ERDC TR-08-5

Engineer Research and Development Center

US Army Corps of Engineers®

Boat-Wave-Induced Bank Erosion on the Kenai River, Alaska

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March 2008

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Final report

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Abstract: The Kenaitze Indian Tribe requested that the U.S. Army Engineer Research and Development Center (ERDC) determine the relative contribution of boat-wake-induced bank erosion to total bank erosion along the Kenai River. The approach used in this study consisted of a delineation of boat wave characteristics along the study reach and a geomorphic and bank stability assessment. This analysis showed that, at specific times of the year and at specific locations, boat wave energy may be a dominant factor. However, on an average annual basis, boat wave energy is secondary to river currents in terms of total bankline recession. Reduction of boat wave energy should focus on areas having large boat passage frequency, such as the drift area at river miles 10–12 and areas where bank erosion is most problematic.

Techniques to reduce boat waves from a *single* boat include the use of flat-bottomed boats, use of 50-hp motors to increase boat speed, keeping boats away from shorelines, and reducing boat weight.

Decreased boat weight and keeping boats away from shorelines are two options that can result in benefits even when significant traffic is present. This study found that boat wakes are one of several factors contributing to bank recession. However, quantification of the relative magnitude of boat wakes to other factors such as river currents could not be determined. The results indicate that boat wakes may be a dominant factor during certain high boat usage times, discharges, and locations along the study reach. Although wake-induced erosion may be a secondary factor in bankline recession, it may be ecologically significant because of its persistence, distribution, and timing. However, bank recession associated with large flood events will likely overshadow the contribution from boat waves.

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Preface

The work reported herein was conducted for the Kenaitze Indian Tribe of Kenai, AK, by the U.S. Army Engineer Research and Development Center (ERDC) during 2005-2007. The field work was performed during July 2005 by personnel of ERDC, personnel of the Tribe, and numerous personnel of state and local agencies in the State of Alaska. ERDC participants in the field studies were Dr. Stephen T. Maynord, Dr. David S. Biedenharn, Fred Pinkard, Thad C. Pratt, and Terry N. Waller, Coastal and Hydraulics Laboratory (CHL); Dr. Craig J. Fischenich, Environmental Laboratory (EL); Dr. Jon E. Zufelt, Cold Regions Research and Engineering Laboratory (CRREL); and Tim E. Nisley, Information Technology Laboratory (ITL). The report was prepared by Drs. Maynord, Biedenharn, Fischenich, and Zufelt.

The authors wish to acknowledge support for this study provided by the following participants:

- Brenda Trefon of the Kenaitze Indian Tribe was the project sponsor and project participant.
- The U.S. Environmental Protection Agency (EPA) provided project funding to the Kenaitze Tribe.
- Lance Trasky was the Project Coordinator.
- Robert Ruffner of the Kenai Watershed Forum provided administrative support, staff assistance, equipment, boats, and reviewed study plans and draft reports.
- Chris Degernes and Jack Sinclair of the Alaska Department of Natural Resources (ADNR), Division of Parks and Outdoor Recreation (DPOR) provided research site boats, equipment, staff, and other help.
- Sue Fisler of the ADNR/DPOR provided permits and help in locating landowners.
- Bill Berkhahn and Jacques Cousteau of the ADNR/DPOR provided advice and transportation.
- Lee McKinley, ADNR/Office of Habitat Management and Permitting, provided boats and operators.
- Mary King of the Alaska Department of Fish and Game (ADF&G) helped in locating study sites, reviewed study plans and draft reports, and provided expert advice and assistance.

- Dean Hughes, ADF&G, helped in locating research sites.
- Carol Barnhill and Francis Innoue of the ADF&G provided maps.
- Kenai River Center Staff provided permits and the Kenai Peninsula Borough staff provided maps and permits.
- The Cities of Soldotna and Kenai provided use of city-owned land.
- Landowner Jeff King provided use of the River Mile 14 study site.
- Landowner Allovise Gehrke provided use of the River Mile 17 study site and electrical power for the instrumentation.
- Peter Thompson of River and Sea Marine provided a boat.
- Volunteers at the boat data gathering sites were as follows: Dennis Gease, Brenda Trefon, Laura Kroto of the Kenaitze Indian Tribe, Wayne Dobson, Jennifer McCard, Dale Dingman, Maxine Dingman, Stephanie Sims, Shannon Donnley, Adam McCullough, George Purdy, Sherry McKenzie, Krista Holman, Jan Yeager, Sheldon Ruffner, Estelle Sandvik, Josselyn Burke, Park Ranger Jacqui Erion, K. Heather Sinclair, Jamie Sinclair, Sara Youngren, Inger Coursen, Dave Soldavini, Jack Reed, Terri Carter, Gail Moore, and Sally Griffin.
- Report reviewers included: U.S. Fish and Wildlife Service – Bill Rice (engineer/hydrologist); U.S. Geological Survey – Janet Curran (hydrologist); Natural Resources Conservation Service – Aimee Rohner (design engineer); EPA - Lorraine Edmond (geologist); University of Alaska at Anchorage – Orson Smith (PhD engineer); Alaska Department of Environmental Conservation – Jim Rypkema (water quality), Kent Patrick-Riley (water quality), Tim Stevens (water quality), Mel Langdon (engineer), Kenwyn George (engineer); ADNR - Lee McKinley (biologist), Jackie Timothy (biologist), Mark Inghram (hydrologist); ADF&G - Bob Clark (biologist), Mary King (biologist), and Lance Trasky.

The study was conducted under the direction of Thomas W. Richardson, Director, CHL; Dr. William D. Martin, Deputy Director, CHL; Dr. Rose Kress, Chief of the Navigation Division, CHL; and Dennis Webb, Chief of the Navigation Branch, CHL. Dr. Beth Fleming was Director, EL; Dr. Robert E. Davis was Director, CRREL; and Dr. Deborah Dent was Acting Director, ITL.

COL Richard B. Jenkins was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

Executive Summary

Bank erosion was observed throughout the Kenai River study reach, from river mile 10 (downstream end) to river mile 21 (upstream end). Bank erosion and deposition are normal and expected fluvial processes that occur in the Kenai River even without human intervention. The observed long-term bank recession rates are generally less than 1 to 2 feet per year, with locally higher rates associated with flood events or large hillslope failures. This study found that boat wakes are one of several factors responsible for bank erosion along the Kenai River within the study area. In addition to previous studies showing the importance of waves to shoreline recession, boat wakes were observed to move bank material in the field study. However, the additive contribution of boat wakes relative to these other factors is difficult to quantify.

Boat passage frequency along the 11-mile study reach varies significantly, with the largest numbers of boats in the downstream end of the reach. Based on counts in July 2005, wave-making boats pass the downstream study sites at a frequency of up to seven times greater than for the upstream sites. As a result, boat wave energy from waves greater than 0.25 ft at the shoreline in the major drift area near RM (river miles from mouth) 10-12 is about ten times greater than the boat wave energy at the shoreline above RM 17. An attempt was made to correlate boat wave energy with bank recession rates. No relationship was found, however.

The contribution of boat wakes relative to other factors such as river currents varies throughout the year. If the peak boating period of July occurs during lower than normal flows (less than about 15,000 cubic feet per second (cfs)), boat wave energy is largely expended on the cobble bank present along much of the river. If the peak boating period of July occurs during higher than normal flows (greater than about 15,000 cfs), boat wave energy attacks the banks above the cobble and boat-wave-induced erosion may be the dominant process. However, bankline recession during these periods may be relatively low based on our analysis. At even higher river flows such as major flood events, the boat wakes appear to become a secondary factor.

The largest shoreline boat waves that occur about 1 percent of the time are capable of moving material exceeding the D_{50} (diameter, average size) of the cobble banks along the river but not the D_{84} (large sizes) that are often used to characterize the stability of cobble banks that have formed by an armoring process. The more frequent "significant" wave height equal to the average of the highest one-third of all waves is capable of moving the D_{50} of the cobble banks at only the highest traffic areas in the downstream 2-mile reach.

The relative contribution of boat wakes and river currents was also evaluated by comparing energy at the bankline from boat waves and energy at the bankline from streamflow. From RM 21 to about 17, computed energy at the bankline from boat waves alone is less than or equal to 5 percent of computed energy at the bankline from streamflow based on the typical 12-hr monitoring period during the 2005 field study. From RM 17 to 12, computed energy at the bankline from boat waves alone is greater than 5 percent and less than or equal to 20 percent of computed energy at the bankline from streamflow based on the typical 12-hr monitoring period during the 2005 field study. From RM 12 to 10, computed energy at the bankline from boat waves alone is greater than 20 percent of computed energy at the bankline from streamflow based on the typical 12-hr monitoring period during the 2005 field study. At the highest sites in the downstream 2-mile reach, computed energy at the bankline from boat waves alone is up to 59 percent of computed energy at the bankline from streamflow based on the typical 12-hr monitoring period during the 2005 field study. These levels of boat wave energy show the relative importance of boat wave energy to streamflow energy, but the combination of streamflow and boat wave energy is not a simple additive relationship.

When comparing streamflow and boat wave energy magnitude for relevant discharges during the entire year, the shoreline boat wave energy is about 16 percent of the shoreline streamflow energy for the highest boat wave energy sites in the downstream 2-mile reach. The percentage for the entire year becomes less during high flow years such as 1995 and significantly less at upstream sites having less boat traffic. This analysis shows that, at specific times of the year and at specific locations, boat wave energy may be a dominant factor, but on an average annual basis it is secondary to river currents in terms of total bank line recession.

During the 1995–1998 period, localized bank recession rates of up to 8 ft/yr were observed. These larger bank recession rates likely reflect the period of record flood that occurred in September 1995 and suggest that major flood events may be the dominant factor with respect to significant bank recession. Although our studies demonstrated that the magnitude of erosion associated with boat wakes is much smaller over the long run than flood-induced erosion, the environmental impacts associated with wake-induced erosion may be significant. Large-scale erosion caused by hydraulic forces during floods serves as an important ecological disturbance that creates new habitats. The recruitment of large woody debris and new spawning gravels on the lower Kenai River, as well as establishment of substrates for vegetation colonization and succession, may depend upon these events. The persistent nature of the wake erosion during the peak boating season, on the other hand, may prevent the colonization of some plant species and may induce elevated turbidity levels in the zone near the bank. The wake energies are not sufficient to entrain woody debris, so some of the benefits of erosion are not realized from this mechanism of bank loss. The spatial distribution of erosion associated with the boat wakes also differs from flood-related erosion, and bank regions that are largely unaffected by floods (e.g., areas on the inside of bends) may be subject to erosion from boat wakes.

Banks along the study reach were classified with respect to susceptibility to erosion from boat wakes and high river flows. The classification scheme was based on long-term erosion rates from Fischenich (2004), field observations in 2005, and basic principles of river mechanics. A primary consideration in the classification scheme was that the presence of vegetation along the bank appears to significantly reduce erosion associated with boat wakes and high flows. The common trait in bank types 2, 6, and 7 is the presence of woody or herbaceous vegetation along the bank. Bank types 3, 4, and 5 lack this vegetative protection; therefore, they appear more susceptible to erosion. It should be noted that, when large flood events occur, all banks may be subject to significant erosion. In areas where bank vegetation has been removed and bank erosion is occurring, an effective management option might be the implementation of bio-engineering measures that have proven successful within the study area. These measures would both restore the disturbed habitat and protect the banks from further erosion. Another possible management option would be to modify boat operation. A discharge threshold concept is presented as a possible method to reduce the boat wake impacts. It should be noted that, even if all boat traffic was eliminated from the river, erosion due to other factors would continue, although at a slower rate in some locations.

The analysis presented herein is based on present levels of boat wave energy. Any future increases in boat wave energy may significantly alter bank erosion levels because existing traffic causes short-term boat wave energy of up to 59 percent of streamflow energy and long-term boat wave energy of up to 16 percent of streamflow energy. Reduction of boat wave energy should focus on areas having large boat passage frequency such as the drift area at RM 10-12 and areas where bank erosion is most problematic. Techniques to reduce boat waves from a *single* boat include (1) use of flat bottomed boats, (2) use of 50 hp motors to increase boat speed, (3) keeping boats away from shorelines, and (4) reducing boat weight. Note that 50 hp motors should not be considered unless present boat weights are maintained. Also note that the finding of decreased wave height from 50 hp motors does not address any safety issues resulting from the increased boat speed or any environmental issues resulting from increased motor sizes. The actual reduction from some of the above boat wave energy reduction techniques in areas of large boat passage will likely be less than for a single boat because of altered boat operation in areas with a large number of waves. The problem with both flat bottomed hull shapes and increased power is that their benefits may not be realized in areas where wave reduction is needed most, namely high traffic areas. Decreased boat weight and keeping boats away from shorelines are two options that can result in benefits even when large traffic is present.

In summary, this study found that boat wakes are one of several factors contributing to bank recession. However, quantification of the relative magnitude of boat wakes to other factors such as river currents could not be determined. The results indicate that boat wakes may be a dominant factor during certain high boat usages times, discharges, and locations along the study reach. Although wake-induced erosion may be a secondary factor in bankline recession, it may be ecologically significant because of its persistence, distribution, and timing. However, bank recession associated with large flood events will likely overshadow the contribution from boat waves.

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic inches	1.6387064 E-05	cubic meters
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
foot-pounds force	1.355818	joules
inches	0.0254	meters
miles (U.S. statute)	1,609.347	meters
square miles	2.589998 E+06	square meters
yards	0.9144	meters

1 Introduction

The Kenaitze Indian Tribe requested that the U.S. Army Engineer Research and Development Center (ERDC) determine the relative contribution of boat-wake-induced bank erosion to total bank erosion along the Kenai River. The specific reach of interest along the Kenai River is the 11-mile reach downstream of Soldotna (Figure 1). Figures 2–5 show details of the study reach from the Soldotna Bridge at RM 21 to the confluence of Beaver Creek at RM 10. Details of the measurement sites and wave gage locations shown on Figures 2–5 are presented later in this report. This study follows an earlier study by ERDC in 2000 in which boat waves were measured on Johnson Lake and on the Kenai River for four different types of boats found on the Kenai River. The approach used in this study consists of a delineation of boat wave characteristics along the study reach and a geomorphic and bank stability assessment.

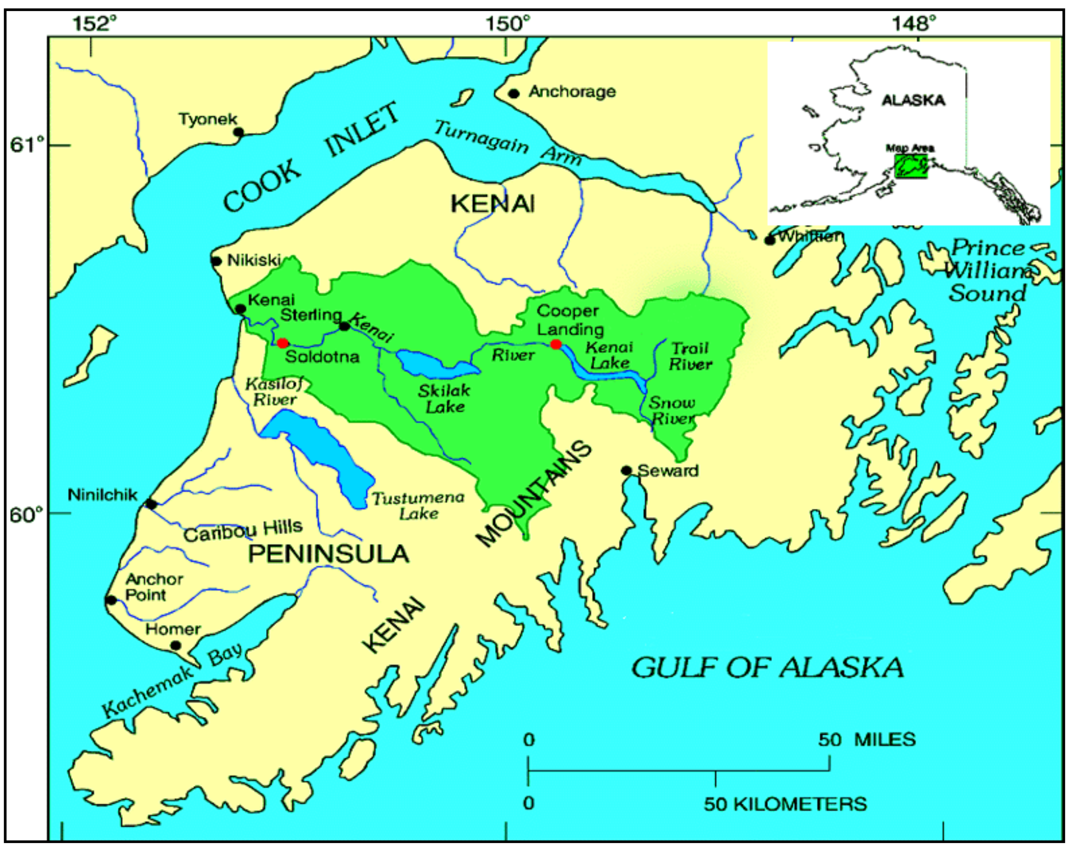

Figure 1. Map of study area and Kenai River Watershed. Study area is 11-mile reach of Kenai River downstream of Soldotna.

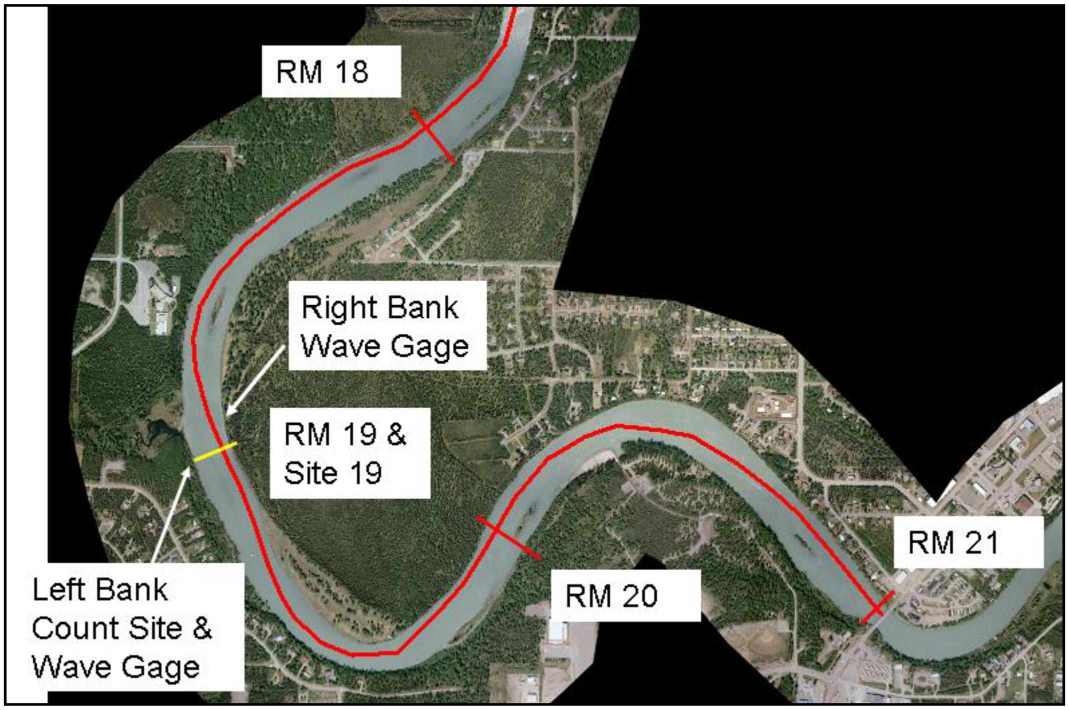

Figure 2. Location of counting and wave measurement sites and primary boat path on Kenai River, RM 21 to 17.6. RM 21 is the upstream end of the study reach.

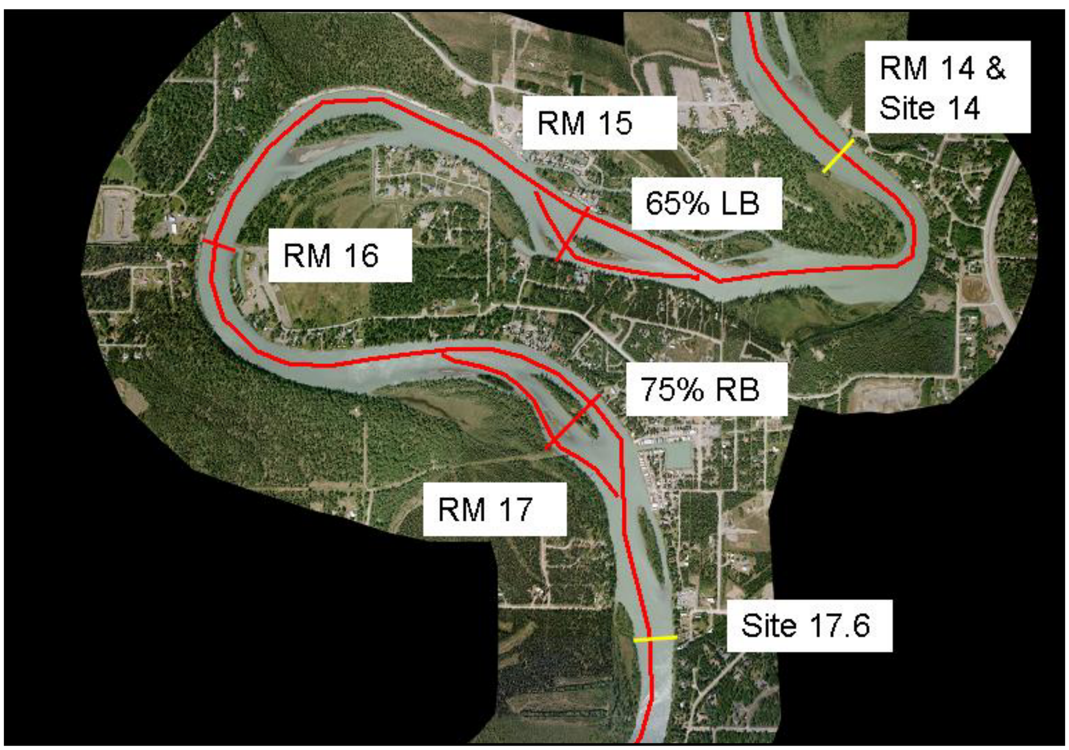

Figure 3. Location of counting and wave measurement sites and primary boat path on Kenai River, RM 17.8 to 14.3.

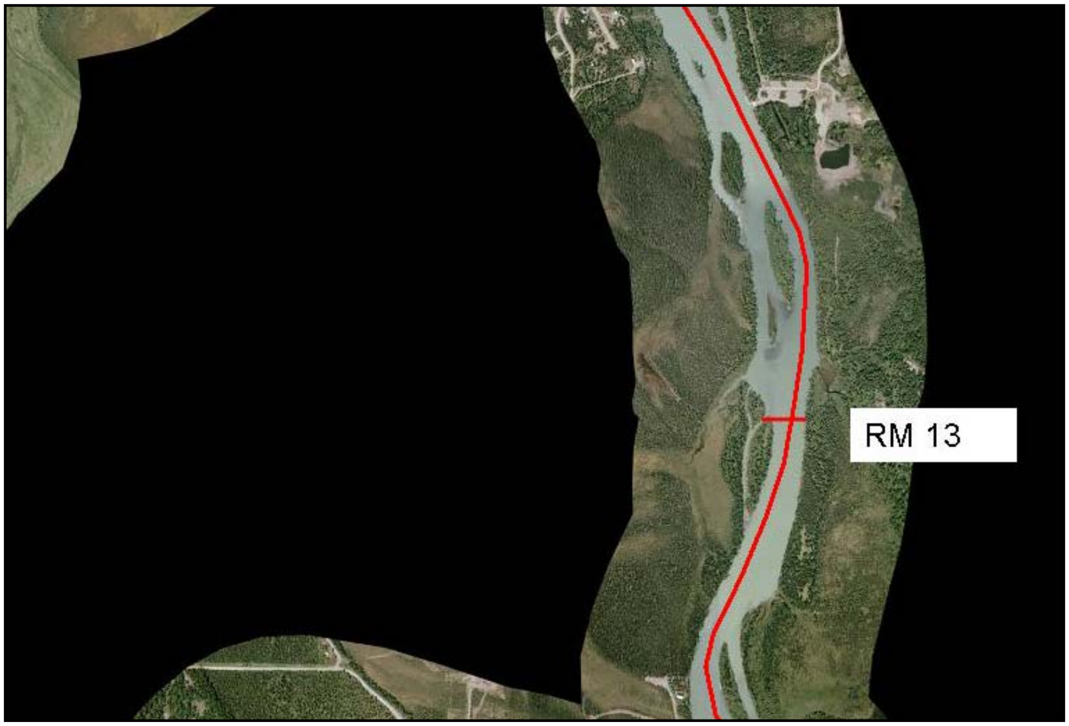

Figure 4. Location of counting and wave measurement sites and primary boat path on Kenai River, RM 13.8 to 12.2.

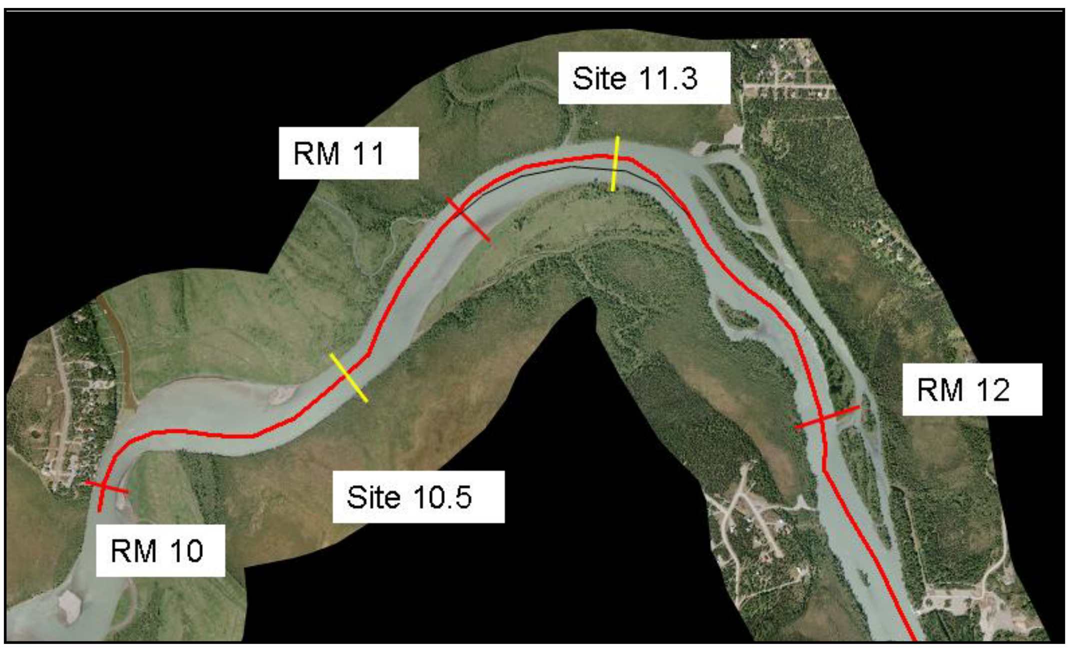

Figure 5. Location of counting and wave measurement sites and primary boat path on Kenai River, RM 12.5 to 10.0. RM 10 is the downstream limit of study reach.

2 Background

The Kenai River is a large proglacial stream draining the Kenai Mountains and portions of the Kenai Peninsula lowlands in south-central Alaska (Figure 1). The watershed includes 2,200 square miles (mi^2) of diverse topography, extending from the icefields of the Kenai Mountains to Cook Inlet. The river's origin is at Kenai Lake, from which it flows generally westerly for 69 miles to Cook Inlet. The average gradient is 0.0012, and the substrates are generally coarse gravels and cobbles, except in the lower 12 miles, where sands and silts predominate due to the lower gradient and tidal influence.

Glacial landforms dominate the Kenai, influencing the character of the river and its streamflow. Four stages during the Naptowne glaciation created moraine features such as Kenai and Skilak Lakes, as well as high terraces from glacial lake deposits interspersed with coarse strata from fluvial outwash. Both the bed material and the channel pattern reflect previous glacial discharges (Scott 1982) and, except for the most downstream 15 miles of the river, are considered "underfit." According to the American Geological Institute, an underfit stream is one that "appears to be too small to erode the valley in which it flows; a stream whose volume is greatly reduced or whose meanders show a pronounced shrinkage in radius. It is a common result of drainage changes effected by capture, by glaciers, or by climatic variations." Mean annual discharge at Soldotna is about 6,000 cubic feet per second (cfs), but flow regulation by the lakes and glacial melt water creates a unique seasonal variability with low discharge levels from mid-fall to late spring and sustained high discharges throughout the summer.

Accelerated stream bank erosion has been recognized as a concern on the Kenai River beginning in the early 1980s. Before that date very little fishing activity occurred on the main Kenai River and riparian development was limited. In the 1970s fishermen learned how to catch both Chinook (also known as King) and Sockeye (also known as Red) salmon in the turbid waters of the main Kenai River stem. The stream bank and boat-based sport fisheries grew rapidly as anglers improved techniques to harvest the abundant Sockeye, Chinook, and Coho (also known as Silver) salmon runs. Riparian areas were subdivided for residential and recreational development to accommodate the growing number of people who wanted live along the river. Concerns about stream bank erosion began to surface as the fisheries and related boat traffic expanded.

In the early 1980s concern about the future productivity of the Kenai River and growing conflicts between users precipitated public demand for reallocation of fishing opportunity, protection of the river, and regulation of some activities. The following actions were taken to address these concerns, some of which may directly or indirectly target power boat-related impacts:

1. The Kenai River was closed to all fishing on Mondays in July after 5 July beginning in 1983. In 1986 this restriction was expanded to no Mondays in May, June, and July. In 1999 the Alaska Board of Fish (BOF) liberalized these regulations to allow a non-guided, drift only fishery on Mondays in July. In 2003 non-guided drift-boat fishing on Mondays was expanded to May, June, and July. The fishing closures and drift-boat-only regulations reduced the number of powerboat trips on the River from several hundred to a much smaller but unknown number on the Mondays when they were in effect.
2. The Kenai River Special Management Area (KRSMA) was created by act of the Alaska Legislature in 1984. This statute gave the Alaska Department of Natural Resources (ADNR) the authority to adopt regulations to protect park lands and public safety within the boundaries of the KRSMA.
3. The U.S. Fish and Wildlife Service closed the section of the Upper Kenai River from Jim's Landing (RM 71) to Skilak Lake (RM 65) to power boats in 1986.
4. The section of the upper river between the power line crossing below Russian River (RM 73) and Skilak Lake (RM 65) was closed to powerboats in 1986 by ADNR regulation. An additional section was added following the revision of the 1997 Kenai River Comprehensive Management Plan that extended the closure from RM 73 to RM 80.7 (power line across the river just upstream from Princess Lodge). This additional closure went into effect in 1998.
5. In 1986 the ADNR adopted regulations limiting boat motors to a maximum of 50 horsepower (hp). The use of hovercraft, personal watercraft (i.e., jet skis), and airboats was prohibited.
6. In 1987 ADNR adopted regulations limiting boat motors to 35 hp.

7. In 1996 the Alaska BOF, which has statutory authority to promulgate regulations to allocate fisheries resources between users and for fisheries and habitat conservation, adopted the Upper Cook Inlet Riparian Habitat Fishery Management Plan. This policy directs the Alaska Department of Fish and Game (ADF&G) to monitor the effects of upper Cook Inlet fresh water sport fisheries on fish habitat. The BOF delegates the authority to ADF&G to close sections of stream bank to bank fishing to prevent degradation and erosion of stream bank habitat.
8. In 1997 ADNR adopted the Kenai River Comprehensive Management Plan and regulations that prohibited power boats on an additional section of the upper Kenai River from the Kenai Princess Lodge to the Russian River Ferry. It also banned personal water craft (jet skis) on a portion of Kenai Lake. This left only a small stretch of the upper river open to power boats, from RM 80.7 to Kenai Lake (RM 82). This stretch has no horsepower limit, but does have a no-wake speed limit.
9. In 1999 the BOF limited Sunday fishing to non-guided anglers only. Some limits had already been placed on Sunday guided fishing in the 1980s.
10. In 2000 the BOF adopted regulations reducing the maximum number of people in guided boats from six to five.

The net effect of these regulations was to reduce the area of the Kenai River subject to boat waves and the number of boat waves that would have otherwise occurred on Sundays and Mondays. These regulations have reduced the effects of power boating activities on the Kenai River, but the amount of reduction has not been quantified and would likely be difficult to measure.

3 Description of the Kenai River Sport Fishery

Wild Kenai River Chinook, Coho, and Sockeye salmon runs support the largest recreational fisheries for these species in Alaska (Pappas and Marsh.2004). Also present are pink salmon and Rainbow Trout. The Kenai River is the only major salmon producing system in Alaska that is road accessible for its entire length from Alaska's population centers in the rail belt. This includes Alaska's two largest cities Anchorage and Fairbanks. Anglers from the continental United States and many foreign countries also travel to the Kenai River to fish its world famous salmon runs. An average of 282,031 angler-days was spent fishing for all species on the Kenai River from 1977 to 2003, and 151,870 angler days spent fishing on the lower Kenai River from 1981 to 2003 (Pappas and Marsh 2004). Sport fishing on the Kenai River extends from early May to freeze up in the fall, with the majority of fishing occurring from May to September.

The sport fishery for Chinook salmon is very popular, and large numbers of anglers participate annually. Chinook salmon fishing is limited by regulation to the 50 miles of the Kenai River downstream of Skilak Lake. Chinook salmon return to the Kenai River in relatively distinct early and late runs. The early run begins in early May and has passed through the study area by late June. Late run Chinook enter the river in early July and continue into early August. The daily bag and possession limit is one Chinook over 20 in. The seasonal limit (1 April – 30 September) is two Chinook. Sport fishing harvest of Chinook and Sockeye salmon typically peaks sometime during the latter two weeks of July.

Most Chinook salmon in the study area are caught from power boats by using the fishing techniques of back trolling or back bouncing. Back trolling anglers slowly drift downstream, using the motor to slow the rate of drift as they pass through sections of the river known to hold large numbers of Chinook salmon, working lures or salmon eggs downstream of the boat. When they reach the end of the drift, they motor upstream and repeat the drift. Anglers who are back bouncing use similar gear, but they use their fishing rods to bounce the lure up and down on the bottom through fishing holes. They use their motors to hold the boat in channel locations, by slowing and controlling the rate of drift. Another technique, formerly the more preferred, is drifting. The boat is oriented in the current, perpendicular to the bank. Anglers cast bait upstream of the drifting boat and allow the bait to bounce along the bottom. Since the boat is moving with the current, the drift is completed more rapidly than with back trolling or bouncing; hence, there are an increased number of upstream boat trips to begin new drifts.

A total of 238,415 hr were expended in the late run Chinook salmon fishery in 2004. Effort was divided between 110,690 guided and 127,725 non-guided fishing hours (Pappas and Marsh 2004). Fishermen harvested 14,494 Chinook salmon (Pappas and Marsh 2004). In 2005 a total of 324 guides were licensed to fish from motor boats on the Kenai River, and many participate in the late run Chinook fishery (Pappas and Marsh 2004).

Power boat activity on the lower Kenai River in July is related primarily to the sport fishery for Chinook salmon. The number of boats is related to the anticipated size and timing of the run (movement of salmon upstream) but may vary in season with daily estimates of the run. Boats on the Kenai River are generally 16-20 ft in length with the majority being 20 ft. Each boat contains the operator and up to five people. Boats are not allowed to have motors larger than 35 hp. Regulations allow larger motor sizes that have been "detuned" to provide the equivalent of 35 hp. Power boat counts made at random 1-hr intervals between the Soldotna and Warren Ames bridges by ADF&G Sport Fish Division personnel in 2005 yielded the ranges shown in Table 1 (Eskelin 2005). These numbers are not directly comparable to the counts conducted in this study because this study focused only on the numbers of boats under way at speeds sufficient to make waves.

Table 1. Power boat counts on Kenai River between the Soldotna and Warren Ames bridges.

Date	Time	No. of Boats
19 Jul 05	0900-1000	446
	0400-0500	161
22 Jul 05	0900-1000	383
	0400-0500	30
23 Jul 05	0700-0800	379
	2200-2300	147

Because Sockeye salmon migrate near stream banks, most Sockeye are caught by bank fishermen or by fishermen in anchored boats. King and Hansen (2001) evaluated shore angler impacts to the Kenai River riparian habitats. Over a short monitoring period between June and August 1998, boat wake level and shore angler use had no significant effect on bank erosion, but increased shore angler effort resulted in increased bare ground. Fishermen often use boats to access Sockeye fishing areas not accessible from the road system. Also, power boats are utilized in the personal use dip net fishery occurring downstream of the Warren Ames Bridge, with some boats accessing this area by motoring downstream from upstream access locations. During the report period, less than 15 percent of the boat activity on the lower river was probably associated with the Sockeye fishery (Eskelin 2005).

The integrity of this fishery depends, in part, upon the character of the streambanks and riparian vegetation. Research has shown that riparian vegetation is beneficial to salmonid species by providing food, shelter, and shade that correspond with fish camouflage. Riparian vegetation influences nearshore water temperatures, decreases nearshore stream velocities and provides resting places for juvenile fish. Juvenile Chinook salmon inhabit areas with water velocities primarily between 0.09 and 0.6 ft/s and rarely use areas with velocities 2.1 ft/s or greater (Burger et al. 1982). In the Kenai River, an estimated 80 percent of the young Chinook salmon are within 6 ft of the bank where water velocities are less than 1 ft/s (Liepitz 1994, Burger et al. 1982). Another study showed most salmonids studied (Dolly Varden, Coho and Chinook salmon) selected focal point velocities between 0.0-0.9 centimeters/second (cm/s) and preferred large woody debris for cover (Dolloff and Reeves 1990). Even a small change to juvenile salmon habitat water velocities and depth may decrease habitat values and salmon survival (Liepitz 1994, Burger et al. 1982, ADNR 1986). Survival of early life stages of salmon is imperative to productive returns of adults. Scientific discussion is considerable about what life stage is actually limiting production of Pacific salmon.

Bank erosion destroys important habitat in the riparian zone. Accelerated erosion rates have been observed along the Kenai River between Beaver Creek at RM 10 and the Soldotna Bridge at RM 21. Figure 6 shows an example of an eroded bank in this reach. A possible cause of this erosion has been attributed to increased boat traffic. In order to test the validity of this assumption, it is necessary to develop a thorough understanding of the morphologic character of the system.

Figure 6. Typical failure of banks along the Kenai River includes loss of vegetative cover and increased erosion at water surface.

4 Delineation of Boat Wave Characteristics along Reach

General

Boat wave delineation consisted of a field study to measure waves at different sites along the river, boat counting to identify traffic variations along the 11-mile reach, and development of a simple empirical model of boat wave characteristics to describe variation of boat wave attack along the 11-mile reach. The field study was conducted from 19-23 July 2005 to coincide with peak boating levels during the late run of Chinook.

2001 and 2005 Kenai River boat wave studies

Prior to this study, Maynord (2001) conducted a study to measure waves from typical Kenai River boats under controlled conditions where no other boats were present. In Maynord (2005), the 2001 measured wave data were used to develop a general boat wave height equation based on boat speed, boat weight, hull type, and distance from the boat. In developing that equation, it became apparent that the speeds of the boat "Willie Predator" (WP) with a 35 hp motor in tests on Johnson Lake were not consistent with other tests. This finding was based on the following comparisons that were all made with six people in the boat:

1. The average speed of WP with 50 hp on the Kenai River was 30.1 mph downbound and 20.2 mph upbound for an average in both directions of 25.2 mph. This speed agrees with the average speed on Johnson Lake of 25.1 mph.
2. The average speed of the boat "Koeffler" (KF) with 35 hp on the Kenai River was 27.3 mph downbound and 17.7 mph upbound for an average in both directions of 22.5 mph. This speed agrees with the average speed on Johnson Lake of 22.3 mph.
3. The average speed of the KF with 50 hp on the Kenai River was 29.1 mph downbound and 21.3 mph upbound for an average in both directions of 25.2 mph. This speed agrees with the average speed on Johnson Lake of 26.0 mph.
4. The average speed of the WP with 35 hp on the Kenai River was 25.3 mph downbound and 16.0 mph upbound for an average in both directions of

20.6 mph. This speed does not agree with the average speed on Johnson Lake of 14.5 mph.

During the Maynord (2001) tests, it was observed that the WP with 35 hp at Johnson Lake had a large amount of bilge water at the end of the tests. The significance of this was not fully understood until the development of the boat wave equation in the Maynord (2005) analysis using boat speed. The average speed on Johnson Lake for the WP with 35 hp should have been about 20.6 mph as shown by the WP with 35 hp tests on the Kenai River. This finding means the WP tests with 35 hp on Johnson Lake are not valid and were not used in the Maynord (2005) study. Subsequent recommendations in this report on techniques to reduce boat waves will reflect the omission of the WP 35 hp data on Johnson Lake.

Site selection

The Kenai River below the Soldotna Sterling Highway Bridge (RM 21) and Beaver Creek was divided into four roughly equal sections represented by sites at river miles from the mouth of the Kenai (RM 10, 14, 17, and 20). Observation of the major drift area near RM 11 resulted in the addition of a fifth representative site. Within these river reaches, four sites were needed to collect data characterizing boat traffic and five to collect boat wave energy data. Site selection criteria included:

- A single uniform stream channel without sandbars or shallows to allow unobstructed wave measurements.
- Road access.
- Electrical power for the cameras.
- Wave gages and computers.
- An unobstructed view of the river for boat data collectors.
- Landowner approval to use sites on both sides of the river.

Sites at or in close proximity to boat launches were excluded. Mary King, ADF&G Kenai River Research biologist, provided expert advice and assistance in identifying and evaluating potential study sites that would characterize boating activity on the lower Kenai River. As project leader, Steve Maynord made the final site section.

Sites that met essential criteria listed above for both boat data and wave data collection were located at RM's 19 right and left bank (viewed looking downstream), 17.6 right and left bank, 14 right and left bank, 11.3 right bank, and 10.5 left bank. Only the sites at RM 19 left bank and 17.6 right bank were accessible by road, and only 17.6 right bank had electrical power. All other sites were accessed by boat, and generators and batteries were used to provide power. An additional boat count and wave data collection site was added at RM 11.3 because this site had the highest level of boat traffic. Approval to use all of the sites was secured from the landowners. Boat counting teams were deployed at each of the five sites during the field study for the days shown in Table 2. The five sites are shown on Figures 1-4 and described in Table 2. Dates and locations of boat wave measurement are also shown in Table 2. Bank classification and cross section for each site are provided later in this report. Additional descriptions of the sites are as follows:

1. **RM 19.0.** The boat data collection site was located on the left descending bank approximately 100 feet upstream from the upstream river access stair at the Slikok Creek public access site. The top of the bank where the observers were stationed is approximately 3 feet above ordinary high water (OHW). The two wave data collection sites at RM 19 were located at the boat data collection site on the left bank and on the right descending bank across the river from the Slikok Creek public access site. Boating activity at RM 19 is primarily from boats traveling from the Centennial Park boat launch to and from fishing areas downstream. It is also at the upper end of the College Hole Chinook salmon fishery, and receives a moderate level of activity from fishing boats cycling through this popular fishery. RM 19 has an exposed rock about 60 ft from the right bank. Although boats can go between this rock and the right bank, the vast majority of boats go in the larger width between the rock and the left bank. This effectively removes boat traffic from being close to the right bank.
2. **RM 17.6.** The boat data collection site was located on the right descending bank on the upstream boundary of Mrs. Alouise Gehrke's property on Knights Drive. The boat data collection site was located on the deck of a small outbuilding approximately 15 feet above the surface of the river at OHW. This site provided a good view of the river. The boat data collection site was approximately 10 yards inland OHW. No wave data were collected at this site. Boating activity at this site is from boats traveling from a number of upstream boat launches and private docks to and from popular Chinook and Sockeye salmon fishing areas downstream. The counting site was also at the head of the popular and productive Graveyard Hole and receives a moderate level of boating activity from fishing boats cycling through this fishery.

