

Appendix F  
Geotechnical Analyses



**US Army Corps  
Of Engineers**

Little Rock District

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**CLEARWATER DAM  
BLACK RIVER, MISSOURI  
MAJOR REHABILITATION PROGRAM  
SEEPAGE EVALUATION  
GEOTECHNICAL ANALYSES**

**February 2004**

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**TABLE OF CONTENTS**

<b>1.0 INTRODUCTION</b> .....	4
<b>2.0 GENERAL</b> .....	4
2.1 SCOPE OF WORK .....	4
2.2 PROJECT DETAILS .....	4
2.3 REGIONAL GEOLOGY .....	5
2.3.1 STRATIGRAPHY.....	5
2.3.2 WEATHERING .....	5
2.3.3 STRUCTURE .....	5
2.4 SEISMICITY .....	6
2.5 HISTORIC SEEPAGE OBSERVATIONS .....	6
2.6 OVERALL CURRENT PROJECT CONDITIONS.....	7
<b>3.0 SINKHOLE DEVELOPMENT AND INVESTIGATIONS</b> .....	7
3.1 SIGNIFICANCE OF SINKHOLE .....	8
3.2 SINKHOLE CAUSES.....	8
3.2.1 NATURAL CAVITY COLLAPSE .....	9
3.2.2 CONSTRUCTION DEFECT .....	9
3.2.3 INTERNAL EROSION/PIPING .....	9
3.2.3.1 MECHANISM A .....	10
3.2.3.1 MECHANISM B .....	10
3.2.3.1 MECHANISM C .....	10
3.3 INVESTIGATIONS .....	10
3.3.1 SURFACE GEOPHYSICAL.....	11
3.3.2 BORINGS .....	11
3.3.3 CROSS HOLE SEISMIC .....	11
3.3.4 DOWNSTREAM GEOPHYSICAL INVESTIGATIONS.....	12
3.4 FOUNDATION DRILLING AND GROUTING –SINKHOLE REPAIR PROJECT. .	13
3.4.1 OBJECTIVES .....	13
3.4.2 DETAILS .....	14
3.4.3 SCHEDULE AND COSTS .....	14
<b>4.0 EXPERT CONSULTATION SOLICITATION</b> .....	14
<b>5.0 SEEPAGE ASSESSMENT</b> .....	15

5.1 NON-HOMOGENEOUS/ISOTROPIC CONDITIONS .....	15
5.2 STEADY STATE SEEPAGE .....	15
5.3 NON-STEADY STATE EFFECT ON CORE .....	15
5.4 BASELINE FLOW NET ANALYSIS (CURRENT CONDITION) .....	16
5.4.1 LEFT ABUTMENT RIDGE SECTION .....	16
5.4.2 VALLEY SECTION .....	16
5.5 SEEPAGE FAILURE SCENARIO AND BREACH SIZE .....	17
5.5.1 VALLEY SECTION .....	17
5.5.2 LEFT ABUTMENT RIDGE SECTION .....	17
<b>6.0 SEEPAGE REMEDIATION MEASURES .....</b>	<b>17</b>
6.1 MEASURE S1 .....	18
6.1.1 DISCUSSION .....	18
6.2 MEASURE S2-A .....	19
6.2.1 DISCUSSION .....	19
6.3 MEASURE S2-B .....	19
6.3.1 DISCUSSION .....	19
6.4 MEASURE S2-C .....	19
6.4.1 DISCUSSION .....	19
6.5 MEASURE S3 .....	19
6.5.1 DISCUSSION .....	20
6.6 MEASURE S4-A .....	20
6.6.1 DISCUSSION .....	20
6.7 MEASURE S4-B .....	20
6.7.1 DISCUSSION .....	20
6.8 MEASURE S4-C .....	20
6.8.1 DISCUSSION .....	21
6.9 MEASURE S4-D .....	21
6.9.1 DISCUSSION .....	21
6.10 MEASURE S4-E .....	21
6.10.1 DISCUSSION .....	21
<b>7.0 RESULTS OF ANALYSES .....</b>	<b>21</b>

**LIST OF PLATES**

F-1 ..... PROJECT LOCATION, VICINITY AND INDEX  
F-2 ..... EXISTING SITE PLAN  
F-3 ..... TYPICAL CROSS SECTIONS  
F-4 ..... GEOLOGIC PROFILE ALONG CENTERLINE OF DAM  
F-6 ..... GROUT CURTAIN PROJECT SITE PLAN  
F-7 ..... ANAL. OF SEEPAGE REDUCTION FROM C/O WALL  
F-8 ..... PLAN OF SEEPAGE REMEDIATION MEASURES  
F-8A..... SEEPAGE REMEDIATION MEASURES I  
F-8B..... SEEPAGE REMEDIATION MEASURES II  
F-8C..... SEEPAGE REMEDIATION MEASURES III  
F-8D..... SEEPAGE REMEDIATION MEASURES IV  
F-8E..... SEEPAGE REMEDIATION MEASURES V

**Appendix F1**

**EXPERT CONSULTANT’S ADVISORY REPORT**

## 1.0 INTRODUCTION

Clearwater Dam has a history of seepage through the left abutment ridge dating from initial impoundment in 1948. A 1981 Comprehensive Seepage Analysis and Report concluded different risk levels for different dam segments. The report concluded that “gradual long term” erosion and solutioning of the limestone bedrock was occurring. It further stated that the dam’s left abutment ridge and left embankment/abutment contact may ultimately require major remediation to ensure the dam’s safe operation over its design life. Recent developments have expanded the concern for the dam’s safety to include the “entire dam” length. On 14 January 2003, discovery of a sinkhole in the valley portion of the dam’s upstream slope has inferred that the entire dam may be experiencing unacceptable seepage distress.

## 2.0 GENERAL

### 2.1 SCOPE OF WORK

The Geotechnical and Civil Section of the Little Rock District was tasked with researching the historical data for the Clearwater project and dissecting the information that specifically relates to the on-going seepage issues at the site. Using this data, and information obtained from recent analytical and subsurface investigations, observed seepage conditions as well as theoretical seepage analyses were completed. Additionally, outside specialists and consultants were hired to assist in determining the potential causes of the sinkhole and developing possible remediation measures that would address the short and long-term seepage issues and extend the life of the dam.

### 2.2 PROJECT DETAILS

Clearwater Dam is approximately 4,300 feet long and primarily designed for flood control. It is comprised of a zoned earth embankment and constructed atop alluvially deposited soils from the nearby Black River and residual soils from the adjacent hills. Other pertinent data on size, storage volume, etc. is presented in the main body of the report. For the purposes of this report, the dam is broken into two primary sections: the valley section and the left abutment ridge section. The project vicinity map and site plan are given on Plates F-1 and F-2, respectively. The typical dam sections are given on Plate F-3.

