US Army Corps
of Engineers®
Little Rock District

Draft

Clearwater Dam, Missouri

Clearwater Dam
Major Rehabilitation Report

DRAFT for Review 3/2/04

March 2004

EXECUTIVE SUMMARY

This Report documents the evaluations conducted to support the decision to address the serious seepage problems at Clearwater Dam under the Corp's Major Rehabilitation Program. This Report is prepared in response to guidance received from CESWD and CECW in March 2003 during ongoing efforts under the Dam Safety Assurance Program, and recognizes the need to proceed forward, under the Major Rehabilitation Program, into more detailed analyses and design for the identified alternative. There are several problems being addressed at Clearwater Dam, and the seepage problem and associated risks was deemed, based upon previous coordination and guidance, to be severe enough to move into the Major Rehabilitation category.

Clearwater Dam was completed in 1948 for flood control, recreation, and fish and wildlife conservation. As originally designed, seepage was allowable resulting in reduced construction costs, and the original design estimated that seepage would be relatively minor and of no significant long term effect. Shortly after completion, seepage was detected and in 1950 became more pronounced than expected. Since that time the Corps has attempted to remediate or reduce the seepage problems using various methods, including installation of two under-drainage systems, installation and ongoing monitoring of 73 piezometers, continual O&M studies and technical investigations, dam safety studies, and construction of a seepage control berm at the base of the dam. In spite of these previous efforts, seepage problems have continued and become worse over time. Ongoing maintenance and repair costs and their frequency, to deal with the seepage issue, have and will continue to increase. Recommendations to correct seepage problems in previous reports have been deferred to the regular O&M program over the past twenty five years. The risk of continued deferral includes increasing probability for a major embankment failure.

The primary seepage problem to be addressed is seepage through and under the dam and left abutment that would cause structural instability and potential catastrophic failure at moderate to high lake levels. As recently as January 2003, sinkholes began developing in the upper half of the dam embankment, and subsequent investigations revealed that erosion is occurring inside the dam due to numerous seepage pathways. The potential failure mode involves the ongoing de-stabilization and weakening of the embankment that eventually would result in a major failure during higher lake levels beyond the conservation pool. There are serious concerns at this time related to the next major flood event that may occur, and the heightened potential for failure based upon the recent sinkhole evidence. An embankment failure during a large flood event could cause up to $200,000,000 in damage and up to 391 deaths.

Several alternatives were considered to increase the reliability of the structure to various degrees. The increase in reliability extends the project life resulting in deferred expenditures for replacement. Economic benefits are based on the following:

a) Increased reliability results in reduced future maintenance expenditures, including the reductions in expenditures for repairing a projected failure that has a high probability of occurrence;

b) With the life of the project extended, there is no structure replacement cost in future funding streams; the replacement cost is considered deferred beyond the period of analysis;

c) Extending the life of the project results in additional average annual flood control and recreation benefits over the extended time frame; and

d) With the potential for the high probability of failure reduced or eliminated, there are recreation benefits that would have been lost during the period of time necessary to accomplish the major repairs.

As part of this report, an Environmental Assessment was conducted to ensure compliance with all applicable environmental laws and regulations, including appropriate public and agency coordination. The Environmental Assessment resulted in [TBD].

The Recommended Plan consists of a concrete cutoff wall along the entire length of the dam centerline. The wall extends into bedrock under the dam and through the impervious core of the dam to prevent seepage and migration both through and under the dam. The total depth of the wall would be 230 feet, starting at the top of the dam and with a length of about 4,300 feet. The total estimated cost of the recommended plan is $68.8 million. The average annual cost of the rehabilitation plan is $4.51 million and average annual benefits are $6.16 million for a benefit to cost ratio of 1.37 to 1.

It is noted that separate studies and analyses are currently being conducted under the Corp's Dam Safety program regarding the seismic design of the dam. The solutions proposed for the seepage problem are not expected to conflict with future solutions that may developed to increase the seismic stability of the structure. Additionally, it is not expected that future seismic solutions will attempt to claim additional seepage benefits. Since the two problems, solutions, and benefits are separable, and the locations of the solutions are not in conflict, implementation of seepage solutions will become part of the existing or without project conditions for the seismic Dam Safety study. Although seismic stability problems are being addressed separately under the Dam Safety Program, it is noted that for a weakened embankment combined with high lake levels during a minor seismic event, economic consequences and loss of life may be greater than estimated in this report.

It is also noted that implementation of the recommended plan in this report does not preclude implementation of additional seepage solutions in the future, if, after monitoring the effects of the implemented plan, it is determined that additional measures may be warranted due to the unanticipated underperformance of the implemented solution. As more knowledge is gained in the future, other evaluations can be conducted to determine if additional iterative solutions are feasible.

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APPENDICES

1.0 PROJECT AUTHORITY

1.1 Purpose and Scope

This Major Rehabilitation study was conducted in response to CESWD and CECW guidance received during the ongoing Dam Safety Study for Clearwater Dam. The Dam Safety Study was evaluating three primary problems - spillway design, seismic stability, and seepage. As a result of the severity of the risks the seepage problem was deemed severe enough to warrant being broken out and addressed separately on a stand alone basis under the Major Rehabilitation Program.

This Report documents the evaluations conducted to support a decision to correct the serious seepage problems at Clearwater Dam under the Corp's Major Rehabilitation Program.

1.2 Authorization

Congress originally authorized the Clearwater Dam and Reservoir project for construction in the Flood Control Act of June 1938 (Public Law No. 761, 75th Congress, 3d Session). The basic legislation relating to the development and use of reservoir areas, under the control of the Department of the Army, for recreational and related purposes is contained in Section 4 of the Flood Control Act of 1944 approved 22 December 1944, as amended by Section 4 of the Flood Control Act 1946, and as further amended by Section 209 of the Flood Control Act approved 3 September 1954 (Public Law 780, 83rd Congress). The project purpose is flood control while providing a permanent conservation pool for recreational use and conservation of fish and wildlife.

[Quote Authority]

2.0 PROJECT LOCATION AND DESCRIPTION

2.1 Location

Clearwater Lake is located on the Black River in Wayne and Reynolds Counties in southeast Missouri. The dam is about 43 miles north of the Missouri-Arkansas State line and is 257 river miles upstream from the mouth of the Black River. It is approximately 5 miles upstream of Piedmont, Missouri, the nearest town with a population of 2,200, and is 125 miles southwest of St. Louis, Missouri (Figure2-1).

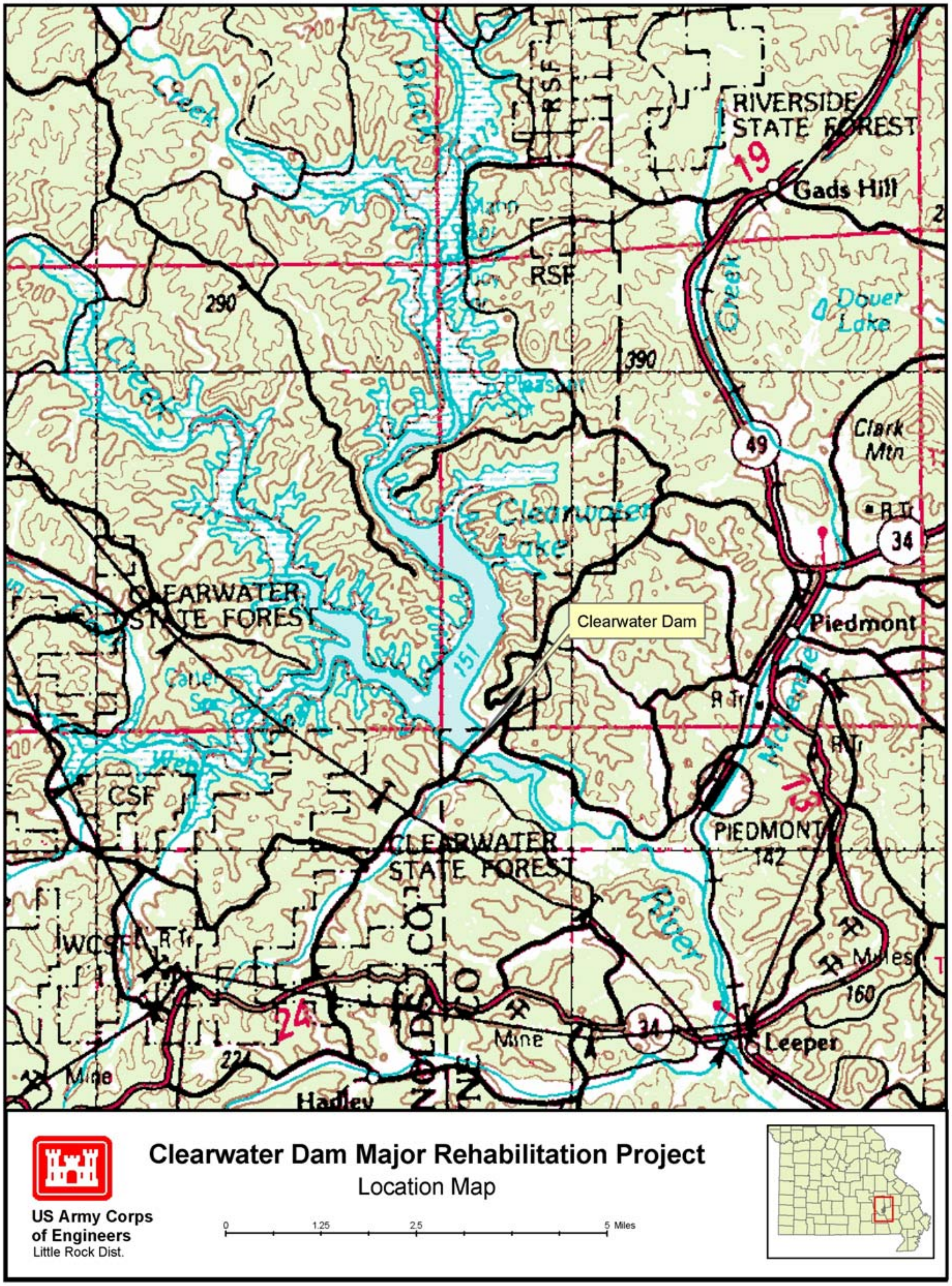
The project is situated in a rural area in the eastern part of the Ozark Plateau in Seismic Zone 2A. The Black River basin contains approximately 8,558 square miles; of that, there are approximately 898 square miles of drainage area upstream of the dam. From the dam site, the Black River flows southeasterly about 35 miles to Poplar Bluff, Missouri (population 17,000); then southerly to the Arkansas-Missouri state line; and southwesterly to the confluence with the White River near Newport, Arkansas.

2.2 Description

Clearwater Lake is one unit of a comprehensive plan for flood control and other purposes in the White River Basin of Arkansas and Missouri and affords protection to the lands in the Black River Valley below the dam. Although the project was authorized and constructed for flood control, a permanent conservation pool with a maximum depth of about 40 feet and a surface area of 1,630 acres is maintained in the area above the dam for recreation and for conservation of fish and wildlife. The use of this permanent conservation pool and the development of areas for recreation constitute an added resource.

The dam is a rolled-fill earth embankment extending across the Black River between the two abutments, has a crest length of approximately 4,225 feet and a maximum height of about 154 feet above the streambed. The dam contains approximately 5,500,000 cubic yards of fill material. The outlet works for releasing impounded waters from the lake consist of a tunnel through the right abutment, an intake structure, a control tower, and a stilling basin and discharge channel. The tunnel, which is circular in section except for the upper and lower transition sections, has an inside diameter of 23 feet and is 1,177 feet long. The stilling basin has a maximum width of 75 feet and is 190 feet long. A total of 37, 600 cubic yards of concrete was used in the construction of the outlet works. The flood control pool is 413,000 acre-feet and the conservation pool is 22,000 acre-feet. The spillway is located in a natural saddle and 1,200 feet from right abutment end of the dam and provides for the passage of 90% PMF flows around the right end of the dam and outlet works.

FIGURE 2-1 PROJECT LOCATION - Missouri



2.3 Pertinent Data

Additional Pertinent Data for Clearwater Dam is provided for reference in the following tables.

