CHAPTER 1

PROJECT AUTHORIZATION

1-01. Construction Authorization. The construction of Tuttle Creek Dam and Reservoir, one unit in the general comprehensive plan for flood control and other purposes in the Missouri River Basin, was authorized by the Flood Control Act of 1938, approved 28 June 1938 (Public Low 761, Seventy-fifth Congress, first session) as modified by the Flood Control Act of 1941, approved 18 August 1941 (Public Law 228, Seventy-seventh Congress, first session) and expanded by the Flood Control Act of 1944, approved 22 December 1944 [Public Law 534, Seventy-eighth Congress, second session (House Document No. 475 and Senate Documents Nos. 191 and 247, Seventy-eighth Congress, second session)].

1-02. Authorized Purposes. The authorized project purposes are: flood control, recreation, fish and wildlife, water quality, water supply, and supplemental releases for navigation on the Missouri River downstream of Kansas City.

a. Water storage capacity of the lake totals over two million acre-feet. Most of this capacity is within the flood control pool, which is generally devoid of water. An estimated additional one million acre-feet of water storage is available in the surcharge pool for use only in critical flood situations.

b. The lake supports a viable fishery, substantial acreage for hiking, bird watching, hunting, horseback riding, and other recreational pursuits. The lake has approximately 112 miles of shoreline at multipurpose pool. Original construction included seven campgrounds and three day use parks offering picnic shelters, picnic tables, camping facilities, and other amenities for the recreating public.

c. Approximately 50,000 acre-feet of water in the multipurpose pool is under contract to the Kansas Water Office to be released from the lake as water supply to downstream users.

CHAPTER 2

PROJECT DESCRIPTION

2-01. General. Tuttle Creek Dam is located on the Big Blue River, 12.3 miles above its confluence with the Kansas River and approximately 6 miles north of Manhattan, Kansas in Riley County. The dam consists of a rolled earthfill and hydraulic fill embankment with a gated, concrete chute spillway on the left abutment and gated twin 20 feet diameter conduits near the right abutment. The pertinent data that characterize the Tuttle Creek Multiple-Purpose Project are presented in what follows:

a. Damsite Location: Pottawatomie and Riley Counties, Kansas, Section 19, R. 8 E., T. 9 S., and Section 24, R. 7 E., T. 9 S., on mile 12.3 of the Big Blue River.

b. Drainage Area Controlled: 9,556 square miles

c. Reservoir Capacity - based on 2000 sediment survey (acre-feet):

Flood control pool	1,871,000
Multipurpose pool (including Sediment storage)	280,000
Total	2,151,000

d. Reservoir Area (acres):

Full pool	53,900
Multipurpose pool	12,500

e. Elevations (feet m.s.l.):

Crest of dam	1159
Spillway weir crest	1116
Multipurpose pool	1075
Full pool (flood-control)	1136
Maximum surcharge pool	1156.85
Outlet invert at intake	1003
Outlet invert at outlet	998.4

f. Dam:

Type	Rolled earthfill
Total length (feet)	7,500
Height above streambed (feet)	156
Height above valley floor (feet)	137

g. Spillway:

Type	Controlled chute
Gates	Eighteen 20(High)x40(Wide) feet
Length (feet)	952

h. Outlet Works:

Conduits	Two in right abutment
Inside diameter (feet)	20
Length (feet)	838
Intake Structure	
Height (feet)	199
Gates	Four 10x20 feet
Stilling Basin	
Depth (feet)	18
Length (feet)	289
Width (feet)	Varies 65 to 160

i. Miscellaneous Data on Record:

Highest pool (feet m.s.l.)	1,137.7 (23 Jul 93)
Maximum discharges (cfs):	
- thru outlet works	
- before dam completion:	29,000 (31 Mar – 10 Apr 60)
- after dam completion:	26,000 (13 Aug 85)
- recent:	16,000 (27 Feb – 2 Mar 01)
- thru spillway	60,000 (23-26 Jul 93)

2.02. Embankment Dam.

a. Construction. Construction of the Project was initiated by the Corps of Engineers in 1952 and closure of the dam was made in July 1959. Storage of water in the reservoir began in March 1962 and the multipurpose elevation was reached in April 1963.

b. Foundation Excavation. All soil material and unsound weathered rock were removed from the foundation of the outlet works. The excavation for embankment foundation was limited to stripping from 1 to 3 feet of topsoil and a 5-foot deep by 75 to 100-foot wide core trench, using bulldozers and scrapers. In the Big Blue River channel area the muck was removed from 300 feet upstream to 600 feet downstream and from 15 to 20 feet of pervious fill placed, which was overlain by a minimum of 18 feet of impervious fill blanket. Areas where the natural (impervious) blanket was thinner were reinforced by a minimum 10 feet of impervious fill under the (upstream) shale-limestone fill.