3. **RM 14.0.** The boat data collection site was located on the right descending bank on top of the bluff at Stewarts Landing. The bluff is approximately 30 feet high providing a good view of boats in the river. The counting site was approximately 10 yards inland from OHW. The wave data collection site was across the river on the left descending bank and approximately 50 feet downstream from the left bank marker used by the boat data collectors to delineate the counting line. The shoreline was too swampy to put wave gages and recording equipment at the marker. Boating activity at RM 14 results from a large number of boats traveling to and from upstream boat launches and private docks to downstream fishing areas and boats traveling from the Stewarts Landing and River Bend Campground boat launches, which are approximately 1/4 mile below the boat data collection site, to and from upstream fishing areas. Boating activity was also moderate from a Chinook and Sockeye salmon fishery located in the data collection area.
4. **RM 11.3.** The boat and wave data collection sites were both located on the right descending bank approximately 1200 ft below Eagle Rock. The site was on top of a 6 ft bank at the water's edge and was within the tidally influenced area of the Kenai River with approximately 6 ft of tidal range during the study. Boating activity at this site was very heavy during the study. Boats participating in the intensive Beaver Creek Chinook fishery, Sockeye salmon fishery, fishing sites below Beaver Creek, and the dip net fishery below the Warren Ames Bridge passed this site. Boats full of fishermen participating in the Beaver Creek Chinook fishery back trolled and back bounced from approximately RM 12 down to about RM 10.8 as they fished. When fishermen reached RM 10.8, they motored back upriver to RM 12 at high speed, joined the line of drifting boats, and repeated the drift. Figure 7 shows boats on the river at RM 11.3 as they drift downstream along the right descending bank while fishing. The predominately upstream traveling boats dominated the wave-making boats counted at RM 11.3.
5. **RM 10.5.** The boat and wave data collection sites were both located on the left descending bank on top of an 8-ft cut bank. This site was approximately 1/2 mile upstream from the confluence of Beaver Creek and the Kenai River and was within the tidally influenced section of the Kenai River with approximately 8 ft of tidal range during the study. The boat data collection site was approximately 4 yd from the water's edge.

Figure 7. Kenai River, looking upstream from boat count and wave measurement site on right descending bank at RM 11.3. Picture shows the major drift area in the study reach. Boats generally drift down the right descending bank and motor up the left descending bank.

Table 2. Boat counting and wave measurement sites.

Day/Site	RM 19.0	RM 17.6	RM 14.0	RM 11.3	RM 10.5
Tuesday – 7/19/2005	Count from LB, WG on LB & RB	Count from RB	Count from RB	Nothing	Count from LB
Wednesday – 7/20/2005	Count from LB	Count from RB	Count from RB	Count from RB	Count from LB, WG on LB
Thursday – 7/21/2005	Count from LB	Count from RB	Count from RB	Count from RB, WG on RB	Count from LB, WG on LB
Friday – 7/22/2005	Count from LB	Count from RB	Count from RB, WG on LB	Count from RB, WG on RB	Count from LB
Lat-Long	N60 28′ 53.9″, W151 07′ 33.4″*	N60 29′ 51″ W151 06′ 15″	N60 30′ 47.8″ W151 05′ 35.6″	N60 32′ 53″ W151 07′ 2″	N60 32′ 24″ W151 07′ 57″
Water surface width during field study**	453 ft	468 ft	432 ft	525 ft	441 ft

*Left bank latitude-longitude. Right bank at N60 28′ 57″ W151 07′ 26″.
**As determined by a Nikon ProStaff Lazer 440 range finder with an accuracy of ± 0.5 m at 440 meters.
Note: LB = Left bank; WG = Wave gage; RB = Right bank.

Water levels, discharges, and tides

The locations used to count boats and measure waves had two locations where water levels were affected by tides and three locations in the non-tidal reach. RM 10.5 and 11.3 were affected by tide-induced water level variation at the site ranging up to about 8 ft during the field study. RM 14.0, 17.6, and 19.0 were unaffected by tides. Mean daily discharges from the U.S. Geological Survey (USGS) gage 15266300 "Kenai River at Soldotna" during the field study were 7/19=13900 cfs, 7/20=14200 cfs, 7/21=14000 cfs, and 7/22=14200 cfs. The average discharge for the 4 days was 14075 cfs. Predicted tides at the mouth of the Kenai River are shown in Figures 8 and 9. The predicted tides are for the Kenai River entrance derived from the Seldovia Gage 9455500, 1983-2001 epoch. Since the tidal variation at low tide is less than the normal depth of the river due to flow, the water level in the tidal reach of the river is dictated by two different mechanisms: river flow and magnitude of the tide. (Note: "Normal depth" is a specific hydraulic term that refers to the depth resulting from the channel slope, channel roughness, channel width and bank slope, and amount of flow in the river. It can be used for the study reach herein to mean the river depth when high tides do not elevate the water level.) During low to intermediate tides, the water level in the river is at normal depth and is determined by the amount of flow in the river. During high tides, the water level in the river is predominately determined by the magnitude of the tide. Water level measurements were conducted on 21 July 2005 to determine the relationship of normal depth in the river at low tides and predicted high tide at the mouth of the river. Figure 8 shows the measurements at RM 10.5 on 21 July that were plotted to have the peak water level measurement coincide with the peak predicted tide at 1800 on 21 July. The lowest measurements define the water level at the normal depth in the river as shown in the figure. Figure 8 shows that, for the tides and discharge in the river during the field study measurements, normal depth water level at RM 10.5 lasted for an average of 8 hr followed by a high tide that lasted an average of 4.5 hr. Over a typical 24-hr period during the field study, high tides lasted for about 7 hr and normal depth lasted about 17 hr. The same tide plot is shown in Figure 9 for water level measurements conducted at RM 11.3 on 21 July. At RM 11.3, normal depth lasted an average of 9 hr followed by a high tide that lasted an average of 3.5 hr. Note that, for different tides, river miles, and discharges, these numbers will change significantly. For example, at lower discharges in the river, time periods of normal depth will have less duration and time periods of high tides will have greater duration. In similar fashion, periods of low tidal range will have greater duration of normal depth periods and less duration of high tides. Both plots are shown relative to mean lower low water (MLLW) at the mouth but should be considered as relative plots because the absolute relationship of water levels at RM 10.5 and 11.3 to predicted tide levels has not been determined. Upstream tidal effects vary with tide magnitude and flow in the river but generally extend as far upstream as “The Pillars” at about RM 12.5.

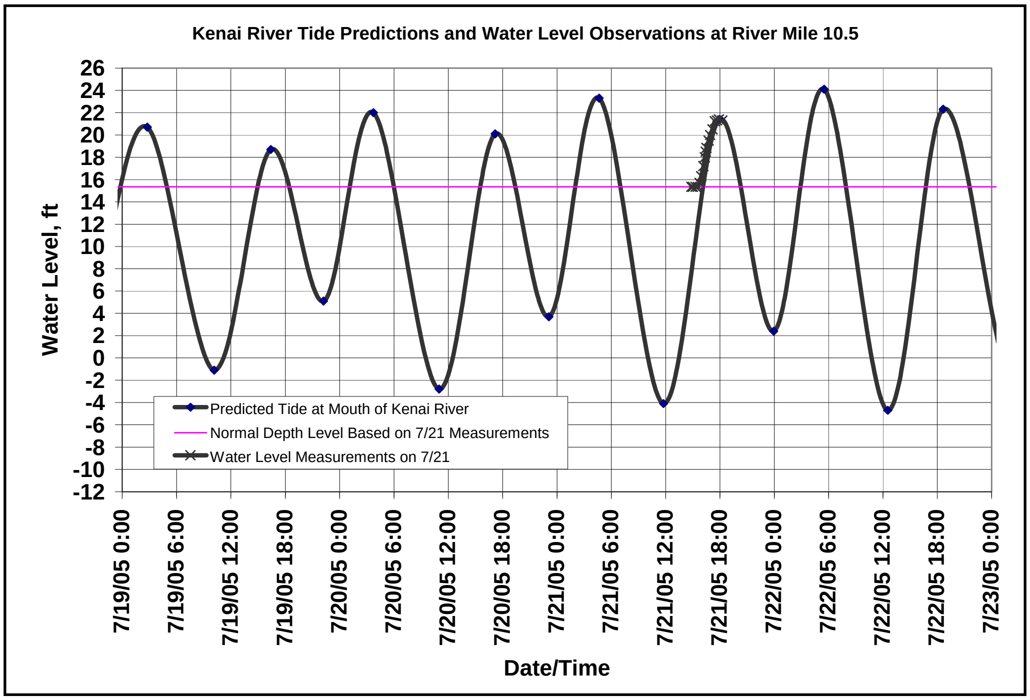

Figure 8. Kenai River tide predictions and water level observations at RM 10.5. Water level datum is the tide elevations at the mouth, which are not absolute elevations at RM 10.5. Minimum water level at RM 10.5 is the normal depth line.

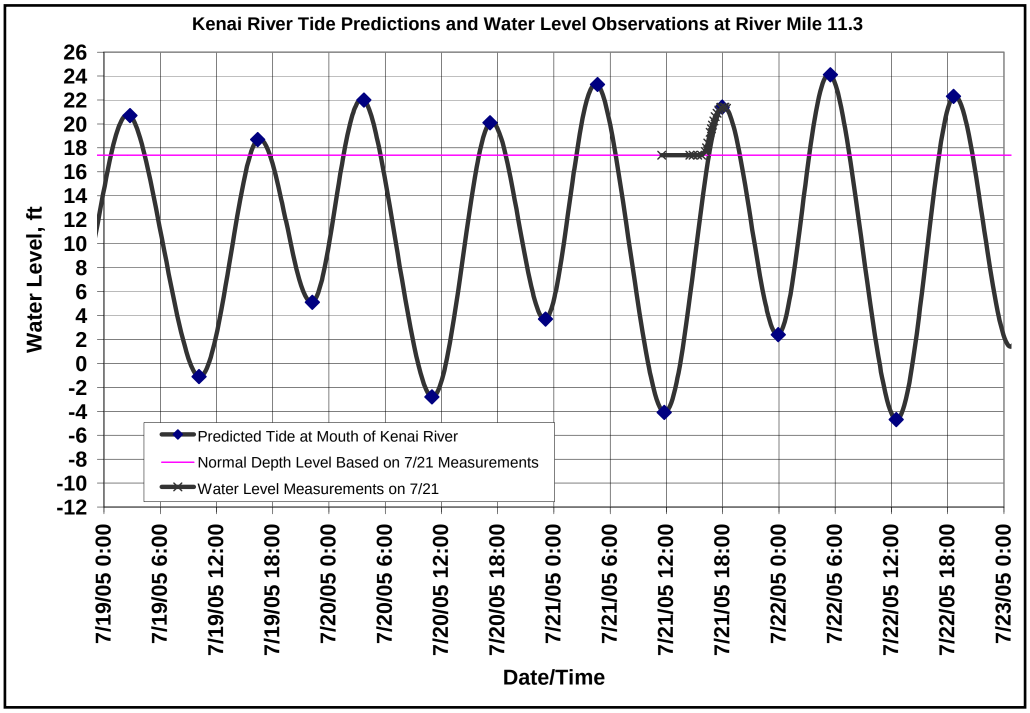

Figure 9. Kenai River tide predictions and water level observations at RM 11.3. Water level datum is the tide elevations at the mouth, which are not absolute elevations at RM 11.3. Minimum water level at RM 11.3 is the normal depth line.

Near-bank cross sections

Near-bank cross sections were measured at the four locations of wave measurement. Figure 10 shows the cross section at RM 10.5 along with water level at the time of survey, water level during the field survey, and maximum tide level during the 21 July water level measurements. The plot is shown relative to MLLW at the mouth but should be considered to show relative elevations because the absolute relationship of water levels at RM 10.5 to predicted tide levels has not been determined. The mean daily discharge during the 21 August 2005 survey of the bank at RM 10.5 and 11.3 was 12,200 cfs. The mean daily discharge during the field study was 14075 cfs. Based on water surface elevation computations near RM 10.5 and 11.3 on the Kenai River reported by Fischenich (2004), the higher discharge had a 0.5 ft higher stage. This stage difference was added to the survey water surface elevation to determine the water level during testing at the near-bank cross section. This approach was checked by comparing measured depths at the wave gages during the field study and found to be in close agreement. The water level measurements on 21 July 2005 were used to place the cross section plot in the MLLW datum used in the tidally affected cross sections. Also shown are the locations of the wave measurement gages.

Figure 11 shows the cross section at RM 11.3 using the same parameters as for RM 10.5. For the non-tidal wave measurement cross sections at RM 14.0 and 19.0, the data were plotted with a local or arbitrary datum as shown in Figures 12-14. The mean daily discharge during the 27 August 2005 survey of the bank at RM 14 and 19.0 was 11700 cfs. The stage difference between the survey date and the testing dates was 0.8 ft at RM 14.0 and 0.95 ft at RM 19.0.

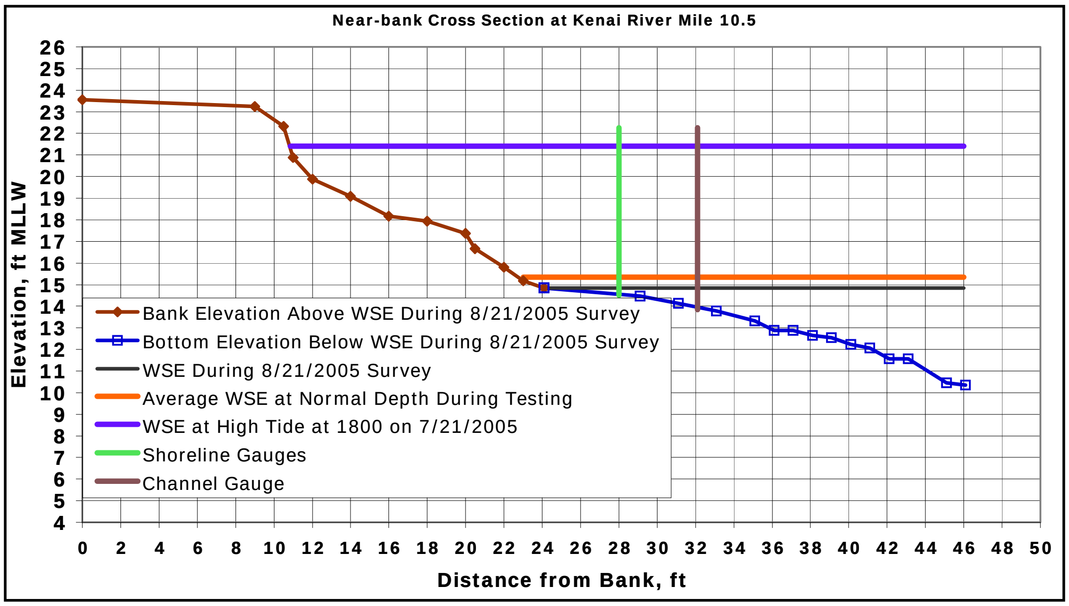

Figure 10. Near-bank cross section at RM 10.5, water surface elevations (WSE), and location of gages. Plot shows that maximum water level variation due to tides during testing on 21 July 2005 was about 6 ft.

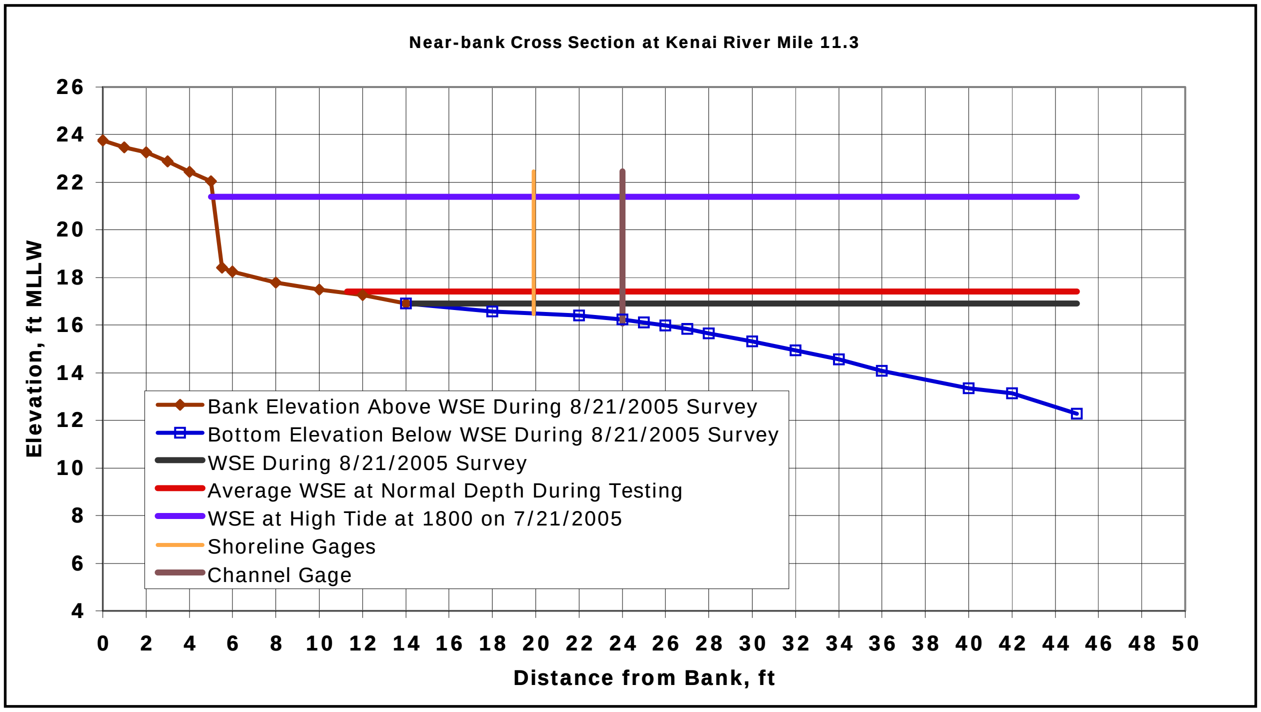

Figure 11. Near-bank cross section at RM 11.3, water surface elevations (WSE), and location of gages. Plot shows that maximum water level variation due to tides during testing on 21 July 2005 was about 4 ft.

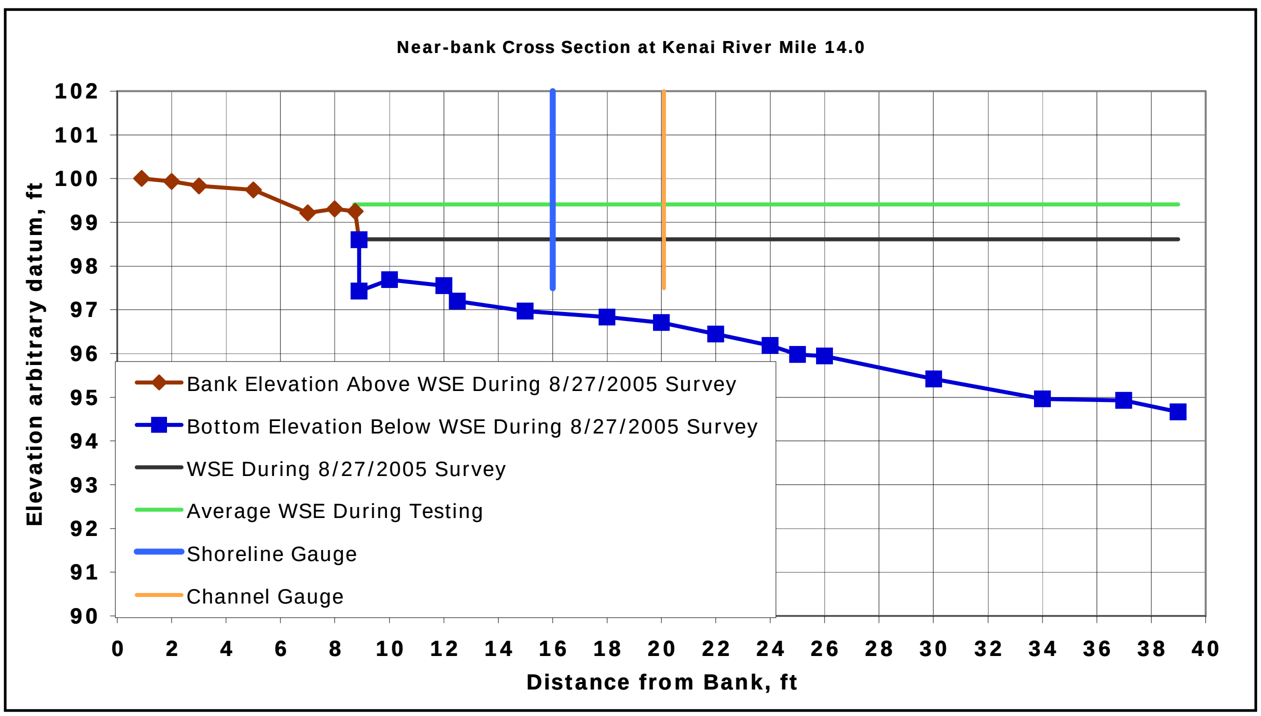

Figure 12. Near-bank cross section at RM 14.0, water surface elevations (WSE), and location of gages. No variation due to tides was present at this site.

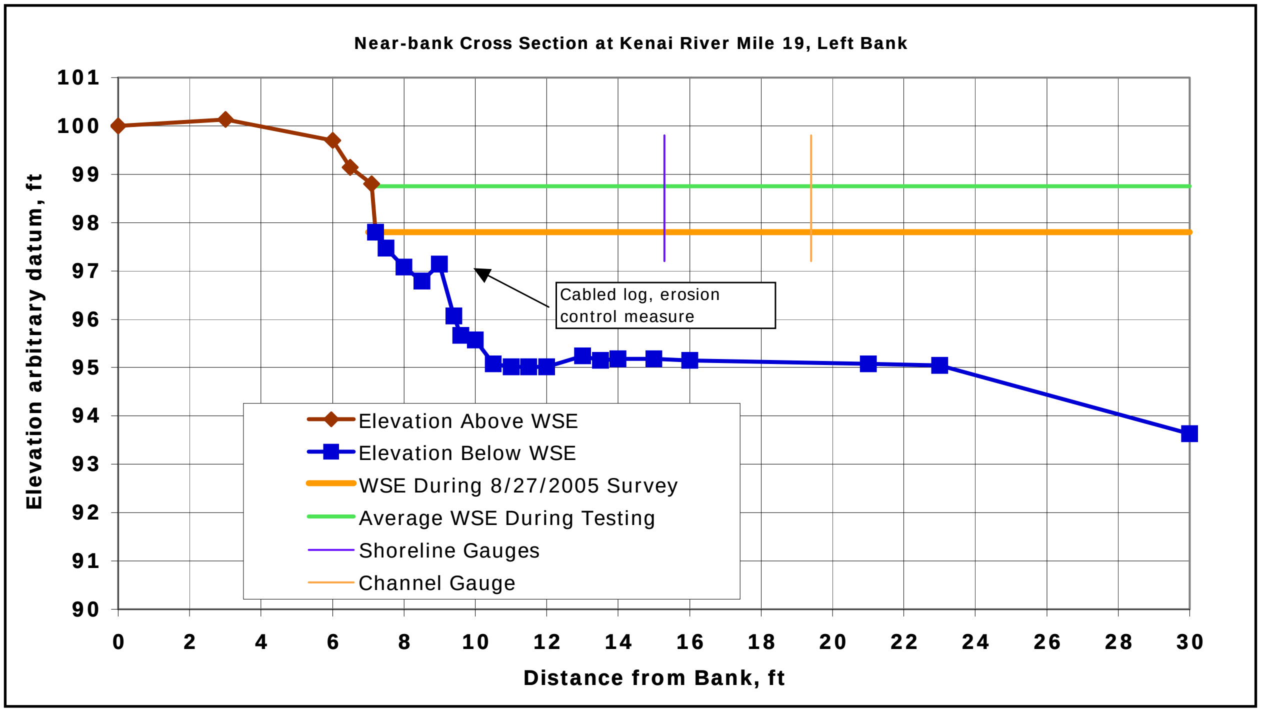

Figure 13. Near-bank cross section at left bank of RM 19.0, water surface elevations (WSE), and location of gages. No variation due to tides was present at this site.

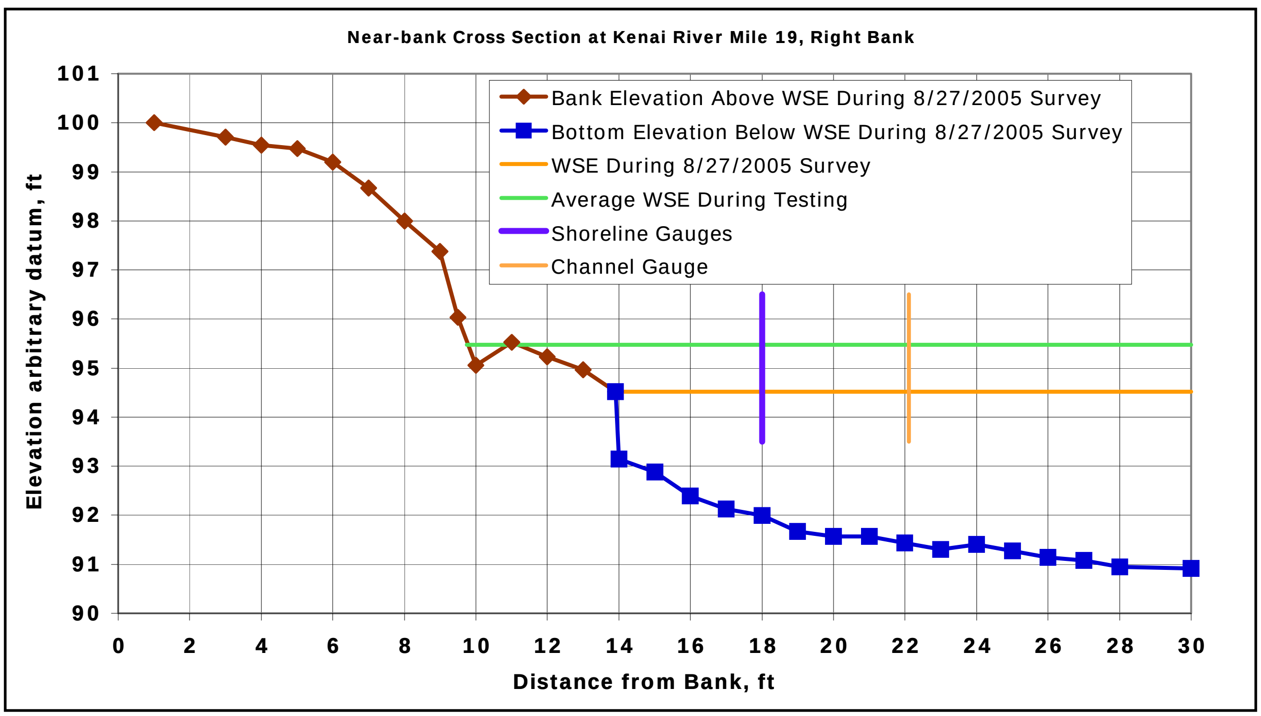

Figure 14. Near-bank cross section at right bank of RM 19.0, water surface elevations (WSE), and location of gages. No variation due to tides was present at this site.

Discharge and velocity measurements

Acoustic Doppler Current Profile (ADCP) and point velocity measurements were conducted on 23 July 2005 as shown in Table 3. ADCP was used to measure discharge and velocity across the river at the five wave measurement sites and seven other sites identified by the bank erosion team. A hand held velocity meter was used to measure velocity at 0.6 × depth below the water surface to obtain an estimate of depth-averaged velocity. The hand held measurements were conducted in the near bank region where the ADCP data collection was not possible due to shallow depths. Appendix A shows cross section and velocity plots from the ADCP.

Table 3. ADCP discharge measurements.

Time on 7/23	River Mile	Discharge, cfs	Tide	Direction	Filename	Left, right bank distance, ft	Velocity plot plate number
0840	10.5	21304	High	L-R	KENA000R	8,3	No plot
0845	10.5	19787	High	R-L	001R	8,3	A1
0855	10.8	18877	High	R-L	003R	9,10	A2
0910	11.3	16638	High	R-L	005R	24,9	A3
0915	11.3	18868	High	L-R	006R	20,25	No plot
0935	12.4	14495	None	R-L	007R	18,10	No plot
0945	12.4	14674	None	L-R	008R	18,?	A4
1100	19.0	13756	None	L-R	009R	8,6	A12
1125	18.5	14264	None	R-L	011R	7,8	A11
1158	17.6	14079	None	R-L	012R	12,6	A10
1235	16.3	15763	None	R-L	013R	8,?	A9
1250	16.3	13700	None	L-R	015R	20,12	No plot
1257	15.9	13027	None	R-L	016R	14,*	A8
1329	14.4	14490	None	L-R	018R	7,12**	A7
1348	14.0	14869	None	L-R	019R	8,12	A6
1415	13.2	14914	None	L-R	020R	12,15	A5
1444	10.5	14147	Low	L-R	021R	16,16	A1
1510	11.3	14676	Low	R-L	022R	10,15	A3

*Unknown because of shallow right bank (RB) area
**RB is in an embayment

Boat counting and boat characteristics–procedures

Boat counting was conducted by two-person teams at five sites along the 11-mile reach of the river. Both volunteers and staff of some of the local and state agencies comprised the counting teams. Boats were counted from 0700-1900 for 4 days from 19-22 July at four of the sites and 3 days from 20-22 July at RM 11.3. For each boat passing a site, the volunteers would record the information shown in Table 4 used to describe each boat. While some of the sites recorded boats that drifted by the site, the focus was on those boats passing the site under power and making waves.

Table 4. Data sheet for Kenai River boats.

Location: River Mile: ________________ Bank (Looking Downstream): Left Right

Date: July 19 20 21 22 23 24 , 2005

Observers:____________________________________

Note: Most important items toward left side of table. Work left to right. Get as many items as possible.

(1) Time	(2) Operation Mode: Plane/Bow up/No wake	(3) Boat Position: Distance or 1/3 Position (Far, Mid, Close)	(4) Direct: Up/Down	(5) Number of People in boat	(6) Boat Type: V-hull/Flat/Other	(7) Comment
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	
	P B N	F M C	U D	1 2 3 4 5 6	V F O	

Several actions were taken to minimize classification errors in data collected. The project leaders held a class for data collectors in how to identify boat hull types, position in the river, operation mode, the number of people in the boat, and up or down direction. Data collectors were taught how to properly fill out Table 4. Pictures of different V-hull, Flat bottom and "Other" boat types were provided in each site manual for reference. Most of the boat data collectors who worked on the project had Kenai River experience and were familiar with the boats used on the river and their operation. Less experienced observers were paired with experienced observers. The most experienced observers were stationed at the sites with the most boat traffic: RM 11.3 and 10.5. Two observers were stationed at each site. This allowed one observer to concentrate on visually identifying boat attributes and the other to enter the data accurately on the form. Dual observers provided two sets of eyes to ensure that the data collected were correct (e.g., the boat was a v-bow and not a flat bottom). Observers worked a 6-hr shift each day and switched duties periodically to minimize fatigue and individual observer error. The data collection sites were selected for a clear view of the river. Two of the sites, RM 14 and 17.6, were located on high banks 20 to 30 ft above the river, allowing observers to look down on the boats and easily determine position in the river. RM 10.5 was located on an 8 ft bank, placing a standing observer's eyes 13 to 17 feet above the river at low and mid-tides. RM 11.3 was sited on a 6 ft bank at mid- and low tides. RM 19 was sited on a low bank. The maximum width of the river at any of the boat data collection sites was 188 yd. It is not hard to determine boat type, number of people, etc. at these distances. All boat data collection was scheduled during daylight hours 0700 to 1900 hr in July when visibility was good and there was 20 hr of daylight. It rained the first morning (19 July), but the clouds were high and visibility was good. The next 3-1/2 days the weather ranged from high broken overcast to clear. Time lapse cameras were mounted at each site to tape the same boats on which observers were collecting data. These tapes can be reviewed to check the accuracy of the observer counts.

The project manager (Lance Trasky) periodically visited sites at RM 10.5, 11.3, 14, 17.6, and 19 and checked to ensure that boat data were collected and entered consistent with project protocols. Visits were made at random times over the course of the project, when other project demands allowed. The project manager made concurrent observations with the observers at those sites to check the accuracy and consistency of the boat classification data entered. The project manager used binoculars and a range finder accurate to ± 0.5 yd at 400 yd to check boat data and position entries and worked with observers to correct the cause of any classification errors noted.

A total of 572 concurrent observations were made at all the boat data collection sites. Table 5 summarizes the types and frequency of observer classification error. A total of 30 (1.04 percent) of the 2,860 pieces of data collected in the error measurement process (572 observations x 5 types of data) were judged to be in error. The largest source of individual classification errors averaged for all sites and observers was operational mode (whether a boat was planing or bow up) at 2.1 percent or 12 of 572 observations. Operational mode classification error for the five sites ranged from 8 percent to zero. Operational mode classification appeared to be a greater problem at some sites and with some observers. The largest number of errors (8 percent) occurred at mile 17.6, where operational mode data from 10 of 124 boats observed were judged to be in error. This error generally occurred when boats were powering up and slowing down as they crossed the counting line and the operational mode (i.e., planing or bow up) was less clearly defined and made classification more difficult. Boat position classification error averaged for all observers and sites was also 2.1 percent or 12 out of 572 observations and ranged from 4.9 percent to zero by site. Position classification errors were observed only at data collection sites RM 11.3 and 19. The largest number of position classification errors (4.9 percent) occurred at RM 11.3 where the position of 10 boats of the 204 sampled in the error measurement process was judged to be in error. Both of these sites were located on low banks and did not provide the elevation perspective provided by the sites on high banks. The river at these two sites was also wider than the other sites. Classification was further complicated at RM 11.3 by the tidal cycle, which caused the bank-to-bank distance to vary by up to 10 yd. The position of boats that were very near the boundary between the middle, middle far, and the far fifth of the river were more difficult to determine, and a few boats that were on the far edge of the middle lane, the middle far lane, or on the near edge of the far lane were misclassified. This may have also been a problem at RM 10.5, which also received a very high level of boat traffic, but the limited number of concurrent observations made at this site did not detect boat position errors. In all cases the erroneous position estimate was only one zone off from the correct distance (e.g., MF instead of F), and in most instances less than 5 yd from the correct classification.

Table 5. Boat classification error by site, 19-22 July 2005.

	Observations		Operation Mode		Boat Position					Direction		Passengers						Boat Hull Type		
Site	Total	Measurement Error Tests	Plane	Bow Up	F	FM	M	MC	C	Up	Down	1	2	3	4	5	6	V	Flat	Other
RM 10.5	5,269	42	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
RM 11.3	7,123	204	0	0	6	4	0	0	0	0	0	0	0	0	2	0	0	0	0	0
RM 14.0	4,514	144	0	2	0	0	0	0	0	0	0	0	0	0	2	0	0	0	0	0
RM 17.6	1,471	124	0	10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
RM 19.0	1,138	58	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	2	0	0
Totals	19,515	572	0	12	6	6	0	0	0	0	0	0	0	0	4	0	0	2	0	0
Percent				2.1	1.0	1.0									0.6			0.3		

Footnotes:
1. Site 14 observers called bow up twice when boat was planing
2. Site 17.6 observers called bow up 10 times when boat was planing
3. Site 11.3 observers called Far 6 times when boat was in Middle Far zone.
4. Site 11.3 observers called Middle Far 4 times when boat was in Middle zone.
5. Site 19 observers called Middle Far 2 times Far zone.
6.Site 11.3 observers called 4 people 2 times when there were 5 people in the boat
7. Site 14 observers called 4 people in a boat 2 times when there were 5 people in the boat
8. Site 19 observers called V bow twice when boat was flat bottomed

The third largest source of classification error averaged for all sites and observers (0.6 percent) was the number of people in a boat. The error by site ranged from 0.9 percent and 1.4 percent. In four instances at two sites, observers estimated that four people were in a boat when there were five (0.9 percent error) and six (1.4 percent error). In the four instances that this error was observed, the boat was in middle far or far zones. The passenger that was not counted was either lying or bending over, or was behind another passenger who was standing up. The fourth largest source of classification errors was misclassification of boat hull type (0.3 percent). At one site observers classified two boats as V-bow when the boat was actually a flat bottom boat (3.4 percent). An older model boat was observed at this site that had a v-bow but actually had a flat bottom.

The larger than anticipated number of boats passing all the research sites and particularly sites RM 11.3 and 10.5 may have been a source of error. When the volume of boat traffic was very high, observers had only a few seconds to determine all five boat attributes and record the data before switching to the next boat. The very high volume of boat traffic at RM 10.5 and 11.3 made classifying and recording boat data challenging. These sites were in the lower Kenai River drift Chinook fishery where boats full of fishermen would drift from approximately RM 12 down to RM 10 as they fished. When fishermen reached RM 10, they motored back upriver to RM 12 at high speed to repeat the drift. Ten to 12 times per day, up to 12 boats would cross the counting line in quick succession. The observers and recorders developed techniques to ensure that correct data were entered for all boats, but entry errors may have occurred for some boats and it is possible that a few boats may have been missed completely. It is unlikely that this is a significant issue. The observer error in classifying boat attributes was relatively low. The 95 percent and 99 percent confidence limit intervals for the observer error sampling results were estimated by using the formula for confidence interval estimates for proportions P ± 1.96 @p and P ± 2.58 @p respectively. The sample proportion for total and individual errors P was used to estimate p (Snedecor and Cochran 1989 pp 210 -211). Using this formula, the 95 percent confidence limits for the total number of observer errors (i.e., 30 out of 2860) is 1.1 percent ± 0.38 percent, and the 99 percent confidence limit is 1.1 percent ± 0.99 percent. The 95 percent confidence limit for the measurement of the two largest types of boat classification error is 2.1 percent ± 1.2 percent and the 99 percent confidence limit is 2.1 percent ± 1.5 percent. Confidence limits were not calculated for other individual boat classification parameters errors, but will be the same or lower.

Boat counting and boat characteristics–results

The numbers of boats counted by site and for each 12-hr monitoring period are shown in Figure 15. Note that the sites can be grouped into three categories. The first category has two sites (RM 19.0 and 17.6) having low levels of boat traffic averaging about 325 boats per 12-hr monitoring period. The second group has two sites (RM 14.0 and 10.5) having medium levels of boat traffic averaging about 970 boats per 12-hr monitoring period. The third category has one site (RM 11.3) having high levels of boat traffic averaging about 2,375 boats per 12-hr monitoring period.

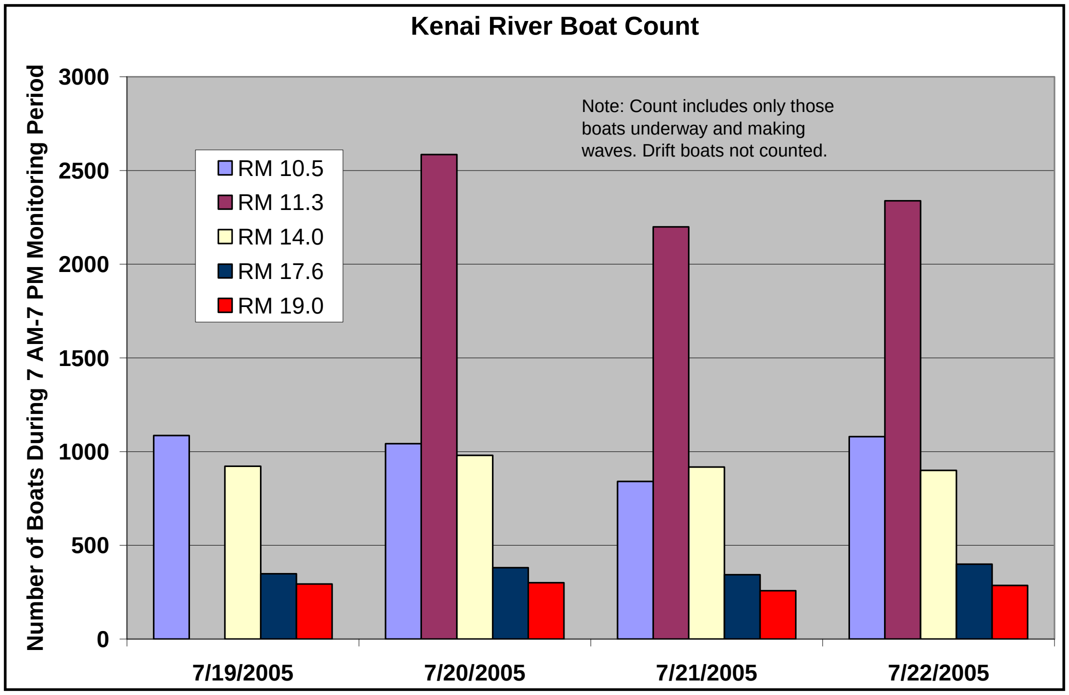

Figure 15. Number of boats passing by each site for each 12-hr monitoring period. Does not include boats not making waves such as drift boats. Note that data were not taken at RM 11.3 on 19 July 2005.

Figure 16 shows the breakdown of operation mode for each site based on the average number of boats per 12-hr monitoring period. Bow up refers to operation at slower speeds and larger boat wakes and is visually characterized by the bow higher than the stern. Planing mode has higher speeds and smaller boat wakes and is visually characterized by the bow being only slightly higher than the stern. While some boats could be planing with large trim angles, this was not frequently observed and would have been difficult for the observers to determine. At all sites, the majority of boats were planing. RM 17.6 has the largest percentage of boats that operated in the bow up mode whereas RM 11.3 has the lowest.