The valley section comprises about 2300 feet of the dam and consists of upstream and downstream pervious shells with an internal inclined clay core. The pervious shells are founded on natural river valley alluvium of varying degrees of density. The clay core is founded in a core trench excavated to bedrock. The bedrock foundation for the clay core was mortar and dental concrete treated approximately from STA 40+50 to STA 53+60. The mortar bed was placed for the purpose of protecting the base of the clay core. It was designed to be 12 inches minimum thickness and spanned full width (upstream to downstream) of the 35 foot nominal width core trench. The clay core was placed on foundation rock, or areas of dental concrete, in the first section of excavation. This area extended from the left abutment toe to approximately 1,000 feet to the right. In the remaining valley sections the core was placed on a mortar bed.

The left abutment ridge forms the foundation for the relatively short 35 feet average height embankment that forms the left most 2000 feet of the dam. The ridge is a fairly massive and wide, averaging about 500 feet width (upstream to downstream), and was expected to serve

reasonably adequately as a natural damming section for low lake levels below about El. 560. However, to assure sealing the upstream slope face of the ridge, an upstream impervious blanket was placed from an estimated elevation of 490 to elevation 560. The blanket terminated between Sta. 33+00 and 34+00. A test grouting section on the left abutment ridge inferred grouting would be impractical; therefore, grouting was not performed along the ridge segment. Moreover, the core base alignment shifts downstream as the core rises onto the ridge. This transition zone in the clay core construction changed from a core trench “to top of rock” in the valley to essentially an inspection trench extending about 10 feet below natural ground on the left abutment ridge.

## 2.3 REGIONAL GEOLOGY

The Clearwater project is located in the highlands region of the Black River Basin. This region is in the eastern part of the Ozark Plateau Province near the Mississippi Embayment. The upper end of the basin lies on the southern flank of the St. Francis Mountains, which form the igneous core of the Ozark dome. Uplands surrounding the Clearwater Dam are a part of the Salem Plateau, the lowest of the plateaus in the province and the only one present in the Black River area. The major structural feature of the area is the gentle regional dip to the southeast, which has been caused by the doming of the Ozark region. Site-specific geology is provided in the following paragraphs.

### 2.3.1 STRATIGRAPHY

Dolomite containing thin chert bands is the only rock exposed in the vicinity of the dam or penetrated by exploratory borings for the dam. The bedrock is of Upper Cambrian Age. Dolomite of the Eminence Formation is present in the upper parts of the abutments and in the spillway excavation. The Potosi Formation underlies the remainder of the dam.

### 2.3.2 WEATHERING

The Potosi formation is extremely weathered along joints and is considered karstic in nature. Weathering along joints and bedding planes has produced pinnacles in the abutment ridges, and irregular surfaces under the greater part of the valley bottom. There are numerous cavities in the underlying bedrock. Some of the cavities, bedding planes and joints are clay filled and some areas consist of disintegrated and partially decomposed rock.

### 2.3.3 STRUCTURE

The strata at the dam strikes east - west on an average dip of less than one degree to the south. One medium fault was encountered in the excavations for the dam about 75 feet downstream of the stilling basin of the outlet works. It has a displacement of 26 feet and trends NE – SW. Several minor faults were exposed in the tunnel construction. Jointing of the rock has been intense, with major systems of joints striking roughly east and west and dipping from 75 degrees to vertical. The average spacing of the joints is 4 inches.

The stratigraphy of the site geology is composed of, generally, three distinctly separate layers of rock at varying degrees of weathering: highly weathered rock, creviced or fractured weathered rock and unweathered rock. A geologic profile of the valley and left abutment ridge is provided on Plate F-4. Test boring locations are shown on the Site Plan, Plate 1. A plan and profile of the clay core trench are given on Plate F-5. Original design documents refer to the highly weathered rock zone as cherty gravel and clay and being very permeable. Crevices or fissures were found

to be present in 13 percent of the total length of the core trench, as indicated from the original mapping of the rock. Therefore, the permeability of this zone is highly variable. The unweathered rock is considered virtually impervious for the purposes of this investigation. This zone is located below a depth of approximately 120 to 130 feet below the existing seepage berm or El 385 to 375.

## 2.4 SEISMICITY

Clearwater Dam lies within a region of moderately high seismic activity and within 60 miles of the New Madrid Seismic zone. This region was the epicenter of several destructive earthquakes in 1811 and 1812. A recent seismic study conducted by USACE's St. Louis District and Engineering Research and Development Center (ERDC) for nearby Lake Wappapello is nearing completion. Although seismic considerations were not within the scope of this report, this recent seismic study and other global seismic stability issues will be incorporated into and augment the final preferred remediation alternative during the design process.

## 2.5 HISTORIC SEEPAGE OBSERVATIONS

Numerous inspections have been performed to document seepage observations throughout the history of Clearwater Lake. These were typically performed any time the lake rises above about elevation 525. These Inspection Reports are on file in the District Office, Design Branch files.

Initial Impoundment to 1972- Impoundment began in 1948 and was complete by 1949. Surface seepage was immediately observed at lake levels exceeding about El. 510. This seepage occurred in the downstream valley floor near the south end of the left abutment ridge in the general vicinity of the discharge point of the shallow depth seepage trench. This trench was constructed under the initial contract along the downstream of the dam. Seepage appeared to increase proportionally as lake level increased. Sometime after 1950, a concentrated seepage exit developed near the toe of the slope of the south end of the left abutment ridge. This location is about 400 feet from the dam axis (roadway centerline) and centered near the location of the existing piezometer E-27.

1972 through 1989 - The general pervasiveness of surface seepage appeared to have increased by 1972, ultimately affecting public use and maintenance of the area now known as River Road Park. This park had been constructed immediately downstream of the dam. Therefore, the first French drain type Drainage System was installed in 1972. Three weirs were installed in the system to monitor seepage flows. The 1972 Drainage System controlled the seepage such that the lake had to rise to about El. 520 before surface seepage emerged near E-27. For lake levels above El. 520, the seepage remained a nuisance problem for the park. It was apparent that seepage was probably passing beneath the 1972 system, which was only placed to about 15 feet of depth. Therefore, the 1980 Drainage System (French drain) was constructed with drilled drain shafts to top of rock. These shafts were filled with pervious material. The 1980 System was located just up slope from the 1972 System and closer to the left abutment ridge to more efficiently intercept seepage moving along the bedrock surface. Surface seepage appeared after the 1980 System installation when the lake reached about El. 530. However, for lake levels above that, seepage remained a problem, appearing as concentrated exits or sheet flow through out the park's northern most side. By 1983, a newly documented major seepage exit developed in the eastern side Highway H-H ditch near the former Glendale Resort Motel. For several high

lake events (exceeding El. 540) during this period, no new exits were noted. The historical exits were located somewhat randomly. However, these exits were mostly along the downstream edge of the left abutment ridge, in the side valley lying just downstream of the ridge and along Highway H-H roadway edge.

