APPENDIX B

HYDRAULICS AND HYDROLOGY
KENAI BLUFF EROSION
TECHNICAL REPORT
KENAI, ALASKA

CONTENTS

FIGURES

1.0 KENAI RIVER BLUFFS PROJECT

The City of Kenai has proposed construction of bank stabilization project along the north bank of the Kenai River near the mouth. Details of the conceptual design are presented in the report "Kenai Coastal Tail and Erosion Control Project", Peratrovich, Nottingham, and Drage, Inc., February 2002. The project, which in part consists of a toe revetment along the bluff, would stabilize a one-mile reach of riverbank from erosion by water currents, wind, and waves (figure 1). This analysis investigates the bluff erosion mechanisms, the revetment's effect on river currents, and areas needing consideration in the construction of revetment along the bluff. This analysis is not an evaluation of the conceptual design.

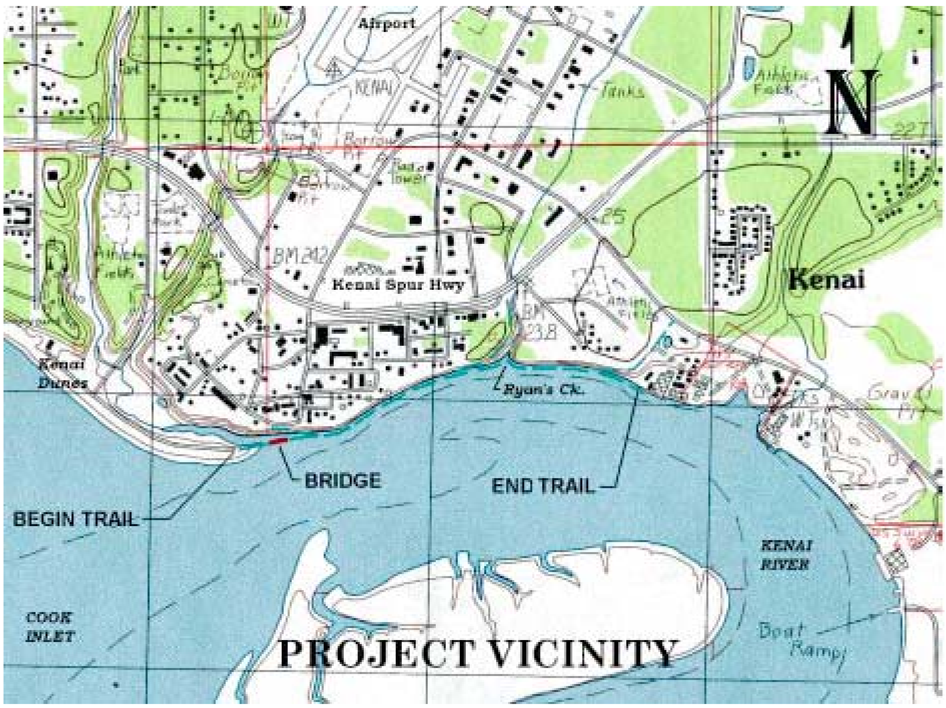

Figure 1. Extent of proposed project

2.0 PHYSICAL ENVIRONMENT

2.1 Topography

The City of Kenai is located on the Nikishka Lowland geomorphological subdivision of the Kenai Lowland. This region is characterized by a modified morainal topography, which is separated by an interlacing pattern of swamps and muskegs developed in abandoned drainage channels and broad depressions. The topography and surficial deposits of the region are primarily the products of repeated Pleistocene glacialtions, which advanced from ice centers in the surrounding mountain ranges. Near the City of Kenai, the Naptowne glacial moraines are fronted by a broad coastal plain consisting of terraced and channeled sand and gravel deposits, which terminate as steep sea bluffs above a series of raised tidal flats. (Tippetts-Abbett-McCarthy-Stratton (TAMS) 1982)

The topography in the area of the Kenai River mouth consists of a bluff approximately 70 feet high opposite a low-lying wetland and tide flat area with a dendridic drainage pattern (figure 2). The topography indicates that the river valley historically has experienced much higher flows (figure 3). Two drainage channels west of the city of Kenai which extend from the south and southwest end of the airport to their confluence behind the dunes at the mouth of the river, could be remnant drainage channels associated with the historical higher flows. These drainage channels appear to play a major part in the local bank stability (figure 4).

The bluff at the mouth of the river is composed of three distinct material types (figure 5). An organic mat top layer that is approximately 2 feet thick, a fine sand layer that is approximately 37 feet thick, and a lower marine deposit layer that can vary from 35 to 45 feet thick. According to Dick Reger, retired geologist with the Alaska Division of Geological and Geophysical Surveys, the lower material is composed of marine deposits that were compacted by a tidewater glacier. Sand and gravel were deposited above the compacted marine deposits through sediment charged efflux jets coming from beneath the approaching glacier. Dropstones from melting icebergs and fine material rained onto the sandy bottom from turbid plumes in front of the glacier (figures 6 and 7).

