FINAL DESIGN ANALYSIS

Lake Ladora Dam Repair

Rocky Mountain Arsenal
Commerce City, CO

July 1997

US Army Corps
of Engineers
Omaha District

OMAHA DISTRICT

3670-1/1

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REHABILITATION OF LAKE LADORA DAM
FINAL DESIGN REPORT

1. **Purpose:** The Final Design Report for Ladora Dam is presented to Rocky Mountain Arsenal with design details, construction drawings, specifications, and a cost estimate for the selected remedial repair for Ladora Dam chosen by Rocky Mountain Arsenal from the alternatives evaluated in the Concept & 65% design submittals. The purpose of this design is to remediate the geotechnical concerns and satisfy the hydrologic requirements and deficiencies necessary for a dam of its size and hazard classification located on army installations.

2. **Scope:** This document is designed to advance the details of the selected alternative presented in the Concept & 65% Design Reports. The selected alternative includes replacement of the south embankment of the dam, the installation of a gravity flow outlet works, and a 100 feet wide earthcut spillway. The spillway includes an earth lined channel with a series of low head drop structures. The rehabilitation will also include placement of riprap protection on the upstream slope of the entire embankment.

3. **Authorization.**

3.1. **Authority.** Memorandum, Rocky Mountain Arsenal, AMCPM-RMM, 6 June, 1996 subject: O&M Project, Lake Ladora Dam, Rocky Mountain Arsenal (RMA).

3.2. **Criteria.** RMA has requested a 65% Design and Construction Cost Estimate identified herein.

4. **Project Description:**

4.1. **Location.** Ladora Dam is located in Section 2, Township 3 South, Range 67 West, in a natural ravine in Adams County, Colorado referred to as Irondale Gulch. Refer to Plate Nos. 1 and 2 for a plan view of the vicinity and location maps.

4.2. **Purpose of Dam.** Ladora Dam and lake is used to store water for the following purposes: 1) Irrigation, 2) store a standby water supply for fire protection, 3) control and measure surface runoff on the arsenal, 4) help direct groundwater movement and location, 5) fish and wildlife and research, and 6) public recreation.

4.3. **Size and Hazard Potential Classification.** In Accordance with guidance from the Army Center for Public Works, the Dam Safety requirements for dams owned or located on an army facility must meet the most stringent of Federal or State dam hydrologic criteria. The current Federal criteria for safety modification for an existing dam is that it should meet a base safety standard. The base safety standard is met when a dam failure related to hydrologic capacity will result in no significant increase in downstream hazard over the hazard which would have existed if the dam had not failed or not existed and still had a flood through the area. For the 100-year storm event, failure of Ladora Dam would increase the peak stages downstream of RMA by less than one foot. This increase is not considered significant. Therefore, the 100-year event is an adequate design inflow routing standard for Ladora Dam. Therefore, the safe hydrologic design of Ladora Dam calls for a spillway discharge capability for the dam, in combination with an embankment crest height, that can handle the 100-year flood event without the embankment being overtopped.

4.3.1. **Size.** Ladora Dam has a maximum height of 22.4 feet and a maximum controlled storage capacity of 280 acre-feet (Standard Operating Manual). According to Federal guidelines, it is classified as a small size dam. A small size dam has a height greater than or equal to 25 feet but less than 40 feet; or a storage capacity greater than or equal to 50 acre-feet but less than 1,000 acre-feet. The State of Colorado also classifies Ladora Dam as a small dam. According to State criteria, a small dam is defined as greater than 20 feet in vertical height but equal to or less than 40 feet in height and 1,000 acre-feet in capacity, or is greater than 100 acre-feet but equal to or less than both 1,000 acre-feet in capacity and 40 feet in vertical height.

4.3.2. **Hazard Potential Classification.** According to Army guidelines, Ladora Dam is classified as having a significant potential for damage in the event of failure. A significant hazard classification is defined as having potential for appreciable property damage (notable agriculture, industry or structures) and no loss of life (no urban developments and no more than a small number of inhabitable structures downstream). The hazard potential was determined by the resulting consequences of failure of the dam on a clear, sunny day and routing of the flood wave downstream. The failure of Ladora Dam would potentially cause failure of Lake Mary Dam and a high railroad embankment located on the Arsenal immediately downstream of Lake Mary. Outflow from these failures would pond behind the Burlington Northern Railroad (BNR) embankment along the northwest boundary of RMA until the ponded water reached an elevation high enough to flow northeast parallel to the tracks. Flows would eventually reach a low point in the BNR embankment about 500 feet north of Ninth Avenue. Failure of the BNR would most likely occur at this location. The resulting outflows would spread out over a rural area northwest of the Arsenal. Without the BNR embankment located along the northwest boundary, Ladora Dam would be classified as having a high potential for damage or loss of life in the event of failure. This is because outflows would pass directly into a residential area instead of being diverted to a rural area. The State of Colorado classifies Ladora Dam as a Class III dam. A "Class III" dam is a dam for which loss of human life is not expected, and significant damage to structures and public facilities (rendering the structures uninhabitable or inoperable) is not expected to occur in the event of a failure.

4.4. **Basin Description.** Ladora Dam and Lake is located in the Irondale Gulch Drainage Basin on RMA about 10 miles northeast of Denver, Colorado. Drainage area at the dam site is 15.86 square miles. The drainage basin has a total length of 8.0 miles and an average width of about 2.0 miles. The topography of the area is gently undulating and drainage patterns are not well defined. Slopes along streams in this area average about 30 feet per mile. The predominant cover in the watershed is grassland. Presently, approximately 30 percent of the Irondale Gulch Drainage Basin upstream of Ladora Dam is urbanized and is anticipated to increase do to the construction of the new Denver airport.

4.5. **General Geology.**

4.5.1. **Physiography.** Rocky Mountain Arsenal (RMA) is located within the Colorado Piedmont Section of the Great Plains physiographic province. This section consists of a late mature to old, elevated plain with a low rolling topography. The Arsenal itself is located on the eastern edge of the broad valley of the South Platte River, east of the foothills of the Front Range of the Rocky Mountains. Topographic relief across the entire Arsenal is approximately 200 feet, with the land surface generally sloping northwest toward the South Platte River.

4.5.2. **Overburden.** The overburden consists primarily of alluvial clays, sands, silts, gravels and some cobbles in various combinations. Above the bedrock, the soils are Quaternary alluvial deposits ranging from 0 to 70 feet in thickness, with irregular, braided channel deposits and lenses characteristic of alluvium. Occasional calcareous cemented zones occur in the alluvium and may vary from several inches to several feet in thickness. The alluvium is overlain in places by more recent deposits of windblown silts and sands.

4.5.3. **Bedrock Stratigraphy.** The Denver and Arapahoe Formations are the bedrock units immediately underlying the Rocky Mountain Arsenal. They consist of deltaic shales, claystones, sandstones, and conglomerates. The Denver Formation is younger and overlies the Arapahoe Formation. The thickness of the Denver Formation at the Arsenal is approximately 250 to 400 feet. Occasional lignite beds are known to occur locally in the Denver Formation.

4.5.4. **Bedrock Structure.** Rocky Mountain Arsenal is situated in the northwestern portion of the Denver Basin. The basin is an oval-shaped structural depression measuring approximately 120 by 70 miles. The basin is filled with approximately 15,000 feet of sedimentary rocks, composed mostly of shales, sandstones and conglomerates, with lesser amounts of limestone. The bedrock immediately underlying RMA has a gentle regional dip to the southeast.

4.5.5. **Seismicity.** The State of Colorado has a relatively low frequency of earthquakes in historic time, with a maximum intensity of Vl on the Modified Mercalli Scale. Three intensity Vl earthquakes occurred in the Denver area in 1965 and 1966, but were attributed to fluid injection at the Arsenal. The affects of other earthquakes were only locally felt, and magnitudes of all earthquakes have generally been less than 4.0 on the Richter Scale. In reference to the Department of the Army TM 5-809-10, "Seismic Design for Buildings," dated October 1992, the Rocky Mountain Arsenal lies within seismic risk zone one where minor damage could be expected from seismic activity. Zone one has a seismic coefficient "Z" value of 0.075.

4.6. **Existing Embankment.** The Ladora Dam embankment extends south to north across a primary drainageway, then bends northeast from a natural knoll and continues across a shallow secondary drainageway. In 1942, the original embankment, designated herein as the south embankment, was raised 5 feet to its present crest elevation and was also extended to the northeast. The extended portion of the dam is designated in this report as the north embankment. The south embankment is approximately 23 feet high at its maximum section and extends for a distance of 1,220 feet. The north embankment is a uniform dike approximately 10 feet in height and extends a distance of 1,080 feet. The crest elevation of both embankments is approximately 5,227.6 feet. The south embankment has a crest width of 25 feet. Its upstream slope is 1V on 2H from the crest to elevation 5215, then flattens to a 1V on 4H slope to natural ground. The downstream slope is 1V on 4H. A 12-foot wide toe berm was constructed on the downstream slope of the south embankment over the original drainage channel and has a 1V on 1.5H slope to natural ground. The north embankment also has a crest width of 25 feet and has 1V on 3H slopes both upstream and downstream.

4.7. **Existing Spillway.** The spillway is located in the left (south) abutment of the south embankment of Ladora Dam. The trapezoidal-shaped spillway is 4.8 feet below the dam crest and has a 50-feet-wide base width, with approximate side slopes of 1V on 3H. The spillway crest has an approximate elevation of 5,222.8 feet M.S.L. and coincides with the centerline of the service road across the south embankment. During 1989, two concrete drop structures were constructed in the spillway channel 124 feet and 321 feet downstream of the spillway crest. The width of both drop structures is 48 feet. These structures were constructed by RMA to relieve a headcutting problem which developed in 1987. Rocky Mountain Arsenal (PMRMA) has constructed several structures in the spillway in 1993. An earth-filled, riprap-protected embankment with three 24 inch CMP's to convey spillway flow was placed across the spillway in order to provide access for a hiking path to the new prairie exhibit around Lake Mary. Rock-filled, gabion baskets were placed within the flowline of the spillway channel in an attempt to relieve the present headcutting. The drop structure closest to the spillway crest has been altered by the inclusion of a flume to measure flows at the downstream end of the structure. Refer to Plate No. 3 for a plan view of the project.

4.8. Existing Outlet Works. The outlet works facilities which exist for Ladora Dam consist of a 8-inch siphon and a pumphouse. The siphon was installed through the maximum section of the south embankment for the purpose of drawing down lake Ladora and filling Lake Mary just downstream. The pumphouse is located between the north and south embankments on a natural knoll. The pumphouse is capable of pumping water from Ladora Lake to industrial buildings located to the north of the lake or to a 500,000 gallon elevated storage tank located to the north of Lower Derby Lake. Records on file at RMA indicate that at one time (prior to embankment reconstruction in 1942) a drainage line extended through the south embankment of Ladora Dam; however, no evidence of this line has been observed during the field inspections conducted to date. It is possible that this drainage line was either buried or removed during reconstruction. Refer to Plate No. 3 for a plan view of the project.

5. Construction History: Information is not available relating to the original construction of Ladora Dam; however, it is known that the dam was constructed for irrigation purposes. In 1942, the crest of the dam was raised 5 feet above its original elevation of 5,222.8 feet M.S.L. in order to enlarge the reservoir. Prior to being raised, the dam had a top width of 8 feet, upstream slope of 1V on 3H and downstream slope of 1V on 2H. The dam had a pipe drainage line extending through the bottom of the fill section; however, as discussed in the paragraph above, no evidence of this line exists today. It is possible that this drainage line was either buried or removed during reconstruction. In the enlargement of the dam, all embankment surfaces, including the crest road and slopes of the dam, were stripped of vegetation and scarified to a depth of approximately 18 inches. The upstream slope was retained at a slope of 1V on 3H in order to prevent new fill from obstructing the original outlet drainage structure.

6. Operating Procedures: The operating procedures for Ladora Dam are the responsibility of the Program Manager Rocky Mountain Arsenal (PMRMA). The operating procedures are documented in the 1995 Water Management Implementation Plan.

7. Problem Evaluation: Ladora Dam in its existing condition does not meet the hydrologic requirements necessary for its size and hazard classification. It has been recommended in past inspection reports performed by the Corps of Engineers-Omaha District that permanent rehabilitation measures be taken to improve the hydrologic adequacy of Ladora Dam and Lake. Such measures should include raising the embankment, enlarging the existing spillway, constructing a secondary spillway, armoring the dam embankment with a material such as Roller Compacted Concrete (RCC), or any combination of these. A survey was recently conducted which a new datum was developed. The following evaluation utilized that datum.

7.1. Hydrologic Evaluation. The hydrologic analysis for the Ladora Dam Rehab involved: developing new elevation-area-capacity relationships for Ladora Reservoir from the new 2' topographic mapping and routing previously developed inflow hydrographs through the reservoir to determine the flow through the new spillway and the required height of embankment.

7.1.1. Elevation-Area-Capacity Relationship. The new 2' topographic mapping was digitized to determine the surface areas for given elevations. The average-end area method was used to determine the elevation-capacity relationships. Table 1 shows the calculated relationships. Figure 1 shows the elevation-capacity relationships in terms of capacities for feet above the spillway crest along with a comparison of the elevation-capacity relationship calculated for the *Irondale Gulch Stormwater Evaluation*, by Woodward-Clyde Consultants, May 1993. As the figure shows, the two relationships are identical.

Table 1
Ladora Reservoir
Elevation-Area-Capacity

Elevation (ft msl)	Area (acres)	Incremental Volume (acre-feet)	Total Volume (acre-feet)
5222	59.4	---	---
5222.8 (Spwy. Crest)	64.6 (Interpolated)	---	---
5224	72.5	82.3	82.3
5226	85.0	157.5	239.8
5228	98.4	183.4	423.2
5230	114.8	213.2	636.4
5232	132.6	247.4	883.8

7.1.2. Dam Safety/Spillway Design Criteria. *The Ladora Reservoir Risk Analysis Report*, May 1994, showed that the 100-year flood event would increase stages downstream of the Arsenal 0.6 feet for the failure and no failure conditions for Ladora Reservoir. Although no economic analysis was performed, it was assumed that there would be no significant increases in flood damages for this increase in flood stage, and the Ladora Reservoir Spillway should be designed to safely pass the 100-year flood event.

7.1.3. Design Inflow Hydrograph. The 100-year inflow hydrograph was previously developed for the *Ladora Reservoir Risk Analysis - Addendum One*, October 1994. An issue in the determination of the 100-year inflow hydrograph is whether or not the outflow from Havana Street Dam, located upstream of Ladora Reservoir, will flow into Ladora Reservoir or bypass Ladora Reservoir. The area between Havana Street Dam and Ladora Reservoir was modelled to determine the percentage of outflow from Havana Street Dam which will flow into Ladora Reservoir. The intervening area between the two reservoirs was modelled as a series of interconnected depression areas or cells. Flows were modelled as either bypassing or flowing towards Ladora Reservoir based on discharge rating curves developed from the weir equation. The elevation-capacity relationships of the interconnected depression areas were developed from the new 2' topographic mapping. The results of the analysis show only a small percentage of Havana Street Dam flows actually reach Ladora Reservoir, and of the flows reaching Ladora Reservoir, the peak flow rate is greatly attenuated and lagged as the flows are routed through the storage cells. Consequently, for the design of the Ladora Rehab, Havana Street Dam outflows were modelled as bypassing Ladora Reservoir.

7.2. **Geotechnical Evaluation.** The existing embankment in its present condition is in need of repair. The following observations of the embankment and its condition are discussed below. This evaluation was the result of the last formal field inspection made by the Corps of Engineers-Omaha District on April, 2, 1996 as well as data obtained during the July 1994 installation of five piezometers.