Table 2-1. Clearwater Dam Engineering Data	
Crest length of dam, feet	4,225
Height of dam (roadway) above streambed, feet	154
Length of outlet tunnel (including transitions) feet	1,177
Diameter of outlet tunnel, feet	23
Width of spillway, feet	190
Height of roadway above spillway crest, feet	41
Volume of earth in dam, cubic yards	5,500,000
Elevation, top of dam (roadway)	608
Elevation, top of flood-control pool	567
Elevation, top of conservation pool (Oct to May)	494
Elevation, top of conservation pool (May to Oct)	498
Elevation, streambed	454
Storage capacity, top of flood-control pool, acre feet	413,000
Storage capacity, top of conservation pool, acre feet	22,000
Area, top of flood-control pool, acres	10,300
Area, top of conservation pool, acres	1,630
Length, top of flood-control pool, miles	172
Length, top of conservation pool, elevation 494, miles	27
Crest length of dam, feet	4,225
Height of dam (roadway) above streambed, feet	154

Table 2-2. Pertinent Data for Clearwater Lake				
			Lake Storage Volume	
Point of Interest	Elevation	Lake Area	Incremental	Cumulative
	(ft, NGVD)	(acres)	(acre feet)	(acre feet)
Top of Parapet Wall	611			
Top of Dam	608			
Maximum Water Surface	610.9*	18,495	577,470	990,470
Reservoir Easement Area Contour	572	11,210		468,000
Top of Flood Control Pool	567	10,400	391,000	413,000
Top of Conservation Pool	494	1,630	21,920	22,000
Invert of Conduit Gate	467			
Streambed at the Dam	454			
* 90% PMF				

Table 2-3. Pertinent Data for Clearwater Dam		
Location	River Mile 257.4 on the Black River, Wayne County, Missouri	
	Approximately 4.5 miles west of Piedmont, MO	
Drainage Area	898 square miles	
	One inch of runoff = 47,890 acre-feet of storage volume	
Dam	Type: Earthen	
	Top of Parapet Wall, feet NGVD	611
	Top of Dam, feet NGVD	608
	Streambed, feet NGVD	454
	Height, above streambed, feet	154
	Crest Length, feet	4,225
Spillway	Type: Side Saddle Uncontrolled	
	Crest Elevation, feet NGVD	567
	Net Length, feet*	470
	Discharge** @ Elevation 574 in CFS	18,000
	@ Elevation 602.5 in CFS	225,000
	Design Discharge, CFS	280,000
	Design Flood Volume, acre-feet	840,000
	Design Flood Volume, inches	17.56
* With spillway excavated to design width		
** At present width after excavation in 1989		
Outlet Works	Type: Concrete Lined Tunnel	
	Diameter, feet	23
	Length, feet (including transition sections)	1,177
	Gates, number/size	3--9’ x 20’
	Discharge @Elevation 567 in CFS	25,000
Construction Dates	Initiated:	1940
	Completed:	1948

3.0 PROBLEM IDENTIFICATION

3.1 Problem Summary

As the dam is currently configured, there is a highly permeable section inside the structure where seepage flows have historically concentrated. This section was originally allowed as part of construction of the dam as a cost saving measure since the associated seepage was not thought at that time to pose a significant problem. However, as early as 2 years after the dam was completed, seepage flows exceeding what was expected were detected. Figure 3-1 shows a depiction of this original design.

Of particular concern was the overburden contact and the section or “window” area which is the space between the right end of the upstream left abutment impervious blanket, and the left end of the embankment core. Since about 1978, the downstream left abutment area has experienced observable seepage at pool elevations above about 530. Various reports and studies concluded the seepage condition gradually worsened up to 1986 and have continued to worsen to the present, including development of sinkholes in January 2003.

Since the initial impoundment of water in 1948, for lake levels exceeding about elevation 510, surface seepage has been documented as occurring along the contact of and downstream of the left abutment ridge which forms the left end of the dam embankment From a comprehensive analysis of seepage, the seepage path was concluded to probably occur through the upper layers of the limestone rock along the entire left abutment ridge.

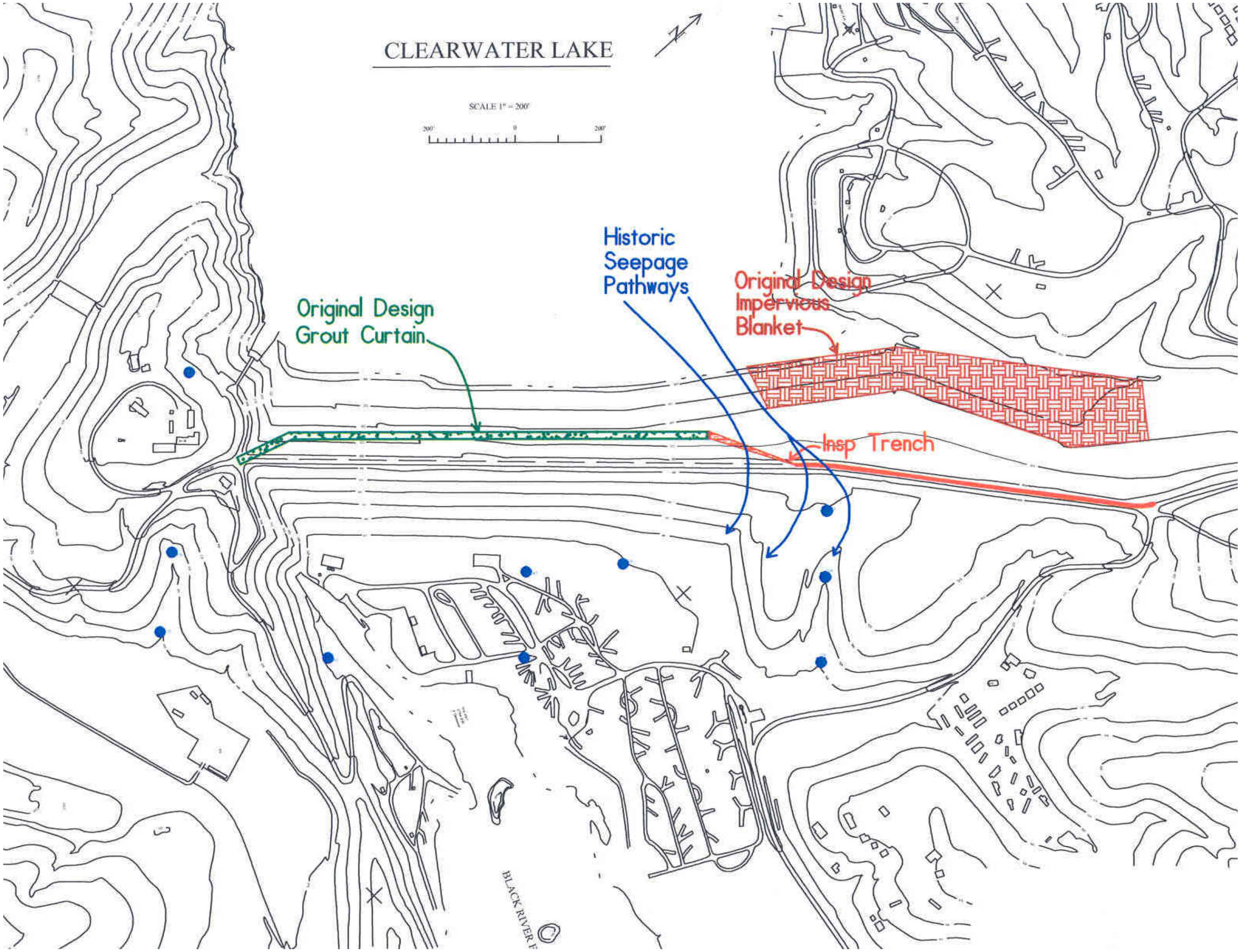
The scope of the seepage problem is flow beneath and through the dam that exits along the downstream left abutment along flow lines mainly through the section in question. The site is underlain with alluvial and residual deposits overlying bedrock that is known to be highly fractured and prone to substantial weathering. During every major flood event there continues to be noticeable quantities of excessive seepage observed downstream of the left abutment. There is internal seepage and piping that occurs during higher lake levels, and there are verified concerns that every high water event is causing additional incremental and cumulative damages. During large flood events or other conditions leading to high lake levels, the existing seepage protection, an impervious clay blanket partially covering the embankment about 2/3 of the way to the top, is overtopped and water is allowed to flow unimpeded throughout the internal shell of the dam, thoroughly saturating the core and causing steady state seepage.

In the projected failure scenario, the interior of the embankment will become saturated during a time period of approximately 48 hours after the estimated flood or other high water event has commenced. Steady state internal seepage would begin to occur throughout and around the dam’s left abutment immediately following thorough saturation of the internal dam structure. After approximately 36 hours, the seepage becomes evident from traditional downstream discharges from the downstream toe and from the base of the left abutment ridge.

After the commencement of a high water event and steady state seepage has been evident downstream for a period of time, discoloration of the discharge will begin to occur.

During the next estimated 11 hours, extensive muddy water will be observed at the discharge areas confirming that internal piping and erosion is taking place. This could lead to the appearance of large visible sinkholes. During the next hour (48 hours after saturation occurs), it is projected that the embankment would begin to settle and rapidly breach. This is thought to primarily occur at the left abutment. An ultimate breach opening (caused by the highest lake level) would measure approximately 100 feet wide at the base, 130 feet high and 360 feet long at the crest, with 1 vertical to 1 horizontal side slopes. This is in contrast to a dam height of 154 feet above the river bed.

FIGURE 3-1 ORIGINAL DESIGN



4.0 PROJECT HISTORY AND SUMMARY OF PREVIOUS WORK

4.1 Original Design Philosophy

The original design allowed the grout curtain to end short of the left abutment and the impervious blanket to extend only along the left abutment. The grout curtain was not intended to completely eliminate seepage through underlying material but instead to limit any seepage. A section of the dam was left partially unprotected against seepage (the "window"). The designers and consultants concluded that seepage paths would be limited to the existing bedrock and overlying residual materials, but the dam could be constructed to remain safe. They also concluded that the amount of seepage could not be predicted prior to impoundment, and that it would be excessively costly to eliminate "all theoretically possible seepage losses now". Construction proceeded with the intention to treat areas of excessive seepage when and where they developed in the future.

4.2 Maintenance & Monitoring History

As ongoing remedial measures, numerous piezometers were added to monitor seepage, and two underdrainage systems were installed in 1972 and 1980 to relieve uplift pressures, lower the phreatic line, and prevent under seepage from emerging above ground along the downstream left abutment of the dam, and in the River Road Park. In addition a seepage berm was constructed to lengthen seepage flow paths. These measures and their limited effectiveness are discussed in additional detail below.

Ten piezometers were installed in 1949 soon after completion of the dam to monitor groundwater levels and/or reservoir-induced seepage. Sixty-three additional piezometers were installed between 1972 and 1979 as a result of increased seepage. From 1980 to 1981, a number of studies were conducted to investigate the continuing seepage problem. An August 1981 comprehensive report documented these studies

Two underdrainage systems have been installed at Clearwater since the dam was constructed. The systems were installed in 1972 and 1980 to relieve uplift pressures and prevent underseepage from emerging above ground along the downstream left abutment of the dam, and in the River Road Park.

1972 System. The system consisted of lateral trenches filled with gravel around a perforated pipe to carry flow to the main line, which was non-perforated. V-notch weirs were installed in the first three manholes in order that flow measurements could be made. The system was effective in reducing the above-ground leakage, to some extent, at fairly low pools, elevation 525-530; however, during the high pools of 1979, when the lake level reached elevation 550.48, the system was highly inadequate, even though the total flow to the system was as much as 300 gpm.

1980 System. During the high pools of 1979, considerable seepage emerged along the toe of the downstream left abutment, through the fill under Highway HH, and into the River Road Park; therefore, a second underdrainage system was installed. The system consists of a perforated pipe, in a gravel filled trench excavated to rock, along the toe of the downstream left abutment. Part of the system was extended along Highway HH to intercept seepage under the roadway fill, and eliminate ponding of water in River Road Park.

4.2.1 Seepage Berm Construction

The earth berm was constructed to control seepage through the left abutment and under the dam. This seepage berm materialized as a result of conclusions drawn from the Comprehensive Seepage Analysis and Report of 1981and subsequent documents. Construction began in October 1987 and was essentially completed in December 1988. The project consisted of constructing an earth seepage berm upstream of the dam along the full length. Borrow material from the spillway was used for constructing the berm, which incidentally allowed the spillway to be widened to accommodate the probable maximum flood spillway discharge. The work also included foundation grouting of the right abutment to help control seepage in that area. A parapet wall was constructed along the top of the dam to control wave action.

