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Technical Memorandum

To: **Gerry Bowen**
Papio-Missouri River Natural Resources District (PMRNRD)

Date: January 10, 2011

From: Bob Mussetter

Subject: Preliminary Observations and Recommendations for Repairs to Elkhorn River IPA

This memorandum describes results of a preliminary evaluation of conditions at, and in the vicinity of, the Elkhorn River IPA that was constructed in the late-1980sto protect adjacent lands from lateral erosion by the river, and subsequently modified in local areas by either extending or repairing damaged sites (**Figure 1**). The memo also includes preliminary recommendations and costs for adding new bank protection or repairing existing bank protection at potential problem sites.

Scope of Work

The preliminary evaluation included the following tasks:

1. Review of available background information related to the project, including the following:

   - The Feasibility Study and Nebraska Natural Resource Fund Applications prepared for the original project by Simons, Li & Associates, Inc. (SLA, 1985).

   - Construction plans and specifications prepared by Wells Engineers, Inc. (WEI, 1987).

   - Inspection reports prepared by the Natural Resource Conservation Service (NRCS) and PMRNRD staff subsequent to the June 2010 high flows.

   - Information from Mr. Gerry Bowen, PMRNRD, regarding the relationship between the original construction plans and the actual installation and subsequent modifications (Figure 1, **Table 1**).

   - Historical aerial photographs of the reach taken on the following dates (recorded discharge at Waterloo gage in parentheses):

     - June 30,1953 (907 cfs)
     - April 26, 1966 (1,150 cfs)
     - May 14, 1981 (411 cfs)
     - June 27, 1988 (381 cfs)
     - October, 2000 (~550 cfs)
     - October, 2004 (~580 cfs)
     - September 22, 2009 (1,040 cfs)
     - June 16, 2010 (37,250 cfs)

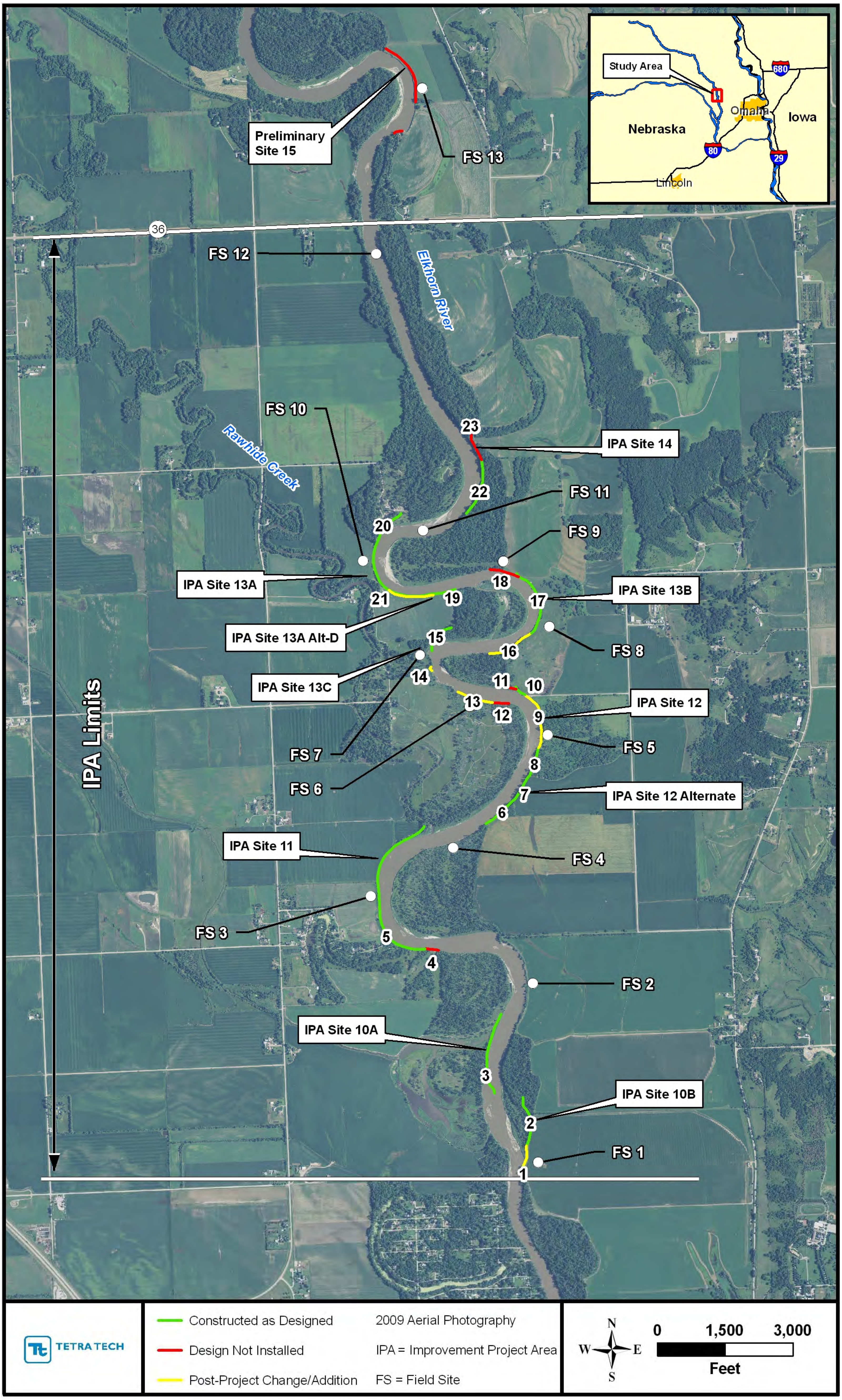

Figure 1. Site map showing the construction sites for the original IPA and the sites that were visited during the October 6, 2010 field reconnaissance. Also shown are label corresponding to the summary notes in Table 1 that were provided by Mr. Bowen (PMRNRD).

Table 1. Summary of construction activities associated with the original IPA (Mr. Gerry Bowen, personal communication, December 2010).			
Note	Site	Status	Description
1	South of 10B	Post-Project Change/Addition	Extended windrow revetment in early 1990's
2	Site 10B	Constructed as Designed	Piled windrow revetment, buried windrows and piled windrow refusal
3	Site 10A	Constructed as Designed	Piled windrow revetment and channel plug
4	Site 11 Alternate A	Design Not Installed	Design alternate (piled windrow revetment) not installed
5	Site 11	Constructed as Designed	Piled windrow revetment, buried windrows and piled windrow refusal
6	Site 12 Alternate A	Constructed as Designed	Design alternate (piled windrow revetment) was installed
7	Site 12	Constructed as Designed	Piled windrow revetment
8	Site 12 Alternate B	Constructed as Designed	Design alternate (piled windrow revetment, not cabled trees) was installed
9	Site 12	Post-Project Change/Addition	Piled windrow revetment was installed but washed away and rock was replaced in the late 1990's
10	Site 12	Constructed as Designed	Piled windrow revetment and piled windrow refusal
11	Site 12 Alternate A	Design Not Installed	Design alternate (piled windrow revetment) not installed
12	Site 13 Alternate A	Design Not Installed	Design alternate (cabled trees) not installed
13	Site 13	Post-Project Change/Addition	Cabled trees installed but washed away, no replacement has been installed
14	Site 13	Post-Project Change/Addition	Placed additional rock in late 1990's
15	Site 13	Constructed as Designed	Buried windrow revetment and piled windrow refusal
16	Site 13B Alternate A	Post-Project Change/Addition	Extended piled windrow revetment in late 1990's where Site 13B alternate (cabled trees) was not installed. Rock obtained from Site 14 stockpile. Rock washed out and not replaced.
17	Site 13B	Constructed as Designed	Piled windrow revetment and piled windrow refusal
18	Site 13B	Design Not Installed	Cabled trees not installed
19	Site 13A Alternate A	Constructed as Designed	Design alternate (piled windrow revetment) was installed
20	Site 13A	Constructed as Designed	Piled windrow revetment, buried windrows and piled windrow refusal
21	Adjacent to Site 13A	Post-Project Change/Addition	Berm constructed in uplands adjacent to Site 13A in early 1990's to prevent river flow from cutting across meander
22	Site 14 Alternate A	Constructed as Designed	Design alternate (piled windrow revetment) was installed. Originally cabled trees were recommended but note in plans changed to windrow.
23	Site 14	Design Not Installed	Piled windrow revetment not installed due to easement problems. Rock was stockpiled but later used on Site 13B Alternate A

2. A site reconnaissance by Dr. Bob Mussetter in the company of Mr. Bowen and Ms. Sara Mechtenberg (Tetra Tech) that was conducted on October 6, 2010. This reconnaissance included the following 13 specific sites *(If viewing in electronic format, click on the field site identification listed below to forward to the relevant section*):

    - Field Site 1
    - Field Site 2
    - Field Site 3
    - Field Site 4
    - Field Site 5
    - Field Site 6
    - Field Site 7
    - Field Site 8
    - Field Site 9
    - Field Site 10
    - Field Site 11
    - Field Site 12
    - Field Site 13

3. Evaluation of observations during the site visit, the historic flows in the reach based on measurements at the Elkhorn River at Waterloo stream gage, the NRCS/PMRNRD inspection reports, and the above listed background information, including an assessment of the performance of the original project.

4. Development of recommendations for additional bank protection or repairs to the existing bank protection.

Site Hydrology

Since the flows that occur in the project reach drive the lateral erosion processes, an understanding of the range, magnitude and duration of the flows is important in assessing the causes of existing erosion problems and potential mitigation measures. Records at the Elkhorn River near Waterloo gage (USGS Gage No. 06800500), located about 2 miles downstream from the downstream IPA boundary at the Highway 275 Bridge, date back to the early part of the 20th century and should accurately represent flows in the project reach. The only significant tributary in the reach is Rawhide Creek that enters from the right (west) side of the river near the upstream end of the IPA. With a total drainage area about 50 mi^2, Rawhide Creek represents less than 1% of the 5,870 mi^2 contributing drainage area at the Waterloo Gage; thus, Rawhide Creek inflows most likely have little impact on the total flow in the river, particularly during intermediate to high river flows that are most important to the lateral erosion processes.

During the 82-year period of continuous mean daily discharge records [Water Year (WY) 1929 through WY2010], the annual runoff volume at the Waterloo gage varied from 301,000 ac-ft in WY1939 to over 2.8M ac-ft in 1993, and averaged about 1.03M ac-ft (**Figure 2**). Interestingly, the runoff volume appears to have increased significantly over the past 3 decades, with the average since WY1982 at about 1.47M ac-ft, compared to ~780,000 ac-ft between WY1929 and WY1981. Based on the mean daily flow duration curves, the median flow during the WY1982 to WY2010 period was 1,260 cfs, and flows exceeded 2,260 cfs about 25 percent of the time and 4,020 cfs about 10 percent of the time (**Figure 3**). The median, 10- and 25-percent exceedence flows during the earlier period were 600, 1,020 and 2,030 cfs, respectively. On a seasonal basis, the flows tend to be elevated from early-March through early-August, with the highest flows typically occurring in June and base-level flows throughout the remainder of the year (**Figure 4**).

Annual peak discharges at the Waterloo gage ranged from 2,120 cfs on November 21, 1931, to 100,000 cfs on June 12, 1944, and the median value was about 14,600 cfs (**Figure 5**). In spite of the increase in runoff volume illustrated in Figures 2 and 3, and the relatively high peak discharge in 2010 (55,000 cfs, **Figure 6**), there does not appear to have been a systematic change in the peak flow regime over the period of record. A flood frequency analysis of the complete data set from WY1929 through WY2010 was performed using HEC-SSP (USACE, 2009) and procedures specified in the Interagency Advisory Committee on Water Data Bulletin 17B (USGS, 1982). Based on this analysis, the 1.5-, 2-, 10-, 50- and 100-year recurrence interval peak flows are 10,400, 14,100, 35,100, 60,700 and 73,700 cfs, respectively. The provisional peak discharge that occurred on June 14, 2010 of 55,000 cfs, the second highest peak in the record, had a recurrence interval of about 36 years (**Figure 7**). For comparison, the 10-, 50- and 100-year peak discharges in the Effective Flood Insurance Study for Douglas County (FEMA, 2010), which were estimated based on only the period of record through WY1975, are 36,200, 69,000 and 88,500 cfs, respectively.

Field Observations

The field reconnaissance was conducted on October 6, 2010, when the recorded flows at the Waterloo gage were about 1,480 cfs. For context in reviewing the photos that will be referred to in the subsequent discussions, this flow is only about 200 cfs higher than the median flow for the entire year (Figure 3), but is a relatively high flow for October when the flows exceeded this level only about 20 percent of the time and the median flow was about 910 cfs during the WY1981-WY2010 period. During the reconnaissance, the reach was traversed on an airboat and stops were made at key locations, working primarily from downstream to upstream.

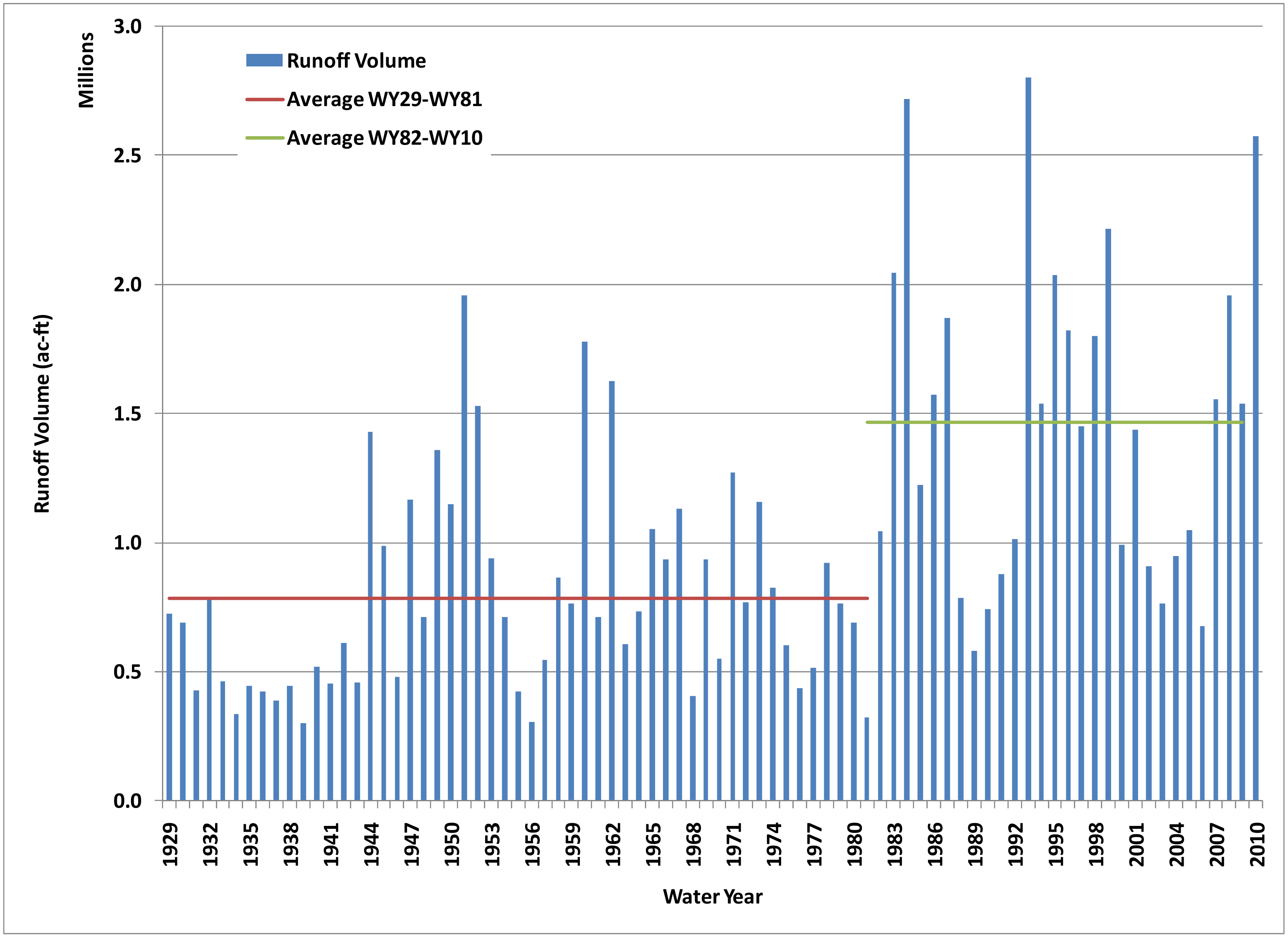

Figure 2. Annual runoff volume at the Elkhorn River at Waterloo, NE gage (USGS Gage 06800500) for the continuous period of record from WY1929 through WY2010. Also shown are the mean values for the periods from WY1929 through WY1981 and WY1982 through WY2010.

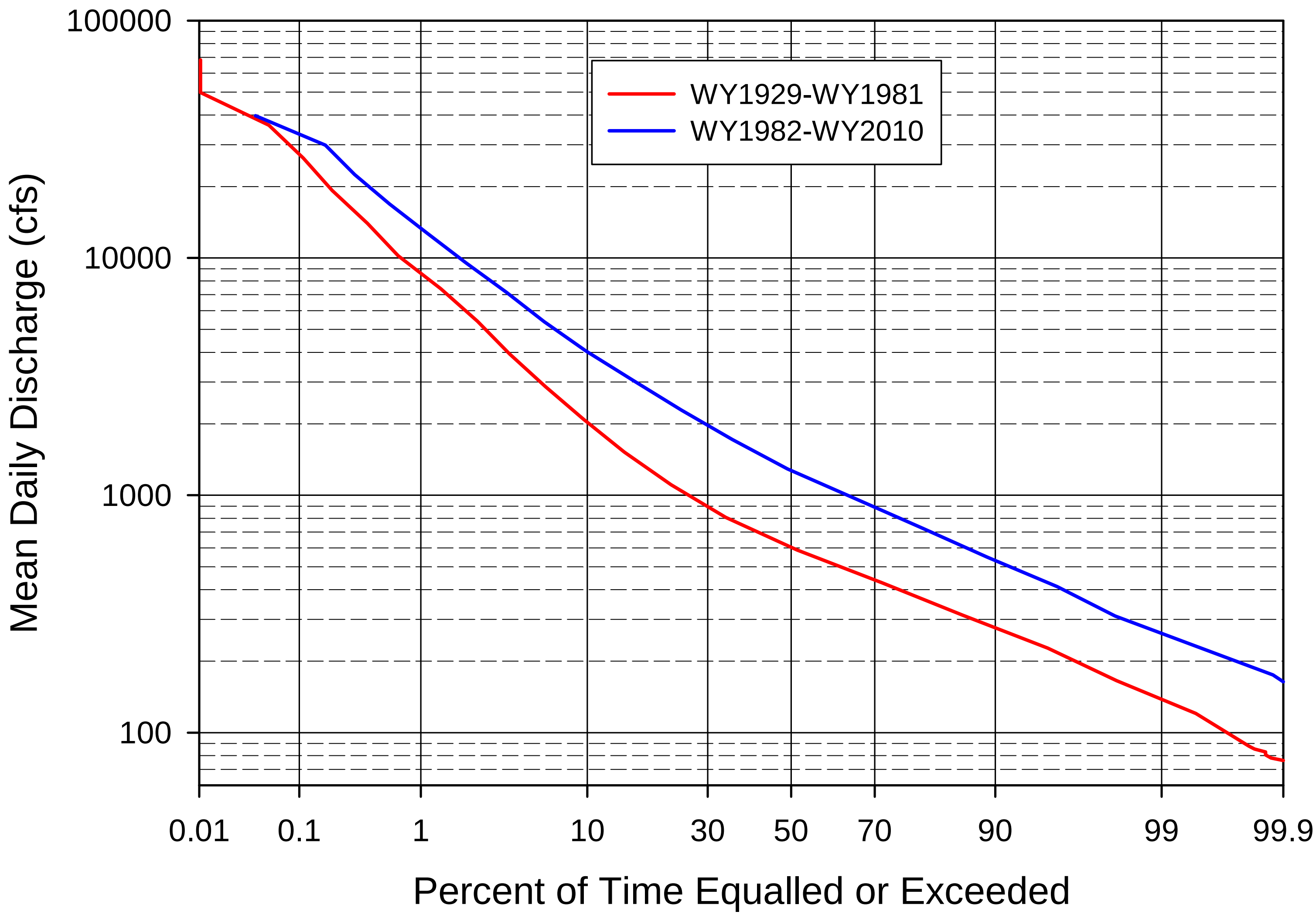

Figure 3. Mean daily flow-duration curves at the Waterloo gage for the periods from WY1929 through WY1980 and WY 1981 through WY2010.

