sheet pile cut-off wall outside the limits of the paved channel section have a 4-ft high concrete cap wall. The sheet pile is shown extending a minimum of 30-ft below the sides of the paved channel section, and approximately 3-ft into shale, to elevation 958.

- A concrete cut-off wall along the downstream edge and top sides of the structure, extending 4-ft below the paved slopes, adjacent top slabs, and invert. The center portion of the concrete cutoff wall is shown as supported by timber or H-piles on 7-ft centers. The piles were to extend to elevation 953 or refusal.
- Two 8-in cast iron pipe intakes, on the upstream face of the weir, at elevations 985.33 and 987.66.
- A 2-ft layer of riprap, on a 6-in layer of bedding, extending 20-ft downstream of the channel paving and 25-ft past the top of banks (the entire width of the sheet pile cutoff wall). The riprap was to be placed flush with the surface of the structure; the downstream edge is shown as vertical, flush with the adjacent ground surface.

The plans did not show riprap or any other slope protection along the upstream face of the structure. The plans showed areas of grading disturbance adjacent to the structure significantly greater in extent than the coverage of the riprap. No information on riprap gradation was found in the plans; presumably it was defined by construction specifications. The plans showed a boring log for the structure site: it indicated a top layer consisting of 4½-ft of loose clayey silt, underlain by approximately 38-ft of silty clays and clayey silts, then by shale. The top of shale was shown at elevation 961.8.

Aerial Photographs. The Delaware River in the project reach appears to be a relatively recent, manmade channel. The 1954 aerial photo from the 1960 Brown County Soil Survey Report appeared to show the river channel not at the current location; rather it followed a meander to the north. The 1957 United States Geological Survey (USGS) map showed a new channel cut through the southern extent of the meander and running west to east generally straight for some 1000-ft.

Subsequent aerial photos dated 1981 and 1991 showed the river channel in the current location with the intake structure and weir in place. Both photos depicted the straight channel having some vegetation and trees on and adjacent to the bank. The 1981 aerial appeared to show the riprap indicated on the original plans; on the right bank and toe, that riprap seemed to be missing in the 1991 aerial photo. It was indeterminate whether riprap remained on the left bank in that picture. Two sandbars appeared on the 1991 aerial photo that were not evident on the 1981 image. The 1991 aerial photo showed a large bar deposit in the middle of the channel downstream of the structure. There also appeared to be a deposit of material along the left toe, just downstream of the original riprap location.

Site Inspection Photographs. The Corps of Engineers (COE) provided inspection photographs from several dates: June 24, 1987; June 30, 1993; November 9, 1993; and June 21, 1995. There was little or no rock protection remaining on the right bank slope downstream of the structure in the 1987, 1993 and 1995 photographs. It appeared that the rock was washed away leaving a near vertical, bare soil slope. There was a visible washout beneath the right bank concrete sill/cutoff wall on the downstream side, which progressively worsened throughout the period covered by the 1987 to 1995 photographs.

Immediately upstream of the structure on the left side there was visible bank erosion in photographs prior to the 1996-1997 construction. For a few hundred feet upstream of the erosion adjacent to the weir structure, the photographs showed that the banks were generally vegetated down to the low flow channel. The photographs (1987 through 1999) consistently showed a steep, bare soil bank on the right bank at the outside of the first channel bend upstream of the structure. The appearance and apparent extent of that steep soil bank did not change much through the period covered by the pictures.

The progression of photographs did not as thoroughly cover the left bank downstream of the structure, but it generally appears that there was some rock slope protection covering the majority of the slope, with exposed soil areas visible near the structure. The pre-project (1996-1997 construction) rock protection appeared loose and not interlocked, had an irregular surface, and consisted of many long, flat rocks. Scattered tall weeds throughout the surface indicated the rock protection was probably thin. A June 1995 picture showed four square concrete slabs with manhole rings in the middle on the left slope downstream from the weir. Approximately 200-ft downstream of the structure, the succession of photographs showed a large sand/gravel bar situated in the middle of the channel. It appeared to be progressing downstream over time.

Previous Repairs. Repairs had been reportedly performed more than once, prior to the original (1996-1997) construction of the bank revetments. This is evidenced by 1987 photographs showing riprap on slopes downstream of what was included in original construction plans, and by some riprap that appeared on 1995 photographs of the right bank just downstream of the structure, where bare soil existed in 1993 photographs. The original streambank stabilization construction plans and documents were prepared in 1994-1995 and construction occurred in late 1996 and early 1997.