c. Grouting was performed in rock at the following locations: embankment left abutment; right bank; under conduits; embankment right abutment; spillway right abutment;

spillway left abutment; under weir; horizontal holes through the left and right bulkhead grout wells, including to seal a minor fault. Grouting operations extended from mid 1956 to mid 1960. The average hole was 100 feet in depth, with the total of approximately 85,000 feet drilled.

d. Zoning. Most of the construction materials for the embankment (shale-limestone fills upstream and downstream, berm fill downstream) resulted from excavation for foundation of outlet works and for spillway. The higher quality rock was placed near the slope surface. The central impervious core is composed of select, high shear strength, impervious, natural floodplain blanket silts, obtained from upstream and downstream borrow areas. In order to ensure the permeability of the downstream shell and thus control through-seepage, sand fill was used in a major portion of the downstream shell, including a horizontal drainage blanket. Part of the sand fill was hydraulically deposited and due to an innovative technology at that time, a dense and stable fill was obtained. Both slopes are protected against erosion with rockfill.

e. Seepage Control. The control of foundation seepage is provided by a line of relief wells along the downstream toe, across the valley. There were originally 43 relief wells; these wells with wooden screens were replaced in 1988 - 1990 with 42 new, modern relief wells, with stainless steel screens. The original old wells were abandoned by lining with plastic screens, but they are still effective.

2.03. Site Seismicity.

a. The dam is located in zone 2A on the seismic zone map of the United States (ER 1110-2-1806, Appendix C).

b. A detailed seismologic analysis identified a “hot spot” (Humboldt seismic zone, along the middle part of Humboldt fault) capable of generating an earthquake with a magnitude (Moment Magnitude) of 6.6 at a minimum epicentral distance of 20 km (12.5 miles) from the dam site. The seismic zones in the vicinity of Tuttle Creek Dam are presented in Figure 2.1.

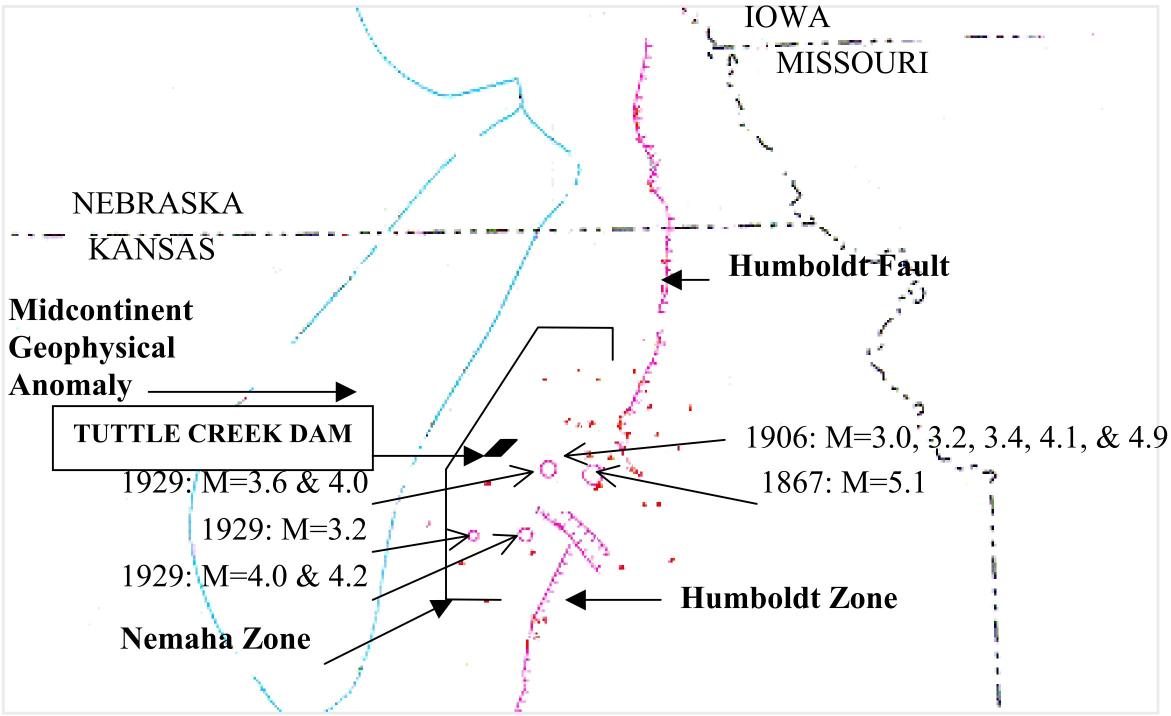

Fig. 2.1. Seismic zones and historic seismic activity in the vicinity of Tuttle Creek Dam.