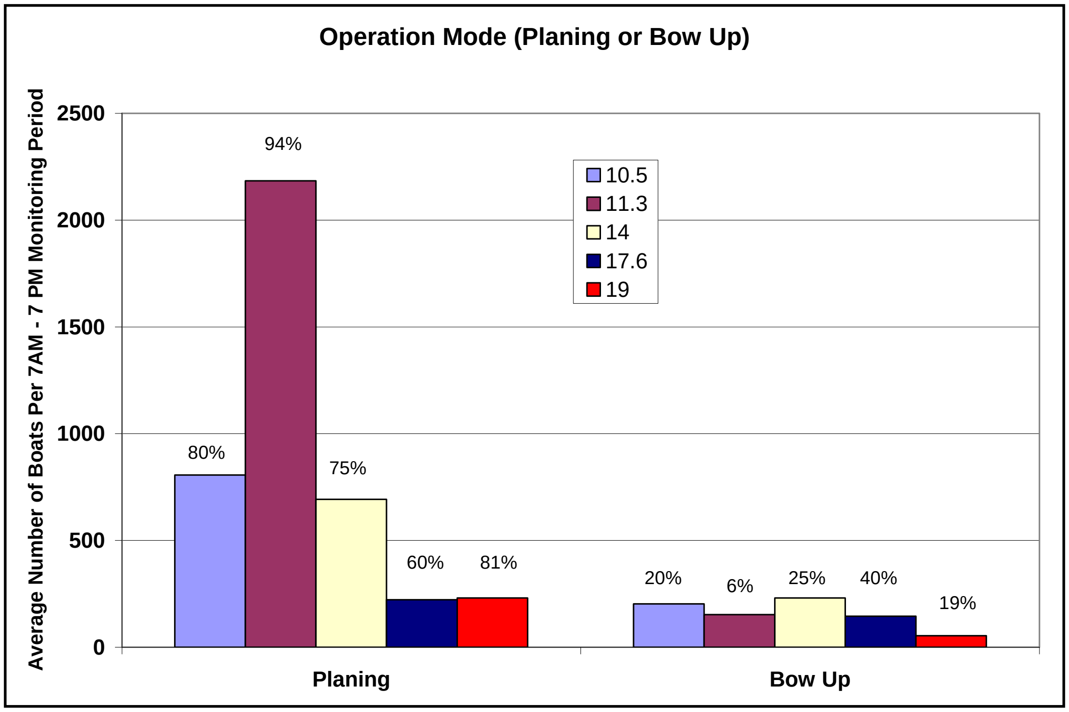

Figure 16. Boat operation mode by site based on average number of wave-making boats per 12-hr monitoring period. Bow Up refers to boats plowing through the water and operating at slower speeds that generally make the largest waves. Does not include boats not making waves such as drift boats. Planing is also referred to as "getting on step."

Figure 17 shows the lateral position of boats by site based on the average number of boats per 12-hr monitoring period. The boat counters were asked to record the lateral position of each boat based on dividing the channel into thirds of close (C), middle (M), or far (F). This type of observation was difficult at RM 19 because the observation site was close to the level of the water. At other sites, the observers were some distance above the water level and characterizing the lateral position was somewhat easier, particularly at RM 14 where the observation location was about 30 ft above the water level. The observers were told that, if a boat was on the borderline of M and F or M and C, the entry could be FM or MC. At all river miles except RM 11.3, the observers used very few designations of FM or MC. At RM 11.3, many boats were observed at FM because the majority of all boats were traveling on the far half of the channel that was toward the left descending bank at RM 11.3. To place the FM into either an F or an M classification used in Figure 17, one-half the FM were placed in F and one-half were placed in M. Lateral positions having almost no traffic, such as the right one-third of the channel at RM 11.3, reflect that this portion of the channel width is a drift area that is generally not used for boats motoring through the reach.

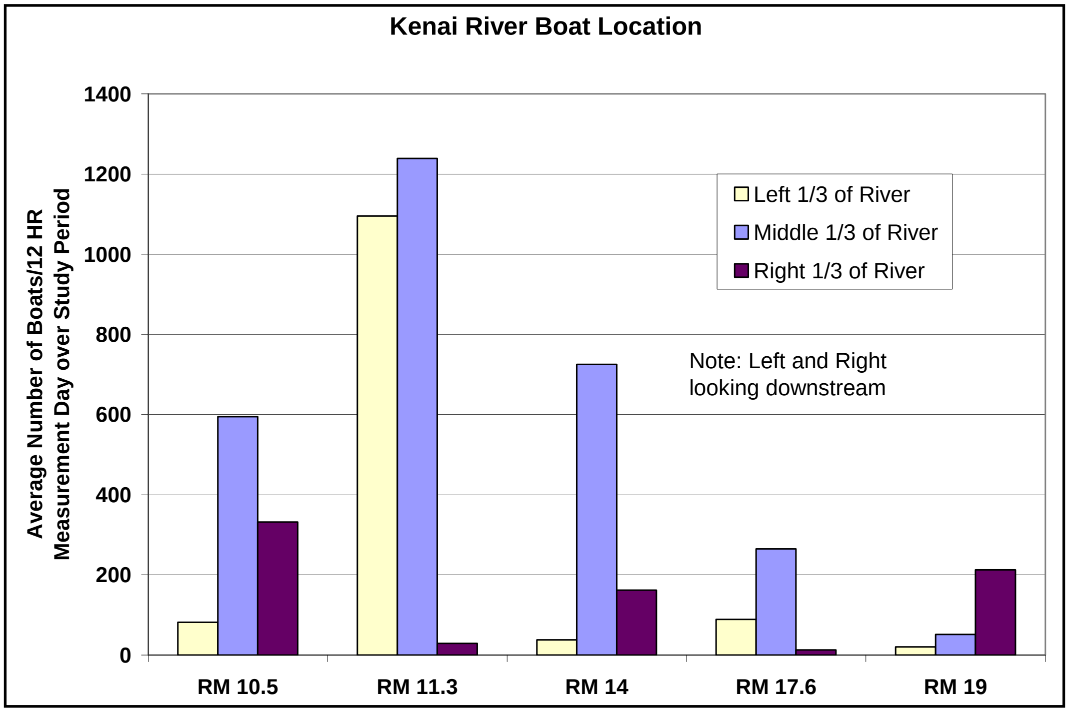

Figure 17. Boat location across the channel by site based on average number of wave making boats per 12-hr monitoring period.

Figure 18 provides the distribution of boat travel direction. The dominant direction of travel for boats underway and creating waves is upstream because much of the fishing is done by drifting downstream. Nowhere is this more evident than at RM 11.3, which is the middle of the major drift area in the study reach. Studies in Maynord (2001) showed that planing conditions resulted in the same maximum wave height for upstream and downstream boats, but the upstream boats had greater period and thus greater energy on the bank. For boats in the bow up mode of operation, both maximum wave height and wave period were higher for upstream boats.

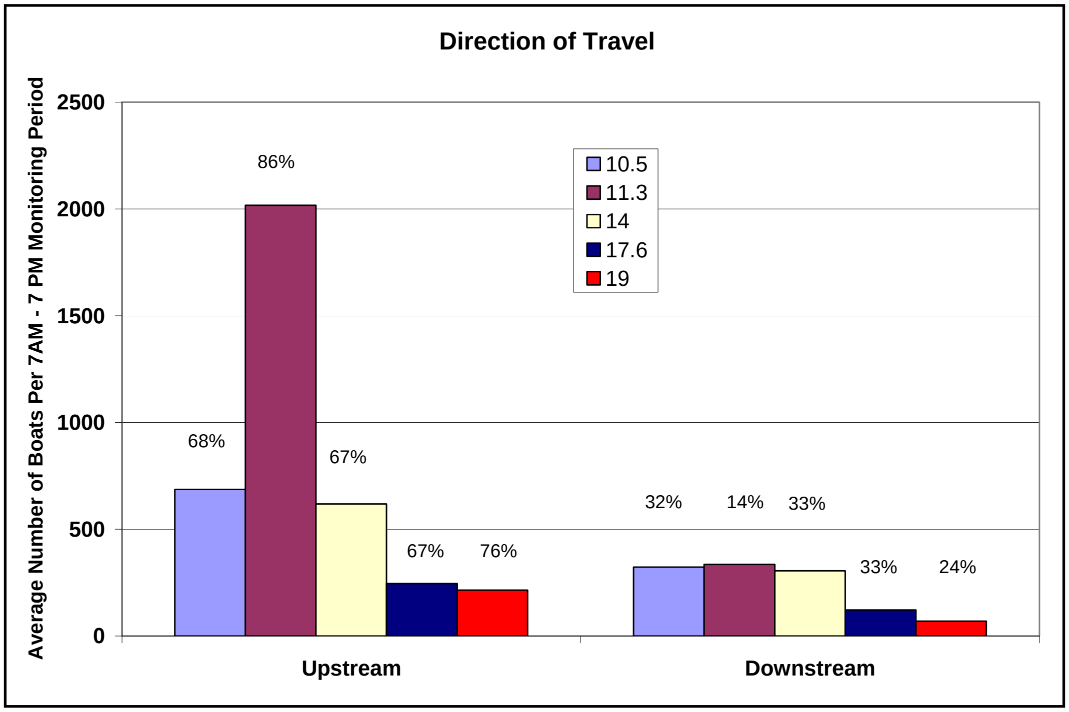

Figure 18. Direction of boat travel by site based on average number of wave-making boats per 12-hr monitoring period. The percentage shown refers to the percent of boats at that site going in the specified direction. Plot shows that most wave-making boats are going upstream.

Figure 19 shows the variation of hull type based on v-hull, flat bottom, and other. V-hull boats are 72 percent of all boats on the river, flat bottom are 27 percent, and other hull types are 1 percent. The data did not show any significant variation of hull type based on site location, and all sites and all dates were lumped together.

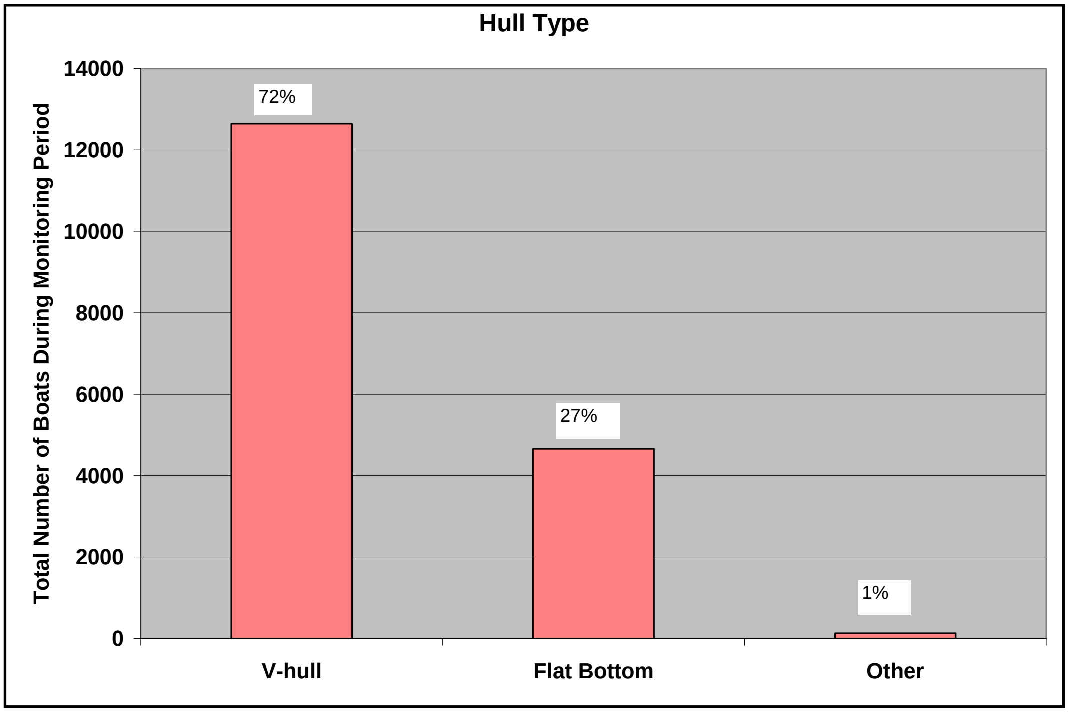

Figure 19. Variation of hull type based on all sites and all dates.

Figure 20 shows the variation of number of people in a boat. Almost one-half of the boats (43 percent) contained five people. The data did not show any significant variation in number of people based on site location, and all sites and all dates were lumped together.

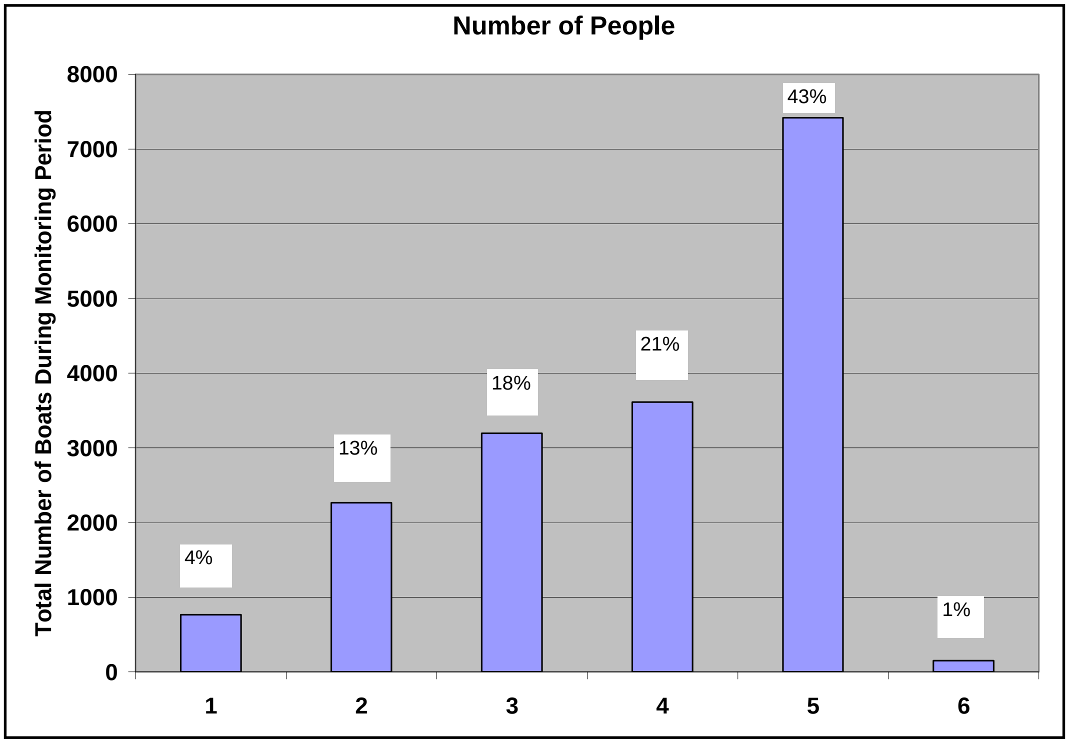

Figure 20. Variation in number of people in boat based on all sites and all dates.

Traffic variation during the day is shown in Figure 21. All sites except RM 11.3 show a relatively consistent amount of traffic with small peaks at lunch and at the end of the day. The drift area at RM 11.3 shows peak traffic in the early morning and decreasing traffic throughout the day. One explanation is that the tidally influenced reaches have less successful fishing during high tides. During the monitoring period from 19 to 22 July, the daytime high tide was generally after about 1500 hr.

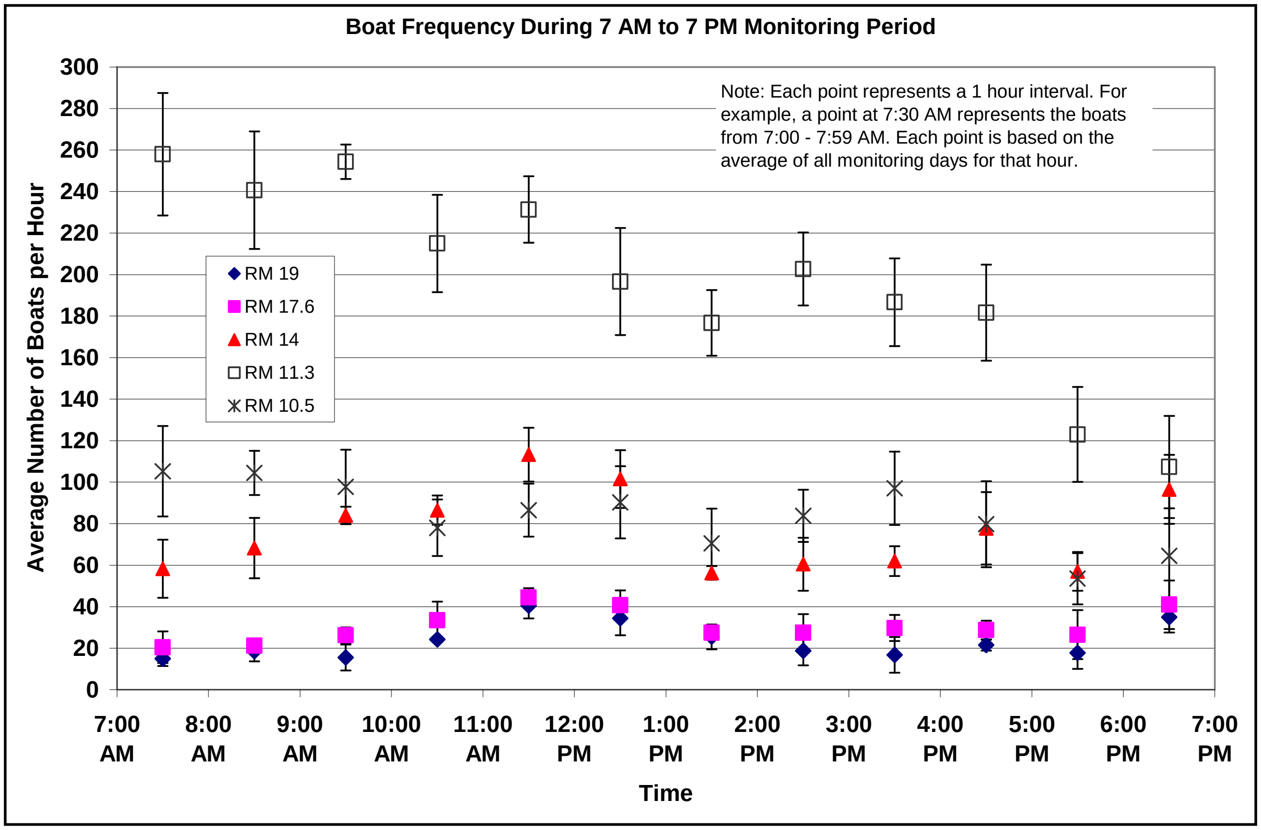

Figure 21. Wave-making boat passage frequency variation during 7AM to 7PM monitoring period. Each point is the average of that hour from the 4 days of boat counts except for RM 11.3, which was counted on only 3 days. Error bars are ± one standard deviation.

Figure 22 shows the variation of average boat traffic along the 11-mile reach. The plot is based on data from the five boat count locations as well trends observed by persons knowledgeable of boat traffic on the river. The trend points are necessary because it was not possible to have enough boat count sites to capture all variations along the river. The trend points are based on turn around points in the major drift area as well as a boat launch point at the upstream end of the study reach. The plot is most representative of the 19-22 July 2005 monitoring period. Only the major drift area at RM 11.3 is included because it was not possible to staff enough count teams to describe all variations in traffic along the reach. Figure 22 will be used to assess boat wave energy variation along the river.

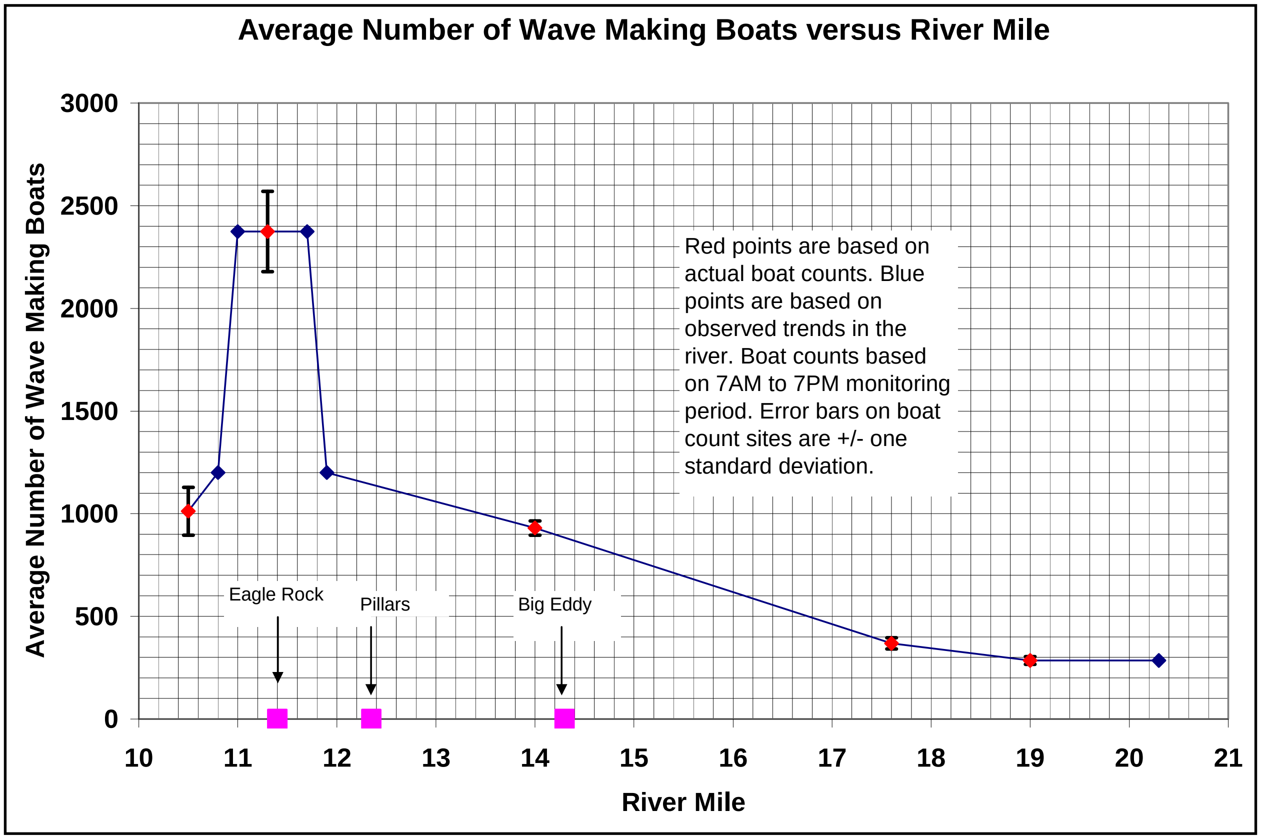

Figure 22. Variation of average number of wave-making boats along study reach per 12-hr monitoring period. Red boat count points based on average of total boats on each of 4 days during field study except for RM 11.3, which was based on 3 days. The blue points based on observed trends are based on turn around points at a drift area or locations of boat launch points.

Wave measurements

Fundamental characteristics of boat waves are presented in the U.S. Army Corps of Engineers (USACE) Coastal Engineering Manual, Chapter 7; PIANC (2003); and the earlier Kenai River study in Maynord (2001).

In the previous Kenai Study (Maynord 2001), individual boat wave tests were conducted on Johnson Lake and on the Kenai River at RM 32. These tests were completely free of the effects of other boats and relatively free of the effects of wave reflections and shallow shoreline depths. The previous tests were intended to define the wave characteristics generated by a single boat. In the wave measurements conducted herein, the objective was to define the wave characteristics attacking the shoreline where shallow depths are present, wave reflections are everywhere, and waves are present from multiple boats. Stated otherwise, the objective of these tests was to measure the shoreline wave environment under realistic or normal conditions.

Correlating shoreline erosion to recreational boat wave characteristics has received only limited attention in the past. Erosion from wind waves has been related to wave power by Kamphuis (1987). Maximum wave height was used by Krusenstierna and Hanson (1989) to characterize boat wave attack of the shoreline. Both maximum wave height and boat wave energy are used herein to quantify the attack of shoreline by boat waves.

Based on USACE (1984), the total energy of a wave system is the sum of its kinetic and potential energy. If all waves are propagated in the same direction, the potential and kinetic energies are equal and the total wave energy in one wavelength per unit crest width is

$$E = \frac{\rho g H^2 L}{8} \quad (1)$$

Where ρ is water density, g is gravitational constant, H is wave height, and L is wave length. Wave energy per unit surface area, termed the specific energy or energy density, is given by

$$\overline{E} = \frac{E}{L} = \frac{\rho g H^2}{8} \quad (2)$$

Wave energy flux, termed wave power, is the rate at which energy is transmitted in the direction of wave propagation across a vertical plane perpendicular to the direction of wave advance and extending down the entire depth. Wave power per unit crest width is equal to

$$P = \overline{E}\, C_g \quad (3)$$

Group velocity C_g is used because it is with this velocity that wave energy is propagated. For waves propagating in deep or transitional water, the group velocity will be less than the phase velocity C that is defined as

$$C = \frac{gT}{2\pi} \quad (4)$$

Where T is the period of the wave. C_g is determined from Appendix C of USACE (1984) based on d/L_o and C. Where d is depth and L_o is deep water wave length. Wave energy expenditure on the bank per unit length of bank over time period Δt is

$$E_w = P\Delta t \tag{5}$$

For wave energy expenditure on the bank per unit length of bank for each wave, Δt = wave period T. Total wave energy expenditure on the bank per unit length of bank during the selected 30-min time interval is the sum of E_w for all waves. Waves are assumed to strike normal to the bank and effects due to reflection are ignored. While these assumptions are not completely met in the wave environment on the Kenai, the intent herein is to provide a relatively simple means of comparing boat wave energy at the shoreline for the different sites on the river. Computed total energies are affected by wave reflection from the bank and are used here to define the trends of the boat wave attack of the shoreline.

A boat traveling off the channel centerline will produce different effects on the two banks based on the different distances from the shoreline. The bank closer to the boat is attacked by fewer waves but the maximum wave height is large. The bank far from the boat is attacked by more waves but the maximum wave height is reduced. Figure 2 from Maynord (2001) of boat wave measurements on Johnson Lake shows the change in wave characteristics with distance from the boat. At 30 ft from the boat, only three waves exceed 0.25 ft and have magnitude (trough to following crest) of 0.40, 0.77, and 0.45 ft. At 95 ft from the boat, six waves exceed 0.25 ft and have magnitude of 0.27, 0.36, 0.46, 0.46, 0.40, 0.28 ft. This difference is important because the larger waves are more likely to move larger bank material. In addition to the total boat wave energy at the bank from all waves, a limiting or threshold wave condition was used to determine boat wave energy at the bank most likely to affect the shoreline. The limiting or threshold wave height depends on bank properties and is generally unknown. Based on Krusenstierna and Hanson (1989), measurable effects of shoreline erosion of medium sands started at maximum wave heights of about 0.13 ft to 0.26 ft. Kenai River shorelines are generally composed of materials considerably larger than medium sand and threshold wave heights should be toward the upper end of the range found by Krusenstierna and Hanson. A threshold wave height of 0.25 ft is used here. Although wind waves are not expected to be a significant erosion factor on this reach of the Kenai River, the 0.25 ft threshold likely removes any wind waves from inclusion in the analysis. This value of 0.25 ft wave height is not proposed as an absolute for effect/no effect but simply as a guide to separating waves that may have an effect on shoreline from those that likely do not have an effect on the shoreline.

The wave measurements were conducted as described in Table 2. Two of the five sites were subject to up to 8.5 ft of tidal variation, which required that the gages be raised and lowered during the tidal cycle. The wave team developed a gage mounting stand that could be easily raised and lowered for gage installation, gage cleaning (because of aquatic vegetation), and to deal with the tidal variation. The stand is shown in Figure 23. Capacitance wire and capacitance rod type gages were used to measure waves for 4 days at five different sites.

Figure 23. Wave stand supporting four wave rods used in wave measurements at RM 10.5. This site is on the left bank (looking downstream) about 0.5 mile upstream of Beaver Creek. Green rope used to raise and lower wave stand for installation, repair, and cleaning of gages as well as raising and lowering to deal with varying water level due to tides.

Wave data were collected almost continuously for 12 hr, which resulted in a huge amount of data. At each site, the 12-hr record was broken down into 30-min intervals to make the data analysis manageable. The 30-min records were selected based on continuous 30-min time intervals when the record was free of gage cleaning and other disruptions. Each 30-min time interval of data was filtered as described in Maynord (2000) to eliminate long period oscillations and any short period spikes in the data not caused by boat waves. Wave height and period was determined for each wave during the 30-min time segment. Selected wave records are presented to demonstrate conditions along the length of the river. Figure 24 shows the measured water level during the 30-min window at RM 19 on the left bank on 19 July between 1320 and 1350. Also shown at the top of the plot is the time of passage of 13 boats during the 30-min window.

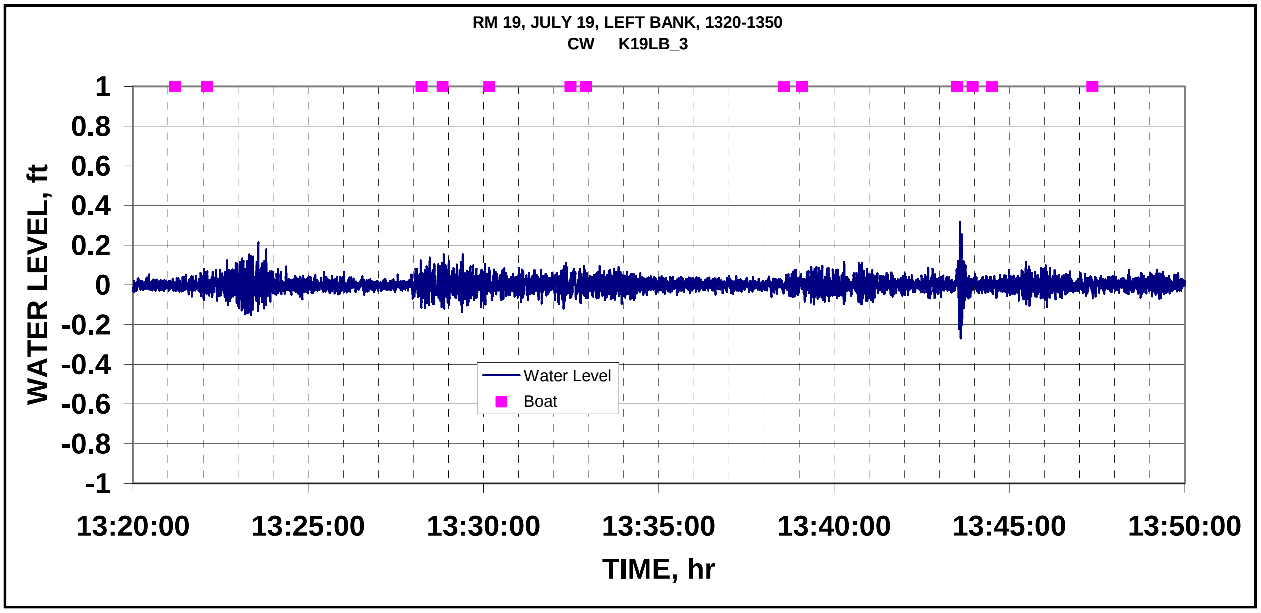

Figure 24. Water level and boat passage at RM 19, left bank, 19 July, 1320-1350.

Figure 25 shows the same time window on the right bank at RM 19. Table 6 shows the boat characteristics recorded from the left bank during the 1320-1350 time window at RM 19.

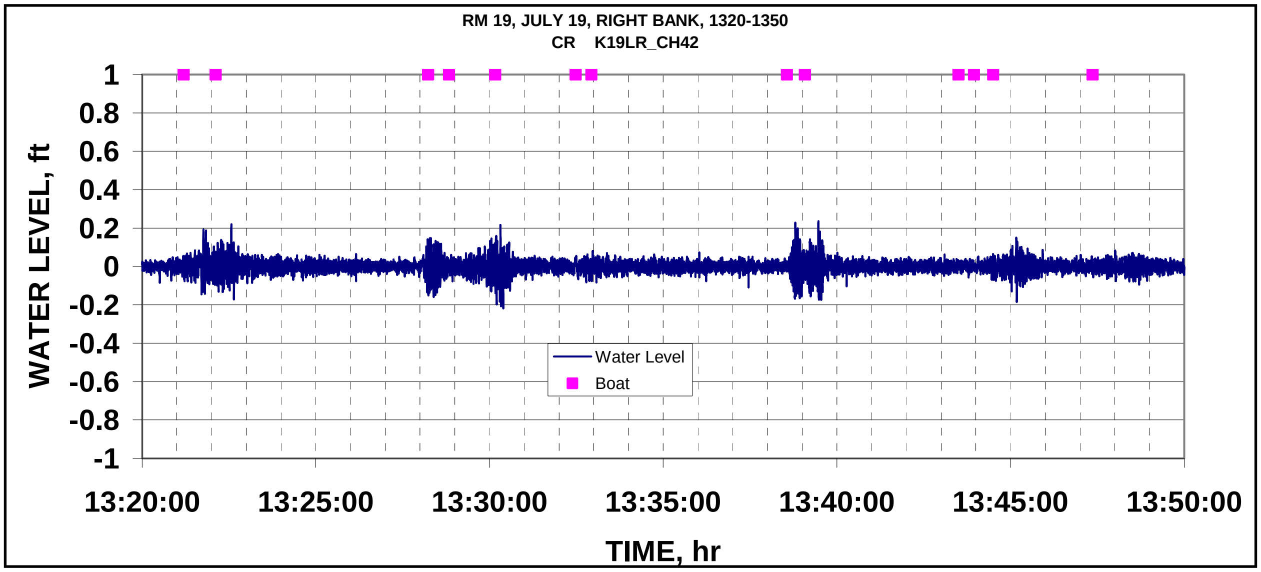

Figure 25. Water level and boat passage at RM 19, right bank, 19 July, 1320-1350.

Table 6. Boat characteristics at RM 19, 19 July, 1320-1350, observed from left bank.

River Mile	Date	Time	Operation	Boat Position	Direction	Number in Boat	Boat Type
19	19	13:21	P	F	D	5	V
19	19	13:22	P	M	U	3	V
19	19	13:28	B	F	D	6	F
19	19	13:28	P	C	U	5	V
19	19	13:30	B	C	D	5	V
19	19	13:32	P	F	D	3	F
19	19	13:32	P	F	U	5	F
19	19	13:38	P	C	U	4	V
19	19	13:39	P	F	U	3	V
19	19	13:43	B	C	U	1	F
19	19	13:43	P	F	U	3	V
19	19	13:44	P	F	U	3	V
19	19	13:47	P	F	U	5	V

P = planing operation, B = Bow-up operation, F = Far 1/3 of channel, M = Middle 1/3 of channel, C = close 1/3 of channel, D = downbound, U = upbound, V = V-hull boat, F = flat-bottomed boat.

Figure 26 shows measured water level and boat passage time for 33 boats at the left bank at RM 14 on 22 July between 1630 and 1700. Figure 27 shows measured water level and boat passage time for 95 boats at the right bank at RM 11.3 on 21 July between 1153 and 1223. Figure 28 shows measured water level and boat passage time for 32 boats at the left bank at RM 10.5 on 20 July between 1330 and 1400.

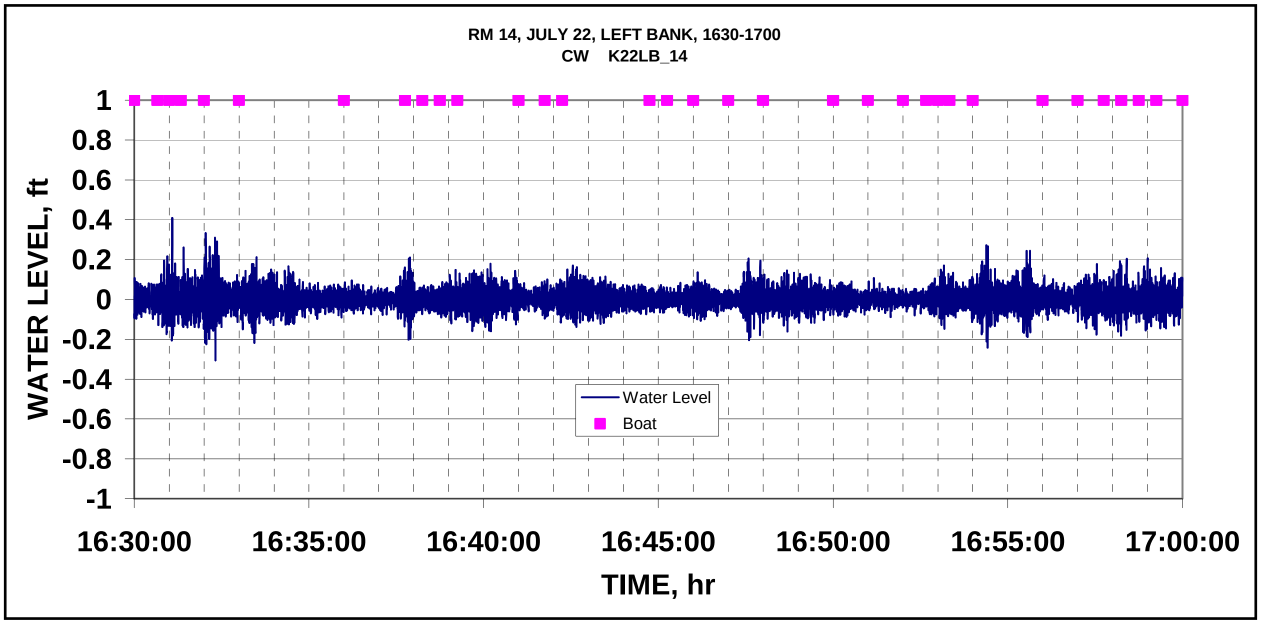

Figure 26. Water level and boat passage at RM 14, left bank, 22 July, 1630-1700.

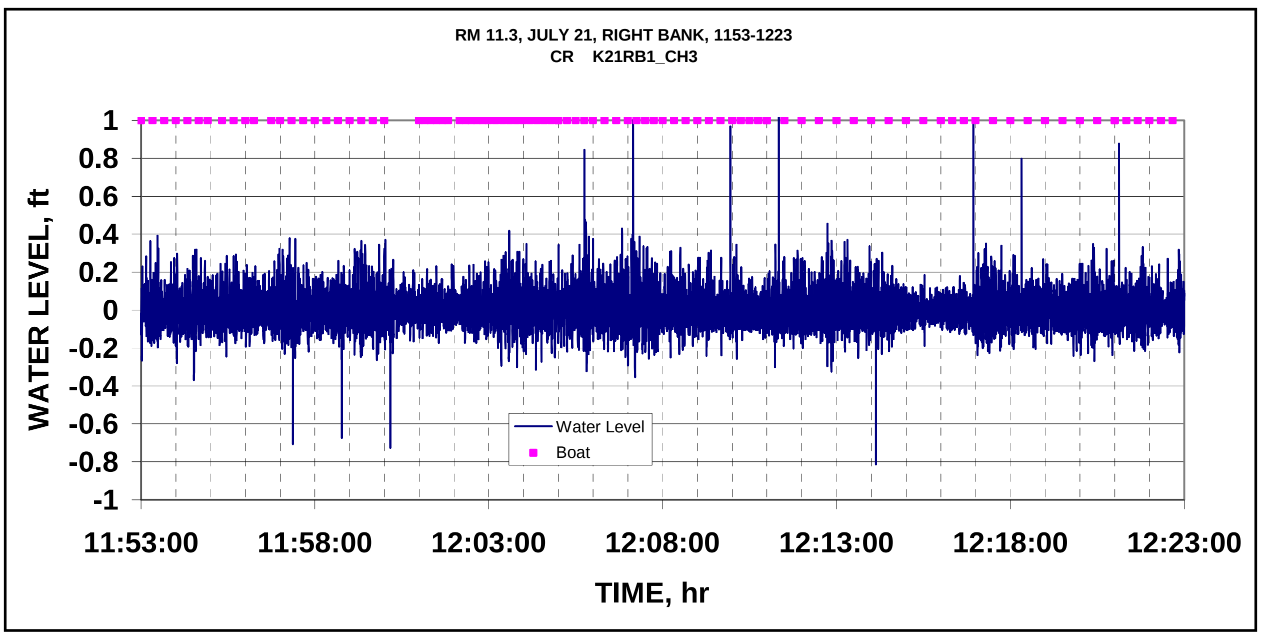

Figure 27. Water level and boat passage at RM 11.3, right bank, 21 July, 1153-1223.

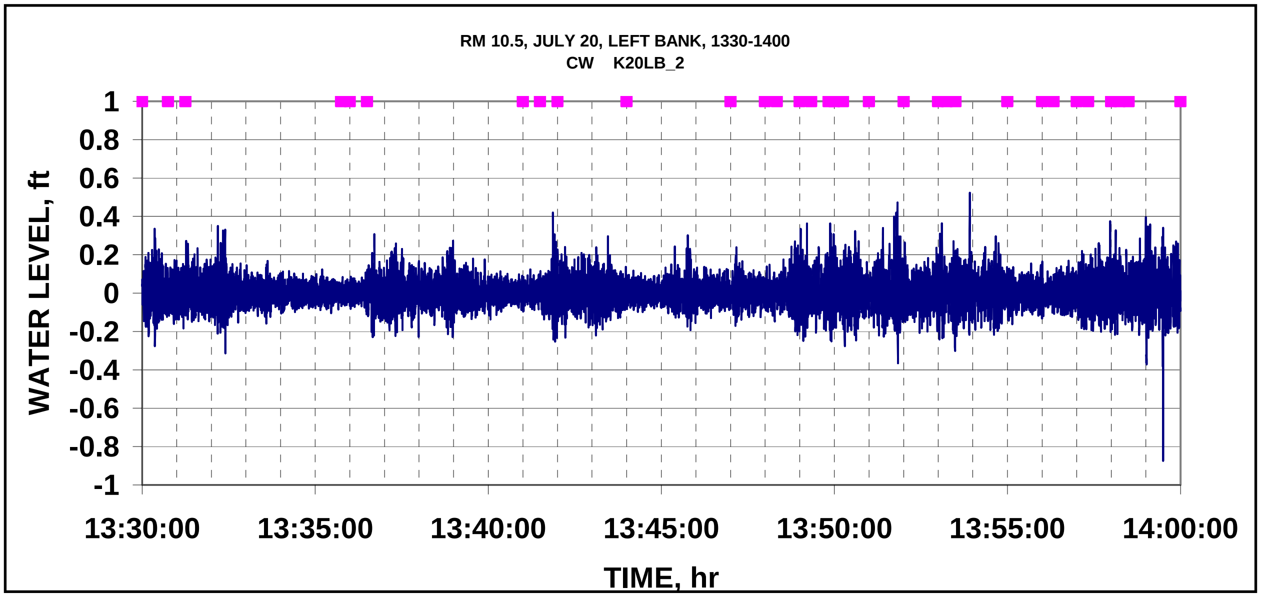

Figure 28. Water level and boat passage at RM 10.5, left bank, 20 July, 1330-1400.

Table 7 provides the following information from the boat wave tests:

- Column 1: Date in July 2005 test was conducted.
- Column 2: River mile/river bank where data were collected. LB = left bank looking downstream, RB = right bank.
- Column 3: Maximum wave height observed during the 30-min window.
- Column 4: $H_{1/100}$ that is the average of the highest 1 percent of the waves. A wave was considered part of the sample to compute $H_{1/100}$ if it exceeded 0.05 ft. Using maximum wave height during the 30-min period can often be skewed by a single boat passing extremely close to the wave gage. The $H_{1/100}$ tends to smooth out extremes of a single boat yet still represent the largest waves. Although use of $H_{1/100}$ is not applicable to waves from a single boat, the wave environment present from many boats likely produces a distribution from which $H_{1/100}$ is a valid descriptor.
- Column 5: Total boat wave energy/unit length of shoreline = sum of energy/unit length of shoreline from all waves during 30-min window using Equations 1-4.
- Column 6: Number of wave-making boats passing during 30-min window.
- Column 7: Total energy per boat = column 5 divided by column 6.
- Column 8: Boat wave energy was also summed for all waves having wave height greater than 0.25 ft.