The stabilization efforts attempted to restore protection to the eroded downstream side slopes and channel bottom and add slope protection to the upstream slopes near the structure by placing a 3-ft minimum depth of rock stone revetment on 2:1 slopes with thickened toes. The stone protection extended for some 45-ft upstream to approximately 75-ft downstream of the structure, with 25-ft long transitions to existing at both ends. The irregular bank surface below the 3-ft revetment depth was to be filled with stone or concrete rubble. A 2-ft thickness of stone was placed on the bottom of the channel for a distance of 25-ft downstream of the intake. A washed-out area under the concrete sill/cut-off wall along the right side of the downstream edge of the structure was filled with gravel. Subsequent high flows in the channel, since the repairs, have caused damage to the rock slope protection.

Weir Raise. To remedy water supply shortages experienced during dry weather, the Kickapoo Tribe constructed a 2-ft temporary weir raise in July of 2000, but it collapsed within a month of construction. The supplementary structure was constructed of steel channels anchored to the existing concrete weir and removable wooden stoplogs.

In May of 2001 another 2-ft temporary weir raise constructed, this time designed and funded by the Bureau of Reclamation. The supplementary structure was constructed of welded steel channels and plates anchored to the existing concrete weir and removable wooden stoplogs. Plans for this raise stated that the stoplogs are to be removed during expected high flows and over winter months. Also, this weir raise was expected to be temporary, lasting only until an alternate water source could be implemented. During subsequent flood events (notably the May 2007 event), this raise suffered significant damaged and is no longer functional. The Federal Emergency Management Agency (FEMA) is currently in the process of replacing the structure.

ANALYSIS OF THE STREAMBANK EROSION PROBLEM

Flow Conditions. The erosion, bank instabilities, and riprap displacement that have occurred since the construction of the intake structure are mainly the result of the impact of the structure on flow conditions in that river reach. Velocity fluctuations that occur at the weir and paved channel section result in flow turbulence. The observed downstream erosion patterns, both in the riprap that was installed in 1996-1997 and in the left and right bank erosion areas, are likely the result of the flow turbulence. Prior experience with turbulent flow conditions at other sites has been that larger riprap than normally required is necessary for calculated velocities. A type of stilling basin has formed immediately downstream of the structure, consisting of a hole in the channel bottom and deposits of stone formerly located in riprap revetments on the channel sideslopes. For some discharges, there are probably secondary hydraulic impacts downstream of this stilling basin. Hydraulic analysis indicates the potential for a hydraulic jump occurring in the channel downstream of the structure; if this actually occurs, the resulting forces and turbulence may be beyond what any reasonable riprap gradation can withstand. The turbulent flow conditions and the stilling basin formation suggested that analysis should be performed using the COE Waterways Experimental Station (WES) Hydraulic Design Criteria 712-1.

Materials. It had been reported that much of the stone in the three-foot thick riprap layer installed in the previous stabilization project had been 6 to 8-inch stone. At the site investigations for this study, most of the stone remaining in place was estimated to be 4 inches to 6 inches in largest dimension. The specifications for the previous project apparently allowed shot rock, and the stone delivered and installed at the site was apparently too small to be stable for the velocities and turbulence that can occur.

The riprap installed upstream of the weir in the prior stabiliztion project is adequate in size, according to preliminary hydraulic analysis and preliminary Riprap15 results.

Photographs and observations of remnants of previous repairs were inconclusive. The material appeared to have been larger, but gradations may not have provided enough range of size to allow fully effective interlocking. The riprap installed with the original construction of the structure did not extend upstream of the structure, therefore providing no protection for the acceleration of flow towards the weir. The original riprap also only extended 20 feet downstream of the weir, an inadequate distance for the increased velocities and turbulence resulting from the structure.

Geotechnical. Based on a review of available boring logs, the shale bedrock encountered at about elevation 960 is covered by a thick alluvial deposit of clay and silt with a few sand seams. The near-surface soils (those above about elevation 1000) are described on the National Resource Conservation Service (NRCS) soil survey as Kennebec silt loam, a moderately well drained material formed in the floodplain. These soils are considered susceptible to erosion, particularly the silty portions.

During the site visits described in this report, no scarps or other indicators of deep-seated slope instability were observed along the river banks upstream and downstream of the weir structure. Some of the remaining riprap from previous repairs was found to be highly friable, likely as a result of weathering. Downstream of the weir, bank instability is apparent with the observed vertical slopes and sloughing that is actively widening the channel on both sides. On the other hand, a vertical slope on the right bank upstream of the weir has not significantly sloughed for several years.

