Kipnuk Engineering Analysis and Design Study

Prepared for:

Kipnuk Traditional Council

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1.0 EXECUTIVE SUMMARY

1.1 Background Summary

Kipnuk is located on the Kuguklik River, four miles from the Bering Sea, approximately 100 miles west of Bethel. Severe erosion of the Kuguklik River bank is cutting into the town site and threatening buildings. The community is subject to storm surges coming off the Bering Sea. Permafrost thaw and/or poor soils contribute to widespread ground settlement in the community. In 2011 Golder Associates conducted a Hazard Impact Assessment (HIA) as part of the Alaska Climate Change Impact Mitigation Program (ACCIMP), administered by the Alaska Department of Commerce, Community, and Economic Development (DCCED). The following summary from the HIA describes the conditions found in Kipnuk:

"Threats to the community were identified such as destructive seasonal and storm-related flooding, riverbank erosion, and ground settlement due to thawing of permafrost. During flooding, school access is sometimes restricted, sewage lagoons and the landfill are sometimes overtopped. The entire community is subject to flooding during severe events. Riverbank erosion rates are estimated from 6 to 9 feet per year, though localized erosion rates vary. Residents reported between 15 and 20 feet of riverbank loss near the fuel transfer facility during the summer of 2009. Kipnuk is underlain by marginally frozen, thaw- unstable ground that is subject to thaw-settlement that results in ground surface subsidence, increasing the degree of flooding and potential loss of foundation support for public and private buildings."

1.2 Purpose of Study

A Hazard Mitigation Plan (HMP) was prepared by the Village of Kipnuk hazard mitigation planning team in 2013. The goal of the Kipnuk Engineering Study was to determine the most suitable combination of solutions to address hazard threats identified in the HMP:

- River Bank Erosion
- Flooding
- Ground Settlement

1.3 Conclusions

Bank stability alternatives are discussed in section 9. Bank stability can be achieved with installation of a sheetpile seawall and/or riprap revetment. Riprap revetment was found to be a more cost effective solutions and is recommended as the bank stability alternative moving forward. If no funding is available for the riprap alternative, and relocation alternative is presented in Section 10.

AECOM estimated storm surge elevations for the 100 year storm surge flood in Section 7. Recommendations for establishing minimum floor elevations above estimated flood levels for future construction are included in Section 7. Recommendations for ground settlement problems are included in Section 8.

2.0 BACKGROUND

The Hazard Mitigation Plan (HMP) prepared by the Village of Kipnuk hazard mitigation planning team identified hazard threats as 1) river bank erosion, 2) flooding, and 3)ground settlement .

Erosion. The HMP considered the severity of impacts of erosion as 'Critical' (potential for critical facilities to be shut down for at least two weeks, and more than 25 percent of property or critical infrastructure being severely damaged). The community is located on the outside bend of the river. The U.S. Army Corps of Engineers (USACE) Kipnuk Erosion Assessment Report completed in 2009 estimated loss of 9 feet of bank per year near the Kugkaktlik Limited (Corp) bulk fuel tank.

The Kuguklik River at Kipnuk is characterized by large discharge rates, fast water velocities, bi-directional tidal flow, and near-bank turbulence. These factors, in combination with fine-grained easily erodible bank soils, contribute to the severe erosion rates. Waves are sometimes generated during fall storms and crash onto the bank, resulting in even more bank erosion.

Flooding. The elevation of Kipnuk is only a few feet above sea level. Flooding results primarily from coastal storm surges during fall storms. The Community is located four miles upstream from the Bering Sea. The HMP considered the severity of impacts of flooding as 'Negligible' (less than 10 percent of property or critical infrastructure being severely damaged). Flooding of roads, homes, boardwalks, or damage to utility infrastructure remains a concern. For this study AECOM provided an estimated elevation of the 100 year flood and provided recommendations for establishing minimum floor elevations for future construction.

Ground Settlement. Settlement can result from soft soils or permafrost thaw. Kipnuk is located in a zone of discontinuous permafrost. Permafrost in this region is relatively warm and can quickly begin to thaw with a slight rise of temperature. Sources of heat include: removal of insulating vegetation layers (tundra); buildings; gravel surfaces on roads, airfields, and boardwalks (radiate summer heat). Silty soils retain water. Settlement of the high ice-content silty soils can occur due to drainage of excess pore water when the soils thaw. The HMP considered the severity of impacts of ground settlement as 'Limited' (more than 10 percent of property or critical infrastructure being severely damaged). While thaw settlement is not life threatening, it can be detrimental to buildings and boardwalks.

3.0 STUDY SCOPE

The goal of the study was to determine the most suitable combination of solutions to mitigate the three hazard threats listed above. The study consisted of the following tasks:

- Field Reconnaissance and Site Survey
- Hydrology and Hydraulic Analysis
- Erosion Analysis

- Flood Analysis
- Ground Settlement Analysis
- Embankment Stability Alternatives

AECOM sub-contracted to DOWL for field and river surveying and Hydraulic Mapping and Modeling (HMM) for hydrology and hydraulic analysis.

Figure 1: Kipnuk 2010

4.0 SITE SURVEY

4.1 Site Description

The study team traveled to Kipnuk in May 2015 for the site survey and field reconnaissance investigation. The site visit coincided with a public meeting and an introductory meeting with the Kipnuk Traditional Council. The team included an AECOM community outreach specialist, AECOM engineers (civil and geotechnical), a hydraulic engineer (sub-consultant Hydraulic Mapping and Modeling), and a surveyor (sub-consultant DOWL).

The site survey included top of bank survey, a hydrographic survey (seven cross sections and a thalweg profile), and a survey of high-water flood elevation marks. High water marks were identified by elders and also by historic flood photographs. The field survey was performed by DOWL from May 26th through May 28th, 2015. Static Global Satellite (GNSS) observations were taken on the primary control station at the Airport, based on Alaska Department of Transportation and Public Facility (ADOT&PF) survey control. All other point locations were determined by Real Time Kinetic (RTK) GNSS. For the hydrographic survey the Kipnuk Traditional Council provided a skiff on which a Leica GNSS RTK receiver was mounted and configured to collect coincident data with an ODOM Hydrotrac Fathometer. The resulting data has an anticipated accuracy of +/- 0.2 feet. A full description of the survey and methods is included in Appendix H.

Local Datums. The 2015 DOWL survey tied into a vertical datum developed by ADOT&PF. The datum was derived by GPS by adding 1000 feet to the NAVD88 Geoid 99 height. The 1000 foot elevation in the ADOT&PF datum corresponds to approximately 4.5 feet in the datum and contours developed for the Kipnuk Community Map in 2004. The elevation difference was estimated through common points in AutoCAD drawings. AECOM did not identify survey information that linked the two datums.

5.0 HYDROLOGY AND HYDRAULIC ANALYSIS

Figure 2: Kuguklik River, Reach 2

The Kuguklik River is a meandering stream that originates about 30 miles east of Kipnuk in an area of flat tundra and lakes. Kipnuk is located on the outer bank of an actively eroding bend of the river.

The area around Kipnuk is flat and poorly drained with numerous lakes and small drainages that flow into the Kuguklik River (ADCED, 2015). The channel is tidally influenced. On the rising (flood) tide, flow comes up the Kuguklik River and flows up the channel adjacent to Kipnuk. Following high tide, the ebb tide flows out the tidal channel to the Kiniak Bay and into the Bering Sea.

Using the channel cross-sections obtained in May 2015 as part of the hydrographic survey, the HMM team built a hydraulic model of the river to estimate the water velocity and stress imposed upon the riverbank soils. Field observations of two-directional flow at the site, along with the HEC-RAS analysis, indicate that the majority of the discharge in the tidal channel is from upstream high-tide storage, not by precipitation-generated flow from the upper watershed. Design flood elevations for the community should therefore be based on tidal surge floods estimates rather than river discharge volumes. A full description of the hydrology and hydraulic analysis is included in Appendix A.

6.0 EROSION ANALYSIS

Figure 3: Kuguklik River, Bank Erosion

Because the most erosive discharge in the tidal channel is from upstream high-tide storage, velocities based on tidal influenced hydraulics were selected as the basis of erosion analysis (see Appendix A). According to geotechnical boring logs, natural soils in the area are composed of fine silts, which can easily erode even with mild currents. Scour undermines the bank, which leads to sloughing of the bank into the river, see figure 3. Without erosion protection the river current and wave action during storms will continue to erode the fine grained soils in the river bank. The estimated height of storm generated waves is about one meter (3 feet). Village elders report that waves typically do not travel past the bank during storm surges.

Based on previous surveys and aerial photographs, the historic and predicted changes in bank location due to river erosion are shown in Figure 4. The projected 2065 bank location assumes the measured erosion rates from 1963 to 2015 will continue for the next 50 years. The estimated 2065 bank is similar to the 2057 estimate by the U.S. Army Corps of Engineers (USACE) in the 2009 Community Erosion Assessment.

Figure 4: Kuguklik River, Past, Present, and Future

6.1 Scour Depths

Bank erosion at the outside bend of the Kuguklik River is exacerbated by significant scour occurring in the bottom of the channel bed. In meandering natural rivers, flow in channel bends can generate secondary flow patterns, which can lead to increased velocities, shear stresses, and scouring. Channel bed scour in this reach is large. The cross-section survey conducted in May 2015 included multiple shots of the channel bed thalweg (deepest point in the river bed). The deepest scour hole was found to be 60 feet deep, located near the bulk fuel unloading facility. Figure 5 shows the 2015 scour depths and thalweg alignment. A profile of the river channel is shown in Figure 7 in Appendix A. A contour map of the scour hole is shown in Figure 8 (also in Appendix A).

As noted in the Hydraulics and Hydrology report (Karle, 2016), currents produced by ship propellers have also been associated with bank and channel erosion. One report describing the erosion at Kipnuk noted that "local erosion may also occur at the barge landing and bulk fuel terminal due to prop wash if the barge keeps its screws turning while unloading" (USACE, 2009). However, other than the proximity of the bulk fuel terminal to the surveyed scour hole, there is no conclusive evidence to determine if prop wash, channel geometry, or a combination of these and other factors are responsible for the large scour hole.

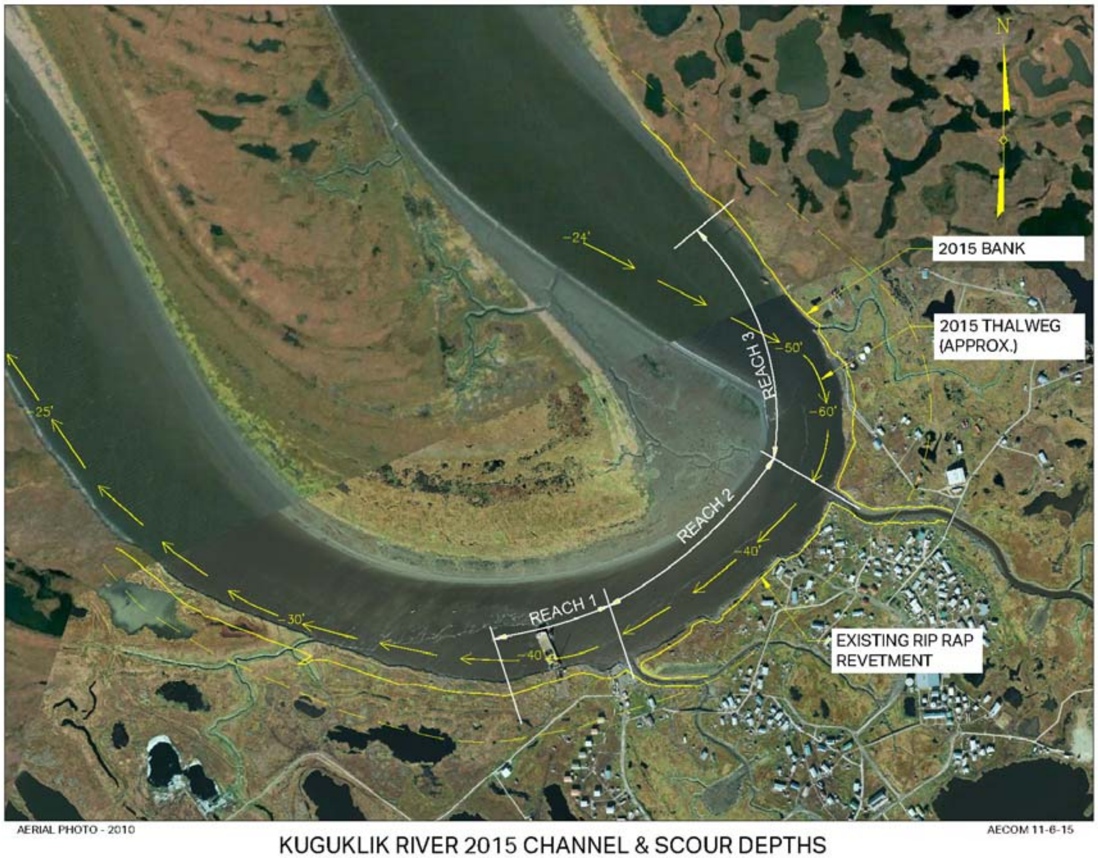


Figure 5: Kugkuklik River & Scour Depths in 2015

6.2 Erosion Analysis

Geotechnical reports describe the underlying soils in Kipnuk as fine grained soils. Bore holes drilled for the Kipnuk Bulk Fuel and Powerplant Facility (Duane Miller Associates, 2007) show silt down to a depth of 40 feet, with fine grained sand below to a depth of at least 70 feet (bottom of bore holes). No permafrost was found in these boreholes. The soils along the river edge are likely composed of similar soils with no permafrost to bind the fine particles. These soils are easily eroded even with mild river currents.

Existing Class II riprap along reach 2 (see figure 5) was installed over 25 years ago. The rock protection has mostly disappeared above the water and only the fabric remains, see figure 6. Much of the rock has either been carried away by ice or dislodged by erosion and wave action. The condition of the rock below the water is unknown, though the revetment appears to have kept the bank erosion in check for many years.

Figure 6: Existing Riprap Revetment Reach 2

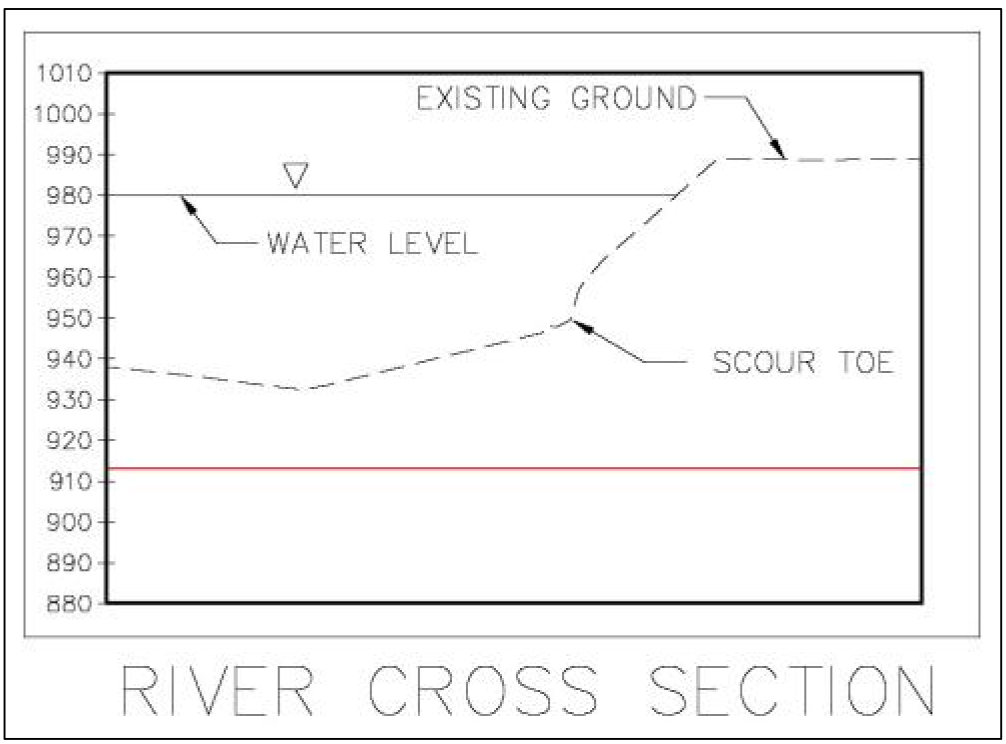


Figure 7: Kuguklik River, Existing Cross Section at Reach 3 Looking Upstream.

Figure 8: Wave Run-Up During October 2005 Storm

7.0 FLOOD ANALYSIS

Flooding results primarily from coastal storm surges during fall storms. Storm surges are temporary increases in sea level that accompany storms in shallow coastal waters. Because the elevation of Kipnuk is only a few feet above sea level, the impact from moderate storm surges can be significant, resulting in inundation.

The ideal approach for estimating flood elevations should be based on modeling validated with anecdotal evidence and testimony from local elders. Detailed modeling to predict storm surge heights in the Kipnuk region is not possible at the present time due to insufficient recorded tidal data. As of 2015 there remains a large data gap along the west coast of Alaska related to weather, tides and historic storm surge heights. The nearest tidal gage station is located at Toksook Bay, 55 miles north of Kipnuk. Two recent studies mapped storm surge inundation in the Yukon-Kuskokwim delta area (Storm-Surge Flooding on the Yukon-Kuskokwim Delta, Alaska, Terenzi et al, 2014 and Modeling Storm-Induced Inundation on the Yukon Kuskokwim Delta for Present and Future Climates, Ravens and Allen, University of Alaska, Anchorage, 2013). Unfortunately at this time the southern-most extent of these models end at the community of Newtok 75 miles north of Kipnuk. The dynamics of a storm surge are controlled by local variations in the ocean bathymetry and coastline geometry. The timing of astronomical high and low tides can increase or lessen the effects of storm surges, especially where the tidal range is high (Wise et al., 1981). In addition, surge heights generally decrease as the storm surges move inland. Therefore, it is difficult to correlate surge height elevations at the mouth of the Kuguklik River with surge height elevations at Kipnuk without direct measurement at both locations.

Due to the data gaps noted above, flood elevation estimates for this report were based on anecdotal evidence and testimony from local elders. During the May 2015 site visit AECOM interviewed elders and identified high water marks from the October 2005 storm surge. The 2005 event resulted in one of the largest storm surges in recent memory (see figures 8 and 9). The 2015 DOWL survey tied these locations into the vertical datum developed by ADOT&PF. The high water mark elevations average 990 feet in the ADOT&PF datum; this elevation corresponds to approximately -5.5 feet in the datum developed by Aero-Metric in 2004 for the Kipnuk Community Map.

The following excerpt was downloaded from the National Oceanic and Atmospheric Administration (NOAA) website for October 17, 2005:

"An intense Bering Sea storm produced west wind gusting to 90 mph across the Pribilof Islands late Sunday night into the early morning hours Monday. The storm bottomed out at 962 MB as it moved northeast into the northern Bering Sea. The combination of the strong wind and long fetch produced a surge that coincided with high tides. Flooding occurred in the Bristol Bay area north to Kipnuk along the Kuskokwim Delta."

For this study we used the October 2005 storm as a baseline to estimate the 100 year storm surge flood elevation, and assumed the 2005 event was at least a 20 year storm. The U.S. Army Corps of Engineers conducted a storm-induced water level prediction study for the western coast of Alaska (Chapman et al, 2009). The study developed frequency-of-occurrence relationships of storm-generated water levels for 17 selected communities along Kotzebue and Norton Sounds, the Bering Sea, and Bristol Bay, including Kongiganak (40 miles east of Kipnuk), and Toksook Bay (55 miles north). The stage-frequency analysis for these communities is shown in the Hydrologic and Hydraulic Report in Appendix A, Table 4.

For this study we averaged the surge levels for Toksook Bay and Kongiganak. The surge level for a 100-year storm in the community of Kongiganak is roughly 3.5 feet higher than a 20-year storm. The surge level for a 100-year storm in the community of Toksook Bay is roughly 2.5 feet higher than a 20-year storm. Note that Kongiganak is located near the head of the Kuskokwim Bay where tidal fluctuations are about 12 feet, compared with 7 feet at Kipnuk (Brower et al, 1988). Therefore, based on tidal range differences and ignoring other local factors, the 100 year storm surge level for Kipnuk region should be somewhat less than that for Kongiganak. The average between Kongiganak and Toksook Bay would place the 100 year storm surge 3 feet higher than 2005 20 year storm surge, or approximately 993 feet above the ADOT NAVD88 datum. There may be opportunities to refine this estimate over time as additional data becomes available.

Note that existing ground elevations in the Kipnuk average between 990 feet and 992 feet. Most of the ground in the community would be inundated by a 100 year storm surge. The containment dike elevation for a recently constructed sewage lagoon is 998 feet, which is well above the estimated 100-year flood level. The old school sewage lagoon that was flooded in 2005 has since been de-commissioned and capped. Although the estimated height of storm generated waves is about 3 feet (see Section 6), community elders report that waves have not typically traveled beyond the river bank during past storm surges. The deeper flood water during a 100 year event could allow wave action beyond the river banks and would be a concern with low sitting buildings.

Most of the public buildings have floor elevations several feet above the surrounding tundra. Floor elevations of many private homes are only a couple feet above the ground surface. We recommend floor elevations of new or re-located structures to be placed 3 feet above the estimated base flood elevation, which would be 996.0 feet (ADOT&PF datum). This analysis does not account for potential future sea level rise due to climate change. Currently there are two elevation datums: the ADOT&PF datum and the datum developed for the 2004 Community Map. A single local datum should be adopted to standardize flood elevations for existing structures and future construction. As the 2004 datum includes negative elevations throughout the community, the ADOT&PF datum is recommend. Community benchmarks could be established (at least 2) on pile caps at the new school, as these piling were driven 70 feet deep and are likely the most stable objects in the community.

Figure 9: Storm Surge October 2005

Recommendations:

- Install a tidal gage in the Kipnuk community to provide reliable long-term data for monitoring tide levels, measuring storm surge heights, and estimating future sea level rise.
- Establish minimum floor elevations for new construction
- Establish a community datum based on the ADOT datum elevations; establish two community benchmarks on pilings of the new school addition

8.0 GROUND SETTLEMENT

The thawing of permafrost, leading to ground settlement and subsidence under structures, was identified as a threat in the Kipnuk Hazard Impact Assessment (Golder 2011). This section addresses the causes of ground settlement and recommendations to help reduce future ground settlement.

8.1 Field Reconnaissance

During the May 2015 site visit AECOM engineers interviewed residents, inspected building foundations, house foundations and general site conditions. Conversations with local residents indicated that ground settlement is a concern although it is a secondary concern compared with the river erosion. The site condition inspection report can be found in Appendix F.

8.2 Previous Geotechnical Explorations

Previous geotechnical studies were reviewed to determine the underlying geologic conditions at Kipnuk. These studies included:

- Boardwalk Improvements, Phase II (Golder, 2011a); total of 43 test holes, 7 performed close to the river bank, utilizing a hand auger to advance through active zone generally to the top of permafrost

- Kipnuk Bulk Fuel and Powerplant Facility (Duane Miller Associates, 2007); borings were located approximately 700 feet to the northeast of the riverbank and drilled to depths of 48 to 67 feet; Split Barrel and Shelby Tube samplers were used to collect soil samples for laboratory testing

- Chief Paul Memorial School Expansion (Golder, 2011); borings were located at the school, approximately 900 feet to the southeast of the riverbank, and drilled to depths of 21 to 70 feet; direct push sampling equipment was used to collect disturbed samples of the non-organic silt and fine sand materials for laboratory testing.

8.3 Subsurface Soils

The subsurface soils were generalized into the following layers based on the available information from previous boring logs. The upper 5 feet is considered to be an active zone, with intermittent frozen soil below this elevation. The soil descriptions for each generalized layer are summarized below.

- 0 to 1.5 feet depth: Loose, wet, dark brown, peak, (Golder, 2011a)

- 1.5 to 5 feet depth: Loose, wet, brown silt, visible ice lenses (1-2mm), (Golder 2011a; Duane Miller Associates, 1999).

- 5 to 45 feet depth: wet, gray to black, medium stiff to stiff, sandy silt. Some fine grained sand, with ice lenses. (Golder, 2011b; Duane Miller Associates, 2007).

- 45 to 67 feet depth: gray, wet, medium dense to very dense, silty sand, some silt, trace organic material (Golder, 2011b; Duane Miller Associates, 2007).

Soil bearing capacity of the top layers of soil is low. Based on field observations well drained soils are quite firm under the foot while saturated soils provide little bearing capacity. An at-grade timber footing pad placed directly over the vegetative mat in a well-drained area could support up to 1000 pounds per square foot, but with some long term settlement expected. (Golder 2011a).

8.4 Permafrost

Kipnuk is located in a zone of discontinuous permafrost. Permafrost in this region is relatively warm, and can quickly begin to thaw with a slight rise of temperature, leading to thaw settlement. Climate trends indicate air temperatures in Western Alaska are rising and could lead to permafrost thaw (Kipnuk Hazard Impact Assessment, Golder 2011). Buildings, on-grade foundation pads, boardwalks, and standing water can also cause heat transfer into underlying soils.

Permafrost conditions vary widely across the community according to previous geotechnical reports. No permafrost was present in boreholes drilled in the vicinity of the Bulk Fuel and Powerplant facility. Discontinuous permafrost was present to depths of 20 feet in several boreholes drilled near the airport and school. The reports show that areas near bodies of water tend to be permafrost free. Much of the permafrost found was considered ‘warm’ and marginally frozen, with measured temperatures close to 32°F.

Figure10: Footing on Undrained Soil

Figure11: Boardwalk Settlement, Low Wet Area

It is difficult to determine if settlement of foundations is the result of permafrost thaw or soft soils without a site specific permafrost investigation. Non-insulating materials placed directly on the ground surface can help transfer heat to permafrost, leading to permafrost thaw. These materials can include wood or gravel. A layer of foam board insulation placed under wood bearing pads could reduce heat transfer to underlying soils and help keep the permafrost in a frozen state. Standing surface water can also warm underlying soils and induce permafrost thaw. Foundation settlement in many cases appears to be the result of undersized footings placed on top of soft soils. Recommendations

Factors the Kipnuk community can control to help reduce permafrost degradation include reduction of heat transfer from buildings and removal of standing water. Ideally, buildings should be founded on pile foundations. The recommended embedment of piling in Kipnuk to resist frost uplift is 38 feet (Golder 2011a). Buildings should be elevated a minimum of 24 inches above the ground to allow cooling from natural convection (McFadden, 2000). Space under buildings should be kept clear for ventilation. Standing water under buildings should be drained wherever possible. The insulating tundra mat should be preserved.

Specific recommendations:

- Develop a community wide drainage program to help drain ponded water. This would help drain saturated soils, improve soil bearing capacity, and could reduce permafrost degradation. Standing water absorbs heat during warm summer days; the heat is absorbed by underlying soils which can lead to permafrost degradation. A community drainage plan would need to be permitted with the U.S. Army Corps of Engineers (see ‘Permitting’ section in this report).

- New or relocated buildings bearing on foundation pads should be placed on well drained soils. Ideally, building sites should be located on high ground, perched above lower wet areas. Increase the size of bearing pads that need to be located over saturated soils.

- Buildings not built on piling should be founded on triodetic frames, or similar, with adjustable legs to facilitate easy re-leveling and establish even load distribution into the floor structure.

- A layer of foam board should be placed under the bottom-most timber layers where buildings are founded on timber foundation pads. This will provide a thermal break which will reduce heat transfer from the surface to underlying soils.

- Consider an installation of a subsidence monitoring system in the community

A community drainage program could identify areas between buildings where surface water accumulation is excessive and causing problems. Constructed drainage swales would need to include measures to remove and replace the top two feet organics (peat) to preserve the insulating properties of the tundra. This peat layer provides an insulating layer to help protect the underlying permafrost from summer heat. Peat has a higher thermal conductivity when frozen than when thawed. (McFadden and Bennet). This allows more heat to escape in the winter than is gained in the summer. The sequence plan below shows how this could be accomplished (Figure 12).

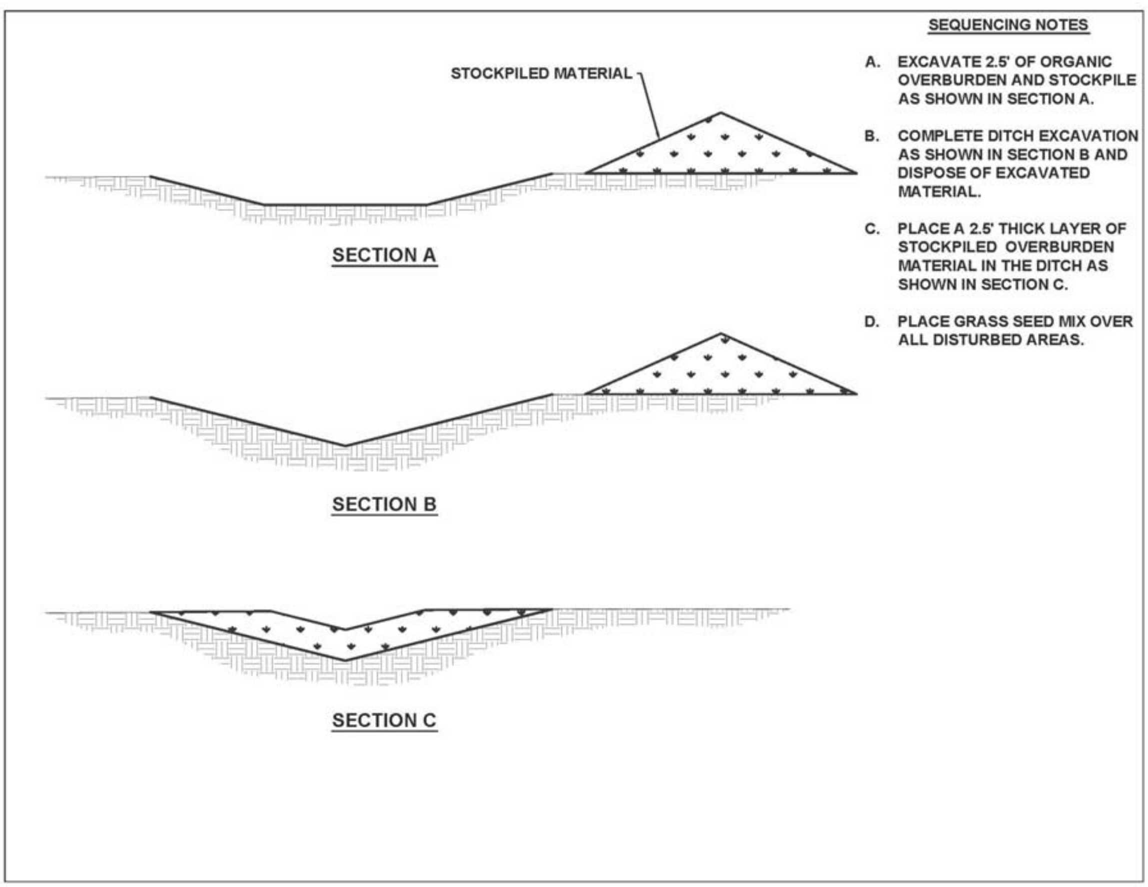


Figure 12: Drainage Swale Excavation Sequence Plan

Figure 13: Foundation Pad on Perched Ground

Figure 14: Adjustable Triodedic Frame

Figure 15: Insulation Between Timber Pads

Figure 16: Boardwalk Footing Placed on Well Drained Soil

9.0 BANK STABILITY ALTERNATIVES

Bank stability alternatives, including sheet pile walls, riprap revetment, and articulating concrete blocks, were evaluated based on the erosion analysis in presented in Section 6. Other types of walls, such as gabion baskets and soldier pile walls were not considered because those systems would be difficult to construct given the depth of the Kuguklik River channel at Kipnuk.

9.1 Riprap Revetments

Riprap design considered several factors, including ice scour, surface wave forces, water velocity, and embankment soils. Results from a hydraulic modeling analysis (HEC-RAS) were used to provide the hydraulic values required for the riprap calculations. Preliminary riprap design criteria and selection of riprap rock sizes are included in the Hydraulics and Hydrology report (Karle, 2016) in Appendix A. Class II and Class IV riprap sizes for this study would meet gradation in accordance with the 2004 State of Alaska Department of Transportation and Public Facilities Standard Specifications for Highway Construction (Class II weighing up to 200 pounds, Class IV weighing up to 2000 pounds).

The design of the top portion of the revetment accounted for ice scour and waves from storms. Class IV riprap would be required at the top of the revetment to resist forces from ice and storm driven waves. Class IV riprap will resist removal by ice better than Class II riprap. Note that much of the existing Class II riprap along reach 2 was likely removed by ice (see figure 18). Articulating Concrete Block matting could also be installed above water instead of Class IV riprap.

Class II riprap is suitable below Low Water based on the river velocity, with a minimum thickness of 1.9 feet (see Appendix A). Thickness would be increased 1.5 times for placement underwater, therefore a three foot thick Class II riprap blanket would be placed between low water and the scour toe. Riprap revetments typically include a buried scour toe at the bottom of the channel. This is not practical in the Kuguklik River due to depth of the channel. A launching toe/launching apron is a practical alternative in this case. This would be accomplished by placing an extra quantity of Class II riprap at the bottom of the channel. Figure 20 shows the riprap revetment concept with a self-launching apron at the toe of the revetment. The self-launching apron would fill in the scour hole as soil is eroded, and keep the scouring in check. Class II riprap would be placed in two slough entrances to reduce erosion at those locations.

Soils. Underlying soils were assumed to be similar to those found in the Kipnuk Bulk Fuel and Powerplant Facility report (Duane Miller 2007) and the Chief Paul Memorial School Expansion report (Golder, 2011). The soils in those locations are fine grained soils. Additional geotechnical investigations along the proposed riprap revetment would be required to confirm assumptions prior to final design.

