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Document # GB-0694

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REGULATORY COMMISSION

October 6, 2003

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Mr. Constantine G. Tjoumas, F.E.
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888 First Street, N.E. Routing Code: PJ-13
Washington, D.C. 20426

RE: Dead River Hydroelectric Project (FERC Project No. 10855)

Dear Ms. Harding and Mr. Tjoumas:

Per Mr. Constantine G. Tjoumas' letter of September 23, 2003, the report entitled "*Silver Lake Dam: Root Cause Report on the May 14th, 2003 Operation of the Fuse Plug Spillway and Subsequent Channel Erosion Resulting in the Uncontrolled Release of Silver Lake*" is being submitted. Our consultant, Washington Group, Inc., prepared this report.

Should you have any comments or questions regarding the information provided, please contact me at (920) 433-1264, or Gilbert Snyder at (920) 433-1411, to arrange either a telephone/video conference or a meeting.

Sincerely,

David W Harpole

David W. Harpole
Vice President – Energy Supply
for Wisconsin Public Service Corporation
as agent for UPPCO

Attachments: (2) copies – Peggy Harding, FERC – CRO
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Silver Lake Dam: Root Cause Report on the May 14, 2003 Operation of the Fuse Plug Spillway and Subsequent Channel Erosion Resulting in the Uncontrolled Release of Silver Lake

DEAD RIVER HYDROELECTRIC PROJECT

Upper Peninsula Power Company

Federal Energy Regulatory Commission
Licensed Project No. 10855

Washington Group International
Integrated Engineering, Construction, and Management Solutions

October 6, 2003

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EXECUTIVE SUMMARY

Background

Silver Lake is located on Michigan's Upper Peninsula, approximately 22 miles northwest of Marquette, Michigan. Silver Lake is the uppermost impoundment of the Dead River Hydroelectric Project (FERC Licensed Project No. 10855) and is used to store spring runoff for regulated releases and power generation later in the year. There is no power generation capability on Silver Lake; the power generation takes place at the downstream Hoist and McClure Hydroelectric Developments.

In 2002 Upper Peninsula Power Company completed upgrades to Silver Lake to increase its spillway discharge capacity to accommodate flows up to the Probable Maximum Flood (PMF). Included in these upgrades were the raising of the crest elevations of the existing dike structures, the removal of one of the dikes (Dike 2) and, in its place, the construction of a new fuse plug spillway. [1]

On Wednesday May 14, 2003 the recently completed emergency[2] fuse plug spillway at Silver Lake operated as a result of inflows from rain that was experienced in the area on Sunday and Monday, May 11 and 12. The fuse plug embankment washout introduced sustained flows to the fuse plug spillway channel. As a result of these sustained flows, which reached approximately 5,000 cfs (prior to erosion of the channel), and which lasted several hours, the fuse plug spillway channel experienced erosion that extended well below the fuse plug embankment foundation. The final erosion channel extended downstream along the entire length of the spillway channel and upstream into Silver Lake. The extension of this erosion into Silver Lake resulted in the release of an additional volume of water, causing peak discharge flows out of Silver Lake to increase to between 30,000 and 32,000 cfs. The volume of water released from Silver Lake, approximately 25,300 acre-feet, was largely contained in the Dead River Storage Basin – the reservoir impounded by Hoist Dam, and whose presence attenuated the discharges downstream. Peak discharge flow rates from Hoist Dam reached approximately 7,600 cfs.

Washington Group International, Inc. (Washington Group) was retained to perform an evaluation of events leading up to and following the breach of the Silver Lake fuse plug spillway. The result of the evaluation is this report on the root causes of the operation of the fuse plug spillway and the release of Silver Lake.

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[1] A "fuse plug spillway" is a type of spillway consisting of an erodible embankment section, which is designed to automatically operate at a design reservoir level, constructed across a spillway channel to provide emergency discharge capacity during extreme events. This report refers to the embankment section as the "fuse plug embankment," or simply "fuse plug," and the spillway channel as the "fuse plug spillway channel."

[2] Chapter II of the FERC Engineering Guidelines for the Evaluation of Hydropower Projects (FERC 1993) defines an emergency spillway as "a spillway that is designed to provide additional protection against overtopping of dams and is intended for use under extreme flood conditions or mis-operation or malfunction of the service spillway."

Summary of Findings

- **Root cause for operation of the fuse plug spillway: The fuse plug embankment and its pilot channels were designed and constructed too low.** The pilot channels are depressions oriented transverse to the axis of the fuse plug embankment, intended to allow the initial flow over the fuse plug to initiate the breach of the fuse plug. The fuse plug crest design elevation was established at El. 1486.5 ft., and the pilot channel invert elevations were designed at El. 1485.5 ft., which was approximately 9 inches below the crest elevation of the concrete ogee service spillway[3] located at the Main Dam. As such, the fuse plug spillway was designed to allow flow through the pilot channel, thus initiating its operation and subsequent breach, before the lake level reached the crest of the concrete ogee service spillway. Fundamentally, the fuse plug spillway became the service spillway. Industry standard references specify that fuse plug spillways should only operate after the normal capacity of the service spillway is exceeded.

- **Root cause for the release of Silver Lake. The fuse plug foundation and spillway channel materials were highly susceptible to erosion.** With the washout of the fuse plug embankment, flows of up to 5,000 cubic feet per second (cfs) were initially introduced to the fuse plug spillway channel. Hydraulic calculations that represent this magnitude of flow combined with the topographic and roughness characteristics of the channel predict the development of turbulent and erosive hydraulic jumps at two locations. In addition, flow velocities in the channel immediately following operation of the fuse plug embankment, (but prior to the subsequent erosion of the channel) are estimated at over 8 feet per second (ft/sec) in the spillway channel, which exceeded published allowable velocities for the in situ materials for an estimated three hours. The erosive power of the turbulence and energy dissipation at hydraulic jump locations and the sustained flows were more than sufficient to trigger and sustain erosion of the fuse plug foundation and spillway channel. One of the hydraulic jump locations occurred where the flows entered the Dead River channel. Geotechnical investigations indicate soils in the vicinity of the Dead River channel consist of loose sands and gravel that are highly erodible. Geotechnical investigations also indicate that the fuse plug embankment foundation and spillway channel soils consist primarily of silty fine sands with variable amounts of gravel and cobbles. These soil conditions are considered highly erodible, particularly when subjected to the hydraulic jump turbulence and the associated erosive power that was experienced at the time. The erosive power probably caused a headcut[4], or drop, somewhere in the channel. The headcut progressed rapidly (within hours) back up the channel to where it crossed the axis of the previously washed out fuse plug embankment and then extended into the lake. Evidence of this head cut can be seen in the photograph below, which was taken at approximately 7:00 pm on May 14, 2003 after the head cut had propagated past the site of the fuse plug embankment axis into Silver Lake. The net result was a significant release of water from Silver Lake. The fuse plug

[3] Chapter II of the FERC Engineering Guidelines for the Evaluation of Hydropower Projects (FERC 1993) defines a service spillway as "a spillway that is designed to provide continuous or frequent regulated or unregulated releases from a reservoir without significant damage to either the dam or its appurtenant structures."
[4] "Headcutting" is the removal of earth material by the combined effect of the erosive power of a jet discharging over an edge and by mass wasting (Annandale, 1995).

embankment was designed to wash out only to El. 1481.0 ft, forming an emergency spillway. Instead, the ground beneath the fuse plug eroded downward an additional 12 to 25 feet.

Headcut propagation in Silver Lake - About 7 pm on May 14, 2003

Conclusions

In conclusion, the events that occurred at Silver Lake can be attributed to the combination of the two causes described above. With regard to the root cause for operation of the fuse plug spillway, overtopping of the fuse plug embankment could have been avoided (or at least limited to a much larger future flood event having a lower probability of occurrence) if the fuse plug pilot channels and crest were set at an elevation that allowed the concrete service spillway to function as intended.

The root cause for the uncontrolled release of Silver Lake is independent of the root cause for operation of the fuse plug spillway. The fact that the fuse plug embankment elevation was designed and constructed too low had no effect on the erodibility of the soils in the fuse plug spillway channel. If the spillway channel soils had been evaluated during design through standard geotechnical sampling, testing and analysis procedures, the highly erosive potential of the soils would have been evident when considering the fuse plug discharge flow characteristics (i.e. the channel velocity and hydraulic jump characteristics). Once the decision was made to construct a fuse plug spillway, erosion of the spillway channel was inevitable without improving the channel's ability to resist erosion by eliminating hydraulic jumps and excessive velocities, or by armoring.

I. INTRODUCTION

I.1 Purpose

The purpose of this study is to investigate the cause or causes of the uncontrolled release of Silver Lake into the Dead River and subsequent downstream reservoirs. This study evaluates the hydrologic events leading to the release, the initial breach of the fuse plug embankment at Silver Lake, and the subsequent erosion of the earthen fuse plug spillway channel.

The evaluations in this report conclude:

1. The fuse plug spillway embankment and its pilot channels were designed and constructed too low in relation to the existing concrete ogee service spillway such that the fuse plug spillway functioned as the service spillway.

2. The fuse plug foundation and spillway channel materials were highly susceptible to erosion with operation of the fuse plug spillway. There was no geotechnical exploration in the area of the fuse plug spillway or downstream, so the erodibility of the spillway channel materials was apparently unknown, or at least unrecognized[1].

The root cause for the uncontrolled release of Silver Lake (No. 2 above) is independent of the root cause for operation of the fuse plug spillway (No. 1 above). The fact that fuse plug embankment elevation was designed and constructed too low had no effect on the erodibility of the soils between the fuse plug embankment and the Dead River channel.

I.2 Scope

In general terms, the scope consisted of determining the cause or causes of the uncontrolled release of Silver Lake. Primarily, the scope of work focused on evaluating the hydrologic factors that contributed to the design and subsequent operation of the fuse plug, the design and operation of the fuse plug itself, and the erosive hydraulic conditions introduced to the highly erodible materials of the fuse plug spillway channel.

Specifically, the scope of the study included the following activities:

- Site inspections were made of the post-breach conditions by Washington Group civil, geotechnical and hydraulic engineers to obtain first hand familiarity with the post-breach exposed foundation conditions and topography. The site inspections were augmented by a review of project documentation including fuse plug design drawings, specifications, the Design Report, FERC Part 12 safety inspection reports, previous flood routing studies, construction reports and as-built project records. A complete listing of project

[1] The absence of subsurface exploration data does not conform with the intent of the FERC's Engineering Guidelines (FERC 1991), which indicate that sufficient subsurface information be obtained to determine the characteristics of the foundation soils and define geologic conditions that may influence the long term operation of the project.

records reviewed by the Washington Group staff in support of this evaluation is included in Appendix A with technical references in Section VI.

- Washington Group worked with STS Consultants Ltd., an engineering firm with local resources in the Upper Peninsula, to survey the site and to develop a subsurface exploration program consisting of drill holes, test pits, surface and cut slope sampling, surface geologic mapping, topographic mapping, in situ and laboratory testing to characterize the engineering properties of the fuse plug foundation and discharge channel.

- The compatibility of the specific fuse plug spillway arrangement, as designed for Silver Lake, with the other lake impoundment dam and spillway structures was reviewed with regard to spill sequence functionality. This addresses the appropriateness of the revised spill sequence that was linked to the new fuse plug configuration.

- A review was conducted of the fuse plug design report, drawings and specifications to compare the Silver Lake design to the standard of practice for fuse plug design.

- Construction quality control (QC) and as-built records were reviewed to compare the fuse plug spillway design requirements with the actual constructed product.

- Geotechnical analyses were performed on the fuse plug structure to evaluate, and if appropriate, rule out potential failure causes related to routine geotechnical factors such as slope instability and seepage/piping.

- A review was conducted of hydrologic conditions leading up to the breaching of the fuse plug and subsequent channel erosion, to ascertain the high water level that was experienced at the fuse plug site during the events of May 2003, and to estimate the recurrence intervals of the precipitation and inflow. STS Consultants gathered hydrologic and hydraulic data that was utilized by Washington Group.

- A hydrologic model representing the Silver Lake watershed and reservoir was established to estimate water levels in Silver Lake leading up to and during the breach of the fuse plug.

- Velocity and water surface profiles in the fuse plug spillway channel were computed to evaluate the erosive effects of the May 14, 2003 discharge.

- The suitability of a grass-lined and wooded fuse plug spillway channel design with respect to existing site conditions was evaluated.

- The erosive power of the May 14, 2003 discharge was evaluated and compared to the ability of the in situ spillway channel soils to resist erosion.

Washington Group's conclusions supporting the root causes for operation of the fuse plug spillway and the uncontrolled release of Silver Lake are presented herein.

I.3 Project Description

The Dead River Hydroelectric Project (FERC Licensed Project No. 10855) consists of the Silver Lake, Hoist, and McClure Developments, located in Marquette County, Michigan. The information in this section pertains only to the Silver Lake development and was obtained from the references listed in Appendix A.

I.3.1 Silver Lake Reservoir

The Silver Lake Reservoir is also called the Silver Lake Basin and is the uppermost impoundment of the Dead River Hydroelectric Project. The reservoir is located approximately 22 miles northwest of the city of Marquette, Michigan. It has an area of about 1,464 acres and a gross capacity of 33,513 acre-feet when the reservoir water surface is at El.1486.25 ft. A site map is shown on Figure I-1.

The Hoist Reservoir, also known as Dead River Storage Basin, is located about 4.6 miles downstream of the Silver Lake Reservoir and connected to it by the Dead River. Numerous tributary creeks drain into the Silver Lake, Dead River and the Hoist Reservoir.