4.2.2 Seepage Condition Before and After Construction of Seepage Berm

Seepage Condition Prior to Construction of the Seepage Berm

Piezometer Activity. In the left abutment, seepage occurred during both flood pool conditions and, to a lesser degree, conservation pool conditions. Seepage through the window area was confirmed by high piezometric levels. Seepage was occurring through the entire left abutment. During heavy rainfall periods, seepage flows were surcharged further by surface flow infiltration. The piezometers showed a moderate rise up to when the pool elevation reaches 510. When the pool elevation exceeds 510, the piezometers slightly reflected a reservoir connection and seepage increases. Surface seepage historically occured when the pool elevation reaches 530.

Weir Flows. During normal (conservation) pool, the flows discharged either into the underdrainage collector system installed in 1972, or through the subsurface rock. During moderate pools (up to elevation 522), some of the flow was intercepted by the underdrainage system installed in 1980. The 1980 system appeared to have intercepted the seepage which had surfaced at the toe of the left abutment during previous flood pools over elevation 510. Since the installation of the 1980 system, surface seepage usually occurs at elevation 530. The 1972 underdrainage collector system flows continuously, except during extreme drought conditions. Flow analysis of the two underdrainage systems indicates the 1980 system intercepts seepage flow that formerly went into the 1972 system at the higher pool levels.

Seepage Condition After Construction of the Seepage Berm

Piezometer Activity. Very few lake levels above elevation 510, have been experienced since completion of the seepage berm; therefore data on the piezometric activity is limited. At a pool elevation of 512, the piezometric levels ranged from 0-15 feet less than the levels prior to construction. The areas for the most drop in piezometric levels were in the left abutment.

Piezometric levels in the valley and right abutment were similar to piezometric levels prior to construction (very low).

Weir Flows. For the 1972 underdrainage system, the flows were less than the flows prior to construction. Prior to construction with a pool level of 510, the flows ranged from approximately 0 to 70 GPM. After construction for a corresponding pool level, the flows ranged from 0 to 20 GPM. Also it appears that lag time has increased since the seepage control construction.

For the 1980 underdrainage system, the flows are similar to the flows prior to construction. For pool levels of elevation 500 and less, the flows have changed very little. For levels above elevation 500, the flows seem to have increased for a given pool. During a period in early 1990, the flow reacted erratically probably due to intense rainfall. It appears that the 1972 system developed a period of lag time before the system began to react. Before construction, the flow reacted simultaneously with the fluctuation of the reservoir. Therefore in this area, the velocity of the seepage might have decreased.

It should be noted that, the reservoir level has only reached elevation 526 once since the seepage control alterations were constructed; therefore, the seepage control measures can't be fully evaluated.

As recently as January 2003, sinkholes have begun forming on the upstream face of the dam. The cause is thought to be directly related to the seepage problems.

4.3 Summary of Historic Maintenance Costs

The expenditure record of Operation and Maintenance (O&M) of Clearwater Lake is shown as Table 4-1. The average annual O&M for the past 5 years is $1,689,000. However, this figure includes all O&M. Estimates were made as to what percentage of total O&M expenditures were attributable to the seepage problem, which varies by year. In addition, other work previously performed dealing with the seepage issue and the costs of that work were assessed. Seepage related O&M expenditure estimates for the with and without project condition are described and evaluated in the Economic Appendix A.

Table 4-1. Clearwater Project Historical O&M Annual Expenditures

Fiscal Year	Expenditure[1] (1,000's)	Fiscal Year	Expenditure[1] (1,000's)	Fiscal Year	Expenditure[1] (1,000's)
1975	157	1985	1,367	1995	1,930
1976	260	1986	1,391	1996	1,828
1977	923	1987	1,536	1997	2,002
1978	841	1988	1,482	1998	2,193
1979	1,247	1989	1,654	1999	2,629
1980	1,863	1990	1,233	2000	2,825
1981	2,404	1991	1,806	2001	4,090
1982	1,250	1992	1,764	Total	45,610
1983	1,526	1993	1,916		
1984	1,532	1994	1,961	Average for 1975 to 2001	1,689

[1]Price level varies

4.4 Previous Reports

a. Clearwater Dam Foundation Completion Report, September 1977.
b. Dam Safety Assurance Program, Clearwater Dam, August 1979.
c. Clearwater Dam Left Abutment Seepage Study, Vol. 1, November 1979.
d. Clearwater Dam Comprehensive Seepage Analysis and Report 1949 to 1981, Vol. I, August 1981.
e. Clearwater Dam Comprehensive Seepage Analysis and Report 1949 to 1981, Vol. II, August 1981.
f. Project Modification for the Dam Safety Assurance Program, Feature Design Memorandum No. 1, September 1983.
g. Project Modification for the Dam Safety Assurance Program, Feature Design Memorandum No. 1, Supplement No. 1: Remedial Plan for Reducing Seepage Problems, September 1983.
h. Project Modification for the Dam Safety Assurance Program, Feature Design Memorandum No. 1, July 1984.
i. Project Modification for the Dam Safety Assurance Program, Reconnaissance Report, Revised May 1986.
j. Clearwater Dam Grouting Completion Report, July 1989.
k. Summary Report for Seepage Berm Construction and Assessment of Seepage, May 1991.
l. Clearwater Lake Spillway Erodibility Study Draft Report, March 1998.

5.0 ENGINEERING CONSIDERATIONS

5.1 Hydrologic and Hydraulic Analyses

Engineering estimates of conditions that could be expected to result in a seepage induced dam breach were based upon a sliding scale of duration and pool elevations. A seepage related failure might occur as a result of low pool elevations of relatively long duration or higher pool elevations of relatively short duration. Pool elevation 505 is equaled or exceeded about 9.3% of the time on an average annual basis and was selected for the low pool elevation breach analysis. For higher pool elevations of relatively short duration it was estimated a dam breach would begin to form if the pool exceeded elevation 575 which is the top of the existing impervious blanket on the upstream face of the dam.

Based upon the pool elevation-frequency curve and the associated routings of the outflow hydrographs, flood elevations downstream are determined for the existing condition (no failure) and the dam failure condition. These differences in flood surface profiles form the basis of the with and without project conditions for economic analysis. The without project condition is discussed in additional detail in Section 8.5 of this report.

Hydrologic and hydraulic analyses were performed for several hydrologic events that might result in a seepage related dam breach – pool at elevation 505.0 feet; 20% PMF resulting in peak pool elevation 581.4 feet with pool receded to elevation 574.7 feet at time of breach; and 100% PMF resulting in peak pool elevation 611.2 feet with pool receded to elevation 593.0 feet at time of breach. The HMR-51 based inflow design flood developed in 1979 for Clearwater Lake was used directly for the 100% PMF breach analysis and scaled for the 20% PMF breach analysis. Time of breach for the 20% and 100% PMF simulations was based on 95 hours of resident time of the pool above EL 575.0 FT (top of seepage berm) prior to beginning of the breach. A base flow of 700 cfs into the reservoir was assumed for the pool at elevation 505.0 feet breach analysis. It is expected for pool elevations below about elevation 505.0 feet there would be enough warning time to avoid a dam failure by evacuating the pool to a safe level before a breach formed. Failure and non-failure analyses were performed for each event. Failure verses non-failure results were compared to determine the difference in flood heights and flood wave arrival time at various points of interest downstream of the dam.

Details of the H&H analyses are presented in Appendix B. Pertinent results of the analyses are summarized below in Tables 5-1, 5-2, 5-3, and 5-4, which are also contained in Appendix B.

TABLE 5-1
Maximum Increase in Flood Heights [FT]: Dam Failure vs. Existing Condition W/O Failure

Distance From Dam [mi]	Location Description	Dam Breach w/ Pool At EL 505 FT	Dam Breach w/ Pool At EL 574.7 FT (20% PMF)	Dam Breach w/ Pool At EL 593.0 FT (100% PMF)
0.8	Vicinity of Dam	7.1	24.1	16.5
4.9	Piedmont/McKenzie Creek	10.3	27.7	15.5
7.0	Hwy 34	9.8	21.7	12.2
7.8	Leeper	9.8	19.5	10.8
9.5	Hwy 49 at Mill Springs	9.2	20.8	11.4
19.2	Hwy 49 at Browns Crossing	10.5	12.2	7.2
22.5	Williamsville	8.7	18.6	7.9
30.0	Hwy 67 near Hendrickson	12.1	20.5	6.2
31.0	Hendrickson	11.0	17.3	5.9
39.6	Hwy W at Hilliard	9.7	13.6	4.3
44.4	Hwy 60 Bypass near Poplar Bluff	7.7	9.8	2.2
45.7	Begin Inter-River Levee at Poplar Bluff (EL +/-343 FT)	7.3	5.0	1.6
49.2	Bus. Hwy 60 at Poplar Bluff	9.0	4.9	1.1
49.4	MO-PAC RR at Poplar Bluff	8.1	3.4	0.7
51.0	South Poplar Bluff	7.3	1.0	0.8
	Pond Area 1 - Northeast of Poplar Bluff	0.0	5.4	0.6
	Pond Area 2 - East of Poplar Bluff	0.0	4.7	0.6

TABLE 5-2
20% PMF Seepage Failure Dam Breach Simulation
Peak Flood Elevations and Times to Peak at Locations below Clearwater Dam

Distance From Dam [mi]	Location	Without Failure			With Failure		
		Elapsed Time[1] [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]	Elapsed Time[2] [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]
0.8	Vicinity of Dam	44.5	468.0	52,724	3.6	492.1	494,205
4.9	Piedmont/McKenzie Creek	45.0	456.6	54,166	3.9	484.3	414,436
7.0	Hwy 34	45.5	445.9	54,111	4.5	467.6	411,843
7.8	Leeper	45.5	442.9	54,092	4.8	462.4	410,303
9.5	Hwy 49 at Mill Springs	46.5	435.1	54,012	5.5	455.9	401,039
19.2	Hwy 49 at Browns Crossing	47.5	403.7	69,472	8.6	415.9	368,265
22.5	Williamsville	49.0	390.1	67,974	9.7	408.7	351,977
30.0	Hwy 67 near Hendrickson	52.0	369.9	62,905	10.7	390.4	337,011
31.0	Hendrickson	52.0	367.3	62,571	11.6	384.6	335,953
39.6	Hwy W at Hilliard	55.0	354.4	69,058	14.3	368.0	300,976
44.4	Hwy 60 Bypass near Poplar Bluff	56.5	345.3	66,436	14.9	355.1	298,002
45.7	Begin Inter-River Levee at Poplar Bluff (EL +/-343 FT)	56.5	343.9	64,810	15.2	348.9	297,581
49.2	Bus. Hwy 60 at Poplar Bluff	56.5	342.2	56,436	15.4	347.1	89,102
49.4	MO-PAC RR at Poplar Bluff	57.5	337.4	56,422	15.9	340.8	78,789
51.0	South Poplar Bluff	58.5	332.5	56,389	17.4	333.5	78,277
	Pond Area 1 - Northeast of Poplar Bluff	66.0	323.2	8,196*	20.0	328.6	218,213*
	Pond Area 2 - East of Poplar Bluff	66.0	315.7	3,508*	22.0	320.4	174,839*

1 - From beginning of PMF spillway release to time of maximum water surface elevation.
2 - From beginning of dam breach to time of maximum water surface elevation.
* Peak inflow to pond area. Inflow to Pond Area 1 is from Black River overtopping Inter-River Levee. Inflow to Pond Area 2 is from Pond Area 1 across MO-PAC RR.

