 47.973892	2,884.7291	3,112.7634	142.49165	0
Slice 32	146.54436	- 47.642677	2,863.5314	3,079.545	134.9803	0

Slice 33	149.25567	-47.459945	2,851.8365	3,062.6635	131.73934	0
Slice 34	152.71066	-47.035012	2,824.6408	3,043.2353	136.59303	0
Slice 35	159.13087	-46.219395	2,772.4413	2,945.6526	108.23442	0
Slice 36	165.55107	-45.403779	2,720.2419	2,848.0699	79.875801	0
Slice 37	171.57088	-44.639028	2,671.2978	2,754.1332	51.761291	0
Slice 38	177.19029	-43.925143	2,625.6092	2,674.222	30.376626	0

SLOPE/W Analysis

Seaside

File Information

Created By: Weakland, Robert T POA
Last Edited By: Weakland, Robert T POA
Revision Number: 22
File Version: 8.2
Tool Version: 8.12.3.7901
Date: 11/5/2014
Time: 3:52:27 PM
File Name: SlopeStability 1500 Seaside.gsz
Directory: O:\EN\Private\ES\ES-SG\1 MAIN PROJECT FILES\3 Civil Works\Craig\Craig harbor Navigation Improvements\Project Data\Analysis\
Last Solved Date: 11/5/2014
Last Solved Time: 3:52:30 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 64 pcf
View: 2D
Element Thickness: 1

Analysis Settings

SLOPE/W Analysis

Kind: SLOPE/W
Method: Spencer
Settings
- Lambda
  - Lambda 1: -1
  - Lambda 2: -0.8
  - Lambda 3: -0.6
  - Lambda 4: -0.4
  - Lambda 5: -0.2
  - Lambda 6: 0
  - Lambda 7: 0.2
  - Lambda 8: 0.4
  - Lambda 9: 0.6
  - Lambda 10: 0.8
  - Lambda 11: 1

PWP Conditions Source: Piezometric Line
Apply Phreatic Correction: No
Use Staged Rapid Drawdown: No
Slip Surface
Direction of movement: Right to Left
Use Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: Yes
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Breakwater

Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion': 0 psf
Phi': 45 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Insitu

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion': 0 psf
Phi': 32 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (-283.02867, -48.53744) ft

Left-Zone Right Coordinate: (-170, -48.09834) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (-97.81049, -37.37366) ft
Right-Zone Right Coordinate: (70, -19.83333) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (-300, -48.60337) ft
Right Coordinate: (300, -43.9825) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
Coordinate 1	-300	-2.9
Coordinate 2	300	-2.9

Points

	X (ft)	Y (ft)
Point 1	-300	-48.60337
Point 2	-113.56868	-47.87912
Point 3	-64.12049	-47.45269
Point 4	0	-45.40455
Point 5	44.55991	-45.11672
Point 6	64.12932	-46.59365
Point 7	109.38485	-46.0899
Point 8	168.76117	-44.09303
Point 9	200.88919	-42.59272
Point 10	225.79381	-42.29448
Point 11	300	-43.9825
Point 12	300	-122
Point 13	-300	-122
Point 14	52.25	-8
Point 15	47.25	-8
Point 16	44.25	-6
Point 17	39.25	-6
Point 18	3.25	18
Point 19	-3.25	18

Point 20	-48.25	-12
Point 21	-54.75	-12
Point 22	-61.125	-16.25
Point 23	-66.125	-16.25

Regions

	Material	Points	Area (ft^2)
Region 1	Breakwater	5,6,7,14,15,16,17,18,19,20,21,22,23,2,3,4	7,406.6
Region 2	Insitu	1,13,12,11,10,9,8,7,6,5,4,3,2	45,533

Current Slip Surface

Slip Surface: 126
F of S: 1.706
Volume: 1,885.5001 ft^3
Weight: 205,593.15 lbs
Resisting Moment: 11,494,244 lbs-ft
Activating Moment: 6,737,966.7 lbs-ft
Resisting Force: 69,988.493 lbs
Activating Force: 41,135.487 lbs
F of S Rank: 1
Exit: (-170, -48.098345) ft
Entry: (2.7318496, 18) ft
Radius: 98.899384 ft
Center: (-129.53539, 80.348236) ft

