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1.0

INTRODUCTION

This report presents the results of a geotechnical study conducted by Woodward-Clyde Consultants (WCC) for the proposed Elkhead Dam expansion.

1.1 PROJECT DESCRIPTION

Elkhead Lake Dam was constructed in 1974 by the Colorado Division of Wildlife (CDOW). It consists of an earth embankment approximately 90 feet high with a crest length of approximately 1,140 feet at elevation 6,375 and a concrete-lined service spillway with a capacity to discharge approximately 17,000 cubic feet per second. The existing embankment has a crest width of 20 feet and upstream and downstream slopes of 3H to 1V and 2.5H to 1V, respectively. The dam is located on Elkhead Creek, a tributary to the Yampa River, about ten miles northeast of the City of Craig in Section 16, Township 7, Range 89W, 6th P.M., Moffat County, Colorado. Figure 1 is a location map. The dam is situated in Water Division 6, Water District 44. The current reservoir capacity at the normal water surface elevation 6,365 is approximately 13,700 acre-feet. CDOW maintains 5,390 acre-feet of storage for recreational purposes, and the remaining 8,310 acre-feet are committed to the Colorado-Ute Electric Association as a back-up cooling water supply for their nearby power generating facilities.

We understand the proposed project will consist of enlarging Elkhead Dam with a 45-foot downstream raise from a crest elevation of about 6,375 feet to a crest elevation of about 6,420. The new dam design would incorporate a new service spillway and outlet structure, as well as adding an emergency spillway and possibly a hydropower generating facility. The Yampa River Basin Alternatives Feasibility Study (1993) recommended enlarging the existing Elkhead Dam and Reservoir as the first component of the recommended alternative for meeting the future water needs of the Yampa River basin.

1.2 PURPOSE AND SCOPE OF STUDY

A field exploration was conducted by the U.S. Bureau of Reclamation (Reclamation) to obtain additional information on subsurface conditions needed for the enlargement design. Previous field investigations were performed at the dam site as described in Section 2.1. Eleven test holes were drilled during this investigation at the locations shown in Figure 2. Samples obtained during this subsurface investigation were tested in Reclamation's laboratory and WCC's laboratory to provide data on the classification and engineering characteristics of the on-site soil and rock. The results of the field and laboratory investigations are presented in Appendixes A, B, and C, respectively.

This investigation was conducted to meet the following objectives:

1. Provide additional information on subsurface conditions and construction materials near the damsite.

2. Characterize the overburden and bedrock.

3. Provide data needed to design the project features, including availability and properties of construction materials.

4. Characterize the geologic setting of the project area in terms of seismic potential, structural geology, and foundation strength.

This report has been prepared to: (1) summarize the data obtained from this investigation and the project record, and (2) to present our analyses and recommendations on the proposed enlargement. Engineering analyses were conducted for the proposed raise as discussed in the Work Plan for Geotechnical Study, Enlargement of Elkhead Dam (W-C 1993).

2.0

PROJECT INVESTIGATIONS

2.1 PREVIOUS INVESTIGATIONS

Previous field and laboratory investigations for Elkhead Dam have included those for the original dam design and those for spillway modifications. The investigations were discussed in reports by Engineering Consultants, Inc. (ECI) and Morrison-Knudsen Engineering (MKE) which are listed in the reference section of this report. Available test hole or test pit logs are in Appendix A. The previously collected information is summarized as follows:

1. In 1968, five auger holes (EC1 - EC5) were drilled along the then-proposed alignment of the dam as part of a feasibility study (ECI 1971, 1972a). Limited laboratory testing was performed, but neither the logs nor the test results were available.

2. In 1971, six auger borings were drilled along another proposed dam alignment, which was called the "Low Dam Design." Eight auger holes were also done for a borrow study. These were done on the right abutment and the flood plain upstream of the then-proposed dam. Laboratory testing performed as part of the 1971 investigation was unavailable except for some summary graphs which can be found in Appendix C.

3. Two drill holes and six auger borings were drilled as part of the investigation for the current dam alignment (ECI 1973). Twenty-six test pits were excavated to locate potential borrow sources. Four test trenches were dug along the proposed spillway alignment, and one was excavated on the slope of the left abutment. Test trench logs were unavailable. Laboratory testing performed as part of the 1973 study was unavailable.

4. In May 1985, seven test pits were excavated as part of a feasibility study on lining the spillway and downstream face of the dam with roller compacted concrete (RCC) (MKE 1985). The purpose of the test pits was to locate RCC aggregate.

5. In September 1985, two test holes (1 or 1-20CL, 2 or 1-220CL) were drilled on the left abutment (MKE 1985). The holes were drilled as part of a study on installing a fuse plug spillway on the left abutment.

Other Elkhead Dam reference materials used in compiling this report are listed in the reference section at the end of the text.

2.2 PRESENT INVESTIGATIONS

The geotechnical investigations for Elkhead Dam were conducted between June 16 and September 28, 1993. The investigation included test holes in the vicinity of the existing dam and proposed dam raise, test holes in the vicinity of the proposed emergency spillway, and test pits in proposed borrow areas. Samples obtained were returned to Reclamation's and WCC's laboratory for testing. Test holes and test pits were sampled by Reclamation. A geophysical survey was also performed in the vicinity of the proposed embankment spillway and emergency spillway. Hadley & Hollingsworth, Ltd. performed the geophysical survey, which is included as Appendix B.

2.2.1 Damsite Investigation

Twelve borings were drilled as part of this investigation. Borings WC-1 through WC-3 were drilled near the existing dam crest. Borings WC-4 through WC-8 were drilled in areas of the proposed dam raise. Borings WC-9, WC-9A, WC-10, and WC-11 were drilled in the vicinity of the proposed emergency spillway crest and channel. Locations for the borings are shown in Figure 2. Logs are given in Appendix A. Appendix A also contains the location maps and available logs of previous borings at the site. Table A-1 lists all borings done for this and previous investigations.

Borings WC-1 through WC-11 were advanced through the overburden materials using a 4-inch diameter hollow-stem auger. Exploration of the underlying bedrock was accomplished with an HQ3 system using a diamond or popcorn bit and a triple-tube core barrel. Reclamation used a Gus Pech No. 1 truck-mounted rotary drill rig to drill the holes.

Samples of the subsurface materials were taken in approximately 5-foot increments with a 2-inch inside diameter (I.D.) California Sampler. The sampler was driven into the various strata with blows from a 140-pound hammer falling 30 inches. The standard penetration test (SPT) resistance values were recorded in the blows per 12 inches of penetration. Samples were also obtained by hydraulically pushing a 3-inch I.D. thin-walled tube sampler (Shelby tube) into the subsoils. Depths at which samples were taken and penetration resistance values along with percentage of core recovery, fracture density, and Rock Quality Designation (RQD) are shown in the logs in Appendix A.

Five of the borings were completed as open standpipe piezometers. The slotted intervals are indicated on the logs and summarized in Table A-3. Well completion reports are included in Appendix A.

2.2.2 Borrow Investigation

Twenty-three test pits, TP-27 through TP-49, were excavated in borrow areas 1, 2, 3, 4, and 6 for this investigation. The pit locations, except for TP-48 and TP-49, are shown in Figure 3. A backhoe was used to excavate the test pits which ranged in depth from 3 to 15 feet. Fifty-pound sack samples were taken at each pit. Several hand-driven California liner samples were also taken; however, only one was useable. Summary logs of the test pits are shown in Appendix A. Previous test pits were excavated for the original borrow study and for a feasibility study for a RCC spillway (MKE 1985). Location maps and logs of these test pits are found in Appendix A. A listing of all test pits is also given in Appendix A.

