WATER DISTRICT NO. 1
OF
JOHNSON COUNTY, KANSAS

KANSAS RIVER INTAKE JETTY IMPROVEMENTS STUDY

BLACK & VEATCH
building a world of difference™
ENERGY • WATER • INFORMATION • GOVERNMENT

PROJECT NO. 99409.108

May 20, 2005

Table of Contents

Kansas River Intake
Jetty Improvements Study
Executive Summary

One of the main sources of drinking water for Water District No.1 of Johnson County, Kansas, (the District) is the intake on the Kansas River. The intake is located about 1,000 ft downstream from the I-435 bridge.

Since the construction of the intake, degradation of the Kansas River bed has compromised the adequate functioning of the intake. A jetty with large dumped rock was constructed in 1967 to maintain enough head to guarantee adequate operation of the intake. Repair and maintenance of the jetty has been an ongoing task. A breach of the jetty that occurred sometime during March 14 - 15 of 2004 and a near breach that occurred in August 2004 prompted the need to evaluate the mechanisms that caused the problems and to provide recommendations addressing reliability of the Kansas River supply.

From the analysis of probable breach causes, two main potential failure mechanisms were identified: (1) undercutting of the downstream slope of the rip - rap due to high flow velocities over the jetty and (2) degradation of the river channel downstream of the jetty. In order to address these issues, seven alternatives were developed, as follows:

Alternative A. Alternative A is based on armoring the jetty with large rocks to withstand the probable flow conditions with the existing jetty downstream slope of approximately 9:1. In addition to periodic maintenance, the downstream riverbed conditions would be monitored and the jetty would be built up to stabilize the jetty and reduce the downstream slope as required to respond to any further downstream degradation.

Alternatives B, B1 and B2. Alternatives B, B1 and B2 are based on stabilization of the jetty against maximum downstream degradation (to bedrock) as a part of the initial construction. The jetty would be stabilized by reducing the downstream slope of the jetty to 16:1 (B and B1) or stepping the downstream slope (B2) and by armoring the downstream face of the slope with large riprap (B and B2) or another lining material such as A-Jacks (B1).

Alternative C. Alternative C adds construction of a sheet pile wall at the upstream face of the jetty to Alternative B.

Alternatives D and D1. Alternatives D and D1 consist of the construction of a cofferdam downstream or upstream from the jetty, respectively.

Other options and variations to the alternatives presented above were also considered. Even though feasibility of these options was addressed, they are not listed as alternatives because (1) they did not provide a suitable solution to address the stability of the jetty and/or (2) they would have resulted in unreasonable costs while providing marginal benefits when compared to the seven alternatives that were listed.

Alternative A could possibly be the least expensive option assuming that degradation of the riverbed downstream from the jetty will be much less than experienced in the past and that tailwater conditions will not deviate from current levels. The opinion of probable project cost of this alternative is $ 7.3 million. If tailwater elevations do not change significantly over time, maintenance costs would be minimized and the opinion of present value cost of this alternative would be $ 8.4 million. However, if the downstream riverbed degrades at previously measured rates, the opinion of present value cost of this alternative would be as high as $ 20.4 million. The main problems with Alternative A are that the risk of failure is much greater than Alternative D and that the present value cost could be quite high, depending on future rate of river degradation, when compared to other alternatives. Unfortunately, there has been no base level control identified that would arrest future degradation.

Alternatives B, B1 and B2 are cost prohibitive due to the volume of material required and other costs. The costs of these alternatives range between $30 and $60 million. Alternative C was discarded as a feasible option because it adds additional costs to Alternative B while providing marginal benefits.

Alternatives D and Dl provide the most permanent solution. The opinion of project cost and the present value cost for these alternatives are approximately $ 12.3 and $ 13.3, respectively. Alternative D was estimated to be marginally less costly than Dl.

However, the excavation of large rock that may have been previously washed out from the existing structure may pose constructability issues and makes Alternative D1 more attractive. Also, a stagnant pool would be created at the intake for Alternative D which could cause icing problems.

Alternative Dl is selected as the recommended plan. Even though this alternative could be more expensive than Alternative A, it provides a more permanent solution and minimizes risk of failure. In addition, future changes in the existing riverbed and the river hydraulic conditions could make Alternative A much more expensive than Alternative Dl.

This plan is different from that proposed by Black & Veatch in 1988. The recent breach and near breach are indications that the stability of the structure is uncertain under current conditions. Ongoing changes in bed elevation and changes in tailwater elevations provide indications that the stability of the jetty may be further compromised for future conditions. The high repair costs associated with the recent breach and the change in downstream riverbed conditions over the years makes a more permanent solution more desirable than it was when proposed back in 1988.

Advice on permitting and potential agencies that will require permits and coordination is provided in Section E of the report. It is important to note that permitting of a permanent structure can be complicated. On the other hand, in meetings the U.S. Army Corps of Engineers (USACE) has supported the construction of a permanent solution.

A value engineering workshop took place on November 19, 2004. The purpose of the workshop was to provide additional review by nationally recognized experts and the USACE and to discuss alternative approaches to resolve the problems associated with the jetty and the intake. The most promising solutions derived from this workshop are slight modifications to the cofferdam alternatives. These modifications should be considered during the design phase as a way to reduce costs.

The stability of the District's jetty as a grade control for the Kansas River is vital to the river stability upstream of the jetty. The jetty stops the Kansas River degradation that is most likely associated with the Missouri River degradation and protects the water supply, real property and environmental interests upstream of the jetty.

Since this grade control is a regional issue rather than a localized District issue, one key recommendation of the value engineering workshop was to pursue financial participation in the construction of a stable structure, not only for the District but also on behalf of upstream interests, from the U.S. Congress and the U.S. Army Corps of Engineers.

Funding requests for an EPA grant have been submitted to the Congressional offices of Senator Brownback, Senator Roberts, and Representative Moore. The status of the funding request is monitored periodically to ensure the offices have the information they need to request funding in the FY 2006 budget.

A meeting was held with the USACE March 31, 2005. The USACE indicated that funding by USACE for jetty improvements is unlikely. An overall river degradation solution is possible after the USACE proposed study of river degradation problems in the KC metro, but final recommendations will not be available for five or more years.

The District should continue to work with USACE to ensure no adverse impacts result from dredging downstream of the jetty. While construction of Alternative D1 eliminates the concern at the jetty, continued degradation will likely reduce the capacity of the existing wellfield.

Recommendations for proceeding with construction of Alternative D1 are as follows:

- Drill exploratory test holes at Grinter's Ferry to verify that it is not a permanent grade control.
- Continue to work with the USACE to obtain funding for the exploratory holes and to identify possible long-term solutions for the marked degradation to protect the wellfield.
- Begin design of Alternative D1 as soon as the test holes at Grinter's Ferry are completed.
- Retain a contractor to be on-call for emergency repairs of the existing jetty until Alternative D1 is complete.
- Continue discussions with the USACE and state agencies regarding permitting of Alternative D1.

Kansas River Intake
Jetty Improvements Study

A. Background

One of the main sources of drinking water for Water District No.1 of Johnson County, Kansas is the Kansas River. An intake was built in 1964 to draw water from the Kansas River. In 1963, the Kansas River streambed near the jetty had an average elevation of 734.5 ft. The sill of the intake was originally built lower than the streambed at an elevation of 732.0 ft.

Since then, degradation of the Kansas River bed has compromised the adequate functioning of the intake. A jetty with dumped rock was constructed in 1967 to maintain enough head for adequate operation of the intake. In 1968, the jetty had to be repaired with larger rock. The river bed continued to degrade and the jetty had to be restored and lengthened in 1969. Successive damage that required repairs was reported in 1970, 1972, 1974, 1975, 1976, 1979, 1983 and 1986. In 1988, a repair and maintenance schedule was proposed. A repair cycle has been maintained since.

Until now, the repair and maintenance plan proved to be an adequate and cost effective way to ensure continuous operation of the intake. However, recent events showed that such strategy may need to be reconsidered.

A breach of the jetty occurred sometime during March 14 - 15, 2004. Prior to the breach, the US Army Corps of Engineers (COE) made a large release of flow (approximately 20,000 cfs) from Tuttle Creek Reservoir because the lake level had risen 11 feet from rainfall. This release, plus added inflows from rainfall in other portions of the Kansas River basin, caused the total flow in the Kansas River at DeSoto to rise from approximately 1,500 cfs to over 35,000 cfs in about one day around March 5th. The flow diminished to about 17,000 cfs around March 7 and remained there until March 10. From March 10 to March 12, the flow increased to about 27,000 cfs and the jetty breach appears to have occurred with a flow of about 26,000 cfs as shown on Figure 1. Using the rating curve shown in Figure 2, the flow appears to have been at least 2 to 3 feet over the top of the jetty.

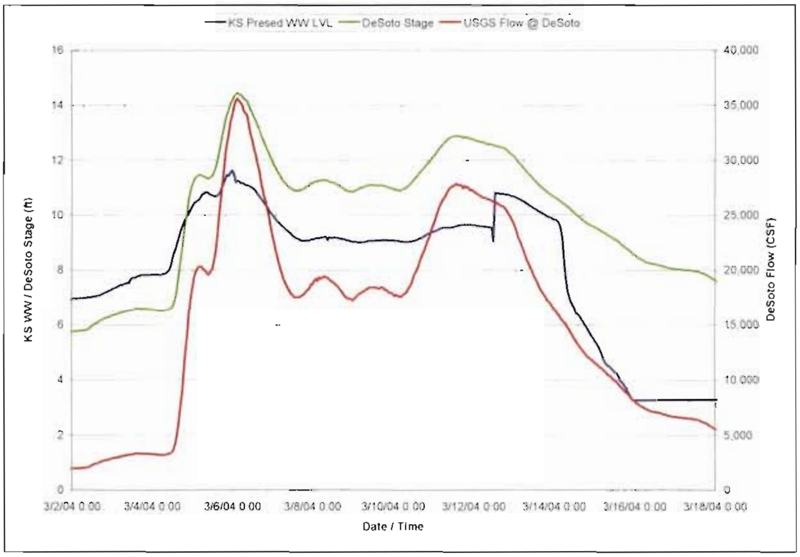


Figure 1. Kansas River Stages and Estimated Flows between 3/2/2004 and 3/18/2004

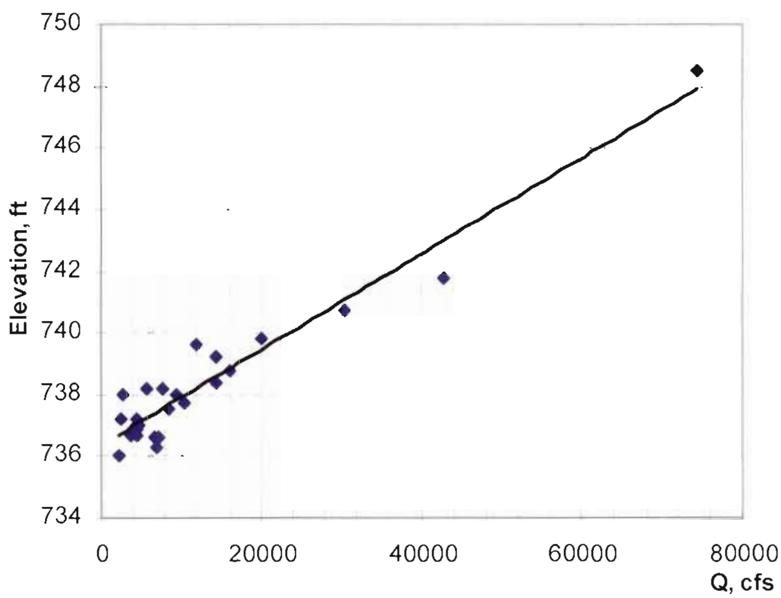


Figure 2. Rating Curve for WaterOne Jetty, 1985

The breach appeared to be about 50 feet wide at its narrowest point shown in Figure 3. The bottom of the breach was at about elevation 720, which is roughly 20 ft below the top of the jetty. Debris consisting of tree trunks and branches was observed on the top of the jetty from the breach to the northern bank of the river. Some debris was also located on the south side of the breach. The majority of the rest of the jetty was relatively free of debris.

Figure 3. WaterOne Jetty Breach

The structural integrity of the jetty immediately prior to the breach and the exact nature of the breach are not known. The walls of the breach above the water level were nearly vertical. The riprap exposed in the breach was a mixture of rock approximately 3 feet in its longest dimension mixed with smaller riprap. Fine-grained soils filled the voids between stones. Downstream from the breach, the flow was very turbulent and some erosion occurred on the north end of the jetty. On the upstream side, sediments were exposed and water level was lower than the sill elevation of 732 ft. It appears that a significant amount of sediment that had collected on the upstream side of the breach was eroded and carried past the jetty through the breach. During a field visit on March 17, the velocity through the breach appeared to be greater than 10 feet per second at a flow of about 7,000 cfs, according to USGS gauge information. The flow in the river diminished to about 5,000 cfs on March 18.

As degradation of the Kansas River bed progresses, the jetty will become more unstable. This report provides an evaluation of the mechanisms that potentially caused the breach. The report also provides recommendations on how to reduce the chance of occurrence of future breaches. The recommendations are based on the alternatives presented in the 1988 Black & Veatch report and other alternatives developed for this study. The analysis includes the advantages, disadvantages and feasibility of each alternative. Opinions of probable project costs are given for feasible alternatives.

B. Existing information summary

1. Topographic and bathymetric data

Topographic and bathymetric data of the jetty and its surroundings were obtained from the survey by Kaw Valley Engineering, Inc. This data consists of survey points and contours that describe the terrain before and after the breach occurred.

The different survey points were combined and used as the basis to create two digital terrain models (DTMs). The DTMs provide a visual description of the jetty geometry before and after the breach (Figure 4 and Figure 5). The "before" DTM was also used to cut cross sections for the hydraulic model and for geotechnical analysis.

Cross sectional data upstream and downstream from the structure for the Kansas River was obtained from the U.S. Army Corps of Engineers, Kansas City.

Geotechnical information was obtained from the 1988 Black & Veatch draft report, and the Kansas Department of Transportation (KDOT). KDOT provided the geotechnical information for the I-435 bridge located just upstream of the jetty.

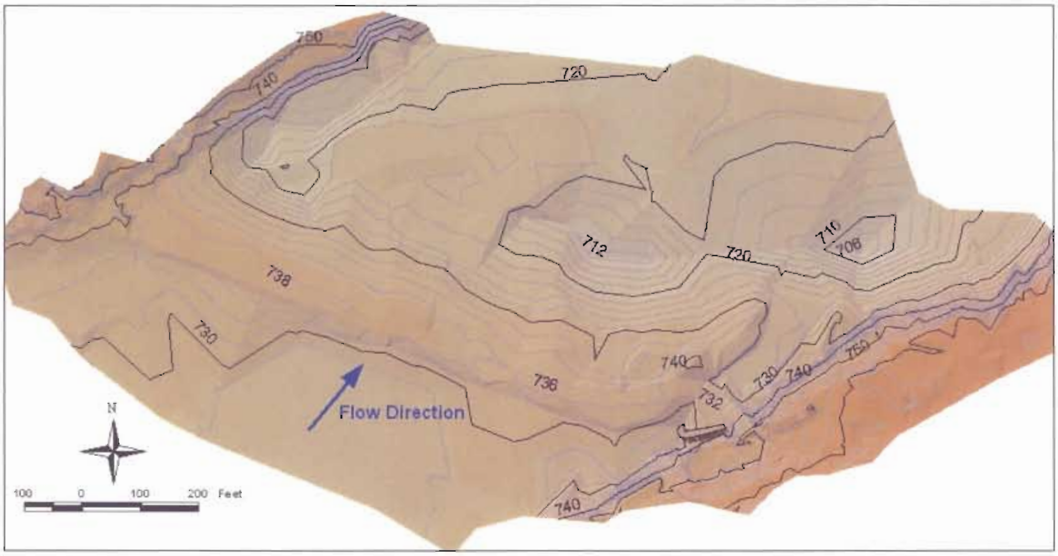


Figure 4. DTM, before breach

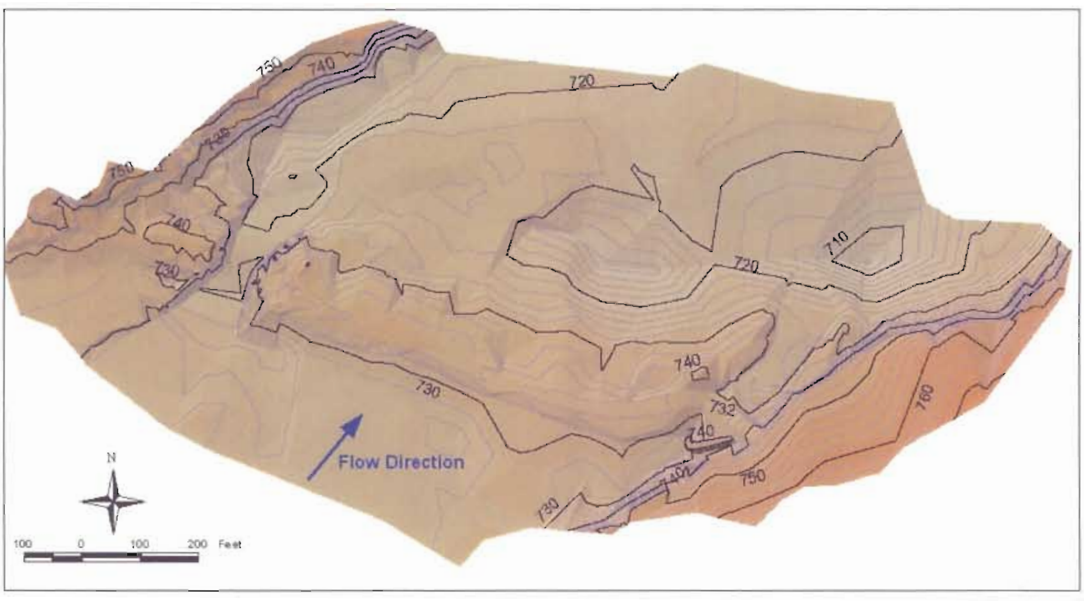


Figure 5. DTM, after breach

C. Failure analysis (potential breach causes)

While the available information is useful in determining conditions at the time of the breach it is not adequate to fully determine the exact cause of the breach. However, discussion and analysis of the potential breach causes is useful in evaluating the most effective solutions.