During site visits, the team did not observe any evidence of near surface slides on riprap covered 2(h):1(v) slopes, but this could be because subsequent or concurrent erosion removed slumped materials and other evidence.

Conclusions. It appears that energy dissipation downstream of the intake structure was not addressed by the original design. The original design also apparently did not consider the upstream effects of the acceleration of flow towards the weir. The riprap stone installed downstream of the structure in the initial stabilization work was too small for the velocities and turbulence that occur downstream of the intake structure. Stone installed previous to 1996 may have been of poor gradation that did not allow adequate interlocking of stones. The riprap stone installed upstream of the structure in the initial stabilization work appears to be adequate.

A hydraulic jump may occur in the channel downstream of the structure for a limited range of discharges. Occurrence of a hydraulic jump would have heavily damaged any of the previous revetments.

SITE HYDROLOGY AND HYDRAULICS

Discharges. The Delaware River has a drainage area of approximately 135 square miles at the project site. A USGS recording stream gage is located 14 miles downstream, near Muscotah, Kansas. The gage has been in operation since 1970. The drainage area at the gage is 431 square miles. The COE computer program HEC-FFA was used to determine expected probability discharges at the gage. Those discharges were then adjusted for the drainage area at the project site. Regression equations as developed for the state of Kansas by the USGS were also used to estimate discharges for various frequencies.

The table below shows the results of the hydrologic calculations for discharges at the project site.

Exceedance Probability	Return Frequency	Gage Analysis	USGS Regression
50%	2-Year	5,330 cfs	5,980 cfs
20%	5-Year	7,980 cfs	10,400 cfs
10%	10-Year	9,830 cfs	15,700 cfs
2%	50-Year	14,100 cfs	28,200 cfs

Normally, the results of gage analyses match regression results more closely. The procedure followed to adjust for the difference in drainage areas between the USGS gage site and the project location may have caused underestimation of the discharges at the intake structure. For expected discharges at the study site, the results of the regression analysis are assumed to be more accurate. Also, it was reported in 2001 by Bill Allen of the Kickapoo Environmental Office that a 2-in rain would cause the channel to flow half to two thirds full, and that bank-full conditions would result from 4 to 5-in of rain. This, in conjunction with the channel capacity calculations described in the following paragraph, supports the higher discharge results (by indicating that bank-full events are not extremely rare).

Hydraulic Modeling. To assess the flow conditions at the intake structure for a wide range of flow conditions, a HEC-RAS model was developed from the geometric information available from surveys. The model extended from about 4,400-ft downstream of the structure to 400-ft upstream. A wide range of discharges was analyzed (100 cfs to 16,000 cfs), and the model was run for mixed flow conditions, starting at both ends from an average channel slope determined from the USGS map. The hydraulics of the river was analyzed for different scenarios, as well as for sensitivity to varied conditions. The n-values were originally assigned relatively low values, for conservatism in hydraulic conditions, because higher downstream velocities would lessen the tailwater depth and increase the possibility for the occurrence of a hydraulic jump. As a result of photographs and field observations of significant flow events, a calibration exercise was performed that increased the n-values.

Results of the Hydraulic Analysis. The weir raise conditions should not have a significant effect on the proposed protection, although the plunge pool design was based on the long term existing plunge pool configuration. HNTB's analysis of the weir raise indicates that the proposed streambank stabilization design (designed for the pre-weir raise condition) will also provide adequate protection for the 2-ft raise as shown in the Bureau of Reclamation plans (RFQ NO. 01SQ600089). This analysis is based on all stoplogs in-place at an equal elevation across the structure. Weir raise effects become unpredictable if weir raise stoplogs are out of place, debris accumulates on the weir, or other undesirable situations (such as an ice jam) arise that cause non-uniform flow patterns across the weir raise. Sustained non-uniform flow has the capability to jeopardize the function and integrity of the proposed stabilization protection. For this reason, proper flow conditions across the weir raise need to be monitored and maintained by water plant personnel as part of ongoing plant operations.

The true size of the existing plunge pool is likely heavily dependent on the major and abrupt drop from the end of the concrete section to the adjacent downstream channel, once the channel slopes have eroded. This has been the predominant condition for many years at the site, except for brief periods subsequent to previous repairs. This abrupt drop will not exist after construction of the proposed bank stabilization project. The impact of the weir raise on the size of the plunge pool is expected to be more than counter-balanced by the elimination of the abrupt drop at the end of the structure.

