Hydrology and Hydraulic Analysis of the Lowe River near the Alpine and Nordic Subdivisions, Valdez, Alaska

Final Report

A Report Prepared For:

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March 2008

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Introduction

Within the past 30 years, the City of Valdez has obtained several flood evaluations of the Lowe River, including the Alpine and Nordic subdivisions (Figure 1). These subdivisions are in a 'flood hazard high velocity zone (FIRM Zone A), and are protected from the river by a series of groins along the right bank. In October 2006, an ADOT&PF dike near Mile 12 of the Richardson Highway failed during a high water event. This breech allowed flood water to flow downstream outside of the Lowe River banks, eventually entering and flooding sections of the Alpine and Nordic subdivisions.

The City groins along the river were reported to have functioned well during the flood event, and there were no reports of flooding from the main channel of the river at that location. However, this event has resulted in renewed interest by the City of Valdez and local residents in obtaining designs and recommendations for flood management at several areas in and near Valdez. This report describes the hydrologic and hydraulic analysis of the Lowe River in the vicinity of the Alpine and Nordic subdivisions.

A number of reports that describe previous studies and surveys of the Lowe River and the flooding issues at the Alpine and Nordic subdivision have been prepared over the past 30 years. Many of these documents were reviewed as part of this project. A partial list of pertinent documents is found in the Bibliography.

Assumptions For Hydrologic Analysis

It is important to note that hydrologic studies are based to a large extent on methods in statistics and probability. Though methods are improving, the long-term forecast of streamflows and river behavior cannot be predicted with much certainty. Additionally, the use of mathematical equations to simulate and predict real events and processes is a difficult process. Unforeseen events, natural or human-caused, can alter the outcome of a modeled prediction.

The Lowe River is a dynamic river; watershed characteristics such as glaciers, large precipitation events, and high sediment loads all combine to make the task of flood analysis very difficult. Of special note is the fact that many areas of Alpine and Nordic subdivisions are at elevations lower than the thalweg of the adjacent main channel. The proper use of the hydrologic analysis in this report will involve developing solutions for a range of flood elevations and magnitudes, rather than focusing on a single result.

Planning for additional flood protection should consider risks that fall outside of the traditional 100-year flood study but may lead to flooding within the subdivisions. Risks could include sediment deposition in the adjacent channel, unexpected upstream dike failure, culvert blockage, severe channel migration, and others. Recurrence intervals for such events could be difficult to assign, leading to a qualitative assessment rather than a quantitative or probabilistic solution.

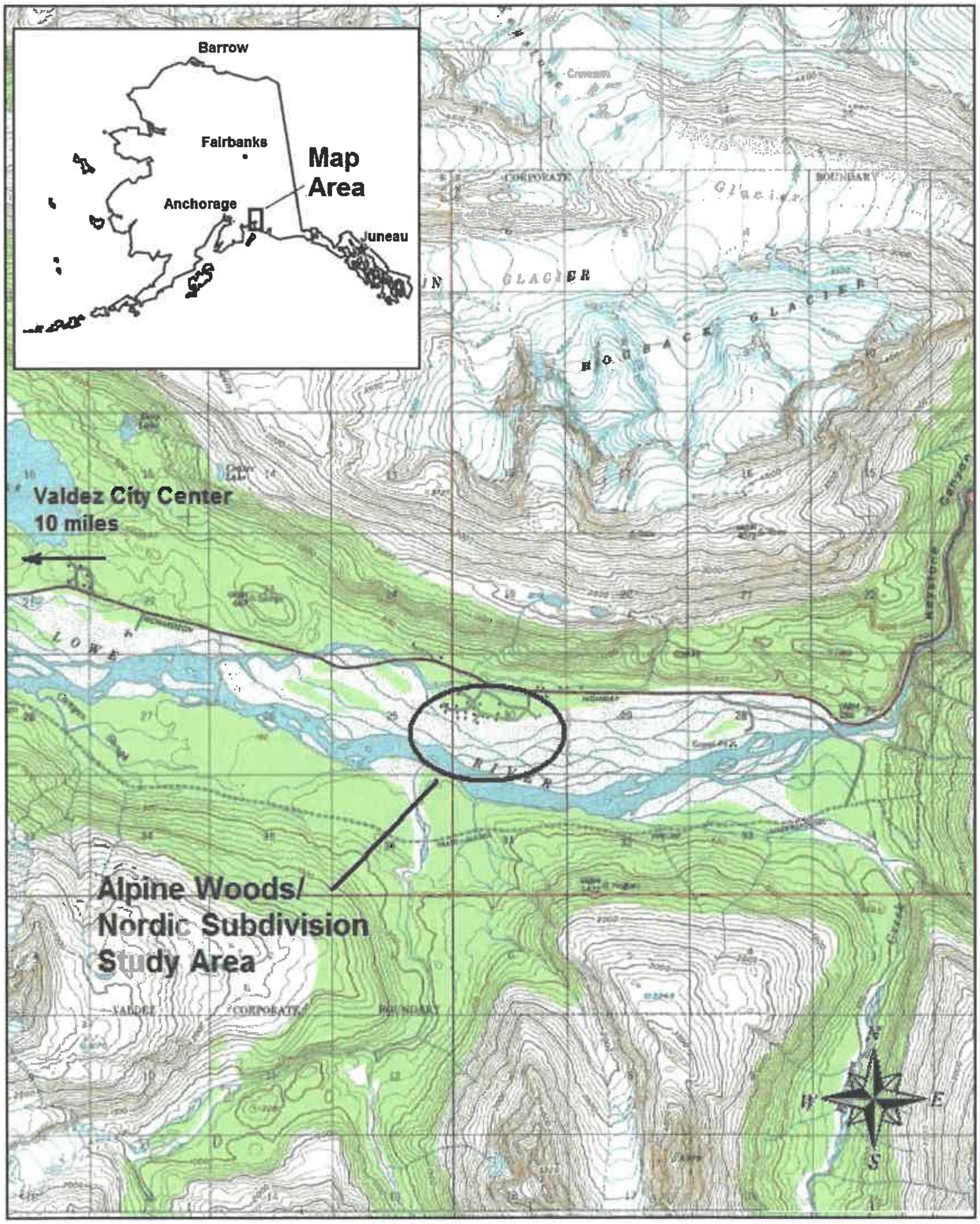


Figure 1. Project location map for Lowe River near Valdez, Alaska.

Hydraulic History

Typical of most of Alaska, little information is available concerning historical floods on the Lowe River. An inactive USGS gaging station (15226600) with a drainage area of 222 square miles is located on the Lowe River in Keystone Canyon, approximately 6 miles upstream from the project site at the Alpine Subdivision (drainage area 328.5 square miles). Six years of peak flow data are available for that site, including two historical floods (isolated high-magnitude peaks that occurred outside the period of systematic data collection).

In addition to a 1995 peak flood of 18,700 cfs, the USGS estimated a maximum peak flow of 42,000 cfs during the October 2006 flooding event. The magnitude of the flood peak was determined by surveying cross-sections through the channel and floodplain immediately following the flood event, noting the high water elevations, and using the slope-area method to determine discharge (David Meyer, USGS, personal communication). The USGS has labeled that event as having a recurrence interval of greater than 100 years.

Hydrology

Since the available peak flow record is so short as to be below the minimum necessary to develop flood estimations based on probability analysis alone, the following methods were used. First, flood magnitude estimations were developed using USGS regression equations for estimating the magnitude of peak streamflows in Alaska, using methods described in Curran et al. (2003). Then a statistical flood-frequency analysis of the annual-maximum peak flows was developed using a log-Pearson Type III probability distribution and the USGS Bulletin 17B methods (USGS, 2006). Using results from the first two methods, a third estimate of weighted values was developed, where weights are based on the years of observed data at the station and the average equivalent years of record for the regional regression equations. Finally, the weighted estimate was adjusted for the larger drainage area at the downstream project site (Curran et al., 2003). Details are found in Appendix 1.

The estimated flood frequency magnitudes for the Q2 through Q500 floods for both sites are shown in Table 1 and Figure 2.

Table 1. Flood magnitudes for Q2 through Q500 flood, Lowe River at project site.

Exceedance Probability (%)	Recurrence Interval (Years)	Discharge (cfs)	
		Lowe River at Keystone Canyon Gaging Station	Lowe River at Alpine Subdivision
50	2	10200	12300
20	5	13200	17000
10	10	15400	20400
4	25	18700	24800
2	50	21300	28300
1	100	24000	31900
0.2	500	31300	40800

It should be noted that these estimated flood magnitude values are somewhat smaller than those used for the Alpine Woods Estates Flood Evaluation conducted in 1983 (WCC, 1983). In that report, the flood values used were derived from a study to review the 1980 Valdez FEMA flood insurance study. For the Lowe River at the confluence with Port Valdez, with a drainage area of 350 square miles, the estimated flood magnitudes were: Q10-31,100 cfs, Q50-46,300 cfs, Q100-54,900 cfs, and Q500-77,500 cfs. These values were calculated for a slightly larger watershed, using regression equations. Additionally, a regression equation was used to estimate the magnitude of glacier dammed-lake releases, based on lake volume. The lake volume data collected for that study were obtained in the late 1970s and have not been updated.

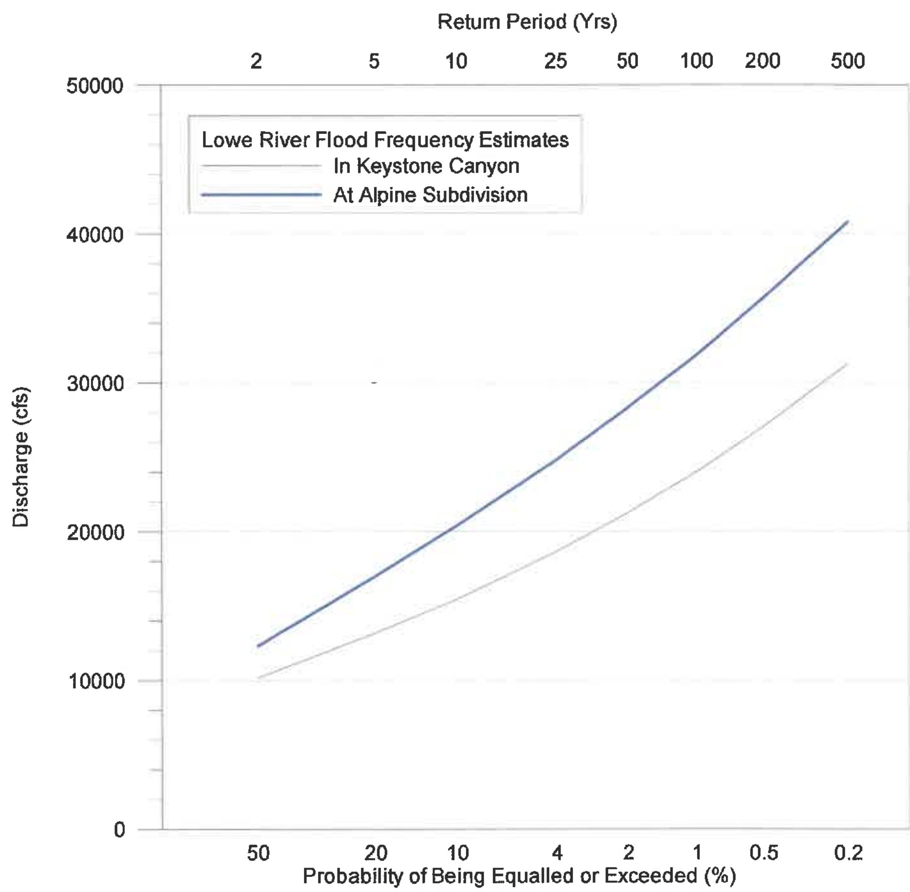


Figure 2. Estimated discharge and exceedance probability for Lowe River.

Hydraulic Modeling

<u>Overview</u>

The hydraulic analysis for the Lowe River at Alpine Subdivision project site consisted of modeling the flow characteristics using the U.S. Army Corps of Engineers Hydrologic Engineering Center water surface profiling computer program HEC-RAS version 3.1.3 for the existing conditions, including the two City of Valdez groins and a newer temporary dike installed immediately following the October 2006 flood event. The basic computational procedure for the HEC-RAS program is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion. The momentum equation is utilized in situations where the water surface profile is rapidly varied, such as at bridges (USACE, 1998).