A transitional filter layer is required to prevent the migration of the bank's fine soil particles through the riprap structure, and permit relief of hydrostatic pressures within the soils. If openings in the filter are too large, soil particles could move through the filter, potentially resulting in internal erosion of the soil (piping) and failure of the bank. A filter layer typically consists of 12 inches of gravel placed over the existing bank prior to placement of the larger rock. The thickness was increased to 18 inches due to placement underwater. The gravel filter layer would be placed underneath the riprap rock in the underwater portions of the revetment. For cost savings, filter fabric can be placed under the Class IV riprap (above mean low water) in lieu of a rock filter layer. Riprap estimates presented in this report assume a layer of filter fabric would be used under the top section of the revetment.

Tie Back. The river bank upstream of reach 3 will continue to erode, though at a slower rate of about 3 feet per year. Tie backs are commonly used at the upstream end of riprap revetments to help control erosion and prevent flanking around the back side of the riprap protection. Tie backs are either riprap filled trenches or windrows of riprap rock aligned perpendicular to the revetment. To reduce the potential for accelerating permafrost warming and creating new subsurface disturbances that may enhance erosion potential, we recommend an above-ground (windrow) tie back at the upstream end. See Figure 17 below.

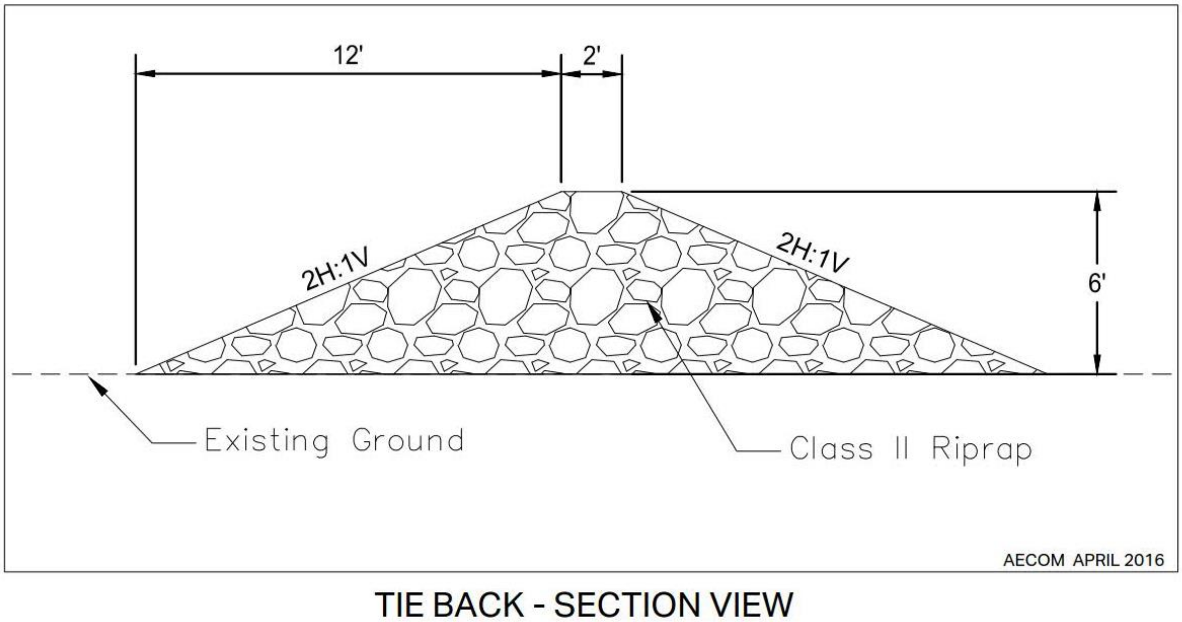


Figure 17: Tie Back Section View

Articulating Concrete Block. As an alternative to Class IV riprap, Articulating Concrete Block (ACB) could be installed at the top of the bank above Low Water. The estimated cost to install ACB matting is approximately equal to the estimated cost for rip rap (see Appendix C Cost Estimates). The ACB mats would be less susceptible to removal by ice than riprap due to their smooth surface. A filter fabric layer would be required under the ACB mats. The top edge of the ACB mats would be buried into the top of the river bank. Organic soils can be placed inside the cavities of the articulating concrete block mats and seeded to re-establish vegetation between the blocks.

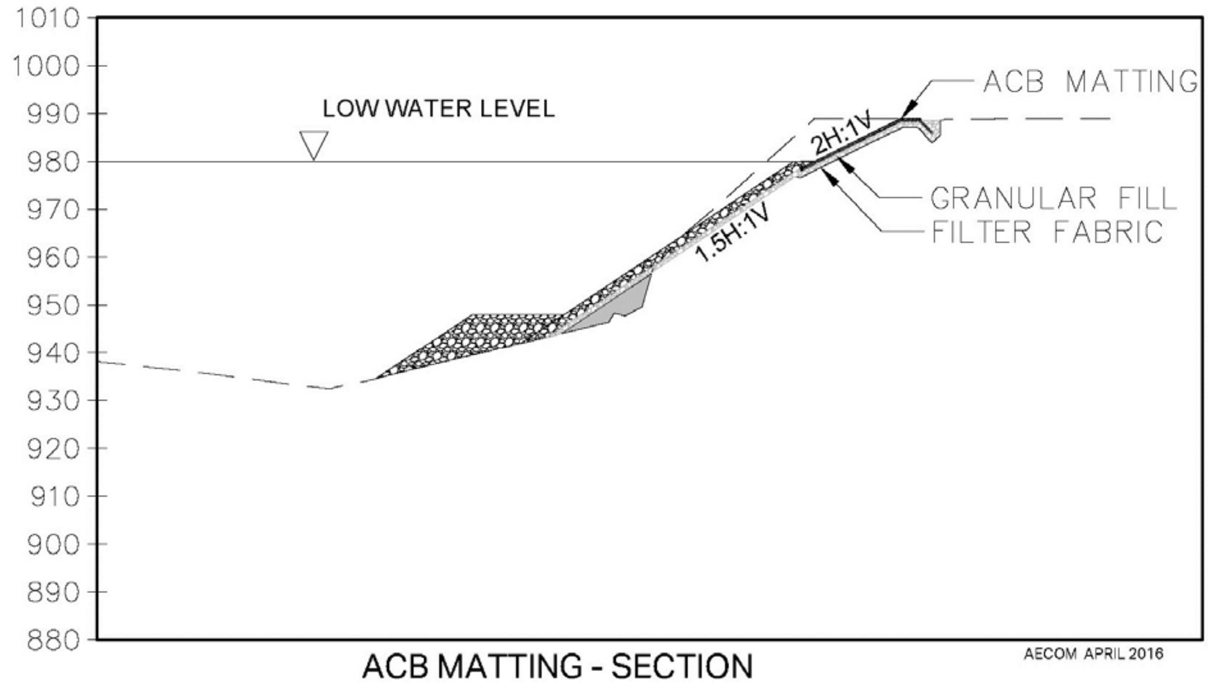


Figure 18: Riprap With Articulating Concrete Blocks Matting

Figure 19: Articulating Concrete Blocks

Riprap Maintenance. Riprap rocks are susceptible to removal by ice scour and periodic replacement of rock should be expected. Riprap revetments should be inspected annually or after major storms as part of an ongoing maintenance program. The maintenance program should budget for periodic replenishment of rock or repair of damaged sections. Damaged areas should be repaired as soon as possible to prevent further erosion. Additional material could be procured during construction and stockpiled in the community for an on-going maintenance program, as well as purchasing of an excavator, (by the Kipnuk Community) required to place the material.

9.2 Alternative 1 Riprap Revetment With Sheetpile Barge Landing (Reach 1, 2, 3)

This alternative is similar to the riprap revetment option suggested in the 2009 U.S. Army Corps of Engineers Community Erosion Assessment. This alternative includes a 190 foot long sheetpile wall at Reach 1 for a barge landing. Currently the community barge landing located at reach 3 is rapidly eroding, with no erosion protection in place. The sheet pile walls (see figure 21) would embed to depths of 40 feet with riprap protection in front and along the sides. The height of the wall above the riprap would be 15 to 20 feet.

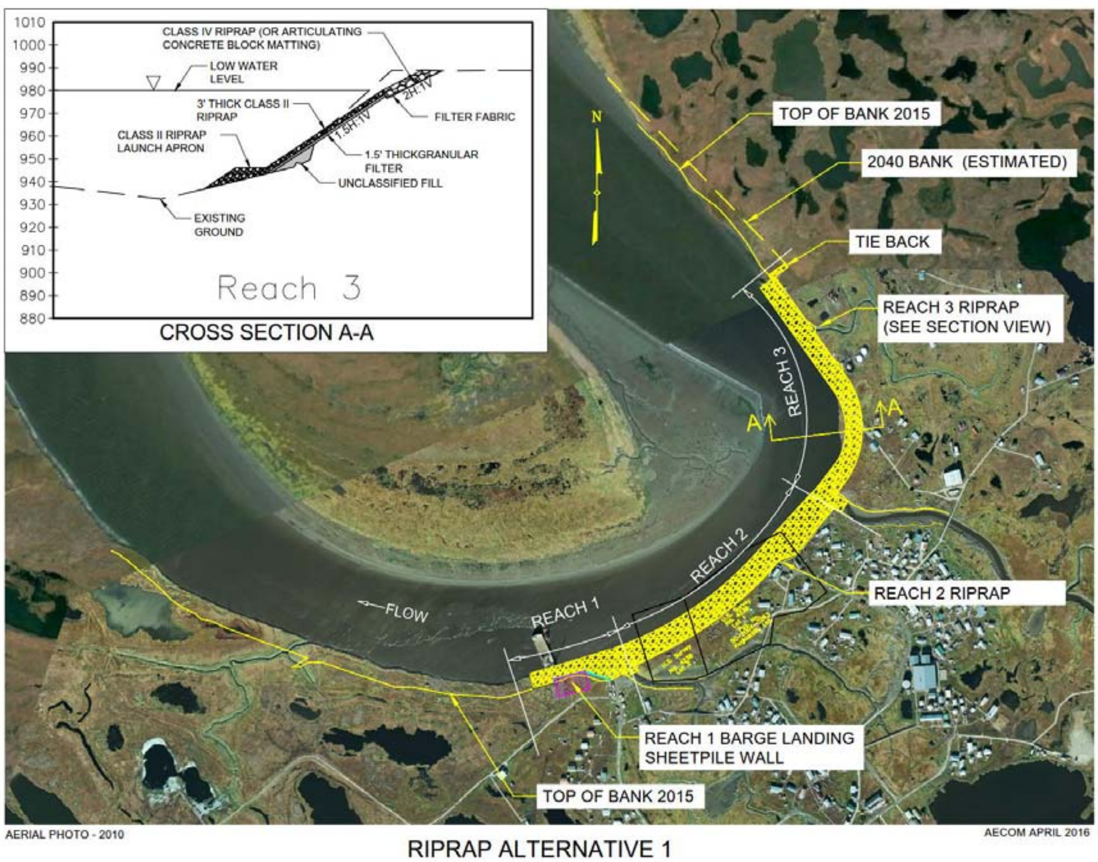


Figure 20: Riprap Alternative1 with Sheetpile Barge Landing

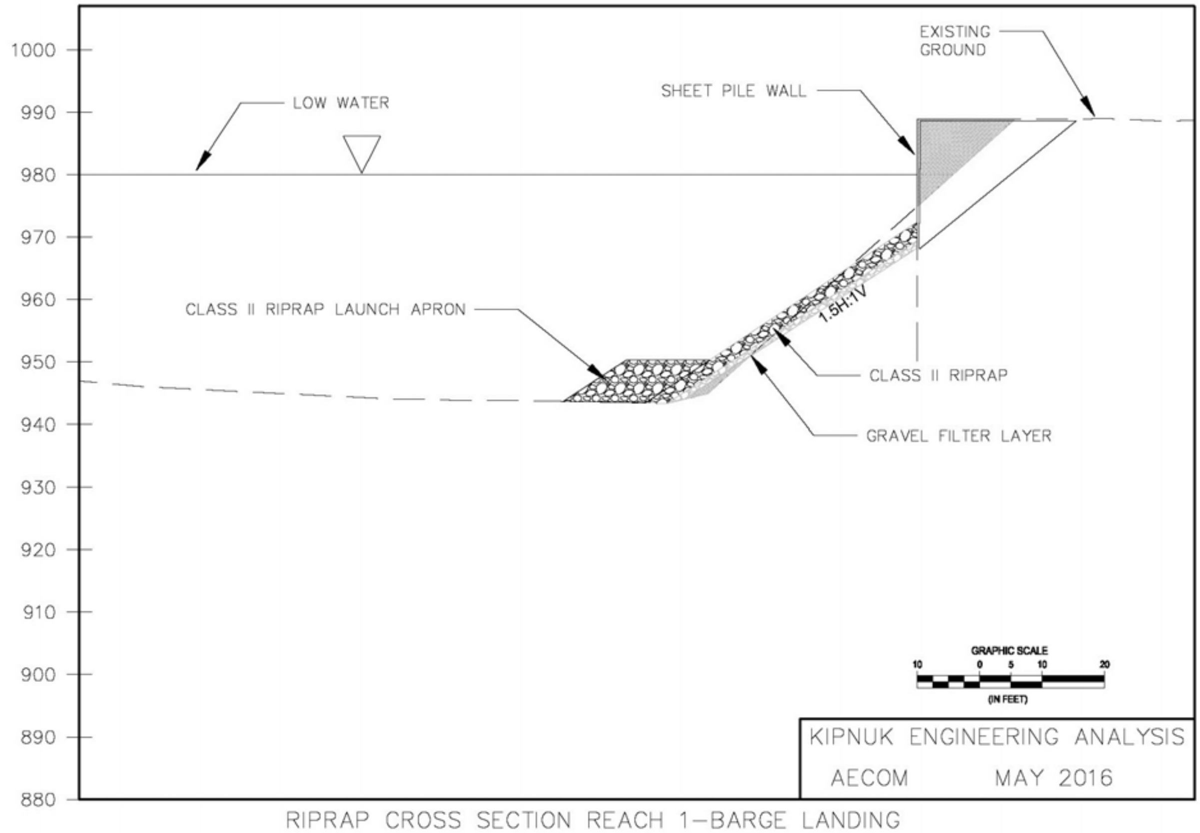
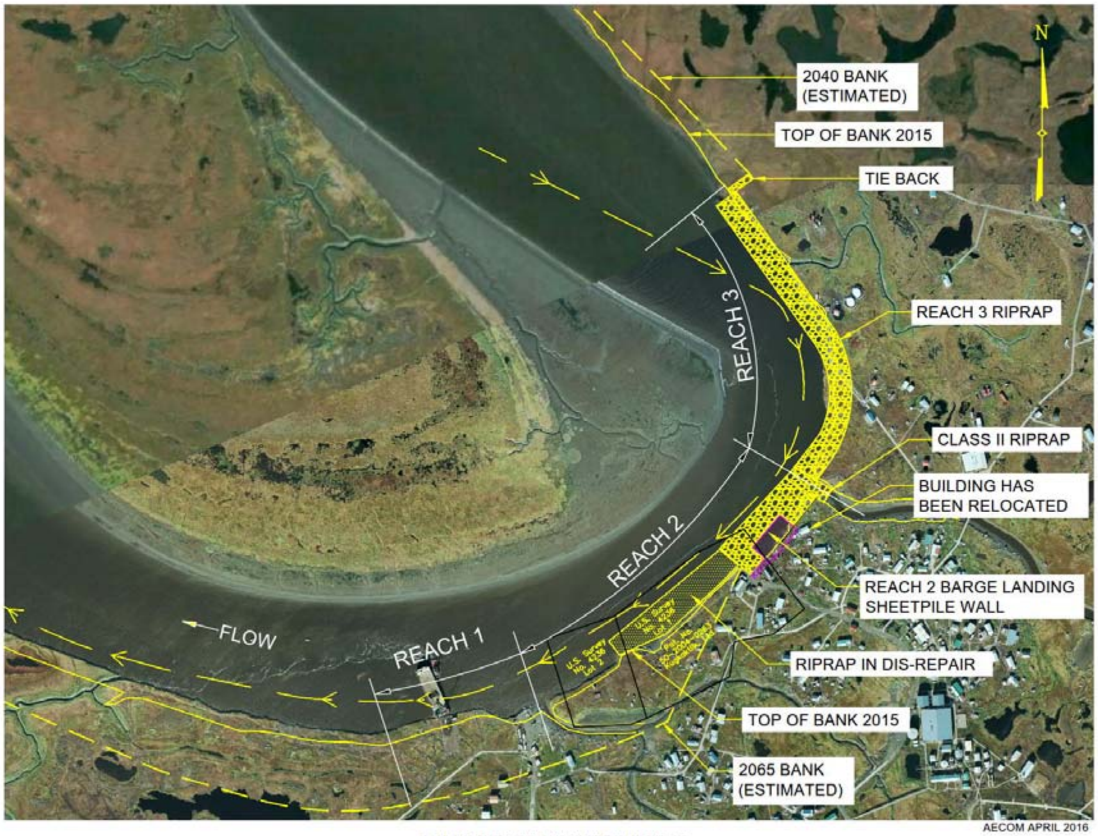


Figure 21: Cross Section Reach 1 Sheetpile Barge Landing

9.3 Riprap Alternative 2 (Reach 3 Riprap with Reach 2 Barge Landing)

Reach 2 Riprap would terminate at the Kipnuk Traditional Council (KTC) Lodge and tie into the existing riprap revetment. The existing revetment is in disrepair (see figure 22 and Section 6.2). The revetment has held erosion in check for many years though it is unclear the condition of the riprap below the water line and how effective the revetment will remain without repair or replacement. Under this alternative a barge landing would be located near the KTC lodge. The southern portion of Reach 2 and Reach 3 would remain unprotected and continue to erode.

Figure 22: Riprap In Disrepair, Reach 2

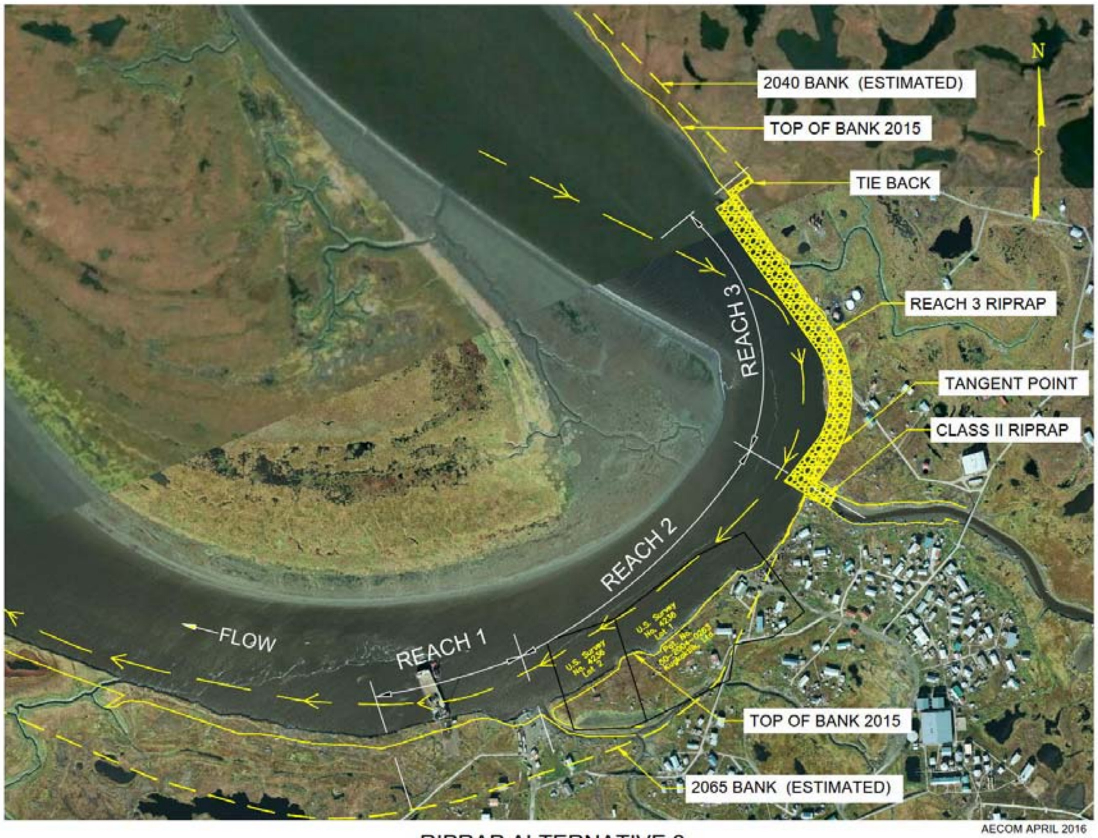


RIPRAP ALTERNATIVE 2

Figure 23: Riprap Alternative 2

9.4 Alternative 3 Reach 3 Riprap

Due to the sharp bend of the Kuguklik River in Reach 3, water shear forces act on the bank at a 45° angle, creating large shear forces and leading to severe erosion rates. Riprap placed along Reach 3 would protect the bank from erosion and direct the river current around and through the sharp bend. Under this alternative Reaches 1 and 2 would remain unprotected and continue to erode. Erosion rates along the unprotected bank downstream of Reach 3 may increase due to turbulence and secondary currents that can develop downstream of the riprap. Design guidelines recommend that a riprap revetment extend 1.5 times the channel width beyond the downstream tangent point of river a bend (FHWA, 1989). Alternative 3 riprap would extend 250 feet beyond the tangent point, or approximately half the channel width. The above criterion is based on laboratory conditions (FHWA, 1989); therefore a hydraulic analysis based on site specific factors (bank material, average velocities, tidal channel influences, etc.) should be performed prior to final design in order to define the required extension length beyond the bend. Lengthening the revetment would likely be needed in future construction phases if first phase funding constraints do not allow construction beyond Reach 3.



RIPRAP ALTERNATIVE 3

Figure 24: Riprap Alternative 3

9.5 Riprap Construction and Maintenance Costs

Construction and maintenance cost estimates for the riprap alternatives are summarized below. More detailed construction cost estimates are included in Appendix G. The estimated cost to import and install Class II riprap is $350 per cubic yard. For this project the estimated cost averages $9,000 per linear foot. This is roughly 100 percent higher than the U.S. Army Corps of Engineers (USACE) estimate in 2009 ($4750 per foot). It is unclear what quantities the USACE assumed for scour protection or unit cost prices for the riprap rock. It is probable that the quantities for this study are significantly greater due to the deep scour hole discovered in 2015. The nearest rock quarry that can provide quality riprap rock is located in Nome, Alaska. The cost to procure the rock from Nome and deliver to Kipnuk is about 75% of the total estimated cost. The cost estimates for Alternatives 1 and 2 assumes a 200 foot long section of sheetpile wall would be installed for a future barge landing ($1.5M).

Table 1: Riprap Construction Cost Summary

Riprap Alternatives Construction Cost Summary					
	Sheet Pile Wall-Barge Landing	**Reach 1**	**Reach 2**	**Reach 3**	**Total**
Alternative 1	$1.5M	$4.5M	$15.2M	$13.2M	$34.4M
Alternative 2	$1.5M	-	$4.5M	$13.2M	$19.2M
Alternative 3	-	-	-	$13.2M	$13.2M

For maintenance costs we assumed that 15 percent of riprap rock would need to be replaced every 15 years.

Table 2: Riprap Maintenance Cost Summary

Riprap Maintenance Cost-Every 15 Years					
	Barge Landing	**Reach 1**	**Reach 2**	**Reach 3**	**Total**
Alternative 1	$.4M	$.6M	$2.6M	$3.0M	$6.6M
Alternative 2	$.4M	-	$.4M	$3.0M	$3.8M
Alternative 3	-	-	-	$3.0M	$3.0M

9.6 Alternative 4 Seawall (Sheet Pile Wall)

This alternative would include a barge landing at Reach 1 and sheet pile walls at Reaches 2 and 3. Riprap would be placed at the entrance of the two sloughs to minimize erosion. Sheet pile walls provide an effective steel barrier that prevents erosion of soft river bank materials, and are not susceptible to ice damage. A second tie-back wall would be required 140 feet behind the bank which would require re-location of several buildings.

Soil Parameters. Soil parameters for preliminary design were obtained from the Kipnuk Bulk Fuel and Powerplant Facility report (Duane Miller, 2007), Kipnuk Boardwalk Improvements, Phase II (Golder, 2011), and the Chief Paul Memorial School Expansion report (Golder, 2011). The material parameters used are considered appropriate for this level of conceptual design. Additional geotechnical investigations along the proposed wall alignment would be required to confirm assumptions of the material parameters and the geologic cross-section of the subsurface materials prior to final design.

Wall Design. The sheetpile walls were designed using Pro Sheet software and checked with the U.S. Army Corps of Engineers CWASHT program. It was estimated that river scour would erode away soils near the base of the wall so that eventually the wall face would span 75 feet from bottom to top. Stronger sheet pile wall systems that were analyzed included a Combo H-Pile wall system and circular cell system. Traditional Z shape sheet pile systems do not develop enough strength to resist the estimated earth pressure forces imposed on a 75 foot tall wall.

A Combo H-Pile system would consist of doubled up H-piles (3.5 feet front-to-back of wall) driven to a depth of 100 feet. Z piles would be placed between the doubled up H-piles. Tie backs near the top of the wall would connect to an anchor wall approximately 140 feet behind the H-Pile wall. The anchor wall would consist of Z shaped sheet piling driven to depths of 35 feet. Steel tie back rods would be 2½" in diameter and placed approximately 8 feet apart, and 15 feet below the ground surface.

Corrosion. The Kuguklik River is a tidally influenced river. Sheet piling installed at Kipnuk would be susceptible to corrosion by salt water. In addition, Kipnuk soils are typically saline (Golder 2011). Piling for marine facilities are generally galvanized or epoxy coated prior to installation to protect against salt water induced corrosion. In addition, cathodic protection systems are typically installed on piling strucutures to help reduce corrosion. There are two types of cathodic protection systems: impressed currents and galvonic (sacrificial anodes). A sacrificial anode system would be recommend for Kipnuk because impressed current systems are difficult and expensive to maintain. Sacrifical anodes would require periodic replacment. Cost estimates for sheet piling structures have assumed expoxy coated steel and a sacraficial anode system.

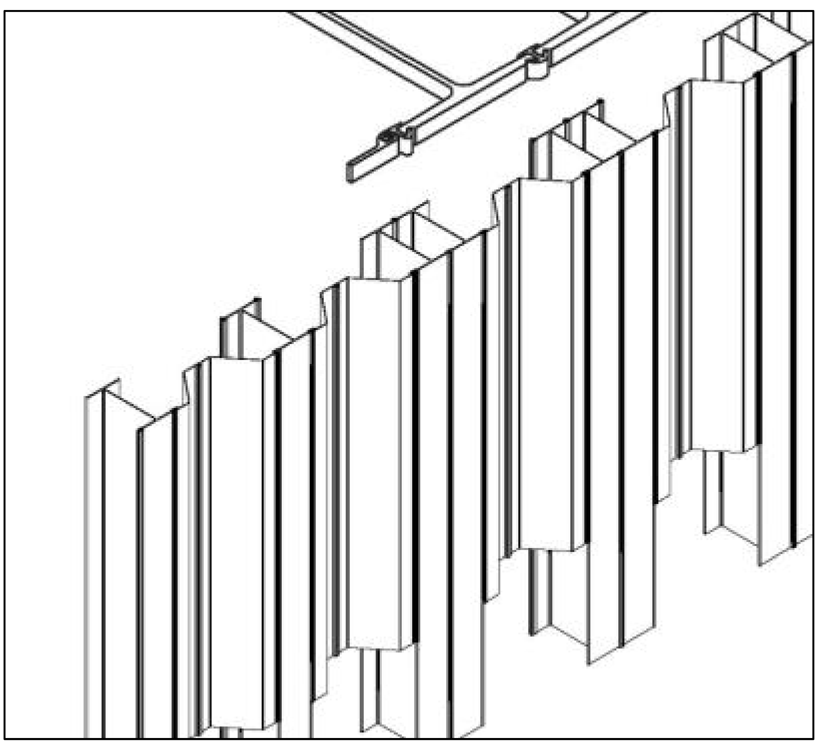


Figure 25: Combo H-Pile Wall System

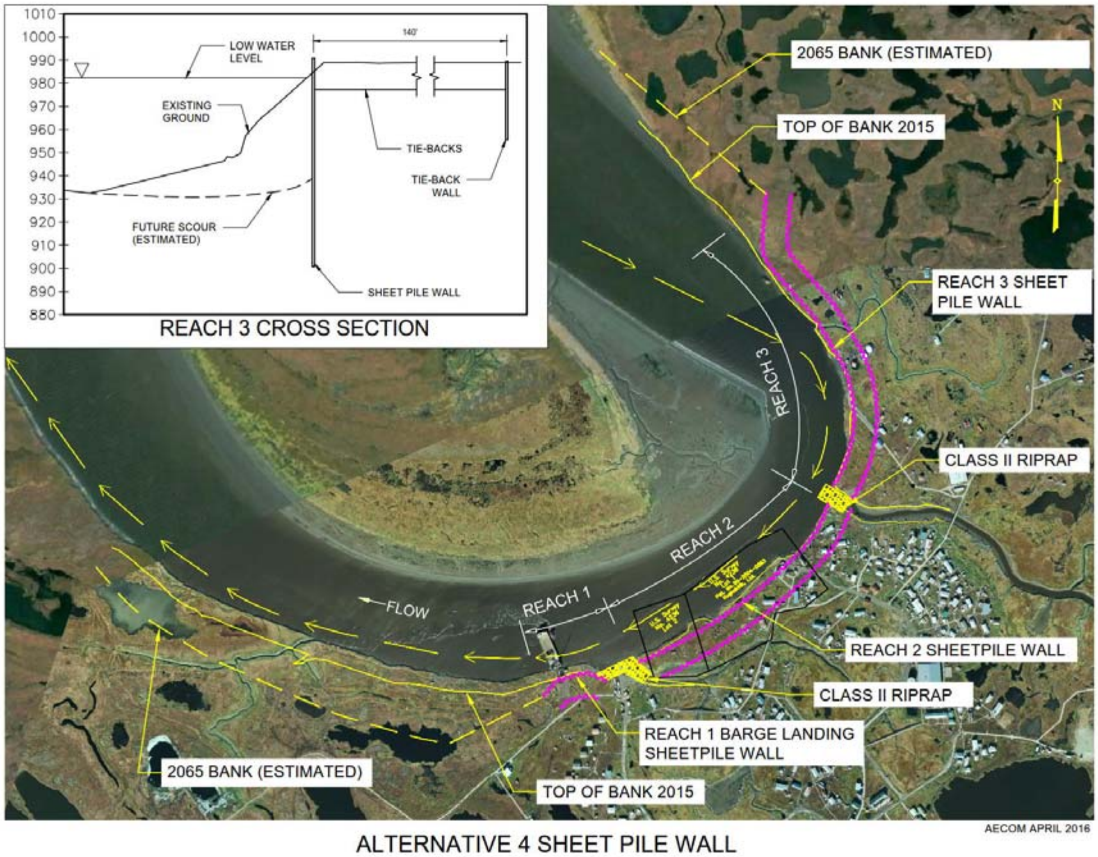


Figure 26: Alternative 4 Combo H-Pile System Sheet Pile Wall

9.7 Alternative 5 Circular Sheet Pile Wall System

The circular cell system would consist of a series of flat sheet piling members connected together to complete a 60 foot diameter circle. The system does not require tiebacks and has worked well at many port facilities with tall dock faces up to 60 feet high. The circular sheets would be installed to depths of 95 feet below the top of bank. The cost would be similar to that of Alternative 4. A detailed cost estimate was not developed for this alternative because the riprap alternatives would provide a more cost effective solution.

Figure 27: Circular Sheet Pile Wall System

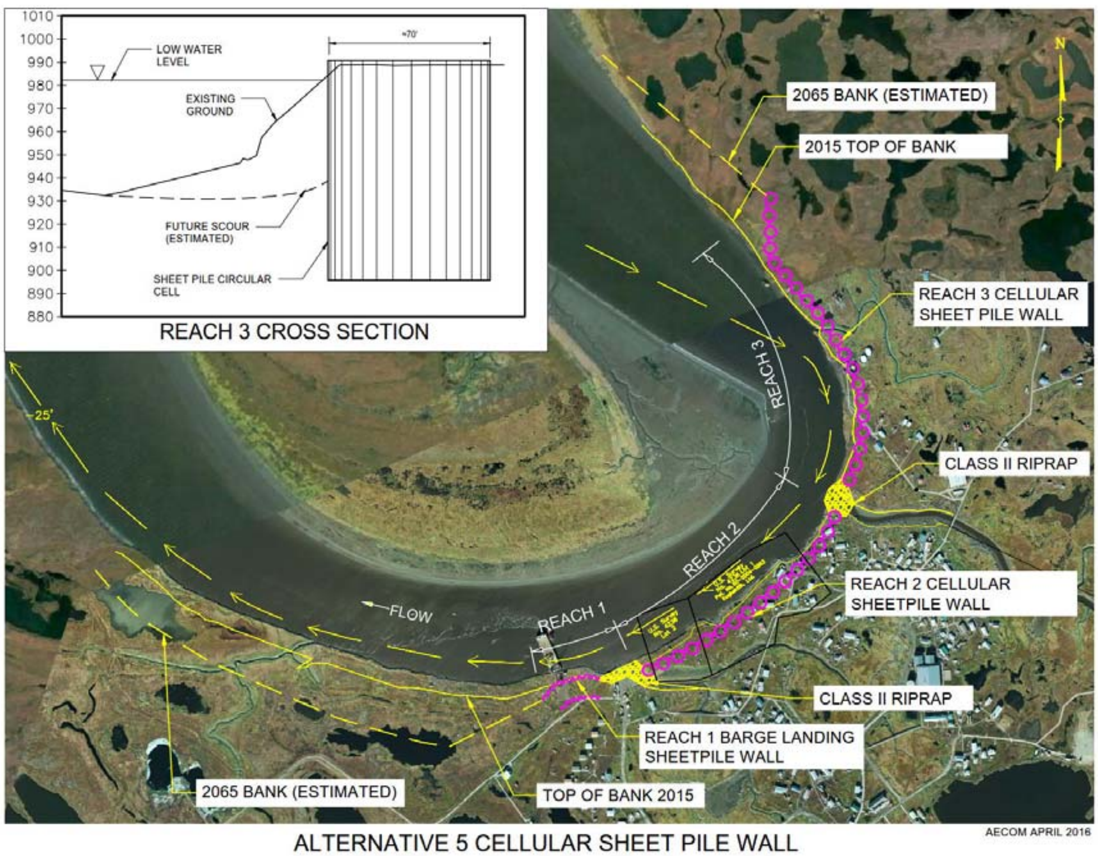


Figure 28: Alternative 5 Cellular Sheet Pile Wall

9.8 Sheet Pile Wall System Construction and Maintenance Costs

The tied-back H-Pile system used in this design would require massive steel components resulting in roughly 6 tons of steel per linear foot of wall. Purchasing, fabrication, and shipping of the steel for the H-Pile system would cost between $3 and $4 per pound of steel, placing a total materials cost of about $20,000 per linear foot of wall. Mobilization, installation, engineering, and permitting costs would be in addition to material costs, bringing the total cost of the wall to roughly $23,000 per linear foot. This cost is generally in agreement with the 2009 USACE estimate of $24,000 per linear foot of wall.