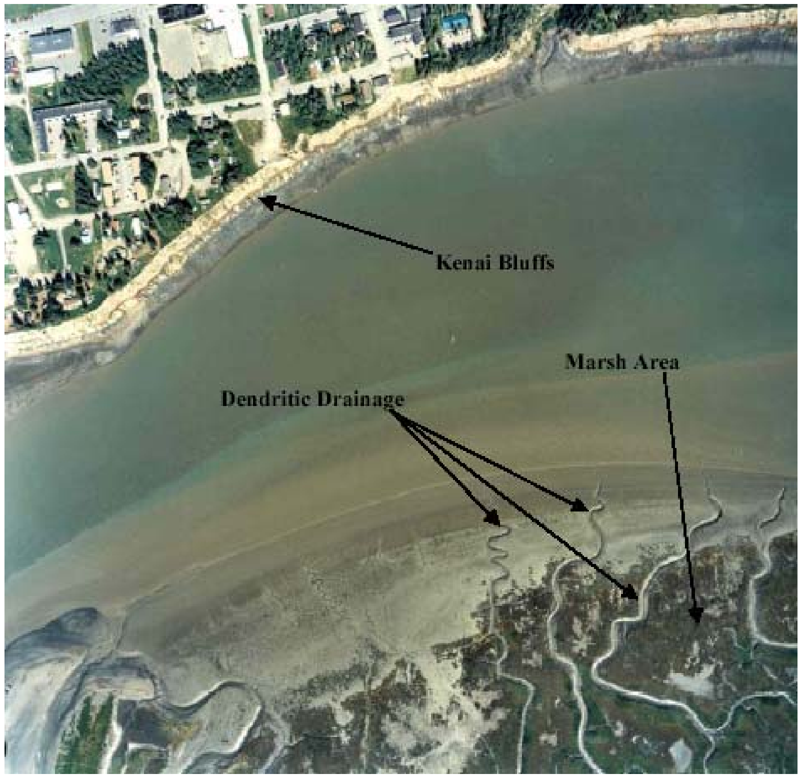

Figure 2. Kenai River Bluffs and wetland opposite the bluffs.

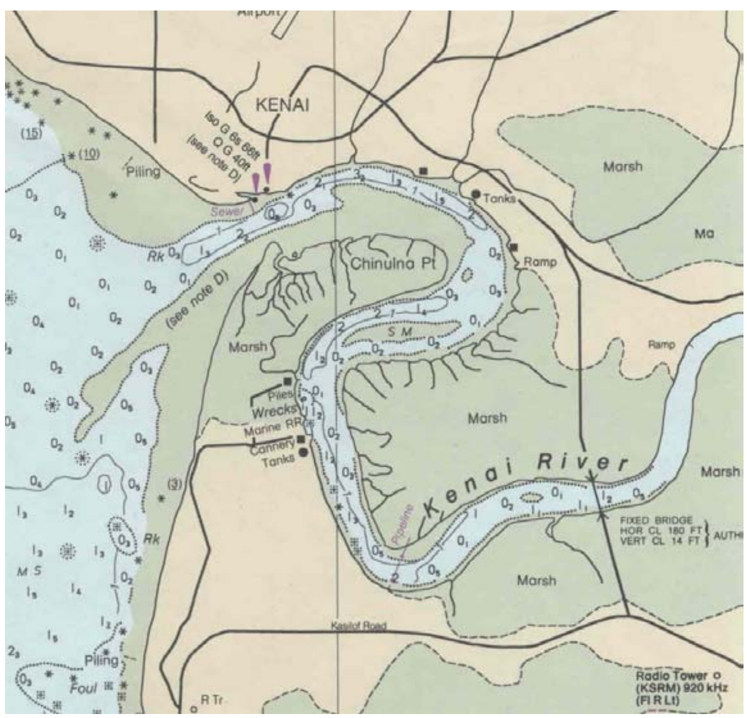

Figure 3. Low-lying areas indicating historic higher flows.

Figure 4. Drainage Channels.

Figure 5. Bluff face with three distinct material types: Marine deposit, sand, and organic mat.

Figure 6. Dropstone embedded in bluff face.

Figure 7. Dropstone.

2.2 Climatology

Temperatures range from an average low of 4°F to an average high of 62°F. Average annual precipitation is 20 inches.

2.3 Tides

The lower portion of the Kenai River is influenced by semi-diurnal tides, with two high waters and two low waters each lunar day. Predicted tidal parameters for the Kenai River entrance are based on the tidal benchmark at Seldovia, Alaska. The extreme tidal parameters are based on a period of record at Kenai from 1985 to present.

Tidal Parameters – Kenai River Entrance

Parameter	Elevation (ft MLLW)
Highest Predicted Tide (10/16/1993)	26.0
Mean Higher High Water (MHHW)	20.7
Mean Tide	11.0
Mean Lower Low Water (MLLW)	0.0
Lowest Predicted Tide (6/14/1995)	-5.4

2.4 River Flow

Kenai River discharge records at Soldotna from 1965 to 2001 show the highest daily mean discharge was 41,400 cubic feet per second (cfs) on September 24, 1995 and the lowest daily mean discharge was 770 cfs on April 1, 1966. Discharge is typically between 1,300 and 15,000 cubic feet per second with average discharges in July, August, and September around 13,000 cfs. Although the flow at the mouth of the Kenai will differ from the flow measured at Soldotna, the flow records at Soldotna are the best historical record available near the mouth of the Kenai and are considered to representative of the freshwater flow that could be experienced at the mouth of the Kenai.

2.5 Waves

Erosion from large wave action is generally limited to the section of the bluff along Mission Avenue due to its open exposure to Cook Inlet. The section of bluff downstream (seaward) of Mission Avenue is protected by coastal dunes and is generally not subjected to wave action. Wave action at the bluff section upstream of Mission Avenue is an infrequent occurrence due to the partial protection provided by the wetlands to the south and the shoal at the river mouth. Wave action at this section requires a combination of storm surge and high tide to overtop these natural barriers. Figure 8 delineates these bluff sections.