Numeric models of study sites are created using stream geometric data. Once the models are constructed and calibrated, estimations of channel velocities and stage are calculated for each cross-section for a range of discharges. The hydraulic analysis is used to develop a map showing flooding extents for the 1-percent-annual-chance (100-year) event. However, regulatory agencies often will require determinations of the 10-percent-annual-chance (10-year), 2-percent-annual-chance (50-year), and 0.2-percent-annual-chance (500-year) flood discharges as well.

Calibration

The cross-section conditions present at a typical braided river site present many unique computational problems for numerical modeling efforts. At low and intermediate flows, the occurrence of flowing water in any of the many channels spaced across the wide braided drainage course appears often as a randomized process. In fact, channels with a higher thalweg elevation may contain significant flow while lower channels on the same section are often dry. Such an effect has been noted by local residents familiar with flow conditions on the Lowe River.

Such conditions cannot be replicated in a numerical model, where hydraulic calculations assume flowing water initiates at the lowest point in a cross-section. This results in a situation where a numerical model cannot be properly calibrated at low flow, even using observed discharges and water surface elevations in numerous channels across the section. The use of hydraulic models for braided rivers is significantly improved for high flood flows, where high water conditions inundate the smaller channels and the mass of flow is essentially contained in one or two major channels. The HEC-RAS program is widely used and accepted in particular for floodplain management and flood insurance studies to evaluate floodway encroachments. However, results from such modeling efforts should be used in conjunction with on-the-ground observations from persons familiar with the river.

A numerical model of the Lowe River at the project site was constructed in HEC-RAS. Nine major cross-sections, labeled 1.0 through 9.0, were surveyed in November 2007; the survey was refined with additional points in December 2007. Techniques were employed to increase the functional stability of the hydraulic analysis process; this is accomplished by increasing the number of cross-sections in the model by interpolating new cross-sections between the surveyed cross-sections. Within the model, some cross-sections were adjusted to insure perpendicularity to the flow. Cross-section locations are found in Figure 3, and Alaska State Plane coordinates (Northing, Easting) for the major cross-section endpoints are found in Appendix 4. The elevation benchmark for the cross-section survey is USC&GS benchmark number E11C964, which was also used for the development of the FEMA FIRM map. See Appendix 5.

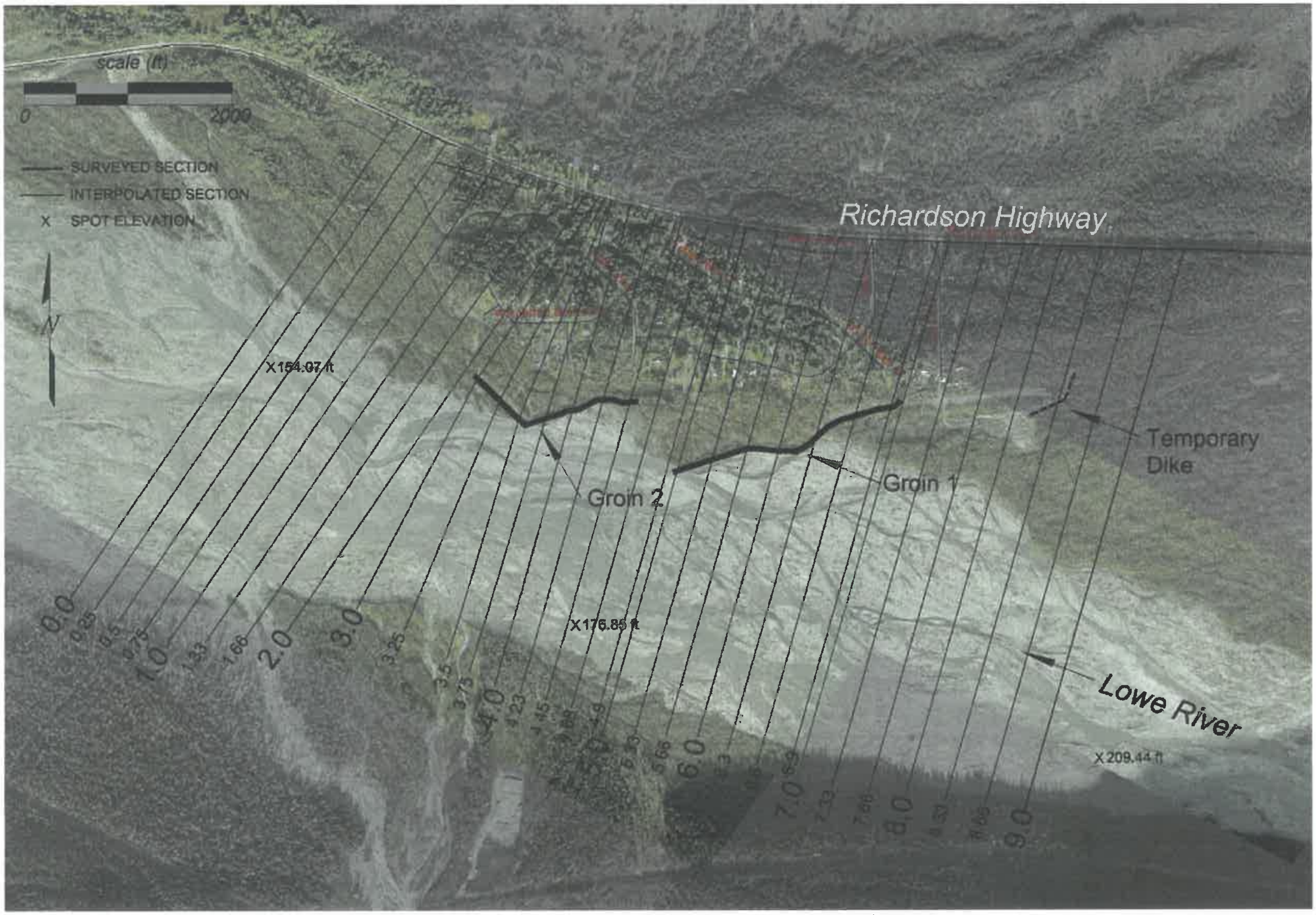


Figure 3. Aerial photograph of project site, with HEC-RAS cross-sections and existing groin locations.

Manning 'n' values were selected for this study based on engineering judgment, published values, and a sensitivity analysis. They were subsequently adjusted during the calibration process. The values used are listed in Table 2.

Table 2. Selected Manning's n values.

Condition	Manning's n
Channels and active floodplain	0.035-0.040
Light vegetation on floodplain	0.10-0.12
Heavy vegetation on floodplain	0.12-0.15

The Lowe River HEC-RAS model was calibrated using data from the October 2006 flood event. The USGS reports that the estimated peak discharge at the Lowe River gaging station in Keystone Canyon is 42,000 cfs. Using standard techniques, this value was adjusted to account for the additional area that drains to the project site downstream of the gaging station. The estimated Lowe River peak discharge at the Alpine Subdivision is 48,650 cfs.

Observers reported a water surface elevation during the October 2006 flood event of approximately 1.5 feet below the top of the levee near Cross-section 6, or 189.5 feet. The model was calibrated to this elevation at the 48,650 cfs flow.

Model Results

Flood calculations were done for the 2-year through 500-year flows. Water surface elevations at selected cross-sections are presented in Table 3, along with top-of-groin elevations at those sections that cross groins.

Table 3. Water surface elevations from HEC-RAS analysis.

Cross-section	Water surface elevation for design floods (feet)				Top of groin elevation (feet) and groin #
	2-year	50-year	100-year	500-year	
0.0	153.31	154.29	154.45	154.81	
1.0	159.53	160.54	160.71	161.10	
2.0	164.50	165.61	165.80	166.16	
3.0	166.66	167.69	167.87	168.27	172.37 (#2)
3.5	170.93	171.89	**172.07***	**172.47**	174.50 (#2)
4.0	174.46	175.53	175.70	176.11	180.51 (#2)
4.45	176.95	178.25	**178.43**	**178.83**	180.50 (#2)
4.90	179.32	180.63	180.82	181.43	
5.0	181.12	182.53	182.73	183.04	186.21 (#1)
6.0	186.89	188.41	**188.61**	**189.03**	191.01 (#1)
6.9	192.12	193.54	193.75	194.22	197.44 (#1)
7.0	192.65	194.06	194.29	194.85	
8.0	199.36	200.89	201.09	201.46	
8.33	202.58	203.77	203.95	**204.40**	207.00 (temp dike)
9.0	208.20	209.21	209.37	209.74	

*Water surface elevations that are less than three feet from the top of groin elevation are noted in bold.

The approximate extent of floodplain inundation for the 100-year flow is mapped in Figure 4. Some variations may be expected in the flood extents during actual flooding conditions, due to river processes and modeling limitations. The major cross-sections, with water surface elevations plotted for the 2-year, 10-year, 100-year, 500-year, and September 1995 floods, are found in Appendix 2.

Discussion

In the cross-section alignments used for this study, the far right sections of the floodplain, especially in the sections that pass through the Alpine and Nordic subdivisions, contain elevations that are lower than the thalweg of the main channel. This is especially true of a drainage channel that runs adjacent to the south side of the Richardson Highway. Other channels that run through the subdivisions are either intermittent or spring-fed. The HEC-RAS model was adjusted through the use of ineffective flow areas to keep the flood flows out of these lower far-right sections, unless the higher elevation points between them and the main channel were first overtopped. This is a feature in HEC-RAS that defines areas of the cross-section that are not part of the active flow area. Ineffective flow areas are denoted in the Appendix cross-sections with cross-hatching.

The extent of flood inundation through the study reach appears to be controlled by several different features along the right bank of the Lowe River in the study area. In the upper section of this reach (Cross-sections 7.0 to 9.0) a high broad ridge 2 to 4 feet in elevation above the bank level confines the flow to the center channel and adjacent floodplains. From Cross-section 7.0 to 8.0, the gravel pit also acts to limit flood extents on the right floodplain. Water entering the upstream end of the pit will drain back into the river at the lower end. Across from the lower end of the pit, an intermittent channel is located between the west end of the air strip and the subdivision. Modeling indicates that the channel's left bank at Cross-section 7.0 may be overtopped by several inches of water during the 100-year flood, introducing flood water into the channel.

Two City groin structures are located along the right bank from Cross-section 6.9 to 3.0. Though these structures were constructed to prevent channel migration, they also act to limit the right extent of flow. Water surface elevations at the design flood generally stay at two feet or more below the top of the groin elevations for the 100-year flood, and slightly less for the 500-year flood.

A gap between Groin 1 and Groin 2 does allow some water to flow laterally through the gap toward the subdivision at flood flows. Though difficult to model exactly with HEC-RAS, local observers have stated that flood water flowing between the two groins does not flow into the subdivisions, but is diverted by an old gravel pit/pond, and flows west through small drainage channels. These features are apparent on the aerial photographs.