CHAPTER 3

CURRENT CONDITIONS

3-01. Current Condition of the Project Features.

a. Embankment. Portions of the embankment dam were hydraulically placed; however, the resulting pervious fill was found to be relatively dense and not susceptible to liquefaction if the design earthquake would occur. In the embankment foundation, sands and silts are potentially liquefiable. Silty clays and clayey silts adjacent to liquefiable sands and silts may also loose most of their shear strength under seismic loading. Liquefaction and/or shear strength loss within portions of the foundation soil would induce large deformation and cracking of the embankment that could potentially induce complete failure.

b. Outlet Works. The outlet works is founded on competent rock (Long Creek limestone and firm, unweathered Hughes Creek shale). The intake tower was considered as both “critical” and “non-critical” and was evaluated for the demands of: (1) the maximum credible earthquake (MCE) as represented by a deterministic mean spectrum and (2) the maximum design earthquake (MDE) that is a 975-year event, respectively. The classification of the intake tower as “critical” or “non-critical” will depend on the decision made with respect to embankment dam remediation based on this study. The intake tower, the conduits, the stilling basin walls, and the access bridge are expected to perform adequately under the MDE, with only minor damage to the tower from the MCE. The stilling basin walls and the access bridge, which are “non-critical features, were found not vulnerable to earthquake damage from the MDE event.

c. Spillway.

(1) Bulkheads, Roadway Slab, and Bridge. The bulkhead monoliths, concrete piers, concrete weir, roadway concrete, and other reinforced concrete features of the spillway weir structure are in good condition. Regular maintenance will ensure their integrity and functionality.

(2) Tainter Gates. During the 1993 flood, the only event in the 40-year history of the project when spillway discharges occurred, the gates performed satisfactory under pool loading to the top of the gates. However, it was determined that the structural stability of the gates is marginal when lifting them under full pool loading. Computations assuming PMF occurrence and impared spillway rating curve with two gates out of service indicated dam overtopping by at least one foot. Therefore, reliability of the gates is critical to the integrity of the dam. Further evaluation of gate integrity and reliability will be performed under the Dam Safety Assurance Program (DSAP).

(3) Chute Walls and Slab. The concrete in the walls appear to be in good condition, but the slab is affected by severe D-cracking. Prolonged discharges may result in partial loss of the slab. This loss would not directly affect the safety of the project. Slab replacement, if needed in the future, would be addressed through normal dam maintenance.

(4) Unlined Chute. Discharges during the 1993 flood event induced severe

erosion of the unprotected portion of the spillway chute. Erosion monitoring allowed the development of a mathematical model that demonstrated that the spillway can likely withstand the design flood (peak discharge in excess of 600,000 cfs) without loss of the ogee structure caused by erosion of the unlined chute if the current erodibility potential of the rock is preserved. In this respect, an interim repair of the unlined chute was constructed in 1997, when erosion knick points were backfilled with grouted rockfill and the upper portion of the unlined spillway was re-graded and seeded in order to prevent further degradation of the rock materials in the near vicinity of the concrete structure.

d. Project Features.

The plan of the embankment, typical cross sections, and some construction drawings of the appurtenant structures are included in Appendix XII.

3-02. Justification and Scope of Proposed Modifications.

a. Embankment Seismic Retrofit.

(1) Design Earthquake. In accordance with COE regulations and recommendations of the consulted experts, the design earthquake for the embankment seismic analysis is the 84th percentile of the deterministic Maximum Credible earthquake (MCE), which is characterized by the following pertinent data:

Table 3-1. Design Earthquake for Embankment.

Parameter	Value
Moment Magnitude	6.6
Epicentral Distance	20 km
Focus Depth	10 km
Peak Ground Acceleration, PGA	0.3g
Duration (bracketed at 0.05g)	10+ seconds
Approximate Return Period (evaluated for the PGA)	3,000 years

(2) Threshold Earthquake. The threshold earthquake was defined as the smallest seismic event which can induce liquefaction in free field at any location along the downstream toe of the dam. Liquefaction occurrence was defined by a calculated factor of safety against liquefaction less than one in a layer of at least 15-foot thickness (e.g. $FS_L \leq 1.0$ based on at least 3 adjacent Standard penetration tests). The following pertinent data characterize this event:

Table 3-2. Threshold Earthquake Characteristics.