CONCEPT DESIGN ALTERNATIVES

In order to meet the goals of an emergency streambank stabilization project at this site, five (5) alternatives were considered. Exhibits showing a schematic drawing for each alternative can be found in the Golden Eagle Feasibility Analysis dated May 29, 2008. Typical sections for the various improvements discussed can also be found in Exhibit 9 of the Feasibility Assessment.

Alternative 1 Riprap Stilling Basin, LPSTP (Longitudinal Peaked Stone Toe Protection), and Slope Revetment (Page 11)

The first 60-ft downstream of the weir would be protected by a 54-in thick large diameter rock blanket. It would match the geometry of the downstream section of the concrete drop structure, with a top elevation of 1004.00 and 2:1 sloping sides. On the left bank, a 90-ft slope revetment transitioning from full bank height (elevation 1004.00) upstream to half bank height (elevation 995.00) downstream would be installed. It would have a thickness of 24-in on the 2:1 side slope. The first 40-ft of slope revetment would be supported at the bottom with a toe blanket and the remaining 50-ft would be supported with an incised section of toe rock. This incised toe would be an inverted trapezoidal section, 36-in thick with a 10-ft flat bottom. The 48-in thick toe blanket would provide toe support for the left bank slope revetment and protect the channel bottom from the transition out of the large riprap blanket plunge pool.

LPSTP would be placed along both the left and right banks. The left bank protection would match the end of the slope revetment on the upstream end with a top elevation of 995.00 and a 2:1 riverward side slope and will have a 1.5:1 landward side slope. It would transition for the first 60-ft in length, down to top elevation 992.00 and a 1.5:1 riverward side slope. The LPSTP would continue downstream for 120-ft on a linear alignment, where it would tie into the existing bank. The right bank would have a combination of slope revetment and LPSTP protection. It would match the end of the large riprap blanket on the upstream end with a top elevation of 995.00 and a 2:1 riverward side slope and would have a 1.5:1 landward side slope. It would transition for the first 40-ft in length, down to top elevation 992.00 and a 1.5:1 riverward side slope. The LPSTP would continue downstream for 195-ft with a linear alignment, where it would tie into the existing bank. Overall, the left bank protection would extend 30-ft further downstream than the right bank protection.

Within the protective works area, the channel would be graded to a flat flowline elevation of 985.00, with the exception of the invert within the large riprap blanket plunge pool. Downstream of the proposed protective works for a length of approximately 200-ft, the existing channel has dual thalwegs and an island accretion of rocky material. The excess material in the center of the channel would be moved toward the split channels near the left and right banks, and graded to drain toward the center and downstream in the channel to an elevation of 984.50 at its terminus.

Alternative 2 Concrete Stilling Basin, LPSTP, and Slope Revetment
The stilling basin in this alternative would have the first 45-ft downstream of the weir protected by a concrete stilling basin with 11-foot high walls instead of the riprap stilling basin. The other features of this alternative are identical to Alternative 1.

Alternative 3 Sheet Pile Stilling Basin, LPSTP, and Slope Revetment
The stilling basin in this alternative would have the first 45-ft downstream of the weir protected by an 11-foot high sheet pile stilling basin with a concrete lined bottom instead of the riprap stilling basin. The other features of this alternative are identical to Alternative 1.

Alternative 4 Sheet Pile Wall
Another alternative that was further assessed subsequent to detailed hydraulic analysis was the usage of sheet piling. For this variation, riprap revetments would not be used for armoring the presently unprotected eroding banks. The sheet piling would be driven along the downstream edge of the intake structure (existing sheet pile only provides formwork for grout work and protection for interface between grout and soil) and down the left bank to protect the waterline. There would also be some sheet pile driven for a short distance down the right bank to prevent the existing concrete weir from being flanked. The scour hole would remain as is and the stream would be left to run its natural course. The scour hole would be allowed to expand, to serve as an energy dissipater.

