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Appendix A
Dam Safety Evaluation

Appendix A, Dam Safety Evaluation, includes the following information:

- **Memorandum for Record. Chatfield Reservoir Reallocation Study FR/EIS, Denver, Colorado**. This memorandum signed by the Chief of Engineering states that there is no impact to the primary flood risk management purpose of Chatfield Reservoir nor is there a change to the system flood control storage evacuation releases during a Level II flood as defined in the FR/EIS.
- **Water Supply Re-Allocation Study Dam Safety Evaluation Chatfield Dam, Littleton, CO**. This report is a geotechnical / structural dam safety evaluation of Chatfield Dam based on a potential permanent increase in the normal reservoir elevation by up to 12 feet.
- **Post-Liquefaction Stability Analyses.** This report presents the results of stability analyses performed on zones of the Chatfield Dam foundation that have been identified as susceptible to liquefaction.

CENWO-ED-GB

23 May 2013

MEMORANDUM FOR RECORD

SUBJECT: Chatfield Reservoir Reallocation Study FR/EIS, Denver, Colorado

1. Reference ER 1110-2-1156, January 2013, "Safety of Dams – Policy and Procedure" and NWDR 1110-1-3, 31 March 2003, "Modifications at Existing Corps-Owned Civil Works Projects."

2. As part of the Chatfield Reservoir Reallocation Study FR/EIS, an analysis of the effect on geotechnical, seismic, and hydrologic conditions due to reallocating 20,600 acre-feet of flood control storage to joint-use flood control and water supply storage at Chatfield Reservoir has been completed.

3. Reallocation would not impact the primary flood risk management purpose of Chatfield reservoir. During Tri-Lakes system flood control storage evacuation for Level I (small flood events), as defined in the Chatfield Reservoir Reallocation Study FR/EIS Appendix B – Tri-Lakes Water Control Plans, the reallocation of flood control storage at Chatfield slightly increases releases and affects the timing and duration of releases made from Cherry Creek and Bear Creek though the primary flood risk management purpose for Cherry Creek and Bear Creek is not affected. There is no change to system flood control storage evacuation releases during Level II (large flood events), as defined in the Chatfield Reservoir Reallocation Study FR/EIS Appendix B – Tri-Lakes Water Control Plans. Omaha District believes there are no dam safety issues that would prevent reallocation of 20,600 acre-feet of the Chatfield flood control pool.

JOHN J. BERTINO, JR., P.E.
Chief, Engineering Division

Water Supply Re-Allocation Study
Dam Safety Evaluation
Chatfield Dam
Littleton, Colorado

US Army Corps of Engineers®

Water Supply Re-Allocation Study
Dam Safety Evaluation
Chatfield Dam - Littleton, Colorado

Water Supply Re-Allocation Study
Chatfield Dam
Geotechnical / Structural Dam Safety Evaluation

EXECUTIVE SUMMARY

Presented herein is a geotechnical / structural dam safety evaluation of Chatfield Dam based on a potential permanent increase in the normal reservoir elevation by up to 12 feet. It is emphasized that this evaluation is based strictly on static loading scenarios and does not address seismic loading. It is vital to address various aspects of design and performance to assure that the proposed modifications do not impact the continued safe operation of the dam and do not pose dam safety concerns. The methodology utilized in this evaluation was to review design assumptions, evaluate instrument data compared to the design assumptions and evaluate historic performance of the project.

The requirement for a Phase I Seismic Study has been identified as a result of a Seismic Safety Review (SSR) of Chatfield Dam. A brief status of the seismic assessment is presented herein; however, seismic loading under new Corps of Engineers criteria (Phase I Study) has not been addressed in this report. The findings in this report are based strictly on normal static loading criteria.

Based strictly on a static evaluation of the project, no conditions have been identified that would prohibit adoption of the Re-Allocation Project. Installation of new piezometers located in the downstream fill, overburden and blanket drain are recommended as additional monitoring devices to assure continued safe operation of the project.

Although no dam safety concerns have been identified for the proposed reservoir loading, based on project performance and the instrumentation program, increased monitoring of the project will be required as part of the routine dam safety program to assure continued safe operation of the dam. This will include the development and implementation of a reservoir raise monitoring plan which would include additional inspection effort, instrumentation data acquisition and data analysis. The Project Surveillance Plan and Emergency Action Plan must also be updated as appropriate.

The analysis presented herein is based on a review of design assumptions, an evaluation of instrument data compared to the design assumptions and an evaluation of historic performance of the project. Any future dam safety concerns (seepage, slope stability, etc.) that may develop during/following the actual reservoir raise may have a direct bearing on the continued long term use of the re-located storage.