Parameter	Value
Moment Magnitude	5.7
Epicentral Distance	20 km
Focus Depth	10 km
Peak Ground Acceleration, PGA	0.2g
Duration (bracketed at 0.05g)	4 seconds
Approximate Return Period (evaluated for the PGA)	1,800 years

(3) Embankment Original Design. The embankment was not originally designed for earthquake effects. The only seismic analysis was for the intake tower stability when a static equivalent horizontal force corresponding to an acceleration of 0.1g was considered.

(4) Current Seismic Analyses of the Dam. The occurrence of the MCE, as defined in Table 3-1, was assumed. The critical accelerogram was determined to correspond to the San Fernando, 1971 earthquake, Castaic Ridge record, N69W component. The ordinates of this accelerogram were scaled with the factor 1.107 for the best fit of the corresponding response spectrum with the response spectrum of the design earthquake. Three types of seismic analyses were performed for the Dam Safety Assurance Program:

- Post earthquake limit equilibrium analysis. The earthquake induced shear stresses were determined using the program WESHAKE6. The resistance of the foundation soil against liquefaction was evaluated based on correlations with Standard Penetration Test (SPT). It was considered that the soil liquefies when the computed factor of safety against liquefaction was 1.1 or less. Post liquefaction residual strength was associated with the zones determined to liquefy. Excess pore pressure at the end of the earthquake was evaluated in the zones with partial liquefaction. The post earthquake stability analysis was performed with the program UTEXAS4 using Spencer's method.

  Limit equilibrium analyses were primarily used to evaluate the extent of the problem zones, the location of liquefiable materials, and the necessary extent of soil stabilization or dam/foundation modification for achieving the global factor of safety in excess of 1.2 for any potential failure surface.

- Post earthquake deformation analysis. The zones expected to liquefy or to develop significant excess pore pressure were determined in the same manner as for the limit equilibrium analysis. Two types of deformation analyses were done: using average SPT data and the computer model DYNAFLOW, and worst case conditions along the dam using the program TARA-3. Both analyses showed that significant deformations should be expected, especially of the lower portions of the slopes. Locally, major loss of freeboard and severe fracturing of the entire embankment are likely.

Based on deformation analysis it was determined that stabilization of soil under the lower portions of the slopes can be effective in limiting post earthquake deformations of the embankment to acceptable values.

- Dynamic deformation analysis. A fully coupled effective stress deformation analysis is currently performed using the computer program TARA-3. The purpose of this analysis is mainly to validate the assumption that an acceptable factor of safety for limit equilibrium (1.2 or greater for the remediated dam) ensures acceptable deformations.

(5) The extent of problem zone includes the following portions along the dam, where the post-earthquake stability was evaluated:

Table 3-3. Summary of Limit Equilibrium Computations.

Stations	Factor of Safety for Post Earthquake Condition	
	Failure Mostly Thru Clay	Failure Mostly Thru Sand
Upstream Slope:		
25+00 to 33+00	1.02	N/A
33+00 to 36+00	N/A	1.03
36+00 to 55+00	0.68	0.84
55+00 to 70+00	0.72	N/A
Downstream Slope:		
25+00 to 30+00	1.33	0.85
30+00 to 35+00	N/A	1.01
35+00 to 42+00	0.68	0.60
42+00 to 70+00	0.83	0.81

Figure 3.1 shows the problem zone and the minimum factors of safety. The plan on Figure 3.1 is not to scale; for the plan showing also the appurtenant structures and other related features see Appendix XII.