7.2.1. **Upstream Slope.** The upstream slopes of both north and south embankments were in fair condition. Areas of erosion scarps, similar to those repaired in 1993, were noted along the waters edge of both north and south embankments. It appeared the scarping was caused by precipitation runoff and wave action. It was recommended that scarping be repaired with the same rock and procedure as was done in 1993. Other than the scarp areas previously repaired, the embankment is virtually unprotected. Riprap which included bricks and large pieces of concrete were found placed along the upstream slope. Riprap of this nature, acts to accelerate the erosion rather than combating it. These unprotected areas were showing more surface erosion on this inspection than on the previous inspection. It was recommended on this and previous inspections, that during rehabilitation of Ladora Dam the entire upstream slope of the embankment should be regraded and a properly designed slope protection be added across the entire face.

It was noted during previous inspections, several large trees were growing in or close to the embankment on the upstream slope of the south embankment at the south abutment/embankment contact and on the upstream slope just south of the pumphouse. The roots of trees can create potential seepage paths through the embankment and foundation. This is especially true for decaying roots if the trees should die. Also, it is possible for these trees to be blown over during a storm potentially creating a hole in the embankment. As recommended during previous inspections, these trees should be removed and the roots grubbed down to about 2 inches in diameter during rehabilitation of Ladora Dam. As was the case during previous inspections, a number of rodent holes were observed on both the upstream and downstream slopes. It was recommended that the rodent holes on the embankment be backfilled with impervious material and the rodents be controlled.

7.2.2. **Embankment Crest.** The embankment crests on both the north and south embankments were in good condition with no unusual cracking, misalignment, differential or excessive settlement observed. The crest road on the south embankment was resurfaced in 1992 and the crest road on the north embankment was resurfaced in 1988. Both asphalt surfaces appeared in good condition.

7.2.3. **Downstream Slope.** The downstream slope of both embankments and the downstream area of the south embankment were inspected and found to be in fair condition. Grass cover was sparse over most of the downstream slopes of both embankments. No visible seepage or wet areas were noted during the inspection of the downstream slope of the north embankment.

A seepage area was noted on the downstream slope of the south embankment near the south abutment/embankment contact just upstream from Lake Mary. This wet area has been observed on previous inspections. Cattail growth and other hydrophilic vegetation is typically heavy in this area. This area can typically be described as moist and soft with some areas of ponded water. Based on past inspections, it appears that the size and amount of ponded water depends from year to year on the pool elevation.

In addition to the previously documented wet area, a new wet area was found during this inspection. The area was located along the north side of the south embankment approximately 30 feet downstream from the toe. It appeared that the area was approximately 2 feet above Lake Mary with dimensions of, at least, 50 feet by 20 feet. This area may have been present during past inspections, but was not visible due to the heavier vegetation. As stated previously, the vegetation was not as heavy during this inspection allowing for a better visual inspection.

7.2.4. **Geotechnical Investigation.** In 1994, five open tube piezometers were installed in the dam in an attempt to identify the phreatic surface within the embankment, determine the hydraulic gradient within the embankment and to classify the embankment materials. Piezometer data taken over a period of time with a wide range of pool elevations enables an evaluation of the effect of the pool on downstream seepage, enable the identification of potential piping areas and enables the identification of potential slope stability problems. The piezometers were installed in two lines (lines A&B). The instrumentation details, embankment cross-sections and data can be observed on Plates 5,6 and 7.

The data from each of the five piezometers exhibits direct responsiveness to the pool with a lag time of 3 or 4 days. The piezometers measures a flat phreatic gradient along both lines of piezometers with little head loss through the embankment. Data was used in a seepage model to obtain the in-situ hydraulic conductivity, seepage quantity, and exit gradient in order to determine if any concerns could develop given an high pool. The conclusion of this analysis is that Ladora Dam seeps water through the pervious foundation and embankment at a high rate into Lake Mary downstream. The seepage model calculated the exit gradient at the toe of the dam and factor of safety against piping. The lowest factor of safety (FS) obtained was 1.1. This FS was calculated with the pool at the dam crest. It is recommended that measures be taken in the design of the rehabilitation of Ladora dam to raise this factor of safety. This analysis can be found in Appendix A of this report.

During installation of piezometers, geotechnical properties were gathered from SPT's and laboratory tests to better evaluate the internal condition of the embankment. The following geotechnical data was obtained.

- Boreholes were field logged and periodically sampled and lab classified according to the unified classification system. The field logs and lab results show a homogenous embankment that varies little in material properties from the foundation. The soils encountered in the embankment at Ladora Dam consisted of silty and clayey sand (SM and SC), likewise, the foundation material consisted of sandy clay (CL) to silty and clayey sands (SM to SC) and varies between 70% and 86% sands. See Appendix A for logs and lab results.

- SPT's were taken in two of the three boreholes located through the crest of the dam. SPT's were taken every 5 feet and extended into the foundation of the 23 feet high dam. These holes are A-6 and B-9 which are both located on the upstream edge of the dam crest. A-6 is located near the maximum dam section and B-9 is located near the left abutment of the dam. The SPT results in both boreholes show that the embankment soils (SC-SM) are in a very loose state with blow counts between (1-4) blows per foot. The left abutment appeared to be in a more dense state (11-14) blows per foot.

- None of the five boreholes were drilled to bedrock; however, the elevation of bedrock (shale) is near 5165 which is taken from the base wide bedrock contour maps.

8. Design.

8.1. **General.** This design analysis presents details on the selected remedial repair of Ladora Dam. The selected remedial repair was determined jointly by the Program Manager for Rocky Mountain Arsenal and the U.S. Fish and Wildlife Service based on information presented in the Concept and 65% design submittals. The remedial repair includes the following:

* Replace the south embankment.
* Raise and regrade the north embankment.
* Gravity flow outlet works.
    - Concrete intake structure.
    - 36in. Reinforced concrete pipe.
    - Grass lined outlet channel.
* Outlet Works diversion into Lake Mary.
* 100 Feet wide trapezoidal earthcut spillway.

8.2. Reservoir Routings.

8.2.1. Design Criteria and Constraints. During the initial phases of this design, an agreement was reached between the COE and Rocky Mountain Arsenal that the spillway and embankment improvements to Ladora Dam would be designed based on the following criteria: The spillway and embankment improvements would be sized using a 100-year design inflow. The designed improvements would provide the minimum of; (1) 3-feet of freeboard above the maximum pool elevation assuming no upstream dam failure (Upper Derby Dam) or (2) would pass the 100-year flood assuming failure of Upper Derby Dam without overtopping the embankment.

8.2.2. Final Reservoir Routings. Reservoir routings were run using the previously mentioned with and without upstream dam failure inflow hydrographs and the design spillway rating curve to determine the top of dam elevation and peak spillway discharge. The final spillway design width was determined through an iterative process in which various configurations of spillway bottom width, side slopes, invert slopes, location of grade control structures, location of crest and crest elevation were examined. The final configuration was arrived at as discussed in the hydraulic design section of this Design Analysis. Table 3 shows the results of the final routings. Both scenarios show that either the condition for upstream dam failures with no freeboard or the condition of no upstream failure with 3' of freeboard would be satisfied with the design embankment crest of approximately 5227.5 ft msl. The embankment will be built to elevation 5228.5 which include one foot of overbuild for settlement purposed. Figures B-3 and B-4 show the inflow, outflow, and pool hydrographs for the with and without upstream dam failure conditions respectively.

Table 3
Final Reservoir Routing Results

Inflow Hydrograph Includes Upper Derby Failure	Peak Inflow (cfs)	Peak Outflow (cfs)	Max. Pool Level (ft. MSL)	Required Freeboard (feet)	Required Embankment Height (ft MSL)
Yes	3130	2340	5226.4	0	5226.4
No	790	680	5224.5	3	5227.5

8.3. Embankment Dam Design.

8.3.1. South Embankment. The south embankment design consists of excavating the existing south embankment to natural ground and replacing it with compacted fill. The compacted fill will consist of a clay core and a clay filled shallow cutoff trench. The rest of the compacted fill will be random fill material from the spillway, existing south embankment, and outlet channel excavations. The new embankment would be constructed to elevation 5228.5 with the elevation of the emergency spillway crest at elevation 5222.8. The hydrologic crest elevation of the embankment is at elevation 5227.6 as determined from hydraulic routings with freeboard as discussed in section 7.1 of this report. The constructed elevation of the embankment placed at EL 5228.5 is to account for anticipated settlement of the embankment and foundation. The new embankment section will include the installation of a sand-gravel drain system which would function as a foundation trench drain. The embankment will consist of zoned fill with a clay core and a three-foot-deep clay cutoff trench in order to reduce through and underseepage. The cutoff trench is designed to improve the foundation that the embankment will be bearing against as well as to reduce underseepage. The plan, profile, and typical cross-section of this design are shown on the construction plans. Prior to excavation of the embankment, the existing topsoil on the embankment slopes, shall be stripped from the embankment slopes and stockpiled for re-use. The existing asphalt road will be required to be demolished and removed off-site. The upstream slope of the dam will require slope protection based on the result of the existing dam history of erosion of the upstream slope from wave action. The majority of the borrow materials, to be used to construct the new embankment, shall be taken from the spillway excavation. Suitable materials excavated from the existing dam may be re-compacted as borrow materials for the new dam. Boring logs including soil classification test data for the spillway excavation area was performed by Harding Lawson Associates. These logs and soil classification data can be found in appendix A of this report.

Dewatering of the existing lake will be required. This work will be done by Rocky Mountain Arsenal prior to construction.

8.3.2. **North Embankment.** The north embankment design consists of excavating, grading, and compacting new earthfill to re-establishing the 3:1 upstream slope and the new embankment crest elevation of 5228.5. Prior to earthwork activities the embankment slope shall require stripping of the existing topsoil and vegetation from the dam and stockpiled for reuse. The existing asphalt road will be required to be demolished and removed off-site.

8.3.3. **Rock Riprap Design.** It is recommended to use the same size, layer thickness, and gradation of rock and bedding materials for the embankment rock riprap as was used on Lower Derby Dam. The following riprap gradation and thickness would be required to be placed on a 3:1 Ladora Dam embankment slope (See construction plans for details).

Gradation of the Rock Riprap:

Percent Lighter than	Stone Weight(LBS)
100	100
30-50	25
10-25	10

Bedding Gradation:

Sieve Size	Percent Passing by WT
2-1/2"	100
2"	95-100
1"	35-70
1/2"	10-30
#4	0-5

8.3.4. Foundation Design. The foundation soils for the south embankment of Ladora Dam are saturated because of the presence of Lake Mary Dam which backs water against the downstream toe of Ladora Dam therefore restricting free drainage. This will present stability and seepage/piping design concerns for the new embankment.

8.3.5. Seepage/Piping Design. A detailed seepage analysis was prepared for the existing dam and the new embankment. The U.S. Army Corps of Engineers Seep2D computer program was used to model the dam. The embankment was modeled for both the high pool and normal pool conditions. The results of the analysis of the existing dam indicate the need for seepage control for the dam. Based on the installation of the piezometers, the foundation soils were found to consists primary of layers of silty-sand (SM), silty-clay (SC), and poorly graded sands (SP). Because of the scarcity of relatively impervious soils such of clays a zoned filled dam is proposed. The cross section consists of a clay core with a random fill shell. Seepage quantities and uplift gradients for the new embankment were computed using the Seep2D model and using the geotechnical design values estimated in appendix A. An impervious core and foundation trench drain were shown to be necessary in order to obtain a satisfactory factor of safety for failure due to piping. Computations can be found in appendix A.

8.3.6. Slope Stability. The following geometric cross-sectional shape of the embankment was developed based on the on-site borrow material identified from site investigation and boring logs in Appendix A.

Top width - The embankment top width is controlled because of the presents of a road over the dam. This road is used for public recreation, employee access and occasional tour bus traffic, as per phone call with Dave Eyre, it is planned to design for the minimum road width of 25 feet.

Slope design - Simplified procedures for preliminary determination of embankment slopes were employed as presented in EM 1110-2-1902, Appendix IV and V. The methods are useful for determining approximate embankment slopes prior to more detailed analyses by methods outlined in the same EM. Based on the calculations below and from experience, the slopes for Ladora Dam shall be no steeper than a 3 to 1. This will also facilitate maintenance of the slope. The analysis can be found in Appendix A.

8.3.7. Settlement Analysis. It is anticipated that settlement of the new south embankment will occur due to consolidation of the foundation soils if lake Mary were drained allowing free drainage of the foundation soils. The drain system will drain the saturated low density sandy material of the upper portion of the embankment causing consolidation of the embankment. The total settlement of the improved embankment was estimated by assuming the effective stress, density, liquid limit, and void ratio of the existing (SC-SM, SP) foundation materials based on the SPT's, Grain-Size Analysis, and Atterberg Limit tests performed. The settlement at the maximum section of the dam was calculated to be 0.9 feet (rounded to 1.0 foot) giving a final elevation of 5228.5 for the crest of the dam. This analysis can be found in Appendix A.

8.4. Outlet Works Design.

8.4.1. Design Criteria and Constraints. In order to meet the dam safety criteria for Lake Ladora, a means to lower the pool behind the dam when the pool elevation is below the spillway crest is required. State of Colorado criteria call for Lake Ladora to be lowered five feet within five days. Based on the area-capacity information for Ladora, approximately 200 acre-feet of storage would need to be evacuated within the five day period. This equates to a sustained discharge of approximately 20 cfs. In order to provide flexibility in regulating the pool elevation of Lake Ladora, stoplog structures were desired to be incorporated into the intake design. Due to intense monitoring requirements for all discharges on and off the Rocky Mountain Arsenal, it was desired to install parshall flumes within the outlet channel to measure releases from the dam. In addition, it was desired to be able to divert water from the outlet channel into Lake Mary, also through a parshall flume.

8.4.2. Hydraulic Design.

8.4.2.1. Outlet configuration. A two-way drop inlet riser was selected to provide outlet capacity for the dam. The uncontrolled overflow weir crest was set one foot below the spillway crest elevation of 5222.8. The outlet pipe consists of a 36-inch concrete cylinder pipe and is approximately 100 feet long. The 36-inch pipe was chosen to allow for future inspection capabilities. Inside dimensions of the riser are 3 feet wide (to match the conduit) and 8 feet long. Invert elevation of the riser at the upstream end is 5211.0 with the elevation at the outlet of the conduit downstream of the dam set at elevation 5210.5. Provisions were made to allow some fluctuations in permanent pool through use of a slot on one side of the two-way riser weir. A 3 foot wide by 3 foot deep notch was placed on the upstream south side of the weir. Through use of 6-inch high stop logs, the permanent pool upstream of the dam can be manipulated for a variety of purposes down to elevation 5218.8. A gated port at the bottom of the riser having an 18 inch diameter is proposed for the outlet work structure to meet the 20 cfs discharge as required by dam safety criteria. The outlet riser was situated as far into the embankment as possible to minimize the length of conduit and length of footbridge necessary to provide access to the structure from the embankment.

8.4.2.2. Rating Curve Computations. The outlet works structure was sized to convey approximately 60 cfs at pool elevation 5222.8 ft. msl, which corresponds to the spillway crest elevation. Conveying flows of up to 60 cfs through the outlet works would eliminate the frequent flows through the spillway which have been partially responsible for the damages due to erosion. In order to determine the effective capacity of the outlet works structure, the following pipe flow equations were computed for the riser and conduit: low weir discharge with stop logs removed; high weir discharge with stop logs in place; smooth pipe flow; rough pipe flow; and orifice flow in the riser. The structure was designed in accordance with Corps of Engineers Hydraulic Design Criteria (HDC) 230-1 to 230-1/2. The weir crest will be rounded to increase efficiency. A weir length of 16 feet (8-feet each side) and a weir coefficient of 3.8 was assumed when computing the upper weir discharge rating curve. The rating curve for the 3' wide notch was computed assuming a weir coefficient of 3.1 because the crest will not be rounded. Orifice flow within the two-way drop inlet is an undesirable situation which can cause gulping and surges in the outlet works and downstream channel. Orifice computations were made to ensure that full pipe flow would occur and fill up the riser before orifice flow could develop within the riser. Conduit capacity was computed for a smooth pipe and rough pipe conditions. The smooth pipe condition assumed a Darcy-Weisbach friction factor of 0.009 and is used to design the energy dissipation requirements at the outlet of the conduit. The rough pipe computations are utilized for capacity purposes. A friction factor of 0.018 was assumed for the concrete conduit. Entrance losses were assumed to be 0.70 and a full exit loss was assumed. The tailwater elevation was initially assumed to be about 60% of the conduit diameter. However, upon placement of the parshall flume in the outlet channel to measure discharges, the maximum tailwater increased to 0.5 feet above to crown of the pipe. This tailwater elevation of 5214 was assumed when computing the head on the pipe for capacity computations. The low flow was computed using the orifice equation and a coefficient of 0.62.