2.2.3 Geophysical Survey

A geophysical survey, consisting of approximately 4,000 feet of seismic refraction profiling, was completed by Hadley & Hollingsworth, LTD. from July 27 to July 31, 1993. The purpose of the survey was to estimate the depth to unweathered bedrock and also to estimate rippability of the subsurface materials. Seven seismic lines, ST-1 through ST-7 were completed and are located in Figure 2. The geophysical survey report is found in Appendix B.

3.0

SITE CONDITIONS

3.1 DAM SITE

Elkhead Lake Dam is located on Elkhead Creek, a tributary to the Yampa River, about 10 miles northeast of the City of Craig, Colorado. The valley section is approximately 1,100 feet in width. The right abutment consists of claystone bedrock that rises quickly to elevation 6,400. The left abutment also rises to elevation 6,400, but less steeply than the right abutment. Some seepage is present on the right abutment, as is further described in Section 8.0. Country Road 28 runs across the crest of the dam.

During the field investigation in July 1993, the reservoir was at the level of the spillway. The embankment appeared to have no noticeable slumps or settlement areas. No trees were noted growing on the embankment, which was otherwise well vegetated. The riprap appeared to be in good condition, not showing appreciable signs of wear.

3.2 RESERVOIR AREA

The proposed 45-foot dam raise will result in a normal water elevation of 6,406. The new capacity of the dam will be approximately 45,000 acre-feet, more than tripling the current capacity of 13,700 acre-feet. This new reservoir area is presently rangeland with some riparian vegetation.

3.3 BORROW AREAS

Six potential borrow sites were identified for the dam raise and are shown, generally, in Figures 1 and 3. The potential borrow sites are located either on hillsides near the reservoir or up ravines of tributaries. The surface vegetation at the potential borrow sites consists of grasses and sagebrush. Most of the areas will be inundated by the new reservoir.

4.0

GEOLOGIC SETTING

Elkhead Dam and Reservoir are located in a portion of the Rocky Mountain Physiographic Province called the Wyoming Basin. This part of the Rocky Mountains is characterized by broad linear mountain range uplifts that are separated by large topographic and structural basins. The basin in which Elkhead Reservoir is located can be considered a southern extension of the Green River Basin. The basin is filled with a thick sequence of Cretaceous and Tertiary aged marine, lacustrine, and fluvial sedimentary rocks. The basin is between the Uinta Uplift to the west, the White River Uplift to the south, and the Park Range Uplift to the east. The Elkhead Mountains form a topographic high north of the reservoir. The topography around the reservoir includes low rolling hills, small mesas, and ridges that have formed due to the weathering and partial erosion of the sedimentary rocks within the basin. The streams have dendritic drainage patterns and partially alluvial filled valley bottoms in which the streams often form meanders. Vegetation consists of grasslands with a few trees and shrubs along the streams. No landslides were noted in the reservoir and dam area during our site visit.

5.0

SITE GEOLOGY

5.1 GENERAL

Elkhead Dam and Reservoir are located approximately 10 miles east of Craig, Colorado. Elkhead Dam was constructed on Elkhead Creek, a few miles upstream of the confluence with the Yampa River. Elkhead Creek has incised a wide stream valley, about 300 to 500 feet deep, in the sedimentary rocks that crop out on some of the ridges and hillsides. The somewhat irregular topography in the valley is due to differential erosion of the sedimentary rocks. Some of the cemented, hard, and erosion-resistant sandstone beds form low cliffs and steep slopes. The sedimentary rock is generally weak and relatively easy to erode; therefore, Elkhead Creek and its tributary streams have formed many deep sloped ravines and stream valleys. The topography in the upper portions of the valley, at elevations above the reservoir, consists of rounded hills, ridges, and small flat to gently sloping mesas. Figure 4 is a geologic map of the damsite area.

The stream valley in the vicinity of the dam is characterized by a sinuous shape and relatively steep valley walls. This type of topography was probably formed by regional uplift of a meandering stream channel, which caused the stream to down cut instead of widen its path. Elkhead Reservoir has an unusual shape because it inundates part of this sinuous stream valley. Over time, the stream valley has become partially filled with alluvium to a depth of about 10 to 15 feet, based on subsurface information at the dam. The present Elkhead Creek meanders through this alluvium at the valley floor, forming channels a few feet deep.

The right abutment of the dam consists of a steep slope formed along the outside of a large meander in the valley. Bedrock is present at a depth of only a few feet on much of this side of the valley. The relatively steep slope on the right abutment becomes less steep at the top of the ridge to the west of the dam. A portion of the ridge was excavated either for dam borrow or for a road cut. The left abutment is on the inside of the meander, on a valley slope that is less steep. Bedrock contains a thin soil and slopewash cover on the lower left abutment and a progressively thicker cover on the gentle slope at the top of the left abutment. The gentle slope at the top of the left abutment continues for several hundred feet to the east. The spillway has been constructed in a cut excavated into the bedrock and soil cover on the left abutment. The bottom of the valley at the dam is flat and is about 700 feet wide.

5.2 BEDROCK STRATIGRAPHY

Bedrock exposed near the dam and reservoir consists of a sequence of Cretaceous to Tertiary aged lacustrine and fluvial sedimentary rocks. These are shown on the geologic map, Figure 4. The sequence consists of mostly sandstones and shales, with minor amounts of sub-bituminous coal. The sequence is part of the Upper Cretaceous Lance Formation and Lower Tertiary (Paleocene) Fort Union Formation. In general, the sedimentary rocks are weak and easily broken apart by hand. The sediments forming the rock originated from sources in the surrounding uplifted areas that were a relatively short distance from the basin in which they were deposited. The sediments contained mica and feldspars that had not completely weathered to clay minerals. The sandstone exposed in the dam abutments is silty and clayey, arkosic, light brown, friable and soft, fine to medium grained, and contains hard to very hard calcareous cemented irregular lenses and concretions 2 to 4 feet thick and some thin shale beds and rip up clasts (broken clay beds within the sandstone). The friable sandstone lacks bedding, jointing, and other structures. The friable sandstone exposed in the spillway cut does exhibit an exfoliation type of weathering, in which thin sheets of the sandstone tends to separate from the bedrock surface in thin sheets before it breaks down into sand-sized material. Dark grey shale with some interbedded black sub-bituminous coal and black shale is present in the road cut area on top of the right abutment.

5.3 SURFICIAL DEPOSITS

Quaternary alluvium and surficial deposits, including soils, slopewash, and colluvium are present at the dam site. The alluvium consists of sand and gravel overlain by sandy clay. The alluvium is about 10 to 15 feet thick and fills the bottom of the stream valley. The clayey alluvium represents overbank deposits along the meanders in the present stream channel. Slopewash and colluvial deposits interfinger with the alluvium along the bottom of the valley slopes. The colluvium consists of sand to large boulder-sized fragments weathered from the bedrock. The slopewash includes fine sand, silt, and clay that move downslope due to precipitation that moves over the surface during heavy rains. The slopewash is mixed with colluvium and soils to form a heterogeneous deposit with variable thickness. On steeper slopes, these deposits are a few inches to a few feet thick. On relatively gentle slopes, the slopewash and soils are up to 10 to 15 feet thick and consist of clayey material. The upper left abutment contains a thick clayey slopewash sometimes underlain by a gravelly terrace deposit. Within the reservoir area, clayey slopewash and soil deposits have accumulated at the bottom of small ravines and adjacent relatively flat slopes.