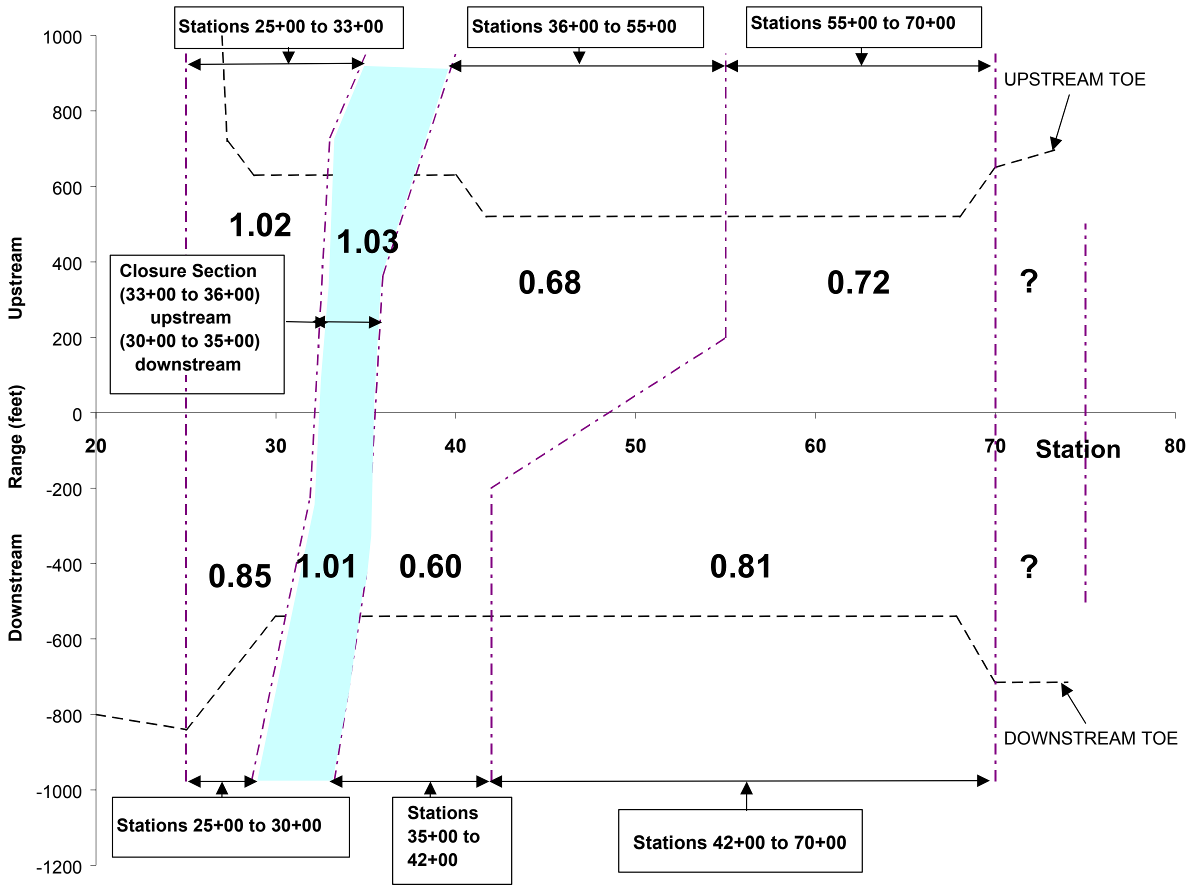

Fig. 3.1. Plan view of dam and minimum factors of safety (large figures in bold) for limit equilibrium after seismic liquefaction.

(6) It is considered that stabilization methods should be applied everywhere the factor of safety for limit equilibrium was found to be less than 1.0. (It is noted that the factor of safety FS = 1.0 was accepted for evaluation of the current condition, as very conservative assumptions were considered; for the design of remediation work based on average soil conditions FS = 1.2 was required.) However, in the first two reaches (closure section and the portion to west of it) there is only one boring in each of the characteristic locations (toes and mid-slopes). Also, there are no borings with SPT data between station 70+00 and the edge of the terrace, station 75+00; therefore, at the current level of investigation it is safe to consider the entire reach, between stations 25+00 and 75+00 (5,000 feet long), a problem zone. The presence of terrace deposits at about station 25+00 should be verified. Additional investigation in Phase III may justify elimination from the problem zone of the portions between stations 25+00 and 36+00 (1,100 feet) upstream and between 25+00 and 35+00 downstream (1,000 feet) and between stations 70+00 and 75+00 for both slopes.

(7) This study concentrated on the primary problem zone, between stations 35+00 and 70+00. For the purpose of cost evaluation it was assumed that the level of effort for

stabilization of the foundation soil in the two zones adjacent to the primary problem zone is about 50% that of the 35+00 to 70+00 reach. Table 3-4 summarizes these assumptions.

Table 3-4. Extent of Problem Zone.

Stabilization Effort	Station Range
As evaluated for the primary problem zone	35+00 to 70+00 (3,500 feet)
Estimated as 50% of the above	25+00 to 35+00 (1,000 feet) and 70+00 to 75+00 (500 feet)

An equivalent length assumed to receive the full amount of stabilization effort, as established for the primary problem zone, is: 3,500 + 0.5 (1,000 + 500) = 4.250 feet. Some construction operations, like for example construction of a working berm, should be applied to the full length of 3,500 + 1,000 + 500 = 5,000 feet.

(8) The depth of the liquefiable sand layer is maximum on each side of the dam (under water in the reservoir and in the open field downstream) and extends with the approximate same thickness under the toe and lower half of the slope. Liquefaction beneath the crest is not anticipated.

Table 3-5. Depth of Liquefiable Sand
in the Reach from Station 35+00 to 70+00.