Two main failure mechanisms have been identified: (1) undercutting of the downstream slope of the rip-rap due to high flow velocities over the jetty and (2) degradation of the river channel downstream of the jetty.

1. Undercutting of the downstream slope

The difference in the upstream and downstream water surface elevations when the jetty is overtopped produces an energy gradient at the jetty. The energy excess is dissipated in two ways: (1) on the downstream slope of the jetty in the form of friction as water moves over the downstream slope, and (2) in the hydraulic jump that could form on the downstream side of the jetty. These two conditions have the potential to move the rip-rap. When the rip-rap moves, a channel could form and produce higher local velocities that progressively worsen the problem.

Debris that accumulated on the top of the jetty may also contribute to a potential breach. The debris could channel the flow over the top of the jetty causing high local velocities, thus increasing the potential for rip-rap displacement. The debris may also cause a greater differential in water surface elevation than normally occurs.

A hydraulic model of the structure was used to evaluate this failure mechanism.

a. Modeling approach

The computer program HEC-RAS was used for this purpose. The model consisted of a two-stage analysis.

The first stage consisted of an overall hydraulic analysis of the Kansas River. The geographic limits of the model were bounded by RM 17-95 and RM 10-35, about 3 miles upstream and about 4 miles downstream from the structure respectively. The approximate jetty location is at RM-14-8. Normal depth was assumed at the downstream end. This first stage analysis provided the headwater and tailwater elevations for the structure for the different flows analyzed.

Analysis shows that for flows in the range observed between March 5 and March 12 (17,000 - 35,000 cfs) the structure would be overtopped along its entire length. Under such conditions the top of the structure acts as a broad crested weir.

The second stage consisted of a localized hydraulic analysis of segments of the structure. This second stage assumes that the flow (q) conveyed by a segment of the crest of the structure will flow perpendicular to the crest and that the flow remains constant over the downstream slope of the jetty. Figure 6 illustrates the situation where a total flow (Q) of 35,000 cfs flows over the jetty. In this example, a local flow (q) of 371 cfs would flow over the selected segment. It also assumes that for a given flow the headwater and tailwater elevations are the same for all sections along the jetty crest. In this stage 1-foot sections representative of different locations along the jetty and weir were selected.

The second stage is based on basic open channel flow principles that include the development of a hydraulic jump on sloping aprons (Figure 7). It provides velocity estimates just before the jump occurs. This velocity is required to estimate recommended rock sizes. Rock size recommendations were obtained based on the COE Hydraulic Design Criteria Sheet 712-1 Stone Stability Velocity vs. Stone Diameter (Sheet 712-1).

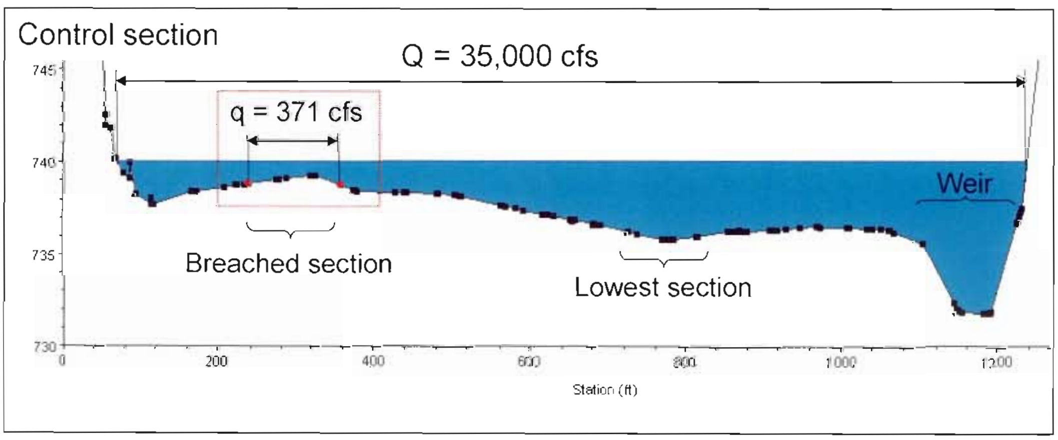


Figure 6. Model test locations.

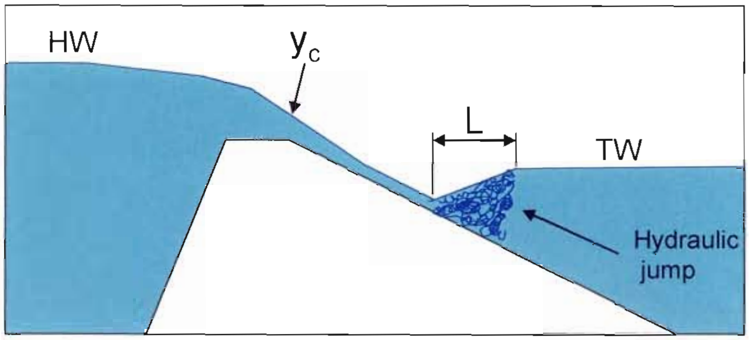


Figure 7. Hydraulic jump on a sloping apron schematic.

Three scenarios were modeled. Scenario (1) is a non-channelized situation; it models a condition where the downstream slope of the jetty has not been altered by flow. This scenario was tested at three locations: the breached section, the lowest section of the jetty and at the weir. Scenarios (2) and (3) model, respectively, the impact of 1-ft and 2-ft deep channels that may have been formed by an initial rock movement at the section that breached. These last two scenarios also approximate the situation where narrow sections of debris that may have accumulated on top of the jetty break away in 1- ft and 2- ft deep sections and channel the flow over the top of a segment of the downstream slope. Scenarios 2 and 3 were tested at the location where the March 14-15 of 2004 breach occurred only.

b. Hydraulic model results

The first stage of the model showed that under existing conditions the most damaging flows would be between 20,000 and 45,000 cfs. At higher flows the tail water submerges the structure. At this point the head difference across the structure is small and the velocities decrease substantially thus the potential for undercutting the downstream slope diminishes.

Table 1. Failure analysis results summary for different streamflow velocities at downstream slope and rock size that would withstand the velocities (limited to spring 2004 breach conditions).

Location		Discharge (cfs)							
(Scenario)		17500	20000	22500	25000	27500	30000	32500	35000
Section that breached in March, 2004.									
** (1) Non-channelized condition	V (ft/s)	10	11	12	12	13	14	15	15
	W50 (ton) *	0.02	0.04	0.06	0.09	0.13	0.17	0.23	0.30
	D50 (ft)	0.8	0.9	1.1	1.3	1.4	1.6	1.8	1.9
*** (2) 1 ft channel	V (ft/s)	13	14	14	15	16	16	17	17
	W50 (ton) *	0.11	0.16	0.22	0.29	0.37	0.46	0.57	0.69
	D50 (ft)	1.4	1.5	1.7	1.9	2.0	2.2	2.4	2.5
*** (3) 2 ft channel	V (ft/s)	15	16	17	17	18	18	18	18
	W50 (ton) *	0.32	0.42	0.53	0.66	0.80	0.96	0.72	0.83
	D50 (ft)	1.9	2.1	2.3	2.5	2.6	2.8	2.5	2.7
Lowest section on jetty									
** (1) Non-channelized condition	V (ft/s)	21	22	22	21	21	22	22	23
	W50 (ton) *	2.18	2.53	2.91	2.09	2.33	2.60	2.88	3.17
	D50 (ft)	3.7	3.9	4.1	3.6	3.8	3.9	4.1	4.2
Weir									
** (1) Non-channelized condition	V (ft/s)	23	22	18	18	19	19	19	19
	W50 (ton) *	3.6	2.9	0.8	0.9	1.0	1.1	1.0	1.1
	D50 (ft)	4.4	4.1	2.7	2.7	2.8	2.9	2.8	2.9

*Assumes specific weigth of rock of 165 lb/ft^3

** non-canalized condition: models a condition where the downstream slope of the jetty has not been altered by flow

*** 1 ft, 2 ft channels: model, respectively, the impact of 1-ft and 2-ft deep channels that may have been formed by an initial rock movement at the section that breached; they also approximate the situation where narrow sections of debris that may have accumulated on top of the jetty break away in 1- ft and 2- ft deep sections and channel the flow over the top of a segment of the downstream slope

Table 1 summarizes the results of the second stage of the model. This table shows maximum velocities expected on the downstream slope for different streamflows. The flows shown on the table correspond to the range of flows registered in the Kansas River between March 5 and March 15. It also presents the size of the smallest rock that would withstand such velocities. The results indicate that even for a non-channelized flow situation, flow over the section that breached in March, could have moved rocks as big as 1.9 ft on the downstream slope of the jetty. A channel caused by either initial movement of particles or a debris washout situation would have worsened the conditions at the section that failed. Field observations indicate that the downstream slope of the jetty at the breach section had rocks on average of 1 ft in diameter (D50).

The results of the hydraulic analysis indicate that the undercutting of the downstream slope is a plausible explanation of the failure. However, degradation of the channel downstream of the jetty may have also influenced the failure of the jetty.

2. Degradation of the channel downstream of the jetty

Degradation of the channel bed downstream of the jetty has been well documented. Between 1918 and 1977 thalweg elevations in the Kansas River at RM 0-6 (0.6 miles upstream from the confluence with the Missouri River) decreased about 15 feet and between 1963 and 1978 the channel bed lowered 14.5 ft just downstream from the jetty intake (Black and Veatch, 1988). The jetty is approximately located at RM 14-8. Figure 8 shows the location of the Jetty and the Kansas River miles.

Several factors have been suggested as the cause for the degradation of the Kansas River. These factors are: (1) dredging of sand and gravel in the lower Kansas River, (2) lowering of the Missouri River stages, (3) construction of major reservoirs upstream, and (4) natural degradation (Black and Veatch, 1988).

Recent findings indicate that degradation of the channel bed may be an enduring phenomenon. Comparison of cross section surveys obtained for 1999 and 2003 indicate an average reduction in bed elevation of more than two feet for RM 14-1 and RM 13- 8; with thalweg (lowest point of a river bed) reductions of 3.8 ft and 8.9 ft respectively. RM 14-1 is located approximately half a mile downstream from the jetty and RM-13-8 is located about one mile downstream from the jetty.

The question has been asked regarding the existence of a geologic formation, hard-point, downstream of the jetty that would stop the riverbed degradation. Two potential locations were identified: (1) Grinter's Ferry and (2) near the Turner Bridge.

In order to verify the existence of the geologic control formations, an onsite investigation was performed at the Grinter's Ferry and Turner Bridge locations. If the geologic control is present, it would be identifiable on both banks of the river. At Grinter's Ferry, the limestone formations were identified on the north bank of the river. A surveyed cross-section showed the formation present from elevation 737 down to ground elevation 720. The river thalweg elevation was identified as elevation 717. No bedrock formations were evident on the south bank. Also, Water District No. 1 of Johnson County's wellfield is located in the floodplain on the south side of the river. Test holes and geologic logs of the wells show no evidence of bedrock above approximate elevation 700. No evidence of a bedrock formation was evident near Turner Bridge during the field investigation. The March 31, 2005 meeting with the USACW confirmed they do not believe that Grinter's Ferry is a permanent geologic control. They also indicated a small amount of funding may be available for assisting the District with exploratory test holes to verify geological conditions at Grinter's Ferry.

Based on the geologic data and the field reconnaissance, it can be concluded that there is no permanent geologic control downstream of the jetty to inhibit river degradation.

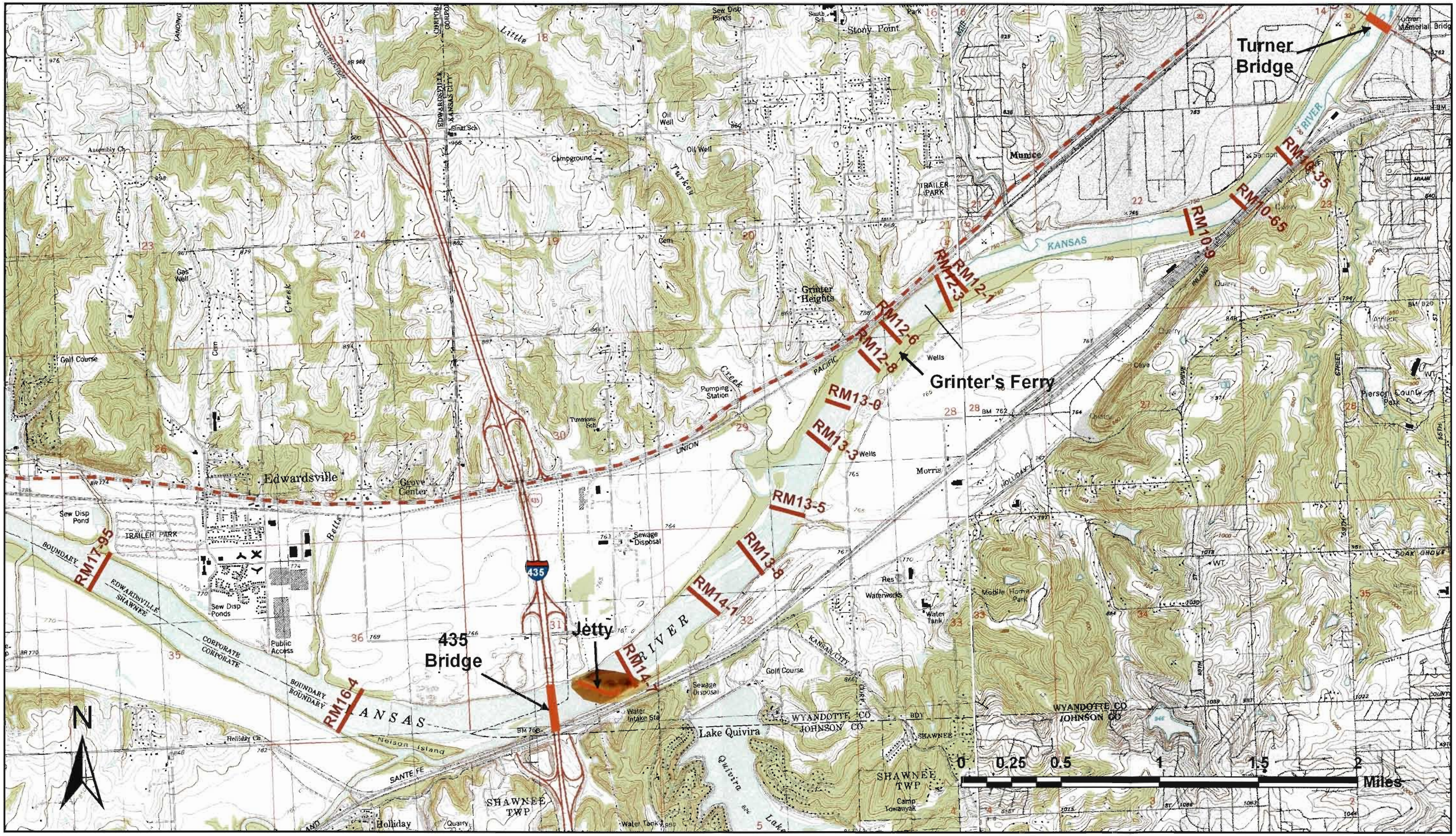
Ongoing degradation has two main effects on the jetty. The first effect is a reduction in tailwater. Tailwater elevation estimates presented in the 1988 Black & Veatch draft report and estimates from a hydraulic model for current conditions were compared. The results indicate that tailwater elevation decreased between 5 ft and 4 ft for flows between 5,000 cfs and 25,000 cfs respectively from 1988 to present. A reduced tailwater increases the head differential at the jetty; this condition produces an increase in the velocities and the shear stress at the downstream slope of the structure. The increase in velocity and shear stress would make the downstream slope of the structure more susceptible to undercutting. The 4 to 5 ft reduction in tailwater elevation estimates represents further evidence that degradation is still an ongoing process in the Kansas River.

