APPENDIX I

HYDROLOGIC AND HYDRAULIC REPORT

Hydrologic and Hydraulic Report

Kwigillingok Airport Improvements Project

Final

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Table of Contents

Table of Figures

Table of Tables

Project Location and Description

The Alaska Department of Transportation and Public Facilities (ADOT&PF) wishes to make improvements at the Kwigillingok Airport (Figure 1). Planned improvements include lengthening and widening the runway, providing additional aircraft parking, and addressing erosion of the runway embankment caused by an adjacent unnamed tidal stream.

Erosion is occurring at both the southwest corner of the existing safety area, and adjacent to the northeast corner of the runway. Most of the stream banks in a tidal channel adjacent to the runway appear to be unstable with active erosion. This ongoing erosion is commonly attributed to the drainage of a large lake southwest of the existing runway in the early 1980s when borrow cells along the west side of the runway became interlinked.

This report includes an analysis of the hydrologic characteristics of Kwigillingok, and a hydraulic analysis of the preferred design for runway embankment erosion protection.

Hydraulic History

The stream that flows adjacent to the west side of the existing runway flows out of a drained lake southwest of the runway. Near the southwest portion of the runway, the embankment forces the stream to make a 90° sharp left turn and flow north roughly parallel to the runway embankment. This appears to be at least partly causing embankment erosion near the toe of fill that is approximately 10’ high.

At the end of the runway, the stream turns right to flow southeast. The erosion on the runway embankment here is similar to that near the southwest portion of the runway, except here it is generally 5’ high. From the end of the runway, the stream meanders for approximately 0.5 miles before joining the Kwigillingok River. The Kwigillingok River then flows in a southerly direction for 3.3 miles before emptying into Kuskokwim Bay.

The channel is tidally influenced. On the rising (flood) tide, flow comes up the Kwigillingok River and flows up the channel adjacent to the runway and into the drained lake. Following high tide, the ebb tide flows out the tidal channel to the Kwigillingok River and Kuskokwim Bay.

Typical of most areas in Alaska, there are no long-term gaging records available for the tidal channel or the Kwigillingok River. Additionally, there is no NOAA tide gage station at Kwigillingok. 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 Kwigillingok River and tidal channel (USACE, 2009).

Hydrology

Kwigillingok is located in a maritime climate, approximately 1 mile from the shore of Kuskokwim Bay. 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

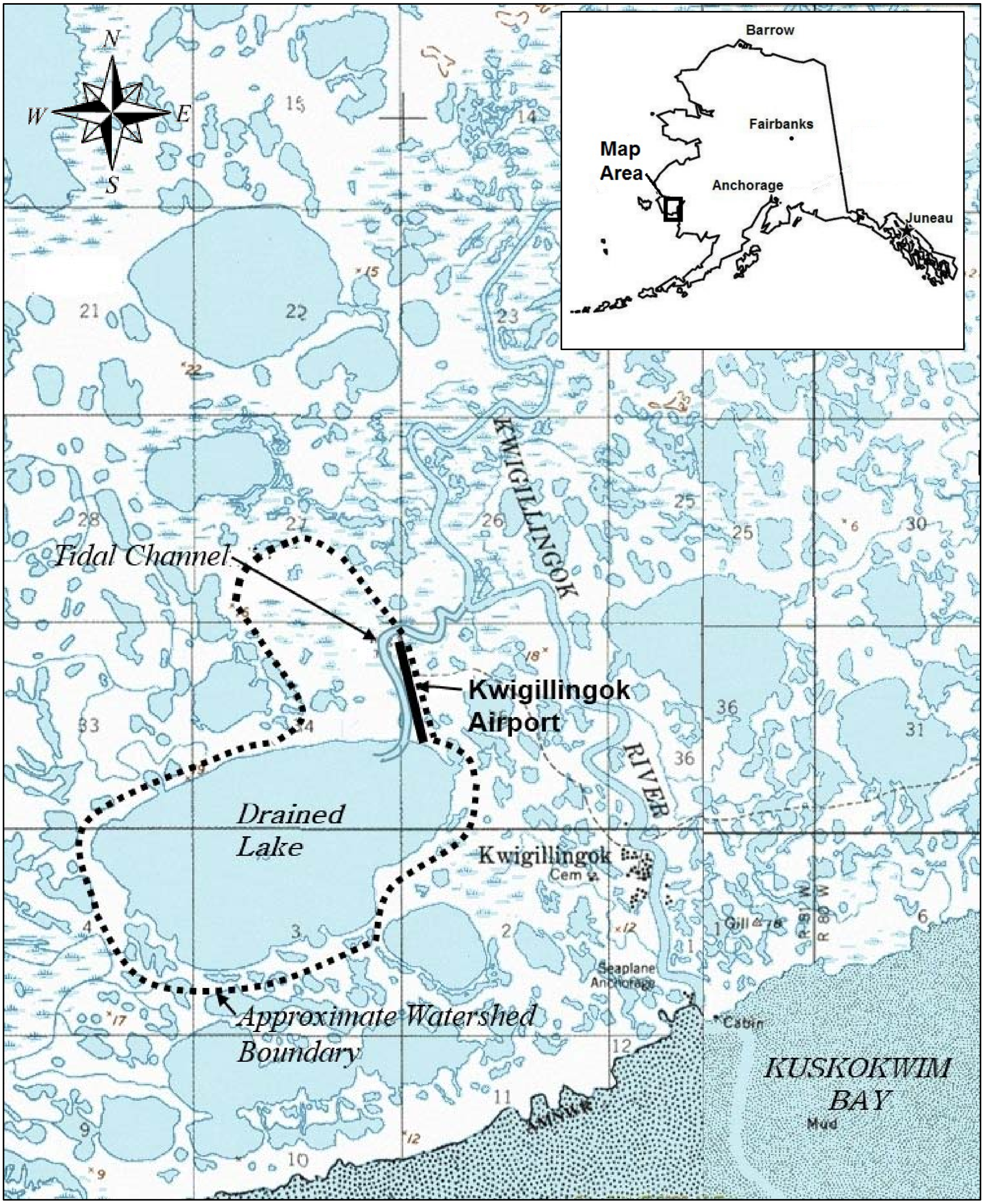

Figure 1. Project location map.

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 Kwigillingok 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, 2012).

Flooding in the Kwigillingok area may be the result of two sources, runoff from precipitation events and/or coastal storm surges. 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. Peak flood magnitudes were estimated for the tidal channel watershed at its confluence with the Kwigillingok River, and for the Kwigillingok River watershed.

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.

Basin characteristics were obtained by planimetric techniques used with USGS 1:63360 quad maps. Due to flat terrain, the planimetered basin characteristics in Table 1 should be considered as an approximation.

Table 1. Watershed characteristics.

	Tidal Channel Watershed	Kwigillingok River Watershed
Drainage Area (mi^2)	2.2	32.8
Area of Lakes and Ponds (%)	61	23
Area of Forests (%)	0	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 two watersheds are 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 these two watersheds 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 tidal channel watershed and the Kwigillingok River watershed are shown in Table 2 and in Appendix 1. 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 Appendix 1.

Table 2. Flood discharges based on precipitation events.

Flood Recurrence Interval	Tidal Channel (cfs)	Kwigillingok River (cfs)
2-year	35	510
5-year	59	745
10-year	77	909
25-year	100	1120
50-year	119	1280
100-year	138	1440
200-year	158	1610

Design Flood Elevation

Project designers require the design flood elevation. 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. Two types of flooding may occur in the Kwigillingok area; runoff from precipitation events and coastal storm surges. 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 tidal channel adjacent to the runway 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 tidal 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 PDC survey crew in August 2011. Fourteen cross-sections, labeled from 637 (downstream) to 9185 (upstream) were used in the model. Station 0 (zero) starts at the confluence of the tidal channel and the Kwigillingok River. 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).