TABLE 5-3
100% PMF Seepage Failure Dam Breach Simulation
Peak Flood Elevations and Times to Peak at Locations below Clearwater Dam

Distance From Dam [mi]	Location	Without Failure			With Failure		
		Elapsed Time[1] [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]	Elapsed Time[2] [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]
0.8	Vicinity of Dam	49.5	487.1	315,684	2.9	503.6	788,792
4.9	Piedmont/McKenzie Creek	49.5	479.7	326,748	3.1	495.2	662,977
7.0	Hwy 34	50.0	464.2	326,528	3.6	476.4	657,808
7.8	Leeper	50.0	459.6	326,391	4.0	470.4	654,793
9.5	Hwy 49 at Mill Springs	50.5	453.5	325,963	4.5	464.9	636,864
19.2	Hwy 49 at Browns Crossing	51.0	416.9	374,947	7.0	424.1	591,638
22.5	Williamsville	51.5	410.3	373,639	7.5	418.2	567,316
30.0	Hwy 67 near Hendrickson	53.0	392.2	371,183	8.8	398.4	541,320
31.0	Hendrickson	53.5	387.1	370,981	9.2	393.0	539,701
39.6	Hwy W at Hilliard	55.0	371.6	394,848	10.8	375.9	528,658
44.4	Hwy 60 Bypass near Poplar Bluff	55.5	357.1	394,329	11.2	359.3	526,521
45.7	Begin Inter-River Levee at Poplar Bluff (EL +/-343 FT)	55.5	350.4	394,241	11.3	352.0	526,406
49.2	Bus. Hwy 60 at Poplar Bluff	55.5	348.5	88,128	11.1	349.6	119,998
49.4	MO-PAC RR at Poplar Bluff	56.0	341.0	79,841	11.6	341.7	101,919
51.0	South Poplar Bluff	56.5	333.5	79,835	12.4	334.3	100,345
	Pond Area 1 - Northeast of Poplar Bluff	54.0	329.7	314,424*	14.0	330.3	424,455*
	Pond Area 2 - East of Poplar Bluff	57.0	321.7	311,532*	15.0	322.3	411,672*

1 - From beginning of PMF spillway release to time of maximum water surface elevation.
2 - From beginning of dam breach to time of maximum water surface elevation.
* Peak inflow to pond area. Inflow to Pond Area 1 is from Black River overtopping Inter-River Levee. Inflow to Pond Area 2 is from Pond Area 1 across MO-PAC RR.

TABLE 5-4
Pool at EL 505.0 FT Seepage Failure Dam Breach Simulation
Peak Flood Elevations and Times to Peak at Locations below Clearwater Dam

Distance From Dam [mi]	Location	Without Failure			With Failure		
			WSEL [ft,NGVD]	Q [cfs]	Elapsed Time[1] [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]
0.8	Vicinity of Dam	-	459.9	4,500	2.8	467.0	45,134
4.9	Piedmont/McKenzie Creek	-	443.6	4,500	5.1	453.9	38,293
7.0	Hwy 34	-	434.0	4,500	6.1	443.8	37,449
7.8	Leeper	-	431.1	4,500	6.4	440.9	36,966
9.5	Hwy 49 at Mill Springs	-	423.4	4,500	7.6	432.6	36,096
19.2	Hwy 49 at Browns Crossing	-	388.2	4,500	13.1	398.7	29,953
22.5	Williamsville	-	377.1	4,500	15.5	385.8	29,003
30.0	Hwy 67 near Hendrickson	-	351.4	4,500	19.9	363.5	26,396
31.0	Hendrickson	-	348.7	4,500	20.2	359.7	26,292
39.6	Hwy W at Hilliard	-	337.3	4,500	26.3	347.0	22,157
44.4	Hwy 60 Bypass near Poplar Bluff	-	331.5	4,500	29.4	339.2	21,129
45.7	Begin Inter-River Levee at Poplar Bluff (EL +/-343 FT)	-	330.6	4,500	30.2	337.9	20,973
49.2	Bus. Hwy 60 at Poplar Bluff	-	324.8	4,500	31.9	333.8	20,325
49.4	MO-PAC RR at Poplar Bluff	-	323.8	4,500	32.3	331.9	20,264
51.0	South Poplar Bluff	-	321.4	4,500	34.4	328.6	19,765
	Pond Area 1 - Northeast of Poplar Bluff	-	N/A	N/A	N/A	N/A	N/A
	Pond Area 2 - East of Poplar Bluff	-	N/A	N/A	N/A	N/A	N/A

1 - From beginning of dam breach to time of maximum water surface elevation. Not applicable for without failure controlled release.
N/A – Black River did not overtop the Inter-River Levee.

5.2 Geotechnical

Based upon the body of geotechnical work to date, including a review by an expert consultant (Poulos), the locations where seepage may not be adequately controlled are listed as follows:

1) Through the bedrock beneath the entire dam,
2) Through the alluvium above the bedrock in the valley segment of the dam,
3) Through the sloping clay core of the dam,
4) Through the core trench,
5) Under and around the left ridge segment founded upon residual soils,
6) Through the resicual soils between the valley and left rige segments,
7) Through the seepage berm and underlaying riprap and alluvium,
8) Along the right abutment contact face.

In January 2003, a 10 foot wide and 10 foot deep sinkhole formed on the upstream face of the dam. The sinkhole occurred following the record high reservoir level experienced in May 2002. The reservoir drained slowly at the time; it took about two weeks to recede to elevation 560 (the sinkhole formed at about elevation 570 and the pool had reached elevation 568), and it took about 2-1/2 months to recede back to normal levels. The Geotechnical expert's (Poulos) opinion is "that the immediate cause of the sinkhole was the flow of water at high reservoir levels under gradients high enough to cause piping of the shell and natural alluvial material into an open joint or solution channel in the bedrock" and that "although the high reservoir levels of the previous year may have been the immediate cause of the sinkhole, it is likely that continuous flow of water through the alluvium at all reservoir levels had caused some piping at various times in the past. The sinkhole probably is the end result of long-term, intermittent piping and more intense piping when the reservoir has been high".

Dr. Poulos provided additional conclusions and recommendations. The full report is included with Appendix F, Geothechnical. A key recommendation was:

"To control seepage through this dam, select and construct a remedial seepage barrier throughout the length of the dam *as soon as practicable.* (Emphasis added).

6.0 ENVIRONMENTAL CONSIDERATIONS

An Environmental Assessment (EA) has been performed as part of this Major Rehabilitation study. The complete EA with Findings is included as Appendix C to this report. None of the alternatives are expected to result in significant adverse environmental impacts.

Note to Reviewer: The draft EA is included for review. After Corps ITR, comments will be addressed prior to coordination and release of the EA for a 30 day public comment period.

7.0 ECONOMIC CONSIDERATIONS

7.1 Federal Interest

The increase in reliability extends the project life resulting in deferred expenditures for replacement. Economic benefits are based on the following:

a) Increased reliability results in reduced future maintenance expenditures, including the reductions in expenditures for repairing a projected failure that has a high probability of occurrence:
b) With the life of the project extended, there is no structure replacement cost in future funding streams; the replacement cost is considered deferred beyond the period of analysis;
c) Extending the life of the project results in additional average annual flood control and recreation benefits over the extended time frame;
d) With the potential for the high probability failure reduced or eliminated, there are recreation benefits that would have been lost during the period of time necessary to accomplish the major repairs. Flood control benefits would not be lost after a major failure since there would be none.

A probabilistic, risk based economic analysis was performed in conducting this Major Rehabilitation Evaluation for Clearwater Dam. Event trees were constructed in order to model the possibilities of occurrences of the dam, given its current condition, and their economic consequences. Life cycle analyses were used to consider the impact of these consequences over the 50-year period of analysis of this evaluation.

Major Rehabilitation guidance requires estimation of total economic costs and benefits of the base condition and alternative solutions. Guidance also requires identification of the recommended plan. The recommended plan will identify the optimum investment, both in terms of proposed actions and timing of proposed actions, given the risk and uncertainty identified during the study. There may be circumstances where the risk and uncertainty is such that more than one plan of action may be considered to reasonably maximize net benefits.

7.2 Without Project Economics

The without project condition represents the current operating condition of Clearwater Dam and takes into consideration the probability of future emergency action and the probability of dam failure given distress. Under the without project condition, the seepage problem at Clearwater is expected to continue over time, leading to an eventual failure of the dam and the rapid release of the pool downstream. It is unknown specifically the conditions that would eventually lead to such a failure or how long it will take for these conditions to develop. However, there is a high degree of confidence that a failure will eventually occur.

The without project condition forecasts a failure over a varying range of time with a range of flood damages downstream depending upon the pool elevation when the failure occurs. The economic evaluation of Clearwater Dam compares economic losses under the without-project condition to the economic losses with the rehabilitation plans in place.

The economic impacts of dam failure are measured in four categories. First, economic losses include distress costs. These costs could include, but are not limited to, the costs of lowering the pool elevation, mobilizing equipment and personnel for investigations, mobilizing equipment and personnel for repairs, and producing public address announcements. These costs exceed the bounds of regular O&M costs at the project. One assumption applied in this analysis is that expenditures for normal O&M at the project are assumed to be the same for both with and without project conditions. The evaluation of each plan includes the quantification of the reduction in distress costs from the without project condition.

The second category of economic loss is agricultural and non-agricultural damages. One of the greatest concerns arising from the structural integrity problems of the dam is the certain catastrophic flood damage caused by a dam failure. The methodology for estimating flood damages is detailed in the Economic Appendix A. Clearwater Dam does not currently prevent all flood damages downstream. Damages were estimated for a with failure scenario and a without failure scenario. The increment of damages between the with and without failure scenarios forms the without project damages which are summarized below:

20% PMF Increment ($ 000's)	
Residential	8,379
Commercial	1,878
Public	735
Other Property	142
Agricultural	36
Transportation	132
Utilities	66
TOTAL	11,368
100% PMF Increment ($ 000's)	
Residential	12,456
Commercial	3,747
Public	2,148
Other Property	82
Agricultural	1
Transportation	48
Utilities	69
TOTAL	18,551

The third category of economic loss is the cost of dam replacement. A basic assumption of the without project condition is that operating the dam is a primary objective. According to ER 1105-2-100, Section X – Major Rehabilitation Studies, “Should the project benefit stream be interrupted due to unsatisfactory feature performance, it is assumed that emergency funds will be available to fix the feature.” In the economic analysis of the without project condition, a dam failure is followed by dam replacement. This assumption also applies in evaluating each rehabilitation plan considered in this analysis.

For the period of time in which the dam is being rebuilt, foregone recreation benefits are counted as the fourth category of economic loss.

7.3 Risk Based Model

A probabilistic, risked based approach, using event trees, life cycle analyses, and Monte Carlo simulation to model the expected reliability of the dam and costs of consequences was employed in this evaluation. Monte Carlo simulation combined the probabilities of occurrence of pool stages, emergency action, and dam failure presented in the event trees with the consequences of various occurrences over a 50-year period of analysis. The expected costs resulting from these analyses of the current condition and operation of the dam was compared to the expected costs with rehabilitation of the dam.

Event Trees

Eleven conditions were evaluated for this report: the base condition and the 10 alternatives. The initiating event of the Clearwater event trees is the probability, or likelihood, of the pool reaching certain elevations under current and repaired conditions. These are based on historical data of annual peak pool elevations of the reservoir. Incremental probabilities of the pool being within a certain elevation range were used in the event trees.

A panel of technical experts estimated probabilities of emergency action at the dam at various pool stages and then probabilities of failure of the dam, given the stage. Each expert was asked to provide estimated probability factors considering that emergency action may be required at the project for five separate lake levels. The process was continued for each of the remediation measures for the possible remaining life of the dam. Additionally, the experts were asked to provide estimated probabilities for failure of the dam given the existence of distress and emergency action. A follow-up discussion was held with each expert to express their probability estimates with descriptive phrases from "A Practical Guide on Conducting Expert-Opinion Elicitation of Probabilities and Consequences for Corps Facilities," (IWR Report 01-R-01). The responses were recorded, and for each category, the median value was determined. This median value represents the probabilities of emergency action and failure given distress that are used in the event trees, life cycle analyses, and Monte Carlo simulations.

Life Cycle Analysis

The consequences of events are related to their probabilities of occurrence in the life cycle analysis. A spreadsheet-type life cycle analysis that calculates the expected costs of distress, emergency action, and failure of the dam was used for this evaluation. The consequences include distress repair costs, flood damage of the downstream area, foregone recreation benefits, and rebuilding costs.

Monte Carlo Sampling

Monte Carlo sampling was performed for values of pool elevation (based on cumulative distribution of peak pool elevations), the occurrence of distress (given its probability based on pool elevation), the occurrence of failure (given its probability based on pool elevation), annual recreation value (based on uniform distribution of recreation benefit), dam repair cost, and dam replacement cost.