Maintenance Cost. Periodic replacement of anodes and coating on a sheetpile wall is recommended every 15 years. Periodic replacement and repair of a Riprap revetment in the two sloughs would be required. Both estimates assume a replacement project would occur every 15 years.

Table 3: Sheet Pile Wall Construction and Maintenance Cost Summary

	Reach 1	Reach 2	Reach 3	Total
Construction Cost	$11M	$37M	$41M	$89M
Maintenance Cost (Every 15 years)	$.4 M	$1.5 M	$2.1 M	$4M

9.9 River Avulsion (Change of Course)

This alternative would attempt to mimic natural river behavior by creating a 'cutoff' meander; this would be accomplished by excavating a channel across the inside meander loop, approximately one mile north of Kipnuk (see Figure 29). This alternative was dismissed and not developed based on the high cost to construct and difficulties described below.

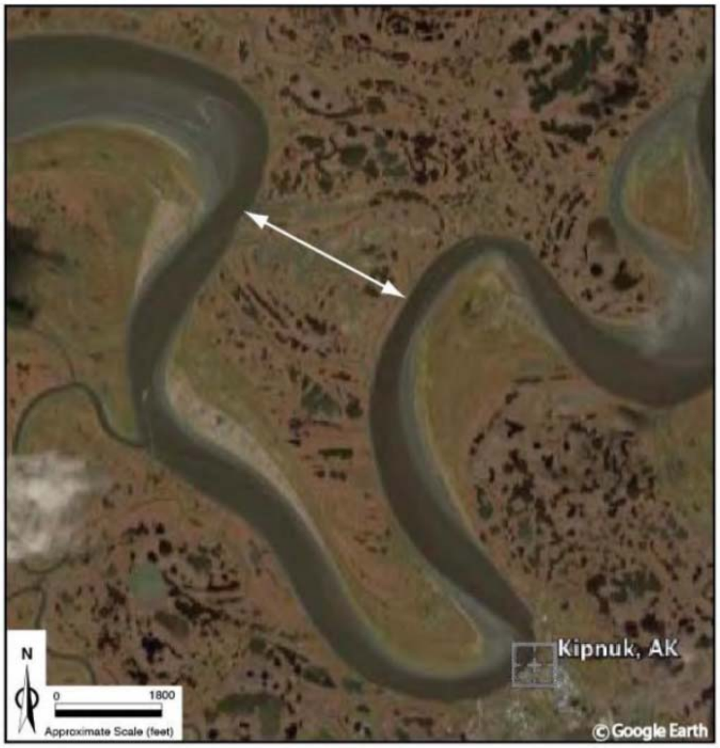


Figure 29: Suggested Location of Cutoff Meander (From Golder 2011)

As the HIA noted, such an avulsion on the Kuguklik River near Kipnuk could take up to 400 years to occur naturally. Meander chutes that form to shorten a river loop are formed by lateral erosion of the bank of the upstream arm of the loop, which causes the stream to cut through the neck of the loop into the downstream arm. Comparison of meander bank positions from 1954 (USGS quad map), 1984, 2003, and 2012 (aerial imagery) indicates that lateral erosion of the loop arms is occurring at a slower pace than the erosion directly in front of Kipnuk. See Figure 30.

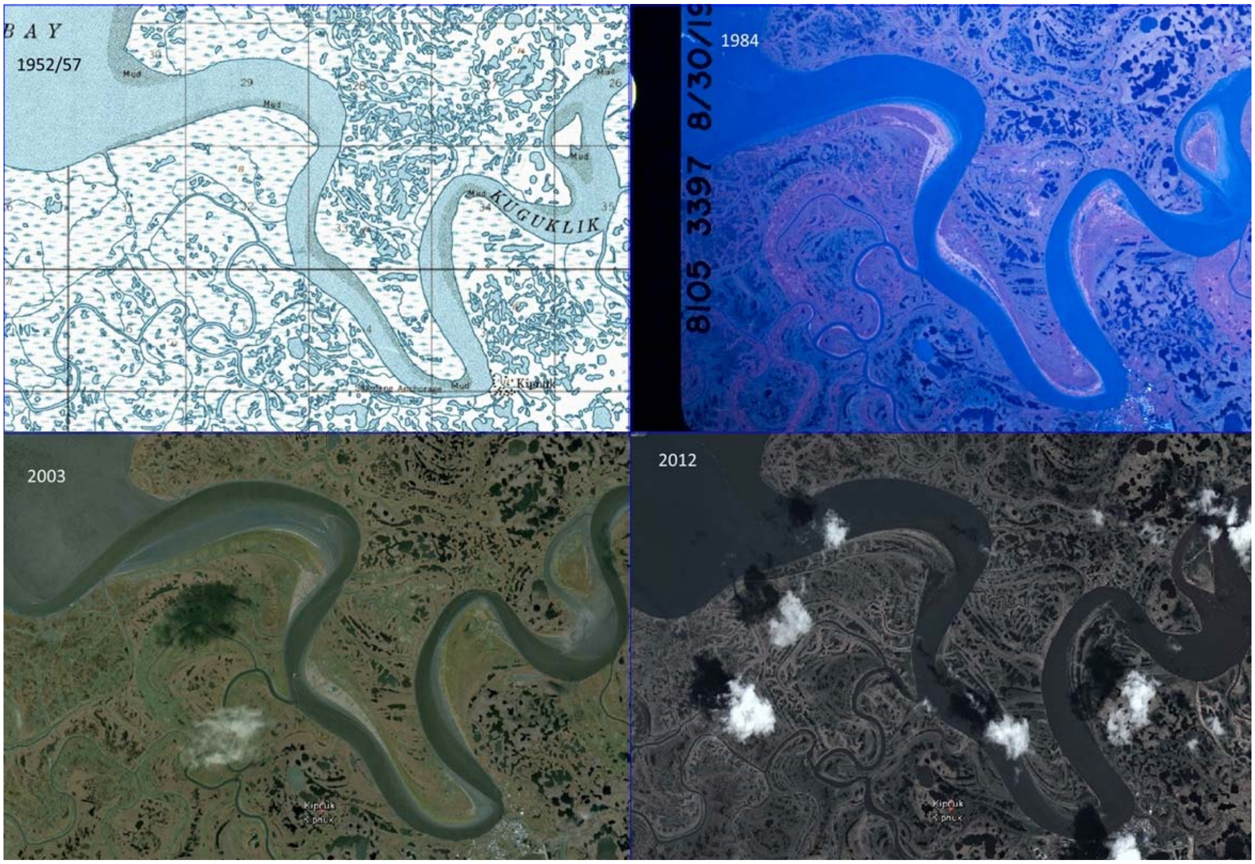


Figure 30: Kuguklik River Meander Loop at Kipnuk; 1954 (Upper Left), 1984 (Upper Right), 2003 (Lower Left), and 2012 (Lower Right).

Forcing an avulsion (excavating a cut-off channel) could result in unintended consequences. A comprehensive technical analysis would be required to understand the potential impacts of such an excavation regarding bank stability, river position, sediment load, and other hydraulic characteristics. Some reports have noted that channel shortening projects may present undesired results, including ongoing meandering, increased sediment loads, and erosion upstream and downstream of the project site (Julien, 2002). One report noted that channel avulsions can trigger the rapid delivery of increased sediment loads into the river at rates that are one to five orders of magnitude larger than those produced by natural lateral bend migration (Zinger et al., 2011). At the confluence of the Wabash and Ohio rivers, a channel avulsion led to significant changes in channel morphology, impeded barge traffic, and necessitated extensive dredging (Zinger et al., 2011). Such sedimentation could create problems for barge traffic in the lower reach, or at the mouth, of the Kuguklik River.

The results of a meander cutoff are commonly thought to include the formation of an oxbow lake in the abandoned channel. However, it is unclear if such a formation would occur, or the shortened channel would even remain stable, on a tidal channel. Twice a day, the Kuguklik River current runs upstream (uphill), pushed by the pressure of the higher water behind it from the rising tide. At the junction of the old channel and a newly excavated channel with a positive slope that is 5 times steeper, a majority of the incoming tidal flow may seek the channel of least resistance and continue to flow up the old channel. An analysis would have to assess the possibility that, because of the large bi-directional flow, an excavated shortcut would fill with sediment and be abandoned.

The formation of an oxbow lake disconnected from the Kuguklik River, if it were to occur, could also have serious consequences for Kipnuk. The village, like many in Alaska, was founded on the river because of the access it provides. Commercial fishermen, barges that deliver heating fuel oil and supplies, and Kipnuk residents all depend on quick and easy access to the village. The ramifications of separating the village from the river would require much consideration. Finally, it is important to note that this alternative would not protect Kipnuk from flooding due to high water events that overtop the riverbank (Golder, 2011).

Forcing an avulsion (excavating a cut-off channel) does not appear to be a cost effective option. Excavation of such a cutoff channel would be massive. Based on the recent channel survey, the smallest channel cross-section (xsec 3823) has a width of 550 feet and an area of 11,500 square feet (at 58,000 cubic feet per second discharge). If constructed to match the existing channel's hydraulic capacity, the total amount excavated would be on the order of 1.7 million cubic yards or more. The cost of the excavation alone would exceed $17 million, based on a unit cost of $10 per cubic yard for excavation. Figure 31 shows the width, depth and area at Cross-section 3823; the design shape and depth for an excavated channel may vary.

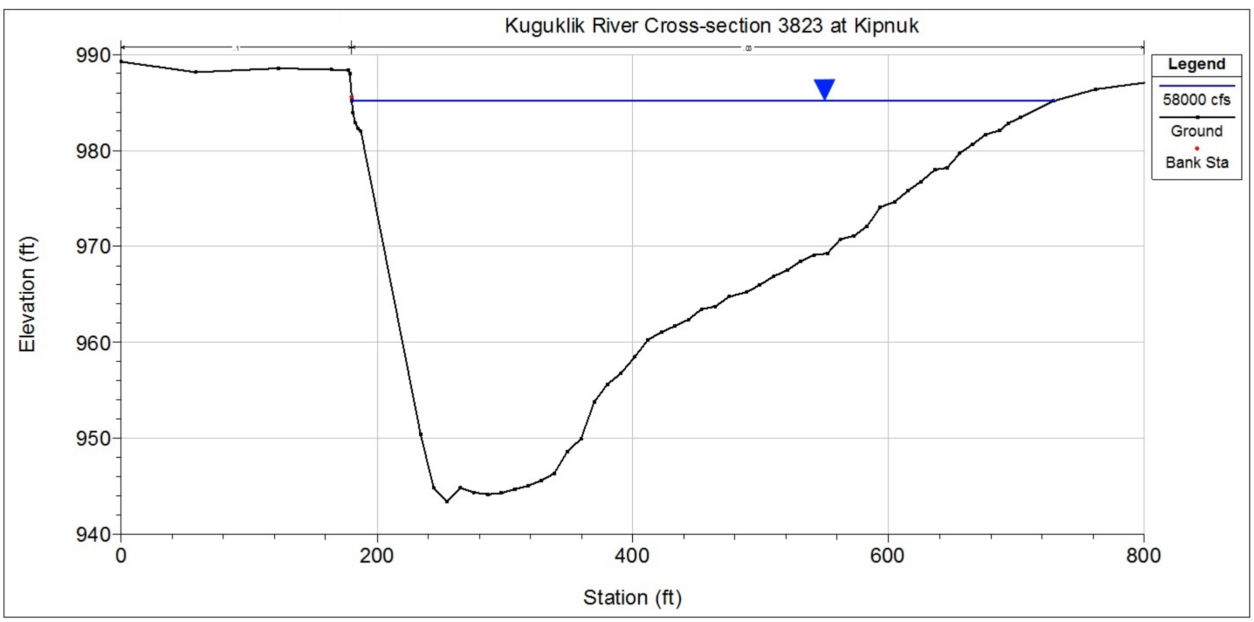


Figure 31: Cross-Section 3823 at 58,000 Cubic Feet Per Second Discharge

Permitting of such a large-scale channel alteration with Federal and State agencies would be difficult, perhaps impossible. The Kuguklik River is a state-cataloged anadromous stream. Permitting agencies would likely be concerned about the ecological disturbances and habitat loss both upstream and downstream of the project location. The excavated material would need to be placed in a massive spoil pile would likely become the dominant land-form feature in and near the Kipnuk area. The spoil pile would need to be placed somewhere out of the storm surge flood-prone zone where it could not erode back into the channel during high water events. Placement of that quantity of material on top of habitat-rich wetlands would be difficult to permit by regulatory agencies.

9.10 Recommended Alternative

AECOM presented Alternatives 1 through 5 to the Kipnuk Traditional Council and community on April 5th, 2016. Alternative 2 was the desired alternative, and Alternative 3 was recognized as the highest priority to be completed in a first phase project. As discusssed in Section 9.4, erosion rates along the unprotected bank downstream of Reach 3 could increase due to turbulence and secondary currents that can develop downstream of the riprap. Construction of Reach 3 alone should not be undertaken unless there is some assurance of funding to implement Alternative 2, or completion of a more detailed hydrulic analysis so the hydraulic impacts of Alternative 3 are well understood. The detailed hydraulic analysis should be completed as part of final design and would consist of a comprehensive river survey (one mile upstream and downstream), and two dimensional HEC-RAS modeling of the river and the proposed riprap revetment.

9.11 Right of Way and Utilities

Easements would be required where the riprap revetment impacts two lots belonging to Kugkaktlik Ltd. There are no known utility conflicts at this time.

10.0 RELOCATION ALTERNATIVE

A relocation alternative could be an option in case no funding can be secured for bank stability projects. Relocation could be planned in phases. Immediately threatened structures would be relocated in a first phase project. Structures not immediately threatened would be included in subsequent relocation phases. The scope of this study did not include a detailed condition assessement of buildings.

10.1 Impacted Property

If bank retreat continues at historic rates, within the next 25 years erosion will endanger several residential buildings, two commercial buildings, several small storage buildings, and three fuel tanks. Overhead electric/telephone distribution lines will be endangered by 2040. Several more residential buildings will be endangered between 2040 and 2065.

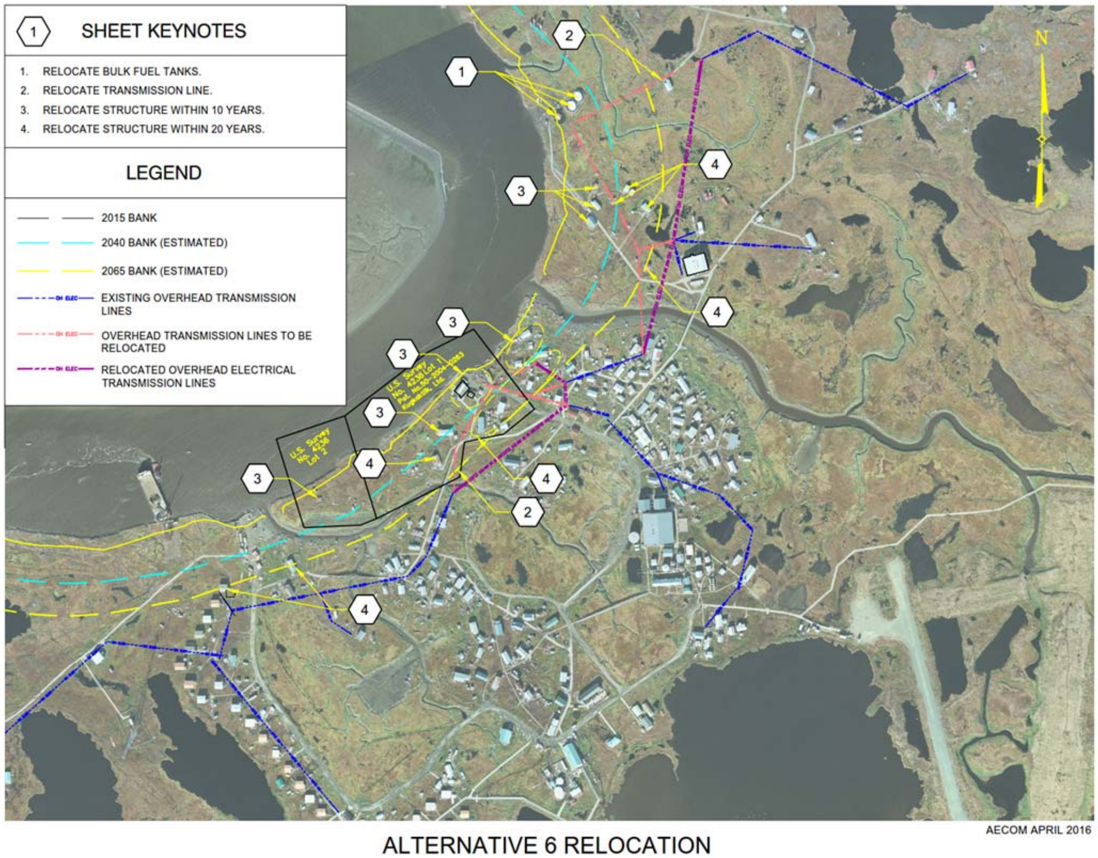


Figure 32: Relocation Alternative

10.2 Planned Building Replacement

Buildings wear out over time and heating systems become outdated. Older buildings often do not meet modern building code standards. Modern buildings constructed with R-50 insulation would require considerably less heating fuel and maintenance than buildings constructed 30 to 40 years ago. The community could develop a long term plan for complete replacement of older buildings instead of relocation. A more detailed assessment and cost analysis of each building would be required if the community decides to pursue that strategy. The assessment would need to weigh the value of each structure, the cost to repair and relocate, and the cost to heat and maintain, versus the cost of new construction.

10.3 Phase I Relocation

Relocation of the Kipnuk Traditional Council Lodge and Garage and a Kugkaktlik Limited (Corp) bulk fuel tank should be a priority under the relocation alternative. These structures should be relocated as soon as possible. The tank was less than 20 feet from the edge of the bank as of May 2015. The tank is supported on a timber foundation within a lined containment area and fenced perimeter. Several connexes and miscellaneous fuel tanks should be relocated in the first phase.

Figure 33: Kugkaktlik Limited (Corp) Tank

Figure 34: Eroding Bank Near Kugkaktlik Tank

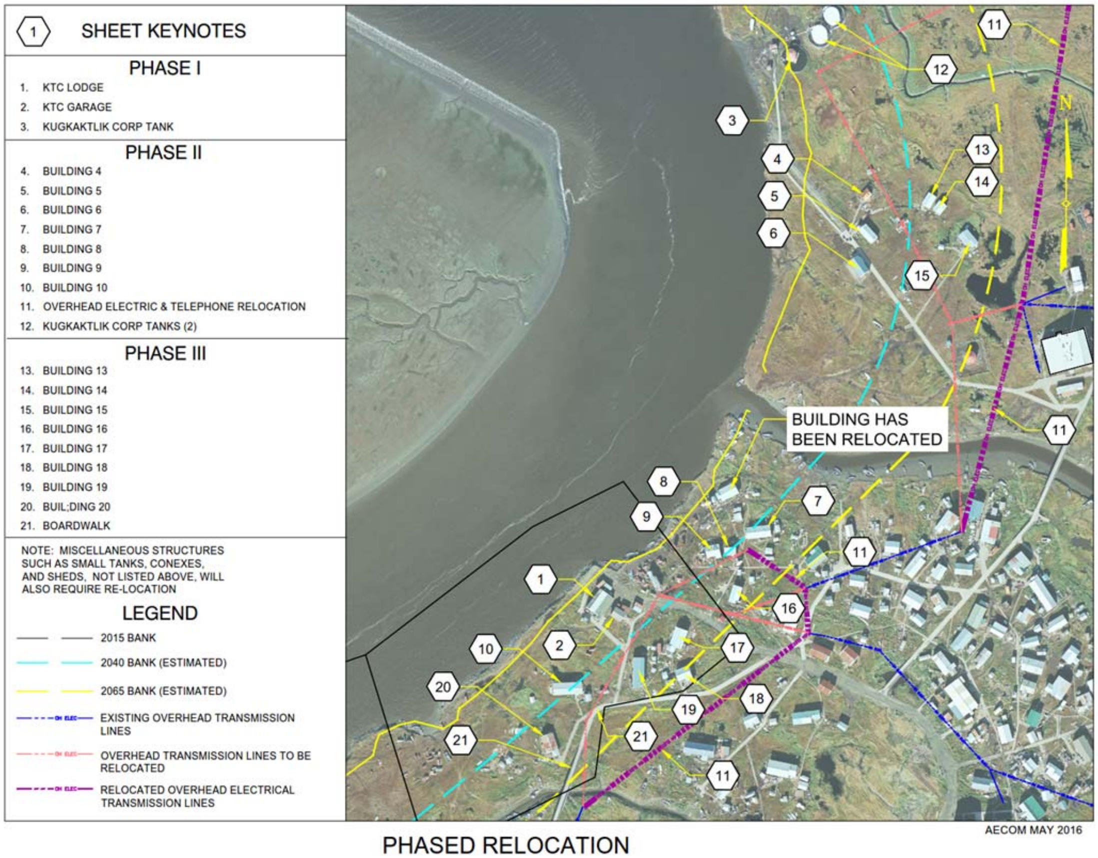


Figure 35: Phased Relocation

Figure 36: Kipnuk Traditional Council Lodge

Figure 37: Miscellaneous Structures-North of Kipnuk Traditional Council Lodge

10.4 Phase II Relocation

This phase would relocate structures within the next 5 to 10 years. Several residential buildings and two Kugkaktlik Limited (Corp) bulk fuel tanks will potentially be impacted by 2025. Overhead power and telephone lines will also be impacted. Figure 35 shows a potential overhead electric and telephone relocation plan in anticipation of the estimated 2035 river bank location. A 2017 project by the Alaska Energy Authority will provide fuel storage upgrades to the community. It is possible that the two Kugkaktlik Limited (Corp) tanks may be decommissioned or re-located prior to 2035. Several smaller tanks south of the Kipnuk Traditional Council Lodge would need to be relocated or properly decommissioned and disposed within the next ten years.

Figure 38: Miscellaneous Structures and Tanks South of the Kipnuk Traditional Council Lodge

Figure 39: Tank Farm-South of Kipnuk Traditional Council Lodge

10.5 Phase III Relocation

This phase would include structures that will need relocation by 2040. These include several residential buildings and 300 feet of boardwalk.

10.6 Relocation Costs

Building relocation assumes a cost of $250 per square foot. This is based on a similar re-location study conducted by AECOM for the community of Kivalina in 2010. The Kugkaktlik tank cost assumes steel perimeter dike and fencing could be re-used. The condition of the tank was not assessed in this study. The costs below assume no fuel leakage from the bottom of tanks has occurred, and therefore large scale environmental cleanup would not be required. If contamination of soils has occurred an environmental cleanup could cost an additional $4.9 million (USACE 2009). Electrical relocation assumes a cost of $20,000 per pole, spaced at 300 feet. Boardwalk costs assume $150 per linear foot.

Table 4: Relocation Cost Summary

Relocation Cost Summary					
	Buildings	**Tanks**	**Electrical**	**Boardwalks**	**Total**
Phase I (As Soon As Possible)	$1.05M	$250k	-	-	$1.3M
Phase II (2025)	$2.2	$1.0M	$300K	-	$3.5M
Phase III (2040)	$2.5M	-	$50K	$50k	2.6M

11.0 PROJECT FUNDING SOURCES

11.1 Bank Stability Alternatives

As of 2016 the U.S. Army Corps of Engineers provided matching funds up to $5 million for civil works projects, with a required match of 35 percent from a sponsoring state agency. This could provide up to $7.5 million for a project but would fall short of funding needed for a riprap revetment project. State sponsorship in 2016 is not likely due to the current State of Alaska budget crisis.

The Denali Commission is an independent federal agency designed to provide critical utilities, infrastructure, and economic support throughout Alaska. Assistance and applications for federal grant money for infrastructure projects can be accessed from the Denali Commission website at https://www.denali.gov/grants.

11.2 Relocation Alternatives

Funding for small projects may be more feasible given the tight federal and state budget constraints in 2016. Sources of State funding include the Division of Homeland Security & Emergency Management (DHSEM). DHS&EM administers FEMA mitigation grants to mitigate future disaster damages such as those that may affect infrastructure including elevating, relocating, or acquiring hazard-prone properties. (http://ready.alaska.gov/plans/mitigation.htm).

The State of Alaska Department of Commerce Division of Community and Regional Affairs (DCRA) administers the HUD/CDBG, FMA Program, and the Climate Change Sub-Cabinet's Interagency Working Group's program funds and administers various flood and erosion mitigation projects, including the elevation, relocation, or acquisition of flood-prone homes and businesses throughout the State. This division also administers programs for State's" distressed" and "targeted" communities. (http://www.commerce.state.ak.us/dca/)

The Department of Housing and Urban Development (HUD) provides a variety of disaster resources. HUD/CDBG provides grant assistance and technical assistance to aid communities in planning activities that address issues detrimental to the health and safety of local residents, such as housing rehabilitation, public services, community facilities, and infrastructure improvements that would primarily benefit low-and moderate income persons. (http://www.hud.gov/offices/cpd/communitydevelopment/programs/)

12.0 PERMITTING

This section provides an overview of permitting requirements for the proposed Riprap revetment and community drainage project discussed in previous sections. A more detailed description of permitting requirements is included in Appendix E.

12.1 USACE Department of the Army Permit

USACE Department of the Army (DOA) Permits would be required for construction of a drainage project in Kipnuk and a bank erosion project (Riprap) along the Kuguklik River. The USACE issues permits under the following authorities: 1) Section 404 of the Clean Water Act, which covers the discharge of dredged or fill material into waters of the U.S., including wetlands and 2) Section 10 of the Rivers and Harbors Act of 1899, which covers work in or affecting navigable water of the United States. Based on aerial photo review the project area appears to be comprised of freshwater wetlands. Because there is no wetland mapping available for the project area, the USACE may require delineation by a wetlands professional to confirm the presence of wetlands and determine the wetlands boundary.

There are two primary types of DOA permits, 1) Nationwide Permit and 2) Individual Permit. The proposed project activities do not appear to fall under any of the 2012 Nationwide Permits available in Alaska; therefore, an Individual Permit would be required. A summary of the 2012 Nationwide Permits can be found at:
http://www.poa.usace.army.mil/Portals/34/docs/regulatory/Summary_Table_2012%20NWPs_14%20Feb%202012.pdf

As the lead Federal permitting agency, the USACE would contact other agencies to determine if the proposed project will have adverse effects on the environment. USACE would:

- Consult with U.S. Fish & Wildlife Service and National Marine Fisheries Service concerning potential Threatened &Endangered species and Critical Habitat
- Consult with National Marine Fisheries Service for an Essential Fish Habitat Assessment
- Coordinate the DOA application with the ADEC to obtain a State Water Quality Certification
- Consult with the State Historic Preservation Office (SHPO) for the presence or absence of historic properties.

Additional permits would include:

- Alaska Department of Fish and Game Fish Habitat Permit (FHP)
- Alaska Pollutant Discharge Elimination System (APDES) Construction General Permit

Any work below the ordinary high water mark of Kuguklik River associated with the bank erosion project would require a FHP. FHPs generally take 30 to 60 days to process. As part of the APDES General Permit, a permittee must prepare a Storm Water Pollution Prevention Plan (SWPPP) which documents the selection, design, installation, and implementation of control measures to minimize pollutant discharges as required by law. The construction contractor would be responsible for preparation of the SWPPP and for obtaining coverage under the APDES General Permit, just prior to construction, as part of the construction contract.

Permitting of chosen alternatives would occur after the final designs are in advanced stages and prior to construction. Other permits in addition to the ones listed above could be required once the scope of the project is fully defined. A more detailed description of permit requirements is included in Appendix E.

12.2 National Environmental Policy Act

The National Environmental Policy Act (NEPA) of 1969 requires that prospective impacts of projects be understood and disclosed prior to a Federal agency issuing a permit or providing funding for a project. If USACE can determine that the environmental impacts from the project fall in a category of actions which do not have a significant effect on the environment then neither an Environmental Assessment (EA) nor an Environmental Impact Statement (EIS) would be required. If the significance of environmental impacts from the proposed action is not clearly established and the activities do not fall under a list of categorically excluded actions from NEPA, the USACE as the lead permitting agency, would need to prepare an EA and may require the applicant to provide appropriate information necessary for the preparation of the EA. The purpose of the EA is to determine if the project will cause significant effects. If the EA concludes that no significant impacts will occur, a Finding of No Significant Impact is prepared and is used to support USACE's permit decision. In the unlikely event that the EA for the proposed activity identifies significant impacts, an EIS would be required. Preliminary discussions with USACE in 2016 indicate that a small EA would probably be required for a river bank erosion revetment project (Riprap) or a community drainage project.

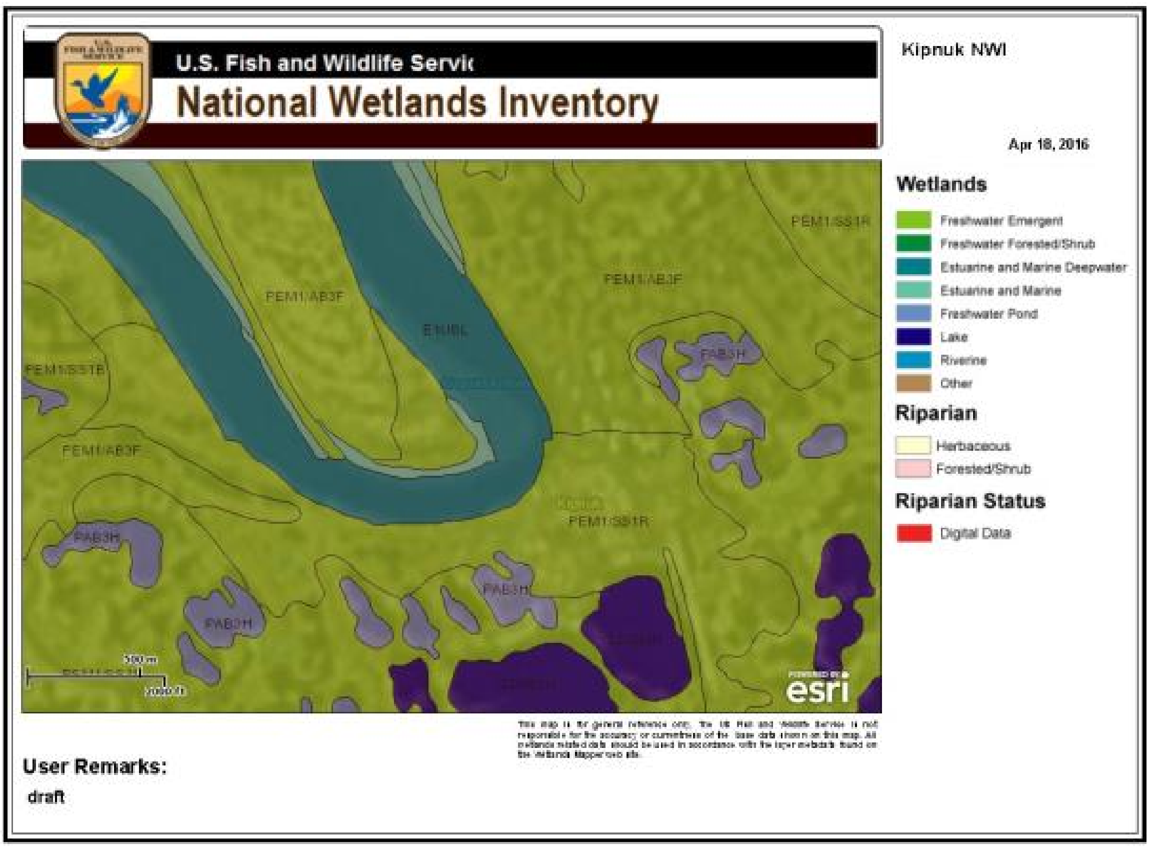


Figure 39: Kipnuk Area Wetlands Map

12.3 Permitting for Community Drainage Project

A community drainage project for the purpose of reducing standing water in the community would require a Section 404 DOA permit from USACE. Federal agencies are reluctant to issue a permit that would in some way alter or modify wetlands unless a compelling case can be made that the action will have beneficial effects. A purpose and need statement that would accompany a DOA permit application would need to make a strong case for environmental benefit. Kipnuk is only a few feet above sea level. Even moderate storm surges can have significant impacts, including inundation. A drainage project to remove standing water in Kipnuk could be justified because it could potentially slow down or stop permafrost degradation in the community. Degradation of permafrost will likely result in continuing ground settlement, leading to more severe flooding from storm surges and converting wetlands to ponds. If the purpose and need statement can demonstrate positive environmental impacts (potentially mitigating permafrost degradation) then compensatory mitigation may not be required.