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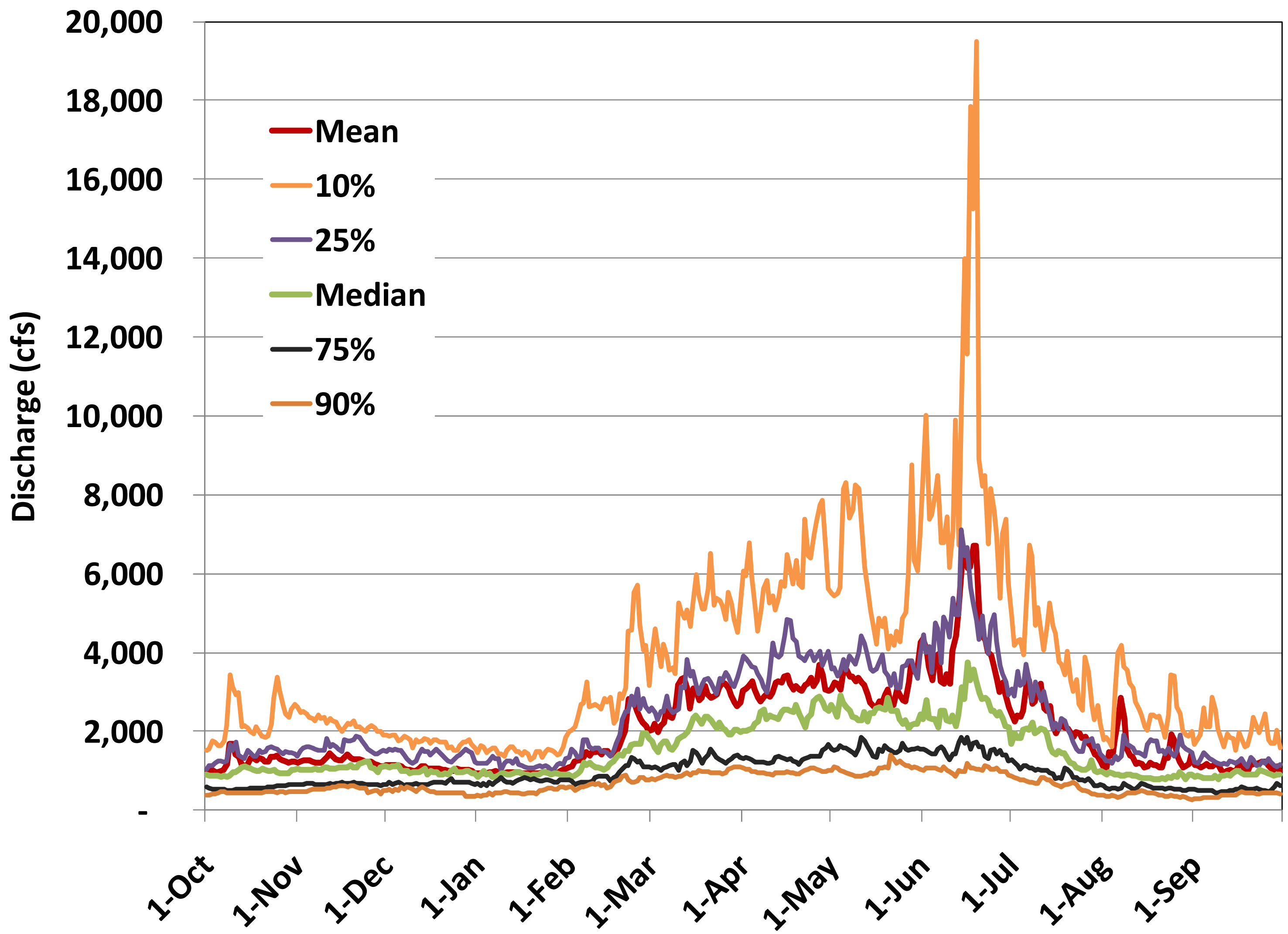


Figure 4. Mean, median hydrographs, along with the hydrographs for various exceedence percentages based on the recorded mean daily flows at the Waterloo gage during the period from WY1982 through WY2010.

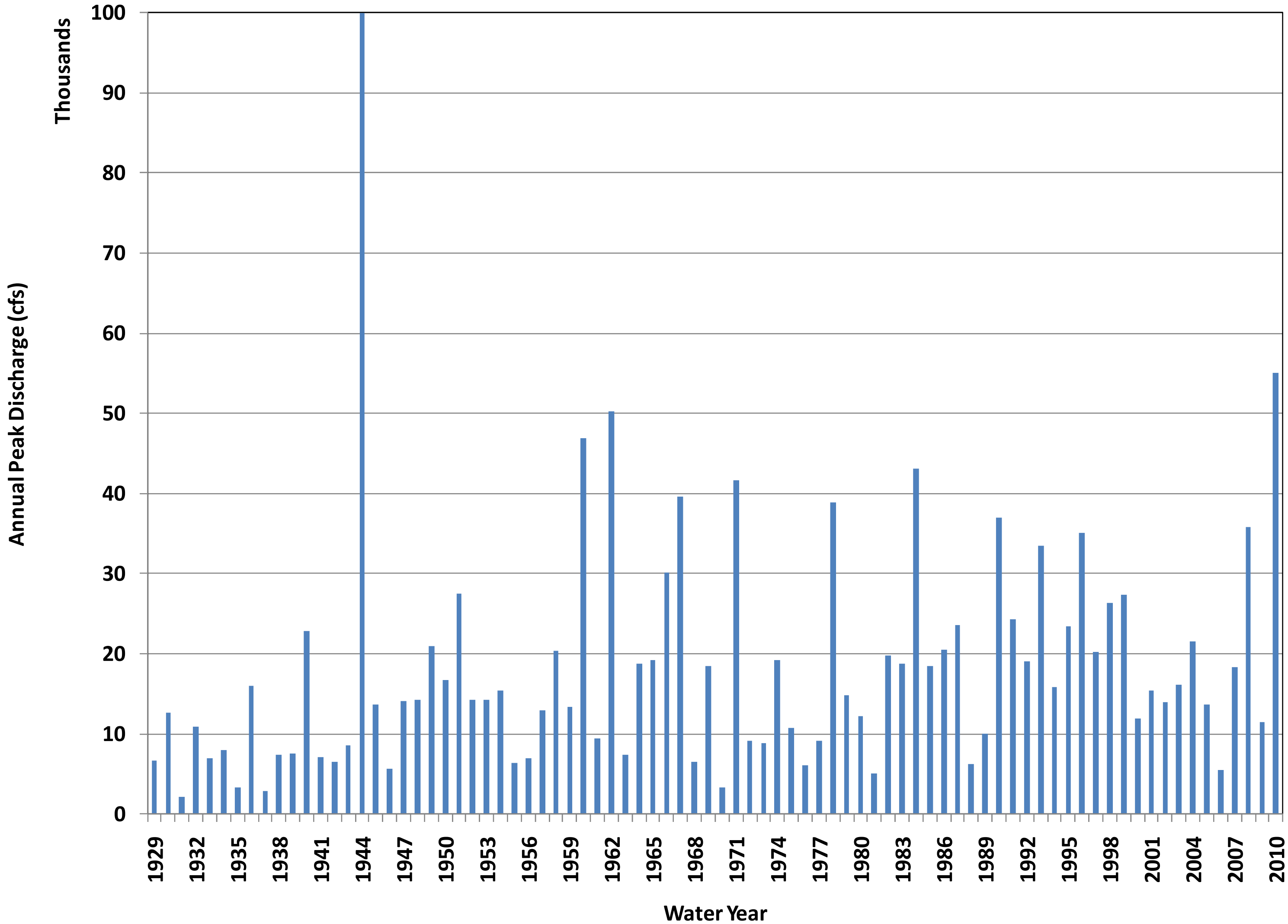


Figure 5. Annual peak discharges at the Waterloo gage for the continuous period of record from WY1929 through WY2010.

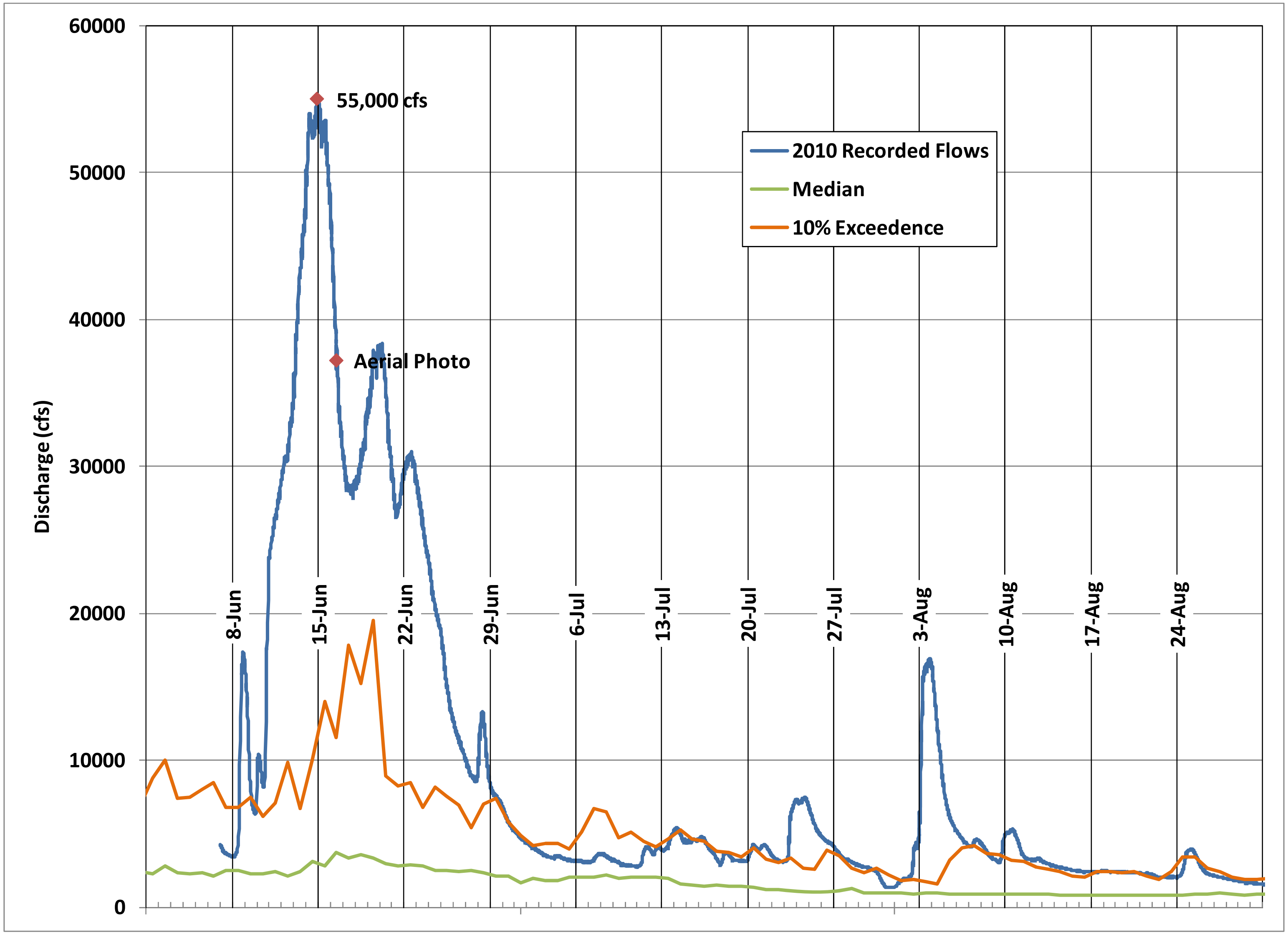


Figure 6. Provisional 2010 high flows measured at the Waterloo gage (15-minute resolution). Also shown is the peak discharge that occurred on June 14, the discharge at the time of the 2010 aerial photograph, and the median and 10% Exceedence (high) flow hydrographs for the period from WY1981 through WY2010.

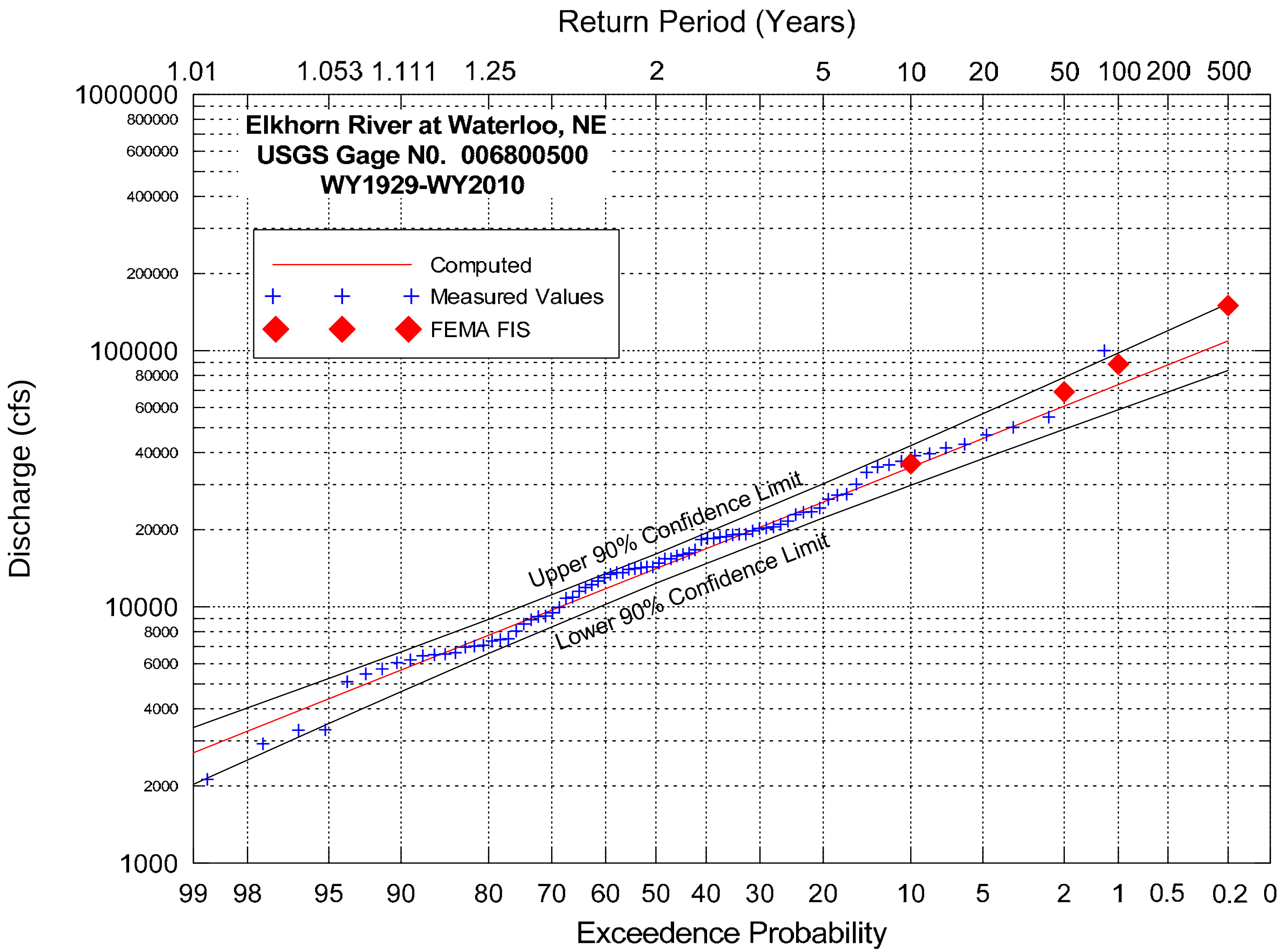


Figure 7. Flood-frequency curve for the Elkhorn River at Waterloo, based on the recorded annual peak flows from WY1929 through WY2010. Also shown are the estimated 10-, 50- and 100-year flood peaks from the Effective Flood Insurance Study (FIS) for Douglas County (FEMA, 2010) that are based on records for the period through WY1975

Director Tesar met with the field group at the Elkhorn Crossing Recreation Area boat ramp and provided insight about the concerns in that area that include erosion of the left (downstream view) river bank that has led to significant deposition along the right bank that now blocks the boat ramp, damage to the bank protection on the right bank downstream from the Recreation Area, and a perception that the channel has widened in the straight reach between the upstream end of Site 14 and Highway 36.

Field Site 1

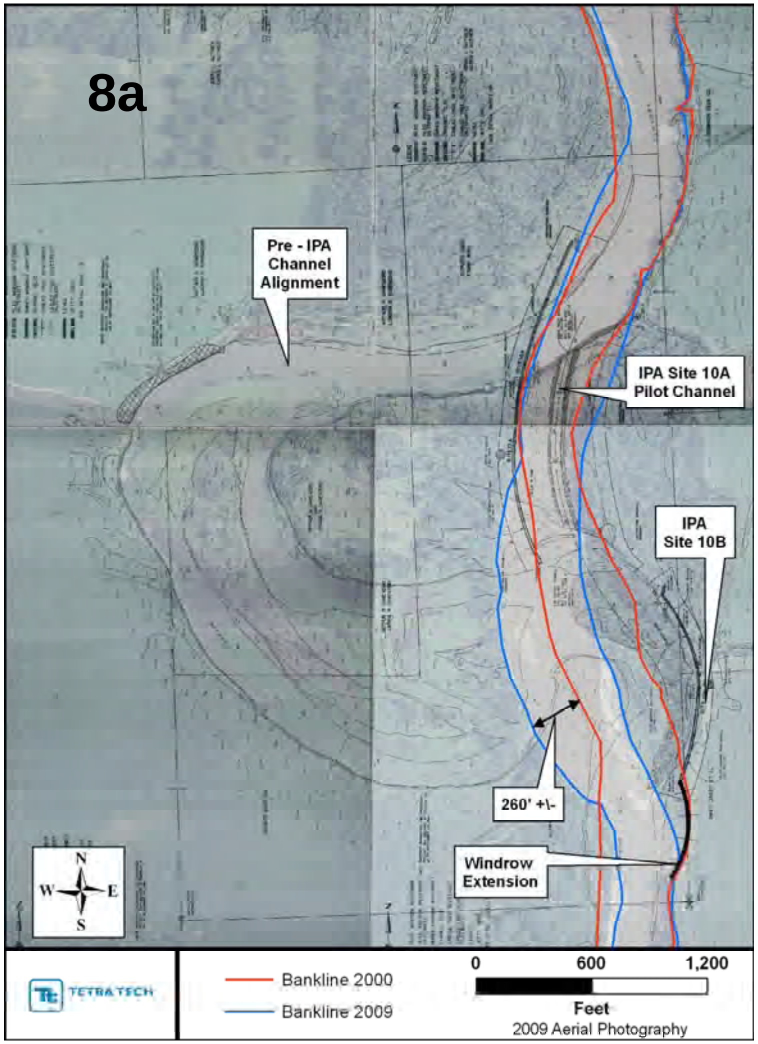
Field Site 1 is located on the left bank just downstream from the original IPA Site 10B, and consists of an approximately 500-foot long extension of the windrow revetment that was installed as part of the original project (**Figures 8a and 8b**). According to Mr. Bowen, the extension was constructed in the early 1990s after completion of the original project to stop erosion of the unprotected bank downstream from Site 10B. As the channel through the meander cutoff that was constructed as part of IPA Site 10A has continued to develop since construction, the river along the original Site 10B and most of the extension site has shifted toward the west through lateral erosion into the right bank, forming a vegetated bankline along the toe of the left bank and abandoning the protection. Based on the 2009 aerial photograph, the river was in direct contact with the bankline along the extension prior to the 2010 high flow, but a large sand bar is now present along the site (**Figure 9**). The rock riprap on the point in the left bank at the downstream end of the windrow extension appears to be intact and functioning as intended, although the rock appears to be relatively thin. The right bank upstream from Field Site 1 will continue to erode to the west and south, approximately as shown by the dashed red lines and arrow in **Figure 10**, expanding the size of the left-bank sandbar. The overbank area into which the river is eroding is the remnant of the former meander bend that was cutoff as part of the original IPA and a small wooded area directly to the south. Whether continued erosion into this area is a potential problem is uncertain, but it is recommended that the area, including the rock point on the left bank (**Figure 11**), be monitored and corrective actions taken if required. These actions could include windrow revetment along the right bank and maintenance of the rock points.

Field Site 2

Field Site 2 is located near the middle of a left-bank jetty field that was constructed prior to the IPA project (**Figure 12**). The jetty field appears to be very stable and is functioning as intended (**Figure 13**). The problem area occurs just downstream from a large jetty near the middle of the jetty field where surface runoff has created a headcut and gully that is eroding back into the adjacent field (**Figures 14 and 15**). Sufficient information is not available to be certain about the source of the runoff that has caused the gullying, but it appears to have developed during the 2010 high-flow event, most likely due to overbank flows returning to the channel. Since it has developed in the low area/minor swale in the overbank area, both future overbank flows and local runoff from the adjacent field will continue to erode and expand the gully. Although expansion of the gully will damage the field, it is unlikely to endanger the stability of the overall dike field and bankline in this area; thus, this problem is not likely to affect the overall stability of the IPA reach.

There are two potential options for repairing and stabilizing the site:

1. Construct an NRCS-type low-level embankment and drop inlet, or



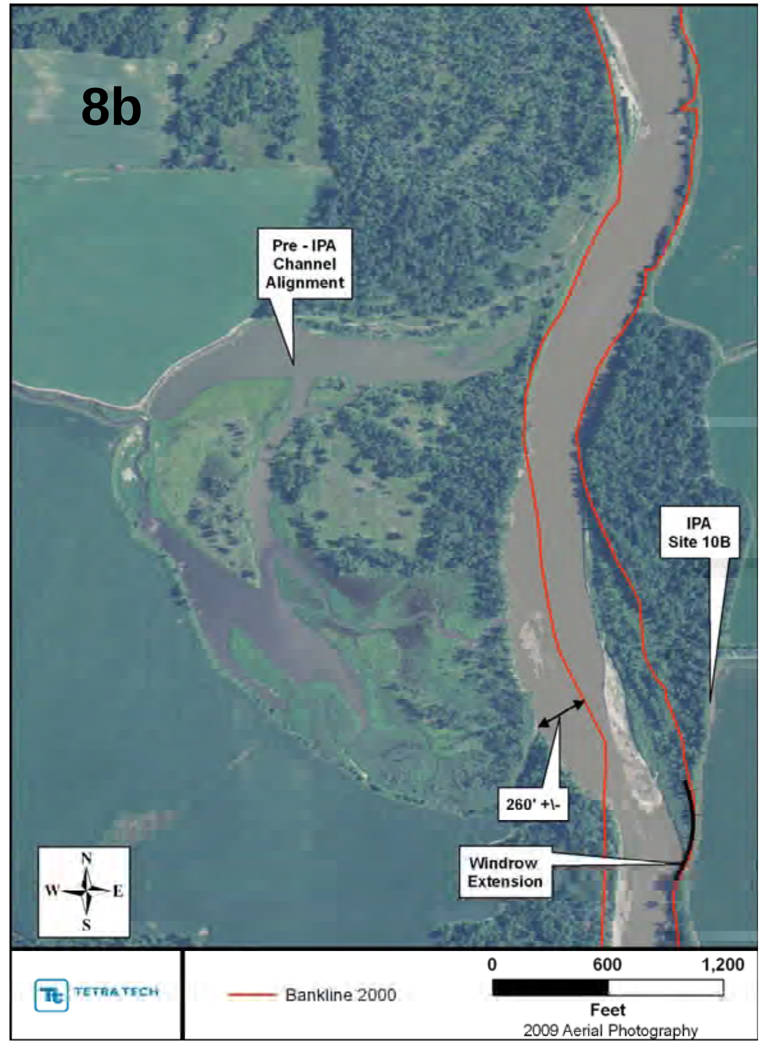


Figure 8a. Elkhorn IPA construction plans at Sites 10A and 10B (Field Site 1) overlaid on the September 22, 2009 aerial photograph. Also shown is the approximate extent of the windrow extension and the river banklines in October 2000 and 2009.