Water Supply Re-Allocation Study
Chatfield Dam
Geotechnical / Structural Dam Safety Evaluation

Table of Contents

Table of Contents i

Paragraph **Page No.**

1. General 1
2. Purpose and Scope 1
3. General Project Description 1
4. Construction History 8
5. Seismic Evaluation 9
6. Slope Protection 10
7. Slope Stability 11
8. Seepage Control 19
9. Movement Review 25
10. Structural Evaluation 27
11. Conclusions 30
12. Recommendations 31

References
Acronyms

Appendix A – General Project Plates

Plate No.	Title
A-1	Vicinity and Location Maps
A-2	Project Plan
A-3	Embankment Grading Plan – Sheet 1
A-4	Embankment Grading Plan – Sheet 2
A-5	Embankment Grading Plan – Sheet 3
A-6	Embankment Grading Plan – Sheet 4
A-7	Embankment Centerline Profile

Appendix B – Stability Anaylsis

Plate No.	Title
B-1	Stability Anaylsis, End of Construction, Sta. 95+00
B-2	Stability Anaylsis, Partical Pool Case, Sta. 95+00
B-3	Stability Anaylsis, Sudden Drawdown Case, Sta. 95+00
B-4	Stability Anaylsis, Steady Seepage Case, Sta. 95+00

B-5	Stability Anaylsis, Part. Pool, Steady Seepage Case, Sta. 104+35
B-6	Stability Anaylsis, Steady Seepage Case, Sta. 68+50
B-7	Summaries of Direct Shear Tests on Soft Seam

Appendix C – Piezometer Data

Plate No.	Title
C-1	Piezometer Location Plan
C-2	Piezometer Observations, Sta. 60+00
C-3	Piezometer Observations, Sta. 68+90
C-4	Piezometer Observations, Sta. 79+00
C-5	Piezometer Observations, Sta. 81+20
C-6	Piezometer Observations, Sta. 82+00 to 88+00
C-7	Piezometer Observations, Sta. 93+00
C-8	Piezometer Observations, Sta. 99+00 to 102+00
C-9	Piezometer Observations, Sta. 102+54
C-10	Piezometer Observations, Sta. 108+65
C-11	Foundation Piezometer Observations, Station 68+80-68+90
C-12	Foundation Piezometer Observations, Station 81+10-81+20
C-13	Foundation Piezometer Observations, Station 93+00
C-14	Foundation Piezometer Observations, Station 101+35-102+00
C-15	Piezometer 502B, Sta. 68+90
C-16	Piezometer 507C, Sta. 81+20
C-17	Piezometer 95-03, Sta. 81+20
C-18	Piezometer 83+00/CTR, Sta. 83+00
C-19	Piezometer 102+00/CTR, Sta. 102+00
C-20	Piezometer 519C, Sta. 102+54
C-21	Piezometer 536, Sta. 68+85
C-22	Piezometer 41, Sta. 68+90
C-23	Piezometer 504A, Sta. 68+90
C-23A	Piezometer 504A, Sta. 68+90
C-24	Piezometer 505A, Sta. 68+90
C-25	Piezometer 505B, Sta. 68+90
C-25A	Piezometer 505B, Sta. 68+90
C-26	Piezometer 560, Sta. 68+90
C-27	Piezometer 79+00, Sta. 79+00
C-28	Piezometer 561 Sta. 93+00
C-28A	Piezometer 561 Sta. 93+00
C-29	Piezometer 102+00/25US, Sta. 102+54
C-30	Piezometer 486, Sta. 102+54
C-31	Piezometer 520A, Sta. 102+54
C-32	Piezometer 522, Sta. 102+54
C-33	Piezometer 558, Sta. 102+54
C-34	Piezometer 555, Sta. 85+50
C-35	Piezometer 563, Sta. 99+00

Appendix D – Survey/Inclinometer Data

Plate No.	Title
D-1	Crest Movement Markers, Vertical Movement
D-2	Slope Movement Markers, Vertical Movement
D-3	Toe Movement Markers, Vertical Movement
D-4	Crest, Slope & Toe Movement Markers, Vert. Movement vs. Time
D-5	Crest, Slope & Toe Movement Markers, Hor. Movement vs. Time
D-6	Horizontal Movement vs. Time, Crest Markers C-9 thru C-16
D-7	Horizontal Movement vs. Time, Crest Markers C-17 thru C-24
D-8	Horizontal Movement vs. Time, Slope Markers
D-9	Horizontal Movement vs. Time, Toe Markers
D-10	Settlement Gages, Sta. 70+00, Vertical Movement
D-11	Settlement Gages, Sta. 90+00, Vertical Movement
D-12	Centerline Top of Dam Elevation
D-13	Intake Structure Plumbline Tilt Observations
D-14	Intake Bridge Piers & Tower, Vertical Movement
D-15	Intake Bridge Deck & Tower, Vertical Movement
D-16	Wet Conduit, Vertical Movement Profile
D-17	Inclinometer Location Plan
D-18	Inclinometer 497, Cumulative Displacement 2002-Present
D-19	Inclinometer 472, Cumulative Displacement 2002-Present
D-20	Inclinometer 488, Cumulative Displacement 2002-Present
D-21	Inclinometer 490, Cumulative Displacement 1997-Present
D-22	Inclinometer 495, Cumulative Displacement 2002-Present
D-23	Inclinometer 496, Cumulative Displacement 2002-Present
D-23A	Inclinometer 496, Cumulative Deviation
D-24	Inclinometer 523, Cumulative Displacement 2002-Present
D-25	Inclinometer 524, Cumulative Displacement 2006-Present
D-26	Inclinometer 525, Cumulative Displacement 2002-2004
D-27	Inclinometer 526, Cumulative Displacement 2006-Present
D-28	Inclinometer 544, Cumulative Displacement 2002-Present
D-29	Inclinometer 545, Cumulative Displacement 2002-Present
D-30	Inclinometer 545, Cumulative Deviation
D-31	Inclinometer 545, CheckSums 2003-2010