- Column 9: Boat wave energy for wave height greater than 0.25 ft per boat equal to column 8 divided by column 6.
- Column 10: File name for future reference.
- Column 11: Time range for 30-min window.
- Column 12: Type of capacitance gage with which data were taken.

Boat wave energy expenditure on the shoreline is summarized as follows:

1. **RM 19.** Average of left and right bank 30-min windows at RM 19 is total boat wave energy of 647 (from (689+605)/2) ft-lb/ft of shoreline. From the data for left and right bank, each boat averages about 52 ft-lb of boat wave energy expenditure per foot of shoreline in a river width of 453 ft. Based on the boat path Figures 1-4 and the boat position data in Figure 17, boats at RM 19 average traveling 75 percent of the channel width from the left bank wave gages. This means that, if all boat path locations at this cross section were averaged, the average location would be 75 percent of the channel width from the left bank. At the right bank wave gages that are downstream, boats average traveling 65 percent of the channel width from the left bank. For boat waves having height > 0.25 ft, each boat averages 14.6 ft-lb/ft of shoreline on the left bank and 20.3 ft-lb/ft of shoreline on the right bank. The difference is due to the sailing line being closer to the right bank. Note that the difference between total energy from all waves of 52 ft-lb per foot of shoreline versus 14.6 ft-lb per foot of shoreline from waves greater than 0.25 ft shows that a significant amount of energy striking the bank is from small waves that do not exceed 0.25 ft.
2. **RM 17.6.** Boat count site only. No wave data were collected at the site because other sites were believed to be more important to understanding the boat wave climate along the river. Techniques will be developed subsequently to estimate boat wave energy at and near this site based on the number of boats passing.
3. **RM 14.** Average of all 30-min windows was a total boat wave energy of 1,653 ft-lb/ft of shoreline. Each boat averages about 40 ft-lb of boat wave energy expenditure per foot of shoreline on the left bank in a river width of 432 ft. Based on the boat path Figures 1-4 and the boat position data in Figure 17, boats at RM 14 average traveling about 56 percent of the channel width from the left bank. For waves having height > 0.25 ft, each boat averages 19.4 ft-lb/ft of shoreline on the left bank.
4. **RM 11.3.** Average of all 30-min windows was a total boat wave energy of 4119 ft-lb/ft of shoreline. Each boat averages about 44 ft-lb of boat wave energy expenditure per foot of shoreline on the left bank in a river width of

525 ft. Note that the test at 1805-1835 hr on 21 July was the only 30-min test record at RM 11.3 at high tide. This value of shoreline boat wave energy was about 60 percent greater than the overall average. This finding is likely due to the change in bank configuration and thus wave reflection at the higher stage. Based on the boat path Figures 1-4 and the boat position data in Figure 17, boats at RM 11.3 average traveling about 35 percent of the channel width from the left bank. For waves having height > 0.25 ft, each boat averages 33.3 ft-lb/ft of shoreline on the right bank.

5. **RM 10.5.** Average of all 30-min windows was a total boat wave energy of 3,810 ft-lb/ft of shoreline. Each boat averages about 112 ft-lb of boat wave energy expenditure per foot of shoreline on the left bank in a river width of 441 ft. Based on the boat path Figures 1-4 and the boat position data in Figure 17, boats at RM 10.5 average traveling about 58 percent of the channel width from the left bank. For waves having height > 0.25 ft, each boat averages 87.5 ft-lb/ft of shoreline on the left bank.
6. Considerable effort was expended to determine why the RM 10.5 site had much higher boat wave energy expenditure/boat than the other sites. The banks at RM 10.5 are smooth, relatively steep, and appear to be highly reflective of boat wave energy, particularly at high tide. As at RM 11.3, the data showed a clear increase in boat wave energy at high tide. All tests before 1500 hr on 20 and 21 July were at low tide. The 10 high tide tests averaged 129 ft-lb/boat, which is about 50 percent greater than the 6 low tide tests that averaged 85 ft-lb/boat. This finding is similar to RM 11.3. As at RM 11.3, this increase is likely due to the changed wave reflection on the bank as the bank configuration and distance of bank from the gages changes with stage. The low tide values averaging 85 ft-lb per boat are still greater than the 40-52 ft-lb per boat calculated at the three upstream sites. RM 10.5 has one characteristic not found at the other sites. Just upstream of the RM 10.5 site is the downstream end of the largest drift fishing reach on the river. At the completion of the drift, most boats rapidly motor upstream to repeat the drift. This results in a large number of boats getting on step (or planing) going upstream just above RM 10.5. Based on the Maynord (2000) study, upbound boats getting on step produce the largest waves of any boat and also have somewhat greater periods as well. This finding is important because both height and wave period determine boat wave energy expended on the shoreline. While it is certainly possible that this site has some specific characteristic that causes much greater computed boat wave energy because of bank configuration, the most likely reason for the greater boat wave energy at this site is boat waves reaching

the gages from the upstream drift area from boats that did not pass through the counting site.

Table 7. Time periods analyzed for boat wave energy and height.

Date	River Mile/ Bank	Hmax, ft	H1/100, ft	Energy, ft-lbs/ ft of bank/ 30 Min	Boats/ 30 Min	Energy/ boat	E for H >0.25 ft/ 30 Min	E/boat, H>0.25 ft	file	Time range	gage type
19	19/LB	0.70	0.45	623	12	51.9	249.7	20.8	K19LB_1	1000-1030	Wire
19	19/LB	0.58	0.39	708	20	35.4	182.2	9.1	K19LB_2	1220-1250	Wire
19	19/LB	0.54	0.39	681	13	52.4	185.9	14.3	K19LB_3	1320-1350	Wire
19	19/LB	0.61	0.40	744	14	53.1	201.3	14.4	K19LB_5	1645-1715	Wire
	average	0.61	0.41	689	15	48.2	204.8	14.6			
19	19/RB	0.42	0.37	612	13	47.1	225.4	17.3	K19LR_CH41	1320-1350	Rod
19	19/RB	0.40	0.35	576	13	44.3	197.8	15.2	K19LR_CH42	1320-1350	Rod
19	19/RB	0.41	0.34	559	10	55.9	220	22.0	K19LR3_CH42	1545-1615	Rod
19	19/RB	0.55	0.42	675	9	75.0	240.3	26.7	K19LR5_CH42	1715-1745	Rod
	average	0.45	0.37	605	11	55.6	220.9	20.3			
22	14/LB	0.60	0.51	2607	62	42.0	1719	27.7	K22LB_10	1200-1230	Wire
22	14/LB	0.67	0.58	1312	31	42.3	598.9	19.3	K22LB_11	1340-1410	Wire
22	14/LB	0.57	0.41	1284	40	32.1	407	10.2	K22LB_12	1500-1530	Wire
22	14/LB	0.62	0.45	1410	33	42.7	498	15.1	K22LB_14	1630-1700	Wire
	average	0.62	0.49	1653	42	39.8	805.7	19.4			
21	11.3/RB	1.31	0.92	4359	95	45.9	3589	37.8	K21RB1_CH3	1153-1223	Rod
21	11.3/RB	0.81	0.62	3303	82	40.3	2388	29.1	K21RB2_CH4	1302-1332	Rod
21	11.3/RB	0.95	0.65	4052	88	46.0	3157	35.9	K21RB3_CH4	1430-1500	Rod
21	11.3/RB	0.97	0.74	4302	63	68.3	3340	53.0	K21RB5_CH2	1805-1835	Rod
22	11.3/RB	0.86	0.71	4865	128	38.0	4080	31.9	K22RB1_CH4	910-940	Rod
22	11.3/RB	0.77	0.67	4529	129	35.1	3700	28.7	K22rb2_ch4	1107-1137	Rod
22	11.3/RB	0.88	0.67	3773	92	41.0	2888	31.4	K22RB3_CH4	1245-1315	Rod
22	11.3/RB	0.77	0.68	4335	104	41.7	3535	34.0	K22RB4_CH4	1415-1445	Rod
22	11.3/RB	1.01	0.63	3558	99	35.9	2613	26.4	K22RB5_CH4	1545-1615	Rod
	average	0.93	0.70	4119	98	43.6	3254.4	33.3			
20	10.5/LB	0.79	0.61	3297	44	74.9	2559.0	58.2	K20LB_1	1140-1210	Wire
20	10.5/LB	0.75	0.61	2634	32	82.3	2038.0	63.7	K20LB2_CH2	1330-1400	Rod
20	10.5/LB	0.80	0.66	3057	32	95.5	2396.0	74.9	K20LB2_CH1	1330-1400	Rod
20	10.5/LB	1.22	0.63	3093	32	96.7	2377.0	74.3	K20LB_2	1330-1400	Wire
20	10.5/LB	0.80	0.65	5367	39	137.6	4596.0	117.8	K20LB3_CH2	1548-1618	Rod
20	10.5/LB	0.88	0.71	6455	39	165.5	5712.0	146.5	K20LB_3	1548-1618	Wire
20	10.5/LB	0.89	0.71	5474	39	140.4	4739.0	121.5	K20LB3_CH1	1548-1618	Rod
20	10.5/LB	0.87	0.69	2791	21	132.9	1802.0	85.8	K20LB_4	1655-1725	Wire
20	10.5/LB	0.72	0.57	2152	21	102.5	1245.0	59.3	K20LB4_CH1	1655-1725	Rod
20	10.5/LB	0.79	0.55	2119	21	100.9	1277.0	60.8	K20LB4_CH2	1655-1725	Rod
20	10.5/LB	0.83	0.71	5264	35	150.4	4589.0	131.1	K20LB_5	1800-1830	Wire
20	10.5/LB	0.89	0.74	4722	35	134.9	3975.0	113.6	K20LB5_CH1	1755-1825	Rod
20	10.5/LB	0.89	0.68	4300	35	122.8	3517.0	100.5	K20LB5_CH2	1755-1825	Rod
21	10.5/LB	0.59	0.52	1784	21	85.0	1129.0	53.8	K21LB_6	1317-1347	Wire
21	10.5/LB	0.65	0.53	2516	33	76.2	1727.0	52.3	K21LB_7	1445-1515	Wire
21	10.5/LB	0.92	0.74	5937	60	99.0	5184.0	86.4	K21LB_8	1530-1600	Wire
	average	0.83	0.64	3810	34	112.3	3053.9	87.5			

Analysis of data

The computed boat wave energy expenditure per foot of shoreline shows consistent trends except for the site at RM 10.5 where the upstream drift area has a potential impact on the measured data by introducing boat wave energy from boats that do not pass the study site. Major conclusions of the data are as follows:

1. Each boat creates a calculated expenditure of boat wave energy on the shoreline of 40-56 ft-lb/ft of shoreline. A significant part of this energy is affected by wave reflection and the magnitude is used only as an indicator. Variation of boat wave energy along the length of the study reach from all boats will strongly correlate with the number of wave-making boats that pass a site on the river.
2. Boat wave energy expenditure from waves having height > 0.25 ft does not follow the trend exhibited by the total boat wave energy. The calculated energy/boat for large waves increases as traffic increases after a certain level of traffic is reached. This answers one of the primary issues of this study being “Are high rates of boat traffic worse than the same number of boats spread out over a greater time period”? For waves that are likely most detrimental to bank stability, the data collected herein show the answer is yes once a threshold of traffic is exceeded. The reason this is true is complex and not completely understood. First, waves from all these boats can combine or negate each other leading to an altered wave environment. Second, and maybe most importantly, as traffic increases and waves increase, boats must often slow down for passenger safety and comfort. As shown in Maynord (2000), when planing boats slow down, wave heights increase and, in some cases, wave periods also increase. Third, the large number of boats present may result in more boats near the shoreline causing a greater number of large waves.
3. RM 10.5 is different from other sites because it has a higher boat wave energy per boat for both total boat energy and boat wave energy for wave height > 0.25 ft. This higher energy appears to result from (a) boats getting on step at the upstream drift area and (b) the influence of tides.

Boat energy equation

A method is needed to convert the boat wave energy calculated from the wave measurement sites to other locations along the river. This boat wave energy equation will use the data for wave height > 0.25 ft to address the waves most likely to affect the shoreline. With large amounts of boat traffic at some sites and effects from waves greater than 0.25 ft increasing with increasing traffic, any approach is going to be approximate because the problem is too complex to develop a physics-based approach.

The boat wave equation in Maynord (2005) was used to calculate how wave height calculated for one bank can be converted to wave height on the other bank, knowing the average sailing line of the boats. The equation from Maynord (2005) for wave height from planing and semi-planing boats is

$$\frac{H}{\nabla^{1/3}} = C F_{\nabla}^{-0.58} \left(\frac{x}{\nabla^{1/3}} \right)^{-0.42} \tag{6}$$

where ∇ is the volume displacement of the boat = total boat weight/unit weight of water, C is a coefficient = 1.0 for v-hull and 0.82 for flat bottom, F is the displacement Froude number = $V/(g\nabla^{1/3})^{0.5}$, V is boat speed, and x is distance from boat. From this equation, the variation of H with distance is

$$H = C_2 \left(x \right)^{-0.42} \tag{7}$$

where C_2 represents the other parameters in the equation. Based on equation 2, the variation of boat wave energy with distance is

$$E = C_3 \left(x \right)^{-0.84} \tag{8}$$

The conversion from wave height to boat wave energy expended on the shoreline used herein is based on wave height alone with no variation due to wave period. This assumption is generally justified because the largest waves in the wave train tend to have the same period regardless of distance. Only those smaller waves preceding the maximum wave tend to have periods greater than the peak wave. These waves are generally small enough to not have wave height that exceeds 0.25 ft. Equation 8 will be the basis for a boat wave energy equation with which to use the observed data in Table 7 to estimate boat wave energy variation along the study reach. The left side of the equation is set equal to the boat wave energy per boat for wave height > 0.25 ft from Table 7. The average distance from boat to shoreline is known from Figures 1-4. The needed relationship is how C_3 varies with number of boats/30 min. Figure 29 shows (boat wave energy/boat) / $(x)^{-0.84}$ plotted against the number of boats during the 30-min period from Table 7 data (except RM 10.5).

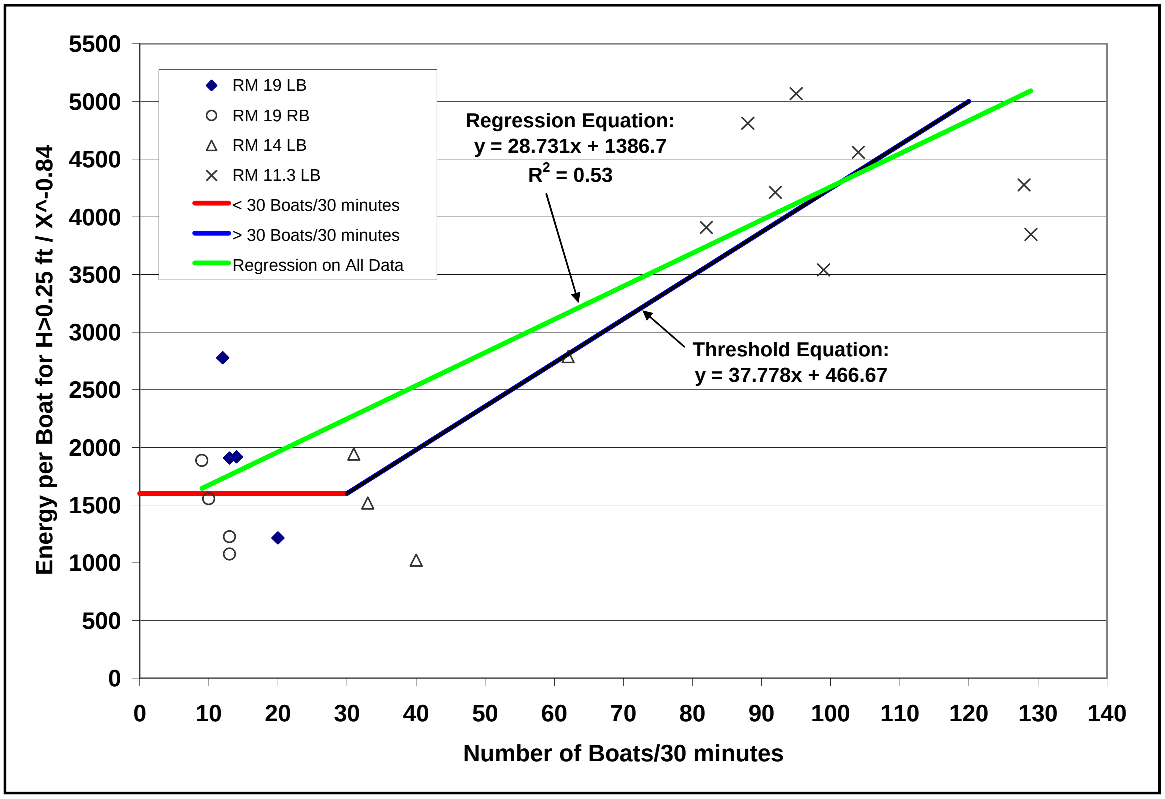

Figure 29. Boat wave energy per boat/ (distance from shoreline to average sailing line)^-0.84 versus number of boats in 30 min.

The equations shown on Figure 29 have a weak statistical basis and are more of a concept model showing how shoreline energy from a boat varies with boating intensity. The concept model based on the collected Kenai River data is that at low levels of boat passage frequency, the energy on the shoreline from waves greater than 0.25 ft from a single boat does not depend on boat passage frequency. As boat passage frequency increases, a threshold in boat passage frequency is reached and the energy reaching the shoreline from waves greater than 0.25 ft from a single boat increases with increasing boat passage frequency. The boat passage frequency threshold for increased shoreline energy is difficult to define with the data and would likely vary from river to river because of channel width variation. The data are scattered but show a constant value of (boat wave energy/boat) / $(x)^{-0.84}$ for boat traffic less than about 30 boats per 30 min, indicating no additive effects of boat traffic. Above about 30 boats per 30 min, the boat wave energy for $H > 0.25$ ft per boat increases with increasing traffic. The boat wave energy equation for boat traffic less than or equal to 30 boats per 30 min is

$$E_{H>0.25\text{ ft}} / \text{boat} = 1600\left(x\right)^{-0.84} \tag{9}$$

where E is in ft-lb/ft of shoreline and x is in feet. For traffic greater than 30 boats per 30 min, the equation is

$$E_{H>0.25\text{ ft}} / \text{boat} = \left(37.8 * \text{boats per 30 min} + 467\right)\left(x\right)^{-0.84} \tag{10}$$

It cannot be emphasized enough that these equations are intended to identify boat wave energy trends along the Kenai River only and not to predict absolute values of boat wave energy on the bank. While a threshold of 30 boats/30 min is used here, the true threshold would be best determined by varying the number of boat passages over a wide range at one location on the river and repeating this process at several other locations on the river. The uncertainty is large because the concept model uses only distance from shoreline to average boat sailing line and boat passage frequency whereas the data are from varying sailing lines, mode of boat operation, travel direction, number of people in boat, and hull type. Application of Equations 9 or 10 requires only the distance of the typical sailing line from the shoreline and the number of boats during a representative 30-min period. For example, consider RM 17.6 where an average of 368 boats passed each day during the 12-hr monitoring period. This represents an average of 368/12/2 = 15.3 boats per 30-min period. On average, boats at RM 17.6 travel 195 ft from the left bank and 273 ft from the right bank. Using Equation 9 (since boat passage is less than 30 boats per 30 min), boat wave energy for H > 0.25 ft/boat on the left bank is 19.1 ft-lb/ft of shoreline. The boat wave energy over the 30-min period on the left bank is 15.3*19.1 = 292 ft-lb/ft of shoreline. Boat wave energy/boat on the right bank is 14.4 ft-lb/ft of shoreline. The boat wave energy over the 30-min period on the right bank is 15.3 * 14.4 = 220 ft-lb/ft of shoreline.

An alternative to the 30 boat/30 min concept that is statistically more acceptable is to fit a single equation to the data and not use a threshold. The equation based on a best fit of the data also shown in Figure 29 is

$$E_{H>0.25\text{ft}} / \text{boat} = \left(28.731 * \text{boats per 30min} + 1386.7\right)\left(x\right)^{-0.84} \tag{11}$$

The threshold Equations 9 and 10 are used here, but Figure 29 shows that the differences between the two approaches is not large.

Using Figures 1-4, the study reach was divided into 69 reaches that were 0.1 or 0.2 mile long based on similar location of the average boat path. For each reach, the distance from the average boat path to the left and right shorelines was determined from Figures 1-4. From Figure 22, the average daily traffic was determined for each reach and that value was converted to an average number of boats per 30 min. Equation 9 or 10 was used to compute the boat wave energy for $H > 0.25$ ft per boat for each reach. The boat wave energy/boat was converted to boat wave energy over the 30-min period by multiplying by the number of boats during that period. Results are shown in Figure 30 and Table 8. Figure 30 also shows the observed data, including the data point at RM 10.5 that was not used in development of the boat wave energy equation. As expected, the boat wave energy equation does not do well at RM 10.5, likely because boats immediately upstream are getting on step in an upstream direction and feeding boat wave energy into RM 10.5 but not passing through the boat counting section. For purposes of determining boat wave energy trends, boat wave energy magnitude downstream of the drift area was estimated as shown by the dashed line in Figure 30. It is quite possible that the area from RM 10.5 to 11.0, where boats are leaving the drift and heading back upstream, has the highest boat wave energy at the shoreline of any site on the river. The boat wave energy equation is based on normal traffic conditions and does not take into account large numbers of boats getting on step. Figure 30 is intended to show trends of boat wave energy along the study reach and not absolute values.

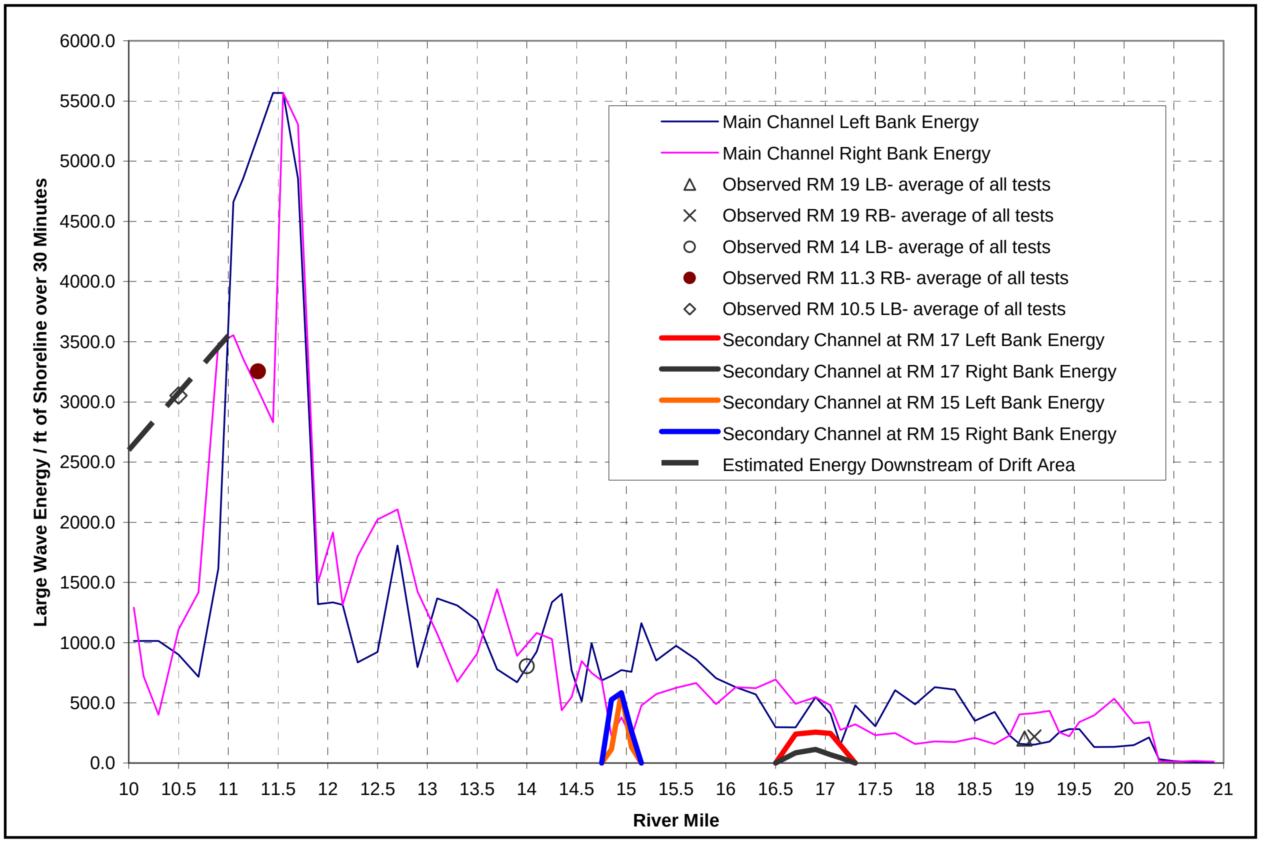

Figure 30. Variation of large (> H=0.25 ft) boat wave energy along study reach based on boat wave energy equation.

Table 8. Reaches used in evaluating boat wave energy along study reach. Note that reaches ending in tenths of a mile are 0.2 mile long and the upstream and downstream limits are 0.1 mile greater and 0.1 mile less, respectively. Reaches having river miles ending in hundredths of a mile are 0.1 mile long and the upstream and downstream limits are 0.05 mile greater and 0.05 mile less, respectively. Distance to left and right banks are from the average boat path shown in Figures 1-4.

River Mile	Distance to Left Bank, ft	Distance to Right Bank, ft	Average Boats/Day	Average Boats/ 30 Minutes	Energy/boat H>0.25 ft Left Bank	Energy/boat H>0.25 ft Right Bank	Energy over 30 Minutes H>0.25 ft Left Bank	Energy over 30 Minutes H>0.25 ft Right Bank
20.9	350	110	10.0	0.4	11.7	30.9	4.9	12.9
20.7	270	125	15.0	0.6	14.5	27.7	9.1	17.3
20.5	190	300	20.0	0.8	19.5	13.3	16.2	11.1
20.35	110	265	25.0	1.0	30.9	14.7	32.1	15.4
20.25	210	120	285.0	11.9	17.9	28.7	212.9	340.6
20.1	320	125	285.0	11.9	12.6	27.7	149.4	329.1
19.9	360	70	285.0	11.9	11.4	45.1	135.4	535.6
19.7	370	100	285.0	11.9	11.1	33.4	132.3	397.0
19.55	150	120	285.0	11.9	23.8	28.7	282.4	340.6
19.45	150	200	285.0	11.9	23.8	18.7	282.4	221.8
19.35	170	170	285.0	11.9	21.4	21.4	254.2	254.2
19.25	260	90	285.0	11.9	15.0	36.5	177.9	433.7
19.1	310	95	285.0	11.9	12.9	34.9	153.5	414.4
18.95	300	100	290.0	12.1	13.3	33.4	160.5	403.9
18.85	200	200	295.0	12.3	18.7	18.7	229.5	229.5
18.7	100	325	305.0	12.7	33.4	12.4	424.8	157.8
18.5	130	240	315.0	13.1	26.8	16.0	352.0	210.3
18.3	70	310	325.0	13.5	45.1	12.9	610.8	175.0
18.1	70	310	335.0	14.0	45.1	12.9	629.6	180.4
17.9	100	380	350.0	14.6	33.4	10.9	487.5	158.8
17.7	80	230	360.0	15.0	40.3	16.6	604.8	249.1
17.5	195	273	385.0	16.0	19.1	14.4	306.0	230.7
17.3	125	200	415.0	17.3	27.7	18.7	479.2	322.9
17.15	360	180	322.5	13.4	11.4	20.4	153.2	274.2
17.05	120	100	345.0	14.4	28.7	33.4	412.3	480.5
16.9	90	90	360.0	15.0	36.5	36.5	547.8	547.8
16.7	200	110	382.5	15.9	18.7	30.9	297.6	491.8
16.5	300	110	540.0	22.5	13.3	30.9	298.9	694.3
16.3	150	135	575.0	24.0	23.8	26.0	569.7	622.4
16.1	140	140	600.0	25.0	25.2	25.2	630.0	630.0
15.9	130	200	630.0	26.3	26.8	18.7	703.9	490.2
15.7	110	150	670.0	27.9	30.9	23.8	861.4	663.8
15.5	100	170	700.0	29.2	33.4	21.4	975.0	624.4
15.3	125	200	730.0	30.4	28.0	18.9	851.8	574.0
15.15	90	260	745.0	31.0	37.4	15.4	1162.3	476.8
15.05	90	400	497.3	20.7	36.5	10.4	756.7	216.1
14.95	90	210	507.0	21.1	36.5	17.9	771.5	378.7
14.85	100	400	520.0	21.7	33.4	10.4	724.3	226.0
14.75	200	200	810.0	33.8	20.3	20.3	686.5	686.5
14.65	135	190	830.0	34.6	28.8	21.6	996.3	747.7
14.55	310	170	845.0	35.2	14.5	24.1	511.3	846.9
14.45	200	300	865.0	36.0	21.4	15.2	769.6	547.4
14.35	100	400	875.0	36.5	38.6	12.0	1405.5	438.6
14.25	110	150	890.0	37.1	36.0	27.8	1336.5	1029.9
14.1	180	150	915.0	38.1	24.3	28.4	927.7	1081.2
13.9	280	200	940.0	39.2	17.1	22.7	671.1	890.3
13.7	250	120	970.0	40.4	19.3	35.8	780.2	1445.2
13.5	160	220	995.0	41.5	28.6	21.9	1187.2	908.6
13.3	150	330	1020.0	42.5	30.8	15.9	1309.7	675.4
13.1	150	200	1045.0	43.5	31.4	24.7	1367.3	1073.8
12.9	300	150	1070.0	44.6	17.9	32.0	796.7	1426.1
12.7	120	100	1100.0	45.8	39.4	46.0	1807.1	2106.2
12.5	280	110	1125.0	46.9	19.7	43.2	923.3	2024.0
12.3	330	140	1150.0	47.9	17.5	35.9	836.6	1719.3
12.15	200	200	1170.0	48.8	27.0	27.0	1314.2	1314.2
12.05	200	130	1180.0	49.2	27.1	39.0	1334.5	1916.3
11.9	210	180	1200.0	50.0	26.4	30.1	1320.3	1502.8
11.7	200	180	2374.0	98.9	49.1	53.6	4856.0	5305.4
11.55	170	170	2374.0	98.9	56.3	56.3	5566.3	5566.3
11.45	170	380	2374.0	98.9	56.3	28.6	5566.3	2832.2
11.3	184	341	2374.0	98.9	52.7	31.4	5208.3	3101.9
11.15	200	310	2374.0	98.9	49.1	34.0	4856.0	3360.5
11.05	210	290	2374.0	98.9	47.1	35.9	4661.0	3554.1
10.9	400	160	1800.0	75.0	21.5	46.5	1614.8	3486.4
10.7	360	160	1100.0	45.8	15.7	31.0	718.1	1419.2
10.5	230	180	1012.0	42.2	21.4	26.3	901.9	1108.1
10.3	200	600	1012.0	42.2	24.1	9.6	1014.3	403.1
10.15	200	300	1012.0	42.2	24.1	17.1	1014.3	721.5
10.05	200	150	1012.0	42.2	24.1	30.6	1014.3	1291.5

Wave height versus size of bank material capable of being moved

In assessing the effects of boat waves, an analysis was conducted to determine what size of bank material can be moved by the waves. In this analysis, two characteristic waves were used to describe the wave climate. First, the average of the $H_{1/100}$ wave height shown in Table 7 was used to characterize the infrequent largest waves on the bank. $H_{1/100}$ is the average of the highest 1 percent of the waves and movement by such infrequent waves likely does not result in significant transport. Second, the significant wave height (H_S) is frequently used in wave investigations and is equal to the average of the highest one-third of the waves. The Hudson equation for stability of armor units on breakwaters and revetments is cautiously used to give an indication of stability because stability coefficients in the Hudson equation for cobble material embedded in smaller material have not been investigated. The Hudson equation is

$$W_{50} = \frac{\gamma_r H^3}{K_D\left(S_r - 1\right)^3 \cot\theta} \qquad (12)$$

where W_{50} is average weight of particle in pounds, γ_r is unit weight of rock = 160 lb/ft^3, H is the wave height, K_D = stability coefficient = 1.8 applicable to rounded material and used for the cobble banks, S_r is the specific gravity of rock = γ_r /unit weight of water, and θ is the bank angle assumed here to be 18.4 deg. W_{50} is converted to average diameter D_{50} using the equation for a sphere. Stone diameter versus wave height is shown in Figure 31 and Table 9. The Hudson equation is strictly applicable to large particles and does not apply to particle sizes where cohesive properties become significant. Based on the boat wave energy plot in Figure 30 and the fact that boat wave energy is a function of wave height squared, estimated wave heights on the left bank at RM 11.3 likely approach an $H_{1/100}$ wave height of 0.91 ft that is capable of moving material of 124 mm (0.41 ft). At the same location, the significant wave height of 0.46 ft is capable of moving 63 mm (0.21 ft) material.

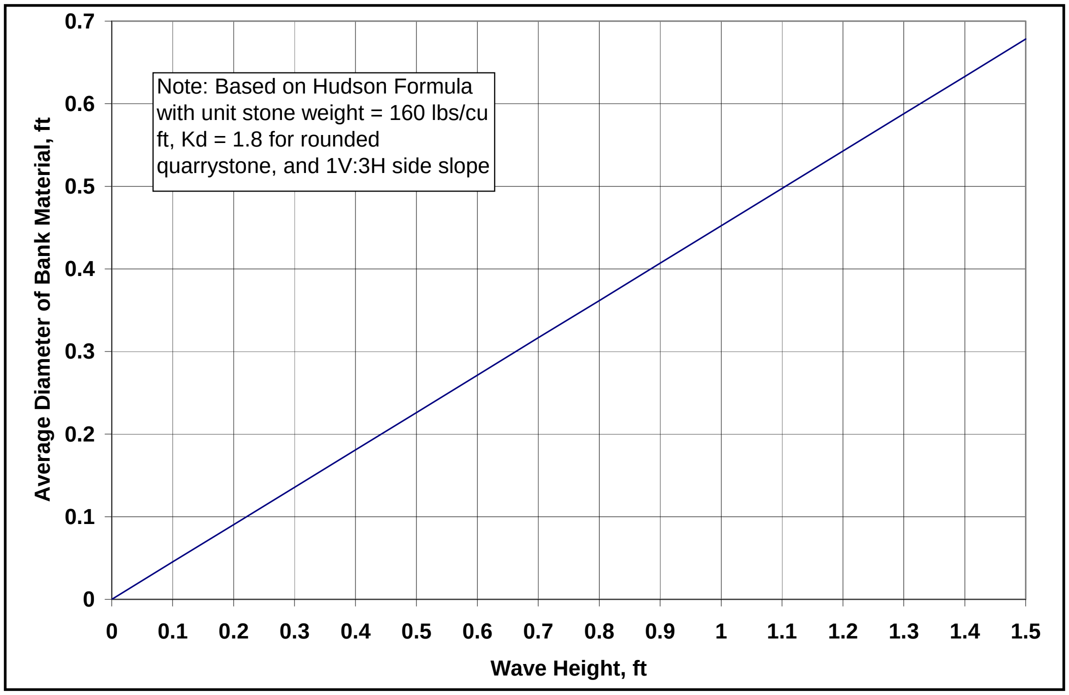

Figure 31. Relationship of bank material size and wave height required for movement.

Table 9. Bank material size capable of being moved by Kenai River boat waves. $H_{1/100}$ is the average of the highest 1 percent of the waves. H_S is the average of the highest one-third of the waves.

River Mile – Bank	Measured Wave Height, ft $H_{1/100}$	Average Material Diameter, mm (ft) from H_{100}	Measured Wave Height, ft H_S	Average Material Diameter, mm (ft) from H_S
19 – Left	0.41	57 (0.19)	0.16	22 (0.07)
19 – Right	0.37	51 (0.17)	0.14	19 (0.06)
14 – Left	0.49	68 (0.22)	0.22	30 (0.10)
11.3 – Right	0.70	96 (0.32)	0.35	48 (0.16)
11.3 – Left	0.91*	125 (0.41)	0.46*	63 (0.21)
10.5 - Left	0.64	88 (0.29)	0.32	44 (0.14)
*Estimated based on boat wave energy magnitude on left and right banks.				

Geomorphic and bank stability assessment

Objectives

The objectives of this component of the study were to conduct a geomorphic assessment of the Kenai River between RM 10 and 21. The geomorphic assessment provides the process-based framework to define past and present channel dynamics, develop integrated solutions, and assess the consequences of remedial actions such as bank stabilization or other structural modifications within the system. A particular focus of this study was the documentation of observed instabilities to determine the dominant causes of erosion and the driving forces behind these processes.

Study approach

The team compiled and evaluated existing data to establish baseline conditions, identify trends, and provide input for later analyses. The data gathered included hydrologic records, sediment data, hydraulic data, construction records, aerial photography and other mapping, channel surveys, and geologic data.

An initial field investigation of the study reach was conducted on 2-6 May 2005 before the summer high flow period. This investigation provided both ground and boat-based observations of the river at very low stage. The banks could be easily inspected for undercutting, block failure, and bed material size. The visit also allowed observation of reaches along the Upper Kenai River where powerboats are not allowed, reaches of the Lower Kenai River where boating pressure is much less, and the Kasilof River. The Kasilof River is a drift fishery (non-motorized) and can be considered to be underfit with large boulders. It has similar vegetation and a few reaches that experience intense bank-fishing pressure.

A detailed field investigation was conducted both by ground and boat in July 2005. During this investigation, the researchers focused upon assessing channel stability and the physical characteristics of the stream that serve as indicators of the dominant geomorphic processes occurring in the basin. They also documented the status of existing structures, locations of problem areas, dominant mechanisms of bank erosion and failure, areas of intense bank use, sediment source and sink areas, vegetative patterns, and other significant morphologic features. Sediment data from the bed, banks, and flood plain were collected and taken to the laboratory for gradation analyses. A geo-referenced digital video was developed for the entire study reach.

The team conducted a number of analyses using historical data and data gathered during the field investigations. Gage records were analyzed to determine trends, major changes, flow duration, and typical hydrographic shape. A Hydrologic Engineering Center River Analysis System (HEC-RAS) model was used to assess the hydraulic characteristics of the study reach. A bank classification scheme was developed and applied to all stream banks within the study area. Historical aerial photographs and other mapping were analyzed to document plan form changes through time. Available geologic and soils data were analyzed to determine spatial distribution trends throughout the system.

Analysis

The primary emphasis of this study was an assessment of the influence of boat wakes upon bank erosion. This entailed studies of the hydrologic character of the river, the hydraulic conditions along and across the river, the characteristics of the sediments in the bed and banks and their erodibility, long-term changes in the bankline positions, and the effects of boat wakes, high discharges, and other stressors upon bankline retreat.

Geology and land use

Figure 32 depicts a portion of a soil type map from the Natural Resources Conservation Service (NRCS 2004) of the lower portion of the study reach. The NRCS soil maps cover the entire study reach. The numbers in each subarea correspond to soil types described in the NRCS report. For example, soil type 535 is a Clunie peat having representative profile of 0 to 33 in. of peat and 33 to 60 in. of silty clay loam. Soil types 611 and 615 are commonly found along the study reach. Soil type 611 is a “Killey and Moose River” soil and has a representative profile of 0 to 2 in. of slightly decomposed plant material, 2 to 6 in. of silt loam, 6 to 29 in. of stratified fine sand to silt loam, and 29 to 60 in. of very gravelly coarse sand. Soil type 615 is a Longmare silt loam and has a representative profile of 0 to 3 in. of moderately decomposed plant material, 3 to 29 in. of silt loam, and 29 to 60 in. of sand. The soils of the Kenai lowlands are generally covered with glacial and terrace gravels (Qtg) and the tidal portions of the Lower Kenai River are composed of alluvial and beach deposits (Qal). Reger and Pinney (1997) describe the last major glaciation of the Kenai lowlands, which provides additional information on probable soil types. They surmise the extents of the four stages of the Naptowne glaciation and the evolution of the modern day channel of the Kenai River. At the maximum extent of glaciation, ice pushing south in what is now the Cook Inlet met ice pushing from Skilak and Tustumena lakes, forming a large impoundment in the vicinity of Sterling (labeled ST on Figure 33) and forming the Sterling Terrace formation. As the Cook Inlet ice receded, meltwater drained along its edge, forming the modern day Kenai River channel above Soldotna and flowing into another impoundment south of

Soldotna, resulting in the Soldotna Terrace formation. The main channel of the Kenai River originally flowed to the southwest, but as the ice continued to recede, the meltwater channel moved to the northwest into the present day Kenai River channel downstream from Soldotna. Massive meltwater discharges eroded the present day Kenai channel into the Sterling and Soldotna Terraces. Further ice sheet recession resulted in reduced discharge into the Kenai River channel, resulting in it generally being "underfit" upstream of Soldotna. The large boulders present in the channel near Soldotna speak of very large prehistoric flows. The Soldotna Terrace is evident at several locations along the Kenai River between RM 21 and 14, where banks are 50 to 100 ft high.

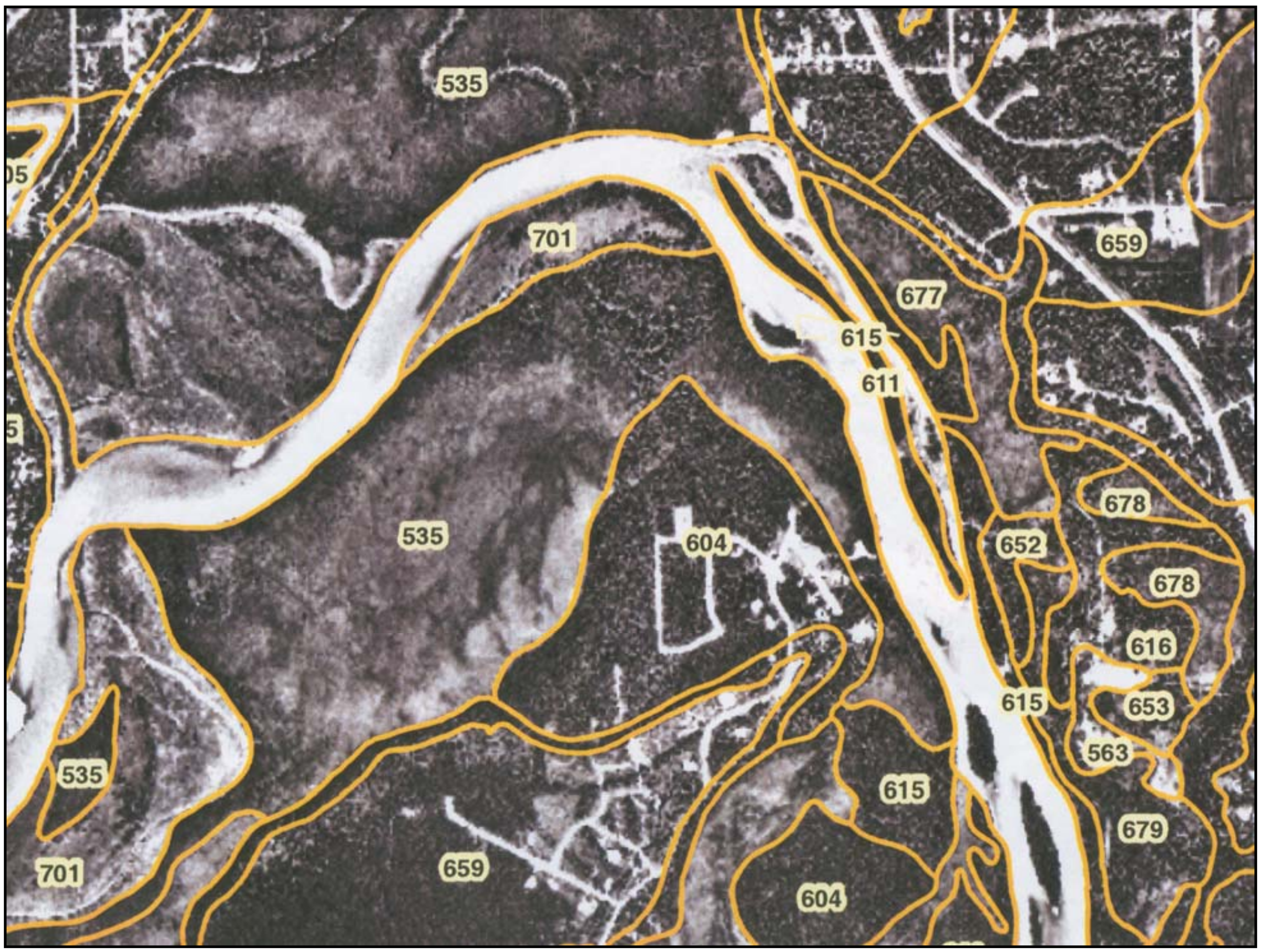

Figure 32. Soil map from NRCS (2004) of lower portion of study reach.