The second effect is related to the structural stability of the jetty. As the degradation approaches the jetty and the sandy riverbed material is removed, it allows the stones in the jetty to settle. This could have been happening prior to the breach.

Figure 8. Kansas River Jetty

Jetty Location and Cross Sections

BLACK & VEATCH



3. Increasing risk of failure due to degradation

Major breaches of the jetty have occurred twice since its construction in the early 1960's. This equates to a probability of failure of approximately once every 20 years. However, the evaluations indicate the likelihood of failure is increasing. The breach and near breach that have occurred this year also appear to confirm the evaluations.

Analyses were conducted of the available stream power, which is the driving force behind rock movement and jetty breaching. Figure 9 compares the stream power when the jetty was first constructed, what it was in 1988, what it is currently, and what it will be when ultimate degradation occurs. The stream power has increased significantly since the jetty was first constructed. The breach and near breach both occurred when the stream power was approximately 20,000 Hp corresponding to a flow of between 15,000 and 35,000 cfs. The chart shows that stream power equal to or exceeding 20,000 Hp will occur much more frequently in the future over a much wider range of flows. Therefore, the probability of failure will increase.

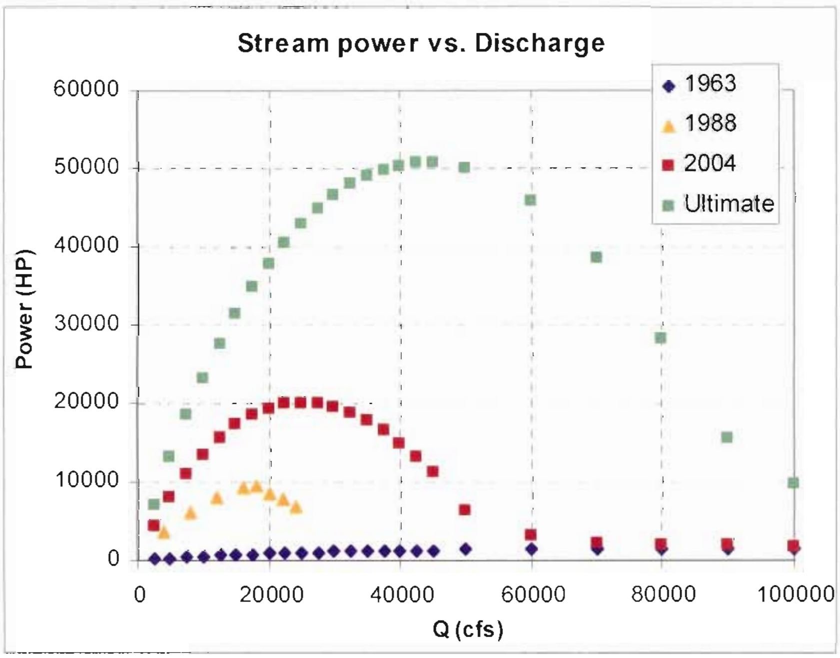


Figure 9. Stream Power vs. Discharge

The jetty does not breach or need significant repairs every time flows occur that create stream power of 20,000 Hp or more. Two other factors appear to be impacting the likelihood of major damage. During both events this year, a major debris jam occurred. This was apparently caused by high flows moving the debris down the river. Following the high flows, the flow diminished for a period of a few days allowing the debris to build up on the top of the jetty. The flows then increased. Damage occurred some time during this process. It is possible that the second period of high flows removed the debris over a relatively narrow width on the top of the jetty. This may have caused the flow to be concentrated where the debris was removed which focused the power of the water on a small area of the jetty leading to a breach.

4. Other potential failure mechanism

Another mechanism that can contribute to settlement of the jetty is not related to channel degradation. Seepage and piping through the jetty material can move the sandy riverbed material though the riprap that has been dumped on top to form the jetty. This movement of sandy bed material may cause settlement of the dumped rock. Localized settlement would channel the flow, producing conditions that would make the downstream slope of the structure more susceptible to undercutting.

D. Alternatives analysis

The alternative analysis includes an update of the alternatives presented in the draft 1988 Black & Veatch report and other alternatives not present in that report. The analysis includes the advantages, disadvantages and feasibility of each alternative. For feasible alternatives, opinions of probable present value cost were prepared.

The alternatives that were studied are:

Modified Plan A from the 1988 Black & Veatch draft report.

Modified Plan B from the 1988 Black & Veatch draft report.

Alternative B1: add lining material to Alternative B.

Alternative B2: stepped downstream slope.

Modified Plan C from the 1988 Black & Veatch draft report.

Modified Plan D from the 1988 Black & Veatch draft report.

Alternative D1: build the cofferdam upstream from existing jetty.

Other Options

The alternatives in the 1988 study where modified or updated based on the analysis presented in section C of this report. All of these alternatives allow the intake to remain in service throughout construction.

The hydraulic stability of the structure was checked for existing and future conditions. Future conditions take into account degradation of the river bed downstream from the jetty.

Based on the geologic mapping at the I-435 Kansas River Bridge, bedrock at the jetty location elevation is estimated to be at elevation 702 ft. Downstream from the structure current stream bed elevations have been measured around 720 ft with low spots at about 705 ft. The worst-case scenario by Simons, Li, and Associates (1985) indicates that by year 2015 the degradation would be 22 ft for the reach downstream from the jetty. Based on Simons, Li and Associates estimates (22 ft in 30 years) and assuming an average current bed elevation of 720 ft, bedrock will be reached in 24 years (year 2028). If a more conservative degradation rate of about 0.95 ft/yr in thalweg persists (3.8 ft in 4 years as indicated in section C.2 of this report for RM 14-1), bedrock will be reached in 20 years (year 2024). Because it is possible that the river bed downstream from the jetty may reach bedrock level within the economic life of the jetty, the stability analysis of future conditions should assume that the downstream bed elevation is at 702 ft and that the tailwater has changed accordingly.

Riprap size is given in terms of the D50 Gradation and should follow COE Hydraulic Design Criteria Sheet 712-1 guidelines. Based on these guidelines the D100 should be larger than 1.26 times the D50 and D15 should be larger than 0.5 times the D50; the thickness of the riprap layer should be 2 times D50.

The cost opinions are based on August 2004 material, equipment and labor costs. Tables with summarized cost calculations are included in Appendix I.

1. Alternative A: modified Black & Veatch 1988 Plan A

Alternative A consists of armoring the downstream slope of the jetty by dumping large rock down to existing riverbed and to periodically update, maintain and repair the jetty. Under existing conditions the 9:1 downstream slope remains unchanged. If river bed degradation persists downstream from the jetty riprap size and slope would have to be modified accordingly.

This alternative requires routine inspections and monitoring. A staff gage should be installed to monitor tailwater elevations. Tailwater elevations and the corresponding streamflow measurements would have to be checked periodically against a current rating curve to detect changes that may jeopardize the stability of the jetty.

a. Riprap design

The selection of riprap size was based on the hydraulic stability analysis of the slope of the structure. The same modeling approach described in section C was used for this purpose. The riprap size was calculated for two locations: (1) the weir section (just downstream from intake sill), and (2) the jetty. For the weir section, a no-breach situation was assumed. For the jetty section the size was calculated based on the low spot at elevation 734. This condition also approximates a 2 to 4-ft deep gap in the debris that could accumulate on other sections of the jetty.

The hydraulic model indicates that under existing conditions rock averaging 5.5 ton (D50 = 5 ft) is needed for the weir section and rock averaging 4.1 ton (D50 = 4.6 ft) for the rest of the jetty. Parts of the jetty may already contain some rocks of about the required size. However, to ensure adequate armor and grading it is recommended the entire top and downstream face of the jetty be reinforced.

As the river bed degrades downstream from the jetty, larger riprap size and or a milder slope would be required. Under ultimate conditions, a 16:1 slope would be required and rock averaging would be needed for the weir section and rock averaging 4.4 ton (D50 = 4.7 ft) for the rest of the jetty (see Alternative B for more detail). Ultimate conditions assume that degradation has reached riverbed at elevation 702 and that the tailwater downstream from the jetty has decreased accordingly.

b. Project cost and present value

The opinion of probable project cost of this alternative is \$ 7.3 million (Table 2). Maintenance costs would depend on the rate of degradation of the Kansas River bed and future changes in tailwater elevations. It is estimated that initial construction will last 16 weeks.

If tailwater elevations do not change over time, maintenance cost would be minimized. Assuming a 1% riprap replacement per year, the opinion of maintenance present value would be \$1.1 million. Assuming a 50-year period at 4 percent interest, the opinion of total present value of this alternative would be **\$ 8.4 million** (Table 2).

If ultimate conditions are reached in 20 years (assuming recent measured trends persist), the opinion of maintenance present value would be \$13.1 million. In this case, the opinion of total present value of this alternative would be **\$ 20.4 million**.

Table 2. Alternative A, estimated quantities and project cost opinion.

Item			Quantity	Unit	Unit cost (\$)	Cost (M\$)
Project Cost						
Riprap Armor layer			76,800	ton*	60	4,608,000
Mobilization/Demobilization:						461,000
					Total Construction Cost:	5,069,000
					Contingencies (25%):	1,267,000
					Total:	6,336,000
					Legal, administrative and engineering expenses (15%):	950,000
					Total Project Cost:	**7,286,000**
Maintenance (assuming no degradation)						
Inspection	\$5,000	/ year over 50 years at 4% interest				107,000
Repair	\$46,000	/ year over 50 years at 4% interest				988,000
					Total Present Value Maintenance Cost:	**1,095,000**
					Total cost (assuming no degradation):	**\$ 8,381,000**

*Assumes a riprap density of 110 lb/ft^3 (takes into account voids between rocks)

2. Alternative B: Black & Veatch 1988 Plan B

Plan B consists of armoring the jetty to withstand ultimate future conditions as presented in Alternative A. The downstream slope was set at 16:1 as proposed in the 1988 report. This alternative assumes future riverbed degradation conditions.

The jetty crest and weir profile would be modified to provide an adequate head at the intake during low flows while reducing the headwater elevation for large flows. The proposed profile is shown in Figure 10.

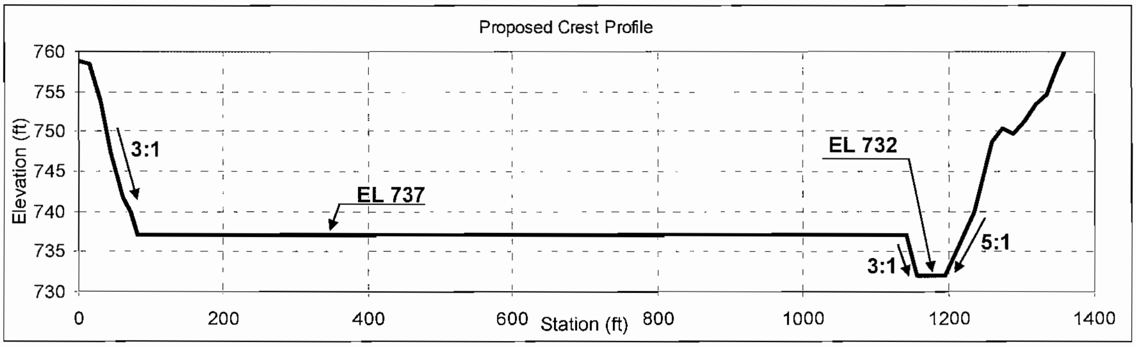


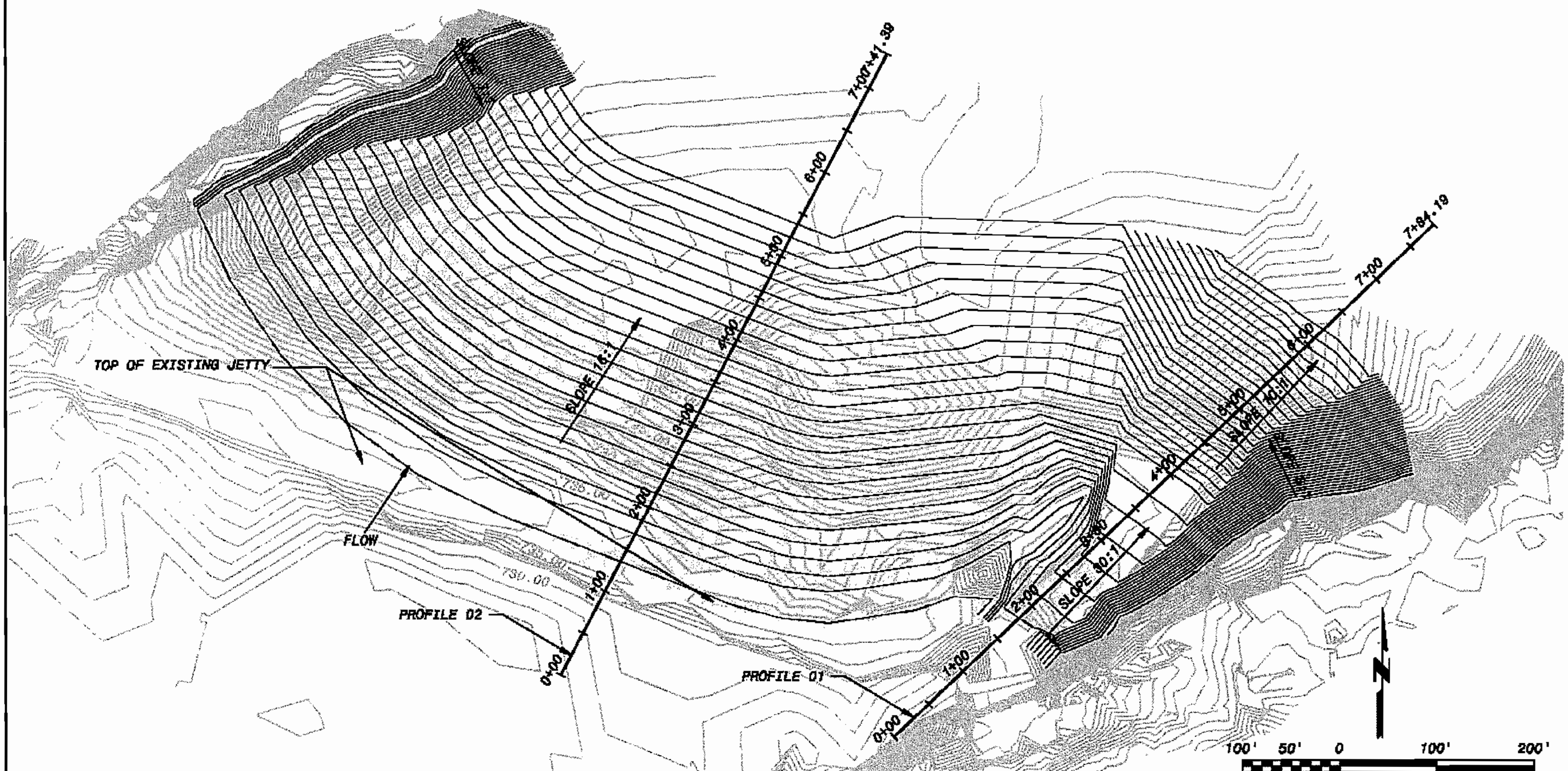
Figure 10. Proposed jetty crest and weir profile.

In order to protect the river banks the jetty shape would be modified. The existing layout diverts the flow towards the north bank causing instabilities. To reduce this effect it is proposed that the north end of the jetty should be curved as shown on Figure 11. This modification would direct flow more towards the center of the channel instead of directing it towards the banks. It is also recommended to armor the river banks downstream from the structure using the same material used in the downstream slope of the jetty.

As indicated in the 1988 report, this alternative does not prevent the washing away of the sands from under the riprap, thus settlement of the structure will continue. Maintenance will be required periodically to repair settlement.

As the stream bed elevation decreases, the toe of the structure would be exposed producing additional instability that may develop into a new breach. To prevent future toe exposure and potential failure, the slope of the structure would have to be extended all the way down to bedrock.

Figure 11 shows a plan view for this alternative. The shaded region shows the area that would have to be filled before the armoring is laid. Figure 12 and Figure 13 show profiles of the proposed 16:1 layout.