The rating curves discussed above are shown on figure 3 in Appendix B. As can be seen on the rating curve, the 18" low flow pipe has a capacity of about 30 cfs with the pool at the crest of the high weir and a capacity of 20 cfs 5 feet below the full pool elevation which meets the drawdown criteria. As the head drops, the capacity decreases. The 3' wide notch at the weir will provide a maximum discharge capacity of about 60 cfs with 3 feet of head at which time the upper weir takes control. Weir control, either the 3' notch or entire 16 foot length of weir will control up to a discharge of about 125 cfs at a pool elevation of 5223.5 which is 0.7 feet above the spillway crest. Above this point, the conduit will control the discharge through the outlet structure. The maximum discharge that can pass through the outlet works is about 145 cfs with water at the top of the dam. The outlet channel was sized to contain this flow without overflowing into Lake Mary.

8.4.3. Structural Design. The outlet works as shown on sheet 14 consist of a cast-in-place concrete intake structure with a pedestrian access bridge constructed from a precast concrete double tee. A concrete steel cylinder pressure pipe extends from the outlet works through the embankment to the diversion structure. The intake tower is approximately 15 ft high by 5 ft wide and 6 ft long. The tower has been designed with the criteria and loading conditions as shown in Appendix C.

8.5. Outlet Works Channel Design.

8.5.1. Hydraulic Design.

8.5.1.1. Design Criteria and Constraints. The outlet works channel was sized to convey the maximum outlet works discharge from the outlet riser. The desires of the Rocky Mountain Arsenal were that the channel bypass Lake Mary and avoid as much as possible the trail system constructed around Lake Mary by the Fish and Wildlife Service(FWS). Diversion capability for small flows into Lake Mary from the outlet channel were desired. In addition, the USGS desired to place parshall flumes in the outlet channel and the diversion into Lake Mary in order to monitor all outflows from Lake Ladora. The USGS will provide a 24" and a 9" parshall flume to place in the outlet channel and Lake Mary diversion respectively. The 24" flume is rated at 23 cfs at 2 feet of head and the 9" flume is rated at about 6 cfs at 1.5 feet of head based on standard discharge tables.

8.5.1.2. Outlet Channel Design. The outlet channel was located along the side hill south of the FWS trail and generally follows the contour of the land. The channel was sized as an 8 foot bottom width channel with 1V on 3H sideslopes. The channel is approximately 1100 feet long and begins at the exit of the outlet conduit and daylights into the spillway at near spillway station 17+50. An invert slope of 0.005 was utilized to match up with the spillway invert elevation at station 17+50. The outlet of the conduit will be ripraped with an 24-inch thick layer of riprap for 20 feet downstream of the outlet. The rock gradation assuming a 24-inch layer thickness is as follows:

% Lighter by Weight	Limits of Stone Weight in pounds	
100	691	276
50	205	138
15	146	62

The assumed channel n-value was 0.030. Three starting water surfaces elevations at the location where the channel daylights into the spillway were evaluated. Case one assumed that the flow goes through critical depth which is a worse case scenario from when looking a maximum velocities. Case two assumed normal depth and case three assumed the maximum spillway flow coincident with the maximum outlet discharge. For case three, the starting water surface was set at the elevation generated by 2380 cfs in the spillway at station 17+50. This case was used to set the left berm elevation for the outlet channel and to compute the tailwater on the outlet conduit. For all three cases, the flow parameters were well within design ranges. During maximum flows in the spillway, the outlet works will be discharging about 140 cfs. The combination of the maximum spillway flow and outlet flow were used to set the left berm elevation along the outlet channel to prevent the flows from flanking or breaching the outlet channel berm and flowing into Lake Mary. The profiles for all three cases converge with 700 feet upstream of where the channel daylights into the spillway. Fig 4 in Appendix B shows a comparison of water surface profiles with normal depth and maximum spillway discharge starting conditions.

The channel velocities are less than 4 feet per second (fps) for flows less than about 100 cfs. Maximum velocities approach 4.5 fps for flows from 120 to 140 cfs. Maximum permissible velocities for unlined, unvegetated channels in silty sand to silty clay material ranges from 2 to 3.5 fps. However, with a vegetated channel in sandy silt material, the maximum permissible channel velocity is 6 fps. See figure 5 in appendix B for a plot of velocities along the outlet channel for a variety of flow rates. The unlikelihood of the high flows and with a source of water from the outlet works readily available, it was assumed that the channel would become vegetated and the expected velocities would be well within the acceptable range. At the maximum discharge of 140 cfs, the depth of flow in the outlet channel is about 2.7 feet.

8.6. Parshall Flume and Diversion Structure Design.

8.6.1. Hydraulic Design. At the request of the Arsenal and USGS, a 24-inch parshall flume was placed in the outlet channel approximately 150 feet downstream of the outlet conduit to promote tranquil entrance conditions to the flume. The furnished parshall flume is undersized for the size of channel and discharge from the outlet works. However, in conversations with the USGS, they are aware of these shortcomings and plan on measuring flows at C-street or some other location downstream when flows exceed the rated capacity of the structure. Therefore, the flume was designed to be overtopped above a discharge of about 50 cfs. The USBR publication Design of Small Canal Structures chapter 5 was utilized in designing the placement of the parshall flume. The 24-inch parshall flume will be placed in the 8-foot bottom width channel and will effectively block the channel for a depth of 2 feet except for the flume opening. This will create a large head loss through the structure. Submergence of the flume will not occur within the rated range (up to about 23 cfs). Placement of the flume in the outlet channel was simulated using the HEC-RAS program which is a one dimensional steady state flow backwater program. HEC-RAS is much like HEC-2 but designed for a Microsoft Windows environment. The flume in the outlet channel raised the maximum water surface profile upstream of the flume by approximately 1.5 feet. The final left overbank berm profile was designed with this in mind. See figure 6 in Appendix B for an example of the effect the flume has on water surface profiles in the outlet channel. The flume will be anchored into the outlet channel with concrete and backfilled. Downstream of the flume, the channel and banks will be armored with 24-inch thick layer of riprap of the same gradation used at the outlet structure for a distance of 10 feet to prevent erosion.

Provisions for diversion of water into Lake Mary from the outlet works was also requested. Nominal flows are required for diversion into the Lake Mary system. Therefore, a smaller channel and flume structure with a capacity from 2-10 cfs is required. The outlet channel invert is about 5-feet higher than the Lake Mary elevation. Therefore, the diversion channel will be very steep and the parshall flume structure will require carefull placement since it will not be effective in a supercritical stream. It is proposed that a sharp crested weir or cippoletti weir be placed in or immediately near the left bank of the outlet channel with provisions for stop logs to be placed upstream of the crest of the weir structure. The stop logs will be utilized to regulate the flow of water from the outlet channel into Lake Mary. The weir will drop about 3 to 4 feet leaving a gentle channel slope and outfall into Lake Mary. According the Design of Small Canal structures, a structure with a weir length of 2-feet will discharge about 6 cfs with one foot of head. This meets the discharge requirements set by the Arsenal. Stoplogs or other channel blocks in the outlet channel can be used to divert a portion of or all the flow into Lake Mary. Provisions shall be made to not place stoplogs in the outlet channel higher than 2 feet in order to limit flows into Lake Mary. This can be done by only placing stoplogs in the parshall flume portion of the channel.

8.6.2. **Structural Design.** The diversion structure as shown on sheet 15 consist of a gated cast-in-place concrete box 6 ft long by 4 ft wide and 7 ft high. The diversion structure has been designed in accordance with the loading conditions and criteria as shown in Appendix C.

8.7. Spillway Design.

8.7.1. **Design Criteria and Constraints.** The spillway design consists of a fixed width 100 feet wide earth-cut channel with 3:1 sideslopes through the left abutment of the dam (looking downstream). Due to the steepness of the channel and material types in the spillway, grade control was required. Alternatives evaluated ranged from a series of low head rock drops to large straight drops to a combination of the two. The selected alternative based on economics, the FWS and Arsenal desires and aesthetics was a series of rock drops. The spillway crest elevation was to remain the same as the existing crest elevation. The spillway crosses one of the main entrances into the Arsenal (C street) downstream of Lake Mary. A 30-inch potable water line is located along the downstream side of C-street and we were to avoid relocation of this water line. The water line was located by trenching with the top elevation at 5200.4 at about the proposed centerline of the new spillway. Therefore, C-street and any culverts through C-street had to be above this elevation. Additionally, there is an existing parshall flume and outlet from Lake Mary that discharges into the existing spillway. The elevation of this flume was 5206.74. Therefore, the spillway invert had to below this elevation for drainage purposes. Other constraints included not impacting existing power lines nor getting too close to Lake Mary to induce seepage from Lake Mary into the spillway.

8.7.2. **Spillway Configuration.** Several combinations of sizes and slopes of spillway were tried including combinations of widths. However, it was found that a constant 100 foot wide spillway with slope of 0.001 would provide the best combination of parameters to meet the constraints and criteria mentioned above. Minor grading is all that is required downstream of C-street. There is an old stock dam embankment located about 300 feet downstream of C-street that must be removed. For this analysis, it was assumed this embankment would be breeched to a width of 150 feet. Arsenal personnel had previously indicated this would be removed under separate contract as part of a remediation project. There is also an area near the natural valley section where the side hill abuts C-street. It was assumed this would be excavated to at least a 75 foot width. Additionally an old access road ties into C-street immediately downstream of where the proposed spillway alignment cross C-street. This road embankment must be removed.

The elevation of C-street was assumed to be at 5206.0 for the width of the spillway (100-feet). The right side of the spillway crossing of C-street will require a berm height of about 3 feet to contain the spillway flows as they cross C-street. The spillway will have a 0.001 slope from C-street upstream to the crest except at the location of the drop structures. The assumed n-value for the spillway channel was n=0.030. The spillway was designed for a peak discharge of 2350 cfs. Velocities in the spillway (exclusive of the rock drops) range from less than 4 fps for discharges less than 1000 cfs to less than 6 fps for discharges up to the design discharge. See figure 7 in Appendix B for the spillway velocity profile upstream of C-street. The velocities for the design discharge of 2350 cfs approach the maximum permissible velocity for a grass line channel in silty sand material. However, it is cost prohibitive to protect against all erosion especially for extremely remote events. In addition, the duration of flows at the peak discharge with upstream dam failure is less than 1-hour. Flow durations range from about 2-hours for discharges above 2000 cfs to a little over 4 hours for a discharge exceeding 1000 cfs with upstream dam failure. Reference the outflow hydrograph on figure 1 in Appendix B for flow durations. Therefore, the erosion resistance of the spillway is satisfactory assuming the channel becomes vegetated. It is recommended that the Arsenal irrigate the spillway and outlet channels and side slopes for one year following construction to ensure a good grass cover and to prevent rill erosion on the side slopes as was experienced with the original spillway. The spillway rating curve is shown on figure 8 in Appendix B. Computed water surface profiles for the spillway are shown on figure 9 of Appendix B.

8.7.3. Low Head Drop Structures

8.7.3.1. Hydraulic Design. This spillway design alternative utilizes the same trapezoidal shape of the spillway described in section 8.5.1 except that three grade control structures are incorporated for grade control. Several drop heights and combinations of drop heights were investigated. The final design resulted in two 5-foot high and one 4.5 foot highsloping rock structures. The 14.5 feet of drop taken out by the drop structures is required to ensure a non-erosive spillway slope between the structures. At C-street, the downstream embankment of the road will be armored with riprap and function as a fourth sloping rock drop. The height of drop at C-street will be about 8 feet. At C-street, the drop height is less and the slope is flatter than currently exists. Some erosion is expected to occur downstream of C-street but this is assumed to be the natural channel beyond that point.

The rock structures consist of a 15-foot flat crest. Downstream of the crest, the structure slopes at a 1V on 10H longitudinal slope. The large rock then continues for another 15 feet downstream of the toe of the drop. The rock is placed a minimum of 4-feet up the side slopes. As water passes down the sloping drop structure the velocities accelerate. Computations to determine the location of the hydraulic jump, maximum velocities and length of hydraulic jump were performed. The maximum velocity occurred at the design discharge of 2350 cfs. The peak velocity determined for all three structures was 14.7 fps. Using ISBASH turbulent flow criteria in HDC 712-1 for sizing the rock resulted in a rock layer thickness of 54-inches. The length of the hydraulic jump was the basis for extending the riprap protection 15 feet downstream from the toe of each drop structure. The plan and profile of the spillway drop structures are shown in the construction drawings. The riprap gradation required for the sloping rock drop structures is as follows:

% lighter by weight	Limits of Stone weight in pounds	
100	7873	3149
50	2335	1575
15	1168	492

Due to the large rock size, it is recommended that a layer of spalls or 12-inch riprap be utilized in conjunction with the bedding layer.

8.8. Box Culvert Design.

8.8.1. Hydraulic Design. To convey low flows and outlet works discharges through C-street without resulting in closure of C-street every time the outlet works flows, provisions were made to provide limited drainage underneath C-street. A 6 ft wide by 3 ft high reinforced concrete structure is required at C Street to convey the outlet works discharges downstream. These structures would require a drop inlet to allow spillway flows of up to 120 cfs from the outlet works channel to pass under the road embankment before overtopping of the road embankment would occur. This structure size is based on a road embankment that is elevated above the invert of the emergency spillway. A low flow channel will be placed in the spillway to convey the outlet works flows through the spillway into the box culvert. The culvert size was based on not impacting the water line on the downstream side of C-street and still passing the outlet works discharge through the culvert. The construction drawings provides profile views of this road structure.

8.8.2. Structural Design. The low flow structure as shown on sheet 16 consist of a concrete box culvert either cast-in-place or precast and shall have a concrete drop inlet structure at the intake end and a headwall apron structure at the exit end. The inlet structure shall consist of a 21 ft long by 9 ft wide concrete structure with side walls sloping from the top of the box culvert down to a 2 ft high drop at the upstream end. The headwall apron structure shall consist of a 30° flared wingwall concrete outlet structure with apron. The low flow structure has been designed in accordance with the loading conditions and criteria as shown in Appendix C.

9. Cost Analysis. A construction cost estimate was completed for the rehabilitation of Ladora Dam. The cost estimate reflects the assumptions set forth in this Design Analysis Report and the construction drawings. Two cost estimates, which are presented in Appendix D in detail, were prepared in order to compare the spillway alternatives. The cost estimates are summarized below:

Alternative:	Cost Estimate:
1. Rehab Lake Ladora Dam with Spillway Alt. no. 1	**$ 1,786,472**
2. Rehab Lake Ladora Dam with Spillway Alt. no. 2	**$ 1,258,637**

APPENDIX A

GEOTECHNICAL DESIGN

1. Design Assumptions
2. Seepage/Piping Analysis
3. Slope Stability Analysis
4. Settlement Analysis
5. MRD Lab Report
6. Embankment Drilling Logs
7. Spillway Drilling Logs
   & Soil Classification

A. Geotechnical Design. This section covers the design assumptions, computations, and details of the new embankment design discussed in the Design Analysis report. The new embankment was analyzed for the following potential problems: 1) seepage and piping 2) settlement and 3) slope stability.

A.1 Design Assumptions. The following design assumptions and values were made based on the field investigations and laboratory testing during installation of the piezometers in July 1994.