The Federal Energy Regulatory Commission's (FERC's) ORDER ISSUING ORIGINAL LICENSE Major Project, dated October 3, 2002 defines the normal water surface level for Silver Lake as 1,486.25 feet National Geodetic Vertical Datum (NGVD). Furthermore, Article 401 – *Shoreline Erosion Control* of the same document requires the water surface levels in Silver Lake be maintained "at all times above the minimum seasonal target elevations and strive to operate the existing project facilities to achieve the start of month target elevations listed below."

Month	Start of Month Target Elevation (feet NGVD)	Minimum Elevation (feet NGVD)
April	1477.5	1477
May	1479	1478.5
June	1481	1480.5
July	1481.5	1480
August	1480	1479
September	1479.5	1479
October	1479.5	1479
November	1479	1478.5
December	1479	1478.5
January	1479	1477.5
February	1477.5	1477
March	1477.5	1477

The past operational practice of Silver Lake has been to allow the lake to fill to the normal water surface level (which is equal to the crest of the concrete ogee service spillway) El. 1486.25 ft, almost every spring since 1996 based on Upper Peninsula Power Company (UPPCO) data. Thus the reservoir stores spring runoff for release during the summer. The low-level outlet gate is manually operated, and is only adjusted occasionally to release the desired flow downstream for hydropower generation or to meet water quality requirements. The FERC license also has specific requirements for maintaining minimum downstream flows.

I.3.2 Embankments

Silver Lake Dam consists of a 1500-foot-long embankment with a maximum height of about 30.5 feet. The existing Silver Lake Dam was constructed between 1944 and 1945 to serve its downstream power plants. The earthfill embankment is separated into three sections by a concrete spillway and a concrete low-level outlet structure. The embankment crest is at El. 1491.5 ft. and the crest width is about 15 ft. Plan views of Silver Lake and the main dam are shown on Figures I-1 and I-2, respectively.

Four small earthen dikes, designated Dike 1 through Dike 4, are located at low topographic saddles along the edge of the reservoir as shown on Figure I-1. As part of the 2002 project modifications, the crests of Dikes 1, 3 and 4 were raised to approximately El. 1491.5 ft. Dike 2 was originally about 369 feet long with a maximum height of 7 feet. The crest was at about El. 1489 ft and had a minimum width of 7 feet. Dike 2 was removed in 2002 and replaced with the fuse plug embankment, which is described in the following section.

I.3.3 Fuse Plug Spillway

Dike 2 was replaced with an emergency fuse plug spillway section in 2002. The fuse plug spillway consisted of inlet and exit (spillway) channels and an erodible fuse plug embankment section. A plan and profile of the fuse plug embankment and spillway channel are presented on Figure I-3. Sections through the fuse plug embankment are presented on Figure I-4. Design characteristics of the fuse plug are as follows:

- Elevation of the crest of the fuse plug embankment, El. 1486.5 ft.
- Fuse plug embankment crest width, 5 feet
- Fuse plug embankment crest length, 292.5 feet
- Bottom elevation of the fuse plug embankment, El. 1481.0 ft.
- Bottom width of the fuse plug embankment at El. 1481.0 ft., 265 feet, the same as the channel width (as shown on Figure I-4, MWH Drawing 20895-C6)
- Impervious core, inclined upstream 1H to 1V, one foot horizontal thickness
- Pervious shell zone, 1.5-inch maximum particle size
- Filter zone, 3/8-inch maximum particle size
- Riprap slope protection zone, 6-inch maximum particle size
- Base width of the inlet and exit channels, 265 feet
- Number of pilot channels, 2
- Invert elevation of pilot channels, El. 1485.5 ft.
- Channel invert elevation at the bottom of the fuse plug embankment, El. 1481.0 ft.
- Side slopes of the channel, 2.5H to 1.0V
- Longitudinal channel slope, flat for 170 feet to station 0+66.5, 1.8 percent slope for 555 feet to El. 1471 ft, existing grade for about 800 feet to the Dead River channel
- Design assumption for time of breach formation, one hour.

I.3.4 Concrete Ogee Service Spillway

The concrete ogee spillway is an ungated overflow spillway 100 feet long with a maximum height of about 9 feet. The spillway is divided into 10 bays, each about 9 feet wide and separated by a concrete pier. Nine of the bays have a crest elevation of approximately 1486.2 ft. The fourth bay from the left has a crest elevation of approximately 1480 ft, but is normally fitted with stoplogs to the elevation of the other bays. The stoplogs are not readily removable and are not used by UPPCO to control the lake elevation. Actual surveyed elevations at each bay are presented in Table I-1. All the bays have slots for flashboards, which can be installed from a walkway on top of the spillway piers. Wingwalls extend upstream and downstream of the structure to retain the embankment.

Table I-1
Concrete Ogee Spillway Crest Elevations

Bay Number	Surveyed Elevation (ft) (by STS Consultants, 2003)
1 (left end)	1486.24
2	1486.26
3	1486.24
4 (concrete sill)	1479.95
4 (top of stoplogs)	1486.04
5	1486.22
6	1486.23
7	1486.22
8	1486.20
9	1486.20
10 (right end)	1486.19

I.3.5 Low Level Outlet

The low level outlet structure is located in the river valley at the maximum section of the dam. It is a concrete gravity non-overflow structure with a crest elevation of 1490.75 ft. and a maximum height of about 32 feet. The downstream face is vertical for the uppermost 8.5 feet, then it slopes at about 0.6H:1V for most of its height, becoming vertical for the bottom 8 feet. Wingwalls, 1.5 feet thick, angle upstream and downstream from the four corners of the structure to retain the main embankment dam.

The upstream side of the structure has two piers that extend into the reservoir, a sloping trash rack, and a manually operated slidegate. Water is discharged through a 4-foot-diameter, 22-foot long pipe, with an upstream invert at about El. 1461 ft. and a downstream invert at about El. 1457 ft.

The low level outlet is equipped with a manual wheeled operator slide gate at the entrance of the 4-foot-diameter, 22-foot long conduit.

I.4 Hydraulic Design

I.4.1 Concrete Ogee Service Spillway Outflow Capacity

The ungated concrete ogee crest service spillway has the capacity to discharge the following flow rates at various lake water levels. The stoplogs at the fourth bay from the left were assumed to be in place.

Table I-2
Concrete Ogee Spillway Discharge Rating

Silver Lake Water Surface Elevation (ft)	Discharge Rate (Provided by UPPCO) (cfs)
1486.21	0
1488.00	724
1490.00	2406
1491.00	3496

The fourth bay from the left of the ogee spillway is fitted with stoplogs. As stated previously in Section 1.3.4, the stoplogs are not readily removable and are not used by UPPCO to control the lake elevation. The elevation of the stoplogs sill is approximately El. 1480 ft, the bay width is 9.0 feet and the breadth of the sill (in the direction of flow) is about 8.5 feet. When all the stoplogs are in place, the top of the wood planks is at approximately El. 1486 ft. The hydraulic capacity of the fourth bay with the stoplogs at El. 1482.5 ft was evaluated by Washington Group. Elevation 1482.5 ft corresponds to the top of stoplog elevation stated in Section 9.0 “Additional Site Improvements” of the Design Report (MWH 2002). The results of the evaluation are presented in Table I-3.

Table I-3
Fourth Bay of Concrete Spillway Discharge Rating

Silver Lake Water Surface Elevation (ft)	Discharge Rate (cfs) (with stoplogs removed to El. 1482.5 ft)
1482.00	0
1483.00	9
1484.00	51
1485.00	105
1485.50	135

I.4.2 Low Level Outlet Structure

The low level outlet structure is located in the river valley at the maximum section of the dam as described in Section I.3.5. The low level outlet has a discharge capacity of about 300 cfs at a water surface level of about El. 1485 ft.

I.4.3 Fuse Plug Spillway Channel

The fuse plug spillway channel was designed to convey the flow through the breached fuse plug embankment to the Dead River confluence at approximately El.1440 ft., 40 feet lower than the base of the fuse plug. The plan view of the spillway channel is shown on Figure I-3. The bottom of the channel is 265 feet wide throughout its length, with 2.5H:1V side slopes. The section extending 100 feet upstream and 70 feet downstream from the centerline of the fuse plug is flat at El. 1481.0 ft. A vertical curve, not defined on the drawing, connected the flat section to the downstream section which sloped at 1.8 percent for a distance of approximately 555 feet. At the end of this reach the cleared channel discharged into the woods, with a slight depression conveying the flow around a gradual bend to the right and to the Dead River channel. The unimproved reach from the end of the cleared channel to the Dead River was about 800 feet long with an average invert slope of 3.75 percent. At the confluence of the fuse plug spillway channel and the Dead River channel, the discharge was conveyed down a steep bank into the Dead River channel.

The operational discharge capacity of the fuse plug spillway (i.e. once the fuse plug embankment is completely washed out down to its design base elevation of 1481.0 ft.) is calculated to be 4,900 cfs. This is based on step-backwater analysis of the outflow channel as designed, discussed in Section III.2.1. A discharge of 4,900 cfs would occur with a lake level of El. 1485.6 ft, 0.1 foot above the design invert of the pilot channels, following a complete washout of the fuse plug embankment. Elevation 1485.6 ft also corresponds to the maximum elevation of Silver Lake on May 14, based on the surveyed high water mark on the concrete spillway at the main dam.

The flow capacity of the fuse plug spillway after breach is dependent on the lake elevation. Since there are no project records of the maximum water surface that existed just prior to breach of the fuse plug embankment, the flow through the fuse plug spillway must be computed using surveyed high water marks present at various locations as documented by STS Consultants (STS 2003g). High water marks in the vicinity of the concrete spillway indicate a maximum lake elevation of 1485.61 ft. Other high water marks observed on trees and on the ground in the vicinity of the fuse plug spillway range from about El. 1484.91 ft to El. 1485.38 ft. For purposes of computing discharge, a lake level of El. 1485.61 ft was used and an initial discharge of 4,900 cfs was computed using a step-backwater model of the outflow channel as designed. This model is discussed further in Section III.2.1.

As mentioned above, some high water marks observed in the woods in the vicinity of the fuse plug were lower than the high water mark at the main dam (El 1485.6 ft). STS Consultants (2003c) stated that high water marks in the wooded areas adjacent to the fuse plug embankment would have been affected by the nominal wave action and turbulence on May 14, making them more difficult to interpret as an accurate high water elevation. The horizontal mark on the vertical concrete structure of the service spillway was easier to measure accurately than those on irregular ground near the fuse plug spillway. Water marks consisting of lines of tree needles and grass may have been deposited as the reservoir level was receding following the fuse plug breach. STS Consultants calculated that wind setup effects were negligible at both the spillway and the fuse plug. There is no physical reason for the maximum lake level to have been any different at the fuse plug spillway than as measured at the service spillway.

I.5 Project Hydrology

I.5.1 Flood Records

The only stream gauge on the Dead River is located downstream of the McClure Powerhouse (USGS No. 04043800) and has recorded daily powerhouse discharge flows since March 28, 1990. The three upstream reservoirs have completely controlled the flow since 1990. Previous flood studies state that it is not possible to accurately report historical floods because of the lack of records. However, at Hoist Dam elevations of the Dead River Storage Basin and turbine discharge are recorded continuously, which makes it possible to calculate historical inflows to the reservoir. This approach was used to evaluate the May 2003 storm.

Gauge records on other Upper Peninsula rivers indicate that historical high flows occurred on April 24, 1960; September 12, 1978; April 20, 1985; and April 17, 2002. The predominance of April dates may indicate that snowmelt is a significant component of the most frequently occurring floods.

I.5.2 Previous Probable Maximum Flood Studies

The Probable Maximum Flood (PMF) is defined as the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in a particular drainage area. It is usually calculated as the result of the Probable Maximum Precipitation (PMP) occurring after significant antecedent rainfall, and oriented over the watershed so as to produce the maximum runoff. The Probable Maximum Precipitation is the greatest depth of precipitation, for a given storm duration, that is theoretically possible for a particular area and geographic location.

The PMF was first calculated for the Silver Lake drainage basin in the Periodic Safety Inspection Report No. 1, October 1993, by Stone & Webster Michigan, Inc. It was refined in the Flood and Spillway Adequacy Analysis Addendum by the same author, dated December 1994. This 1994 Addendum was the first document to define the hazard classification of Silver Lake Dam as High-Hazard. A High-Hazard dam requires that the Inflow Design Flood (IDF) be the PMF.

The Probable Maximum Precipitation was determined from the Electric Power Research Institute (EPRI) publication "Probable Maximum Precipitation Study for Michigan and Wisconsin" (1992). The total precipitation depth was 21.6 inches in 72 hours for a warm-season synoptic storm. A runoff model of the Silver Lake watershed was developed using U.S. Natural Resource Conservation Service (NRCS, formerly the Soil Conservation Service, SCS) methodology, which is commonly used for ungauged watersheds (SCS, 1972). One key parameter of the SCS methodology is the Curve Number, which determines the per cent of rainfall which becomes runoff, on the basis of soil and land-use types throughout the watershed. The Curve Number for the watershed was determined to be 53. Another key parameter is the lag time, which determines the peakedness of the hydrograph. The lag time for Silver Lake was determined to be 1.9 hours.

Using the above precipitation and runoff parameters, the PMF inflow hydrograph to Silver Lake was calculated using the HEC-1 flood routing computer program (USACE, 1990), and the peak lake level and outflow were calculated using the DAMBRK program (NWS, 1984). The December 1994 Stone and Webster report assumed the lake level at the start of the storm to be El. 1483.5 ft[2]. The purpose of the 1994 analysis was to determine the spillway adequacy, so it was assumed that neither the dam nor any of the dikes would fail if overtopped. The PMF peak inflow was determined to be 40,700 cfs; the maximum outflow was 12,100 cfs, and the maximum lake level was El. 1491.3 ft. The 1994 Addendum concluded that the main dam would be overtopped by about 0.6 feet and the dikes by larger amounts. The Silver Lake spillway was determined to be inadequate for passage of the PMF. The report included recommendations to lower the crest of the existing spillway from El. 1486.25 ft. to El. 1483.5 ft., and to install a 60-foot wide auxiliary spillway with a crest elevation of 1484 ft. Also, it was recommended to provide 3 feet of freeboard above the embankment crest, which would require raising the main dam and dikes.