Slip Slices

	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
Slice 1	-167.31163	-48.344215	2,908.4298	2,966.2202	36.111468	0
Slice 2	-161.93488	-48.835956	2,939.9012	3,035.3718	59.656644	0
Slice 3	-156.55814	-49.327697	2,971.3726	3,104.5233	83.20182	0
Slice 4	-151.1814	-49.819438	3,002.8441	3,173.675	106.747	0
Slice 5	-145.80465	-50.311179	3,034.3155	3,242.8266	130.29217	0
Slice 6	-142.32494	-50.60651	3,053.2166	3,268.5467	134.55318	0
Slice 7	-141.47553	-50.65363	3,056.2323	3,222.3558	103.80548	0
Slice 8	-138.70487	-50.50974	3,047.0234	3,198.4797	94.640366	0
Slice	-133.13626	-	3,037.5672	3,204.7022	104.43755	0

9		50.361987				
Slice 10	-127.42425	-50.349582	3,036.7733	3,204.5848	104.86025	0
Slice 11	-121.81835	-50.538615	3,048.8713	3,269.6105	137.93314	0
Slice 12	-116.31857	-50.929084	3,073.8614	3,324.4631	156.59332	0
Slice 13	-113.56649	-51.124475	3,086.3664	3,618.456	332.4865	0
Slice 14	-110.28169	-50.93415	3,074.1856	3,591.3832	323.18092	0
Slice 15	-103.22776	-50.35933	3,037.3971	3,666.1466	392.8863	0
Slice 16	-95.918795	-49.406655	2,976.4259	3,684.1909	442.26068	0
Slice 17	-88.843505	-48.269985	2,903.679	3,713.0664	505.76136	0
Slice 18	-82.070061	-45.840648	2,748.2014	3,305.2315	557.03011	0
Slice 19	-75.656947	-42.152278	2,512.1458	3,083.6001	571.45433	0
Slice 20	-69.302316	-38.497544	2,278.2428	2,863.9898	585.74702	0
Slice 21	-65.55314	-36.341283	2,140.2421	2,669.0268	528.7847	0
Slice 22	-63.05314	-34.954707	2,051.5012	2,552.8984	501.39721	0
Slice 23	-57.9375	-32.148518	1,871.9051	2,407.2847	535.37956	0
Slice 24	-53.50634	-29.717801	1,716.3393	2,203.4034	487.06408	0
Slice 25	-50.25634	-28.116417	1,613.8507	2,091.7325	477.88183	0
Slice 26	-44.8375	-25.633862	1,454.9672	1,962.9941	508.02686	0
Slice 27	-38.0125	-22.507098	1,254.8543	1,801.2771	546.42283	0
Slice 28	-32.84088	-20.137803	1,103.2194	1,743.8178	640.59839	0
Slice 29	-28.242217	-17.30377	921.84128	1,610.8028	688.9615	0
Slice 30	-22.563132	-13.24753	662.24192	1,528.5592	866.31725	0
Slice 31	-16.010559	-7.059705	266.22112	1,116.5777	850.3566	0
Slice 32	-10.477469	-0.86099	-130.49664	871.16249	871.16249	0

Slice 33	-5.953705	5.1714026	-516.56977	595.96293	595.96293	0
Slice 34	-0.25907519	13.582393	-1,054.8731	238.77198	238.77198	0

APPENDIX C
Rock Quarry Trip Report 2014

DEPARTMENT OF THE ARMY
ALASKA DISTRICT, U.S. ARMY CORPS OF ENGINEERS
P.O. BOX 6898
JBER, AK 99506-0898

CEPOA-EN-G-GM

MEMORANDUM FOR THE RECORD

SUBJECT: Trip Report – Site reconnaissance to evaluate potential material sources for the Craig Harbor Navigational Improvements project.