7.4 Loss of Life With and Without Dam Failure

Reach	Percent PMF	LOL With Failure	LOL Without Failure	Incremental Difference
1	20	27	0	27
	100	38	0	38
2	20	3	0	3
	100	5	0	5
3	20	347	1	346
	100	348	1	347
Subtotals	20	377	1	376
	100	391	1	390

8.0 ALTERNATIVE DEVELOPMENT & EVALUATION

8.1 Basis of Formulation

This chapter presents the plan formulation rationale used for this report. The Corps of Engineers six step planning process specified in ER 1105-2-100 was used to develop, evaluate, and compare the array of candidate plans that have been considered. Steps in the plan formulation process include:

1. The specific problems and opportunities to be addressed in the study were identified, and the causes of the problems were discussed and documented. Planning goals were set, objectives were established, and constraints were identified.
2. Existing and future without-project conditions were identified, analyzed and forecast. The existing condition resources, problems, and opportunities critical to plan formulation, impact assessment, and evaluation were characterized and documented.
3. The study team formulated alternative plans that address the planning objectives. An initial set of alternatives was developed and will be evaluated at a preliminary level of detail.
4. Alternative project plans will be evaluated for effectiveness, efficiency, completeness, and acceptability. The impacts of alternative plans will be evaluated as specified in the Principles and Guidelines and ER 1105-2-100.
5. Alternative plans will be compared. A benefit-cost analysis will be conducted to prioritize and rank rehabilitation alternatives.
6. A plan will be selected for recommendation, and a justification for plan selection will be prepared.

8.2 Opportunities

There is an opportunity to significantly reduce future O&M expenditures related to the seepage problem that has developed since dam construction. The opportunity to make one-time economically justified expenditures is associated with the opportunity to fulfill federal responsibilities when a problem develops to a point requiring serious corrective action. Deferral of the solutions may result in significant dam failure modes resulting in loss of life and severe economic and legal consequences. There is an opportunity to avoid these at this this time by recognizing that past efforts have turned out to be piecemeal type solutions within limited funds, and that a comprehensive solution is now warranted.

8.3 Constraints

Planning constraints represent restrictions that should not be violated. The planning constraints identified in this study are as follows:

Availability of Water

Existing federal, state, county, tribal and private water laws, and agreements may impact plan formulation and implementation. Any proposed project must take into account ownership/control of water rights.

Maintenance of Floodway Capacity

Rehabilitation alternatives should not be pursued if such would substantially reduce the flood control capacity of the project.

Recreation

Projects must be formulated in such a way as to avoid impacts to existing and planned recreational facilities in adjoining areas. Reductions in existing recreation benefits should be minimized.

Cultural Resources

The study area may contain cultural resource sites which must be considered in development of any specific plan. The structure is under consideration for historic designation, and significant changes to its appearance should be avoided.

Endangered Species

Any potential project would be required under the Endangered Species Act to not jeopardize the continued existence of threatened or endangered species or to destroy or adversely modify their habitat.

Local Acceptability

Any plan must be generally publicly acceptable.

Displacement of People

The Uniform Relocation Assistance and Real Property Acquisition Policies Act of 1970 requires any local sponsor acquiring land for a project involving the Federal government to comply with the provisions of this act.

Real Estate

Real Estate costs can significantly affect project costs. Since right-of way costs may not be uniform with respect to location within the study area or width of acquisition, real estate costs represent a constraint on the location and dimensions of potential alternatives. Existing federal, state, county, tribal and private land ownership will impact real estate appraisals and acquisitions if project features require lands not currently part of the project.

8.4 Without Project Condition (No Action Plan)

Clearwater Dam currently provides flood control and recreation benefits. The average annual flood control benefit under present day conditions is about $4.1 million. Recreation use is about 360,000 visits annually with benefits of about $2.1 million annually.

Under the without project condition, the seepage problem at Clearwater is expected to continue and worsen over time, leading to an eventual failure of the dam and the rapid release of the pool downstream. It is unknown specifically the conditions that would eventually lead to such a failure or how long it will take for these conditions to develop. However, there is a high degree of confidence that a failure will eventually occur, with lower probability in the nearer term and high probability over the next 30 years. The without project condition assumes a failure over a varying range of time with a range of flood damages downstream depending upon the pool elevation when the failure occurs. The estimated population at risk downstream is about 9,000 people. The incremental increase in downstream damages with a failure is about $11 million for a 20% PMF event. Total downstream damages with a failure would amount to about $183 million for the 20% PMF and about $200 million for the 100% PMF event. In addition to downstream economic damages, a failure is projected to cause up to 391 deaths. With a failure, there will be a large cost involved in repair of the dam, and the without project condition assumes that the dam will be repaired in the event of a failure. During the time period repairs are being made recreation benefits will be lost. After the repairs are complete, the without project condition is that the life of the dam will be renewed to 50 years.

Operation and maintenance expenses related to seepage are expected to increase and become more frequent over time as the dam's condition continues to deteriorate. These expenses then would level off or be reduced after the dam fails and is repaired. The without project condition assumes that increasing percentages of the O&M budget will be required to address the seepage problem on an annualized basis, and that these piecemeal expenditures to defer risks into the future will not prevent the eventual failure mode. The first sinkhole appeared on the dam face in 2003 requiring additional unanticipated expenditures for investigations and repairs. These types of actions are expected to continue in the absence of a comprehensive solution.

The existing structure with the upstream seepage berm and impervious blanket to elevation 575 is shown at Figure 8-1 and 8-2.

FIGURE 8-1 Existing Structure

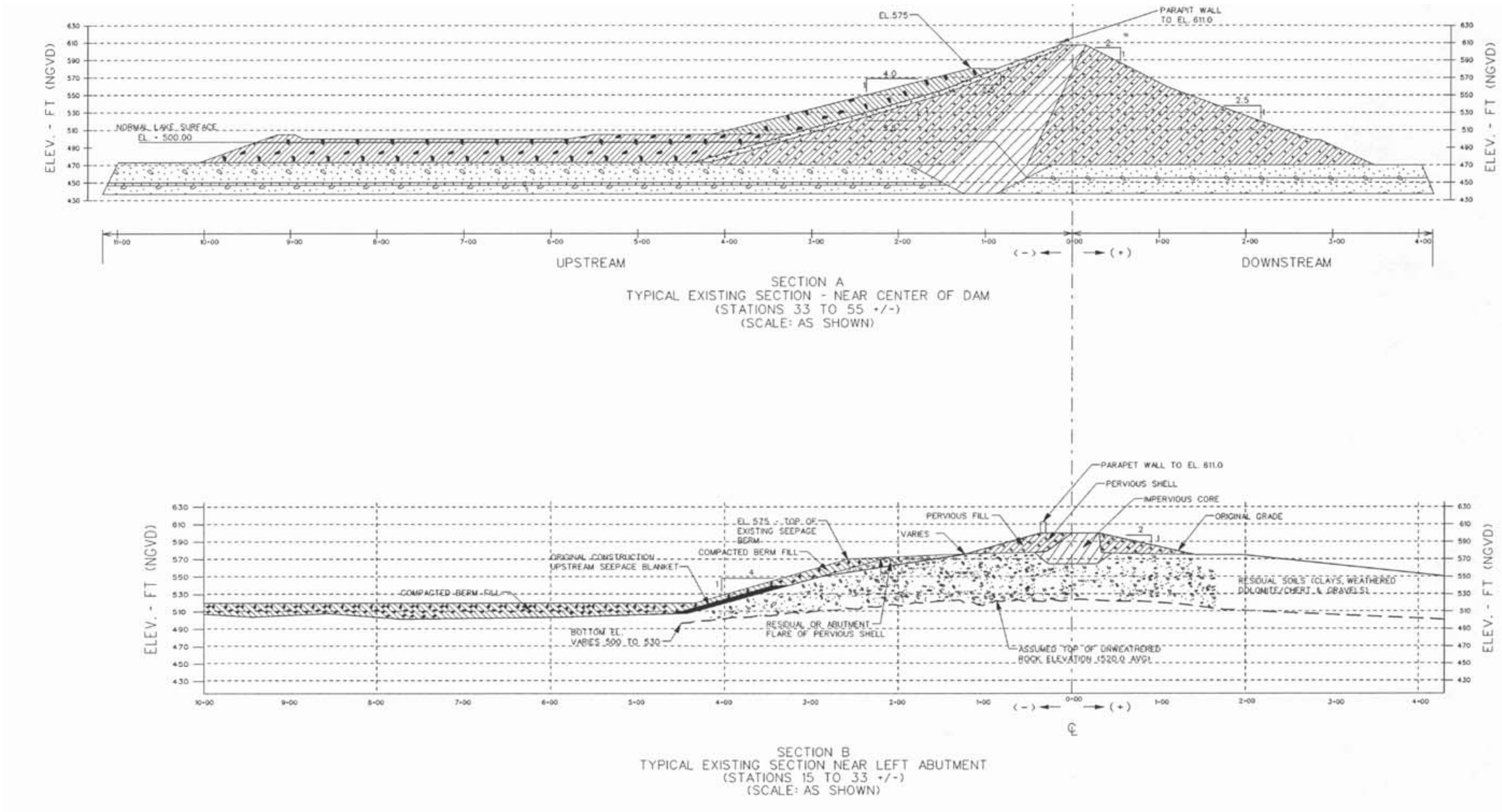


FIGURE 8-2 Existing Structure with Upstream Impervious Blanket on Face of Dam, & Seepage Berm to the Left

8.5 Objectives

Federal Planning Objectives

Planning objectives and constraints provide a framework for the development of alternative plans. As planning objectives for this investigation, it is in the Federal interest to:

- Contribute to National Economic Development (NED) through the reduction of failure hazards and concomitant reliability increases related to the identified seepage problems.

NED contributions include increases in the net value of national output of goods and services and can be measured in terms of monetary outputs such as reductions in O&M, flood damages and emergency response costs.

Specific Planning Objectives

Specific planning objectives were developed to guide formulation of a rehabilitation plan. Those objectives are:

- Reduce or eliminate seepage through the embankment,
- Reduce or eliminate seepage under the embankment,
- Prevent a catastrophic failure of the embankment and pool release.

8.6 Alternative Development

8.6.1 Alternative Development Rationale

The alternatives are developed for the purposes related specifically to the requirements for a Corps of Engineers Major Rehabilitation Report. As such, the alternatives described in this report are not of sufficient design detail to be constructed. Following the completion of this report and environmental compliance, detailed design analysis and preparation of plans and specifications would take place. Alternatives were formulated to address a Federal rehabilitation project to:

a. Comply with NEPA and other environmental laws and regulations;
b. Reduce future maintenance costs;
c. Increase the reliability of the structure and extend the project life;
d. Maintain existing benefit levels;
e. Significantly reduce risks of catastrophic or gradual failure modes;
f. Contribute to the National Economic Development Account (NED);
g. Provide decision makers with information that could be utilized to help determine the balance between initial construction costs and the costs and risks of continued deferral.

8.6.2 Alternative Development and Evaluation Process

The process involves developing and refining, through successive iterations, alternative solutions to the defined problem. Solutions are evaluated based upon the degree to which they address study objectives and take advantage of identified opportunities while remaining within the limitations imposed by the identified constraints. The general justification criteria that are required to be met are as follows:

- *Technical Feasibilit*y: Solutions must be technically capable of performing the intended function, have the ability to address the problem, and conform to Corps of Engineers technical standards, regulations, and policies;
- *Environmental Feasibilit*y: Solutions must comply with all applicable environmental laws, including the National Environmental Policy Act;
- *Economic Feasibilit*y: Solutions must be economically justifiable in that the economic benefits or, in the case of ecosystem restoration NER (non-monetary) benefits, must exceed the economic costs, in accordance with applicable regulations, policies, and procedures; and
- *Public Feasibilit*y: Solutions must be publicly acceptable as evidenced and documented through a process that incorporates the public's input on potential solutions.

Initially, specific measures were developed to satisfy the four feasibility criteria. Measures are stand alone features that address the defined problems. There are numerous measures that can be utilized, depending upon site location, technical considerations, environmental conditions, and a host of other factors. In determining the set of measures to be evaluated for this report, consideration was given to extensive Corps experience with similar situations, technical considerations based upon the specifics of the area, flood control considerations for maintaining the existing level of protection, and risk reduction strategies to reduce the hazards associated with a dam failure.

Clearwater Dam has been the subject of numerous Dam Safety studies and constructed remedial measures over many years. In selecting specific measures for documentation in this report, previous measures and their evaluation are included in the initial array for purposes of completeness. No specific distinction was initially made between dam safety-type measures, major rehabilitation-type measures, or old measures versus newer measures. All were included in the initial array either for documentation of previous considerations or for potential inclusion or combination with other measures.