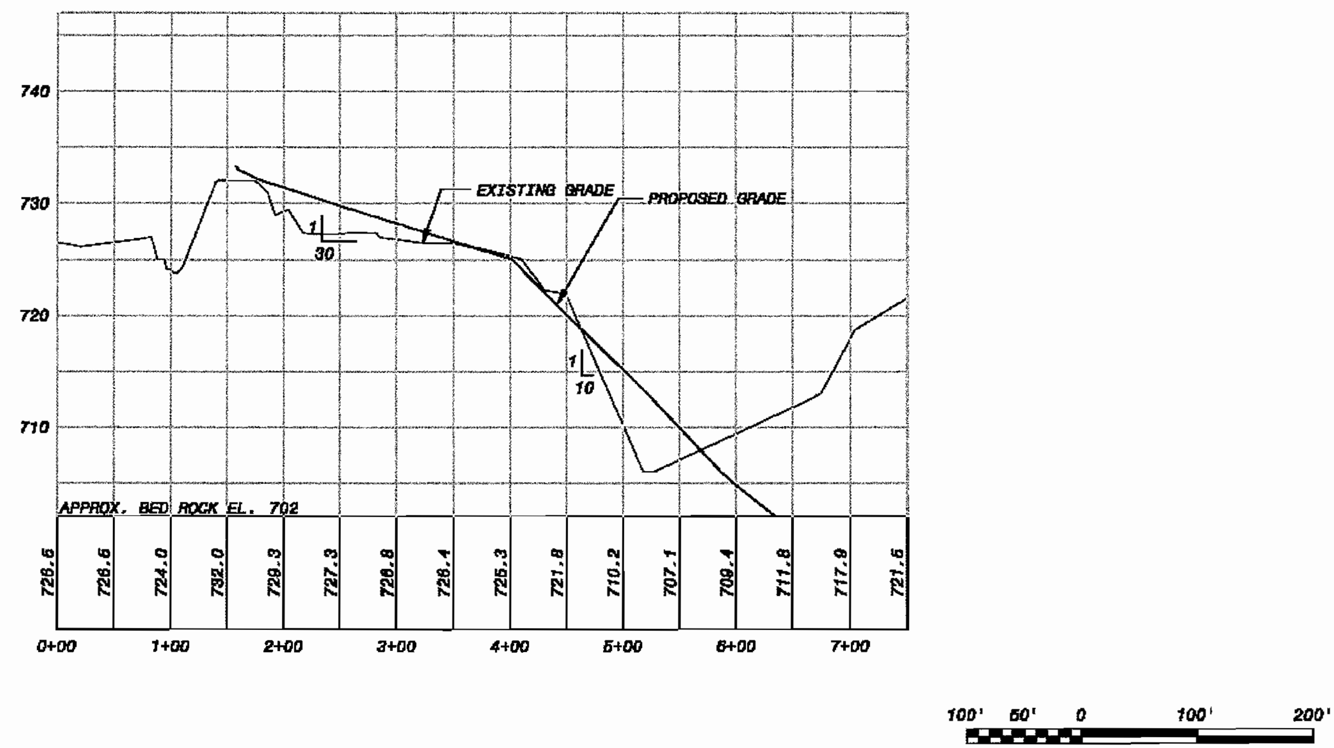


AREA TO BE FILLED TO
OBTAIN 16:1 SLOPE

ALTERNATIVE B PLAN

BLACK & VEATCH
building a world of difference

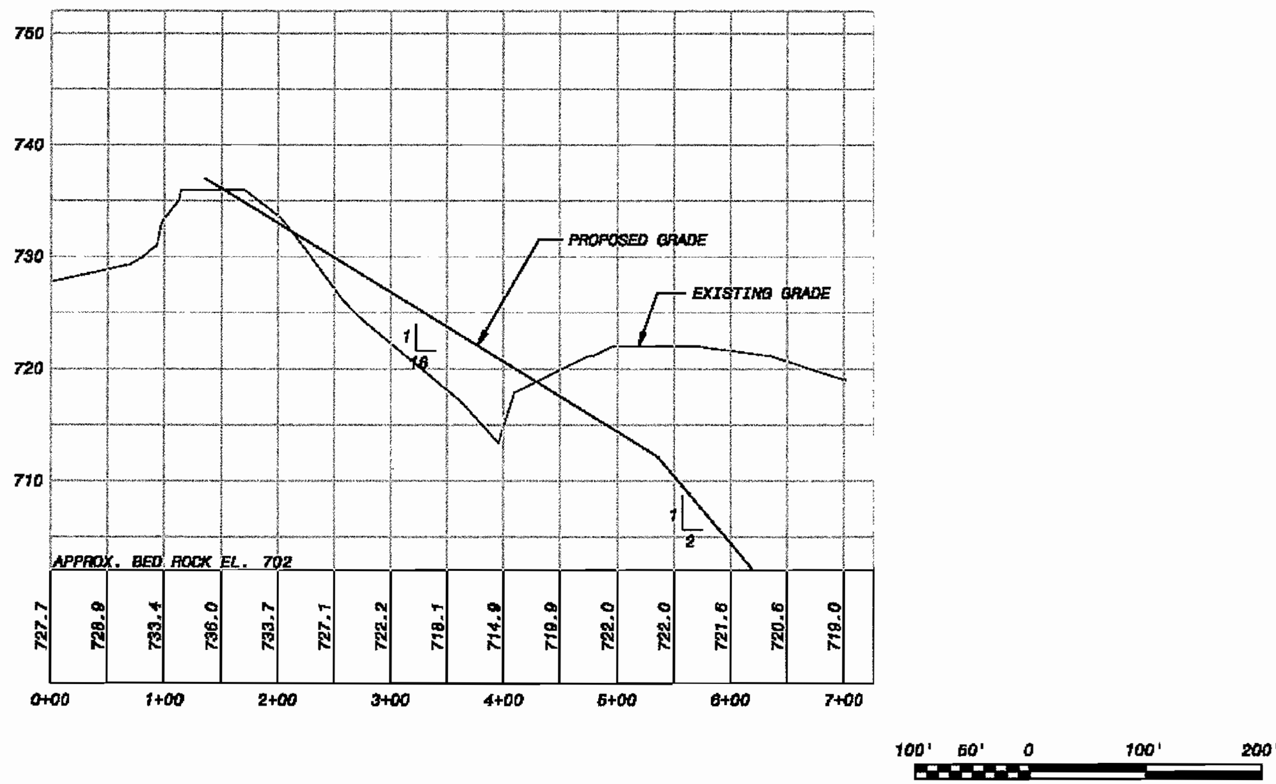
FIGURE 11



BLACK & VEATCH

ALTERNATIVE B PROFILE 01

FIGURE 12



BLACK & VEATCH

ALTERNATIVE B PROFILE 02

FIGURE 13

a. Riprap design

The riprap size was chosen based on the hydraulic stability analysis of the slope of the structure. The same modeling approach described in section C.1.a was used for this purpose. The riprap size was obtained for two locations: (1) the weir section (just downstream from intake sill), and (2) the jetty. For the weir section, a no-breach situation was assumed. For the jetty section, the size was calculated based on the low spot at elevation 734. This condition also approximates a 2 to 4-ft deep gap in the debris that could accumulate on other sections of the jetty. The extent of the riprap slope was determined to prevent the hydraulic jump from occurring at the toe of the structure, including a 10% safety margin. The hydraulic model indicates that the existing 30:1 slope of the upper part weir section, that extends about 220 ft from the weir crest, can remain as is. The toe of this section would have to be modified so that its slope is 10:1 instead of about 5:1. The armor layer for the rest of the jetty should extend 400 ft from the crest at a 16:1 slope. From there on, the toe of the structure should be extended down to bedrock. Due to the large size of rock required, extending the toe down to bedrock can be accomplished by adding one more layer of rock at the toe.

The hydraulic model indicates that rock averaging 7.7 ton (D50 = 5.6 ft) would be needed for the weir section and rock averaging 4.4 ton (D50 = 4.7 ft) for the rest of the jetty.

b. Project cost

The estimated project cost for this alternative is **$ 29.1 million**.

3. Alternative B1: add lining material to Alternative B

Use of different materials including new technologies such as A-jacks, articulated concrete block, etc. were investigated as possible lining materials that would replace the large rock required for Alternative B. The idea is to use smaller rock to define the 16:1 slope of the structure and then armor the smaller rock with one of the materials considered. The lining materials considered were gabion mattresses, articulated concrete block, grouted riprap and A-Jacks.

Articulated concrete block and grouted riprap were excluded as feasible lining materials because: (1) potential for settlement of the existing jetty and (2) installation of these materials requires dry conditions. Settlement may compromise the stability of these lining materials. If settlement occurs, cracks would form in the grouted riprap and the articulated concrete block may be dislocated, thus their effectiveness against erosion would be lost. Dry conditions are difficult to obtain without building a structure, such as a cofferdam, that would prevent water from overtopping and flowing through and under the jetty. Building a cofferdam is a solution in itself; it would be counterintuitive to build a temporary structure that is more stable than the permanent structure.

Gabion mattresses can be installed in wet conditions; however, "if wet conditions exist for long periods of time in the area surrounding the site, the delivery of rock materials may be impossible or extremely problematic." (Freeman and Fischenich, 2000). Maccafferi's Bank Protection reference manual reports a maximum allowable tractive force (sheer stress) for gabion products of 7 lb/ft^2; estimates indicate that under existing conditions sheer stresses exceed 10 lb/ft^2 at the jetty downstream face. Stability and constructability issues make this option unfeasible.

The hydraulic stability analysis was based on the velocity and sheer stress estimates obtained for Alternative B. Extrapolation from laboratory measurements indicates that A-jacks (AJ-96) resist velocities up to 44 ft/s with sheer stress up to 152 (lb/ft^2) (Scour Design Manual, Armortec, posted on the internet). Based on these numbers and model results for ultimate conditions the sheer stress minimum safety factor would be 2.3 and the velocity minimum safety factor would be 1.8.

After careful consideration of these materials it was determined that except for the use of A-jacks, the other materials would be inappropriate for lining the jetty.

As with Alternative B, this alternative does not prevent the washing away of the sands from under the riprap, thus settlement of the structure may continue. Maintenance will be required periodically to repair settlement. To prevent future toe exposure and potential failure, the slope of the structure should be extended all the way down to bedrock.

a. Project cost

The estimated project cost for this alternative is **\$ 59.0 million**.

4. Alternative B2: stepped downstream slope

Plan B2 is a modification to Alternative B. The 16:1 slope would be replaced by a stepped downstream slope (Figure 14). The idea is that by stepping the 16:1 slope, the volume of material required to build the stepped slope would be less than that for Alternative B. Larger step sizes reduce the needed volume of the material. However, the step size is limited by the hydraulic stability of the slope. A large step would result in a longer slope which allows the flow to accelerate more, resulting in larger flow velocities, thus larger rocks would be required to stabilize the structure.

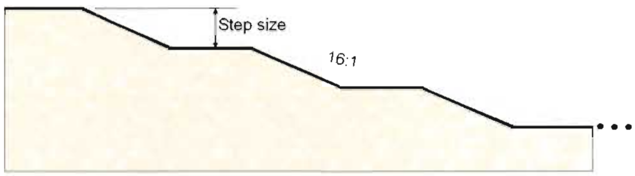


Figure 14. Stepped slope schematic

a. Step design

The selection of the step size was performed by limiting the required rock size (D100) to 7 ft (16 ton, possibly the largest rock size available and manageable). The riprap size was done based on the hydraulic stability analysis of the sloped sections of the structure using a 9:1 slope. The result from the hydraulic analysis indicates that the maximum step size would be 2 ft.

b. Project cost

The estimated project cost for this alternative is **$ 28.7 million**.

5. Black & Veatch 1988 Plan C

Plan C from the 1988 Black and Veatch report proposes the construction of a sheet pile wall driven to bedrock at the upstream face of the jetty, in addition to changes proposed for Plan B.

The 1988 report stated that the sheet pile wall lessens the flow of water into the sands that underlie the riprap jetty, thereby reducing the possibility of a washout. However, further analysis of the situation indicates that the structure will be overtopped under high flow conditions. Flow over the structure will infiltrate and the possibility of a washout cannot be discarded. The 1988 report also indicates that this alternative is about 2.5 times more costly than Plan B.

Further consideration of this alternative is unnecessary because it already proven to be costly while providing marginal benefits.

6. Alternative D:
Black & Veatch 1988 Plan D

Plan D from the 1988 Black and Veatch report is by far the most permanent solution. This plan proposes the construction of a sheet pile cellular cofferdam downstream from the structure (Figure 15 and Figure 16). The proposed cofferdam would look similar to the one presented in Figure 15; which is a photograph of the cofferdam built in the Kansas River in Topeka, Kansas. The proposed cofferdam would substitute for the existing jetty. The existing jetty would not need to be removed.

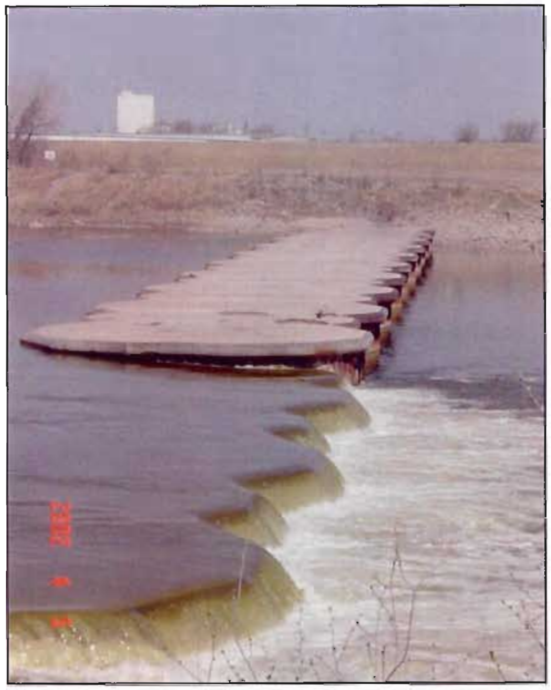
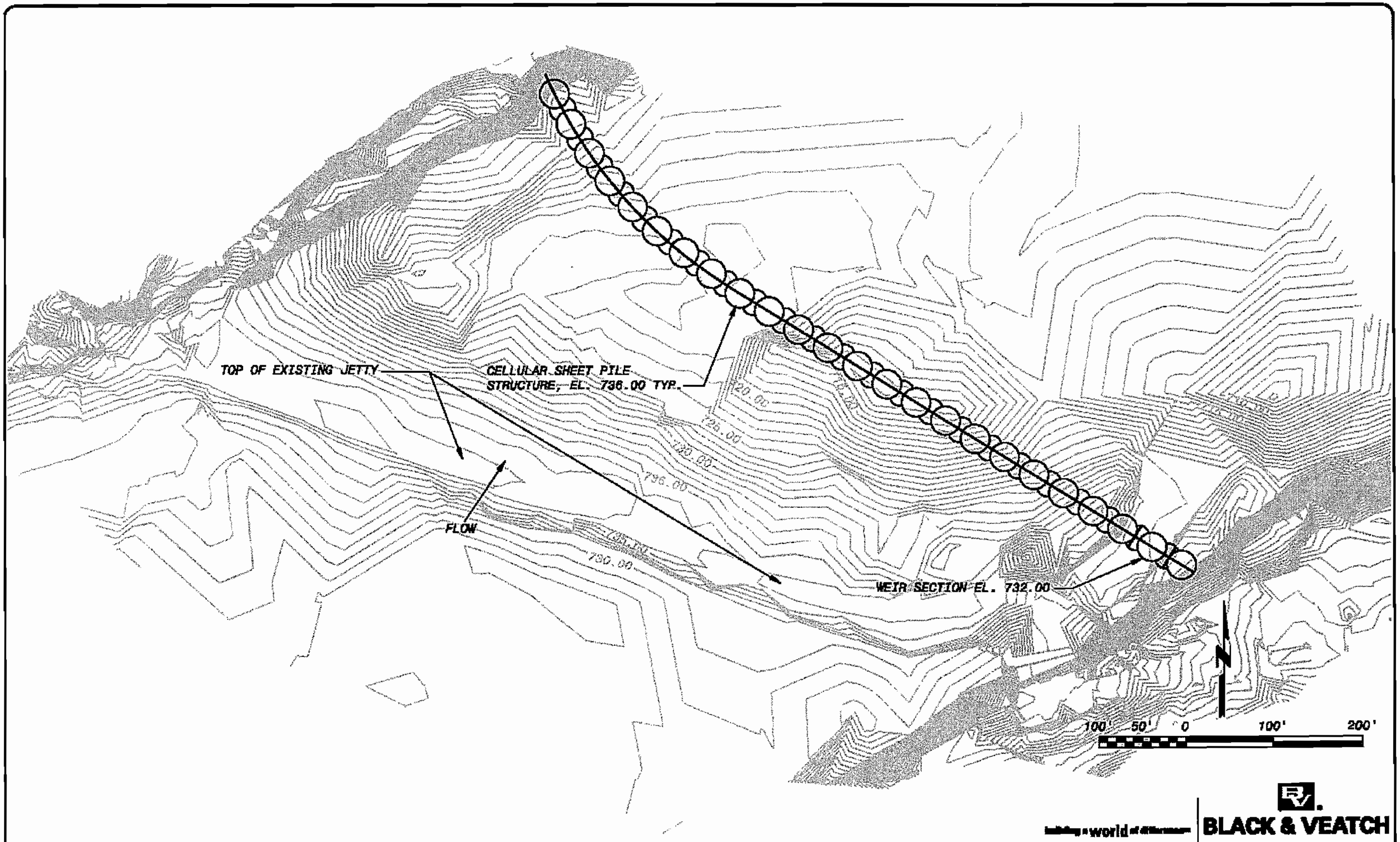


Figure 15. Cofferdam in the Kansas River in Topeka, Kansas.

a. Coffer dam design

Preliminary calculations indicate that the coffer dam cell size should be of 34 ft in diameter. A total of 24 cells and 23 interconnecting arcs would be needed to build the cofferdam across the river (Figure 16). The coffer dam would have a crest elevation of 736.0 across the river. It would include a low water weir near the south bank with a crest elevation of 732.0. The weir would be formed by lowering one of the cells of the coffer dam. The cofferdam would be curved to form an angle greater that 90° with respect to the riverbanks to divert flow away from the riverbanks and toward the center of the river. This curved shape would reduce erosion potential to the riverbanks.