Figure 8b. September 22, 2009 aerial photograph of Field Site 1 (IPA Sites 10A and 10B) (discharge ~1,040 cfs) with construction plan overlay removed. Also shown are the banklines from the October 2000 aerial photograph.

Figure 9. Panoramic view of Site 10B and the downstream end of Site 10A taken on October 6, 2010. Note the vegetated bankline along the windrow extension, large sand bar along the base of the bank, and erosion of the right bank (left side in photo).

Figure 10. Expected trajectory of future bank erosion on the left bank at Field Site 1 (IPA Site 10A).

Figure 11. View looking downstream of rock-protected point at the downstream end of the windrow extension at Site 10B.

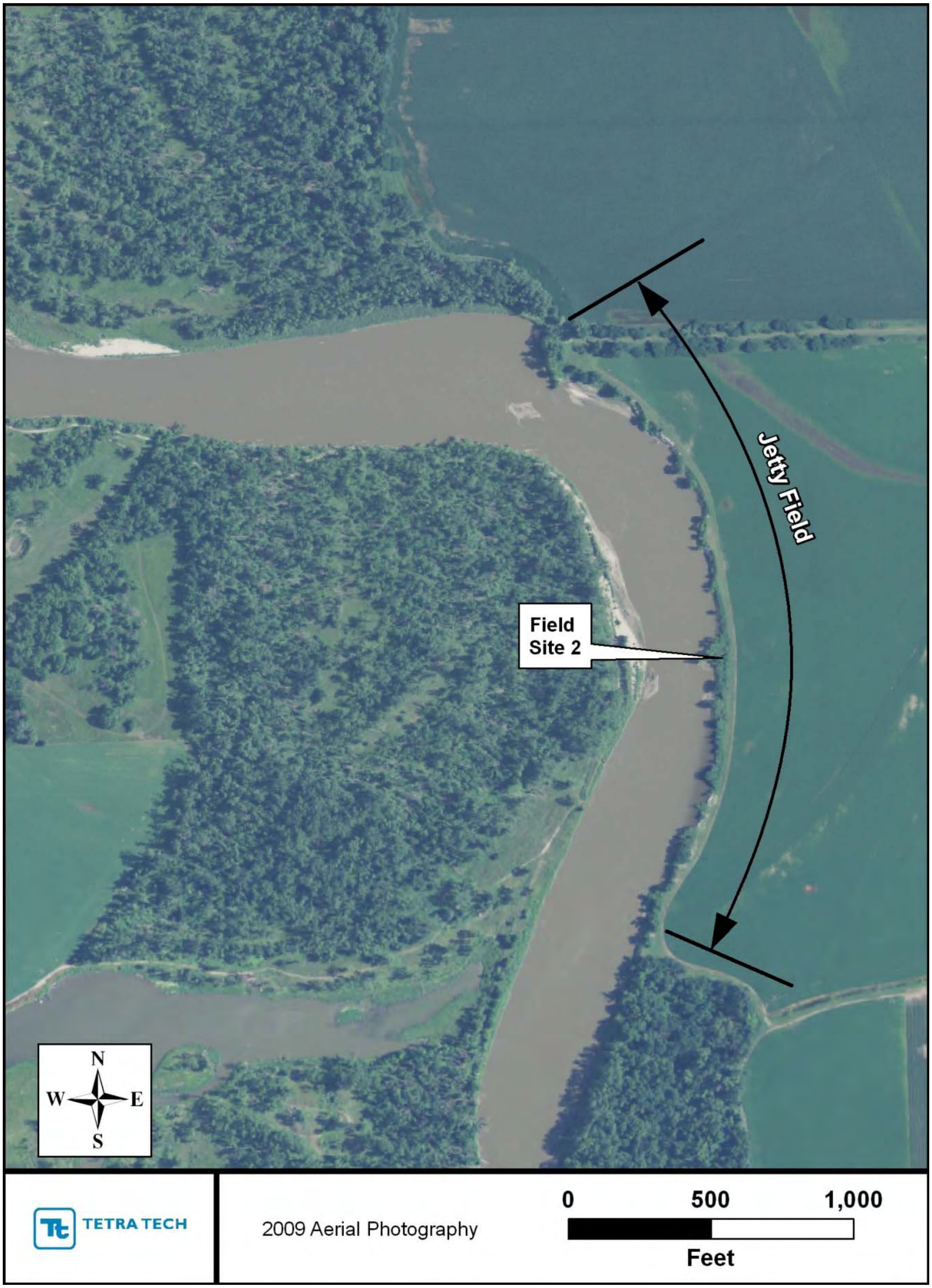


Figure 12. Aerial photograph of the left-bank jetty field and Field Site 2 taken on September 22, 2009, when the discharge was ~1,040 cfs.

Figure 13. View looking upstream along the upstream portion of the jetty field from Field Site 2.

Figure 14. Rock jetty and mouth of headcut just downstream from the jetty at Field Site 2.

Figure 15. View looking downstream in the headcut at Field Site 2.

2. Construct rock grade-controls or a rock chute in the head cut from the top to the base of the bank, and formalize a drainage swale that would connect to the upstream end of the protection to insure that future flows do not flank the installation.

The first solution would be problematic if the erosion is due to overbank flows, because the embankment would potentially block flows from returning to the channel at this location, essentially pushing the problem farther downstream. The first solution would also require additional grading of the area in order to concentrate the overland flows and provide adequate depths and prevent blockage at the drop inlet. The second solution would work regardless of the runoff source, but care would need to be taken in the placement of the revetment to avoid local scour around the protection that could make the problem worse.

Given the available topography, the second solution is recommended. For this solution, the existing bank would be graded to develop a more defined drainage swale and then protected by the placement of rock riprap of an appropriate size along the surface (**Figure 16**). It is also recommended that a granular filter be placed under the rock rather than filter fabric, which has caused rock to launch in nearby locations. Care will need to be taken to tie the upstream and downstream ends of the revetment into the existing spur dike rock and channel bank, respectively, as well as to continue the placement of rock down to an appropriate toe-down depth below the channel to protect against local scour. The layer of riprap should extend up to the top of bank, where it should be thickened and keyed into the bank to prevent overtopping flows from undercutting the revetment.

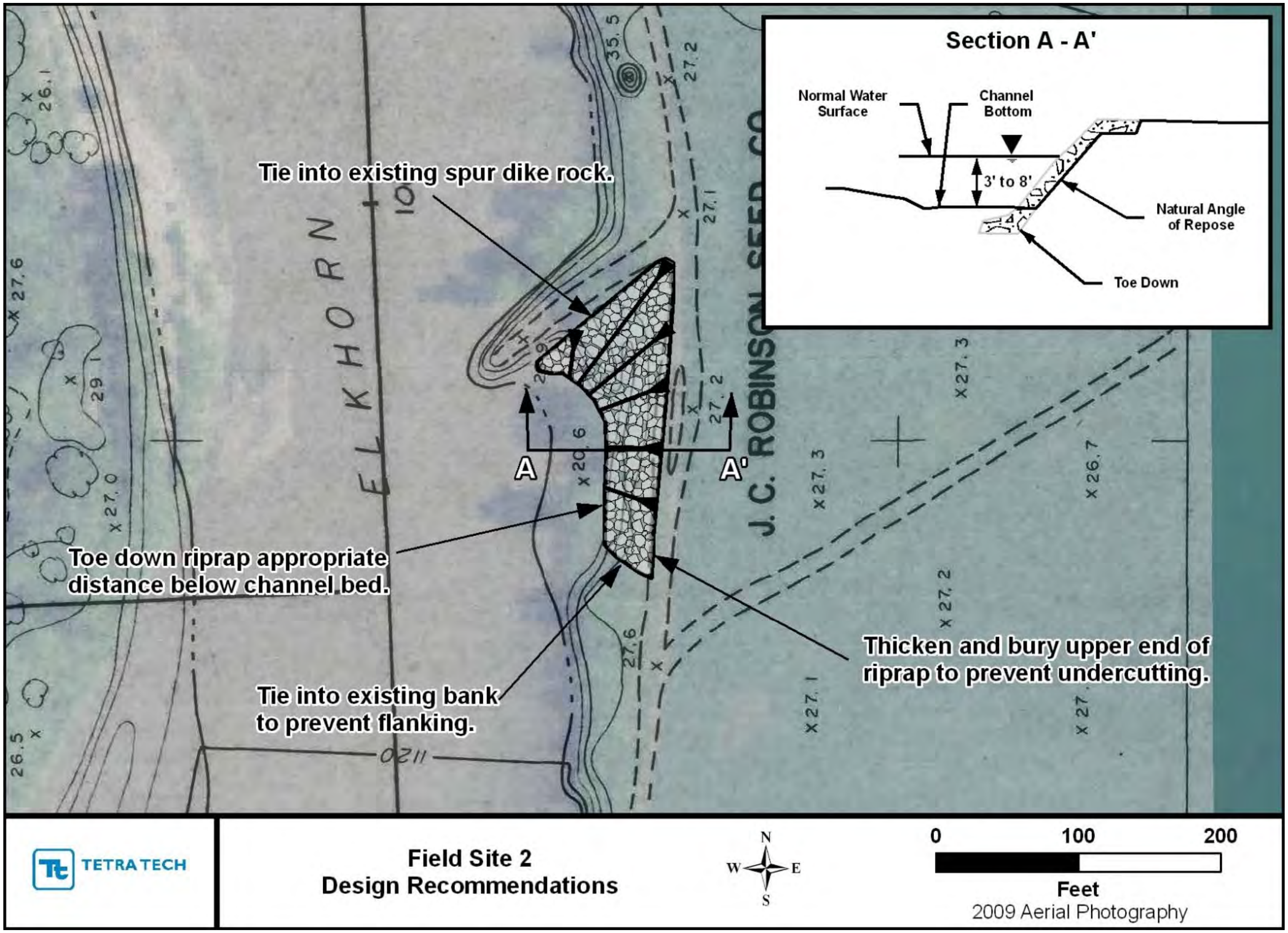


Figure 16. Recommended stabilization measures for Field Site 2.

Field Site 3

Field Site 3 is located along an approximately 900-foot reach in the middle of IPA Site 11 where the windrow revetment that was applied at a rate of 3.0 tons/foot has been fully launched (**Figure 17**) and there are now gaps in the rock that could allow undesirable bank erosion during future high flows (**Figure 18**). At the upstream end of the site, the rock is in place at the toe of the bank, but the part of the bank above the ~1,500 cfs water-surface is retreating behind the rock (**Figure 19**). With additional upper-bank retreat at this location, the channel will expand on the back side of the ridge formed by the rock, and the existing bank protection will no longer be effective. The bankline along the entire length of IPA Site 11 has shifted very little since the project was constructed (**Figure 20**). Approximately one-fourth of the rock in the windrow revetment in the portion of IPA Site 11 downstream from Field Site 3 remains to be launched, and this portion of the site appears to be stable and functioning as intended (**Figure 21**). Most of the rock has launched in the portion of IPA Site 11 between Field Site 3 and the mouth of Rawhide Creek and this bankline also appears to be stable (**Figure 22**). It is recommended that additional rock be added to the bank at Field Site 3 at a rate of 2.5 to 3 tons/foot over a total length of about 900 feet to prevent additional erosion of this bank that could impact the adjacent overbank area, and potentially allow flanking of the downstream portion of IPA Site 11.

Field Site 4

Field Site 4 is an approximately 1,500-foot length of eroding left-bank immediately downstream from IPA Site 12, where the bankline has retreated laterally on the outside of the bend by over 150 feet since construction of the project (**Figures 23 and 24**). In fact, as shown in Figure 23, nearly all of the bank retreat occurred after October 2004. The left bank retreat has been accompanied by accretion on the opposite (right) bank, resulting in little, if any, change in average channel width in this portion of the reach (**Figure 25**). The overbank into which the bankline at Field Site 12 is eroding is a wooded area that developed through accretion on the inside of the bend at IPA Site 11. This type of erosion is typical of meander bend development in which the bends tend to migrate laterally and in the downstream direction (Brice and Blodgett, 1978; Shen and Schumm, 1981), with accompanying accretion on the inside of the bend. Whether the indicated erosion at Field Site 4 is a problem is uncertain. If PMRNRD considers it to be problematic, the erosion can be checked by installing windrow revetment at the rate of 2.5 tons/foot, consistent with IPA Site 12 that appears to be stable and performing as intended.

Field Site 5

Field Site 5 is located on the left bank near the middle of IPA Site 12 where the bankline has retreated laterally by up to 100 feet over a length of approximately 600 feet since October 2004 (**Figures 26a and 26b**). Some rock is still present at the base of the bank, and concrete rubble has apparently been added to the site since the original construction (**Figure 27**). The IPA plans called for piled windrow revetment from approximately the downstream end of Field Site 5 through the bend upstream from the site. The construction plans also called for cabled-tree revetment for about 550 feet immediately downstream of Field Site 5, but windrow revetment was placed in lieu of the cabled trees (Table 1, Note 8). The portions of the site up- and

Figure 17. Aerial photograph of Field Site 4 taken on September 22, 2009, when the discharge was ~1,040 cfs.

Figure 18. Panoramic view of Field Site 3 showing the launched windrow rock and eroding banks.

Figure 19. View looking downstream at the upstream end of Field Site 3 where the windrow revetment has fully launched and the rock is now forming a low-elevation bench above which the bank is retreating.

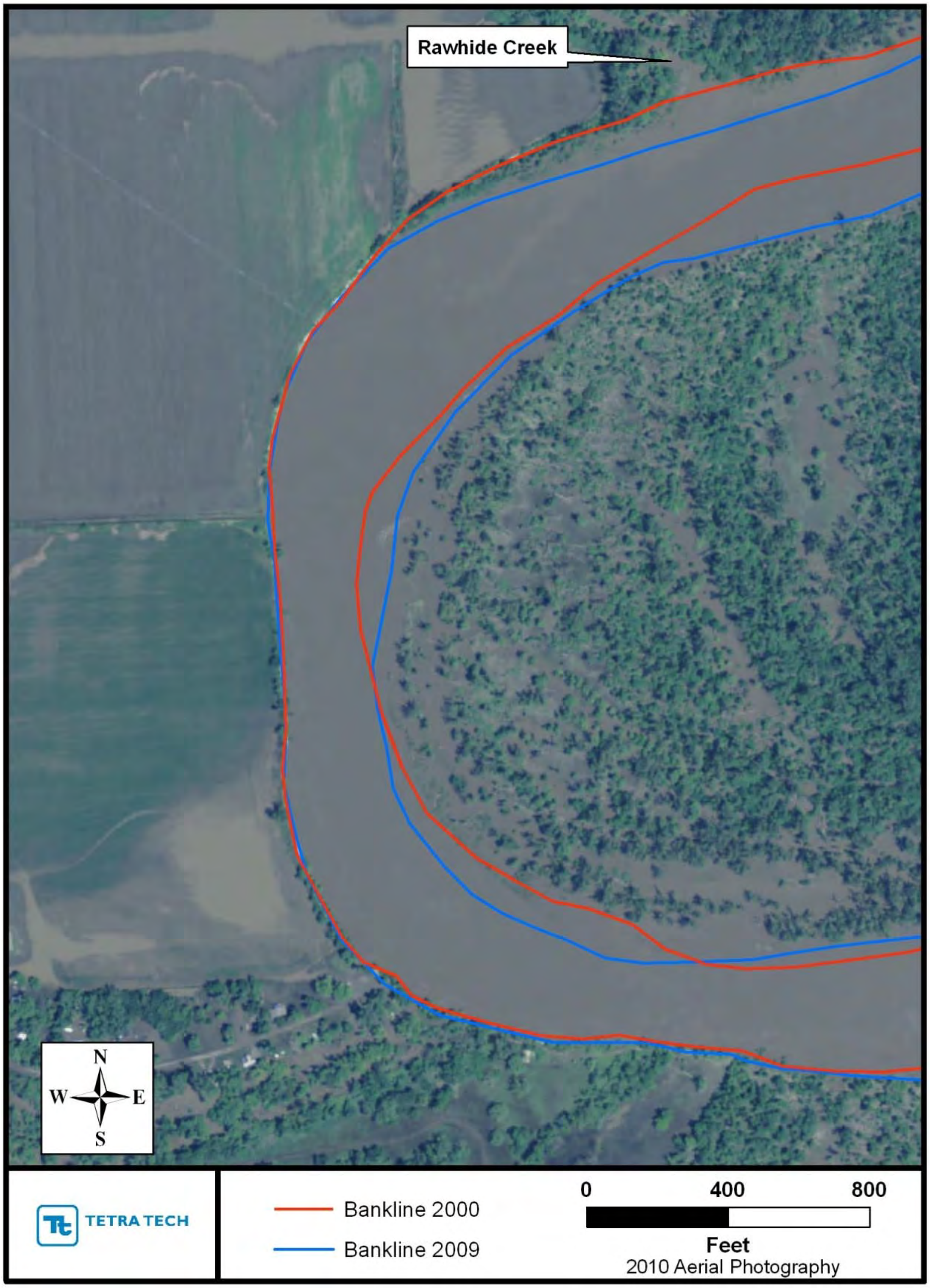


Figure 20. Aerial photograph at IPA Site 11 taken on June 16, 2010, when the discharge was ~37,250 cfs. Also shown are the banklines from the October 2000 and September 2009 photographs.

Figure 21. Downstream portion of IPA Site 11 where the about one-fourth of the original windrow revetment remains to be launched.

Figure 22. View looking upstream at the upstream portion of IPA Site 11 where most of the windrow revetment has launched, and the bankline appears to be intact and stable. The upstream end of Field Site 3 is visible on the left side of the photo.

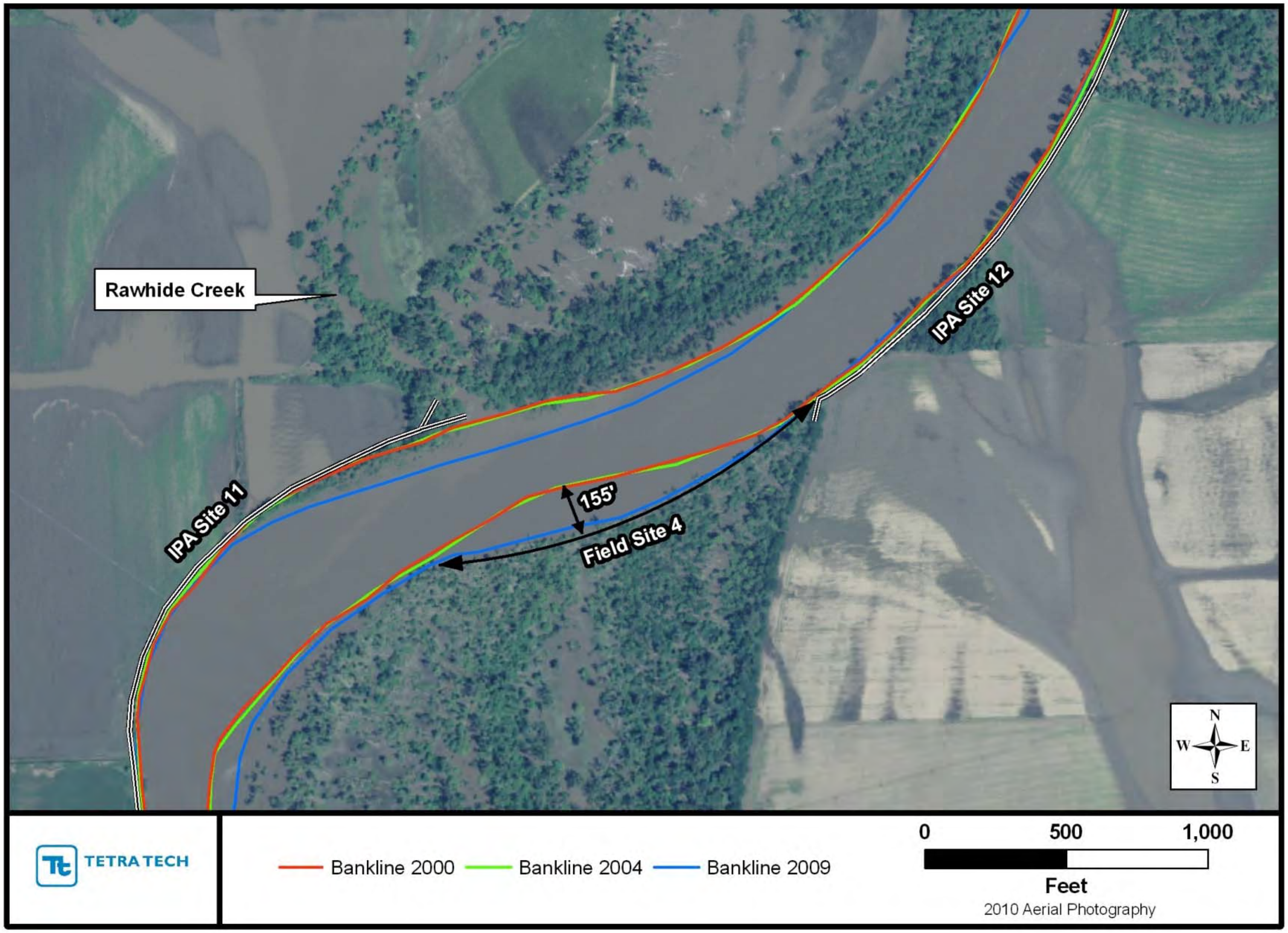


Figure 23. Aerial photograph of the downstream end of IPA Site 12 and Field Site 4 taken on June 16, 2010, when the discharge was ~37,250 cfs. Also shown are the banklines from the October 2000 and September 2009 photographs.