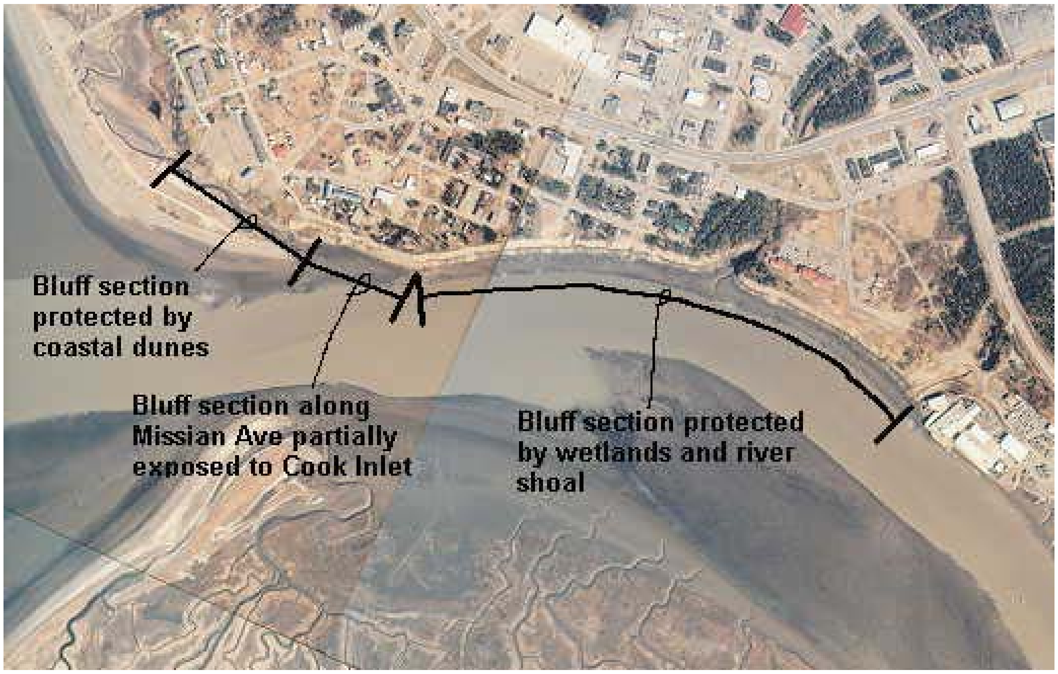

Figure 8. Wave Climate Delineation Along Bluff

2.6 Erosion History

Analysis of aerial photos from 1976 to 1999 performed by the University of Alaska Anchorage, indicate that the average volume of material lost from the bluff is approximately 32,150 cubic yards per year. A rough evaluation of the bluff lines drawn in the report indicates that on average the bluff is eroding at a rate of 2.5 feet per year.

3.0 EROSION MECHANISMS

Bluff erosion along the Kenai River mouth is a product of several conditions, with varying degrees of contribution. The mechanisms acting on the bluff include:

- Wind scour on the face of the bluff
- Wave action at the toe of the bluff
- Groundwater seepage and piping from the upper sand layer
- Groundwater seepage, piping, and hydrostatic pressure build up in the lower clay layer
- Runoff draining over the bluff
- Freeze thaw effects weakening the bluff

3.1 Wind

Isolated occurrences of wind erosion on the upper sandy portion of the bluff were noted during an April 2003 site visit. Wind erosion is minor relative to the total erosion along the bluff face.

Wind does play a role in the transport of the fine sediment. This is evidenced by the build up of sand on the face of the dunes and along fence lines after a dry windy period. While the volume of sediment transported appears minor when viewed as a yearly volume, over time this could account for a large quantity of material transported.

3.2 Waves

The bluff section along Mission Avenue is most susceptible to large wave action due to its open exposure to Cook Inlet. Wind data necessary for a wave analysis at this section of bluff is not available. Therefore, this analysis was beyond the scope of this study.

The section of bluff downstream (seaward) of Mission Avenue is protected by coastal dunes. Erosion of the dunes would be required for this section to be exposed to waves.

Wave action at the bluff section upstream of Mission Avenue is an infrequent occurrence. There must a combination of storm surge and high tide for the bluff to be impacted by wave action since the wetland and shoal at the mouth of the river create a natural barrier to waves by limiting the wave heights.

A brief analysis was performed to determine the wave height that could impact the bluff. This analysis looked at only one wave direction and did not consider the effects of refraction. A more detailed analysis would require the use of pressure charts to accurately define the wind fields that could generate waves in Cook Inlet. The wave that could impact the bluff was evaluated by examining two wave conditions: the unbroken wave height that could travel past the shoal and the wetland, and the wave potential to reform after being broken.

A wave traveling towards the bluff would travel over the shoal and wetland. Survey of the wetland area across the river indicates that it has an elevation of up to 23 feet MLLW so the water level needs to be greater than 23 feet to even begin to support a wave climate. Mean higher high water is 20.7 feet, which would not overtop the wetland, so a surge event coupled with a high tide would be needed for the wetland to be capable of supporting a wave. While tide data at Kenai is available, storm surge data is not available. To approximate a storm surge at

Kenai a list of storm surge observations compiled by the National Ocean Service and listed in table 3-6 of the Shore Protection Manual was consulted. Kodiak is the closest suitable station to Kenai. The extreme high water level recorded for Kodiak is 3.7 feet above mean high water. When combined with the mean higher high water elevation for Kenai, a highest water level of 24.4 feet MLLW (20.7 feet + 3.7 feet) is obtained. This will be the water level over the wetland while the tide is at the mean higher high water elevation, once the tide turns this water level will begin to fall. A 24.4-foot water level leaves 1.4 feet of water over the wetland. Typically waves break in a water depth that is 0.78 times the wave height, applying this to the wetland results an unbroken wave of one foot that could be supported in 1.4 feet.