On 08/15/ 2011, PDC, Inc surveyors surveyed a series of water surface elevations during a period from 12:00 pm to 3:41 pm local time at the upstream tidal observation location on the tidal channel, and a set of simultaneous water surface elevations at both the upstream and downstream tidal observation locations at 3:41 pm local time, approximately 2 hours following the observed high tide (see Figure 2). The difference between simultaneous water surface elevations (0.13 feet) and the distance between the tidal observation locations (8,550 feet) were used to estimate a normal depth slope for a calibration of the HEC-RAS model. The estimated slope during the ebb tide is 0.000015 ft/ft. See Appendix 2 for tidal observation and stage data.

By matching the observed water surface elevations in the HEC-RAS model, the discharge in the channel at the time of the survey was estimated to be 227 cfs. See Table 3 for the HEC-RAS results, including hydraulic characteristics at all cross-sections.

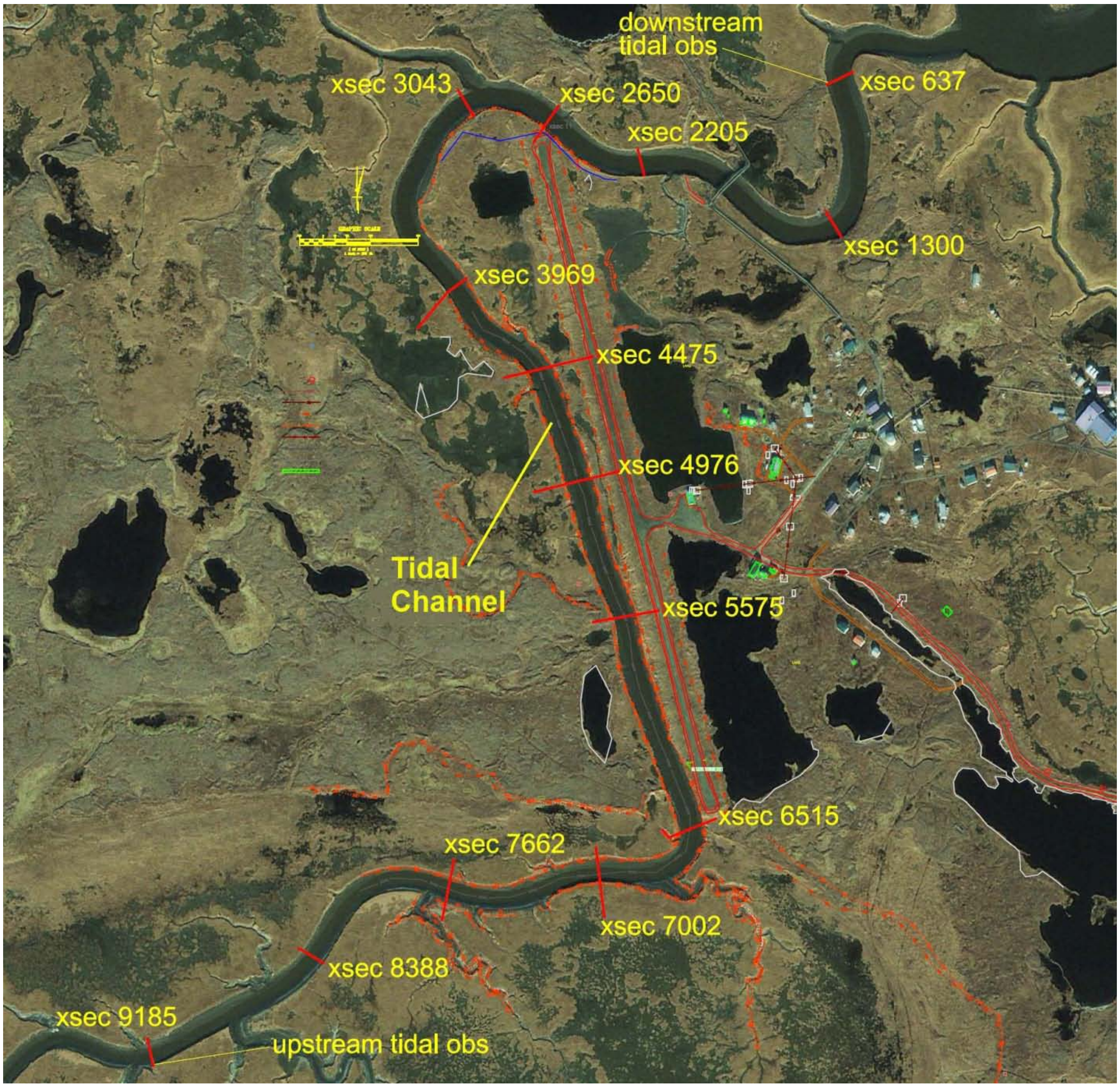

Figure 2. Cross-sections and tidal observation locations for HEC-RAS hydraulic analysis.

A comparison of the estimated flood magnitudes in Table 2 with the channel hydraulic analysis shows that the outgoing flow that occurred in the tidal channel on 08/15/2011 (227 cfs) was substantially greater than the predicted precipitation-based 100-year (138 cfs) and 200-year (158 cfs) floods for the tidal channel watershed. As no large storm events had occurred immediately preceding the August survey, the flow 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)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)
9185	227.0	0.98	6.78	0.78	291.04	78.35
8388	227.0	1	6.75	0.69	328.85	87.28
7662	227.0	-0.54	6.73	0.56	407.94	96.48
7002	227.0	-0.36	6.72	0.55	409.34	86.31
6515	227.0	-0.69	6.72	0.50	453.65	102.61
5575	227.0	-1.1	6.70	0.50	455.45	94.91
4976	227.0	-0.79	6.7	0.49	467.80	105.48
4475	227.0	-1.43	6.69	0.48	468.48	94.99
3969	227.0	-2.35	6.68	0.48	477.40	92.20
3043	227.0	-2.18	6.67	0.46	492.66	96.65
2650	227.0	-2.25	6.66	0.38	597.16	103.02
2205	227.0	-2.71	6.65	0.41	549.10	98.10
1300	227.0	-0.92	6.63	0.42	536.97	102.41
637	227.0	-2.57	6.62	0.36	635.75	114.59

Tidally affected river crossings are characterized by both 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 may not be 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 Kwigillingok area, 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. The community of Kongiganak, located approximately 10 miles east of Kwigillingok, was included in the study. The stage-frequency analysis for Kongiganak is found in Table 4. Stage units are feet mean lower-low water (ft MLLW).

Table 4. Stage-frequency analysis for Kongiganak, AK (Chapman et al., 2009).

Return Period (years)	5	10	15	20	25	50	100
Surge Level (ft MLLW)	10.31	12.57	13.92	14.84	15.43	17.03	18.28
Std. Deviation (ft)	0.52	1.05	1.21	1.38	1.28	1.35	1.28

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). Though near-shore bathymetric data wasn't available for this study, Kwigillingok and Kongiganak share similar characteristics, including aspect and an open sea to the south. This suggests that results from the USACE storm surge stage-frequency analysis for Kongiganak are applicable for use at Kwigillingok.

To estimate the Kwigillingok MLLW elevation, eight years of daily predicted tide levels were obtained for a nearby subordinate station located at Apokak Creek (NOAA, 2012). For the period of record, the MLLW was calculated as the mean of the lower twice-daily low tide levels. The estimated MLLW at Apokak Creek is 0.04 feet.

A lack of data prevents a direct correlation of the Apokak MLLW datum to MLLW at Kwigillingok. However, the 1:40 p.m. high tide measurement at Kwigillingok for August 15, 2011 (8.69 feet) is similar to the 2:37 p.m. high tide prediction for Apokak (8.6 feet). Based on the lack of additional tide data, and the proximity of the Apokak Creek station to Kwigillingok, the estimated MLLW at Kwigillingok is 0.0 feet.

Based on the stated assumptions, the 100-year storm surge elevation at Kwigillingok is estimated at 18.3 feet.