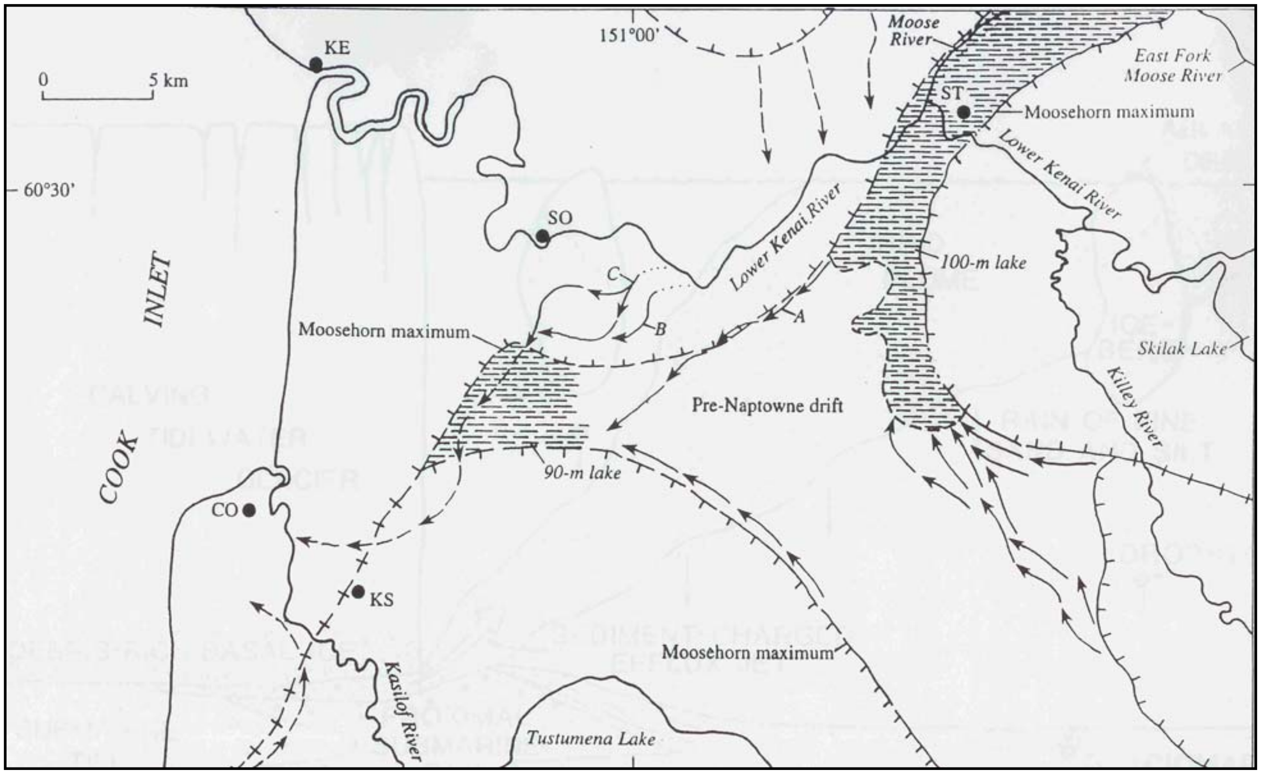

Figure 33. Maximum extent of the Naptowne glaciation (from Reger and Pinney 1997).

Figure 34 shows a land use map. While waterfront land ownership is primarily private, there are large tracts of borough and city-owned land as well as the Kenai River Special Management Area (KRSMA) which is regulated by the ADNR to preserve stream banks and maintain water quality. The public lands can and do receive land use and land access restrictions by the ADNR and the ADF&G, to protect against bank damage and bank retreat. Areas that receive too much bank-fishing pressure are often restricted until the bank vegetation has had a chance to reestablish and some areas have been permanently closed to bank fishermen, or stairs have been built to allow fishermen to enter the water without trampling the banks.

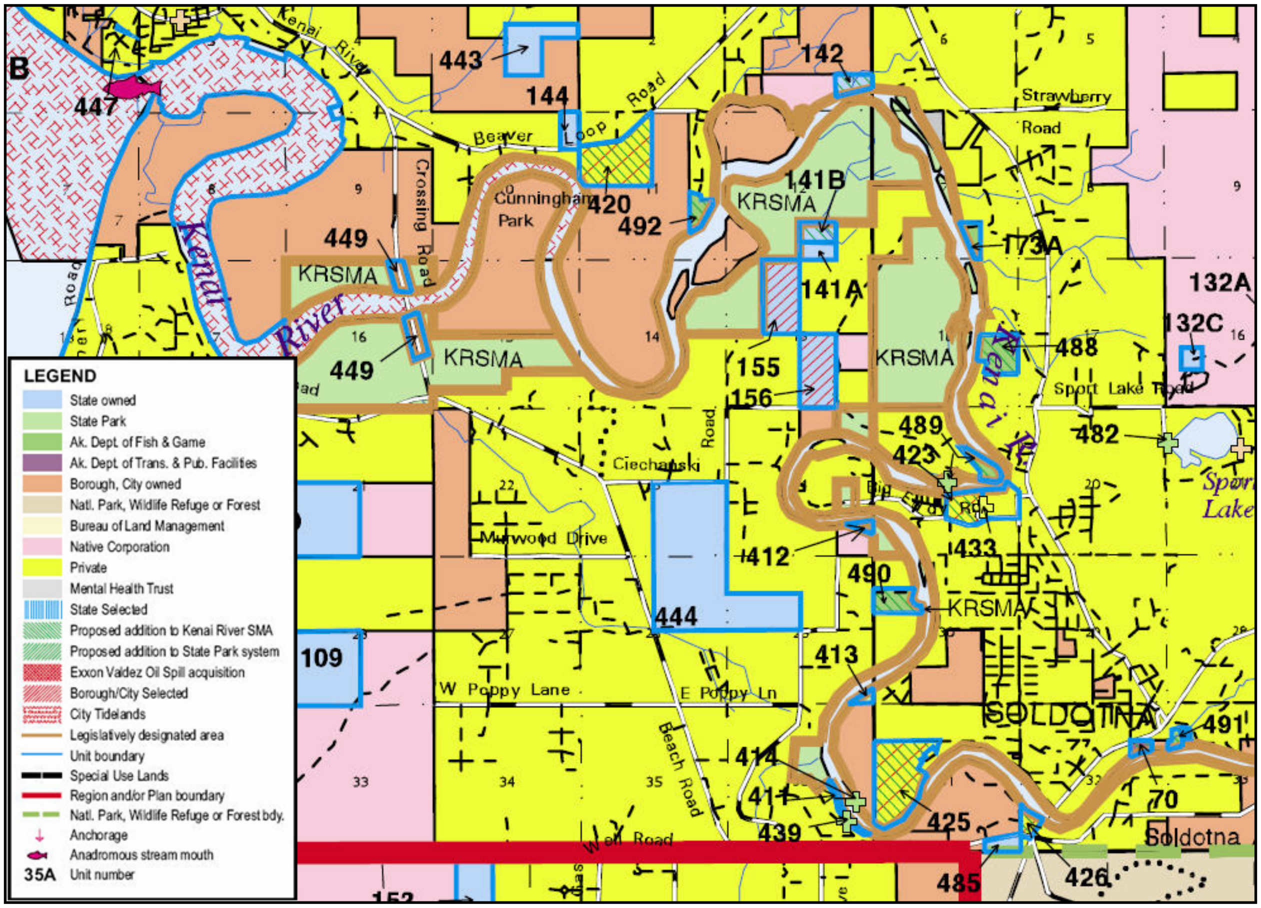

Figure 34. Land use of the study area (from ADNR 2006).

A system of state, federal, and private grants allows private land-owners to stabilize riverbanks, as long as the techniques used improve fish habitat. The cost of restoration is shared 50/50 with the grantor. The Kenai River Center assists landowners in developing plans for "fish friendly" bank stabilization projects. Figure 35 shows the properties (public and private) within the study reach that have conducted bank restoration and rehabilitation projects during 1995-1999.

Hydrology

The study area is located within the lower portion of the Kenai River watershed, where sustained high summer flows occur from a combination of melt water and storm runoff. Drainage area at the USGS Soldotna Gage at the upstream end of the project reach is 1951 square miles. Figure 36 is a hydrograph showing the average mean daily flow for the period of record (1965 to present) in dark blue, the historic daily minimum (yellow), historic daily maximum (red), and the mean daily discharge for 2005 (light blue). Winter baseflows (January through April) are about 1500 cfs with discharge beginning to rise due to snowmelt at the beginning of May. On average, flows reach 5000 cfs by 1 June and increase to 10,000 cfs by late June. High flows are sustained through mid-August with peaks generally reaching 15,000 cfs. The discharge decreases through late summer and fall, receding to 10,000 cfs by October, and to 5000 cfs by November.

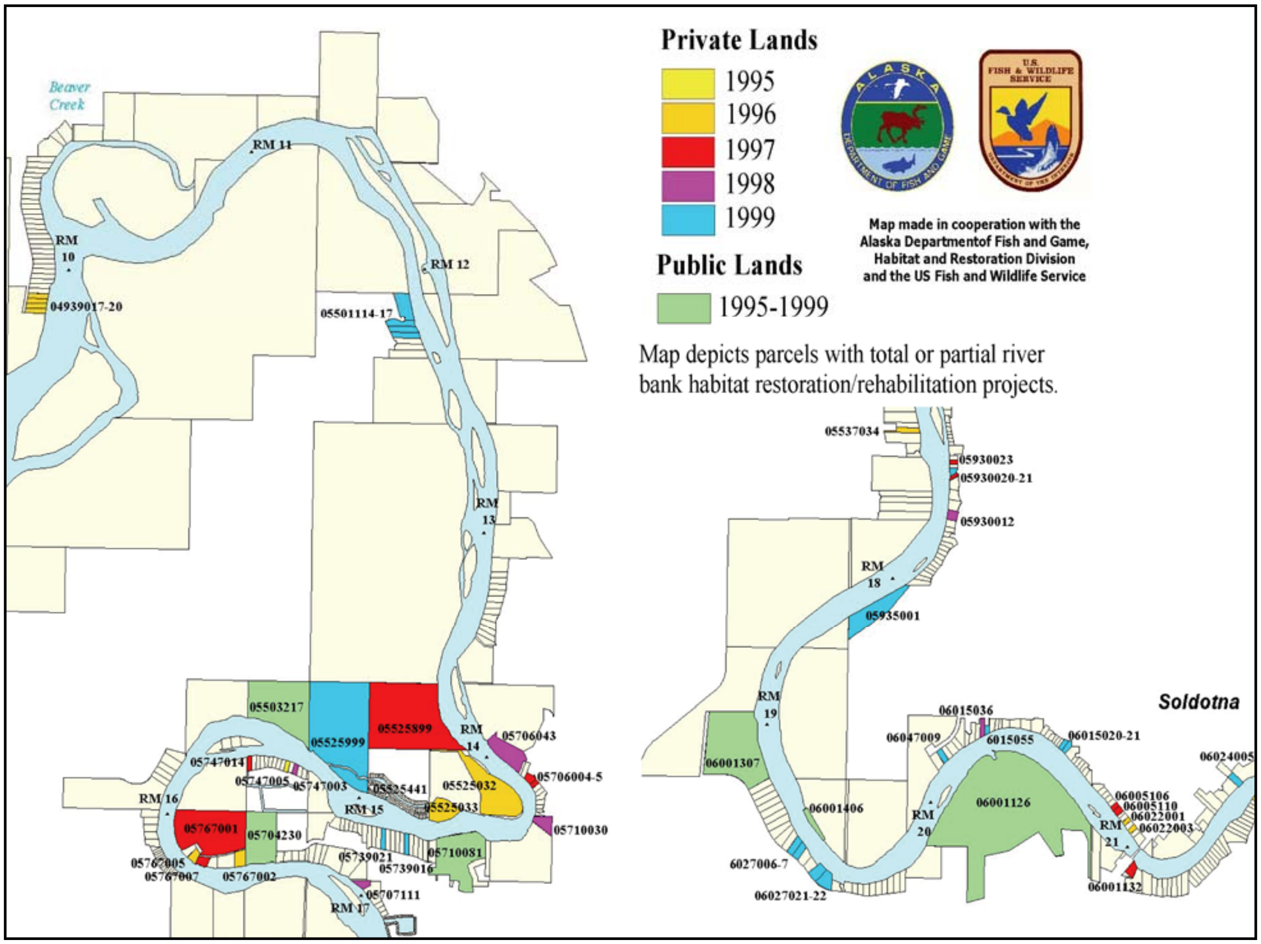

Figure 35. Map depicting permitted stabilization projects on the lower Kenai River.

The highest recorded discharge at Soldotna is 42,200 cfs, which occurred 24 September 1995. Many of the maximum discharges depicted in Figure 36 are due to intense late summer and fall low pressure systems that move into the Gulf of Alaska from the central Pacific Ocean. Two such systems in 2002 produced flooding throughout the Kenai Peninsula during late October and then again in late November. In fact, the maximum daily discharges for 24 October through 25 December all occurred during 2002.

The 2005 daily discharge record in Figure 36 shows that there can be considerable variation in the timing of the summer rise and recession in the late summer and fall. It also shows another discharge feature unique to northern rivers; the Jokulhlaup or glacial lake release. The Snow River (a tributary flowing into Kenai Lake) has a glacier on it that releases about every 3 years. Meltwater builds up behind and under the glacier until it is floated up and water releases as a surge. This surge can be seen in the 2005 daily discharge record of Figure 36 at the beginning of November. While usually not devastating, discharges can increase by several thousand cfs quite rapidly, resulting in increased sediment transport and erosion.

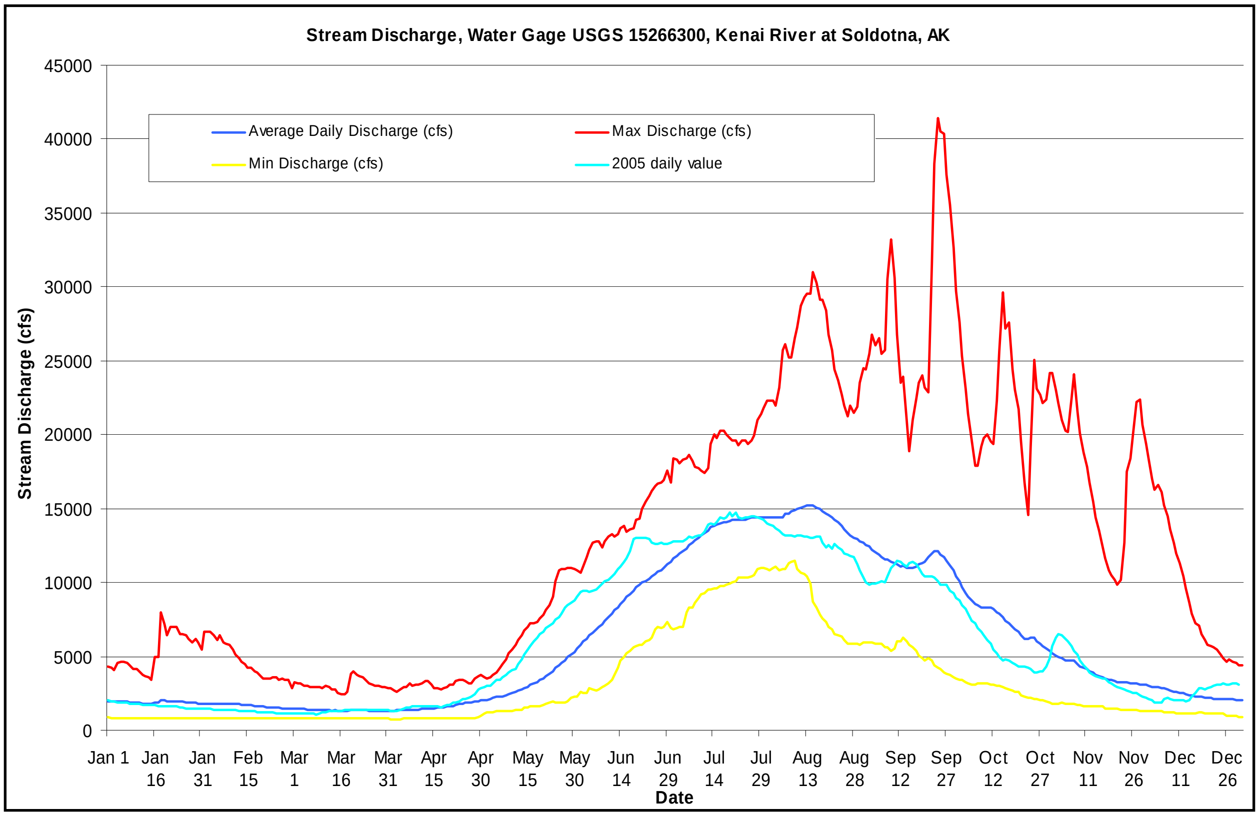

Figure 36. Hydrograph of the Kenai River at Soldotna.

Since the focus of the study was to determine the effects of boat traffic on bank erosion, it is appropriate to discuss the “seasons” of boat traffic (all estimates of boat traffic throughout the year are based on personal communication with Dean Hughes, ADF&G). Boat traffic is non-existent for December through April due to the very low discharge and stage levels and the presence of ice in and along the river. While discharges increase beginning in May, boat traffic is usually very light, restricted to “locals” with jet-powered outboard motors who have a thorough knowledge of rocks, bars, and shallows. By mid-June, discharge and stage is sufficient to allow boating along the entire reach from Soldotna to the mouth. Fishing pressure and boat traffic increases near the end of June and is quite heavy in July. The late run Chinook salmon season is in full swing for the month of July with heavy boat traffic at the popular holes. A section of the Kenai River near the mouth of Slikok Creek (RM 19) opens to Chinook fishing on 15 July, resulting in increased boat traffic in that reach. The Chinook fishery closes on 31 July and, as a result, the boat traffic reduces by about one-half. A strong Coho salmon fishery often continues into October but, after 1 September, most boat traffic is locals and is concentrated in the morning hours. Late September and October boat traffic reduces due to low discharges and stages.

Additional restrictions include only non-guided fishing on Sundays (which effectively cuts the power boat traffic in half), and drift-only fishing on Mondays in June, July, and August. Overall, the boating traffic is a function of the river discharge (and thus stage), fishing pressure, fishing regulations, and tourist activity. Table 10 presents these factors, which combine to show a correspondence between the highest discharges and highest fishing pressure to occur throughout the month of July. The discharge ranges in Table 10 are based on the average mean daily discharge for the period of record.

Table 10. Discharge and fishing pressure.

Time Period	Discharge range (cfs)	Fishing Pressure (Boat Traffic)	Users
December – April	1,300 – 3,000	None	None
May	2,100 – 5,500	Minimal	Resident
Early June	5,500 – 8,800	Light	Resident
Late June	8,800 - 11,500	Medium to Heavy	Resident/Guided
July	11,500 - 14,500	Heavy	Resident/Guided
August	12,700 - 15,200	Medium	Resident/Guided
September	10,400 - 12,700	Light	Resident
October	5,200 - 10,400	Minimal	Resident
November	3,100 – 5,200	None	None

An annual peak flow analysis was conducted to determine the frequency of flooding greater than the annual high flows in July and August. Figure 37 shows a plot of the discharge frequency curve based on a simple plotting position formula.

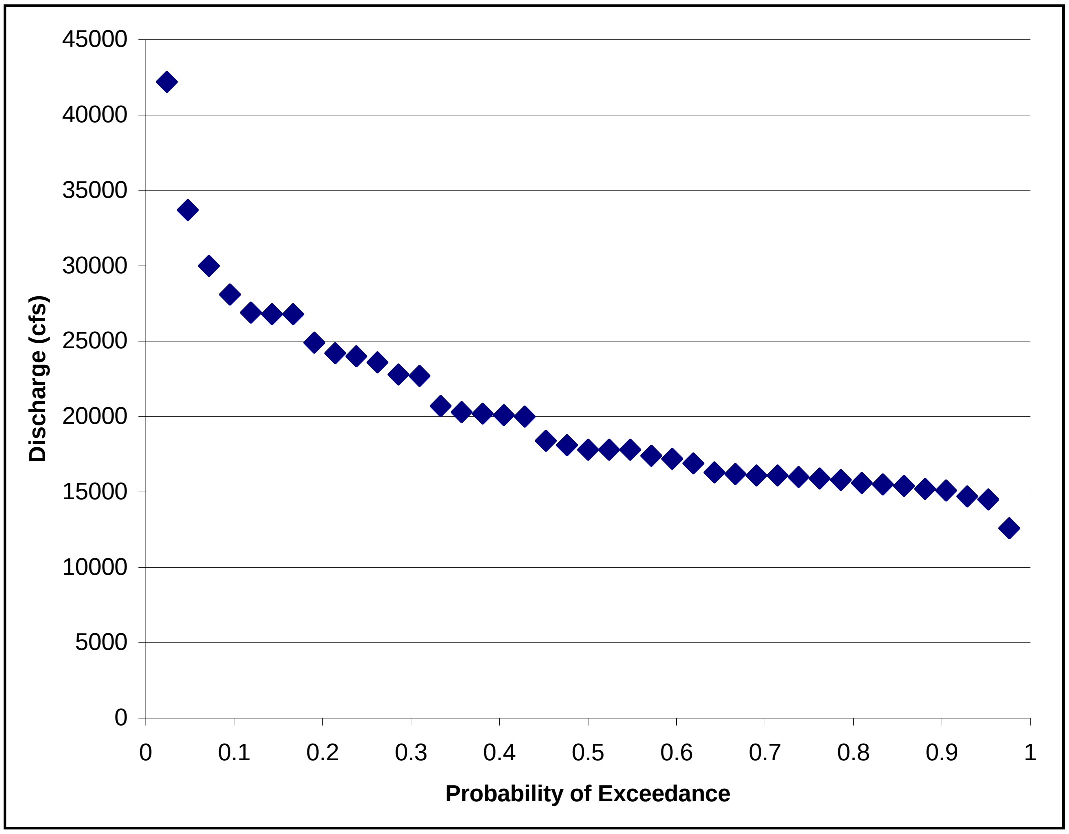

Figure 37. Discharge frequency curve for the Kenai River at Soldotna.

Figure 37 shows that, while the average annual peak flow (based on the average mean daily values of Figure 36) is approximately 15,000 cfs, the 2-yr discharge is approximately 17,500 cfs, the 3-yr is 20,500 cfs, and the 5-yr is 25,000 cfs.

Another way to look at the effects of hydrology on bank erosion is to develop a discharge-duration curve. In general, the discharge reaches 15,000 cfs every summer during July and August. For most sections along the study reach, this corresponds to the vegetation/erosion line. Extended periods of discharge over 15,000 cfs can result in increased erosion and/or vegetation loss. Figure 38 presents a discharge-duration curve for the study reach based on the record for the Soldotna USGS gage, which plots the discharge against the number of days per year that the discharge is equaled or exceeded. The dark blue line represents the mean daily discharge values for the period of record for the gage, while the red line plots data for 2005 (a year in which the maximum flow did not reach 15,000 cfs) and the green line plots 2002 (a year with very high flows). If bank erosion is more probable at discharges above 15,000 cfs, the figure shows that 2002 was a year with accelerated bank erosion due to the 38 days that year with discharges above 15,000 cfs.

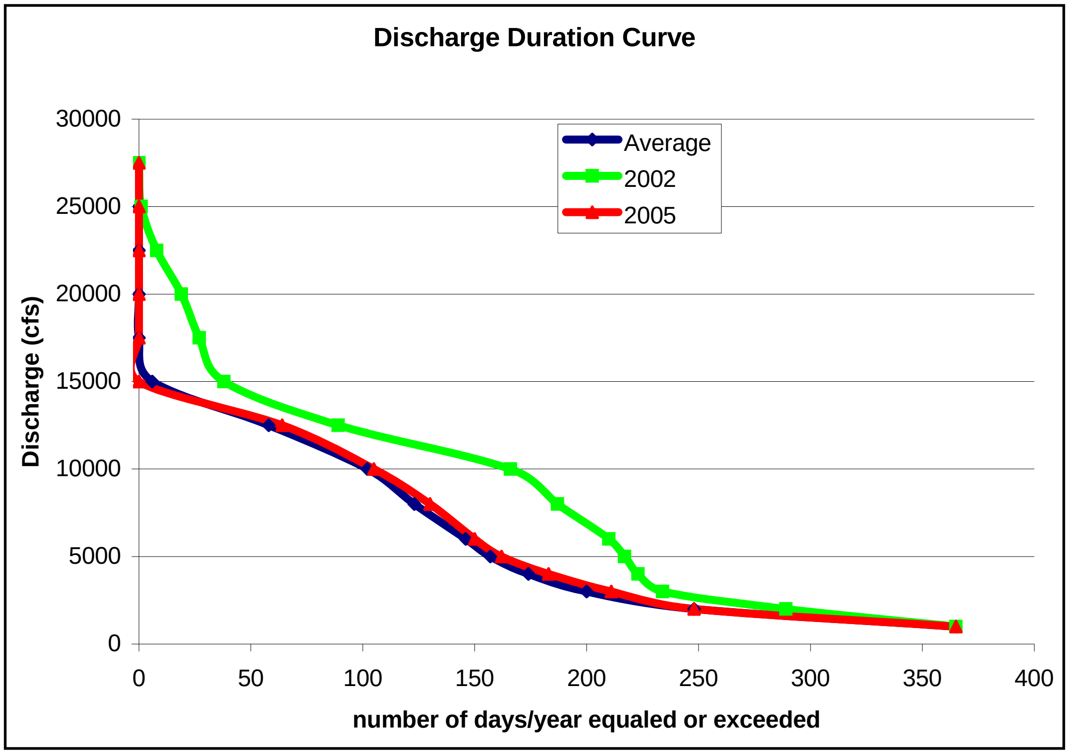

Figure 38. Discharge - duration curve.

Hydraulics

The hydraulics of the study reach were examined to determine the effects of discharge on river width, stage, and velocity. Figure 39 shows a rating curve for the river at the USGS Soldotna gage based on discharge measurement data. The gage is located at RM 21 where the river is entrenched into the Soldotna Terrace. Due to the entrenchment, the rating curve exhibits a fairly linear increase in stage with increase in discharge. Other sections of the river within the study reach have side channels, low floodplains, or are in the tidal zone and thus have a much different rating curve shape.

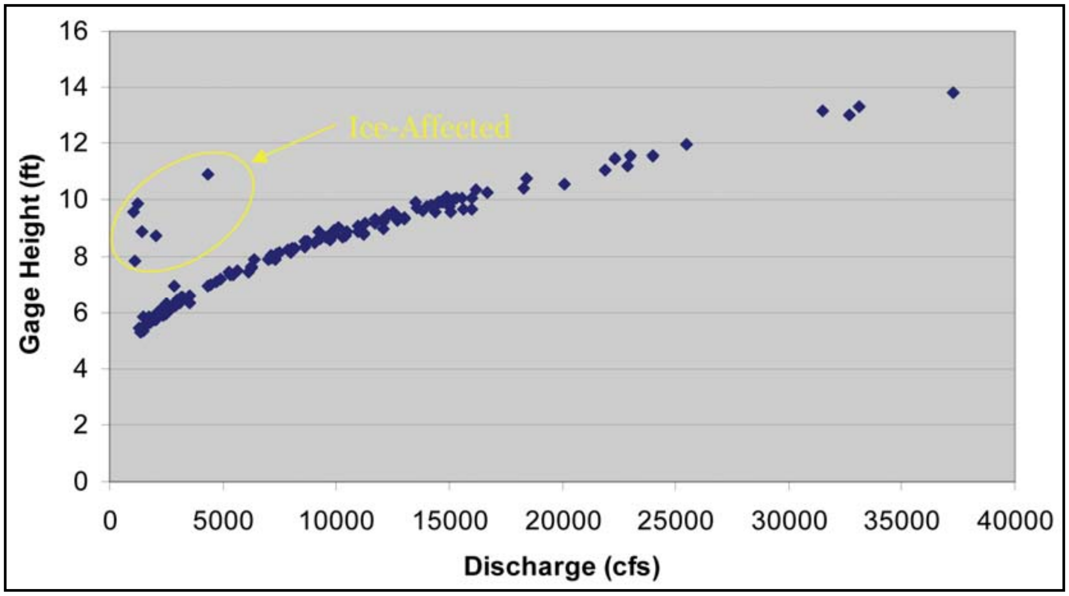

Figure 39. Rating curve for the Kenai River at Soldotna.

Figure 39 shows that the increase in stage at the Soldotna gage due to an increase in discharge from 3,000 cfs (discharge during May 2005 field trip) to 15,000 cfs (discharge during July 2005 field study) is approximately 4 ft. Figures 40 through 42 show comparisons of the stage on 4 May 2005 (discharge of 3030 cfs) and 22 July 2005 (discharge of 14,700 cfs) at RM 20, 17, and 13, respectively. The figures show that the bed materials exposed in the 4 May 2005 photographs are sufficient to resist erosion at the toe of the banks. The stage of 22 July 2005 corresponds to the annual peak stage. It is evident that stages higher than those depicted in Figures 40-42 (whether due to higher discharge or wave height) may result in the erosion of bank materials, causing slumping or block failure of the banks. The 22 July 2005 photograph in Figure 42 also demonstrates the wave environment at this location, a popular fishing hole, where boats drift downstream through the reach and then power back upstream before their next drift. Figure 40 shows the greatest effect of width changes due to increased discharge. At some locations within the study reach, wide gravel bars are exposed during low flows but become submerged by 3 to 4 feet of fast moving water during the heavy fishing seasons.

Figure 40. Stage at RM 20 (4 May 2005 and 22 July 2005).

Figure 41. Stage at RM 17 (4 May 2005 and 22 July 2005).

Figure 42. Stage at RM 13 (4 May 2005 and 22 July 2005).

A HEC-RAS model was developed for the study reach to assess hydraulic conditions over the range of expected flows. Cross sections used to represent channel geometry were developed from an existing HEC-RAS model furnished by the University of Alaska Anchorage (UAA), augmented with cross sections obtained by the Alaska District of the Corps of

Engineers for a flood insurance study. The Alaska Department of Transportation is also conducting a scour study at the Sterling Highway bridge, and data concerning the cross sectional shape of the river and bridge geometry were available. The modeled cross sections extend from RM 0 to RM 22 at the USGS gage in Soldotna (at the Sterling Highway bridge). The model was calibrated by UAA for use in a study near the mouth of the river (O. Smith, personal communication, 2003) and no additional calibrations of upstream reaches were performed. While the cross section spacing and vertical control were not optimal for the study reach, the HEC-RAS model could provide information on relative changes in stage, velocity, stream power, and width with changes in discharge.

Several discharges were chosen to model the range of flows expected throughout the year but also for those expected on a less frequent basis. Table 11 shows the discharges modeled and the conditions relating to their occurrence. Modeling runs were conducted with both low and high tidal conditions at the mouth of the Kenai River. The tidal effects on water levels are a function of the river discharge and the tide level. Maximum upstream effects would occur with the highest tide level and lowest river discharge. At summer discharge levels, the tidal effect is much less significant upstream of RM 12.

Table 11. Discharges modeled with HEC-RAS.

Discharge (cfs)	Modeling condition
5,000	Low discharge conditions (November through May)
10,000	June discharge - medium to heavy boating
15,000	Annual high discharge - heavy boating season
17,500	2-yr discharge
20,000	3-yr discharge
25,000	5-yr discharge
30,000	10-yr discharge

Sediments

Sediments found in the banks along the lower Kenai River are diverse, consisting primarily of poorly sorted glacial till on high bank terraces, and deposits of fine sands, silts, and clays on lower banks below RM 13. Downstream of RM 13, the banks of the Kenai River are generally low in height, and consist primarily of clays and silts, with some fine sands. Samples obtained 8-10 July 2003 and 2-5 May 2005 at RM 9.1, 9.6, 11.0, and 11.3 are representative. The maximum sediment size found in the banks passes a 0.25-mm sieve. The D_{50} of these sediments is about 0.02 mm.

Bank samples collected upstream (at RM 13.2, 15.9, 17.6, 18.3, 19.5, 19.7, and 20.2) demonstrate a coarser and more poorly sorted grain size distribution. The average D_{50} of these samples is just under 2 mm, and about two-thirds of the sediments in the banks fall within the sand classes, with the remainder consisting mostly of gravels. Sediments in the terraces along the Kenai River are often layered due to a complex pattern of glacial advancement and retreat in the region, with associated glacial damming and deposition. The multiple layers can include impervious clay layers, volcanic ash deposition layers, and alluvial gravel layers, which can act as piping conduits. Appendix B presents the grain size distributions of the bank sediments collected for analysis.

Sediments on the bed of the Kenai River grade from coarse to fine with distance downstream. Wolman pebble counts of bar sediments obtained under low flow conditions (3800 cfs at Soldotna) on 24-25 May 2003 (Fischenich 2004) show that the D_{50} upstream of RM 13 is about 2 in., and a larger fraction of coarse sediments are found with distance upstream. Figure 43 presents an example grain size distribution, and Table 12 provides an average of bar samples collected upstream of RM 13. Pebble counts were not conducted downstream of RM 13 because the sediment is too fine. However, a study by Kinetic Laboratories (1998) included samples at RM 2, 3, 5, 10, and 13. The precise location of these samples is not noted in the report, but they demonstrate a d_{50} in the fine sand/coarse silt range. Lenses of gravel are present on the bed in this lower reach, particularly adjacent to high terrace/bluff features, but the bed material appears to be primarily sands.

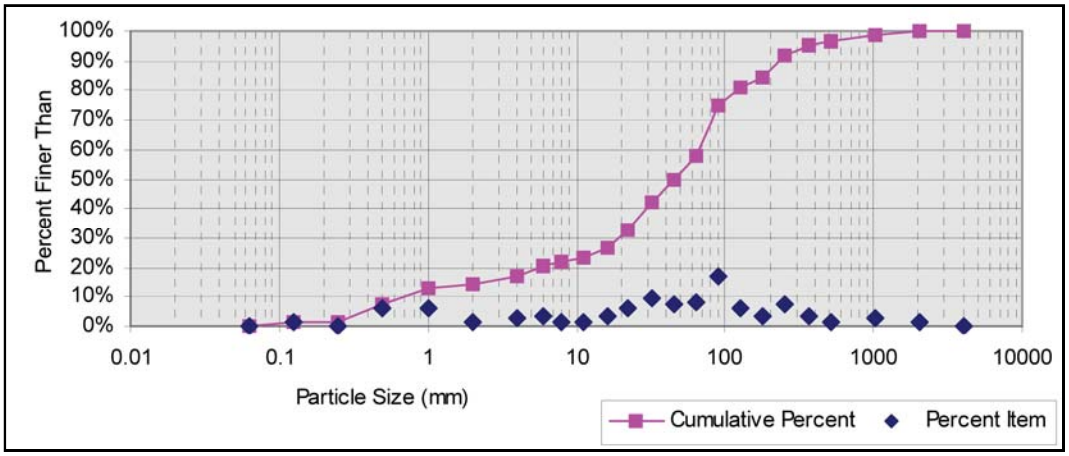

Figure 43. Grain size distribution of bar sediments – Kenai River near Big Eddy (RM 16).

Table 12. Average of bar samples collected upstream of RM 13, Kenai River.

Size percent less than (mm)					Percent by substrate type					
D_{16}	D_{35}	D_{50}	D_{84}	D_{95}	silt/clay	sand	gravel	cobble	boulder	bedrock
3.117	24.22	46.1	174	356	0%	14%	43%	34%	8%	0%

Stability of cobble banks from boat wave attack

The Table 12 bar sediment sizes show a D_{50} and D_{84} of 46 and 174 mm, respectively. While D_{50} is often used to describe stability of revetments, stability of armored areas is often described by the larger fractions such as D_{84}. Comparing Tables 9 and 12 shows that the $H_{1/100}$ waves are capable of moving material exceeding the D_{50} of the bar sediments but not the D_{84}. The more frequent significant wave height H_S is capable of moving the D_{50} of the cobble banks at only the highest traffic areas in the downstream 2-mile reach. Some of the downstream 2 miles of the study reach is composed of cohesive material that is not applicable to this analysis.

Geo-referenced videos

A geo-referenced digital video was conducted during the boat trips in May and July 2005 using the Red Hen positioning system. The geo-referenced video allows the user to view the digital video and a map of the river showing the location of the boat at the same time. The video provides the best photographic information about the wave and boat counting sites. The videos cover the entire study reach from RM 10 to 21. A copy of the videos was provided to the project sponsors.

Bank classifications

One component of the Kenai River study was to identify the various bank classifications along the study reach. The classification scheme developed by Fischenich (2004) was modified to include seven categories of banks. Bank classifications were assigned to both the right and left banks along the entire study reach and are shown in Table 13 and Figures 44–46. The limits for the bank classification reaches shown in Table 13 and Figures 44–46 reflect the dominant bank type for that reach. It should be noted that any reach may have short, localized areas that exhibit different bank characteristics than the reach as a whole. It is also important to recognize that a transition zone often exists between bank types, which makes it difficult to establish exact reach limits. The seven categories of banks are summarized below.

Figure 44. Bank and island classifications for approximate RM 21 to 17.5.

Figure 45. Bank and island classifications for approximate RM 17.5 to 13.

Figure 46. Bank and island classifications for approximate RM 13 to 10.

Table 13. Right and left bank classification, Kenai River, RM 10 to 21.

River Mile	Right Bank Classification	River Mile	Left Bank Classification
20.6 - 21.1	1	20.8 - 21.1	7
19.9 - 20.6	2	20.2 - 20.8	1
19.7 - 19.9	4	20.1 - 20.2	4
19.1 - 19.7	6	19.8 - 20.1	6
19.0 - 19.1	4	19.5 - 19.8	2
17.9 - 19.0	6	18.9 - 19.5	6
17.6 - 17.9	2	18.8 - 18.9	1
16.6 - 17.6	1	18.2 - 18.8	3
16.5 - 16.6	7	18.0 - 18.2	4
16.0 - 16.5	1	17.7 - 18.0	7
15.6 - 16.0	7	17.5 - 17.7	6
15.5 - 15.6	6	17.4 - 17.5	1
15.4 - 15.5	1	16.5 - 17.4	7
15.2 - 15.4	7	16.4 - 16.5	2
14.6 - 15.2	1	15.4 - 16.4	3
14.5 - 14.6	7	14.6 - 15.4	1
14.4 - 14.5	6	13.9 - 14.6	6
14.0 - 14.4	2	13.8 - 13.9	4
13.9 - 14.0	1	13.6 - 13.8	1
13.5 - 13.9	7	13.0 - 13.6	7
13.4 - 13.5	6	12.9 - 13.0	6
13.1 - 13.4	4	12.4 - 12.9	7
12.9 - 13.1	7	12.2 - 12.4	1
12.6 - 12.9	4	11.1 - 12.2	7
12.5 - 12.6	6	10.0 - 11.1	5
11.4 - 12.5	7	*	
11.3 -11.4	6		
10.8 - 11.3	7		
10.4 - 10.8	5		
10.0 - 10.4	6		
*Note that RM 10 is downstream end of reach			

Type 1

Type 1 banks have been stabilized with various types of streambank stabilization. Common stabilization techniques consist of root wads, spruce tree revetments, coir logs, and riprap. Stabilization may be continuous throughout a reach or may occur in a more intermittent manner. The assumption is that Type 1 reaches are stable against imposed river forces or will be maintained should local failures occur. Figures 47–51 show several typical Type 1 banks (stabilization techniques). All permits issued on the Kenai River in the past 15 years have been for the purpose of fish habitat restoration, and have been restricted to bio-engineering measures. Riprap stabilization has been limited to the protection of public structures.

Figure 47. Type 1 bank stabilized with riprap.

Figure 48. Type 1 bank with spruce tree habitat restoration.

Figure 49. Type 1 bank with root wad habitat restoration.

Figure 50. Type 1 bank with coir log habitat restoration.

Figure 51. Type 1 bank with willow plantings/ladder access habitat restoration.

Type 2

These high till banks are covered with dense woody vegetation. Bank recession along these banks is minimal, primarily due to the presence of woody vegetation. Bank heights are generally less than 50 ft. Type 2 banks are limited to areas upstream of RM 13 and are located along the outside of meander bends. Figure 52 shows a typical Type 2 bank. Although Type 2 banks may have some upper bank erosional processes such as piping and shallow translational failures, these local failures do not appear to be the result of hydraulic forces. These banks appeared to be resistant to erosion at the water line due to the presence of vegetation or gravel benches. Soil horizon layers were not evident, although this may have been due to the presence of vegetation.

Figure 52. Type 2 bank, which is resistant to erosion at water line.

Type 3

These high till banks are sometimes overlying glacial lacustrine deposits, which are predominantly void of vegetation. The primary erosion mechanism appears to be erosion at the bank toe by both boat wakes and currents during high flows. The toe material above the average summer time water level is comprised of easily erodible sands, gravels, and cobble supplied from upper bank failures. Typically, a relatively flat bench of cobble armor extends about 4 to 5 ft out from the bank toe below the water line. Type 3 banks are located along the outside of meander bends. Type 3 erosion is limited to the reach upstream of RM 15.4. Figure 53 shows a typical Type 3 bank. Figure 54 shows a schematic of the features of a Type 3 bank at RM 17.

Figure 53. Typical Type 3 bank, predominantly void of vegetation.