13.0 REFERENCES

Alaska Department of Commerce, Community, and Economic Development (ADCED). 2015. Alaska Climate Change Impact Mitigation Program. Accessed at http://commerce.state.ak.us/dcra/planning/accimp/kipnuk.htm.

Chapman, R.S., S.C. Kim, and D.J. Mark, 2009. Storm-Induced Water Level Prediction Study for the Western Coast of Alaska. Chapman, R.S., S-C Kim, and D.J. Mark. US Army Corps of Engineers, Engineer Research and Development Center Letter Report, October 2009. Vicksburg, MS.

Duane Miller Associates, 2007. Geotechnical Investigation and Engineering Recommendations, Kipnuk Bulk Fuel and Power plant Facility, Kipnuk, Alaska. July 25, 2007.

Federal Highway Administration (FHWA), 1989. Design of Riprap Revetment, Highway Engineering Circular No. 11. U.S. Department of Transportation, March, 1989.

Golder, 2011a. Geotechnical Exploration, Kipnuk Boardwalk Improvements, Phase II. Golder Associates. March 17, 2011.

Golder, 2011b. Chief Paul memorial School Expansion, Kipnuk, Alaska. Golder Associates. June 10, 2011.

Golder, 2011. Hazard Impact Assessment, Kipnuk, Alaska. October 31, 2011.

Hazard Mitigation Plan, 2013. Native Village of Kipnuk Hazard Mitigation Plan. September 6, 2013. The Village of Kipnuk Hazard Mitigation Planning Team.

Julien, P.Y. 2002. River Mechanics. Cambridge University Press. 456 p.

McFadden, 2000. Permafrost Technology Foundation, Design manual for New Foundations on Permafrost. Dr. Terry McFadden, September, 2000.

McFadden and Bennet, 1991. Construction in Cold Regions. Terry T. McFadden and F. Lawrence Bennet. Published by Wiley 1991.

U.S. Army Corps of Engineers (USACE), 2009. Community erosion assessment, Kipnuk, Alaska. 26 January 2009. Alaska District, United States Army Corps of Engineers, Civil Works Branch.

Wise, J.L., A.L. Comiskey, and R. Becker, Jr. 1981. Storm surge climatology and forecasting in Alaska. Arctic Environmental Information and Data Center, University of Alaska, Anchorage.

Zinger, J.A., B.L. Rhoads, and J.L. Best. 2011. Extreme sediment pulses generated by bend cutoffs along a large meandering river. Nature Geoscience 4, 675-678.

Appendix A:
Hydrology and Hydraulic Report

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Hydrologic and Hydraulic Report

for

Kipnuk Engineering Analysis and Design Study

Prepared for:

Kipnuk Traditional Council
PO Box 57
Kipnuk, AK 99614

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Fairbanks, AK 99709

May 2016

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Project Location and Description

The Village of Kipnuk is located on the Kuguklik River 85 miles southwest of Bethel, four miles inland from the Bering Sea. The community is located on the outside bend of the river. Kipnuk faces a number of hazard threats, including: 1) river bank erosion, 2) ground settlement due to permafrost, and 3) flooding. A Hazardous Mitigation Plan (HMP) was prepared by the Village of Kipnuk hazard mitigation planning team in 2013. Based on the hazard threats detailed in the HMP, the Kipnuk Traditional Council requires assistance with determining the most suitable combination of solutions to mitigate the three primary hazard threats listed above.

This report includes an analysis of the hydrologic characteristics of Kipnuk and the Kuguklik River, and a hydraulic analysis of bank erosion and potential erosion protection methods.

Hydrology

Kipnuk is located in a maritime climate, approximately 4 miles from the shore of Kinak Bay on the Bering Sea coast. The coast is bordered by sea ice in the winter, and the surrounding coastal area is treeless and dotted with numerous small lakes. Although the mean annual temperatures are similar to inland sites at the same latitudes, the seasonal range of temperatures is much lower and the winds are much higher. Annual precipitation at Kipnuk averages 22 inches, with 43 inches of snowfall annually. Summer temperatures range from 41 to 57 °F, and winter temperatures average 6 to 24 °F (ADCED, 2009).

The Kuguklik River (also spelled Kugkaktlik) is a meandering stream that originates about 30 miles east in a flat tundra and lakes complex area. Kipnuk is located on an actively eroding bend of the river. The area around Kipnuk is flat and poorly drained with numerous lakes and small drainages that flow into the Kuguklik River (ADCED, 2015).

The channel is tidally influenced. On the rising (flood) tide, flow comes up the Kuguklik River and flows up the channel adjacent to Kipnuk. Following high tide, the ebb tide flows out the tidal channel to the Kinak Bay and the Bering Sea.

Typical of most areas in Alaska, there are no long-term gaging records available for the Kuguklik River. Additionally, there is no NOAA tide gage station at or near Kipnuk. Anecdotal information from the U.S. Corps of Engineers describes the effects of several fall storm surges during the 1970s, and a recent report documents the rate of erosion along the banks of the Kuguklik River (USACE, 2009).

In the Kipnuk area, flooding occurs most commonly from coastal storm surges and runoff from precipitation events. Since no gaging information exists for any nearby streams, precipitation-related flood magnitude estimations were developed using USGS regression equations for estimating the magnitude of peak streamflows in Alaska.

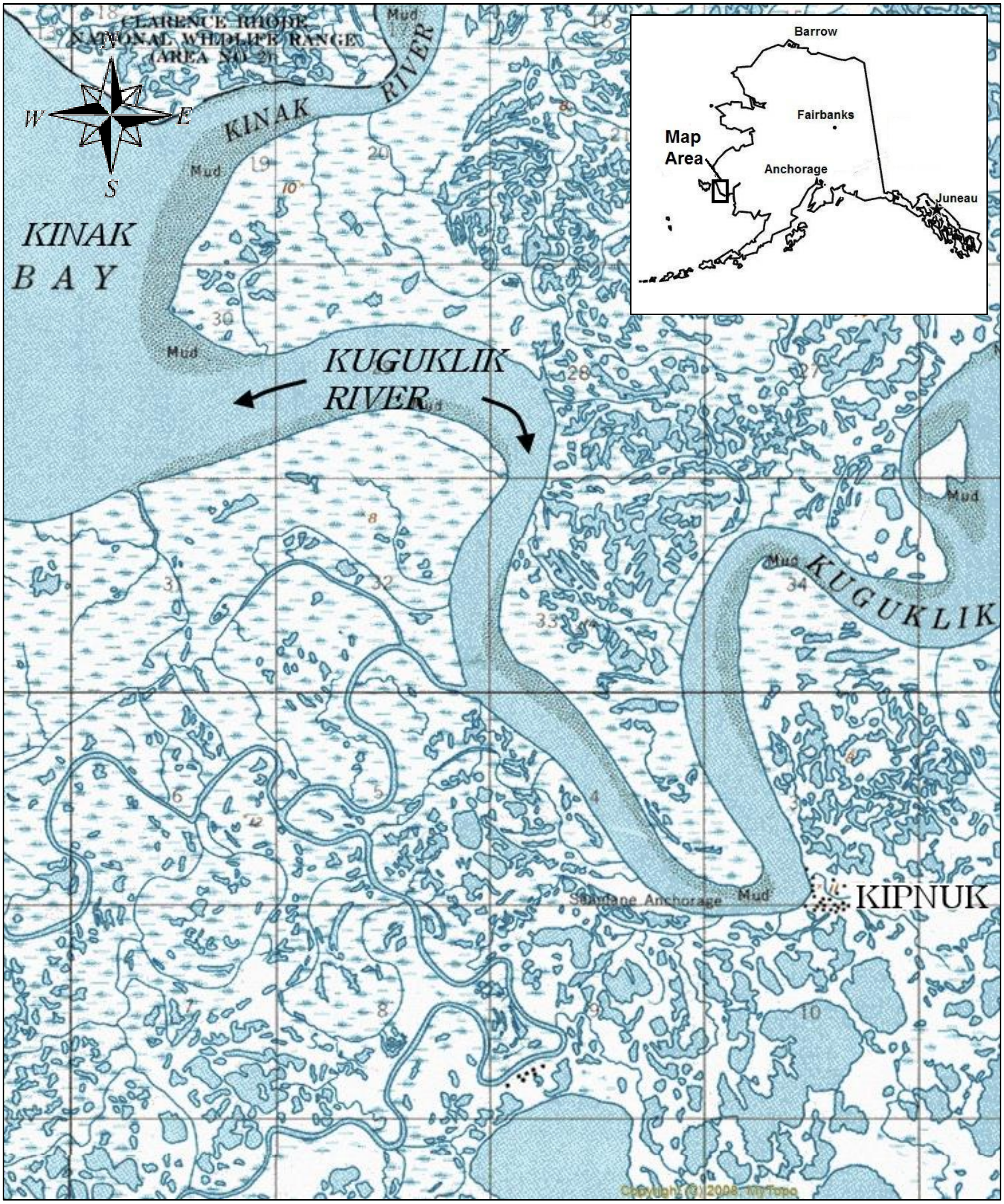


Figure 1. Project location map.

The latest USGS regression method for estimating peak streamflows at ungaged locations is described in the USGS Water Resources Investigations Report 03-4188 (Curran et al., 2003). Basin characteristic information is used in the USGS regression analysis. For Region 6, the characteristics include:

- drainage area upstream from the site,
- percentage of lakes and ponds area,
- percentage of forest areas.

Drainage basin area was obtained from the USGS Watershed Boundary Dataset (USGS, 2015). Other basin characteristics were obtained by planimetric techniques used with USGS 1:63360 quad maps. Due to flat terrain and the ubiquitous presence of lakes, ponds and wetlands, the planimetered basin characteristics in Table 1 should be considered as an approximation.

Table 1. Watershed characteristics.

	Kuguklik River Watershed
Drainage Area (mi^2)	149
Area of Lakes and Ponds (%)	38
Area of Forests (%)	0

The range of the 'lakes and ponds area' variable used to develop the regression equations for Streamflow Analysis Region 6 is 0 to15 %. The percentages of the 'lakes and ponds' areas for the Kuguklik River watershed is significantly larger than the high end of the range. Lakes and ponds act as temporary storage areas during floods, and tend to dampen peak flood magnitudes. Therefore, the peak flood magnitudes for this watershed may be smaller than predicted by the regression equations.

For flooding caused by precipitation events, the estimated magnitudes for the 2-year flood through the 500-year flood for the Kuguklik River watershed are shown in Table 2. The adequacy of the regression equations can be evaluated by several measures. Confidence limits provide a measure of the error in a particular prediction. The 5% and 95% confidence limits provide a 90% prediction interval for a particular site. Because this watershed is ungaged, has limited historic hydraulic information, and has boundaries that are difficult to delineate, the lower and upper confidence limits were calculated and included in Table 2 .

Table 2. Flood discharges based on precipitation events.

Flood Recurrence Interval	Kuguklik River (cfs)	Confidence Limits 5%	Confidence Limits 95%
2-year	1730	823	3640
5-year	2370	1100	5090
10-year	2800	1250	6260
25-year	3330	1390	7980
50-year	3730	1470	9420
100-year	4120	1540	11000
200-year	4500	1590	12800
500-year	5020	1640	15400

Design Flood Elevation

Project designers require the design flood elevation. For many projects, the design flood has a recurrence interval of 100 years, also referred to as having a 1-percent probability of being equaled or exceeded in any given year. In Kipnuk, flooding occurs most commonly from coastal storm surges and runoff from precipitation events. Analyses of both types of floods were conducted to determine the type and water surface elevation of the governing 100-year flood.

Precipitation-Based Flooding

A hydraulic analysis was conducted to determine whether the estimated 100-year precipitation-based flood will result in a higher water surface elevation in the Kuguklik River channel adjacent to Kipnuk than the typical daily high tide elevation. The analysis involves modeling the tidal channel's flow characteristics using the HEC-RAS water surface profile modeling program. The program was used to estimate and compare the discharge in the channel during a non-storm ebb tide flow following high tide to the tidal channel precipitation-based 100-year flood. If the 100-year discharge is less than that of a non-storm ebb tide discharge, that would indicate that precipitation-based floods may not be the correct choice for establishing the design flood elevation.

A numerical model of the tidal channel was constructed in HEC-RAS, using cross-sections surveyed by a project surveyor in May 2015. Five cross-sections, labeled from River Station 0 (units of feet) (downstream) to 7497 (upstream) were used in the model. Station 0 (zero) starts about 4500 feet downstream from the village of Kipnuk. See Figure 2 for cross-section location. Field observations, published tables, and engineering judgment were used to determine estimates of the Manning's n values. The selected n values used in the model are 0.03 (channel) and 0.10 (floodplain).

By matching the observed water surface elevations in the HEC-RAS model, the peak tidal discharge in the channel at the time of the survey was estimated to be between 37000-58000 cfs. See Table 3 for the HEC-RAS results, including hydraulic characteristics at all cross-sections.

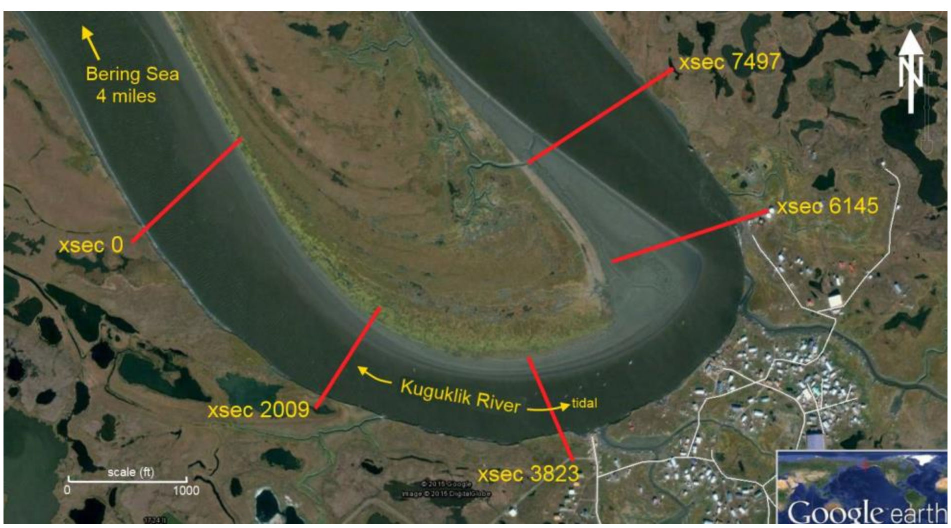


Figure 2. Cross-sections and tidal observation locations for HEC-RAS hydraulic analysis.

A comparison of the estimated flood magnitudes in Table 2 with the channel hydraulic analysis shows that the outgoing tidal channel flow (37000-58000 cfs) was substantially greater than the predicted 100-year (4120 cfs) and 500-year (5020 cfs) floods for the tidal channel watershed. The flow observed during the May survey was considered typical.

Table 3. Results from HEC-RAS analysis of existing tidal channel.

Cross-section	Q total (cfs)	Min Ch El (ft)	W.S. El (ft)	Ave Vel Chnl (ft/s)	Top Width (ft)
0	58000	959.88	984.41	4.67	974.61
2009	58000	953.98	984.89	4.89	645.01
3823	58000	943.44	985.23	5.02	550.29
6145	58000	936.07	985.75	3.28	588.28
7497	58000	961.25	985.80	4.18	1080.78

Tidally affected rivers are characterized by both downstream river flow and tidal fluctuations. Field observations of two-directional flow at the site, along with the HEC-RAS analysis, indicate that the majority of the discharge in the tidal channel is from upstream high-tide storage, not by precipitation-generated flow from the upper watershed. Flood flows and associated water surface elevation increases from precipitation events are likely insignificant compared to daily ebb and flood tide levels and discharges. This indicates that precipitation-based floods are not the correct choice for establishing the design flood elevation.

Storm Surge

Storm surges are temporary abnormal changes in sea level that accompany storms in shallow coastal waters. Impacts to low-lying coastal areas in western and northern Alaska can be significant, as a result of both inundation and increasing the effective height of waves.

Some work on analysis and modeling of storm surges in Alaska has occurred. A statistical model was developed from the Alaska storm surge climatology developed by Wise et al. (1981). Regression analysis was used to correlate surge height with various parameters. For the Kipnuk area (Sector 11), the 50-year surge height is 11.6 feet above mean high water (MHW); the 100-year surge height is 12 feet above MHW.

The U.S. Army Corps of Engineers conducted a storm-induced water level prediction study for the western coast of Alaska (Chapman et al, 2009). The study developed frequency-of-occurrence relationships of storm-generated water levels for 17 selected communities along Kotzebue and Norton Sounds, the Bering Sea, and Bristol Bay, including Kongiganak, Mekoryuk, and Toksook Bay were included in the study. The stage-frequency analysis for these communities is found in Table 4. Stage units are feet mean lower-low water (ft MLLW).

Table 4. Stage-frequency analysis for three locations near Kipnuk, AK.

Location	Distance From Kipnuk (miles)	Return Period (years)				
		5	10	25	50	100
Kongiganak	40	10.31	12.57	15.43	17.03	18.28
Toksook Bay	55	8.45	10.19	11.93	12.98	13.86
Mekoryuk	81	5.53	6.41	7.23	8.19	9.27

The hydrographic parameters influencing the formation of storm surge are a gently sloping seafloor near shore and sufficient open sea to allow for a long fetch (Wise et al., 1981). Near-shore bathymetric data wasn't available for this study.

Residents report that during storm surges, strong winds from the north generate large waves in the Kuguklik River when the channel is full, due to the long fetch of the river. Storm surges create abnormally high water levels in the channel, and the generated waves crash onto the vertical bank next to the Village, resulting in severe bank erosion.

Bank Erosion Analysis

According to an analysis of erosion by the Corps of Engineers, erosion is affecting three different areas in Kipnuk (USACE, 2009):

- Reach 1 is a 1,000 foot reach that starts at the barge landing and stretches downstream. Average erosion rate 7.0 feet per year.
- Reach 2 is 1,500 feet long and stretches from the barge landing to the slough near the tribal offices. Average erosion rate 6.0 feet per year.
- Reach 3 is 1,700 feet long. Average erosion rate 9.0 feet per year.

The COE report attributes the erosion to episodic events related to fall storm surges and related high waves and flooding. However, one or more other causes of erosion are likely contributing to the high rate of erosion, and are discussed below.

Pore-Water Pressure

Positive pore-water pressure can lead directly to streambank erosion and instability. In addition to increasing the weight of the bank, pore-water pressure reduces the effective friction (normal stress) between soil particles, thereby weakening the soil and allowing particles to be dislodged. Bank erosion from positive pore water pressure is commonly attributed to areas with shallow water tables and non-cohesive bank materials such as gravels and sand. However, a short literature review found papers that focus on the importance of accounting for positive and negative pore-water pressures of unsaturated cohesive materials when considering stream stability, bank erosion, and channel widening. Simon and Collison (2001) note that pore-water pressure within cohesive riverbeds will increase during the rising limb of a flood hydrograph (or tidal inflow). If the water level falls rapidly on the receding limb, bed pore-water pressure will also fall, though the impermeability of the soil delays pressure equalization. As a result, upward-directed seepage occurs to eliminate the pressure differential, and leads to rupture and erosion of the streambed, or to partial liquefaction of the upper part of the bed

Similarities between the Simon and Collison study sites and the Kuguklik River bank erosion at Kipnuk include the soil type (silt) and the large rapid variation in the tidal elevations, which occurs approximately every 6 hours. For example, the mean tidal range at the Nelson Island Toksook Bay NOAA subordinate tide station, 55 miles from Kipnuk, is 7.8 feet.

Ice

River ice can contribute to riverbank erosion in several ways. Freeze-thaw cycling and the formation of ice in riverbanks disrupt bank soil structure, which reduces soil strength. This often results in in-situ bank sediment being dislodged and moved downslope via gravity. In fact, numerous Kipnuk residents reported during the field visit in May 2015 that shore ice, frozen fast to the bank during the winter and dislodged at breakup, was the cause of the majority of bank erosion.

Additionally, there are several types of ice movements that can remove and transport in-situ bank soils and accumulated sediments, including: 1) ice retreat from the bank that results in plucking and rafting of material frozen into and lying on the ice, 2) ice thrust into the bank that disrupts the sediments and soils, and 3) ice shear parallel to the bank that abrades and gouges the bank (Gatto, 1993).

Frost Heave Phenomenon

Frozen soils frequently have intermittent layers of ice in the soil mass that range in thickness from barely visible to ten of millimeters or more. Segregation of ice is caused by a thermodynamic imbalance created by the advancing freeze front within the soil. Ice lens formation in fine-grained soils is responsible for frost heave. Three conditions must occur simultaneously for frost heaving to occur, including: 1) a prolonged period of subfreezing temperatures, 2) frost-susceptible soils (silts are more frost-susceptible than either sands or clays), and 3) a source of water.

In general, water moves from warm to cold, from high-moisture zones to low moisture zones, and from regions of low solute concentrations to high solute concentrations. As the soils freeze from the top downward, the thermal gradient will induce an upward flow of water to the freezing front. Freezing of soil water creates a strong sink for water and induces an upward movement of water. The resultant ice lens formation results in frost heaving (Henry, 2000). In addition to an upward displacement of the ground surface, segregated ice lenses may also form vertically in areas of vertical cuts or faces (such as a stream bank). In this condition, the frost heave results in an outward displacement of the vertical face. See Figure 3.

The potential for frost heaving to occur in the vicinity of the Kipnuk area is high. Bethel has a subarctic climate, with a minimum monthly temperature below freezing for 7 continuous months. More importantly, residents that we talked with during the field visit reported that visible ice lenses were commonly observed in the bank of the Kuguklik River in the past. However, the residents reported that ice lenses have either moved lower (relative to the ground surface) or disappeared altogether in recent years.

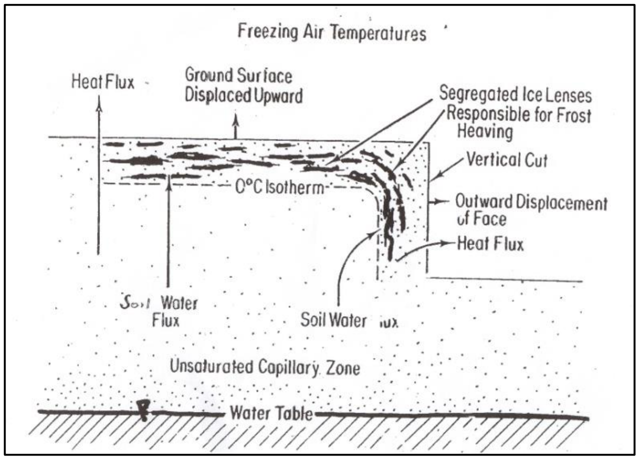


Figure 3. Vertical and horizontal surface displacement from frost heave.

Thermal Degradation

Changing thermal conditions may be responsible for melting permafrost and subsequent bank erosion. Reports documenting the effects of coastal shore erosion from warming or melting permafrost, and thermokarsting (thawing process associated with disturbance of the surface thermal regime in areas of ice-rich permafrost) are readily available. Researchers have noted thermally induced erosion of areas with high ground ice content, including hillslopes and river channels (Rowland et al., 2010).

The University of Alaska Geophysical Institute Permafrost Lab (GIPL) has developed a model specifically to assess the effect of a changing climate on permafrost. The GIPL model calculates the active layer thickness and mean annual ground temperature (Romanovsky and Marchenko, 2015). Changes to permafrost temperatures in Alaska for two time periods are found in Figure 4. The area of southwest Alaska in general, and the coastal ecosystems on the Yukon-Kuskokwim Delta in particular, are at high risk for increased soil temperatures and melting permafrost.

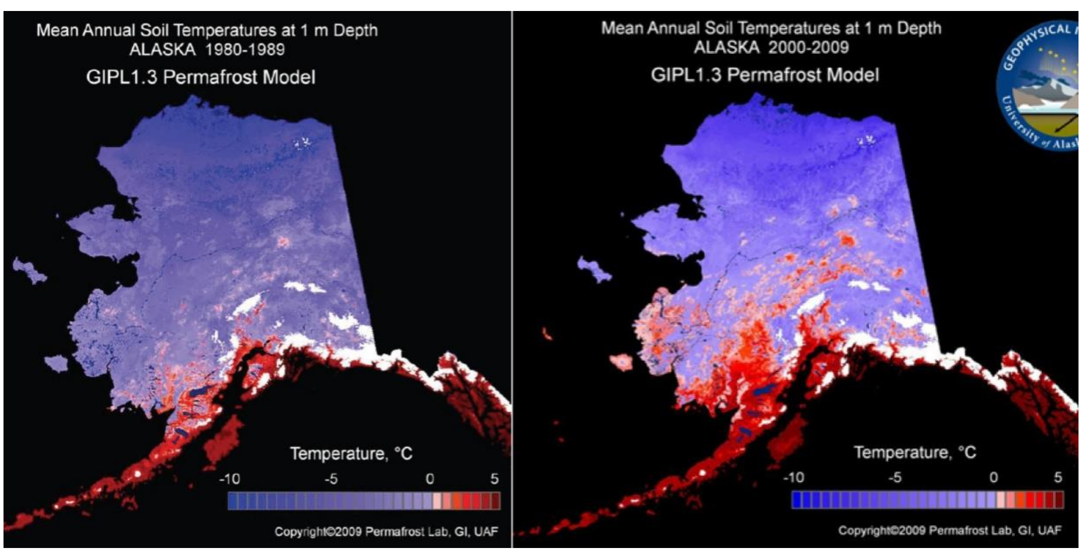


Figure 4. Change in ground temperatures. From UAF (2015).

Boat Wash

Water movements generated by boat motion have long been recognized as a contributing factor to bank erosion. In addition to waves propagating from the bow of a boat underway and impacting the bank, currents produced by ship propellers have also been associated with bank and channel erosion, In fact, a report describing the erosion at Kipnuk noted that "local erosion may also occur at the barge landing and fuel terminal due to prop wash if the barge keeps its screws turning while unloading" (USACE, 2009).

During our May field trip to Kipnuk, the fuel barge arrived in the evening and offloaded fuel and other supplies. The articulated tug-barge Cavek and AVEC-208 is large: the tug is 76 feet in length, and the barge is 208 feet in length. The tug is powered by three diesel engines delivering a total of 1800 horsepower. The engines drive 40-inch propellers in nozzles (Professional Mariner, 2015). Though the barge is designed to pin itself against the shore in fast moving rivers, we noted continuous prop wash occurred during maneuvering and the entire period that the barge was stationary and offloading. See Figure 5 and 6.

Figure 5. The Avec-CAVEK 208 fuel barge offloading at Kipnuk, May 2015.

Figure 6. Prop wash from the Avec barge, while stationary against the bank.

Wave Runup and Erosion

Wave runup is the maximum vertical extent of wave uprush on a beach or structure above the still water level. Wave runup may cause erosion by directly impacting the bank, dislodging material, and redistributing it to the foreshore and nearshore. Wave runup promotes bank erosion by carrying failed bank material away from the bank.

At Kipnuk, strong winds from the north can occur during storm surges. The winds generate large waves in the Kuguklik River when the channel is full, due to the long fetch of the river. Storm surges create abnormally high water levels in the channel, and the generated waves crash onto the vertical bank next to the Village, resulting in severe bank erosion.

Scour

Bank erosion at the outside bend of the Kuguklik River is likely exacerbated by significant scour occurring in the channel bed in front of the bank. In meandering natural rivers, curved flow will generate secondary flow patterns, which will cause scouring on the outside of a bend. Channel bed scour in this reach is large.

To help define the extent of the scour hole in the Kuguklik River, the cross-section survey conducted in May 2015 included multiple shots of the channel bed thalweg. A profile of the river channel, including the thalweg and adjacent bank, is found in Figure 7. A contour map of the scour hole is found in Figure 8.

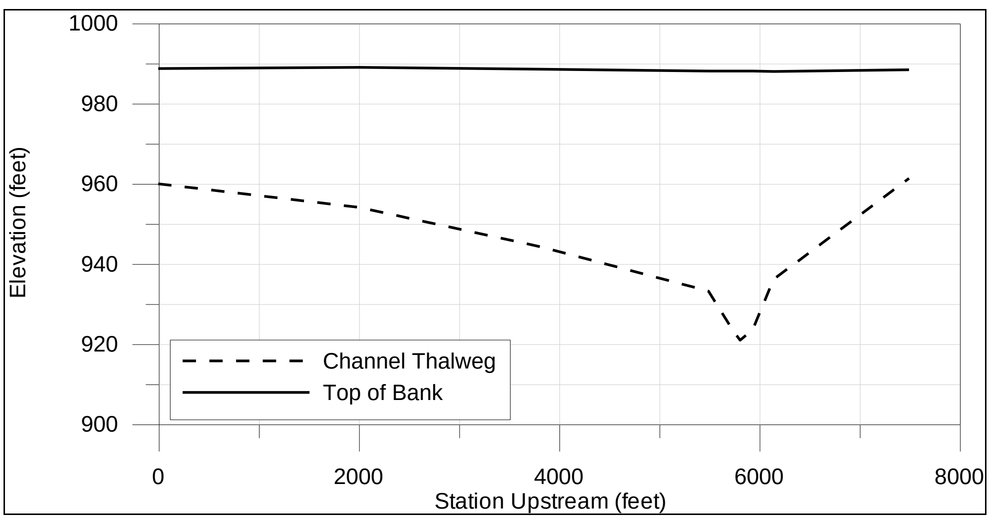


Figure 7. Profile of Kuguklik River channel at Kipnuk.

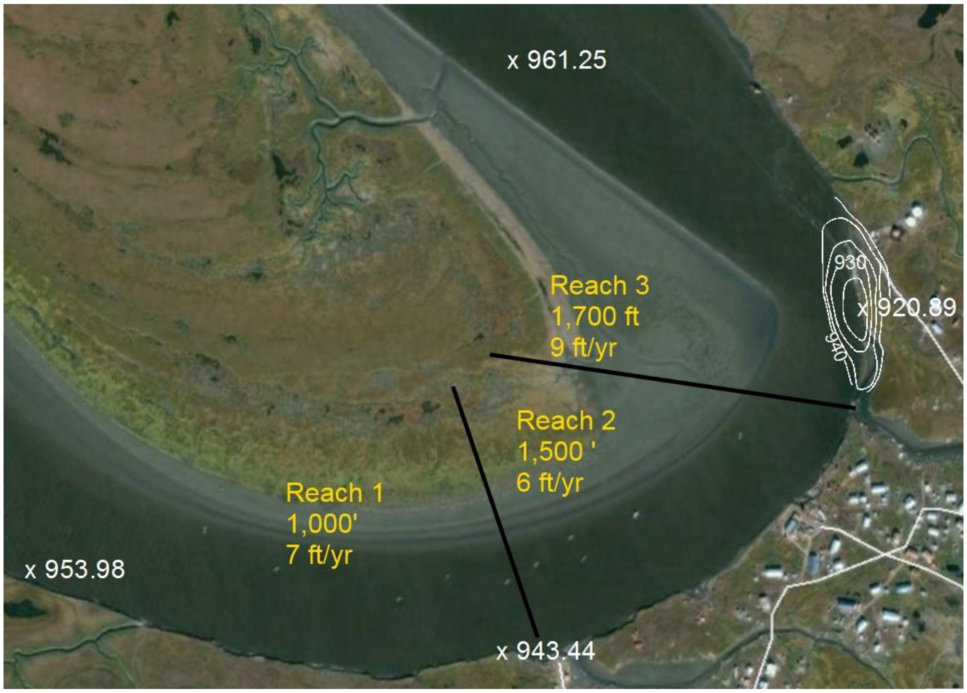


Figure 8. Rates of bank erosion for three reaches, and surveyed scour hole near Kipnuk.

Figure 8 includes the rates and locations of bank erosion at three reaches delineated by the Corps of Engineers (USACE, 2009), and channel thalweg elevations upstream and downstream of the scour hole. Note that the scour hole is located in the reach identified as having the highest bank erosion rate. Also note that the scour hole is in the narrowest part of the channel, and where the large bend has the smallest radius of curvature.

Hydraulic Design Considerations

The objective of this project is to analyze the hydrologic characteristics of the Kuguklik River, and develop methods to reduce or eliminate bank erosion at Kipnuk. The assessment of the bank erosion problem indicates that severe and accelerated bank erosion has occurred in the recent past, is ongoing, and will likely continue to be a characteristic trait of this river reach for the foreseeable future.

Most bank erosion protection techniques may be classified as either a resistive method or a redirective method. Resistive methods, such as riprap or sheetpile, are designed to protect against shear stress, and are generally used in a continuous application. Redirective techniques, usually discontinuous, are techniques that redirect the flow and energy of the river or stream away from the area of the eroding bank (McCullah, 2004). Examples of redirective techniques include rock vanes and bendway weirs. Such in-stream structures pose a threat to both large and small boats from boat hull strikes, and therefore were not considered suitable for use in this segment of the Kuguklik River.

The excessive height and steep angle of the eroding bank along the Kipnuk reach create unique and challenging conditions for designers considering remedies. Channel shape and estimated velocities are important characteristics when designing bank protection. Results from the HEC-RAS analysis are found in Appendix 2, including velocity distribution across the cross-section. Note that the flow distribution results should be used cautiously, as they are based on a one-dimensional model. A true velocity and flow distribution varies vertically as well as horizontally (USACE, 2010).

Channel Scour Calculations

One of the most common causes of bank revetment failures is from floods scouring bed material from the toe of the revetment structure. With no information or data on long-term bed elevation changes in the Kuguklik River, a scour analysis of the channel at Kipnuk focused on general scour.

> *General Scour:* Flow contraction at a revetment project may result in removal of material from the bed across all or most of the channel width. Other general scour conditions include flow around a bend where the scour may be concentrated near the outside of the bend. General scour is different from long-term degradation in that general scour may be cyclic and/or related to the passing of a flood (USDA, 2007).

The numerical computation of scour requires site specific stream data, including the bed material D_{50}, design discharge, and top width and hydraulic radius at the design discharge, which are derived from a hydraulic analysis. Results from the HEC-RAS modeling analysis were used to provide the required hydraulic values for the scour calculations. Soil parameters were provided by AECOM, and are listed in Appendix 3.