Design Elevation

The design elevation for the Kwigillingok airport is guided by requirements that the runway should be raised to a level above the 100-year flood elevation. Based on the tidal channel hydraulic analysis and the storm-induced water level prediction study for the western coast of Alaska (Chapman et al, 2009) and other assumptions described above, the estimated surge level and design build elevations for the Kwigillingok airport are found in Table 5.

Table 5. Design elevation for Kwigillingok runway.

100-year storm surge elevation	18.3 ft
design elevation (1-foot freeboard)	19.3 ft
design elevation (3-foot freeboard)	21.3 ft

Bank Erosion Analysis

According to a 1993 ADOT&PF memo, the lake southwest of the existing runway was drained in the early 1980's when borrow cells along the west side of the runway became interlinked. A series of aerial photographs show that the channel has continued to straighten and widen. Though the lake has drained, tidal currents continue to affect channel geometry.

The tidal channel is eroding the southwest corner of the existing safety area and the bank adjacent to the northeast corner of the runway. Determining the reason for the bank erosion along the tidal channel is necessary for designing embankment erosion protection and assessing tidal channel alignment alternatives.

Four possible causes of erosion were considered: pore-water pressure, frost heave phenomenon, long-term permafrost melting, and boundary shear stress. The first three causes are discussed qualitatively; a quantitative analysis was conducted to estimate bank erosion from boundary shear stress. The analysis was based in part on logs of test holes bored by the ADOT&PF in the vicinity of the Kwigillingok airport in February 2012. The logs reveal that subsurface soils in the project area are primarily silts, silts with sand, and silts with organics.

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 Kwigillingok airport include the soil type (silt) and the large rapid variation in the hydrograph. The spring range (mean difference between high and low tidal levels during “spring tides”) for the Apokak Creek entrance NOAA subordinate tide station, used for this project as a reference station for Kwigillingok, is 12.0 ft. The variation in tide levels occurs approximately every 6 hours.

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 Kwigillingok airport is high. Bethel has a subarctic climate, with a minimum monthly temperature below freezing for 7 continuous months. Bore holes show predominantly silty soils in the region. Additionally, ice lenses of 3.5 to 6 inches thick were noted in 4 test holes; TH12-11, TH12-15, TH12-16, and TH12-44. Though a water table was not evident in the test hole logs, the relatively high water content and degree of saturation of the silt layers may provide enough water for upward or lateral movement and the development of ice lenses.

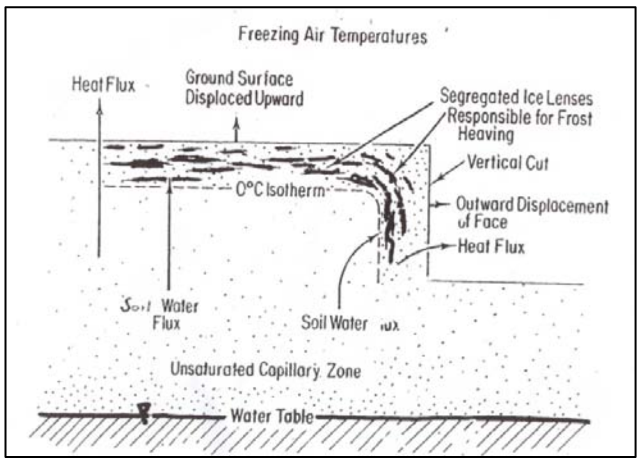

Figure 3. Vertical and horizontal surface displacement from frost heave.

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).

In August 1972, five test holes were placed by ADOT&PF along the centerline of the proposed runway at Kwigillingok, between the southern edge of the runway (adjacent to the lake) and the parking apron at mid-runway. Two additional test holes were also bored; one beyond the northern end of the runway and one on the access road. Six of the holes indicated permafrost to depth of hole (10-20 ft) and the seventh hole indicated frozen soil to 7.5 ft and unfrozen ground between 7.5 and 20 ft (Moores, 1972)

In February 2012, 47 test holes were bored by ADOT&PF in the vicinity of the runway and proposed expansion areas adjacent to the runway. At nine of the holes at the northern and southern ends of the runway, data loggers with temperature probes were installed and recorded temperature data from March 2012 through September 2012. The data indicates that at most of the holes, seasonal frost now occurs rather than permafrost. At one hole, soils at 8 and 12 ft depth did not freeze for the duration of the logging period (Steff Browne, personal communication, October 22, 2012). Soil temperatures did not appear to be influenced by tidal activity.

A recent study (Karle, 2014) concluded that thawing permafrost and subsequent thermokarsting of soils and vegetation that had been disturbed by construction activities and/or activities by local residents was likely responsible for the establishment and downcutting of the tidal channel over time in the early 1980s. However, there is currently not enough data available to determine if melting permafrost is responsible for the ongoing bank erosion.

Erosion by Shear Stresses

The USDA Bank Stability and Toe Erosion Model (BSTEM) was utilized to estimate erosion of the bank and bank toe by hydraulic shear stress. The model estimates boundary shear stress from channel geometry and considers critical shear stress and erodibility of two separate zones with potentially different materials: the bank and bank toe (USDA, 2012).

Cross-section 6515, surveyed at the high eroding bank at the southwest corner of the runway, was used to provide the bank geometry for the analysis. Soil layer data, including thickness and material type, were obtained from the log for boring hole TH12-16. The input reach slope was based on water surface elevations surveyed simultaneously at two locations on the tidal channel, upstream and downstream of the runway. The analysis was run at the high tide water surface elevation surveyed by PDC, Inc in August 2011. The model does not use a discharge value. Analyses were conducted for two flow durations: 4 hours, representing two 2-hour high tide levels in one day, and 1460 hours, representing high tide levels for approximately one year.

Rates of erosion for the bank, bank toe, and bed are found in Table 6 and Appendix 3.

Table 6. BSTEM model results for existing tidal channel.

Cross-section 6515		1 day duration	1 year duration
high tide	average boundary shear stress (Pa)	0.30	0.30
	maximum lateral retreat (cm)	1.1	195

In summary, four potential reasons for bank erosion were considered. There is currently not enough data available to determine if melting permafrost is responsible for bank erosion. The prevalence of easily erodible silt soils, ice lenses, a subarctic climate, and large rapid variations in the tidal hydrograph indicate that pore water pressure and frost heave may both have a role in the ongoing bank erosion. However, rates of erosion due to these factors are extremely difficult to quantify.

The quantitative analysis of channel shear stress indicates an erosion rate for the existing channel of approximately 6 feet per year. Though specific rates may vary with better data for the

BSTEM model, the analysis indicates that the material type, depth, and water velocity found in the tidal channel can be responsible for bank erosion by hydraulic shear stress.

The BSTEM analysis also indicates that erosion rates will increase slightly if the channel length is shortened. It is important to note that the increased erosion predicted by the model is due to the steeper slope of the shortened channel(s). However, it is not clear if a steeper slope in a tidal channel will result in a steeper water surface slope. In non-tidal streams, gravity and riverbed friction are the primary forces that determine the water's velocity and depth. However, in this tidal channel, where the amount of daily tidal flow far exceeds upland flows, the upstream velocity and depth are determined by the downstream tailwater (tidal elevation).

Analysis of Preferred Alignment Alternative

PDC, Inc. Engineers has prepared a scoping report that describes the development and evaluation of several airport alternatives. Preliminary proposals generally involved altering the size and/or location of the runway embankment and associated safety areas, and analyses were conducted to review the hydrologic aspects of each alternative. The preferred design includes the realignment of the tidal channel to a route approximately 400 feet west of the runway (Figure 4). By moving the channel away from the runway, this design eliminates the need for erosion protection along the west side of the runway embankment.

To realign the tidal channel, a new channel approximately 2500 feet in length must be excavated. To determine how channel geometry affects the channel hydraulic performance, four channel shapes were selected for analysis. The four geometries are described in Table 7 and shown in Figure 5.

Table 7. Four channel design geometry alternatives.