Alternative 5 Riprap Stilling Basin with Stabilized Slopes
The soils behind the large riprap blanket and the slope revetment would be stabilized to provide slopes that satisfy criteria specified in EM 1110-2-1913. Horizontal layers of geogrid reinforcement vertically spaced at 4-ft are proposed. The geogrid would be embedded in 12-in layers of sand (6-in above and 6-in below). The remaining 3-ft of material between sand layers would be soil fill from the site. Filter fabric wrapped around the riverward face of these 3-ft layers would prevent migration of fines into the gravel bedding layer below the riprap. The top elevation of the stabilized slopes

would match the top of the faced rock protection. A rock layer would be placed below the soil, sand and geogrid stabilization. The displaced existing riprap on the site would be recovered for this rock layer. The bottom of this rock layer would be at elevation 980.00 for the large riprap blanket and slope revetment with toe blanket, and elevation 981.00 for the slope revetment with incised toe.

CONCLUSIONS AND RECOMMENDATIONS

Protection of the site is technically and economically feasible. The first priority is to protect the water line and intake structure. The second priority is to stabilize the left bank downstream, to protect the access road. The third priority, while not required by the project, is to stabilize the right bank downstream in order to help control flow conditions at the intake structure. Protection on the right bank would help prevent an asymmetrical scour hole and potential flanking of the weir structure, so it is felt to be important.

A "No Action" alternative would likely result in eventual damage to the structure and require its complete replacement, as it is not believed that the piling at the structure would be adequate for its support if most of the underlying soil washes out. Replacing the structure is a viable alternative, especially if the new structure incorporated an energy dissipater; however, replacement would be far more expensive than stabilizing the downstream channel.

Biostabilization techniques were determined not to be a feasible option for this project based upon the turbulent flow conditions and excessive velocities through the proposed stabilization area. In addition, the scour hole that is already present needs to be addressed by the chosen solution.

The hydraulic analysis shows that unstable conditions (potential hydraulic jump) may exist downstream of the weir due to the abrupt drop from the end of the concrete section to the adjacent downstream channel. All alternatives except Alternative 4 address this issue. The sheet pile wall protects the road and water line, but it does nothing for the existing scour hole or erosion elsewhere. Alternative 4 is also significantly more expensive than the other alternatives.

Alternatives 2 and 3 could have safety issues because the sheetpile or concrete will leave high vertical walls. As people have been known to fish the area, safety measures such as railing and/or barriers would be needed at a minimum. That cost is not currently included in the estimates, which already show higher costs than Alternatives 1 and 5.

The geotechnical investigation performed in 2000 included slope stability analysis. Results indicated that the slope stabilization measures described in Alternative 5 are needed to meet the design criteria specified in EM 1110-2-1913 for levee slopes. Because (1) site observations to date have found no evidence of deep slides and (2) the consequences of a bank failure downstream of the intake weir are not commensurate with failure of a levee, the additional costs associated with Alternative 5 do not appear to be warranted.

Alternative 1 is the recommended solution for the site. This riprap stilling basin, along with the LPSTP and slope revetment meets all of the priorities of the proposed project and is the most economical.