Significant revisions to the PMF analysis were made in "Flood Routing of Probable Maximum Floods in Dead River Basin" by Harza Engineering Company (the designer), March 2001. The peak inflow to Silver Lake was reduced from 40,700 to 36,460 cfs. The report contains no explanation for the reduction in flow. The maximum reservoir level was reduced from El. 1491.3 ft. to El. 1491.03 ft., and the peak discharge with no dam or dike failures was reduced from 12,100 to 9,350 cfs. The peak discharge reductions are primarily due to reducing the initial reservoir elevation to 1481.5 ft. The report states:

> "For all the runs as discussed in the earlier sections, the initial reservoir water levels at the Silver Lake, Hoist and McClure Reservoirs are based on the maximum target water surface as addressed in the December 1995 report, Recommended Modifications for The Dead River Hydroelectric Project. It is learned during this supplement flood routing analysis that the water levels at those reservoirs as negotiated between UPPCO and Michigan Department of Environmental Quality, Surface Water Quality Division (MDEQ-SWQD) are different from those presented in the December 1995 report."

The normal maximum level cited in the Design Report, Section 5.3, (MWH 2002) for Silver Lake is stated to be El. 1481.5 ft. However, the October 3, 2002 FERC License, Appendix A Section 1.1, indicates all the monthly specified elevations are *minimum* elevations. The FERC License states the normal maximum operating reservoir level is El. 1486.25 ft. The stated normal maximum level in the Design Report conflicts with the water levels stated in the FERC license. There is no description in the Flood Routing report (Harza 2001) or in the Design Report (MWH 2002) of any measures to be taken to maintain El. 1481.5 ft. as a maximum lake level. Section 9.0 of the Design Report (MWH 2002) states "stop logs in the fourth bay of existing concrete spillway from the left will be removed to elevation, El. 1482.5." The basis for removal of the stoplogs and selection of El. 1482.5 ft is not provided in the report. The stoplogs were removed (due to deterioration) and replaced to the same elevation with no decrease in stoplog elevation as part of the dam modification project in late 2002.

[2] Coincidentally, this is the estimated elevation of Silver Lake at the beginning of the May 2003 storm (based on UPPCO recorded observations). This purpose of this section of the report is to document previous PMF studies; evaluations and elevations stated in this section should not be confused with evaluations and elevations of the May 2003 event which is discussed in Section I.5.4.

In the Design Report (MWH 2002), minor adjustments were made to the SDF routing to conform to the design of the fuse plug. The flood analysis indicated a maximum lake level of El. 1488.45 ft. and the maximum PMF discharge through the fuse plug was 19,230 cfs, with additional outflow through the spillway. Assuming the crests of the dam and saddle dikes were raised to El. 1491.5 ft, there would be three feet of freeboard as previously recommended.

I.5.3 Recurrence Interval Floods

It is normal practice when designing a multi-part spillway system to determine the frequency or recurrence interval at which the capacity of the service spillway is exceeded and the emergency spillway must be utilized. The "recurrence interval" or the "return period" of a storm or flood is defined as the inverse of the probability of the event being equaled or exceeded in any year. For example, a rainstorm with a 1 percent probability in any year is called a 100-year storm; a 5 percent probability is called a 20-year storm. This does not imply that such storms occur 100 years apart, or 20 years apart, but that these would be the average of the intervals over a very long period of time. The gauge record on the Dead River is not sufficient to calculate the probability of the magnitude of flood peaks. Instead, flood peaks associated with storms of given probability are calculated from probable rainfall depths using the calibrated hydrologic model discussed in Section I.5.4. The probabilities of rainfall depths are determined from Bulletin 71, "Rainfall Frequency Atlas of the Midwest" by Huff and Angel, 1992, (the "Atlas"). As with the PMF studies, it is assumed that the most significant floods occur as a result of heavy warm-season rainfall.

There are no known records of a recurrence interval floods evaluation for the project. The Harza 2001 study, "Routing of Probable Maximum Floods in Dead River Basin" states:

> "Due to the large storage at the Silver Lake between the normal maximum operating water level and the top elevation of fuse plug at Dike 2, the fuse plug would not be breached for a 1,000 year flood."

This is the only prior reference to any return period flood. However the 2001 PMF routing report does not contain any documented analysis, explanation or results supporting or defining a 1000-year flood. The Atlas states, "One must be very cautious in extrapolating the derived frequency relations beyond the limits of the data. Thus if rainfall frequency relations have been derived from an 80-year data sample, it is reasonable to assume that the relations should be satisfactory for estimating the expected 100-year event, but certainly not the 500-year event. This is too far beyond the limits of the data." Therefore no events of greater than a 100-year recurrence are defined in the Atlas. The designer did not provide any basis for it's 1000-year flood in the Design Report (MWH 2002).

The point precipitation depths from the Atlas for the Dead River vicinity, and for various storm durations and return periods, are given in Table I-4. The Atlas states that rainfall depth over a 25-square mile area, similar to the Silver Lake watershed, would be 97 percent of the point precipitation for a 24-hour duration.

Table I-4
Precipitation Depth and Recurrence Interval

Recurrence Interval (years)	Point Precipitation from the "Atlas" (Huff and Angel, 1992) page 126 (inches)		
	Duration		
	1-HR	24-HR	72-HR
1	0.92	1.95	2.39
2	1.12	2.39	2.96
5	1.41	3.00	3.69
10	1.64	3.48	4.29
25	1.96	4.17	5.11
50	2.22	4.73	5.79
100	2.50	5.32	6.49

Synthetic floods based on the above warm-season rainfall depths have been created using the calibrated HEC-HMS model (HEC, 2001a) discussed in Section I.5.4. For warm-season floods, the initial loss and constant loss parameters are increased 20 percent from the values determined by calibrating the model to the May storm. This reflects the increased interception of precipitation by foliage, and the absence of any frost in the ground, both of which will result in less excess precipitation and lower flood peaks from a given amount of precipitation. Time distributions of rainfall are as defined in Huff and Angel (1992), Table 11, and the hourly rainfall increments for each synthetic storm are given in Appendix B-I. The results are shown below in Table I-5. The HEC-HMS summary tables for each recurrence interval are provided in Appendix B-I.

Table I-5
Synthetic Flood Runoff and Peak Inflow

Recurrence Interval (years)	Total Rainfall (inches) (24 sq. miles)	Total Watershed Runoff (inches) (24-hour rainfall)	Peak Inflow (cfs) (24-Hour Rainfall)
2	2.32	0.78	494
5	2.91	1.23	809
10	3.38	1.60	1062
25	4.05	2.15	1420
50	4.59	2.58	1706
100	5.16	3.04	2001
May 11-12 storm	2.94 (24-hr) 3.85 (72-hr)	1.58	910

The hydrologic conditions prevailing during the May 2003 flood, and the modeling thereof are discussed below. The storm is best defined as a 24-hour storm since 70 percent of the total rainfall occurred from noon on the 11th to noon on the 12th, preceded by only minor bursts. Figure I-5 presents a plot of rainfall depth, duration and probability according to Huff and Angel (1992). By observation of the STS Consultants' precipitation data plotted on Figure I-5, the event is between a 4- and 5-year, 24-hour storm (2.94 inches, between 2.39 and 3.00 inches on Table I-4) and between a 6- and 7-year, 72-hour storm (3.85 inches). Figure I-6 shows similar plots of peak discharge and total storm runoff for the 24-hour synthetic storms. By peak inflow and by total runoff, compared with the above 24-hour and 72-hour synthetic storms, the May precipitation event was a 7.5-year flood or a 9-year flood, respectively.

I.5.4 Event of May 9-15, 2003

The event of May 14 and the antecedent rainfall have been analyzed in detail by STS Consultants (STS 2003a, STS 2003e). Washington Group has utilized STS Consultants' input data and has independently verified the calibration of their flood model. The purpose of this section is to determine from the available data, the relative severity and the time history of the flood event which led to the operation of the Silver Lake fuse plug spillway.

Precipitation

STS Consultants, in consultation with the National Weather Service, developed an estimated time distribution of rainfall over the Silver Lake and Dead River Storage Basin watersheds for the period May 9 through May 12, 2003 (STS 2003a). There are no rainfall gauges within either watershed except one maintained by UPPCO at Hoist Dam. Data were developed using the surrounding rain gauges and Doppler radar images. The time distribution of the rainfall was determined from radar images and was ground-truthed using the nearest hourly recording rain gauge at Humboldt, MI.

On the basis of the STS Consultants' analysis, the total rainfall depth for the 4-day period was 4.0 inches over the watershed contributing to Silver Lake Storage Basin, and 4.05 inches over the watershed between Silver Lake and Hoist Dam. The most concentrated period of rainfall was a 28-hour period between 8:00 am on May 11 and 12:00 noon on May 12. In this period 3.3 inches fell over the Silver Lake watershed. No more rain fell between noon on May 12 and the time of the fuse plug operation.

Calibrated Flood Model

STS Consultants (2003e) developed a calibrated flood model using the HEC-HMS computer program (USACE, 1990) and the available data on precipitation, river flows and reservoir levels. A previous hydrologic model had been developed by the designer to determine the Inflow Design Flood (Harza 2001), but due to the lack of measured flows on the Dead River, it is uncertain whether a calibration or verification of this model was performed.

Data available for calibration of the STS Consultants model consisted primarily of the water level records of Silver Lake and the Dead River Storage Basin. At Silver Lake, the only data on lake levels are a recorded elevation of 1483.5 ft. on May 7, and the high water mark surveyed after the breach at El. 1485.6 ft and observations of the falling lake level after 6:45 pm on May 14. The Dead River Storage Basin level is automatically recorded every hour, and can be used to determine the rate of inflow both due to the rainfall runoff that occurred before the breach, and due to the release from Silver Lake after the breach.

The HEC-HMS computer program allows the analyst to choose among all the widely accepted flood modeling techniques. The two principal aspects of the model which require calibration are loss rates and hydrograph transformation. Loss rate is the rate at which rainfall is absorbed by the ground and does not become part of the flood. The total rainfall minus the losses is called *excess rainfall,* which becomes part of the flood. Hydrograph transformation means determining the rate at which the rainfall that is not absorbed travels overland and downstream to a point of interest. STS Consultants determined that the initial and constant loss rates and the Clark unit hydrograph methods were the most suitable considering the available data. The Curve Number Method was also used to determine losses in order to make use of the available soil hydrologic classification data.

Curve Numbers were initially determined for all the subwatersheds in the model. These numbers indicated that the Silver Lake watershed is expected to have smaller losses than the watersheds downstream, due to having lesser amounts of sand and silty sand (STS 2003e). However to match the observed volume of inflow to the Dead River Storage Basin for the May 2003 event, better agreement with the observed inflow was obtained using the initial and constant loss approach. Figure I-7 shows the inflow hydrograph to Dead River Storage Basin calculated from the recorded elevations (the irregularity of the curve is due to small errors in the measurement of water levels). The dotted line is the HEC-HMS model output, showing good agreement in both peak discharge and volume.

The Silver Lake runoff volume is the difference between the reservoir storage before and after the rainfall. The initial and constant loss parameters were set at smaller values than those used for the downstream watersheds, in proportion to the difference in runoff indicated by the difference in Curve Numbers discussed above. The adjusted loss parameters accurately reproduced the volume of runoff, resulting in the lake elevation calculated by the model being equal to the observed high water mark of El. 1485.6 ft on May 14. Figure I-8 shows the inflow and lake elevation hydrographs for Silver Lake calculated by the model.

The Clark unit hydrograph parameters determine the rate of increase and decrease of flow in response to a given amount of excess rainfall. The parameters are the Time of Concentration, T_c, and the Storage Constant, R. The T_c was estimated by hydraulic analysis of the flow path from the furthest part of a watershed to its outlet. The Storage Constant must be determined by calibration. These parameters had been determined by the designer for use in their PMF model, but the T_c values are too short to give a good match of the May 2003 observed inflow to Dead River Storage Basin. STS Consultants increased the designer's T_c values in equal proportion, resulting in T_c = 12 hours for the Silver Lake watershed and R = 15 hours.

The model was also run with the designer's values for Tc to determine the sensitivity of the model to variation in Tc. The designer's values were shorter than the STS Consultants' values, by a ratio of 1/3.75. The results with the designer's Tc values are also shown on Figures I-7 and I-8. The peaks occur earlier and are more responsive to short-term variations in rainfall. However, the peak inflows are higher than those of the STS Consultants model. This is because the rainfall distribution does not have any sharp peak, but is quite steady for a 21-hour period. Figure I-7 shows that the STS Consultants model gives closer agreement to the May 2003 event for the peak inflow to the Dead River Storage Basin. The peak with the designer's Tc values occurs earlier and is higher. in comparison with the STS Consultants' calibrated model, as shown in Figure I-7. Appendix B-I contains supporting input and output from the HEC-HMS model for both cases.

Figure I-7 shows that the HEC-HMS flood model developed by STS Consultants is capable of accurately reproducing the results of the rainfall of May 9-12, 2003. Figure I-8 shows the model's estimate of the flood inflow to Silver Lake, which was not recorded or observed except for the starting and maximum lake levels. The lake began to rise very slowly through May 9 and 10 in response to small amounts of precipitation, then began to rise rapidly after noon on May 11. The estimated peak inflow to the lake is 905 cfs at 1 pm on May 12; after this time the rain ended and the inflow fell off. However, the lake continued to rise gradually because the level was below the fuse plug pilot channel invert elevation and the outflow was limited to approximately 10 cfs through the low level outlet. The model indicates that the lake reached El. 1485.6 ft. about noon on May 14 (as shown on Figure I-8), and operation of the fuse plug spillway occurred soon afterward.