Downstream from Cross-section 3 and the lower end of Groin 2, the design flow is generally contained on the right floodplain by the bank, and by a narrow band of ground at a slightly higher elevation of 1 to 2 feet. Again, this analysis is confirmed by local observers who reported that the vegetated floodplain downstream of and to the immediate

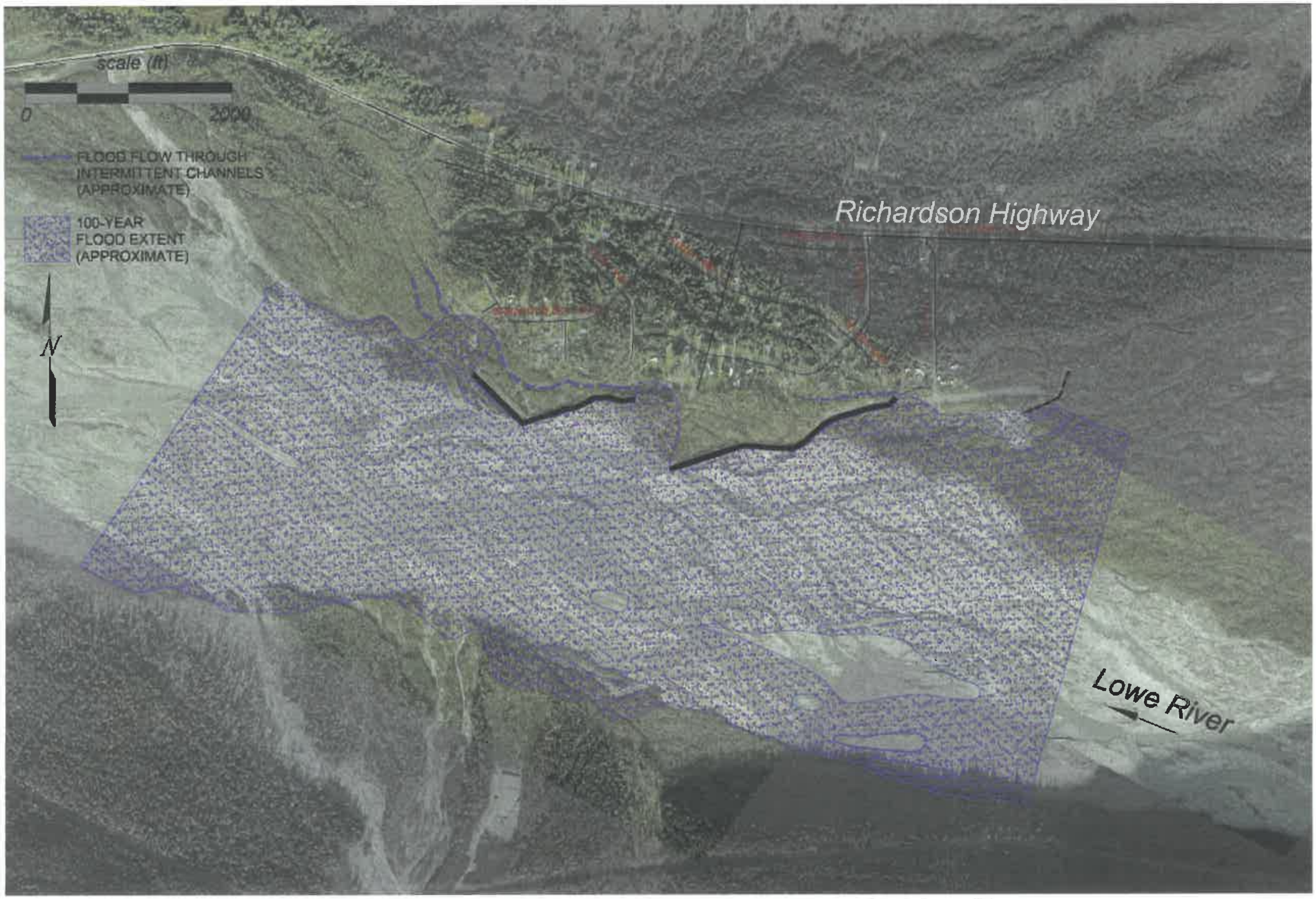


Figure 4. HEC-RAS modeled extent of flooding for the 100-year flood, Lowe River.

west of the subdivisions was not inundated during the October 2006 flood, though smaller channels through that floodplain were at or above capacity.

The HEC-RAS model shows that on several of these downstream cross-sections, the high point on the right side of the floodplain is overtopped during the 500-year flood, with inundation occurring to the Richardson Highway. However, this analysis is somewhat uncertain, as it conflicts with reports from observers during the October 2006 flood. No large floodplain inundation or highway overtopping was reported in this section.

It is important to note that HEC-RAS is a one-dimensional model, and topographic variations on the ground that were not captured during the cross-section surveys can cause inaccuracies during the flood analysis and mapping. Unmapped high areas can act to constrict flows, while unmapped areas can be inundated by backwater.

Additional Flood Protection

Results from the HEC-RAS analysis indicate that the Alpine and Nordic subdivisions are generally outside of the modeled 100-year floodplain. However, physical topography can be altered before or during a flood, which may result in flooding extents that are different than the modeled results. For example, the cause of subdivision flooding during the October 2006 event was a breeched dike upstream, which allowed water to travel down the right floodplain out of the Lowe River right bank. Random events during extreme floods, such as dike or culvert failures and highway embankment breaches, were not modeled for the development of the 100-year flood plain map.

The Lowe River is a large braided river with coarse bed material, and flows in several dividing and uniting, relatively wide and shallow channels. The primary causes of braiding are an abundant sediment load, large and sudden discharge variations, erodible banks, and a steep gradient. Such conditions can readily lead to lateral channel migrations, and flooding in areas that are generally dry.

The HEC-RAS results should be used to assess existing flood protection structures, and help guide the design of additional flood protection measures for the Alpine and Nordic subdivisions. Several suggestions for designs and improvements to existing structures are found below:

Existing Structures-Top Elevations

The profile graph in Figure 5 shows the water surface elevation of the river at the 100-year and 500-year flood levels, along with the top elevations of the two City of Valdez groins. Some sections of the two city groins and the new temporary dike do not meet a 3 foot minimum freeboard requirement for the 100-year flood elevation, as described in Section 65.10 of the FEMA National Flood Insurance Program (NFIP) regulations (FEMA, 2003).

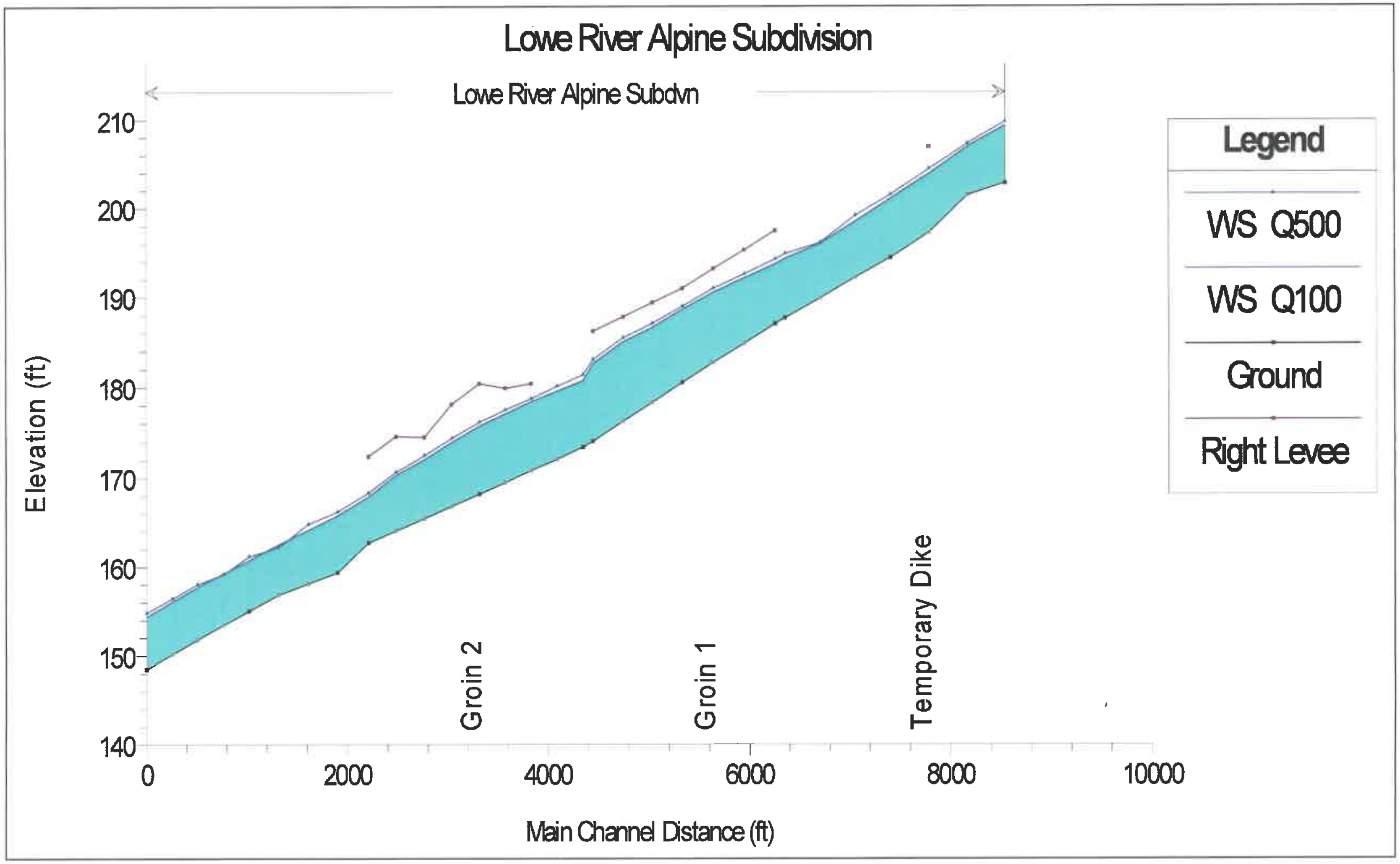


Figure 5. Profile of the 100-year and 500-year flood elevations.

New designs for groin improvements or extensions may include increasing the top elevations to meet the minimum FEMA freeboard requirements, especially if new levee design/certification is conducted by a registered professional engineer, rather than a federal agency. If the U.S. Army Corps of Engineers is the certifying agency, FEMA allows the use of a risk based analysis to design levee crest heights as an alternative to the 3 feet of freeboard above the 100-year flood.

Existing Structures-Riprap Facing

The riprap used to face and protect the existing groins should be evaluated to determine whether the proper size and gradation of rock was used. The factors that determine riprap size include water velocity, water depth, and bank angle.

Average velocities at each cross-section are calculated as part of the HEC-RAS hydraulic analysis. For the 100-year flood, average velocities are found for each of the cross-sections that are located at a groin or dike in Table 4.

Table 4. Average channel velocities for the Q100 at cross-sections crossing groins.

Groin 1		Groin 2		Temporary Dike	
Cross-section	Average Velocity (ft/sec)	Cross-section	Average Velocity (ft/sec)	Cross-section	Average Velocity (ft/sec)
5	6.4	3.0	6.6	8.33	5.1
5.33	5.1	3.25	5.7		
5.66	5.6	3.5	6.0		
6.0	5.5	3.75	5.3		
6.3	4.7	4.0	4.8		
6.6	4.5	4.25	4.6		
		4.45	5.0		
		4.68	2.0		

In addition to the average velocities, HEC-RAS has an option that allows users to plot estimated velocity distributions across a cross-section. It is important to note that these estimated velocities are based on the results of a one-dimensional hydraulic model, and that true velocity and flow distribution varies vertically as well as horizontally. The velocity distribution at the 100-year flood for Cross-section 5 is found in Figure 6. At this section, the maximum estimated velocity of 11 feet per second is at the groin.

Existing Structures-Culverts

At least one 36 inch culvert is installed on the City Groin #1, and provides hydraulic connectivity between the Lowe River and the protected side (Figure 7). Such culverts should be removed or plugged to reduce flows into the protected area during floods.