Figure 24. Panoramic view looking downstream at Field Site 4 taken on October 6, 2010.

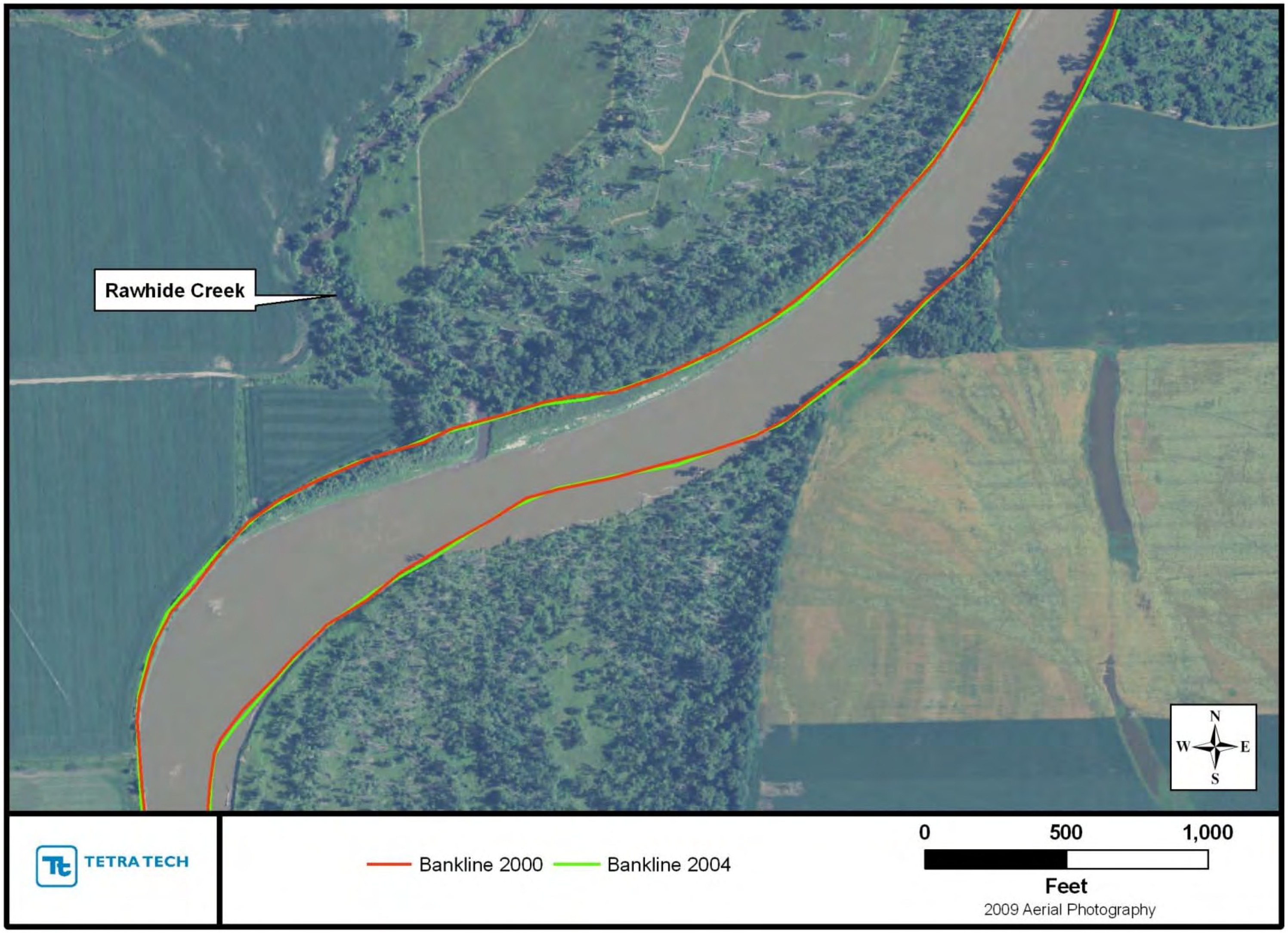


Figure 25. Aerial photograph of the downstream end of IPA Site 12 and Field Site 4 taken on September 22, 2009, when the discharge was ~1,040 cfs.

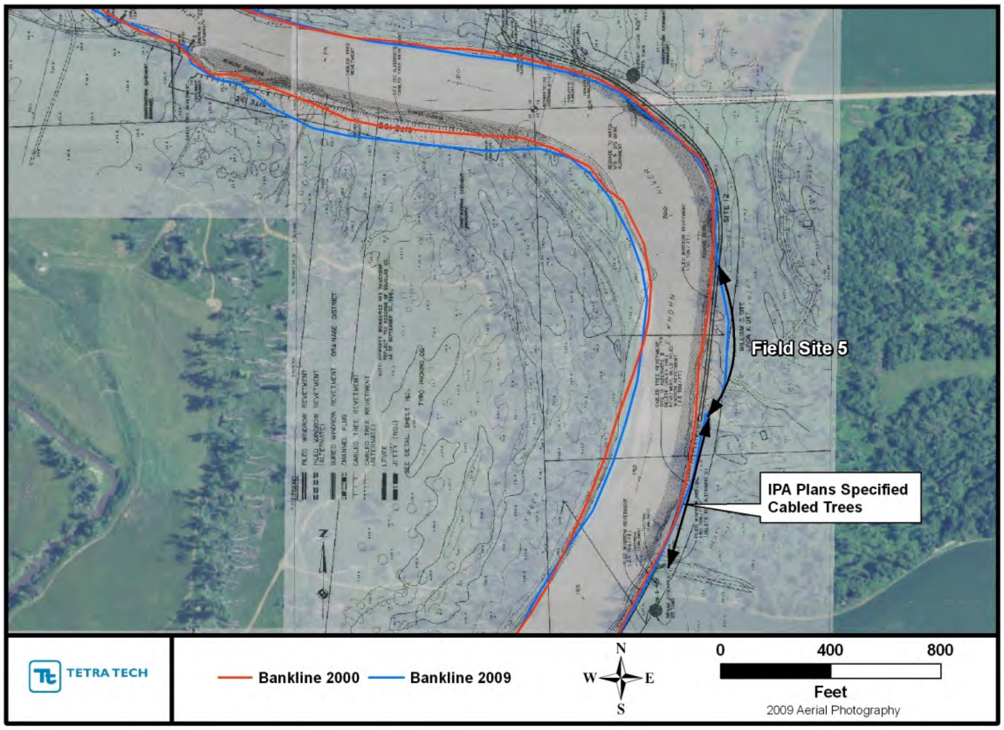


Figure 26a. Construction plans for the middle portion of IPA Site 12 overlaid on the September 2009 aerial photograph. Also shown are the October 2000, October 2004 and September 2009 banklines showing the ~100-foot bank migration at Field Site 5.

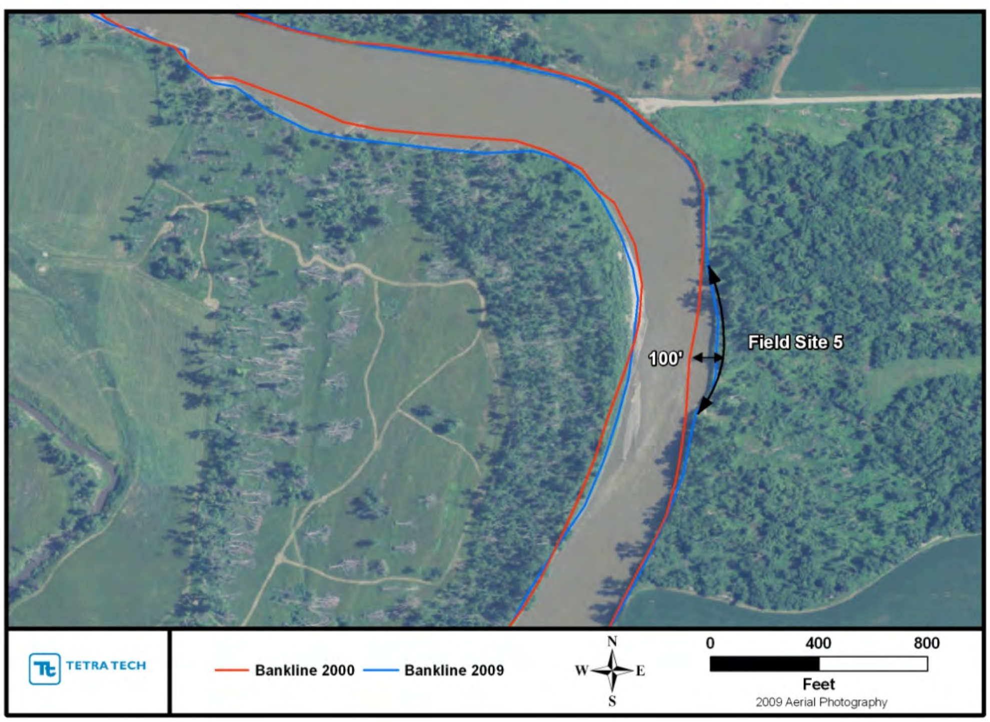


Figure 26b. September 22, 2009 aerial photograph (Discharge ~1,040 cfs) of Field Site 5 showing the October 2000 and October 2004 banklines.

Figure 27. Panoramic view of Field Site 5 taken on October 6, 2010.

downstream from Field Site 5 appear to be in good condition, but Field Site 5 is unstable and eroding, with significant gaps in the existing rock/concrete rubble protection. The overbank area along this site consists of moderately thick woods. If continued lateral erosion into the wooded area is considered to be a problem, it may be possible to check the erosion with additional windrow revetment, applied at a rate of 2.5 to 3.0 tons/foot. Before such additional rock is installed, however, it would probably be prudent to study this area in more detail, including evaluation of the construction notes, to gain a better understanding of why the indicated bank erosion has occurred to insure that the recommended fix is, in fact, appropriate.

Field Site 6

Field Site 6 is an approximately 1,400-foot long reach of eroding right bank where cable-tree revetment was installed as part of the original IPA project (Site 13C) (**Figures 28a and 28b**). The bankline along this site is quite unstable, including significant areas of undercut, slumping banks (**Figures 29 through 31**), and it migrated laterally by over 110 feet in some locations between construction of the original project and the September 2009 aerial photograph. The June 2010 aerial photograph indicates additional lateral erosion at this site of up to 20 feet in some locations. The overbank area at this site is sparsely to moderately vegetated with large trees. If continued lateral erosion into this area is problematic, the erosion can be checked with windrow revetment applied at a rate of 3.0 tons/foot.

Field Site 7

Field Site 7 is a riprap point at the downstream end of the levee that was constructed in the late 1990s where the rock has been stripped off the underlying filter fabric (**Figures 32a, 32b and 33**). Although the erosion hazard does not appear to be severe at this location, the site can be repaired by replacing the rock with riprap of an appropriate size. It is recommended that a granular filter be placed under the rock rather than the filter fabric that was previously used to limit the likelihood that the rock will simply slide off the underlying filter.

It is interesting to note that the left bank opposite Field Site 7 has migrated to the south by over 150 feet in some locations and the right bank adjacent to the levee and buried windrow at the upstream end of Site 13C has accreted by a similar amount.

Field Site 8

Field Site 8 is an approximately 300-foot long scallop that has developed in the left bank just downstream from a pre-existing rock spur located at the downstream end of the windrow revetment for Site 13B (**Figures 34 and 35**). The bankline at this location retreated into the overbank by nearly 80 feet downstream from the rock spur. The IPA construction plans called for cabled-tree revetment for approximately 1,000 feet downstream from the rock spur. The cabled-trees were not installed during the original construction, but windrow revetment was placed in this area in the late 1990s (**Figure 36;** Table 1, Note 16). With the exception of the scalloped area, the remainder of the bankline appears to be relatively stable at the present time.

Recommended repairs at Field Site 8 include realigning the bankline to eliminate the scallop, and placement of appropriately sized rock riprap on the realigned bankline (**Figure 37**).   The length of the realigned bankline would be about 350 feet, and the area that would need to be filled between the existing and new bankline is about 17,000 ft^2. Assuming a bank height of 12

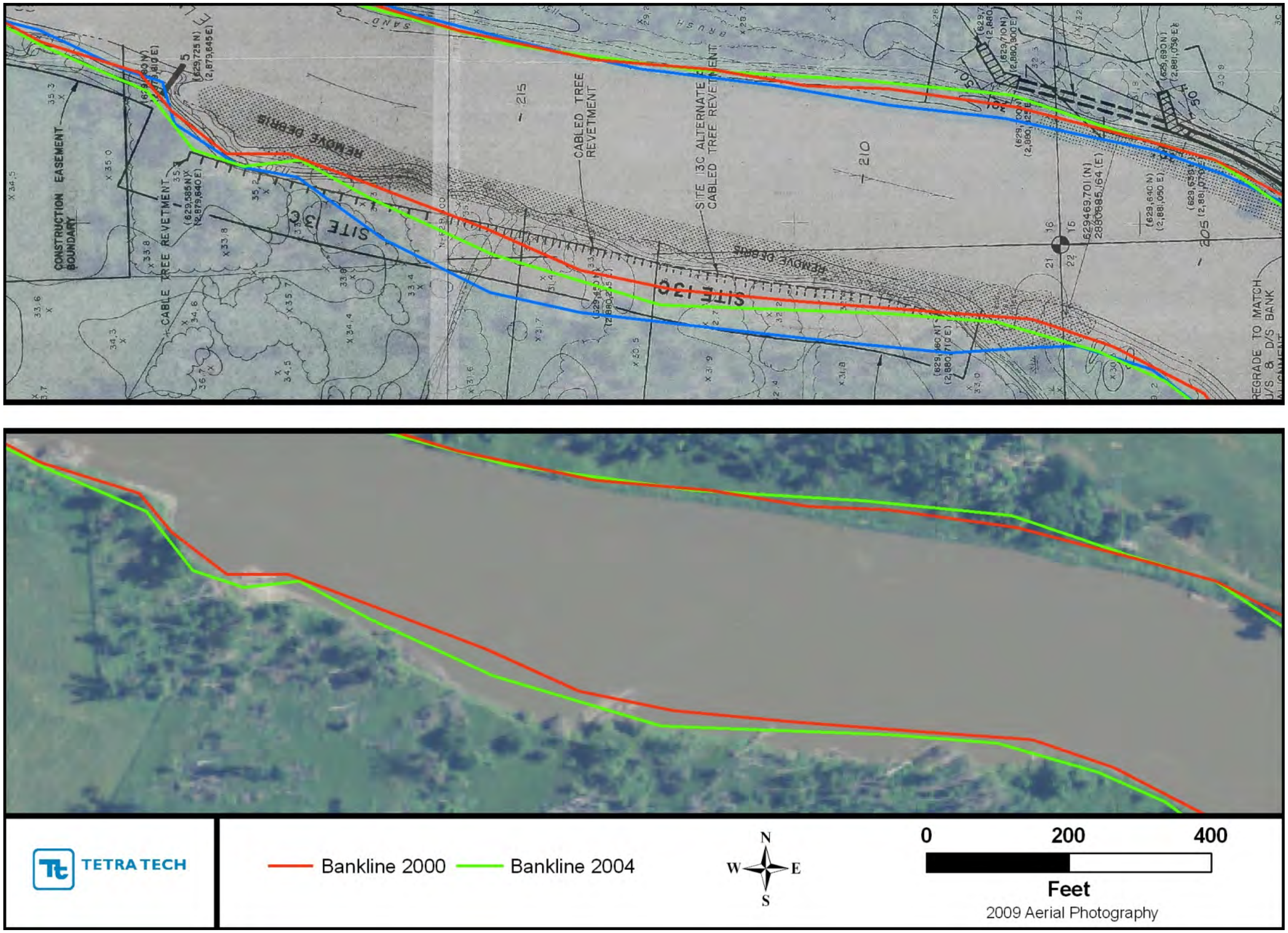


Figure 28a. Construction plans for the cabled-tree revetment at IPA Site 13C (Field Site 6) overlaid on the September 2009 aerial photograph. Also shown are the October 2000, October 2004 and September 2009 banklines.

Figure 28b. September 22, 2009 aerial photograph (Discharge ~1,040 cfs) of IPA Site 13B and Field Site 6 showing the October 2000 and October 2004 banklines.

Figure 29. View looking downstream of the eroding bankline at Field Site 6.

Figure 30. View looking near middle of the eroding bankline at Field Site 6.

Figure 31. View looking upstream of the eroding bankline at Field Site 6.

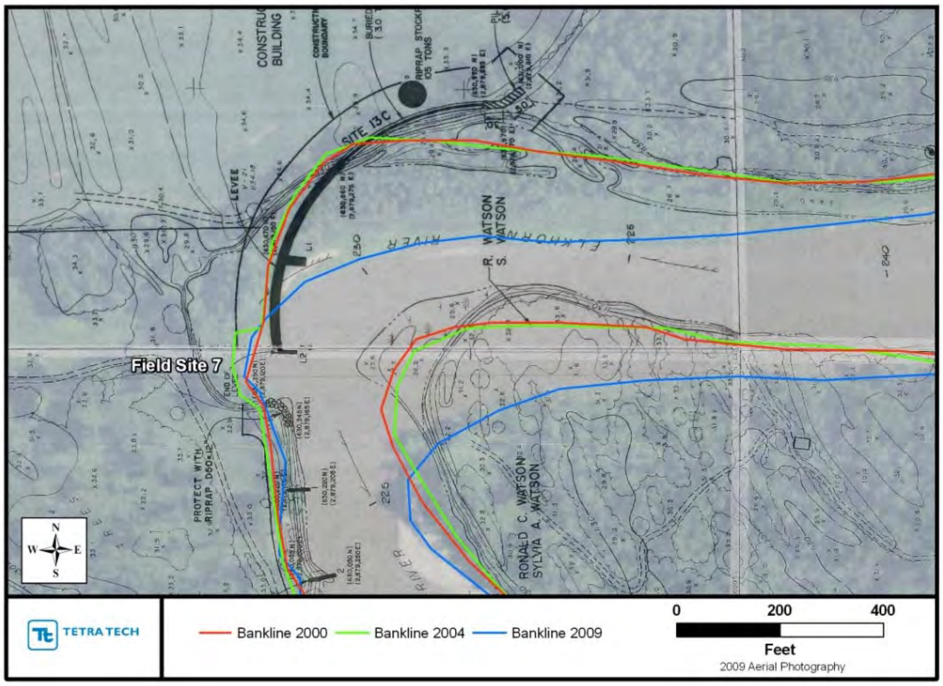


Figure 32a. Construction plans for the rock jetties, levee and buried windrow revetment at IPA Site 13C and the recently constructed rock point at Field Site 7 that was damaged during the 2010 high flow overlaid on the September 2009 aerial photograph. Also shown are the October 2000, October 2004 and September 2009 banklines.

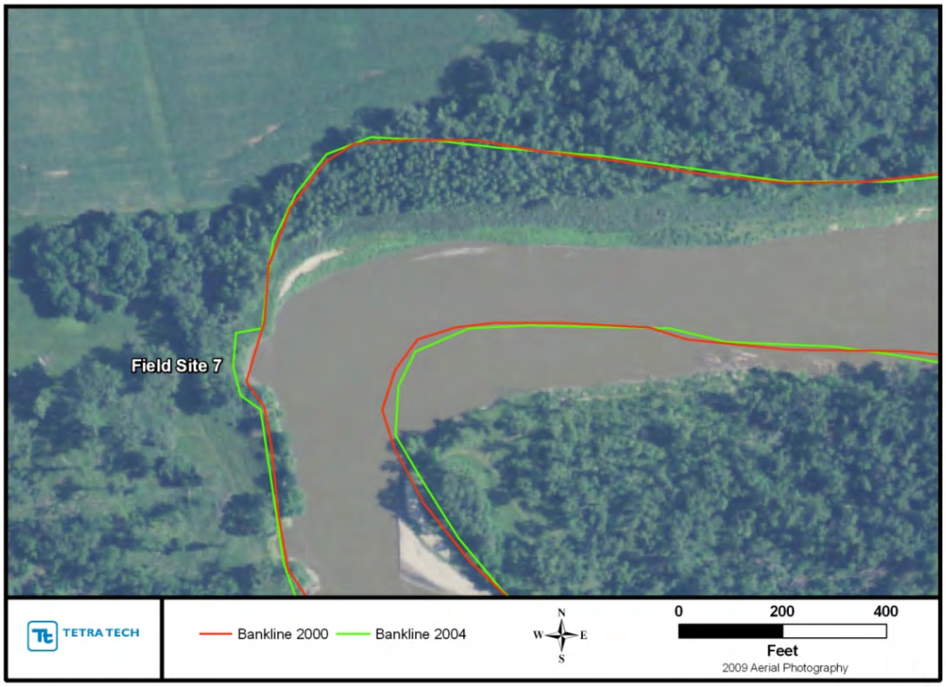


Figure 32b. September 22, 2009 aerial photograph (discharge ~1,040 cfs) of the upstream portion of IPA Site 13B and Field Site 7 shown in Figure 31a, including the October 2000 and October 2004 banklines.