The rainfall depth, the runoff and the peak discharge of the May 9-13 event all indicate that the recurrence interval was between 5 and 10 years. In other words, the probability of such a flood occurring in any year is between 10 percent and 20 percent. The fuse plug embankment was overtopped and eroded due to a relatively high-frequency event because its design did not take into account the possibility of the lake level being as high as El. 1483.5 ft at the start of the storm. In fact, lake levels reached El. 1485 ft in the spring in at least seven of the previous nine years. Lake levels are discussed in greater detail in Section II.

I.6 Geologic and Geotechnical Setting

Information presented in this section was mostly obtained from available project documents and STS Consultants' geotechnical exploration program presented in "Draft of Geological and Geotechnical Data Report for Silver Lake Dam Breach" dated August 20, 2003 (STS Consultants 2003f). This information is provided as relevant background information for this Root Cause Report.

I.6.1 Regional and Site Geology

The regional geology of the Upper Peninsula of Michigan is composed of Paleozoic and Precambrian rocks unconformably overlain by numerous Quaternary (Wisconsinian) age glacial outwash and morainal till deposits, varying in thickness from a thin veneer to over 100 feet.

The Silver Lake basin is located in an area of complex geomorphology. The surficial features reflect the results of glacial and post-glacial erosion and depositional processes combined with a topographically variable bedrock surface. This region was subjected to multiple glacial advance and recession events in the past, with most of the landforms related to glacial advance and retreat. Silver Lake basin is the location of a natural lake that formed in an area of ice stagnation during the last glacial retreat. Buried blocks of ice left in the location of the lake slowly melted and ultimately left the depression. Geologic units in and around the basin include coarse-textured glacial till, glacial outwash sand and gravel deposits, and postglacial alluvium and lacustrine deposits. Both till and outwash deposits are in evidence at the site.

1.6.2 STS Consultants' Geotechnical Exploration

STS Consultants' geotechnical exploration included drilling six soil borings at the site using a combination of solid stem augers and rotary drilling methods. The soils were generally sampled using a 2-inch diameter split barrel sampler driven 18 inches with a 140-pound hammer falling 30 inches in accordance with ASTM D1586, "Standard Test Method for Penetration Test (SPT) and Split-barrel Sampling of Soils". The "N" value (Standard Penetration Resistance) is obtained by counting the number of blows of the hammer for the final 12 inches of driving. This value provides a qualitative indication of the in-place density of cohesionless soils (sand and gravel). A 3-inch diameter SPT sampler was used at selected locations to collect a larger soil sample for testing.

The relative density term corresponding with the "N" value, as used by STS Consultants, is summarized in Table I-6 below.

Table I-6
Relative Density Based on SPT-N Values

SPT-N, blows per foot	Relative Density
0 to 3	Very Loose
4 to 9	Loose
10 to 29	Medium Dense
30 to 49	Dense
50 to 80	Very Dense
>80	Extremely Dense

Laboratory tests on boring samples included grain size, plasticity, consistency, and moisture content. Soils were classified in general accordance with ASTM D 2487 "Standard Classification for Engineering Purposes."

STS Consultants performed field density testing to document the in situ density. Tests were performed at remaining fuse plug foundation areas referred by STS Consultants as "relics." In situ tests were performed at Relic Nos. 1 and 2, in test pits 1, 2 and 3, as well as along the eroded channel at various locations and elevations. A total of 70 tests were performed. Grain size tests were also performed in the laboratory on all retrieved samples from the sand cone tests.

The field density testing was performed according to ASTM D1556 "Standard Test Method for Density and Unit Weight of Soil in Place by the Sand Cone Method", often referred to as "sand cone test." The test is performed as follows: A test hole is hand excavated in the soil to be tested and all the material from the hole is saved in a container. The hole is filled with free flowing sand of a known density, and the volume is determined. The in-place wet density of the soil is determined by dividing the wet mass of the removed material by the volume of the hole. The water content of the material from the hole is determined and the dry mass and in-place dry density are calculated using the wet mass of the soil, the water content and the hole volume.

Bulk samples were obtained by hand or with an excavator from the banks of the eroded channel. Samples from each zone were taken from both the east and west banks. Laboratory grain size analysis of glacial till materials (Zones 1 through 4) were performed on 23 samples from the east bank and 23 samples from the west bank. Composite bulk samples were also obtained from the relics in fuse plug foundation area.

Laboratory tests on the bulk samples included Modified Proctor compaction to obtain the maximum dry density and optimum water content of the soil, grain size analysis, specific gravity, Atterberg limits, and direct shear tests on remolded samples to measure the drained cohesion intercept and drained friction angle of the soil. Tests were performed in accordance with relevant ASTM Standards described in the STS Consultants report (STS Consultants 2003f).

I.6.3 Description of Geologic Soil Zones[3]

The soils present at the site were grouped into six geologic zones according to appearance, color, structure, apparent density, and texture. Zones 1 through 4 (with Zone 1 at the surface and Zone 4 at the bottom) are till zones found in the banks of the breach channel. Zone 5 is the glacial outwash sands of the old Silver Lake basin. Zone 6 consists of geologically recent (post-glacial) river deposits found more than 1,000 feet downstream of the former fuse plug in the current Dead River channel. Descriptions of each of the soil zones provided below are based on STS Consultants descriptions.

<u>Zone 1 Surficial Till</u>
Zone 1 is the uppermost soil stratum in the exposed banks. Zone 1 material predominantly consists of unstratified and unsorted sand and silt mixtures with some angular gravel, cobbles and boulders. Based on grain size analyses of 8 samples, the amount of fines varied from about 5 to 23 percent with an average of 13 percent. The amount of sand for Zone 1 ranged from about 52 to 89 percent, with an average of 75 percent. The combination of sand and fines ranged from 69 percent to 100 percent, with an average of 88 percent. No in place density testing was performed in Zone 1.

<u>Zone 2 Till</u>
Zone 2 underlies Zone 1 material and consists of brown till, generally unstratified and unsorted medium dense to very dense sand and silts with some rounded gravel and cobbles. Based on grain size analyses of 45 samples, the amount of fines varied from about 6 to 27 percent, with an average of 13.5 percent. The amount of sand for Zone 2 ranged from about 57 to 88 percent, with an average of 77 percent. The combination of sand and fines ranged from 72 to 99 percent, with an average of 91 percent. In place dry density on 19 samples ranged from 101.0 pcf to 125.7 pcf with an average of 114.4 pcf.

<u>Zone 3 Till</u>
Zone 3 material consists of unconsolidated unstratified, reddish brown to brown, medium dense to extremely dense silty sands and sand-silt mixture with some rounded gravel and cobbles. The structure of Zone 3 soils is blocky and friable with macro-fractures that could have been caused by desiccation or stress-relief since the sandy till was over-consolidated by a former ice sheet. The presence of the fractures may have contributed to an increased rate of erosion of this lower

[3] The term "zone" is used in this report for consistency with the STS Consultants site geologic report (STS 2003f). The "zones" may also be considered geologic "units."

till unit due to the dislodging of large blocks of till. Based on grain size analyses of 23 samples, the amount of fines varied from about 9 to 26 percent, with an average of 16 percent. The amount of sand for Zone 2 ranged from 42 to 87 percent, with an average of 72 percent. The combination of sand and fines ranged from about 51 to 98 percent, with an average of 87 percent. In place dry density on 8 samples ranged from 106.9 pcf to 127.5 pcf with an average of 120.8 pcf.

Zone 4 Till

Zone 4 forms the base of the observable soils within the eroded channel. Zone 4 soils consist of unstratified, unsorted gray glacial till, dense to extremely dense, silty sand with varying amounts of rounded gravel, cobbles and boulders. Macro-fractures, as described for Zone 3 above, were visible at the exposed surface. The presence of the fractures may have contributed to an increased rate of erosion of this lower till unit similar to Zone 3. Based on grain size analyses of 18 samples, the amount of fines varied from about 6 to 35 percent, with an average of 18 percent. The amount of sand for Zone 4 ranged from about 51 to 79 percent, with an average of 67 percent. The combination of sand and fines ranged from about 67 to 100 percent, with an average of 85 percent. In place dry density on 4 samples ranged from 120.7 pcf to 146.4 pcf with an average of 133.1 pcf.

Zone 5 Glacial Outwash Sands

Zone 5 consists of relatively clean (few fines) sands with some gravel and occasional boulders. Based on grain size analyses of 28 samples, the amount of sand for Zone 5 ranged from 33 to 97 percent, with an average of 81 percent. The combination of sand and fines ranged from about 73 to 100 percent with an average of 97 percent. For purposes of this report, the relatively thin lacustrine deposits overlying the glacial outwash sands are also included in Zone 5.

Zone 6 River Channel Deposits

Zone 6 consists of relatively clean, well-graded sands with some fine gravel. Based on grain size analyses of 55 samples, the amount of fines varied from about 1 to 30 percent, with an average of 7 percent. The amount of sand for Zone 6 ranged from about 27 to 99 percent, with an average of 81 percent. The combination of sand and fines ranged from 31 to 100 percent, with an average of 88 percent. In place dry density on 39 samples ranged from 90.7 pcf to 128.9 pcf with an average of 108.0 pcf. Zone 6 soils had the lowest average in place density of all six soil zones.

II. ROOT CAUSE FOR OPERATION OF THE FUSE PLUG SPILLWAY: THE FUSE PLUG EMBANKMENT ELEVATION WAS DESIGNED AND BUILT TOO LOW

II.1 Fuse Plug Spillway was Designed as a Service Spillway

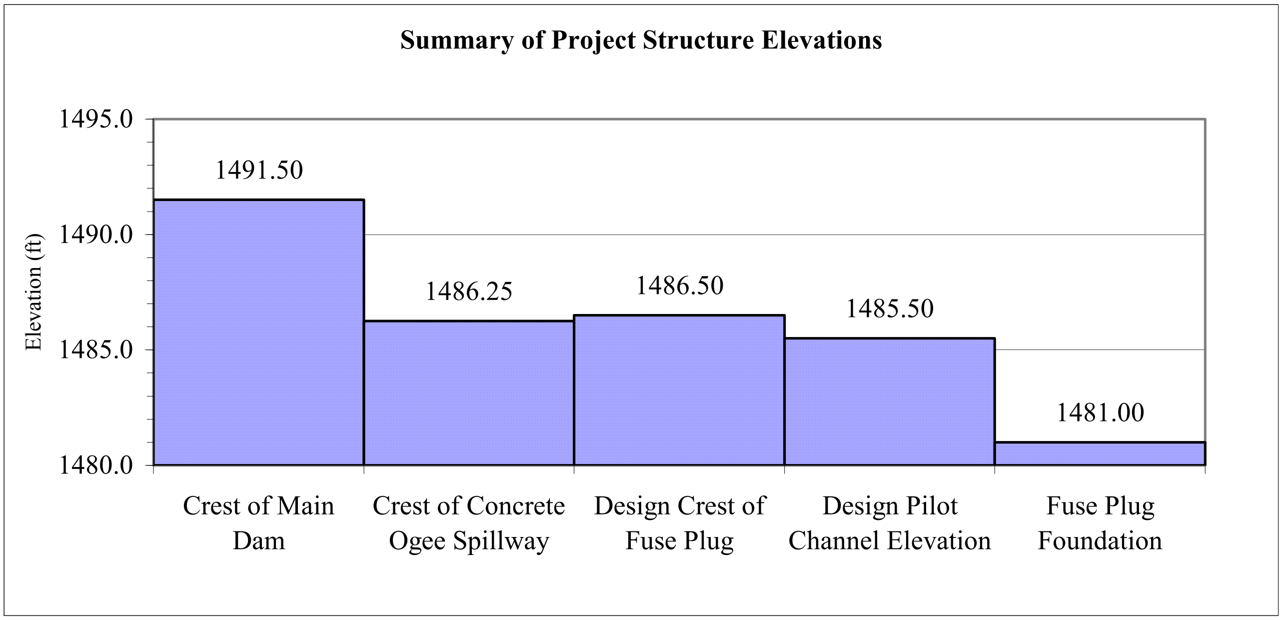
The standard practice for fuse plug design described in the United States Bureau of Reclamation (USBR) publication "Guidelines for using fuse plug embankments in auxiliary spillways" (Pugh, 1985) includes establishing elevations such that the fuse plug spillway is breached only after the normal capacity of the service spillway is exceeded. Pugh (1985) defines a fuse plug as follows:

> "A fuse plug is a zoned earth and rockfill embankment designed to wash out in a predictable and controlled manner when the flow capacity needed exceeds the normal capacity of the service spillway and the outlet works."

The Silver Lake fuse plug design approach was contrary to this key standard design concept. By setting the pilot channel elevation below the crest of the concrete ogee service spillway, the fuse plug spillway was designed to allow flow through the pilot channel before the lake level reached the crest of the concrete ogee service spillway. Fundamentally, the fuse plug spillway became the service spillway. As stated above, fuse plug spillways should only operate after the normal capacity of the service spillway is exceeded.

II.2 Project Structure Elevation Summary

Relevant design elevations for the project structures are summarized on the chart below. From this summary it is immediately apparent that the pilot channel elevations were set 0.75 ft below the crest of the concrete ogee spillway. The pilot channel elevation should have been set above the elevation of the concrete ogee spillway crest. Chapter II of the FERC Engineering Guidelines for the Evaluation of Hydropower Projects (FERC 1993) defines an emergency spillway as "a spillway that is designed to provide additional protection against overtopping of dams and is intended for use under extreme flood conditions or mis-operation or malfunction of the service spillway." The elevations established for the fuse plug design as stated in the Design Report (MWH 2002) resulted in operation of the fuse plug spillway as the service spillway.