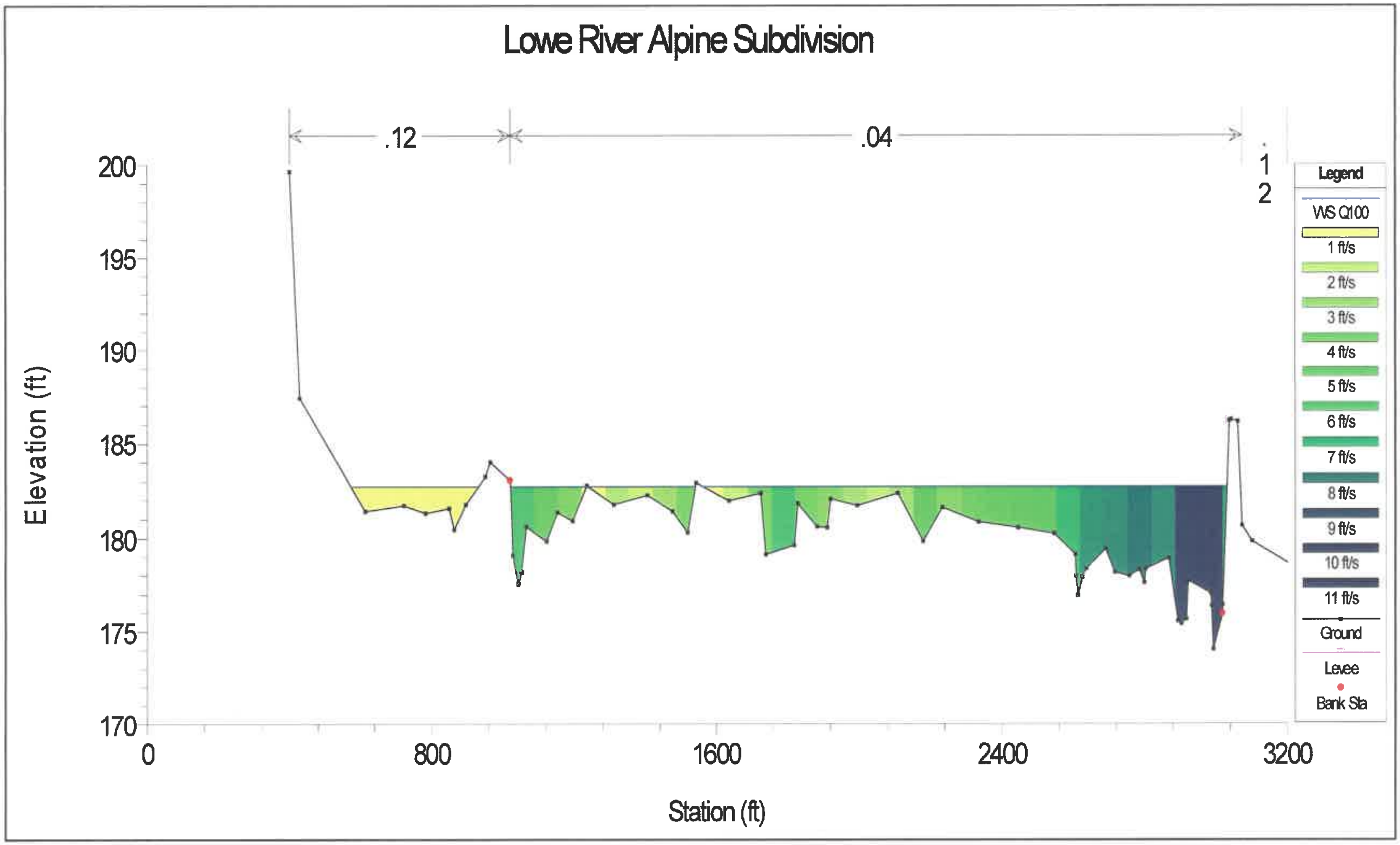


Figure 6. Velocity distribution across the channel at Cross-section 5.

Figure 7. Culvert in Groin 1 connects Lowe River to protected area.

Upstream of Existing City Structures

The HEC-RAS analysis shows that in the upper study area, the 100-year inundation extents from a river flood event are controlled by a high broad ridge approximately halfway between the right river bank and the Richardson Highway. Additionally, flow coming from east to west on the right floodplain will be intercepted by the new temporary dike and diverted into the large gravel pit adjacent to the river.

For flood events such as the October 2006 flood, additional protection may be obtained by several methods. These include extending a new dike to the west and north of the subdivisions, to capture and train flow traveling outside of the right bank down the floodplain. In lieu of constructing a new groin or dike, the existing temporary dike might be utilized by increasing its top height and length, and improving drainage from the lower end of the gravel pit to the river. However, methods and materials used to construct the temporary dike may not meet standard specifications or design requirements. A qualified inspection will be required to determine the competency of the temporary dike as part of a permanent flood protection structure.

A large intermittent stream flows into the subdivisions from the east. This channel is observed in Figure 3 just north of the temporary dike. Local residents report that it is generally groundwater-fed and flows only certain times of the year. Any new levee structure that may be extended north to the high point of the right floodplain will cross this channel and require some sort of closure device. The closure device, which allows channel flow during normal operations, is a movable and essentially watertight barrier, and would be used in flood periods to close an opening in the levee, securing but not increasing the levee design level of protection.

Several earlier flood control concept plans included a dike or ring levee that wraps around the entire subdivisions, terminating at the road (Engles and Engles, 2007). The hydraulic analyses for those design features were not available for review during this project. However, the existing HEC-RAS analysis (this report), field observations that note lack of channels or historic flood activity, and observations from residents during the October 2006 event indicates little probability of river flooding between the intermittent channel described above and the Richardson Highway. The construction or extension of a dike or levee in this section appears to be unnecessary. However, increasing the capacity of the culverts for Nordic Drive and other roads that drain the ditch immediately adjacent to the south side of the Richardson Highway would greatly improve drainage conditions during high water events. Subdivision drainage is discussed in another report.

Floodplains and gravel bars are often excellent sources of gravel in Alaska, and several borrow pits are located within or adjacent to the right floodplain upstream of the subdivisions. Any future area gravel pits should be situated such that they do not encourage lateral channel migration into the floodplain. They should also be located away from any existing or planned groins or levee structures, to reduce the threat of scour and toe erosion.

In-Between Existing City Structures

Local residents have noted that though water flows through the gap between the upstream and downstream City groins during flood events, the flows are generally diverted downstream before reaching the subdivisions. However, adding a groin section between the existing two groins will provide additional structural protection to those groins by preventing the river from outflanking or eroding them from behind. Computer modeling should be used to determine the necessary height and desired freeboard of the groin extension.

Downstream of Existing City Structures

The HEC-RAS analysis shows that in the reach downstream from Cross-section 3 and the lower end of Groin 2, the design flow is generally contained on the right floodplain by the bank, and by a narrow band of ground at a slightly higher elevation of 1 to 2 feet. Minor flooding at a few residences in the southwest corner of the subdivision was perceived to be from the river around the lower end of the groin, and the HEC-RAS analysis indicates that low spots between the surveyed cross-sections could lead to flood flow in that direction.

Additional flood protection would be obtained by extending the lower end of Groin #2 downstream for a distance of several hundred to several thousand feet. The groin could follow the small ridge running down the right floodplain approximately 400-600 feet to the north of the right bank. A downstream extension of Groin 2 would further reduce the potential for downstream backwater flooding into the subdivision, especially given the steep channel slope and few downstream obstructions.

As mentioned earlier, earlier flood control designs included plans for a dike or ring levee to wrap around the lower subdivision and terminate at the road (Engles and Engles, 2007). Though an analysis of topographic data indicates little probability from downstream backwater flooding on the north side of the lower end of Groin 2, additional modeling is recommended.

Additionally, perennial spring-fed streams flow from the subdivision; the channels are easily observable in Figure 3. At least one of the channels supports anadromous fish, and is likely classified as a fish-bearing stream by ADFG (Figure 8). A ring levee that connects Groin 2 to the Richardson Highway will intersect this channel, and will likely have to include a gravity outlet that meets fish passage requirements. If subsequent modeling determines that high exterior stages would occur during a flood event, the outlet would have to be equipped with a gate to prevent riverflows from entering the protected area. In addition to the gate, a pumping station would then be needed to discharge the interior flow over or through the levee.

Figure 8. Perennial anadromous stream flows west from subdivisions.

Design-Hydraulic Modeling

The general recommendations described above for additional flood protection through new groin extension or levee construction projects are illustrated in Figure 9. Additional hydraulic analysis is required to finalize design parameters such as length, height of dike/levee, size of required riprap, and others. The HEC-RAS program should be used to conduct the design hydraulic analysis and dike/levee modeling.

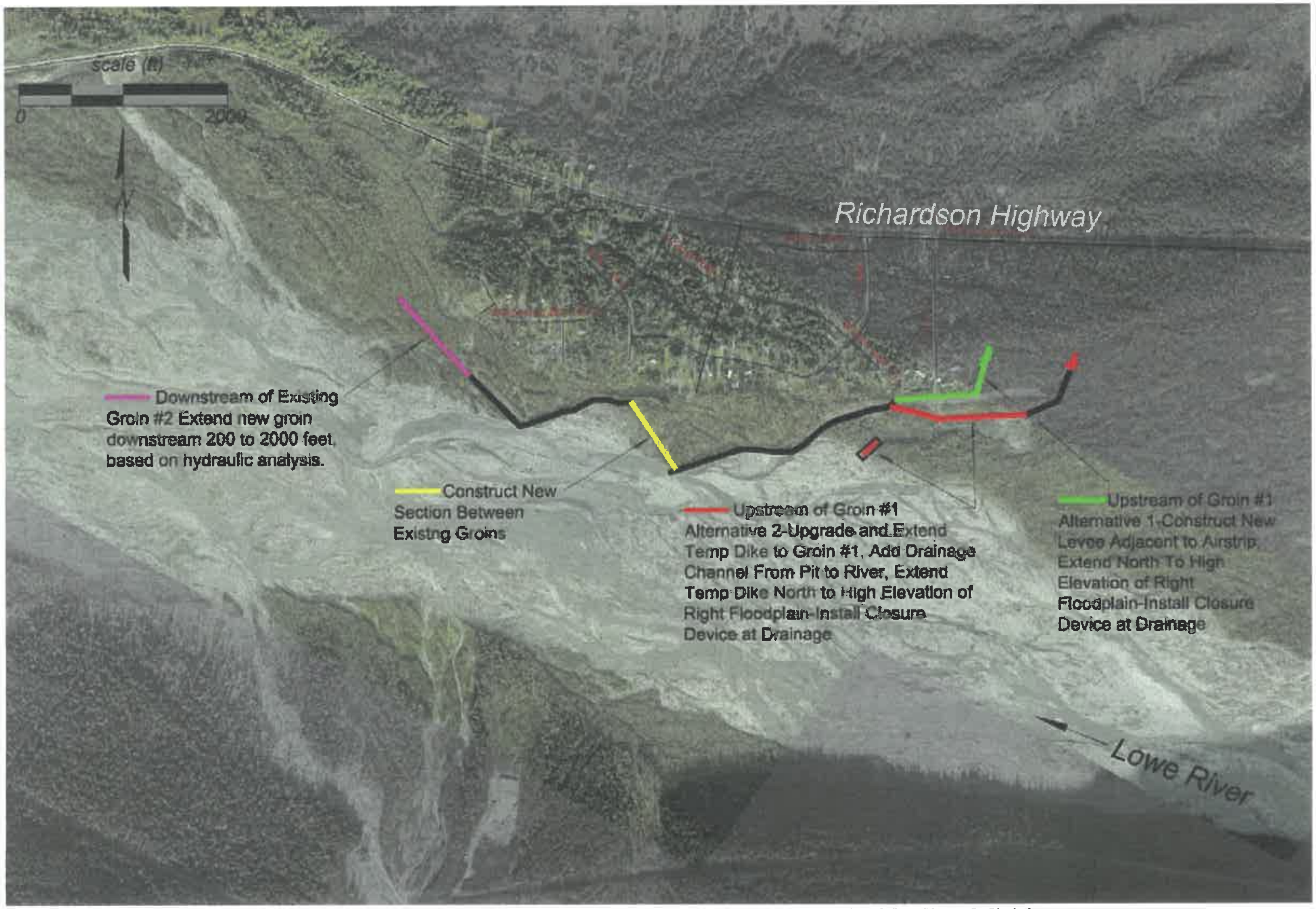


Figure 9. Options for additional flood protection at Alpine/Nordic subdivisions.

References

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

Engles, J., and L. Engles. 2007. October 2006 Alpine/Nordic subdivisions flood event; root cause analysis report. April 2007. Subarctic Consultants, Valdez, AK.

Federal Emergency Management Agency (FEMA). 2003. Guidelines and specifications for flood hazard mapping partners. Appendix H: guidance for mapping of areas protected by levee systems.

Jones, S.H., and C.B. Fahl. 1994. Magnitude and frequency of floods in Alaska and conterminous basins of Canada: U.S. Geological Survey Water-Resources Investigations Report 93-4179, 122 p., 2 plates.

Prakash, A. 2004. Water resources engineering; handbook of essential methods and design. American Society of Civil Engineers Press.

Woodward-Clyde Consultants (WCC). 1983. Alpine Woods Estates flood evaluation. Prepared for the City of Valdez. October 3, 1983.

Bibliography

Prepared by FEMA for the City of Valdez

- Flood Insurance Study, City of Valdez, Alaska, August 1982.