Geotechnical Strength Parameters. The following parameters were estimated based on material type & gradation, lab test results of atterberg limits, along with the SPT's results:

Soil Type: sandy clay and silty sand (SC-SM)
Saturated unit weight = 120 lb/cu.ft.
Angle of internal friction(Φ) = 30°
Allowable bearing pressure = 2,000 lb/sq.ft.

Hydraulic Conductivity: Saturated permeability or hydraulic conductivity (K_H) can be estimated for a compacted state using Hazen's method given in EM 1110-2-1906 for the D_{10} size of a clean sand material.

$$D_{10} = 0.06\text{mm} = 0.0006 \text{ cm}$$

$$K_H = 100\ D_{10}^2 = 100\ (0.0006\text{cm})^2 = 4 \times 10^{-5} \text{ cm/sec}$$

The hydraulic conductivity of the existing in-place embankment material was estimated using the piezometer data and the Seep2D computer model. The piezometers are believed to be adequately measuring the phreatic surface and head loss through the embankment. This data including piezometer data, observed for a constant reservoir elevation, was used in the Seep2D computer model to estimate the in-situ hydraulic conductivity existing within the dam embankment. This was done in a trial and error fashion by adjusting the vertical and horizontal conductivity and using a typical ratio for (SC-SM) soils of K_H/K_V between 5 and 10 until the flownet matched the measured phreatic surface. The values obtained for the in-situ horizontal and vertical hydraulic conductivity for the existing embankment are shown below. See the seepage analysis for the existing embankment for more detail.

$$K_H = 5 \times 10^{-3} \text{ cm/sec}$$

$$K_V = 1 \times 10^{-3} \text{ cm/s}$$

$$K_H = 5 \times K_V$$

From, knowing that the existing dam is in a loose density state (low SPT blow counts), it can be reasonably assumed that K for a well compacted dam will have a much lower value for the same soil materials. The Hazen value is a better estimate of K_H for the new embankment. Therefore, the following values were selected:

Random Fill: $K_H = 2.5 \times 10^{-4}$ cm/s

$K_V = 5 \times 10^{-5}$ cm/s

$K_H = 5K_V$

Clay Core: $K_H = 2.5 \times 10^{-6}$ cm/s

$K_V = 5 \times 10^{-7}$ cm/s

$K_H = 5K_V$

Foundation: $K_H = 5 \times 10^{-3}$ cm/s

$K_V = 1 \times 10^{-3}$ cm/s

$K_H = 5K_V$

A.2 Seepage/Piping Design. The seepage condition was evaluated for determining 1) uplift gradients along the toe of the dam, 2) the location of the phreatic surface through the dam and the exit point, 3) pore pressures, and 4) seepage quantities using the design lake level elevation. To improve the estimate of these parameters, data from two lines of piezometers (seepage pipes) were gathered. The following geotechnical information and data was obtained during the field installation and monitoring of the five piezometers at Ladora Dam.

The results of the Seep2D analysis are as follows:

Seepage flowrate:

Discharge(Q) @ Spillway Crest Elev. - 5222.8 ft. = 40.5 Cu.Ft./day
Discharge(Q) @ 100-Yr flood Elev. - 5227.6 ft. = 75.6 Cu.Ft./day

Uplift Exit Gradient: i_{ex} = 0.72 @ Reservoir Elev. = 5227.6 ft.

$$i_{cr} = \frac{\gamma_b}{\gamma_w} = \frac{49.6\frac{lb}{yd^3}}{62.4\frac{lb}{yd^3}} = 0.8$$

$$FS = \frac{i_{cr}}{i_{ex}} = \frac{0.8}{0.72} = 1.11$$

The results of this analysis show that the existing embankment exhibits a high seepage rate of 75.6 Cu.Ft./day and a low factor of safety for uplift (1.11) for a earthen dam of its height and size. Therefore a blanket/foundation drain is recommended to be designed to both intercept foundation seepage and to increase the factor of safety for piping of the foundation. A clay core and cutoff trench is required to reduce the relative high quantity of the seepage and increase the headloss of the embankment and foundation seepage.

The drainage system shall be placed just into the foundation between the foundation soils and the embankment fill in elevation. In plan, the drain should be placed downstream of the clay core of the dam.

A SEEP2D analysis was prepared for the new embankment comparing the effectivness of the clay core, cutoff trench, and drainage blanket. The results indicate that all these measures are needed to control seepage and to raise the factor of safety for uplift to 1.5.

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PROJECT Ladora Dam Rehab		SHEET NO. 1	OF 5
ITEM Replacement Dam Alt. - Design Assump		BY DDR	DATE 8-16-96
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As stated in the evaluation section of the existing dam alternative in the "Concept design report" the embankment fill zone was found to be in a low density state and has currently a high phreatic water surface with a low factor safety at the downstream toe of the embankment for uplift. There exist remedial treatment repairs to improve the existing dam as described in the "Raise existing dam alt.", however replacement of the embankment as opposed to repair has several advantages in this case

1. Better know the material in the embankment & foundation and can remove any suspect material.
2. Can reduce the seepage rather then strictly controlling it therefore providing more water for its attended use.
3. Reduce the expense of upstream slope protection by flattening the slopes.
4. Make dam more pleasing to the "eye" by using landscape Arch. ideas.
5. Improve the foundation and steepen the embankment slopes to reduce the "foot print" of the dam taking up less area for the Lake Mary area downstream.

The following disadvantages are:

1. Lake Ladora would need to be drained
2. the cost of rehab. would be higher as shown in the cost analysis in the "Concept design Report"

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Laboratory Testing & Field Investigation:

No laboratory testing was completed for the "Concept Design phase" for the rehabilatation of Ladora Dam. However some geotechnical testing and data gathering was obtained during the 1994 installation of piezometers (5 total) into Ladora dam under the RMA dam safety inspection program which is conducted annually by the Corps-Omaha district. The data is presented in the "Concept design Report", including the MRD Lab report and piezometer data graphs. The following testing were completed:

1. Boreholes were field logged and periodically sampled and Lab Classified according to the Unified Classification System. Three bore hole were taken through the embankment and two were taken at the toe.

   The field logs and lab results show that a homogeous embankment exist and that the material very little in material properties in the embankment and foundation. The foundation however has lenses of coarser Sand zones. The material in the embankment and foundation is classified as a SC-SM which varies between 70%-80% sands.

2. SPT's were in two of the three boreholes located through the crest of the embankment. SPT's were taken every 5 feet and extended through the contact of the foundation of the 23 ft. dam. These boreholes are A-6 & B-9, both located on the U/s edge of the dam crest. Bore hole A-6 is located through the maximum section of the dam and bore hole B-9 is intall through the left abutment/embankment contact of the south embankment. In both boreholes the SPT results show that the embankment soils (SC-SM) are in a very loose state with blow counts are in the range (1-4) per foot. The left abutment are in a more dense state (11-14) per foot.

3. None of the five boreholes were drilled to bedrock, however the elevation of the bedrock (shale) is near elevation 5165 which is taken from the basewide bedrock contour maps.

4. Piezometers were installed in July '94 and monthly readings have been taken and graphed along with lake level elevations in Lake Ladora and Lake Mary (tailwater). The PZ's are installed in two percendicular lines (Lines A & B) through the south embankment. The piezometers are because of the location, screen length, homogenius of the embankment

OMAHA DISTRICT	COMPUTATION SHEET	CORPS OF ENGINEERS	
PROJECT Ladora Dam Rehab		SHEET NO. 4	OF
ITEM Design Assumptions		BY DDR	DATE 8-16-96
		CHKD. BY	DATE

and foundation as well as water elevation found during drilling it is believed that the piezometers are adequately monitoring the phreatic surface and that the embankment materials are in a saturated state.

5. As well as soil classification grain size analysis and gradation were also determined for the existing embankment material. Atterbergs test were also taken for the embankment materials

Design Assumptions

The following design assumption and values are made based on the field investigations and lab testing during installation of the Piezometers in July '94.

Hydraulic Conductivity:

saturated permeability or hydraulic conductivity (K_H) can be estimated in a compacted state using the hazen's method given in EM 1110-2-1906 using the D_{10} size.

$$D_{10} = .06mm = .0006cm$$

$$K_H = 100\, D_{10}^2 = 100(.0006)^2 = 4\times10^{-5}\ cm/sec.$$

OMAHA DISTRICT	COMPUTATION SHEET	CORPS OF ENGINEERS	
PROJECT Ladora Rehab		SHEET NO. 5	OF 5
ITEM Design Assumptions		BY DDR	DATE 8-16-96
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Hydraulic cond. con't.

The Hydraulic cond. of the existing in place embankment material was estimated using existing steady state conditions and back figured using piezometer data and lake elevation accociated with the piezometer water levels and using the Seep 2D seepage model. See the seepage analysis for the "Raise the existing Dam alternative for more detail.

$K_H = 5 \times 10^{-3}$ cm/sec
$K_V = 1 \times 10^{-3}$ cm/s
$K_H = 5 \times K_V$

Therefore, Knowing that the existing dam is in a loose density state (given low SPT blow counts) it can be reasonably assumed that K for a well compacted dam will be much lower value for the same soil materials. The Hazen value will be a better value for the new embankment

Therefore use:

Embankment = $K_H = 4 \times 10^{-5}$ cm/s
$K_V = 1 \times 10^{-5}$ cm/s
$K_H = 5 K_V$

Foundation = $K_H = 5 \times 10^{-3}$ cm/s
$K_V = 1 \times 10^{-3}$ cm/s
$K_H = 5 K_V$

A.2 Seepage/Piping Design. The seepage condition was evaluated for determining 1) uplift gradients along the toe of the dam, 2) the location of the phreatic surface through the dam and the exit point, 3) pore pressures, and 4) seepage quantities using the design lake level elevation. To improve the estimate of these parameters, data from two lines of piezometers (seepage pipes) were gathered. The following geotechnical information and data was obtained during the field installation and monitoring of the five piezometers at Ladora Dam.

The results of the Seep2D analysis are as follows:

Seepage flowrate:

Discharge(Q) @ Spillway Crest Elev. - 5222.8 ft. = 40.5 Cu.Ft./day
Discharge(Q) @ Dam Crest Elev. - 5227.6 ft. = 75.6 Cu.Ft./day

Uplift Exit Gradient: i_{ex} = 0.72 @ Reservoir Elev. = 5227.6 ft.

$$i_{cr}=\frac{\gamma_b}{\gamma_w}=\frac{49.6\frac{lb}{yd^3}}{62.4\frac{lb}{yd^3}}=0.8$$

$$FS=\frac{i_{cr}}{i_{ex}}=\frac{0.8}{0.72}=1.11$$

The results of this analysis show that the existing embankment exhibits a high seepage rate of 75.6 Cu.Ft./day and a low factor of safety for uplift (1.11) for a earthen dam of its height and size. Therefore a blanket/foundation drain is recommended to be designed to both intercept foundation seepage and to increase the factor of safety for piping of the foundation. A clay core and cutoff trench is required to reduce the relative high quantity of the seepage and increase the headloss of the embankment and foundation seepage.

The drainage system shall be placed just into the foundation between the foundation soils and the embankment fill in elevation. In plan the drain should be placed downstream of the clay core of the dam.

OMAHA DISTRICT	COMPUTATION SHEET	CORPS OF ENGINEERS	
PROJECT Ladora Dam @ RMA		SHEET NO. 1	OF 12
ITEM Seepage Analysis		BY 2DL	DATE 8-2-96
		CHKD. BY	DATE

Evaluate the seepage condition including determining uplift gradients along the toe of the dam, the location of the phreatic surface thru the dam, pore pressures, and seepage quantities. To improve the estimate of these parameters two lines of piezometers (Seepage Pipes) were installed to help gather this data. Locations of these piezometers are shown on Plate No. 1. The following geotechnical information and data was obtained during the field installation and monitoring of the five piezometers at Ladora Dam.

1. bore holes were field logged and periodically sampled and lab classified according to the Unified Classification sys. The field logs and lab results show a homogeous embankment that very little in material properties from the foundation. The material is classified as a SC-SM, and very between 70% to 86% sands. See Appendix A for logs and lab results
2. SPT's were taken in two of the three boreholes located through the crest of the dam. SPT's were taken every 5 Ft. and extended into the foundation of the 23 ft high dam. This holes are A-6 & B-9 which are both located on the upstream edge of the dam crest.

MRO Form 1550, 1 DEC 83

☆ U.S.G.P.O.: 1993-556-577

OMAHA DISTRICT	COMPUTATION SHEET	CORPS OF ENGINEERS	
PROJECT Ladora Dam DRMA		SHEET NO. 2	OF 12
ITEM		BY SDR	DATE 8-2-96
Seepage Analysis		CHKD. BY	DATE

A-6 is located near the maximum dam section and B-9 is located near the left abutment of the dam. The SPT results in both boreholes show that the embankment soils (SC-SM) are in a very loose state with blow counts between (1-4) per foot. the left abutment showed to be in a more dense state (B-7)

3. None of the five boreholes were drilled to bedrock, however the elevation of bedrock (Shale) is near 5165 taken from the basewide bedrock contour maps.

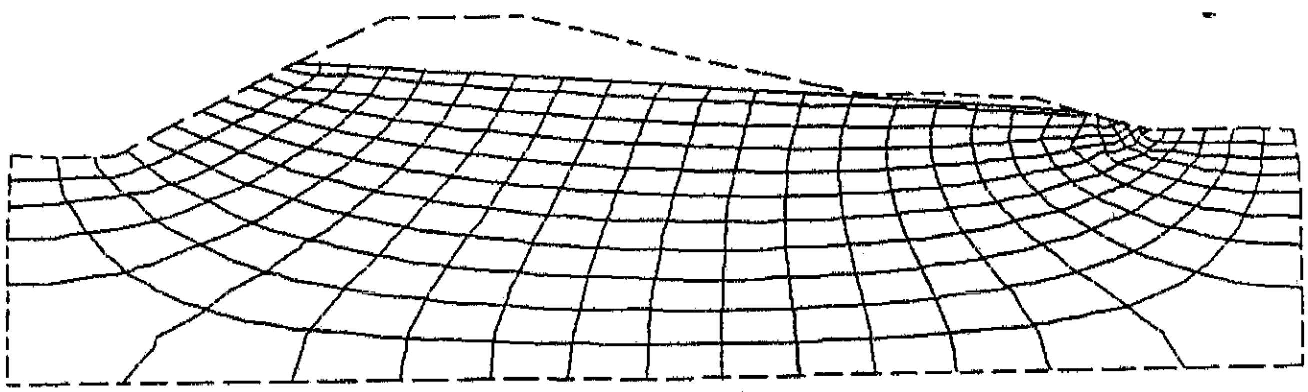
4. the piezometers have been installed in July '94 and monthly readings have been taken for each instrument along with the reservoir elevation for both Lake Ladora and Lake Mary (TW). the piezometers are installed in two lines (Lines A & B). See Plate No. 2 for piezometer locations and screen depth locations. Because of homogenious of the embankment and foundation the piezometers [illegible] adequately [illegible]. the phreatic surface was near as drawn in the embankment. This data including piezometer data, observed for an existing reservoir elevation which was known to have been constant for a long period were used in the SEEP2D computer model. This res. head and seepage profile was used to estimate the insitu hydraulic conductivity existing within the dam embankment. This was done in a trial and error fashion by adjusting the vertical and horizontal conductivity and using a ratio of K_H/K_V between 5 and 10 until the flow net matched the measured

OMAHA DISTRICT	COMPUTATION SHEET	CORPS OF ENGINEERS	
PROJECT Lodora Dam Rehab		SHEET NO. 3	OF 12
ITEM Seepage Analyses		BY DDR.	DATE
		CHKD. BY	DATE

Phreatic surface. Design values for hydraulic gradient and seepage quantity were obtained for the design reservoir by using the insitu hydraulic Conductivity and running Seep2D prg. using the design reservoir head.