Waves broken on the wetland would have the potential to reform after they are broken if there is sufficient wind and fetch to support wave growth. Wind records from the Kenai Municipal Airport have recorded sustained 30-knot winds. While this is an infrequent occurrence, this wind was used to determine the potential for wave reformation after correction for measurement height and land effect. The fetch used for wave growth after being broken was one-half mile. With these conditions a conservative estimate of the potential wave generation is a one-foot or smaller wave being able to reform and impact the bluff.

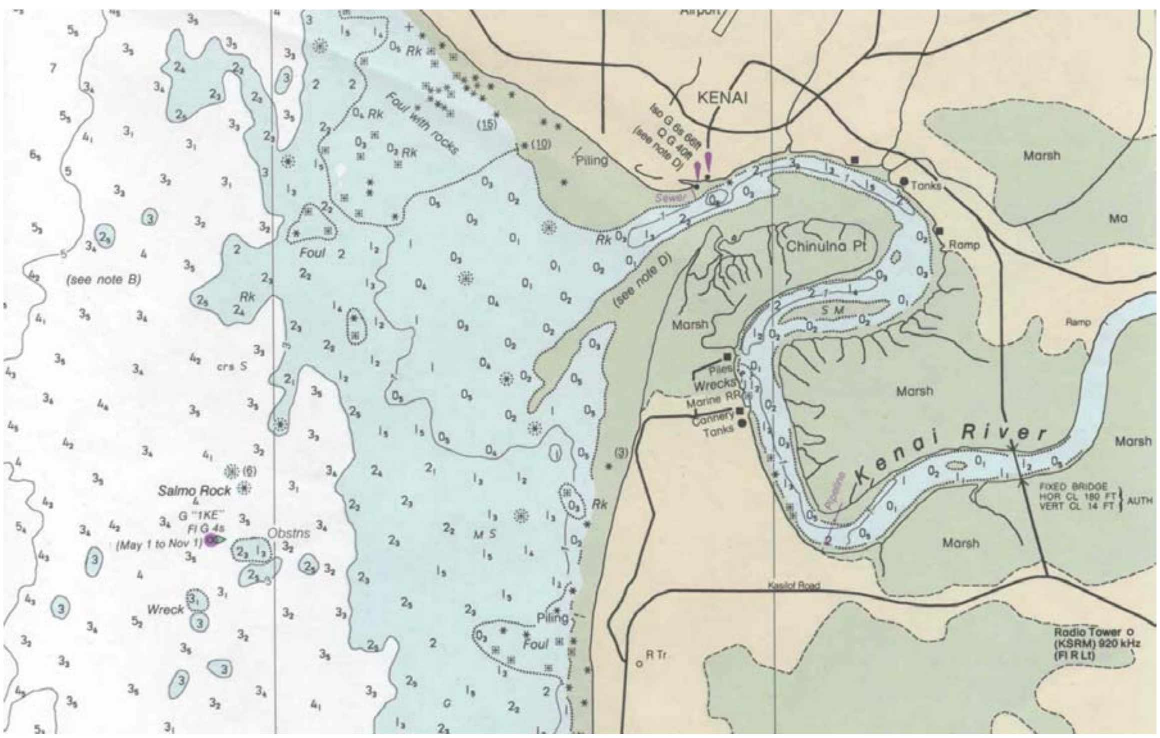

Figure 9. Bathymetry at the Kenai River mouth.

Figure 10. Topography affecting wave behavior at the mouth and along the bluffs.

Figure 11. Breaking wave in front of the bluffs.

3.3 Groundwater Seepage

At the interface of the sandy material and the marine deposit, groundwater seeps out of the bluff face. The water source for the seepage appears to be the wetland area behind the airport. This area contains a large volume of water that can supply a constant source of recharge for the unconfined aquifer that is exposed at the bluff face. The upper sand layer is very porous and readily transmits surface water to the impermeable marine deposit layer. Once the impermeable layer is contacted, the groundwater flows along this surface and out of the bluff face.

Two areas east of the proposed project site appear to interrupt the groundwater flow with a natural channel (figure 12). These areas appear to provide a gradient away from the bluff face and towards two channels that meet behind the dunes at the mouth of the river. In this local drainage the bluffs are more stable and seepage out the bluff face is not observed (figure 13). Figure 14 shows a transition area between the bluff where the groundwater is channelized away from the bluff face and where seepage is observed out of the bluff face.