Prepared by Woodward-Clyde Consultants for the City of Valdez

- Hydrologic Study for the City of Valdez, June 1979.
- Phase II Hydrological Study-Summary of Collected Data, October 1979.
- Alpine Woods Subdivision Flood Evaluation Proposal, July 1983.
- Alpine Woods Estates Flood Evaluation-Flooding calculations, September 1983.
- Alpine Woods Estates Flood Evaluation, October 1983.
- Alpine Woods Estates Detailed Flood Evaluation, April 1984.

Prepared by the US Corps of Engineers for the City of Valdez

- Flood Plain Information, Mineral Creek, Valdez, Alaska. July 1976.
- Special Flood Hazard Report, Alpine Woods Estates Subdivision, Valdez Alaska, January 1984.

Prepared by CH2MHill, for the City of Valdez

- Lowe River Stabilization/Relocation Study-Task No. 9-Program for Immediate Action for the Alpine Woods Estates/Nordic Subdivision Area. August 1990.
- Lowe River Stabilization/Relocation Study, December 1990.

City of Valdez or other documents

- City of Valdez land ownership maps and photographs. City of Valdez (?). Date unknown.
- Valdez Comprehensive Plan. City of Valdez (?). Date unknown.
- City of Valdez Floodplain Ordinance.
- Valdez Coastal Management Plan-Scoping Report/Plan Evaluation-Phase 1, Resource Inventory, prepared by Valdez Coastal District, Valdez Planning Commission, Bechtol Planning and Development, and Arctic Slope Consulting Group, Inc., June 2004.

Appendix 1-Hydrologic Analysis

The USGS Bulletin 17B guidance (annual flood frequency analysis) requires at least 10 years of data before conducting a probability analysis. Since the available peak flow record for the Lowe River is so short as to be below the minimum necessary to develop flood estimations based on probability analysis alone, the following methods were used. First, flood magnitude estimations were developed using USGS regression equations for estimating the magnitude of peak streamflows in Alaska. Estimations were developed for the Lowe Rive at the USGS gaging station site in Keystone Canyon.

The latest USGS regression method for estimating peak streamflows at ungaged locations is described in the USGS Water Resources Investigations Report 03-4188 (Curran et al., 2003). Basin characteristic information is used in the USGS regression analysis. For Region 3, the characteristics include:

- drainage area upstream from the site
- percentage of the total drainage area shown as lakes and ponds
- mean minimum January temp
- mean annual precipitation averaged over the drainage area.

Drainage areas and areas of lakes and ponds were obtained by planimetric techniques used with USGS 1:63360 quad maps. The mean annual precipitation value for the watershed was obtained from Plate 2 of the Jones and Fahl report (1994). Basin characteristics are as follows:

Drainage areas - 222 square miles (Lowe River-Keystone Canyon);
Area of lakes and ponds - 0.0 %
Mean min January temp - 4 degrees F
Mean annual precipitation - 100 inches

The USGS report provides several methods to evaluate the accuracy and limitations of the regression equations. One measure of predictive ability of each equation is the average equivalent years of record, or the number of years of systematic streamflow data that would have to be collected for a given site to estimate the streamflow statistic with accuracy equivalent to the estimate from the regression equations. Methods are also provided to estimate the average standard of error of prediction. Finally, confidence limits provide a measure of the error in a particular prediction. The 5% and 95% confidence limits provide a 90% prediction interval for a particular site. These values are listed in Table 5 to provide the user with an understanding of the accuracy of the equations. Additional description of these methods is found in Curran et al. (2003).

A statistical flood-frequency analysis of the annual-maximum peak flows was developed using a log-Pearson Type III probability distribution and the USGS Bulletin 17B methods (USGS, 2006). USGS records at the USGS gaging station site in Keystone Canyon provided 7 annual peak flow values; two were treated as historic peak values outside of the systematic record (1995 and 2006), and one peak was not used in the analysis because of data problems. Results and confidence limits are found in Table 6.

Table 5. Flood frequency estimations and accuracy of regression equations.

Recurrence Interval (years	Discharge (cfs)	Standard Error (+%)	Standard Error (-%)	Confidence Limits		Equivalent Years
				5%	95%	
2	8770	45.6	-31.3	4700	16400	0.9
5	12200	45.0	-31.0	6580	22600	1.4
10	14600	45.5	-31.2	7860	27300	2.0
25	17900	47.0	-32.0	9420	33900	2.7
50	20400	48.8	-32.8	10500	39400	3.1
100	22900	51.0	-33.8	11600	45400	3.5
200	25700	53.6	-34.9	12600	52300	3.8
500	29400	57.4	-36.5	13800	62400	4.1

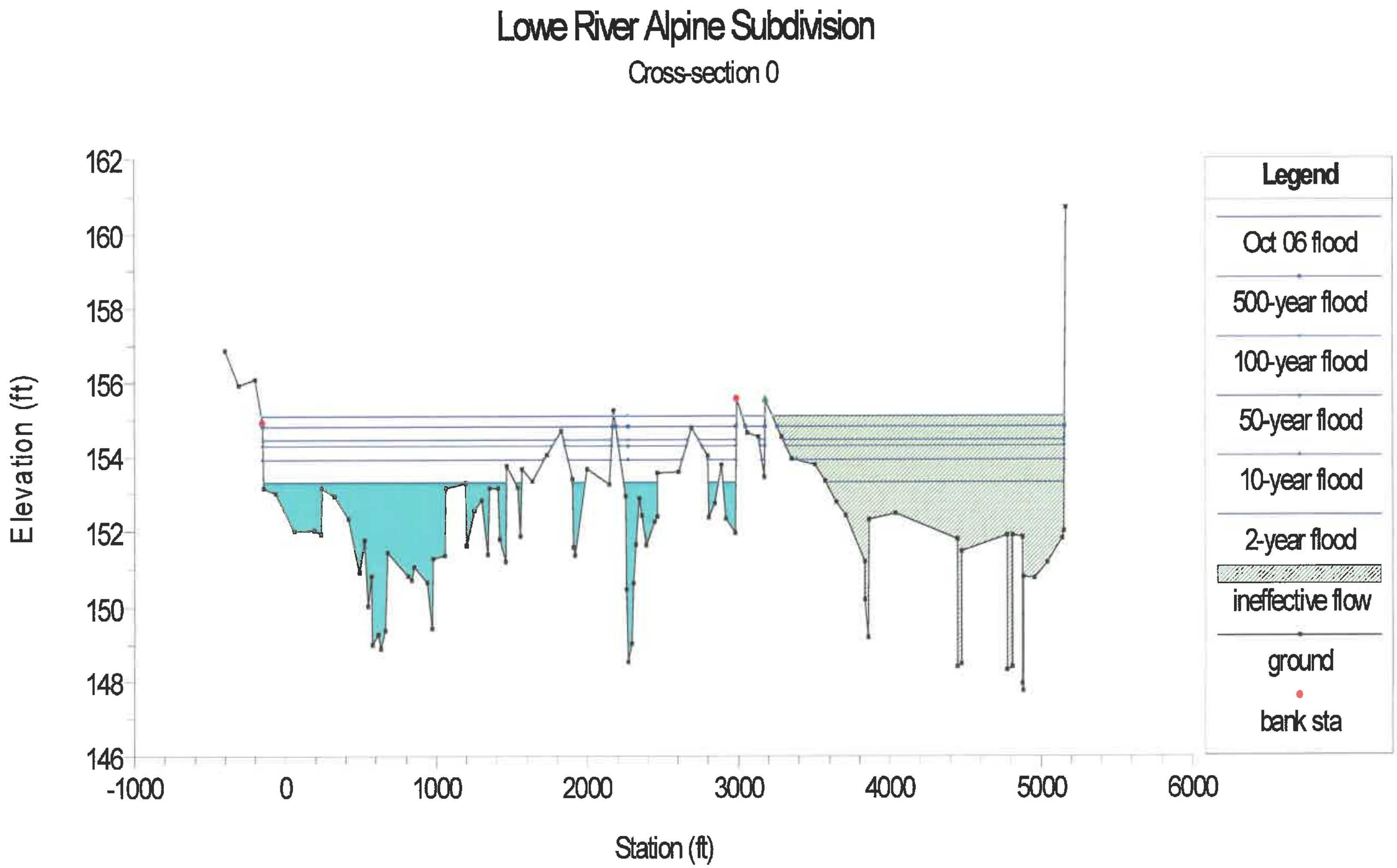
To improve estimates, Bulletin 17B recommends that a generalized skew computed from nearby long-term stations be used to weight individual station skews within a region. In Alaska, average station skews for each region, are estimated and provided in Curran et al. The average skew coefficient of 0.16 was used in the analysis.

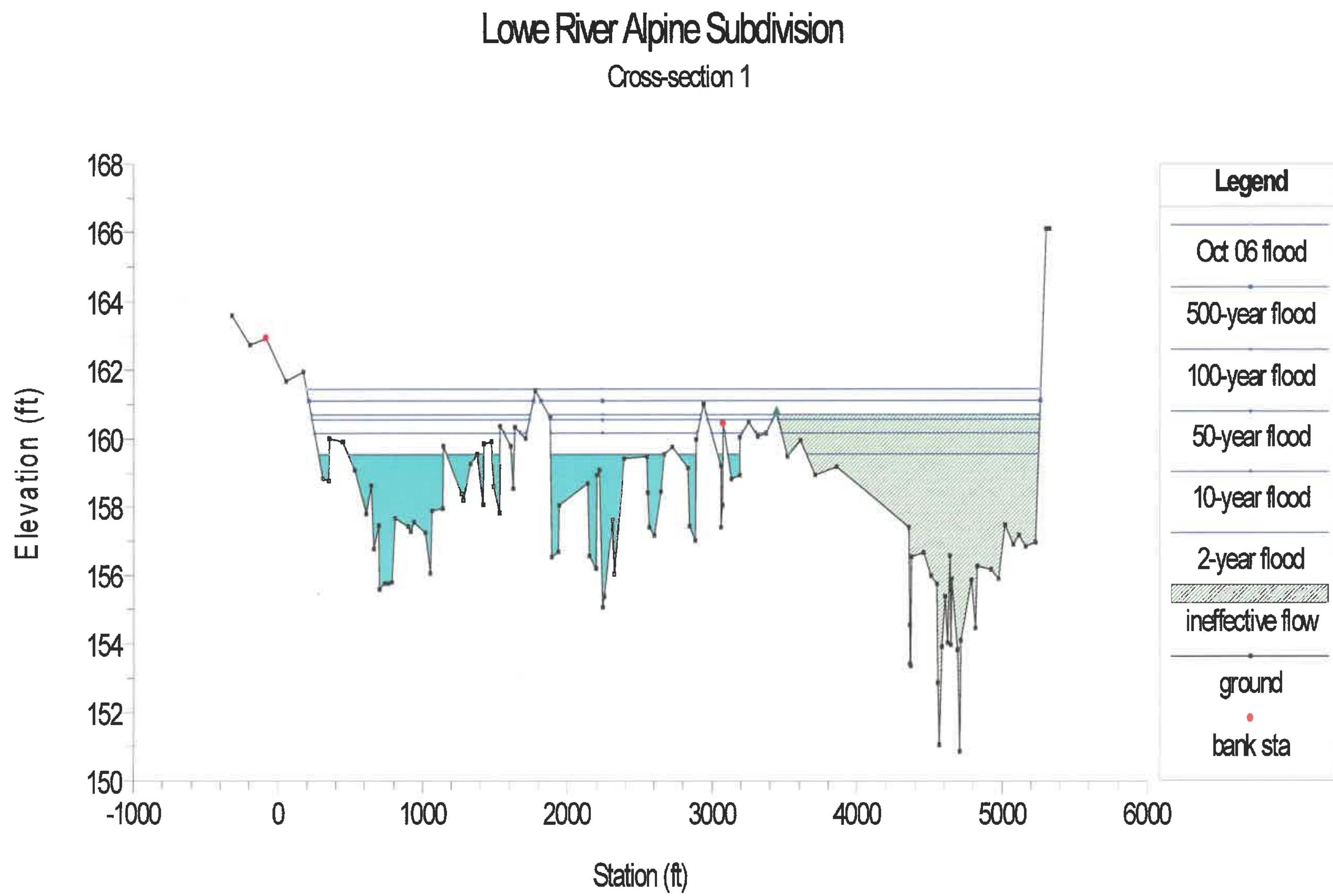
Table 6. Probability estimates and confidence limits.

