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ABSTRACT

Discharges up to 60,000 cfs that lasted 21 days caused extensive erosion of the unlined spillway chute excavated in alternating layers of limestone and shale. An empirical model allowed to evaluate the extent of erosion anticipated for future events. Parametric calculations showed that relatively low discharges a long period of time are critical for the induced damage. It was determined that the spillway can safely pass the design discharge if weathering of rock exposed by erosion is prevented. Accordingly, the interim repair was designed to protect the rock units, especially the critical limestone layers, from weathering. Although it is expected much of the repair to be eroded during future spillway flows, it will ensure the spillway can safely pass the next discharge event.

KEYWORDS

Rock mechanics, Weathered rock, River bed erosion, Spillway discharges, Rock erodibility, Empirical models, Erosion repair.

INTRODUCTION

Tuttle Creek reservoir, a flood control project located in Eastern Kansas, has a 9,628 square miles watershed and an estimated maximum design outflow through the excavated spillway of 612,000 cfs. The dam is a rolled-earth/rock fill embankment about 140 feet high and 7,500 feet in length. The spillway is located on the left abutment of the dam and consists of a concrete weir structure, concrete lined chute, and an unlined chute. The weir structure is 839 feet wide with 18 tainter gates 20 feet high and non-overflow bulkheads. The concrete apron is as wide as the weir, 600 feet in length and terminates with the flip bucket which includes a cutoff wall.

The unlined portion of the chute was excavated 839 feet wide at the flip bucket and narrows to only 200 feet wide at the lower end. The chute drops over 86 feet in a horizontal distance of 3,400 feet. The average channel gradient from the cut off wall to the valley floor was approximately 2.5%. The chute curves 90 degrees to the right with an approximate radius of 1,400 feet. The chute bottom consists of alternating limestone and shale units. The major limestone units are relatively durable and do not erode easily, however the shale units are generally soft and erodible. The alternating layers are susceptible to head cutting caused by erosion of the shales, leading to undermining of the erosion resistant limestones.

A major flood event, with an estimated 100-year return period, occurred in the summer of 1993. Discharges through the spillway occurred for the first time, after 30 years of operation of the project. Spillway releases lasted 21 days reaching a peak discharge of 60,000 cfs. Nearly 400 000 cy of material were eroded from the unlined chute resulting in a series of escarpments ranging in height from 4 to 26 feet with some concentrated head cutting.

An attempt was made to estimate further possible headcut advance during future storm events using the empirical model developed by U.S. Department of Agriculture (USDA) researches (Moore et al, 1994, Temple and Moore, 1994). However, this model was based primarily on soil spillways and did not fit the actual observed rates of rock erosion at Tuttle Creek spillway. A modified model was, therefore, developed using the same approach established by USDA researchers, but different calibration points. The Kansas City District (KCD) model was conceived in such a manner to overpredict erosion. Although the KCD model is essentially site specific, based primarily on data obtained from the Tuttle Creek event, it is believed it can be used for erosion potential evaluation in similar rock conditions.

The KCD model was used to evaluate the extent of erosion anticipated for future events at Tuttle Creek spillway. Based on this evaluation repairs were designed to ensure the spillway can safely pass the next discharge event.

AREA GEOLOGY

The site geology is characterized by alternating layers of limestone and shale. The limestones are medium hard to hard and the shales are generally soft, easily erodible. Table 1 shows the simplified geologic column of the rock units encountered in the spillway.

Table 1. Geologic Column.

Rock Unit	Thickness (feet)	Rock Type	Drying/Wetting Test Result
Burr	4.1	Limestone	
Legion Shale	1.5	Shale	
Sallyards Limestone	1.9	Limestone	5% loss after 13 cycles
Roca Shale:			85% loss after
Zone A	4.3	Shale	4 cycles
Zone B	1.0	Limestone	
Zone C	3.0	Shale	100% - 1 cycle
Zone D	1.7	Limestone	
Zone E	5.1	Shale	100% - 1 cycle
Zone F	0.6	Limestone	
Zone G	2.6	Shale	100% - 1 cycle
Zone H	0.7	Limestone	100% - 1 cycle
Zone I	4.3	Shale	100% - 1 cycle
Howe Limestone	4.1	Limestone	
Bennett Shale	3.5	Shale	
Glenrock Limestone	2.6	Limestone	
Johnson Shale:			
Zone A	5.5	Shale	100% - 1 cycle
Zone B	1.5	Limestone	0 loss, 10 cycles
Zones C&D	6.6	Shale	100% - 1 cycle
Zone E	1.5	Shale	
Zones F&G	8.5	Mudstone	100% - 1 cycle
Long Creek:			
Zone A	2.0	Limestone	
Zones B&C	5.6	Limestone	
Hughes Creek:			
Zone A	1.6	Shale	
Zone B	0.5	Limestone	
Zones C&D	4.5	Shale	