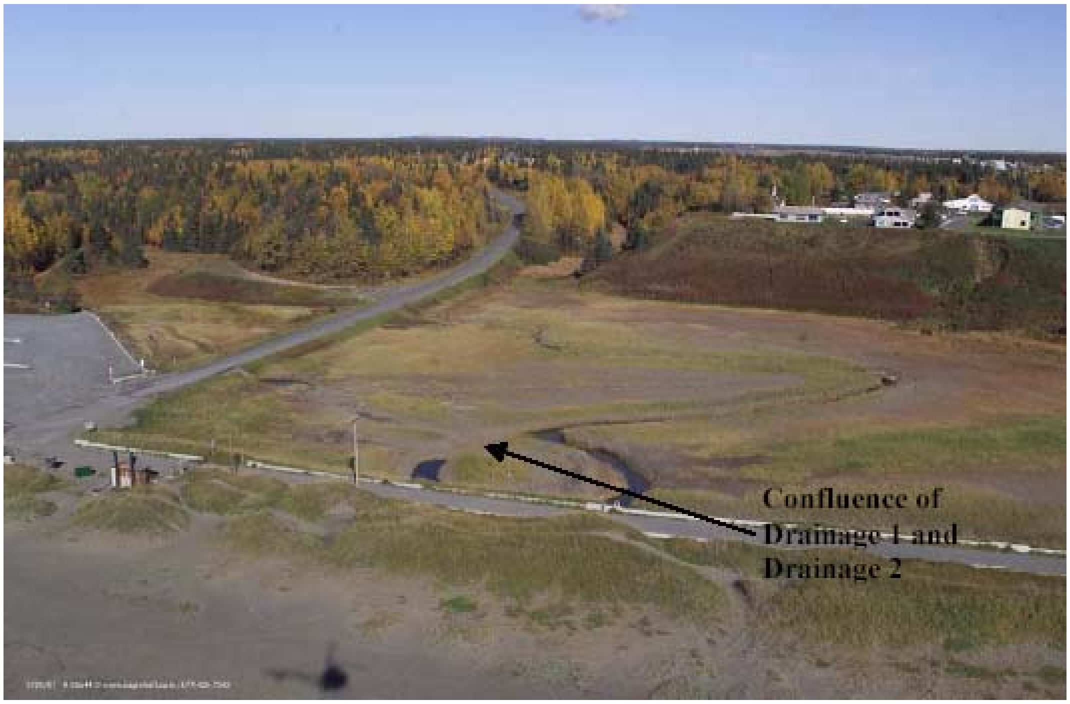

Figure 12. View of the two drainages behind the dunes

Figure 13. Stable bluff in the area behind the drainage.

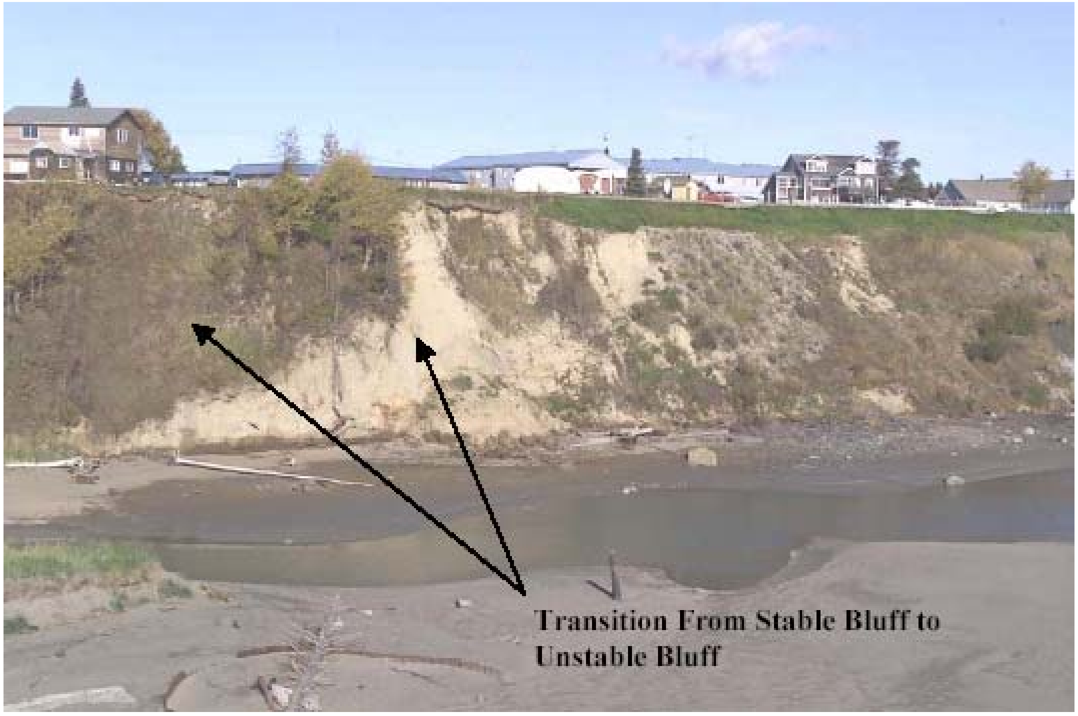

Figure 14. Transition area to seepage out of the bluff face

Bluff damage due to seepage is evident by the piping holes along the face of the bluff. At the interface between the sand layer and the marine deposit, piping holes are visible along the entire length of the proposed project area (figures 15 and 16). This indicates that the water is seeping along preferential flow paths and removing material from the sand layer. As the material is removed, the support for the material above weakens until it collapses and localized failure is experienced on the upper bluff (figure 17). The failed material covers the lower bluff and then is washed away by high tides, storms, and rain. The cycle of piping out the lower material and subsequent failure then begins again.

The bluff damage from the groundwater seepage is not limited to impacts seen on the upper bluff sand layer. The water seeping out of the bluff face travels over and sometimes through fractures in the marine deposit. The water traveling over the marine deposit eventually erodes channels into the deposits. In areas where the water travels through the marine deposit, the water pressure appears to build up and pipes out of the bluff base (figure 18). This was observed along the bluff after the heavy rainfall events in the fall of 2002 where blocks of marine deposit material had piped out and left large voids in the bluff base. The failed material was gone the following year having been washed away or scoured away by ice over the winter.