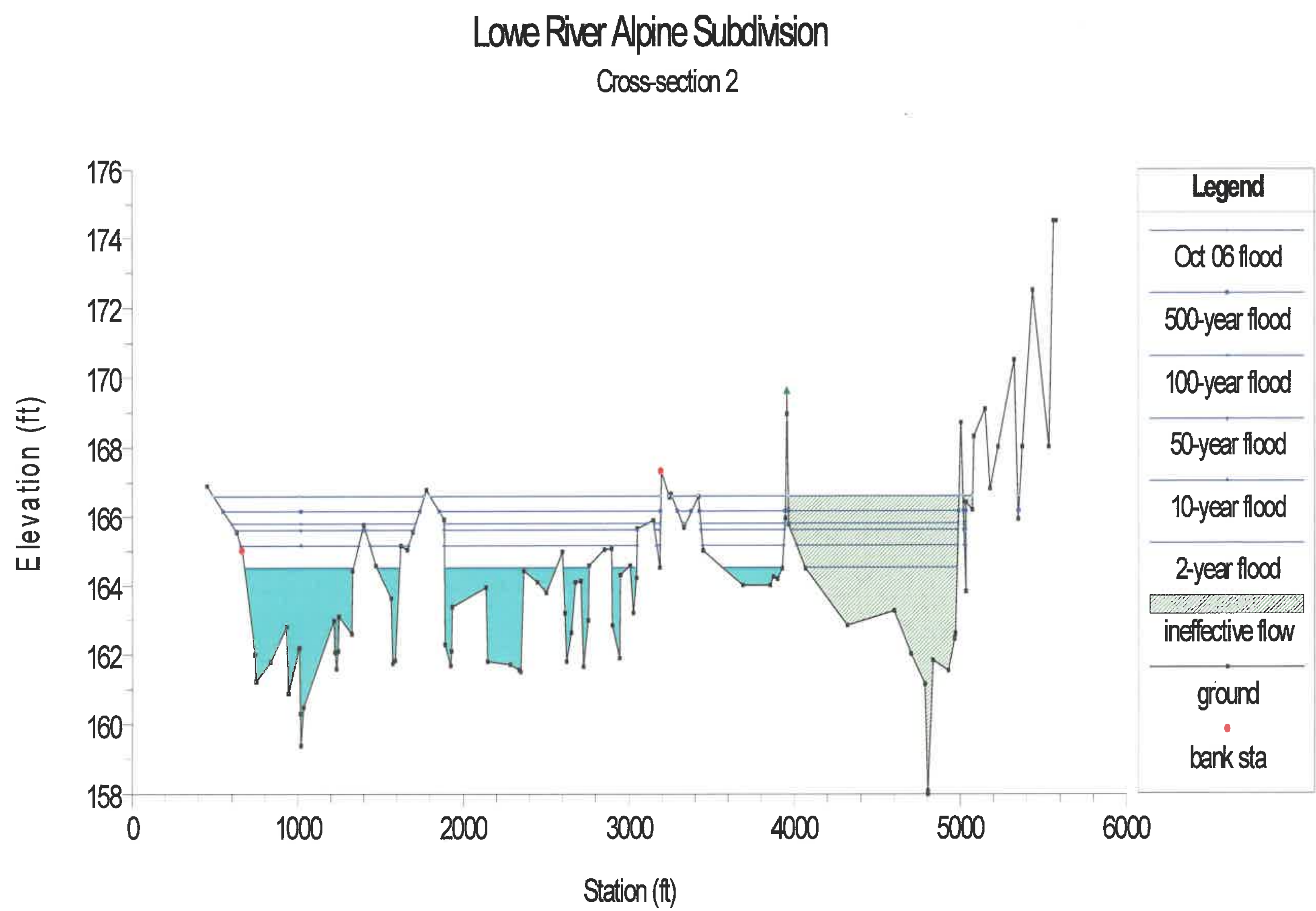
Annual Exceedance Probability	Bull. 17B Estimate (cfs)	Confidence Limits	
		Lower	Upper
0.9950	6557	2189	8640
0.9900	6732	2363	8810
0.9500	7363	3056	9440
0.9000	7813	3612	9917
0.8000	8496	4540	10710
0.6667	9305	5730	11820
0.5000	10380	7320	13780
0.4292	10910	8042	15020
0.2000	13430	10650	24190
0.1000	15740	12300	37430
0.0400	19020	14170	64710
0.0200	21730	15530	95920
0.0100	24690	16900	140200
0.0050	27910	18290	202700
0.0020	32660	20210	325200

Following that estimation, a weighted estimate was developed from both the regression estimate and the probability distribution, using methods described in Curran et al. The weights are based on the years of observed data at the gage, and the average equivalent years of record for the regression equation. Finally, the new weighted estimate was then adjusted for the larger watershed at the ungaged site (328.5 square miles at Alpine Subdivision). The weighted estimates for both sites are found in Table 1 and Figure 2.

Appendix 2-HEC RAS Results



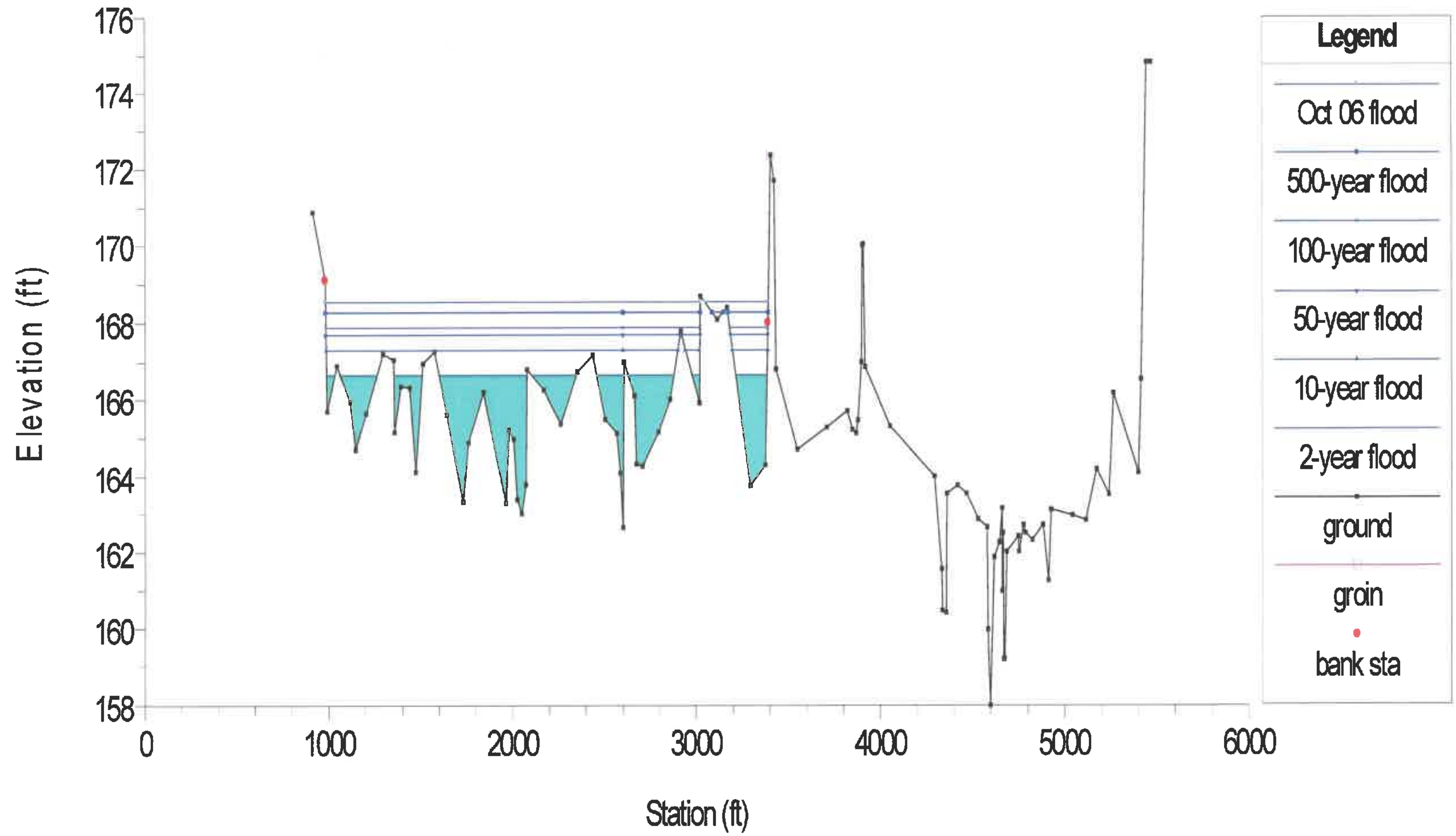

Lowe River Alpine Subdivision
Cross-section 1
Elevation (ft)
168
166
164
162
160
158
156
154
152
150
-1000
0
1000
2000
3000
4000
5000
6000
Station (ft)
Legend
Oct 06 flood
500-year flood
100-year flood
50-year flood
10-year flood
2-year flood
ineffective flow
ground
bank sta

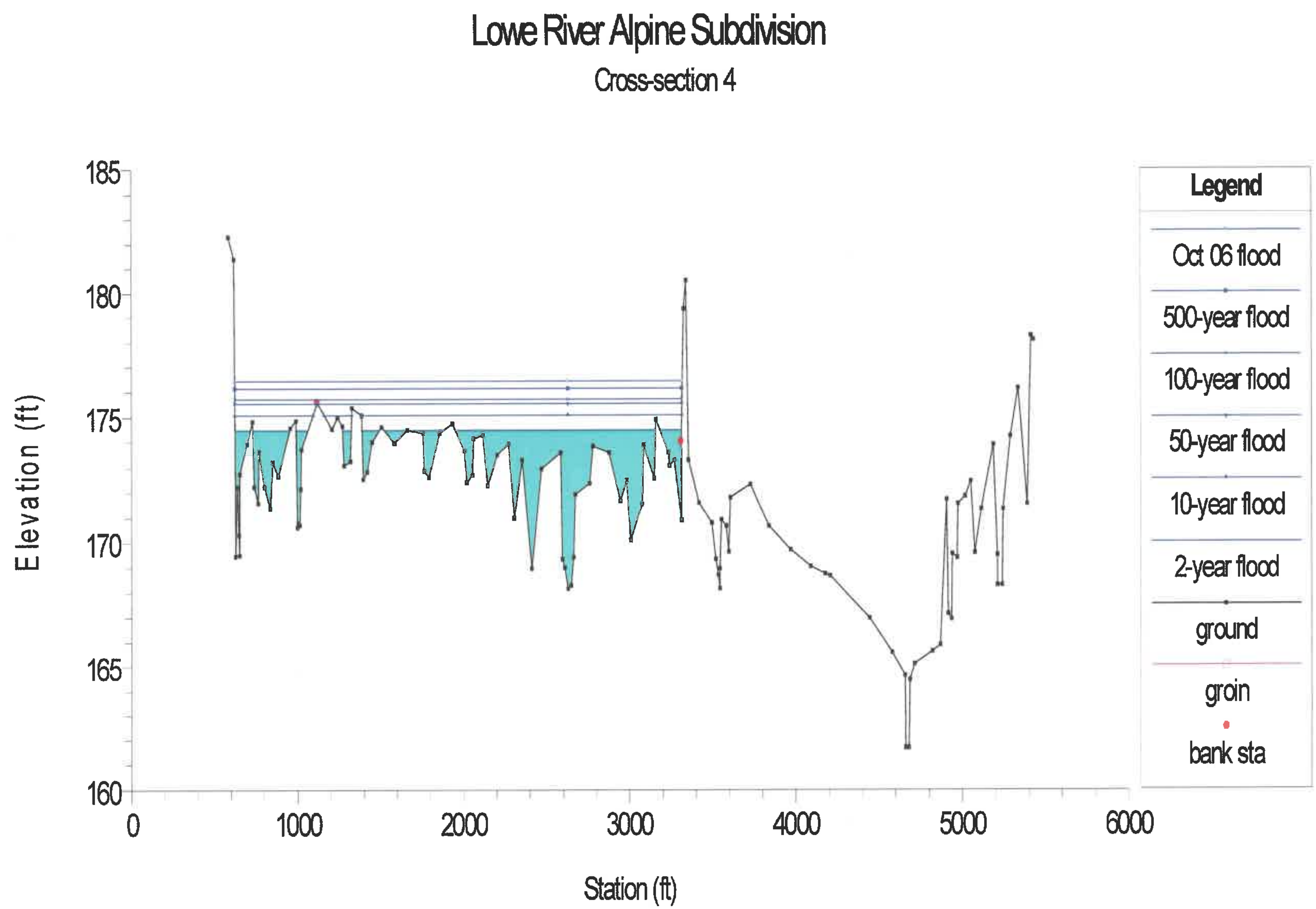


Lowe River Alpine Subdivision
Cross-section 2
Elevation (ft)
176
174
172
170
168
166
164
162
160
158
0
1000
2000
3000
4000
5000
6000
Station (ft)
Legend
Oct 06 flood
500-year flood
100-year flood
50-year flood
10-year flood
2-year flood
ineffective flow
ground
bank sta