DATE: 2 May 2014

1. Purpose: Between 24 and 25 April 2014, Christine Morgan (Cost Engineering) and Coleman Chalup (Geotechnical and Materials Section) traveled to Craig Alaska to look at the proposed harbor site and determine if any quarries in the area are capable of producing adequate rock for the proposed harbors breakwater. The team met with several local quarries owners during the two days in order to see if existing quarries are available and obtain information.
2. Schedule:
    a. 24 April, 2014
        i. Traveled to Craig, Alaska
        ii. Met with James Carl from Shaan Seet Inc.
    b. 25 April, 2014
        i. Traveled to St. Johns island
        ii. Met with Paul Thibodeau from Southeast Road Builders Inc.
        iii. Traveled to Anchorage, Alaska
3. Discussion
    a. <u>Shaan Seet Quarries –</u> Upon arrival in Craig the team met with James Carl from Shaan Seet Inc. and traveled to three (3) of their quarries near Craig. The first one visited was the Lower 62 Pit (Figure 2). There is an upper 62 pit, however it is unusable. They had stockpiles of rock consisting of 2 to 4 foot diameter stone that were dull when struck with a rock hammer indicating that the stone was elastic and strength was greater than 15,000 psi, had a more open composition than tight and small pieces would break away. The quarry face had veins of highly fractured rock with low fractured rock and could produce a large variety of stone sizes. However James commented that this pit had a hard time meeting some specs but was unaware of which ones they were. There was also a barge landing across the main road from the quarry that could be used during construction activities. If this quarry was to be used testing would have to be used to determine if the stone would be adequate which could make this quarry viable. The other two quarries that we traveled to were the 4.5 mile (Figure 3) and 5 mile quarries. These two quarries would be unable to produce B or A rock and were typically used for crushing the stone into gravel for road construction. The 5 mile quarry also had a locked gate and we were not able to see the actual quarry. James also said that they would have to improve the road before this quarry would be usable. We were also told about another quarry at 8-9 miles down the road called Wolf Lake that could produce A and B material but due to the distance and rough road it did not seem like a viable quarry. Another quarry on St. Johns Island (about 4 miles away to the southeast from the proposed harbor site) is planned on being developed and also had an existing quarry. He arranged for a boat ride the next morning for the team to investigate. The

St. Johns quarry (Figure 4) had some stockpiled material from the last project (15 to 20 years ago) that used the site for a Canadian breakwater. This site had stone from 2 to 5 feet in diameter stockpiled and the quarry face looked like it could produce similar sized stone. The stone made a sharp tone when struck with a rock hammer indicating that the stone was elastic and strength was greater than 15,000 psi, had a tight composition and did not break apart easily. This stone appeared to be competent and be a viable quarry for the breakwater project.

b. Southeast Road Builders – The team met with Paul Thibodeau from Southeast Road builders quarry (Figure 5) just outside of Klawock, Alaska. This quarry had all sizes of stone that ranged from 3 inch minus to A on site. The stone had dimensions of +4 feet (A), 1-2 feet (B), and 3 inch minus (Small Core). The rock made a sharp tone when struck with a rock hammer that indicated it was elastic and strength was greater than 15,000 psi, had a tight composition and did not break apart easily when struck. Paul also said that for his company to transport their material they would truck it to the site and stock pile the material on site since they do not have a barge landing adjacent to the quarry. He also indicated that they have passed DOT and highway specs that include LA Abrasion, Degradation, Fractures, and Sodium Sulfate tests that have been performed in the past. This quarry appears to be a viable option to produce materials for the breakwater although it would have to still undergo more testing. Paul also took us to the Black Rock Quarry (Figure 6) where they could produce B and Core material. The majority of the material produced at this site was core, due to the highly fractured faces of the quarry and occasionally made B rock. This quarry would only be viable for B and Core material and would not produce any A size material which would not make it a viable quarry unless it was needed for strictly core material.