Hydraulic simulation in HEC-RAS indicates that lowering one of the cells down to elevation 732.0 would maintain the headwater elevation above the minimum level required for operation of the intake (Elevation 734.0), for stream flows above 400 cfs. If additional head is required the low water weir length can be reduced by adding concrete walls on the sides of the sill.



ALTERNATIVE D

FIGURE 16

b. Project cost

The opinion of probable project cost of this alternative is **$12.3 million** (Table 3). It is estimated that the construction will last 1.5 years. Because maintenance requirements are minimal, the probable total present value is essentially the same as the probable project cost.

Table 3. Cost opinion for Alternative D.

Item	Quantity	Unit	Unit cost ($)	Cost ($)
Project Cost				
Excavation				
	50,700	yd^3	10	507,000
Place 8' depth fill in excavation				
	16,900	yd^3	30	507,000
Place sheet piles				
	102,900	yd^3	40	4,116,000
Place remaining fill in cells and arcs				
	25,900	yd^3	30	777,000
Grout front face of sheet piles at top of rock				
	1,000	yd^3	250	250,000
Construct concrete cap on cells				
	1,100	yd^3	500	550,000
Place large riprap at downstream side of cells				
	9,400	yd^3	75	705,000
Remove sheetpile structure:				
	1		350,000	350,000
Mobilization/Demobilization:				
				776,000
			Total Construction Cost:	8,538,000
			Contingencies (25%):	2,135,000
			Total:	10,673,000
			Legal, administrative and engineering expenses (15%):	1,601,000
			Total Project Cost:	**$ 12,274,000**

7. Alternative D1: build cofferdam upstream from existing jetty

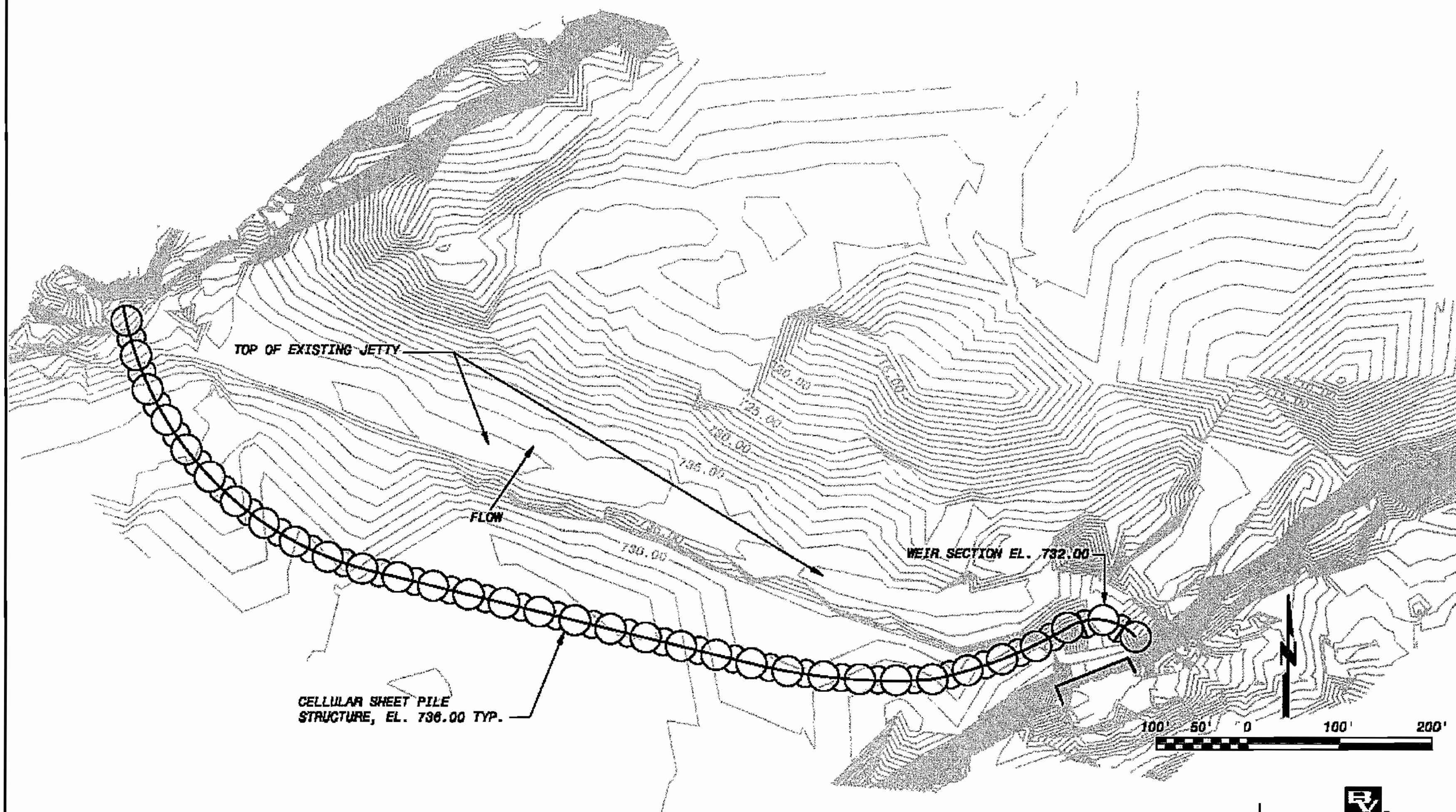
Alternative D1 consists of building the cofferdam just upstream of the structure instead of downstream (Figure 17). The size and specifications are essentially the same as with Alternative D. A total of 33 cells will be required.

a. Project cost

The opinion of probable present value of this alternative is **$ 13.3 million** (Table 4). It is estimated that the construction will last 1.5 years. Because maintenance requirements are minimal, the probable total present value is essentially the same as the probable project cost.

Table 4. Cost opinion for Alternative D1.

Item	Quantity	Unit	Unit cost ($)	Cost ($)
Project Cost				
Excavation				
(for cells over jetty only)	5,800	yd^3	10	58,000
Place 8' depth fill in excavation				
(for cells over jetty only)	1,600	yd^3	30	48,000
Place remaining fill in cells and arcs				
(for cells over jetty only)	4,300	yd^3	30	129,000
Place sheet piles				
	141,800	yd3	40	5,672,000
Compact sand in cells and arcs using vibroflotation				
	1	ls	750,000	750,000
Verification borings for vibrocompaction				
	2,100	ft	25	53,000
Add fills to cells and arcs				
	11,700	yd^3	25	293,000
Grout front face of sheet piles at top of rock				
	1,300	yd^3	250	325,000
Install concrete cap on cells				
	1,500	yd^3	500	750,000
Remove sheetpile structure:				
	1		350,000	350,000
Mobilization/Demobilization:				
				843,000
			Total Construction Cost:	9,271,000
			Contingencies (25%):	2,318,000
			Total:	11,589,000
			Legal, administrative and engineering expenses (15%):	1,738,000
			Total Project Cost:	**$ 13,327,000**



building a world of difference™ BLACK & VEATCH

ALTERNATIVE D1

FIGURE 17

8. Other options considered

a. Stepped dams downstream from the jetty

This option considers the construction of a series of stepped dams downstream from the existing jetty. The first dam would raise the tailwater conditions for the existing jetty. Raising the tailwater would make conditions better for the existing jetty but it exports the instabilities to the downstream face of the second rock dam, thus requiring more dams. The number of dams would be limited by tailwater conditions in the river, at some point adding a dam downstream does not improve the conditions of the previously added dam.

Under existing conditions the maximum step that would have an effect on improving the jetty stability is 2.9 ft. A larger step would not impact the stability of the existing jetty because the flow conditions on its downstream face would remain unchanged. A 2.9 ft step would make conditions marginally better, thus a shorter step is desirable. If a 2.5 ft step is used two more dams would be required.

The existing jetty has been built gradually since 1967. Because the rock has been added on top of previously dumped rock, settlement of the structure has occurred slowly over time. This condition has allowed the jetty to be relatively stable in terms of settlement. If a new jetty is build on top of the river bed, settlement would be a major factor of concern. Maintenance due to settlement would be much higher than that for the existing jetty and would offset the costs making it a more expensive option. To prevent settlement, rock would have to be placed down to bedrock as shown in Figure 18. The shaded region represents the area that would have to be filled with rock. The 720' line represents the approximate average elevation of the riverbed. In this case the project costs were estimated to be in excess of $ 45 million.

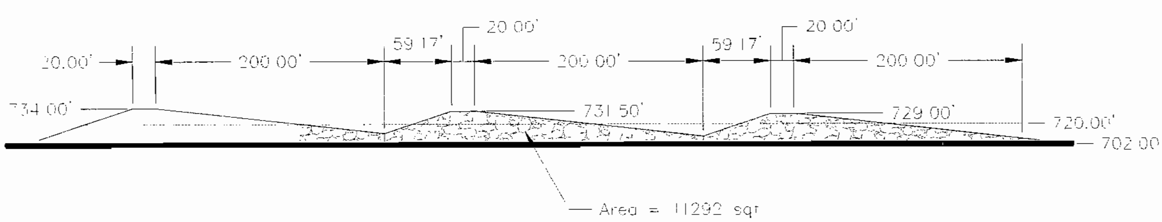


Figure 18. Stepped dams schematic

b. Milder slope

This option explores the possibility of using a milder slope that would allow the use of smaller rock (W50 = 5 ton) or concrete rubble (W50 = 1 ton). The hydraulic model indicates that a 19:1 slope would be required for the 5-ton rock and a 72:1 slope for the 1-ton rubble. The slope would extend for about 500 ft and 1,570 ft respectively. The amount of material required to obtain such slopes makes these alternatives more costly than Alternative B with estimated project costs of about $ 34 million and $31 million respectively.

c. Armor top of the jetty

This option proposes to armor the top of the jetty with a smoother surface to avoid debris accumulation. The problems associated with this option are similar to those associated with Alternative B1; settlement of the jetty may compromise the armor integrity. Even if a slicker surface is used, such as a concrete cap, there is no guarantee that debris will not build up on top of the jetty. Armoring the downstream face of the jetty would still be required, thus the smoother armor at the top of the jetty would represent additional costs.

d. Bentonite

The use of bentonite was suggested as a way to prevent seepage through the jetty. However, this option does not address the hydraulic stability of the downstream face of the jetty.

e. Replace intake with collector wells.

The Kansas River intake has a supply capacity of 65 mgd. A collector well system would have to be able to meet this capacity.

Development of a horizontal collector well supply along the Kansas River was studied in 1999 and 2003. These investigations indicated that the existing wellfield capacity could be expanded to about 15 mgd with the addition of a collector well for approximately $4.4 million in 1999 dollars (Black and Veatch, 1999).

Black and Veatch (2003) explored the Kansas River alluvium from the I-435 bridge west to the Johnson County line. It was determined that collector wells could be installed to develop a 50 mgd supply. The total present value of the development of these collector wells was estimated to be $50.2 million. The 2003 study required the construction of a new treatment plant. In this case, however, because the Kansas River intake would not be operational, the Hansen water treatment plant can be used instead of building a new treatment plant. The Sunflower main would have to be repaired and a new transmission pipeline would have to be built from the collector wells to the Hansen treatment plant with a conceptual project cost of about $25 million. The conceptual total present value of such a project would be about $75 million. The cost of this option is prohibitive compared to the cost of other alternatives presented in this report.

Infiltration galleries were also evaluated in the 1999 Black and Veatch report and found to be cost prohibitive.

f. Lowering the intake sill and adding low-head pumps.

Lowering the intake sill would allow the top of the jetty to be lowered, thus reducing the likelihood of breaching. If the top of the jetty (including the weir section) is lowered, the existing intake would not be operational, in which case a new intake would have to be constructed at a lower elevation. The new intake would pump into the existing intake cells.

This option does not remove the jetty, it just reduces its height; therefore the smaller jetty would still have to be maintained to prevent a breach. The potential of a breach would be low but the risk remains and would increase as the river degrades.

The conceptual construction cost for a new intake would be about $8.5 million. Taking into account maintenance of the intake the conceptual total present value would be about $14 million. Additional costs would be incurred to lower the jetty and to remove the sediment that has accumulated upstream from the jetty.

The intake would be similar to the intake that the District has on the Missouri River. Because the Kansas River freezes over and the pumps would have to be available at all times the use of submersible pumps out in the river was not considered feasible.

Moreover, it is not clear how to lower the jetty without producing major environmental impacts. Sediment upstream from the jetty is currently at elevation 728 and the jetty acts as a grade control. If the top of the jetty (including the weir section) is lowered, the elevation of the grade control will change. This could impact stream bank stability, location of sand bars, and sediment patterns of the river bed for significant distance upstream.

Based on all these factors including questionable reliability of the jetty and unacceptable environmental impacts this option is not viable.

9. Alternatives comparisons

a. Alternatives advantages and disadvantages

Alternative A

Alternative A builds from the existing structure. This alternative does not provide protection at the toe of the jetty structure and does not prevent the washing away of the sands from under the riprap. These two conditions may trigger another failure. Additional costs may be incurred if other failures occur. Even though it is believed that settlement has slowed down significantly, if settlement occurs, more rocks may have to be added on top of the dam. Maintenance will be required periodically to update the structure for changing tailwater conditions, to repair potential settlement and to replace weathered rocks.

Alternative B

Alternative B also builds from the existing structure. However, it minimizes the risk of another failure under existing and future conditions by armoring the jetty to withstand ultimate future conditions. However, the large volume of material required to fill and armor the 16:1 slope, and the volume of material that has to be excavated, make this alternative cost prohibitive with an estimated project cost of about $29.1 Million. In addition to the initial construction costs, this alternative will require periodic repair and maintenance to overcome settlement and to replace weathered rocks. The high cost of this alternative discards it as a viable option.

Alternative B1

The advantage of this alternative over Alternative B is that transportation and placement of the A-Jacks is not as challenging as it may prove to be with the large rocks required for B. A-Jacks may also be more stable than riprap.

This alternative is even more expensive than Alternative B. Therefore, this alternative is not economically feasible.

Alternative B2

Alternative B2 has essentially the same advantages and disadvantages as Alternative B. The hydraulic conditions pose limitations on the step size. These limitations translate into insignificant reductions of the volume of rock required to build this structure. Therefore, the economic benefit of this alternative compared to alternative B is marginal and is also not economically feasible.

This alternative also presents constructability issues. Building 2 ft steps using 7 ft rocks could be difficult.

Alternative D

The main advantage of this alternative is that it would be a permanent structure. Maintenance of the structure would be relatively low. This alternative would handle ongoing scouring and head cutting.

Another advantage of this alternative is that by increasing the dam crest elevation and changing the weir geometry it may be possible to increase the submergence of the intake for low flow conditions. The coffer dam and the weir would have to be designed following FEMA requirements (see section E3 of this report).