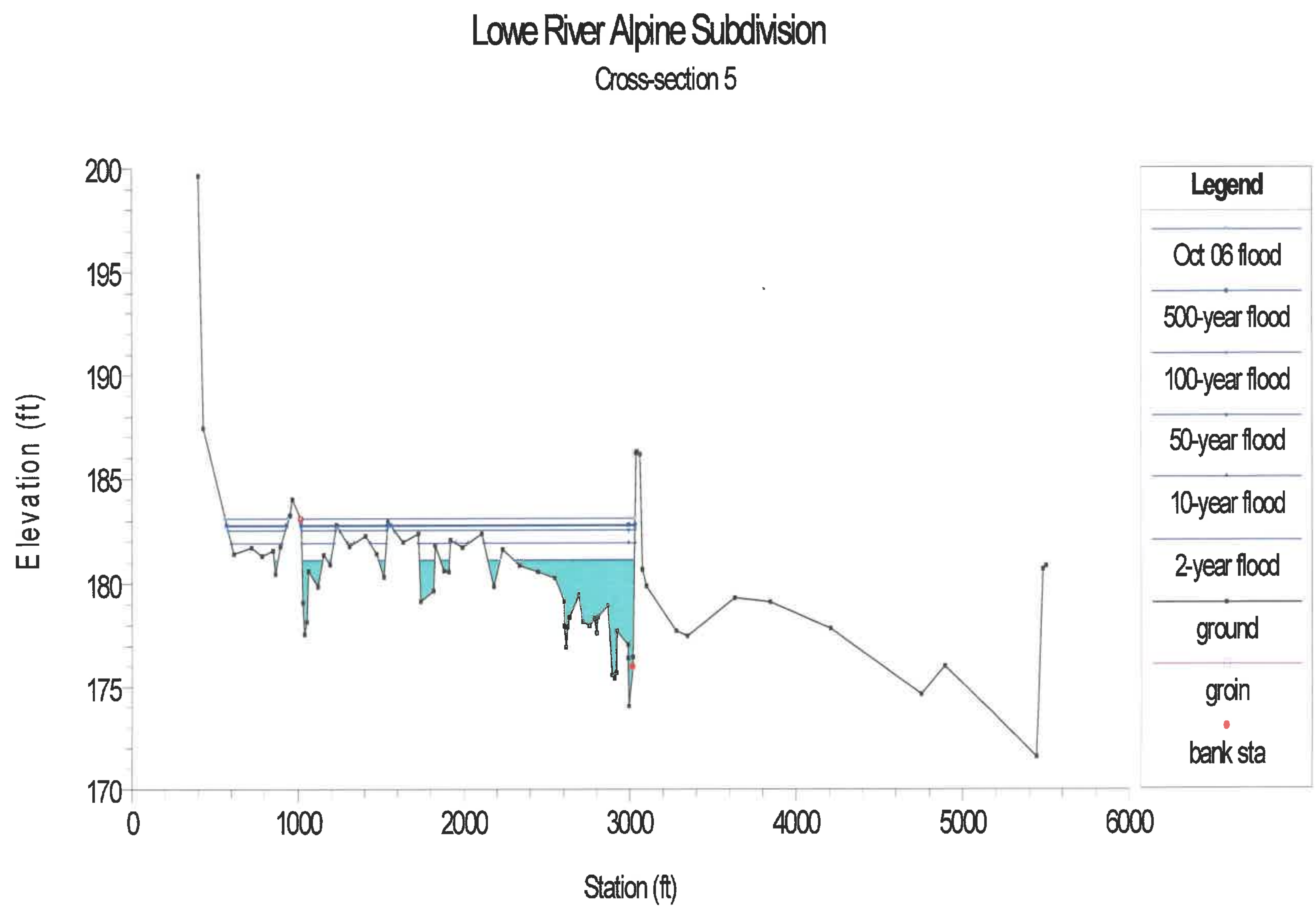
Lowe River Alpine Subdivision

Cross-section 3

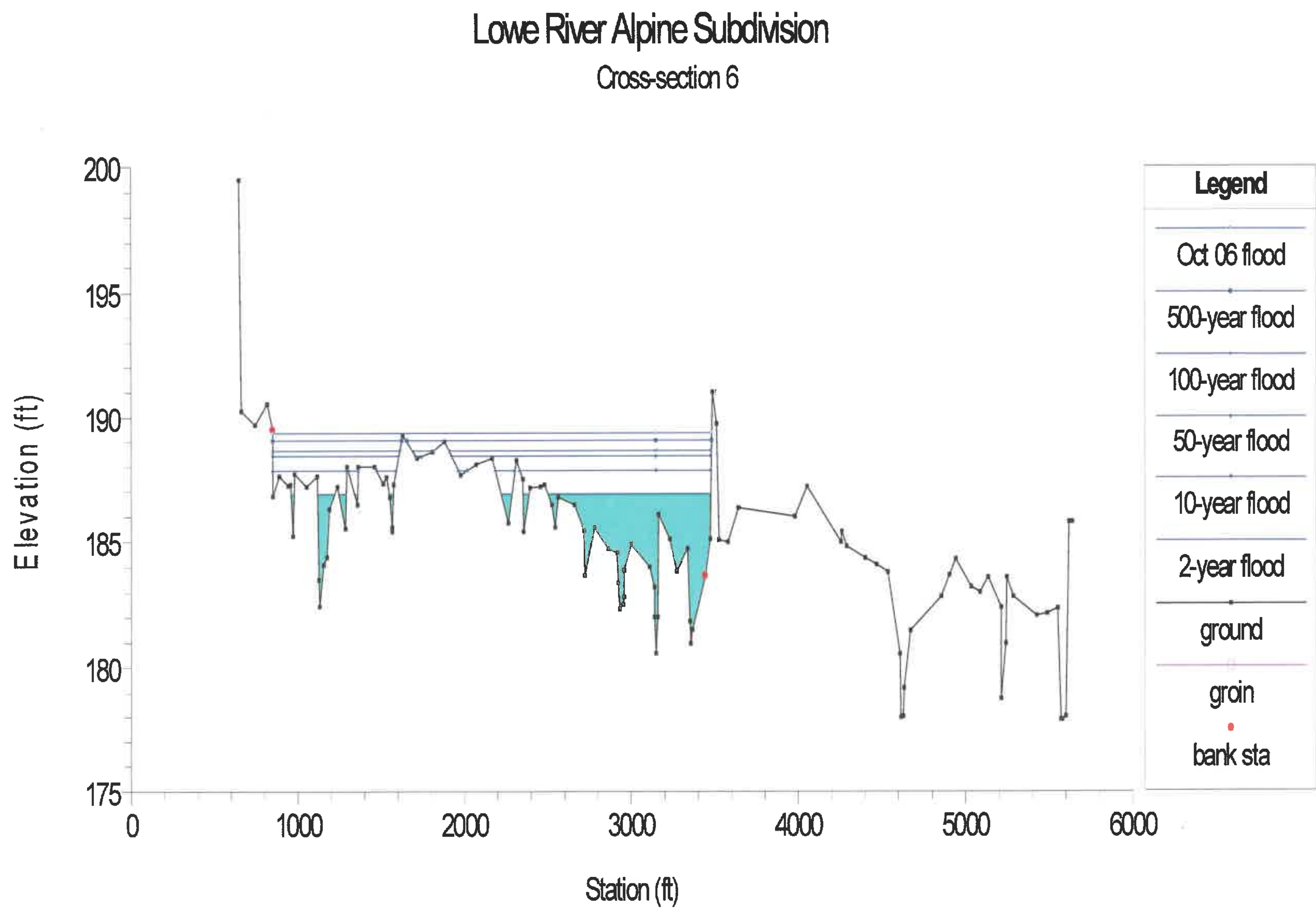


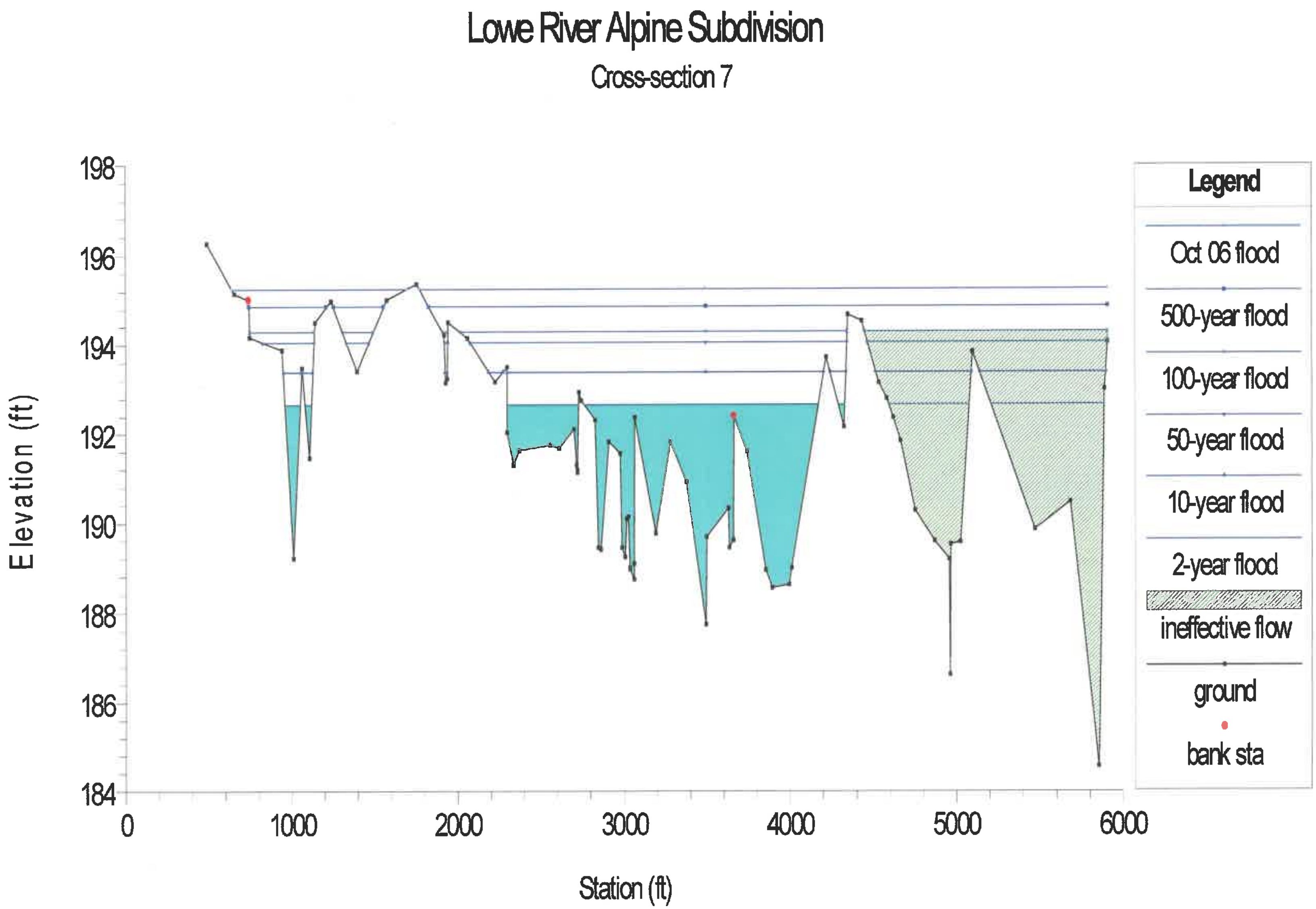


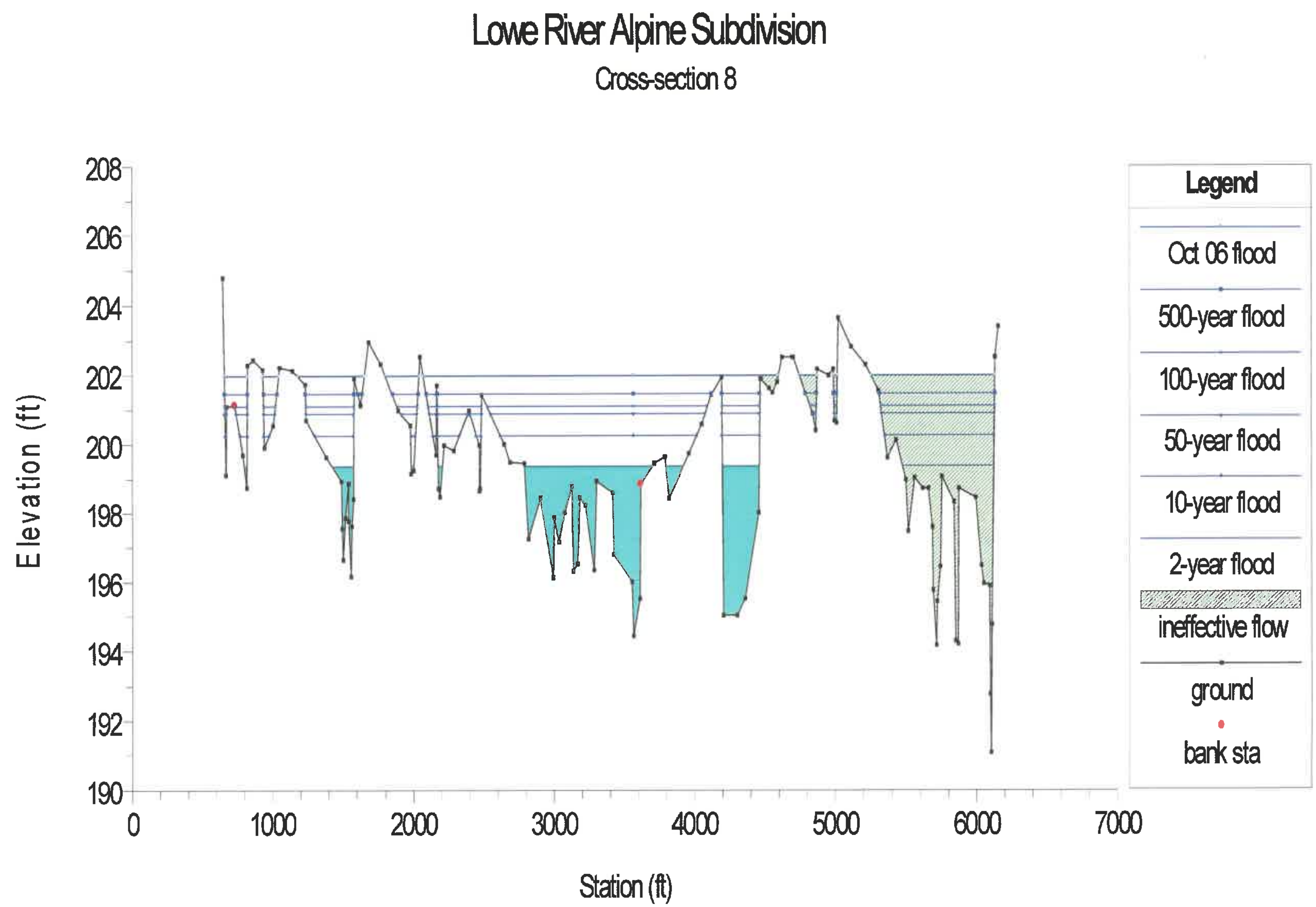
Lowe River Alpine Subdivision
Cross-section 4
Elevation (ft)
185
180
175
170
165
160
0
1000
2000
3000
4000
5000
6000
Station (ft)
Legend
Oct 06 flood
500-year flood
100-year flood
50-year flood
10-year flood
2-year flood
ground
groin
bank sta

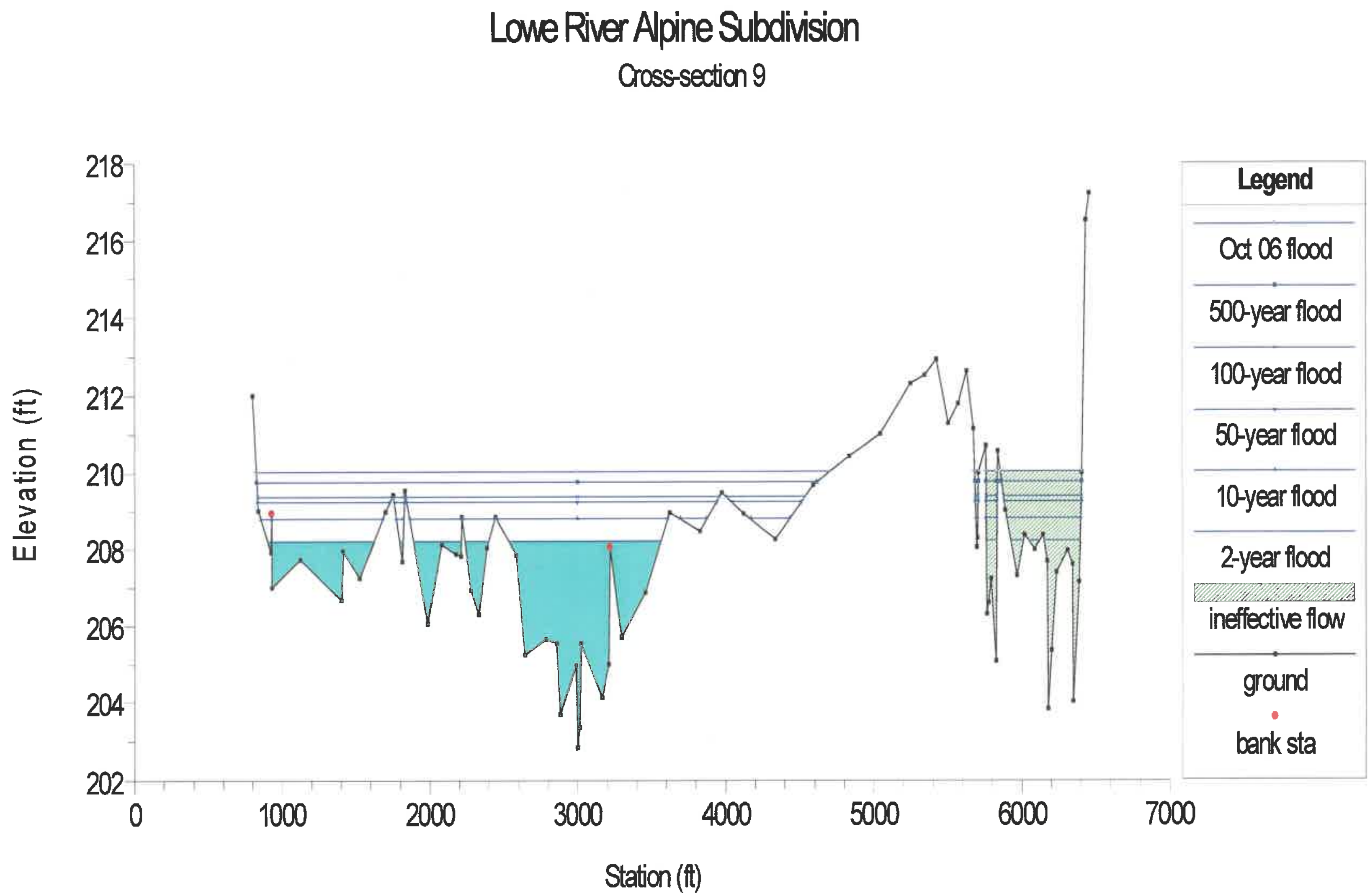


Lowe River Alpine Subdivision
Cross-section 5
Elevation (ft)
200
195
190
185
180
175
170
0
1000
2000
3000
4000
5000
6000
Station (ft)
Legend
Oct 06 flood
500-year flood
100-year flood
50-year flood
10-year flood
2-year flood
ground
groin
bank sta


Lowe River Alpine Subdivision
Cross-section 6
Elevation (ft)
200
195
190
185
180
175
0
1000
2000
3000
4000
5000
6000
Station (ft)
Legend
Oct 06 flood
500-year flood
100-year flood
50-year flood
10-year flood
2-year flood
ground
groin
bank sta


Lowe River Alpine Subdivision
Cross-section 7
198
196
194
192
190
188
186
184
Elevation (ft)
0
1000
2000
3000
4000
5000
6000
Station (ft)
Legend
Oct 06 flood
500-year flood
100-year flood
50-year flood
10-year flood
2-year flood
ineffective flow
ground
bank sta


Lowe River Alpine Subdivision
Cross-section 8
208
206
204
202
200
198
196
194
192
190
Elevation (ft)
0
1000
2000
3000
4000
5000
6000
7000
Station (ft)
Legend
Oct 06 flood
500-year flood
100-year flood
50-year flood
10-year flood
2-year flood
ineffective flow
ground
bank sta


Lowe River Alpine Subdivision
Cross-section 9
Elevation (ft)
218
216
214
212
210
208
206
204
202
0
1000
2000
3000
4000
5000
6000
7000
Station (ft)
Legend
Oct 06 flood
500-year flood
100-year flood
50-year flood
10-year flood
2-year flood
ineffective flow
ground
bank sta

Appendix 3-Glossary Of Flood Control Structures

Various terms have been used to describe the flood control structures along the right bank of the Lowe River. The following definitions are provided to assist the reader with definitions and terms.