Design A	Design B	Design C	Design D
Shallow V-shaped, no flat channel, banks 7:1 for 6 vertical feet then 2:1 for 4 vertical feet	Shallow V-shaped channel with 10 foot flat middle, banks 7:1 for 6 vertical feet then 2:1 for 4 vertical feet	V-shaped channel with 20 foot flat middle; banks 4:1 for 8 vertical feet then 2:1 for 4 vertical feet	V-shaped channel with 30 foot flat middle, single slope banks 4:1

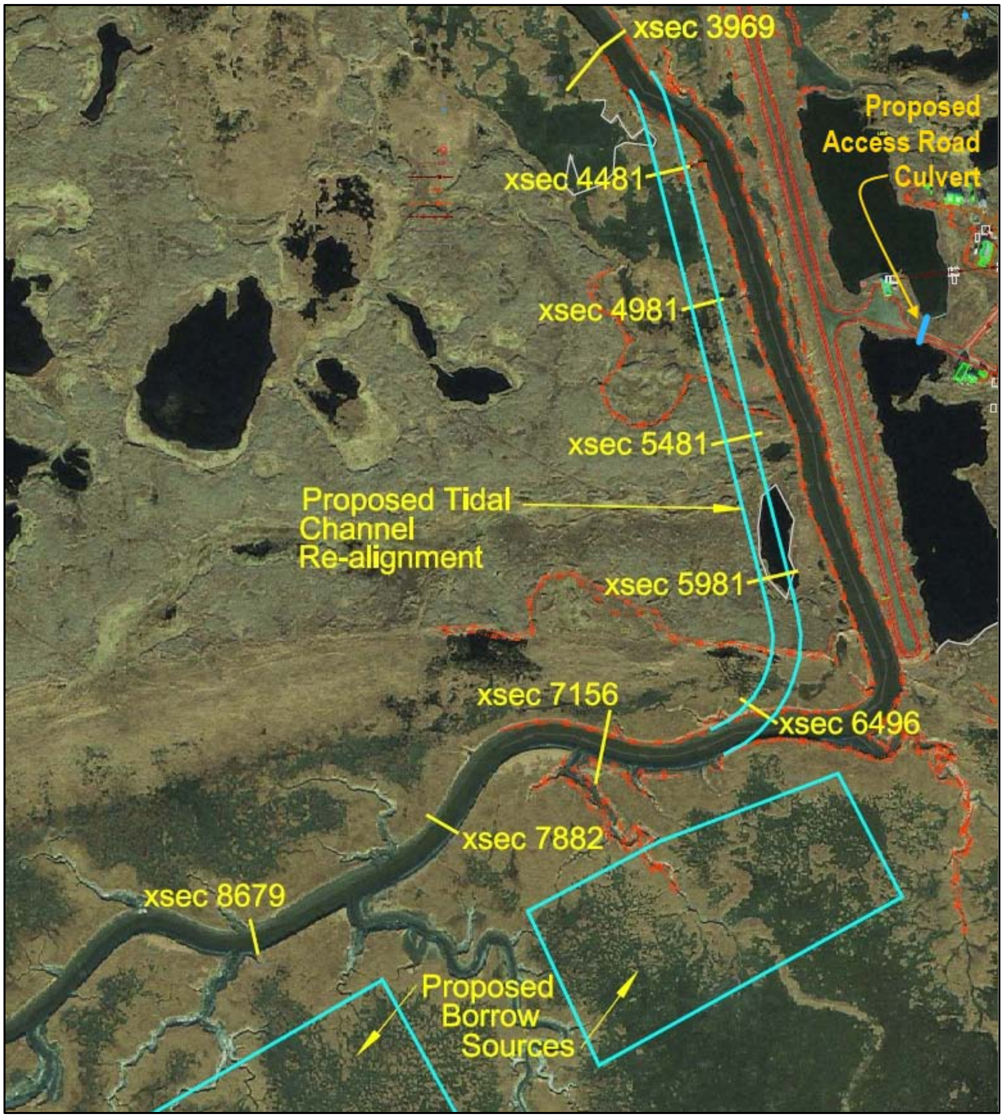

Figure 4. Proposed tidal channel realignment and HEC-RAS cross-sections.

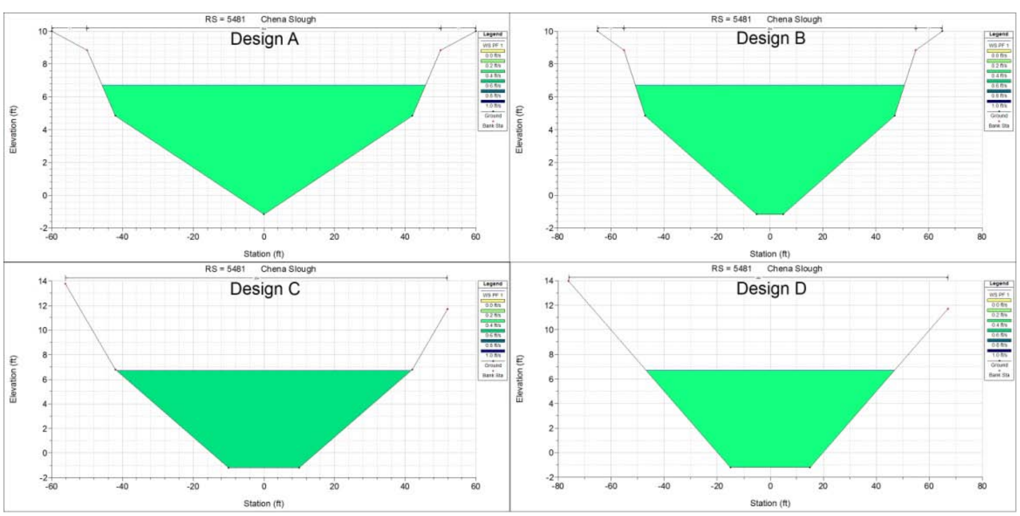

Figure 5. Design shapes for tidal channel realignment.

HEC-RAS Analysis

The tidal channel realignment results in a shorter channel than the existing channel it replaces. The realignment shortens the existing 3270-foot reach by 506 feet, upstream of Cross-section 3969. The slope of the existing channel is 0.0006 ft/ft. The slope of the shortened channel is 0.0007 ft/ft. The hydraulic modeling is based on the assumption that the same quantity of water will flow upstream during the flood tide and downstream during the ebb tide, even though the channel is shorter and slightly steeper. Results shown are from Cross-section 5481, located approximately in the center of the new channel alignment. A comparison of average channel velocities and other characteristics for the four channel design geometries is shown in Table 8. Channel shapes and water surface elevations for the 4 design geometries are shown in Figure 5.

Table 8. HEC-RAS results for 4 channel design alternatives.

HEC-RAS Results				
	Design A	Design B	Design C	Design D
Flow Area at Cross-section 5481 (Q = 227 cfs)	416.0 ft^2	494.0 ft^2	408.3 ft^2	488.6 ft^2
Average Velocity at Q= 227 cfs	0.55 ft/s	0.46 ft/s	0.56 ft/s	0.46 ft/s
Top Width at 227 cfs (wsel = 6.70 feet)	91.5 ft	101.5 ft	83.3 ft	93.7 ft

Erosion by Shear Stresses

The BSTEM was utilized to estimate erosion of the bank and bank toe by hydraulic shear stress for the four channel design alternatives. For each alternative, the analysis was conducted at cross-section 5481, located mid-channel. Soil layer data from adjacent boring hole logs were used. Layer 1 (0' to 4') was resistant cohesive silt; layer 2 (4' to 12') was erodible cohesive silt. Two tide levels were modeled: mean tide at 5.2 feet MLLW, and high tide at 9.9 ft MLLW. The duration of flow was 1460 hours (2 hours per tide cycle x 2 cycles/day x 365 days/year). The model results are found in Table 9 and Appendix 4.

Table 9. BSTEM model results for 4 channel design alternatives.