Slope	Elevation (feet m.s.l.)
Upstream	994 to 1010 (16 feet)
Downstream	990 to 1010 (20 feet)

It is noted that in some cases the soil stabilization is required several feet below the liquefied layer, in order to ensure acceptable factors of safety for deep potential slip surfaces.

(9) It was assumed that the cohesive soil in the foundation blanket is susceptible to large deformations and, consequently, to significant loss of strength if it is in direct contact with the liquefiable sand. Therefore, the blanket between the original ground surface (approximate elevation 1025) and the top of the liquefiable sand (elevation 1010) was considered to require stabilization. This is a 15-foot layer under both the upstream and the downstream slopes.

b. Hydrologic Adequacy.

(1) Introduction. As part of the Tuttle Creek DSAP, the existing inflow design hydrograph (IDH) for this lake was reevaluated and the project performance in response to that IDH was determined. Further, the base safety condition (BSC) for this lake was determined. These evaluations are presented in Appendix III. Also, the required freeboard for this lake was reevaluated using the current criteria. The revised IDH was furnished to CENWD for approval in accordance with the provisions of ER 1110-8-2(FR). The revised IDH was approved by letter of 18 September 2001. The processes used in the study, and details about the findings will be presented in Section 7-01.b. of this report, and the recommended remediation measures will be presented in Section 7-06.

(2) Findings. The hydrologic studies found that a hydrologic deficiency, as defined in EP 1110-2-1155 *Dam Safety Assurance Program* exists at Tuttle Creek Dam. When functioning as designed and authorized, the static lake surface in the lake will infringe on the required freeboard of the dam. Further, the base safety condition for the project was found to be associated with the probable maximum precipitation. The project relies on 18 tainter gates to pass severe flood events without damage to the dam, and there are identified structural deficiencies with those gates. Additional hydrological studies found that the hydrologic deficiency of the project could be seriously exacerbated by gate failure.

(3) Required Modifications. There are two modifications proposed to deal with the identified hydrologic deficiency of Tuttle Creek Dam. They are a.) a short anchored "Jersey Barrier" type concrete wall installed on the upstream shoulder of the dam crest, in place of the existing guard rail, to provide the required freeboard for wind-driven waves, and b.) remedial strengthening of the spillway tainter gates to insure reliable performance during serious flood events.

c. Tainter Gate Reliability.

(1) The design of the Tainter gates did not consider wave loading. Other loads used in the original design do not meet current criteria. A preliminary reanalysis indicated the gate struts are overstressed. Overstressing of the gates could result in failure of one or more gate members. This situation could result in uncontrolled releases through the gate bays or the inability to make controlled releases through the gates. As discussed elsewhere in this report, the loss or impaired ability to make releases from even two of the eighteen Tainter gates would result in the overtopping of the dam during the Probable Maximum Flood. As such, Tainter gate and general spillway modification is considered critical to the safety of the dam and will be addressed under the Dam Safety Assurance Program.

(2) Structural Reinforcement. The exact extent of modification of the Tainter gates is not currently known since detailed remedial design evaluation is not part of the Evaluation Report process. However, other Corps of Engineers Tainter gates have been modified by adding bracing members, adding strut cover plates and modification/replacement of the trunnion pins and bearings. All of these measures are potential solutions that will be considered to strengthen the gate. The trunnion anchorage beams will require evaluation. The anchorage beams consist of embedded members. Corrosion of the beams has been observed

where the beam enters the concrete. Depending on the extent of the corrosion and loss of section of the embedded beams, the trunnion anchorage may require strengthening. In addition to direct modifications to strengthen the gates and possibly the anchorage beams, during December 2001 gate exercises, it was determined that over travel of the gates is limited by the configuration of the gate hanger support bracket, i.e., the gate dogging system. Over travel of the gates is required to ensure the gates can be opened sufficiently to meet the reevaluated spillway discharge. Modification or replacement of gate equipment and gate dogging system may be appropriate to ensure personnel and equipment safety during large discharges. These equipment modifications would be performed concurrent with other gate modifications.

(3) Painting. As part of the structural modifications, the removal of existing paint and repainting of the gates will be required. The extent of paint removal and repainting will be determined during design of the modifications. After consideration of all factors, including hazardous waste requirements, it may be determined that the extent of paint removal and repainting is sufficiently significant to justify repainting of all gate surfaces.

(4) Environmental Considerations. The Tainter gates surface paint and primer has been determined to contain lead of sufficient concentration that Resource Conservation and Recovery Act (RCRA) regulations are likely to be applicable. As such, special paint removal, collection, treatment, and disposal methods are likely to be required. The Tainter gates paint will also be tested for Polychlorinated Byphenols to determine the applicability of Toxic Substances Control Act (TSCA) regulations.