1989 to May 2001- Cumulatively, the pervasive expanse of surface seepage, inferring a significant amount of subsurface seepage movement, justified further study in pursuit of a seepage remediation project. The 1989 Seepage Berm and Parapet Wall Project was constructed to extend the seepage path, slow the seepage gradient through the left abutment and reduce the initiation and amount of surface seepage. The goal of this project was not to halt seepage; i.e., a goal that was viewed as economically impractical at the time. An economically achievable remediation goal was to further reduce gradients from the already relatively low values that existed at normal lake, thus extend the life of the dam out to its' intended design life. This project also utilized excavated material from the spillway. This effort widened the spillway from 75% to about 94% of PMF required spill capacity. Furthermore, it added stability improvement by flattening the upstream slope and placing a berm against the toe. Thus, the project was considered a very economically prudent remediation. Subsequent to completing the berm, one lake event rose to about El. 547 in 1993 and no surface seepage was observed. No other significant high lake level test of the berm's gradient reducing effect materialized until the May 2001 Pool of Record (POR).

May 2001 POR - The Pool of Record was set at El. 566.6 in May 2001. This lake level exceeded the previous high of El. 565.4 recorded in the spring of 1957. Seepage gradients for this record appeared similar to previous high lake conditions; however, there was a major retarding effect on seepage observed for this event. This inferred the benefit that the seepage berm had provided in lengthening the seepage path. Surface seepage at the historical exit near piezometer E-27, traditionally the first observed exit for seepage as the lake rose, did not begin flowing until nine days after the lake had peaked at El. 566.8. This was approximately two weeks after the lake had passed El. 530 where surface seepage had emerged at E-27 prior to the Seepage Berm Construction.

## 2.6 OVERALL CURRENT PROJECT CONDITIONS

This report specifically focuses on the matter of internal seepage through and around the dam embankment and its affects of the sinkhole development and possible failure. However, the Clearwater project has four identified deficiencies that are currently being investigated as to their individual severity; two affecting the Spillway, and two affecting the embankment. These deficiencies are: a) Spillway Erodibility, b) Spillway Hydraulic Capacity, b) Embankment Seepage, and c) Embankment Seismic Stability. Although each problem and failure mechanism could likely occur individually, they are related through one or more common mechanism or failure scenarios. These would be: site geology, dam breach failure mechanism, common lake level at failure, common extent of damage-Loss of Life (LOL) downstream, and shared solution or beneficial side effect from one deficiency being solved.

## 3.0 SINKHOLE DEVELOPMENT AND ANALYSES

A 10 feet diameter sinkhole was discovered on 14 January 2003, on the upstream slope at elevation 570+/- . The physical location was surveyed at Sta. 39+87 as taken from the roadway.

Photographs and measurements were taken by Clearwater Project personnel and relayed to the District Office. District personnel immediately responded and evaluated the sinkhole and its temporary repair. A subsequent inspection and trip report was produced and is on file in the District Office. The following is a general summary of the events and observations of the initial sinkhole investigation:

The measurements of the sinkhole were documented to be 10 ft diameter x 10 ft deep. The sinkhole appeared to be a near perfect vertical cylinder into the embankment. At a depth of 20 feet, the sinkhole appeared to neck down to a diameter of 5 feet of visible clay material. At the ultimate excavation depth of 25 feet, the neck of the sinkhole was measured at 3 feet in diameter. The sinkhole was refilled using on site clay borrow material which was excavated and hauled from a nearby stockpile located along the upstream left abutment. The borrow material consisted of clay (CL/CH) with cobbles. This material was placed in approximately 10 to 12 inch loose lifts and compacted thoroughly between lifts with the remote operated sheep foot roller. The surveying measurements and computations were completed to document the exact location of the sinkhole.

Approximately two weeks after the sinkhole discovery, bubbles were observed rising from the lake bottom when the lake was thinly frozen. This occurred approximately 100 ft upstream of the seepage berm's lake edge, near Sta. 45+00+/- . The bubbling persisted for about two weeks. Other bubbling locations were observed briefly for a few days at the lake edge and upstream of Sta. 36+/- . This anomaly may or may not be related to subsurface seepage. However, no surface seepage was observed downstream in the historical exit areas during the sinkhole inspection, surface repair or during the period of observed bubbling.

### 3.1 SIGNIFICANCE OF SINKHOLE

The sinkhole's relationship or influence on the seepage issue and assessment of the dam's safety is as follows. Prior to the sinkhole discovery, the valley section of the dam had been assumed stable and functioned as intended. The left abutment ridge section was the only area considered at risk from a) long term deterioration, and b) rapid erosion at the window area via a maximum gradient induced single high lake event that could pass over the top of the impervious blanket , and cause a rapidly deteriorating situation. With the sinkhole discovery, the remaining valley section of the dam became seriously suspect for the first time since the original evaluation of the observed seepage in 1950. A significant expenditure to justify, study and remediate the global seepage issue is now anticipated since the sinkhole could represent a major deterioration in the dam's core.

### 3.2 SINKHOLE CAUSES

Sinkholes can be characterized by two general categories: Finite and Progressively Developing. The Finite category consists of those sinkholes that result from a finite size void collapse. The Finite sinkholes tend to close slowly over time but may collapse instantaneously under certain circumstances. Progressively Developing sinkholes are those that result from a "progressively increasing" void that forms then collapses. They tend to enlarge, typically collapses rather quickly, the collapsed material is removed, the void reforms, re-collapses, continuing in this fashion until a large hole is visible at the surface of the dam. Finite voids tend to not be detectible at the surface when they collapse. This is primarily because, over time, the overlying

material slowly moves under gravity into the forming void. The area of disturbance above subsequently spreads laterally outward as subsidence moves vertically. For example, this condition might occur if buried wood debris rotted and formed a void. This process is more of a soil mechanics “settlement” condition, for which only a low area or sag in the surface is observable. In order to fit into the Progressively Developing category, material has to be progressively removed. This removal progresses either; intermittently at low to marginally high lake levels or continuously at very high levels. Removal occurs from the sinkhole root or base area “as the collapse occurs” such that the sinkhole size increases proportionally in a short time and works itself vertically to the surface as a distinct shaft. The excavation of the sinkhole inferred it to be shaft like. It is stressed that this is a general characteristic of void collapse and each category can evolve similarly and appear identically. Possible causes of the sinkhole are listed, in no particular order, in the following paragraphs. There may be other less probable explanations than those subsequently presented and that materialize as investigations unfold.