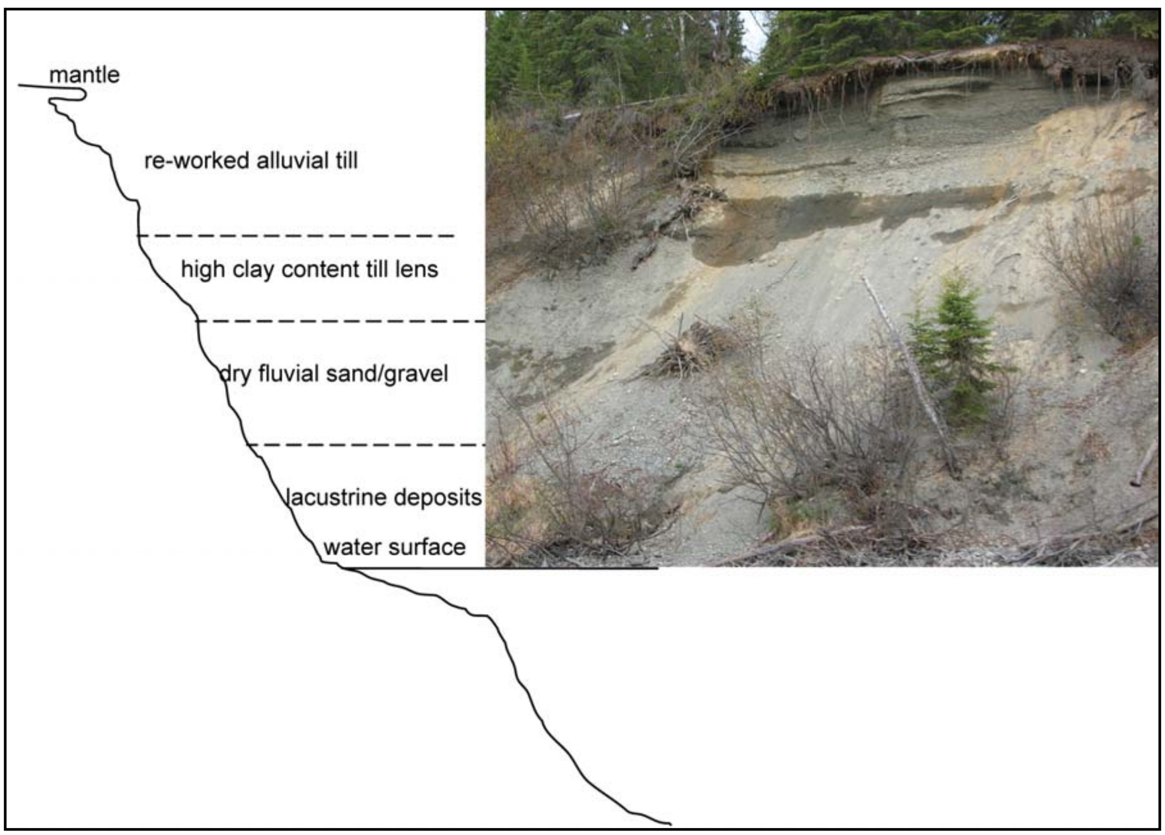

Figure 54. Schematic of Type 3 bank features. Water surface at a discharge of about 15,000 cfs.

Type 4

Type 4 banks are relatively low (5 to 10 ft) and composed of a mix of till and alluvium ranging from silts to cobbles. A flat bench of cobbles generally armor the toe near the average summer time water level. The upper bank is generally near vertical except for a thin root/soil mantle that frequently drapes over the exposed bank face. The two factors that appear responsible for the erosion of these banks include: (1) boat wakes which remove material from the lower bank, thereby triggering upper bank failures and (2) fluvial entrainment at higher flows (1 to 5 year recurrence interval flows). Although boat wakes do cause the removal of some of the lower bank materials, the cobble bench helps dissipate some of these forces.

Figure 55. Typical Type 4 bank showing root/soil mantle draped over exposed bank face.

Type 5

Type 5 banks are relatively low (5 to 8 ft), comprised predominately of clays, silts, and fine sands. These banks are limited to downstream of about RM 11. These banks are highly susceptible to boat wakes, which undercut the banks and set up upper bank failures. Because these banks occur within the tidal zone, wide tidal fluctuations affect the susceptibility of these banks to erosion. Figure 56 shows a typical Type 5 bank.

Figure 56. Typical Type 5 bank, which is highly susceptible to boat wakes.

Type 6

Type 6 banks are relatively low (3 to 5 ft) and covered by a dense growth of herbaceous vegetation. Type 6 banks are often undercut and often overly cobble alluvium. Although the lower slopes of these banks are nearly vertical, the dense grasses serve to absorb the forces of the boat wakes when water levels are near ordinary high water. These banks slowly erode from boat waves and currents, which maintains the overhang. Banks are susceptible to trampling, with fishermen stepping on the unsupported overhanging bank until it breaks off. Type 6 banks generally occur where sediment deposition occurs that allows colonization by grasses. Consequently, the recession rate on these banks is negligible. Figure 57 shows a typical Type 6 bank. Figure 58 shows the effects of bank trampling by anglers, which can damage the integrity of the bank, making it susceptible to erosion. Figure 59 demonstrates the failure of these banks. Trampling and points of access into the water during high flows result in areas where boat wakes and current forces can be accentuated. This effect will eventually lead to block failure of the soil/root mass and open a section of the shore to further erosion. Figure 60 shows the dense herbaceous vegetation and root mass.

Figure 57. Typical Type 6 bank, low and covered with herbaceous vegetation.

Figure 58. Trampling caused by bank anglers on Type 6 bank.

Figure 59. Type 6 bank failure mechanism.

Figure 60. Root mass of herbaceous vegetation of Type 6 bank.

Type 7

Type 7 banks are relatively low (4 to 10 ft) and covered with woody vegetation. Overall erosion rates are minimal. However, these banks typically exhibit irregular bank lines, with localized scallops that may concentrate erosive forces from boat wakes. This irregular bank line may reduce velocities near the bank, effectively reducing erosion from high flows. In some locations, these localized erosion areas can enlarge and begin to exhibit characteristics of Type 4 banks. Although the bank materials may vary from clays, silts, and fine sands (generally downstream of RM 13) to gravels and cobbles, the distinctive characteristic of the Type 7 reach is the presence of woody vegetation. Figure 61 shows a typical Type 7 bank.

Figure 61. Typical Type 7 bank covered with woody vegetation.

Bank types at wave measurement and boat counting sites

At the wave measurement and boat counting site at RM 19.0, both left and right banks were Type 6 banks. At the boat counting site at RM 17.6, the left bank was a Type 6 bank and the right bank was on the border between a Type 1 and 2 bank. At the wave measurement and boat counting site at

RM 14.0, the left bank was a Type 6 bank and the right bank was on the border between a Type 1 and 2 bank. At the wave measurement and boat counting site at RM 11.3, both left and right banks were Type 7 banks. At the wave measurement and boat counting site at RM 10.5, both left and right banks were Type 5 banks.

Bank erosion processes and historical planform analysis

Several studies have reported upon the mechanisms of bank erosion and failure along the Kenai River. Barrick (1984) documented erosion rates and suggested that the loss of vegetation, boat wakes, and improperly designed erosion control practices were the primary factors associated with rates of retreat. Scott (1982) provided some information on this reach of the river with aerial photographs from 1950-51, 1972, and 1977 used to estimate bank erosion rates. He states that the channel is relatively stable between RM 21.8 to 17.6 with estimated rates of erosion of less than 1 ft/yr. Scott suggested that the retreat of low banks is triggered by freeze-thaw, and that flows erode non-cohesive soil lenses of sediments at the toe of high banks, which triggers upper bank failures. Scott pointed to the loss of bank vegetation and streamside use as significant contributing factors. Table 14 shows the erosion rates for the Kenai River within the study area based on the Scott (1982) data. It should be noted that erosion rate should be used with caution because any rate that is stated may or may not include an extreme event that could substantially skew the estimated erosion rate.

Table 14. Erosion rates as determined by Scott (1982).

RM	Erosion rate (ft/yr)	Notes
21.8 to 17.6	<1.0	Entrenched channel, relatively stable
17.6 to 13.4	2.0	Some armoring
13.4 to 9.0	5.0	No armoring, tidal below RM 12

Inghram (1985) disputed much of Scott's analysis, stating that the areas listed as experiencing erosion were those that would typically be expected to be migrating and eroding (from a planform view). He extended the analysis of aerial photographs to include those taken in 1984 and found a generally stable river with several areas of increased erosion over "background" rates. Inghram stated that, while human activity has increased erosion in some localized reaches of the river, it may be impossible to quantify how much is human-induced and how much is natural.

Reckendorf (1989) and Reckendorf and Saele (1993) documented erosion along the Kenai River and determined that erosion rates were related to both natural and anthropogenic factors. Freeze-thaw, particle entrainment by flows and boat wakes, and vegetation loss through removal and trampling were listed as primary triggers of the observed erosion.

Dorava and Moore (1997) conducted a detailed study of the effects of boatwakes on streambank erosion on the Kenai River. Their study found that erosion in segments of the upper river where boat use is restricted was about 75 percent less than that in the most popular boating areas of the lower river. They concluded that the prevalence of boat wave energy relative to the energy from river currents during a specific peak flow period in 1996 indicated that boatwakes produced a substantial contribution to bank erosion at the specific sites studied. They noted, however, that this conclusion should not be applied throughout the river where other erosion mechanisms such as tides, foot traffic, or slumping may dominate, or where streamflow and boat activity conditions vary.

Fischenich (2004) identified several areas of bank erosion along the Kenai River between RM 21.1 and RM 0 by comparing aerial photographs from 1965 and August 1995 coupled with field investigations. He found indicators of each of the factors identified by previous investigators and separated the lower river into two reaches based upon predominant bank characteristics. Downstream of RM 13, boat wakes, freeze-thaw, and piping were the predominant mechanisms of bank loss. Material loss in the lower or middle zones of the banks is often followed by translational failure of the upper bank or, where low cohesive banks are found, cantilevered failure of the upper banks. Fischenich suggested that fluvial entrainment during high floods followed by upper slope failures and dry soil fall were the predominant mechanisms of failure in the reach from RM 13 to RM 22. Other contributing factors in this reach included freeze-thaw, piping, ice scour, and trampling. He determined that banks that were well vegetated were relatively stable, but disturbed segments of the bank were generally retreating. Table 15 shows the erosion rates for the study area based on comparison of the 1965 and 1995 aerial photography.

Table 15. Erosion rates based on aerial photo analysis of Fischenich (2004).

RM	Bank (RB, LB)	Erosion rate (ft/yr)
20.8 to 20.8	LB	2.0
20.6 to 20.6	LB	1.2
20.1 to 19.8	RB	0.8
19.3 to 19.3	LB	1.6
19.1 to 19.0	RB	0.8
18.9 to 18.1	LB	1.2
18.3 to 18.0	RB	1.0
17.7 to 17.6	RB	0.4
16.2 to 15.9	LB	0.6
15.7 to 15.4	LB	1.6
14.0 to 14.0	RB	2.2
13.5 to 13.3	RB	0.4
13.0 to 13.0	LB	1.6
11.5 to 11.0	RB	1.2
11.2 to 10.8	LB	0.6
11.2 to 10.7	RB	0.6
10.8 to 10.3	LB	0.6
10.7 to 10.1	RB	0.4
10.1 to 10.0	LB	1.0

King (draft) conducted a feasibility study to evaluate the ability of modern aerial photogrammetry techniques using older photographs to measure changes in bank position. Area A of the King study (RM 15-21) overlaps with the present study. King evaluated aerial photographs comparing 1975 and 1985, 1985 and 1998, and 1975 and 1998 and presented results in terms of a uniform loss over the entire 6-mile reach. Based on the analysis, uniform loss over the 6-mile reach varied from 0.2 to 0.4 ft/yr.

For this study, high quality images of the river taken in August 1998 were obtained and compared to the August 1995 aerial photographs. These two dates bracket the flood of record in September 1995. While the photographs covered a period of only 3 years, it was felt that the episodic changes of the flood of record would stand out against the lower erosion rates typically encountered (generally less than 2 ft/yr as reported by Fischenich 2004). Both banks were analyzed and broken into reaches by bank type as described above. Several evenly spaced measurements were made for each reach to determine an overall bank erosion rate for each bank type. While this worked well for most reaches, there were a few obvious locations where localized erosion was quite large and most likely attributed to the 1995 flood of record. Due to the quality of the images, erosion rates of less than about 1 ft/yr (absolute measurement of 3 ft over the 3-yr span of the photographs) were considered below the level of detection/resolution and thus labeled as no change. Bank Type 1 is defined as a bank that has been stabilized and, as such, was determined to exhibit no change over the 3-yr span.

Tables 16 and 17 list the erosion rates for the right and left banks, respectively, separated by bank type. Figure 62 is a plot of erosion rate versus river mile for both the right and left banks. As indicated in Tables 16 and 17, the erosion rates for 1995–1998 are generally higher than those reported by Fischenich (2004) and Scott (1982). These higher erosion rates probably reflect that impacts of the 1995 record flood. The plot of erosion rate versus RM (Figure 62) also indicates that the erosion rates are higher upstream of about RM 16. An attempt was made to determine if any relationship existed between erosion rate and bank type. As shown in Figure 63, no clear relation exists between erosion rate and bank type during the 1995–1998 period. Figure 64 is a plot of bank erosion rate versus boat wave energy for the 1995–1998 period. As shown in Figure 64, the highest boat wave energies are associated with erosion rates of about 1 to 2 ft/yr, while the higher erosion rates (5 to 8 ft/yr) occurred where boat wave energy was lower. This lack of a relationship between erosion rates and boat wave energy, and bank types coupled with the extreme erosion rates observed during this period, suggest that the 1995 flood was the dominant factor during this short time period and that the impacts of boat wakes may have been relatively minor.

Table 16. Present study right bank average erosion rates based on 1995–1998 time period (no change indicates erosion rates less than 1 ft/yr).

RM	Bank Type	Average erosion rate (ft/yr)
21.1 to 20.6	1	0
20.6 to 19.9	2	1.3 with local areas up to 7
19.9 to 19.7	4	6
19.7 to 19.1	6	5
19.1 to 19.0	4	2
19.0 to 17.9	6	1.1 with local areas up to 6
17.9 to 17.6	2	8.3
17.6 to 16.6	1	0
16.6 to 16.5	7	2.5
16.5 to 16.0	1	0
16.0 to 15.6	7	5
15.6 to 15.5	6	no change
15.5 to 15.4	1	0
15.4 to 15.2	7	1
15.2 to 14.6	1	0
14.6 to 14.5	7	no change
14.5 to 14.4	6	no change
14.4 to 14.0	2	no change
14.0 to 13.9	1	0
13.9 to 13.5	7	no change
13.5 to 13.4	6	3
13.4 to 13.1	4	no change
13.1 to 12.9	7	no change
12.9 to 12.6	4	no change
12.6 to 12.5	6	no change
12.5 to 11.4	7	2 with local areas to 8
11.4 to 11.3	6	no change
11.3 to 10.8	7	2.5
10.8 to 10.4	5	2.5
10.4 to 10.0	6	2

Table 17. Present study left bank average erosion rates based on 1995–1998 time period (no change indicates average erosion rates less than 1 ft/yr).

RM	Bank Type	Average erosion rate (ft/yr)
21.1 to 20.8	7	1
20.8 to 20.2	1	0
20.2 to 20.1	4	no change
20.1 to 19.8	6	3
19.8 to 19.5	2	7.2
19.5 to 18.9	6	7.1
18.9 to 18.8	1	0
18.8 to 18.2	3	5.2
18.2 to 18.0	4	1.5
18.0 to 17.7	7	2
17.7 to 17.5	6	5.3
17.5 to 17.4	1	0
17.4 to 16.5	7	2
16.5 to 16.4	2	1
16.4 to 15.4	3	2 with local areas to 7
15.4 to 14.6	1	0
14.6 to 13.9	6	no change
13.9 to 13.8	4	7
13.8 to 13.6	1	0
13.6 to 13.0	7	1.3
13.0 to 12.9	6	no change
12.9 to 12.4	7	1 with local areas to 3
12.4 to 12.2	1	0
12.2 to 11.1	7	1 with local areas to 5
11.1 to 10.0	5	2.1

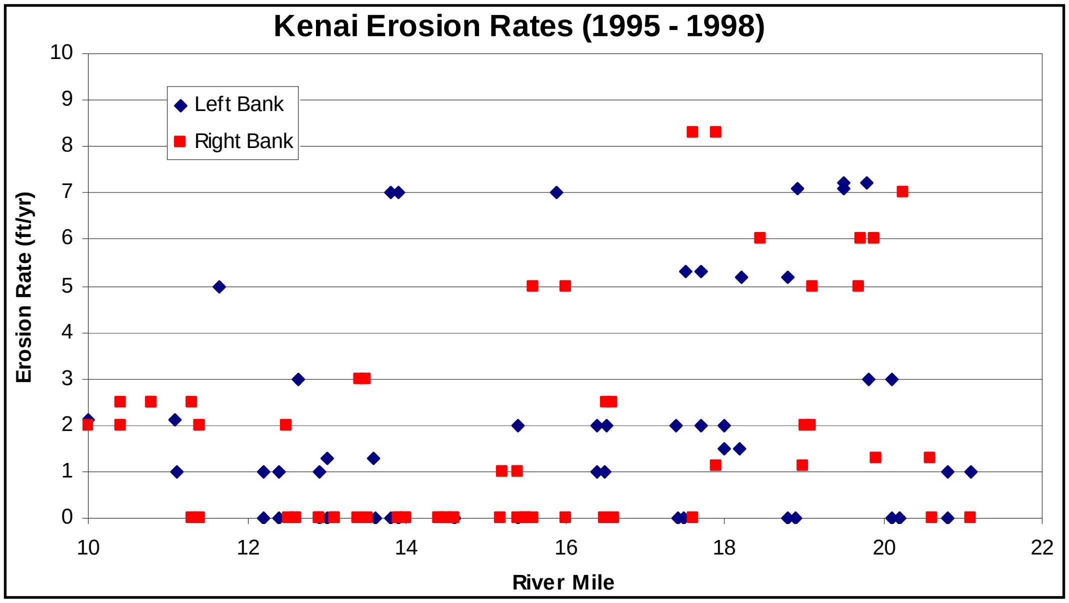

Figure 62. Average erosion rate versus RM based on comparison of 1995 and 1998 aerial photography.

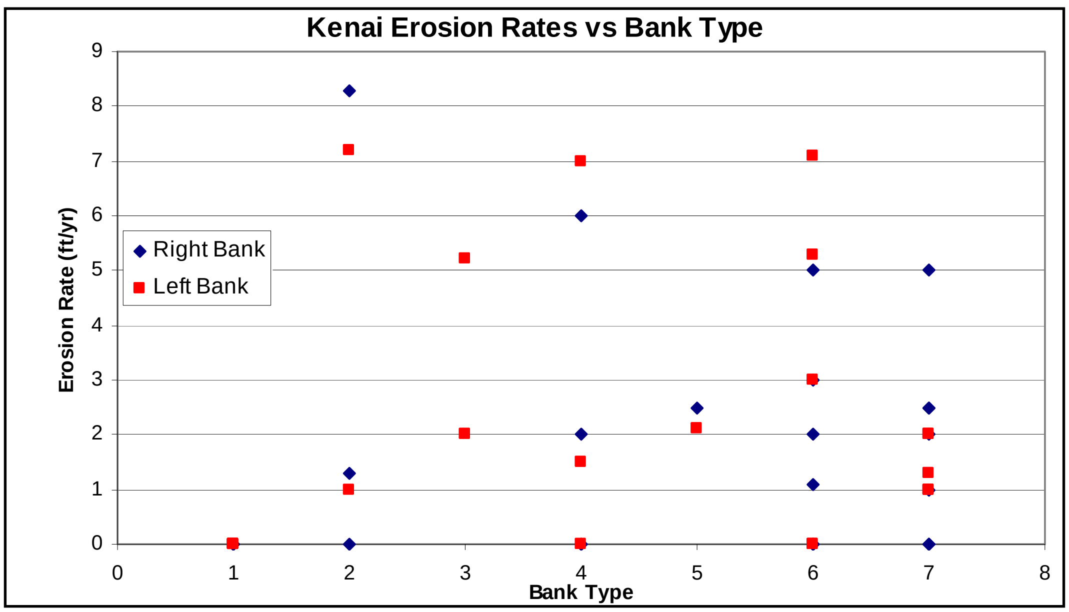

Figure 63. Erosion rate versus bank type based on comparison of 1995 and 1998 aerial photography.

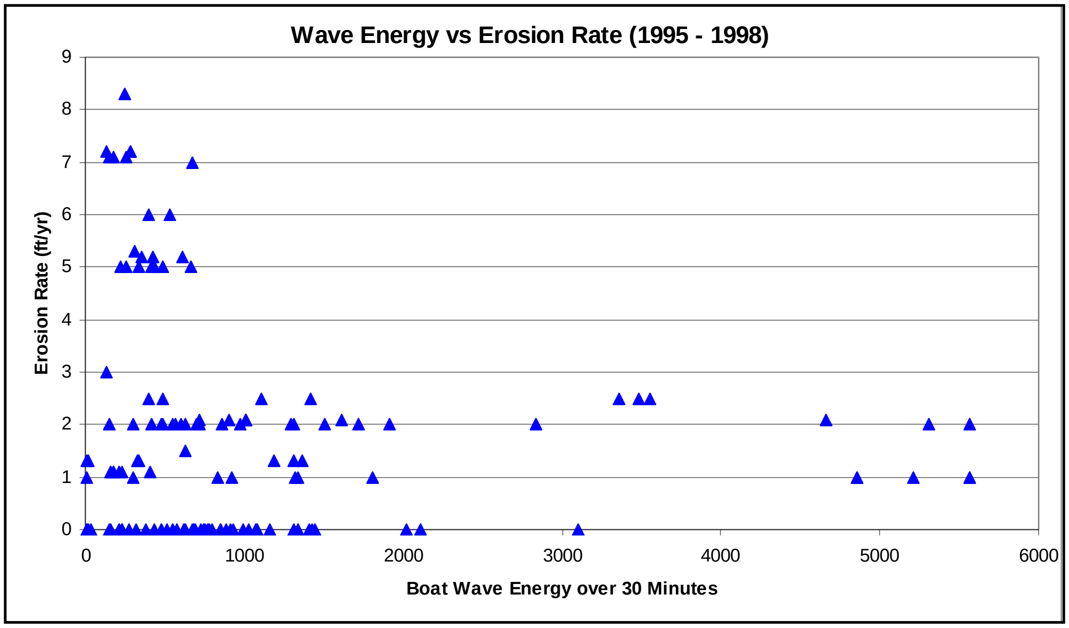

Figure 64. Bank erosion rate (1995–1998) versus boat wave energy.

5 Comparison of Energy from Boat Waves versus Streamflow

While there are differences between energy from boat waves and streamflow, a comparison of the two energies on the shoreline provides an order-of-magnitude type of analysis. One of the main differences is that wave attack is episodic and roughly perpendicular to the shoreline whereas streamflow is relatively constant and parallel to the shoreline. This difference prevents a simple addition of energy from boat waves and streamflow. The power in streamflow is determined from

$$P = \gamma Q(\Delta H) \tag{13}$$

where P = power, Q = discharge, γ = unit weight of water = 62.4 lb/ft^3, and ΔH is the head difference across a unit length of the stream that is equal to the slope. Using the maximum value of average daily discharge from Figure 36 of 15,000 cfs and a slope of 0.0012, P is equal to 1123 ft-lb/sec per ft of channel. Over a 30-min window as used to evaluate boat wave energy, the total streamflow energy per foot of channel length is power*time = 1123*30*60 = 2.02(10)^6 ft-lb. As noted in Hill et al. (2002), the appropriate streamflow energy to use for comparison to boat wave energy is the streamflow energy in the near bank region. Hill et al. used 0.9 percent, 5.2 percent, and 2.5 percent of total streamflow power to define the bank region in streams that were about 410, 50, and 150 ft wide, respectively. Average channel width of the 5 measurement sites on the Kenai River was 464 ft. The boat wave periods of about 1.5 sec have deepwater wavelength of about 11.5 ft. Waves are characterized as deep water if the depth exceeds 0.5 times the wavelength. Wave characteristics are not significantly affected once depth exceeds about 0.4 times the wavelength or about 4 ft for the 11.5 ft wavelength typical of Kenai boat waves. Using a 4-ft depth to define the near bank region, the Kenai River cross sections show an average distance of about 18 ft from the shoreline to a depth of 4 ft. In this 18 ft wide near-bank region, area was about 35 ft^2 and average velocity was about 2 ft/sec, resulting in a discharge of 70 cfs in the near bank region. Using the near-bank discharge of 70 cfs with the channel slope of 0.0012 results in a near-bank streamflow power of 5.2 ft-lb/sec per foot of shoreline that is 0.46 percent of the total channel power. Note that the bank percentage used here of 0.46 percent of total is comparable to extrapolating the Hill et al. (2002) values to a channel width of 464 ft that results in 0.54 percent of total channel power. Over the 30-min evaluation window, streamflow energy in the near bank zone is 5.2*30*60 = 9360 ft-lb per foot of shoreline. Note that, in this order-of-magnitude type of analysis, no distinction is made for different reaches along the river such as inside and outside of bends.

Although bankline streamflow and boat wave energy can be calculated, their relationship to bank recession is not known for environments such as the Kenai River. The approach used here is to define three levels of boat wave bankline energy using both near-bank streamflow energy and boat wave energy along the study reach.

1. Boat wave energy/ft of shoreline less then 5 percent of the shoreline streamflow energy at a discharge of 15,000 cfs. — This level of boat wave energy is less than 0.05*9360 = 468 ft-lb over a 30-min period. This level of boat wave energy is generally found upstream of RM 17.
2. Boat wave energy/ft of shoreline of 5 to 20 percent of the shoreline streamflow energy at a discharge of 15,000 cfs. — This level of boat wave energy is 468 to 1870 ft-lb over a 30-min period. This boat wave energy level range is generally found between RM 17 and 12.
3. Boat wave energy/ft of shoreline greater than 20 percent of the shoreline streamflow energy at a discharge of 15,000 cfs. — This level of boat wave energy is greater than 1870 ft-lb over a 30-min period. This boat wave energy range is generally found between RM 12 and the downstream end of the study reach at RM 10. This boat wave energy level may also be found at other areas where a large number of boats are getting on step. At the highest sites in this downstream reach at RM 11.55 in Table 8, computed shoreline energy from boat waves alone is up to 5566/9360*100 = 59 percent of computed shoreline streamflow energy.

In addition to the above short-term comparison of boat wave energy and streamflow over the same 30-min window for conditions typical of the field study, a long-term comparison was made over relevant conditions during 1 year. Relevant conditions herein are defined as discharges greater than 10,000 cfs. Flows less than 10,000 cfs were not included because boat waves would not reach vulnerable banks at these flows. Using the average daily discharge from Figure 36, 10,000 cfs occurs between about 21 June and 30 September. The long-term analysis is broken down into the time periods shown in Table 18. Because boat count data are not available for the entire period, estimates were made using the description of the boat traffic “seasons” in Table 10, boat counts collected in this field study, and limited data in Dorava and Moore (1997) that show that average August boat counts are 34 percent of average July boat counts at RM 16 on the Kenai. Traffic numbers are based on the number of wave-making boats counted in this study at RM 11.3 to represent conditions in the downstream 2-mile reach. Average daily boat passage during the 12-hr monitoring period was 2,383 boats. Since the 12-hr period did not encompass the entire number of boats during the entire day, the total number of boats was based on ratioing the 12-hr numbers up to 14 hr for a total of 2780 boats per day. This peak number was assumed to apply to the last half of July. Boat numbers during all other time periods were assumed to be a percentage of the peak number of 2780 boats per day as shown in Table 18. The table shows the representative discharge used for each time period based on the average daily discharge curve of Figure 36. Bank energy is calculated using 0.46 percent of the total stream power from Equation 13. The number of boat days is reduced by 1 day per week in June and July to account for Mondays being drift fishing only. Equations 9 and 10 were used to determine the boat wave energy for $H > 0.25$ ft per boat. The boats are assumed to be located one-third of the channel width from the shoreline that is equal to 150 ft in the roughly 450 ft wide channel. Based on this long-term analysis, boat wave energy at the shoreline is about 16 percent of streamflow energy at the shoreline for the conditions specified above. In the upstream reaches where traffic is much less and during high flow years, this percentage will be reduced. The boat wave addition of 16 percent of energy above the streamflow energy is significant and is likely to increase erosion. Techniques for estimating the additional erosion based on the 16 percent increase from boat waves are not available.

Table 18. Long-term comparison of boat wave and streamflow energy based on boat counts at RM 11.3.

Date	Discharge, cfs	# of flow days	Total bank energy from flow, ft-lbs	% of peak boats	# boats per day	# of boat days	Energy per boat, ft-lbs	Total bank energy from boats, ft-lbs
6/21-6/30	10500	10	3124832	70	1946	9	46.0	805384
7/1-7/15	13000	15	5803260	85	2363	13	54.4	1669634
7/16-7/31	14500	16	6904391	100	2780	14	62.7	2441005
8/1-8/15	14900	15	6651429	50	1390	15	34.8	726197
8/16-8/31	14000	16	6666309	30	834	16	23.8	317314
9/1-9/15	12000	15	5356855	20	556	15	23.8	198321
9/16-9/30	11500	15	5133653	10	278	15	23.8	99161
		Total Flow Energy	39640729				Total Boat Energy	6257016
						% Boat/Flow	Energy =	16

6 Environmental Impacts of Boat Waves

Although these studies demonstrated that the magnitude of bank recession associated with boat wakes is much smaller over the long run than flood-induced bank recession, the environmental impacts associated with wake-induced erosion may be significant. Large-scale erosion caused by hydraulic forces during floods serves as an important ecological disturbance that creates new habitats. The recruitment of large woody debris and new spawning gravels on the lower Kenai River, as well as the establishment of substrates for vegetation colonization and succession, may depend upon these events.

The persistent nature of the wake erosion during the peak boating season, on the other hand, may prevent the colonization of some plant species and may induce elevated turbidity levels in the zone near the bank. Figure 65 shows near-bank turbidity at the boat count site on the right bank at RM 11.3. The wake energies are not sufficient to entrain woody debris, so some of the benefits of erosion are not realized from this mechanism of bank loss. The spatial distribution of erosion associated with the boat wakes also differs from flood-related erosion, and bank regions that are largely unaffected by floods (e.g., areas on the inside of bends) may be subject to erosion from boat wakes.

Figure 65. Near-bank turbidity at wave measurement site at RM 11.3.

7 Managing Wave Impacts

Of the previous studies addressing methods of managing vessel wakes, Permanent International Association of Navigation Congresses (PIANC 2003) is a good summary of the various techniques that are available and many of these have and are being considered and used on the Kenai River. PIANC (2003) divides management measures into vessel design, operational measures, and non-operational measures. Each of these is discussed as follows:

1. Vessel design. Hull form is the primary means of managing wakes with vessel design. This approach has been adopted by some Alaska state agencies in their adoption of flat bottom boats partially as a result of Maynord (2001) studies showing reduced maximum wave height with flat bottomed boats compared to v-hull boats. PIANC(2003) notes that one factor that generally cannot be reduced by hull vessel design is wave period, which is important in determining shoreline impacts.
2. Operational measures from PIANC (2003) that might be applicable to the Kenai River:
    a. Increasing shoreline distance from vessel.
    b. Relocating where speed changes are made to avoid focusing wake at a particular location. On the Kenai River, this measure could include controlling where boats get on and off step.
    c. Modifying the schedule to reduce impacts that may be associated with predictable shoreline use or environmental factors.
    d. Training boat operators so they understand type of boat operation and wake generation that is most harmful to the shoreline.
    e. Ensuring that the navigation of the vessel conforms with the courses and speeds established for each leg of the route.
    f. Ensuring the vessel is trimmed to minimize wake.
3. Non-operational measures from PIANC (2003) that might be applicable to the Kenai River. Designing new sea walls and quay walls or retro-fitting existing ones with wave-absorbing materials to reduce wave amplification by reflection. This approach is already underway on the Kenai River with the various habitat restoration methods used on the river.

The PIANC (2003) document also provides numerous references addressing wake management, particularly from high speed ferries.

8 Results and Discussion

Wave measurements and boat traffic observations were conducted to determine variation of boat wave forces along the study reach. Boat wave energy from waves greater than 0.25 ft was used here to quantify shoreline attack. Boat traffic variation along the 11-mile study reach was determined using five counting stations and traffic trends observed by experienced users of the river as shown in Figure 22. A boat wave energy equation was developed based on the computed boat wave energy from the wave measurements and the traffic counts. The traffic data, the average boat path data, and the boat wave energy equation were used to determine the trends of boat wave energy expended on the shorelines of the Kenai River as shown in Figure 30.

The analysis presented herein is based on present levels of boat wave energy. Any future increases in boat wave energy may significantly alter bank erosion levels because existing traffic causes short-term boat wave energy of up to 59 percent of streamflow energy and long-term boat wave energy of up to 16 percent of streamflow energy.

Reduction of boat wave energy should focus on areas having large boat passage frequency such as the drift area at RM 10-12 and areas where bank erosion is most problematic. Techniques to reduce boat waves from a *single* boat are as follows:

1. Use flat bottomed boats. Based on Maynord (2005), maximum wave heights are 22 percent higher with a v-hull boat with all other factors such as boat speed, length, and weight being equal. Using the boat wave equation from Maynord (2005), the v-hull WP at 3170 lb with six people traveling at 20.6 mph produces a 42 percent larger wave than the flat bottom KF at 2650 lb with six people traveling at 22.4 mph. It is not known if flat bottom boats are generally lighter and faster than v-hull boats used on the Kenai River.
2. Allow use of 50 hp. Based on the equation in Maynord (2005) and speeds observed in the 2001 study, the v-hull maximum wave height was reduced 11 percent and flat bottom was reduced 7 percent, all other factors being equal, when going from 35 to 50 hp. This reduction in maximum wave height applies only to planing boats and results from the boat drafting

and/or trimming less at the higher boat speed. Based on the measurements in the 2001 study, the v-hull WP had a 12 percent reduction based on the Kenai River tests. The flat-bottom KF had a 15 percent reduction on Johnson Lake and a 2 percent reduction based on Kenai River tests. Based on all results, the 50 hp motor will reduce maximum wave height by about 10 percent. If boat weights are allowed to increase above their current weights, allowing 50 hp motors will actually increase wave heights. Note that this finding does not address any safety issues resulting from the increased boat speed or any environmental issues resulting from increased motor sizes. This finding addresses only the reduction in maximum wave height.

3. Stay away from shorelines. The Kenai River has an average width in the study reach based on the five measurement sites of about 450 ft. A boat 75 ft from the bank produces a maximum wave 34 percent larger than a boat 150 ft from the bank.
4. Reduce boat weight. One advantage of the flat-bottomed boats tested in the 2000 study was their lighter weight. The 20 ft KF weighed 2650 lb with six people whereas the 20 ft WP weighed 3170 lb with six people. Comparing the same boat hull at the same boat speed, that weight difference alone will cause a maximum wave height of about 11 percent greater with the heavier boat.
5. Although likely difficult to enforce, boats should be encouraged to stay away from speeds that result in the largest waves. For Kenai River boats, the Maynord (2001) study showed the maximum wave-making speeds were about 9 mph relative to the water. In addition to the largest wave height, waves at these speeds had larger periods leading to larger energy on the bank, particularly for boats heading upstream.

Wave height reduction will be most effective in areas having large boat passage frequency such as the drift area at RM 10-12. The actual reduction from some of the above techniques in areas of large boat passage frequency may be less than stated above for single boats. For example, flat bottom boats are far more uncomfortable to ride in when waves are present than v-hull boats. Flat bottom boats generally have to slow down more than v-hull boats when wave conditions are present. The net result is that flat bottom boats will be traveling slower and, because of their slower speed, causing waves closer to the wave height from the v-hull that did not have to slow down as much in wave conditions. This same line of reasoning is why the 50 hp increase may have less reduction than given above for a single boat. When traffic is high and boats have to slow down for passenger safety and comfort, the increased speed and thus decreased wave height from the increased power cannot be realized. The problem with both flat bottomed hull shapes and increased power is that their benefits may not be able to be realized in areas where wave reduction is most effective, namely, high traffic areas. Decreased boat weight and keeping boats away from shorelines are two options that can result in benefits even when large traffic is present.

All banks along the study reach were classified according to the geomorphic scheme developed for the Kenai River. This scheme was based on the erosion rates between 1965 and 1995, field observations in 2005, and basic principles of river mechanics. Based on this analysis and 2005 field observations, it appears that, of the seven bank types, the most susceptible to erosion are Types 3, 4, and 5. These banks appear vulnerable to both boat wakes and high river flows, primarily due to absence of protective vegetation. While bank protection schemes could effectively address both mechanisms of failure, managing boat traffic alone will not eliminate erosion and retreat for these bank types. It should also be noted that the above correlation between bank type and erosion rates may not hold during large flood events such as the flood of September 1995. Comparison of the 1995 and 1998 aerial photography not only produced some of the largest erosion rates observed in the study reach, but also indicated that the bank type is not a good indicator of erosion locations.

Bank Types 2, 6, and 7 are generally less susceptible to erosion due in large part to the presence of vegetation along the banks. The woody vegetation and irregular bankline associated with Type 2 and 7 banks appear to reduce the near-bank velocities and effectively minimize erosion from river currents, but may be less effective in damping boat wave energy. The dense herbaceous vegetation along Type 6 banks effectively protects against both boat wakes and river currents. However, these banks are susceptible to trampling by shore anglers. Although the Type 2, 6, and 7 banks are relatively stable at present, localized areas of erosion can be found along these banks, and it is important to recognize that large floods such as the 1995 event can cause significant erosion here. The loss of the protective vegetation along Type 2, 6, and 7 banks could also lead to future instabilities. Therefore, these banks should be managed to preserve vegetation and monitored routinely to ensure their long-term sustainability.

Various methods have been used along the Kenai River ranging from bio-engineering measures such as root wads, spruce trees, spruce tree revetments, and coir logs to traditional riprap structures. For the past 15 years, only bio-engineering measures have been allowed for the restoration of fish habitat, while riprap stabilization has been limited to protection of public structures. Field observations indicate that, where properly installed, these methods are functioning well and appear to be effective in addressing both boat wake and flow-induced erosion provided they are well maintained.

The boat wake analysis results (Table 8) were correlated with the bank types and the results are shown in Figure 66. As shown in Figure 66, variability in the distribution of boat wave energy is considerable among the bank types. However, the results do indicate that, during the study period, the highest boat wave energy is expended upon the Type 7 and 6 banks.

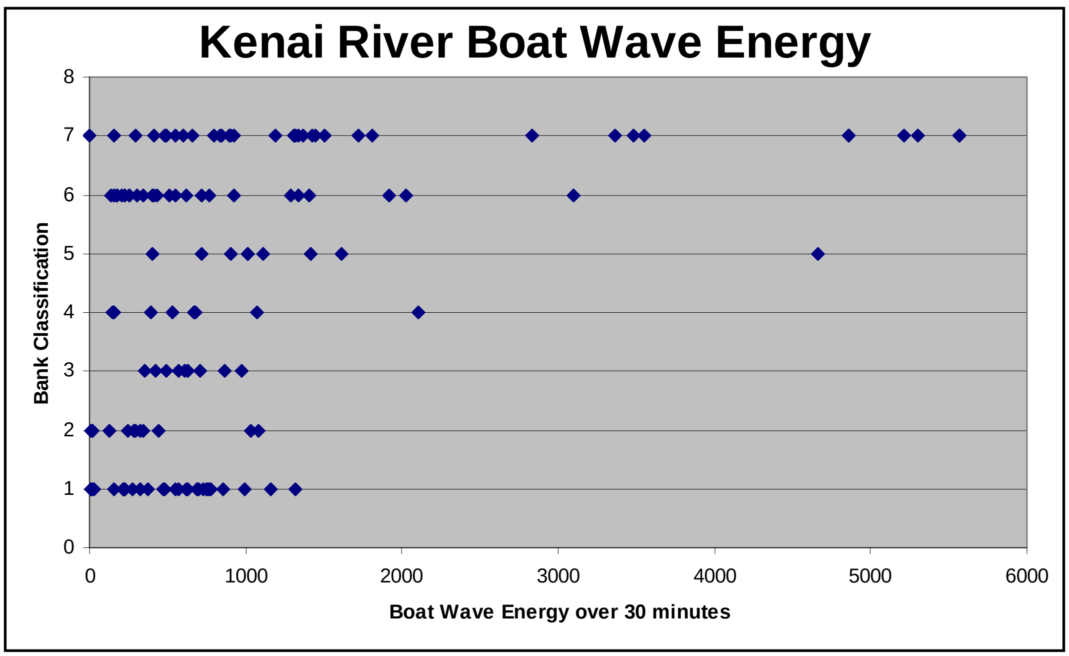

Figure 66. Relationship between bank types and boat wave energy.

A dominant feature along most banks upstream of the tidal zone (about RM 13) is the bench of cobbles ranging from about 4 to 10 in. in diameter that gently slopes riverward from the steeper upper bank. These features are illustrated in Figures 67–70. Field observations during July 2005, when flows were about 14,700 cfs, indicated that boat wakes break on these coarse benches with little movement of the material. This observation suggests that these cobble benches have armored through removal of fine material to resist the forces of boat wakes and river currents. However, the banks above the cobble bench are generally much less erosion resistant. Hydraulic analysis using HEC-RAS indicates that a flow of about 15,000 cfs will inundate the cobble benches (Figure 70). The long-term average annual peak discharge is 15,000 cfs, which generally occurs July to August. This 15,000 cfs flow may provide a guide to when the potential for bank erosion is increased due to boat wakes. Boat wakes will break upon the cobble bench at flows less than about 15,000 cfs, and the potential for bank erosion is minimal. However, if the cobble bench is inundated (flows are around 15,000 cfs or greater), then the upper bank becomes exposed to the forces of boat wakes and is much more susceptible to erosion. At these higher flows, the forces of the river currents are also more pronounced and the overall potential for erosion also increases.

Figure 67. Cobble bench on Type 4 bank at low flow in May 2005.

Figure 68. Cobble bench along Type 6 bank.

Figure 69. Cobble bench showing size of material.

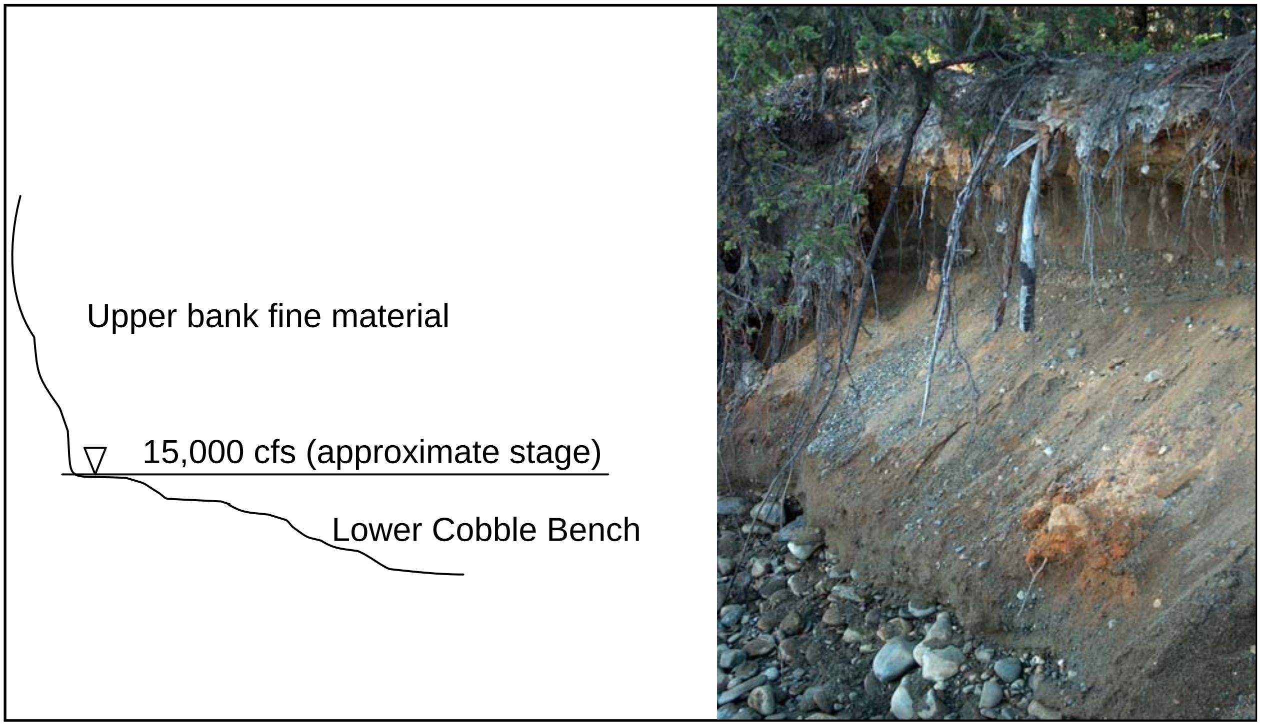

Figure 70. Typical bank profile showing relationship between lower cobble bench, upper bank, and 15,000 cfs flow.