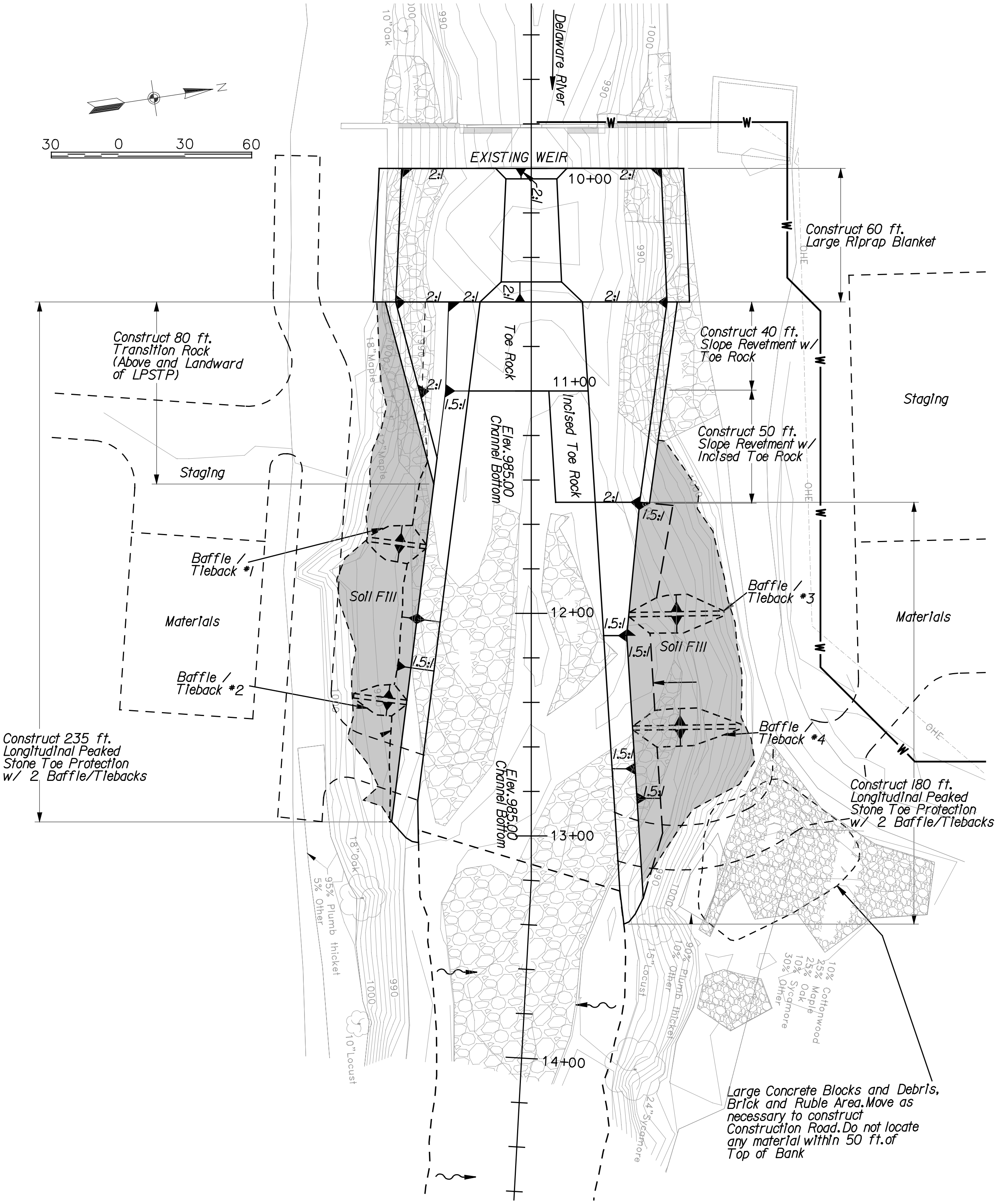
SOURCES OF INFORMATION

Aerial photographs were acquired from the NRCS Brown County Field Office in Hiawatha, Kansas. Plans for the intake structure, previous site inspection photographs, and the 1995 bank stabilization plans were provided by the Kansas City District (KCD) COE.

The following is a list of the information used for this paper:

- USGS Horton NW Quadrangle, 1:24,000 (based on 1957 aerial photography).
- USGS Kansas City 1 x 2-Degree Series Map, 1:250,000 (based on 1951-1952 and 1973 aerial photography).
- Aerial Photographs dated 1981 and 1991.
- NRCS Soil Survey aerial photograph, dated 1954.
- Brown County NRCS soil properties description sheet.
- Construction plans for Kickapoo Water Supply System, including Water Intake Structure plans.
- KCD COE Bank Protection plans (dated 1994-1995).
- USGS stream gage data, Delaware River.
- Detailed Project Report: Delaware River, Water Intake Weir, Kickapoo Tribe of Kansas, Brown County Kansas; CWIS Number 92942; March 1994; KCD COE.

Golden Eagle Section 14 Emergency Streambank Protection Feasibility Assessment; May 2008; Kansas City District Corps of Engineers; Eric Lynn, Project Manager.



Delaware River
30 0 30 60
EXISTING WEIR
10+00
Construct 60 ft. Large Riprap Blanket
Construct 80 ft. Transition Rock (Above and Landward of LPSTP)
Toe Rock
Construct 40 ft. Slope Revetment w/ Toe Rock
11+00
Staging
Incised Toe Rock
Elev. 985.00 Channel Bottom
Construct 50 ft. Slope Revetment w/ Incised Toe Rock
Staging
Baffle / Tieback #1
Soil Fill
Baffle / Tieback #3
Materials
12+00
Materials
Soil Fill
Baffle / Tieback #2
Baffle / Tieback #4
Construct 235 ft. Longitudinal Peaked Stone Toe Protection w/ 2 Baffle/Tiebacks
Construct 180 ft. Longitudinal Peaked Stone Toe Protection w/ 2 Baffle/Tiebacks
Elev. 985.00 Channel Bottom
13+00
14+00
Large Concrete Blocks and Debris, Brick and Ruble Area. Move as necessary to construct Construction Road. Do not locate any material within 50 ft. of Top of Bank