c. Harbor location – The team walked the proposed harbor (Figure 7) location from wharfs, land, and also made a stop along the outside of the wharfs when returning from St. Johns quarry by boat. Below the water surface and on the beach cobbles and various boulders were noted that may cause issues while dredging although dredging may not be necessary for this project. During construction, and the demolition phase, it will be necessary to remove all of the old piling from the existing and past wharfs. Some of the wharfs are still intact and others are long gone but the creosote piling remains. There is also evidence of metal debris and trash along the bottom that will have to be removed during demolition but the extent and content is unknown at this time. It is recommended that the use of an underwater camera be used to get an understanding of the extent of the debris along the bottom. There are also several open places within 500 feet and directly upland of the proposed harbor that could be used to stock pile material during construction.

d. Representative photographs can be found in the attached figures and are also located at:

O:\EN\Private\ES\ES-SG\1 MAIN PROJECT FILES\3 Civil Works\Craig harbor Navigation Improvements\Photos\Craig site visit 4-25-14

Coleman Chalup, P.E.
Civil Engineer, CEPOA-EN-G-GM

Attachments:
Figure 1: Craig Navigational Improvement site visit location map
Figure 2: Shaan Seet Quarry – Lower 62
Figure 3: Shaan Seet Quarry – 4.5 mile
Figure 4: St. John's Quarry
Figure 5: Southeast Road Builders Quarry – 7 mile
Figure 6: Southeast Road Builders Quarry – Black Rock Quarry
Figure 7: Proposed Harbor

References:

- ASTM International. (2005). Standard Guides for Using Rock-Mass Classification Systems for Engineering Purposes. In *ASTM D 5878 - 05*. West Conshohocken, PA: Author.
- US Army Corps of Engineers. (1985). Geotechnical Descriptions of Rock and Rock Masses. In *Technical Report GL-85-3*. Washington, DC: Department of the Army USACE.

US Army Corps of Engineers Alaska District	**Craig Navigational Improvement Site Visit** **Craig, AK**	**Figure 1** **2 May 2014**