CHAPTER 4

HISTORY OF MAINTENANCE AND REHABILITATION

4-01. General. Since the project completion there were several expenditures for maintenance, as listed in the following sub-chapter (4-02). There were no major rehabilitations or dam safety modifications until present.

4-02. Major Maintenance Activities.

a. Embankment:

(1) Repair of upstream riprap, in 1987-1988 (Contract DACW41-87-C-0081). The work performed included:

- Construction of a rock access road. Quarry-run rock: 17,700 tons.
- Placement of type A riprap on the upstream face of the dam: 10,170 tons.

The expenditures to complete this item were:

• Plans and specs:	$15,400
• Engineering during construction:	$30,800
• Construction:	$250,000
Total:	$296,200

(2) Relief well replacement, in 1989-1991 (Contracts DACW41-89-C-1310 and DACW41-91-C-0003). The work consisted of:

- Installation of 42 relief wells, totaling 2,366 ft.
- Development and pump testing of wells: 860 hours.

The expenditures to complete this item were:

• Plans and specs:	$134,000
• Engineering during construction:	$173,700
• Construction:	$536,200
Total:	$843,900

(3) Lining of old relief wells in 1991 (Hired Labor Contract). There were 43 wooden relief wells to be lined with 4" PVC well screens and the space between liner and the original 8" wood screen and riser filled with gravel pack, sand and grout on top.
The expenditures to complete this item were:

• Plans and specs:	---
• Engineering during construction:	---
• Construction:	$40,400
Total:	$40,400

(4) Long Creek Limestone drain on the left abutment near the downstream toe, installed in three stages during 1950s and restored in 1994 (Hired Labor Contract). The drain is approximately 250 feet long, 100 feet wide, and consists of 5-foot crushed stone layer protected

with either an 18-inch riprap or an impervious layer.
The expenditures to complete this item were:

- Plans and specs: ---
- Engineering during construction: ---
- Construction: $19,100

Total: $19,100

(5) Construction of window drains in the pervious drain, in 1994 (Hired Labor Contract). There were 10 window drains consisting of 25 feet long 6-inch slotted polyethylene pipes surrounded by 30x30-inch crushed stone wrapped in filter fabric, buried within the dam pervious blanket, connected with 6-inch PVC solid pipes crossing the toe road.
The expenditures to complete this item were:

- Plans and specs: ---
- Engineering during construction: ---
- Construction: $27,400

Total: $27,400

b. Outlet Works:

(1) Modification to intake tower: stoplog alterations and cathodic protection, in 1968 (Contract DACW41-68-C-0165). The work performed included:

- J-bulb seals for the stoplogs.
- Cathodic protection for the service gates.

There are no records to document the expenditures for completion of this item.

(2) Gate repair: floor plate, anode installation and oil tank modification, in 1973 (Contract DACW41-73-C-0080). The work performed included:

- Steel plates for the passageway floor to repair cavitation.
- Anodes for the service gate.
- Oil tank modification.

There are no records to document the expenditures for completion of this item.

(3) Installed reinforced concrete slab in basin and built-up roof on intake tower, in 1975 (Contract DACW41-75-C-0035). The work performed included:

- Dewatering of stilling basin and debris removal;
- Existing concrete removal: 1,118 sq. ft.;
- Placement of reinforcement anchors in stilling basin slab: 7,200 anchors;
- Installation of reinforcing steel mat (17,600 lb.) and placement of abrasive resistant concrete overly slab: 606 cu. yd.;
- Removal of drummy concrete in stilling basin divider wall and replacement with new concrete: 7 cu. yd.;
- Installation of chain link fence near upstream end of stilling basin divider wall;
- Replacement of built-up roof on intake tower.

The expenditures to complete this item are not available. The bids for construction (including spillway repair, see item 12) ranged between $213,493 and $570,785; the government estimate for booth items b(3) and c(1) was $315,680.