ROCK ERODIBILITY CHARACTERIZATION

The characterization of the rock units for hydraulic erodibility was made in accordance with the methodology developed at U.S. Department of Agriculture by J. Moore and D. Temple. According to Moore et al (1994) the spillway erosion process can be divided in three sequential phases for purposes of mathematical quantification:

- Phase I: Erosion resulting in the local failure of the vegetal cover, if any, and the development of the concentrated flow,
- Phase II: The downward and downstream erosion leading to the formation of a vertical or near vertical headcut, and
- Phase III: The upstream advance of the headcut with associated widening and deepening.

In the case of Tuttle Creek spillway it is believed that the first two phases had a relatively short duration as compared with the third phase. Measurements of the headcut advance, that were used to calibrate the KCD model, became available starting with the sixth day of discharge, when the erosion process was evidently in the third phase of development all along the unlined portion of the spillway. Therefore, only phase III parameters were evaluated and used in mathematical quantification of the spillway erosion.

In the analysis of phase III of the erosion process, there are three main steps:

1. Determination of an erosion threshold, when the hydraulic action starts to induce headcut advance,
2. Quantification of the headcut advance in the upstream direction, and
3. Headcut deepening during the phase III erosion.

For the first two steps of phase III analysis the characterization of the materials in spillway is given by the "erodibility index" (K_h). The headcut deepening should be related to an erosion rate similar to that used to characterize the behavior in phase II (Temple and Moore, 1994); however, the data available were not sufficient for evaluation of the erosion rate of materials as encountered in the Tuttle Creek spillway. Therefore, only the upstream advance of the headcut was considered in the KCD model.

EROSION INDEX

According to Moore et al (1994) the erosion (or erodibility) index, K_h, represents a measure of the resistance of the material to erosion and has the general form:

$$K_h = M_s \times K_b \times K_d \times J_s$$

where:

M_s = material strength number of the earth material.

For rock, $M_s = 0.78\ (UCS)^{1.09}$ for UCS ≤ 10 Mpa (104.4 tsf) and M_s = UCS for UCS > 10 MPa, where UCS is the unconfined compressive strength (ASTM D-2938).

K_b = block/particle size number. For rock and rock-like materials the primary method of calculation of this parameter is $K_b = RQD/J_n$ where RQD = Rock Quality Designation, and J_n = Joint Set Number. RQD is a standard parameter in drill core logging and represents the sum of the length of core pieces greater than 0.1 m (4") divided by the total core run length (usually 1.5 m ≅ 60"), expressed in percent. The joint set number is a scale factor representing the effect of different individual discontinuity spacings relative to the average discontinuity spacing; this factor accounts for the shape of the material units or, alternatively, the relative occurrence of different joint sets.

K_d = discontinuity/inter-particle bond shear strength number. $K_d = J_r/J_a$ where J_r = joint roughness number, which represents the degree of roughness of opposing faces of a rock discontinuity, and J_a = joint alteration number, which represents the degree of alteration of the materials that form the faces.

J_s = relative ground structure number, which represents the orientation of the effective dip of the least favorable discontinuity with respect to stream flow. The number takes into account the effect of the relative shape of the material units (as determined by joint set spacings) on the ease with which the stream penetrates the ground and dislodges individual material units.

Details of the procedure for evaluating the parameters that form the headcut erodibility index, including tables that show the range of variation of the parameters with rock type, are presented by Moore (unpublished). The evaluation of headcut erodibility index for various rock units in Tuttle Creek spillway is presented in Table 2. The parameter J_s was determined taking into account the following parameters, valid for all rock units: Spillway Flow Direction = 135 azimuth degrees; Bedrock Strike = 330 azimuth degrees; Bedrock Dip Direction = 240 azimuth degrees; Rock Dip = 1^0.

Table 2. Calculation of Erosion Index.