Quarry Face

Core material

A material

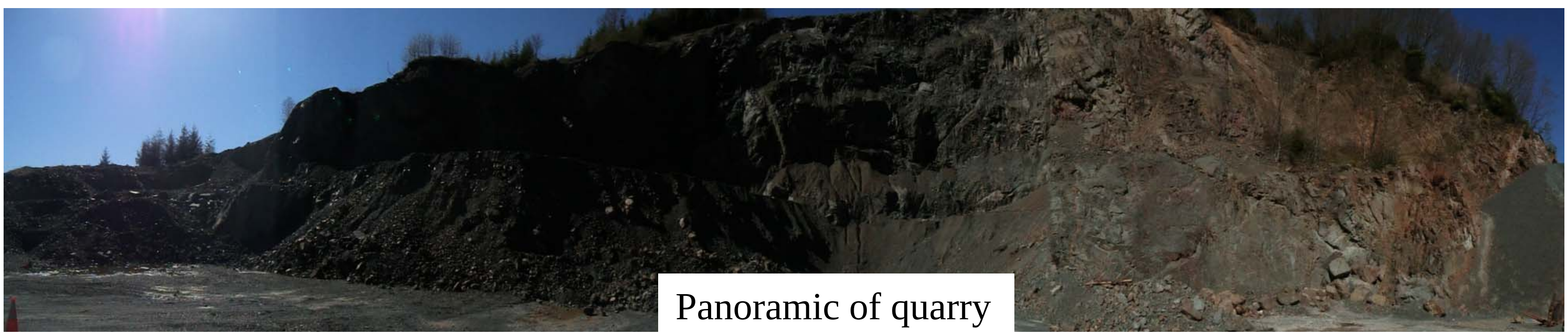
Barge Landing

Panoramic of quarry

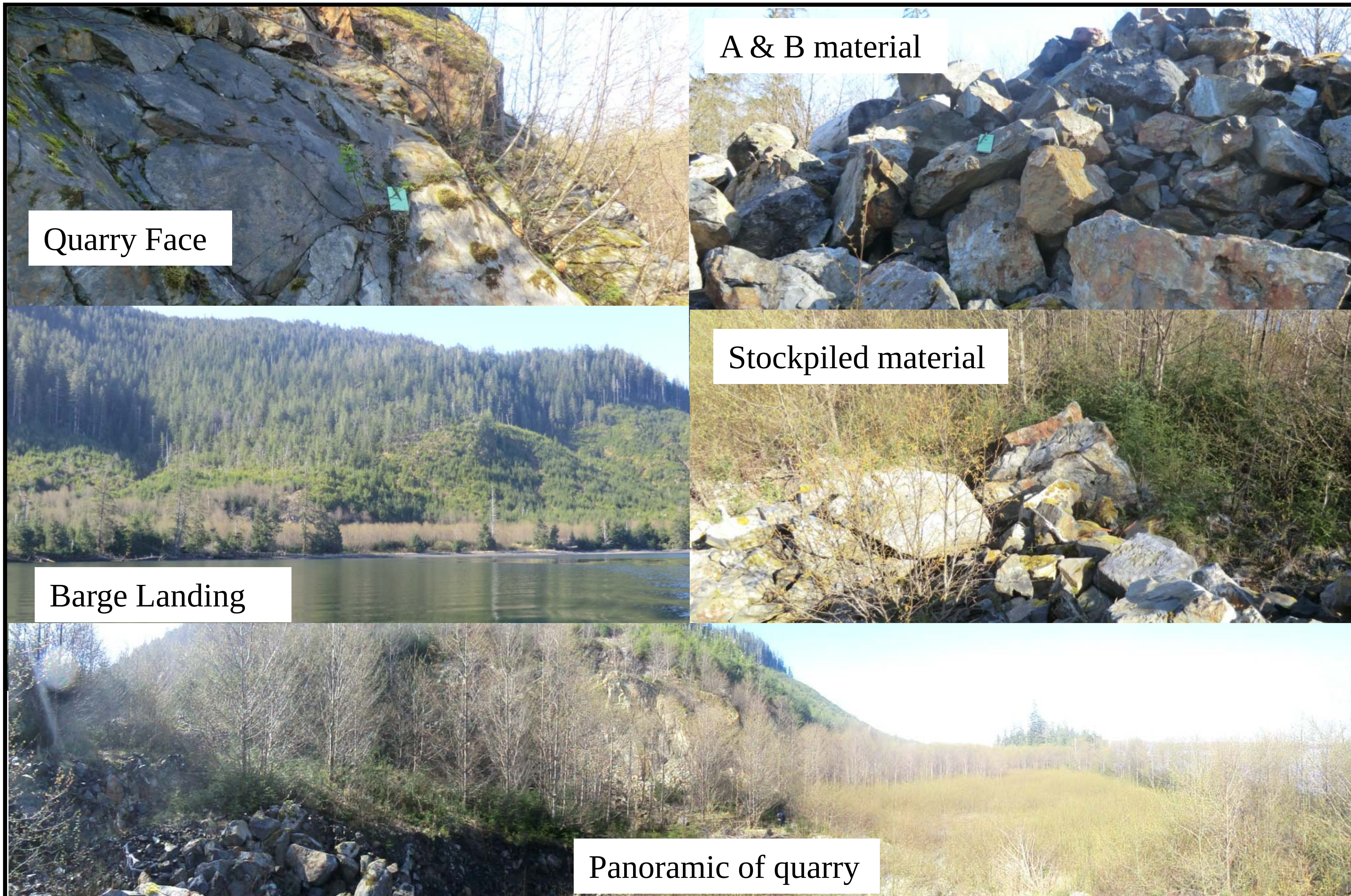
US Army Corps of Engineers **Alaska District**	**Craig Navigational Improvement Site Visit** **Shaan Seet Quarry – Lower 62** **Craig, AK**	**Figure 2** **2 May 2014**