5.4 STRUCTURAL GEOLOGY

The prominent geologic structure at Elkhead Dam and Reservoir is the regional dip of the bedding, which has a northeast strike and a dip of about 5 to 10 degrees to the north to northwest. The strike and dip may be somewhat variable, with steeper dips in the northern part of the reservoir. The presence of irregular lenses within the sandstone and undulatory bedding contacts combined with a lack of bedding in some of the sandstone results in variable individual strike and dip measurements. The regional orientation of beds can be measured by viewing outcrops in the reservoir area from a distance to get an overall average strike and dip. Folding may also be present at the site, which would cause a range in bedding dip angles to be observed.

Most of the friable sandstone beds are thick bedded to massive and the shale is laminated to thinly bedded. Some contain cross bedding. Sandstone exposed in the spillway cut slopes contained shale rip up clasts, suggesting that much of the original sedimentary structures, such as bedding, were destroyed by the processes that formed the rip up clasts. The hard cemented sandstone lenses and concretions contain close to widely spaced joints. The joints appear to be randomly oriented. The shale rip up clasts had laminated bedding with gray and black beds. The shale is typically fissile with thin partings that are sometimes carbonaceous. Some of the shale is interbedded with very thinly bedded sub-bituminous coal.

6.0

SEISMOTECTONIC SETTING

6.1 GENERAL

Elkhead Dam and Reservoir are located within the Uinta-Elkhead seismotectonic province as described by Kirkham and Rogers, 1981. Faults within this province consist mainly of northwest trending structures that may be the northern most extension of the Rio Grande Rift Zone that crosses through central Colorado. The faults are believed to be surface expressions of deep crustal structures in the Precambrian basement rocks. This area is probably a zone of crustal thinning at the contact between the 2.5 billion year old Precambrian rocks in Wyoming and the 1.8 billion year old Precambrian rocks in Colorado. Historical seismicity in the region includes Intensity V and less than Magnitude 5 events.

6.2 PRINCIPAL FAULT SYSTEMS

Kirkham and Rogers (1981) mapped potentially active faults in the area. The faults can be grouped into three areas: (1) faults in the Cross Mountain - Craig area, (2) faults in the Elkhead Mountain area, and (3) faults in the area between Steamboat Springs and Tonapas. The faults in these areas as well as additional faults are shown in Figure 5. The faults between Cross Mountain and Craig consist of an echelon horst and graben structures. Some of the fault scarps have had Quaternary and Holocene movement. In the Elkhead Mountains area, west to northwest trending high angle normal faults show signs of Tertiary and Quaternary movement. A number of relatively long (up to 22 miles long) faults with Tertiary and Quaternary movement have been mapped between Steamboat Springs and Tonapas, Colorado. The structures are mostly high angle normal faults and may be structurally related to the Rio Grande Rift Zone to the south.

6.3 EARTHQUAKE RECORD AND HAZARD EVALUATION

Seismicity within Colorado and the vicinity is summarized in Figure 6. The data shown in this figure is based on the database compiled by the National Geophysical Data Center. A review of this database indicates that an Intensity VI earthquake occurred in the axial basin in the Cross Mountain and Craig area in 1891. Other small magnitude earthquake activity has been documented in this area. In the Elkhead Mountains area, the historical seismicity consists of events of less than Magnitude 5. In the area between Steamboat Springs and Tonapas, historical seismicity includes Intensity V and less than Magnitude 5 events.

Algermissen (1969) has mapped the site and vicinity within Seismic Zone 1, which is a zone of low seismic risk where earthquakes of Intensity VI and smaller may occur. Kirkham and Rogers (1981) indicate that this portion of Colorado should be included in Seismic Zone 2 because "several earthquakes of intensity VII have been recorded historically in Colorado and geological evidence suggests large earthquakes have happened in the recent past and are likely to recur in the future." Kirkham and Rogers (1981) suggest a Magnitude 5.5 to 6.5 maximum credible earthquake for the Uinta-Elkhead seismotectonic province. Based on fault length, a Magnitude 5.5 event may be possible about 20 km from the site and a magnitude 6.5 event may be possible about 70 km from the site. A Magnitude 5.5 event, at a distance of 20 km to the nearest fault and at a focal depth of 20 km, would yield a ground acceleration of about 0.15g. A Magnitude 6.5 event at a distance of 70 km at a focal depth of 20 km, would yield a ground acceleration of about 0.2g.

6.4 SEISMIC DESIGN CRITERIA

Low risk structures at the site can be designed based on the seismic Zone 1 criteria as mapped by Algermissen (1969). This criteria is incorporated into building codes in many areas of the United States for low risk structures such as single-family dwellings. The peak bedrock acceleration for design of the low risk structures is 0.1g.

The criteria used for the dam should be based on the seismic hazard evaluation of potentially active faults in the area. A peak bedrock acceleration of 0.2g should be used for design of the dam and any structures that might affect the integrity of the dam.

7.0

SUBSURFACE CONDITIONS

7.1 FOUNDATION SOILS

Field investigations for this study showed that the thickness of the alluvium ranges from 9 to 20 feet from the edge to the center of the floodplain. This alluvium was characterized in the 1973 geology report (ECI 1973) as 5 to 7 feet of clayey, sandy silt underlain by 3 to 8 feet of gravel. The current borings placed in the floodplain indicate a sandy clay and/or clayey sand and gravel soil above an interlayered claystone and sandstone bedrock. Boring WC-5 (Figure 2) showed the thickest alluvium, approximately 12 feet of clay overlying 6 feet of sand and gravel. The sandy clay material was soft to medium stiff, having SPT blowcounts from 3 to 16 blows per foot. The blowcounts in the sand and gravel were 45 blows per foot and over. A profile section, taken at the proposed dam crest is shown in Figures 7 and 8, incorporating all nearby log data from this and previous investigations.

Foundation material on the abutments consisted of a residual clayey soil on the right abutment and a gravel terrace deposit on the left abutment. The right abutment had approximately four feet of residual soil overlying bedrock. The far left abutment had approximately 3 feet of clayey gravel overlying bedrock. Borings along the proposed spillway, also located on the left abutment, showed about one foot of clayey overburden increasing to 9 feet.

Foundation conditions in the area of the proposed emergency spillway consisted of residual clayey sand to sandy clay. The thickness of this deposit increased from 3 feet at the proposed crest (WC-10) to 14 feet at the end of the proposed channel (WC-11).

The results of the geophysical survey generally showed similar bedrock depths as the borings. Overburden was generally found to consist of a weaker soil overlying a denser soil overlying dense bedrock. The seismic velocities did not indicate a clear boundary for unweathered rock; rather, the transition from overburden to rock was found to be gradual. Most of the upper 10 feet of the bedrock was considered rippable from the compressional wave velocities recorded.

Laboratory testing was performed on samples taken from the drill holes. The purpose of these tests was to classify the material and/or calculate the index properties and strength characteristics. Testing for the soils was performed at either Reclamation's laboratory in Grand Junction or WCC's laboratory in Denver. The laboratory data and summary tables are found in Appendix C. The classification and index property testing done on the samples consisted of: (1) grain size analyses, (2) Atterberg limits, (3) natural water contents, (4) specific gravity, and (5) dry unit weight. Other tests which were run included water soluble sulfate tests and unconfined compression tests.