If the decision is made that boat wave energy needs to be reduced, the above threshold discharge concept may provide a potential management option. The appropriate threshold discharge could be 15,000 cfs, or something less if a more conservative approach is taken. For instance, if it is assumed that boat wakes are about 1 ft, then a flow of 12,000 cfs could be selected because the stage at 12,000 cfs is about 1 ft below that for 15,000. During the period of the summer when flows are at or above the threshold discharge, boat traffic could be modified to reduce boat wave energy. Table 19 shows the measured wave heights at five locations along the study reach with the corresponding threshold discharges. It is important to note that, downstream of about RM 13, the tidal effects may render the threshold concept invalid, or at a minimum make its implementation impractical. As shown in Table 19, the wave heights upstream of RM 13 are less. Thus, a threshold discharge of about 14,000 cfs might be considered if this concept was enacted.

Table 19. Maximum measured wave heights and associated threshold discharges.

Wave Measurement Station	Measured Maximum Wave Heights, ft	Discharge Above Which Erosion is Likely	Dates of Concern in 2005
RM 19.0	0.6	14,200 cfs	17-31 July
RM 14.0	0.6 ft	14,000 cfs	14 July – 1 August
RM 11.3	0.9 ft	12,000 cfs	18 June - 26 August
RM 10.5	0.8 ft	13,200 cfs	10 July - 12 August

It is important to note that bank erosion will continue along the Kenai River, even if some type of boat wake management options is enacted. In fact the largest erosion rates measured in the study area occurred during the 1995–1998 time period and probably reflects the September 1995 period of record flood. The fact that this flood occurred during September when boat traffic is reduced suggests that the significant erosion observed during this period was due to the high river currents and not boat wakes. Therefore, these large, relatively infrequent flood events appear to be a dominant factor with respect to bank erosion.

Summary

Bank erosion was observed throughout the Kenai River study reach from RM 10 (downstream end) to RM 21 (upstream end). Bank erosion and deposition are normal and expected fluvial processes that occur in the Kenai River even without human intervention. The observed long-term bank recession rates are generally less than 1 to 2 ft/yr, with locally higher rates associated with flood events or large hillslope failures. This study found that boat wakes are one of several factors responsible for bank erosion along the Kenai River within the study area. In addition to previous studies showing the importance of waves to shoreline recession, boat wakes were observed to move bank material in the field study. However, the additive contribution of boat wakes relative to these other factors is difficult to quantify.

Boat passage frequency along the 11-mile study reach varies significantly, with the largest numbers of boats in the downstream end of the reach. Based on counts in July 2005, wave-making boats pass the downstream study sites at a frequency of up to seven times greater than for the upstream sites. As a result, boat wave energy from waves greater than 0.25 ft at the shoreline in the major drift area near RM 10-12 is up to 10 times greater than the boat wave energy at the shoreline above RM 17. An attempt was made to correlate boat wave energy with bank recession rates; however, no relationship was found.

The contribution of boat waves relative to other factors such as river currents varies throughout the year. If the peak boating period of July occurs during lower than normal flows (less than about 15,000 cfs), boat wave energy is largely expended on the cobble bank present along much of the river. If the peak boating period of July occurs during higher than normal flows (greater than about 15,000 cfs), boat wave energy attacks the banks above the cobble and boat-wave-induced erosion may be the dominant process. However, bankline recession during these periods may be relatively low based on this analysis. At even higher river flows such as major flood events, the boat wakes appear to become a secondary factor.

The largest shoreline boat waves that occur about 1 percent of the time are capable of moving material exceeding the D_{50} of the cobble banks along the river but not the D_{84} size that is often used to characterize the stability of cobble banks that have formed by an armoring process. The more frequent "significant" wave height equal to the average of the highest one-third of all waves is capable of moving the D_{50} of the cobble banks at only the highest traffic areas in the downstream 2-mile reach.

The relative contribution of boat wakes and river currents was also evaluated by comparing energy at the shoreline from boat waves and energy at the shoreline from streamflow. From RM 21 to about RM 17, computed energy at the shoreline from boat waves is less than or equal to 5 percent of computed energy at the shoreline from streamflow based on the typical 12-hr monitoring period during the 2005 field study. From RM 17 to 12, computed energy at the shoreline from boat waves alone is greater than 5 percent and less than or equal to 20 percent of computed energy at the shoreline from streamflow based on the typical 12-hr monitoring period. From RM 12 to 10, computed energy at the shoreline from boats alone is greater than 20 percent of computed energy at the shoreline from streamflow based on the typical 12-hr monitoring period. At the highest sites in the downstream 2-mile reach, computed energy at the shoreline from boats alone is up to 59 percent of computed energy at the bankline from streamflow based on the typical 12-hr monitoring period. These levels of boat wave energy show the relative importance of boat wave energy to streamflow energy, but the combination of streamflow and boat wave energy is not a simple additive relationship.

When comparing streamflow and boat wave energy magnitude for relevant discharges during the entire year, the shoreline boat wave energy is about 16 percent of the shoreline streamflow energy for the highest boat wave energy sites. The percentage for the entire year becomes less during high flow years such as 1995 and significantly less at upstream sites having less boat traffic. This analysis shows that, at specific times of the year and at specific locations, boat wave energy may be a dominant factor, but on an average annual basis it is secondary to river currents in terms of total bank line recession.

During the 1995–1998 period, localized bank recession rates of up to 8 ft/yr were observed. These larger bank recession rates likely reflect the period of record flood that occurred in September 1995 and suggest that major flood events may be the dominant factor with respect to significant bank recession. Although our studies demonstrated that the magnitude of erosion associated with boat wakes is much smaller over the long run than flood-induced erosion, the environmental impacts associated with wake-induced erosion may be significant. Large-scale erosion caused by hydraulic forces during floods serves as an important ecological disturbance that creates new habitats. The recruitment of large woody debris and new spawning gravels on the lower Kenai River, as well as the establishment of substrates for vegetation colonization and succession, may depend upon these events. The persistent nature of the wake erosion during the peak boating season, on the other hand, may prevent the colonization of some plant species and may induce elevated turbidity levels in the zone near the bank. The wake energies are not sufficient to entrain woody debris, so some of the benefits of erosion are not realized from this mechanism of bank loss. The spatial distribution of erosion associated with the boat wakes also differs from flood-related erosion, and bank regions that are largely unaffected by floods (e.g., areas on the inside of bends) may be subject to erosion from boat wakes.

Banks along the study reach were classified with respect to susceptibility to erosion from boat wakes and high river flows. The classification scheme was based on long-term erosion rates from Fischenich (2004), field observations in 2005, and basic principles of river mechanics. A primary consideration in the classification scheme was that the presence of vegetation along the bank appears to significantly reduce erosion associated with boat wakes and high flows. The common trait in bank types 2, 6, and 7 is the presence of woody or herbaceous vegetation along the bank. Bank types 3, 4, and 5 lack this vegetative protection and, therefore, appear more susceptible to erosion. It should be noted that, when large flood events occur, all banks may be subject to significant erosion. In areas where bank vegetation has been removed and bank erosion is occurring, an effective management option might be the implementation of bio-engineering measures that have proven successful within the study area. These measures would both restore the disturbed habitat and protect the banks from further erosion. Another possible management option would be to modify boat operation. A discharge threshold concept is presented as a possible method to reduce the boat wake impacts. It should be noted that, even if all boat traffic were eliminated from the river, erosion due to other factors would continue, although at a slower rate in some locations.

The analysis presented here is based on present levels of boat wave energy. Any future increases in boat wave energy may significantly alter bank erosion levels because existing traffic causes short-term boat wave energy of up to 59 percent of streamflow energy and long-term boat wave energy of up to 16 percent of streamflow energy. Reduction of boat wave energy should focus on areas having large boat passage frequency such as the drift area at RM 10-12 and areas where bank erosion is most problematic. Techniques to reduce boat waves from a *single* boat include use of flat bottomed boats, use of 50 hp motors to increase boat speed, keeping boats away from shorelines, and reducing boat weight. Note that 50 hp motors should not be considered unless present boat weights are maintained. Also note that the finding of decreased wave height from 50 hp motors does not address any safety issues resulting from the increased boat speed or any environmental issues resulting from increased motor sizes. The actual reduction from some of the above boat wave energy reduction techniques in areas of large boat passage will likely be less than for a single boat because of altered boat operation in areas with a large number of waves. The problem with both flat bottomed hull shapes and increased power is that their benefits may not be able to be realized in areas where wave reduction is needed most, namely high traffic areas. Decreased boat weight and keeping boats away from shorelines are two options that can result in benefits even when heavy traffic is present.

In summary, this study found that boat wakes are one of several factors contributing to bank recession. However, quantification of the relative magnitude of boat wakes to other factors such as river currents could not be determined. The results indicate that boat wakes may be a dominant factor during certain high boat usages times, discharges, and locations along the study reach. Although wake-induced erosion may be a secondary factor in shoreline recession, it may be ecologically significant because of its persistence, distribution, and timing. However, bank recession associated with large flood events will likely overshadow the contribution from boat waves.

Additional study

This study has revealed the need for several follow-on tasks to better define some factors that are not well known. The analysis of bank recession rates indicates there is a general trend of low rates of erosion over time, interspersed with high flow events that cause considerable bank loss. This finding is based on the various bank recession rate studies using aerial photographs. Aerial photographs on a regular 3-yr cycle are recommended. In order to better delineate the rates of erosion, it is suggested that certain reaches of the river be surveyed (DGPS) every year during the low water conditions of early May and also following any high flow events (flows greater than 20,000 cfs). The reaches to be surveyed would include representative areas of high and low boat traffic, vegetated and non-vegetated areas, and reaches where one would expect erosion (and no erosion) based on the planform analysis. These measurements would need to be conducted over enough years to provide general long-term rates but also the episodic rates associated with flooding events and high boat use.

Boat counts were conducted during the peak river usage time of middle to late July. These counts were spread over the study reach with some counts corresponding to known fishing holes. Boat usage patterns for other times during the season (May through October) were obtained from discussions with local users, ADF&G personnel, and stage records. It is suggested that additional boat counts take place during other time periods to better define river usage throughout the season. It is also probable that the peak boat usage during the Coho salmon season (August-October) differs in location from the Chinook salmon season (May-July). One alternative is that these counts could take place on the second Saturdays in May, June, July, August, and September. Boat counts should separate boats that are underway and making waves from drift boats not making waves.

It was evident that the high usage areas (particularly RM 10-12) experienced significant boat traffic. Boats did not simply run up river at high speed (on step) and then float down through the entire fishing hole. Navigation through these high use areas often requires slowing and avoiding drifting boats, boats with a fish on line, or for safety reasons to avoid large waves. It is recommended to map the high usage areas and determine how boats access these areas; whether they exercise a "conveyer belt" power upstream-drift through technique or a more random process. It would also be helpful to understand what areas of the hole are used the most. It might then be possible to define travel lanes and drift lanes through these high usage areas.

Another unknown is the stability of banks that have been formed by armoring under combined wave and current attack. This study would best be done in a large laboratory facility where both waves and currents can be simulated. This type of work would answer how the energies from waves and streamflow can be added together to define bank stability. Consideration should be given to a study evaluating the impact boat waves on turbidity. Turbidity measurements should be conducted throughout the day and along the study reach at locations corresponding to high/low/no boat traffic, tidal and non-tidal reaches, various bank material types, and various discharges.

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Appendix A: Cross Section and Depth-Averaged Velocity Transects from ADCP

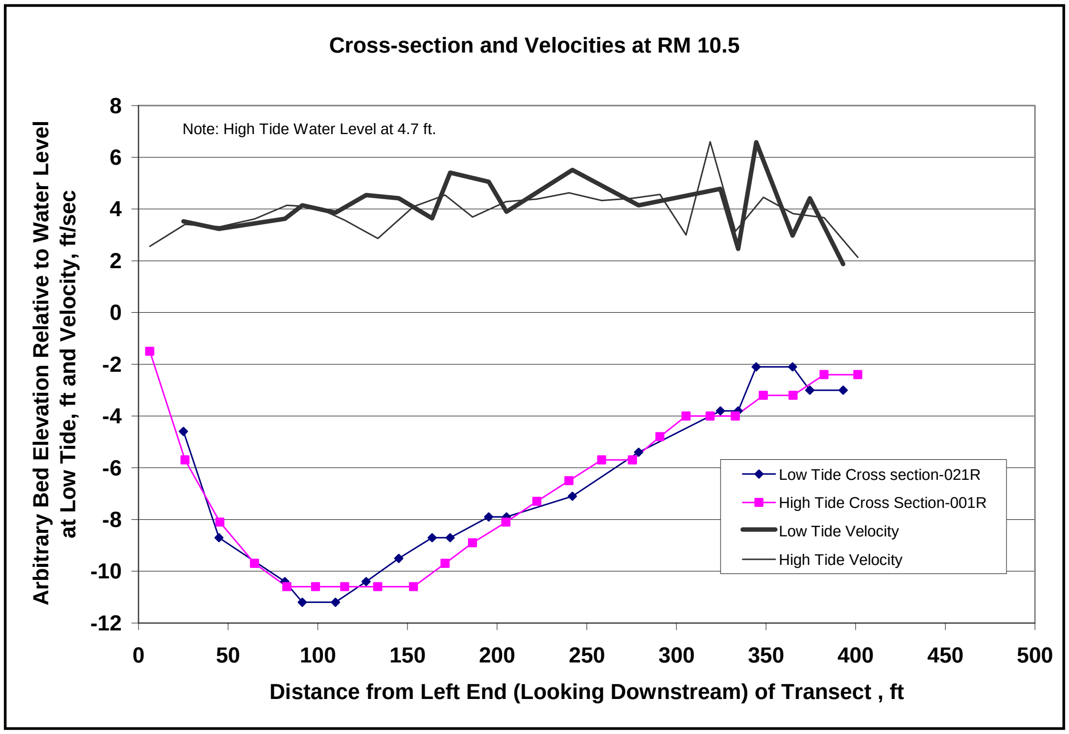


Figure A1. RM 10.5.

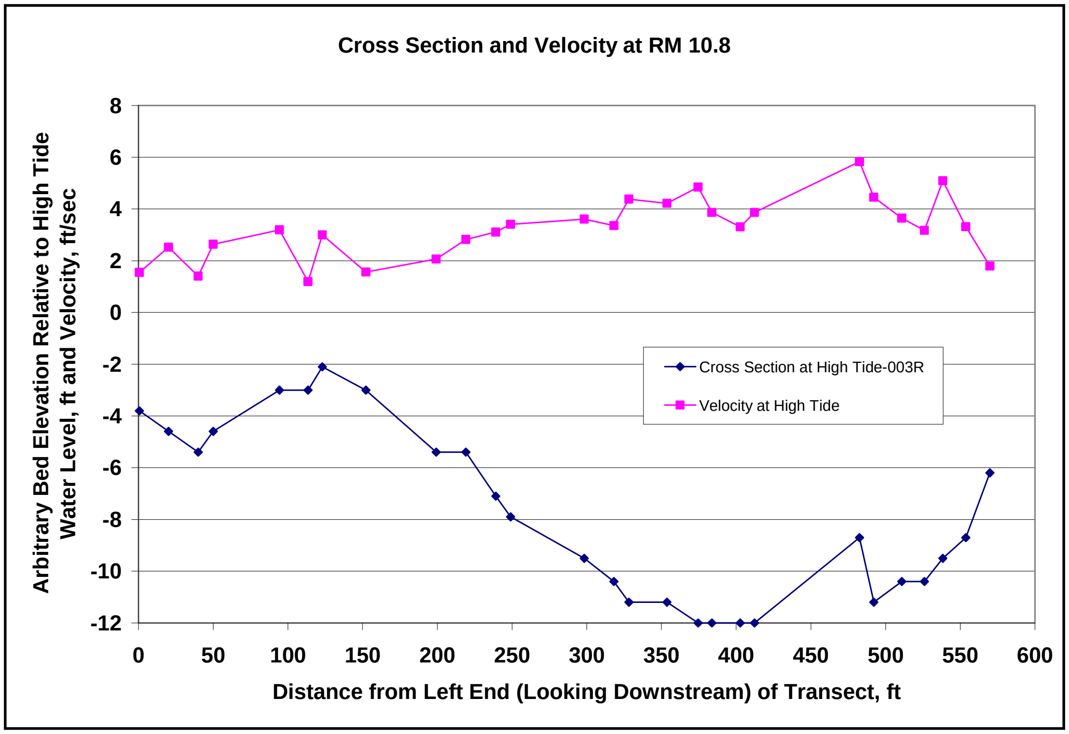


Figure A2. RM 10.8

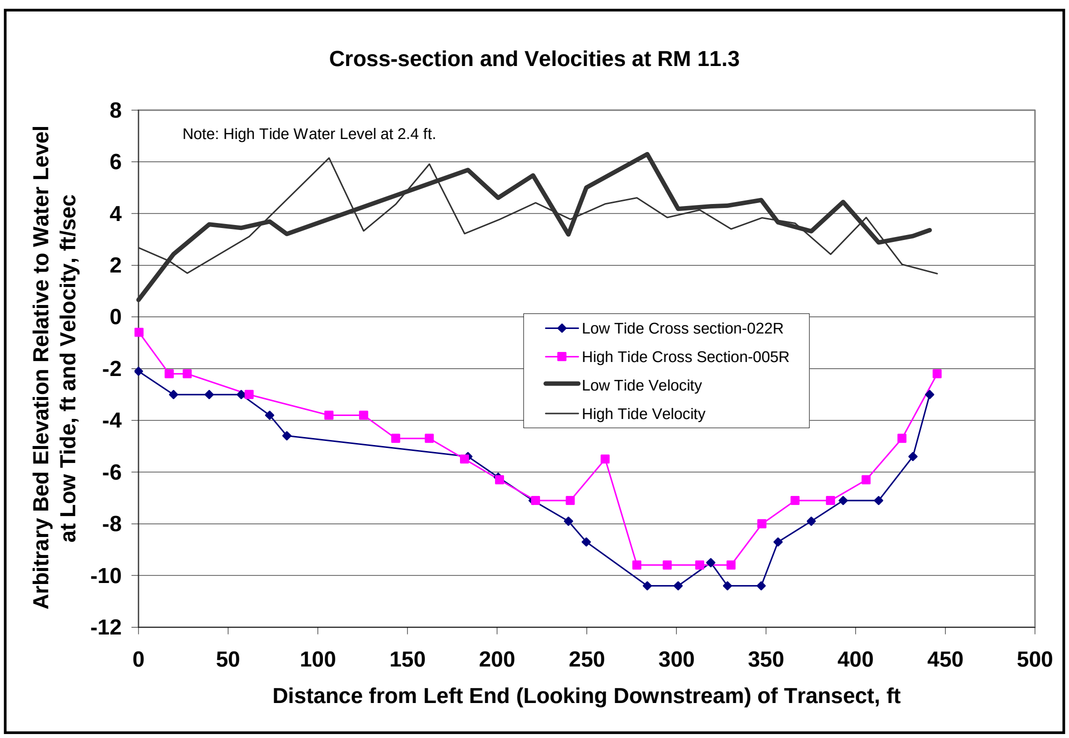


Figure A3. RM 11.3

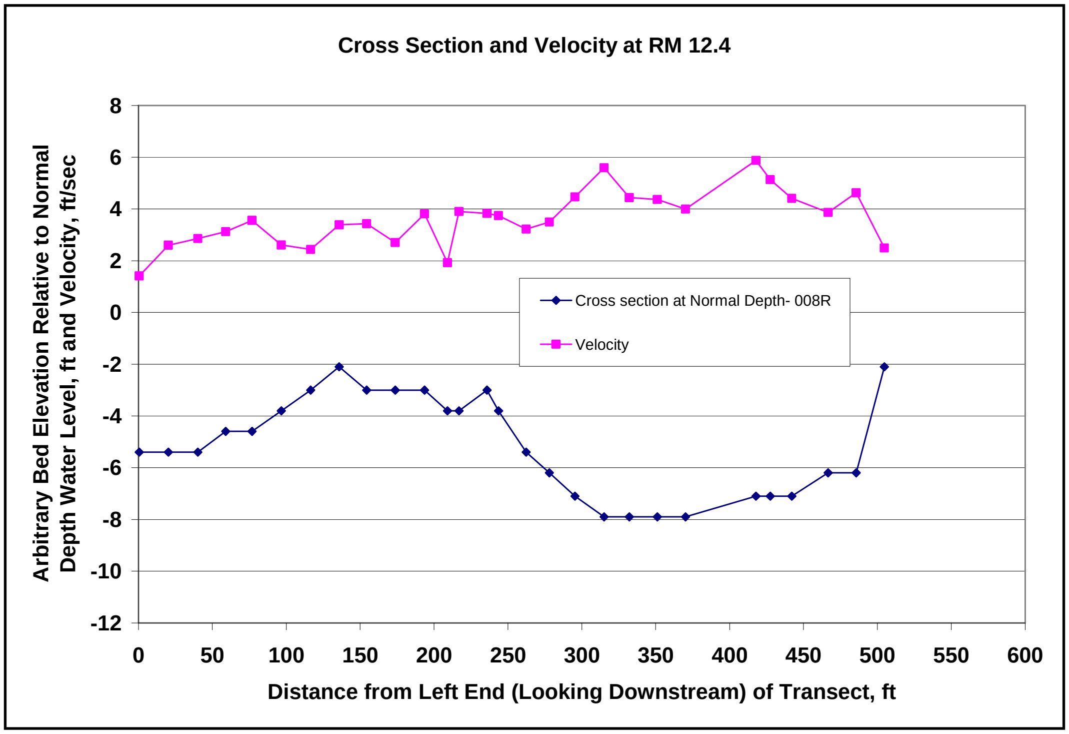


Figure A4. RM 12.4

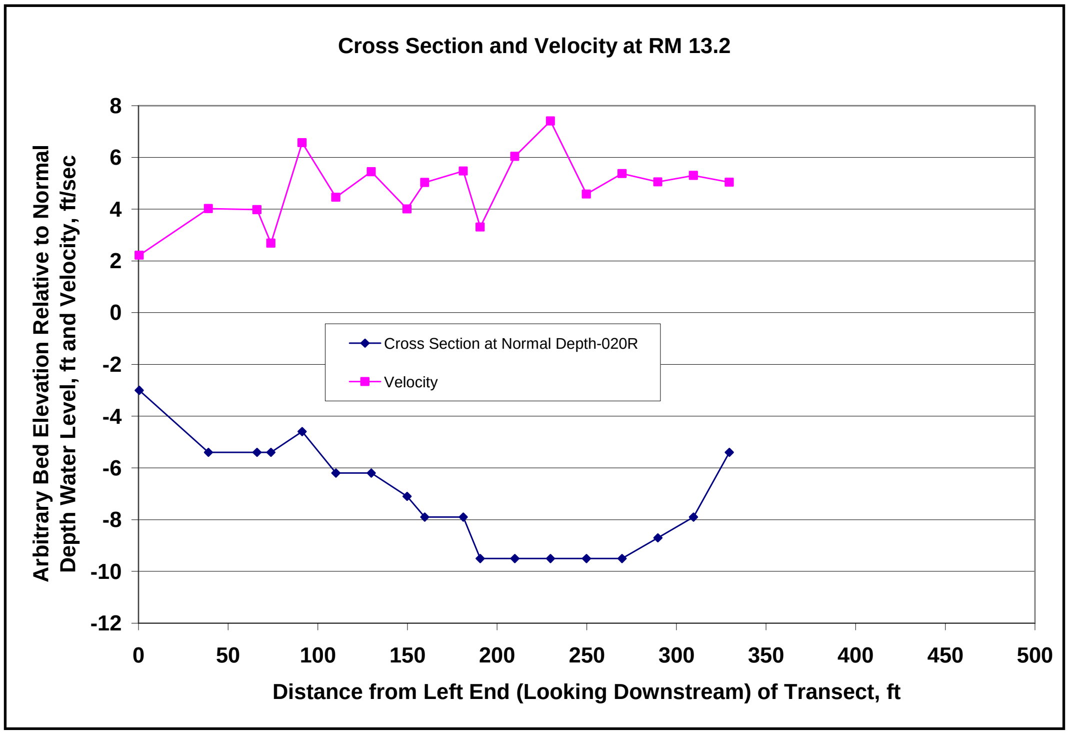


Figure A5. RM 13.2

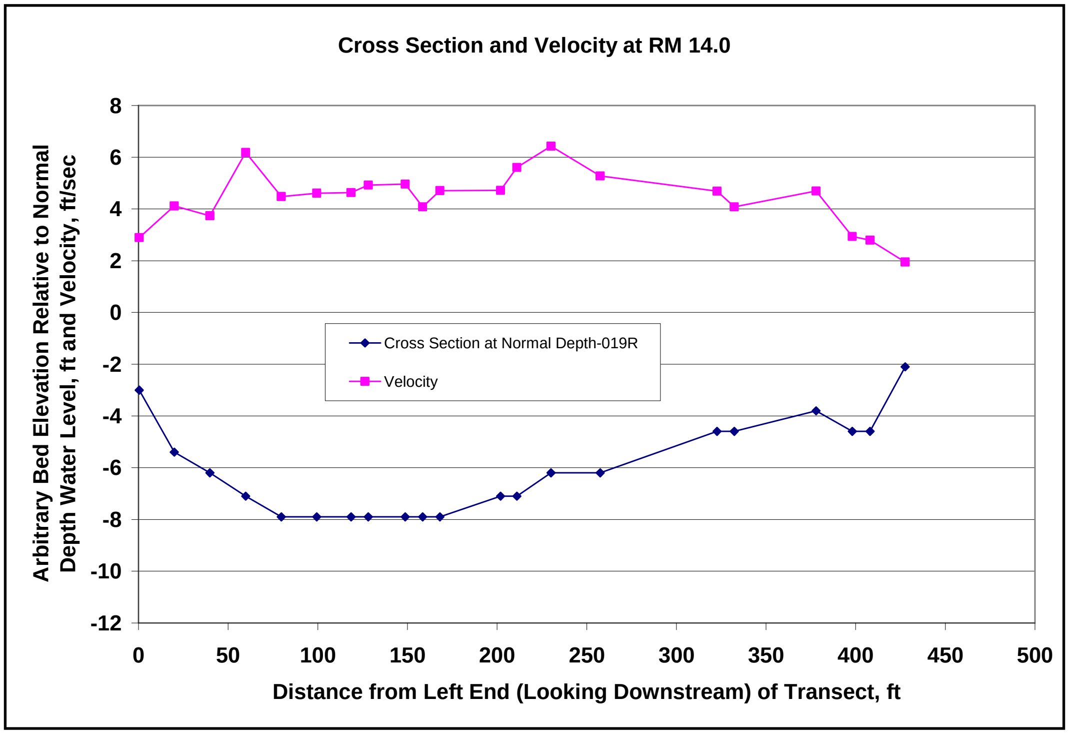


Figure A6. RM 14.0.

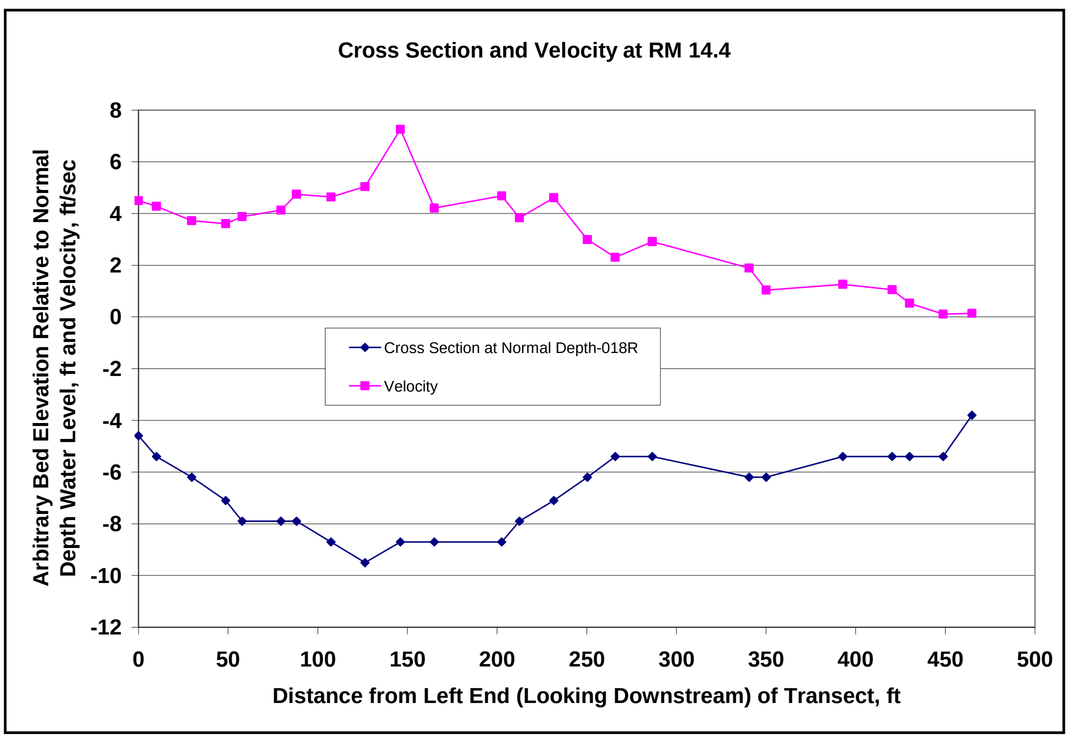


Figure A7. RM 14.4

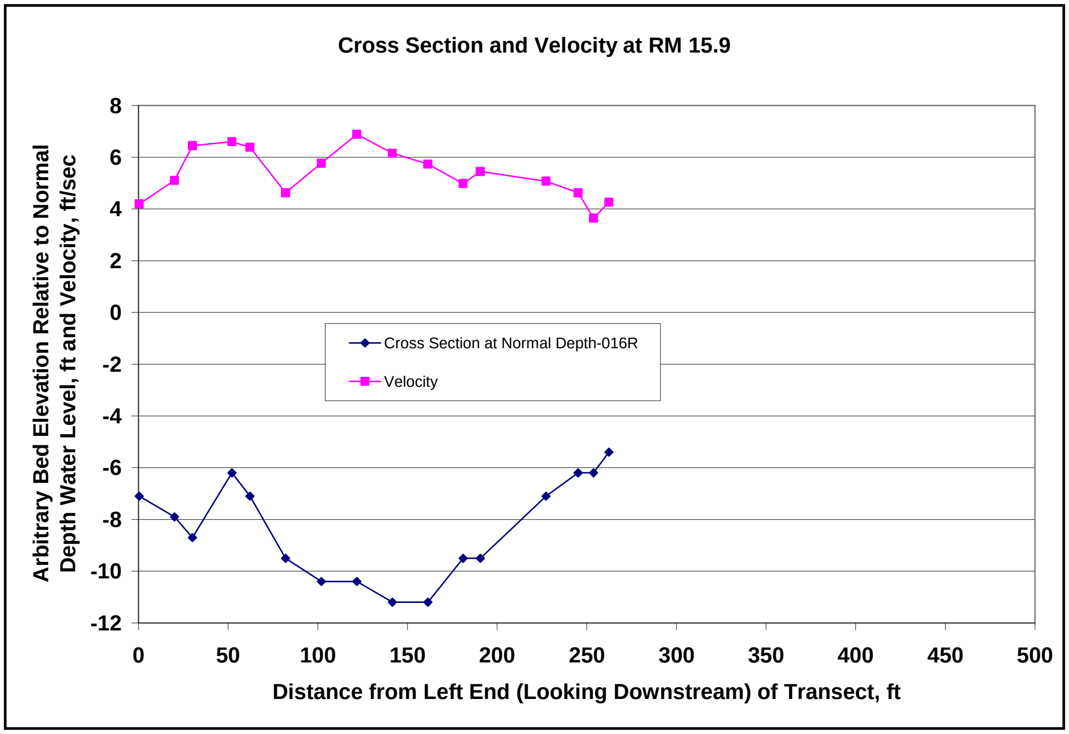


Figure A8. RM 15.9

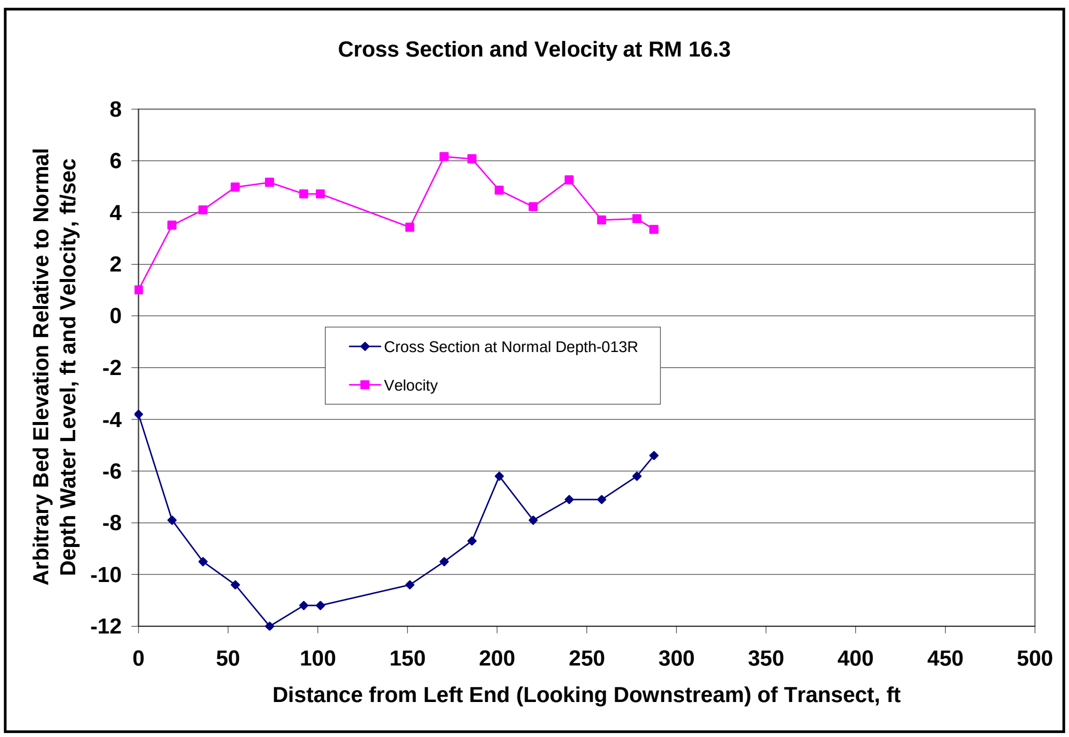


Figure A9. RM 16.3

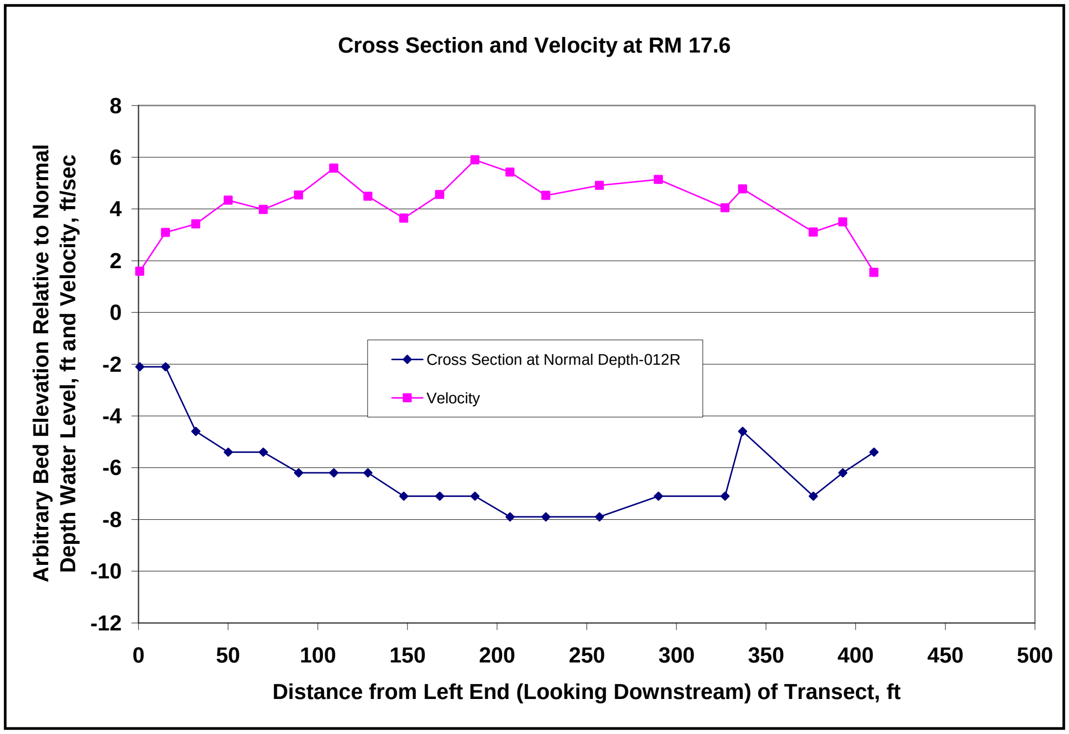


Figure A10. RM 17.6

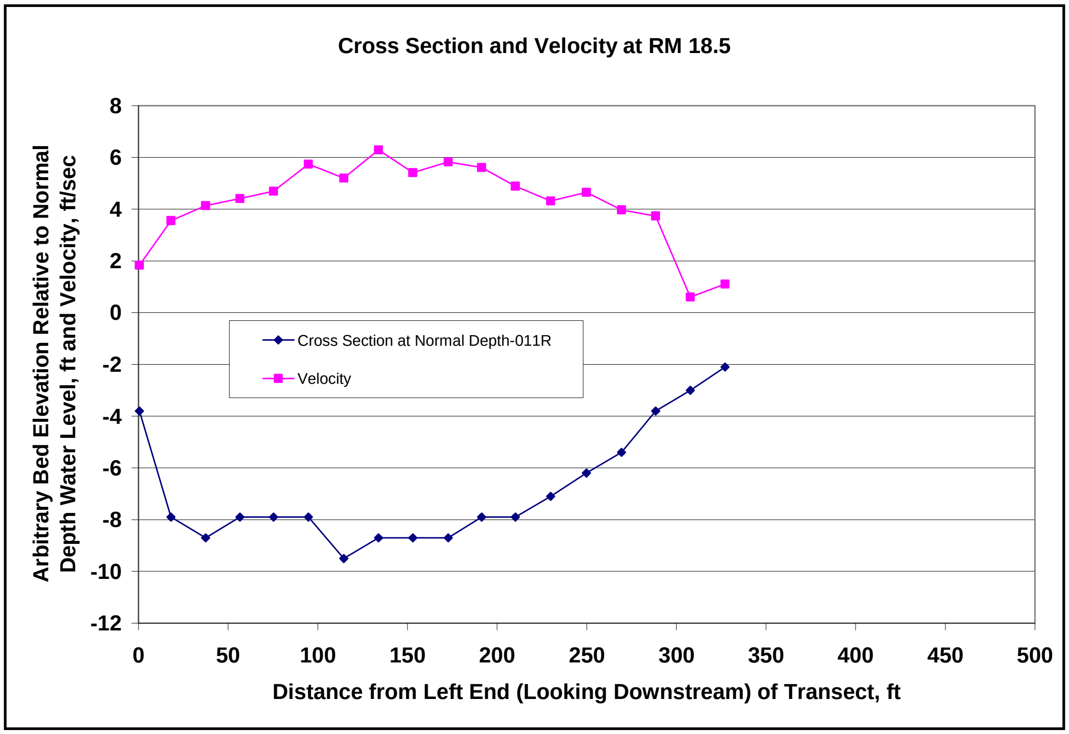


Figure A11. RM 18.5

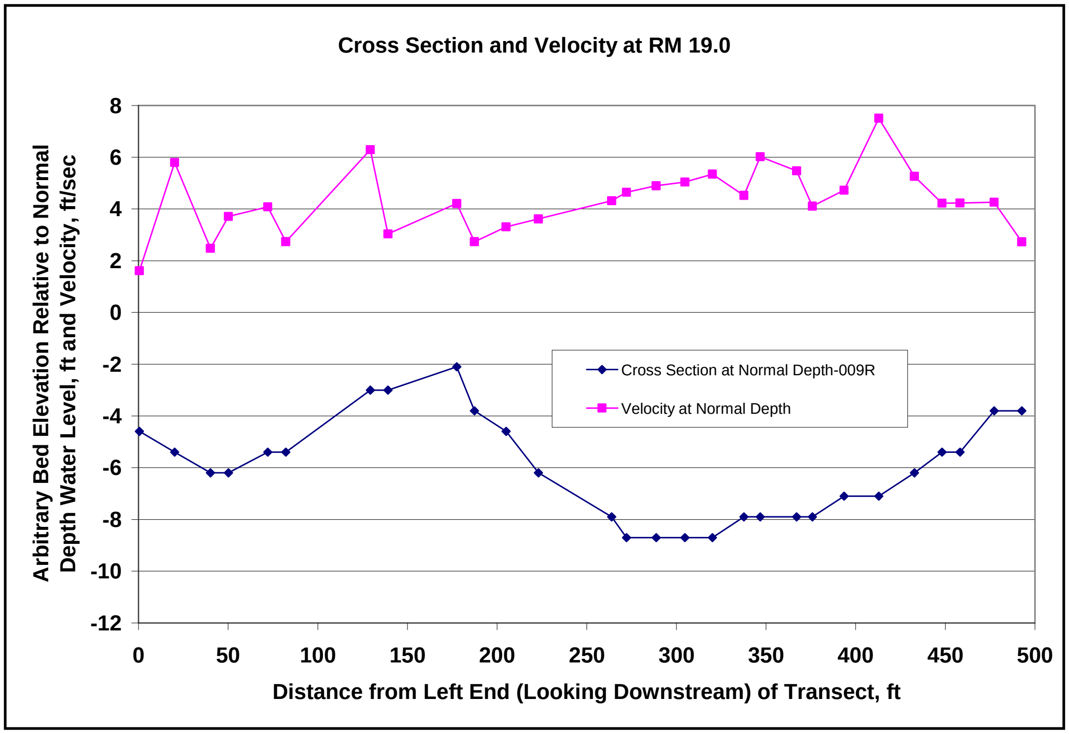


Figure A12. RM 19.0

Appendix B: Sediment Sampling Data

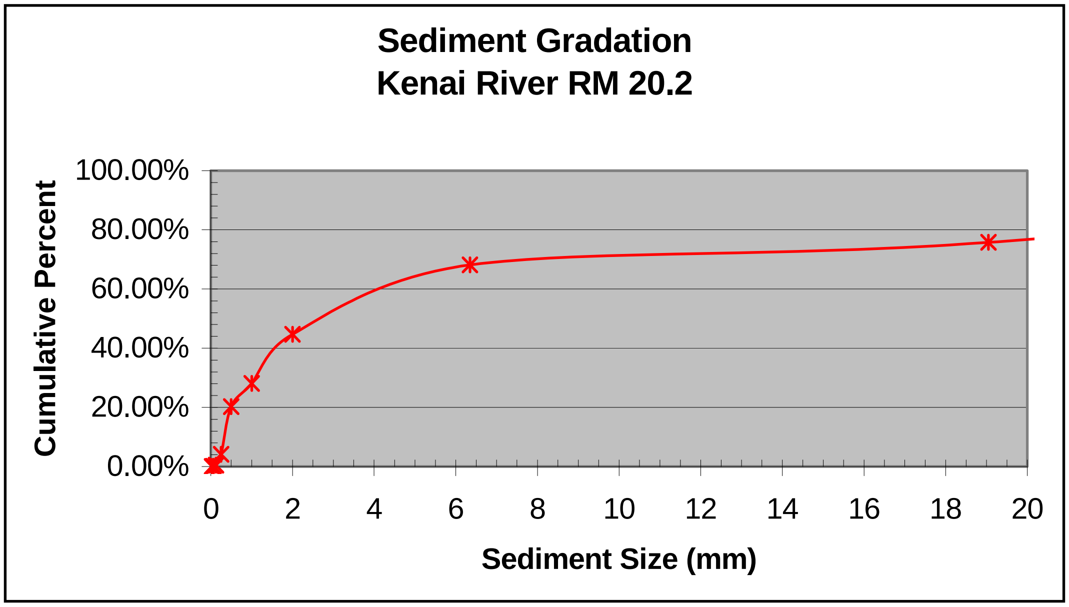


Figure B1. Sediment gradation at RM 20.2.