3.2.1 NATURAL CAVITY COLLAPSE - A Finite sinkhole cause that results from collapse of a pre-construction existing cavity in a crevice in the bedrock. Soil arching, as is the principle of support mechanism in Roman Aqueduct arch/columns, allows voids to form in the cracks/crevices beneath the alluvial deposition in the valley floor. However, the natural river inundation/recession process tends to collapse these voids over geological time, often forcing soil migration into solution voids 50 feet or more below the rock surface. Borings at the site have encountered soil deposits (typically clay) in solution features over 40 feet depth below the rock surface. Another factor is that the dam has been in place for 53 years, with numerous cycles of lake rise and fall that would increase the collapsing effect. A point to note is that the lake has cycled numerous times in the 14 years since completion of the seepage berm. The length of time for which no sinkholes were found casts suspicion on this mechanism since natural cavity collapse would have likely occurred after only a few lake rise/fall cycles. While this cause cannot be ruled out, its’ probability must be lowered to reflect the lengthy time of lake operation during which no collapse was observed.

3.2.2 CONSTRUCTION DEFECT - This mechanism is probably a cause that would exhibit itself as a Finite category sinkhole, but may also be Progressively Developing under certain circumstances. Previous failures at other dams have revealed poor construction practices to include tree root wad burial, low-density compaction, etc. This mechanism very likely could be associated with the inability of the original dam constructors to adequately clean out deep crevices (known to penetrate 25+/- feet). These crevices are typically very narrow at the sidewall apex “crevice bottom”. Thus, no further excavation could physically occur. Dental concrete would then have been placed on unconsolidated soils susceptible to high gradient erosion. Construction photos clearly show sloughing of excavation slopes and other suspect construction practices. Concrete was found 43 feet upstream of the core trench in the sinkhole investigation boring SH-4.

3.2.3 INTERNAL EROSION/PIPING – This, typically Progressively Developing sinkhole, cause is related to internal erosion from high gradients occurring in the near vicinity of the water seal interface between clay core and the prepared foundation rock in the core trench. It could also occur in the foundation rock below dental concrete or mortar bed covering. This cause would lead to a sinkhole in which the void size progressively increased. It should also be noted

that the original construction reports and plans clearly document the presence of two particularly large fissures and their subsequent filling/repair. These fissures were located in the center of the clay core excavation trench. They also happen to lie almost directly beneath the sinkhole. These foundation conditions are illustrated in Plate F-5.

Internal erosion/piping can be further subdivided into three progression extents depending on the exact location of the internal erosion or piping mechanism that could have formed the sinkhole. These mechanisms are described as follows.

3.2.3.1 MECHANISM A - Non-colloidal materials (non-plastic soils such as sand-gravel mixes that comprise the alluvial foundation and pervious shells) could be transported into the rock foundation. The foundation rock contains a highly irregular conduit system of solutioned opened or filled cavities. When these cavities are filled with transported material, the resulting sinkhole evolution stops. A sinkhole formed by this mechanism is considered Finite and would not progressively develop. This is due, in part, to the material's ability to lodge at pinch points in this irregular walled and convoluted path system. Additionally, the material could be filtered by the downstream strata overlying the path. This does not mean that numerous solution paths could not be forming other sinkholes similarly at other locations, it simply means a single sinkhole, once formed and collapsed to the surface, would halt in development; and once the sinkhole is refilled/repared, it would not reform.

3.2.3.2 MECHANISM B - Non-colloidal materials are being transported to an exit face which exists along the solution path system, allowing these "sand-gravel" materials to pipe from within/beneath the dam (i.e., they do not lodge at pinch points). This mechanism requires an exit face through which material is progressively removed from beneath the embankment or an enormously large cavity exists that could receive a huge amount of transported material. Since no ground surface exit face exists (groundwater only encounters the surface at extremely high lake levels) and a possible "river bed" exit face is over 1000 feet distance from the sinkhole and extremely large sinkholes are not suspected, there is a very low probability that materials are moving that distance without "pinching off" as in Mechanism A.

3.2.3.3 MECHANISM C – Colloidal plastic soils (silts-clays as comprise the dam core and which are graded too small in particle size to "pinch off") could be progressively transported downstream. This mechanism is very detrimental to the dam because colloidal material (material finer than No. 200 gradation sieve size – 0.075mm) erodes under high seepage gradient and can pass unhindered/unfiltered into the downstream strata of sand/clean gravels, or into bedrock fracturing, exiting downstream undetected into the tailwater. This mechanism can go undetected for years as has commonly occurred when sewer line rainfall infiltration (only developing under a few feet of gradient head) has led to rather large sinkhole collapses under city streets. Thus, if this condition exists and due to the large seepage gradients occurring during a lake levels greater than El. 530, cavity/sinkhole enlargement can be significant and can progressively develop to the extent that failure occurs via dam crest collapse/lowering.

### 3.3 INVESTIGATIONS

### 3.3.1 SURFACE GEOPHYSICAL

The Engineering Research Design Center (ERDC) Geotechnical Laboratory was contacted on 22 January 2003 to assist in identifying the path and origin of the sinkhole through geophysical investigations. The ERDC recommended the Kansas Geological Survey (KGS) to provide the necessary expertise based upon ongoing R&D projects with similar expectations. The KGS is a proponent of innovative techniques under evaluation by the U.S. Army's effort to detect underground tunnels and cavities. KGS personnel completed Surface Wave Surveys (shallow depth resolution) and High Resolution Reflection (extended depth resolution) surveys across the surface sinkhole area on 14 March 2003 with the following conclusions:

The surface wave data inferred the surface sinkhole path was elongated to the right below 45-foot depth. It does not extend more than 8-foot downstream, or more than 16-foot upstream at this horizon.

Reflection data inferred that at 45-foot depth the disturbance to be about 15-foot wide upstream/downstream, and extending from 8-feet left of the sinkhole to 8-feet right of the sinkhole. From there, it drifts strongly to the right. At 70-foot depth it extends to 10-feet left of the sinkhole to over 40-foot right of the sinkhole. Going deeper, the disturbed material appears to meander around resistant layers that form bridges or subside intact. At the top of the impervious core "wedge" there is indication of subsidence which continues to move to the right and intercept a fractured bedrock surface in alignment with the large clay-filled joint encountered during construction, between 35 to 80-feet right of the sinkhole.

Reflection data also reveals a second disturbed area located 35-feet left of the sinkhole, extending from bedrock to approximately 75-feet from the surface. This is the location of the second of the two large clay-filled joints or fissures (20' wide X 25' deep) encountered during construction.

### 3.3.2 BORINGS

During 8 April to 13 May 2003, Bowser-Morner, INC, installed six "Vibra-Sonic" borings surrounding the questionable bedrock area delineated by the prior KGS investigations. These borings are identified with the prefix "SH" as shown on Plate F-6. The borings were Drilled into bedrock and sleeved with 4-inch ID PVC casing to provide improved resolution of the disturbed area through subsequent cross-hole seismic investigations. Drill action and sample recovery indicate that:

Borings adjacent to the core trench indicate the impervious clay core to be continuous to the top of rock or the concrete contact with the original cutoff trench.

Borings SH-1 through SH-4 suggest continuous bedrock with no loss of drill water return.