There are other potentially functional designs that could have been considered with pilot channel elevations above the crest of the concrete ogee spillway. The purpose of this discussion is not to present alternative designs, but rather to illustrate that the proper elevations for any design would have to be determined using engineering evaluations to accomplish the objectives of the design – but in any case the pilot channel elevation should be above the crest of the concrete spillway. For example, one design would involve setting the fuse plug pilot channel elevation some established height above the service spillway crest to allow the service spillway to provide initial discharge capacity. If lake levels continued to rise and exceed the pilot channel elevation, the fuse plug spillway would have provided the additional capacity, as suggested by standard design practice (USBR, Pugh 1985). In addition, if hydrologic and hydraulic studies indicate that large storm events (i.e. PMF) cause lake levels to continue to rise and the embankment dam sections are in danger of overtopping, then the embankment dams should be raised to accommodate the required storage and freeboard. The elevations established for this selected fuse plug design resulted in operation of the fuse plug spillway during a flood event with a probability of excedence in any year of 10 to 20 percent.

II.3 Fuse Plug Pilot Channel Elevation

In standard fuse plug design (refer to Section IV for complete description of fuse plug spillway design guidelines), the pilot channel elevation is critical. The purpose of the pilot channel is to concentrate the initial breach flow at controlled locations with easily erodible materials such that the fuse plug erodes laterally from the pilot channels. Therefore, the Silver Lake fuse plug was designed to breach the first time the reservoir level increased from any given starting elevation to the pilot channel elevation 1485.5 ft. The existing design was such that any precipitation or storm event that resulted in an increase in the Silver Lake level above El. 1485.5 ft would cause fuse plug spillway operation prior to utilizing the concrete service spillway, which has a nominal crest elevation of 1486.25 ft. In this manner, the fuse plug spillway becomes the primary spillway for flow, with the concrete service spillway being used only if the storm event is extreme enough to continue raising the lake level above El. 1486.25 ft with the fuse plug spillway in use. For certain small precipitation events, such as the May 2003 storm, the fuse plug breach provided sufficient spillway capacity (prior to erosion of the channel) to lower the lake level such that there was no discharge over the concrete spillway.

II.4 Lake Levels

The Federal Energy Regulatory Commission's (FERC's) ORDER ISSUING ORIGINAL LICENSE Major Project, dated October 3, 2002 defines the normal water surface level for Silver Lake as 1,486.25 feet National Geodetic Vertical Datum (NGVD). Furthermore, Article 401 – *Shoreline Erosion Control* of the same document requires the water surface levels in Silver Lake be maintained "at all times above the minimum seasonal target elevations and strive to operate the existing project facilities to achieve the start of month target elevations listed below."

Month	Start of Month Target Elevation (feet NGVD)	Minimum Elevation (feet NGVD)
April	1477.5	1477
May	1479	1478.5
June	1481	1480.5
July	1481.5	1480
August	1480	1479
September	1479.5	1479
October	1479.5	1479
November	1479	1478.5
December	1479	1478.5
January	1479	1477.5
February	1477.5	1477
March	1477.5	1477

The past operational practice of Silver Lake has been to allow the lake to fill to the crest of the concrete ogee service spillway, El. 1486.25 ft. This has occurred almost every spring since 1996 based on UPPCO data as shown on Figures II-1 and II-2. Thus the reservoir stores spring runoff for release during the summer. The low-level outlet gate is manually operated, and is only adjusted occasionally to release the desired flow downstream for hydropower generation or to meet water quality requirements. Figure II-1, which is taken from the 1999 FERC Part 12D report (Stone and Webster, 1999), indicates the Silver Lake level was maintained above El. 1481.5 ft. for the majority of the time between 1992 and 1999. Figure II-1 also shows the lake level exceeded El. 1486.25 ft (i.e. flow over concrete service spillway) for varying periods of time in 1996, 1997 and 1998. Figure II-2 shows recorded lake levels from June 2001 through May 15, 2003. The lake level exceeded El. 1486.25 ft in June 2001 and in April and May, 2002. It was kept near this level until about July 4 in both years, when releases were increased for downstream use. If the fuse plug spillway had been installed in 1996 as designed , it would have washed out almost every spring.

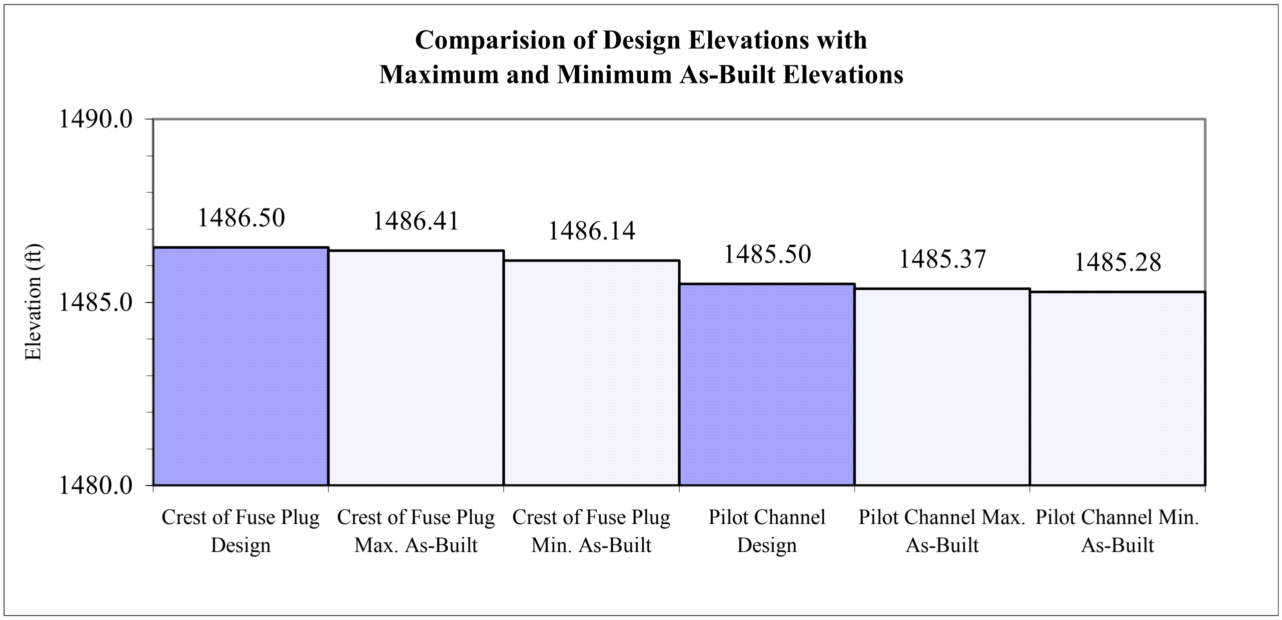
Design documents (MWH 2002) indicate an assumed normal maximum lake level of El. 1481.5 ft, which conflicts with the normal lake operating elevation provided in the license document as stated above. Regardless of the basis for this elevation, the designer's hydrology and hydraulics evaluations for the 2002 design modifications should have verified the ability of the project operational features (i.e. the low level outlet and bay 4 of the concrete spillway with stoplogs as it was configured) to control the lake elevation and prevent the lake level from increasing to El. 1485.5 ft (pilot channel invert elevation).

Washington Group's hydrologic and hydraulic analyses indicate that for the project as configured and operated to be in accordance with the FERC license requirements, it was unreasonable to assume that the normal maximum lake elevation 1481.5 ft. would be the starting elevation for any flood event. By assuming the lake could be maintained at El. 1481.5 ft. with confidence, the designers relied on the lake's four feet of storage capacity between El. 1481.5 ft and El. 1485.5 ft. During normal operations, the only way to control the lake elevation is by operating the low level outlet gate. Washington Group's analyses indicate that inflows during a moderate storm event would exceed the outlet capacity of the gate, resulting in an increase in lake level. Analyses also indicate that the gate has adequate capacity to lower the lake back to El. 1481.5 ft after a moderate storm runoff event, but only if it is operated during the event. Such operation is inconsistent with the procedures used to operate the project. Therefore, it is unreasonable to assume that 100 percent of the lake's storage capacity between El. 1481.5 ft and El. 1485.5 ft is available for storm runoff storage. In fact, past operating practices have been to fill this storage every spring, if possible, for release during the summer.

As described in Section I.4.1, the fourth bay from the left of the ogee spillway is fitted with stoplogs. The designer stated in Section 9.0 of the Design Report (MWH 2002) that stoplogs in bay 4 will be removed to El. 1482.5 ft. The stoplogs theoretically can be added or removed to control flow through bay 4, although there is no means to readily do so and the regular operation of the project did not involve addition or removal of stoplogs. Overall, removing the stoplogs is inconsistent with the manner in which the project is operated and with the FERC license that states the normal operating level is at the concrete ogee spillway crest El. 1486.25 ft. There is no requirement in the project FERC license to remove the stoplogs to El. 1482.5 ft. The stoplogs were removed (due to deterioration) and replaced to the same elevation with no decrease in stoplog elevation as part of the 2002 dam modification project.

II.5 As-Built Fuse Plug Embankment

As previously discussed, pilot channel elevations are critical to the proper function of the fuse plug. As-built surveyed elevations were documented on Coleman Engineering drawing WSK745-S1 prepared for Moyle Construction (the contractor who built the fuse plug) dated November 5, 2002. The as-built drawing indicates the actual pilot channel elevations were slightly lower than the design elevations. A comparison of the design elevations with maximum and minimum as-built elevations for the fuse plug is presented below.



The survey data indicates all elevations were constructed lower than the design elevations. The lower pilot channel elevations result in less available storage; and therefore, breach of the fuse plug occurred at a slightly lower lake elevation than the design breach elevation. There is no record indicating the pilot channels elevations were raised, subsequent to the as-built survey, to correct the elevation deficiency.

II.6 Key Factors Causing the Operation of the Fuse Plug Spillway

In summary, the key factors causing operation of the fuse plug spillway are as follows:

- The emergency fuse plug spillway design elevations stated in the Design Report were inappropriate and resulted in operation of the fuse plug spillway as the service spillway.

- The design was based on a normal maximum lake level of El. 1481.5 ft, which is inconsistent with project operations and the FERC license dated October 2002 which defines the normal water surface level for Silver Lake as El. 1486.25 ft.

III. ROOT CAUSE FOR THE RELEASE OF SILVER LAKE: THE FUSE PLUG FOUNDATION AND SPILLWAY CHANNEL MATERIALS WERE SUSCEPTIBLE TO EROSION

III.1 Exploration and Testing of Foundation and Spillway Channel Materials

The fuse plug spillway design was apparently completed without performing any geotechnical exploration to sample and evaluate the site soils that would ultimately provide a foundation for the fuse plug embankment. The project design documents do not address typical geotechnical aspects (density, grain size, etc.) of the foundation or spillway channel soils.

Limited soils data in the vicinity of the fuse plug embankment was obtained during construction and is documented in the Final Construction Report dated December 2002. The data includes four field density tests and one grain size analysis. The grain size analysis indicates 95 percent of the material was finer than the No. 4 sieve (i.e. 95 percent sand and finer) and about 78 percent of the material was *fine* sand (finer than the No. 40 sieve). A laboratory Standard Proctor density test (ASTM D698) was performed on the gradation sample. The test indicated a maximum dry density of 114.6 pcf. The four field density tests indicate a range of compaction from 96.7 to 100 percent of the Standard Proctor maximum density, corresponding to an in place density range from about 110 to 115 pcf.

After operation of the fuse plug and subsequent erosion of the spillway channel, only three small areas of the original foundation and spillway channel remained. These areas, called "relics" were photographed, tested, and documented by STS Consultants (STS Consultants, 2003f) as part of an extensive geotechnical exploration program. STS Consultants performed a geotechnical investigation in the vicinity of the fuse plug foundation (including the relics) and spillway channel and developed geotechnical profiles of the left and right sides of the channel based partly on borings performed adjacent to the channel and primarily on geologic/ geotechnical reconnaissance of the exposed eroded slopes. Descriptions of the geotechnical conditions presented in this report are based on field observations of the site by Washington Group engineers on May 22 and results of STS Consultants' exploration program.

A plan view showing the site and locations of exploration points is presented on Figure III-1 (STS Consultants 2003f, Sheet 3 of 5). The profiles of the left and right sides of the channel are presented on Figures III-2 and III-3, respectively (STS Consultants 2003f, Sheets 1 and 2 of 5). As indicated on the profiles, the geotechnical investigation identified five soil units ("zones") of glacial origin and one zone of post-glacial river channel deposit. Zones 1 through 4 are glacial till, Zone 5 is glacial outwash and lacustrine deposits, and Zone 6 is the post-glacial Dead River alluvial channel deposits. Summary descriptions of the site soil zones are provided in Section I.6.

A summary of the STS Consultants gradation and density testing is provided on Table III-1. The average median grain size (D_{50}) for till Zones 1 through 4 does not vary widely; i.e., the D_{50} is in the fine to medium sand range for all 4 zones. The density of the till deposits tends to increase with depth as indicated on Table III-1. The lower density materials located at the surface (Zone

2) are more prone to erosion than the denser materials in Zones 3 and 4. All of these predominantly sandy glacial till materials are considered highly erodible.

The D_{50} for the Zone 6 river channel deposits is in the medium sand range. Although the Zone 6 deposits are most likely former till deposits that were washed downstream, these deposits tend to have a smaller percentage of fine material than the parent till because normal flows would have transported the fines and left larger materials behind. The size of material transported depends on the velocity of the flow. More importantly, the transport and deposition process resulted in formation of a loose deposit that is highly erodible, particularly when subjected to the velocities, erosive power and hydraulic jump conditions that were experienced during the fuse plug breach. The in place density tests summarized on Table III-1 confirm that the Zone 6 river channel deposits have the lowest average density of all zones tested (108 pcf).