Figure 33. View looking upstream at the damaged rock point at Field Site 7. Note exposed filter fabric that was placed under the rock.

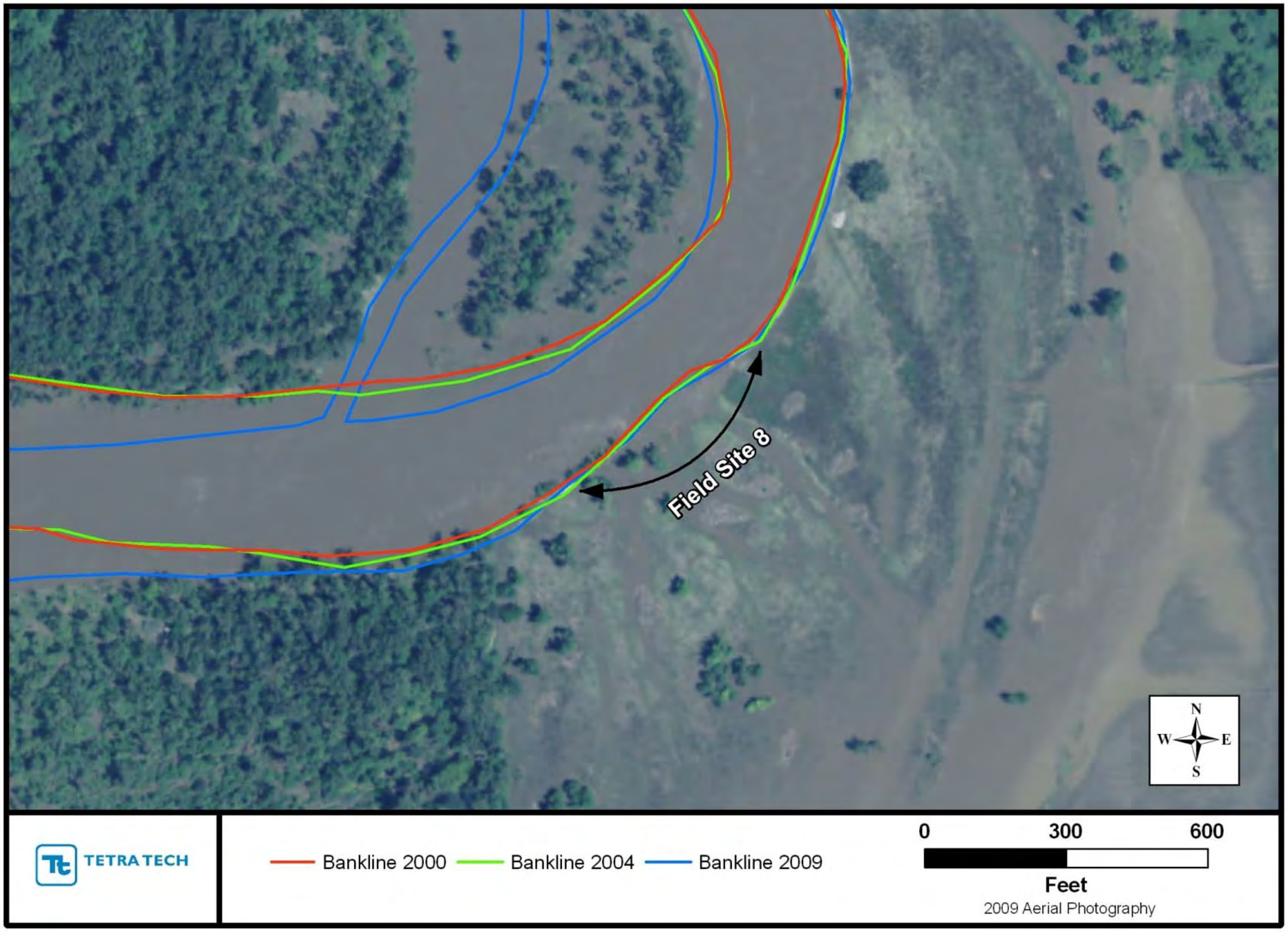


Figure 34. Aerial photograph taken on June 16, 2010 (discharge ~37,250 cfs) of Field Site 8. Also shown are the banklines from the October 2000, October 2004 and September 2009 aerial photographs.

Figure 35. View looking downstream flows from the pre-existing rock spur at Field Site 8 across the scallop in the left bank that developed during the June 2010 high flow.

Figure 36. View looking upstream along the bankline in the downstream portion of IPA Site 13B where cabled-tree revetment was called for in the construction plans. Note the launching rock along the bank. A portion of Field Site 8 is visible on the far left side of the photo.

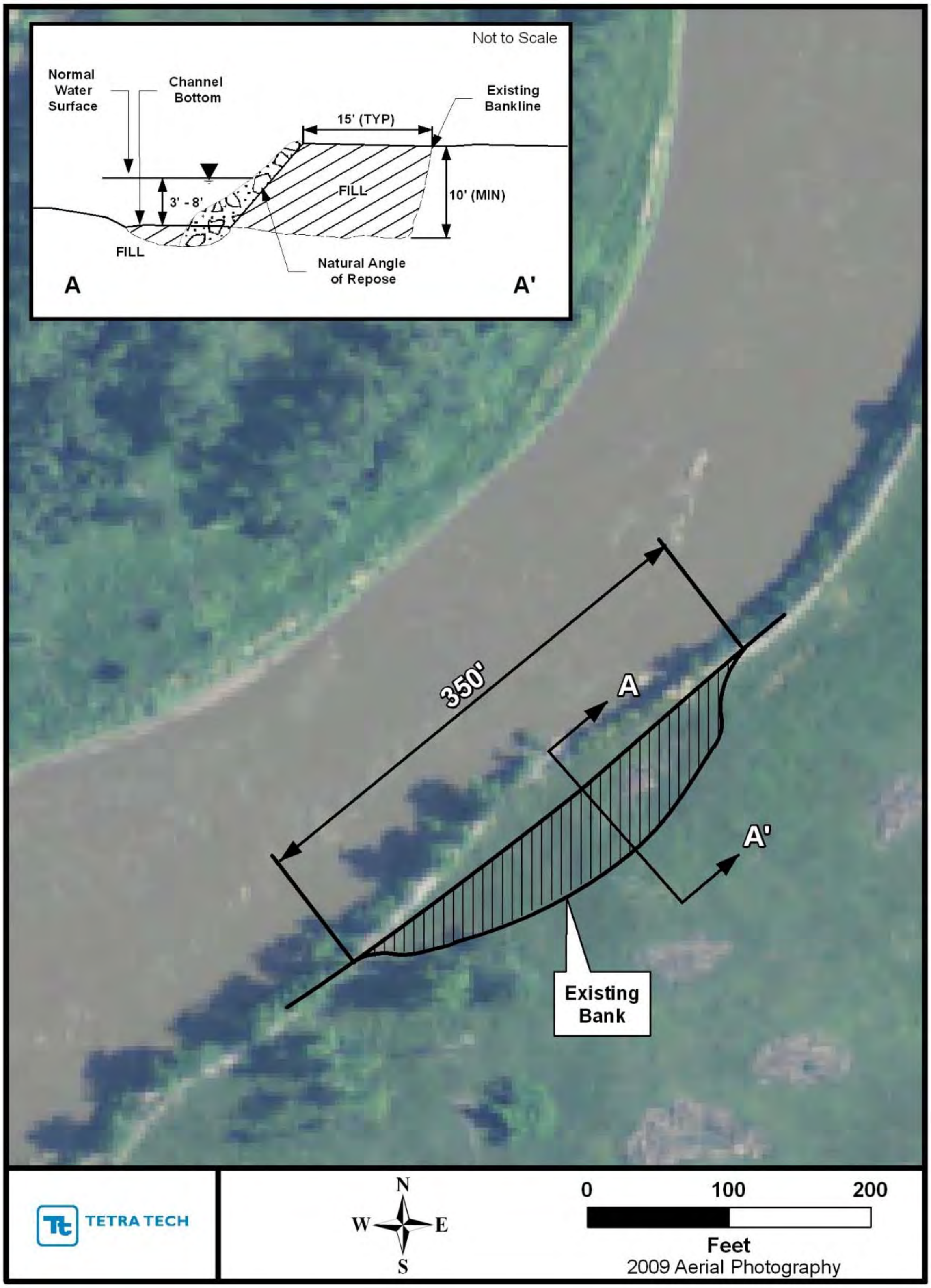


Figure 37. Recommended re-alignment of bankline at Field Site 8 and typical cross section of the rock riprap that should be placed along the realigned bank.

feet, based on the topography in the IPA plans, plus approximately 5 feet of local scour on the outside of the bend, a total fill volume of about 7,600 yd^3 would be required.

The upstream portion of Site 13B, where piled windrow revetment was installed at a rate of 3.0 tons/foot, appears to be relatively stable and functioning as intended. Most of the windrow rock has been launched , with 10 to 15 percent of the rock still in-place at the top of the bank along much of the site (**Figures 38 and 39**).

Field Site 9

The IPA construction plans for Site 13B called for the piled windrow revetment to end about 300 feet downstream from the edge of the existing wooded area, with about 540 feet of cabled-tree revetment extending upstream from the end of the windrow (**Figure 40**). The cabled-trees were not installed ; however, windrow revetment was placed in this part of the site at some point either during or after construction of the IPA (**Figure 41**). This portion of the site is also in good condition and is functioning as intended. A modest amount of bank erosion is now occurring in the approximately 300-foot reach just upstream from the existing rock (Figures 41 and **42**). Woody debris is currently providing some protection for the site, but continued erosion is possible. A large woody debris jam has formed along the right bank just upstream from the site that appears to be constricting high flows against the left bank at Field Site 9 (**Figures 43 and 44**). A similar, but somewhat smaller, debris jam, was present about 400 feet downstream from the current location as late as 2004 that may have contributed to the erosion tendencies at the upstream end of IPA Site 13B (**Figure 45**). The bankline at Field Site 9 has migrated laterally into the wooded area by up to 50 feet in some places since October 2004.

While Field Site 9 is not in a critically unstable condition at the moment, it should be monitored after future high flows, and the windrow revetment extended upstream at an application rate of 3.0 tons/foot, if necessary. Although the debris jam likely provides habitat value, removal of at least the part that projects into the main current would also relieve some of the erosional pressure on the left bank.

Field Site 10

IPA Site 13A consisted of approximately 3,000 feet of buried windrow revetment, with an additional ~500 feet of piled windrow revetment at the downstream end (**Figure 46**). An application rate for the rock of 3.5 tons/foot was specified for the entire site. A portion of the upstream end of the site tied into the Elkhorn Crossing Recreation Area. A berm was constructed in the early-1990s along the portion of the site shown in yellow in Figure 1 to prevent overbank flows from cutting across the meander bend (Table 1). Most of the site downstream from the recreation area appears to be in good condition, with the bulk of the windrow rock launched along the toe and mid-height portion of the banks (**Figure 47**). With the exception of an approximately 400-foot long area near the downstream end of IPA Site 13A, where the bankline appears to have migrated by up to 25 feet, the bankline along the entire site downstream from the Recreation Area has remained in about the same place since completion of the IPA (**Figure 48**). There are, however, gaps in the launched rock at several locations over an approximately 1,000-foot reach immediately downstream from the Recreation Area (Field Site 10, **Figures 49 and 50**). These gaps can be repaired by placing appropriately-sized rock along the toe and intermediate-height portion of the banks. Where appropriate, additional piled windrow rock can be placed along the top of the banks at a rate of 2 to 2.5 tons/foot to prevent further damage if the upper part of the bank continues to erode.

Figure 38. View looking downstream from near the downstream rock stockpile at IPA Site 13B.

Figure 39. View looking upstream from near the downstream rock stockpile at IPA Site 13B.

Figure 40. Construction plans for IPA Site 13B overlaid onto the September 2009 aerial photograph. Note that windrow revetment has been installed at the upstream end where the plans call for cabled-tree revetment.

Figure 41. View looking upstream end of the existing windrow revetment at IPA Site 13B, and the moderately eroding bank at Field Site 9, just upstream from the windrow revetment.

Figure 42. View looking downstream at the upstream end of the existing windrow revetment at IPA Site 13B, and the moderately eroding bank at Field Site 9.

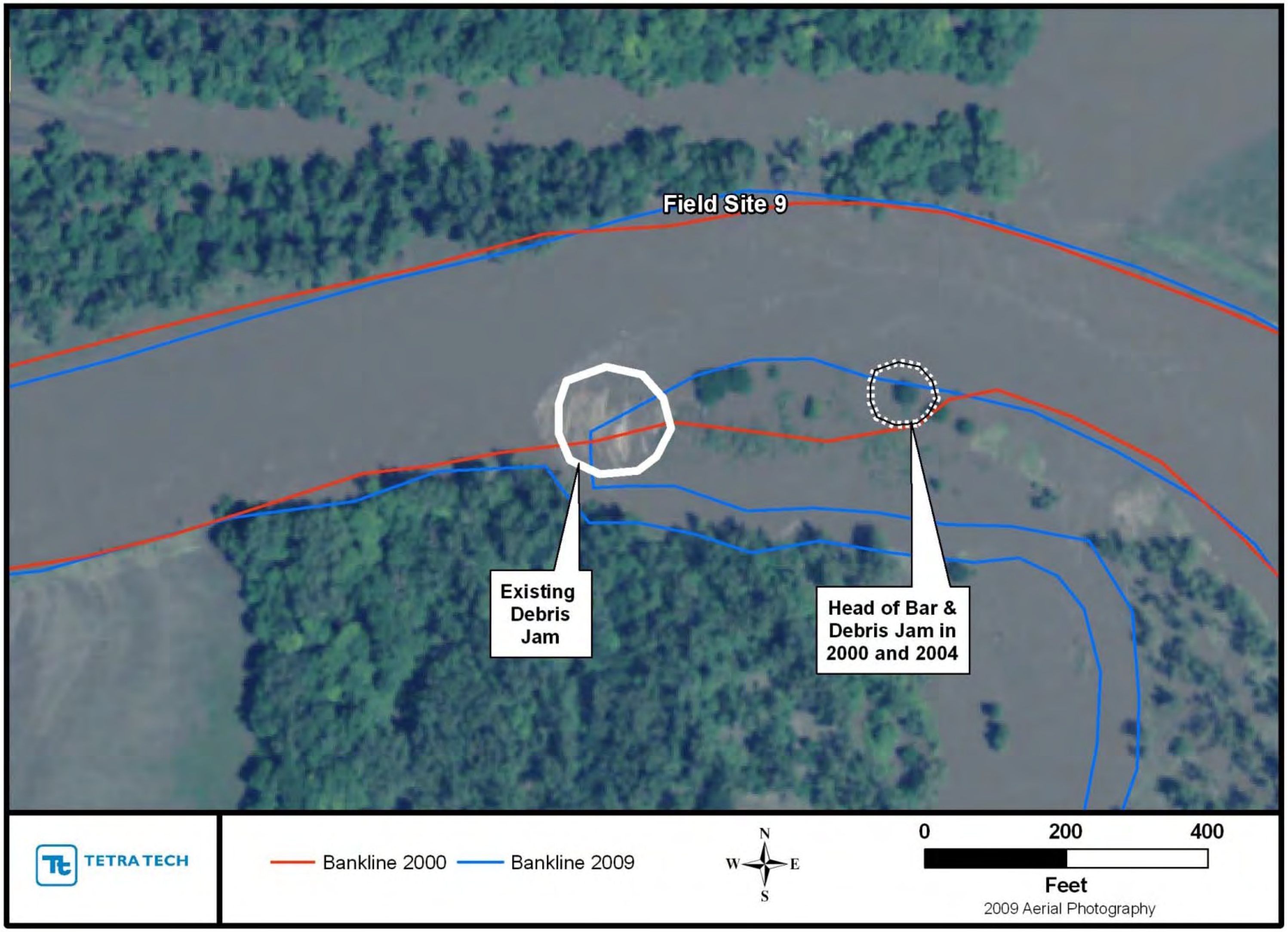


Figure 43. June 16, 2010 aerial photograph (Q~37,250 cfs) in the vicinity of Field Site 9 and the right bank debris jam located just upstream. The approximate location of the head of the point bar and debris jam shown in the October 2000 and October 2004 aerial photos, and the banklines from the October 2004 and September 2009, are also shown.

Figure 44. View looking downstream of the existing debris jam. Field Site 9 is visible near the right side of the large trees along the left bank in the background.

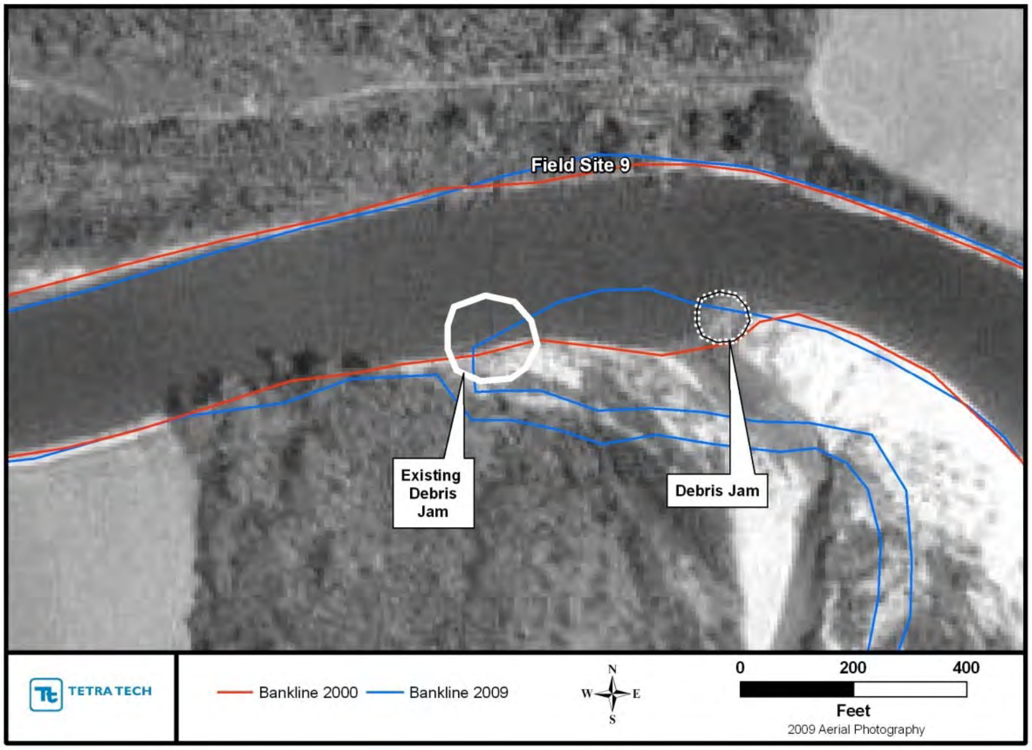


Figure 45. October 2000 aerial photograph (Q~550 cfs) in the vicinity of Field Site 9. The head of the right-bank point bar and debris jam shown in this photo was also in this approximate location in the October 2004 aerial photo. The approximate location of the head of the point bar and existing debris jam, as well as the October 2004 and September 2009 banklines, are also shown on the figure for reference.

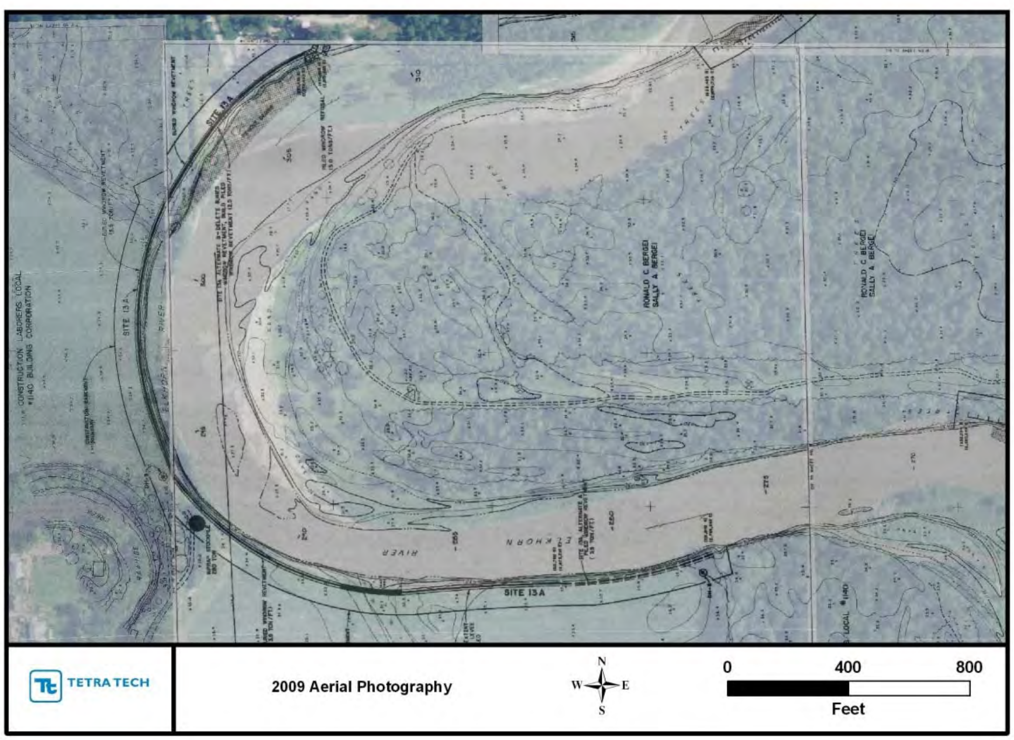


Figure 46. Construction plans for IPA Site 13A overlaid onto the September 2009 aerial photograph.

Figure 47. View looking upstream of the middle portion of the buried windrow revetment at IPA Site 13A.