Unit	Thickness (feet)	UCS (psi)	M_s	RQD	J_n	J_r min	J_r max	J_a min	J_a max	J_s	K_h min	K_h max
Sallyards Limestone	1.9	3910	27.0	79.3	2.2	1	3	1	2	0.88	428	2570
Roca Shale, Zone A (Sh)	4.3	356	2.1	20*	5.0	1	3	1	4	0.88	1.8	22.2
Zone B (Ls)	1.0	2480	17.1	89.3	3.3	1	3	1	2	0.88	204	1220
Zone C (Sh)	3.0	850	5.4	15*	5.0	1	2	1	6	0.88	2.4	28.5
Zone D (Ls)	1.7	1580	10.9	50*	5.0	1	3	1	4	0.88	24	288
Zone E (Sh)	5.1	1655	11.4	40*	5.0	1	3	1	4	0.88	20	241
Zone F (Ls)	0.6	1068	6.9	25*	5.0	1	3	1	4	0.88	7.6	91
Zone G (Sh)	2.6	825	5.2	40*	5.0	1	3	1	4	0.88	9.1	110
Zone H (Ls)	0.7	980	6.3	71.0	3.3	1	3	1	2	0.88	60	358
Zone I (Sh)	4.3	773	4.8	40*	5.0	1	3	1	4	0.88	8.4	101
Howe Limestone	4.1	2835	19.6	94.0	3.3	1	3	1	2	0.88	246	1470
Bennett Shale	3.5	247	1.4	15*	5.0	1	2	1	6	0.88	0.6	7.4
Glenrock Limestone	2.6	6633	45.7	95.0	2.7	1	3	1	2	0.88	707	4250
Johnson Shale, Zone A (Sh)	5.5	331	1.9	30*	5.0	1	3	1	4	0.88	2.5	30
Zone B (Ls)	1.5	3620	25.0	60*	3.3	1	3	1	4	0.88	100	1200
Zones C&D (Sh)	6.6	969	6.2	45*	5.0	1	3	1	4	0.88	12.3	147
Zone E (Sh)	1.5	1220	8.0	50*	5.0	1	3	1	4	0.88	17.6	211
Zones F&G (Sh)	8.5	359	2.1	30*	5.0	1	3	1	4	0.88	2.8	33
Long Creek, Zone A (Ls)	2.0	4101	28.3	91.1	3.3	1	3	1	2	0.88	344	2060
Zones B&C (Ls)	5.6	2962	20.4	90.0	3.3	1	3	1	2	0.88	245	1470
Hughes Creek, Zone A (Sh)	1.6	1795	12.4	40*	3.3	1	2	1	2	0.88	66.1	264
Zone B (Ls)	0.5	3858	26.6	70*	3.3	1	3	1	3	0.88	166	1490
Zones C&D (Sh)	4.5	931	5.9	40*	3.3	1	2	1	2	0.88	31.5	126
Zone E (Ls)	2.1	3446	23.8	50*	3.3	1	3	1	3	0.88	106	952
Zone F (Sh)	1.7	1395	9.2	40*	3.3	1	3	1	3	0.88	32.7	294

Notes: * Estimated; Ls = Limestone; Sh = Shale; Shaded areas = Relatively more erodible shales.

THE 1993 EROSION EVENT

Releases were gradually increased during a 4-day period until they reached the peak discharge of 60,000 cfs. Without any further change in gate opening, the discharge gradually decreased to zero during the following 17 days. An aerial photo at the end of releases is presented in Fig. 1.