Forty-feet into bedrock, borings SH-5 and SH-6 both experience water loss and uneven drill action at approximately 179-foot depth

### 3.3.3 CROSS HOLE SEISMIC

Cross-Hole investigations were desired to provide the detail on the origin of the sinkhole with the degree of success dependant on the completion of the previous techniques. The entire investigative process evolved from consultations with the staff of Canada's British Columbia Hydropower, which had a strikingly similar sinkhole develop on an earthen dam of like construction. Corroborating suggestions were also provided by the ERDC and the KGS, with recommendations to enlist the Bureau Of Reclamation (BOR) to conduct the investigations. The BOR completed fifteen cross-hole seismic profiles between the six borings on 8 June 2003 with the following conclusions:

Three-dimensional P-wave and S-wave velocity models computed from 15 crosshole seismic surveys clearly show reduced-strength alluvial and embankment materials near the sinkhole and overlying a wide bedrock fissure at Clearwater Dam. This tends to infer that the extension of the bedrock fissure mapped in the cutoff trench during dam construction (located to the southwest of the sinkhole under investigation) is clearly seen in P-wave velocity images. The fissure is elongated in the upstream-downstream direction and vertical. The fissure, as indicated by the P-wave velocity images, is about 15 feet wide at the top of rock and narrows slightly with depth. It extends throughout the depth range of the survey - i.e., the fissure extends below elevation 400 feet. The fissure to the left of the sinkhole, noted during the original dam construction, was not within the geophysical scope. The S-wave velocities in the lower embankment (below about elevation 485 feet) show a distinct low-velocity zone that is directly over the interpreted fissure in bedrock, indicating that the embankment materials above the fissure have been loosened and weakened with time. The average decrease in S-wave velocity within this zone is about 20%. At shallower depths, the low-velocity zone becomes smaller and tends to be more concentrated toward the dam axis. However, a relative low-velocity zone in the vicinity of the downstream edge of the crosshole site (just to the right of borehole SH-3) persists to within 25 feet of the ground surface. Because of decreased resolution above this depth, we cannot determine if the low velocities extend to shallower depths. Other areas of decreased S-wave velocity are also present. Of note are sharply decreased velocities within the upper clay core near borehole SH-6 and within upper embankment materials (above 45 feet depth) near borehole SH-4. Both of these anomalous areas, as well as other less distinct low-velocity anomalies, appear to be related to (mainly) vertical piping of embankment materials in the vicinity of the bedrock fissure. Downhole natural gamma and neutron logs indicate spatially variable densities in the clay core and lower pervious shell materials, consistent with material piping interpreted from the S-wave velocities. Several of the low-density zones indicated by the borehole logs directly correlate with low S-wave velocities computed from the crosshole data.

### 3.3.4 DOWNSTREAM GEOPHYSICAL INVESTIGATIONS

Geophysicists from the ERDC Center of Expertise completed seepage investigations along the downstream areas of the dam and valley on 19 July 2003. Self-Potential, Resistivity and several Electronic/Magnetometry techniques were applied in arrays parallel to the dam axis to detect possible seepage paths. Anomalies were detected among the different methods employed, but did not display a consistency to suggest any positive seepage paths. Repeating investigations of this type at elevated pool levels are typically required. However, no additional surveys are planned at this time.

### 3.4 INTERIM REMEDIATION AND INVESTIGATION

The sinkhole presence is being addressed as a critical situation until it can be confirmed otherwise, or until a long term remediation is designed, approved, funded and constructed to control the historical seepage distress. It was determined that further investigation and interim remediation efforts should be undertaken as soon as possible to mitigate the risk of further collapse of the discovered sinkhole and the surrounding areas. This effort was entitled the “Foundation Drilling and Grouting – Sinkhole Repair Project”. A site plan of the grouting program is shown on Plate F-6.

The grouting contract consists of drilling and grouting to: determine if the sinkhole path has penetrated and disturbed the clay core, to evaluate the condition of the foundation rock in the area of the sinkhole and to affect a temporary repair by grouting. This project also includes an extensive program of clay core and foundation bedrock sampling and testing.

The contact surface between the core’s erodible soil and non-erodible foundation bedrock (or dental concrete/mortar treatment) is the most likely location of through seepage that could lead to dam failure. Both permeability and pressure testing will be performed at the clay core/foundation interface for each grout hole drilled. In addition, continuous sampling and close space drilling (5-foot interval) should produce an excellent testing program of the interface area. If the interface contact zone is found undisturbed, free of foreign materials, of low permeability at the interface (that is the foundation holds pressure during the testing), over the entire length of the grout curtain, then it would be reasonable to assume the interface zone in the entire remaining valley section of the dam is in good condition. However, this is only a statistical inference and may require physical or further geophysical verification.

The grout curtain program consist of using balanced and stable grout with no bleed or shrinkage and will be monitored by the computer aided grouting system software which is considered “state of the art” in the grouting industry. All appropriate tests will be made prior to commencing grouting. This should result in an excellent grouting program. The foundation rock in this area shows extensive weathering and the most advanced grouting methods and equipment is necessary to construct a successful grout curtain. In addition to constructing a grout curtain, close space drilling (grout holes on 5-foot centers) will produce a clearer picture of the geology in the sinkhole area. Specifically, the rock hardness of the unweathered bedrock must be adequately defined in order to estimate the preferred equipment excavation methods and equipment.

#### 3.4.1 OBJECTIVES

The adopted objectives of a viable interim remediation alternative were: a) repair possible core erosion and reduce dam failure risks in the near term, and to a lesser degree, be expandable as construction details were revealed that supported an emergency continuation of interim remediation along the remaining dam length b) be relatively inexpensive since the long term remediation would likely duplicate the benefits of the interim construction c) be quickly implemented prior to the next flood season d) provide correlation with earlier geophysical testing, instrumentation data and observations as to the sinkhole cause mechanism and

disturbance locations and e) detect possible piping features or defects within core, or the absence thereof. If detected, these possible defects could support the drastic measure of significantly lowering the lake. The absence of defects would infer the integrity of the entire dam's "core and prepared foundation interface seal", and secondarily, prepared foundation surface, dental concreting condition, and original grout curtain quality. The Grouting alternative fully met the short-term action goals. The grouting alternative simultaneously provided close line drilling, sampling, and testing of the core, core trench foundation and dental concrete preparation. Additionally, if the theorized cause of sinkhole was, in fact, related to the "piping beneath dental concrete" cause, then grouting construction would probably be relatively low cost since rather low grout take volumes would occur through most of the proposed treated dam segment. Grouting was also considered as a long-term alternative but was eliminated as a "stand alone" long-term alternative. This is due primarily to the unlikelihood that the left abutment could be grouted in an economic manner. The interim grouting project in the valley could be incorporated as a viable alternative under the future long-term remediation design, depending on its success.