Local scour downstream from the dam would be expected due to turbulence at the toe. Under existing downstream conditions the maximum scour hole expected would lower the streambed down to elevation 703 ft (just above bedrock). Future conditions would lower the streambed down to bedrock (elevation 702 ft). Large rock may be required to protect the bedrock at the toe of the cofferdam. Bedrock properties and conditions would have to be determined during the design of the coffer dam. Bedrock properties and conditions would indicate whether measures to protect the bedrock at the toe of the coffer dam would be needed. Even under ultimate conditions, tailwater would be high enough to contain the turbulence close to the coffer dam. Estimates indicate that under ultimate conditions the area disturbed by turbulence would be contained within 230 ft from the downstream face of the coffer dam.

This alternative may require periodic sluicing or dredging upstream as more sediment deposition is expected.

One main construction issue for this option may include excavation of large rock that may had been previously washed out from the existing structure and buried under the sand in the riverbed. Test holes should be drilled to obtain a representative sample of riverbed material. Fifty test holes spaced between 25 and 50 ft would provide an adequate sample. Results from the riverbed samples would provide an indication of the magnitude and requirements of excavation.

Alternative D1

It has essentially the same advantages and disadvantages as Alternative D. The difference is that there is a reduction in the likelihood of finding large rocks upstream from the existing jetty. This may eliminate the need to excavate and fill before placing the sheet piles. As with Alternative D, test holes should be dig to obtain a representative sample of riverbed material. Results from the riverbed samples would provide an indication of the magnitude and requirements of excavation.

This alternative requires six more cells and additional considerations have to be made to stabilize the existing diversion wall and intake structure sill.

b. Overall comparison

Alternative A is the least expensive option assuming that degradation of the riverbed downstream from the jetty will be minimal and that tailwater conditions will not deviate from current levels. This alternative may not prevent another failure and will require periodic maintenance to repair settlement and to replace weathered rocks. Because there is no base level control to stop degradation it is likely that degradation would continue. If downstream conditions change, larger rocks or a flatter downstream slope may be required. A flatter downstream slope increases the volume of material required thus costs will increase correspondingly. Because it is likely that the riverbed will continue degrading the present value of Alternative A would be much higher than that of Alternatives D and D1.

Alternatives B, B1 and B2 explore the requirements for future conditions. Alternatives B, B1 and B2 are costly and only provide a partial solution. To avoid another failure, the entire downstream face of the jetty must be armored down to bedrock. The hydraulic analysis indicates that rock of 7.7 ton (D50 = 5.6 ft) or larger is required. The layer thickness required is two times the D50. Thus, the volume of rock required to take the 16:1 slope down to bedrock is massive making this option extremely expensive. Alternative B2 proposes stepping the downstream slope to reduce the amount of material required; however, hydraulic limitations translate into insignificant reductions of the volume of rock required to build this structure. Substituting the large rocks with A-Jacks may provide a stability advantage but with a substantial increase in costs. These three alternatives do not provide a solution to the settlement of the structure. As the structure settles more armoring material has to be added over time, thus costs increase even more.

Alternative C was discarded because this alternative adds additional costs to Alternative B while providing marginal benefits.

Alternatives D and Dl provide the most permanent solution. The excavation of large rock that may have been previously washed out from the existing structure may pose constructability issues that may make Alternative Dl more attractive.

E. Permitting

Since all of the proposed alternatives include activity directly in the Kansas River there are several local, state and federal agencies that will require some type of permit for the project. Potential agencies that will require permits and coordination include Bonner Springs/Wyandotte County, Kansas Department of Agriculture - Division of Water Resources (KDWR), Kansas Department of Health & Environment (KDHE), Kansas Department of Wildlife & Parks (KDWP), U.S. Army Corps of Engineers (USACE), U.S. Environmental Protection Agency (EPA), and the Federal Emergency Management Agency (FEMA) as part of its National Flood Insurance Program (NFIP). In addition to these governmental agencies, various other stakeholder groups, including Friends of the Kaw, will most likely have comments regarding the proposed facilities.

Technically, the permit applications are to be submitted after final plans have been completed. Since this may not fit with fast-track design and construction schedules, the permit applications can be submitted prior to final plan completion as long as the location and overall dimensions of the facilities do not change.

1. Corps of engineers permits

The Kansas River is a navigable river and as such falls under the USACE jurisdiction under Section 10 of the Rivers and Harbors Act and Section 404 of the Clean Water Act. The USACE jurisdiction is from the ordinary highwater mark and below. This has been defined as elevation 742.0 in this section of the Kansas River. Based on this information, a Section 10 and Section 404 permit will be required from the USACE for any project that is undertaken in the river. As part of the permit process clearances will be obtained from other federal agencies including U.S. Fish & Wildlife (USFWS) and EPA.

Pre-project meetings were held on April 14, 2004 and March 31, 2005 that included members of the USACE Hydrology and Hydraulic section and from the Regulatory Branch, Black & Veatch staff and WaterOne staff. In those meetings, the USACE expressed no objections to developing a permanent solution to WaterOne's continued problems with the jetty. They expressed concerns regarding a rock-fill structure and indicated that any USACE project would have been constructed using alternate materials.

Due to the impact on the river, an individual permit will most likely be required for the project. After the application is submitted, the Corps of Engineers will develop a public notice requesting comments on the project for a period of up to 30 days from federal, state, local agencies and the public. The Corps reviews the comments and may request additional information from the submitter. The corps then makes a decision whether the project should be permitted. Typically the process can be completed within a three month timeframe.

2. Endangered species act

There are two species listed on the federal Threatened and Endangered Species list that are applicable to the Kansas River. The first is the Piping Plover bird species that utilizes sand bars on the river for nesting. The construction of the proposed alternatives will most likely not impact this since any proposed improvements will not increase the elevation of the jetty.

The second species is the Pallid Sturgeon. The USFWS will provide comment back to the USACE regarding the impact of the proposed jetty construction on the Pallid Sturgeon. It will be important to involve the USFWS in the design process for the project. The USFWS office for Kansas is located in Manhattan, Kansas. The Missouri office located in Columbia, Missouri will also be a useful asset as they are regarded as the Pallid Sturgeon experts in this region.

3. State of Kansas permits

The State of Kansas will review any proposed project based on placing a structure on a stream with a drainage area greater than 240 acres, the impact of the construction activities to water quality in the river, the impact to species listed on the State's Threatened and Endangered list, and the impacts to recreation on the river.

The KDWR will require that a Stream Obstruction Permit be obtained for the construction of a structure in a stream with a drainage area greater than 240 acres. Due to the size of the structure, it will most likely be permitted as a dam. The Stream Obstruction Permit application is sent to 12 state agencies for review and comment. The formal review period is for 30 days, however the overall process often takes six months or more to complete depending on staff workload. One of the most influential agencies that review the applications is the KDWP.

In addition to KDWR, an NPDES permit for construction activities disturbing greater than one-acre must be acquired from KDHE. KDHE administers the NPDES program in the State of Kansas for EPA. A Notice of Intent (NOI) must be completed and submitted with a Storm Water Pollution Prevention Plan (SWPPP). The SWPPP outlines the procedures and best management practices that will be followed to minimize the erosion and other pollution associated with construction activities to the river. The NOI is required to be submitted 15 days prior to the start of construction. It is advised that it be submitted before this time to incorporate any agency comments.

It is unlikely that this project will required a complete Environmental Impact Study (EIS); however, it is always a possibility if enough evidence is presented to the permitting authorities that the proposed project can dramatically impact the river. Typically a less rigorous environmental assessment is submitted with the permit applications and is sufficient to obtain the permit.

4. Recreation

Both the USACE and KDWP have previously expressed the need to assess the impact to recreation for any proposed construction on the Kansas River. The river is used for canoeing and kayaking. In addition, the river, particularly along the existing jetty, is used by fisherman. Proposed improvements to the existing jetty should consider options to facilitate both of these uses of the river, while maintaining public safety. Construction of a canoe portage or a boat chute may be required.

5. Federal Emergency Management Agency requirements

This section of the Kansas River has an effective Flood Insurance Study in-place and as a result has a FEMA/NFIP defined floodway. By definition of the floodway, any construction that occurs within the floodway zone must be evaluated using the effective hydraulic model for the river and shown to produce no increase in water surface elevation for the 100-year flow. Alternatives D and D1 would require an Engineering "No-Rise" Certification. The "No-Rise" Certification must be issued prior to the construction of the improvement by a registered professional engineer in the State of Kansas and presented to the governmental agency responsible for administering the flood insurance program for the stream. This is typically the City or County where the stream is located.

The "No-Rise" Certification would have to be addressed during the design phase of the project. Alternatives D and D1 propose a crest elevation that is on average lower than that of the existing jetty, thus water surface elevation should not increase during the 100-year flow. The cofferdam crest elevation would be designed to prevent any rise in water surface elevation for the 100-year flow.

6. Environmental advantages of the jetty

In addition to providing the necessary water surface elevations at the Kansas River intake, the jetty provides a necessary grade control for the Kansas River. Without the jetty, the river degradation that exists downstream of the jetty, would continue upstream and result in loss of threatened and endangered species habitat, degrade or destroy upstream infrastructure, and result in the loss of property adjacent to the river from failing river banks. This degradation would negatively impact well fields for municipalities upstream of the jetty and threaten their water supplies.

The degradation downstream of the jetty is potentially related to the Missouri River degradation in this region. The USACE is attempting to get a study of the Missouri River degradation and its effects on its tributaries, including the Kansas River, approved by Congress. They have partial-funding available to initiate the project, but need additional commitment of funds from Congress to complete the study. If the study is funded, it will likely take five or more years to identify solutions to the Kansas River degradation. As a grade control structure, the existing jetty has arrested the upward migration of the river degradation on the Kansas River. The Corps of Engineers' study could lead to potential federal funding for jetty rehabilitation.

The removal of the jetty, natural or otherwise, would result in river instability from the current jetty location potentially all the way to Lawrence, Kansas. This river instability would result in damage to real property, and adversely impact water supplies taken from the river. The instability and resulting degradation would also negatively impact the habitat along the river and potentially destroying habitat for several threatened and endangered species.

Since the stability of the jetty is regionally important, there may be opportunity to partner with upstream property owners and water suppliers to maintain the stability of the jetty. In addition, this regional importance may allow the project to fit under one or more of the USACE or other Federal Agencies programs for funding. It is not clear on the schedule for Federal Agencies funding, but sometimes with Congressional urging, the funding can be made available more quickly than normal. A meeting with the USACE was held March 31, 2005 to discuss their interest in the project and funding options. USACE staff indicated that it is unlikely that they would fund improvements specifically for the jetty.

F. Protection of the intake from debris

Debris accumulates at the intake at the existing concrete filled sheet pile diversion structure near the face of the intake. Removal of the sheet pile structure would mitigate the accumulation of debris.

Under Alternative D, the sheet pile structure can be removed. The cofferdam proposed in Alternative D could be designed to provide the appropriate headwater at the intake. Alternative D1, as proposed, requires the removal of the sheet pile structure.

G. Value Engineering Workshop

A value engineering workshop took place on November 19, 2004. The following people participated in the workshop:

Jim MacBroom	Milone & MacBroom
Robert Prager	Intuition Logic
Ken Stark	US Army Corps of Engineers
Paul Corkill	Water Dristrict No.1 of Johnson County, Kansas
Tom Schrempp	Water Dristrict No.1 of Johnson County, Kansas
Michael Horsley	Black and Veatch
Jeff Henson	Black and Veatch
Don Baker	Black and Veatch
Pablo Gonzalez	Black and Veatch

The purpose of the workshop was to discuss alternative approaches to resolve the problems associated with the jetty and the intake. Four broad categories of recommendations and potential solutions where identified and are discussed in this section of the report. The four categories are:

Systemic river problems/solutions
On-site alternative solutions
Design alternatives
Other recommendations

Statements from Jim MacBroom and Robert Prager can be found in their letters included in Appendix II.

1. Systemic river problems/solutions

The causes and extent of the Kansas River degradation are key variables controlling the stability of Water District No. 1's jetty. The potential causes and extent of the degradation were discussed as part of the value engineering workshop.

Based on the previous geotechnical and geologic studies in the river basin, the entire river valley was scoured to a depth that is greater than the riverbed elevation of today. This scouring most likely occurred during the glacial period of this region. As the glacial flows were reduced, the alluvial sediments of today's river valley were deposited. It is this alluvial material that the Kansas River is currently eroding.

The Missouri River in the Kansas City region is currently degrading and much attention has been given to this issue because of the threat to the many water supply intakes along the river. An issue that has not received nearly as much press is the effect of the Missouri River degradation on its tributaries, such as the Kansas River. While a rigorous study has not been completed on the subject, the information that has been reviewed as part of this study indicates that the Kansas River degradation is most likely associated with the Missouri River degradation. Since there is no geologic control evident between the District's jetty and the Missouri River, there is no reason to believe that the Kansas River degradation downstream of the jetty will cease until the Missouri River degradation ceases.

The VE participants agreed that the dredging is unlikely to be the cause of the Kansas River degradation. However, it was recommended that the District continue to work with the USACE to ensure that downstream dredging does not adversely impact riverbed elevations.

These conclusions are important to the District because they indicate the stability issue of the jetty and is not a localized District problem, but is a regional Kansas River and Missouri River stability issue. In other words, the existing jetty acts as a grade control for the entire Kansas River upstream to any point of geologic bed elevation control. If no such control exists between Kansas City and Lawrence, Kansas, the jetty provides control for the river bed elevation from the District's intake to the Bowersock Dam in Lawrence.

The VE participants agreed that the stability of the District's jetty is key to the real property, water supply and environmental stability of the Kansas River. As a result, the participants agreed that the District should not shoulder the entire economic burden of constructing a stable jetty. It was recommended that communication be started with all parties and entities upstream of the jetty with an interest in the stability of the Kansas River to determine interest in participating in the repair of the jetty. Communications have begun with USACE as discussed previously, and U.S. Congressional Representatives. A package requesting a $2 million appropriation from EPA for the jetty was sent to Representative Moore, Senator Brownback, and Senator Roberts office. This package will be used by their offices to justify appropriations for FY 2006 funding.

2. On-site alternative solutions

The on-site alternative solutions include modifications and additions to the alternatives presented in section D.

a. Modifications to the existing jetty

Use rubble instead of riprap as fill material

Alternative A uses the downstream face of the existing jetty as the base for the armoring rock. As Alternative A progresses over time to accommodate changes in tailwater elevation, more rock would have to be added on top of the rock that was previously placed. Therefore, no fill material is needed for Alternative A.

Alternative B does require fill material. The fill material only represents 20% of the amount of material required for Alternative B and only 11% of the construction costs. The armoring rock represents the remaining 80% of the material and 78% of the construction costs. Even if the fill material can be acquired and placed for free, the cost for Alternative B would still be high, that is, in excess of $28 million.

Inflatable dam

An inflatable dam system wouldn't be appropriate for any permanent solution due to the amount of debris that flows and icing conditions in the Kansas River. The debris and/or ice would damage the inflatable dam which would have to be repaired or replaced periodically.

The Obermeyer Spillway Gate system is an alternative to inflatable dams that may be appropriate to control the flow in the Kansas River. The Obermeyer Spillway Gate system would need to be anchored on top of a permanent structure. Building a permanent structure across the river would be equivalent to building the cofferdam proposed in section D of this report. The installation of the gates would therefore be an additional expense, making this option cost-prohibitive.

Lower crest cofferdam

The crest elevation of the cofferdam as proposed in section D of this report was set at elevation 736 ft to approximate existing conditions at the jetty. It may be possible to reduce costs by lowering the crest of the cofferdam.

Lowering the crest of the cofferdam below elevation 734 ft would prevent the intake from functioning properly during low flow conditions; i.e. submergence at the intake will be below requirements. Assuming it is possible to set the crest elevation of the cofferdams at elevation 734 ft the costs for Alternatives D and D1 will be reduced by about 5% (~$500,000). The minimum elevation for the cofferdam and the design of the weir section that would allow appropriate operation of the intake would have to be analyzed during the design phase of the project, if the cofferdam alternative is selected. This option should be further evaluated during the preliminary design.

Spillway

Increasing the size of the existing weir section would allow more water through the section, thus the head and flow over the rest of the jetty would be reduced. On the other hand, the larger flow through the weir section would require the placement of rock that is larger than the rock that is already in this section. Larger rock than that which is in place would be extremely difficult to obtain from local quarries. Transportation and placement of such rock would be challenging and essentially impossible with the equipment that has been used so far. Therefore a more permanent weir section or spillway would be required.

Estimates indicate that a weir section with a crest at elevation 732 would have to span virtually the entire width of the jetty to prevent headwater levels over elevation 736 during problematic flow conditions (~25,000 cfs). At the same time a spillway larger than the existing weir section would prevent the intake from functioning properly during low flow conditions; i.e. submergence at the intake will be below requirements. These results show that this is not a feasible option.