	Alternative Channel Design	A	B	C	D
mean tide	average boundary shear stress (Pa)	0.11	0.11	0.11	0.11
	maximum lateral annual retreat (cm)	20.5	20.5	41.7	9.3
	total eroded bank area (m^2)	0.57	0.57	0.32	0.33
high tide	average boundary shear stress (Pa)	0.25	0.25	0.15	0.21
	maximum lateral annual retreat (cm)	71.1	71.7	58.6	61.9
	total eroded bank area (m^2)	3.38	3.38	2.18	2.08

Given the lack of detailed soils data and annual tide elevation information, the results from the BSTEM model should be viewed as a general predictive tool, rather than for specific erosion rates. For all alternatives, the maximum lateral erosion rates for the high tide elevation are relatively similar, within 20 percent or so. Design C does indicate the lowest rate of annual erosion. Additionally, the Design C channel shape mimics some existing cross-sections, especially at the upper 4 feet where the banks steepen to a 2:1 (H:V) slope. That feature was noted in several of the surveyed cross-sections.

The cross-sectional area of Design C is somewhat smaller than the other alternatives. This may have the advantage of restricting the upstream flow during the flood tide. As a result, overall discharge in both directions may be smaller than with the other designs, which have larger cross-sectional areas. However, a smaller constructed cross-section will likely erode over time such that it matches the adjacent channel geometry.

The flat 20-ft channel bottom of Design C may be easier to construct, as heavy equipment will have a level platform to operate on.

Tidal Channel Realignment Recommendations

- At the upstream connection, the new channel turns to the north in a left-hand turn away from the older (and subsequently abandoned) existing channel. A large radius of curvature will reduce the bank erosion on the outside bend of the new channel. The radius of curvature for the bend should be no less than 400 ft, to match the existing channel geometry.

- At the downstream connection, the new channel turns slightly to the northwest to rejoin the existing channel. The alignment correction is much less severe, and a smaller radius of curvature, similar to channel bends downstream of the new alignment, can be used.

- New channel junctions: the new channel segment should be blended smoothly into the existing channel at the upstream and downstream connections. At the channel junctions, the existing channel segments should be backfilled with material excavated from the new channel alignment. Countermeasures should be constructed to reduce the potential of backfill erosion. At each end within the channel segment to be filled, geotextile-encapsulated soil lifts should be constructed approximately 20 feet back from the channel toe. The face and tops of the two soil lift structures should be covered with backfill such that there is a seamless transition along the banks and tops of banks from the existing channel to the new channel segment. A preliminary design, based on recommendations from Mitch Miller (ADOT&PF), is shown in Figure 6 (Janke, 2013).

- At the new channel junctions, special effort should be focused on establishing a vegetative mat along the top of the new banks. The vegetative mat should cover the new fill at the junctions and provide a continuous coverage along both banks, from existing channel to new channel to existing channel.

- The new channel alignment is expected to intersect several smaller ponds in the area west of the airstrip. Ponds that are 2 feet or less in depth are not expected to have much effect on channel performance or stability. Ponds that are deeper and are only partially dissected by the new channel alignment may eventually erode such that the bottom of the pond matches the channel thalweg. Additionally, dissected ponds may result in wider channel widths over time. A potential solution would be to fill any deep pond remnants that are dissected but outside of the new channel boundaries.

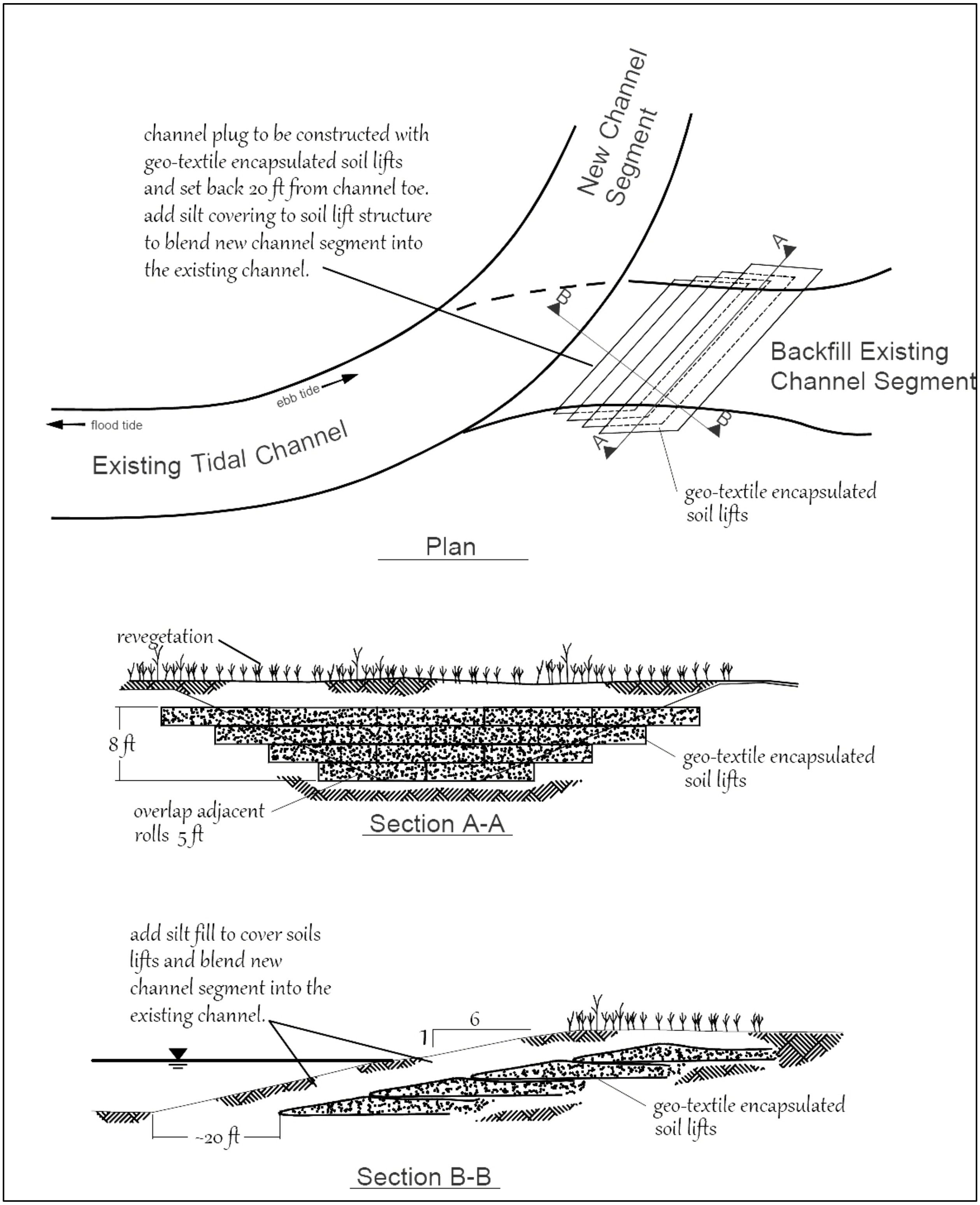

Figure 6. Geo-textile encapsulated soil lift design, to reduce erosion of backfill. Structures are placed in the existing channel segment at the upstream and downstream junctions with the new channel segment.

- The existing channel to be abandoned should be filled with silt to the elevation of the existing undisturbed ground if possible. If sufficient fill is not available, then the channel should be filled starting at both junctions and moving toward each other. Any gaps in the fill should result in a single shallow pond near the center of the existing channel.

- The top of the fill should be revegetated such that the old channel width has a full and complete covering. Rolled erosion control blankets or other bio-degradable methods may be used to assist the revegetation effort. Transplanted vegetation mats harvested during the new alignment excavation may be utilized. This action is especially critical at the junctions, where efforts should focus on restoring a robust and seamless vegetative mat.

- Two borrow sources are proposed to be located in the drained lakebed to the south of the tidal channel (see Figure 2). Within the limits of property boundaries, material quality and other restrictions, the borrow sources should be located as far as possible from the tidal channel. There should be no excavated connection or trench between the borrow pits and the tidal channel.