RMA.WPT

Ladora Dam
Total Flowrate = 40.420



MROED-HE (SMCRM-ISF-E/9 Mar 87) 1st End Clemetson/plf/FTS 864-4580
SUBJECT: Additional Storm Drainage Waters onto Rocky Mountain Arsenal Property

DA, Omaha District, Corps of Engineers, 1612 U.S. Post Office and Courthouse
Omaha, Nebraska 68102-4978 24 March 1987

TO: Commander, Rocky Mountain Arsenal, ATTN: SMCRM-ISF-E, Commerce City, Co. 80022-2180

1. This is in response to your letter dated 9 March 1987, subject as above. We have included additional information beyond your request, which we hope will assist you in your response to the letter from the City and County of Denver.

2. The Omaha District of the U.S. Army Corps of Engineers has, in the past few years, evaluated surface runoff conditions in the watersheds located on and upstream from the Rocky Mountain Arsenal (RMA). These hydrologic evaluations have included investigations of existing and future conditions with various levels of urbanized development in the upstream watersheds. Our evaluations are documented in the following reports:

a) "Evaluation of the Existing and Future Flood Potential on the Rocky Mountain Arsenal" - report, March 1983.

b) "The Effect of the Parkfield Development on Flood Characteristics of the Rocky Mountain Arsenal" - report, June 1983.

c) "Dam Inspection Report - Rocky Mountain Arsenal" - series of 9 reports, July 1983.

d) "Hydrology for Rocky Mountain Arsenal Lower Derby Dam Spillway Rehab" report, June 1985.

e) "Additional Hydrologic Data Report - Lower Derby Dam" - report, January 1986.

f) "Review and Assessment of Proposed Denver Residential Development - Upland Filing No. 1" - letter to Commander RMA, 9 July 1986.

3. These studies have shown that the potential for flooding in the Irondale Gulch basin is eminent, with associated flood hazards being serious even under existing conditions. Although our studies evaluate flooding under hypothetical scenarios, it has been demonstrated under prototype conditions that the flood threat in the Irondale Gulch basin is real. This was evidenced in May 1973 when Upper Derby Dam was overtopped by flooding conditions and subsequently failed. Consequently, Lower Derby Dam was overtopped, but fortunately, did not fail. If it had failed, the resulting flood damages could have been catastrophic. As you are well aware, there has been additional development in the Irondale Gulch basin since 1973. If the same storm that occurred in May 1973 reoccurred under current development conditions, the flooding potential on the RMA could be significantly greater.

4. In the Irondale Gulch basin, surface runoff from upstream areas enters the RMA via three primary drainage channels. These include the Highline Lateral, the Uvalda Street Interceptor, and the Havanna Street Interceptor. Total drainage area of the Irondale Gulch basin upstream from the RMA is about 11.5 square miles. As of 1982, approximately 32 percent of the Irondale Gulch basin had been developed. The majority of the undeveloped areas lie in the basins that drain into the Highline Lateral and the Uvalda Street Interceptor, which convey waters into the Derby Lakes on the RMA. The Derby Dams and Lakes are for water supply purposes only and were not designed to store or bypass flood runoff. Consequently, under current urbanized conditions, their spillway capacities are grossly inadequate. Any additional development in the upstream areas which would cause an increase in the volume of runoff would increase the existing flood threat and increase the risk of potential dam failure(s) on the RMA.

5. Although our assessments of individual proposed developments (i.e. Parkfield and Upland) indicated that the effects of each development on flooding characteristics at the RMA is minimal, consideration should be given to the cumulative effect of other developments in the Irondale Gulch basin upstream of the RMA. Previous Corps studies indicate that development of the entire Irondale Gulch basin upstream of the RMA would increase the once in 100-year flood runoff volume into the Derby Lakes by about 60 percent. For more frequent events, the percentage increase in runoff volume is even greater. Considering the present inadequate spillway capacity of the Derby Lakes, this could result in a much more serious problem.

6. Based on the 1982 level of development, significant runoff could enter the RMA via the Highline Lateral during storms with frequencies less that once in 5 years, depending on antecedent moisture conditions. This is equivalent to a storm total point-rainfall of about 1.3 inches in one hour, 2 inches in 6 hours or 2.6 inches in 24 hours. Significant runoff is defined for purposes here as the discharge which exceeds the channel capacity. With additional development upstream, storms more frequent than once in 2 years could result in significant runoff. A storm of this frequency is approximately 0.8 inches in one hour, 1.3 inches in 6 hours, or 2 inches in 24 hours.

7. It is our understanding that current regulations in the Denver area only require developers to provide enough detention storage in their development so that the peak runoff rate for a storm with a frequency of once in 100 years will not exceed the rate for historic (undeveloped) conditions. Although other drainage criteria may be proposed, we are not aware that they are currently required in the Denver area. Developer's may argue that because of the 100-year peak discharge requirement, they will not increase the flood threat to the RMA. This statement is entirely false, since the critical factor affecting the flood threat to the RMA dams located in the Irondale Gulch basin is the total runoff volume and not the peak discharge rate. Therefore, unless total retention storage is proposed as part of the development, additional development in the Irondale Gulch basin upstream from RMA will increase the risk of dam failure. Although the increased runoff due to an individual development may be considered minimal, the increased runoff due to several developments collectively would be considered significant.

8. Because of interest being expressed in the past few years by several developers to convey their storm drainage onto the RMA and the desires of the Department of Public Works to cooperate with the RMA without restricting all upstream development, the following recommendations are made for your consideration:

a) Develop a comprehensive plan which would allow development upstream of the RMA and protect the RMA interests. It is recommended that this plan be an effort involving the city and county of Denver and the Urban Drainage and Flood Control District.

b) In the meantime, require all new upstream development to include sufficient storage for total retention of any increased runoff due to the new development. If this is acceptable to the developer, the plans should be submitted for review by our office.

9. If you have any questions regarding this information, please contact Doug Clemetson at FTS 864-4580.

FOR THE COMMANDER:

Wallace R. Ster
for EDWARD W. SIZEMORE, P.E.
Chief, Hydrologic Engineering Branch
Engineering Division

CF:
MROED-MF (Williams)
MROOP-N

Lower Ladley Lake file

DEPARTMENT OF THE ARMY
ROCKY MOUNTAIN ARSENAL
COMMERCE CITY, COLORADO 80022-2180

REPLY TO
ATTENTION OF:

S: 23 March 1987

SMCRM-ISF-E

9 March 1987

SUBJECT: Additional Storm Drainage Waters onto Rocky Mountain Arsenal Property

U.S. Army Corps of Engineers
6014 USPO & Courthouse
ATTN: MROOP-N (Mr. John Morton)
215 N. 17th Street
Omaha, NE 68102-4978

1. Reference letter, City & County of Denver, 2 Mar 87, SAB (Encl 1).

2. Request you furnish Rocky Mountain Arsenal (RMA) information as to the size precipitation event which would be sufficient to allow water to enter RMA via Irondale Gulch.

3. Request you furnish Rocky Mountain Arsenal information as to the size precipitation event occurring in an undeveloped scenario to allow water to enter RMA via Irondale Gulch.

4. Request information NLT COB 23 March 1987 to enable us to make a timely response to City & County of Denver.

5. POC for this action is Lloyd B. Howe, FTS 330-1167.

6. RMA - Providing Soldiers the Decisive Edge.

FOR THE COMMANDER:

David L. Heim

DAVID L. HEIM
Director, Installation Services

Encl

415-10.3 RMA 87-000-155

-ca, 3/ 4/5:

CITY AND COUNTY OF DENVER

FEDERICO PEÑA
Mayor

DEPARTMENT OF PUBLIC WORKS

OFFICE OF THE MANAGER
CITY AND COUNTY BLDG.
DENVER, COLORADO 80202

March 2, 1987

Lt. Colonel Craig M. Dexter
Commanding Officer
Rocky Mountain Arsenal
Commerce City, Colorado 80022

Re: Impacts of Additional Storm Drainage Waters onto Arsenal Property

Dear Colonel Dexter:

Our Wastewater Management Division has proven itself to be quite successful in keeping upstream landowners from developing their properties in such a manner as to increase peak storm drainage flows so as to cause damage to downstream landowners. Although we view this as primarily being a common law relationship between landowners, our experienced engineers who oversee development and resulting storm drainage patterns diligently attempt to prevent major problems. However, whenever development occurs, some increase in total volume of storm waters must necessarily follow.

In the last two months, we have been advised orally that the Arsenal will not accept additional volumes of water, as a general matter, from upstream developments. The consequences of ignoring the Arsenal's wishes in this regard would presumably be some sort of legal action involving the cost of treating contaminated water from Arsenal property.

If all development which lies within drainage basins leading to Arsenal property is to be halted or even subjected to the Army's pre-approval, the consequences could be staggering. To the extent possible, of course, we will seek to cooperate in achieving your needs without even questioning your authority to make such requests. However, in order to make rational decisions in response to current and near-future requests regarding developments, we need a clearer statement of your position. Please advise us as to whether or not the Arsenal is requesting that all upstream development, regardless of size or distance from Arsenal property, be either halted or pre-approved for some period of time.

Your response in the very near future would be greatly appreciated. Without hearing from you in the next thirty days, we will have to assume that we may continue development and drainage decisions with only the normal restraints and considerations. This would not apply to the Chambers Tower Filing No. 1 development, of which your engineers have expressed specific disapproval as currently proposed.

Thank you for your assistance in clarifying the situation for us.

Sincerely,

John S. Mrozek

John S. Mrozek
Manager
Department of Public Works

cc: George C. Rupert, Wastewater Management Division
Steven J. Coon, Assistant City Attorney

ADDITIONAL HYDROLOGIC DATA REPORT

Lower Derby Dam
Rocky Mountain Arsenal

Background. In the original sizing of the new spillway, a risk analysis was performed on the dam with the risk center being the non-federal land located downstream from the northwest boundary of Rocky Mountain Arsenal as was detailed in the previous report dated October 9, 1985. As the previous report stated, for inflows greater than 140% of the SPF, whether or not Lower Derby Dam fails or holds does not have a significant impact on the water surface level downstream at this location. This is due to the relatively large volume of runoff at 6400 acre-feet due to a 140% SPF storm over the entire basin (6.48 inches of runoff over 18.5 square miles) as compared to the relatively small combined storage volumes at Upper and Lower Derby dams of 1075 acre-feet. Thus, for minimum standards, the spillway was designed to safely pass the 140% SPF inflow hydrograph which would have a peak of 10,100 cfs and also at least 1 foot of freeboard. Based on our preliminary geotechnical analysis, a spillway with a bottom width of 275 feet and a 1 foot raise in the top of dam was the optimum configuration. It would have 2 feet of freeboard at 140% SPF or a 1 foot freeboard of 170% SPF as shown on Plate 2. Since the State only requires 1 foot of freeboard, the inflow design flood selected for this study was 170% of the SPF, which has an inflow peak of 12,400 cubic feet per second.

New Studies. Additional conditions of development and no development on the Rocky Mountain Arsenal were required by the State of Colorado to evaluate potential liabilities associated with property damage or loss of life. The three conditions required to make these evaluations are listed below:

a. Routings of 170% SPF with no dams in the basin.

b. Routings of 170% SPF with 275 feet spillway on Lower Derby with no failure of Lower Derby.

c. Routings of 170% SPF with 275 feet spillway on Lower Derby with Lower Derby failing when the water level in the reservoir is within one foot of the top of embankment.

An analysis was made of the water surface level at three cross sections downstream from the RMA northwest boundary for each condition, assuming an eventual failure of the Burlington Northern Railroad embankment in each case.

The routings were performed using HEC-1 to route the flow until it reached the BNR railroad embankment at the northwest boundary of RMA. The flow was then split, with part of the flow assumed to go parallel to the embankment in the north easterly direction, while the remainder of the flow was assumed to overtop and fail the BNR railroad embarkment. The embankment was assumed to have a final failure bottom width of 100 feet with 1 on 1 sideslopes and required 20 minutes to form. The failure hydrographs for the three routings conditions are shown on Plate 3. The volume of water resulting from the failure of the railroad embankment for the three conditions listed above are 2290 Acre-feet, 2540 Acre-feet and 2970 Acre-feet respectively. The hydrographs were inserted into a MRD-Harders routing model of the area downstream of the BNR railroad embankment to determine the peak discharges and stages for three different locations, sections 10.00, 9.62, and 8.82 which are listed below for each condition:

Section 10.00

Peak Discharge (cfs)	Peak Stage (MSL)	Condition
9855	5133.8	170% SPF - Lower Derby fails
9624	5133.6	170% SPF - No dams in basin
8847	5133.3	170% SPF - Lower Derby holds

Section 9.62

Peak Discharge (cfs)	Peak Stage (MSL)	Condition
9728	5131.2	170% SPF - Lower Derby fails
9357	5131.1	170% SPF - No dams in basin
8747	5130.9	170% SPF - Lower Derby holds

Section 8.82

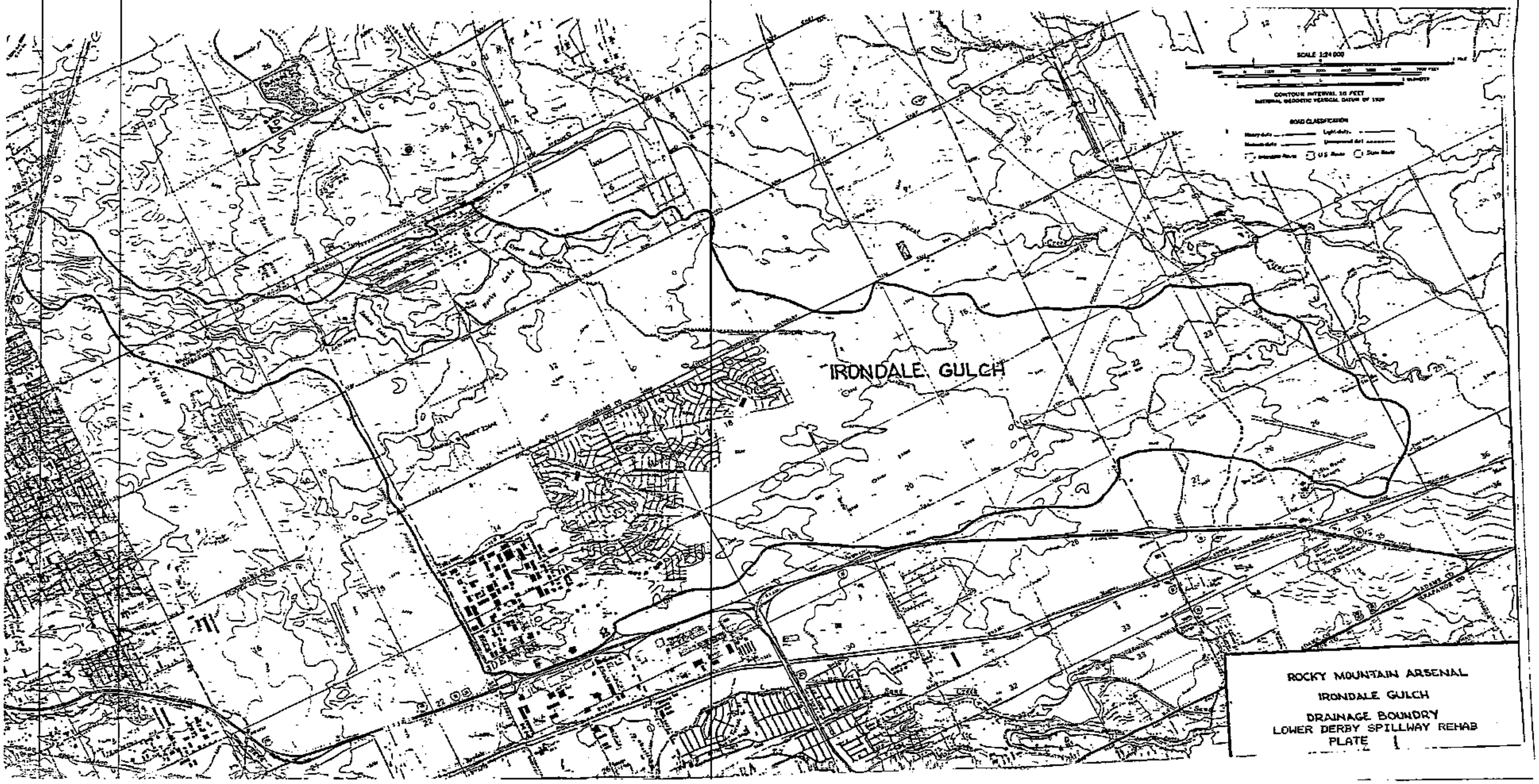
Peak Discharge (cfs)	Peak Stage (MSL)	Condition
9675	5121.0	170% SPF - Lower Derby fails
9260	5120.9	170% SPF - No dams in basin
8690	5120.9	170% SPF - Lower Derby holds

Rating tables which include the effects of backwater condition have been developed from the output at the Harders model and are shown on Plates 4, 5, and 6.