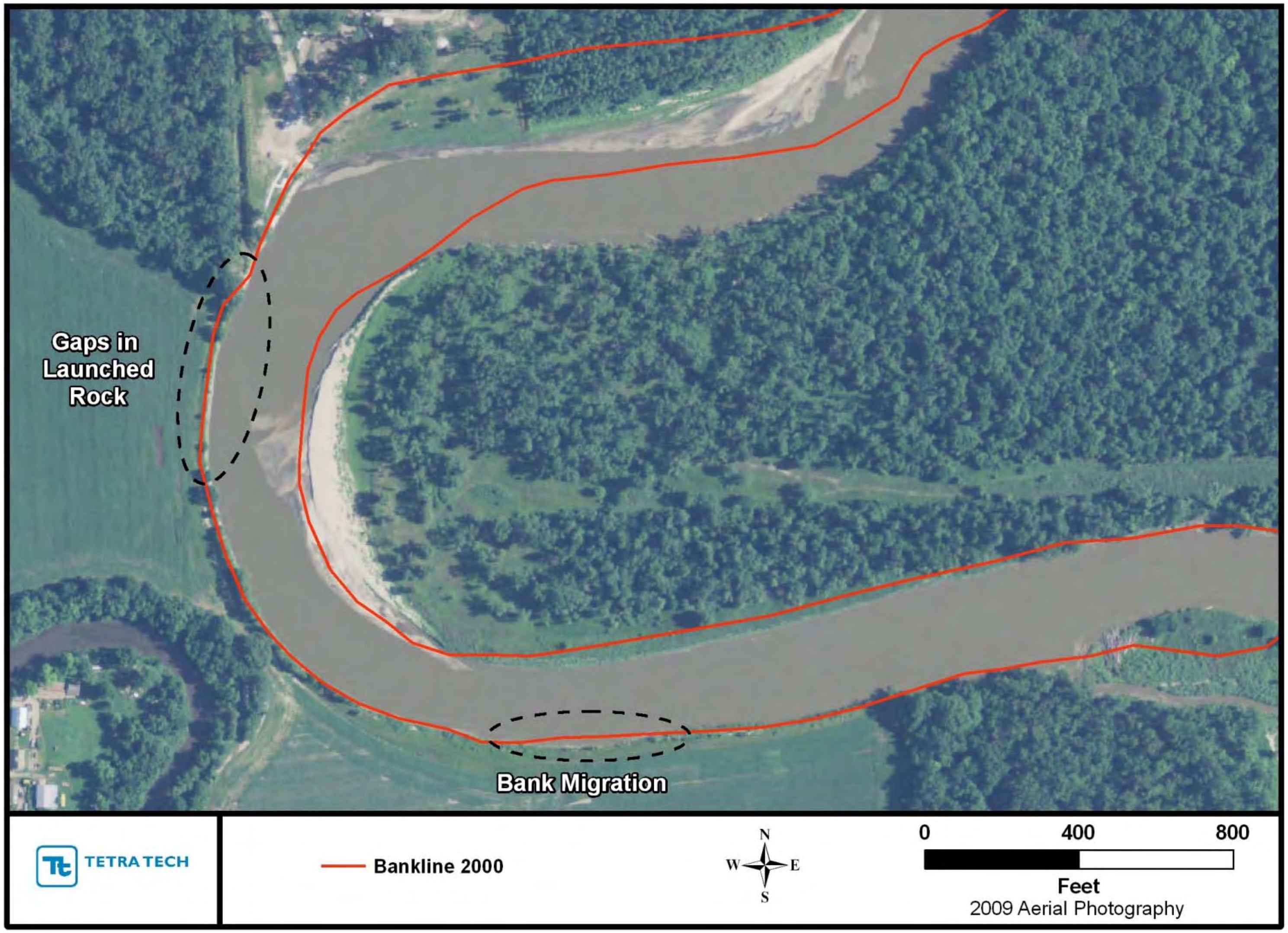


Figure 48. September 2009 aerial photograph of IPA Site 13A (Field Site 10). Also shown is the bankline in the October 2000 aerial photograph.

Figure 49. Right bank at the edge of the downstream edge of the wooded area at the Elkhorn Crossing Recreation Area (IPA Site 13A and Field Site 10).

Figure 50. View looking downstream of the right bank downstream from the wooded area at the Elkhorn Crossing Recreation Area (IPA Site 13A and Field Site 10).

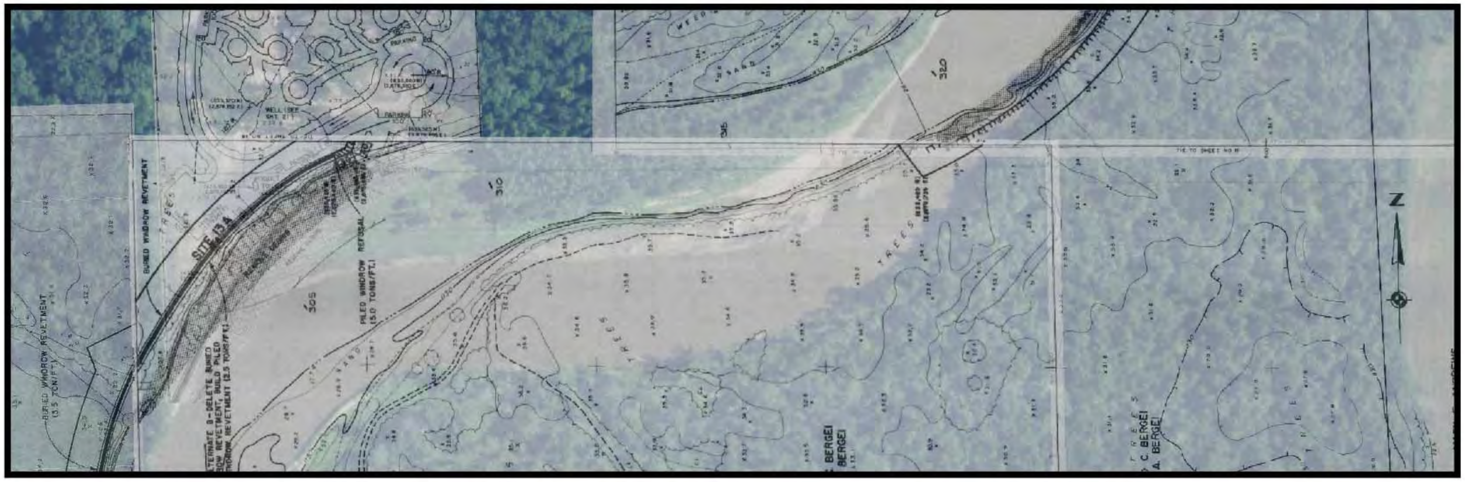
Field Site 11

At the time the IPA was constructed, the right riverbank was directly in contact with the overbank surface on which the Elkhorn Crossing Recreation Area was developed, and IPA Site 13A was, therefore, extended upstream past the facilities for protection (Figure 46). The construction plans also originally called for about 1,400 feet of cabled-tree revetment on the left bank beginning about 1,600 feet upstream from the Recreation Area boat ramp, with an additional approximately 550 feet of piled windrow revetment upstream from the cabled-trees (IPA Site 14); however, the plans were subsequently modified to include windrow revetment in place of the cabled trees (Table 1, Note 22). With the exception of the red area at Note 23 in Table 1, the site was constructed in accordance with the modified plans. Due to landowner constraints, the portion indicated by Note 23 was not constructed.

The left bank between the Recreation Area and the downstream limit of the proposed cabled-tree revetment migrated by over 150 feet in some locations between the time of the mapping that was used for the IPA design and October 2000, but remained in about the same position during the relatively dry period between October 2000 and October 2004 (Figure 2, **Figures 51a and 51b**). Between October 2004 and September 2009, this bankline migrated southward by 220 feet, and it experienced an additional 50 to 60 feet of migration at the time of the June 16, 2010 aerial photograph (**Figure 52**). In total, the left bank in this portion of the reach migrated by over 400 feet in some places and approximately 10 acres of the left overbank was eroded away between the time of the mapping and June 2010. Consistent with the typical behavior of migrating bends (Brice and Blodgett, 1978; Shen and Schumm, 1981), the right bank adjacent to the Recreation Area has accreted by up to 400 feet in conjunction with this erosion, abandoning the upstream portion of the bank protection at IPA Site 13A and forming a large sand bar that now blocks the boat ramp (**Figures 53 and 54**).

As discussed above, meander bends tend to evolve by migrating laterally and in the downstream direction. Analysis of bend geometry data for a large number of rivers throughout the world indicates that erosion rates in bends tend to increase with increasing bend sharpness, as measured by the ratio of radius of curvature to channel width (R_c/W), up to R_c/W values in the range of 2 to 4, and the erosion rates tend to decrease rapidly in sharper bends (i.e., $R_c/W<2$) due to energy loss in the bend (Hickin, 1975; Nanson and Hickin, 1986; Begin, 1981; Odgaard, 1987; Bagnold, 1960). Carey (1969) and Page and Nanson (1982) showed that in very tight bends (i.e., Rc/W <2), deposition actually occurred on the outside of the bend. Under these conditions, the rate of lateral migration essentially stops, and there is a high probability that the bend will cut off.

The average channel width through the bend at IPA Sites 13A and 14 is about 300 feet, and the radius of curvature of the bend at IPA Site 13A is about 670 feet; thus, R_c/W is approximately 2.2, the approximate maximum sharpness (or minimum R_c/W) at which the bend should develop without cutting off. Prior to construction of the IPA, this bend followed the typical evolutionary sequence indicated by Brice and Blodget (1978) and Shen and Schumm (1981) (**Figure 55**). The bank protection provided by the IPA effectively stopped continued downstream migration, and the bend is now compressing through downstream migration (i.e., erosion of the left bank) of the upstream bend at IPA Site 14. The left bank opposite the Recreation Area eroded southward at an average annual rate of about 9.3 feet/year between late-1984 and October 2000, and migration effectively stopped during the relatively dry period between 2000 and 2004. The annual erosion rate increased substantially to about 46 feet/year between October 2004 and September 2009, and the bank eroded by an additional approximately 75 feet between



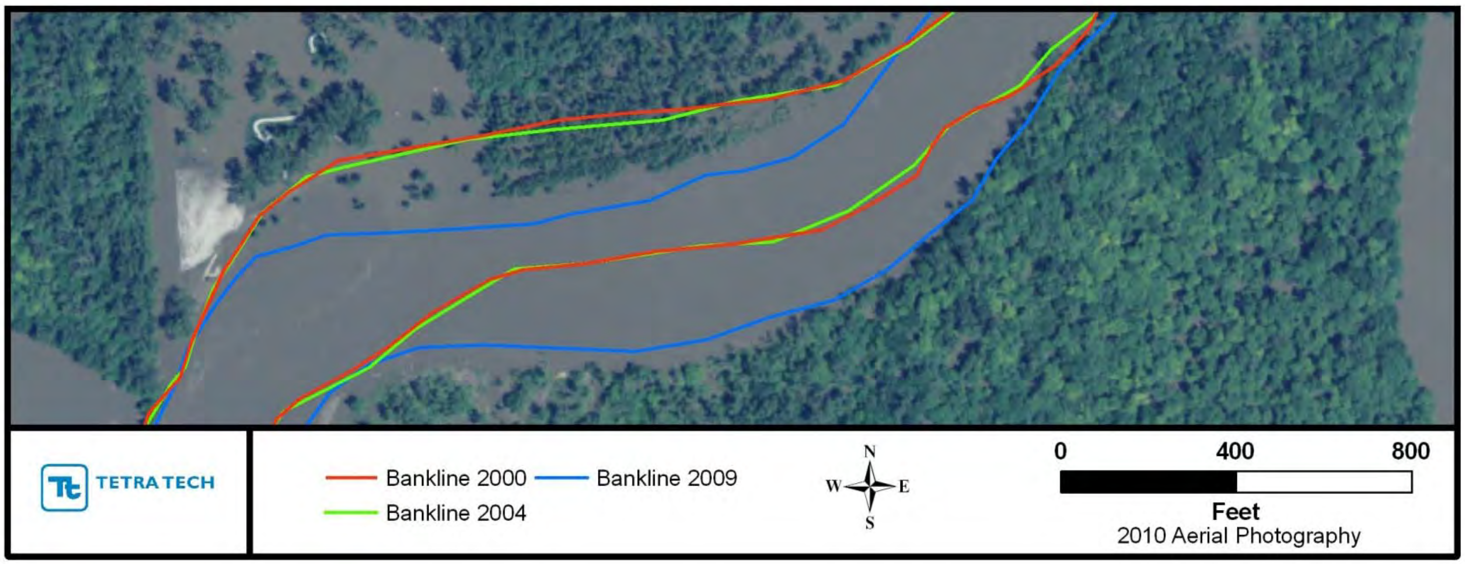


Figure 51a. Construction plans for the upstream portion of IPA Site 13A and downstream portion of IPA Site 14 overlaid onto the September 2009 aerial photograph. Also shown are the banklines from the October 2000 and October 2004 aerial photograph.

Figure 51b. Same image as Figure 51a with construction plans removed.

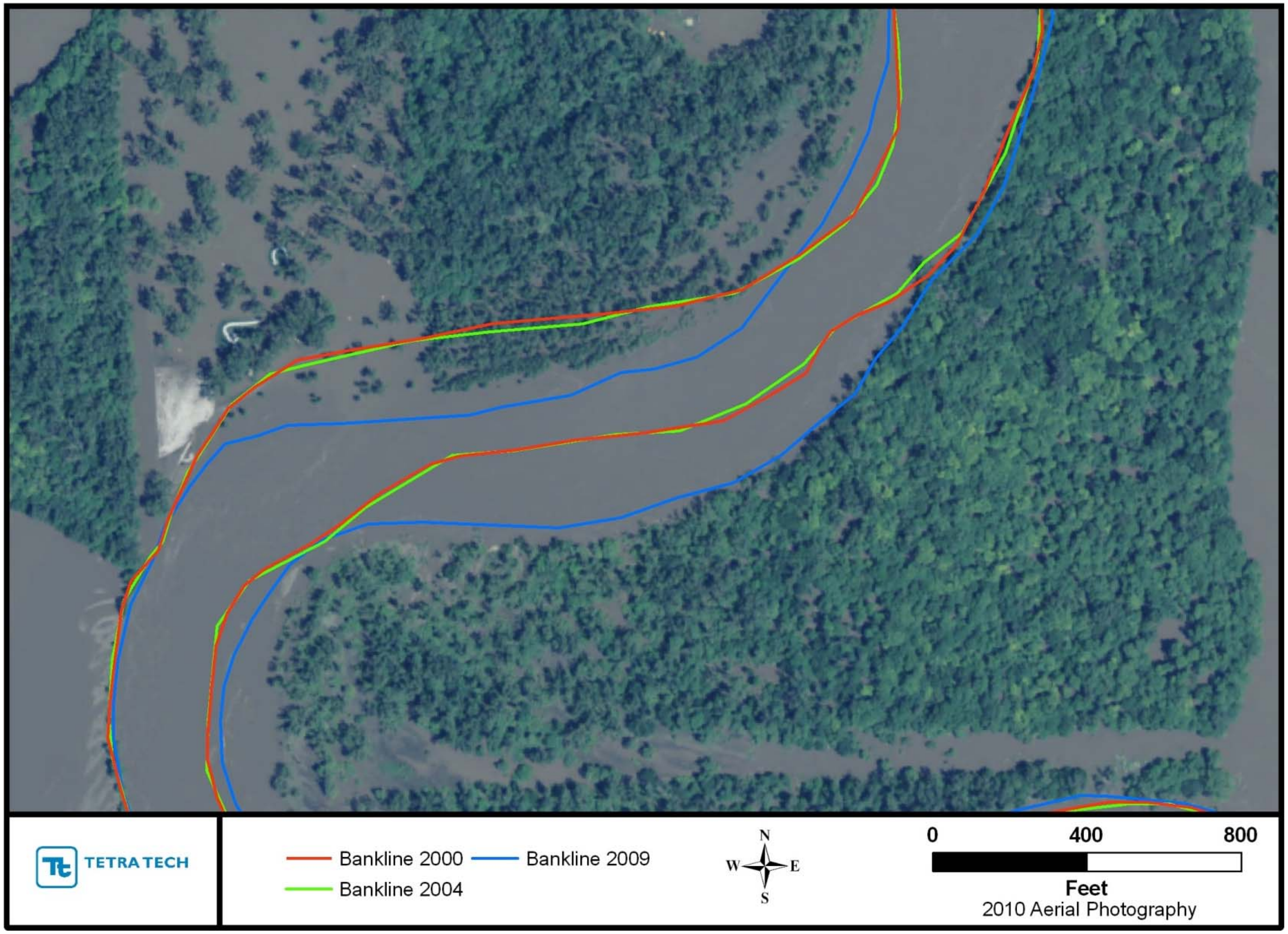


Figure 52. June 16, 2010 aerial photograph Field Site 11, between IPA Sites 13A and 14 showing the additional 55 to 60 feet of lateral migration after September 2009.

TETRA TECH

Figure 53. Blocked boat ramp and upstream end of riprap bank protection at the Elkhorn Crossing Recreation Area.

TETRA TECH

Figure 54. View looking upstream of the large sandbar that has formed along the right bank at the Elkhorn Crossing Recreation Area. The boat ramp shown in Figure 52 is just behind the mound on the left side of the photo.

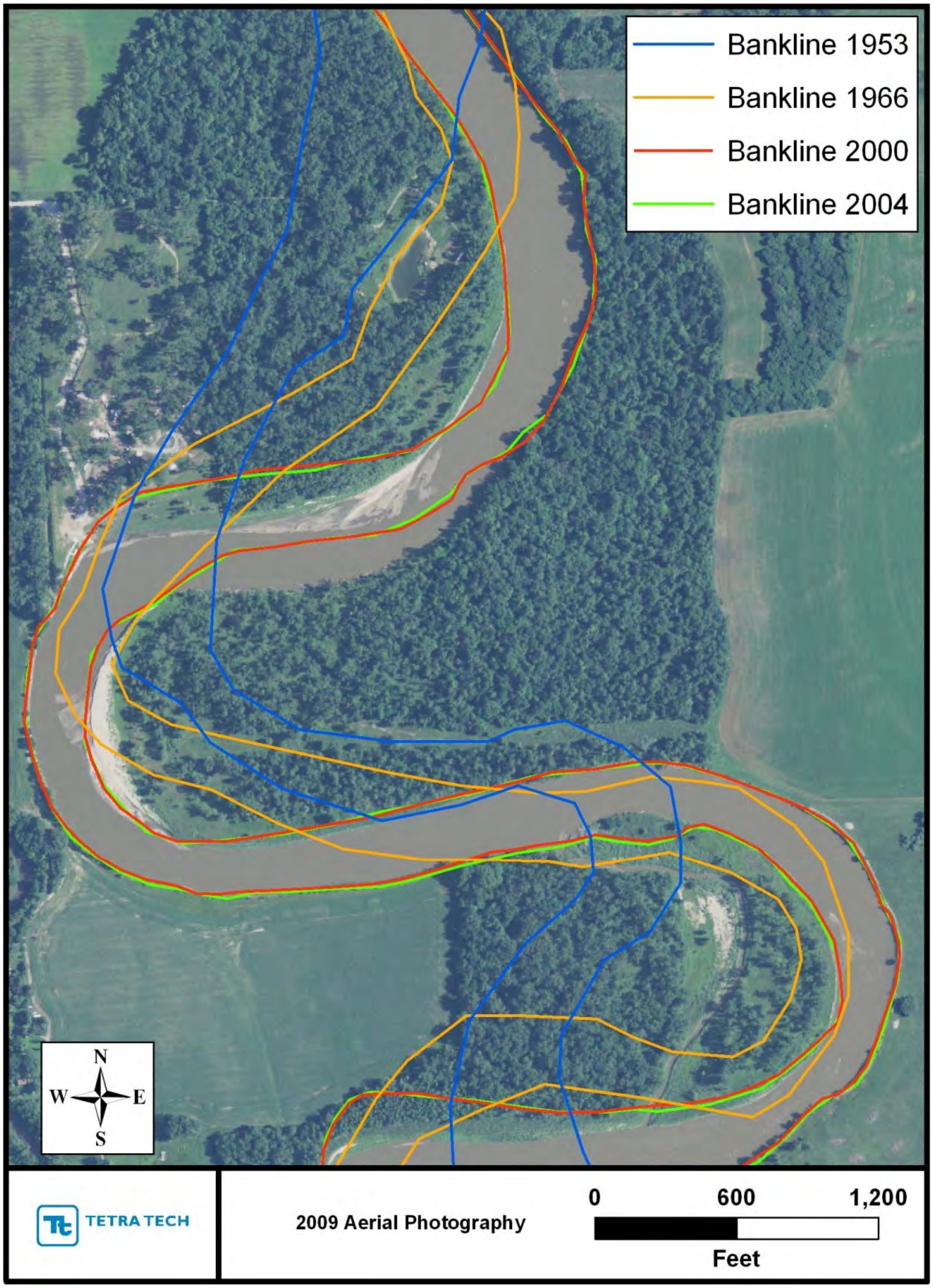


Figure 55. September 2009 aerial photograph showing the bankline locations in June 1953, April 1966, October 2000, and October 2004.

September 2009 and mid-June 2010 (**Figure 56**). [It should be noted that most of the erosion between the 2009 and 2010 aerial photos most likely occurred during the rising limb, peak and early part of the falling limb of the 2010 hydrograph (Figure 6). Data are not available to determine how much additional erosion occurred during the later part of the 2010 high-flow hydrograph.] Future erosion rates in this bend may slow, based on the findings of Hickin (1975) and others, because the bend at IPA Site 13A is approaching its maximum sharpness. Considering the topography and amount of trees and other vegetation in the wooded area through which the cut-off would occur, it seems unlikely that the bend would abruptly cut off, but this must be considered as a possibility.