East Embankment Shoreline

The east side of the runway embankment is located adjacent to two long lakes. Prevailing winds are north/south. The greatest fetch length is also in the north/south direction; the east/west fetch is substantially shorter. A review of photographs taken during the June 2011 field trip to Kwig shows no noticeable erosion of the runway embankment along the lengths of the two lakes.

A shallow embankment slope and successful grass seeding effort should provide erosion protection along the east runway embankment. A rolled erosion control blanket may be used to provide immediate erosion protection following construction until revegetation occurs.

Access Road Culvert

A pond is adjacent to the east portion of the runway south of the apron and adjacent to the south side of the airport access road. This pond occasionally drains north across the airport access road surface and into a second pond. A culvert should be installed across the access road to keep the water below the runway and apron surface and off the road surface. See Figure 4.

The elevation of the culvert inlet invert will control the water surface elevation of the pond. PDC surveyors measured the water surface elevation at multiple points at the edge of the south pond near the proposed culvert crossing area. Survey points 11856 and 11666 were used to determine the water surface elevations of the south and north ponds (12.0 ft and 10.5 ft respectively).

The culvert should be long enough to extend from the south pond to the north pond, approximately 85 feet in length. The USDA program WinTR-55 was used to determine the peak discharge from the south pond drainage basin. The rainfall intensities for the 24-hour 50-year and 24-hour 100-year storms for the Kwigillingok area were obtained from the National Weather Service, which provides precipitation frequency estimates for Alaska (NOAA, 2013). Due to flat terrain, delineation of the drainage area that drains into the south pond is difficult. Two sub-areas were delineated within the small drainage area, with an estimated combined area of 15.6 acres.

The FHWA HY-8 culvert analysis program was used to size the culvert. A 2.0-ft diameter corrugated metal pipe culvert should provide adequate hydraulic capacity for the 24-hour 100-year design storm and minimize long-term maintenance needs. See Appendix 5 for details. Erosion-resistant aprons should be constructed at the inlet and outlet.

23 CFR

There are no regulated 100-year floodplains on this project.

Conclusion

Flooding in the Kwigillingok area may be the result of two sources, runoff from precipitation events and/or coastal storm surges. An analysis of both types of floods concludes that coastal storm surges are the dominant 100-year flood.

The 100-year storm surge elevation at Kwigillingok is estimated at 18.3 feet. The 1-foot freeboard design elevation for the Kwigillingok runway is 19.3 feet.

The tidal channel is eroding the southwest corner of the existing safety area and the bank adjacent to the northeast corner of the runway. Four possible causes of bank erosion were considered: pore-water pressure, frost heave phenomenon, long-term permafrost melting, and boundary shear stress. There is currently not enough data available to determine if melting permafrost is responsible for the ongoing bank erosion. The prevalence of easily erodible silt soils, ice lenses, a subarctic climate, and large rapid variations in the tidal hydrograph indicate that pore water pressure and frost heave may both have a role in the ongoing bank erosion.

The preferred airport design includes the realignment of the tidal channel to a route approximately 400 feet west of the runway. The primary advantage is to eliminate the need for erosion protection along the west side of the runway embankment. To realign the tidal channel, a new channel approximately 2500 feet in length must be excavated.

At the upstream connection, a large radius of curvature should be constructed to reduce the bank erosion on the outside bend of the new channel. The radius of curvature for the bend should be no less than 400 ft, to match the existing channel geometry.

The new channel segment should be blended smoothly into the existing channel at the upstream and downstream connections. Counter-measures such as geotextile encapsulated soil lifts should be constructed to prevent bank erosion at the junctions.

The new channel alignment is expected to intersect several smaller ponds in the area west of the airstrip. Though smaller ponds are not expected to have much effect on channel performance or stability, deeper dissected ponds may eventually erode such that the bottom of the pond matches the channel thalweg. Additionally, dissected ponds may result in wider channels over time.

The existing channel to be abandoned should be filled with silt to the elevation of the existing undisturbed ground if possible. The top of the fill should be revegetated such that the old channel width has a full and complete covering, especially at the junctions. Rolled erosion control blankets, harvested vegetated mats, or other bio-degradable methods may be used to assist the revegetation effort.

The borrow sources should be located as far from the tidal channel as possible.

A culvert should be installed across the access road between the south and north ponds adjacent to the east of the runway to keep the water below the runway and apron surface and off the road surface.

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Appendix 1-Flood Magnitudes

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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: Tidal Channel Watershed
 Region 6
 Drainage area, in square miles:                              2.20
 Percent of area in lakes and ponds:                         61.0
 Forest cover, in percent:                                     0.0

    T     DISCHARGE    SE (+%)     SE(-%)     CONFIDENCE LIMITS   EQ. YEARS
            (cfs)                                5%         95%
     2         35.       61.8      -38.2         16.         79.        2.4
     5         59.       64.2      -39.1         26.        135.        3.3
    10         77.       69.1      -40.9         32.        184.        4.1
    25        100.       76.9      -43.5         39.        260.        5.0
    50        119.       83.4      -45.5         43.        327.        5.4
   100        138.       90.4      -47.5         47.        405.        5.8
   200        158.       97.7      -49.4         51.        494.        6.1
   500        186.      108.0      -51.9         55.        631.        6.4
 WARNING - Lakes+Ponds out of range of observed data
 Range:    0.00 to   15.00 for Region 6

 Flood frequency estimates for
 Site: Kwig River Watershed
 Region 6
 Drainage area, in square miles:                             32.80
 Percent of area in lakes and ponds:                         23.0
 Forest cover, in percent:                                     0.0