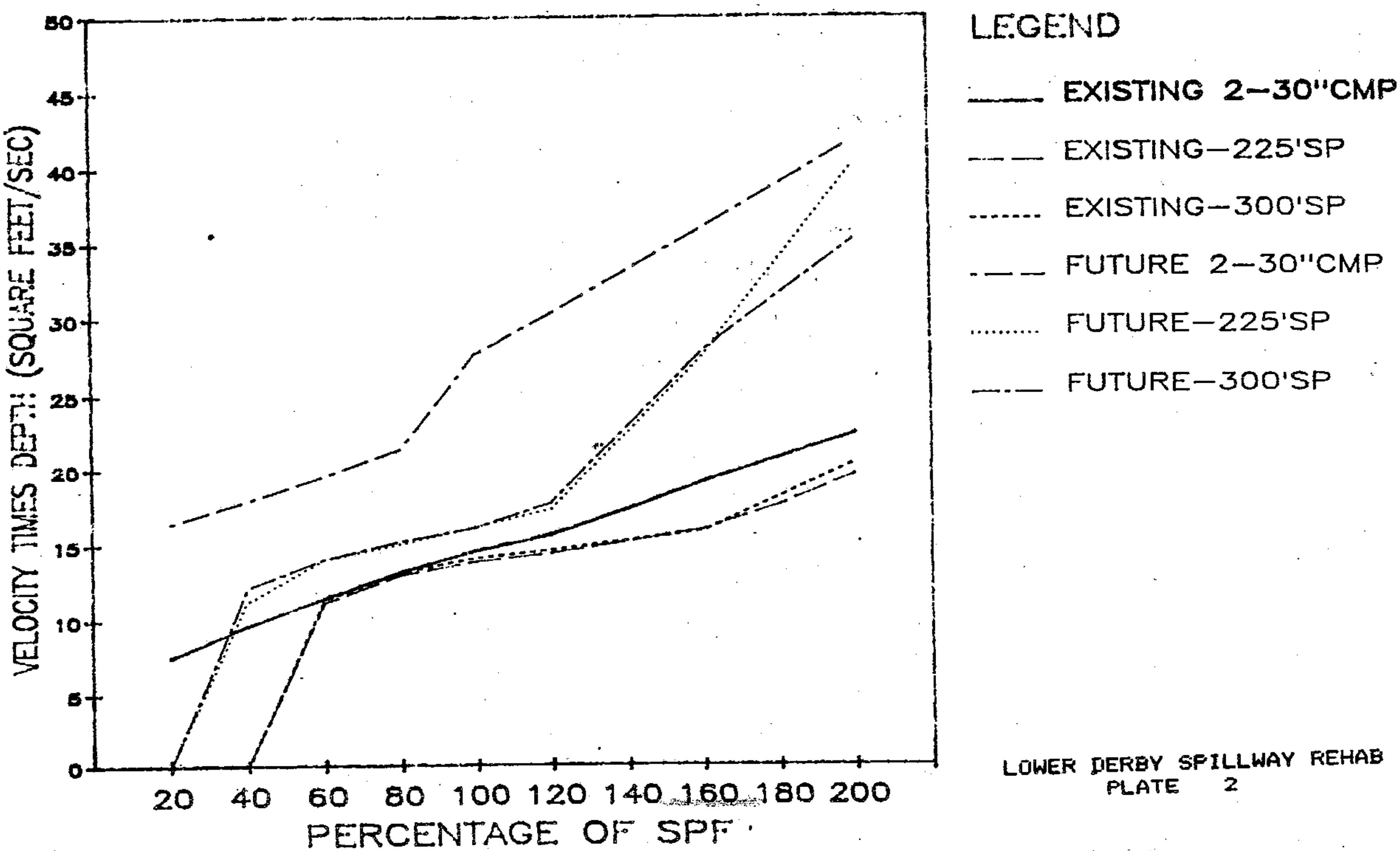
As the above data indicates, there is minimal difference in water surface levels for the three conditions. To put this into perspective, as defined by the State of Colorado, the potential liability of property damages would be the difference in water surface elevations downstream had the dam not been constructed and if the rehabilitated dam would fail at maximum pool. The largest difference in water surface level for this condition would only be 0.2 foot.

Also, defined by the State of Colorado, the liability for loss of life is the difference in water surface elevations downstream with the rehabilitated dam safely passing the spillway design flood versus failure of this dam at maximum pool during the same event. For this condition the maximum difference would be only 0.5 foot.

From these results we can conclude that the spillway rehabilitation project with a spillway of 275-foot bottom width would cause an insignificant amount of additional liability on downstream locations.

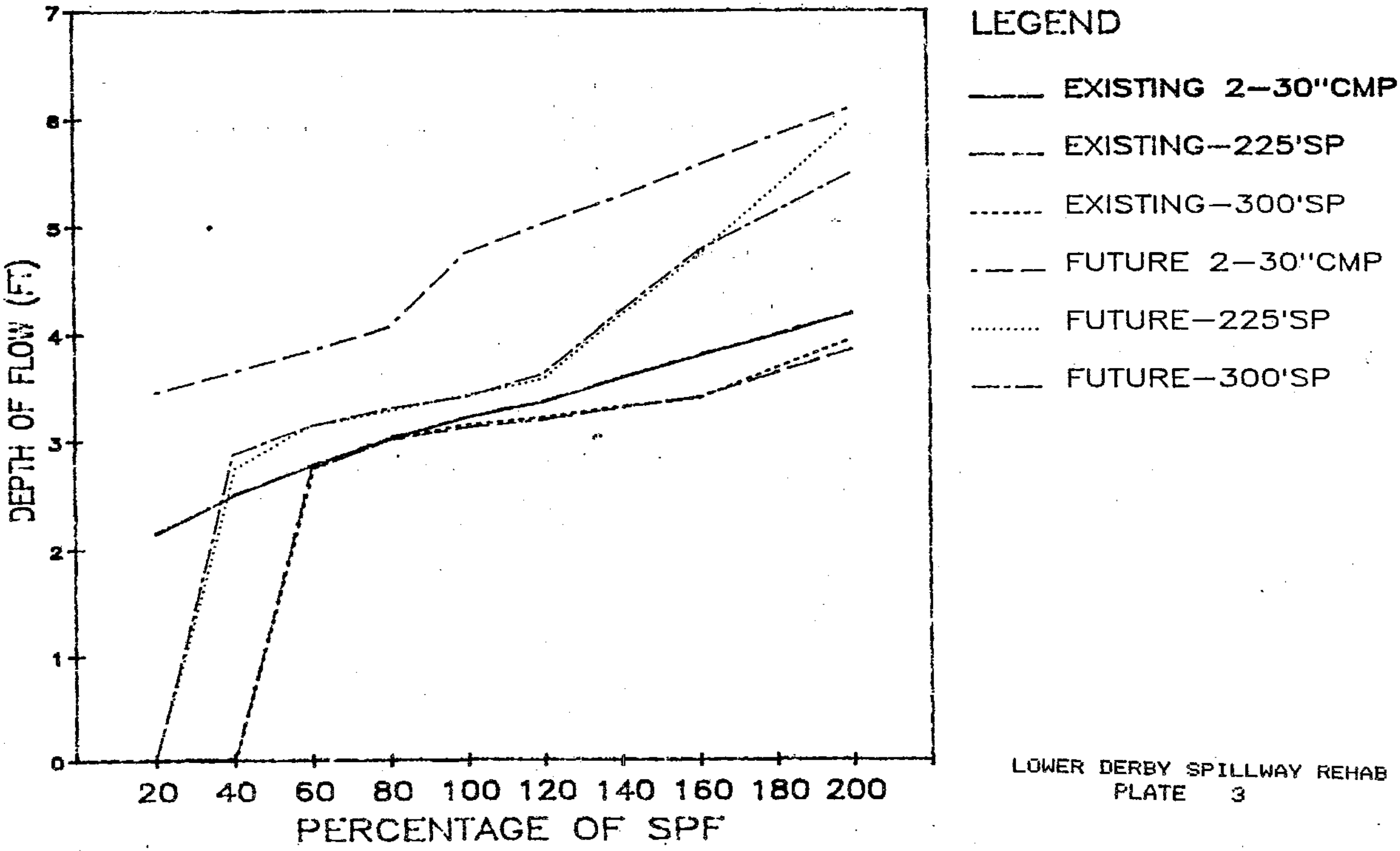

IRONDALE GULCH
SCALE 1:24 000
CONTOUR INTERVAL 10 FEET
ROAD CLASSIFICATION
ROCKY MOUNTAIN ARSENAL
IRONDALE GULCH
DRAINAGE BOUNDRY
LOWER DERBY SPILLWAY REHAB
PLATE 1