The sandbar at the boat ramp obviously creates a problem for use of the ramp, and PMRNRD has expressed a desire to modify the site in a manner that will remove and prevent future formation of the sandbar. Since this sandbar is formed by accretion on the inside of the bend that is associated with migration of the opposite bank, the only effective way to cause the river to remove the sandbar and insure that it does not re-form is to shift the left bank opposite the site back to its approximate position prior to formation of the sandbar. This could be achieved by constructing a series of rock jetties or bendway weirs in the eroded area along the left bank, with the ends of the structures at approximately the 2004 bankline (**Figure 57**), but the cost would be very high (cumulative length of the spurs would be ~1,700 feet, not considering the necessary tie-back to prevent flanking). It is questionable whether the necessary environmental permits could be obtained for such a project. Two other alternatives could be considered for dealing with the sedimentation issues at the boat ramp, both of which would require stabilization of the existing bankline at IPA Site 14 to insure their continued viability:

1. Periodically dredge an opening to the boat ramp, as necessary, and
2. Re-locate the boat ramp downstream beyond the point of the existing sandbar (**Figure 58**).

Option 1 may not be practical during periods of moderate to high flows that typically occur in the late-spring and early-summer because the dredged area would most likely rapidly fill-in , requiring dredging on a very frequent basis to insure accessibility of the ramp. Option 2 may be the most effective approach if the erosion on the opposite bank is checked and the sandbar does not continue to grow in the downstream direction. Based on the Recreation Area Boundary shown on the second figure that accompanied the July 14, 2010 memorandum from Mr. Winkler, Director of PMRNRD to Mr. Chick, State Conservationist with the Natural Resources Conservation Service (NRCS) that was provided to Tetra Tech as part of the background material, there appears to be room to move the boat ramp downstream and remain within the Recreation Area boundary (**Figure 59**). Based on the preliminary alignment shown in Figure 58, the boat ramp would be roughly twice the length of the existing ramp. With additional site information, it may be possible to extend the parking area to the southwest, shortening the required length of the boat ramp.

During the field visit, Director Tesar indicated that the rock that is visible in Figure 57 that was placed in 2009 has slumped 2 to 3 feet down into the channel, and he is concerned that the installation may eventually fail. Sufficient data are not available confirm this observation, but it appears to be reasonable based on the existing elevation at the top of the bank compared to the top of the rock. If the observation is correct, it may be necessary to repair the installation to insure continued stability, and these repairs could be done in conjunction with relocating the boat ramp.

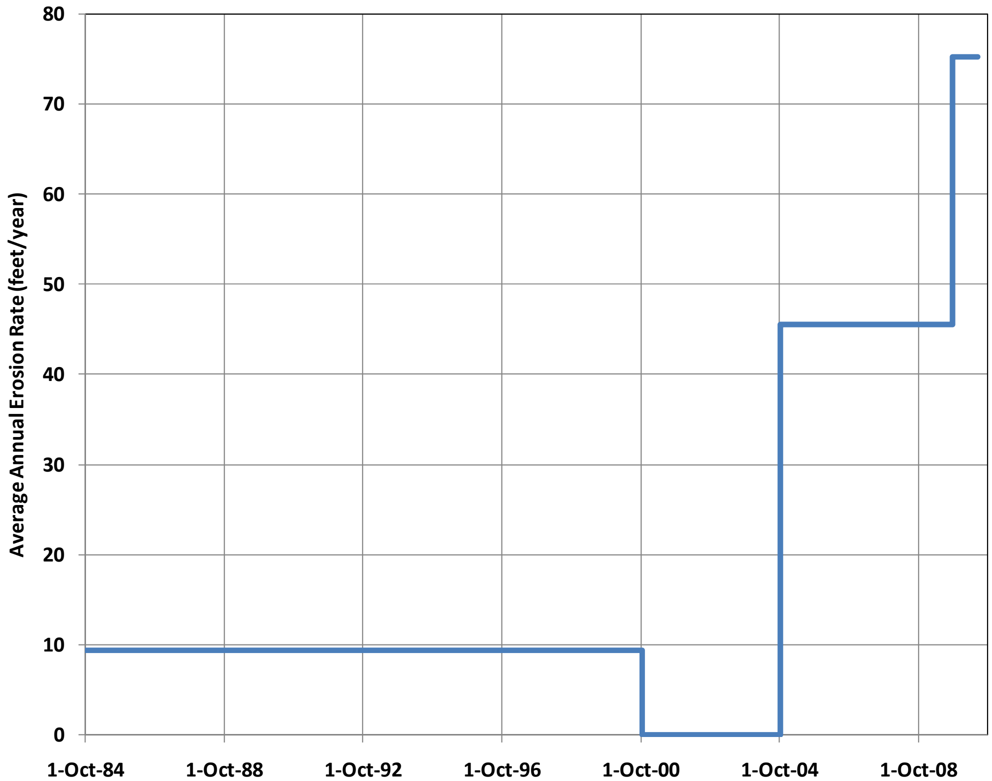


Figure 56. Average annual erosion rates in the bend at the downstream end of IPA Site 14 between late-1984, when the aerial photographs used to develop the mapping were taken and mid-June 2010.

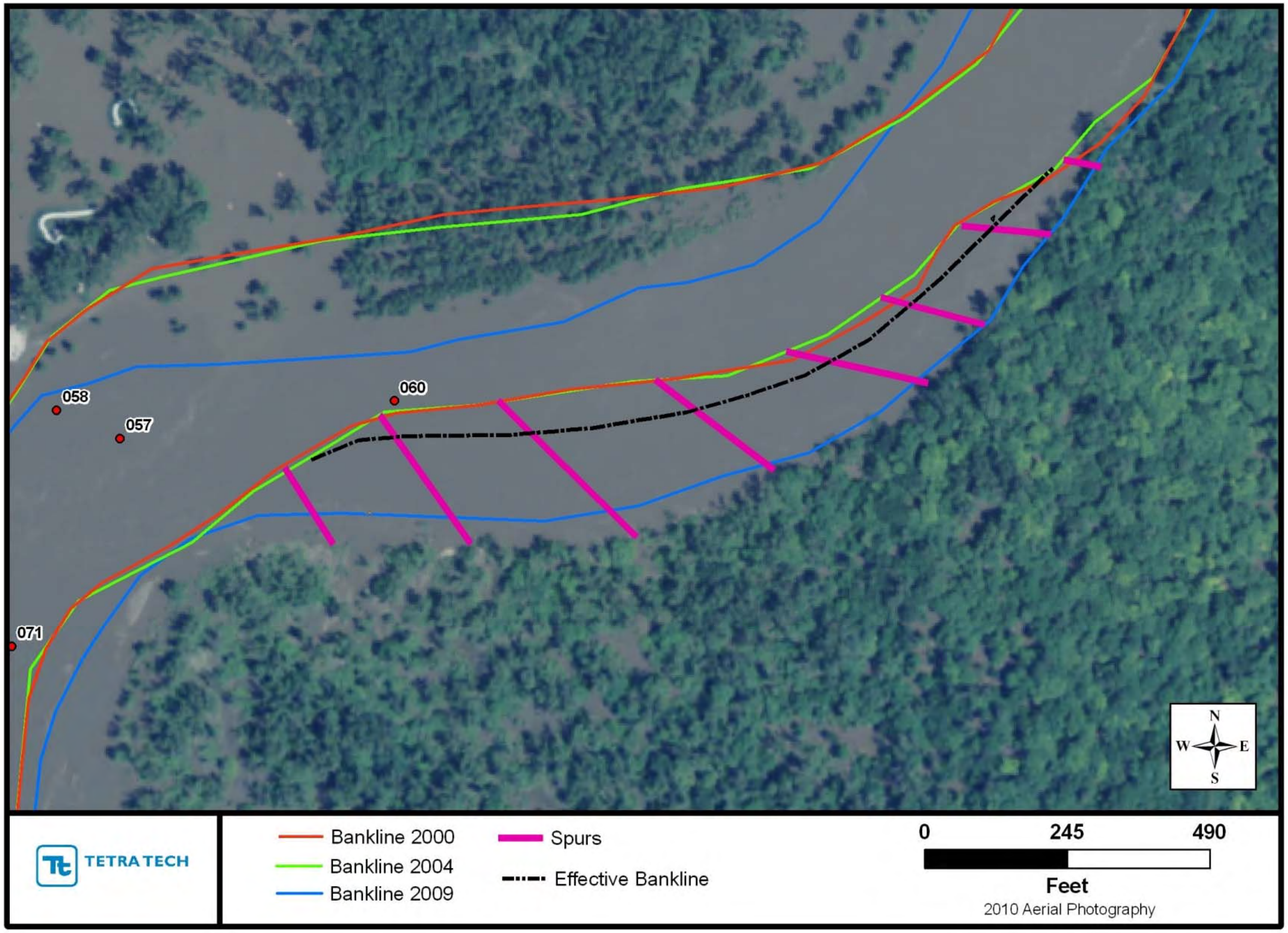


Figure 57. Preliminary layout of spur dike field that would restore the river alignment in the vicinity of the Elkhorn Crossing Recreation Area to eliminate the sandbar at the boat ramp.

Figure 58. Looking downstream at the end of the sandbar that blocks the Elkhorn Recreation Area boat ramp toward the riprap bank protection that was installed in 2009. It may be possible to relocate the boat ramp to the rock-protected area downstream from the sandbar.

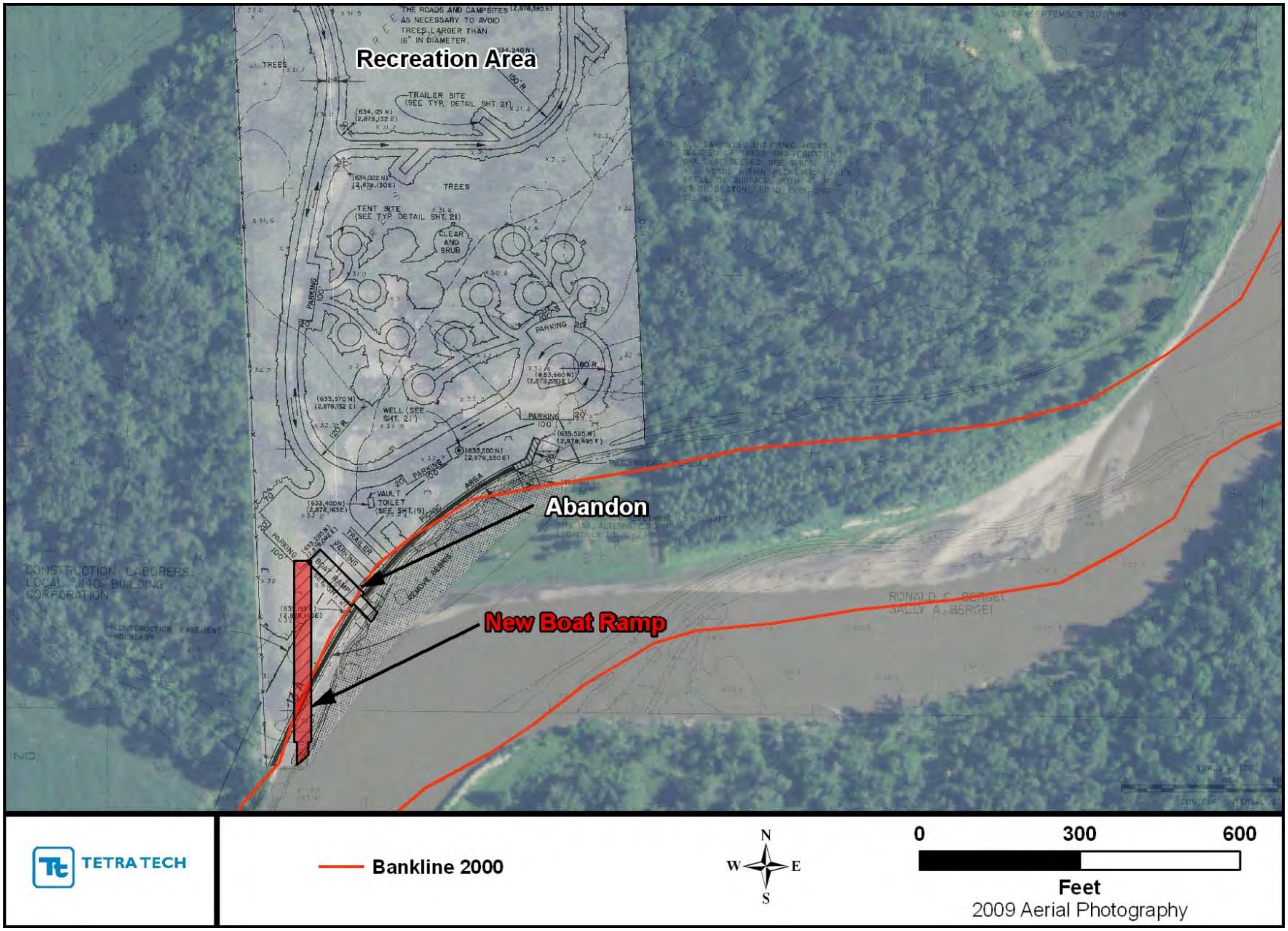


Figure 59. Suggested relocation of Elkhorn Crossing Recreation Area boat ramp.

Field Site 12

During the field discussions, Director Tesar indicated that there is concern that the river is widening in the approximately one-half-mile reach downstream from the Highway 36 Bridge. Some erosion of the right bank for several hundred yards downstream from the bridge was observed during the site visit, but the left bank appears to be relatively stable (**Figures 60 and 61**). Comparison of the banklines in the 2000, 2004, 2009 and 2010 aerial photos indicates that there has been little, if any widening in this reach over at least the past decade, except along the left bank for about 550 feet downstream from the bridge (**Figure 62**). It also appears that the left bankline has retreated by up to 30 feet over a distance of about 400 feet beginning about 650 feet downstream from the bridge, and the right bankline may have retreated by up to 30 feet over a distance of about 800 feet, beginning about 1,400 feet downstream from the bridge. These relatively small distances are likely within the resolution of the photographs; thus, there is considerable uncertainty as to whether this amount of lateral erosion actually occurred. In any case, the average width of the channel in the approximately 0.7-mile reach extending from the end of the left-bank widening in the first 550 feet downstream from the bridge was 320 feet in October 2000, and the average width in this part of the reach is the same in the September 2009 aerial photo. Based on the 1996 construction plans, the span length of the Highway 36 Bridge between the abutments is also about 320 feet.

Based on the above information, it does not appear that this reach of the river is systematically widening. It is interesting to note, however, that the line of riparian trees in the right overbank that provide a measure of bank stability is relatively thin for about 750 feet downstream from the bridge. If the observed erosion in this area is creating a hazard to the adjacent property, the erosion could be checked with windrow revetment similar to the installations for the IPA, or the rock could be placed directly on the bank. If the windrow option is chosen, it would probably be necessary to remove the existing trees. It is possible that the bank could be built outward by the necessary amount to allow placement of the riprap on a minimum 2.5H:1V slope without constricting the river sufficiently to affect the upstream water-surface profile at high flows. Whether this is, in fact, the case would require detailed hydraulic modeling.

Field Site 13

This site, which is located outside the boundaries of the IPA, consists of the bend located about one-half-mile upstream from Highway 36 that has migrated to the east by about 300 feet to the east since October 2000 (**Figure 63**). During the site visit, Director Tesar indicated that there is concern that this bend will continue to migrate, causing the river to flank into the left (east) overbank, potentially abandoning the Highway 36 Bridge. This site was considered in the evaluation to assess whether future changes at the site could potentially affect the stability of the downstream river within the IPA boundaries.

An abandoned meander channel is clearly visible in both the 2009 and 2010 aerial photographs that extends from just upstream from the apex of this bend downstream to just above Highway 36 where it reconnects to the river (Figures 63 and **64**). Bank protection in the form of rock and concrete rubble has been placed on the bank to protect the properties in the left overbank at the upstream end of the wooded area at the downstream end of the bend (**Figure 65**). The protection has effectively prevented additional erosion of the bankline at this location, but the bankline just upstream has continued to erode. At the present time, there is a relatively sharp hook in the bankline where it transitions from the eroding zone to the protected zone. Although the erodible portion of the bank does not appear to have migrated significantly during the portion of the 2010 high flow before June 16, when the aerial photo was taken, it likely will continued to

Figure 60. View looking upstream through the Highway 36 Bridge.

Figure 61. View looking upstream through the right end of the Highway 36 Bridge.

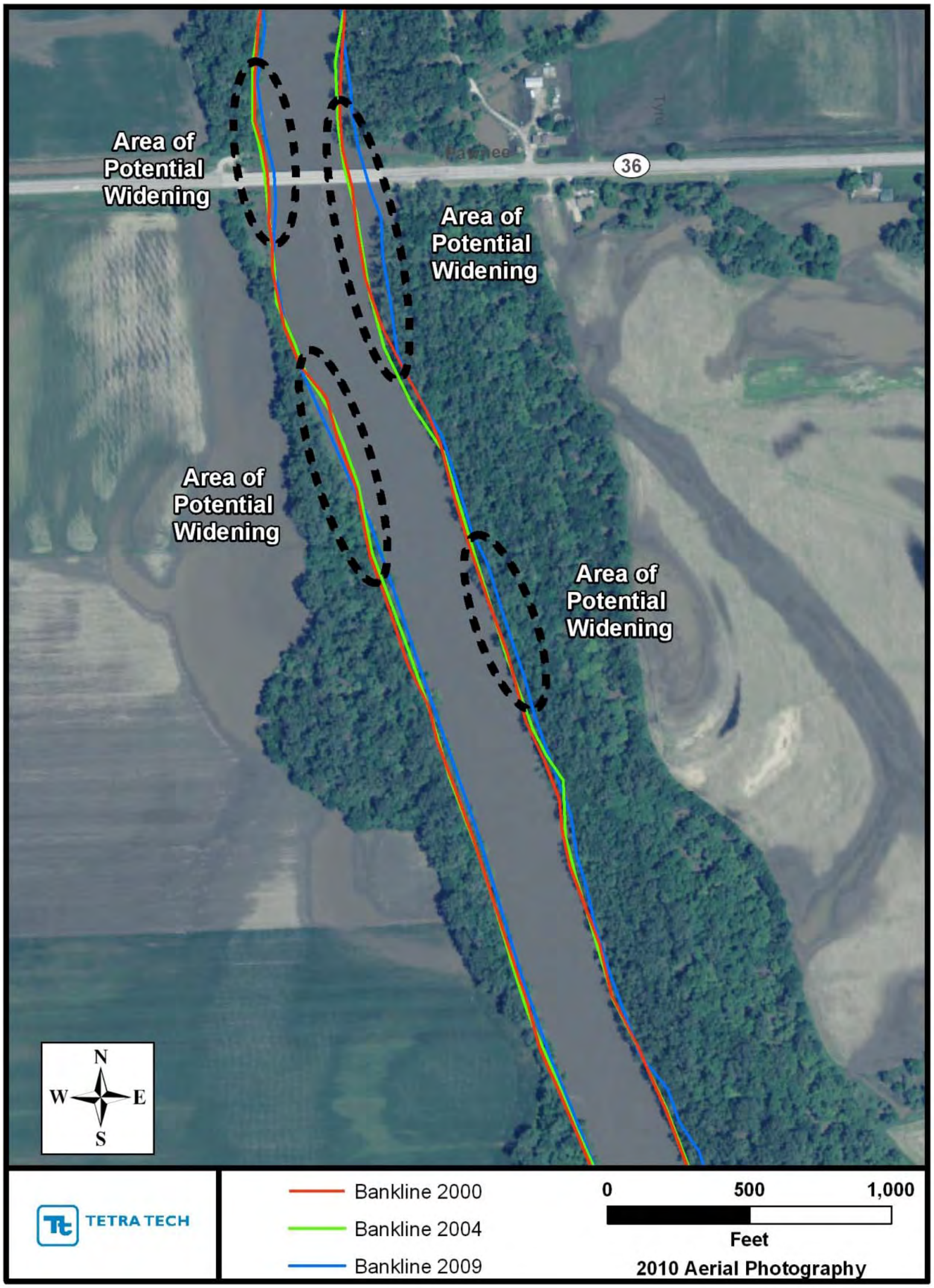


Figure 62. 2010 aerial photograph of the reach downstream from the Highway36 Bridge, showing the banklines from the 2000, 2004, and 2009 aerial photos (Field Site 12).