In other areas the water seeps to the base of the bluff resulting in areas of high pore pressure making a traverse of the bluff base very soupy. The seepage pathway through the marine deposit is also susceptible to deterioration from freeze thaw effects. The water freezing in the fractures in the bluff will expand to increase the fracture and weaken and or fail the bluff material. High tides, waves, or high flows at the base of the bluff remove the failed material that accumulates and the erosion process starts over. The groundwater seepage from the bluff face is a major contributor of erosion of the bluff face.

Figure 15. Piping hole in the bluff face.

Figure 16. Series of piping holes in bluff face.

Figure 17. Eroded upper bank material at the base of the bluff and channels cut in more impermeable marine deposit.

Figure 18. Piping at the bluff base.

3.4 Overland Flow

Uncontrolled flow over the top of the bluff would erode the sand in the upper bluff. It appears that in some areas along the bluff an effort has been made to control overland flow. A drainage swale has been put in place along Mission Avenue (figure 19). This has helped to control the overland flow in this area, but the bluff is still affected by the action of the other erosional forces previously discussed.

Figure 19. Drainage swale for overland flow.

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4.0 GEOTECHNICAL EVALUATION

Soil borings were advanced at four locations (figure 20) in support of a slope stability and groundwater flow analysis. Soil samples collected during the drilling effort indicated that the sand layer was approximately 37 feet and was underlain by a clay layer that was 36 to 45 feet thick. Piezometers were installed in each of the borings to allow groundwater elevation measurements to be taken periodically. Groundwater measurements taken in October 2003 and April 2004 did not indicate much variation in the water level elevation.

Figure 20. Location of soil borings

Water level elevation for October 2003 and April 2004

	TB1 Water Elevation [ft]	TB2 Water Elevation [ft]	TB3 Water Elevation [ft]	TB4 Water Elevation [ft]
October 2003	62.74	60.57	58.23	61.24
April 2004	62.40	60.26	Plugged	60.95

Gradient lines constructed from the water elevation measurements confirmed the water flow is towards the bluff. The flow gradient in October and April was 0.01 foot per foot.

A pump test was performed on TB2 in October 2003. Using the Hvorslev Method, the hydraulic conductivity of the aquifer was determined to be 3.08×10^{-3} feet per second. This is in the range of the hydraulic conductivities for well-sorted sands, glacial outwash cited in C.W. Fetter's *Applied Hydrogeology*. The pump used for the test had difficulty achieving a large draw down, and the recovery was very rapid, so the test results are approximate only.

The calculated hydraulic conductivity was used in the Dupuit equation to determine the aquifer's unit flow of 2.5×10^{-3} ft^3 per second for the October and April measurements. Assuming the upper soil layer is uniform with no impermeable lenses channeling flow from the face of the bluff, and a proposed protection length of 4,500 feet, then the amount of water moving through or behind the bluff proposed for protection is 972,000 ft^3 per day or 7.3 million gallons per day.

As a check on the flow, the volume of water available for transport from the wetland (figure 21) was estimated. The area of the wetland was estimated to be 11 square miles. All precipitation that occurs was assumed to be made available to the aquifer and not stored since the wetland is already is a saturated condition. The reported average annual precipitation is 20 inches, which results in 511 million cubic feet of water over the wetland. This precipitation was then divided evenly over a year and resulted in 10.5 million gallons a day moving through the aquifer. This is just the flow potential, the effects of local drainage were not been considered and the validity of distributing the flow evenly throughout the year was not tested; however, it provides a check on the calculated flow through and behind the bluff, which appears to be reasonable.

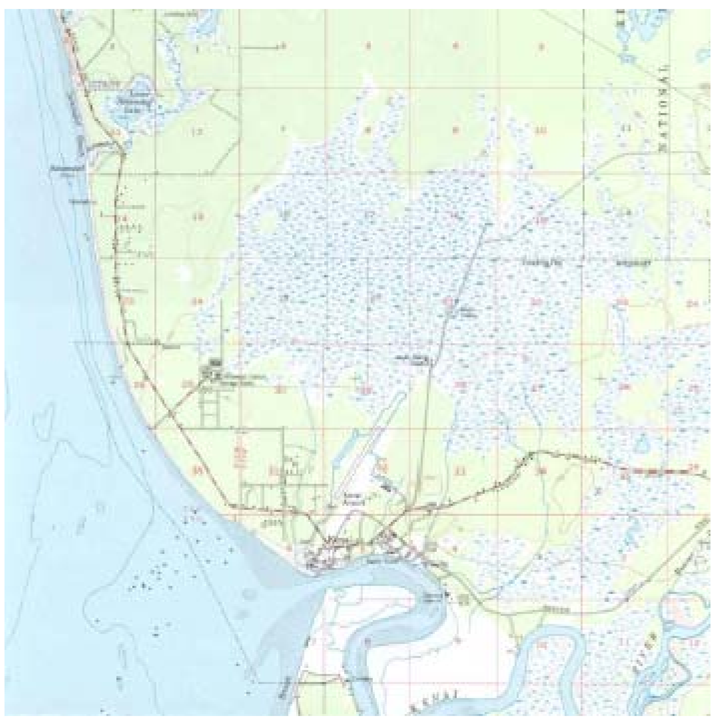

Figure 21. Wetland supplying aquifer

5.0 POTENTIAL EFFECTS

5.1 Potential Effects

Several concerns of potential adverse effects from bluff stabilization were addressed in the study. These concerns include:

- Effect of revetment placement on river flow
- Effect of the bluff stabilization on the dune stability

5.2 Effect of Revetment Placement on River Flow

The general effect of the proposed project on the flow of the river was evaluated using the Hydrologic Engineering Center River Analysis System (HEC-RAS). The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head).