Quarry Face

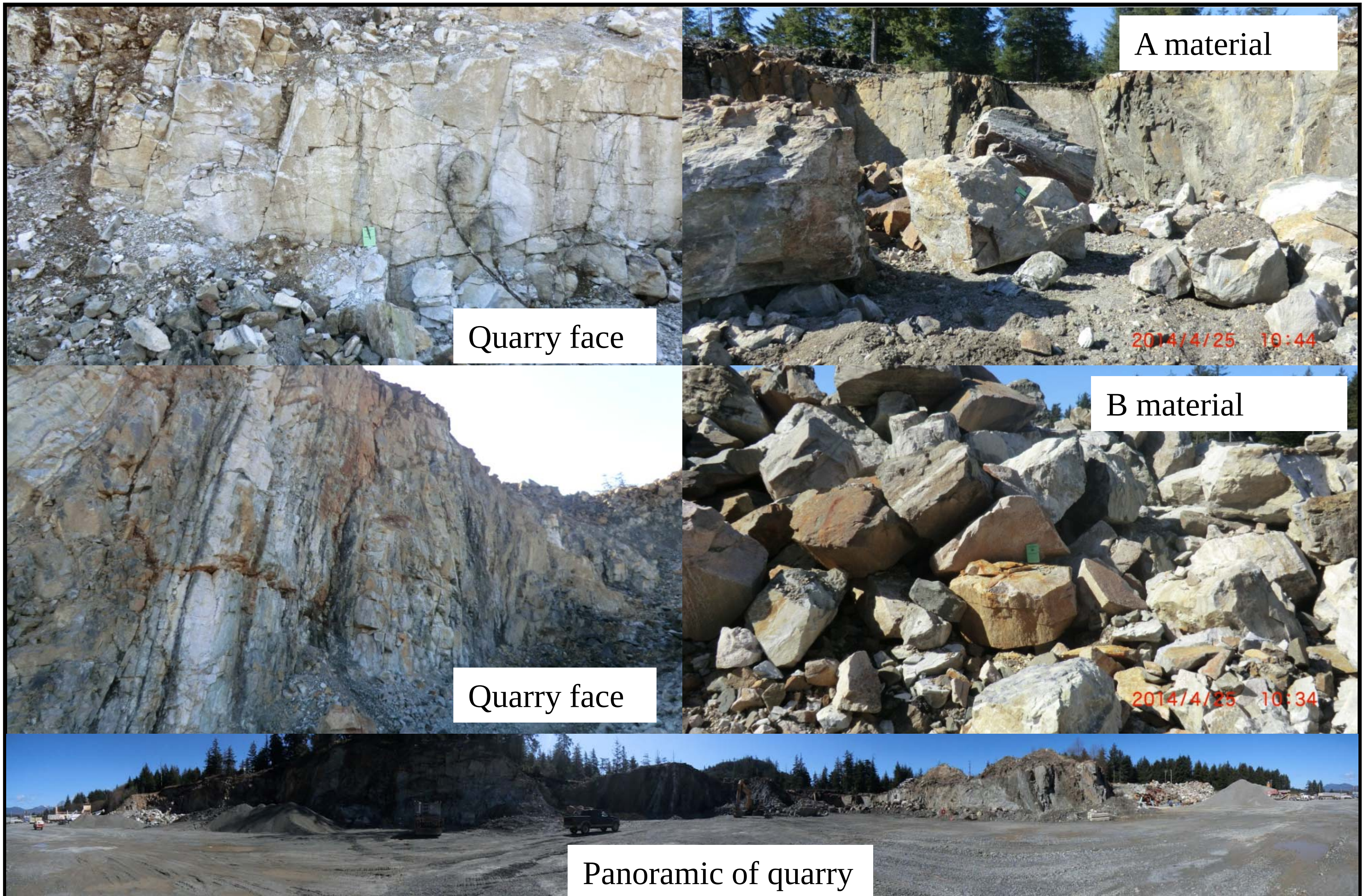

Material before crushing


Panoramic of quarry

US Army Corps of Engineers **Alaska District**	**Craig Navigational Improvement Site Visit** **Shaan Seet Quarry – 4.5 Mile** **Craig, AK**	**Figure 3** **2 May 2014**



US Army Corps of Engineers **Alaska District**	**Craig Navigational Improvement Site Visit** **St. John's Quarry** **Craig, AK**	**Figure 4** **2 May 2014**