8.6.3 Initial Measures

An interdisciplinary study team identified a variety of measures which could be used to address the seepage problem. The broad categories of measures initially investigated are listed below. For the evaluation of the measures discussed below see Section 8.D, Preliminary Evaluation of Measures.

Nonstructural Measures

N1 Operate As a Dry Reservoir

Operating the project as a dry reservoir would increase the flood control benefits that are being realized at Clearwater Dam and decrease the amount of the seepage through the dam.

N2 Dam Breaching During Emergencies

Emergency breaching of Clearwater Dam could occur at predetermined rainfall amounts, pool elevations, and Black River stages. This measure would be employed as a last resort in the event of an emergency situation.

N3 Remove Structure

The Dam could be decommissioned and removed, eliminating future O&M expenditures.

N4 Downstream Emergency Planning

A flood warning system and emergency action plan update could be implemented for downstream reaches. This could consist of actual flood warnings, and notifications by local officials and media channels to residents in coordination with the Corps in events where dam failure risks were deemed to warrant activation of emergency response plans.

N5 Changes to the Water Control Plan

This alternative includes such operational changes as making larger releases at lower lake levels during rainfall events or lowering the conservation pool elevation to increase flood storage. Changes to the operations plan would be designed to lower the conservation pool and reduce the seepage flows through and under the dam.

N6 Expand Seepage Monitoring

The existing seepage system could be expanded to provide more accurate determinations of seepage flow paths and flow rates. Seepage monitoring is a crucial tool in identifying any questionable changes in subsurface groundwater flow or possible internal erosion or damage. If any noticeable changes occur an action plan can be implemented.

Structural Measures (Schematics Follow Descriptions)

S1 Extend the Impervious Fill Blanket

At the present, an impervious soil blanket covers much of the upstream embankment slope of the dam to elevation 575 ft. The existing blanket could be extended to the top of the dam (bottom of parapet wall) in order to withstand internal infiltration and saturation problems. Such an extension would eliminate the seepage through the top portion of the embankment that currently occurs with a 20% PMF or larger event.

S2 Construct a Cement/Bentonite Slurry Cutoff Wall in the Upstream Seepage Berm

A cement/bentonite slurry cutoff wall could be installed within the upstream seepage berm. This measure would involve the mixture of a cement/bentonite slurry that is pumped into an excavation that extends from the ground surface to bedrock. This cutoff wall would essentially prevent groundwater flow through the overburden soils beneath the dam. Three variations were developed:

S2-A: Slurry Cutoff Wall To Rock Located 500 ft Upstream of Existing Dam Toe Without Extension of Impervious Blanket.

Construct a bentonite cement cutoff wall penetrating to rock. This would be placed upstream of the dam toe and through the existing seepage berm. It would begin at the right abutment and extend out approximately 500 feet onto the seepage berm and terminate into high ground near the left abutment. The total length would be about 4,300 feet. The cutoff would extend to a depth of about 70 feet to top of rock.

S2-B: Slurry Cutoff Wall To Rock Located 500 ft Upstream of Existing Dam Toe With Extension of Impervious Blanket

This is a combination of Measures S1 and S2A - A slurry cutoff wall in combination with extending the impervious blanket.

S2-C: Slurry Cutoff Wall To Rock Located at Existing Upstream Embankment Toe of Dam With Extension of Impervious Blanket

This measure is the same construction as Measure S2-B but the location would be moved to the upstream toe of the dam instead of 500 feet away.

S3 Construct a Cement/Bentonite Slurry Cutoff Wall with Bedrock Keyway

A cement/bentonite slurry cutoff wall could be installed within the upstream seepage berm, 500 feet upstream of the toe, with the mixture of a cement/bentonite slurry pumped into an excavation that extends from the ground surface into a keyway cut into bedrock. This type of keyed in cutoff wall would essentially prevent groundwater flow with a greater degree of certainty than the cutoff wall that rests upon the bedrock. Essentially the previous measure S2-B is expanded in scope to include deep intermittent concrete cutoff wall panels extended 60 feet into rock where defects or voids are detected. The cutoff wall would have a depth of between 70 feet to rock and 130 feet for the deep cutoff panels areas.

S4 Secant Pile or Rockmill Cutoff Wall - Concrete

As a means of reducing the amount of seepage that has been allowed to infiltrate beneath and through the dam and left abutment, a concrete secant pile or rockmill cutoff wall were considered. These structures would utilize concrete instead of the slurry mix and meet or penetrate through the bedrock in order to form a positive cutoff from lakeside groundwater flow through solution channels and cavities within the karstic bedrock. Five variations were developed:

S4-A: Concrete Cutoff Wall To Rock Located at Existing Upstream Embankment Toe of Dam With Extension of Impervious Blanket

This measure is the same configurations as S2-C at the toe of the dam, with the cutoff wall extending 70 feet to bedrock, but construction would consist of concrete grout instead of a slurry mix.

S4-B: Concrete Cutoff Wall Into Rock Located at Existing Upstream Embankment Toe of Dam With Extension of Impervious Blanket

Same as S4-A except that the cutoff wall (either secant pile or rockmill method) would be extended into rock continuously (60 feet into rock) with concrete. The total length would be about 4,300 feet.

S4-C: Concrete Cutoff Wall Into Rock Located Through the Dam and Through the Centerline of the Clay Core Trench With Extension of Impervious Blanket

Placement of the concrete cutoff wall would be through dam and through the centerline of the clay core trench. Penetration of the wall would begin along the existing impervious blanket at El. 575 of the dam's upstream face and extend 60 feet into rock for a total depth of 200 feet. Length and width would be the same. A berm would have to be constructed for access and to allow for a 30 feet wide working platform. Extension of impervious blanket will be required to prevent seepage inflow directly into the window area. The total length would be about 4,300 feet.

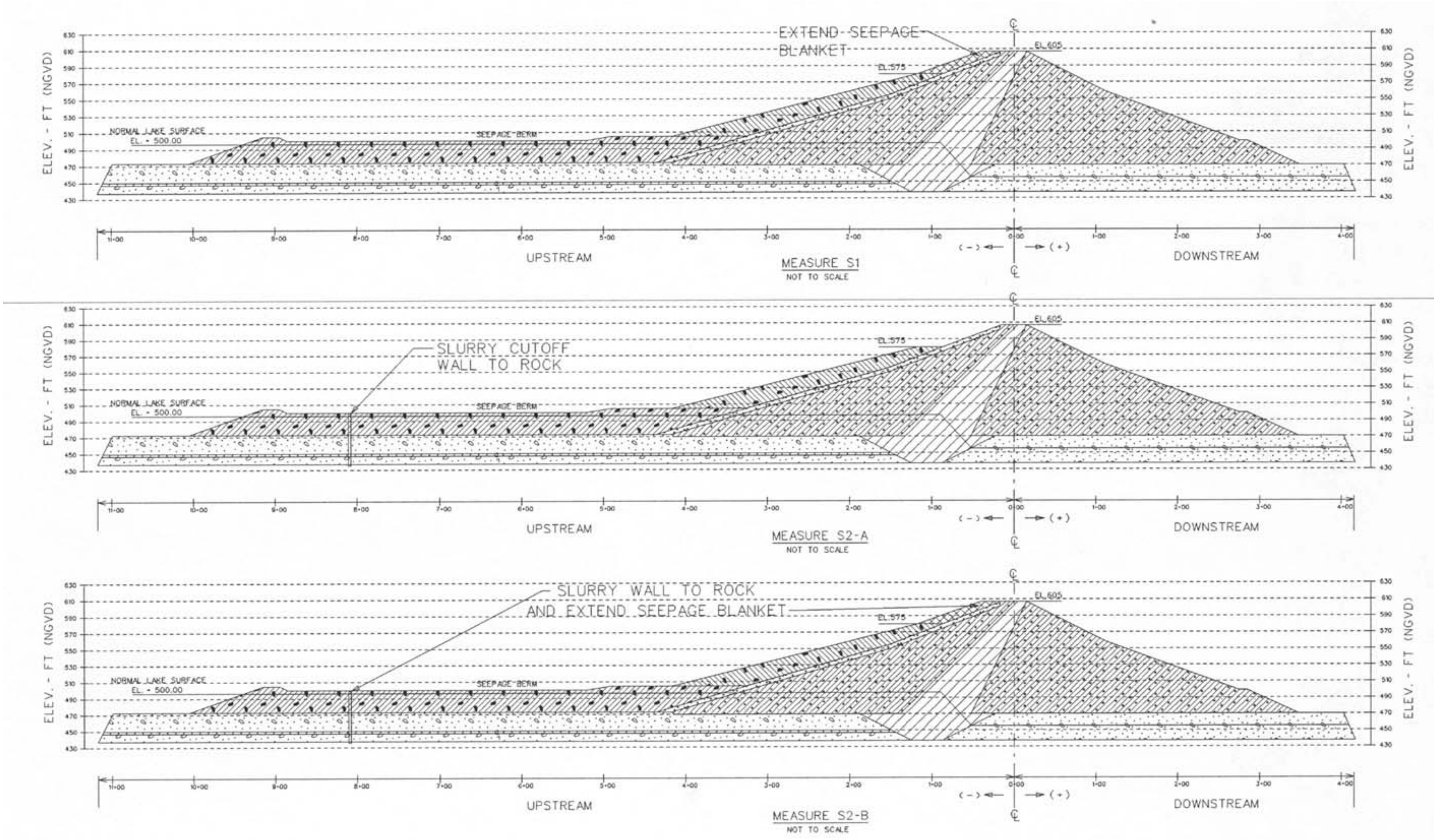
S4-D: Concrete Cutoff Wall Into Rock Located Through the Centerline of the Dam Alignment

Same as Measure S4-C except the cutoff wall location would move to the centerline of the dam alignment. The total depth of the wall would be 230 feet with the same length and thickness. The total length would be about 4,300 feet. Extension of the impervious blanket would not be included. The roadway surface of the dam would provide adequate access for construction. Highway HH would be closed during construction activities and traffic would be routed along Highway 34.

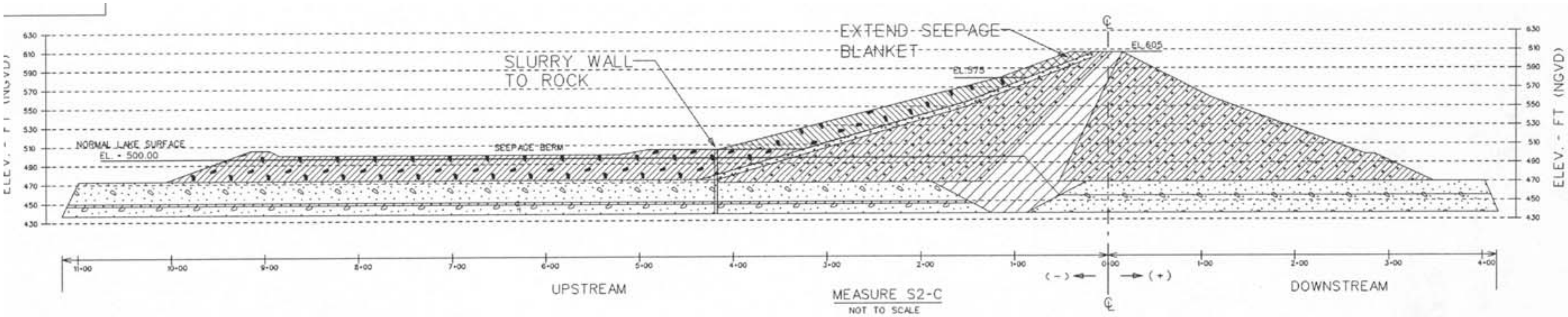
S4-E: Concrete Cutoff Wall Into Rock Located Through the Dam and Through the Centerline of the Clay Core Trench from Sinkhole to End of Left Abutment With *Extension of Impervious Blanket*

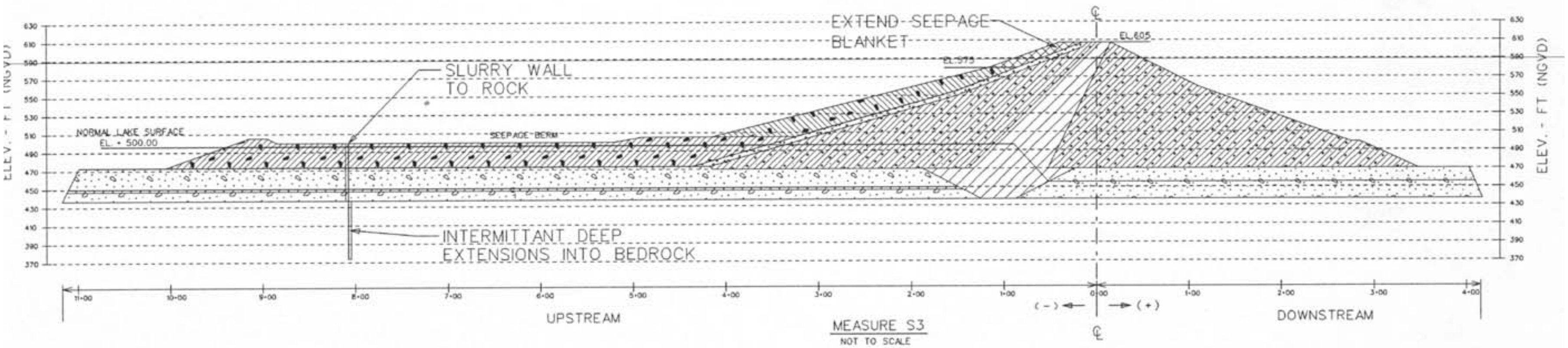
Same as S4-C except total length would be shortened to approximately 2800 feet.