Levees: Manmade structures, usually earthen embankments, designed and constructed in accordance with sound engineering practices to contain, control, or divert the flow of water so as to provide protection from temporary flooding.

Ring levees: Levees that completely encircle or "ring" an area subject to inundation from all directions.

Setback levees: Levees that are built on the land side of existing levees, usually because the existing levees have suffered distress or are in some way being endangered, as by river migration.

A levee system usually consists of a main levee, tie back levees, a gravity outlet, and pumps. Some levee systems may also include pressure conduits, closure structures, ring levees, setback levees, sublevees, and spur levees.

Dikes: Embankments constructed of earth or other suitable materials to protect land from overflows or to regulate water (from FEMA, 2003).

Groins: Groins are dikes extending from the bank of the river to a specified distance, which may usually be up to the normal waterline. They are constructed to protect the bank against erosion or to control channel meanders. Groins are more effective when constructed in series. They may be oriented perpendicular to the bank or at angles inclined slightly upstream or downstream (from Prakash, 2004).

Appendix 4-Cross-section Coordinates

Table 7. Major cross-section endpoints, in Alaska State Plane coordinates. Cross-sections are viewed looking downstream.

Cross-section	Left Endpoint		Right Endpoint	
	Northing (ft)	Easting (ft)	Northing (ft)	Easting (ft)
0.0	2576792	1632277	2581380	1635435
1.0	2576297	1633177	2580948	1636378
2.0	2576446	1634353	2580672	1637260
3.0	2576672	1635029	2580642	1637375
4.0	2575922	1636358	2580561	1637679
5.0	2575438	1637392	2580347	1638786
6.0	2575434	1638313	2580255	1639686
7.0	2575003	1639258	2580214	1640740
8.0	2574860	1640308	2580161	1641817
9.0	2574737	1641457	2580117	1642995

Appendix 5-Elevation Reference for Cross-section Survey