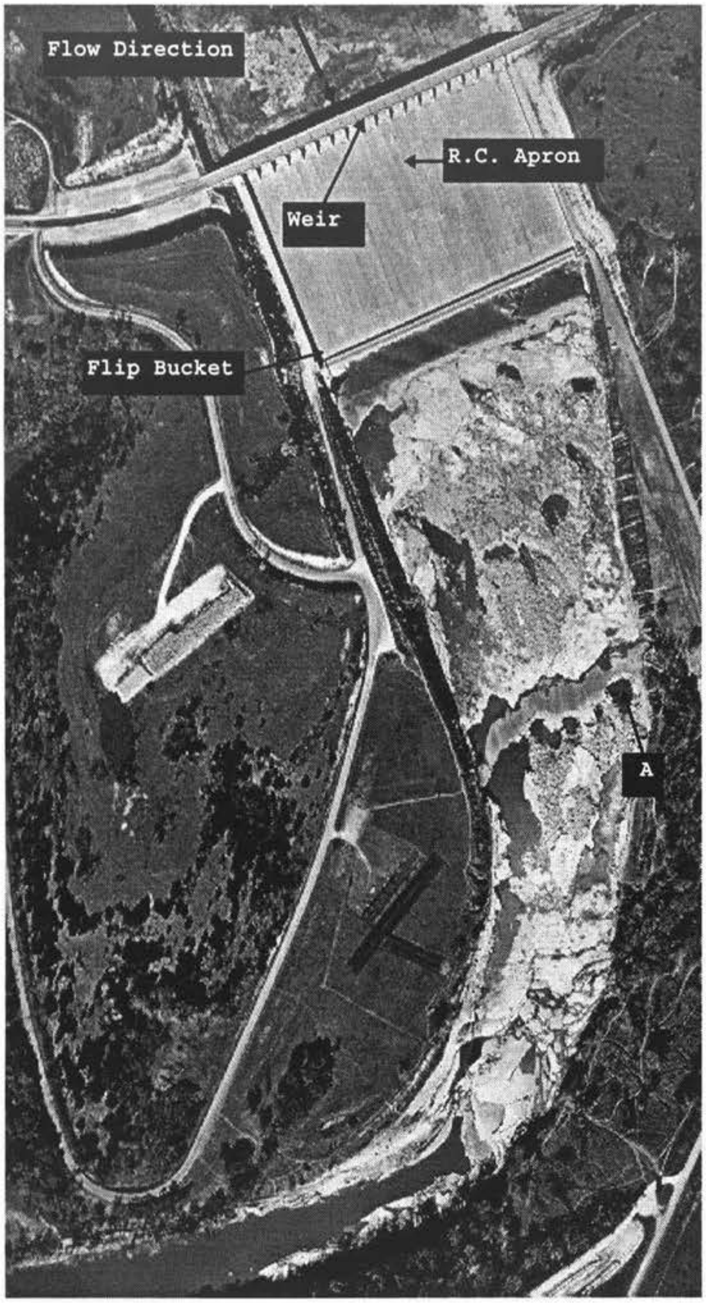

Fig. 1 Post - Erosion Aerial Photo.

By comparing contours determined from aerial photographs before and after the flow through the spillway, the total eroded distance along each headcut was determined, as shown in Table 3. The measured erosion was defined as the horizontal distance along the middle of the hard layer (generally limestone) at the top of the escarpment. Fig. 2 shows the profile along the most eroded zone, before and after spillway releases.

Table 3. Total Erosion at the End of 1993 Event.

Head-cut No.	Limestone (Ls) on Top and Shale (Sh) Underneath	Average Thickness (feet)		Erosion at Middle of Ls (feet)	
		Layer	Total	Max	Average
1	Burr Ls &	4.0	6.0	130	130
	Legion Sh	2.0			
2	Sallyards Ls &	1.9	6.2	130	86
	Roca, Zone A Sh	4.3			
3	Roca, Zone B Ls &	1.0	4.0	250	129
	Roca, Zone C Sh	3.0			
4	Roca, Zone D Ls &	1.7	15.0	210	82
	Roca, Zones E...I	13.3			
5	Howe Ls &	4.1	7.6	65	38
	Bennett Sh	3.5			
6 + 7	Glenrock Ls &	2.6	26.2	173	125
	Johnson A...G	23.6			
8	Long Creek Ls &	7.6	18.0	275	172
	Hughes Creek Sh	10.4			

The headcut erosion was not uniform. The advance was relatively high at weak points and at other locations the erosion was minimal. All headcuts had a relatively hard layer (usually limestone) on top and easily erodible rock underneath. Fig. 3 presents a characteristic headcut immediately after the erosion event.

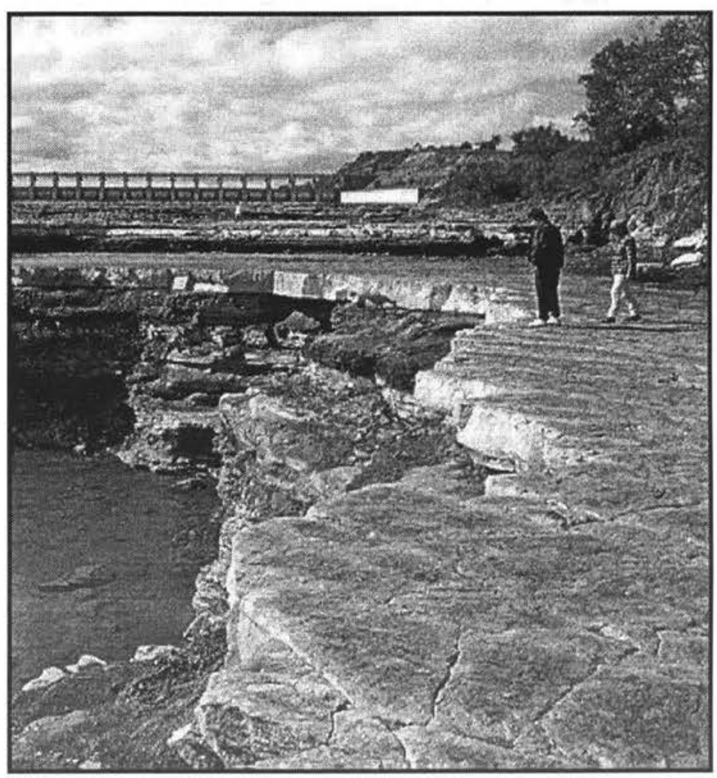

Fig. 3 Characteristic headcut (point A on Fig. 1).