### 3.4.2 DETAILS

The initial section of the grout curtain is 180 ft. length and is extendable to 400 ft. length if the final segments are added. It is anticipated that the entire 400 ft. length will be constructed. The 400-foot grout curtain includes the sinkhole at Sta. 39+87+/-, and the two major cavity features on either side at approximately stations 40+10 and 39+50. Payment bid items were developed to allow contract adjustment of the final grout curtain length. The grout curtain is to be located atop the original grout curtain line, at the core trench centerline in order to assess the grout curtain's current water tightness, to determine core and core interface integrity, and to reseal the foundation at the location where the best chance of doing so will occur and require lower grout takes. This location also allows placing equipment for ease of operation, on the impervious blanket top end horizontal surface at el. 575, directly over the core trench. Another factor is that the upstream edge of the core base can be expected to have eroded and raveled from concentrated seepage into rock. If eroded, granular material would likely have been transported into the core, but not necessarily through the core and the core may remain essentially as constructed with ample seal along most of its' base. If the grout boreholes are placed to close to the upstream core base edge, it may infer a poor interface seal when the opposite is the case. This error would affect the assessment of the core's integrity, thus selection of a long-term remediation.

### 3.4.3 SCHEDULE AND COSTS

This effort has been contracted and Notice to Proceed will be given on December 4, 2003. It will require approximately 6 months construction time at a cost of approximately \$2.1 Million dollars.

## 4.0 EXPERT CONSULTANT SOLICITATION

An independent outside consultant was sought in an effort to assist the Little Rock District (SWL) in analyzing the methodology, parameters and material characteristics used in the seepage and sinkhole failure mechanism investigation. Additionally, the consultant would provide expert counsel and conclusions on all viable alternatives and conceptual designs and assist in the selection and justification of the recommended remediation alternative to correct the historical seepage condition.

Dr. Steve J. Poulos was highly recommended and his expertise and services were acquired by the Little Rock District (PR&C #2307936) on August 18, 2003. Dr. Poulos made a project site visit with other SWL personnel during the week of August 25, 2003. Dr. Poulos was provided with copies of historical construction data and other information that was critical to his investigation. The Little Rock District was very appreciative of the level of expertise and thoroughness of his assistance. His advisory report containing conclusions and recommendations are attached as Appendix F1. Additionally, Dr. Poulos' resume is given in Appendix F2.

## 5.0 SEEPAGE ASSESMENT

### 5.1 NON-HOMOGENEOUS/ISOTROPIC CONDITIONS

Karst geology (such as at Clearwater) is inherently non-isotropic, non-homogeneous; therefore, presents a degree of difficulty in predicting seepage conditions. However, if the weathering condition of the rock is extremely advanced, seepage generally begins to trend as if moving through an isotropic, homogeneous material. This is evident by examining the general paralleling trend of the phreatic surface contouring developed from piezometric data collected over nearly 35 years at Clearwater Dam. This pattern suggests the use of standard seepage analysis methods may be generally appropriate to infer seepage conditions. It is stressed that these methods only infer seepage gradients and paths, and isolated gradient anomalies may exist at any given subsurface point. If conservative assumptions are routinely adopted through out the analyses, the resulting predictions of seepage are bounded on the conservative side of the true conditions and become a reliable tool on which engineering decisions can be based.

### 5.2 STEADY STATE SEEPAGE

This condition becomes a factor when assessing the rate of rise of the phreatic surface relative to the lake rise. Inspection of piezometric data generally indicates that piezometers (specifically those located downstream of the core trench) lag behind lake peak by two to six weeks. Prior to placement of the 1989 seepage berm, most piezometers along and upstream of the core trench/dam axis reacted rapidly to lake rise with very little lag time. This condition changed upon completion of the berm but the extent has not been well analyzed due to the small number of high lake events since 1989. Weir flows in the 1972 and 1980 drainage systems lag similarly to general piezometric activity. The construction of the 1989 seepage berm extended the lag time considerably as indicated by the following observation. The Steady State Seepage development time is estimated for use in developing the Failure Scenarios and time development rate of breaching as subsequently discussed.

### 5.3 NON-STEADY STATE EFFECT ON CORE

This effect is related to the significant extent of time required to re-establish a new phreatic surface through the core, which would be associated with a temporary rise in lake level. As lake rises, water begins saturation of the core face above the pre-rise phreatic surface (i.e., the saturated soil line through the core), and continues flowing into the core at the partial-saturation suction/permeability rate for the particular soil type in the various lifts of the core. This suction/permeability rate for a moderately plastic soil is somewhat higher than the saturated permeability rate, but is still very low relative to most marginally to high permeability soils such as exist in the Pervious Shells. Saturation of the core continues as the lake rises until a new

phreatic surface is established at steady state seepage, if the lake is allowed to remain at the higher elevation a sufficient time for the mechanism to complete. For simplicity, the saturation line is often approximated as a straight line from headwater to tailwater although it actually is shaped in a concave fashion as the lake begins to rise, moving to a parabolic surface at completion. For clayey soils with permeability of  $1.0 \times 10^{-6}$  or lower, feet per sec, this time event is lengthy, and theoretically occurs over several months to a year or more. The higher lake levels that can occur in the flood pool, above about El. 520, are present for one month or less, and never more than three months. Therefore, the core probably never fully saturates to the gradient line associated with the higher lake levels. It is important to note that this saturation time rate mechanism is difficult to predict since very small seams of more permeable soils lying within the core will affect the steady state establishment time. This phenomenon allows dams and cores to safely tolerate very high rises in lake so long as the high levels are vacated in a reasonable time period; and which is the case for standard operation of the Clearwater Lake when in surcharge pool (lake above spillway crest). This condition is cited because Clearwater doubtfully meets design criteria for core thickness to gradient ratio, and filtering zone against the downstream core face. Samples will be directly obtained during the interim grouting project. These samples will be inspected for possible core migration. This condition will be addressed during design phase to determine the maximum gradient allowable through the core.

#### 5.4 BASELINE FLOW NET ANALYSIS (CURRENT CONDITION)

##### 5.4.1 LEFT ABUTMENT RIDGE SECTION

Referencing Plate 7, two dimensional (cross-sectional) flow net analyses were performed to estimate the Current Condition theoretical seepage gradient profile at the Left Abutment Ridge Section (aka. the “Window” area near Sta. 31+00). The theoretical analysis agrees well with the observed seepage gradient profile developed from past piezometric measurements. Also, Future Condition two dimensional analyses were performed to infer the degree of cutoff that would exist with the Recommended Alternative in place to cutoff the alluvial seepage path thus forcing all flow into bedrock through the gravel-sand filled crevices, solution conduits-fractures that were likely not grouted during original dam construction. The permeability of the gravel filled creviced rock was approximated by weighting the amount of crevice flow area to the total rock flow area. The permeability of each material was then considered in the proportions of the crevice area. This proportional weighting inferred that permeability through the creviced/fissured rock is about 10 percent of the permeability of assumed clean gravel lying within the crevices. It is important to note that the gradients would reduce to zero if the creviced rock permeability were essentially impermeable. The theoretical analysis merely infers that the success of a seepage retarding remediation measure is dependent on the actual permeability of the rock.