    T     DISCHARGE    SE (+%)     SE(-%)     CONFIDENCE LIMITS   EQ. YEARS
            (cfs)                                5%         95%
     2        510.       54.9      -35.4        246.       1060.        1.6
     5        745.       56.8      -36.2        351.       1580.        2.2
    10        909.       60.7      -37.8        411.       2010.        2.7
    25       1120.       67.1      -40.2        476.       2640.        3.3
    50       1280.       72.5      -42.0        516.       3180.        3.7
   100       1440.       78.2      -43.9        550.       3790.        3.9
   200       1610.       84.3      -45.7        579.       4460.        4.1
   500       1830.       92.8      -48.1        611.       5470.        4.3
 WARNING - Lakes+Ponds out of range of observed data
 Range:    0.00 to   15.00 for Region 6
```

Appendix 2-Surveyed Tidal Elevations

Simultaneous Stage Data

Pt no.	Northing	Easting	Station Elevation	Description	Water Surface Elevation
19057	49029.83	27296.65	6.57	Upstream Tidal Obs Location	6.77' observed at 3:41pm local on 8/15/11
19056	53160.55	30135.94	7.58	Downstream Tidal Obs Location	6.64' observed at 3:41pm local on 8/15/11

Tidal Observations

Tidal observations were observed at point 19057, the Upstream Tidal Observation Location, from 12:00pm local to 3:00pm local on 8/15/11

Time (local)	Water Surface	
12:00 PM	7.01	
12:10 PM	7.29	
12:20 PM	7.56	
12:30 PM	7.87	
12:40 PM	8.04	
12:50 PM	8.34	
1:00 PM	8.40	
1:10 PM	8.54	
1:20 PM	8.62	
1:30 PM	8.67	
1:40 PM	8.69	*Observed High Tide
1:50 PM	8.67	
2:00 PM	8.62	
2:10 PM	8.51	
2:20 PM	8.32	
2:30 PM	8.12	
2:40 PM	7.95	
2:50 PM	7.75	
3:00 AM	7.52	
3:41 AM	6.77	

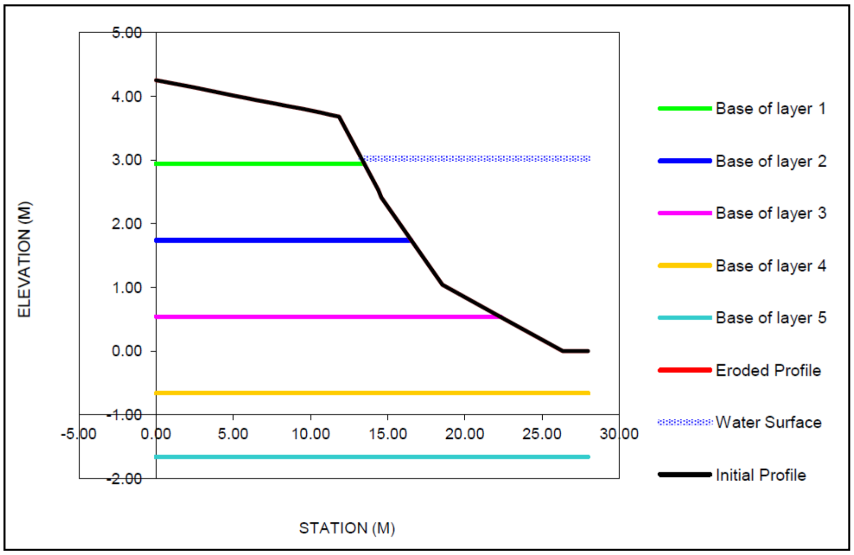
Water Surface Elevations

Water Surface Elevation Location 1

Pt no.	Northing	Easting	Elevation	Description
11268	50389.65	30065.20	12.35	Water Surface Elevation
11269	50356.79	30074.49	12.28	Water Surface Elevation
11270	50350.89	30095.66	12.32	Water Surface Elevation
11271	50327.70	30094.37	12.21	Water Surface Elevation
11272	50292.79	30068.45	12.05	Water Surface Elevation
11273	50246.78	30020.03	12.15	Water Surface Elevation
11274	50215.02	29963.42	12.22	Water Surface Elevation
11275	50187.92	29910.82	12.36	Water Surface Elevation
11276	50160.87	29885.91	12.24	Water Surface Elevation
11277	50132.00	29852.16	12.13	Water Surface Elevation
11278	50137.69	29808.99	12.17	Water Surface Elevation
11279	50146.68	29776.62	12.16	Water Surface Elevation
11280	50126.23	29752.34	12.19	Water Surface Elevation
11281	50103.53	29736.55	12.31	Water Surface Elevation

P:\2010\F10101\L\Survey Memo\PDC Kwig Hydro Survey Data 2011

Appendix 3- Erosion Analysis Graphical Output From BSTEM Model for Existing Tidal Channel



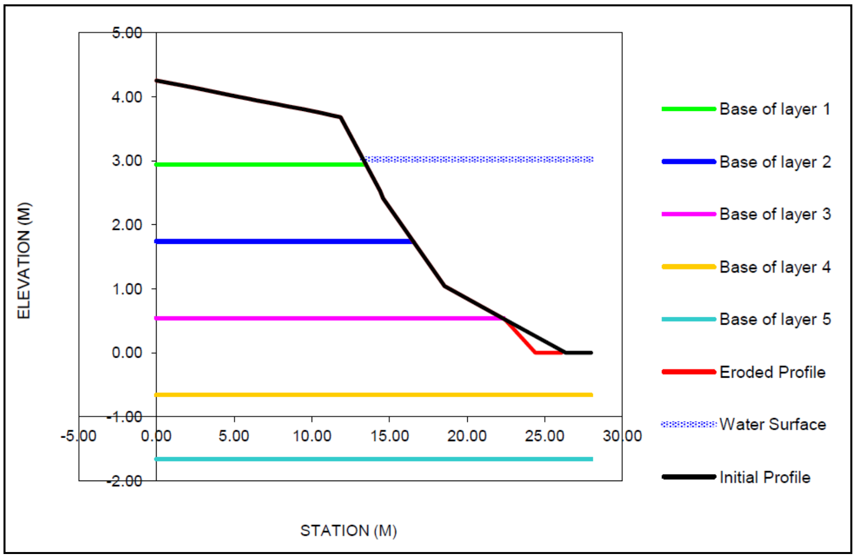