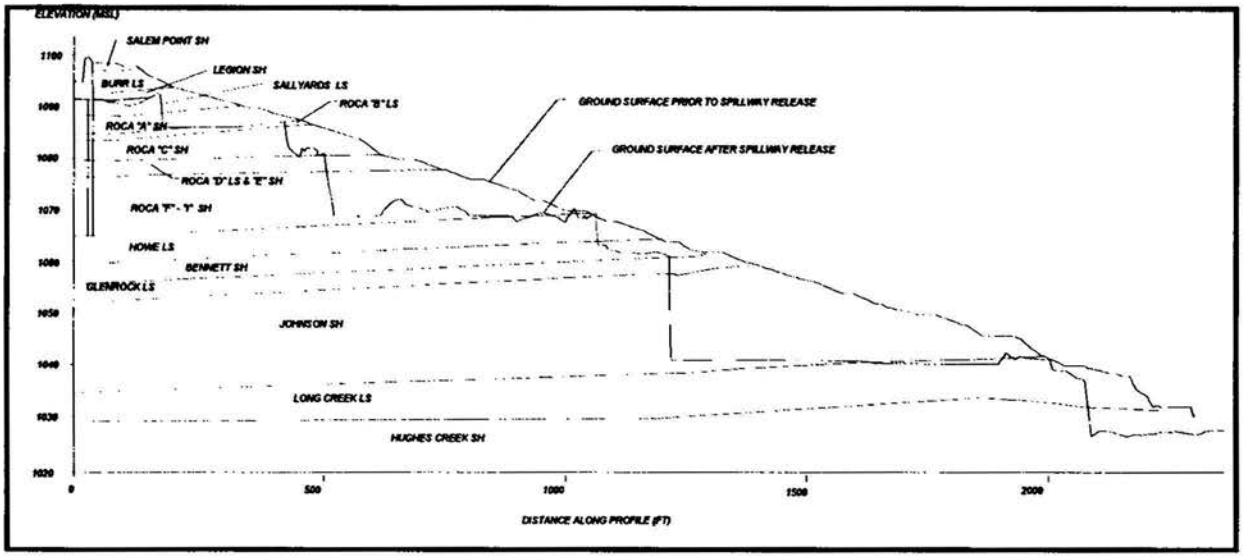

Fig. 2 Profile along right side of the spillway.

EROSION MONITORING

During the releases daily photographs were taken at nine different sites along the spillway, to document regression of headcuts. Daily panoramic photos were reviewed and erosion advance in the upstream direction was estimated at selected knick points on top of the headcuts, where the flow was visibly affected by the ledge.

This method of estimating erosion rate was somewhat inaccurate, but provided reasonable determination of average erosion rate for periods of time varying between 4 and 16 days. These rates, and the corresponding average flows during the same periods of time, were used together with rock erodibility indices to develop a mathematical model of headcut upstream advance.

Aggregate headcut erodibility indices were used to characterize erodibility of multiple layers. They were evaluated using the formula (Temple and Moore, 1994):

$$K_h = \exp\,[\Sigma h_i \ln(K_{hi}) / (\Sigma h_i)] \qquad (1)$$

where:

K_{hi} = Erodibility index of each rock unit exposed by the headcut (K_h-values in Table 2);

h_i = Thickness of each rock unit exposed by the headcut (also shown in Table 2).

Average values of the headcut erodibility indices for a specific range of degree of weathering were also based on a logarithmic equation:

$$\text{Average } K_h = \exp\,[(\ln K_{min} + \ln K_{max}) / 2] \qquad (2)$$

Table 4 presents the aggregate headcut erodibility indices for all headcuts monitored during the releases.

Table 4. Headcut Erodibility Indices

Head-cut No.	Rock Units	Thickness (feet)	Erodibility Index Min ↓	← w →	Medium ↓	← u →	Max ↓
2	Sallyards / Roca A	6.2	10	17	30	53	95
3	Roca B / Roca C	4.0	7	13	23	41	73
4	Roca D / Roca E…I	15.0	14	26	48	89	160
3+4	Roca B / Roca C…I	19.0			41	75	140
5	Howe / Bennett	7.6	15	26	45	76	130
6	Glenrock / Johnson A	8.1	15	27	47		
7	Johnson B / C…G	18.1	8	14	26		
6+7	Glenrock / Jn A…G	26.2	9	17	31	57	100
5+6+7	Howe / Var. shales	33.8			35	65	120
8	Long Cr. / Hughes A	9.2	210	320	500	760	1200
9	Hughes B / C…F	8.8	47	73	110	180	280
8+9	Long Creek / Hughes	18.0	100	160	240	370	580

Notes: w = average weathered; u = average unweathered; shaded areas = combined headcuts.