VELOCITY TIMES DEPTH VS PERCENT OF SPF AT NORTHWEST BOUNDARY OF RMA FOR EXISTING AND FUTURE URBAN CONDITIONS



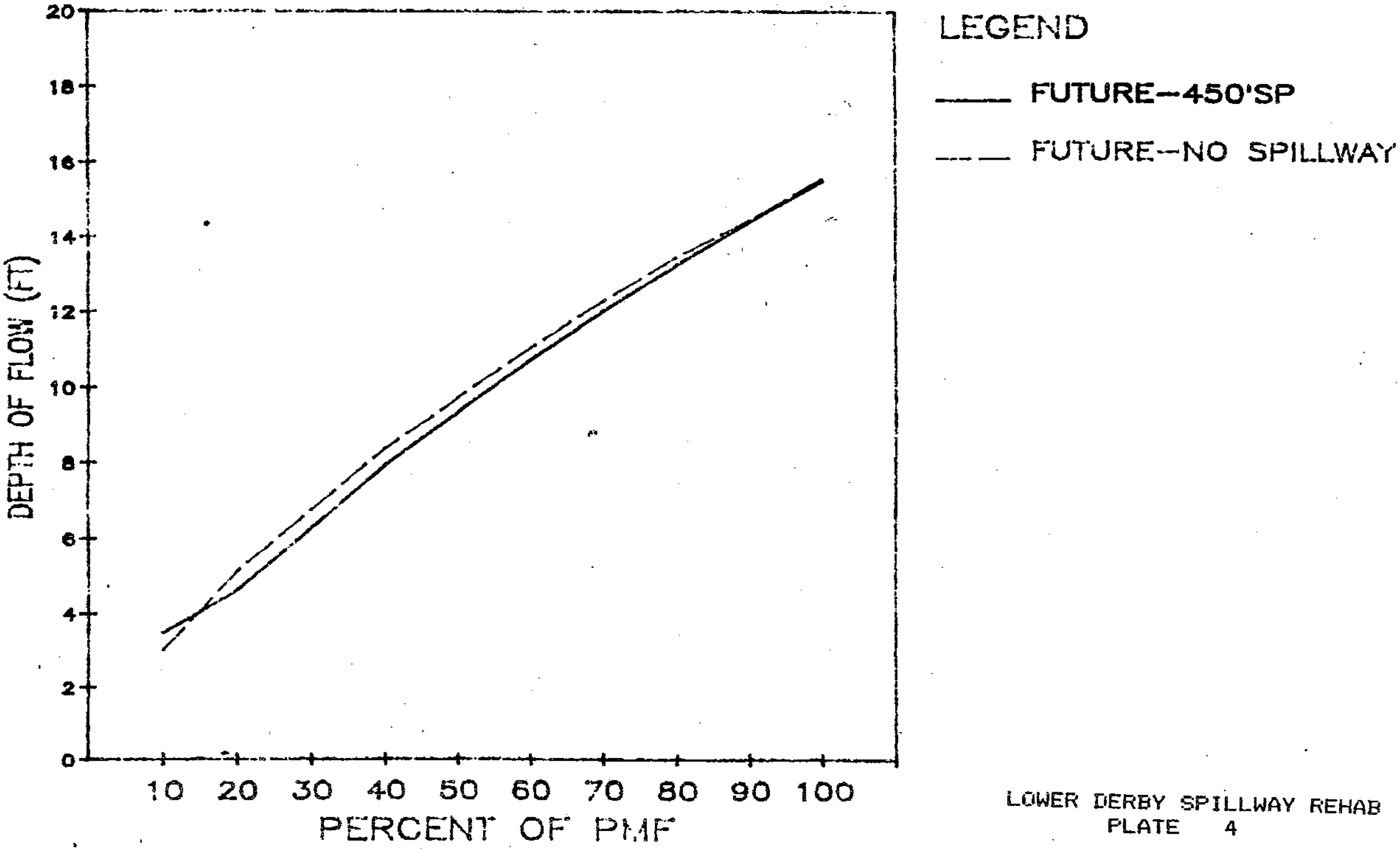
LOWER DERBY SPILLWAY REHAB
PLATE 2

DEPTH OF FLOW VERSUS PERCENTAGE OF SPF AT NORTHWEST BOUNDARY OF RMA FOR EXISTING AND FUTURE URBAN CONDITIONS



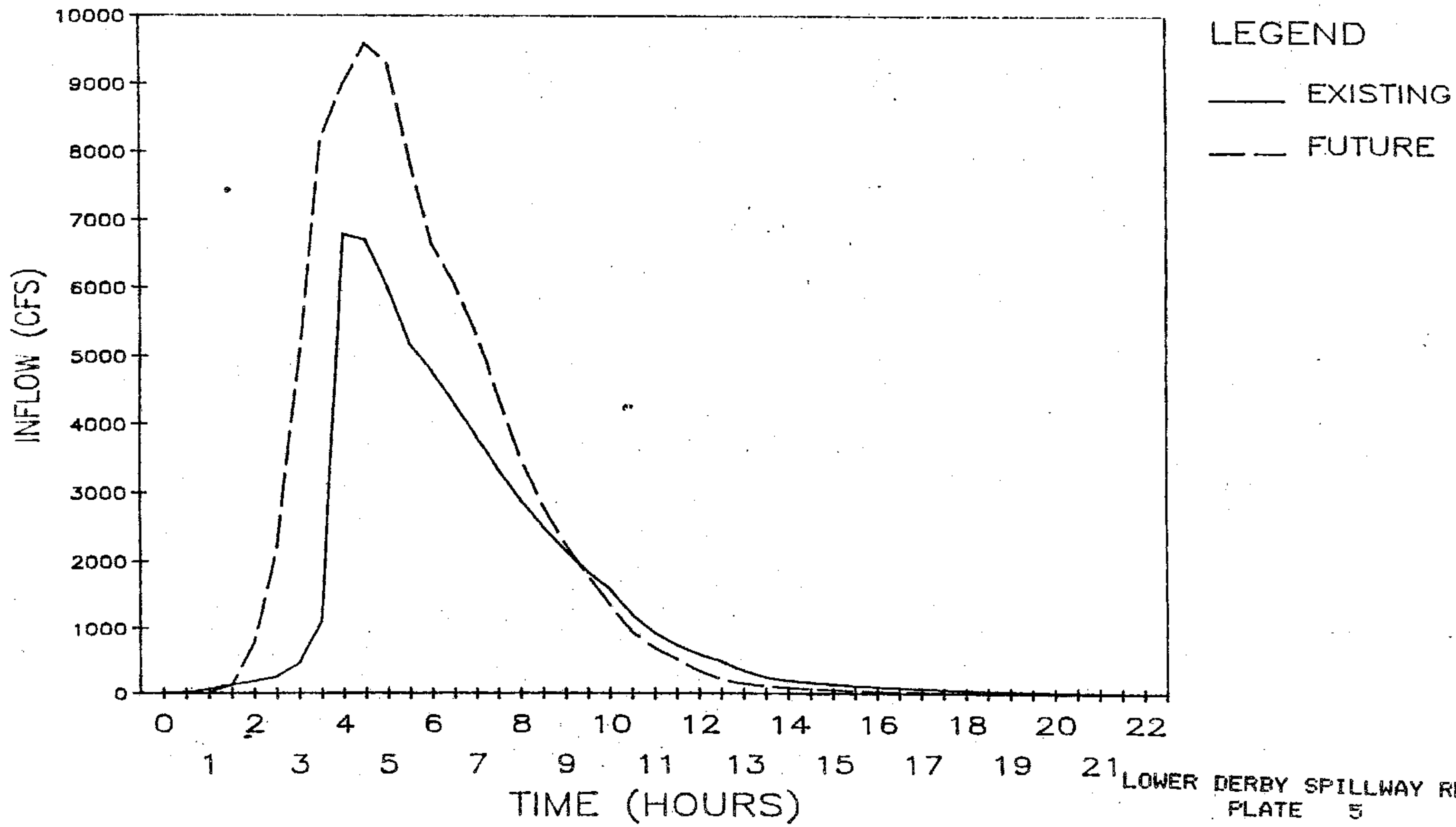
LOWER DERBY SPILLWAY REHAB
PLATE 3

DEPTH OF FLOW VS PERCENT OF PMF AT NORTHWEST BOUNDARY OF RMA PMF STORM—URBAN CONDITIONS



LOWER DERBY SPILLWAY REHAB
PLATE 4

INFLOW HYDROGRAPH FOR LOWER DERBY DAM
140% OF SPF
INFLOW INCLUDES OUTFLOW FROM FAILURE OF UPPER DERBY



LOWER DERBY SPILLWAY REHAB
PLATE 5

MROED-HE 24 June 1985

SUBJECT: Repair and Maintenance of Lower Derby Dam - Rocky Mountain Arsenal

Commander
Rocky Mountain Arsenal
ATTN: SMCRM-ISF (Mr. Jim Green)
Commerce City, Colorado 80022-2180

1. Reference our letter of 24 April 1985. We have completed our hydroloic analysis as discussed in that letter, and two copies of the report are enclosed so that one copy can be forwarded to the State of Colorado, Department of Water Resources, for their review and approval.

2. We have recommended an emergency spillway that has a 275-foot bottom width with a crest elevation set at elevation 5247.2. Its shape is trapezoidal, having 1 on 3 side slopes. The design was based on routing 140 percent of the standard project flood for future urbanized condition through Lower Derby Lake without increasing the maximum lake level above elevation 5253.9 which is 1 foot below the top of the dam. The deviation from using the PMF flood for the design as indicated in our last letter was based on a risk analysis which is discussed in the report. Our estimated cost for the excavation of this new spillway is about $662,000; this compares to just over $500,000 included in our preliminary estimate of the total cost of the project which was enclosed in our last letter to you.

3. We will proceed with our final design effort as soon as we receive notification that your office and the state of Colorado have approved the hydrologic design. If there are any questions, call Mike Kelly at FTS 864-3220 or Wally Stern at 864-4582.

FOR THE COMMANDER:

Encls

T. R. KELL, P.E.
Chief, Engineering Division

CF: w/o encls
MROED-A
MROED-D
MROED-GB
MROED-GE
MROED-HD
MROED-MF

Stern/bsh/4582
Horihan
Monzingo
Hokens
Kell

Enclosure 3

HYDROLOGY FOR
ROCKY MOUNTAIN ARSENAL
LOWER DERBY DAM
SPILLWAY REHAB

U.S. Army Corps
of Engineers
Omaha District

June 1985

Table of Contents

Tables

Plates

HYDROLOGY FOR
Rocky Mountain Arsenal
Lower Derby Dam Spillway REHAB

Introduction. This report presents the results of the hydrologic analysis that was prepared to upgrade Lower Derby Dam to a hydrologically safe dam. This study was made in response to a request from the Rocky Mountain Arsenal Command.

Location. Lower Derby Dam is located in the southwest 1/4 of Section 1, Township 3 South, Range 67 West, inside of Rocky Mountain Arsenal (RMA) in Adams County, Colorado and is considered a high hazard dam.

Lower Derby Dam drains a portion of the Irondale Gulch drainage basin.

Dam Crest = 5253.4 M.S.L. = staff gage of 23.2'

Drainage Area = 9.69 square miles

A map of the drainage basin is shown on plate 1.

Geology. The bedrock in the RMA area is the Laramie Formation which consists of poorly indurated gray, silty and sandy clay, and brown to gray silty, clayey fine sand that is generally impervious. The bedrock within the RMA slopes from southeast to northwest and is cut by numerous buried channels and gullies. The overburden material is comprised predominantly of alluvial sands and gravels interbedded with discontinuous silt and clay layers. These alluvial deposits are 10 to 30 feet thick over most of the RMA area.

Topography and Vegetative Cover. The topography of Irondale Gulch consists of undulating ground with low rolling hills. Vegetation consists mainly of short and tall grasses with some scattered trees along the lakes and along the channels and in certain low areas.

Rainfall and Runoff. The hydrologic design for Lower Derby Dam was developed for the present degree of land development and for a projected future degree of land development. The future conditions involve a general degree of development in the area east of Montbello. The general area was estimated to be 40 percent impervious. The hydrologic design involved using two types of rainfall, a 26" - 6 hour Probable Maximum Precipitation (PMP) based on Hydrometeorological Report No. 55; and an 8" - 6 hour Standard Project Storm (SPS) based on Engineering Bulletin 52-8. Given below are the inches of runoff and volume of runoff for present and future conditions for both storms.

Table 1

Rainfall - Runoff

	Inches of Runoff	Volume of Runoff (acre-ft)
SPF-Storm		
Present Conditions	3.57	1,840
Future Conditions	4.63	2,390
1.4 X SPF		
Present Conditions	5.00	2,580
Future Conditions	6.48	3,350
PMF-Storm		
Present Conditions	21.74	11,230
Future Conditions	23.04	11,900

Flood Routings. Inflow hydrographs for Irondale Gulch were developed from the Environmental Protection Agency's Storm Water Management Model (SWMM). Inflow to Lower Derby includes the outflow from Upper Derby as it overtopped and was assumed to fail. The hydrographs were routed through the reservoir and then routed downstream through Ladora Lake, Lake Mary, ponding formed behind the railroad embankment immediately downstream, and ponding formed behind the Burlington Northern Railroad embankment at the northwest corner of the aresenal. Outflows would pond behind the BNR embankment until it would eventually fail. A rectangular flood plain channel of 300 ft. width, a slope of

the embankment. The depth of flow for the channel was calculated for the peak discharges due to various spillway sizes for Lower Derby. The velocity times depth and depths were calculated for a range of ratios of the Probable Maximum Flood (PMF) runoff and Standard Project Flood (SPF) runoff and are shown on Plates 2, 3, and 4.

A Risk Analysis. The selected design flood to upgrade Lower Derby Dam from unsafe to safe was made on the basis of a risk analysis. The damage center for this analysis was downstream of the Northwest RMA Boundary as shown by the general flood outline in red on Plate 1. Plates 2, 3, and 4 display the information that was used to analyse the potential damage from a hydraulic standpoint rather than an actual dollar figure. Since all significant flood events in this area create velocity times depth values in excess of 8, incremental flood depth alone was selected as the key parameter for making incremental damage comparisons. It was assumed that a difference in flood depths of less than 1 foot would not cause a major change in damages. On the basis of our analysis of with and without the spillway rehabilitation project, a difference larger than 1 foot occurred when an event smaller than 140 percent of the future SPF was considered. This relationship is shown on plate 3. Therefore the future SPF as shown on plate 5 was selected as the design storm for determining the spillway size to make Lower Derby a safe project. The existing SPF is also shown for comparison purposes.

Spillway Design. Several iterations of routings of the future SPF, were made through Lower Derby Reservoir using a spillway crest elevation of 5247.2 and various spillway widths to determine what size would be required to provide 1 foot of freeboard with a the Design flood. The results of these studies showed that a spillway with a 275-foot bottom width would provide the necessary freeboard with a top of dam elevation of 5253.9. The area-capacity curve and the 275-foot rating curve are shown in Tables 2 and 3.

A 1-foot high fuse plug will be used in the spillway to prevent its frequent use as the design spillway crest has been set at normal pool level. This 1-foot fuse plug will provide 7Ø acre feet of storage or about 4.4 inches of runoff from the 19Ø acre local inflow below Upper Derby Lake.

Table 2
Lower Derby Reservoir
Area - Capacity Table

Elev. (msl)	Elev. (Staff Gage)	Area (Acres)	Capacity (Ac-ft)
5231.25	Ø.8	Ø	Ø
5232.Ø	1.8	4	5
5233.Ø	2.8	9	1Ø
5234.Ø	3.8	17	2Ø
5235.Ø	4.8	22	3Ø
5236.Ø	5.8	27	5Ø
5237.Ø	6.8	32	7Ø
5238.Ø	7.8	37	1ØØ
5239.Ø	8.8	41	13Ø
524Ø.Ø	9.8	45	16Ø
5241.Ø	1Ø.8	5Ø	2ØØ
5242.Ø	11.8	53	25Ø
5243.Ø	12.8	56	3ØØ
5244.Ø	13.8	62	35Ø
5245.Ø	14.8	66	42Ø
5246.Ø	15.8	7Ø	49Ø
5247.Ø	16.8	73	56Ø
Spillway Crest			
5247.2	17.Ø	74	58Ø
5248.Ø	17.8	77	64Ø
5249.Ø	18.8	81	71Ø
525Ø.Ø	19.8	85	8ØØ
5251.Ø	2Ø.8	89	87Ø
5252.Ø	21.8	93	97Ø
5253.Ø	22.8	96	1Ø5Ø
Top of Dam			
5253.9	23.7	99	113Ø
5254.Ø	23.8	1ØØ	114Ø
5255.Ø	24.8	1Ø3	123Ø

Table 3

275' Spillway Rating

Staff Gauge (ft)	Discharge (cfs)
Spillway Crest	
17.0	0
18.0	480
19.0	1450
20.0	2590
21.0	4210
22.0	5980
23.0	8150
Top of Dam	
23.7	9760

DEPARTMENT OF THE ARMY
OMAHA DISTRICT, CORPS OF ENGINEERS
1612 U.S. POST OFFICE AND COURTHOUSE
OMAHA, NEBRASKA 68102-4978

REPLY TO
ATTENTION OF

MROED-GE

12 February 1987

SUBJECT: Rehab Lower Derby Dam

OD 6/85

Commander, Rocky Mountain Arsenal
ATTN: SMCRM-ISF (Mr. Jim Green)
Commerce City, Colorado 80022-2180

1. We have completed incorporating Rocky Mountain Arsenal and Missouri River Division review comments into the plans and specifications for rehabilitating Lower Derby Dam. Enclosed are 10 sets of plans and specifications (Enclosure 1) as requested to accompany your DD Form 1391, to provide to the State of Colorado for their review, and for your review. In order to meet our project schedule, we would appreciate it if final State and RMA comments be received by this office no later than 6 March 1987. Construction is currently scheduled for the period of approximately July through November 1987. Sets of final contract plans and specifications which had been approved by all required offices will be distributed at the time of project advertising.

2. As indicated during past conversations with your office, RMA will prepare DD Form 1391 in order to obtain technical and administrative approval for funding this project. Enclosed is a Current Working Estimate (Enclosure 2) dated 9 February 1987 for assistance in completing this form. A final contract estimate will be completed at the time of project advertising.

3. As has been requested by your office, the Omaha District will advertise for construction and provide construction management for this project. Rehab Lower Derby Dam advertising date is currently scheduled for 6 April 1987.

4. Personnel from Omaha District, Engineering Division, Geotechnical Branch, Monitoring and Evaluation Section will inspect the outlet works conduit and box structure and also the manhole located on the embankment upon completion of dewatering and clean out by the contractor. Omaha District Construction Division will contact Monitoring and Evaluation Section when dewatering and clean out is accomplished. It is planned that the 1987 annual inspections of Ladora Dam, Havanna Street Dam, and Lower Derby Dam will occur at this time when Omaha District personnel are inspecting the items indicated above.

12 February 1987

5. On 7 January 1987, Missouri River Division accepted our design comments and we were directed to proceed with finalization of the plans and specifications. It was requested by Missouri River Division that your office be fully informed that with the rehabilitation work as designed, the project will satisfy the State of Colorado safety requirements but the project will not meet hydrologic dam safety requirements for Corps owned projects. The Corps hydrologic criteria for dams in this area of Colorado is the Probable Maximum Flood based on HMR #55. Our simple explanation for not using the PMF, as explained to Missouri River Division, is that rehabilitation costs using the HMR #55 criteria were prohibitive. Since the State of Colorado has approval authority for the proposed repairs and modifications of the project, it was agreed between this office and the State Engineer's office to adopt the incremental damage procedure described in the State's Dam Safety Branch Design Review Manual.

6. It is emphasized here that the maximum pool level for Lower Derby Lake is 5247.2 M.S.L., which is the elevation of the base of the fuse plug. The lake should not be kept or operated at a level where it would be up on the fuse plug, between elevations 5247.2 and 5249.2. The purpose of constructing the fuse plug across the inlet of the spillway is to provide reservoir storage behind the fuse plug for small flood events, thereby preventing frequent use of the spillway. (The fuse plug is designed to wash out when overtopped by a large spillway flow.) If the lake is kept or operated at a level which is up on the fuse plug, this would defeat the purpose of the plug by eliminating the reservoir storage required to prevent frequency spillway flows. Small spillway flows over the fuse plug will cause partial erosion of the plug, resulting in repair costs for maintaining the fuse plug cross-section. Also, with the lake up on the fuse plug, the plug will become saturated and cause erosion of the plug's pervious fill.

7. Any questions regarding the distribution of funds should be directed to Glen Mitchell of MROED-MF, extension 4508. Questions regarding technical or scheduling matters should be directed to Dennis Gaare of MROED-GE, extension 3220.

FOR THE COMMANDER:

2 Encls

T. R. Kell
T. R. KELL, P.E.
Chief, Engineering Division

Estimate No. 16,813 9 February 1987

Rehab Lower Derby Dam

Rocky Mountain Arsenal, Colorado

Item No.	Description	Quantity	Unit	Unit Cost	Total Cost
1.	Clearing and Grubbing			L.S.	$ 35,500.00
2.	Excavation	242,500	C.Y.	$2.82	$683,850.00
3.	Embankment	4,000	C.Y.	$0.80	$ 3,200.00
4.	Riprap	4,020	Ton	$28.50	$114,570.00
5.	Bedding Material	2,800	Ton	$27.00	$ 75,600.00
6.	Crushed Rock Surfacing	1,400	Ton	$15.00	$ 21,000.00
7.	Topsoil	21,000	C.Y.	$3.00	$ 63,000.00
8.	Seeding and Mulching	15	Acre	$1,000.00	$ 15,000.00
9.	Drop Structure			L.S.	$ 12,000.00
10.	"D" St. Alterations			L.S.	$ 58,577.00
11.	Outlet Works Conduit and Channel Work			L.S.	$ 17,317.00
12.	Fuse Plug			L.S.	$ 9,500.00
	Total Contract Cost				$ 1,109,114.00
	Cost Growth (Sept 1987) $\frac{1550}{1518}$				$ 23,381.00
	Subtotal				$ 1,132,495.00
	Contingencies 10%				$ 113,205.00
	Subtotal				$ 1,245,700.00
	S&A 5.5%				$ 68,500.00
	Total Project Cost				$ 1,314,200.00

Enclosure 2

Lower Derby Lake file

~~N~~ 1/13/87

MRDED-TH (MRDMD-A/24 Jan 86) 4th End Swoboda/dr/7327
SUBJECT: Rehabilitate Lower Derby Lake at Rocky Mountain Arsenal, CO

DA, Missouri River Division, Corps of Engineers, PO Box 103,
Downtown Station, Omaha, NE 68101-0103 7 January 1987

TO: Commander, Omaha District, ATTN: MROED

1. The responses furnished in the 3rd Endorsement regarding the rehabilitation of lower Derby Dam are adequate and you are directed to proceed with finalization of the plans and specifications. We continue to emphasize that the proposed alterations to lower Derby Dam will only meet the State of Colorado minimum standards assuming the spillway fuse plug erodes. This may or may not happen even under the revised 15 ft. wide fuse plug section, therefore, inclusion of the 2 ft fuse plug as planned will add a certain degree of risk of overtopping of the project.