Gated spillway

A hydraulic analysis was performed to determine the size of the spillway that would reduce the flow velocity enough to provide adequate stability for the rest of the jetty under existing and future conditions. The hydraulic analysis indicates that the spillway should be 150 ft long and 6 ft high; the crest of the spillway would have to be at elevation 729 ft and the crest of the jetty would have to be lowered to elevation 735 ft across its entire length. In order to have adequate submergence at the intake, the spillway would need control gates that would be closed during low flow conditions. The Obermeyer Spillway Gate system or equivalent gate systems may be appropriate to control the flow over the spillway.

In order to build the spillway, water would have to be diverted to at least maintain the top of the cofferdam cells dry. To this effect, a temporary cofferdam would have to be built on the upstream side of the construction area. The number of cofferdam cells alone would represent more than the amount required for the cofferdam proposed in section D. Additional construction cost would be incurred for the placement of the cofferdam cells because the existing rock would have to be excavated, possibly down to bedrock. The cost of the spillway gate system is relatively low compared to the rest; however, the total cost of this option would be greater than Alternative D.

This option does not remove the jetty, it just reduces its height; therefore the smaller jetty would still have to be maintained to prevent a breach. The potential of a breach would be low but the risk remains and would increase as the river degrades.

This option can be excluded because both the risk of failure and the construction cost are potentially greater than Alternatives D and D1.

Articulated concrete slab

The use of articulated concrete slab was proposed as another lining material for either Alternative A or B. Just the cost of the concrete that would be required to build a 3 ft thick concrete slab that could be placed on top of the current slope (9:1) would be around $15 Million. This makes this alternative economically unfeasible.

Gabions

The use of gabions was brought up again during the value engineering workshop. This option was excluded as a viable option due to life of mesh issues. Because of the debris transported by the Kansas River the gabion wire mesh would be subject to abrasion that would significantly reduce the gabion mesh life. Corrosion problems could also be expected.

b. Structures that provide tailwater control

The idea behind these solutions is to raise the tailwater conditions for the existing jetty. Higher tailwater conditions would add stability to the existing jetty. Two options were suggested: (1) wing dikes and (2) low crest dam downstream.

Hydraulic analyses were performed to determine the height of the downstream dam and to determine the size of a constriction that would raise the head water enough to allow the existing structure to remain stable.

The results indicate that the opening in between the wing walls would have to be 50 ft under existing conditions and 20 ft under ultimate stream degradation conditions. The wing walls would have to be a permanent structure such as the cofferdam proposed in section D. Because it would be difficult to modify this structure once built, it should be built for ultimate degradation conditions. The top of the cofferdam would have to be at elevation 736 ft. The cost of the wing walls would be about 2% (~$250,000) lower than the cofferdam presented as Alternative D. Because the wing walls do not provide enough head water to maintain the required submergence at the intake during low flow conditions, the operation of the intake would still depend on the integrity of the existing jetty. The existing jetty would require some maintenance to replace weathered rock and to overcome potential settlement. The wing walls and the lower crest dam would have a project cost that is marginally lower than Alternative D. Maintenance costs, however, would make this option less attractive. In addition, the risk of failure of the jetty, though small, cannot be discarded.

Other structures such as debris dams where discussed in the workshop. The advantage of using such structures is not clear. The crest of a debris dam would have to be at elevation 735 ft to provide any benefit to the existing jetty, in which case the debris dam would have to be almost as stable as the existing jetty. This condition would make this option equivalent to the stepped dams considered in section D.8.a of this report. The high costs of this alternative discard it as a viable option.

c. Modifications to the existing intake

An alternative to fixing the existing jetty would be to build a new intake at a lower elevation. This option was considered in section D8f of this report.

d. Contingency plan, rock stockpile

During the value engineering workshop the possibility of having a rock stockpile as a contingency plan was discussed. The idea is to reduce costs and reduce response time in case a breach occurs in the future.

As previously indicated the jetty had to be repaired two times during 2004. The repair costs were significantly different on these two occasions. The differences in costs fluctuate depending on the time given to the contractor to start the repair. The main factor driving the cost differences seems to have been equipment mobilization and availability. The cost of rock, however, did not seem to be affected by response time requirements. Because equipment mobilization and availability seems to be the limiting factor, stockpiling rock would not provide a significant economical benefit whenever repairs are needed in the future. However, retaining a contractor to be on-call for repair needs would reduce the time required to respond to a breach and would minimize price fluctuations. A bid package has been advertised for on-call jetty repairs.

3. Other recommendations

a. US Army Corps of Engineers, 1946 Flood Control Act Section 14

Section 14 of the 1946 Flood Control Act was mentioned during the value engineering workshop as a potential funding source. Section 14 of the 1946 Flood Control Act allows obtaining federal funds to build bank protection works to protect important and essential public facilities including municipal water supply systems. Section 14 provides federal funding up to $40,000 for studies and up to $1,000,000 for project construction. If the study or the project exceeds these limits the difference must be provided by local cash contributions. It is anticipated that the feasibility of using Section 14 funds would be evaluated in the USACE river degradation study. More information could be obtained at the following web site:

http://www.mvp.usace.army.mil/fl_damage_reduct/default.asp?pageid=2

b. Further analysis on degradation

The data analyzed in section C2 of this report suggests a continuous degradation of the Kansas River bed. The data used in this analysis, however, is limited. During the value engineering workshop two sources of additional data where identified: (1) Turner Bridge stage gage information and (2) stereo-photo interpretation.

It was not possible to locate historical stage data for the Kansas River at Turner Bridge. According to the National Weather Service Website "Forecasts for the Kansas River at Turner Bridge are issued as needed during times of high water, but are not routinely available" (http://www.crh.noaa.gov/cgi-bin/ahps.cgi?eax&tnrk1#Historical). The Pleasant Hill/Kansas City, MO office of National Weather Service office indicated that no historical data or other information is available for this gage. The USGS NWIS web site does not hold any stage or flow records for the Kansas River at Turner Bridge.

Interpretation of historical sets of stereo-photos may provide additional data regarding degradation. Such analysis, however, is beyond the scope of this report and would probably not yield results different than those reached in previous sections of this report.

c. Further risk analysis

Additions to the analysis of stream power and its correlation to failure events were suggested in the value engineering workshop. However, there are no detailed records of past partial failures, thus it is difficult to associate partial or full failures with a given trigger event. In addition, other variables, such as the amount of debris that is transported by the Kansas River at a given time, are not known; thus it is impossible to associate a probability of occurrence to such variables.

H. Summary and Recommendations

From the analysis of potential breach causes, two main failure mechanisms were identified: (1) undercutting of the downstream slope of the rip-rap due to high flow velocities over the jetty and (2) degradation of the river channel downstream of the jetty.

Results show that, without taking into account the effects of degradation, undercutting of the downstream slope is likely over a range of flows. The high flow velocities registered around the time when it failed probably moved the rocks on the downstream slope of the jetty.

Degradation of the channel bed has persisted since the construction of the intake, and there is no indication that it will stop before the streambed reaches bedrock. Degradation produces a combination of effects: (1) it promotes settlement of the jetty, and (2) reduces the tailwater elevation as the streambed elevation gets lower. Each one of these effects increases the chances of failure due to undercutting.

In order to address these problems, six alternatives were analyzed. The first Alternative (A) consists on armoring the downstream slope of the jetty by dumping large rock down to existing riverbed and to periodically update and repair the jetty. The second third and fourth Alternatives (B, B1 and B2) propose the stabilization of the structure by reducing to 16:1 (B and B1) or stepping (B2) the downstream slope of the jetty and armoring the downstream face of the slope with large riprap (B and B2) or another lining material such as A-Jacks (B1). The fifth Alternative (C) adds to Alternative B the construction of a sheet pile wall at the upstream face of the jetty. The last two Alternatives (D and D1) propose the construction of a cofferdam downstream and upstream from the jetty respectively. Other options and variations to the alternatives presented above were also considered. Even though feasibility of these options was addressed, they are not listed as alternatives because (1) they did not provide a suitable solution to address the stability of the jetty and/or (2) they would have resulted in unreasonable costs while providing marginal benefits when compared to the six alternatives that were listed.

Alternative A may be the least expensive option assuming that degradation of the riverbed downstream from the jetty will be minimal and that tailwater conditions will not deviate from current levels. However, because there is no base level control to stop degradation it is likely that degradation will continue. If the downstream riverbed degrades at previously measured rates the present value of Alternative A may be much higher that that of Alternatives D and D1.

Alternatives B, B1 and B2 are costly and only provide a partial solution to the problem and Alternative C was discarded based on the 1988 Black & Veatch draft report. Alternative C adds additional costs to Alternative B while providing marginal benefits. Alternatives D and D1 provide the most permanent solution.

Alternative D and D1 are selected as the best plans. Even though these alternatives may be more expensive than Alternative A, Alternatives D and D1 provide a more permanent solution. Any change in existing riverbed and the river hydraulic conditions would offset the costs of Alternative A potentially making Alternative A much more expensive than Alternatives D and D1.

Additional alternative approaches were discussed in the value engineering workshop (section G of this report). The most promising solutions derived from this workshop are slight modifications to the cofferdam alternatives. These modifications should be considered during the design phase as a way to reduce costs.

Alternative D1 is the recommended plan. While its probable project cost is higher than Alternative D, the construction of Alternative D1 is anticipated to be less risky than Alternative D. The large rocks that have been washed downstream from the jetty may be embedded in the riverbed in the proposed location of Alternative D which could significantly increase construction costs. Alternative D would also cause a stagnant pool to form at the intake. This could create icing problems during the winter. The proposed schedule for construction of Alternative D1 is as follows:

Preliminary design, final design, and permitting-12 months

Advertisement and award-1 month

Construction-18 months

The stability of the District's jetty as a grade control for the Kansas River is vital to the river stability upstream of the jetty. The jetty stops the Kansas River degradation that is most likely associated with the Missouri River degradation and protects the water supply, real property and environmental interests upstream of the jetty. Since this grade control is a regional issue rather than a localized District issue, one key recommendation of the value engineering workshop was to pursue financial participation in the construction of a stable structure, not only for Water One but also in behalf of upstream interests, from the U.S. Congress and the USACE.

This plan is different from that proposed by Black & Veatch in 1988. The recent breach is an indication that the stability of the structure is uncertain under current conditions. Ongoing changes in bed elevation and changes in tailwater elevations provide indications that the stability of the jetty may be further compromised for future conditions. The high repair costs associated with the recent breach and the continued river degradation makes a more permanent solution more desirable than it was when proposed back in 1988.

It's important to note that permitting of a permanent structure can be complicated. Advice on permitting and potential agencies that will require permits and coordination is provided in section E.

A meeting with USACE to discuss jetty stability issues, possible USACE participation in a solution, and funding was held March 31, 2005 (See Appendix III). USACE staff indicated funding for a specific solution to jetty stability is not likely. A study of riverbed degradation on the Missouri River including the Kansas River is planned, but will not be completed for two or three years. Any recommendations from the study will not be constructed until at least four years after the study is complete. USACE staff did indicate funding may be available to assist in drilling exploratory test holes at Grinter's Ferry to verify geologic conditions.

The District applied for an EPA grant to Senator Brownback, Senator Roberts, and Representative Moore to help pay for Alternative D1 for FY 2006 funding. The District is in periodic contact with their offices to ensure they have the information they need.

I. References

Black and Veatch. (1988). *Water Works Master Planning Study for Water District No. 1 of Johnson County Kansas; Volume IV - Kansas River Degradation Study.*

Black and Veatch. (1999). *Hydrogeological Evaluation Part 1 Existing Wellfield Evaluation.* Water District No. 1 of Johnson County Kansas.

Black and Veatch. (2003). *Water District No. 1 of Johnson County Kansas Water Supply Master Plan.*

Freeman G. E. and J. C. Fischenich. (2000). *Gabions for Streambank Erosion Control.* Paper presented by the US Army Corps of Engineers, Ecosystem Management & Restoration Research Program.

Koutsourais, M. (1994). *A Study of Articulated Concrete Blocks Design to Protect Embankment Dams.* Case History presented by ARMORTEC.

Peterka, A.J. (1964). *Hydraulic Design of Stilling Basins and Energy Dissipaters.* United States Bureau of Reclamation, United States Department of the Interior.

Simons, Li, and Associates. (1985). Recommendation for a Plan to Regulate Commercial Dredging on the Kansas River. U. S. Army Engineer District, Kansas City Corps of Engineers.

United States Bureau of Reclamation (1987). *Design of Small Dams.* United States Department of the Interior, third edition.

United States Army Corps of Engineers. *Hydraulic Design Criteria, Sheet 712-1, Stone Stability Velocity vs. Stone Diameter.*

APPENDIX I

COST TABLES

Alternative A, estimated quantities and project cost opinion

Item	Quantity	Unit	Unit cost ($)	Cost (M$)
Project Cost				
Riprap Armor layer	76,800	ton*	60	4,608,000
Mobilization/Demobilization:				461,000
			Total Construction Cost:	5,069,000
			Contingencies (25%):	1,267,000
			Total:	6,336,000
			Legal, administrative and engineering expenses (15%):	950,000
			Total Project Cost:	**7,286,000**

Maintenance (assuming no degradation)			
Inspection	$5,000	/ year over 50 years at 4% interest	107,000
Repair	$46,000	/ year over 50 years at 4% interest	988,000
		Total Present Value Maintenance Cost:	**1,095,000**
		Total cost (assuming no degradation):	**$ 8,381,000**

*Assumes a riprap density of 110 lb/ft^3 (takes into account voids between rocks)

Maintenance (assuming that recent measured trends persist)			
Inspection	$5,000	/ year over 50 years at 4% interest	107,000
Improvements**	$888,050	/ year over 20 years at 4% interest	12,069,000
Repair	$46,000	/ year over 50 years at 4% interest	988,000
		Total Present Value Maintenance Cost:	**13,164,000**
		Total cost (assuming that recent measured trends persist):	**$ 20,450,000**

*Assumes a riprap density of 110 lb/ft^3

** Improvements include additional material added over 20 years to reach the 16:1 required for ultimate conditions

Alternative B, estimated quantities and project cost opinion

Item		Quantity	Unit	Unit cost ($)	Cost (M$)
Project Cost					
Rock Fill (subgrade material required to obtain the desired slope)					
		68,000	ton*	33	2,244,000
Excavation (required to obtain the 16:1 slope down to bedrock)					
		30,000	yd^3	10	300,000
Riprap Armor layer					
		264,000	ton*	60	15,840,000
Mobilization/Demobilization:					
					1,838,000
				Total Construction Cost:	20,222,000
				Contingencies (25%):	5,056,000
				Total:	25,278,000
				Legal, administrative and engineering expenses (15%):	3,792,000
				Total Project Cost:	**29,070,000**
Maintenance (Present Value)					
Inspection	$5,000				107,000
Repair	$158,000				3,394,000
				Total Present Value Maintenance Cost:	**3,501,000**
				Total cost:	**$ 32,571,000**

*Assumes a riprap density of 110 lb/ft^3

Alternative B1, estimated quantities and project cost opinion

Item	Quantity	Unit	Unit cost ($)	Cost (M$)
Project Cost				
Rock Fill (subgrade material required to obtain the 16:1 slope)				
	68,000	ton*	33	2,244,000
Excavation (required to obtain the 16:1 slope down to bedrock)				
	30,000	yd^3	10	300,000
96" A-Jacks Armor layer				
	497,000	ft^2	70	34,790,000
Mobilization/Demobilization:				
				3,733,000
			Total Construction Cost:	41,067,000
			Contingencies (25%):	10,267,000
			Total:	51,334,000
			Legal, administrative and engineering expenses (15%):	7,700,000
			Total Project Cost:	**59,034,000**
Maintenance (Present Value)				
Inspection	$5,000	/ year over 50 years at 4% interest		107,000
Repair	$348,000	/ year over 50 years at 4% interest		7,476,000
			Total Present Value Maintenance Cost:	**7,583,000**
			Total cost:	**$ 66,617,000**

*Assumes a riprap density of 110 lb/ft^3

Alternative B2, estimated quantities and project cost opinion

Item	Quantity	Unit	Unit cost ($)	Cost (M$)
Project Cost				
Rock Fill (subgrade material required to obtain the desired slope)				
	61,000	ton*	33	2,013,000
Excavation (required to obtain the 16:1 slope down to bedrock)				
	28,000	yd^3	10	280,000
Riprap Armor layer				
	264,000	ton*	60	15,840,000
Mobilization/Demobilization:				
				1,813,000
			Total Construction Cost:	19,946,000
			Contingencies (25%):	4,987,000
			Total:	24,933,000
			Legal, administrative and engineering expenses (15%):	3,740,000
			Total Project Cost:	**28,673,000**
Maintenance (Present Value)				
Inspection $5,000				107,000
Repair $158,000				3,394,000
			Total Present Value Maintenance Cost:	**3,501,000**
			Total cost:	**$ 32,174,000**