Since the beginning of phase III of erosion headcuts Nos. 2 through 9 had been formed. During the erosion process, due to different erosion rates, headcut No. 7 combined with No. 6, forming a new, higher headcut. It is probable that further erosion would result in formation of higher headcuts, as illustrated in Table 4 on the shaded lines. Such a combination of headcuts is possible when in some places a headcut advances upstream at a higher rate than the headcut above it.

MATHEMATICAL MODELLING

The most comprehensive model of headcut advance (phase III erosion) at the time of Tuttle Creek spillway erosion event was that developed by USDA researchers (Temple and Moore, 1994). However, because that model was based mostly on spillways excavated in soil, the USDA model did not fit the data obtained from erosion monitoring at Tuttle Creek. Therefore, a site specific model was developed using Tuttle Creek data as well as a limited number of USDA data for rock spillways. In development of the KCD model the general form of the USDA equation for evaluation of the rate of headcut advance, R, was assumed:

$$R = C\,(A - A_0) \qquad (3)$$

where:

C = parameter function of headcut erodibility index;
$A = (qH)^{1/3}$, parameter expressing the erosive capability of the flowing water;
q = the volume of flow over the headcut per units of width and time;
H = the drop in the energy grade line as the flow passes over the headcut, approximately equal to the headcut height;
A_0 = parameter representing the threshold energy required to generate headcut advance.

Details of KCD model derivation were given elsewhere (Perlea et al, 1997). The obtained relationship is:

$$R = \exp[3.77 - 0.57\,\ln(K_h)] \times [(qH)^{1/3} - 1.19\,(K_h)^{1/2.25}] \qquad (4)$$

where:

R = rate of headcut advance, in feet/day;
K_h = aggregate headcut erodibility index;
q = unit discharge, in cfs/foot;
H = height of headcut, in feet.

The graphical representation of the KCD model is shown in Fig. 4. On the graph are also plotted the location of the points used in calibration.

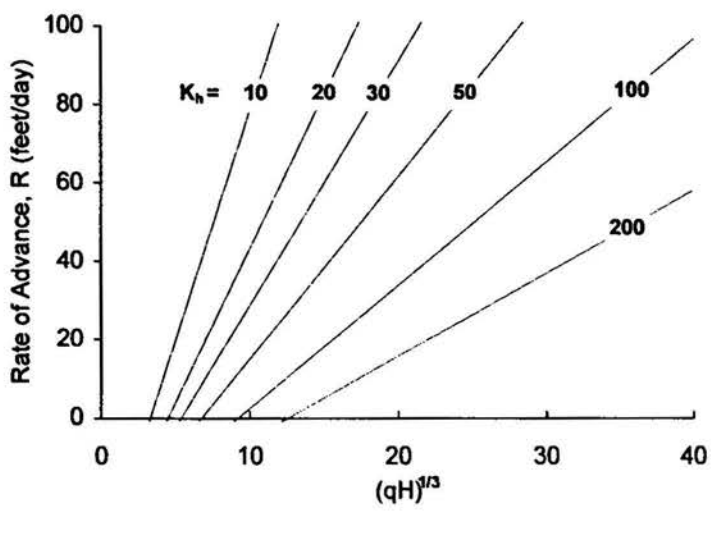

Fig. 4 KCD Headcut Advance Model.

ESTIMATION OF TUTTLE CREEK SPILLWAY BEHAVIOR DURING FUTURE RELEASES

Parametric studies were performed using the KCD model to determine conditions that can lead to failure of the concrete structure, starting from spillway geometry after 1993 erosion:

- Three types of discharge hydrographs: (1) the discharge hydrograph occurred in 1993, with a peak discharge of 60,000 cfs and a duration of 21 days; (2) the design outflow hydrograph, with a peak flow of 612,000 cfs and a duration of 11 days; and (3) constant discharges between 20,000 and 600,000 cfs and the duration needed to induce failure.
- Different longitudinal profiles along the eroded spillway, including a regraded profile to minimize initial concentration of flow.
- Two variants from the point of view of degree of weathering: (1) unweathered rock, condition believed to characterize the headcuts in spillway immediately after the 1993 erosion; (2) weathering to approximately the same extent as existing before the 1993 event, i.e. after several decades of exposure to atmospheric weathering factors.