US Army Corps
of Engineers
Alaska District

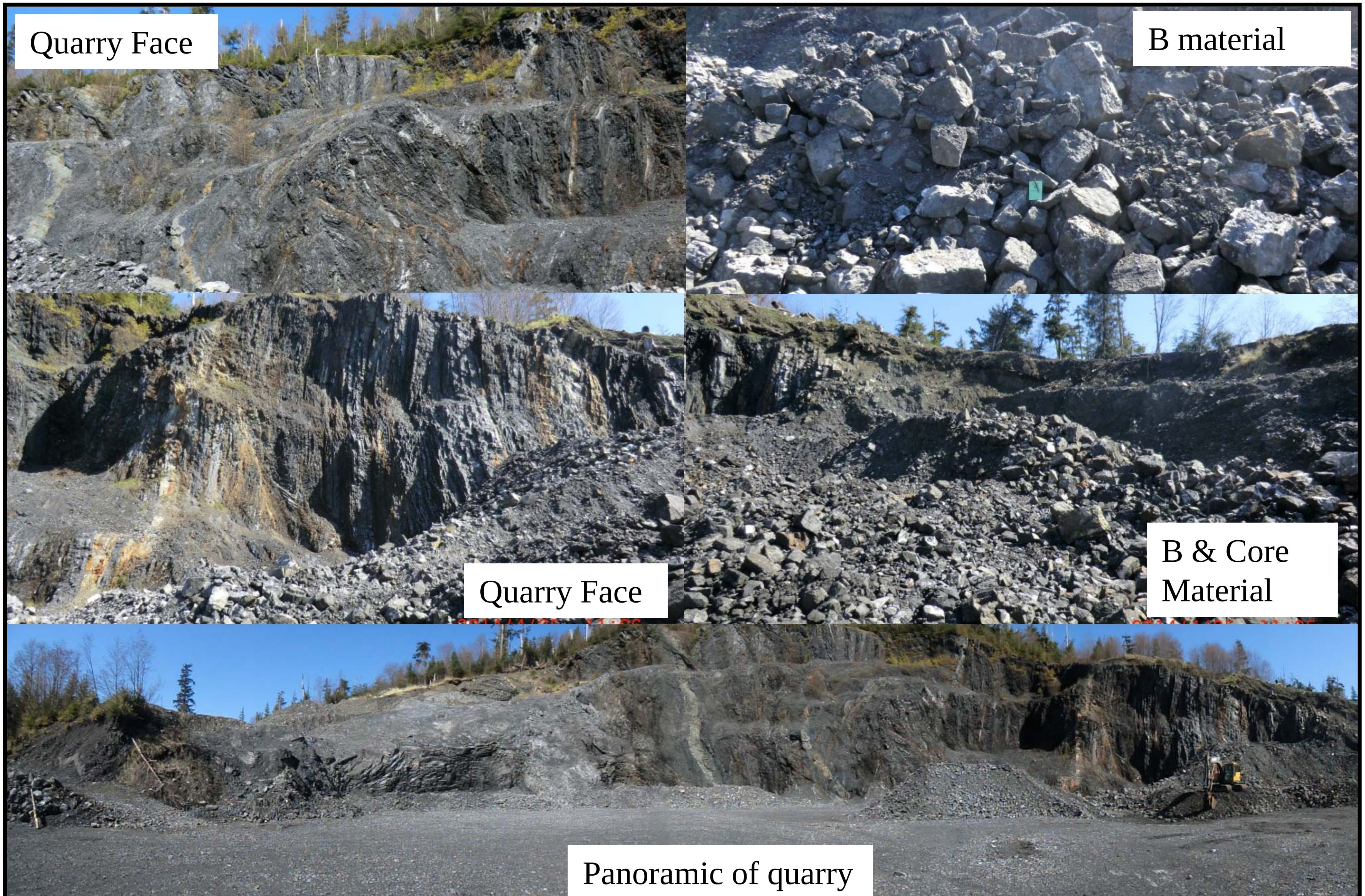
Craig Navigational Improvement Site Visit