Most of the foundation soils classified as sandy clays, with some clayey sands and a few silts. Percent fines in these soils ranged between 50 to 99 percent. The dry unit weights varied from 89 to 121 pcf, with specific gravities between 2.50 and 2.68. Atterberg limit tests showed liquid limits between 20 to 47 and plasticity indexes from nonplastic to 24. The natural water contents ranged from 1 to 33 percent.

7.2 EMBANKMENT SOILS

Soils from the current embankment were sampled in two drill holes in the crest. The embankment materials were found to be a medium stiff to stiff sandy clay with occasional layers of clayey sand. Blowcounts in the embankment averaged 17 blows per foot in the upper 50 feet of WC-2 and 22 blows per foot in WC-3. No alluvial material was found underlying the embankment, probably because the drill holes intersected the cutoff trench.

Fifteen samples from the existing embankment were tested for index properties. These tests indicated that the embankment fill generally consists of sandy clays (CL) with some clayey sands (SC). Dry unit weights of the fill range from about 102 to 115 pounds per cubic foot (pcf), with natural water contents ranging from about 13 to 21 percent. Liquid limits of these soils were from 23 to 41 percent with plasticity indexes from nonplastic to 17. A specific gravity of 2.60 was measured for the one embankment sample tested.

Laboratory consolidated-undrained (CU) triaxial tests were conducted on one undisturbed embankment sample to evaluate the strength characteristics of the soil. The triaxial test data, including Mohr circle plots and stress path plots are given in Appendix C.

Both total and effective stress parameters were calculated. The embankment sample, a CL, showed an effective friction angle of 35 degrees with an effective cohesion of 0 psf. The total stress parameters consisted of a friction angle of 24 degrees and zero cohesion.

Unconfined compressive strength tests were performed on the sandstone bedrock samples removed in the borings. The strengths ranged from 675 to 2,275 psi.

7.3 GROUNDWATER

No records were found of any water level measurements in the dam or foundation after original dam construction. Five of the drill holes for this study, WC-1 through WC-4, and WC-7, were completed as open well piezometers in both the dam and foundation. Water level readings were only recorded for dam piezometer WC-3 at the time of drilling, which showed a water level 29 feet below the ground surface. Readings taken in August 1993 in foundation piezometer WC-4 indicated that the water level was about 8 feet below the valley surface, which was consistent with the pre-dam water levels recorded in 1973 (ECI 1973). The reading for Piezometer WC-7, on the left abutment, showed a water level about 37 feet below the ground surface. September readings were taken in all piezometers and are listed in Appendix A. No change was found in piezometers WC-4 and WC-7.

7.4 CONSTRUCTION MATERIALS

7.4.1 Soils

Six potential borrow sites have been identified within the reservoir area. These have been designated as Borrow Areas 1 through 6 and are shown in Figure 3 with the exception of Borrow Area 3 which was too far upstream to fit on the map. It is shown in Figure 1. Test pits were placed in all of these borrow locations except for Borrow Area 5. This area was well sampled in the original borrow investigation of the dam (ECI 1973).

Samples were tested from sixteen test pits in this investigation. The borrow materials classified as silty to clayey sands (SP-SM, SM, SC), sandy to silty clays (CL) and a few silts (ML) and gravels (GW-GM, GC). Percent fines on the borrow material ranged from 5 to 94 percent. Liquid limits were between 23 and 42, with plasticity indexes from nonplastic to 20. The minus 4 specific gravities ranged from 2.47 to 2.77. Apparent and bulk specific gravities were also run.

Water soluble sulfate tests were conducted on five samples taken from drill holes either at the emergency spillway area (northwest of the dam) or the left abutment and spillway area of the proposed dam. This test measures the percentage of water soluble sulfates and the corresponding degree of attack on concrete exposed to the soil. Test results indicated negligible attack.

Two CU triaxial tests were conducted on remolded samples from TP-34 and TP-42. The remolded test pit samples classified as sandy clays. Effective friction angles ranged from 24.5 to 26 degrees with effective cohesions ranging from 0 to 7 psf. Total stress friction angles ranged from 14 to 15 degrees with 5 to 7 psf cohesion.

Five pinhole dispersion tests were conducted on samples from the test pits. Dispersion is a process by which certain types of clays erode easily when in contact with water. All classified as nondispersive.

Compaction tests were performed on 23 samples from the test pits. Optimum water contents ranged from 13 to 23 percent with maximum dry unit weights ranging from 93 to 114 pcf.

The borrow materials generally consist of weathered sandstone and claystone, residual clayey sand and sandy clay, clayey and silty slopewash deposits, and gravelly terrace deposits. The seismic refraction survey indicated that approximately the upper 10 feet of the unweathered bedrock should be rippable. A description of the borrow areas sampled and an estimate of the volume of material available follow. Volumes were based on average depths of soil or weathered bedrock encountered in the test pits. The quantities given do not distinguish between embankment fill and random fill.

Estimated Volume of Potential Borrow Areas

Area	Soil (cy)	Rock (cy)
1	250,000	250,000
2	450,000	100,000
3	200,000	60,000
4	300,000	70,000
5	60,000	150,000
6	200,000	80,000

Borrow Area 1 is located near the boat ramp on the west side of the existing reservoir. Test pits already exist at this location from previous studies. Test pits TP-27 through TP-31 are in this area and pits TP-45 through TP-47 are on the hill above the ramp. The lower pits revealed mostly residual sands and clays (0 to 2.5 feet) overlying highly weathered claystone and sandstone. The upper pits (TP-45 through TH-47) contained up to 10 feet of clayey soil. Some of the cleaner sand was not suitable for use as embankment fill. The upper three feet of bedrock was typically easy to break by hand.

Borrow area 2 is located on a knoll which juts out into the existing reservoir northeast of Borrow Area 1. Test pits TP-32 through TP-35 are located in this area. The material in these pits was mostly silty and clayey slopewash deposits at least 10 feet in thickness. A gravel terrace deposit was noted under one of the shallower deposits. No bedrock was encountered to the depths excavated in these pits.

Borrow Area 3 is located up a ravine about a mile upstream and northwest of the dam. Test pits TP-48 and TP-49 are located here. Both test pits contained residual clayey soils to at least 10 feet in depth. No bedrock was encountered to the 10-foot depth excavated.

Borrow Area 4 is located up a ravine on the east side of the reservoir, about 1/2 mile upstream of the dam. Test pits TP-39 through TP-44 are located in this area. The material consists of mostly residual silt and clay overlying bedrock. The residual soil varied in thickness from 1.5 to 14 feet. The depth to bedrock varied from 3.5 feet to more than 15 feet in depth.

Borrow Area 5 is located northeast of the dam. Previous test pits indicated 0 to 3 feet of sandy clay with occasional cobbles overlying weathered sandstone and claystone. No additional test pits were excavated.

Borrow Area 6 is located on the left abutment of the existing dam. Test pits TP-36 through TP-38 are located in this area. The pits revealed approximately 4 to 9 feet of silty and clay slopewash deposits. The shallower slopewash was underlain by a gravel terrace deposit. No bedrock was encountered to the depths excavated.