The following conclusions were made based on observed performance of the unlined spillway during the 1993 flood in conjunction with the KCD erosion model:

- The spillway can likely withstand the spillway design flood (peak discharge of 612,000 cfs) without loss of the ogee structure caused by erosion of the unlined chute. However, it is recognized that significant rock erosion would occur, requiring an immediate repair. It is expected that the Sallyards limestone and Roca Zones A, B, and C would be completely removed leaving a scarp 15 feet in height some 20 to 30 feet downstream of the cut-off wall. The existing Glenrock/Johnson Shale scarp would be expected to combine with the overlying Howe/Bennett Shale scarp to form a headcut 34 ft in height. While this headcut would be considered the most significant with respect to loss of the cut-off wall and subsequently the weir structure, it is not expected to advance to the cut-off wall during a single spillway design event.
- Long duration low flow events may represent more significant risk than the spillway design event. Based on a comparison of total flow volume required for headcut advancement to the ogee structure, it appears that discharges in the range of 30,000 to 50,000 cfs may in fact be more critical than very large discharge events. The erosion model indicates that a continuous flow of 50,000 cfs (same order of magnitude as the 1993 event) could threaten the ogee structure only after a duration of 50 to 70 days, depending on the extent of rock weathering in the unlined chute. It is believed that such an event would be extremely rare.

- The Sallyards limestone, the Roca, Zone B limestone, and the Roca, Zone D limestone were found to be quite erosion resistant during the 1993 event, but were left exposed to weathering. Since all three of these limestone layers are less than 2 feet thick, they were expected to weather full depth, thereby allowing more rapid deterioration of the underlying shales. Erosion of this material will be minimized by restoring some of the protective covering that existed prior to the 1993 erosion.

REPAIR MEASURES

An interim repair was designed and constructed in the period October 1996 through August 1997.

Objective of Interim Fix

Based on the parametric studies it was estimated that failure of the weir structure during future flood events is very unlikely if the current weathering condition of the rock units is preserved and no significant concentration of flow exists at the beginning of an event. Therefore, interim repair work was needed in order to:

- Protect the rock units exposed to weathering. By doing this, the erosion indices of the rock units at the beginning of a future flood event will be approximately equal to the indices experienced at the end of the 1993 releases.
- Prevent concentration of flow at the beginning of a future flood event. With the spillway regraded, initial flows will be evenly distributed across the spillway. Although concentration of flow during subsequent releases will not be prevented, this effort will significantly extend the anticipated useful life of the spillway.

The interim repairs are not designed to provide permanent protection against erosion during spillway releases. However, they are designed to survive significant runoff from precipitation and will provide protection during minor releases. The interim repairs were designed to not preclude construction of a permanent repair, if future studies economically justify a permanent repair. Portions could be incorporated in permanent repairs.

Grading

The spillway was regraded from the flip bucket to the Howe Limestone, a distance of about 900 feet along the spillway. This regrading incorporates a series of 1 vertical on 100 horizontal and 1 vertical on 6 horizontal slopes consisting of cut and fill operations utilizing the existing shale (earth) materials. An average of two feet of earth material was placed over the limestone layers to retard weathering. The plan of regrading is shown on Fig. 5.

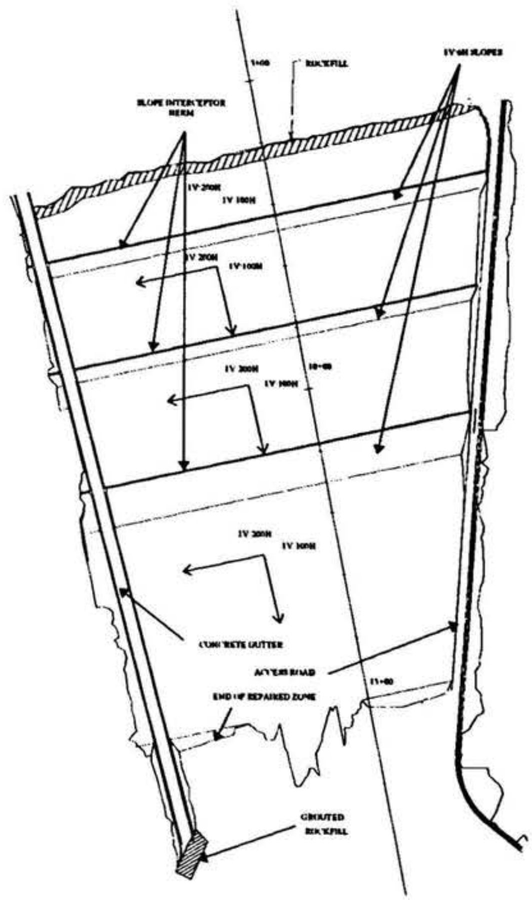

Fig. 5 Regrading plan.