(4) Gate repair: floor plates, in 1981 (Contract DACW41-80-C-0097). The work performed included installation of steel plates for passageway floor to repair cavitation damage. The expenditures to complete this item were:

• Plans and specs:	$5,700
• Engineering during construction:	N/A
• Construction:	N/A
Total:	$5,700+

(5) Replaced emergency gate guides, in 1998 (Contract No. DACW41-96-C-0088).
The expenditures to complete this item were:

• Plans and specs:	$110,600
• Engineering during construction:	$10,600
• Construction:	$481,300
Total:	$602,500

(6) Replaced wheel tracks and painted gates and equipment, in 2001 (Contract DACW41-00-C-0004).
The expenditures to complete this item were:

• Plans and specs:		$21,700
• Engineering during construction (including S&I):	FY00	$30,000
	FY01	$76,000
	FY02*	$90,000
• Construction:	FY00	$125,000
	FY01	$207,000
	FY02*	$334,000
	Total:	$883,700

c. Spillway:

(1) Removed and replaced concrete and drain pipes in small sections of spillway chute, in 1975 (Contract DACW41-75-C-0035). The work performed included:

- Replacement of the existing flapgate;
- Removal and replacement of concrete within 5 zones: 24.2 cu. yd.;
- Replacement of drainage system embedded in the existing concrete;

The expenditures to complete this item are not available. See information on government estimate and bids for construction at b(3) above.

(2) Removal and replacement of deteriorated reinforced concrete, modification of storm inlets, and concrete joint sealing, in 1976 (Contract DACW41-76-C-0069). The work

performed included:

- Roadway repair. Asphalt coating removal: 42 sq. ft.; spall repair: 90 ft.; roadway, curb, sidewalk, and parapet concrete sawing, removal and replacement: 593 sq. ft.; roadway, curb, and sidewalk joint sawing and sealing: 1195 ft.;
- Bulkhead concrete removal and replacement: 162 cu. ft.;
- Modification of storm inlets: 19 inlets.

The expenditures to complete this item are not available.

(3) Repair of spalled concrete, in 1983 (Hired Labor Contract). The work performed included:

- Removal of spalled concrete, installation of anchors, and placing repair concrete.
- Excavation of slab anchors, testing and evaluation.

The expenditures to complete this item are not available.

(4) Spillway bulkhead and bridge roadway repair, in 1987 (Hired Labor Contract). The work performed included:

- Removal of roadway concrete including curb, sidewalk, and portions of the parapet and bulkheads.
- Replacement of the removed concrete with new reinforced concrete.

The expenditures to complete this item totaled $29,000.

(5) Repair of unlined spillway erosion, in 1996 (Contract DACW41-96-C-0079). The work consisted of:

- Filling the erosion knick points with grouted rockfill: 21,200 tons of rockfill and 5,000 cu. yd. grout;
- Covering of exposed rock with an average 2 feet of soil, grading for uniform slope: 70,000 sq. yd.;
- Topsoil: 11,650 cu. yd.; Seeding and mulching: 14.4 acres;
- Concrete gutter: 1,100 feet long, 25 feet wide;
- Access road: 1,300 feet long, 15 feet wide.
- Fencing: 1,500 linear feet.

The expenditures to complete this item were:

• Plans and specs:		$ 207,000
• Engineering during construction:		$40,900
• Construction:		$1,146,200
	Total:	$1,394,100

CHAPTER 5

PROJECT USE

The project purposes are flood control, recreation, fish and wildlife, water quality, water supply, and supplemental releases for navigation on the Missouri River downstream of Kansas City. The project currently satisfies the authorized purposes and should continue to do so during, and after, evaluation, design, and construction of the remedial modifications, if approved.

At multipurpose pool level, Tuttle Creek Lake is the second largest body of water in Kansas. About 12,000 acres of project lands are licensed to the Kansas Department of Wildlife and Parks for wildlife management. Of the 11 public use areas, 4 are granted to the Kansas Department of Wildlife and Parks for operation as State Parks. In Fiscal Year 2000, the project accumulated more than 2,654,000 visitor-hours of public use. The 1,871,000 acre-feet of flood control storage regulates flows on the Big Blue River, a major contributor to Kansas River floods. Of the 185,000 acre-feet of multipurpose storage allocated for water, 50,000 acre-feet has been marketed to the State of Kansas for municipal and industrial water supply storage. The remaining multipurpose storage supports low streamflow supplementation on the lower Big Blue and Kansas Rivers and navigation on the Missouri and Mississippi Rivers.

The historical flood damages prevented by Tuttle Creek Dam by year are presented in Table 5.1, on the next page.

Table 5.1. Tuttle Creek Lake Historical Flood Damages Prevented

Fiscal Year	Flood Damages Prevented ($1000)
1960-1980	$296,850
1981	$148
1982	$7,854
1983	$26,204
1984	$48,869
1985	$256
1986	$25,761
1987	$69,684
1988	$0
1989	$42,747
1990	$31,911
1991	$1,619
1992	$106,281
1993	$1,250,128
1994	$8,297
1995	$696,783
1996	$280,965
1997	$120,202
1998	$5,705
1999	$881,364
2000	$167
2001	$0
Total	$3,901,795