Several methods were used to estimate general scour, including Zeller, Lacey, Blench, Neill, Maynord Bend, Thorne Bend, COE bend, and USBR (PBS&J, 2008). The Lacey, Blench, Neill, Maynord, Thorne, and COE equations also include a bend scour component in the calculations. Results were averaged to provide a mean estimated scour depth. The selected design discharge for scour estimation was 58,000 cfs.

The upstream radius 'R' of the river bend at Kipnuk was estimated at 1130 feet. With a design channel top width 'T' of 640 feet, the degree of curvature 'R/T' is 1.8, which is classified as a severe bend.

Calculations included the D_{50} and D_{90} for two bed material types, sand and silt. Results are found in Table 5. Predicted scour elevations, and the existing scour hole elevation between Sections 3823 and 6145 are found in Table 6.

Table 5. Estimated scour depths below streambed (ft).

Cross-section & Grain Size	USBR Mean Velocity	Lacey	Neill	Blench	Zeller Bend	Maynord Bend	Thorne Bend	COE Bend	Mean
6145 – silt	22.58	22.58	40.4	na	8.01	17.31	na	na	22.18
6145 - sand	22.58	22.58	40.4	12.85	8.01	17.31	na	na	20.62
5371* – silt	16.58	16.58	33.87	na	6.76	25.88	na	na	19.93
5371* - sand	16.58	16.58	33.95	11.53	6.76	25.88	na	44.66	22.28
4597* - silt	15.64	15.64	34.51	na	6.76	25.88	na	na	19.69
4597* - sand	15.64	15.64	34.51	12.15	6.76	25.88	na	45.7	22.33
3823 - silt	15.75	15.75	32.94	na	5.83	25.16	34.84	na	21.71
3823 – sand	15.75	15.75	32.94	13.44	5.83	25.16	34.84	41.48	23.15
2009 – silt	13.8	13.8	23.55	na	2.94	14.67	20.5	na	14.88
2009 - sand	13.8	13.8	23.55	12.08	2.94	14.67	20.5	19.6	15.12

Table 6. Estimated scour elevations at cross-section (ft).

Cross-section & Grain Size	Thalweg (ft)	Mean Scour Depth	Predicted Scour Elevation	Existing Scour Hole Elevation Between 3823 & 6145
6145 – silt	936.07	22.18	913.89	921.0
6145 - sand	936.07	20.62	915.45	
5371* – silt	938.53	19.93	918.6	
5371* - sand	938.53	22.28	916.25	
4597* - silt	940.98	19.69	921.29	
4597* - sand	940.98	22.33	918.65	
3823 - silt	943.44	21.71	921.73	
3823 – sand	943.44	23.15	920.29	
2009 – silt	953.98	14.88	939.10	
2009 - sand	953.98	15.12	938.86	

Riprap Revetment

Rock riprap is the most widely used type of revetment in the United States (FHWA, 1989), and has been used by the US Corps of Engineers in one location on the Kuguklik River near the project location to protect the embankment at Kipnuk. The advantages of riprap include the following:

1. The riprap blanket is flexible and is not impaired or weakened by minor movement of the bank caused by settlement or other minor adjustments.
2. Local damage or loss can be repaired by placement of more rock.
3. Construction is not complicated (FHWA, 1989).

Installed riprap protects a bank from the stresses of higher velocity water along it. This armoring is not meant to alter the flow of the river, but typically does cause some local scour. In addition, riprap usually provides protection only to the section of bank that is armored (NRCS, 2004).

Riprap can be very expensive. For example, freight/haul costs can significantly affect the cost of these revetments.

Riprap design is well-established. River characteristics needed for riprap design include:

- design flood discharge and elevation,
- velocity, bed material gradation,
- scour depth excavation,
- cross-section geometry.

Riprap Gradation

Two methods were used for a preliminary riprap size analysis, including HEC-11 (FHWA, 1989) and the USACE method. Values for the average depth of flow and average velocity at the 58,000 cfs design discharge were developed from the HEC-RAS analysis. The HEC-11 results were more conservative, and are listed in Table 7 in a Corps of Engineers gradation.

Table 7. Preliminary HEC-11 riprap gradation analysis.

FHWA Gradation, Gradation Class Facing, Layer Thickness 1.9 ft		
Percent Smaller By Size	Rock Size (ft)	Rock Size (lbs)
D100	1.3	200
D50	0.95	75
D10	0.40	5

The Corps of Engineers provides typical riprap gradations, with max and min limits. The gradation that must closely agrees with the FHWA gradation for the riprap analysis is:

Table 8. Corps of Engineers riprap gradation.

Layer thickness, ft	W_{100}, lb		D_{100}, ft		W_{50}, lb		D_{50}, ft		W_{15}, lb		D_{15}, ft		D_{30}, ft	D_{90}, ft
	max	min	max	min	max	min	max	min	max	min	max	min	min	min
1.75	463	185	1.75	1.29	137	93	1.17	1.02	69	29	0.93	0.70	0.85	1.23

The COE gradation most closely agrees with a State of Alaska Class II riprap specification. Preliminary analysis shows a 2-foot layer of Class II riprap should be sufficient for everything lower than tidal LLW, if the riprap extends all the way down to the lowest elevation predicted by the scour calculation. However, if the blanket cannot be placed at the scour elevation, then the riprap launching apron should be used. See details below.

Granular Filter for Riprap Layer-Preliminary Gradation

The filter is a transitional layer to prevent the migration of the fine soil particles through the riprap structure, and permit relief of hydrostatic pressures within the soils. If openings in the filter are too large, excessive flow piping through the filter can cause erosion and failure of the bank material below the filter.

For rock riprap, the design method involves determining the filter ratio, which is defined as the ratio of the D_{15} of the coarser layer to the D_{85} of the finer layer. An additional requirement for stability is that the ratio of the D_{15} of the coarser material to the D_{15} of the finer material should exceed 5 but be less than 40:

$$\frac{D_{15}\text{ (coarse layer)}}{D_{85}\text{ (finer layer)}} < 5 < \frac{D_{15}\text{ (coarse layer)}}{D_{15}\text{ (finer layer)}} < 40$$

Based on previous geotechnical studies, a typical cross-section at the Kugiklik River bank for the analysis was simplified into the following layering:

- 0 to 45 feet – Silt
- 45 to 100 feet – Silty Sand (fine grained)

For purposes of preliminary design estimates, the following soil parameters are assumed for the riverbank:

D_{15} = 0.3 mm D_{85} = 1.0 mm (from USDA, 1994)

Based on gradation estimations of the bank silt and Class II riprap, the suggested filter layer has the following gradation:

D_{15} = 5 mm D_{85} = 50 mm

Additionally, to prevent gap-grading, the material should meet the following coefficient of uniformity requirement:

$$CU = \frac{D_{60}}{D_{10}} \leq 6$$

The minimum thickness of a filter layer is 6 inches. In moving water where large riprap particles will be used, the filter layer should be 12-15 inches in thickness.

Note that typically, project specifications call for a 50% increase in layer thickness if the riprap is to be placed underwater. Construction of riprap structures under the water line is always problematic because of depth of water and direction and magnitude of the current.

Riprap Launching Apron

A launching toe/launching apron is a practical alternative to excavating the bottom of the channel and placing riprap down to the lowest elevation predicted by the scour equation. Where additional bed scour is anticipated at or near the toe of the bank, an extra quantity of rock is placed on the streambed next to the toe. This extra rock forms a thick apron, the intent of which is to progressively launch riprap into the scour zone as scour is occurring. Some reports indicate stones in the overlying layer have been oversized. See Figure 9.

Table 9 includes preliminary volume estimates at each cross-section for the Class II riprap launch apron, which is calculated as the volume required to install the riprap blanket down to the estimated scour elevation.

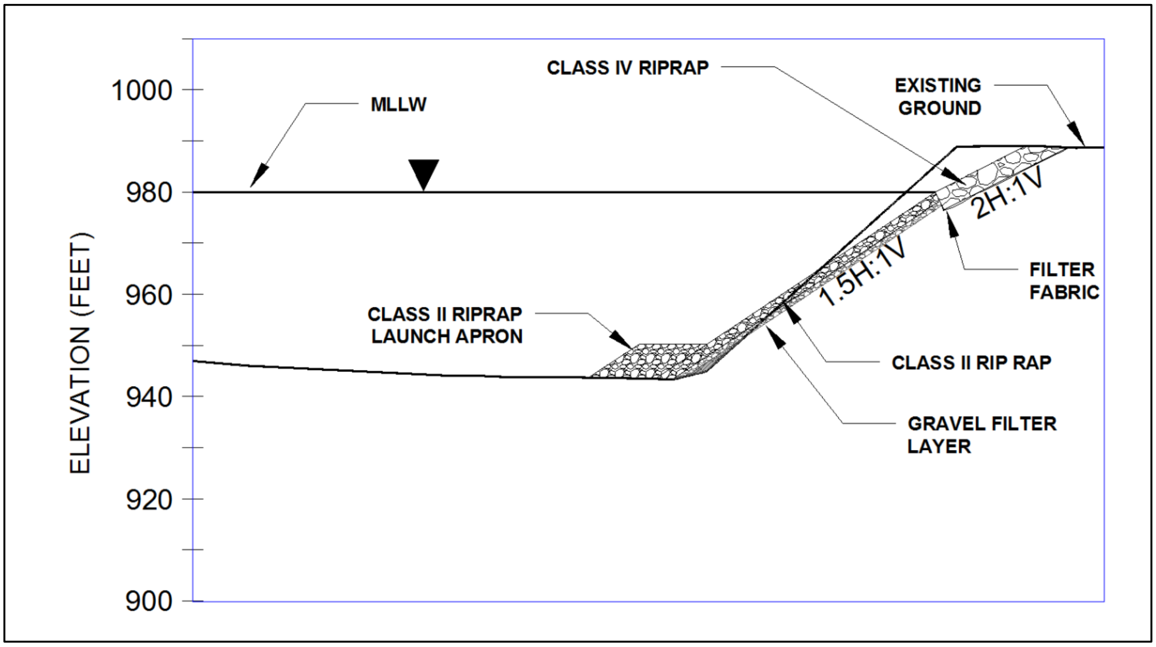


Figure 9. Typical riprap launch apron; may vary in height and width. Additional Class IV riprap for ice plucking protection.

Table 9. Preliminary estimates for riprap launch apron.

Cross-section	2009	3823	4597	5371	6145
Volume Riprap Launch Toe (yd^3/yd)	9.1	8.9	6.1	8.4	9.4
Additional Riprap for Underwater Placement (yd^3/yd)	4.6	4.5	3.1	4.2	4.7
Total Riprap Volume (yd^3/yd)	13.7	13.4	9.2	12.6	14.1

Assumes slope at or laid back to 1.5:1 H:V.
Riprap blanket thickness for bank slope 1.9 ft.
Additional riprap for uncertainties with underwater placement is 50%.
All volume estimates in units of cubic yard per linear yard.

Added Protection From Storm Waves and Ice Plucking

As noted earlier, larger riprap or articulated concrete blocks may be beneficial to protect the upper bank from storm waves and ice plucking. Available guidance (CRREL, 1996) indicates that the maximum size (D_{100}) of rocks should be twice the ice thickness for shallow slopes (1V:3H), or three times the ice thickness for steeper slopes (1V:1.5H).

This would indicate Class IV riprap. ADOT&PF specifications - 50-100% weighing 2000 pounds or more, approximately 2.5-4 ft.

The Class IV riprap would require a thicker layer of the Class II underlying riprap to support it on a slope. The thickness would have to match the maximum D_{100} diameter of the larger riprap. The Class IV riprap band should extend vertically from MLLW (Mean Lower Low Water-average height of the lowest tide) to the top of the bank.

In lieu to the thicker underlayer, the upper slope could be laid back to a shallower slope to reduce the required size and/or offer additional stability. See Figure 9.

Summary

The Kuguklik River is a tidally-influenced meandering stream that originates about 30 miles east in a flat tundra and lakes complex area. Kipnuk is located on an actively eroding bend of the river. The area around Kipnuk is flat and poorly drained with numerous lakes and small drainages that flow into the Kuguklik River.

USGS regression equations were used to develop a flood frequency analysis for the Kuguklik river at Kipnuk. However, due to watershed characteristics that are outside of the range of those used to develop the regression equations, the actual flood magnitudes may be much smaller than predicted by the regression equations. Field observations of two-directional flow at the site, along with the HEC-RAS analysis, indicate that the majority of the discharge in the tidal channel is from upstream high-tide storage, not by precipitation-generated flow from the upper watershed. Flood flows and associated water surface elevation increases from precipitation events are likely insignificant compared to daily ebb and flood tide levels and discharges. In addition, large storms in western Alaska and their accompanying storm surges are likely responsible for the largest flood events in and around Kipnuk.

The bank of the Kuguklik River is eroding along the entire length of the community at Kipnuk. The average erosion rate is estimated at between 6 to 9 feet per year, depending on location. Possible reasons for the ongoing bank erosion include: shear stress, pore-water pressure, thermal degradation, river ice, boat wash, wave runup, and channel scour. A 40-foot deep scour hole exists in the channel just upstream of the community.

An analysis was conducted to determine the hydraulic characteristics of the Kuguklik River channel at various flow levels. Cross-sections on the Kuguklik River channel were surveyed in May 2015 and used to create a numerical model of the channel. The model was used to predict water velocities and channel widths at the outgoing tidal bankfull discharge flow. Results show that high velocity rates are present in the channel.

Based on the results of the HEC-RAS analysis, hydraulic design considerations were presented. Preliminary calculations to predict general scour in the channel were developed. Estimated scour depths range from 15 to 23 feet below the channel bottom. As hardening a bank can lead to increased bed scouring, incision, and a shift of the thalweg toward the outside bend, the large scour depths must be considered when designing bank revetment.

Preliminary riprap revetment requirements for rock sizing, filter layers, launching apron dimensions, and added protection from storm waves and ice plucking were developed. Class II riprap will likely provide adequate protection for the bank below MLLW with a revetment slope of 1.5H:1V. As noted earlier, Class IV riprap or articulated concrete blocks may be beneficial for protecting the upper bank above MLLW from storm waves and ice plucking. Additional data and calculations are required for final design.

References

Alaska Department of Commerce, Community, and Economic Development (ADCED). 2009. Alaska Community Information Database Community Information Summaries. Accessed at http://commerce.state.ak.us/dnn/Portals/4/pub/Kipnuk_Final%20Report.pdf.

Alaska Department of Commerce, Community, and Economic Development (ADCED). 2015. Alaska Climate Change Impact Mitigation Program. Accessed at http://commerce.state.ak.us/dcra/planning/accimp/kipnuk.htm.

Chapman, R.S., S-C Kim, and D.J. Mark. 2009. Storm-induced water level prediction study for the Western Coast of Alaska. US Army Corps of Engineers, Engineer Research and Development Center Letter Report, October 2009. Vicksburg, MS.

Cold Regions Research & Engineering Laboratory (CRREL). 1996. Ice action on riprap small scale tests. US Army Corps of Engineers CRREL Report 96-12. Authors D.S. Sodhi, S.L. Borland, and J.M. Stanley.

Curran, J. H., D. F. Meyer, and G.D. Tasker. 2003. Estimating the magnitude and frequency of peak streamflows for ungaged sites on streams in Alaska and conterminous basins in Canada. U.S. Geological Survey, Water-resources investigations report 03-4188. Anchorage, AK.

Federal Highway Administration (FHWA). 1989. Hydraulic Engineering Circular No. 11, Design of riprap revetment.

Gatto, L.W. 1993. Riverbank conditions and erosion in winter. Proceedings of the workshop on environmental aspects of river ice. T.D. Prowse, editor, National Hydrology Research Institute, Saskatoon, Saskatchewan, NHRI Symposium Series No. 12, p. 43-56.

Henry, K.S. 2000. A review of the thermodynamics of frost heave. Technical Report ERDC/CRREL TR-00-16. US Army Corps of Engineers. September 2000.

National Oceanic and Atmospheric Administration (NOAA). 2012. Tidal station locations and ranges. Accessed at http://tidesandcurrents.noaa.gov/tides08/tab2wc2c.html.

Post, Buckley, Schuh & Jernigan (PBS&J) Scour Analyses Spreadsheet, 2008, Version 1.2.

Professional Mariner. 2015. Specially designed shallow-draft ATBs head north to serve remote Alaskan villages. Accessed at http://www.professionalmariner.com/American-Tugboat-Review-2012/Specially-designed-shallow-draft-ATBs-head-north-to-serve-remote-Alaskan-villages.

Rowland, J. C., C. E. Jones, G. Altmann, R. Bryan, B. T. Crosby, G. L. Geernaert, L.D. Hinzman, D. L. Kane, D. M. Lawrence, A. Mancino, P. Marsh, J. P. Mcnamara, V.E. Romanovsky, H. Toniolo, B. J. Travis, E. Trochim, and C. J. Wilson. 2010. Arctic landscapes in transition: responses to thawing permafrost. Eos, Transactions, AGU, 91, 229-236.

Simon, A, and A. Collinson. 2001. Pore-water pressure effects on the detachment of cohesive streambeds: seepage forces and matric suction. Earth Surf. Process. Landforms. 26, 1421-1442. John Wiley & Sons, Ltd.

US Army Corps of Engineers (USACE). 2009. Community erosion assessment, Kipnuk, Alaska. 26 January 2009. Alaska District Corps of Engineers Civil Works Branch.

US Army Corps of Engineers (USACE). 2010. HEC-RAS River analysis system. Hydraulic reference manual, version 4.1, January 2010. CPD-69.

US Geological Survey (USGS). 2015. Watershed boundary dataset. Accessed at http://nhd.usgs.gov/wbd.html.

University of Alaska (UAF). 2015. Permafrost laboratory. Accessed at http://www.permafrost.gi.alaska.edu/content/modeling.

Western Regional Climate Center (WRCC). 2012. Bethel WSO Airport, Alaska (500754), 1981-2010 monthly climate summary. http://www.wrcc.dri.edu/cgi-bin/cliMAIN.pl?ak0754.

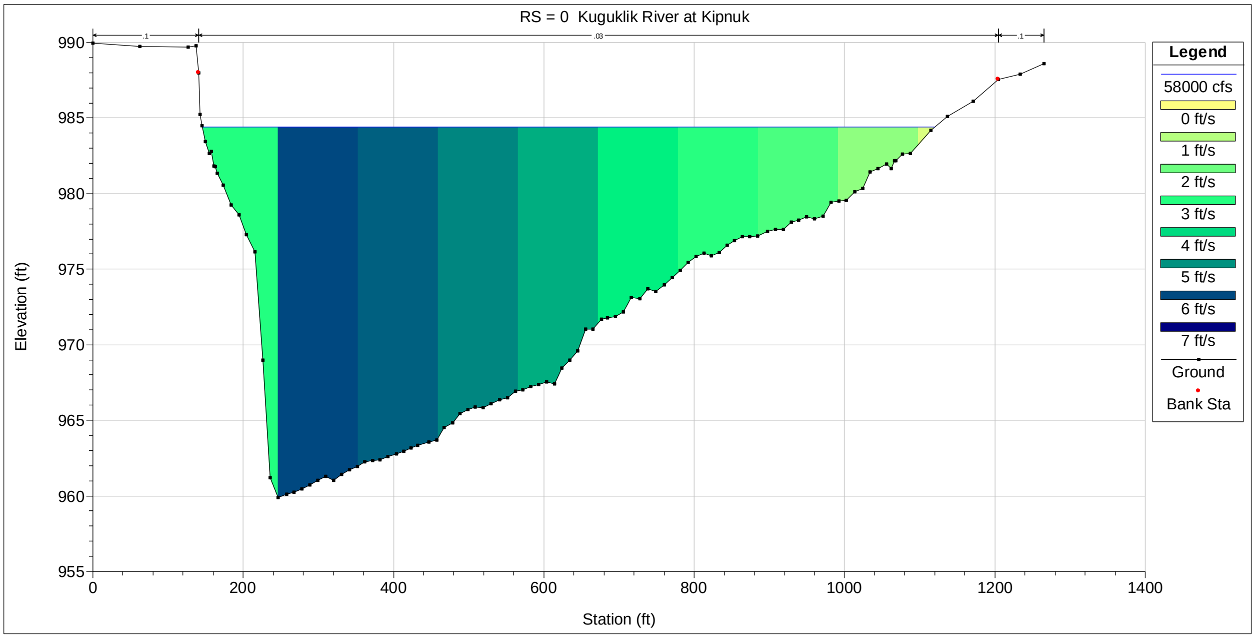
Wise, J.L., A.L. Comiskey, and R. Becker, Jr. 1981. Storm surge climatology and forecasting in Alaska. Arctic Environmental Information and Data Center, University of Alaska, Anchorage.

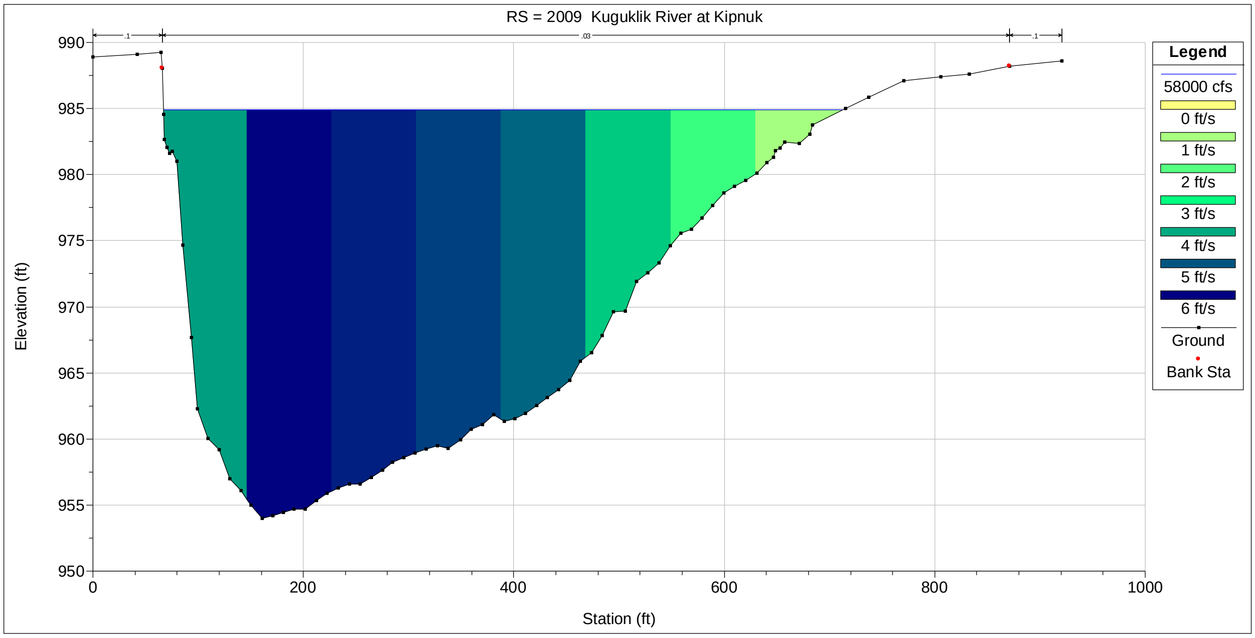
Appendix 1-Flood Magnitudes

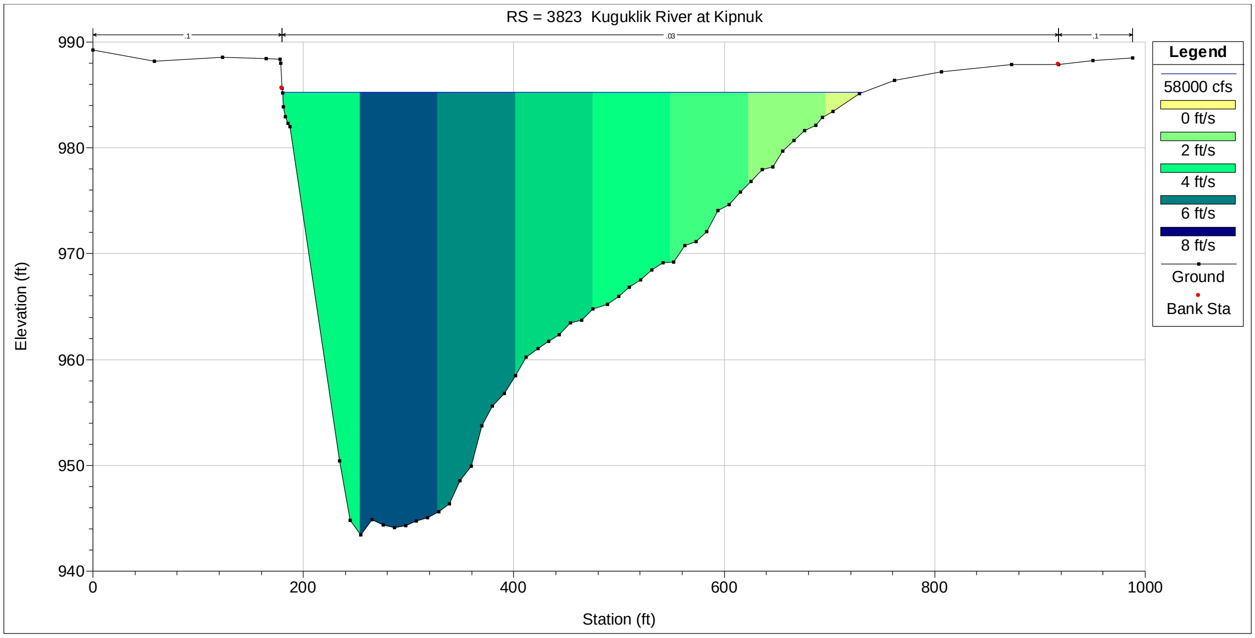
```
This program computes estimates of T-year floods
 for ungaged sites in Alaska based on the
 report "Estimating the Magnitude and Frequency of Peak
 Streamflows for Ungaged Sites on Streams in Alaska and
 Conterminous Basins in Canada", WRIR 03-4188
 See the above publication for equations
* No warranty, expressed or implied, is made by the
* USGS as to the accuracy and functioning of the
* program and related program material.
  VERSION 10/04/03
 ++++++++++++++++++++++++++++++++++++++++++++++++++


 Flood frequency estimates for
 Site: Kuguklik River At Kipnuk
 Region 6
 Drainage area,in square miles:                          149.00
 Percent of area in lakes and ponds:                       38.0
 Forest cover, in percent:                                  0.0

    T     DISCHARGE    SE (+%)     SE(-%)     CONFIDENCE LIMITS   EQ. YEARS
            (cfs)                                  5%          95%
     2       1730.       56.0      -35.9        823.        3640.       1.1
     5       2370.       58.0      -36.7       1100.        5090.       1.5
    10       2800.       62.1      -38.3       1250.        6260.       1.9
    25       3330.       68.7      -40.7       1390.        7980.       2.3
    50       3730.       74.3      -42.6       1470.        9420.       2.5
   100       4120.       80.2      -44.5       1540.       11000.       2.7
   200       4500.       86.5      -46.4       1590.       12800.       2.8
   500       5020.       95.2      -48.8       1640.       15400.       3.0
 WARNING - Lakes+Ponds out of range of observed data
 Range:    0.00 to   15.00 for Region 6
```

Appendix 2-HEC-RAS Cross-sections with Velocity Distribution

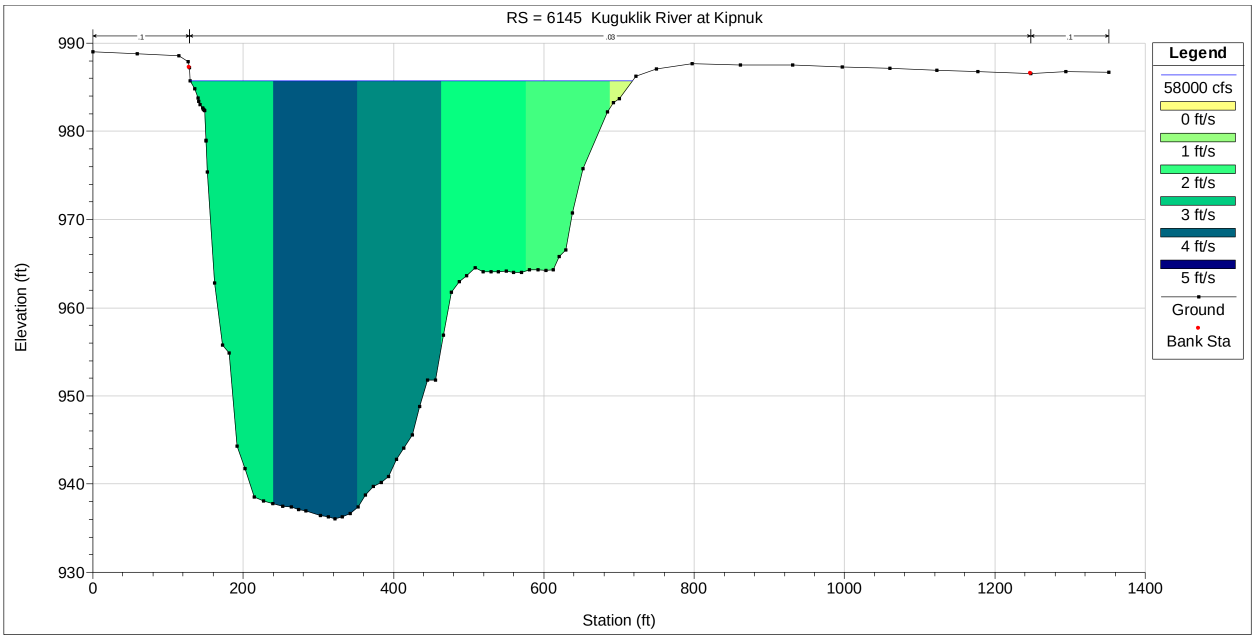
All cross-section stationing increases left to right looking downstream. See Figure 2 for cross-section location.



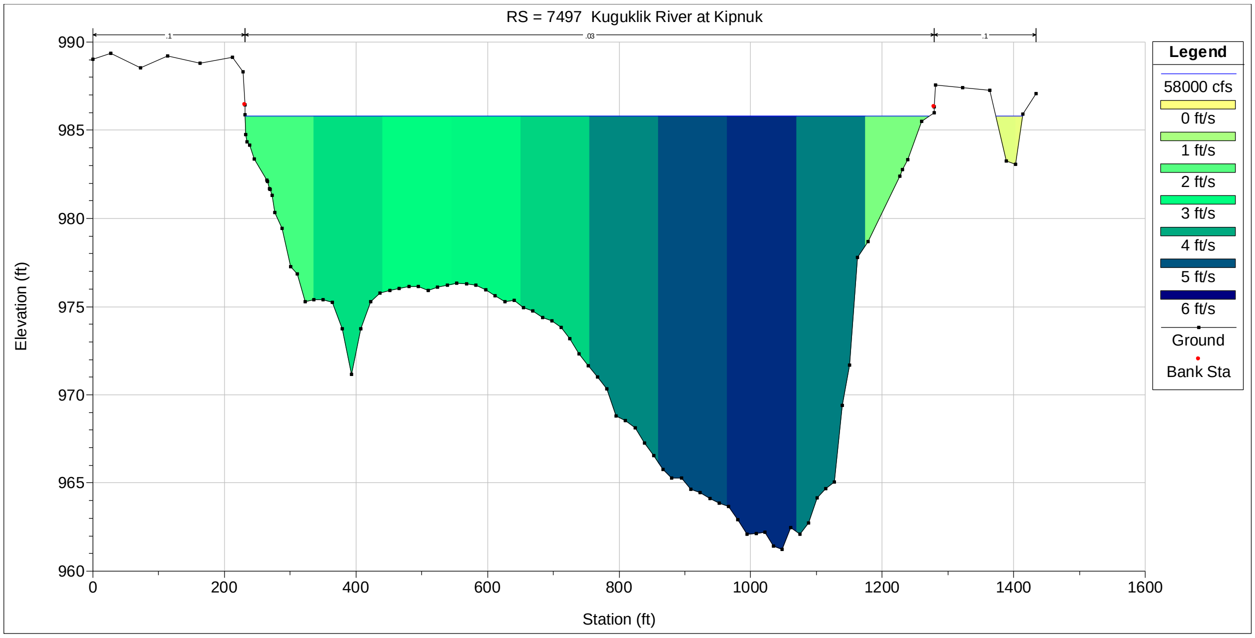




RS = 3823 Kuguklik River at Kipnuk
Legend
58000 cfs
0 ft/s
2 ft/s
4 ft/s
6 ft/s
8 ft/s
Ground
Bank Sta
Elevation (ft)
Station (ft)



RS = 6145 Kuguklik River at Kipnuk
Legend
58000 cfs
0 ft/s
1 ft/s
2 ft/s
3 ft/s
4 ft/s
5 ft/s
Ground
Bank Sta
Elevation (ft)
Station (ft)


RS = 7497 Kuguklik River at Kipnuk
Elevation (ft)
Station (ft)
Legend
58000 cfs
0 ft/s
1 ft/s
2 ft/s
3 ft/s
4 ft/s
5 ft/s
6 ft/s
Ground
Bank Sta

Appendix 3-Soil Parameters for Scour and Riprap Calculations

(Following information provided by AECOM)

Numerous geotechnical studies have been performed in the village of Kipnuk and surrounding areas however geotechnical borings along the Kugiklik riverbank and proposed alignment of the sheetpile wall have not been performed at this date. The geologic condition (typical cross-section and material parameters) along the Kugiklik riverbank have been assumed based on information collected from these previous studies. The material parameters are considered appropriate for the "feasibility" level design, used during this study. Additional geotechnical investigations along the proposed wall alignment are recommended to confirm assumptions of the material parameters and the geologic cross-section of the subsurface materials during final design.
The following geotechnical studies and information were used during the development of the geologic conditions along the Kugiklik riverbank:

- **Boardwalk Improvements, Phase II (Golder, 2011a):** Shallow borings performed for the Boardwalk Improvements, Phase II (Golder, 2011). Total of 43 test holes, 7 performed close to the river bank. Hand auger used to advance through active zone generally to the top of permafrost.
- **Kipnuk Bulk Fuel and Powerplant Facility (Duane Miller 2007):** Borings located approximately 700 feet to the northeast of the riverbank were drilled to depths of 48 to 67 feet. Split Barrel and Shelby Tube samplers were used to collect soil samples for laboratory testing.