*Assumes a riprap density of 110 lb/ft^3

Alternative D, estimated quantities and project cost opinion

Item	Quantity	Unit	Unit cost ($)	Cost ($)
Project Cost				
Excavation				
	50,700	yd^3	10	507,000
Place 8' depth fill in excavation				
	16,900	yd^3	30	507,000
Place sheet piles				
	102,900	yd^3	40	4,116,000
Place remaining fill in cells and arcs				
	25,900	yd^3	30	777,000
Grout front face of sheet piles at top of rock				
	1,000	yd^3	250	250,000
Construct concrete cap on cells				
	1,100	yd^3	500	550,000
Place large riprap at downstream side of cells				
	9,400	yd^3	75	705,000
Remove sheetpile structure:				
	1		350,000	350,000
Mobilization/Demobilization:				
				776,000
			Total Construction Cost:	8,538,000
			Contingencies (25%):	2,135,000
			Total:	10,673,000
			Legal, administrative and engineering expenses (15%):	1,601,000
			Total Project Cost:	**$ 12,274,000**

Alternative D1, estimated quantities and project cost opinion

Item	Quantity	Unit	Unit cost ($)	Cost ($)
Project Cost				
Excavation				
(for cells over jetty only)	5,800	yd^3	10	58,000
Place 8' depth fill in excavation				
(for cells over jetty only)	1,600	yd^3	30	48,000
Place remaining fill in cells and arcs				
(for cells over jetty only)	4,300	yd^3	30	129,000
Place sheet piles				
	141,800	yd3	40	5,672,000
Compact sand in cells and arcs using vibroflotation				
	1	ls	750,000	750,000
Verification borings for vibrocompaction				
	2,100	ft	25	53,000
Add fills to cells and arcs				
	11,700	yd^3	25	293,000
Grout front face of sheet piles at top of rock				
	1,300	yd^3	250	325,000
Install concrete cap on cells				
	1,500	yd^3	500	750,000
Remove sheetpile structure:				
	1		350,000	350,000
Mobilization/Demobilization:				
				843,000
			Total Construction Cost:	9,271,000
			Contingencies (25%):	2,318,000
			Total:	11,589,000
			Legal, administrative and engineering expenses (15%):	1,738,000
			Total Project Cost:	**$ 13,327,000**

Other options considered, conceptual project costs (Section D.8)

D.8.a. Stepped dams downstream from the jetty

Item		Quantity	Unit	Unit cost ($)	Cost (M$)
Project Cost					
Rock Fill (subgrade material required to obtain the desired slope)					
		451,000	ton*	33	14,883,000
Excavation (required to obtain the 16:1 slope down to bedrock)					
		304,000	yd^3	10	3,040,000
Riprap Armor layer					
		171,000	ton*	60	10,260,000
Mobilization/Demobilization:					
					2,818,000
				Total Construction Cost:	31,001,000
				Contingencies (25%):	7,750,000
				Total:	38,751,000
				Legal, administrative and engineering expenses (15%):	5,813,000
				Total Project Cost:	**44,564,000**
Maintenance (Present Value)					
Inspection	$5,000				107,000
Repair	$103,000				2,213,000
				Total Present Value Maintenance Cost:	**2,320,000**
				Total cost:	**$ 46,884,000**

*Assumes a riprap density of 110 lb/ft^3

D.8.b. Milder slope

Small rock (W50 = 5 ton) using a 19:1 slope

Item	Quantity	Unit	Unit cost ($)	Cost (M$)
Project Cost				
Rock Fill (subgrade material required to obtain the desired slope)				
	84,000	ton*	33	2,772,000
Excavation (required to obtain the 19:1 slope down to bedrock)				
	41,000	yd^3	10	410,000
Riprap Armor layer				
	306,000	ton*	60	18,360,000
Mobilization/Demobilization:				
				2,154,000
			Total Construction Cost:	23,696,000
			Contingencies (25%):	5,924,000
			Total:	29,620,000
			Legal, administrative and engineering expenses (15%):	4,443,000
			Total Project Cost:	**34,063,000**
Maintenance (Present Value)				
Inspection $5,000				107,000
Repair $184,000				3,953,000
			Total Present Value Maintenance Cost:	**4,060,000**
			Total cost:	**$ 38,123,000**

*Assumes a riprap density of 110 lb/ft^3

Rubble (W50 = 1 ton) using a 72:1

Item	Quantity	Unit	Unit cost ($)	Cost (M$)
Project Cost				
Excavation (required to obtain the 72:1 slope down to bedrock)				
	280,000	yd^3	10	2,800,000
Rubble				
	518,000	ton*	33	17,094,000
Mobilization/Demobilization:				
				1,989,000
			Total Construction Cost:	21,883,000
			Contingencies (25%):	5,471,000
			Total:	27,354,000
			Legal, administrative and engineering expenses (15%):	4,103,000
			Total Project Cost:	**31,457,000**
Maintenance (Present Value)				
Inspection $5,000				107,000
Repair $171,000				3,673,000
			Total Present Value Maintenance Cost:	**3,780,000**
			Total cost:	**$ 35,237,000**

*Assumes a riprap density of 110 lb/ft^3

APPENDIX II

Value Engineering Workshop
Experts Statements

Engineering,
Landscape Architecture
and Environmental Science

MILONE & MACBROOM®

December 16, 2004

Mr. Donald Baker
Water Resources Group
Black & Vetch Corporation
8400 Ward Parkway
Kansas City, Missouri 64114

RE: Kansas River Jetty
MMI #2650-01-1

Dear Mr. Baker:

It was a pleasure to meet you last month at the Value Engineering Workshop on the Kansas River Jetty for the Water District #1 Intake. The workshop included a powerpoint presentation on the history and performance of the intake jetty, a site inspection, and a discussion of potential alternate repairs. I have prepared the following comments on my observations and thoughts on the above subject.

UNDERSTANDING

1. The Kansas River is a low gradient plains river. Unlike the adjacent Arkansas and Platte River basins, its' headwaters exclude snow fed mountainous areas and therefore depends on direct rainfall and ground water. The overall channel is generally wide and shallow with numerous sand bars and multiple flow paths. The alignment is sinuous, with some lateral confinement where the channel reaches the floodplain and valley wall. The channel east of the jetty is reportedly incised to the Missouri River.

2. The Kansas River has a watershed area of 59,756 square miles at the DeSoto USGS gauge and a median annual flood of about 60,000 CFS. The gauge recorded only one annual peak flood over 100,000 CFS from 1974 to 1992, since then there have been three events over 100,000 CFS. NOAA records show only a slight increase in long-term precipitation.

3. It is my understanding that the loose stone jetty was built in 1967 to divert water to the right bank water supply intake, and to help maintain upstream river water levels. The jetty extends across the full channel width.

4. The jetty has a history of surficial erosion of its riprap, requiring periodic repairs.

5. The downstream channel has reportedly degraded about 15 feet over the past 40 years, reducing tailwater levels.

6. The water elevation differential (head) at the jetty has correspondingly increased as the tailwater decreased, such that the "jetty" now acts as a pervious dam.

7. It has become increasingly difficult and costly to maintain the jetty.

8. It is believed that the periodic accumulation of debris along the jetty crest tends to concentrate over flows, encouraging erosion.

9. The present observed erosion channel across the jetty surface is about 60 feet wide with two distinct headcuts, one of which is very near the crest. This type of channel evolution usually is an indicator of erosion that began at the downstream end.

10. Black & Vetch Corp has investigated the jetty, recent breach, and alternative repair strategies.

COMMENTS

1. My impression is that the jetty (built in 1967) began to have erosion that required repairs so early in its life (1970, 1972, 1974, etc.) that there is a fundamental weakness in the loose rock jetty concept. This is supported by a comparison of the "repair" years versus the annual USGS peak flood flows (there is no pattern). The dumped rock on the jetty face has variable voids, gradation and sizes, making it prone to occasional erosion.

2. Downstream channel degradation and the corresponding lower tailwater could decrease the jetty's surface stability, due to both hydraulic jumps and their position lower on the jetty, near the critical toe position.

3. During our site visit, I noticed extensive seepage in the mid-level of the jetty, some of it with turbid water. The lack of a low permeability core creates seepage forces that reduce riprap stability.

4. Channel degradation is generally due to reduced base levels, increased flow rates, or modified sediment regimes. It appears that Missouri River degradation may influence the Kansas River, and that downstream sand and gravel mining is probably a contributing factor. A quick check of USGS suspended sediment data at Wamego, Kansas (1958-1975) indicates that there was a sharp reduction after 1962, presumably due to dams on the Kansas River tributaries. The combination of reduced sediment loads, plus dredging some of the remaining bedload, could account for channel degradation. If a channel is initially in equilibrium, then dredging even just a fraction of the sediment load could induce degradation.

5. I noted that the upstream channel has extensive gravel bars, a shallow water depth, and a high width to depth ratio. Aerial photographs and my subsequent site inspection reveal a distinct change in sediment conditions just upstream of the Route 7 bridge near Wilder, where additional gravel mining occurs. Similarly, the gravel mining just downstream of the jetty is expected to have a negative impact.

6. The computed flow velocities over the weir are in the range of 20 feet per second, very fast even for a mild channel slope with uniform flow. When combined with a steep slope, turbulence, and seepage forces it is not surprising that riprap failures have occurred.

In conclusion, the jetty is functioning as an in-stream run of the river dam, but lacks an impervious core, cut off walls, and erosion proof spillway. Consequently, it is prone to damage.

REMEDIAL ACTION

1. The Black & Vetch report discusses several alternative approaches to protecting use of the water supply intake. I concur that alternate D (cofferdam) is the best long-term solution, simply because rebuilding the jetty's loose rock face has had only temporary benefits and due to continuing degradation.

2. The continuing downstream channel degradation is a community wide issue due to its potential impact on infrastructure (bridges, powerplant, pipe crossings) and environmental impact. Maintaining the jetty is the only known way to prevent massive upstream headcutting and erosion. I strongly recommend that a systematic community wide approach be adopted as this jetty (dam) impacts many properties and the river's ecological health.

3. Several interim measures are available for use until a long-term solution is in place. They include ceasing downstream gravel mining, pumping upstream gravel over the jetty to the toe, use of an upstream cable debris boom, and injecting grout into the rock mass of the jetty.

It was a pleasure to meet with you and discuss the above project. Please contact me if you have any questions.

Very truly yours,

MILONE & MACBROOM, INC.

James MacBroom

James G. MacBroom, P.E.
Senior Vice President

2650-01-1-d1404-ltr.doc

INTUITION
LOGIC
450 N. New Ballas Rd., Suite 264N St. Louis, MO 63141
314.432.2543 314.432.5812
1306 Autumn Trace Amelia Island, FL 32034
904.261.5555

December 22, 2004

Jeff Henson, P.E.
Black & Veatch
8400 Ward Parkway
Kansas City, MO 64114

Dear Jeff,

I reviewed the draft report, "Water District No. 1 of Johnson County, Kansas, Kansas River Intake Jetty Improvements Study" dated November 12, 2004, and participated in the Value Planning workshop. The mosaic of aerial photos presented by Jim MacBroom at the workshop and the background data presented in the report and at the workshop assisted in understanding the project. I concur with Dr. MacBroom that the existing jetty is a major geomorphic control feature. Upstream of the jetty, sediment is stored in alternating bars and the channel is more sinuous. Downstream of the jetty, there is little stored sediment and the channel is incising. The jetty acts as a major grade control protecting the Kansas River above the intake. After the workshop, I reviewed a project we did for Overland Park. A similar condition exists on the Blue River.

In your report you describe three failure mechanisms that may be occurring concurrently. I agree with the forensic analysis that was performed. The data support the assertion that a major head cut is moving upstream and undermining the downstream toe of the jetty. I concur with your conclusion that there is no effective geologic control downstream of the jetty. The shape of the eroding downstream face observed in the field and in the output of the hydraulic profile support the results of your analysis indicating that a hydraulic jump is occurring on the jetty. I concur that the accumulation of woody debris on the levee may result in local concentrations of flow and scour. It is also likely that riprap is settling in the sand bed under the jetty as fine-grained material is piped through the structure.

You analyzed the rock size necessary to resist scour using COE Hydraulic Design Criteria Sheet 712-1 Stone Stability Velocity vs. Stone Diameter (Sheet 712-1). The predicted rock may be undersized. The USACE engineering manual EM 1110-2-1601, 30 Jun 94, Section 3-7 Stone Size, c., (1) recommends increasing the safety factor in the rock sizing equation when there are logs and other debris. Also this equation does not consider the uplift condition that was observed onsite. Uplift forces should be considered when sizing rock.

It is by LOGIC that we prove
but by INTUITION that we discover

I concur with the alternatives evaluation and risk assessment presented in the report. My one exception is that the riprap may be under-sized, further increasing the cost for Alternatives A and B.

An additional alternative may be to install upstream-pointing wing dikes downstream of the jetty. The dikes should be oriented to shift scouring flow away from the banks and to keep the thalweg in the center of the channel. The hydraulic roughness and shape of the wing dikes can be used to raise the tailwater elevation during a damaging flow to protect the downstream face of the existing jetty. If may be necessary to protect the jetty from concentrated flows near the thalweg.

The team members also discussed that the Water District's efforts to protect their intake is benefiting everyone upstream. If the jetty were allowed to fail, a major headcut would move upstream resulting in bank failures and scour. I concur that dialogue should begin to involve all of the stakeholders. Additional funding sources should be developed to assist the Water District in the defense of this regional problem.

Thank you for the opportunity to participate in the workshop. If I can be of further assistance please contact me.

Best regards,

Robert Prager

Robert Prager, P.E.
Principal River Engineer

APPENDIX III
USACE Meeting Notes
March 31, 2005

Water District No. 1
Corps of Engineers Meeting Notes
Kansas River Intake Weir
March 31, 2005

Attendees:

John Grothaus – Corps of Engineers (Chief Plan Formulation Section)
Allen Tool – Corps of Engineers (Chief of Hydrology and Hydraulics)
Gordon Lange – Corps of Engineers (Hydrology and Hydraulics)
Don Meier – Corps of Engineers (Hydrology and Hydraulics)
Josh Marx – Corps of Engineers (Regulatory)
Mike Armstrong - WaterOne
Tom Schrempp – WaterOne
Jeff Henson – Black & Veatch
Mike Horsley – Black & Veatch

Presentation:

Jeff Henson presented a review of the weir problem with powerpoint slides and handouts. Main areas covered were as follows:

- Background and breach history
- Failure analysis (potential breach causes)
- Alternatives analyses
- Corps of Engineers Interests

Discussion:

Nomenclature. Appropriate name for structure is "weir".

Grinter's Ferry. Corps said no indication of bedrock at Grinter's Ferry but would have to do boring to be sure. River appears to be going around hard spot caused be glacial deposits washed in from creek. B&V mentioned evidence of anticline since there is an apparent change in elevation of formations.

Records. Corps said additional cross-section data and aerial photos available for Kansas River – contact Ken Stark.

Dredging. One dredger below weir. They will be checking whether 2 foot average elevation loss (bank to bank) for 5 miles rule per EIS is violated. If WaterOne has evidence dredging is a problem they will consider.

Funding of Weir Upgrade to Withstand Continued Degradation of River. Corps has no authority to act on problem. Authority on Kansas River limited to Lakes and specific levees. Missouri River is different, greater authority, includes navigation, bank stabilization, and levees. For the Kansas River, there is a Section 14 funding mechanism but the funding limit is $1,000,000 which must pay for total project on a 65%/35% basis and must include construction. Section 14 would not be applicable to weir. Section 14 was used for bank stabilization when the Eudora bridge was threatened.

Kansas River Basin Authority. Kansas River Basin Authority has been proposed, which would give them more opportunity to fund projects. Will send congressional briefing package to WaterOne.

KWO Study on Degradation. Mentioned KWO Study on Degradation. TAC is looking at three reasons for degradation:

- Reservoirs
- Dredging
- Mo. River

Establishing a Control Point at Grinter's Ferry. No solution has been developed at this time. Would have to evaluate problem before arriving at a solution.

Degradation Study. Missouri River degradation study has been authorized but not yet funded. Would also look at Kansas River up to some point, perhaps to the WaterOne weir, since that could be the limit of the effect of Missouri River degradation on the Kansas River. After the study, they would address solutions. Timing would be 2 to 3 years for study, 1 to 2 years to figure out solution, and then construction. Would not fix weir, but perhaps Grinter's Ferry.

Permitting. Some items to be considered:

- Need portage around weir.
- Environmental has wanted weir removed in past.
- Keep same elevation.
- Fish ladder.

EPA / STAG Funding. Corps has no problem with WaterOne seeking EPA / STAG funding.

PAS Funding. Funding may be available through Corps PAS funding for small study, perhaps fish ladder or drilling test borings at Grinter's Ferry. Corps to send details to B&V.