To perform the HEC-RAS analysis, five depth profile cross sections were input into the HEC-RAS program to describe the river reach. One cross section was located upstream of the project and one cross section was located downstream of the project. Three cross sections were located within the river reach along the bluff. Flow velocities were evaluated for different discharge and tide level conditions in the with- and without-project condition. The river discharge was based on the Soldotna data for maximum flow, minimum flow and average summer flow. The tide conditions were based on the high and low tide conditions for the days that experienced the maximum and minimum flow. The highest and lowest predicted tide for the years 1985 to 2003 was used for evaluation of the average summer flow condition. The tide elevation was the starting water surface elevation. No flow measurements were made as part of this study. The following scenarios were analyzed:

RUN #	RIVER FLOW [cfs]	TIDE [ft]	PROJECT CONDITION
1	770	0.5	Without project
2	770	18.5	Without project
3	770	0.5	With project
4	770	18.5	With project
5	13,000	-5.4	Without project
6	13,000	26.0	Without project
7	13,000	-5.4	With project
8	13,000	26.0	With project
9	41,400	-0.3	Without project
10	41,400	23.2	Without project
11	41,400	-0.3	With project
12	41,400	23.2	With project

Model results indicate very low velocities during high tide conditions with an average velocity in the main channel less than 1.5 feet per second (fps). Channel velocities during low tide conditions ranged from slack water to 6.6 fps.

Model results indicate that the project would have minimal effect on the average river velocity. At low tide, the project would not change the average river velocity. Under high tide conditions the project would have minimal effect on the average velocity. The maximum increase by the project is 0.1 fps; an increase from 0.9 foot per second to 1.0 foot per second. Results of the HEC-RAS analysis follow:

Run #	River Flow [cfs]	Tide [ft]	Project Condition	Average Velocity In Project Area [ft/s]		
				Overbank by Wetland	Main Channel	Overbank by Bluff
1	770	0.5	Without project	0.01	0.16	No water
2	770	18.5	Without project	0.01	0.03	0.02
3	770	0.5	With project	0.01	0.16	No water
4	770	18.5	With project	0.01	0.03	0.02
5	13,000	-5.4	Without project	No water	4.67	No water
6	13,000	26.0	Without project	0.16	0.41	0.25
7	13,000	-5.4	With project	No water	4.67	No water
8	13,000	26.0	With project	0.16	0.41	0.25
9	41,400	-0.3	Without project	0.85	6.60	No water
10	41,400	23.2	Without project	0.55	1.48	0.82
11	41,400	-0.3	With project	0.85	6.60	No water
12	41,400	23.2	With project	0.55	1.49	0.88

It is estimated from the current project drawings, that the lowest elevation of the project will be at 10 feet MLLW. This was arrived at by overlaying the proposed project drawings on the survey data (figure 21). Using 10 feet MLLW as the lowest project point, an evaluation was performed to determine the percent time that the project will be in the river. An analysis of the predicted tides for 2003 indicates that a portion of the project will be in the river approximately 55% of the time. This assumes that the water surface elevation is governed by the tides. The average flows along the revetment, when wetted, will be relatively slow. Observation of the material indicates that it is very stiff and would likely be unaffected by minor, temporary turbulence from slow moving water. During the time that the structure is not wetted, the toe will be available for periodic inspection for scour.

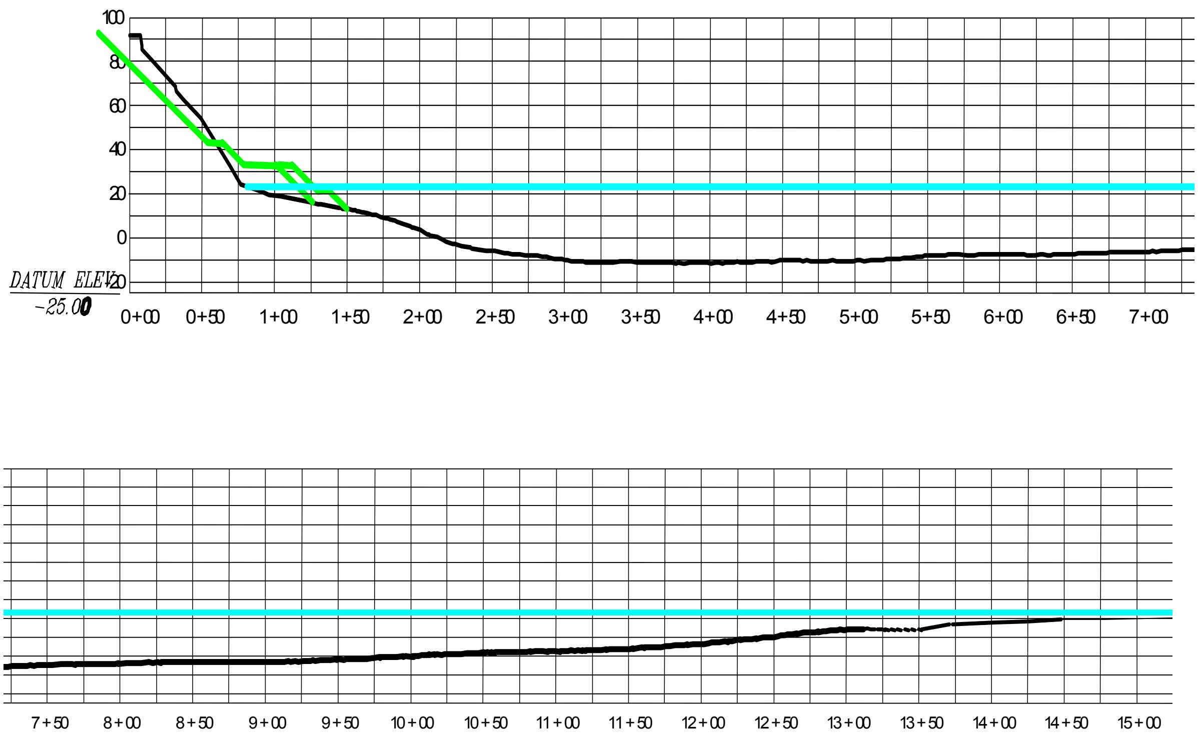

Figure 21. Bluff project overlain existing bluff. Water level is mean flow with extreme high tide