Three locations identified as Point US (for upstream at toe of dam slope), CL (for centerline of dam), and DS (for downstream toe of dam slope) are identified on Plate 7 to show theoretical gradient change along the seepage path.

##### 5.4.2 VALLEY SECTION

The Valley Section seepage gradient profile was not analyzed theoretically because the observed conditions correctly define the Current Condition seepage profile as approximately horizontal

lines in the shells with a near straight line drop through the core. Simplistically stated for the Valley Section, HW exists upstream of the core, near TW exists downstream of the core, with the maximum gradient ( $i_{max}$ ) drop across the core equaling 1.3 at normal lake, and approaching 3.8 at maximum lake.

## 5.5 SEEPAGE FAILURE SCENARIOS AND BREACH SIZE

The following failure scenarios and possible breach sizes were estimated based on the current dam configuration and internal physical properties. Further description and hydraulic analyses can be found in the hydraulic appendix of the Clearwater Major Rehabilitation Report.

### 5.5.1 VALLEY SECTION

As lake rises, saturation of the dam occurs by primary seepage movement via alluvial gravelly soils and bedrock fracturing lying below the Seepage Berm horizontal portion. The upstream shell of the embankment is estimated to become saturated to the new peak lake level at approximately 48 hours after that lake level has been reached. This point is the start of the Steady State Seepage Condition at which the gradient is the steepest and possesses the greatest energy for causing internal erosion. Internal erosion occurs within dam structure. A period of approximately 36 hours may pass, at this point, until seepage becomes evident from traditional downstream discharges from the downstream toe and from the base of the left abutment ridge. Failure is estimated to occur at approximately 20% PMF. Steady state seepage has been evident downstream for 36 consecutive hours; discoloration of the discharge will begin to occur. During the next 11 hours, extensive muddy water will be observed at the discharge areas. This would infer that internal piping and erosion is taking place. This would lead to the appearance of large visible sinkholes. During the next hour (48 hours after saturation occurs), it is anticipated that the embankment would begin to settle and rapidly breach. The breach is anticipated to occur to the right descending side of the left abutment/embankment groin, between the groin and the January 2003 sinkhole location. An ultimate breach opening is assumed to measure approximately 100 feet wide at the base, 130 feet high and 360 feet long at the crest, with 1 vertical to 1 horizontal side slopes.

### 5.5.2 LEFT ABUTMENT RIDGE SECTION

A breach size and time sequence for this section was not developed extensively since the Valley Section breach was considered far more plausible, much faster developing, and to a larger spill opening size thus far more destructive to the downstream area. The Left Abutment Ridge breach opening will definitely halt when erosion progresses down to the relatively unweathered rock surface lying at about elevation 520 at the midsection of the ridge. The breach will develop in relatively firm residual clay that is moderately resistant to erosion. A breach width of a few hundred feet to no more than 400 feet is estimated to form very slowly, and thus will only progress downward to a relatively shallow depth during the breach spill event, definitely not developing any near to the maximum depth extent at El. 520. The breach elevation view should be trapezoidal to somewhat triangular with the apex near mid opening. While this breach scenario was not used in the Breach Analysis for inundation prediction, it is considered a very probable development, essentially equal the Valley Section breach probability because of the extreme state of weathering and relative pervious nature of the soils along the left abutment ridge/window area.

## 6.0 SEEPAGE REMEDIATION MEASURES

The following structural remediation measures were considered for implementation within the project site. The site plan indicating the locations of each measure is given as Plate 8. Section views of each measure are shown on Plates 8A through 8D.

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

Each alternative was developed considering the best alignment that would optimize or minimize various issues and risks related to cost, success of controlling gradient or eliminating seepage, and experiencing future damage or deterioration to the seepage barrier/cutoff or dam. Four alignments are believed the most viable: Alignment (a) on the Seepage Berm’s horizontal surface (El. 502+/-), between the upstream end and the toe of the dam’s upstream slope. Alignment (b) at the toe of the existing embankment. Alignment (c) within and along the core trench (provides the best opportunity to restore integrity of the dam’s core - the dam’s original seepage control feature). Alignment (d) along the dam axis (centerline of roadway) which lies 105+/- feet downstream of the Core Trench/Grout Curtain (provides the best opportunity to form a new seepage cutoff downstream of possible defects that formed over the 53 year life of the dam and that possibly currently exist).

Alignments (a), (b) and (c) would require the impervious blanket extension, and alignment turns in order to follow existing topography, or aid in constructing, and/or for intersecting internal dam features to which the barrier/cutoff would connect. Alignment (d) would not require the extension in order to form a complete impervious seal, nor any major alignment turns. However, this alignment would require a costly detour of traffic during construction. The cost of a detour would be significant even for a temporary situation, requiring a near full scale road and bridge construction to safely re-route traffic across the Black River. Therefore, alignment (d) would likely result in a permanent Highway H-H relocation, in the event that a detour during construction could not be arranged, and produce a secondary benefit that public access to the top of the dam would be terminated. This effort would decrease risks associated with terrorist or natural disaster disturbance of that route.

### 6.1 MEASURE S1 Extend the Impervious Fill Blanket

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

### 6.1.1 DISCUSSION

Measure S1 is considered only a partial solution to the global seepage issue because it does not address the primary concern of subsurface seepage and piping. The estimated contract cost of this measure \$8.5 million with a construction time frame of approximately 15 months. Measure S1 is not recommended as a stand-alone solution.

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

Construct a bentonite cement cutoff wall penetrating to rock. This would be placed upstream of the dam toe and through the existing seepage berm. It would begin at the right abutment and extend out approximately 500 feet onto the seepage berm and terminate into high ground near the left abutment. The total length would be about 4,300 feet. The cutoff would extend to a depth of about 70 feet to top rock. The impervious blanket would not be extended.

### 6.2.1 DISCUSSION

Measure S2-A only addresses seepage through the alluvial soil strata and does not attempt to reduce seepage in the underlying bedrock. High lake levels (+ EL 575) would lead to immediate saturation and high flow gradients into the unprotected window area if the seepage berm were not extended. Furthermore, the risk of relying on the impervious blanket along the dam face as a barrier is relatively high. The estimated contract cost and construction time are \$3.9 million and 6 months, respectively. Measure S2-A is not recommended.

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

Same as Measure S2-A except the impervious blanket would be extended.

### 6.3.1 DISCUSSION

Measure S2-B only addresses seepage through the alluvial soil strata and does not attempt to reduce seepage in the underlying bedrock. Furthermore, the risk of relying on the seepage blanket on the dam face as a barrier is relatively high. The estimated contract cost and construction time are \$12.3 million and 20 months, respectively. Measure S2-B is not recommended.