Table III-1
Summary of Foundation and Spillway Channel Material Testing

Soil Zone Number	Gradation Testing			Density Testing	
	Number of Gradation Samples	Average D_{50} (mm)[1/]	Average Percent Sand and Fines	Number of Density Tests	In place Avg. Dry Density (pcf)
1 (Glacial Till)	8	0.40	88.1	0	-
2 (Glacial Till)	45	0.29	90.9	19	114.4
3 (Glacial Till)	23	0.46	87.1	8	120.8
4 (Glacial Till)	18	0.32	85.2	4	133.1
5 (Glacial Outwash)	28	0.25	96.6	0	-
6 (Dead River Channel Deposits)	55	0.84	87.9	39	108.0

[1/] D_{50} is the median grain size

III.2 Computed and Permissible Flow Velocities and Hydraulic Jump Characteristics

III.2.1 Fuse Plug Discharge Velocity Evaluation

This section of the report includes a discussion of the designer's approach to evaluating velocities and Washington Group's evaluation of the discharge velocity. The Design Report (MWH 2002) states that the maximum velocity at the entrance of the inlet channel is 9.1 ft/sec with a flow depth of about 7.45 feet at the maximum (PMF) flow of 19,230 cfs. The report does not state the velocity downstream of the fuse plug. However, the velocity can be approximated using the water surface profile for the maximum outflow of 19,230 cfs on Figure 8 of the report. The geometry of both the approach channel and the downstream channel at 1.8 percent slope are similar, and the total flow at each section is the same. Figure 8 (of the Design Report) indicates the depth in the 1.8 percent slope area is about 4.8 feet. Therefore, the velocity on the 1.8 percent slope, which the report describes as supercritical flow, must exceed 9.1 ft/sec by a ratio of approximately 7.45/4.8, or about 14 ft/sec. A hydrologic model was developed by Washington Group to confirm the expected velocity for the PMF in the 1.8 percent slope area as described below.

To determine design velocities in this reach, Washington Group performed a standard step method calculation. The HEC-RAS computer program was used (Hydrologic Engineering

Center, January 2001, HEC-RAS River Analysis System, Version 3.0: User's Manual, Hydraulic Reference Manual. U.S. Army Corps of Engineers, Davis, CA). HEC-RAS is capable of handling subcritical, supercritical and mixed flow. Sixteen cross sections were prepared from Drawing 20895-C5, and from recent topographic mapping with one-foot contours, at varying distances. The locations of the cross sections are shown on Figure III-4. Note that the cross section numbering is different from the channel stationing on the topographic maps due to computer program requirements. Cross sections located downstream of the limits of the topography on MWH Drawing 20895-C5 must be regarded as approximate, since the recent map shows the topography after the erosion event. The topography along the Dead River channel was much altered by both erosion and deposition. The three cross sections in this area made use of the bank topography, the average elevation of the floodplains, and an average 1 percent slope of the channel bed estimated from the USGS 5-meter scale topographic map.

The variability of flow resistance in the channel due to surface roughness is represented in step-backward calculations by Mannings' roughness coefficient N. Higher N values result in lower velocities. The Manning's N values used by Washington Group were the same as those used by the designer, 0.04 in the graded, grassed channel and 0.08 in the woods. The designer used SCS's (1985) recommended value of Manning's N = 0.04 to represent a vegetated (grassed) spillway. Velocities could be higher if the grass was not well-established at the time of a discharge event, since the SCS reference recommends an N value of 0.02 for an earth spillway (ungrassed). In the downstream reach representing the original Dead River channel, Manning's N was taken as 0.06 based on field observations of typical reaches of the river.

A range of steady flows from 1,000 to 19,230 cfs was analyzed. Section 2800 represents the lake, Sections 2440 to 2422 are the fuse plug embankment location, and Section 1772 represents the end of the 1.8 percent spillway channel slope at El. 1471 ft. Figure III-5 presents a plot of the water surface profiles, and Figure III-6 presents a plot of the velocities for the PMF and a flow of 4,900 cfs, which is the estimated flow during the initial breach and washout of the fuse plug embankment. The lake elevation corresponding to a given outflow is taken as the energy gradient elevation at Section 2800, since the velocity head at Section 2800 is small. The results of the analysis are summarized in Table III-2. Details from the HEC-RAS output are provided in Appendix B-III. These include the computed water surface elevations and channel velocities for all the cross sections, and the Froude number, which indicates subcritical or supercritical flow.

Table III-2
Calculated Maximum Velocity in Fuse Plug Channel

Discharge (cfs)	Maximum Velocity (ft/sec)	Section of Maximum Velocity	Lake Elevation (ft)
1,000	4.95	1900	1482.98
2,000	6.22	1900	1483.84
3,000	7.14	1900	1484.54
4,000	7.85	1900	1485.13
4,900[(1)]	8.38	2328, 2100, 1900	1485.61
6,000	9.20	2100	1486.14
7,000	9.85	2100	1486.59
10,000	11.56	2100	1487.80
15,000	14.32	1772	1489.49
19,230	15.82	1772	1490.74

(1) A discharge of 4,900 cfs is comparable to the estimated May 14, 2003 discharge from Silver Lake immediately after the breach and washout of the fuse plug embankment.

The maximum velocity for 4,900 cfs is 8.38 ft/sec at Sections 1900, 2100 and 2328. For the designer's estimated PMF discharge of 19,230 cfs, the maximum velocity ranges from 13.06 ft/sec at Section 2328 up to 15.82 ft/sec at Section 1772, the end of the graded, cleared channel. The PMF velocities are in agreement with the velocities implied by Figure 8 of the Design Report, but the velocities are not documented in the Design Report.

Figure III-6 shows the velocities computed along the spillway channel for the fuse plug embankment breach outflow 4,900 cfs and for the PMF outflow of 19,230 cfs. Section 1400, which is located in the woods about 370 feet downstream from the end of the cleared channel, is a low point on the velocity profile, as the Froude number drops to about 0.4. Further downstream there is a maximum velocity at Section 1100. This is just above the drop over the bank of the original Dead River channel, which is indicated by the last contour lines visible on Figure I-3 (MWH Drawing No. 20895-C5). Flow is supercritical at this section for all flows over 3,000 cfs, but subcritical at the next section downstream. A hydraulic jump (see Illustration 1) occurs between these two sections due to the decrease in slope of the riverbed. A hydraulic jump is also predicted at Section 1772, the end of the cleared channel, because of the increase in Manning's N. HEC-RAS results indicate that flow is supercritical at the upstream section for all flows, and subcritical at Section 1772 for flows from 1,000 to 10,000 cfs. In Appendix B-III, supercritical flow is indicated by a Froude number (column heading *Froude # Chl*) greater than or equal to 1.0.

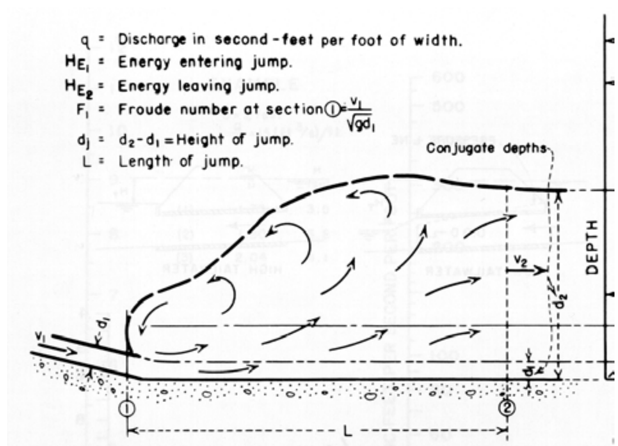


Illustration 1 - Hydraulic Jump
(Design of Small Dams, USBR 1987)

A hydraulic jump is described in "Hydraulic Design of Flood Channels" (COE, 1991) as:

> "an abrupt rise of the water surface in the region of impact between rapid and tranquil flows. Flow depths before and after the jump are less than and greater than critical depth, respectively. The zone of impact of the jump is accompanied by large-scale turbulence, surface waves, and energy dissipation. The hydraulic jump in a channel may occur at locations such as:
>
> (1) The vicinity of a break in grade where the channel slope decreases from steep to mild.
> (2) . . ."

"Rapid flows" and "tranquil flows" mean the same as "supercritical" and "subcritical" flows, respectively. The hydraulic jump downstream of Section 1100 is explained by (1) above; the jump at Section 1772 is due to increased flow resistance rather than decreased grade.

It should be noted that all velocities computed by the HEC-RAS model are average values across the channel width. In the graded channel the velocities will not deviate significantly from the average. However, once the flow enters the woods, the lateral velocity distribution is unknown. It is likely that some areas with openings between the trees would experience velocities much higher than the average computed velocity. The hydraulic jumps predicted by HEC-RAS at Section 1772 would likely take the form of standing waves against clusters of trees that obstruct flow. The turbulence of the standing waves would increase the potential for uprooting the trees and exposing the soil underneath to a high rate of erosion. Erosion and subsequent headcutting caused by hydraulic jumps may have started at any location within the reach between the end of the graded channel and the point where the flow discharged into the main channel of the Dead River, particularly given the condition of the loose Zone 6 river channel deposits within that reach. Figure III-7 shows the locations of the predicted hydraulic jumps with cross hatching superimposed on the site plan. The location of the downstream hydraulic jump coincides with the location of Zone 6 river channel deposits. Due to this coincidence, it is likely that the severe erosion and headcutting began in this vicinity.

An analysis was performed to determine the duration of velocities exceeding the designer's recommended allowable velocities of 7.5 ft/sec (for extreme conditions) and 6 ft/sec (for normal conditions). The case considered was a flood, which just exceeds the elevation of the pilot channel inverts. This is similar to the conditions of May 14, 2003, with a lake level of El. 1485.6 ft. To determine the minimum duration, it was assumed that the inflow to the lake has dropped to the base flow level, 24 cfs, by the time the lake reaches El. 1485.6 ft. It is assumed that the fuse plug embankment breach then develops fully, as designed, in one hour. The outflow, as a function of lake elevation, was calculated in a spreadsheet using 15-minute time increments. The spreadsheet is provided in Appendix B-III. The analysis indicates the discharge is greater than 3,500 cfs, which corresponds to a maximum velocity of 7.5 ft/sec, for 3 hours (see HEC-RAS output in Appendix B-III). The discharge is greater than 1,900 cfs, which corresponds to a maximum velocity of 6 ft/sec, for 9.5 hours. During the May 14 event, evidence indicates that the development of the fuse plug foundation erosion and resulting release of the lake below El. 1481 ft. began approximately four hours after the fuse plug spillway operation (STS Consultants 2003d). At that time, the HEC-RAS model indicates the discharge would have been about 3,200 cfs and the maximum velocity would have been about 7.3 ft/sec. Downstream of the graded channel, the existence of localized higher velocities, hydraulic jumps, and sandy soils all combine to increase erosion of in situ materials.

III.2.2 Review of Published Permissible Flow Velocities

Published guidelines for permissible flow velocities on earthen spillway channels were reviewed. In Chapter 2 of the US Army Corps of Engineers publication "Hydraulic Design of Flood Control Channels" (EM 1110-2-1601, July 1991) a range of suggested maximum channel velocities is provided for various channel materials. A summary of selected materials relevant to Silver Lake is provided in Table III-3 below.

Table III-3
Maximum Channel Velocities
(US Army Corps of Engineers, EM 1110-2-1601, 1991)

Channel Material	Mean Channel Velocity (ft/sec)
Fine Sand	2.0
Coarse Sand	4.0
Fine Gravel	6.0
Earth - Sandy Silt	2.0
Grass-lined Earth (slopes less than 5%)	
Bermuda Grass on Sandy Silt	6.0
Kentucky Blue Grass on Sandy Silt	5.0

The US Natural Resource Conservation Service (NRCS) (formerly the US Soil Conservation Service) provides maximum permissible velocities for channels lined with grass. The NRCS maximum permissible velocities for the relevant slope range are summarized on Table III-4 below.

Table III-4
Maximum Permissible Velocities for Grass Lined Channels
(US Natural Resource Conservation Service, Source: SCS 1985, Table 7-1)

Type of Cover	Slope Range (percent)	Permissible Velocity (ft/sec)	
		Erosion-resistant soils	Easily eroded soils
Bermuda Grass	0-5	8	6
Buffalo grass, Kentucky bluegrass	0-5	7	5
Sod-forming grass mixtures	0-5	5	4
Other grasses	0-5	3.5	2.5
Remarks: The values apply to average, uniform stands of each type of cover. Use velocities exceeding 5 ft/sec only where good covers and proper maintenance can be obtained.			

V.T. Chow (Open Channel Hydraulics, 1959) also recommends using the NRCS maximum velocities as shown on the table above.

The USBR guidelines for armoring of channels provides minimum particle sizes required to resist erosion. For a mean velocity of 6 ft/sec., the diameter required to resist erosion is about 2.7 inches (gravel size). Similarly, the USBR's publication on hydraulic design of stilling basins indicates a diameter of about 2.2 inches (gravel size) is required to resist erosion. As discussed in Section III.1, the in situ foundation and channel soils consisted of mostly sand size particles with an average D_{50} of about 0.4 mm.

Based on our review of published values of maximum permissible velocities, the maximum velocity for grass-lined channels with easily erodible soils ranges from 2 to 6 ft/sec. Therefore, the spillway channel design that included flow velocities in the range of 6 ft/sec for over 8 hours accepted the potential for significant erosion. The ability of the channel soils to resist erosion is only maximized after a grass cover has developed. Exposed channel soils would be eroded at velocities lower than 6 ft/sec. The degree to which the channel was covered with grass at the time of the breach is unknown, but it is unlikely to have been sufficiently established by May 14 to prevent erosion.