  Twelve moisture/density tests were performed on silty to fine sandy materials. The dry density of the samples ranged from 70 pcf (silt) to approximately 97 pcf (silty SAND). Four Direct Shear Tests performed on fine sandy materials and two Unconsolidated Undrained Triaxial (TXUU) tests performed on siltier materials. Results indicate the materials were non-plastic. The direct shear tests estimated the internal friction angle ranged between 30 to 35 degrees for three in-situ samples and was estimated to be 26 degrees for one remolded sample. The two TXUU results estimated the undrained shear strength of the inorganic silts was approximately 600 to 750 psf.
- **Chief Paul Memorial School Expansion (Golder, 2011b)**: Borings located approximately 900 feet to the southeast of the riverbank were drilled to depths of 21 to 70 feet. Direct push sampling equipment was used to collect disturbed samples of the non-organic silt and fine sand materials for laboratory testing.

  Two Direct Shear tests performed on remolded samples and estimated the internal friction angle was 38 degrees (silt) and 34 degrees (sand). As the results of the silt are much higher than published literature values of this material, a lower value of 34 degrees was used by Golders in their engineering analysis.

Permafrost:
Permafrost noted in previous borings. Permafrost conditions vary widely across the community.
Typical cross-section
Subsurface layering was generalized into the following layers based on the available information from previous borings. The upper 5 feet is considered to be an active zone, with intermittent frozen soil below this elevation. The soil descriptions for each generalized layer are summarized below.

- 0 to 1.5 feet depth: Loose, wet, dark brown, PEAT, (from drill logs, Golder 2011a)
- 1.5 to 5 feet depth: Loose, wet, brown SILT, visible ice lenses (1-2mm), (from drill logs Golder 2011a & Duane Miller Associates 1999).
- 5 to 45 feet depth: wet, gray to black, medium stiff to stiff, sandy SILT. Some fine grained sand, with ice lenses. (from drill logs Golder 2011b & Duane Miller Associates 2007).
- 45 to 67 feet depth: gray, wet, medium dense to very dense, silty SAND, some silt, trace organic material (from drill logs Golder 2011b & Duane Miller Associates 2007).

Material properties
The dry density ranged from 70 pcf to approximately 97 pcf (Golder 2011b). Due to minimal number of density tests completed on in-situ samples, the unit weight of the soil will be assumed to be 105 pcf for the lateral soil pressure/force calculations. This value is closer to typical published values for silt and sandy silt materials and will provide more conservative results. When estimating shear strength parameters for the silt, the lower in-situ density values will be considered to maintain conservative assumptions.
Using material descriptors from logs: un-frozen silt described as soft. Silt is considered to be non-plastic and assumed to behave as a non-cohesive soil (c'=0).
Internal friction angle:

Published literature values of the silt ranges between 26 to 30 degrees (ML) with relative compaction of 0% to 25%. (Figure 6, USS, Steel Sheet Piling Design Manual, 1894). In-situ tests results estimated values ranging from 26 to 38 degrees. A conservative value of 27 degrees for the Silt is considered appropriate for this level of design.
Published literature values of the silty sand ranges between 29 to 32 degrees (SM) with relative compaction of 25% to 50%. (Figure 6, USS, Steel Sheet Piling Design Manual, 1894). In-situ tests results estimated values ranging from 30 to 34 degrees. A conservative value of 30 degrees for the silty Sand is considered appropriate for this level of design.
Wall/Soil friction angle = 11 degree (fine sandy silt, non-plastic silt, Table 4, USS, Steel Sheet Piling Design Manual, 1984)

Geologic Conditions and Design parameters assumed along the Kugiklik riverbank:

Based on previous geotechnical studies, a typical geologic cross-section and material parameters at the Kugiklik riverbank (Kipnuk) were selected for use for the feasibility design of the sheetpile retaining wall.
The typical cross-section for the analysis was simplified into the following layering:

- 0 to 45 feet – Silt
- 45 to 100 feet – Silty Sand (fine grained)

Material properties - Design Values

SILT:

- Unit weight = 105 pcf
- Internal friction Angle = 27 degrees
- cohesion, c′ = 0 psf

Silty SAND:

- Unit weight = 105 pcf
- Internal friction Angle = 30 degrees
- cohesion, c′ = 0 psf

Assumptions:

- Depth 0 to 45 feet - Soil Profile consists of Silt with trace sand, ice lenses.
- Depth 45 to 100 feet - Soil Profile consists of silty Sand.
- Discontinues lenses of thawed, soft silty soils are interbedded within frozen silt.
- intermittent frozen soil exists 5 feet below ground surface,
- Groundwater is located at ground surface behind sheet-pile wall
- Shear strength of frozen soil is higher than unfrozen soil.
- Shear strength of soil will be considered un-frozen for conservatism.
- Silt is non-cohesive.

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Appendix B:
Survey Report

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KIPNUK EROSION SURVEY
KIPNUK, ALASKA
SURVEYING AND MAPPING REPORT

Prepared for:

AECOM
700 G Street, Suite 500
Anchorage, Alaska 99501

Prepared by:

DOWL
4041 B Street
Anchorage, Alaska 99503
(907) 562-2000

DOWL HKM Project Number: 1127.61897.01

June 2015

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TABLE OF CONTENTS

LIST OF ACRONYMS

ADOT&PF.................... Alaska Department of Transportation & Public Facilities

GNSS....................Global Navigation Satellite System

NAD83....................North American Datum 1983

NGS....................National Geodetic Survey

QA....................Quality Assurance

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HORIZONTAL & VERTICAL CONTROL SUMMARY

1.0 INTRODUCTION

This project consists of hydrographic surveying a portion of the Kuguklik River at the community of Kipnuk, Alaska, as well as collecting local high water flood marks in the community, existing bank conditions, and Photo ID points for historical imagery comparision.

2.0 HORIZONTAL CONTROL SUMMARY

A field survey was performed by DOWL from May 26th through May 28th, 2015. Static Global Navigation Satellite System (GNSS) observations were taken on the primary control station. All other control points locations were determined by RTK GNSS

Coordinates are based on Record of Survey for the Kipnuk Boardwalk Improvements Phase II project.

3.0 HORIZONTAL CONTROL STATEMENT

COORDINATE SYSTEM

This project is located entirely within the Kipnuk Adjustment, A U.S. Survey Foot local surface grid coordinate system developed by the Alaska Department of Transportation. Control information was obtained from the ADOT&PF survey control sheet The Kipnuk Boardwalk Improvements Phase II recorded as plat 2011-17 in the Bethel Recording District.

TRANSLATION PARAMETERS

To convert Local Coordinates to NAD83(CORS96) Alaska State Plane Zone 8 U.S. Survey Feet Coordinates:

- Rotate coordinates Counter-Clockwise about 30,000N, 50,000E 1°42'24",
- Translate +2,140,346.6464 North, +1,951,944.7322 East,
- Scale about 0, 0 using 1.0000488464.

4.0 VERTICAL CONTROL SUMMARY

Elevations are based on A U.S. Survey Foot local datum developed by the Alaska Department of Transportation. Control information was obtained from the ADOT&PF survey control sheet The Kipnuk Boardwalk Improvements Phase II recorded as plat 2011-17 in the Bethel Recording District. The Elevation for Point 551 was held as 993.09 Feet. All other elevations were determined by RTK GPS surveying methods.

5.0 SURVEY METHODS

All Topographic Survey information was collected using RTK GNSS Surveying equipment. For the Hydrographic Survey data, A Leica GNSS RTK Receiver was configured to collect coincident data with an ODOM Hydrotrac Fathometer. The resulting data has an anticipated accuracy of +/- 0.2’.

6.0 QUALITY ASSURANCE

Quality Assurance (QA) methods and procedures outlined in the statement of services were reviewed with our staff and adhered to. Some examples of QA methods include the following:

- All equipment utilized during this project was checked for accuracy, and adjusted when necessary, prior to commencing any work.

- Check shots taken and compared within 0.12 feet horizontally and 0.11 feet vertically.

7.0 SURVEYOR'S CERTIFICATION

I, A, William Stoll, Alaska Land Surveyor #12041, do hereby certify that the information contained herein is the result of work performed by me or by others working under my direct supervision.

STATE OF ALASKA
49TH
A. William Stoll
LS-12041
6/7/15
REGISTERED PROFESSIONAL SURVEYOR

Appendix C:
Cost Estimates

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Rip Rap Alternative 1 Cost Estimate

Reach 1- Rip rap					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	6,483	2,268,972	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
Rip Rap Class IV	CY	380	1,206	458,111	Unit cost based on $360/cy materials delivered from Platinum, plus $20/cy to install
Gravel Filter Layer	CY	150	1,889	283,333	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Sheet Pile	LS			1,365,000	See ** below
Geotextile Fabric	SY	2	1,167	2,334	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				656,663	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$5,534,413*	
$/Linear Foot				*$11,069*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$6,034,413*	

**Barge Landing Sheet pile wall: 350' long x 60' tall x $65/sf = $1.365 million

Reach 2- Rip rap					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	25,565	8,947,750	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
Rip Rap Class IV	CY	380	5,536	2,103,511	Unit cost based on $360/cy materials delivered from Platinum, plus $20/cy to install
Gravel Filter Layer	CY	150	8,602	1,290,250	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	5,300	10,600	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				1,852,817	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$14,704,928*	
$/Linear Foot				*$8,524.60*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$15,204,928*	

Reach 3- Rip rap					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	21,086	7,380,139	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
Rip Rap Class IV	CY	380	5,140	1,953,130	Unit cost based on $360/cy materials delivered from Platinum, plus $20/cy to install
Gravel Filter Layer	CY	150	8,447	1,267,083	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	5,083	10,166	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				1,591,578	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$12,702,096*	
$/Linear Foot				*$5,773.68*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$13,202,096*	

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Rip Rap Alternative 2 With Barge Landing Cost Estimate

Reach 2- Rip rap					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	6,455	2,259,367	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
Rip Rap Class IV	CY	380	877	333,387	Unit cost based on $360/cy materials delivered from Platinum, plus $20/cy to install
Gravel Filter Layer	CY	150	2,098	314,650	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	1,483	2,967	Unit cost based on ADOT&PF bid tabs (Kipnuk)
**Barge Landing	LS			1,365,000	
Contingency				641,306	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$5,416,676*	
$/Linear Foot				*$3,140.10*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$5,916,676*	

**Barge Landing Sheet pile wall: 350' long x 60' tall x $65/sf = $1.365 million

Reach 3- Rip rap					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Furnish and Install - Reach 3	CY	350	21,086	7,380,139	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
Rip Rap Class IV	CY	380	5,140	1,953,130	Unit cost based on $360/cy materials delivered from Platinum, plus $20/cy to install
Gravel Filter Layer	CY	150	8,447	1,267,083	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	5,083	10,166	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				1,591,578	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$12,702,096*	
$/Linear Foot				*$5,773.68*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$13,202,096*	

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Rip Rap Alternative 2 Without Barge Landing Cost Estimate

Reach 2- Rip rap					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	6,855	2,399,367	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
Rip Rap Class IV	CY	380	1,574	597,979	Unit cost based on $360/cy materials delivered from Platinum, plus $20/cy to install
Gravel Filter Layer	CY	150	2,298	344,650	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	1,483	2,967	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				501,744	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$4,346,707*	
$/Linear Foot				*$2,519.83*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$4,846,707*	

Reach 3- Rip rap					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Furnish and Install - Reach 3	CY	350	21,086	7,380,139	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
Rip Rap Class IV	CY	380	5,140	1,953,130	Unit cost based on $360/cy materials delivered from Platinum, plus $20/cy to install
Gravel Filter Layer	CY	150	8,447	1,267,083	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	5,083	10,166	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				1,591,578	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$12,702,096*	
$/Linear Foot				*$5,773.68*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$13,202,096*	

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Rip Rap & ACB Alternative 1 Cost Estimate

Reach 1- Rip rap & ACB					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	6,483	2,268,972	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
ACB Matting	SY	430	1,167	501,667	Unit cost based on $410/yard materials delivered from Seattle, plus $20/cy to install
Gravel Filter Layer	CY	150	1,889	283,333	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Sheet Pile	LS			1,365,000	See ** below
Geotextile Fabric	SY	2	1,167	2,334	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				663,196	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$5,584,502*	
$/Linear Foot				*$11,169*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$6,084,502*	

**Barge Landing Sheet pile wall: 350' long x 60' tall x $65/sf = $1.365 million

Reach 2- Rip rap & ACB					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	25,565	8,947,750	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
ACB Matting	SY	430	5,300	2,279,000	Unit cost based on $410/yard materials delivered from Seattle, plus $20/cy to install
Gravel Filter Layer	CY	150	8,602	1,290,250	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	5,300	10,600	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				1,879,140	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$14,906,740*	
$/Linear Foot				*$8,641.59*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$15,406,740*	

Reach 3- Rip rap & ACB					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	21,086	7,380,139	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
ACB Matting	SY	430	5,083	2,185,833	Unit cost based on $410/yard materials delivered from Seattle, plus $20/cy to install
Gravel Filter Layer	CY	150	8,447	1,267,083	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	5,083	10,166	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				1,626,483	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$12,969,705*	
$/Linear Foot				*$5,895.32*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$13,469,705*	

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Rip Rap & ACB Alternative 2 With Barge Landing Cost Estimate

Reach 2- Rip rap & ACB					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	6,455	2,259,367	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
ACB Matting	SY	430	840	361,200	Unit cost based on $410/yard materials delivered from Seattle, plus $20/cy to install
Gravel Filter Layer	CY	150	2,098	314,650	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	1,483	2,967	Unit cost based on ADOT&PF bid tabs (Kipnuk)
**Barge Landing	LS			1,365,000	
Contingency				645,478	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$5,448,661*	
$/Linear Foot				*$3,158.64*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$5,948,661*	

**Barge Landing Sheet pile wall: 350' long x 60' tall x $65/sf = $1.365 million

Reach 3- Rip rap & ACB					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Furnish and Install - Reach 3	CY	350	21,086	7,380,139	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
ACB Matting	SY	430	5,083	2,185,833	Unit cost based on $410/yard materials delivered from Seattle, plus $20/cy to install
Gravel Filter Layer	CY	150	8,447	1,267,083	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	5,083	10,166	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				1,626,483	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$12,969,705*	
$/Linear Foot				*$5,895.32*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$13,469,705*	

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Rip Rap & ACB Alternative 2 Without Barge Landing Cost Estimate

Reach 2- Rip rap & ACB					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Class II	CY	350	6,855	2,399,367	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
ACB Matting	SY	430	1,507	647,867	Unit cost based on $410/yard materials delivered from Seattle, plus $20/cy to install
Gravel Filter Layer	CY	150	2,298	344,650	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	1,483	2,967	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				509,228	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$4,404,078*	
$/Linear Foot				*$2,553.09*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$4,904,078*	

Reach 3- Rip rap & ACB					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
Rip Rap Furnish and Install - Reach 3	CY	350	21,086	7,380,139	Unit cost based on $330/cy materials delivered from Nome, plus $20/cy to install
ACB Matting	SY	430	5,083	2,185,833	Unit cost based on $410/yard materials delivered from Seattle, plus $20/cy to install
Gravel Filter Layer	CY	150	8,447	1,267,083	Unit cost based on $122/cy materials delivered from Platinum, plus $28/cy to install
Geotextile Fabric	SY	2	5,083	10,166	Unit cost based on ADOT&PF bid tabs (Kipnuk)
Contingency				1,626,483	15%
Mobilization/Demobilization				500,000	
Sub-Total Estimated Construction Costs				*$12,969,705*	
$/Linear Foot				*$5,895.32*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$13,469,705*	

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Rip Rap Quantities, Reach 1, 2 & 3

Reach 1	Area (ft^2)	Length (ft)	Vol(Yd^3)	Weight (tons)
Class IV	93	350	1,206	1,929
Class II	129	295	1,409	2,255
Class II Toe	90	541	1,803	2,885
Class II Wall	113	265	1,109	1,775
Total Class II			4,322	6,915
Total Class II x 1.5			6,483	10,372
Gravel Filter Layer			1,259	2,015
Gravel Filter Layer x 1.5			1889	3,022

Reach 2	Area (ft^2)	Length (ft)	Vol(Yd^3)	Weight (tons)
Class IV	94	1590	5,536	8,857
Class II	183	1590	10,777	17,243
Class II Toe	90	1590	5,300	8,480
Class II slough at south	145	180	967	1,547
Total Class II			17,043	27,269
Total Class II x 1.5			25,565	40,904
				0
Gravel Filter Layer	89	1590	5,241	8,386
Gravel at S. Slough	74	180	493	789
Total Gravel Filter			5,734	9,175
Gravel Filter Layer x 1.5			8602	13,763

Reach 3	Area (ft^2)	Length (ft)	Vol(Yd^3)	Weight (tons)
Class IV	91	1525	5,140	8,224
Class II	132	1525	7,456	11,929
Class II Toe	90	1525	5,083	8,133
Tie Back Class II	96	125	444	711
Class II Slough at south	145	200	1,074	1,719
Total Class II			14,057	22,492
Total Class II x 1.5			21,086	33,738
				0
Gravel Filter Layer	90	1525	5,083	8,133
Gravel at Slough at south	74	200	548	877
Total Gravel Filter			5,631	9,010
Gravel Filter Layer x 1.5			8,447	13,516

Note: Quantities based on measurements in AutoCAD.

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Alternative 2 Rip Rap Quantities, Reach 2

With Barge Landing Wall

Reach 2-Barge Landing	Area	Length ft	Vol(Yd3)	weight (tons)
Class IV	94	252	877	1,404
Class II	183	252	1,708	2,733
Class II Wall	147	200	1,089	1,742
Class II Toe	90	452	1,507	2,411
Total Class II			4304	6,886
Total Class II x 1.5			6,455	10,329
Gravel Filter Layer	69	445	1,398	2,238
Gravel Filter Layer x 1.5			2,098	3,356
Fabric	30	445	1483.333	

Without Barge Landing Wall

Reach 2	Area	Length ft	Vol(Yd3)	weight (tons)
Class IV	94	452	1,574	2,518
Class II	183	452	3,064	4,902
Class II Toe	90	452	1,507	2,411
Total Class II			4570	7,312
Total Class II x 1.5			6,855	10,969
Gravel Filter Layer	69	445	1,532	2,451
Gravel Filter Layer x 1.5			2,298	3,676
Fabric	30	445	1483.333	

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Sheetpile Wall Alternative 4 Cost Estimate

Reach 1 - Barge Landing					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
HZ Piling-materials cost	LF	524	6,845	3,584,291	$1.10 per lb
AZ14 Piling-materials cost	LF	170	6,845	1,160,993	$0.80 per lb
AZ50 Piling-materials cost	LF	150	5,782	869,593	$0.80 per lb
HZ Piling- driving cost	Each	2,500	60	150,120	
AZ Piling - driving cost	Each	2,000	60	120,096	
Shipping	Pound	0.67	5,796,679	3,883,775	Budgetary Quote Alaska Marine Lines
#20 Anchor Rod furnish & install	Each	5,800	30	174,000	
Epoxy Coating - Headwall	SF	6	67565	405,389	Waterside Only
Contingency				517,413	5%
Mobilization				500,000	
Sub-Total Estimated Construction Costs				*$11,365,670*	
$/Linear Foot				*$22,731*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$11,865,670*	

Reach 2 -Seawall					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
HZ Piling-materials cost	LF	524	21,906	11,469,733	$1.10 per lb
AZ14 Piling-materials cost	LF	170	21,906	3,715,177	$0.80 per lb
AZ50 Piling-materials cost	LF	150	18,502	2,782,696	$0.80 per lb
HZ Piling- driving cost	Each	2,000	192	384,307	
AZ Piling - driving cost	Each	1,500	613	918,980	
Shipping	Pound	0.67	18,549,371	12,428,079	Budgetary Quote Alaska Marine Lines
#20 Anchor Rod	Each	5,800	160	928,000	
Epoxy Coating - Headwall	SF	6	216208	1,297,245	Waterside Only
Contingency				1,696,211	5%
Mobilization				500,000	
Sub-Total Estimated Construction Costs				*$36,120,428*	
$/Linear Foot				*$22,575*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$36,620,428*	

Reach 3 - Seawall					
Item Description	**Unit**	**Unit Cost**	**Quantity**	**Cost $**	**Comments**
HZ Piling-materials cost	LF	524	24,644	12,903,449	$1.10 per lb
AZ14 Piling-materials cost	LF	170	24,644	4,179,574	$0.80 per lb
AZ50 Piling-materials cost	LF	150	20,815	3,130,534	$0.80 per lb
HZ Piling- driving cost	Each	2,000	216	432,346	
AZ Piling - driving cost	Each	1,500	689	1,033,852	
Shipping	Pound	0.67	20,868,043	13,981,589	Budgetary Quote Alaska Marine Lines
#20 Anchor Rod furnish & install	Each	5,800	180	1,044,000	
Epoxy Coating - Headwall	SF	6	243233	1,459,401	Waterside Only
Riprap	CY	300	400	120,000	
Contingency				1,914,237	5%
Mobilization				500,000	
Sub-Total Estimated Construction Costs				*$40,698,981*	
$/Linear Foot				*$22,611*	
Engineering, Permiting, Geotech				500,000	
Sub-Total Estimated Engineering Costs				*$500,000*	
Total Estimated Cost				*$41,198,981*	

Notes: 1. Quantities obtained from AutoCAD
2. Unit costs from comparable bid tabs
3. Sheet pile costs based on quotes from sheet pile suppliers

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Phase I Relocation Cost Estimate

Item Description	Unit	Unit Cost	Quantity	Cost $	Comments
KTC Lodge	SF	250	2,280	570,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
KTC Garage	SF	250	600	150,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Kugkaktlik Tank	SF	350	700	245,000	Assume no leaking or large sacle contamination cleanup required
Minor structures	LS	50000	1	50,000	Small Tanks, conexes, sheds
Contingency				193,000	20%
Mobilization				100,000	
		Total Estimated Cost		$1,308,000	2016 Dollars

Phase II Relocation Cost Estimate Year 2025

Item Description	Unit	Unit Cost	Quantity	Cost $	Comments
Building 4	SF	250	400	100,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 5	SF	250	810	202,500	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 6	SF	250	1,100	275,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 7	SF	250	1,400	350,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 8	SF	250	500	125,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 9	SF	250	900	225,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 10	SF	250	1,800	450,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Minor structures	LS	50000	1	50,000	Small Tanks, conexes, sheds
Overhead Electric & Telephone	Pole	20,000	12	240,000	Assume poles at 300' spacing
Kugkaktlik Tanks (2)	SF	350	2,400	840,000	Assume no leaking or large sacle contamination cleanup required
Contingency				571,500	20%
Mobilization				100,000	
		Total Estimated Cost		$3,529,000	2016 Dollars
Net present value (4% discount rate) adjusted for inflation (2%inflation rate)				$2,952,909	

Phase III Relocation Cost Estimate, Year 2040

Item Description	Unit	Unit Cost	Quantity	Cost $	Comments
Building 13	SF	250	630	157,500	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 14	SF	250	530	132,500	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 15	SF	250	1,000	250,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 16	SF	250	850	212,500	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 17	SF	250	1,300	325,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 18	SF	250	1,000	250,000	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 19	SF	250	1,750	437,500	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Building 20	SF	250	1,350	337,500	Unit cost based on $250 per square foot, Kivalina Estimate, 2010. Area from AutoCAD Drawing
Minor structures	LS	50000	1	50,000	Small Tanks, conexes, sheds
Boardwalk	LF	150	300	45,000	
Contingency				439,500	20%
Mobilization				100,000	
		Total Estimated Cost in 2040		$2,637,000	2016 dollars
Net present value (4% discount rate) adjusted for inflation (2%inflation rate)				$1,639,480	

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SHEET KEYNOTES

PHASE I

1. KTC LODGE
2. KTC GARAGE
3. KUGKAKTLIK CORP TANK

PHASE II

4. BUILDING 4
5. BUILDING 5
6. BUILDING 6
7. BUILDING 7
8. BUILDING 8
9. BUILDING 9
10. BUILDING 10
11. OVERHEAD ELECTRIC & TELEPHONE RELOCATION
12. KUGKAKTLIK CORP TANKS (2)

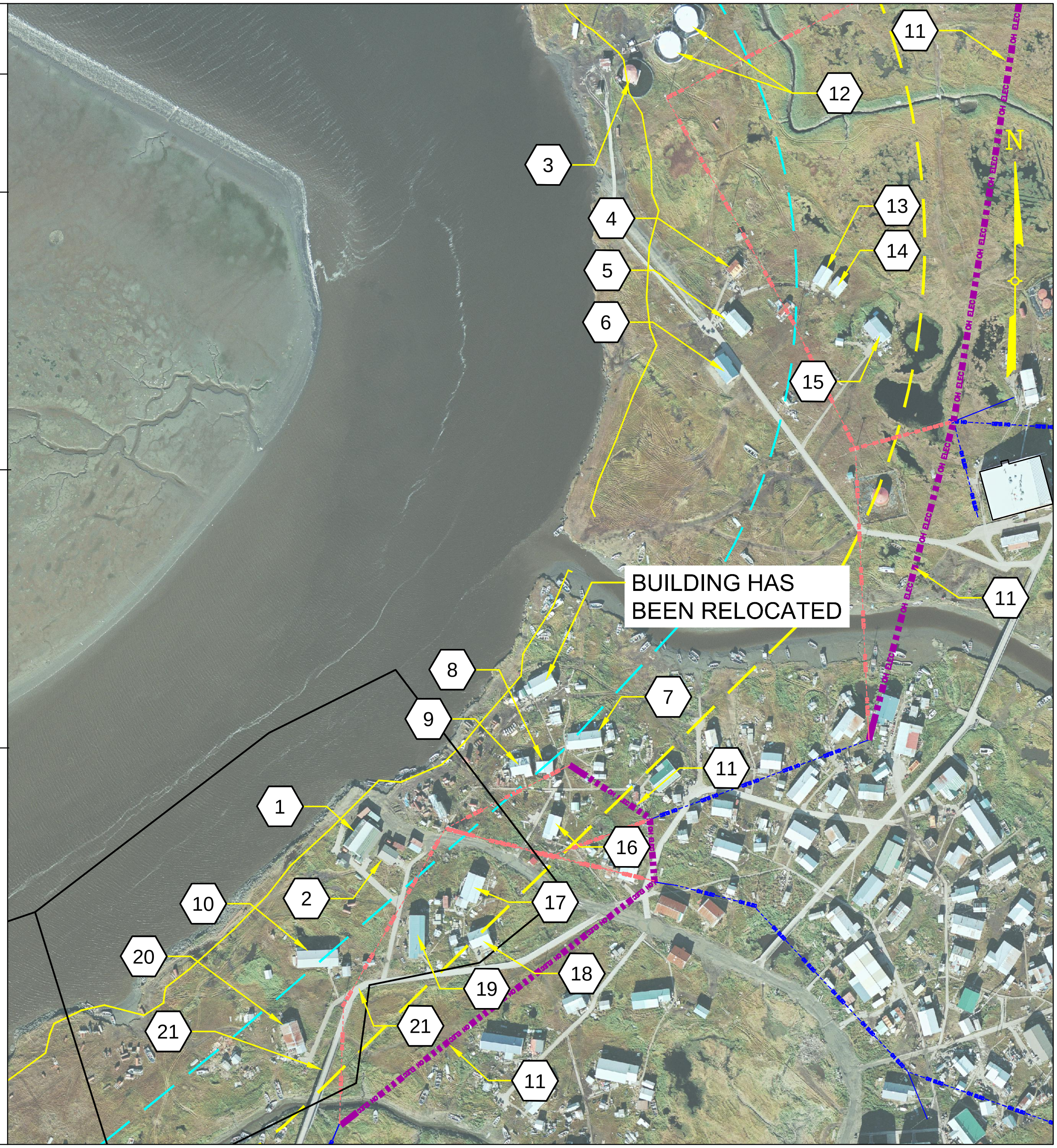
PHASE III

13. BUILDING 13
14. BUILDING 14
15. BUILDING 15
16. BUILDING 16
17. BUILDING 17
18. BUILDING 18
19. BUILDING 19
20. BUIL;DING 20
21. BOARDWALK

NOTE: MISCELLANEOUS STRUCTURES SUCH AS SMALL TANKS, CONEXES, AND SHEDS, NOT LISTED ABOVE, WILL ALSO REQUIRE RE-LOCATION

LEGEND

- 2015 BANK
- 2040 BANK (ESTIMATED)
- 2065 BANK (ESTIMATED)
- OH ELEC — EXISTING OVERHEAD TRANSMISSION LINES
- OH ELEC — OVERHEAD TRANSMISSION LINES TO BE RELOCATED
- OH ELEC — RELOCATED OVERHEAD ELECTRICAL TRANSMISSION LINES



AECOM MAY 2016

PHASED RELOCATION

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Appendix D:
Sheetpile Wall Design Parameters

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Basis of Design

1. Introduction
This Project involves the preliminary design of an anchored sheetpile bulkhead at the Village of Kipnuk to mitigate impacts of erosion caused by the Kuguklik River.

2. Geographic Conditions
The Village of Kipnuk is located on the Kugkaktlik (Kuguklik) River 85 miles southwest of Bethel, four miles inland from the Bering Sea. The community is located on the outside bend of the river.

3. Design Criteria

1.1 Design Reference Codes and Standards

- ASCE 7-10
- AASHTO Bridge Design Specification 7th edition
- USS Steel Sheet Piling Design Manual 1984
- USACE EM 1110-2-2504
- USACE EM 1110-2-2502

1.2 Materials

1. Sheet Piling – ASTM A572 Grade 65 (65ksi)
2. Anchor Rods – ASTM A722 Grade 150
3. All structural steel to be hot rolled sections.

1.3 Loading

1. Dead Load
    - Weight of Materials
2. Live Load
    - 120 psf surcharge
4. Seismic Load
    - Seismic loads are expected to be relatively small and were not considered at this preliminary phase of work.
5. Ice Load
    - Ice Loads are not expected to control the design and were not considered at this preliminary phase of work.
6. Wave Loads

- Wave loads have not been considered at this preliminary phase of work.

7. Seepage Loads
   - Seepage loads have not been analyzed at this preliminary phase of work.

1.4 Tidal Data

Water elevations are from a GPS survey conducted May 2015. Survey points were tied into the vertical datum developed by the Alaska Department of Transportation. The datum was derived by GPS by adding 1000 feet to the NAVD88 Geoid 99 height.

Low Water: +980’

Tidal Lag: 9’, assumes water level is at top of bank (+989’) behind the wall and at the low water level (+980’) in front of the wall

1.5 Geotechnical Criteria

Soil Properties from: “estimated soil parameters – riverbank” 11/20/2015

Layer 1: Silt 0’ – 45’ Below Ground Surface
Unit Weight – 105 pcf
Submerged Density – 43 pcf
Soil Friction Angle – 27 degrees
Soil Wall Friction Angle – 11 degrees

Layer 2: Silty Sand
Unit Weight – 105 pcf
Submerged Density – 43 pcf
Soil Friction Angle – 30 degrees
Soil Wall Friction Angle – 11 degrees

- Deep seated failure has not been analyzed at this stage of the design
- Scour depth: +913’ (from document tilted “wall depths 11-16-15”)
- Discontinuous permafrost exists in the area, including along the river bank. The effects of permafrost have not been considered at this preliminary phase of the work, issues might include difficulty installing the piles through the permafrost (especially the sheets), downward loads on the tie rods due to settlement from thawing permafrost, and reduced anchor wall capacity due to thawing permafrost. Additional geotechnical investigation would be required to confirm permafrost locations.

1.6 Load Combinations

ASD Load Combinations:

- LC2: D+L+H

1.7 Design Software

The following computer software is used in the analysis and design of the structural elements:

1. Prosheet V2.2

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Soil Parameters Kipnuk Sheet Pile Wall Design

Numerous geotechnical studies have been performed in the village of Kipnuk and surrounding areas however geotechnical borings along the Kugiklik riverbank and proposed alignment of the sheetpile wall have not been performed at this date. The geologic condition (typical cross-section and material parameters) along the Kugiklik riverbank have been assumed based on information collected from these previous studies. The material parameters are considered appropriate for the "feasibility" level design, used during this study. Additional geotechnical investigations along the proposed wall alignment are recommended to confirm assumptions of the material parameters and the geologic cross-section of the subsurface materials during final design.

The following geotechnical studies and information were used during the development of the geologic conditions along the Kugiklik riverbank:

- **Boardwalk Improvements, Phase II (Golder, 2011a):** Shallow borings performed for the Boardwalk Improvements, Phase II (Golder, 2011). Total of 43 test holes, 7 performed close to the river bank. Hand auger used to advance through active zone generally to the top of permafrost.
- **Kipnuk Bulk Fuel and Powerplant Faciltiy (Duane Miller 2007):** Borings located approximately 700 feet to the northeast of the riverbank were drilled to depths of 48 to 67 feet. Split Barrel and Shelby Tube samplers were used to collect soil samples for laboratory testing.
  Twelve moisture/density tests were performed on silty to fine sandy materials. The dry density of the samples ranged from 70 pcf (silt) to approximately 97 pcf (silty SAND). Four Direct Shear Tests performed on fine sandy materials and two Unconsolidated Undrained Triaxial (TXUU) tests performed on siltier materials. Results indicate the materials were non-plastic. The direct shear tests estimated the internal friction angle ranged between 30 to 35 degrees for three in-situ samples and was estimated to be 26 degrees for one remolded sample. The two TXUU results estimated the undrained shear strength of the inorganic silts was approximately 600 to 750 psf.
- **Chief Paul Memorial School Expansion (Golder, 2011b)**: Borings located approximately 900 feet to the southeast of the riverbank were drilled to depths of 21 to 70 feet. Direct push sampling equipment was used to collect disturbed samples of the non-organic silt and fine sand materials for laboratory testing. Two Direct Shear tests performed on remolded samples and estimated the internal friction angle was 38 degrees (silt) and 34 degrees (sand). As the results of the silt are much higher than published literature values of this material, a lower value of 34 degrees was used by Golders in their engineering analysis.