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

Same as Measure S2-B except the location of cutoff wall is moved to the upstream toe of the existing dam. There is no change in reliability for change in location.

### 6.4.1 DISCUSSION

Measure S2-C only addresses seepage through the alluvial soil strata and does not attempt to reduce seepage in the underlying bedrock. Furthermore, the risk of relying on the impervious blanket on the dam face as a barrier is relatively high. The estimated contract cost and

construction time are \$12.9 million and 22 months, respectively. Measure S2-C is not recommended.

#### 6.5 MEASURE S3 Slurry Cutoff Wall To Rock Located 500 ft Upstream of Existing Dam Toe Including Deep Panels into Rock With Extension of Impervious Blanket

Same as Measure S2-A except the impervious blanket would be extended and deep intermittent concrete cutoff wall panels would be extended 60 feet into rock where defects or voids are detected. The cutoff wall would have a depth of between 70 feet to rock and 130 feet for the deep cutoff panels.

##### 6.5.1 DISCUSSION

Measure S3 attempts to address seepage below the rock surface with intermittent deep concrete panels. The confidence that seepage could be adequately cut off with this measure is relatively low. Furthermore, the risk of relying on the impervious blanket on the dam face as a barrier is relatively high. The estimated contract cost and construction time are \$21.9 million and 24 months, respectively. Measure S3 is not recommended due to the tremendous concern of relying on the seepage berm and impervious blanket. It has been noted that these features do not have a properly designed filter zone and have not been tested or subject to a high head differential.

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

Same as Measure S2-C except concrete would be used in lieu of bentonite slurry.

##### 6.6.1 DISCUSSION

Measure S4-A only addresses seepage through the alluvial soil strata and does not attempt to reduce seepage in the underlying bedrock. Furthermore, the risk of relying on the impervious blanket on the dam face as a barrier is relatively high. The estimated contract cost and construction time are \$32.0 million and 29 months, respectively. Measure S4-A is not recommended.

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

Same as Measure S3 except that the cutoff wall (either secant pile or rockmill method) would be extended into rock continuously (60 feet into rock) with concrete. The total length would be about 4,300 feet. Steel reinforcement may be required to a minimal depth into bedrock to account for cracking and seismic conditions.

##### 6.7.1 DISCUSSION

Measure S4-B addresses seepage below the rock surface with a continuous deep cutoff wall. However, the risk of relying on the impervious blanket on the dam face as a barrier is relatively high. The estimated contract cost and construction time are \$73.1 million and 40 months, respectively. Measure S4-B is viable but not recommended.

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

Same as Measure S4-B except placement would be through dam and through the centerline of the clay core trench. Penetration of the wall would begin along the impervious blanket at El. 575 of the dam's upstream face and extend 60 feet into rock for a total depth of 200 feet. Length and width would be the same. A berm would have to be constructed for access and to allow for a 30 feet wide working platform. Extension of impervious blanket will be required to prevent seepage inflow directly into the window area. The total length would be about 4,300 feet.

### 6.8.1 DISCUSSION

Measure S4-C addresses seepage below the rock surface with a continuous deep cutoff wall. There would be minimal risk of relying on the impervious blanket as a barrier due to the location of the wall (at top of existing seepage blanket). Difficulty in excavating through the embankment shell material and into rock should be considered. The estimated contract cost and construction time are \$72 million and 39 months, respectively. Measure S4-C is a recommended plan.

## 6.9 MEASURE S4-D Concrete Cutoff Wall Into Rock Located Through the Centerline of the Dam Alignment

Same as Measure S4-C except the cutoff wall location would move to the centerline of the dam alignment. The total depth of the wall would be 230 feet with the same length and thickness. Traffic would be rerouted via detour of about 5 miles to the main Highway. The total length would be about 4,300 feet. Extension of the seepage berm would not be required.

### 6.9.1 DISCUSSION

Measure S4-D addresses seepage below the rock surface with a continuous deep cutoff wall. The seepage berm extension would not be required because the upstream face of the dam would be protected by the centerline location of the cutoff wall. Difficulty in excavating through the entire embankment shell material and into rock should be considered. The estimated contract cost and construction time are \$68.8 million and 38 months, respectively. Measure S4-D is a recommended plan.

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

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

### 6.10.1 DISCUSSION

Measure S4-E addresses seepage below the rock surface with a deep cutoff wall. However, the designers are not confident that the shortened length would mitigate the possibility of detrimental seepage inflow. The estimated contract cost and construction time are \$50.5 million and 34 months, respectively. Measure S4-E is viable but not recommended.

## 7.0 RESULTS OF INVESTIGATION AND ANALYSES

The following points are offered for consideration as a conclusion to the Geotechnical Analyses Appendix to the Clearwater Major Rehabilitation Study Report:

- 1) Seepage conditions at Clearwater have been often observed throughout the life of the project. Many efforts have been attempted in an effort to minimize the seepage effects that had become a nuisance downstream of the dam. These efforts consist primarily of: The 1972 and 1980 drain systems and the 1989 seepage berm. These efforts did not address long-term seepage effects because any attempt to construct a seepage barrier was considered economically impractical.
- 2) A sinkhole occurred in the upstream portion of the embankment in January 2003. This sinkhole appearance and the subsequent investigations have cast doubt on the integrity of the entire dam structure.
- 3) The sinkhole investigation and geophysical studies indicate that the sinkhole shaft is relatively vertical and emanates from the base of the dam. Although the exact cause of the sinkhole has not been determined, it is believed that some anomalies within the karstic bedrock combined with a form of internal seepage and erosion/piping are the causative agents of the sinkhole development.
- 4) The ability of the existing filter zones within the dam structure to perform as designed have also been questioned. Limited testing of the core, shell and alluvial material suggest that the filter zones are sub standard.
- 5) The aforementioned deficiencies have led the design team to consider both interim and long-term remediation measures of the sinkhole and entire dam. The measures considered attempt to address the seepage effects for the projects potential lake levels.
- 6) These measures consist of: deep (into rock) cutoff walls that essentially halt seepage and shallow depth (to rock) barrier walls to retard seepage.
- 7) A foundation drilling and grouting – sinkhole repair project is currently being conducted as an interim measure to repair possible damage of the core caused by the sinkhole. Additionally, much qualitative design information will be ascertained from the drilling and piezometer data obtained during the implementation of this project. This information will be utilized in the further development of the preferred long-term remediation measure.
- 8) An economic analysis of the various remediation measures is currently being completed. The final selection of the preferred alternative relates to the acceptance of risk relative to the current project condition, benefits and potential for catastrophic failure. However, Measures S4-C and S4-D appear to be the most satisfactory choices from a reduction of risk perspective.

## Appendix F1

# EXPERT CONSULTANT'S ADVISORY REPORT