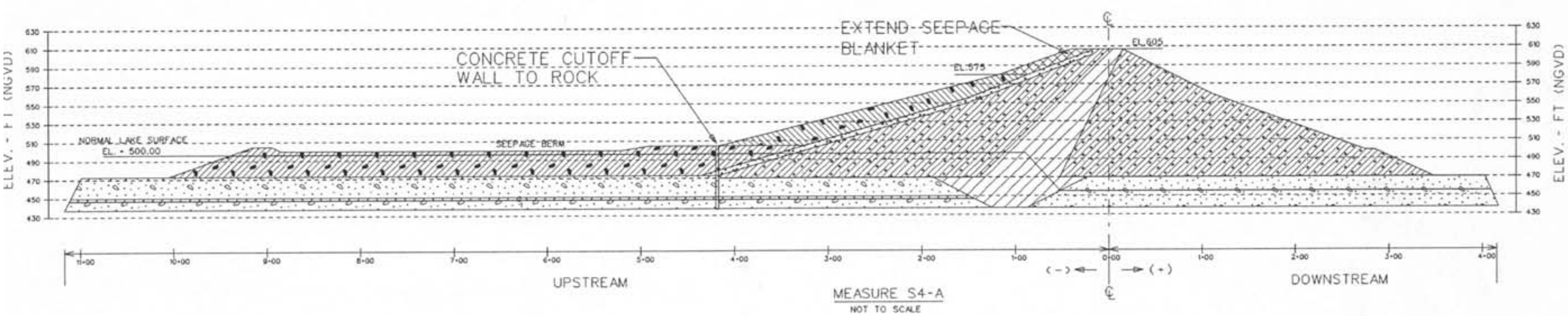
Alternative Schematics



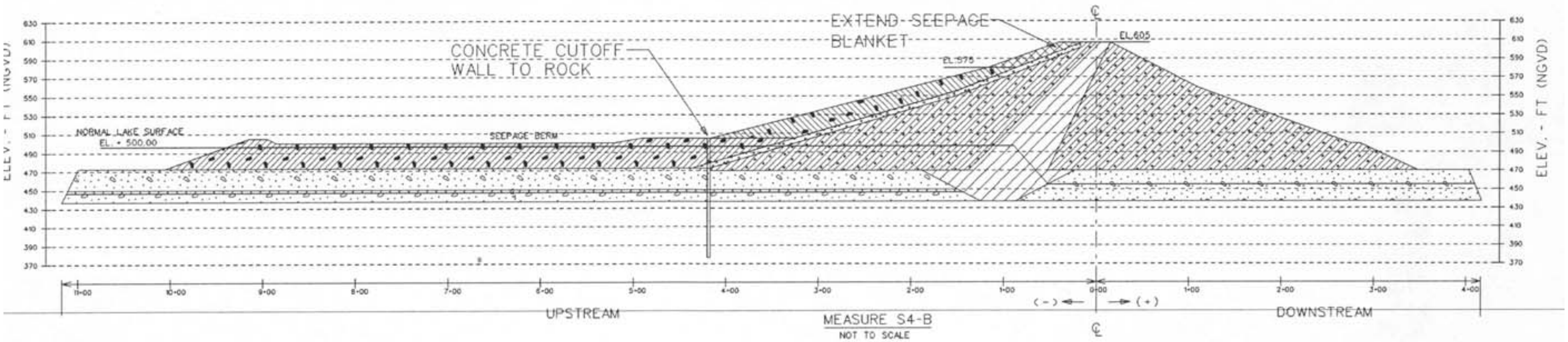
Alternative Schematics

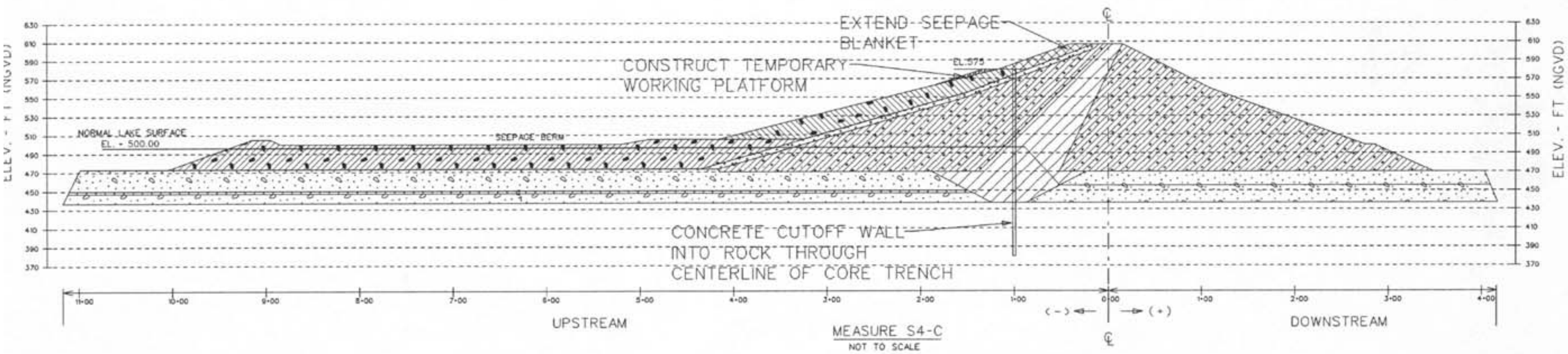






Alternative Schematics





8.6.4 Preliminary Evaluation and Screening of Alternatives & Measures

Each measure or alternative combination of measures, along with its associated elements, was evaluated in terms of the criteria. All criteria must be adequately met since any one criterion can serve to eliminate a measure from further consideration. Those measures/elements satisfying all the criteria were carried forward for additional development and evaluation while those that were shown not to meet the criteria were eliminated from further consideration.

Nonstructural Measures

N1 Operate As a Dry Reservoir

Operating the project as a dry reservoir would increase the flood control benefits that are being realized at Clearwater Dam and decrease the amount of the seepage through the dam. However, a dry reservoir would eliminate the ancillary project benefits for recreation and fish and wildlife. The local communities would not support this option. The increase in flood control benefits is offset by the loss of recreation benefits. This measure was screened from further consideration.

N2 Dam Breaching During Emergencies

Emergency breaching of Clearwater Dam would be a last resort and is not a desirable alternative. A controlled breach of the dam would occur at predetermined rainfall amounts, pool elevations, and Black River stages that were beyond the scope of this report to compute. Further, any breach of the embankment would accelerate as it progresses an uncontrolled rate and have similar disastrous results as would a dam failure. Flooding would be catastrophic and project annual benefits would be lost. A purposeful dam breach may be considered in the future during a particularly severe emergency situation and would remain an option. A dam breach is not considered a technically feasible solution to the seepage problem and does not meet the planning objectives, and was screened from further consideration.

N3 Remove Structure

The annual costs of the removing the structure, environmental cleanup and restoration, plus the loss of the project's current average annual net benefits greatly exceeds the annual OMRR&R cost savings. Structure removal is screened from further consideration

N4 Downstream Emergency Planning

A flood warning system would be impractical for the downstream reaches below Piedmont. The area immediately below the dam could receive benefits from an upating of the emergency reponse system and increased interagency coordination, possibly saving lives. Implementation of such a program would require development of emergency response plans with downstream interests. This measure would not be costly or complex to implement, and should be considered further in the future as a stand alone system, potentially under the Continuing Authorities Program. Flood Warning and Emergency Planning would not accomplish the objective of increasing structure reliability or reducing the risk of failure and is screened from further consideration.

N5 Changes to the Water Control Plan

This alternative includes such operational changes as making larger releases at lower lake levels during rainfall events or lowering the conservation pool elevation to increase flood storage. Increasing the flood control releases would have negative impacts to the protected reaches downstream, causing disbenefits especially to the fertile farmlands south of Poplar Bluff. The farming interests have opposed higher regulating stages for years and have been successful in lowering regulating stages through deviations to the water control plan. Decreasing the conservation pool elevation would have insignificant flood control benefits. The lower elevations of the pool have inadequate storage volumes to make an appreciable impact to flood reduction. Also, it would result in recreation disbenefits and lake beneficiaries would oppose such a move. This measure is screened from further consideration.

N6 Expand Seepage Monitoring

Seepage monitoring should be continued regardless of what additional design changes would be implemented in the future. This is primarily due to the history of observed seepage occurrences that have been documented over the lifetime of the project and the karstic geologic formations that the dam and the area are founded on. With seepage monitoring, if any noticeable changes do occur during the implemented seepage monitoring regime, the ways and quantity of monitors can be modified in order to best understand the seepage mechanism that could be happening at any given time. Expansion of seepage monitoring does not address the seepage problem or reduce the risks of failure and would be pursued only if fixing the seepage problem could not be accomplished. Although expansion of the seepage monitoring is not ruled out for future O&M activities, this measure is not carried forward for additional evaluation in this report.

Structural Alternatives/Measures

Most long-term remediation alternatives include extending the impervious blanket to the top of the dam to preclude lake infiltration into the pervious shell. If this were not done as part of most alternatives considered, a continuous water seal would not be formed and the associated alternative would fail to control seepage at very high lake levels. The impervious blanket extension is considered a “partial” solution and was evaluated as an increment.

Each alternative was developed considering the best alignment that would optimize or minimize various issues and risks related to cost, success of controlling gradient or eliminating seepage, and experiencing future damage or deterioration to the seepage barrier/cutoff or dam.

Alternative Economic Comparisons

A detailed Economic Analysis was conducted on all of the structural alternatives. The reliability of each alternative was compared against the without project condition at various lake levels. In general, as lake levels rise the risk of failure becomes greater and the reliability of the structure decreases. However, the reliability decreases less under the with project condition associated with each alternative. These changes in reliability were input in a Monte Carlo style analysis. They represent the degree to which each alternative solves the seepage problem. The alternatives with the highest reliability (and highest costs) were compared against other lower cost alternatives with lesser degrees of increased reliability. The Economics Appendix has complete details on the methodology and basis of the alternative comparisons. Table 8-1 below details the summary information from the Economic Analysis. A discussion follows the Table outlining the economic and technical considerations employed in selecting the Alternatives for the Final Array.