III.2.3 Designer's Approach to Permissible Flow Velocities

The designer's approach (MWH 2002) to evaluating permissible flow velocities was to compare the velocity at the channel inlet to guidelines established by the Natural Resource Conservation Services which include a permissible velocity for a grassed channel of 6.0 ft/sec (see Section III.2.2). The design report states the permissible velocity can be increased 25 percent to 7.5 ft/sec for a flood event with occurrence frequency less than 100 years. This is in accordance with the Natural Resource Conservation Service's Technical Report 60, "Earth Dams and Reservoirs" (SCS 1985). The designer determined the entrance velocity would exceed the permissible velocity for 4.4 hours during the PMF event. Washington Group's analysis of velocities in the spillway channel for the May 2003 event, as described in Section III.2.1, indicates the maximum downstream channel velocity exceeds 7.5 ft/sec in the graded channel for a flow of 4,000 cfs, from Section 2328 to Section 1900. As shown on Table III-2, the maximum velocity is greater than 6.0 ft/sec for all flows greater than 2,000 cfs and greater than 7.5 ft/sec for all flows greater than 4,000 cfs. Therefore, flows much lower than the PMF flow, comparable to the flow resulting from the initial breach of the fuse plug, would result in erosion of the discharge channel soils. The designer's justification for allowing 4.4 hours of flow with velocities greater than 7.5 ft/sec is questionable because the location of that velocity is at the entrance, rather than on the sloping discharge channel where velocities may be 50 percent greater.

As discussed in Section II.2.1, the maximum velocity in the outflow channel for a flood that just exceeds the elevation of the pilot channel invert (i.e. similar conditions to the May 14 breach of the fuse plug) would be greater than 7.5 ft/sec for 3 hours, over 6 ft/sec for 9.5 hours, and would have reached a maximum value of nearly 8.4 ft/sec.

III.3 Evaluation of Erodibility of Soils

Section III.2 discussed permissible velocities for earthen channels based on experience and standard accepted practice. A more analytical approach to evaluating erodibility that involves determining the rate of energy dissipation of water was published in 1995 by George Annandale. The approach, described in "Erodibility" in the Journal of Hydraulic Research compares the rate of energy dissipation, or "erosive power," provided by flowing water to the erosion resistance of soil and rock. The approach was developed using data collected from numerous sources that documented cases with and without erosion. That data was evaluated and a threshold line was established above which erosion occurs.

The Annandale approach includes multiple methods for calculating the rate of energy dissipation, or erosive power, depending on the hydraulic conditions. Washington Group evaluated the erodibility of the fuse plug spillway channel at several locations using the Annandale approach. The erosive power (per foot of channel width) for flow on the cleared and graded 1.8 percent slope is determined by the following equation:

Power (Watts/ft) = Unit Weight of Water (pcf) * Flow (cfs) * Energy Slope (ft/ft) * Unit Length (ft) / Width (ft)

Where: Unit Weight of Water = 62.4 pcf
Flow = 4,900 cfs (at initial fuse plug spillway operation)
Energy slope = slope of graded, cleared channel, 1.8 percent or 0.018
Channel Width = 265 ft

Power (Watts/ft) = 21, or 68 Watts/m (metric is used in the Annandale paper)

The term used to describe the ability of the soils to resist erosion is the head cut erodibility index, K_h (or simply the "erodibility index"). Annandale (1995) uses a method by H. Kirsten (1982) to determine the erodibility index. For soils (without grass), the erodibility index is determined empirically based on grain size, density and strength. For the glacial till Zones 1, 2, 3 and 4, the erodibility index was determined conservatively assuming dense to very dense soils, a residual friction angle of 32 degrees, and a D_{50} of 0.46 mm (from Section III.1). The erodibility index for glacial till zones is computed to be 1.22 x 10^{-8}. Figure III-8 presents the threshold line for cohesionless granular materials based on data provided by the USBR. The data point representing the erosive power on the graded slope and the erodibility index for the glacial till materials is presented on Figure III-8. The point is clearly above the threshold line by several orders of magnitude where erosion would be expected.

The erosive power for locations where a hydraulic jump occurs is based on energy dissipation and is determined using output from the HEC-RAS program and the following equation:

Power (Watts/ft) = Unit Weight of Water (pcf) * Flow (cfs) * Energy Dissipated (ft) / Width (ft)

Where: Unit Weight of Water = 62.4 pcf
Flow = 4,900 cfs (at initial breach)
Energy slope = slope of channel, 1.8 percent or 0.018
Channel Width = 265 ft
Energy Dissipated = $\Delta E = E_1$ (before jump) – E_2 (after jump)
Energy Dissipated at hydraulic jump at end of graded channel = 107 Watts/m
Energy Dissipated at hydraulic jump at entrance to Dead River Channel = 3000 Watts/m

The locations of the hydraulic jumps indicated by HEC-RAS are shown on Figure III-7 along with the geologic zones at each location. Glacial till is present at the location of the first hydraulic jump, and the soil erodibility index is the same as presented above, 1.22 x 10^{-8}. Loose river channel deposits (Zone 6) are present at the location of the second hydraulic jump. The erodibility index was determined assuming loose soils, a residual friction angle of 32 degrees, and a D_{50} of 0.84 mm (from Section III.1). The erodibility index for river channel deposit is computed to be 1.85 x 10^{-8}. The river channel deposit erodibility index is slightly greater than the erodibility index for the glacial till because the D_{50} is greater. Figure III-9 presents the data points representing the erosive power at the hydraulic jump locations and the corresponding erodibility indices for the glacial till and river channel deposits. Both points are above the threshold line by several orders of magnitude where erosion would be expected.

In order to provide an adequate level of erosion resistance to the power provided by the initial fuse plug discharge on the graded 1.8 percent slope, an erodibility index of at least 1.7 x 10^{-2} would be necessary based on the Annandale threshold line. This erodibility index corresponds to a D_{50} grain size of about 1.8 inches (gravel size). This approach to evaluating erodibility indicates that the in situ foundation and channel soils would be subject to erosion during operation of the fuse plug. A greater level of erosion resistance is required at the locations of the hydraulic jumps.

Although the erodibility evaluation indicates that erosion may have been initiated at any point on the 1.8 percent slope portion of the channel, the greatest concentration of erosive power was probably in the reach between the end of the graded channel and the main channel of the Dead River, where at least two hydraulic jumps likely occurred. The Annandale evaluation on Figure III-9 indicates the erosive power associated with energy dissipation at the hydraulic jump locations. The locations of these hydraulic jumps, coupled with the presence of sandy and easily erodible soils, were probably the most likely locations of the initial head cut through the in situ soils.

The continuous source of flow from Silver Lake provided the energy to maintain the erosive process. The location of the head cut progressed rapidly back up the channel to where it crossed the axis of the previously washed out fuse plug and then extended up to the lake. The net result was an uncontrolled and significant release of water from Silver Lake to levels well below the fuse plug base at El. 1481.0 ft. Washington Group's evaluation indicates if the head cut had not developed, erosion of fuse plug foundation and channel soils would have occurred. However, the most probable cause of the uncontrolled release of Silver Lake is turbulence and energy dissipation associated with hydraulic jumps and subsequent headcutting of in place soil materials.

III.4 Analysis of Development of the Uncontrolled Release of Silver Lake

The purpose of this section is to estimate the rate of release of the Silver Lake reservoir storage, and the rate of development of the breach of the fuse plug embankment foundation. The results provide insight into the erosion mechanisms and the possible shape of the breach channel as time passed. The rate of the breach development was estimated by calculating the rate of outflow from the record of the lake elevations. Reasonable assumptions were made about the shape of the breach channel, based on photographs taken during the event, and surveys of the fully developed channel after the event. The capacity of the channel as a function of lake elevation was determined for six stages of the channel development. The details of the analysis are discussed in Appendix B-III and summarized below.

Profiles of the six assumed profiles of the channel bed are shown on Figure III-10. HEC-RAS profile computations were performed for these six profiles. The channel models were configured so that critical flow occurs at an upstream control section. The control section represents the location of head cutting, where the lake plunges into the outflow channel. A plot of the above estimated control section elevations vs. time is shown in Figure III-11. Two cases were analyzed; Case 1 in which the Silver Lake outflow hydrograph was calculated from Dead River Storage Basin inflows, and Case 2 in which the outflow hydrograph was calculated from Silver

Lake elevation measurements. Both cases show the fuse plug embankment washing out as designed in one hour, between 1 pm and 2 pm. In the next hour the lake level drops slightly. No erosion of the base of the fuse plug is necessary to cause the slight lake level drop, but erosion may be occurring downstream. After 3 pm, the critical section (at the fuse plug axis location) begins to erode, the discharge increases, and the lake level begins to drop more rapidly. Case 2 shows the critical section degrading more rapidly at first, and more irregularly. The rapid drop of the critical section toward midnight for Case 1 is not realistic. However the upper portion of the curve could be accurate if a horseshoe-shaped crest developed, and provided sufficient capacity to produce the observed drop in the lake level.

Either analysis indicates that in order to produce the observed drop in the level of Silver Lake and the observed inflow to the Dead River Storage Basin, the saddle must have eroded downward from the base elevation of the fuse plug to about El. 1457 ft between 3 pm on May 14 and about 4 am on May 15. The peak outflow, on the order of 30,000 to 32,000 cfs, occurred between 9:00 pm on May 14 and 1:00 am on May 15. In either case the fuse plug foundation must have eroded downward by 11 feet or more at the time of the peak discharge. After 4:00 am on the 15th, the breach channel was sufficiently flat, less than a 1 percent slope, that little further erosion occurred and the lake emptied out to about El. 1458 ft at decreasing rates of discharge. The hydraulic analysis indicates that headcutting and critical control sections must have existed to provide sufficient hydraulic capacity for the rate of outflow observed.

III.5 Elimination of the Rock Trench

A rock trench shown on the drawings was part of the fuse plug spillway design until it was eliminated with FERC's approval on November 5, 2002. If the rock trench had been constructed as designed, it is doubtful that it would have prevented the erosion and deep undercutting that resulted after the fuse plug breach, particularly if the erosion head cut began at the downstream end of the graded channel or at the Dead River channel. As previously stated, analyses indicate the predominantly sandy materials were not capable of resisting erosion and scour when subjected to the fuse plug breach flows and hydraulic jump conditions.

III.6 Key Factors Causing Release of Silver Lake

In conclusion, the erosion and subsequent headcutting of the spillway discharge channel which resulted in the complete release of Silver Lake was made possible by several factors including:

- Presence of in situ highly erosive soils
- Achieving erosive velocities at flows lower than considered in the design
- Accepting a permissible velocity higher than what is recommended for the conditions present.
- Accepting a higher and potentially erosive velocity for an extended period of time.
- The occurrence of hydraulic jump(s) in highly erodible soils, particularly at the end of the graded channel and at the confluence with the Dead River channel.

Acceptance of higher and potentially erosive velocities and hydraulic jump conditions for an extended period of time may have been considered tolerable if the design included a spillway liner, energy dissipation, or a positive cutoff or sill to prevent headcutting of the channel from progressing beyond the axis of the fuse plug dike. The rock trench eliminated from the design would not have functioned as a positive cutoff or sill and the headcutting process would not have stopped at the rock trench.

IV. DISCUSSION OF OTHER POTENTIAL CAUSES

As part of Washington Group's effort to establish the root cause for the uncontrolled release of Silver Lake, it was necessary to attempt to rule out conventional failure mechanisms and other factors not associated with high water levels and overtopping. Examples of conventional failure mechanisms include slope instability and piping of the fuse plug embankment fill materials and foundation soils. This section presents a review of the fuse plug embankment design itself as well as a review of the adequacy of the fuse plug design to resist conventional failure mechanisms. Design and construction records were reviewed to determine if the design was appropriate and if the structure was constructed in strict accordance with the design. Construction deviations from the design, including apparent fuse plug axis and spillway channel alignment inconsistencies, were evaluated to determine if the deviations have significant impact on the function of the fuse plug embankment.

IV.1 Fuse Plug Design

A properly designed fuse plug embankment section will "operate" and completely erode down to its design foundation level when overtopped. The United States Bureau of Reclamation (USBR) has performed hydraulic model studies and has published two principal documents that provide engineers with design guidelines for fuse plugs. Detailed descriptions of fuse plug design concepts are provided in "Hydraulic model studies of fuse plug embankments" written by Clifford A. Pugh at the USBR Engineering and Research Center in Denver, CO, in December 1985. Another USBR document, "Guidelines for using fuse plug embankments in auxiliary spillways" was published as ACER Technical Memorandum No. 10 in July 1987. All of the details of these publications are not repeated in this report. Instead, this section of the report critically evaluates the Silver Lake fuse plug design using the cited USBR publications.

The basic concept of a fuse plug embankment involves placement of easily erodible material downstream of a narrow inclined core such that overtopping will wash out the material supporting the core resulting in collapse of the core and breach of the entire section. The Silver Lake fuse plug embankment was designed using this concept. Pugh (1985) documented the results of hydraulic model studies and determined erosion rates for various geometric configurations. The model studies utilize dimensional ratios to compare hydraulic models with prototype designs. Table IV-1 summarizes the basic design dimensions taken from the design drawings.