It is estimated that the raised embankment will require between 800,000 to 900,000 cubic yards of material. It is normal practice to locate at least twice the estimated quantity of material for preliminary design. Based on the information obtained from the test pit investigation, there should be adequate earthfill and random fill borrow.

7.4.2 Sand and Gravel

A borrow area for sand was identified about 11 miles west of Craig on County Road 15 at the Hard Rock Sand and Gravel commercial borrow pit. A gradation was run on the sand sample, shown in Appendix C, which indicates that it will be filter compatible with the embankment core material. The size of the borrow area is sufficient for the quantities needed for design.

7.4.3 Riprap

A suitable riprap source was located about 5 miles south of Craig at the S&K Construction borrow area. The rock consists of a fine grained basalt. This was the source of riprap used in the original construction. No additional testing was performed on this source. There is sufficient quantity for the proposed dam construction.

8.0

SEEPAGE ANALYSES

8.1 EMBANKMENT SEEPAGE

Seepage has been occurring on the right abutment contact and there is standing water at the toe of the dam. No noticeable seepage has been documented on the downstream face, which is an indication that the embankment is performing well as a barrier to seepage. The lack of piezometer records to date make it hard to draw any further conclusions about the water level in the dam. The readings for Piezometer WC 4, located on the floodplain downstream of the current dam, show a water level at 8 feet below ground surface, which is consistent with the pre-dam geology reports (ECI 1973).

A seepage analysis was conducted to estimate the quantity of flow through the new embankment. Raising the reservoir from elevation 6,365 to 6,406 will increase the hydraulic head and the seepage flow rate. The quantity of flow through the raised embankment was estimated as approximately 10 to 50 gpm, assuming an adequate foundation cutoff is provided. For this analysis, seepage in the alluvium material was assumed to be interrupted by the cutoff trench and so flow was limited to that through the embankment. A permeability of 1 x 10^{-5} cm/s was assumed for the clayey sand/sandy clay embankment materials. A sloping chimney drain having an assumed permeability of 10^{-2} cm/sec was modelled. Due to the uncertainty in these permeabilities, the drainage system should be designed for at least one order of magnitude over the estimate.

The seepage gradient to the old chimney drain due to an increased reservoir head was investigated. This gradient was estimated as approximately 0.4 and is not expected to cause problems with piping.

8.2 HISTORICAL FOUNDATION AND ABUTMENT SEEPAGE

A system of drain pipes and abutment grout curtains were installed in Elkhead Dam during May 1975, when the reservoir was at the spillway crest elevation and seepage was noted in both abutment areas (ECI, 1975). Seepage through the right abutment was estimated at 200 gpm and emerged 10 to 15 feet downstream of the embankment-abutment contact. Seepage on the left abutment appeared as a wet area 25 to 50 feet right of the spillway training wall. The seepage through both abutments was noted to be clear. A system of perforated drain pipes was installed near the seepage exit points on both abutments. The system consisted of trenches with 4-inch perforated drain pipe backfilled with 3/4-inch gravel. The drains were installed to collect all visible seepage.

Three left abutment holes and five right abutment holes were drilled in an attempt to provide an impervious grout cut-off. The drilling was done by a company with no previous grouting experience. The three left abutment holes all took substantial grout. Grouting for the right abutment was done over two days. Two holes were drilled on the first day, both taking substantial grout. The three holes drilled on the following day took little to no grout and it was thought that they were plugged by communication with one of the previously grouted holes. Measurements of seepage flows were not recorded after the program was finished, except to note that they were minor and were stable after the grouting had occurred.

Existing seepage continues to be noted on the lower right abutment of the dam. Lack of evidence of piping indicates that the seepage through the abutment at this time does not have an exit gradient sufficient to cause erosion. To measure this flow, three weirs have been set up on the right abutment, and were measured three times in June 1993. The general location of the weirs is shown in Figure 2 and the flow measurements are given below.

FLOW READINGS AT RIGHT ABUTMENT

Estimated Flow (gpm)

Date	Weir 1	Weir 2	Weir 3
6-17-93	0.28	5.01	0.94
6-23-93	0.28	5.03	0.91
6-29-93	Dry	5.08	1.00

It is likely that the seepage may be occurring along the contact between the embankment and bedrock foundation. A field observation of the right abutment area found a hard, cemented horizon containing lenses and concretions of sandstone at the base of the abutment which could also be routes for flow. Drill fluid losses in the borings ranged from 0 to 100 percent, which indicated some significant permeability in the bedrock in some areas. The 100 percent loss came from WC-3 in the bedrock below the embankment.

Raising the reservoir height another 40 feet will increase the hydraulic head within the right abutment.

The seepage quantity could increase by at least one order of magnitude. Installation of a blanket drain will route this seepage to the collection system, or a positive cutoff could be constructed to reduce this seepage quantity. These options are presented in Section 10.

9.0

EMBANKMENT STABILITY ANALYSES

9.1 GENERAL

Stability analyses were performed for the proposed Elkhead Dam enlargement under end-of-construction, steady state, rapid drawdown, and seismic loading conditions. The material properties used for each of the analyses are described in each section and were chosen based on the results of the recent laboratory testing, SPT blowcounts, and our experience with similar materials. The program "UTEXAS3" was used to perform the stability analyses, using Spencer's Method to compute the theoretical factor of safety for circular shear surfaces. These theoretical factors of safety assist in evaluating the design of the embankment section. Considerable experience-based judgement must be used in applying the results of these analyses because of the simplifying assumptions inherent in the methods and the use of estimated strength parameters which, in themselves, are not amenable to accurate determination.

Figure 9 shows two proposed cross-sections for the dam raise. The upper cross-section, which was presented in the Alternatives Feasibility Study (Hydrosphere 1993) was modelled for the stability analyses. Figure 10 shows the simplified format used for the computer analyses. The new crest was placed at elevation 6,420 with the normal reservoir elevation at 6,406 ft. The side slopes on the dam are 3H:1V upstream and 2.5H:1V downstream. The 3H:1V downstream slope proposed in the 1993 report was considered conservative and was not analyzed. A chimney/blanket drain was modelled under the new crest to interrupt seepage and carry it to the downstream toe. The foundation was modelled as having 10 feet of clayey alluvium overlying 10 feet of sand and gravel alluvium and the bedrock.

9.2 PREVIOUS ANALYSES

Previous analyses were performed on the existing embankment (crest elevation 6,375) by Engineering Consultants, Inc. (ECI 1971, 1972a, 1973). Testing programs were performed in association with each of those reports, and material properties were presented in each report based on the results of the testing programs. The final material properties and strengths used by ECI are given below and were based on the results of 1972 and 1973 laboratory tests. Of these tests, only the results of the 1972 direct shear tests were available and are found in Appendix C.

		Drained Strength	
Material	Max Dry Density (pcf)	(degrees)	c (psf)
Embankment	125	27	200
Clay Foundation	100	18	200
Gravelly Sand Foundation	120	36	0

No undrained parameters were given. A pore pressure factor r_u of 0.15 was used for embankment and foundation soils.

The ECI analyses used the modified Fellenius method and the Morgenstern-Price methods to compute the theoretical factors of safety (ECI 1972a, 1973). Seismic stability analyses were also done, using a horizontal earthquake acceleration of 0.1g, which ECI considered conservative for western Colorado.

9.3 MATERIAL PROPERTIES

The material properties assumed for the analyses included unit weights and shear strength properties. Materials used in the analysis included the existing embankment, the proposed embankment, the existing and proposed drains, the proposed upstream riprap, the upper clayey alluvium foundation, the lower sandy gravel alluvium foundation, and the sandstone/claystone bedrock.