TABLE 8-1 ALTERNATIVE ECONOMIC COMPARISONS (NOTE: All Numbers in 000's, rounded)

	S1	S2A	S2B	S2C	S3	S4A	S4B	S4C	S4D	S4E
Subtotal Construction	5,958	2,244	8,198	8,738	14,340	22,282	59,553	59,058	56,480	51,682
PED/EDC	357	135	820	699	1,728	1,114	1,191	1,181	1,130	1,034
S&A	596	337	984	1,049	1,728	2,674	4,466	4,429	4,236	3,876
First Cost (Incl. Contingency)	8,467	3,854	12,258	12,856	21,892	31,955	73,057	71,948	68,812	57,963
IDC	276	44	547	636	1,190	2,211	6,905	6,616	6,151	4,592
Gross Investment	8,743	3,898	12,805	13,492	23,082	34,166	79,962	78,563	74,963	62,556
Annualized (5-5/8&, 50 yrs)	526	234	770	812	1,388	2,055	4,810	4,725	4,509	3,763
O&M	-	-	-	-	-	-	-	-	-	-
Total Annual Cost	526	234	770	812	1,388	2,055	4,810	4,725	4,509	3,763
Expected Annual Benefits	502	1,292	2,252	2,250	3,351	2,279	3,357	5,782	6,161	3,509
Net Benefits	-24	**1,058**	**1,482**	**1,438**	**1,963**	224	-4,452	**1,056**	**1,652**	-254
Benefit/Cost Ratio	.95	5.52	2.92	2.77	2.41	1.11	.70	1.22	1.37	.93
Statistical Derivation Rating of Technical Feasibility (0 to 50)	1.6	4.4	9.0	9.0	15.0	9.2	15.1	33.5	39.9	16.1

S1: Extend Impervious Blanket
S2A: Slurry Wall to Rock 500' U/S
S2B: =S1 + S2A
S2C: =S1 + Slurry Wall to Rock at Toe
S3 : =S2B with added bedrock keyway
S4A: =S2C with concrete instead of slurry wall
S4B: S1 + Concrete wall into Rock at Toe
S4C: S1 + Concrete wall through Clay Center into Rock
S4D: Concrete wall into Rock through Dam Center
S4E: Shortened Variation of S4C

In general, the alternatives were screened for the Final Array in the following manner. Several alternatives are economically justified. Of those alternatives, the alternative(s) producing the highest net benefits were further screened for potential inclusion in the Final Array. Then, the alternative that provides the highest technical feasibility will be included in the Final Array. This process resulted in a Final Array consisting of the no action plan, the most economic plan(s), and the best technical plan that is economically justified.

S1 Extend the Impervious Fill Blanket

The existing impervious soil blanket covers the upstream embankment slope of the dam up to elevation 575 ft. Alternative S1 is considered only a partial solution to the global seepage issue because it does not address the primary concern of subsurface seepage and piping. The estimated cost of this measure $8.5 million with a construction time frame of approximately 15 months. S1 is not technically sufficient as a stand alone solution. As a stand alone solution S1 will not be included in the Final Array.

S2 Construct a Cement/Bentonite Slurry Cutoff Wall

A cement/bentonite slurry cutoff wall was evaluated both alone and in combination with Measure S1. Three variations were evaluated. All were economically justified.

S2-A only addresses seepage through the alluvial soil strata and does not attempt to reduce seepage in the underlying bedrock. High lake levels (+ EL 575) would lead to immediate saturation and high flow gradients into the unprotected window area if the impervious blanket were not extended. Furthermore, the risk of relying on the blanket along the dam face as a barrier is relatively higher. The estimated cost and construction time are $3.85 million and 6 months. Due to the lower benefits and lesser degree of technical feasibility, S2-A will not be included in the Final Array.

S2-B also only addresses seepage through the alluvial soil strata and does not attempt to reduce seepage in the underlying bedrock. Furthermore, the risk of relying on the seepage blanket on the dam face as a barrier remains relatively higher. The estimated cost and construction time are $12.3 million and 20 months. Due to the lower benefits and lesser degree of technical feasibility, S2-B will not be included in the Final Array

S2-C only addresses seepage through the alluvial soil strata and does not attempt to reduce seepage in the underlying bedrock. Furthermore, the risk of relying on the seepage blanket on the dam face as a barrier is relatively higher. The estimated cost and construction time are $12.9 million and 22 months. Due to the lower benefits and lesser degree of technical feasibility, S2-C will not be included in the Final Array.

S3 Construct a Cement/Bentonite Slurry Cutoff Wall with Bedrock Keyway

A cement/bentonite slurry cutoff wall could be installed within the upstream seepage berm, with the mixture of a cement/bentonite slurry pumped into an excavation that extends from the ground surface into into bedrock. This type of keyed in cutoff wall would essentially prevent groundwater flow with a greater degree of certainty than the cutoff wall that rests upon the bedrock. Alternative S3 addresses seepage below the rock surface with intermittent deep concrete panels. The confidence that seepage could be adequately cut off with this measure is lower than higher cost alternatives partially due to the risk of relying on the impervious blanket on the upstream dam face. The estimated cost and construction time are $21.9 million and 24 months. Alternative S3 produces the highest net benefits and will be included in the Final Array. From a technical perspective it is a less reliable solution than higher cost alternatives.

In evaluating S2B, S2C and S3 together, the increment associated with the bedrock keyway was identified. S2C is similar in overall cost to S2B, and the benefits are essentially the same between the two. S2B and S2C differ from one another only in location; S2B is located 500' upstream of the toe whereas S2C is located at the toe. In evaluating S3 against other justified alternatives the net benefits increase incrementally.

S4 Secant Pile or Rockmill Cutoff Wall - Concrete

S4-A addresses seepage through the alluvial soil strata and does not attempt to reduce seepage in the underlying bedrock. Furthermore, the risk of relying on the impervious blanket on the dam face as a barrier is relatively higher. The estimated cost and construction time are $32 million and 29 months. Due to the lower benefits and lesser degree of technical feasibility, S4-A will not be included in the Final Array.

S4-B addresses seepage below the rock surface with a continuous deep cutoff wall. However, the risk of relying on the impervious blanket on the dam face as a barrier is relatively higher. The estimated cost and construction time are $73.1 million and 40 months. S4-B is not economically justified and will not be included in the Final Array.

S4-C addresses seepage below the rock surface with a continuous deep cutoff wall. There would be less risk of relying on the impervious blanket as a barrier due to the location of the wall (intersecting at top of existing impervious blanket). Difficulty in excavating through the embankment shell material and into rock was considered as well as the required construction of a working platform. The estimated cost and construction time are $72 million and 39 months. S4C has lower net benefits and has the second highest reliability rating. S4C will not be included in the Final Array.

S4-D addresses seepage below the rock surface with a continuous deep cutoff wall. The impervious blanket extension would not be required because the upstream face of the dam would be protected by the centerline location of the cutoff wall. The estimated cost and construction time are $68.8 million and 38 months. S4-D is economically justified and is the most technically feasible alternative. It provides the greatest annualized benefits of all of the alternatives, but due to the cost the net benefits are slightly lower than S3. From a technical perspective it is the most reliable solution among all of the alternatives, i.e. it solves the problem with the highest degree of confidence. Due to the highest technical feasibility with economic justification S4-D will be included in the Final Array.

Secondary highway HH would be closed during construction activities and this would necessitate traffic to detour using Highway 34. A detour bypass road and bridge around the dam was initially evaluated as part of this alternative and was not economically justified. In addition, construction of a new road and bridge in the river would have environmental impacts requiring mitigation, and permitting issues. The detour bypass component of this alternative was subsequently dropped in favor of a standard road closure scenario. The benefits shown in the above table include a reduction due to estimated detour times and distances. These re-route costs are also lower than the costs of constructing a bypass road. During the next phase of study and detailed design, lower cost alternatives to the standard road closure scenario can be evaluated.

S4-E addresses seepage below the rock surface with a deep cutoff wall. The reliability and technical feasibility and net benefits are less than other alternatives. S4-E will not be included in the Final Array.

8.7 Alternatives - Final Array

Alternatives or measures not screened from further consideration are included in the final array of alternatives for more detailed evaluation. In addition, those alternatives consisting of the Final Array are subjected to detailed evaluation and analysis for environmental compliance purposes. The Final Array of alternatives is subjected to the following specific process to enable selection of a plan:

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Technical merits are further considered,

- Costs and Benefits are verified/refined,
- Environmental compliance for each Final Array alternative is evaluated,

The Final Array of Alternatives is as follows:

1. The No Action Plan
2. Alternative S3
3. Alternative S4-D

Technical Considerations

The following technical points were considered for the Alternatives in the Final Array:

1) Seepage conditions at Clearwater have been often observed throughout the life of the project. Many efforts have been attempted in an effort to minimize the seepage effects that had become a nuisance downstream of the dam. These efforts do not address long-term seepage effects and the problem remains and is worsening over time.
2) A sinkhole occurred in the upstream portion of the embankment in January 2003. This sinkhole appearance and the subsequent investigations have cast doubt on the integrity of the entire dam structure.
3) The sinkhole investigation and geophysical studies indicate that the sinkhole shaft is relatively vertical and emanates from the base of the dam. Although the exact cause of the sinkhole has not been determined, it is believed that some anomalies within the karstic bedrock combined with a form of internal seepage and erosion/piping are the causative agents of the sinkhole development.
4) The ability of the existing filter zones within the dam structure to perform as designed is questionable. Limited testing of the core, shell and alluvial material suggest that the filter zones are sub standard.
5) The aforementioned deficiencies have led the project team to consider both interim and long-term remediation measures of the sinkhole and entire dam. The measures attempt to address the seepage effects for the all potential lake levels. These measures consist of: deep (into rock) cutoff walls that essentially halt seepage and shallow depth (to rock) barrier walls to retard seepage.
6) A foundation drilling and grouting – sinkhole repair project is currently being conducted as an interim measure to repair possible localized damage of the core caused by the sinkhole. Additionally, much qualitative design information will be ascertained from the drilling and piezometer data obtained during the repair. This information will be utilized in the plans and specifications phase of the recommended plan.
7) S4-D would be the most satisfactory choice from a reduction of risk perspective. It has the greatest technical confidence and reliability in solving the seepage problem. S4-D also has the highest level of total benefits. Alternative S3 which has the highest net economic benefits has a lower level of technical reliability. S4-D has the greatest statistical increase in the long term survivability of the dam.

Economic Considerations

1) Lower cost alternatives that utilize the cement/bentonite slurry have the greatest net benefits over methodologies employing concrete/grout. Specifically Alternative S3 produces the highest net benefits, however the reliability is low and risk is higher. In contrast, a small reduction in the net benefits is associated with a large increase in reliability and much lower risk.

Environmental Compliance

1) None of the alternatives are expected to result in significant adverse environmental impacts.

8.8 Conclusions

Based upon the above evaluations, the best overall choice for implementation is alternative S4-D. This plan addresses both components of the seepage problem and is economically justified with the second highest net benefits and the highest technical feasibility of all of the alternatives. This plan best meets the objectives of the major rehabilitation study while conforming to the constraints when compared to the other measures and alternatives evaluated.

The only alternative producing higher net benefits than S4-D is alternative S3. However, the technical feasibility rating of S3 is low. The increase in the net benefits of S3, when compared against the large increase in reliability of S4-D, indicates that benefits can be increased if much higher levels of risk are taken. When considering the additional potential for loss of life, and the federal responsibility towards public health and safety associated with this Corps constructed, operated, and maintained facility, assumption of such risks is deemed unacceptable.

9.0 RECOMMENDED PLAN

The recommended plan is S4-D. It consists of a concrete cutoff wall along the entire length of the dam centerline. The wall extends into bedrock under the dam and through the impervious core of the dam to prevent seepage and migration both through and under the dam. The total depth of the wall would be 230 feet, starting at the top of the dam and with a length of about 4,300 feet.

10.0 MAJOR REHABILITATION CLASSIFICATION

Studies to address the seepage problems at Clearwater Dam focus on improving the reliability and significantly extending the physical life of the structure. For Clearwater Dam, the recommended plan cost of addressing the seepage problem is $68.8 million and the work will extend over a period of 3 years and is consistent with guidance received from CESWD and CECW to address the seepage problem at Clearwater under the Corps Major Rehabilitation Program.

The long term reliability of the structure will be improved resulting in decreased maintenance costs and extending the life of the structure. The net economic benefits have been estimated at $1,652,000 annually and the benefit-cost ratio is 1.37 to 1.

No changes have been proposed to the operation of Clearwater Dam. There is no change in project outputs beyond the original design. The proposed rehabilitation work will enable the project to realize original design outputs (flood damage reduction) over the extended life of the project. No benefits are claimed for the Efficiency Improvement category.

11.0 OTHER CONSIDERATIONS

11.1 Cost Sharing

Clearwater Dam is owned and operated by the U.S. Army Corps of Engineers, Southwest Division, Little Rock District. There are no non-federal cost sharing requirements for the rehabilitation activities recommended in this report.

12.0 RECOMMENDATION

A full consideration of the costs, benefits, environmental effects, and technical considerations including the risks associated with each the alternatives, has been performed, and with consideration of the federal responsibility towards public health and safety, I recommend that Plan S4-D, as generally described and evaluated herein, be implemented and that detailed design activities for Plan S4-D be initiated.

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DATE

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Benjamin H. Butler, P.E.
Colonel, US Army
District Engineer