Table IV-1
Fuse Plug Design Dimensions

Dimension (refer to Pugh, 1985, USBR for definitions)	Value
H = height of fuse plug above foundation	5.5
B = base width on foundation	27
b = distance from upstream edge of core to downstream toe of fuse plug	18.5
W = width of fuse plug crest	5
J = breadth of spillway crest (in direction of flow)	200
Phi = core inclination angle (degrees)	45
t = thickness of core (measured perpendicular to inclination)	0.71
T = thickness of filter downstream of core	1.41
P = top width of pilot channel	7
h = depth of pilot channel	1
D = depth of water above foundation level at point of breach (measured when water is 0.5 ft deep in pilot channel)	5
L = length of fuse plug section	265

Table IV-2 compares the design dimension ratios based on the actual design data with the upper and lower limits of the hydraulic models.

Table IV-2
Comparison of Design to Model Studies

Ratio	Project Ratio	Model limits Lower	Model limits Upper	Within Limits?	Comments
W/H	0.91	0.40	0.80	no	Large crest width, probably for constructibility; acceptable
B/H	4.91	4.40	4.80	no	Large base due to crest width, side slopes standard; acceptable
b/H	3.36	3.10	4.00	yes	Within model limits
phi	45	30	45	yes	45 degrees is standard
T/H	0.26	0.12	0.12	no	Thick downstream filter
t/H	0.13	0.04	0.12	no	Nearly within model limits
L/H	10.91	0.00	3.24	no	Large distance to pilot channel compared to small H.
P/H	1.27	0.24	3.20	yes	Pilot channel standard design
h/H	0.18	0.12	0.24	yes	Pilot channel standard design
D/J	0.03	0.07	0.21	no	Large breadth of crest is acceptable
D/H	0.91	0.60	0.92	yes	Breach depth of flow within model limits
sand filter	yes	no	yes	yes	Sand filter is standard in design

Although several of the design dimensional ratios are outside the model limits, the design is generally considered adequate to function as intended.

The USBR (1987) design guidelines are summarized in Table IV-3 along with the comments relative to the Silver Lake fuse plug design.

Table IV-3
Comparison of Design to USBR Guidelines

Summary of Guidelines (USBR 1987)	Silver Lake fuse plug design
1. Fuse plug embankments must be designed in accordance with current, established standards.	Some aspects of the fuse plug embankment design (e.g. elevation, dimension ratios) are not in accordance with the established standards; although the design was adequate to perform as a fuse plug.
2. Site conditions should be favorable for fuse plug location and operation.	The fuse plug spillway channel foundation soils were erodible and therefore, not a favorable location for a fuse plug spillway. This design aspect is discussed in Section III.
3. In general, fuse plugs should be designed to operate only for floods with recurrence intervals that are long relative to the economic life of the project. Specifically, fuse plugs should not be designed to breach for floods with recurrence intervals less than 100 years.	By setting the fuse plug pilot channel elevation below the concrete service spillway elevation, the fuse plug was designed to operate before the service spillway operates. This design aspect is discussed in Section II. The storm event that resulted in breaching of the fuse plug was less than a 100-year event.
4. Fuse plugs should be designed so the rate of increase in reservoir outflow as the fuse plug washes out is acceptable. Splitter walls with variable elevation control sections or pilot channel elevations can facilitate this requirement.	The fuse plug had no splitter walls and no variable elevation control sections. It was designed to wash out entirely when overtopped. Without variable elevation control sections, the fuse plug spillway was designed to release at an increased rate of flow.
5. The elevation of the control section in the channel containing the fuse plug should not be lower than the top of active conservation capacity elevation, unless temporary loss of active conservation is acceptable.	The elevation of the control section in the channel is El. 1481.0 ft., which is 0.5 ft lower than the reported design normal maximum water level, about 4 ft lower than the historical seasonal maximum water level and 5.25 ft lower than the water level cited in the project licensing documents.
6. A well-conceived operation and maintenance program is necessary to ensure that the fuse plug will operate as designed. The program should be defined in the standard operating procedures and should include prevention of pedestrian and motorized traffic and vegetative growth on the fuse plug.	The design included traffic barriers and signage to prevent motorized and pedestrian traffic.
7. A fuse plug must be constructed of durable earth and rock materials which may not need to function as intended until many years after construction.	Project records indicate the fuse plug was constructed of durable earth and rock materials.

As noted above, several key design guidelines were not followed for the Silver Lake fuse plug. The two most important guideline deficiencies, which constitute the root causes of the uncontrolled release of Silver Lake, were numbers 2 and 3 above. Each of these is discussed in greater detail in Sections II and III.

IV.2 Overall Slope Stability

A slope stability analysis was performed to compute factors of safety against slope failure. Failure of fuse plug embankment slopes is not a suspected cause of failure; however, a slope stability analysis was performed to verify adequate factors of safety. The analysis was performed using the slope stability program UTEXAS3 and Spencer's method, a limit equilibrium technique. Geometry was selected based on the dimensions shown on design drawings. Shear strengths were estimated based on available as-built gradation and density tests and on engineering judgment. The estimated shear strengths for the various embankment zones included a friction angle that varied from 25 to 35 degrees (depending on gradation and density) and zero cohesion.

The analysis was performed for two load case scenarios; (1) steady-state seepage at El. 1481.5 ft. and (2) rapid fill from El. 1481.5 ft. to the pilot channel design invert at El. 1485.5 ft. Various search routines were used in the program to determine upstream and downstream shear surfaces with the lowest factors of safety. The lowest computed factor of safety is about 1.4 for the steady-state seepage case. Factors of safety were slightly higher for the rapid fill case because the increased upstream water pressure tends to have a stabilizing effect and the primarily granular materials do not lose strength during a rapid loading condition.

With conservative strength and loading condition parameters, the fuse plug embankment slope stability analysis indicates adequate factors of safety against shear surface failure. Therefore, it is unlikely that the initial breach of the fuse plug was caused by a pre-overtopping slope stability failure.

IV.3 Upstream Slope Durability

The upstream slope protection consisted of riprap sized 100 percent finer than 6 inches and as much as 70 percent finer than 3 inches. Using wind and wave setup data provided by STS Consultants (STS 2003c), Washington Group evaluated the durability of the upstream slope protection by performing a wind/wave run-up analysis for the lake level and wind conditions present on May 14, 2003. The purpose of the analysis was to determine if wave action could have caused damage and subsequent degradation of the fuse plug pilot channel below the as-built elevation. The analysis indicated that the slope protection that existed was adequate for the conditions present on May 14. Therefore, it is doubtful that the breach was prematurely initiated by wind and wave action causing instability of the upstream slope.

IV.4 Gradation and Density of Materials

The gradations and densities of materials used in construction of the fuse plug were reviewed to evaluate if incompatible gradations or low densities contributed to conventional failure mechanisms such as piping or structure instability.

The specified gradation limits of the Zone 2 filter were appropriate for the Zone 1 core based on a comparison with USBR filter criteria. Four as-built gradation tests indicate the actual material used was within the limits specified. Therefore, the filter gradation was designed and built appropriately.

The shell zone was designed as a well graded sand and gravel material. USBR filter criteria indicate the specified gradation limits for the shell were too broad for it to serve as a "filter" (i.e. to prevent piping of the filter into the shell) for all permissible Zone 2 filter gradations. However, the shell zone does satisfy filter criteria for the actual tested Zone 2 filter gradations. One shell zone gradation test (number 9) failed to meet the specification requirement for the Zone 3 shell; although the test indicated less than 1 percent out of range on one sieve. The specification did not include percentage limits for the No. 4 sieve, although quality control testing included the No. 4 sieve. All shell gradation test results were beyond the limits of the specification envelop when the No. 4 sieve results are considered. This means the shell material had less sand than what was intended, even though the specification was met. Overall, considering the specified and as-built gradations, the materials are considered compatible and piping is not considered a likely failure mechanism.

With regard to compaction of the Zone 1 core, four out of seven core density tests (numbers 6, 7, 13 and 19) failed to meet the specified 90 percent compaction requirement. Percent compactions of 84.7, 87.6, 87.5 and 86.3, respectively were measured. Also, one of the Zone 2 filter density tests (number 22) indicated a compaction of 89.1 percent; just below specified 90 percent compaction requirement. There are two main consequences of having lower than specified densities: (1) the available shear strength of the material is reduced, and (2) the permeability is increased. Even though more than half the Zone 1 core density tests failed to meet the specification requirement, the core was narrow and did not provide a significant strength contribution to the overall stability of the dike. The slightly lower strength was reflected in the material parameters in the stability analysis described in Section IV.2. The increase in permeability of the core would result in increased seepage; however, since core was well-protected against piping by the Zone 2 filter, the seepage would have been controlled.

IV.5 Settlement

The fuse plug dike may have settled a minor amount shortly after construction was completed in late 2002. If any settlement occurred it would most likely be attributed to settlement of disturbed surface soils at the fuse plug foundation rather than settlement due to elastic compression or consolidation of deeper soils. The project specifications required inspection of the fuse plug foundation after excavation of the existing dike and prior to placement of fuse plug dike materials. There is no record available documenting inspection of the fuse plug foundation. Granular materials tend to settle immediately; and therefore, settlement of the disturbed, surficial granular materials would have occurred during, or shortly after, placement of fuse plug embankment materials. The specification required that the final elevations are "intended to be the final surfaces after placement, compaction and settlement during construction."

Settlement due to compression of deeper soils is unlikely since (1) the stress increase at depth is generally low and (2) soil sampling and standard penetration testing of the in-situ materials adjacent to the erosion channel indicate the presence of dense, granular glacial till materials. The amount of potential settlement is difficult to quantify since the condition of the fuse plug foundation materials is unknown. Based on subsurface conditions encountered in borings B-2, B-3 and B-5 and the right abutment, the computed elastic settlement of the till soils due to the fuse plug embankment loading is less than 0.1 inch.

IV.6 Fuse Plug Embankment and Spillway Channel Alignments

The fuse plug embankment and spillway channel alignments as shown in the Design Report (MWH 2002) are inconsistent with the as-built locations. STS Consultants (2003g) performed surveys at the site and determined that a different horizontal control was used by the fuse plug

Contractor's surveyor, Coleman Engineering, to lay out the fuse plug and spillway channel. No difference in vertical control was found. The use of different horizontal control resulted in a rotational misalignment of about five degrees. Since the misalignment is minor, it did not have any impact on the root cause of operation of the fuse plug spillway or erosion of the fuse plug foundation and spillway channel materials as set forth in this root cause evaluation report.

V. CONCLUSIONS

The events that occurred at the Silver Lake Dam can be attributed to the two causes described in this report which are summarized as follows.

The fuse plug embankment was designed and built too low at an elevation below the primary concrete ogee spillway. The pilot channel design elevations were 9 inches below the crest of the concrete ogee spillway El. 1486.25 ft. The May 2003 rain event, estimated to be a 5- to 10-year storm, resulted in the lake level rising to El. 1485.6 ft, which was just above the as-built pilot channel invert elevations of 1485.37 and 1485.28 ft. This caused operation of the fuse plug spillway on May 14, resulting in an initial discharge of about 4,900 cfs. Substantially greater flows occurred in the hours that followed the initial discharge due to erosion, hydraulic jumps, and headcutting of the spillway channel (see below). Contrary to fuse plug design practice, the concrete ogee spillway was not utilized as the primary spillway to pass the flow since the lake level did not reach the concrete ogee crest elevation, El. 1486.25 ft. The fuse plug spillway design elevations were inappropriately established and the design appears to be based on a normal maximum lake level of El. 1481.5 ft, which is inconsistent with project operations and the FERC license dated October 2002.

As the 4,900 cfs flow from operation of the fuse plug spillway discharged down the graded 1.8 percent slope, analyses indicate velocities exceeded permissible values and some erosion of the glacial till soils probably occurred. As the flow entered the woods at the end of the graded channel, analyses indicate a hydraulic jump occurred due to the increase in flow resistance. The turbulent flow and energy dissipation associated with the hydraulic jump probably began to remove vegetation and uproot small trees. Local, higher velocity channels were probably established through the woods where subsurface soils were being exposed and eroded by the high flows. Analyses also indicate a second hydraulic jump occurred at the point where the flow discharged into the existing Dead River channel. Geotechnical investigations performed after the failure indicate that at this location, the river channel deposits are even looser than the highly erodible glacial till along the graded slope of the spillway channel. The combination of high velocity flow, hydraulic jump turbulence and loose river channel deposits provided an environment for headcutting to begin. Although the locations of the hydraulic jumps represent the most likely locations where headcutting began, the actual location of the beginning of the headcutting is not known with certainty. The headcut may have begun at any point or multiple points along the graded channel or in the woods. Regardless of the origin of the headcut, hydraulic analyses indicate it progressed relatively rapidly up the channel to the fuse plug embankment axis and beyond. The result was the uncontrolled release of Silver Lake through the eroded fuse plug spillway channel at an estimated maximum flow rate of 30,000 cfs.

The erosion and subsequent headcutting of the spillway discharge channel described above were made possible by several factors including:

- Presence of in situ highly erosive soils
- Achieving erosive velocities at flows lower than considered in the design
- Accepting a permissible velocity higher than what is recommended for the conditions present.

- Accepting a higher and potentially erosive velocity for an extended period of time.
- The occurrence of hydraulic jump(s) in highly erodible soils, particularly at the end of the graded channel and at the confluence with the Dead River channel.

The root cause for the uncontrolled release of Silver Lake is independent of the root cause for operation of the fuse plug spillway. The fact that fuse plug embankment elevation was designed and constructed too low had no effect on the erodibility of the soils between the fuse plug embankment and the Dead River channel. If the spillway channel soils had been evaluated during design through standard geotechnical sampling, testing and analysis procedures, the highly erosive potential of the soils would have been evident when considering the fuse plug discharge flow characteristics (i.e. velocity and hydraulic jump characteristics). Once the decision was made to construct a fuse plug spillway, erosion of the spillway channel was inevitable without improving the channel’s ability to resist erosion by eliminating hydraulic jumps and excessive velocities, or by armoring.

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(Note: A complete list of Silver Lake Project Documents is included in Appendix A)

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