5.3 Effect of the Bluff Stabilization on the Dune Stability

The dunes at the mouth of the Kenai River are an ephemeral feature that owes its existence to wind blown sand, beach grass, and a favorable littoral transport system. The orientation of the dune and the channel behind the dune could be an indication of a west to east littoral transport or it could also be an indication of the best hydraulic gradient to the Kenai River.

The dune appears to be nourished by wind blown sand, which is trapped in the beach grass, allowing the dune to grow until a storm event. The wind blown sand is sacrificed during storm events and the grass remains to capture more sand and renourish the dunes. The key to dune preservation is the preservation of the dune grasses. Figures 22 and 23 show the dune before and after a storm event.

While the mechanism for dune renourishment appear to be wind blow sand transport, the source of sand is uncertain since the coastal bluffs and upper layer of bluff along at the mouth both contain sand similar to the dune material. This results in the follow potential sources:

- Sand that is transported along the coastal beach from northwest to southeast at the river mouth

- Sand moved from the river system, deposited on the shoal at the mouth, and moved back to shore during storm events
- Sand moved from the base of the bluffs along the shore and deposited on the beach at the mouth by tidal currents

Figure 22. Dune during heavy use before storm event

Figure 23. Scarped face of dune after a storm

5.3.1 Coastal Sand

The shoreline exposure and orientation to Cook Inlet indicate a dominant northwest to southeast littoral transport system for the north river bank. Evidence for this transport is found northwest of the mouth of the river in a headland area with numerous boulders lying on the tide flat. The headland and boulder field act like a large groin. The oblique aerial photograph at the Kenai Airport offers an excellent view of the groin effect of the headland (figure 24 and 25). The headland is indented in its lea and fine sand is evident at this site where the "groin" starves the beach of coarser sediment and allows fine material to deposit in the eddy formed behind the groin. Further to the southeast in the direction of the dune the coarser material rejoins the beach and is carried along the base of the bluff. The material is continuous from where it rejoins the shoreline to the dune zone.

The sand sediment along this stretch of beach appears to be a thin mantel of eroded bluff material with a maximum layer thickness of a few feet underlain by marine deposit material similar to the lower half of the river bank bluff. Numerous pockets along this beach show areas of high pore pressure characteristic of the saturated marine deposit material under the surface sediment. It is suspected that most of the surface sediment is lost to offshore of the tide flat during major storms and only minor amounts are transported southeast for windblown transport to the dune face.

Figure 24. View of the Kenai River and indicator of sediment transport direction

Figure 25. Ground view of the headland acting as a groin

5.3.2 River Bluff Sand

An estimate of the sediment contribution from the bluff proposed for revetment is 5 to 7 percent of the sediment load in the river according to Sediment Impact Assessment conducted by the Engineering Research and Development Center (see Appendix C). Two alternatives exist to make the bluff sand available for dune nourishment. First, bluff sediment must be transport down river and drop out at the shoal at the river mouth. Sediment at the shoal would be available for wave transport to the beach in front of the dune, then blown by wind to the dune face. Survey data is not available to determine if the shoal is growing and it is unclear if the shoal is composed of sand similar to the bluff sand. This scenario requires a storm to transport the eroded bluff material to the beach. The storm that have that potential are infrequent and are of short duration.

A second alternative is transport of the eroded bluff sand along the river's edge by tidal currents during high tide events. The sediment would not move a long distance but t would move close tot the mouth of the river and would be available for wind blow transport to the dune face during low tide events.

To be made available for dune nourishment, bluff sediment would first be transported down the river to the mouth, where, if it were to stay in the system, it would drop out at the shoal at the mouth. Survey data is not available to determine if the shoal is growing and it is unclear if the shoal is composed of sand similar to the bluff sand. The material that would accumulate on the shoal would then be available for wave transport to the beach in front of the dune, and then windblown transport to the dune face. This scenario requires a storm to transport the eroded bluff material to the beach. The storms that have that potential are infrequent and are of short duration.

5.3.3 Most Likely Sediment Source.

While the source of the dune nourishment source is likely a combination of all of the sources discussed, geographical features provide the most compelling evidence that the major source of dune nourishment is the material from the north around the headland and not river buff material.

5.4 Further Studies and Recommendations For Design Work.

If the City pursues the revetment project it is strongly recommended that the issue of the groundwater seeping out the bluff face be addressed as the first order of work. Any solution that does not address the groundwater will not solve a major source of erosion. Options that may be considered to address the groundwater include: a cutoff wall and pump system to intercept the groundwater, draw down wells to reduce the water table along the bluff face, a horizontal drain system into the face of the bluff to collect and divert the water, a free draining retaining system to hold back the bank material and still allow free drainage of water from the face of the bluff, or creation of drainage channels to alter the groundwater gradient. Additional information can be found at the web site: http://www.montauklighthouse.com/erosion.htm to see how another community with similar soil and groundwater issues, and a more severe wave climate approached their erosion control.