Figure 7. Results from BSTEM analysis of bank erosion at existing tidal channel cross-section 6515, for 1 day (upper) and 1 year (lower).

Appendix 4- Erosion Analysis Graphical Output From BSTEM Model for Four Tidal Realignment Channel Designs

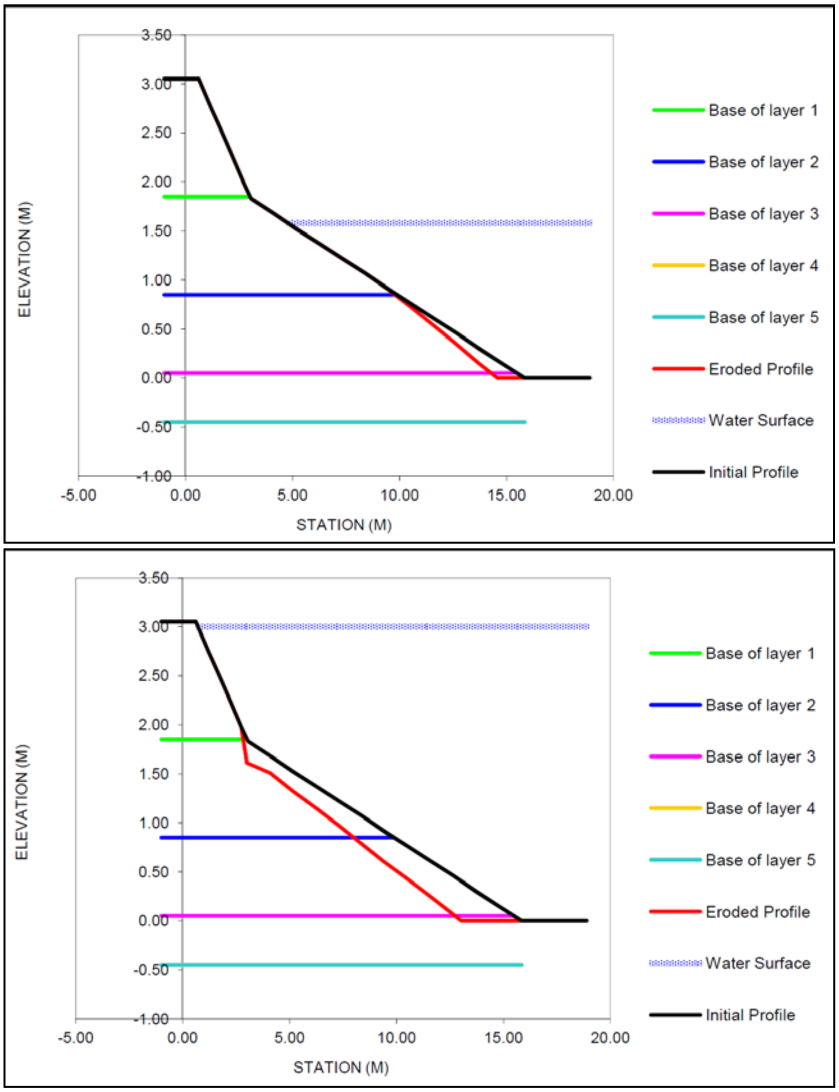


Figure 8. Designs A & B, mean tide (upper) and high tide (lower).

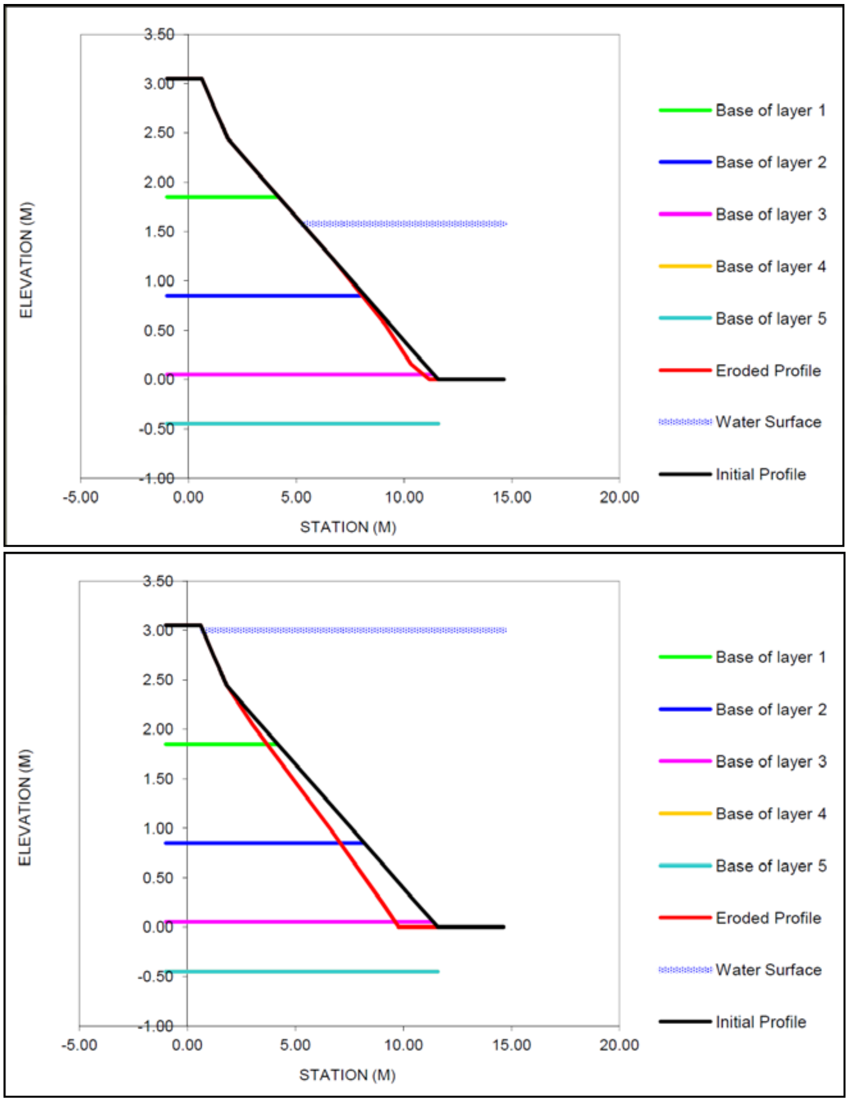


Figure 9. Design C, mean tide (upper) and high tide (lower).

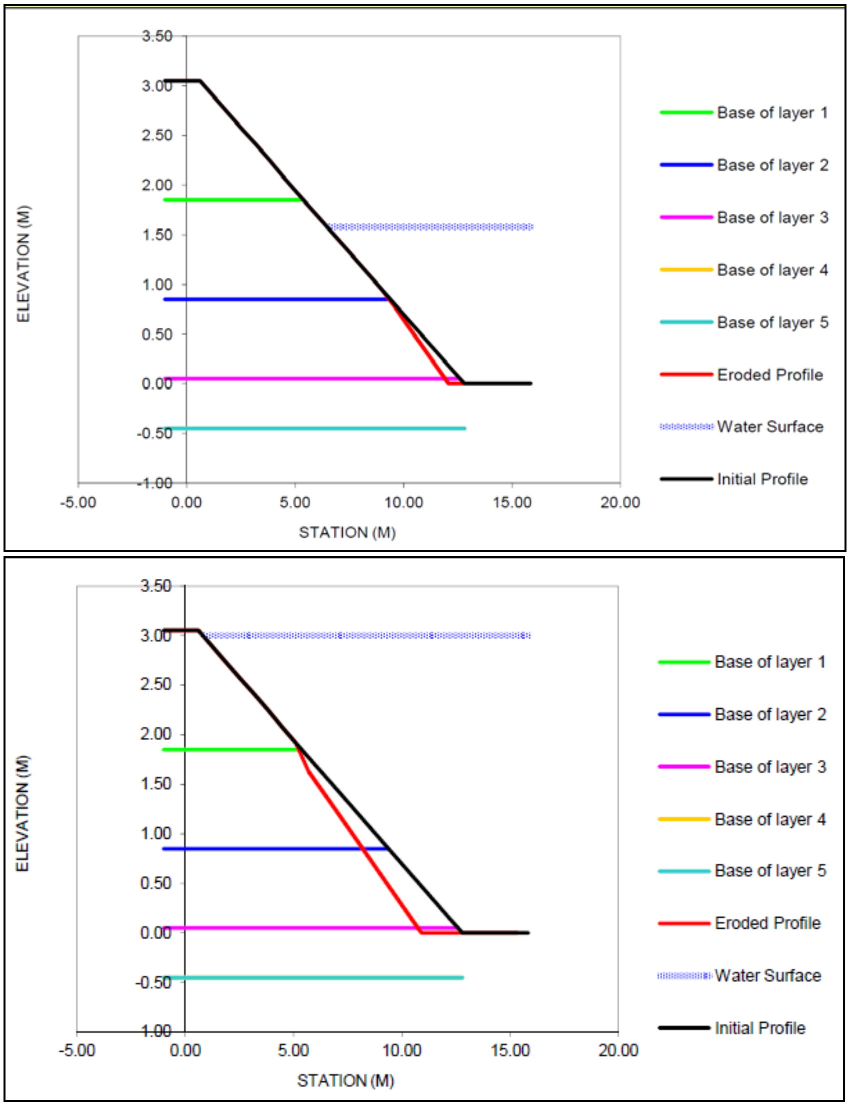


Figure 10. Design D, mean tide (upper) and high tide (lower).

Appendix 5- Access Road Culvert Design

A rainfall runoff hydrologic model was constructed using the WinTR-55 software program. WinTR-55 contains procedures for estimating runoff and peak discharges in small watersheds. WinTR–55 is a single-event rainfall-runoff, small watershed hydrologic model. Two sub-areas were delineated within the small watershed that drains into the south pond, with an estimated combined area of 15.6 acres. For Subarea 1 (13.2 acres), a CN number (65) was selected based on a predominant cover of brush, weed and grass mix in good condition on silty soils. For Subarea 2 (2.4 acres), a weighted CN number (72) was selected based on a combination of a vegetated area and a portion of the gravel runway embankment. Due to flat terrain, the delineated subbasin areas should be considered as approximations.

The National Weather Service provides updated precipitation frequency estimates for Alaska, including the Kwigillingok area; see http://www.nws.noaa.gov/oh/. Storm data used in the analysis are found below:

Table 10. NOAA Atlas 14 point precipitation frequency estimates for Kwigillingok.

Rainfall Return Period (yr)	2	5	10	25	50	100
24-hr Rainfall Amount (in)	1.27	1.62	1.91	2.33	2.68	3.06

Based on the rainfall estimates and watershed characteristics, the model determined the Q50 to be 0.43 cfs, and the Q100 to be 0.65 cfs. Discharge results for the Q50 and Q100 were then used in HY8 to analyze culvert characteristics.

The following culvert for the access road was analyzed: 2.0-foot diameter Corrugated Metal Pipe, 85-ft length, culvert inlet 12.0 ft, culvert outlet 10.5 ft, 1.0 ft of fill over culvert. Results are found in the Hydraulic Summary below:

<table>
<tr><td colspan="3">HYDRAULIC SUMMARY</td></tr>
<tr><td colspan="3">Drainage Area = 15.6 acres</td></tr>
<tr><td>Exceedance Probability</td><td>2% (Q50)</td><td>1% (Q100)</td></tr>
<tr><td>Design Discharge</td><td>0.43 cfs</td><td>0.65 cfs</td></tr>
<tr><td colspan="3">Design High Water Elevation at Q50 = 1.64 ft Below Culvert Inlet Crown</td></tr>
<tr><td colspan="3">Anticipated Additional Backwater at Q100 = 0.0 ft</td></tr>
<tr><td colspan="3">Design Discharge at Hw/D = 1.5 is 17.8 cfs</td></tr>
<tr><td colspan="3">Capacity at Roadway Overtopping = 17.8 cfs</td></tr>
</table>