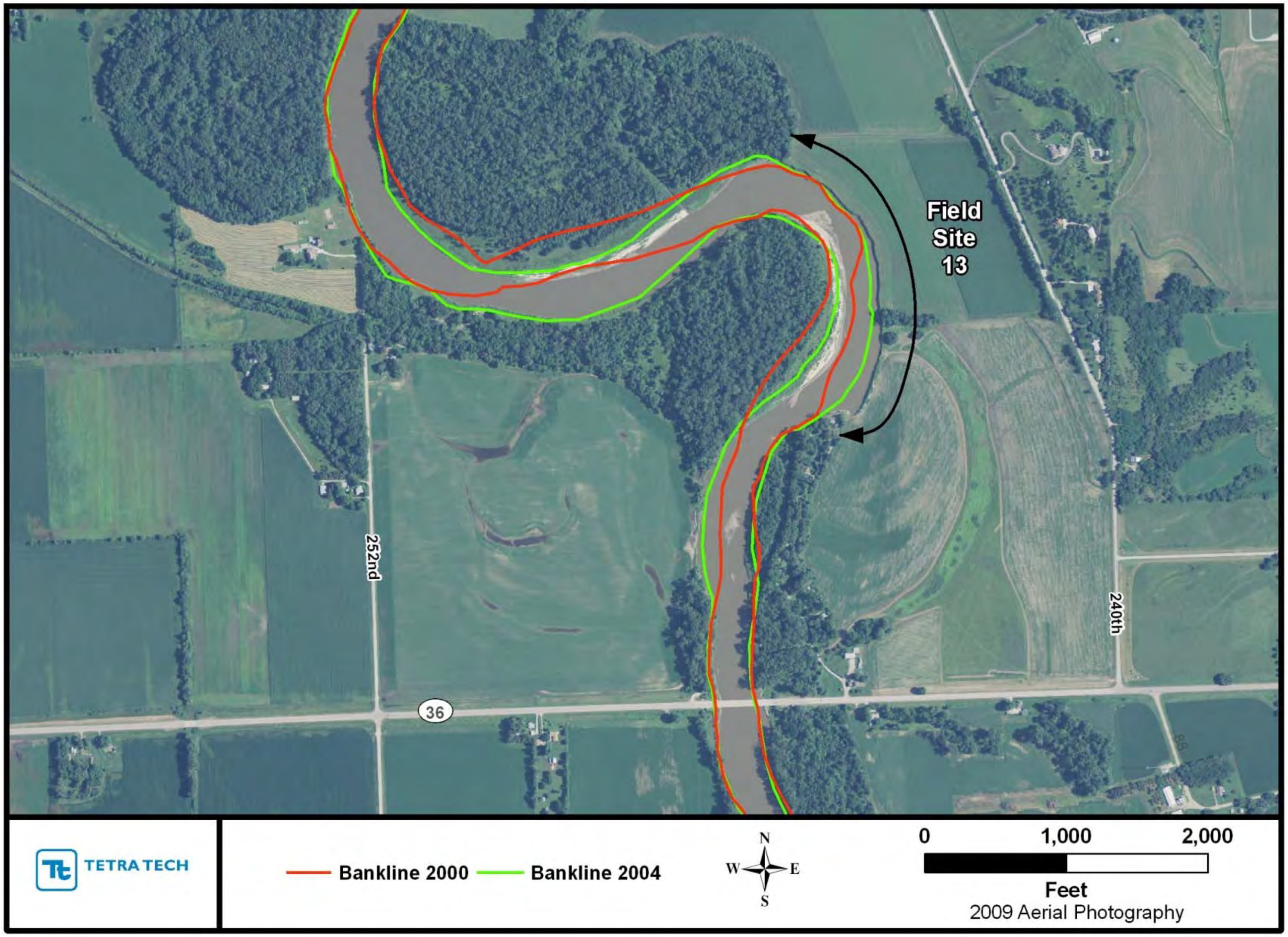


Figure 63. 2009 aerial photograph of the eroding bend at Field Site 13, showing the banklines in 2000 and 2004.

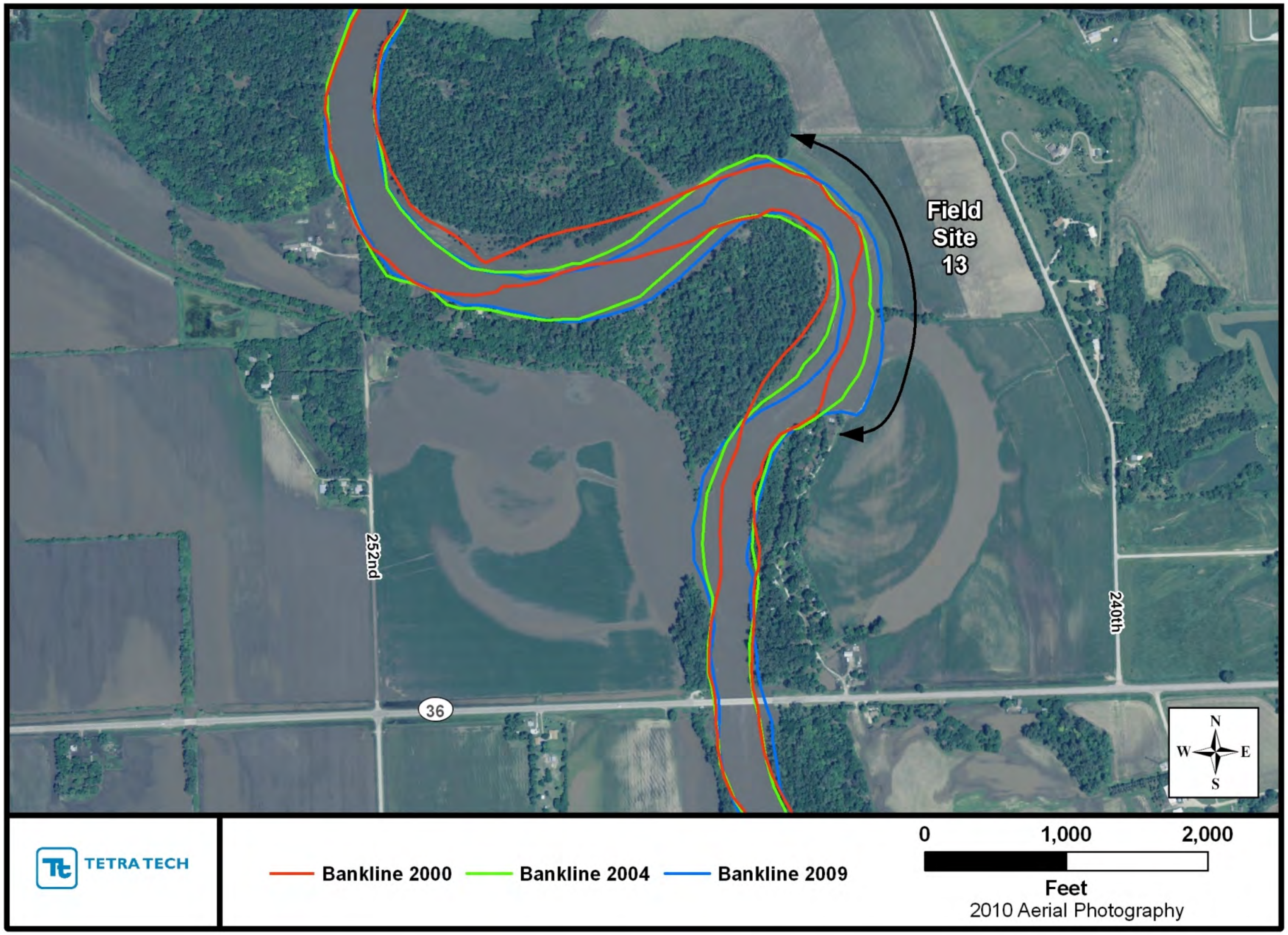


Figure 64. 2010 aerial photograph of the eroding bend at Field Site 13, showing the banklines in 2000, 2004 and 2009.

Figure 65. Looking downstream at the protected bank at the downstream end of Field Site 13.

erode during future high flows. Considering the relative stability and existing bank protection along the downstream wooded area, it is unlikely that the bend will avulse to the east and impact Highway 36 or the bridge crossing. Considering the geometry of the bend, the most likely scenario is that, as the bend continues to develop, the radius of curvature will decrease sufficiently to encourage the bend to cut off on the inside, effectively straightening the river upstream from the bend. Continued development of the bend will, however, cause additional erosion in the adjacent field, and could endanger the homes that are located in the wooded area (many, or perhaps most, of which were damaged by flooding during the 2010 high flows).

To prevent continued erosion of this bend, the erodible bankline should be restored to its approximate position in 2004 and protected over a distance of about 750 feet in the downstream portion of the bend, and piled windrow revetment should be placed on the top of the bank along the remainder of the erodible portion of the bend (~1,500 feet). The 2004 bankline can be restored by either backfilling the eroded area and placing rock along the new bankline, or by constructing a series of rock spurs (**Figure 66**). Based on the amount of rock required for the spurs, it appears that backfilling and protecting the new bankline would be the most economical method to restore the 2004 bankline in the downstream 750 feet of the site. Although sufficient topographic data are not available to accurately estimate the appropriate application rate for the piled windrow revetment, it is assumed for purposes of estimating the probable cost that the rate would be in the range of 3.0 tons/ft, similar to many of the downstream sites within the IPA.

Estimated Costs

A preliminary estimate of the cost to implement the suggested repairs or new protection discussed in the previous sections was prepared based on the type of repair and size of the site (**Table 2**). Unit costs for new or additional windrow revetment shown in Table 1 were estimated by determining the cost per linear foot for the most applicable IPA site from the original construction cost estimate (SLA 1985, Table 6.1), and escalating those costs by a factor of 2.3 to account for the change in the Engineering New Record (ENR) Construction Cost Index (CCI) from 1984 through 2010 (**Table 3**). (The CCI for Kansas City and Denver, the two closest cities for which local information was available, increased by factors of 2.48 and 2.06 over the period, respectively, and the national average increased by 2.16.) Using the information in SLA (1985), the estimated construction cost, adjusted to 2010 costs, averaged about $204/LF for all 21 sites that were considered in the original analysis, and the average unit cost for the 5 sites within the IPA was $181/LF. The SLA (1985) estimated construction cost at each site was escalated by 1.5 to account for "operation, maintenance, and replacement" and an additional 10 percent was added for final design, construction supervision and contingencies. Although the contingency factor appears to be relatively small, the overall escalation factor appears to be reasonable. The SLA (1985) estimates also indicated that the actual revetment costs made up 60 percent of the total estimated cost at each site, on average, with clearing and grubbing making up about 0.5 percent and site access about 2.9 percent.

Based on the above assumptions, the estimated cost for the individual sites range from about $27,000 to repair the recently installed, but damaged, rock point at Field Site 7, to about $645,000 to realign the bank and provide windrow rock protection at Field Site 13 upstream from Highway 36. The estimated cost to apply windrow revetment to the left bank opposite and upstream from the Elkhorn Crossing Recreation Area and relocate the boat ramp downstream to a non-depositional area is about $562,000. The cumulative, estimated cost for all of the sites totals approximately $2.9M. Table 1 also provides a ranking of the suggested priority for the sites.

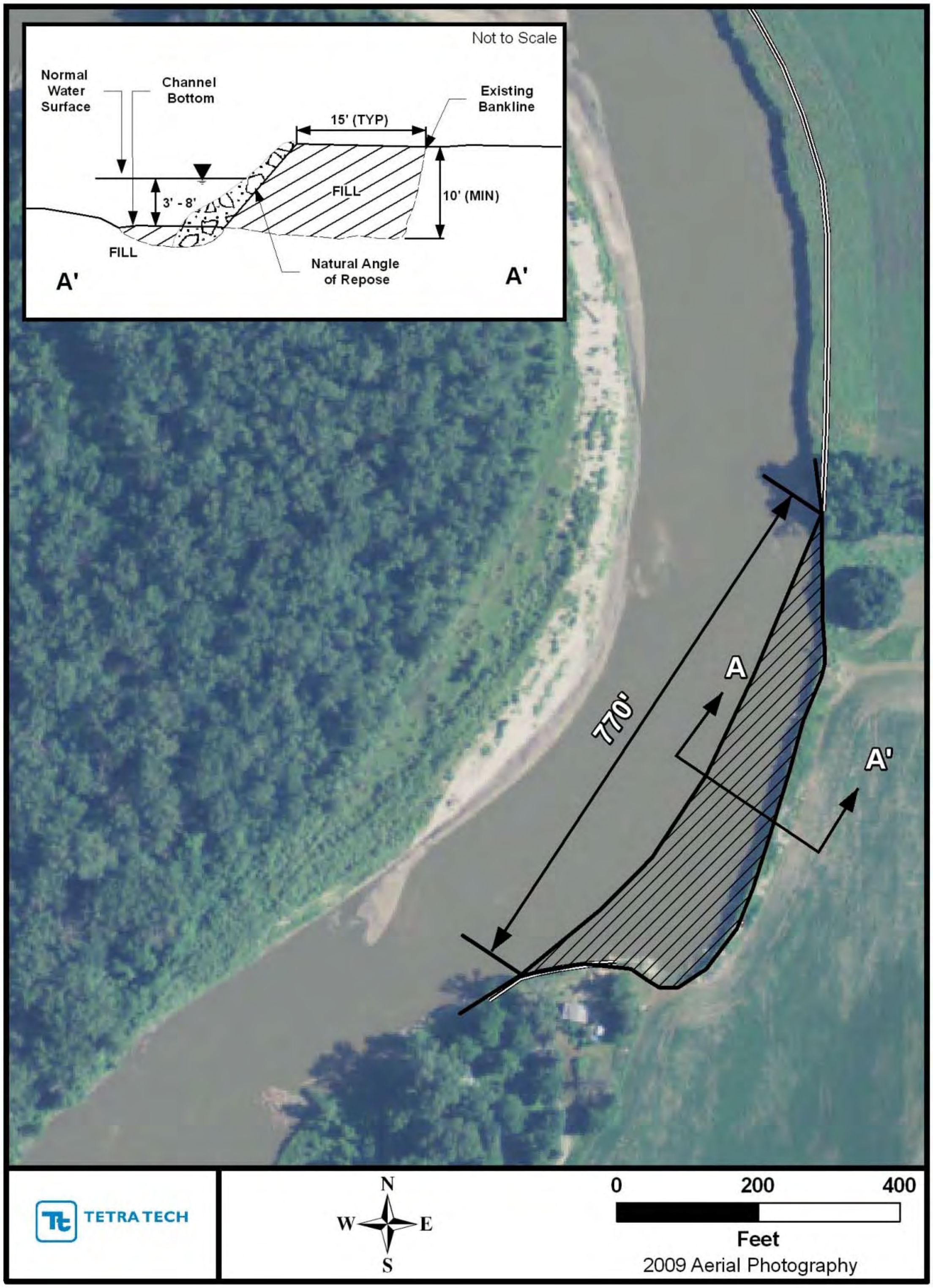


Figure 66. Preliminary recommended for bank protection at Field Site 13.

Table 2. Summary of field sites, recommended repairs and estimated cost.								
Site Number	Site Location	Length (ft)	Previous Application Rate (t/ft)	Unit Cost ($/ft)	Approximate Total Cost	Priority	Proposed Fix	Applicable Figure
2	pre-existing Site	N/A			$ 119,000	High	Re-grade and protect drainage swale	Figure 15
3	11	900	3	$ 183	$ 247,000	High	Additional piled windrow revetment	Figure 15
4	12	1500	2.5	$ 180	$ 405,000	Low-Medium	Additional piled windrow revetment	Figure 22
5	12	600	3	$ 180	$ 162,000	Low-Medium	Additional piled windrow revetment	Figure 25
6	13C	1400	Cabled trees	$ 182	$ 382,000	Medium	Additional piled windrow revetment	Figure 27
7	13C	100	End of levee	$ 182	$ 27,000	Medium	Repair riprap point	Figure 31
8	13B	400	Cabled trees	$ 363	$ 145,000	Medium	Realign bank line and back fill to eliminate scallop	Figure 36
9	13B	300	5	$ 182	$ 82,000	Low	Additional piled windrow revetment	Figure 39
10	13A	1000	3.5	$ 182	$ 136,500	HIgh	Repair gaps in existing windrow revetment	Figure 47
11	14	2000		$ 180	$ 562,000	High	Windrow revetment on left bank and relocate Elkhorn Crossing Boat Ramp	Figures 50 and 59
12	N/A	400		$ 180	$ 108,000	Low-Medium	Windrow revetment	Figure 61
13	15	2250	3	$ 287	$ 645,000	High	Realign bank line and back fill to eliminate scallop, windrow revetment	Figure 65
Total					**$ 2,901,500.0**			

Table 3. Analysis of estimated construction costs from SLA (1984, Table 6.1) for 2010 conditions.											
Site Number	Revetment	Clear and Grub	Access	Length (ft)	Cost of the Site	Total cost of site (1)	Rev %total	C&G % total	Access %/total	Total Cost/ft	
										1984	2010 (2)
1	$ 71,000	$ 370	$ 3,040	1,520	$ 74,510	$ 119,216	60%	0.3%	2.5%	$78	$180
2	$ 157,050	$ 200	$ 6,860	3,430	$ 164,110	$ 262,576	60%	0.1%	2.6%	$77	$176
3	$ 222,300	$ 550	$ 9,760	4880	$ 232,610	$ 372,176	60%	0.1%	2.6%	$76	$175
4	$ 133,200	$ 570	$ 5,800	2,900	$ 139,570	$ 223,312	60%	0.3%	2.6%	$77	$177
6	$ 30,910	$ 300	$ 2,320	600	$ 33,530	$ 53,648	58%	0.6%	4.3%	$89	$206
10a,b	$ 265,500	$ 1,190	$ 12,330	5780	$ 279,020	$ 446,432	59%	0.3%	2.8%	$77	$178
11	$ 226,050	$ 700	$ 9,500	4,750	$ 236,250	$ 378,000	60%	0.2%	2.5%	$80	$183
12	$ 91,800	$ 1,140	$ 3,960	1980	$ 96,900	$ 155,040	59%	0.7%	2.6%	$78	$180
13a,b,c	$ 497,350	$ 3,450	$ 29,520	10730	$ 530,320	$ 848,512	59%	0.4%	3.5%	$79	$182
14	$ 112,950	$ 1,410	$ 5,750	2450	$ 120,110	$ 192,176	59%	0.7%	3.0%	$78	$180
15a,b	$ 379,940	$ 2,780	$ 15,820	7,920	$ 398,540	$ 637,664	60%	0.4%	2.5%	$81	$185
16	$ 126,000	$ 600	$ 10,160	2,680	$ 136,760	$ 218,816	58%	0.3%	4.6%	$82	$188
17	$ 197,100	$ 670	$ 8,640	4,320	$ 206,410	$ 330,256	60%	0.2%	2.6%	$76	$176
19a,b,c,d,e	$ 761,630	$ 4,390	$ 40,730	12650	$ 806,750	$ 1,290,800	59%	0.3%	3.2%	$102	$235
20a	$ 142,200	$ 1,660	$ 6,720	3,100	$ 150,580	$ 240,928	59%	0.7%	2.8%	$78	$179
20b	$ 42,130	$ 200	$ 3,200	330	$ 45,530	$ 72,848	58%	0.3%	4.4%	$221	$508
21	$ 57,600	$ 300	$ 7,720	1220	$ 65,620	$ 104,992	55%	0.3%	7.4%	$86	$198
22	$ 65,250	$ 2,080	$ 6,080	1,390	$ 73,410	$ 117,456	56%	1.8%	5.2%	$85	$194
23a,b,c	$ 511,920	$ 4,200	$ 35,340	10570	$ 551,460	$ 882,336	58%	0.5%	4.0%	$83	$192
24	$ 88,650	$ 260	$ 3,820	1910	$ 92,730	$ 148,368	60%	0.2%	2.6%	$78	$179
25	$ 69,750	$ 900	$ 7,230	1,490	$ 77,880	$ 124,608	56%	0.7%	5.8%	$84	$192
26	$ 470,700	$ 1,690	$ 16,400	8,200	$ 488,790	$ 782,064	60%	0.2%	2.1%	$95	$219
27a	$ 334,800	$ 3,500	$ 13,990	6,500	$ 352,290	$ 563,664	59%	0.6%	2.5%	$87	$199
28a,b,c,d	$ 889,060	$ 3,970	$ 30,920	15,030	$ 923,950	$ 1,478,320	60%	0.3%	2.1%	$98	$226
Average	**All Sites**						**59%**	**0.4%**	**3.4%**	**$89**	**$204**
	IPA (Sites 10-14)						**59%**	**0.5%**	**2.9%**	**$79**	**$181**

(1) Includes 50% of total constrution cost for operation, maintenance and replacement, plus 10% total construction cost for final design, supervision and contingencies.

(2) 1984 costs escalated by 2.3, based on the change in Engineering New Record Construction Cost Index between 1984 and 2010.

NOTE: For purposes of this reconnaissance-level evaluation, it was assumed that the rock sizing and application rates used for the original design are appropriate for the recommended repairs and additional protection. The preliminary recommendations were developed based on the available topographic information and qualitative site observations. As a result, the cost estimate and specific dimensions of the individual sites should be used for planning and longer-term budgeting purposes only. Additional, site-specific surveys and quantitative investigations, including hydraulic modeling and scour analysis, should be conducted to support permitting and detailed design of the sites.

REFERENCES

Bagnold, R.A., 1960. Some Aspects of the Shape of River Meanders. USGS Professional Paper 282-E.

Begin, Z.B., 1981. Stream Curvature and Bank Erosion: A Model Based on the Momentum Equation. Journal of Geology, v. 89.

Carey, W.C., 1969. Formation of Floodplain Lands. ASCE Journal of Hydraulic Engineering, v. 95.

Federal Emergency Management Agency, 2010. Flood Insurance Study, Douglas County, Nebraska and Incorporated Areas, FIS Number 31055CV000C, May, 127 p.

Hickin, E.J., 1975. The Development of Meanders in Natural River-Channels. American Journal of Science, v. 274.

Nanson, G.C. and Hickin, E.J., 1986. A statistical analysis of bank erosion and channel migration in western Canada. Bulletin of the Geological Society of America 97:497-504.

Odgaard, A.J., 1987. Streambank Erosion along Two Rivers in Iowa. Water Resources Research, v. 23, no. 7.

Page, K. and Nanson, G.C, 1982. Concave Bank Benches and Associated Floodplain Formation. Earth Surface Processes and Landforms, v. 7.

Simons, Li & Associates, Inc. 1985. Feasibility Study and Nebraska Natural Resource Fund Application for the Lower Elkhorn River Improvement Project Area—Bank Stabilization and Recreational Development Project. Prepared for Papio Natural Resources District, Omaha, Nebraska, May 10.

U.S. Army Corps of Engineers, 2009. HEC-SSP Statistical Software Package. Version 1.1, Hydrologic Engineering Center, 214 p.

U.S. Geological Survey, 1982. Guidelines for Determining Flood Flow Frequency, Bulletin #17B of the Hydrology Subcommittee, 194 p.

Wells Engineers, Inc. 1987. Construction plans for Elkhorn River Bank Stabilization, King Lake Segment, Papio Natural Resources District.