Material properties for the riprap and drain material were based on our experience with similar materials. Material properties for the existing embankment were based on index tests conducted on material from WC-2 and WC-3, the triaxial test results from drill hole WC-2, and our experience with similar embankments. Material properties for the alluvium were based on SPT blowcounts, index properties from samples, and our experience. Material properties for the new embankment were based on results of index testing on the borrow pit material, the two triaxial tests from Test Pits 34 and 42, and our judgement. The strength for the bedrock material was based on the results of the unconfined compression tests and our experience with similar materials.

Material strengths for the analyses included those for both drained (effective stress) and undrained (total stress) conditions. Due to the permeable nature of the drain material and the sand and gravel alluvium, these materials were assumed to exhibit their drained strengths under all the loading conditions used. The embankment fill, both new and existing, and the clayey alluvium were given undrained strength parameters for loading conditions where excess pore pressures were expected to exist in these materials. The following parameters were used:

Material	Saturated Unit Wt (pcf)	Drained Strength (degrees)	c (psf)	Undrained Strength (degrees)	c (psf)
Existing Embankment	130	33	0	24	0
New Upstream Riprap	145	45	0	--	--
Drain/Filter Material	123	34	0	--	--
New Embankment (Raise)	128	26	250	15	600
Clay Foundation	118	18	200	14	600
Gravel and Sand Foundation	127	37	0	--	--
Bedrock	140	40	2000	--	--

9.4 ANALYSES

Analyses were conducted on the raised dam for the following cases:

(1) Upstream Slope, Steady State Seepage Condition

(2) Downstream Slope, Steady State Seepage Condition

(3) Downstream Slope, End of Construction Condition

(4) Upstream Slope, Rapid Drawdown Condition

(5) Upstream Slope, Pseudo-Static Condition

(6) Downstream Slope, Pseudo-Static Condition

The steady state seepage condition represents a period when the excess pore pressure generated during construction has dissipated in the embankment and foundation. The reservoir is at normal pool elevation 6,406 feet and the phreatic surface within the embankment has fully developed. This loading condition represents the working condition of the dam. Drained parameters were used for the analyses on the upstream and downstream slopes. An automated search for circular arcs with the lowest theoretical factor of safety was performed. The minimum factor of safety was 2.0 for the upstream slope and 1.5 for the downstream slope. The critical circles are shown in Figure 10.

The end of construction case was performed on the downstream slope only. This loading condition represents the period following construction of the dam when dissipation of excess pore pressures have not occurred in the less permeable materials. For this case, the undrained material strengths were used in the embankment materials and clayey alluvium. In low stress situations where the drained strengths were actually weaker than the undrained strengths, the lesser of the two strengths was used. A phreatic surface was input in the analysis at the foundation level. An automated search was also used in these analyses. The minimum theoretical factor of safety computed was 1.2.

The rapid drawdown case was performed for the upstream slope. This case models the maiximum reservoir drawdown, when it can be performed at a rate fast enough to cause undrained behavior of the embankment. The Alternatives Feasibility Study (1993) stated that the outlet works was designed to meet the minimum criteria set by the State Engineer of a drawdown of 5 feet of head within 5 days time. Given this information, a rapid drawdown analysis was performed with the reservoir dropping from 6406 to elevation 6401. The phreatic surface in the dam was kept at its initial, normal reservoir level. The analysis was performed using the procedure recommended in the UTEXAS3 manual. A circular search resulted in a failure surface with a minimum factor of safety of 1.9. The original drawdown analysis by ECI (1973) assumed a 35-foot drawdown in 90 days, with a minimum factor of safety of 1.4. For comparison, a 35-foot drawdown was also modelled on the raised embankment. The critical failure surface had a minimum factor of safety of 1.2. These surfaces are shown on Figure 11.

The seismic stability condition was previously performed (ECI 1972a) using a pseudostatic analyses and entering a horizontal gravity acceleration on the dam. This investigation also performed a pseudostatic analysis and examined the liquefaction potential of the dam and foundation materials.

According to the State of Colorado (1988), the pseudo-static load coefficient should be one-half of the predicted peak bedrock acceleration (gs). For this pseudostatic analysis, a horizontal acceleration of 0.1g was applied to the dam since the peak horizontal bedrock acceleration was estimated to be 0.2g. The reservoir was assumed to be at elevation 6,406 feet. Material strengths used were the same as those used for the steady state cases. A circular search was used to locate the critical circle for the steady state upstream and downstream cases. The minimum factors of safety computed were 1.1 and 1.0 for the upstream and downstream slopes, respectively.

The potential for liquefaction of the Elkhead Dam embankment and/or foundation soils during design earthquake loading was evaluated using the general approach proposed by Seed and Idriss (1982). The soils in the embankment and foundation were categorized according to their material classification. Clayey soils with water contents less than 90 percent of their liquid limits are considered nonliquefiable. Following this criteria and using the results of laboratory testing presented in Appendix C, the sandy clay and clayey sand embankment material and the sandy clay alluvium were identified as not likely to liquefy.

The potential for liquefaction of gravelly soils has been discussed recently in the geotechnical literature (Sykora 1990; Crova 1992), but has historically not been observed. The information available from these and other articles is not sufficient, however, to make an evaluation of the liquefaction potential of the sand and gravel alluvium under the dam. Reclamation's Design Standards (1989) for seismic analysis recommend that if the shear wave velocities are in excess of 1,200 ft/s, the deposit may be considered nonliquefiable. Most of the seismic traverse results have shear wave velocities meeting this criteria. We generally do not feel liquefaction is an issue for the gravel alluvium at this site.

9.5 CONCLUSIONS

Results of the stability analyses are shown below. It can be seen that all cases satisfy the State of Colorado (1988) minimum required factor of safety.

Condition	Existing Dam Factor of Safety (ECI 1973)	Proposed Raise Factor of Safety (WCC 1993)	Required Safety Factor State of Colorado (1988)
Downstream Steady State	1.6	1.5	1.5
Upstream Steady State	--	2.0	1.5
Downstream	1.4	1.2	(1)
End-of-Construction			
Rapid Drawdown	1.4	1.9	1.2
Downstream Seismic	1.5	1.0	1.0
Upstream Seismic	1.2	1.1	1.0

(1) No minimum given in current regulations.

10.0

EMBANKMENT DESIGN AND CONSTRUCTION RECOMMENDATIONS

10.1 FOUNDATION TREATMENT

10.1.1 Excavation of Foundation Soils

Excavation of portions of the soils will be required to reduce settlement, seepage, and to provide a firm foundation for fill placement. Foundation preparation will involve removal of soil, slopewash, colluvium, and weathered bedrock on the abutments and portions of the alluvium in the bottom of the valley.

The right abutment contains a thin (1 to 5 foot thick) cover of colluvium and slopewash on the relatively steep slope just downstream of the existing embankment. The top of the abutment has a more gentle slope and surficial materials have already been stripped for the road cut or possible existing borrow source.

The left abutment has a relatively thick (10 to 15 feet thick) cover of clayey soil near its crest and a thin layer of slopewash along the contact area which should be removed. The clayey soil could be suitable borrow material for portions of the embankment.

Borings indicate the alluvium filling the bottom of the valley is 10 to 20 feet thick. The upper clayey alluvium would need to be removed while the dense sand and gravel alluvium could be left beneath the embankment.