Permafrost:

Permafrost noted in previous borings. Permafrost conditions vary widely across the community.

Typical cross-section

Subsurface layering was generalized into the following layers based on the available information from previous borings. The upper 5 feet is considered to be an active zone, with intermittent frozen soil below this elevation. The soil descriptions for each generalized layer are summarized below.

- 0 to 1.5 feet depth: Loose, wet, dark brown, PEAT, (from drill logs, Golder 2011a)
- 1.5 to 5 feet depth: Loose, wet, brown SILT, visible ice lenses (1-2mm), (from drill logs Golder 2011a & Duane Miller Associates 1999).
- 5 to 45 feet depth: wet, gray to black, medium stiff to stiff, sandy SILT. Some fine grained sand, with ice lenses. (from drill logs Golder 2011b & Duane Miller Associates 2007).
- 45 to 67 feet depth: gray, wet, medium dense to very dense, silty SAND, some silt, trace organic material (from drill logs Golder 2011b & Duane Miller Associates 2007).

Material properties

The dry density ranged from 70 pcf to approximately 97 pcf (Golder 2011b). Due to minimal number of density tests completed on in-situ samples, the unit weight of the soil will be assumed to be 105 pcf for the lateral soil pressure/force calculations. This value is closer to typical published values for silt and sandy silt materials and will provide more conservative results. When estimating shear strength parameters for the silt, the lower in-situ density values will be considered to maintain conservative assumptions.

Using material descriptors from logs: un-frozen silt described as soft. Silt is considered to be non-plastic and assumed to behave as a non-cohesive soil (c'=0).

Internal friction angle:

Published literature values of the silt ranges between 26 to 30 degrees (ML) with relative compaction of 0% to 25%. (Figure 6, USS, Steel Sheet Piling Design Manual, 1894). In-situ tests results estimated values ranging from 26 to 38 degrees. A conservative value of 27 degrees for the Silt is considered appropriate for this level of design.

Published literature values of the silty sand ranges between 29 to 32 degrees (SM) with relative compaction of 25% to 50%. (Figure 6, USS, Steel Sheet Piling Design Manual, 1894). In-situ tests results estimated values ranging from 30 to 34 degrees. A conservative value of 30 degrees for the silty Sand is considered appropriate for this level of design.

Wall/Soil friction angle = 11 degree (fine sandy silt, non-plastic silt, Table 4, USS, Steel Sheet Piling Design Manual, 1984)

Geologic Conditions and Design parameters assumed along the Kugiklik riverbank:

Based on previous geotechnical studies, a typical geologic cross-section and material parameters at the Kugiklik riverbank (Kipnuk) were selected for use for the feasibility design of the sheetpile retaining wall.

The typical cross-section for the analysis was simplified into the following layering:

- 0 to 45 feet – Silt
- 45 to 100 feet – Silty Sand (fine grained)

Material properties - Design Values

SILT:

- Unit weight = 105 pcf
- Internal friction Angle = 27 degrees
- cohesion, c' = 0 psf

Silty SAND:

- Unit weight = 105 pcf
- Internal friction Angle = 30 degrees
- cohesion, c' = 0 psf

Assumptions:

- Depth 0 to 45 feet - Soil Profile consists of Silt with trace sand, ice lenses.
- Depth 45 to 100 feet - Soil Profile consists of silty Sand.

- Discontinues lenses of thawed, soft silty soils are interbedded within frozen silt.
- intermittent frozen soil exists 5 feet below ground surface,
- Groundwater is located at ground surface behind sheet-pile wall
- Shear strength of frozen soil is higher than unfrozen soil.
- Shear strength of soil will be considered un-frozen for conservatism.
- Silt is non-cohesive.

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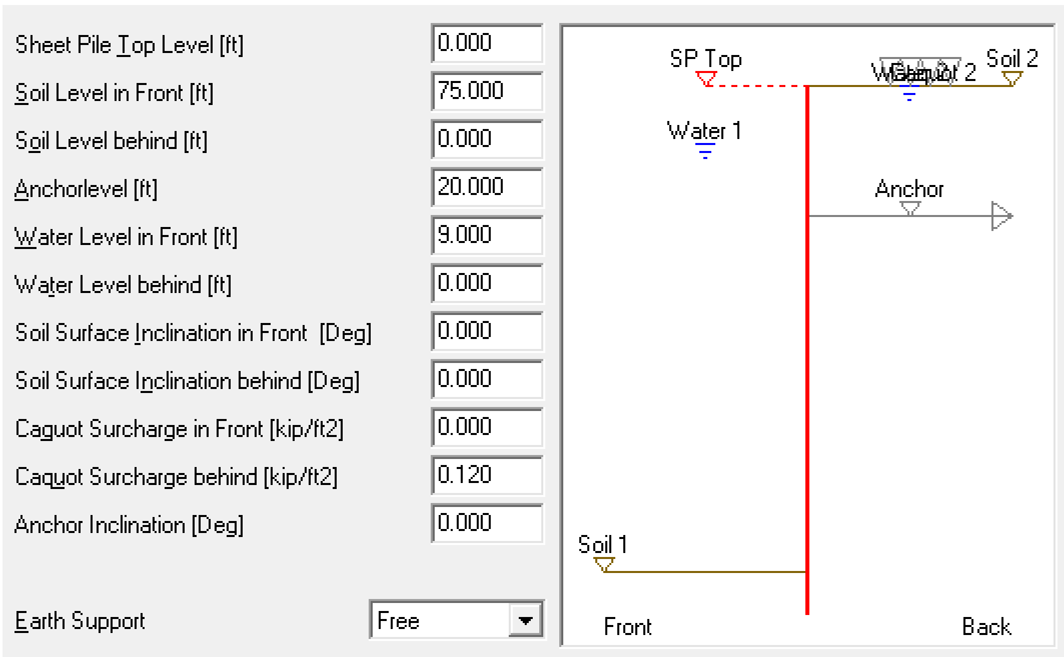
Kipnuk Sheetpile Wall Preliminary Design

Find the required length and section modulus of an anchored sheetpile wall with geometry shown below. Model in prosheet using "free-earth" support and a 120psf surcharge behind the wall. Assume the water level behind the wall is at the soil surface and the water in front of the wall is 9 ft below the top (approximate low tide level).



Soil Properties From "Estimated Soil Parameters - riverbank":

Soil 1 (0'-45')	Soil 2 (45' +)
$\phi_1 := 27 \cdot deg$	$\phi_2 := 30 \cdot deg$
$\delta_1 := 11 \cdot deg$	$\delta_2 := 11 \cdot deg$
$\gamma_m := 105 \cdot pcf$	$\gamma_m := 105 \cdot pcf$
$\gamma_{sub} := 43 \cdot pcf$	$\gamma_{sub} := 43 \cdot pcf$

Find length of headwall piles and req'd embedment:

First apply a safety factor to the soils on the passive side of the wall as recommended by USACE manual EM-1110-2-2504

$FSP := 1.5$ Table 5-1 EM-1110-2-2504

Factor of safety is applied to the soil friction angle (ϕ2) on the passive side of the wall using Eq. 5-1

$$\phi_{eff} := \tan(\phi_{eff}) = \frac{\tan(\phi_2)}{FSP} \text{ solve } \rightarrow \operatorname{atan}(0.66666666666666666667 \cdot \tan(30.0 \cdot deg)) = 21.052 \cdot deg$$

Use this value for ϕeff in prosheet for the friction angle of soil on the passive side of the wall

Geo Data | Soil Layers | Concentrated Forces | Userdefined Pressures | Boussinesq | Sheet Pile Section

Layers in Front

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	1000.000	0.105	0.043	2.789	21.052	-11.000	0.000

☑ Automatic Kph Values + −

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kah	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	45.000	0.105	0.043	0.338	27.000	11.000	0.000
Layer 2	1000.000	0.105	0.043	0.301	30.000	11.000	0.000

☑ Automatic Kah Values + − Copy from Above

All Values | Extremal Values | Pile Check

		Depth [ft]	
Sheet Pile Top Level [ft]	0.000		
Sheet Pile Tip Level [ft]	113.813		
Sheet Pile Length [ft]	113.813		Required sheetpile length is 114 ft,
Included OverLength [ft]	0.000		
Vertical Equilibrium [kip/ft]	-0.109		
Anchor Force (horiz.) [kip/ft]	-62.056		Required anchor force; use 62 k/ft

Find Required Steel Section for Anchored Walls

Chapter 6 EM-1110-2-2504

Structural design of the sheetpile wall will be done without a factor of safety applied to the friction angle of the soil on the passive side (i.e. FSP = 1) so as not to compound factors of safety.

Geo Data | Soil Layers | Concentrated Forces | Userdefined Pressures | Boussinesq | Sheet Pile Section

Layers in Front

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	1000.000	0.105	0.043	4.288	30.000	-11.000	0.000

☑ Automatic Kph Values + −

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kah	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	45.000	0.105	0.043	0.338	27.000	11.000	0.000
Layer 2	1000.000	0.105	0.043	0.301	30.000	11.000	0.000

☑ Automatic Kah Values + − Copy from Above

Find maximum moment calculated by prosheet for the above soil input (geometry and loading remains the same)

All Values | Extremal Values | Pile Check

		Depth [ft]
Minimal Moment [kipft/ft]	-837.701	58.944
Maxmimal Moment [kipft/ft]	98.184	20.000

$M_{max} := 837\text{ft}\cdot\text{kip} = 1.004 \times 10^{4}\cdot\text{in}\cdot\text{kip}$ per foot of wall

Find Minimum Section Modulus. use A572 Gr. 65 Steel

$f_y := 65 \cdot \frac{kip}{in^2}$

$\Omega_b := 1.67$

$M_a \le \frac{M_n}{\Omega_b}$

$M_n = f_y \cdot s_{min}$

The minimum section modulus will be given by

$M_{max} = \frac{f_y \cdot s_{min}}{\Omega_b}$

$s_{min} := \frac{M_{max} \cdot \Omega_b}{f_y} = 258.054 \cdot in^3$

Try HZ 1080M B-24 M / AZ 14-770 Wall

$s := 261.1 in^3$

$I := 5417 in^4$

<u>Check Flexure:</u>

Allowable moment $M_a := \frac{f_y \cdot s}{\Omega_b} = 846.881 \cdot ft \cdot kip$ Capacity

Previously calculated $M_{max} = 837 \cdot ft \cdot kip$ Demand

Demand/Capacity Ratio $\frac{M_{max}}{M_a} = 0.988$ $< 1 ...OK$

Design Anchor Wall, Tie Rods, and Wale

Per EM-1110-2-2504 tie rods and wales should be designed for the anchor force found during the stability design (with a FS applied to the passive soils). Conservatively design the anchor wall to resist these forces as well.

Use prosheet to model the anchor wall

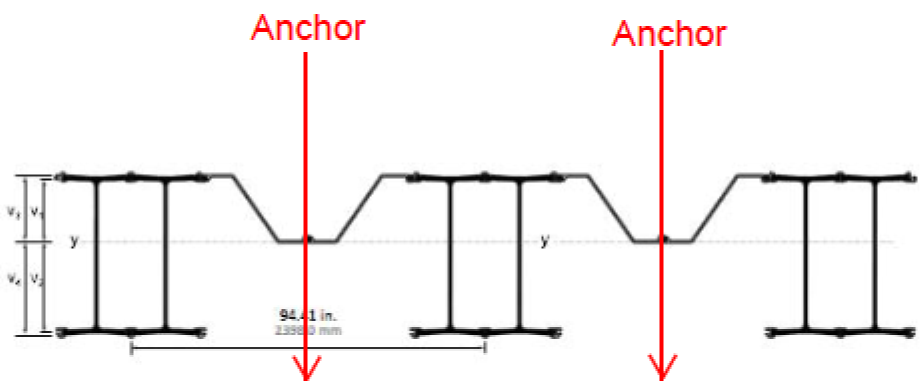
Find required depth of penetration using reduced passive side soil properties

All Values | Extremal Values | Pile Check

		Depth [ft]	
Name	AZ 48		
Inertia [in4/ft]	847.024		
Modulus [in3/ft]	89.280		
Area [in2/ft]	14.481		
Mass [lbs/ft2]	49.279		
Steel Grade [lb/in2]	65000.000		
Minimal Moment [kipft/ft]	-261.932	28.491	
Maxmimal Moment [kipft/ft]	4.037	42.336	
Normal Forces at Max. Moment [kip/ft]	0.000	28.491	
Normal Forces at Min. Moment [kip/ft]	0.000	42.336	
Deflection at Min. Moment [ft]	-0.237	28.491	
Deflection at Max. Moment [ft]	-0.160	42.336	
Min. Stress at Min. Moment [lb/in2]	-35204.566	28.491	
Max. Stress at Min. Moment [lb/in2]	35204.566	28.491	
Min. Stress at Max. Moment [lb/in2]	-542.552	42.336	
Max. Stress at Max. Moment [lb/in2]	542.552	42.336	
Safety > Req. Safety = 1.500	1.846		AZ 48 Section OK for Flexure
Sheet Pile Top Level [ft]	0.000		
Sheet Pile Tip Level [ft]	42.763		
Sheet Pile Length [ft]	42.763		Use 43' Pile
Included OverLength [ft]	0.000		
Vertical Equilibrium [kip/ft]	0.000		
Anchor Force (horiz.) [kip/ft]	62.056		

Find minimum anchor rod section

Try using one anchor for each HZM section (spacing = 99").



$F_{Anch} := 62 \cdot \frac{kip}{ft}$ Required Anchor Force per Foot of Wall

$S := 99 \cdot in = 8.25 \cdot ft$ Distance Between Anchor Rods

$T_{rod} := F_{Anch} \cdot S = 511.5 \cdot kip$ Tension Per Rod

Find required Diameter of rod using 150ksi steel

$f_{yrod} := 150 \cdot ksi$

$\Omega_t := 2.0$

$A_{rod} := \frac{T_{rod} \cdot \Omega_t}{f_{yrod}} = 6.82 \cdot in^2$ Minimum allowable area of steel

$D_{rod} := \sqrt{\frac{4 \cdot A_{rod}}{\pi}} = 2.947 \cdot in$ Use 3" Anchor rod every 8.25 feet

Size Wales per EM-1110-2504

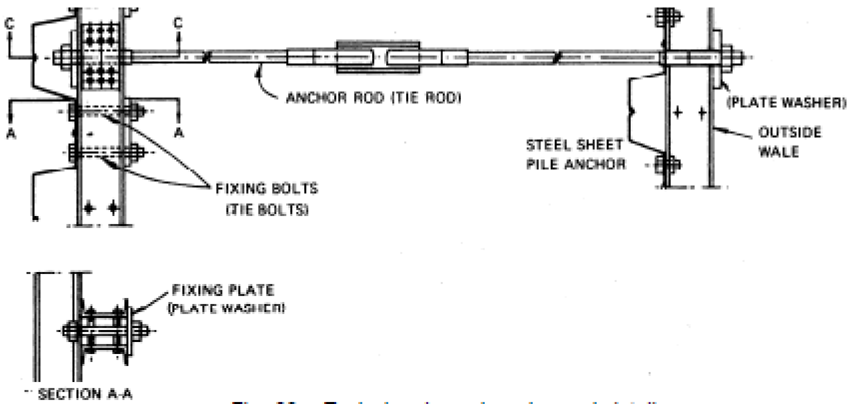


Fig. 39 - Typical wale and anchor rod details

For sizing purposes, the response of a wale may be assumed to be somewhere between that of a continuous beam on several supports (the tie rods) and a single span on simple supports. Therefore, the maximum bending moment for design will be somewhere between

$M_{max} = (1/10)Td^2$ (three continuous spans - simply supported)

$M_{max} = (1/8)Td^2$ (single span - simply supported)

where T = the anchor pull in pounds per foot (before increase).
d = distance between rods in feet (center to center)

$$T := F_{Anch} = 62 \cdot \frac{kip}{ft}$$

$$d := S = 8.25 \cdot ft$$

Conservatively..

$$M_{max} := \frac{1}{8} \cdot T \cdot d^2 = 527.484 \cdot kip \cdot ft$$

Minimum Required section modulus for 50ksi steel will be

$f_y := 50 \cdot ksi \qquad \Omega := 1.67$

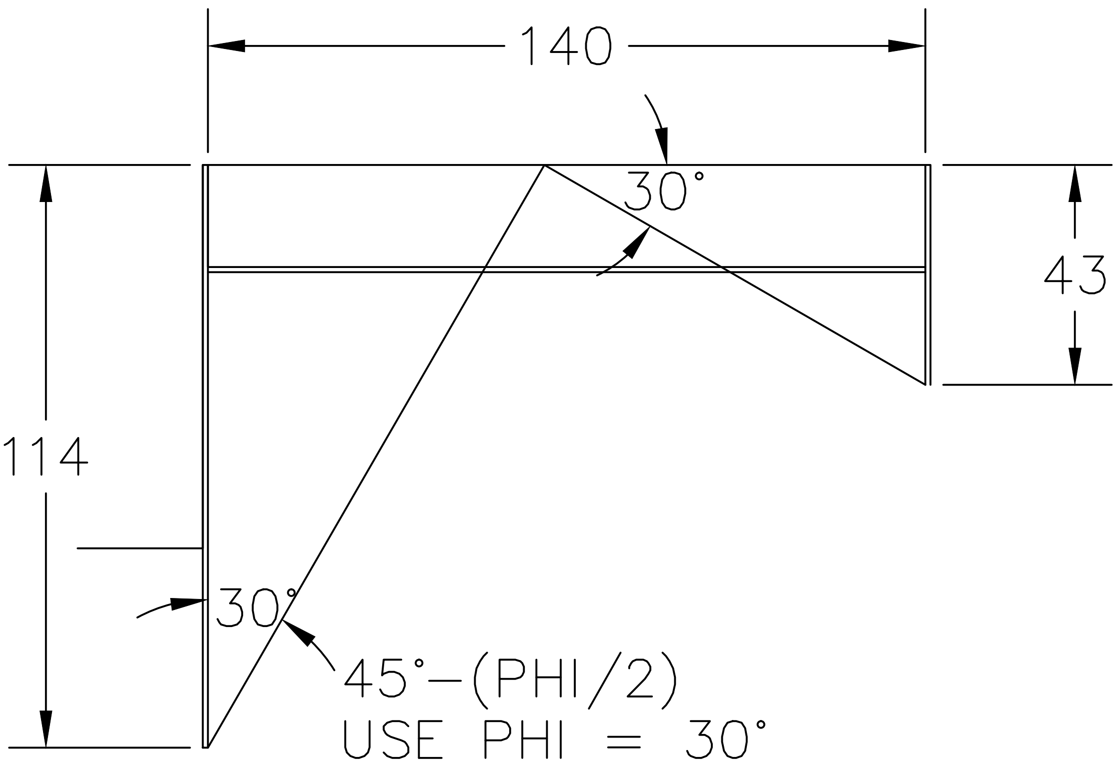
$$S_{min} := \frac{M_{max} \cdot \Omega}{f_y} = 211.416 \cdot in^3$$

Try using (2) WF shapes per wale (instead of channels as shown above)

$$S_{req} := \frac{S_{min}}{2} = 105.708 \cdot in^3$$

Use (2) W21X50 per wale

KIPNUK WALL
TIE ROD LENGTH



Appendix E:
Permitting Requirements

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Kipnuk Engineering Analysis

Permitting Research

April 2016

Project Scope

The purpose of the Kipnuk study is to determine the most suitable combination of solutions to mitigate three hazards: erosion, ground settlement, and flooding. The permitting research presented below is focused on Kuguklik River bank stabilization work.

Natural Resources

AECOM researched publically available information on agency websites to summarize the natural resources in the vicinity of the project area. This information can be used to guide project design and evaluate potential impacts associate with project construction. A summary of AECOM's research is provided below.

AECOM consulted the U.S. Fish and Wildlife Service's (USFWS) IPaC online system[1] for a preliminary assessment of natural resources potentially impacted by the proposed project. This web application is designed to assist private citizens and public employees who need information to assist in determining how their activities may impact sensitive natural resources managed by the USFWS. Based on the IPaC Trust Resources Report generated for the project location, the Spectacled Eider (*Somateria fischeri*), which is listed as a Threatened species by the USFWS, and multiple species of migratory birds may occur in the project vicinity. There are no critical habitats in this location. An official Threatened and Endangered (T&E) species list can be obtained from the USFWS during project permitting.

AECOM reviewed USFWS National Wetlands Inventory (NWI) mapping[2] to verify the presence of wetlands in the project area. According to the NWI map generated for the project location, the area consists of estuarine / marine deepwater and freshwater emergent wetlands. The NWI mapping is prepared from analysis of high attitude imagery and often does not involve on-the-ground verification. An onsite delineation of wetlands may help expedite project permitting (see further discussion under the permitting section).

The Alaska Department of Fish and Game (AD&G) Fish Resource Monitor[3] was viewed to identify anadromous water bodies in the vicinity of the project. According to these maps, the Kuguklik River is an anadromous water body (AWC Code: 335-10-16600-2197-0040).

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[1] USFWS IPaC: http://ecos.fws.gov/ipac/

[2] USFWS NWI Mapping: http://www.fws.gov/wetlands/Data/Mapper.html

[3] ADFG Fish Resource Monitor: http://extra.sf.adfg.state.ak.us/FishResourceMonitor/?mode=culv

The Office of History and Archaeology maintains a data repository called the Alaska Heritage Resources Survey (AHRS) with information on reported cultural resources in Alaska[4]. AECOM was granted permission to use the AHRS Mapper to identify the presence or absence of reported cultural resource sites in the vicinity of projects for baseline research. According to the AHRS Mapper, there are several reported sites around Kipnuk. The exact locations and type of sites are not publically available. An evaluation of the project area by a cultural resource professional may help expedite project permitting (see further discussion under the permitting section).

The National Marine Fisheries Service provides an Essential Fish Habitat (EFH) Mapper[5] as an online tool for the public to identify habitats identified as necessary to fish for spawning, breeding, feeding or growth to maturity. According to the EFH mapper, no EFH or Habitat Areas of Particular Concern were identified at the project location.

Permitting

The following permits / agency consultation would likely be required for construction of a bank stabilization project along the Kuguklik River. Permitting of a chosen alternative would occur after the final design is in advanced stages and prior to construction. Other permits in addition to the ones listed below could be required once the scope of the project is fully defined.

- U.S. Army Corps of Engineers Department of the Army Permit and National Environmental Policy Act
- Consultation for T&E species and Critical Habitat
- National Marine Fisheries Service EFH Assessment
- State Water Quality Certification
- Alaska Pollutant Discharge Elimination System Construction General Permit
- Alaska Department of Fish and Game Fish Habitat Permit

U.S Army Corps of Engineers Department of the Army Permit and National Environmental Policy Act

The U.S Army Corps of Engineers (USACE) issues permits under the following authorities: 1) Section 404 of the Clean Water Act, which covers the discharge of dredged or fill material into waters of the U.S., including wetlands and 2) Section 10 of the Rivers and Harbors Act of 1899, which covers work in or affecting navigable water of the U.S.. These permits are often referred to as Department of the Army (DOA) permits. Based on AECOM's research, estuarine / marine deepwater and freshwater emergent wetlands are present in the project area and would be affected by project activities.

[4] AHRS Mapper: https://dnr.alaska.gov/parks/oha/ahrs/ahrs.htm

[5] EFH Mapper: http://www.habitat.noaa.gov/protection/efh/efhmapper/index.html

There are two types of DOA permits, 1) Nationwide Permit and 2) Individual Permit. The proposed project activities do not appear to fall under any of the 2012 Nationwide Permits available in Alaska; therefore, an Individual Permit would be required. A summary of the 2012 Nationwide Permits can be found at:
http://www.poa.usace.army.mil/Portals/34/docs/regulatory/Summary_Table_2012%20NWPs_14%20Feb%202012.pdf

There are three primary steps to the Individual Permit process:

1) Pre-application (Optional). Meeting between the applicant, USACE, and resource agencies (Federal. State, or local) prior to submittal of a written application. The basic purpose of the Pre-application meeting is to facilitate discussions about a proposed activity before the applicant makes irreversible commitments of resources (funds, detailed design, etc.) The pre-application process is intended to provide the applicant with an assessment of the viability of some of the more obvious alternatives available to accomplish the project purpose, to discuss measures for reducing the impacts of the project, and to inform the applicant of factors the USACE must consider in it decision-making process.

2) Formal Review. After USACE receives a written application they will determine if the application is complete. Additional information may be requested if the application is determined incomplete. Once the application is determined complete the USACE will begin their formal review. During the formal review USACE determines if they have authority over project activities and issues a Public Notice to initiate public review. Depending on the accuracy of the NWI mapping, the USACE may require the applicant to conduct a wetlands delineation by a wetlands professional in order to make their Jurisdictional Determination.

3) Evaluation and Decision. During this time the USACE evaluates the environmental impacts of the proposed project and all public comments received during the public comment period and explains its decision in a decision document. This document may include a Categorical Exclusion, an Environmental Assessment (EA), Environmental Impact Statement (EIS), a statement of findings or record of decision, a section 404(b)(1) guidelines evaluation (if necessary), and a public interest review evaluation. The USACE's decision to issue or deny the permit is usually made within 120 days of receipt of application.

DOA application materials include:

- Written Application – Form ENG 4345. Includes information on the project location, description, purpose and need of the project, type of discharge, and surface area and volume of fill material.
- Mitigation Statement – statement to demonstrate how impacts to U.S. waters and wetlands will be avoided, minimized, and compensated. The mitigation statement needs to be submitted with the written application. Information on applicant mitigation statements can be found at:

http://www.poa.usace.army.mil/Portals/34/docs/regulatory/applicantproposedmitigationstatements.pdf.

- Project/Property Location Map – overhead view of property that shows where proposed project will be built. Include landmarks, useful features, property boundaries, proposed project boundaries, MTSR, and latitude / longitude.
- Project Plan View – Drawing to show the plan view of the proposed project, including details and dimensions of the proposed project (e.g., length, width, fill dimension and volumes).
- Project Cross-Section View – Drawing showing the cross section of proposed project. Include all dimensions and quantities associated with the proposed project.

National Environmental Policy Act:

The National Environmental Policy Act (NEPA) of 1969 requires that prospective impacts of projects be understood and disclosed prior to a Federal agency issuing a permit or providing funding for a project. If the significance of environmental impacts from the proposed action is not clearly established and the activities do not fall under a list of categorically excluded actions from NEPA, the USACE as the lead permitting agency, will need to prepare an EA and may require the applicant to provide appropriate information necessary for the preparation of the EA. The purpose of the EA is to determine if the project will cause significant effects. If the EA concludes that no significant impacts will occur, a Finding of No Significant Impact is prepared and is used to support USACE's permit decision. In the unlikely event that the EA for the proposed activity identifies significant impacts, an EIS would be required. Preliminary discussions with USACE in 2016 indicated that a small EA would probably be required for a river bank erosion revetment project (sheet pile walls, rip rap, articulating concrete block matting, etc.) or a community drainage project.

DOA Permit Strategy:

An informal pre-application meeting with the USACE Regulatory Division and appropriate resource agencies would be beneficial to discuss the proposed project prior to submitting an application. During the pre-application meeting the applicant can seek regulatory input regarding potential environmental impacts, ways to reduce or minimize impacts, and determine if any studies, data, or analyses are needed to support the permitting effort (e.g., wetlands delineation and data to support an EA, as necessary).

The project area falls within the North Section which is coordinated through the Alaska District Office located on JBER in Anchorage. Contact information is provided below:

Alaska District Office
P.O. Box 6898
JBER, Alaska 99506-0898
(907) 753-2712

The Purpose and Need (P&N) for the project needs to be well crafted. Incorporating a safety component provides a strong case for permitting. An alternative analysis summary can also helpful to demonstrate how the proposed design was selected and ideally that it has the least environmental impact.

The mitigation statement is another key component of the permit application. Some initial thoughts on mitigation are provided below for future discussion.

Avoidance – Avoidance may not be possible due to the nature of the project. A discussion of negative effects that could result with no action (i.e., continued erosion threatening village housing and infrastructure) will be helpful to stress the need for the project.

Minimization – If possible, the applicant should demonstrate that the project footprint has been minimized to the maximum extent. A discussion of other alternatives considered will also be helpful if there are alternatives with greater impacts that were eliminated or if there are alternatives with less impacts that are not feasible.

Compensation – Compensatory mitigation means, for the purposes of the USACE regulatory program, the restoration, establishment, enhancement, or protection/maintenance of wetlands and/or other aquatic resources for the purpose of compensating for unavoidable adverse impacts which remain after all appropriate and practicable avoidance and minimization have been achieved. If possible, demonstration that the project would enhance aquatic habitat (i.e., by mitigating continued bank erosion) could be used as a starting proposal for compensatory mitigation. However, typically unavoidable impacts are required to be mitigated at a 2:1 (or greater) ratio of mitigation to direct impacts

Consultation for T&E Species and Critical Habitat

The Endangered Species Act requires Federal agencies to consult with the USFWS and the National Marine Fisheries Service (NMFS), as appropriate, if any activity that requires Federal authorization may affect T&E species and critical habitat. The USACE, as the Federal permitting agency, will make a determination on the effect the proposed project may have on any listed (or proposed) T&E species and their critical habitat and determine the appropriate consultation with the USFWS and NMFS. As a result of the consultation process, the USACE may add special conditions to the DOA permit to ensure the activity does not jeopardize T&E species or adversely modify critical habitat. Based on AECOM's preliminary research there are no critical habitat concerns but Spectacled Eiders, which are listed as a Threatened species by the USFWS, may occur in the project vicinity.

Essential Fish Habitat Assessment

The Magnuson-Stevens Fishery Conservation and Management Act requires the identification of EFH, which is defined as waters necessary for fish for spawning, breeding, feeding, or growth to maturity. This law requires Federal agencies to consult with the NMFS on proposed actions that are permitted, funded, or undertaken by the agency that may adversely affect EFH. USACE will make a determination of project impacts on EFH and will consult with the NMFS during the DOA permitting process. Any comments that the NMFS may have concerning EFH will be considered in their final assessment. Based on AECOM's preliminary research, no EFH was identified at the project location.

State Water Quality Certification

Any applicant for a federal license or permit to conduct an activity that might result in a discharge into navigable waters, in accordance with Section 401 of the Clean Water Act of 1977, also must apply for and obtain a certification from the Alaska Department of Environmental Conservation (ADEC) that the discharge will comply with the Clean Water Act, the Alaska Water Quality Standards, and other applicable State laws. By agreement between the USACE and ADEC, application for the DOA permit also serves as application for the State Water Quality Certification. The USACE will coordinate with ADEC to obtain the certification prior to issuing the DOA permit. As necessary, special conditions may be added to the DOA permit and/or water quality certification to ensure water quality standards are met.

Alaska Pollutant Discharge Elimination System Construction General Permit

If the project footprint (e.g., ground disturbance) is greater than one acre, coverage under ADEC's Alaska Pollutant Discharge Elimination System Construction General Permit (CGP) will be required. To obtain coverage under the CGP the operator will need to file a Notice of Intent and prepare a Stormwater Pollution Prevention Plan prior to commencing construction. The goal of the CGP is to minimize erosion and reduce or eliminate the discharge of pollutants, such as sediments carried in storm water runoff from construction sites through implementation of appropriate control measures. Instructions for filing a Notice of Intent and preparing a SWPPP can be found at: http://dec.alaska.gov/water/wnpspc/stormwater/sw_construction.htm.

ADFG Fish Habitat Permit

ADFG has the statutory responsibility for protecting freshwater anadromous fish habitat and providing free passage for anadromous and resident fish in fresh water bodies. Any activity or project that is conducted below the ordinary high water mark of an anadromous stream requires a Fish Habitat Permit (FHP). FHPs are required for the following activities:

- construct a hydraulic project, or
- use, divert, obstruct, pollute, or change the natural flow or bed of a specified river, lake, or stream, or
- use wheeled, tracked, or excavating equipment or log-dragging equipment in the bed of a specified river, lake, or stream.

The Kuguklik River is an anadromous waterbody and any work below the ordinary high water mark will require a FHP. FHPs generally take 30-60 days to process. Instructions for completing a Fish Habitat Permit application can be found at: http://www.adfg.alaska.gov/index.cfm?adfg=uselicense.main.

Cultural Resources

Section 106 of the National Historic Preservation Act requires the USACE to take into account the effects that activities authorized by DOA permits are likely to have on historic properties. The USACE will consult the latest published version of the AHRS for the presence or absence of historic properties and will coordinate with the State Historic Preservation Office (SHPO) during the DOA permitting process. Any comments that SHPO may have concerning presently known archaeological or historic data that may be lost or destroyed by work under the DOA permit will be considered in their final assessment. Based on AECOM's research, there are several reported cultural resource sites around Kipnuk. The exact locations and type of sites are not available using the AHRS Mapper. Depending on the nature and proximity of the reported sites to project activities, SHPO may request an evaluation of the project area by a cultural resource professional to identify potential historic properties that could be affected by the proposed project.

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Appendix F:
Foundation Assessment-Kipnuk Buildings

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Kipnuk Community Foundation Reconnaissance Survey May 2015

A reconnaissance of the previous geotechnical studies was reviewed prior to the site visit. Site inspections of various building foundations were performed between May 26th to 28th, 2015. During the site visit, photographs and visual inspections of the major building foundations, house foundations and general site conditions were conducted. Conversations with local residents was undertaken throughout the visit. Some of the residents expressed concerns about permafrost degradation and mentioned minor problems (doors not shutting etc) associated with some building foundations. However when residents were asked what problems the community was facing, riverbank erosion was the first hazard that almost every person /discussed, and follow-up questions pertaining to permafrost were required before residents would discuss foundation issues. It was obvious the community is most concerned about the riverbank erosion.