Additional surface grading was necessary to provide drainage for surface water and to prevent at the same time significant erosion of the material protecting rock from weathering. The spillway should experience only minor erosion, easily repaired by regular maintenance, under the following categories of water flow:

- runoff resulting from precipitation on the spillway and adjacent areas, including on the concrete apron (directed through the notch cut in the flip bucket);
- losses through the closed gates when the pool level is higher than the weir crest; the experience showed that the these losses may completely fill the notch and slightly overflow the flip bucket crest.

This objective of surface draining was met by providing a system of slope interceptor ditches with sloping the spillway bottom 1% in direction transverse to spillway axis, parallel to the natural dip of the limestone layers. The flow through the notch and from the interceptor ditches is contained in a concrete paved gutter that discharges the water at a location were erosion is not a concern.

Regrading Material

Four concepts were investigated for interim repairs to the spillway:

1. Regrading with natural shales. This design concept, which was actually adopted, started with filling the major

headcuts with grouted rock. The rock was placed in succesive 2-foot layers covered with high slump concrete that filled practically all voids. The grouted rock zones were intended to prevent early concentration of flow in areas of known weakness. A detail of this concept is shown on Fig. 6.

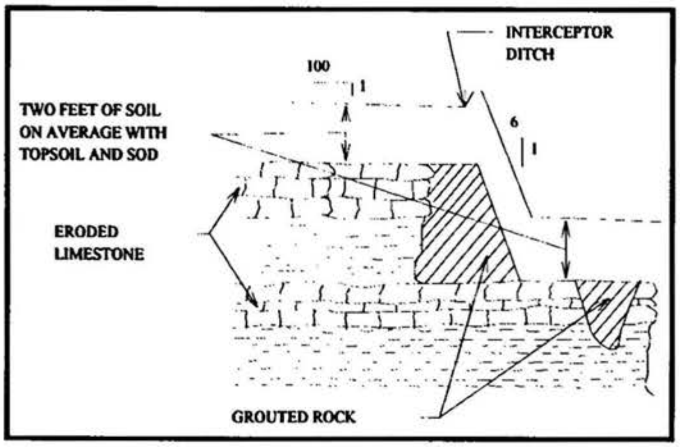

Fig. 6 Profile Through a Grouted Rock Zone.

An average of 2 feet of soil over the rock exposed by erosion is intended to control further weathering. No compaction requirement was required except for the concrete gutter and access road subgrades. Erosion by precipitation water and minor releases is controlled by establishing a sod cover on top of a 6-inch layer of top soil over all the earth material used for regrading.

2. Regrading with lime stabilized shales. Data from literature and pH tests (as recommended in TM 5-822-14 "Soil Stabilization for Pavements" by DOT) suggested that 5 - 7% lime would provide a significant increase in strength of the backfill material. Greater weathering protection and durability of the regraded earth material than the previous option were expected.
3. Regrading with cement stabilized shales. This alternative is similar to the previous two, except that no sod cover is necessary, as the treatment with cement would provide some degree of erosion resistance to runoff and relatively small discharges from reservoir. Data from literature indicated that significant increase in strength (and probably in resistance to erosion also) can be obtained with 9 - 12% cement; however, determination of the optimum percentage of cement to be used would have been required a laboratory test program.
4. Shotcrete application to 1:1 slopes. This design concept is similar to the previous three, in that it consists of filling the major knick points with grouted rock and regrading the spillway on 900 feet downstream from the flip bucket, but the regrading would incorporate 1:1 slopes in the place of 1 (vertical) to 6 (horizontal) slopes. To prevent the erosion of the 1:1 slopes an 8-inch thick layer of shotcrete, anchored into rock, would be applied to these slopes. Erosion of the toe of the 1:1 slopes is, however, very likely, even during small spillway discharges.

CONCLUSIONS

Considerable spillway releases during a flood event in 1993 at Tuttle Creek dam in eastern Kansas provided new valuable data on erodibility of unlined spillways. The KCD model for headcut advance evaluation has been an useful tool in evaluating the risk and designing remedial measures at Tuttle Creek spillway. It is the authors opinion that this model can be used for rough evaluation of spillway erodibility in similar rock conditions. It should be kept in mind that, because of the significant scatter in calibration data, the model was intentionally conceived to overpredict damages.

The repair design concept was not intended to restore the initial spillway condition or to prevent further damages during future discharges. It, however, provides weathering protection to the underlying rock layers. It also provides minimal protection against the concentration of flow during small spillway discharges. The main aspect of the repair was the establishment of a good sod cover, which will protect the slopes from the formation of erosion rivulets due to precipitation runoff, and the restabilization of known weak points with the use of grouted rock.

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