Foundation preparation in areas of weathered bedrock should involve removal of 1 to 2 feet of the top of bedrock, to shape the abutment and foundation bedrock surfaces.

10.1.2 Construction Dewatering

Handling of surface and groundwater during construction will require diverting Elkhead Creek upstream of the existing embankment and dealing with seepage in excavations.

Since the reservoir will not be drained during construction, a cofferdam and temporary diversion may be necessary to isolate the existing outlet works and replace it with a new outlet works, possibly at a lower elevation. The cofferdam could be constructed with sheet piling or an earth embankment.

The foundation excavation will require dewatering during construction in the lower portions adjacent to the existing stream channel, due to seepage from the alluvium and bedrock. Based on the conditions encountered during our investigation, dewatering may be accomplished by use of temporary interceptor trenches at the upstream and downstream edges of the excavation. Intercepted seepage can then be removed by pumping.

10.1.3 Abutment and Foundation Cutoff

Several options have been considered for reducing foundation underseepage for the embankment raise. The selection of an option will depend on the dam raise alternative chosen and cost. In dam raise Alternative 1, the cutoff trench at the existing dam centerline would be extended up the abutments beneath the new core, and deeper underseepage control options would be implemented. In Alternative 2, a new cutoff would be constructed beneath the dam raise, incorporating a cutoff trench and deeper underseepage control options. Both alternatives are shown on Figure 9.

Historically, Elkhead Dam has had seepage from the abutments, as described in Section 8. The benefits of remedial grouting to reduce underseepage at this site are uncertain, based on the information available. This has been considered in our underseepage evaluation. Although the abutment seepage is the primary concern, underseepage reduction options described below also treat the valley floor foundation beneath the main section of the embankment. Detailed seepage analyses performed during preliminary designs should address whether or not valley floor foundation treatment is desirable from an economic standpoint.

For dam raise Alternative 1, underseepage control would be accomplished by the existing and extended cutoff trench underlain by either a grout curtain or a cement-bentonite slurry diaphragm wall. Either method would entail excavating or drilling from the crest of the existing dam into unweathered bedrock. The cutoff would extend up both abutments to a level above the normal water level.

The diaphragm wall beneath the cutoff trench would have a lower risk for seepage loss, but would have to extend through the existing dam to encounter the foundation. Therefore, the constructed wall within the embankment would be extra cost, in terms of underseepage control. The existing dam crest could be excavated to a lower excavation to reduce the height of the diaphragm wall, but this would require additional earthwork.

In our opinion, the more practical solution for Alternative 1 is the grout curtain beneath the cutoff trench. The grout curtain could be constructed by drilling and grouting, in stages, a series of holes from the old dam crest extending up both abutments. Grouting should decrease the permeability of the treated area and reduce seepage. This method is not as positive a cutoff as a diaphragm wall, but probably is better suited to the geometry of this design. The preliminary design should address this issue in more detail. For initial estimation purposes, a single curtain is recommended, with grout holes extended about 20 feet into unweathered bedrock beneath the dam and up to about 100 feet into bedrock in the abutments. These same depths should be considered for a diaphragm wall.

Reduction of underseepage for Alternative 2 consists of excavating a new key trench about 5 feet into bedrock, and construction a cement-bentonite slurry diaphragm wall in the bedrock. Unlike Alternative 1, the diaphragm wall could be constructed from a prepared foundation surface at the base of the cutoff trench. It would not be excavated through the existing embankment, which makes the wall a better option for this alternative.

The diaphragm wall would be constructed by excavating a 2- to 3- foot wide trench in bedrock under a bentonite slurry. The slurry is used to help stabilize the trench sides. The trench would then be backfilled with plastic concrete to displace the slurry and form the wall. The wall should provide an almost "impermeable" barrier with typical permeabilities of less than 0.1 foot per year. For estimation purposes, the wall should extend 20 feet into unweathered bedrock under the dam and up to about 100 feet into bedrock on the abutments. As a less expensive, but also less positive option, a grout curtain could also be constructed for this alternative. Both should be further studied in order to understand the relative degree of economic savings and assumed risk of underseepage.

10.2 EMBANKMENT GEOMETRY

Engineering analyses for the proposed dam were performed to evaluate suitable dam sections for the site conditions and available on-site construction materials. Two alternative sections are proposed, based on reuse of the existing drain system or abandoning the existing drain and establishing a new core. The two alternatives are shown in Figure 9. Both of the recommended embankments are zoned, rolled-fill embankments with a 25-foot wide crest, a 3:1 or flatter upstream slope, and 2.5 or flatter downstream slope.

Alternative 1 has a core raised from the existing dam and forming the upstream face of the raised dam. The downstream shell would consist of random material available from the borrow areas and required excavation. A blanket drain would be placed between the core and shell which would discharge at the toe of the downstream slope.

Alternative 2 involves removal of a portion of the existing dam toe and drain system, constructing a new core, and then a new chimney drain downstream of the core. Upstream and downstream shells of random material are shown. The chimney drain would connect to a blanket drain that would discharge at the toe of the downstream slope.

10.3 DRAINS

An 8-foot wide internal chimney drain and a 4- to 5-foot thick horizontal blanket drain should be provided to prevent internal piping and erosion of the downstream slope and to control the phreatic surface in the embankment.

Material used for construction of the chimney drain and drainage blanket should be designed to meet the filter and drainage requirements. These materials should be placed in near-horizontal lifts and compacted with vibratory compaction equipment.

10.4 CAMBER AND SETTLEMENT

A camber should be provided on the crest of the embankment so that the crest elevation will not be reduced by foundation and embankment settlement. The original dam design called for a camber of up to 2 feet (elevation 6,377) along the central portion of the dam crest. It appears that settlement of about 1.5 feet has occurred based on a level survey conducted in October 1993. The new fill will cause settlement of the old embankment and the foundation. There will also be settlement of the new embankment due to self weight. This settlement is not expected to exceed about 2 percent of the embankment height. It is expected that a major portion of this consolidation will occur during construction before the embankment is completed. A camber of two feet at the center line of the embankment tapering to zero at the abutments is recommended.

10.5 UPSTREAM SLOPE PROTECTION

The upstream slope of the embankment must be protected against rain and snowmelt runoff and wave action. The most common form of protection is riprap. The material should be a well-graded mixture of rock placed by end-dumping on the slope. A bedding layer beneath the riprap is required to act as a filter to keep wave action from leaching out the embankment fill.

The rock for the riprap should be high quality rock capable of withstanding the environmental and wave conditions. The rock should have a specific gravity of at least 2.55 (ASTM C-127), a weight loss of 10 percent or less under the magnesium or sodium sulfate soundness test (ASTM C-888), a weight loss of 40 percent or less when tested for resistance to abrasion (ASTM C-535), and a weight loss of 5 percent or less when tested for resistance to disintegration by freeze-thaw testing (AASHTO T-103).

The riprap should extend from 5 feet above the high water line to at least 5 feet below the low pool elevation. Below this elevation, the slopes should be protected by less expensive riprap or an earth berm to protect against waves of the first filling or abnormally low pools. The riprap thickness should be sized based on wave action. The riprap should be processed to eliminate excessive fines. The riprap should be placed on a 12-inch thick bedding to prevent piping of the embankment fines through the riprap layer. As an alternative, the bedding may consist of a geotextile fabric covered with 12 inches of sand.