Southeast Road Builders Quarry – 7 Mile

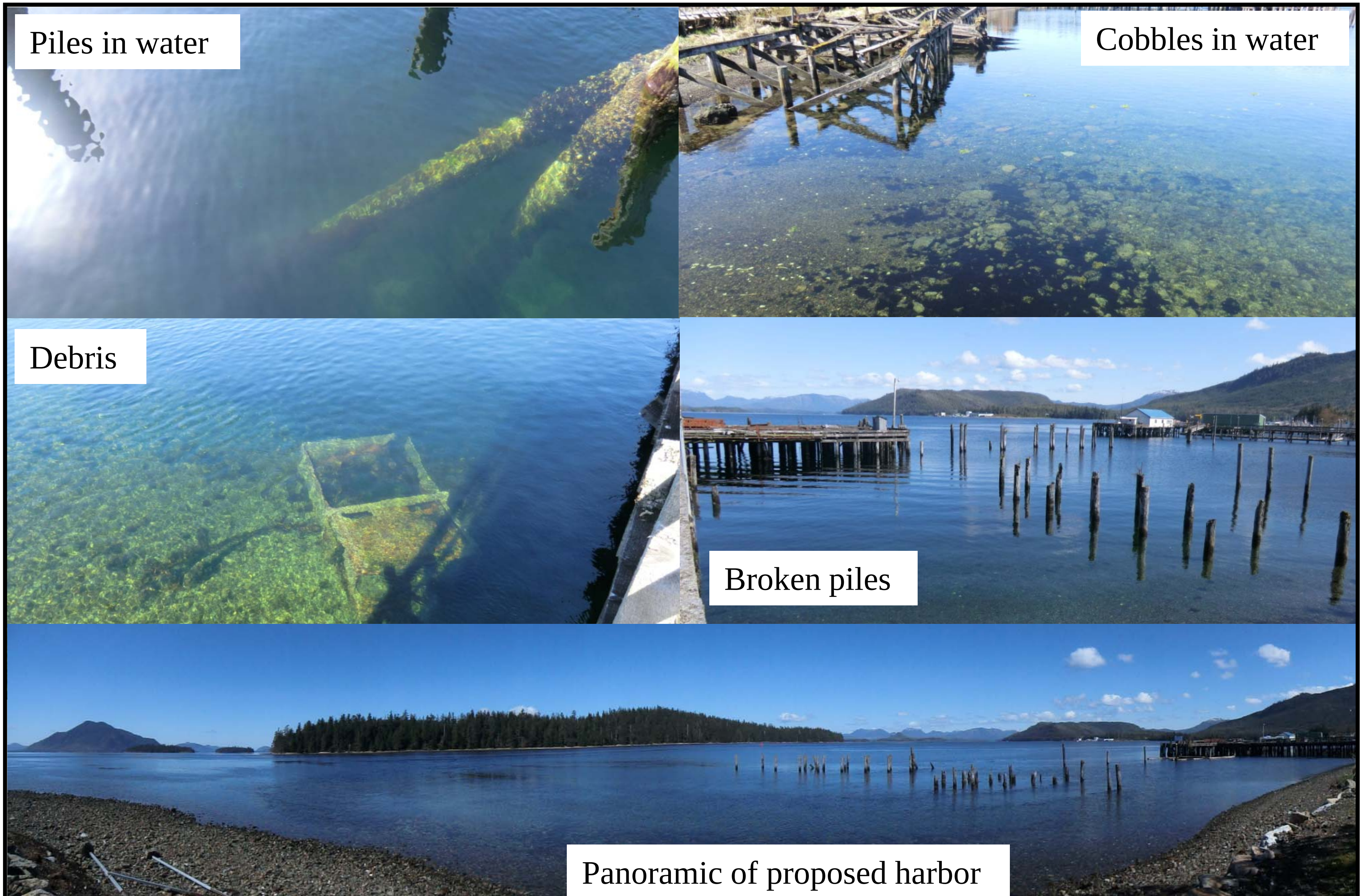
Craig, AK

Figure 5

2 May 2014



US Army Corps of Engineers Alaska District	**Craig Navigational Improvement Site Visit** **Southeast Road Builders Quarry – Black Rock Quarry** **Craig, AK**	**Figure 6** **2 May 2014**



US Army Corps of Engineers

Alaska District

Craig Navigational Improvement Site Visit

Proposed harbor

Craig, AK

Figure 7

2 May 2014