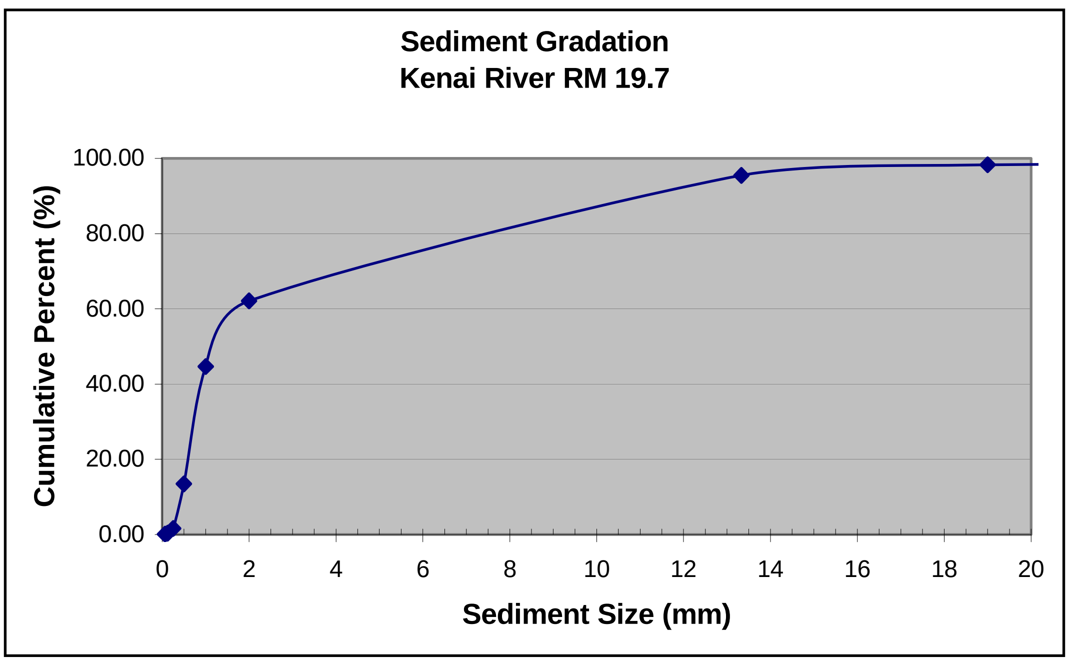


Figure B2. Sediment gradation at RM 19.7.

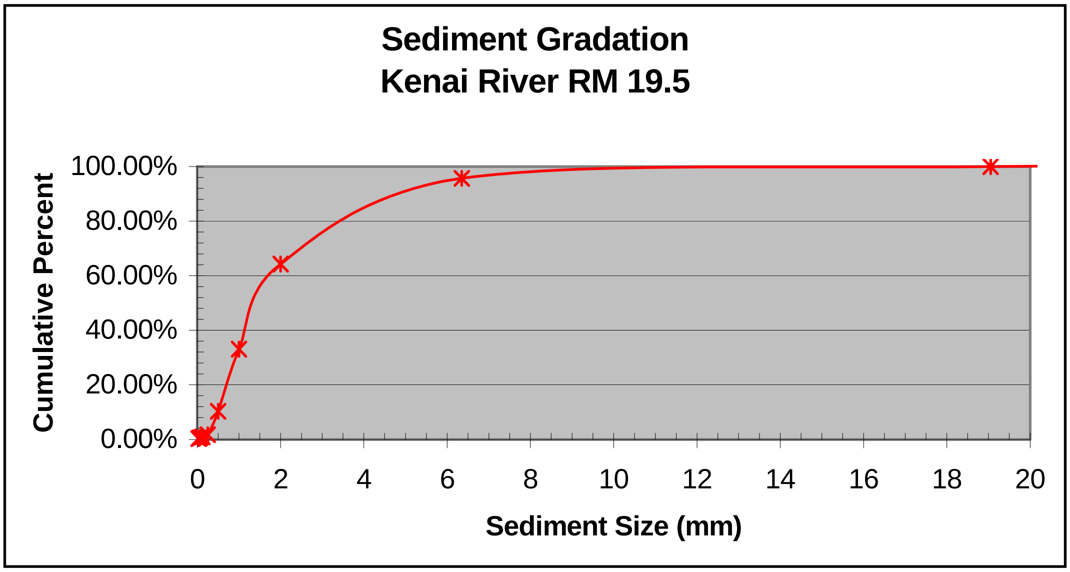


Figure B3. Sediment gradation at RM 19.5.

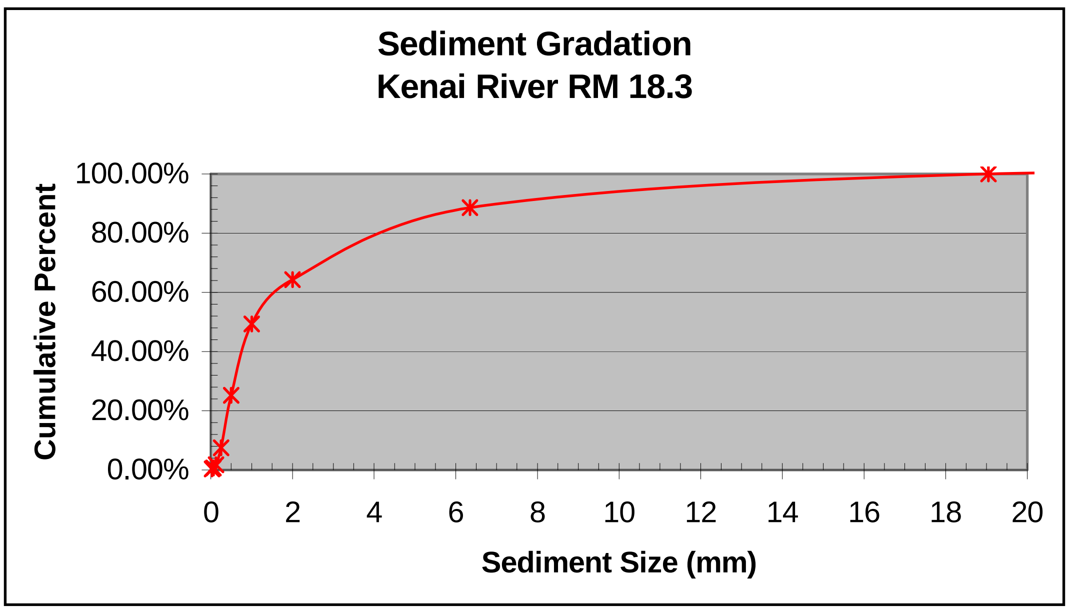
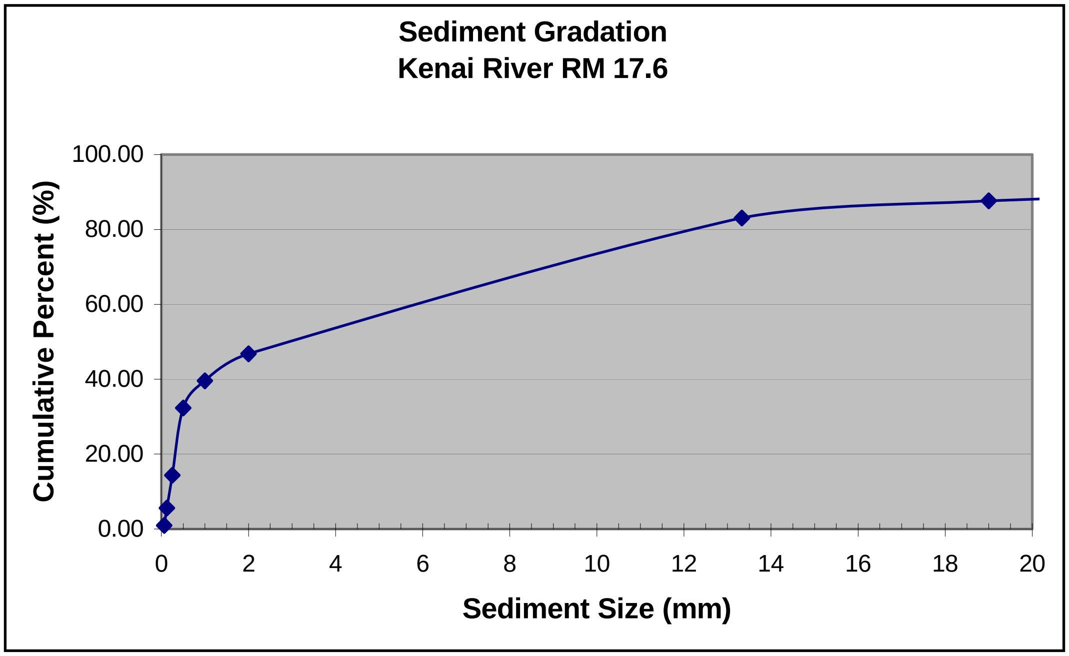


Figure B4. Sediment gradation at RM 18.3.



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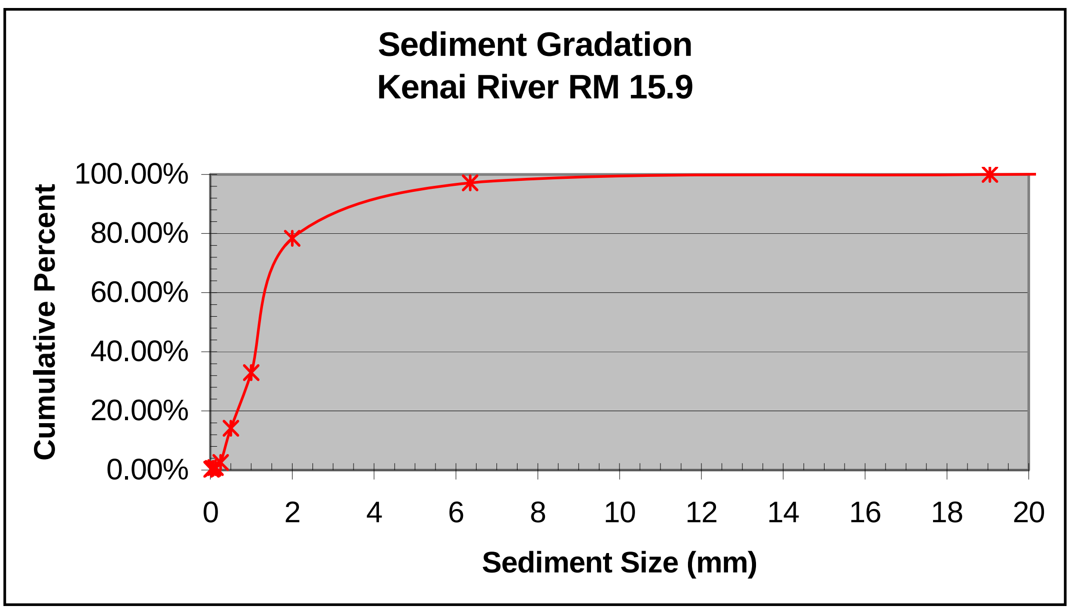


Figure B6. Sediment gradation at RM 15.9.

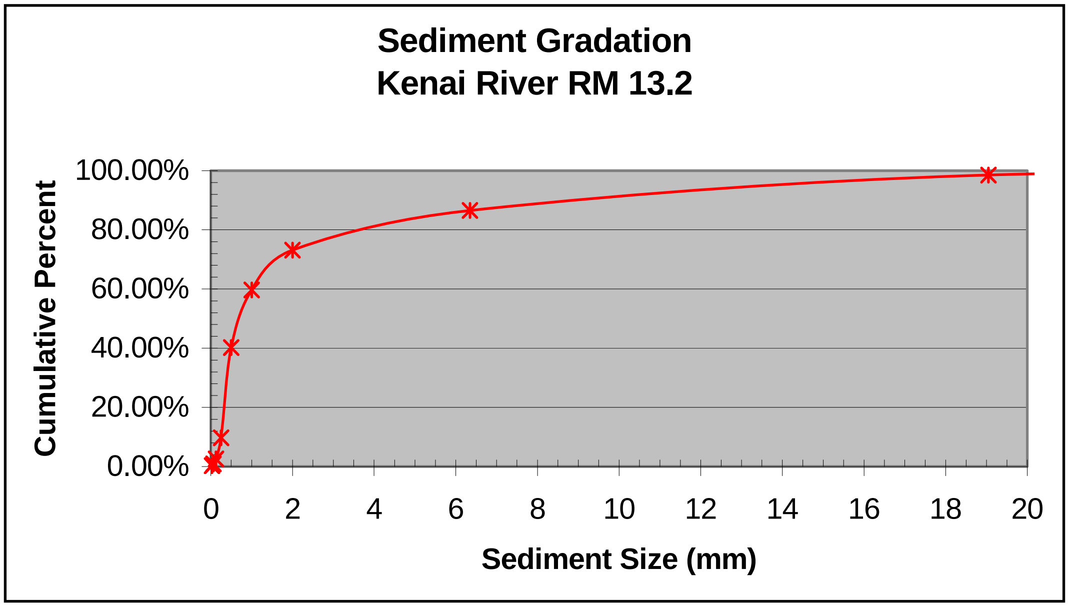


Figure B7. Sediment gradation at RM 13.2.

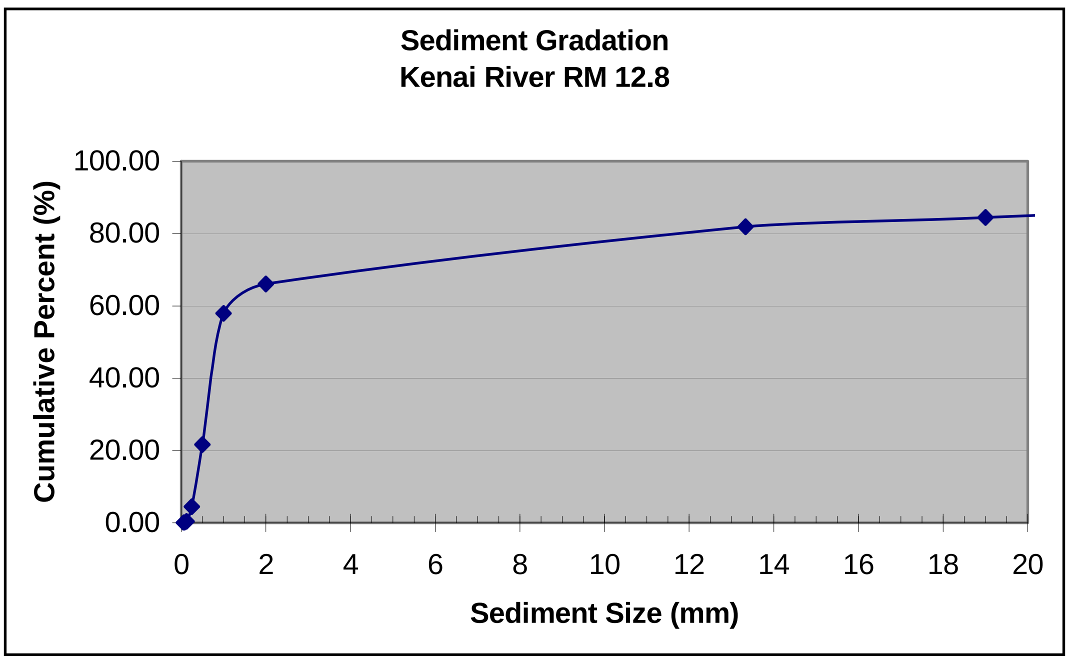


Figure B8. Sediment gradation at RM 12.8.

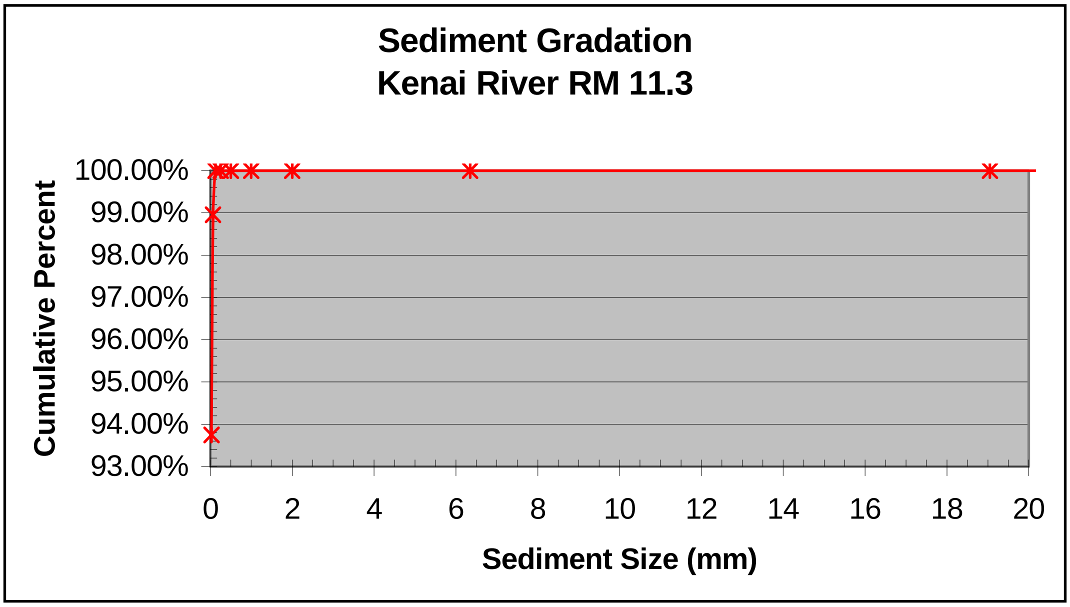


Figure B9. Sediment gradation at RM 11.3.

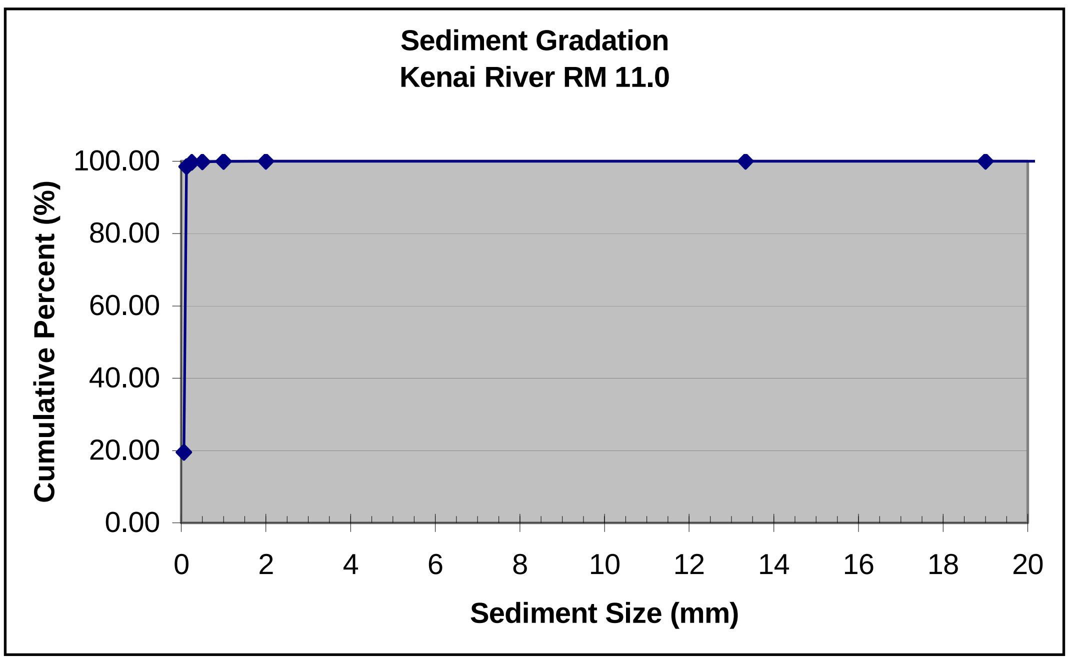


Figure B10. Sediment gradation at RM 11.

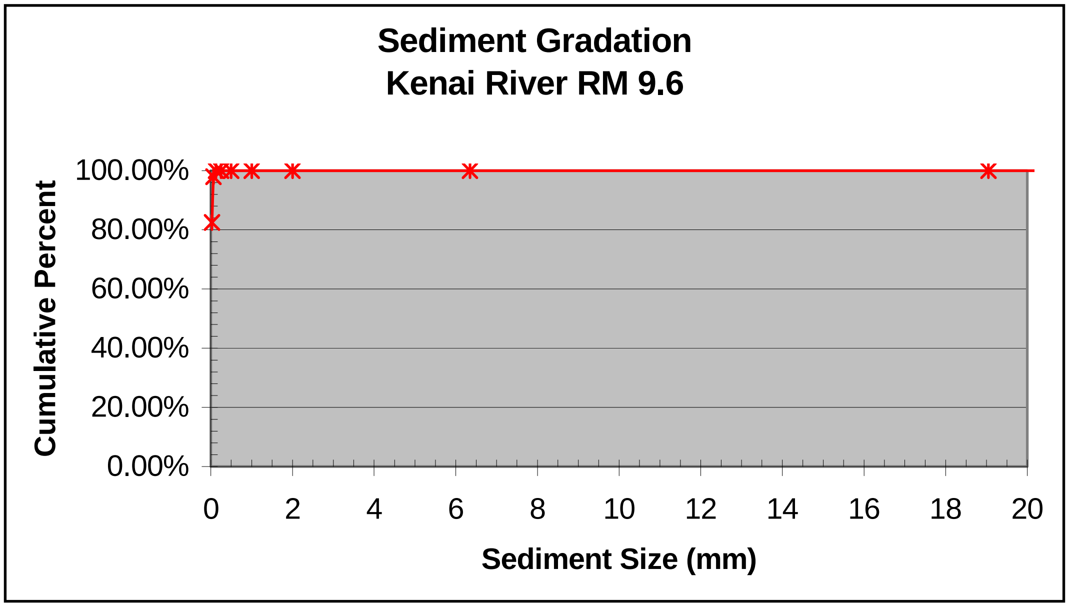


Figure B11. Sediment gradation at RM 9.6.

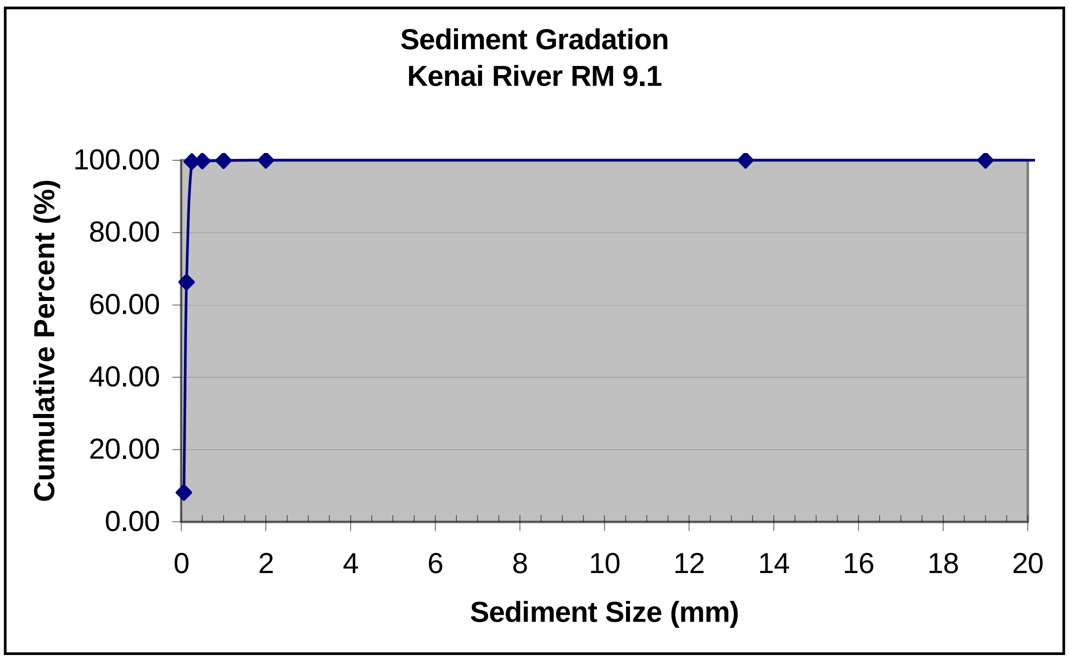


Figure B12. Sediment gradation at RM 9.1.

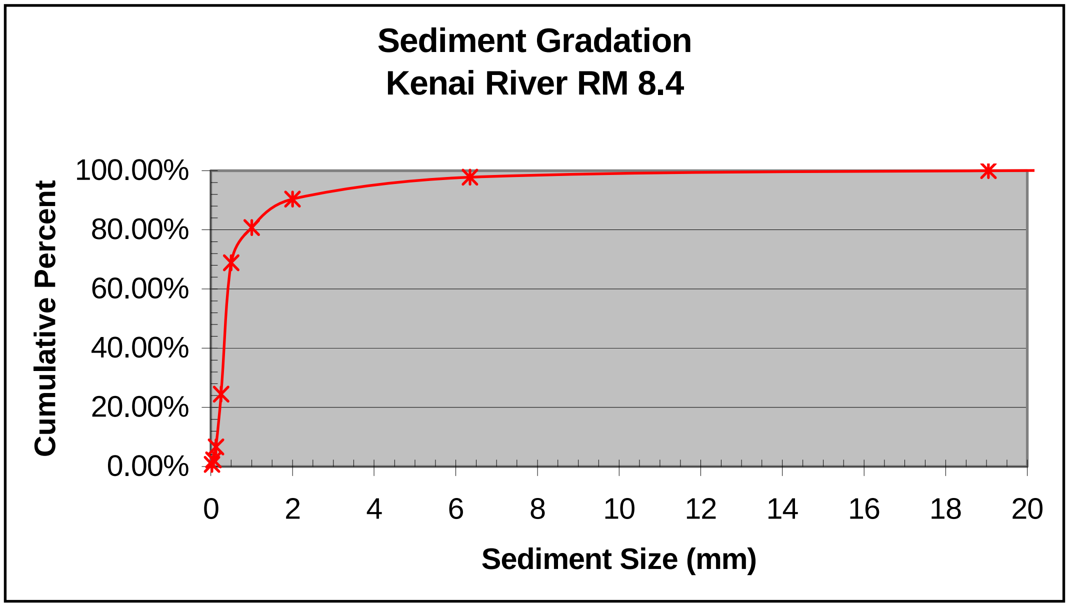


Figure B13. Sediment gradation at RM 8.4.

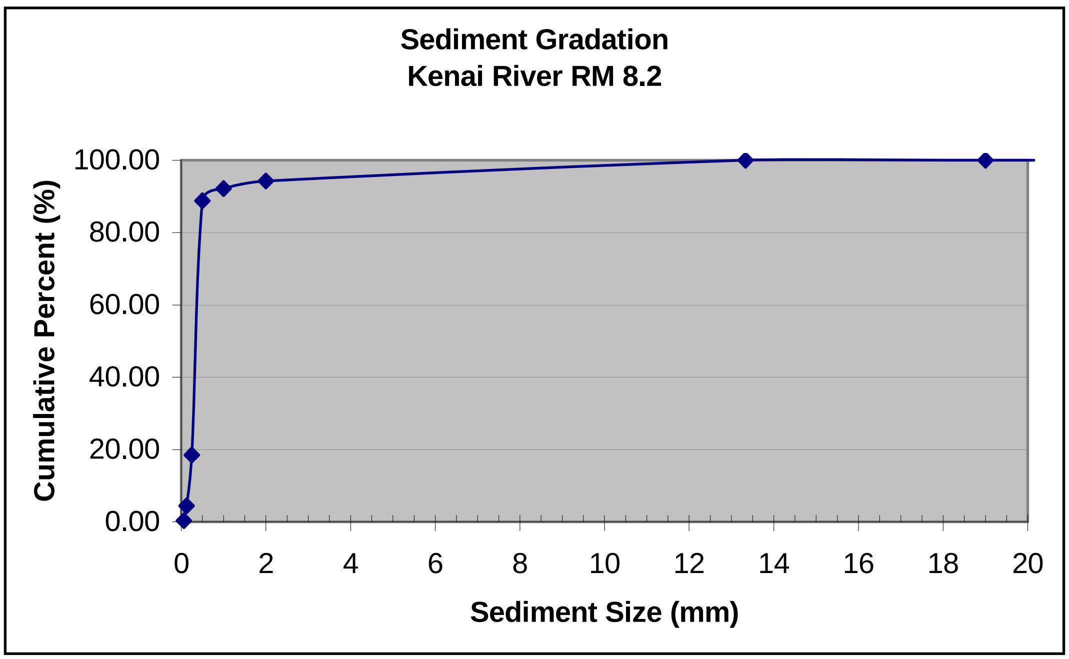
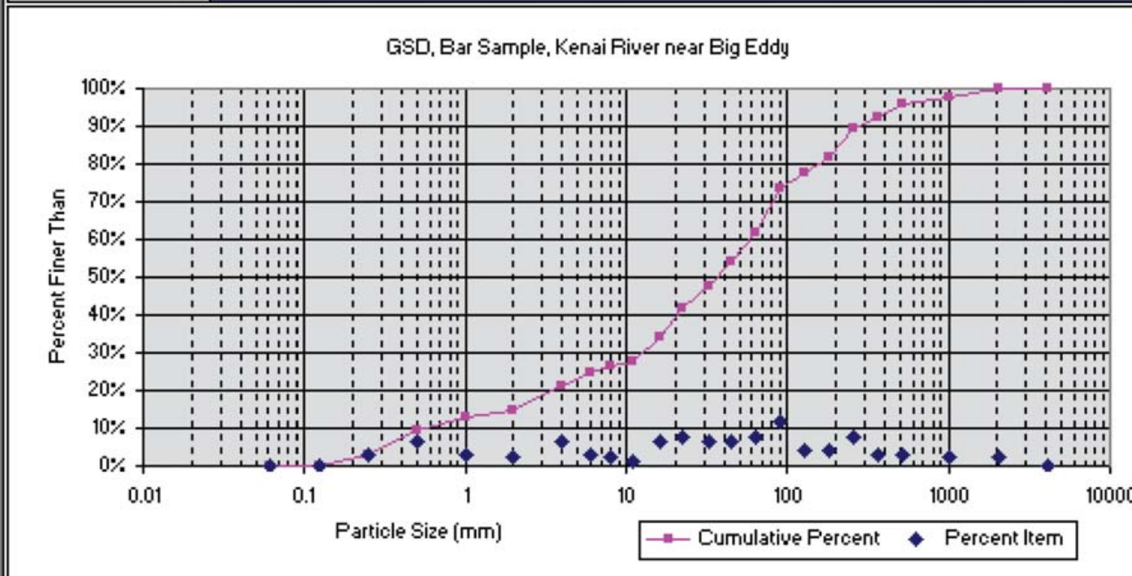


Figure B14. Sediment gradation at RM 8.2.

Pebble Count

Material	Size Range (mm)		Count
silt/clay	0	0.062	0
very fine sand	0.062	0.13	0
fine sand	0.13	0.25	3
medium sand	0.25	0.5	6
coarse sand	0.5	1	3
very coarse sand	1	2	2
very fine gravel	2	4	6
fine gravel	4	6	3
fine gravel	6	8	2
medium gravel	8	11	1
medium gravel	11	16	6
coarse gravel	16	22	7
coarse gravel	22	32	6
very coarse gravel	32	45	6
very coarse gravel	45	64	7
small cobble	64	90	11
medium cobble	90	128	4
large cobble	128	180	4
very large cobble	180	256	7
small boulder	256	362	3
small boulder	362	512	3
medium boulder	512	1024	2
large boulder	1024	2048	2
very large boulder	2048	4096	0
bedrock			0
	Total Particle Count:		94

stream:	Kenai River
	Centennial Park
	Bar Sample
Note:	



Size percent less than (mm)				
D16	D35	D50	D84	D95
2.255	16.67	35.9	199	472

Percent by substrate type					
silt/clay	sand	gravel	cobble	boulder	bedrock
0%	15%	47%	28%	11%	0%

Figure B15. Bar sample of Kenai River (Centennial Park).

Pebble Count

Material	Size Range (mm)		Count
silt/clay	0	0.062	0
very fine sand	0.062	0.13	1
fine sand	0.13	0.25	0
medium sand	0.25	0.5	5
coarse sand	0.5	1	5
very coarse sand	1	2	1
very fine gravel	2	4	2
fine gravel	4	6	3
fine gravel	6	8	1
medium gravel	8	11	1
medium gravel	11	16	3
coarse gravel	16	22	5
coarse gravel	22	32	8
very coarse gravel	32	45	6
very coarse gravel	45	64	7
small cobble	64	90	14
medium cobble	90	128	5
large cobble	128	180	3
very large cobble	180	256	6
small boulder	256	362	3
small boulder	362	512	1
medium boulder	512	1024	2
large boulder	1024	2048	1
very large boulder	2048	4096	0
bedrock			0
Total Particle Count:			83

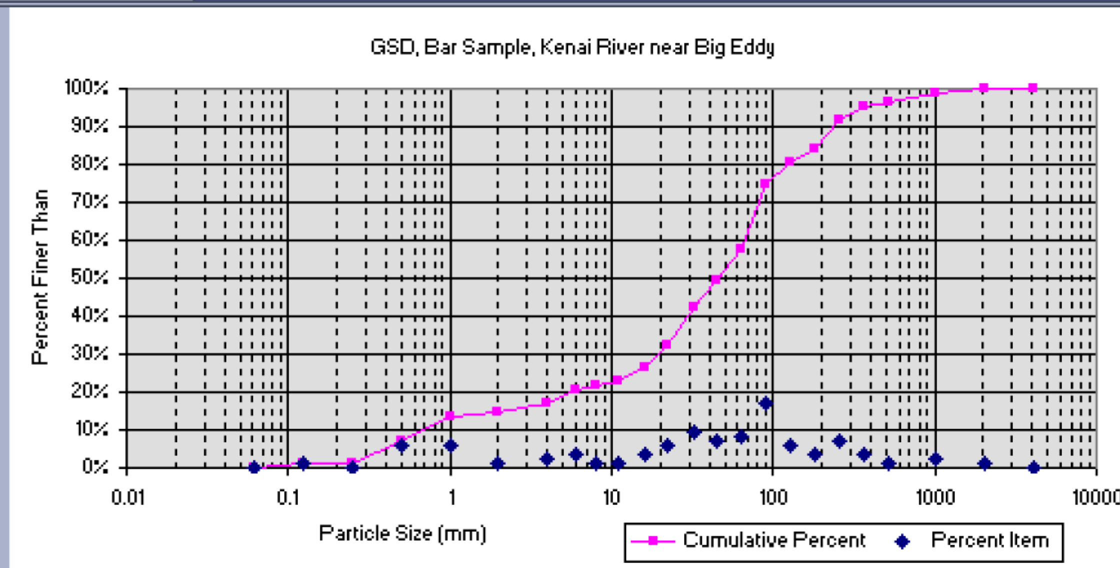
Pebble Count,

stream: **Kenai River**

Big Eddy

Bar Sample

Note:



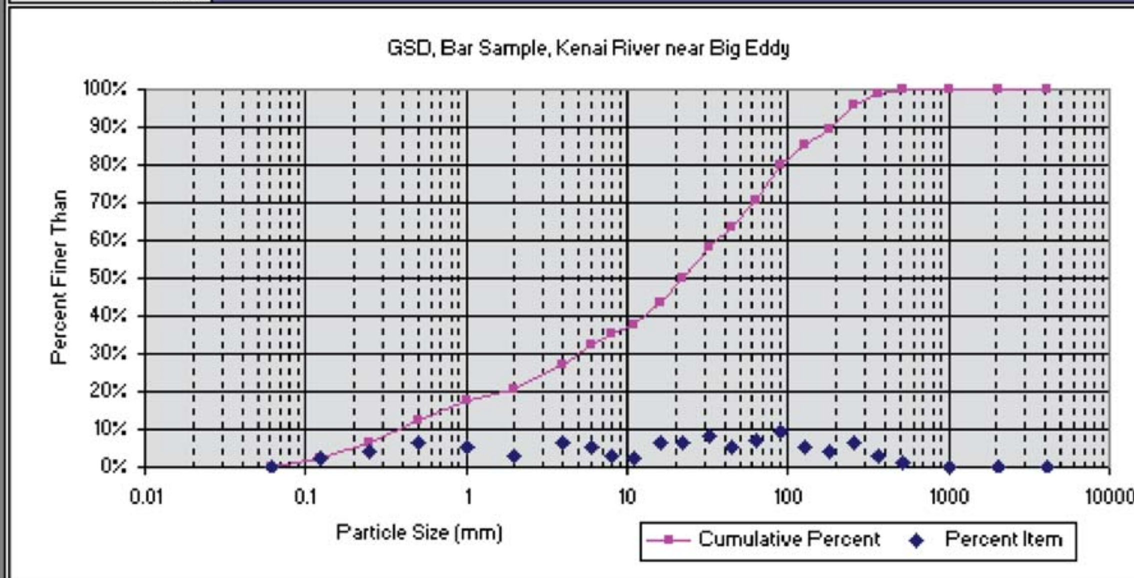
Size percent less than (mm)					Percent by substrate type					
D16	D35	D50	D84	D95	silt/clay	sand	gravel	cobble	boulder	bedrock
3.117	24.22	46.1	174	356	0%	14%	43%	34%	8%	0%

Figure B16. Bar sample of Kenai River (Big Eddy).

Pebble Count

Material	Size Range (mm)		Count
silt/clay	0	0.062	0
very fine sand	0.062	0.13	2
fine sand	0.13	0.25	4
medium sand	0.25	0.5	6
coarse sand	0.5	1	5
very coarse sand	1	2	3
very fine gravel	2	4	6
fine gravel	4	6	5
fine gravel	6	8	3
medium gravel	8	11	2
medium gravel	11	16	6
coarse gravel	16	22	6
coarse gravel	22	32	8
very coarse gravel	32	45	5
very coarse gravel	45	64	7
small cobble	64	90	9
medium cobble	90	128	5
large cobble	128	180	4
very large cobble	180	256	6
small boulder	256	362	3
small boulder	362	512	1
medium boulder	512	1024	0
large boulder	1024	2048	0
very large boulder	2048	4096	0
bedrock			0
Total Particle Count:			96

stream:	Kenai River
	Pillars
	Bar Sample
Note:	



Size percent less than (mm)					Percent by substrate type					
D16	D35	D50	D84	D95	silt/clay	sand	gravel	cobble	boulder	bedrock
0.797	7.70	22.0	116	244	0%	21%	50%	25%	4%	0%

Figure B17. Bar sample of Kenai River (Pillars).

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)*
March 2008

2. REPORT TYPE
Final report

3. DATES COVERED *(From - To)*

4. TITLE AND SUBTITLE
Boat-Wave-Induced Bank Erosion on the Kenai River, Alaska

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)
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5d. PROJECT NUMBER

5e. TASK NUMBER

5f. WORK UNIT NUMBER

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
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8. PERFORMING ORGANIZATION REPORT NUMBER
ERDC TR-08-5

9. SPONSORING / MONITORING AGENCY NAME(S) AND ADDRESS(ES)
Kenaitze Indian Tribe
P.O. Box 988, Kenai, AK 99611

10. SPONSOR/MONITOR'S ACRONYM(S)

11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES

14. ABSTRACT
The Kenaitze Indian Tribe requested that the U.S. Army Engineer Research and Development Center (ERDC) determine the relative contribution of boat-wake-induced bank erosion to total bank erosion along the Kenai River. The approach used in this study consisted of a delineation of boat wave characteristics along the study reach and a geomorphic and bank stability assessment. This analysis showed that, at specific times of the year and at specific locations, boat wave energy may be a dominant factor. However, on an average annual basis, boat wave energy is secondary to river currents in terms of total bankline recession. Reduction of boat wave energy should focus on areas having large boat passage frequency, such as the drift area at river miles 10–12 and areas where bank erosion is most problematic.
Techniques to reduce boat waves from a *single* boat include the use of flat-bottomed boats, use of 50-hp motors to increase boat speed, keeping boats away from shorelines, and reducing boat weight.
Decreased boat weight and keeping boats away from shorelines are two options that can result in benefits even when significant traffic is present. This study found that boat wakes are one of several factors contributing to bank recession. However, quantification of the relative magnitude of boat wakes to other factors such as river currents could not be determined. The results indicate that boat wakes may be a dominant factor during certain high boat usage times, discharges, and locations along the study reach. Although wake-induced erosion may be a secondary factor in bankline recession, it may be ecologically significant because of its persistence, distribution, and timing. However, bank recession associated with large flood events will likely overshadow the contribution from boat waves.

15. SUBJECT TERMS

	Bank erosion	Rivers	Waves
Boat waves	Bank recession	Streams	
Boats	Boat operation	Wave height	

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	c. THIS PAGE			19b. TELEPHONE NUMBER *(include area code)*
UNCLASSIFIED	UNCLASSIFIED	UNCLASSIFIED		148	

Standard Form 298 (Rev. 8-98)
Prescribed by ANSI Std. 239.18