Site Report:

Kuguklik Fuel farm: consists of three steel fuel tanks located on the northern edge of the community, upstream of Traditional Council office (one Red and 2 white tanks). The smaller of the three tanks is sitting on four rafts of wooden sleepers. These wooden sleepers showed some slanting towards the center of the tank, indicating differential settlement of the foundation soils. This tank is now located within 20 feet of the edge of the river bank and needs to be relocated in the next couple of years. This section of river bank has been aggressively eroding and steel posts and sheet metal have been installed along a short section the riverbank to minimize and slow this process in front of the tank. As the river bank erodes closer to the fuel farm, the permafrost is expected to degrade further.

The two larger tanks are located at least 100 feet from the riverbank and were constructed on raised cement treated (Golder, 2011) embankments. These embankments raise the base of the tanks approximately 3 feet above the surrounding ground surface. Additionally, four thermosyphon's extend beneath these tanks, however two of the four thermosyphon's are damaged. Due to the vegetation and spill containment surrounding the tanks, it was difficult to visually assess if these larger tanks foundations are moving.

Figure 1: Smaller Tank

Figure 2: Two Larger Tanks

National Guard Buildings: Three buildings in this group are all supported by triodetic steel frames and pad footers. The Southernmost building showed minimal-to-no movement and was in the best condition of the three buildings. It has adjustable (12-inch adjustment) legs connecting the triodetic frame to the footers. These footers were the largest pad footers (3-foot x 3-foot) of the three buildings and each contained 2 to 4-inches of insulation between the tundra and wooden footers. Some of the footers have compressed up to 3-5 inches into the ground surface. The main (painted green) building was tilting significantly towards the west and the triodetic frame was disconnected from some of the footers on the east side. Installing adjustable legs between the triodetic frame to the footers would facilitate re-level the building. The triodetic frame of the older building (constructed in the 1960's) to the east, was supported by small (1-foot to 1.5 foot square) footers and in some places the footers were missing and the frame was sitting directly on the ground surface. The frame should be elevated and re-leveled on adequately sized footers installed beneath it.

Figure 3: National Guard Armory

Figure 4: Sunken Footer

Post Office: in the post office building is supported by 6-inch diameter pipe piles and is raised approximately 4 feet above the ground surface. The foundation experiences differential movement and is routinely jacked/re-leveled every couple of years. This was evident when inspecting the foundation as the pile caps were not level and timber shims were observed at many of these caps. Some of the internal walls were cracked and the post office staff reported some doors jamming during winter months. Corrosion was observed on all piles.

Figure5: Post Office

Water Treatment Plant and Washeteria: Was constructed in 2009 and is located on the south edge of town. The foundation consists of steel piling with black polyethylene used to reduce the adfreeze bond strength within the active layer. The building is raised approximately 5-6 feet above the ground surface to cool the foundation using natural convection. This foundation appears to be performing well with no signs of differential movement of the pile caps or distress to the interior of the building.

Figure 6: Washeteria Piling

City Water Storage Tanks: Located beside the WTP and Washeteria. The smaller tank is supported by larger steel H-piles while the larger tank is founded on slender (4-inch diameter) thermal steel piles. Both tank foundations showed signs of differential settlement.

Board Walks: Some intersections are located in standing/ponded water. Some older boardwalks (pre-2006?) are not anchored and could float during floods. This has not been a problem over past decade and unlikely they will move due to minimal current.

Figure 7: Boardwalk in Ponding Water

Traditional Council Building: The building was constructed in 1974 on timber adfreeze “freeze back” piling with a passive refrigeration system (Golder 2011). Traditional Council staff reported the building moves during winter months, doors become difficult to close during this period. Significant differential elevations between the pile caps was observed during visual inspection of the foundation.

Chief Paul Memorial School is constructed using passive refrigerator piling (thermal piles) and has a history of differential movement since construction. Test borings performed in 1988 at the school site encountered frozen soils consisting of surface peat, organic silt and silt. In the 1990's differential movement was reported and in 1996 Artic Foundations Inc performed a pile cap survey. The survey measured differential movement since construction of ¼ to 4.5 inches. At this time it was determined the movement was caused from heave due to over cooling of the piles and a number of thermal pile were decommissioned. In 2011 Golder also performed a pile cap survey and reported differential movement since the 1996 survey of ¼ to 4.5 inches.

Figure 8: Old School

Figure 9: New School Addition

During our inspection of the Chief Paul Memorial School foundation, insulation damage and standing water observed beneath the school floor in 2011 (Golder, 2011) was still present. It appears the ponding water is periodically pumped from beneath the school as a small 2-inch trash pump was set-up beside one of the ponded areas. At the time of the site visit, the interior of the school was being renovated and was not inspected for structural or cosmetic damage to the internal walls/doors. The pile caps were visually assessed for differential movement and settlement on the perimeter piles of the West, North and East sides of the building. Some piles on the west side of the building showed upwards movement of up to 1-inch, indicating these piles may have heaved in this location. Piles on the east side of the building appeared to be relatively consistent with each other when visually assessed. Two piles on the north side of the building showed up to 1-inch settlement, however it was not clear if these were constructed this way as there were old shims that were inserted in the gap and appears to have been there for many years. The observed movement of the pile caps on the perimeter of the building were within the range (1/4-inch to 4-1/2 – inch) of differential movement previously recorded during pile cap surveys and the differential movement does not appear to be increasing at this date.

If future movement is observed and is causing distress to the building, another pile cap survey could be performed and measurements compared to previous surveys. Additional piles could be decommissioned or decommissioned, as needed.

An expansion to the Chief Paul Memorial School has been under construction for the past 2 years. The expansion consisted of new buildings and an outdoor playground situated on an elevated decking and supported by 8.5-inch diameter steel piles. The majority of the expansion appeared to be completed at the time of the site visit, however some internal renovations were still being performed. The building foundations consisted of 18-inch diameter galvanized steel piles and were reportable driven to refusal at depths of up to 70 feet. The floor of the building is raised approximately 5 to 6 feet above the ground surface, providing an airspace for natural convection

Halibut Processing Plant: Is supported by triodetic steel frames with adjustable legs and pad footers. The building was tilting and the triodetic frame was disconnected from some of the footers. Adjusting the legs and re-cribbing the footers could be completed to re-level the building. The triodetic frame appears ridged and the beams it supports do not appear to have deflected in the north-south directions. The building remaining in the same location as it was during the 2011 inspection by Golders and has not yet been relocated.

Figure 10: Halibut Plant, Unsupported Legs

Since 2011, an number of new building and/or structures were constructed at Kipnuk. These are summarized below:

General Store: this is a new structure that was constructed in 2014 using slender steel piles. The floor of the building is elevated approximately 30 to 36-inches from the ground surface providing space to cool the ground under the building and reinforce the permafrost in winter months. To date the foundation visually appears to be performing well and shows little to no signs of movement.

Church Buildings: Two church buildings were constructed in 2014 and are located on the northeast edge of town. The larger Church building is supported by an adjustable triodetic steel frame on square 2-foot x 2-foot pad footings. The smaller building/residence is supported by wooden beams, cribbing and pad footings. The pad footing for both buildings are resting on granular gravel backfill. The granular backfill appears to have been placed on a geo-fabric directly on the tundra and used to level the footing. The foundations appear to be adequately supporting the buildings and to date they did not show signs of differential movement.

Figure 11: Church Footings

Figure 12: Church Residence Footings

Residential Houses: Most houses kept opening under house clear to allow cooling of foundation in winter. This was generally well done around the community. Some houses stored equipment/supplies, but this didn't appear to be a problem, or a common occurrence.

Figure13: Pile Supported Residence

Figure14: Residence Footing on Undrained Soil

<u>Kipnuk Recommendations:</u>

- Surface drainage plan development would help drain ponded water, improving the " walking on the ground" and minimize the permafrost degradation. When walking around the site, we observed that areas that were drained, showed less soil compression under-foot that areas that were saturated or had standing water. High groundwater and surface water appeared to contribute to the permafrost degradation as the boardwalk showed compression/settlement many areas where water was ponding.
- Drainage would improve the bearing capacity of the soils and should help reduce the permafrost degradation.
- Increase the size of the footings under the boardwalks in location where there is ponded water.
- Raise the boardwalk by propping up the footings in areas of ponded water to reduce wood rotting etc.

- Adjustable legs added to some of the building supported by triodetic frames would facilitate easy re-leveling and even load distribution into the foundation.

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Appendix G:
Cross Sections

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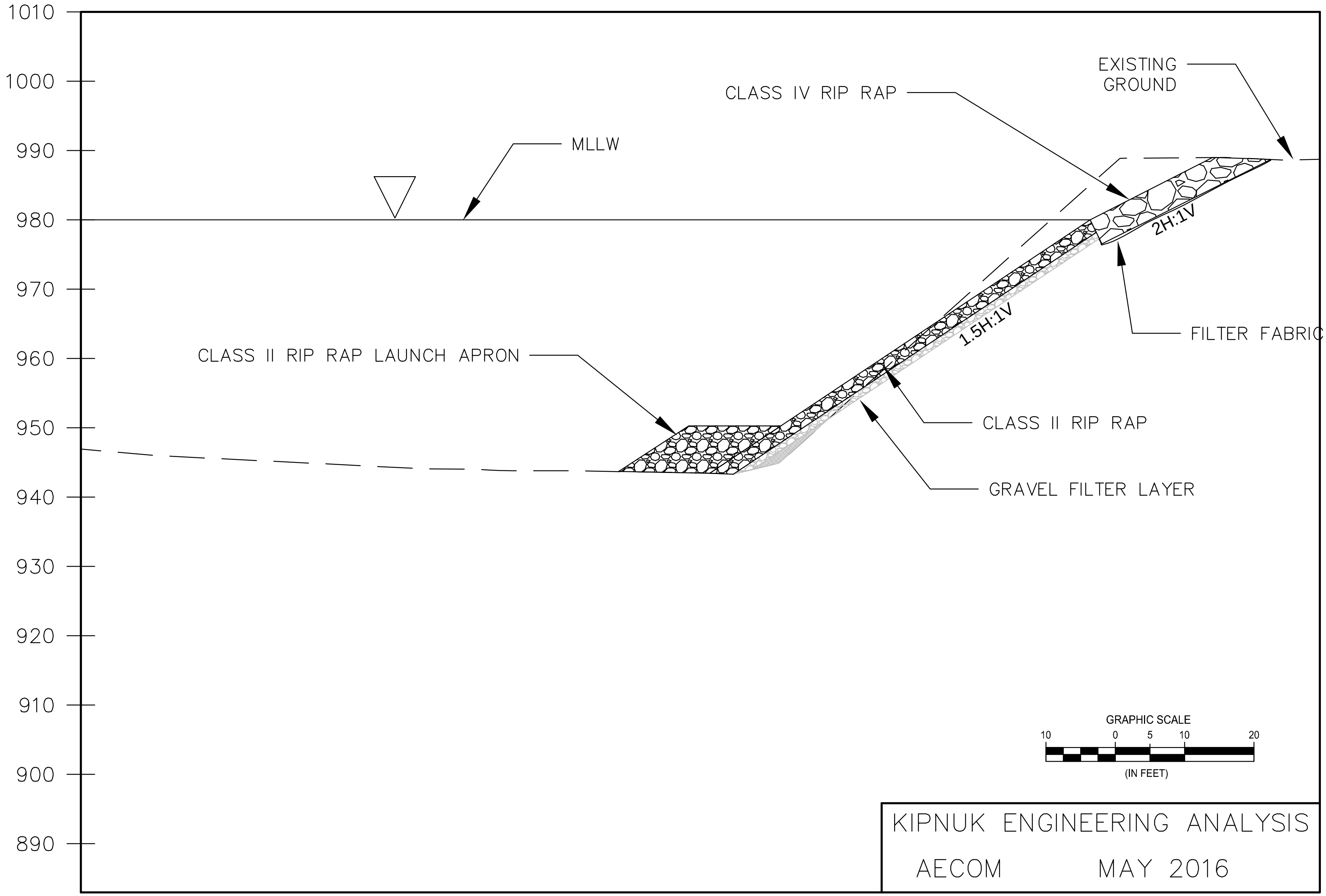

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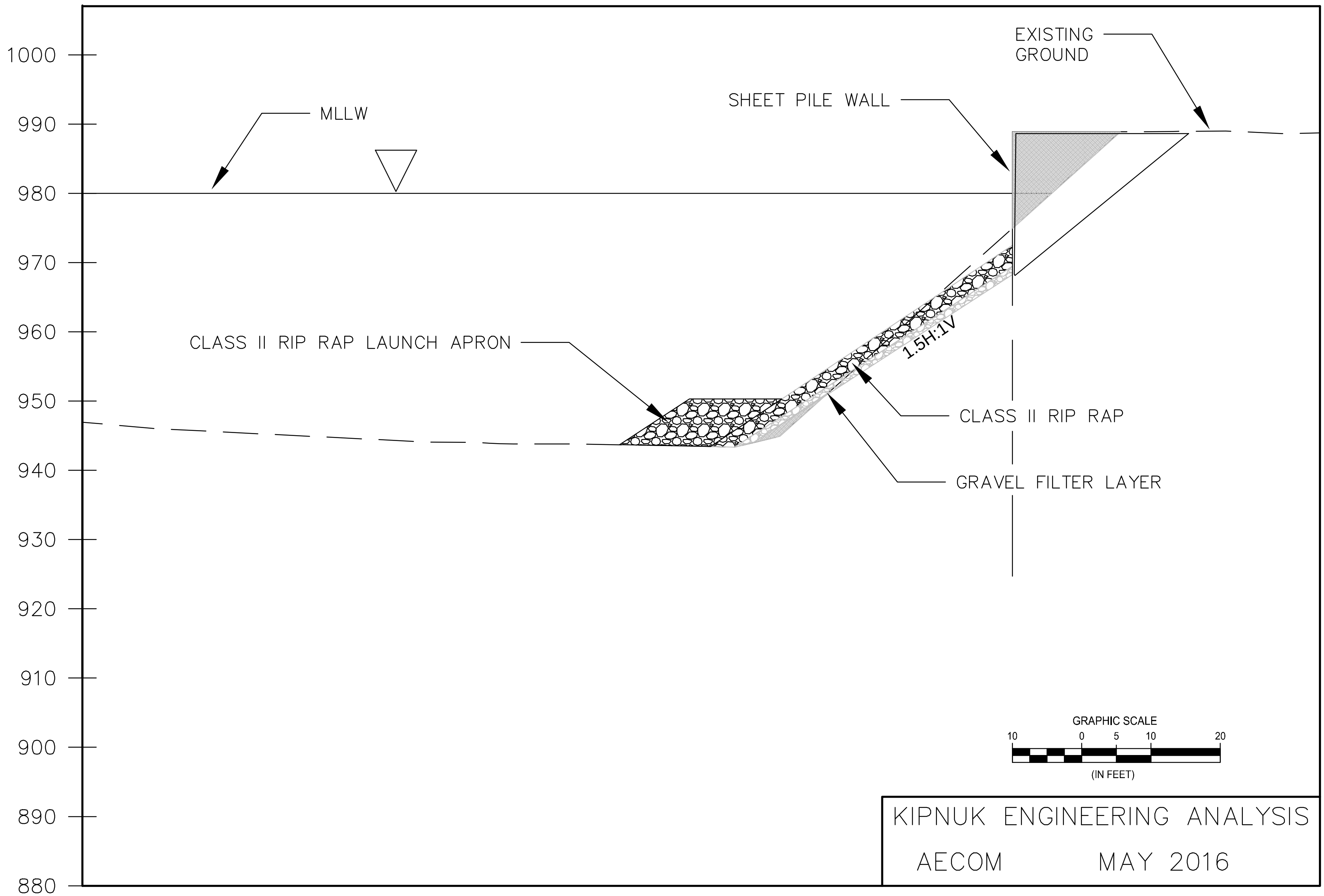

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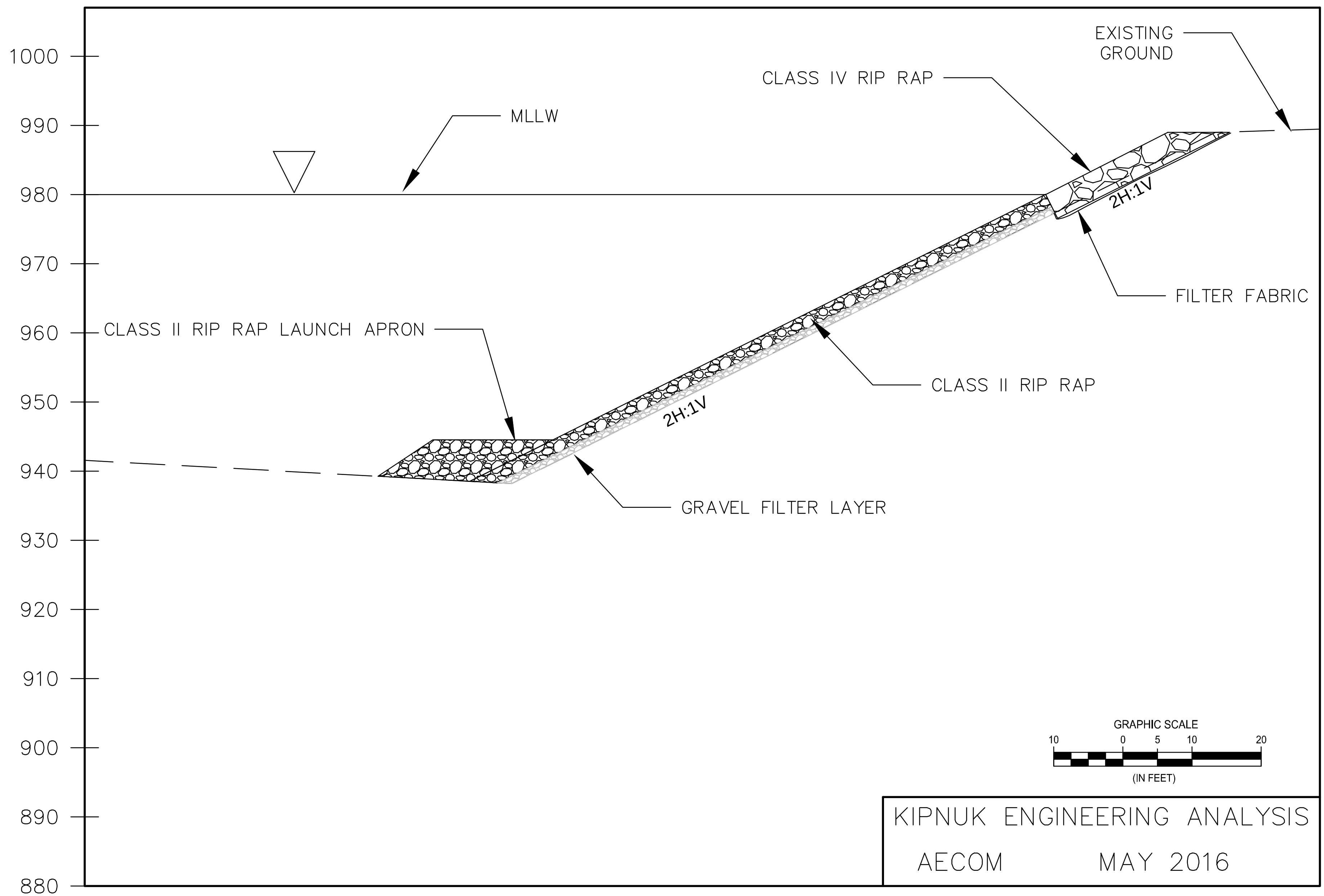

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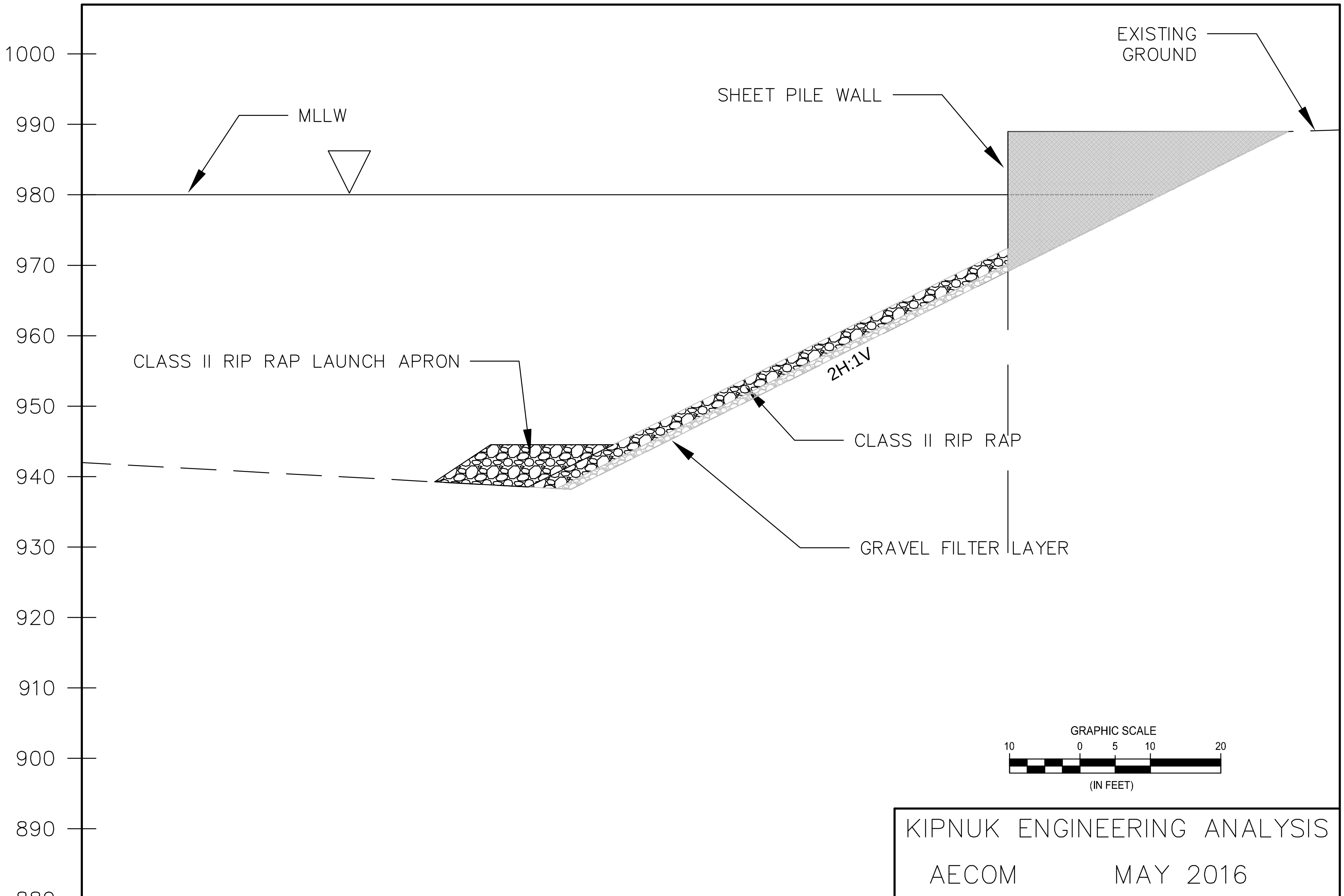

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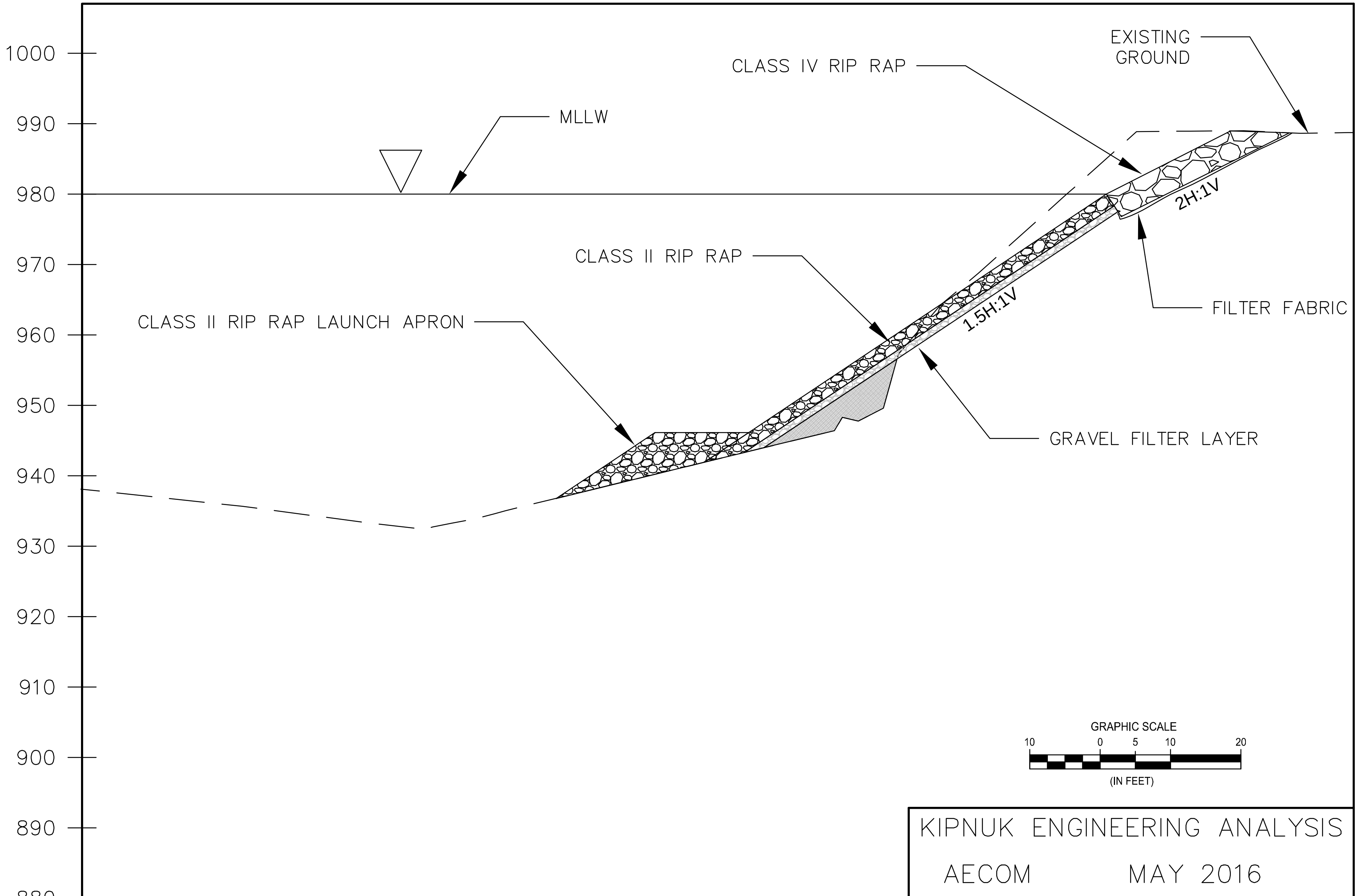

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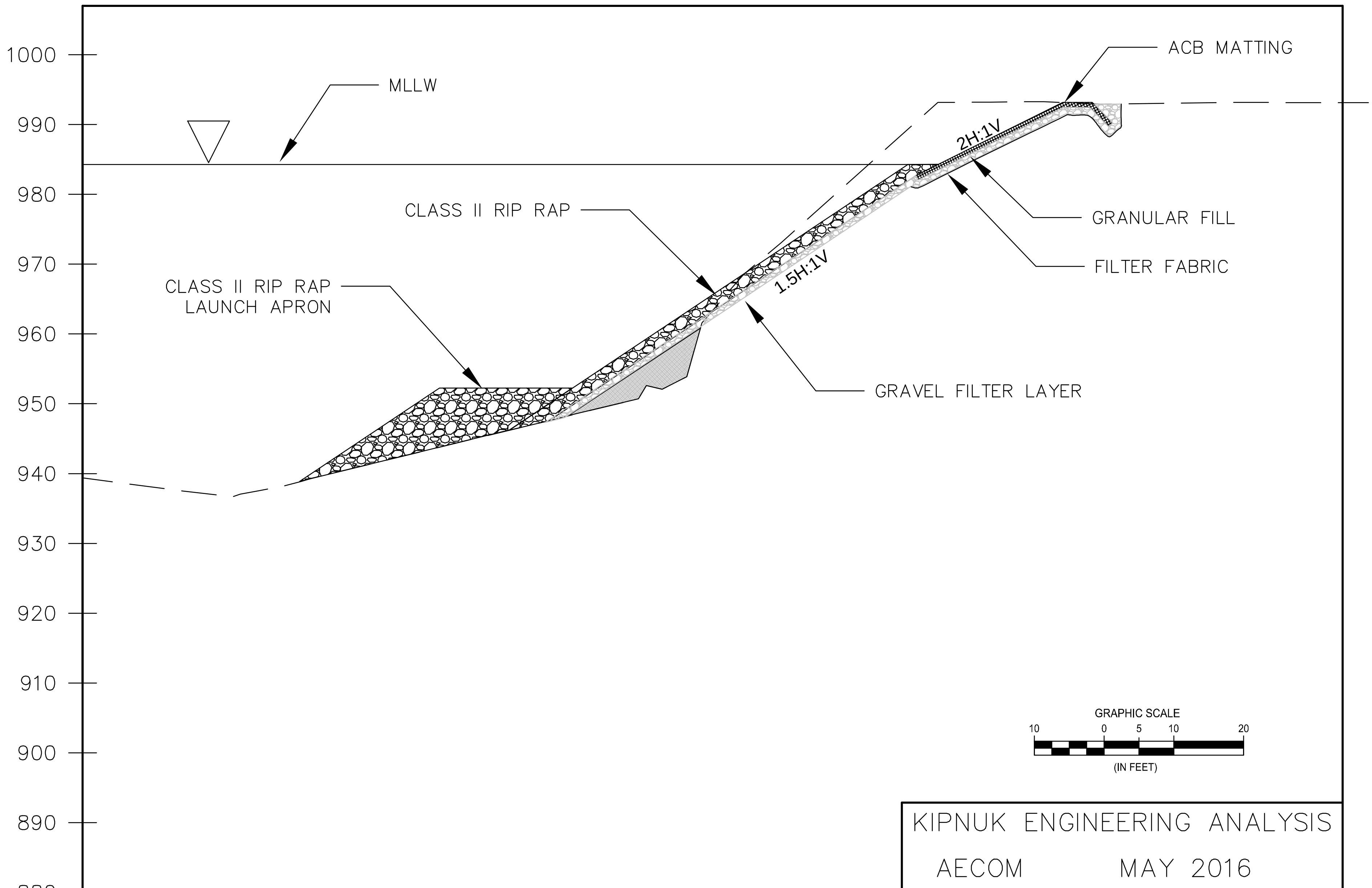

KIPNUK ENGINEERING ANALYSIS

AECOM MAY 2016

RIP RAP WITH ACB MATTING (REACH 3)

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Appendix H:

Community Meeting Notes

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KIPNUK Community Meeting
5.27.2015
3:00 p.m.
Kipnuk Traditional Council Building

Public Outreach Specialist, Donne Fleagle, introduced the Program Manager and herself to the community. There were approximately 12 people present. She explained the basic facts of the grant.

The Program Manager explained the project and said that there will be one more trip to the Community probably in October to explain the results of the Engineering Analysis. He asked for feedback on erosion, flooding and permafrost thawing. He stated that a river survey, a hydraulic survey and a flood study would be done. The Flood study will note the base flood elevation. He stated that the grant will look at a combination to slow down or stop river bank erosion. Appears thaw settlement of permafrost is occurring.

<u>QUESTIONS:</u>

1. HOW IS YOUR STUDY DIFFERENT THAN THE ONE THAT WAS DONE FOR THE TANK FARM? This engineering analysis takes into account hydrology and hydraulic solutions.
2. DID YOU SEE THE STUDY? Yes. We will review it and consider all solutions. It depends on river currents and ice. We have to consider all the solutions. Sheet pile walls are problem. SIMILAR TO BETHEL? We have not seen what is in Bethel. THE GROUND HERE IS CLAY MUD, TUNDRA, NO REAL ROOT SYSTEMS.
3. WHY IS HYDRAULIC? Wave action, scour, ice, velocity.
4. WHEN WILL THE REPORT BE DONE SO THE COUNCIL CAN REQUEST FUNDS? WE HAVE HAD TO MOVE OUR GAS STATION TWO – THREE TIMES. By the end of the year.
5. IS DIVERTING WATER UPSTREAM AN OPTION? We've talked about it but it's not a reasonable option to the consequences such as access.
6. WILL YOU BE LOOKING AT THE RIVER OR SLOUGHS? Mainly the river but we are focusing on the main problem although any plan will need to look at the sloughs.
7. WILL YOU SUBMIT THE REPORT TO THE USACE? No
8. WILL YOU FIX THE PROBLEMS? No. This grant does not fund a solution.

COMMENTS:

1. Sheet Pilings did not work by the tanks.
2. Boulders are not effective over all. They make an eddy sometimes.

3. It floods in the fall, not the spring. The ground is hard and ice is destructive.
4. Low pressures push water u here – a storm surge.
5. Main Creek comes through the village.

KIPNUK Community Meeting
4/05/2016
1:30 p.m.
Kipnuk Traditional Council Building

Kipnuk 4-5-16

Project Manager and Civil Engineer , Peter Crews, described the banks stabaility alternatives 1 through 6 to the community. There were approximately 13 people present, including Kipnuk Traditional Council (KTC) members. Peter explained the high cost of sheet pile alternative and the less costly option of riprap revetment.

KTC members conferred in Yupik following presentation. Alternative 3 was recognized as the most cost effective option and the accepted alternative. KTC members requested that alternative 3 be shortened on the north end, with future phase to extend as needed, in order to bring down the cost of a first phase project. They said the river erosion on the north end is less. Alternative 2 was also favored, with the included barge landing, but there seemed to be agreement that Alternative 3 (riprap reach3) should be the priority project.