2. You are reminded that even though the project will satisfy State of Colorado safety requirements, the project does not and will not meet Corps safety requirements. Our criteria for dams in this area of Colorado is the PMF based on HMR 55. It is requested that RMA be fully informed of the above prior to proceeding with the proposed modifications.

FOR THE COMMANDER:

CF:
MROED-G
MROED-H

WILLIAM P. TODSEN, P.E.
Chief, Engineering Division

MROED-GE (MRDMD-A/24 Jan 86) 3rd End Letak/bl/4447
SUBJECT: Rehabilitate Lower Derby Lake at Rocky Mountain Arsenal, CO

DA, Omaha District, Corps of Engineers, 1612 U. S. Post Office and Courthouse, Omaha, NE 68102-4978 1 December 1986

TO: Commander, Missouri River Division, ATTN: MRDED-TH

1. Reference 2nd Endorsement, dated 5 September 1986, subject same as above.

2. Responses to the submitted comments are as follows:

a. The impacts of a Lower Derby Dam failure on downstream Ladora Lake are shown in Table 1 below. The table shows peak reservoir levels in Lower Derby and Ladora Lakes for a range of flood events. Due to the embankment soil type and the relative steepness of the embankment, it was assumed that failure of Lower Derby Dam would begin when the embankment is overtopped (Elev. 5255.0). Reservoir levels for Ladora Lake are given assuming no failure of Ladora Dam; however, failure would most likely begin to occur when the embankment is overtopped.

TABLE 1

MAXIMUM RESERVOIR LEVELS FOR LOWER DERBY LAKE*
(ASSUMING FAILURE OF LOWER DERBY DAM BEGINS AT ELEV. 5255)

		NEW 275' SPILLWAY ON LOWER DERBY		
FLOOD EVENT	EXISTING SPILLWAY AT LOWER DERBY	NO FUSE PLUG	FUSE PLUG ERODES (INSTANTANEOUS FAILURE)****	FUSE PLUG DOES NOT ERODE
100-YEAR	5255.9	5251.1	5251.1	5252.9
SPF	5255.9	5251.9	5252.1	5253.9
140% SPF	5255.9	5253.1	5253.1	5254.9
170% SPF	5256.2	5253.9	5254.0	5255.5

TABLE 1 - CONTINUED

MAXIMUM RESERVOIR LEVELS FOR LADORA LAKE**
(ASSUMING NO FAILURE OF LADORA DAM)
(TOP OF DAM - ELEV. 5225)

FLOOD EVENT	EXISTING SPILLWAY AT LOWER DERBY	NEW 275' SPILLWAY ON LOWER DERBY		
		NO FUSE PLUG	FUSE PLUG ERODES (INSTANTANEOUS FAILURE)****	FUSE PLUG DOES NOT ERODE
100-YEAR	5226.8***	5225.1	5225.1	5224.7
SPF	5227.4***	5226.2	5226.2	5226.2
140% SPF	5227.7***	5226.6	5226.6	5226.6
170% SPF	5228.0***	5227.0	5227.0	5228.0***

* Reservoir routings were conducted for the entire drainage basin upstream of Lower Derby Dam
** Reservoir routings were conducted for the entire drainage basin upstream of Ladora Dam
*** Reservoir elevations include the effects of a Lower Derby Dam failure
**** Failure of the fuse plug occurs when reservoir reaches the fuse plug crest elevation (5249.2)

As the table indicates, Ladora Dam is hydrologically inadequate and is incapable of passing the outflow from a Lower Derby failure without overtopping. Improving Lower Derby's spillway capacity will improve Ladora's hydrologic capacity for flows up to the 100-year event. Corps and State of Colorado minimum criteria for the hydrologic capacity of a dam of Ladora Dam's size (small) and hazard classification (significant) is the 100-year event. With Lower Derby Dam's proposed spillway, Ladora Dam would be borderline adequate, having reservoir elevations right at crest elevation for the 100-year event. For flows greater than the 100 year event, but less than 170% of the SPF, an improved spillway will prevent Lower Derby Dam from overtopping; however, outflows from Lower Derby would be large enough to overtop and fail Ladora Dam.

b. The rating curve shown in Table 3 of the Hydrology Analysis is not the correct rating curve for a 275' spillway. The rating curve for a 225' spillway was inadvertantly substituted. The 275' spillway rating curve used in the actual computer modeling is shown below in Table 2.

TABLE 2

275' SPILLWAY RATING CURVE

ELEVATION	DISCHARGE (cfs)
5247.2	0
5248.3	683
5249.5	2,175
5250.2	3,160
5250.8	4,290
5251.4	5,555
5252.7	8,473
5254.0	11,880

c. The fuse plug was not taken into account in the original reservoir routings. Additional reservoir routings were performed using a range of flood events and four different scenarios to determine the impacts that the fuse plug has on these routings. The results of the routings were previously presented in Table 1. As the table indicates, the proposed spillway for Lower Derby Dam can still pass 140% of the SPF for the worst possible case of the fuse plug not eroding. The risk analysis conducted as part of the original Hydrology Analysis has shown that for inflows greater than 140% of the SPF, whether Lower Derby Dam fails or holds does not have a significant impact on the water surface level downstream of RMA.

d. Three alignments for the proposed spillway at Lower Derby Dam were examined in detail. The three alignments included the proposed right abutment alignment, an alignment closer to the right abutment and entering the headwater of Ladora Lake a few hundred feet downstream, and a left abutment alignment. The latter two alignments were eliminated from consideration as explained below. It is pointed out that the proposed alignment of the spillway is the alignment desired by the Rocky Mountain Arsenal.

A spillway alignment on the left abutment at the location of the existing spillway was eliminated from consideration for the following reasons. A site investigation and examination of contour maps of the areas to the south and southwest of Lower Derby Lake revealed a very flat terrain. An extremely long (5200 feet) channel would be required to properly direct spillway flows back to Ladora Lake. In addition, three roadways would need to be relocated or modified. An extensive amount of trees and other vegetation would need to be removed in the area of the existing spillway for this alignment. This area would most likely require a continual maintenance program to keep the trees and vegetation from returning. Because of the considerable number of cubic yards of excavation required for the channel, the relocation of the three roadways, and the continual maintenance problems expected, this alignment was considered to be extremely cost prohibitive.

A spillway alignment closer to the right abutment of the dam and entering the headwater of Ladora Lake a few hundred feet downstream of Lower Derby Dam was eliminated from consideration for the following reasons. Locating the spillway near the Ladora Lake headwater would require a drop of 18 feet in a distance of approximately 800 feet. The result would be velocities in excess of 10 fps. The 18 foot drop would have to be contained with a sharp bend of the spillway. The sharp bend would result in a concentration of flow and increased velocities along the right bank of the spillway. The large drop and the necessity of placing the drop within a bend would require the spillway to be a cost prohibitive concrete structure.

e. An inspection of the outlet works conduit has already been incorporated into the specifications for the rehab work (Section 2M). The inspection is to be conducted by Corps personnel and any work identified as a result of the inspection will be added to the contract by modification.

f. The cost of the SAF straight drop is less than 2% of the total project cost. A trapezoidal concrete channel would be about half the cost of the SAF straight drop; however, the trapezoidal channel is not an efficient energy dissipater and its performance cannot be guaranteed. Since the SAF straight drop is an efficient and reliable hydraulic structure and the cost is not excessive, it is recommended the SAF straight drop be constructed.

g. The maximum normal operating reservoir elevation for Lower Derby Dam is 5247.2. The fuse plug was incorporated into the design of the spillway in order to prevent frequent use of the open cut spillway during small flood events. The fuse plug prevents use of the spillway until the reservoir reaches elevation 5249.2. Elevation 5249.2 exceeds the 100-year pool elevation for the Lower Derby drainage basin. It is emphasized that this is the drainage basin between Lower Derby Dam and Upper Derby Dam and not the entire drainage basin above Lower Derby Dam.

The fuse plug in the Lower Derby Dam spillway is a 2-foot high plug with an impervious blanket on the upstream face and base. An erodible pervious material is utilized in the main portion of the plug. The fuse plug is the same design as used at Bowman-Haley Dam in North Dakota, except for the riprap placed at the upstream face and the height of the plug. Riprap is placed on the upstream face of the fuse plug because the maximum normal operating pool is at the toe of the fuse plug.

Since the spillway at Bowman-Haley has not been used, the fuse plug has not been tested to determine if it will fail. It is anticipated that the Lower Derby Dam fuse plug, as designed utilizing the erodible pervious material in the downstream portion, would erode from the downstream toe upstream to the impervious face. The impervious face, upon losing its foundation, would collapse. The riprap on the face would also collapse, lowering the height of the riprap to approximately the height of the largest rocks plus some underlying collapsed impervious material. The continuing flow of water over the riprap would disperse the rock allowing nearly unobstructed flow into the spillway.

It is felt that the spillway flow velocities at the fuse plug are sufficient to wash away the erodible ~~pervious~~ material; however, to provide greater assurance that the Lower Derby Dam fuse plug will fail, the top width of the erodible pervious material will be constructed to 15 feet rather than 30 feet as was proposed.

As indicated in the hydrologic analysis presented in this endorsement (Paragraph 2a. and 2c.), the presence of the fuse plug, whether or not it erodes, does not threaten the integrity of Lower Derby Dam. The fuse plug also does not pose any additional threat to the hydrologic adequacy of Ladora Dam as compared to the no fuse plug condition.

3. We would appreciate an expeditious response to this endorsement so that the plans and specifications for the rehabilitation of Lower Derby Dam can be finalized.

FOR THE COMMANDER:

O. H. Asleso

for T. R. KELL, P.E.
Chief, Engineering Division

MRDED-TH (MRDMD-A/24 Jan 86) 2nd End Swoboda/dr/7327
SUBJECT: Rehabilitate Lower Derby Lake at Rocky Mountain Arsenal, CO

DA, Missouri River Division, Corps of Engineers, PO Box 103, Downtown Station, Omaha, NE 68101-0103 5 September 1986

TO: Commander, Omaha District, ATTN: MROED

1. The submitted Hydraulic and Hydrology Analysis for the rehabilitation of Lower Derby Lake at Rocky Mountain Arsenal has been reviewed and a field inspection of the area was conducted on 16 July 1986. The Hydrology Analysis of Lower Derby Dam is approved subject to resolution of comments a, b and c. Comments d thru g are furnished regarding the proposed rehabilitation of Lower Derby Dam:

a. The Hydrology Analysis should discuss the impacts of a lower Derby Dam failure on the downstream Lake Ladora. The hydrologic safety of Lake Ladora should be addressed.

b. There is a discrepancy between the spillway rating curve developed from the dimensions and equations shown in the Hydraulic Analysis and the rating curve shown in Table 3 of the Hydrology Analysis.

c. The hydrology write-up should indicate if the 2-foot high fuse plug was accounted for in the reservoir routings.

d. It is recommended that the alignment and location of the proposed spillway be reevaluated to determine if a more cost effective alignment can be found. The proposed location is through one of the highest areas existing on RMA and thus the most costly. There appears to be a spillway route, requiring less excavation, closer to the right abutment of the dam and entering the headwater of Lake Ladora a few hundred feet downstream of lower Derby Dam.

e. It is recommended that the outlet works conduit be inspected to determine its condition.

f. The trapezoidal concrete channel downstream of the box outlet structure appeared to be functioning adequately, however, the side slopes have severely cracked. Repair or replacement of the concrete is recommended in lieu of constructing a SAF drop structure at the end of the box outlet.

g. The proposed spillway modifications for Lower Derby Dam call for the placement of a 2-foot high fuse plug in the spillway. The need for this fuse plug is questioned in light of the small amount of flood control storage existing in this reservoir. It is also questioned if the fuse plug will in fact erode as planned when overtopped. Recommend you evaluate the erodibility of the proposed fuse plug and the consequences if it does not erode.

FOR THE COMMANDER:

5 Encls
wd

WILLIAM P. TODSEN, P.E.
Chief, Engineering Division

MROED-G (24 Jan 86) 1st End
SUBJECT: Rehabilitate Lower Derby Lake at Rocky Mountain Arsenal, CO

DA, Omaha District, Corps of Engineers, 6014 U. S. Post Office and Courthouse, Omaha, NE 68102-4978 27 February 1986

TO: Commander, Missouri River Division, ATTN: MRDMD-A

1. Reference is made to your basic letter dated 24 January 1986, subject as above.

2. Enclosed for your review and approval are the Hydraulic Design Analysis (enclosure 2), Hydrology Analysis (enclosure 3), and a reply to your review comments (enclosure 4).

3. Although the hydrologic analysis shows the development of the Probable Maximum Flood using HMR #55 criteria, it does not go into detail as to why a decision was made to use an incremental damage analysis instead of this event for design considerations. A simple explanation for not using the Probable Maximum Flood is that the rehabilitation costs using the HMR #55 criteria were prohibitive. Since the State of Colorado has approval authority for the proposed repairs and modification of this project, we decided to adopt the incremental damage procedure described in the State's Dam Safety Manual. The results of this study were reviewed by the State and a letter indicating approval was received February 18, 1986. A copy of this letter is attached as enclosure 5.

4. If you are still interested in a field inspection of Lower Derby Dam after review of the above mentioned enclosures, please notify either Mike Kelly, MROED-GE, at Ext. 4444 or Dennis Gaare, MROED-CB, at Ext. 4553.

FOR THE COMMANDER:

5 Encls
Added 4 encls
2. Hyd Des Analysis
3. Hyd Analysis
4. Comments
5. Letter

T. R. KELL, P.E.
Chief, Engineering Division

CF: w/o encls
MROED-MF
MROED-HD
MROED-HE
MROED-GE

DEPARTMENT OF THE ARMY
MISSOURI RIVER DIVISION, CORPS OF ENGINEERS
P. O. BOX 103, DOWNTOWN STATION
OMAHA, NEBRASKA 68101-0103

MRDMD-A

24 January 1986

SUBJECT: Rehabilitate Lower Derby Lake at Rocky Mountain Arsenal, CO

Commander, Omaha District
ATTN: MROED-MF

1. Reference is made to your letter dated 24 Dec 85, subject as above, to this office.

2. Approval of the submitted plans and specifications is withheld pending a review of the dam safety aspects of the proposed rehabilitation of Lower Derby Dam. Request that MROED submit their Hydraulics and Hydrology evaluation to MRD for review and approval.

3. A joint MRD/MRO field inspection of Lower Derby Dam should be scheduled to field review the proposed modifications. This could coincide with the final design review conference at Rocky Mountain Arsenal with RMA and state in attendance. Please contact Al Swoboda, MRDED-TH, to arrange a suitable date.

4. Data submitted with above-referenced letter have been reviewed and our partial comments are enclosed. Request that a reply be made to this office by returning the enclosed comments properly annotated as to disposition.

FOR THE COMMANDER:

Encl

WILLIAM P. TODSEN, P.E.
Chief, Engineering Division

CF:
MROED (wo/encl)