10.6 DOWNSTREAM SLOPE PROTECTION

The downstream slope of the embankment should be protected against rain and snowmelt runoff and wind erosion. In general, grass turf is a desirable means of protection. This is obtained by seeding a layer of fertilized topsoil. Vegetation which conceals the outer surface from ready visual inspection should not be used. Trees and shrubs should not be permitted. With a downstream slope of 2.5 horizontal to 1 vertical, it may not be practical to mow the slope, so a dry grass should be used.

10.7 EMBANKMENT FILL PLACEMENT

The placement of embankment soil should be monitored by continuous observation and testing. Fills should only be placed on approved subgrade, and earthwork should meet project specifications. Fill should not be placed on a frozen foundation, a problem which occurred during the original construction of the dam and necessitated removal and replacement of that portion of the fill and foundation. Materials should be free from segregated zones. The materials should be placed in 8 to 12-inch lifts and compacted to high density. The materials should be placed at moisture contents between -1 percent and +3 percent of optimum. Material properties and in-place compaction should be tested prior to acceptance.

11.0

SPILLWAYS AND OUTLET WORKS RECOMMENDATIONS

11.1 GENERAL

Appurtenant structures for the dam include a service spillway, an emergency spillway, an outlet works, and a terminal building. The outlet works will be a buried conduit. The service and emergency spillways will be constructed as open cuts.

11.2 SPILLWAYS

Due to the hydrologic conditions of the project, it is anticipated that the emergency spillway will be used infrequently.

The emergency spillway will have an approximate 250-foot bottom width and will require an excavation approximately 10 to 20 feet deep. The base of the spillway will extend into the underlying sandstone bedrock. The side slopes of the spillway will be excavated through sandy clays and clayey sands. It is anticipated that slopes and base of the spillway channel will be grass lined.

The excavated slopes for the emergency spillway channel should be a maximum of 2H:1V or flatter in both the soils and bedrock. This slope is required for stability and to allow topsoil and seeding of the spillway.

As part of the foundation preparation, a service spillway will be excavated into the bedrock on the left abutment. The bedrock in this area should be easy to moderately difficult to rip; however, at least 1 or 2 cemented sandstone lenses should be expected in the excavation. Cut slopes in the bedrock are expected to be stable on 1:1 slopes. Cut slopes in the soils and slopewash are expected to be stable on 2H:1V slopes.

The spillway at the proposed location shown on Figure 2 will be built in embankment fill. Locating the spillway approximately 750 feet to the east on the left abutment will position the spillway in bedrock, which is more desirable.

11.3 OUTLET WORKS

The new outlet works will consist of an inlet tower, a 6-foot diameter conduit running beneath the embankment, and an optional hydroelectric power generating facility near the toe of the dam. These would be located immediately to the west of the new spillway channel. The conduit will be about 500 feet long and should be placed in a trench excavated into bedrock beneath the overburden soils. Design of the conduit should consider structural loading and seepage along the exterior. As a minimum, partial or full encasement with concrete should be considered in the section below the core of the embankment. The remaining length of the conduit could also be encased or backfilled with material meeting the requirements for embankment core material. The excavation for the conduit should be such as to permit maximum use of machine compacting equipment and hand tamping of the backfill should be kept to a minimum.

The inlet and outlet structures should be founded with a mat foundation on bedrock designed for a maximum bearing pressure of 4,000 psf.

12.0

INSPECTION AND MONITORING

12.1 GENERAL

Inspection and monitoring of the dam continues long after the construction is finished and are part of the normal maintenance of a dam. Problems with the dam may develop over time or not become apparent until certain reservoir levels are reached. A regular schedule of visual inspections and the monitoring are crucial in detecting early signs of problems and correcting them before they can adversely affect performance. We recommend the following inspection schedule and the following instrumentation. These recommendations should be incorporated into the Operations and Maintenance (O&M) manual for the dam.

12.2 VISUAL INSPECTION

A visual inspection of the dam should be performed weekly for the first six months after construction. The State of Colorado (1988) requires that Class I and Class II dams be observed at least twice a month when the reservoir water level is greater than half the full storage capacity. Inspections should also be done following any major storm or seismic event. Visual inspections should be done in accordance with methods acceptable to the State Engineer.

12.3 INSTRUMENTATION

12.3.1 Piezometers

We recommend that at least five open standpipe piezometers be installed to monitor the changes in phreatic surface within the dam. Two piezometers should be placed along the crest of the dam. They should be protected from crest traffic and any regrading of the crest road. Another two piezometers should be placed at each abutment. A piezometer should also be placed at the toe of the dam. Additional piezometers may be needed, depending on the final dam design. The piezometers in the existing dam will likely be destroyed during construction. During construction, they should be grouted and abandoned.

During initial refilling, the water levels in the piezometers should be read at every 5-foot change in the reservoir level below elevation 6,365 feet, or at least once every four weeks. Piezometers should then be read weekly until the reservoir reaches its new high water level. Subsequent readings should be taken monthly during the first year of operation and quarterly thereafter. The data should be reviewed by a qualified professional familiar with the design and construction of the dam.

12.3.2 Movement Monuments

WCC recommends that ten movement monuments be installed in 2 lines along the crest and midway down the downstream slope. After their initial location is known, these points are subsequently resurveyed to detect vertical or lateral displacements in the dam. The monuments should be surveyed quarterly during the first year of operation and annually during the next four years. Subsequent surveys should be conducted as needed. The results should be reviewed by a qualified professional familiar with the design and construction of the dam.

12.3.3 Weirs

WCC suggests that a weir be installed along the downstream toe ditch, if feasible. The weir will allow measurement seepage exiting the drains. Locating the weir may prove difficult given the current design of the toe ditch. As the design is finalized, consideration should be given to installation of this valuable monitoring tool.

12.3.4 Inclinometers

WCC recommends that at least 2 inclinometer casings should be installed in the crest of the dam to monitor potential movements in the embankment. The inclinometer casings can be installed in conjunction with the open well piezometers.

13.0

LIMITATIONS

Professional judgments on subsurface conditions are presented in this report. They are based partially on the evaluation of the technical information gathered from this investigation, partially on the evaluation of the technical information gathered by other investigators, partially on our understanding of the characteristics of the dam, and partially on our experience with subsurface conditions in the surrounding area. We do not guarantee the performance of the project in any respect, only that our engineering work and judgments rendered meet the standard of care of our profession.

Test holes drilled and test pits excavated for this report were spaced to obtain a reasonable understanding of subsurface conditions for the intended purposes of this report. Variations from the conditions portrayed, which are not indicated by the test explorations, may occur. Judgments made should consider this potential variability.

Dam raises involve the use of materials and procedures which by their nature vary so much that it is not possible to cover all eventualities in design. This necessitates that competent and experienced personnel are used for guidance in decisions during construction. We recommend construction observations during the raise by trained and experienced personnel to take advantage of any opportunities to recognize differing conditions, thus minimizing the risk of having undetected conditions which may adversely affect the raise of Elkhead Dam.

14.0

CREDITS

The analyses and designs presented in this report were made under the supervision of Mr. Richard J. Tocher, Project Manager and Mr. Daniel Johnson, Peer Reviewer. Mr. Richard Romano was the WCC representative during the field investigation conducted by the U.S. Bureau of Reclamation. Ms. Jessica Marshall performed the analysis and drafted this report.

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Project Manager

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