Appendix E – Intake Structural Stability Analysis

Plate No.	Title
E-1	Intake Tower Structural Analyses

Appendix F – Project Management Plan (PMP)

Water Supply Re-Allocation Study
Chatfield Dam
Geotechnical / Structural Dam Safety Evaluation

1.0 General. It has been proposed to raise the normal elevation of the Chatfield Reservoir by up to 12 feet, from El. 5432 feet to El. 5444 feet, for the purpose of water supply storage. The Water Re-allocation Study will actually evaluate three options, (1) no raise, (2) a five foot raise and (3) a twelve foot raise. The final adopted plan will not increase the maximum surcharge reservoir elevation. The historic maximum reservoir elevation at Chatfield Dam is 5447.58 feet. All elevations referenced hereinafter are NGVD 1929 Datum.

2.0 Purpose and Scope. The purpose of this report is to evaluate potential dam safety concerns based on a permanent increase in the reservoir elevation. The evaluation is based strictly on static loading; however, historic information on previous seismic evaluations will be presented. It is vital to address various aspects of design and performance to assure that the proposed modifications do not impact the continued safe operation of the dam and do not pose dam safety concerns. This evaluation will address the following areas of interest.

- ✓ Seismic
- ✓ Slope Protection
- ✓ Slope Stability
- ✓ Seepage
- ✓ Movement
- ✓ Structural

3.0 General Project Description. The Chatfield Dam and Reservoir Project is composed of a rolled earthfill dam, an ungated concrete spillway and stilling basin, an outlet works intake structure, two-barrel conduit and a stilling basin. A small flood detention dam, the Spring Gulch embankment, is located at the extreme right abutment of the main dam. General project drawings of the Chatfield Project are presented in Appendix A for information. Plate Nos. A-1 and A-2 present a general location plan and project plan. A list of pertinent data is as follows.

PERTINENT DATA

DRAINAGE AREA

Entire South Platte River Basin	24,030 square miles
Total above Chatfield Dam Site	3,018 square miles

RESERVOIR DATA (1991 Data)

	Elevation (feet m.s.l.)	Gross Storage (Acre-Feet)	Surface Area (Acres)
		Initial	
Maximum Surcharge Pool (Spillway Design Flood)	5521.6*	351,366	5,977
Flood Control Pool (Spillway Crest)	5500.0*	235,098	4,770
Multipurpose Pool	5432.0*	28,369	1,423
Streambed	5380.0*	0	0

* NGVD 1929

ELEVATIONS

Top of Dam*	5527.0 feet m.s.l.
Maximum Surcharge Pool (Spillway Design Flood)*	5521.6 feet m.s.l.
Spillway Crest*	5500.0 feet m.s.l.
Top of Multipurpose Pool* (Prior to Re-Allocation)	5432.0 feet m.s.l.
Top of Sediment Pool*	5426.0 feet m.s.l.

* NGVD 1929

DAM

Type	Rolled Earthfill
Maximum Height Above Riverbed	147 feet
Height Above Valley Floor	137 feet
Crest Length	13,136 feet
Fill Volume	17,255,100 cubic yards

OUTLET WORKS

Intake, Type	Tower with Access Bridge
Service Gates	
Number	2
Type	Hydraulically-Operated Slide
Size	5.5 feet X 13 feet
Emergency Gates	
Number	1
Type	Wheel-Wire Rope Hoist
Size	5.5 feet X 14.25 feet
Low-Flow Releases	
Gate Type	Gate-within-a-Gate
Size	2 feet X 2 feet
Auxiliary Conduit	
Size	72-inch diameter

Upstream Gate, Type	Butterfly
Number	1
Size	72-inch
Downstream Gate, Type	Butterfly
Number	2
Size	60-inch and 48-inch diameter
Bulkhead	
Number	1
Size	8 feet X 19.5 feet
Conduit Type	Twin Oblong
Maximum Height (inside dimension)	16.0 feet, each barrel
Maximum Width (inside dimension)	11.0 feet, each barrel
Length	756 feet
Invert Elevation, at Intake*	5385.0 feet m.s.l.
Invert Elevation, at Outlet*	5375.0 feet m.s.l.
Discharge Capacity (@ Elev. 5500.0)	8,300 cubic feet per second
Stilling Basin, type	Conventional Hydraulic Jump
Width	37 feet
Length	91 feet
Floor Elevation*	5358.0 feet m.s.l.
End Sill Elevation*	5361.0 feet m.s.l.

* NGVD 1929

SPILLWAY

Type	Ungated Chute
Bottom Width	390 feet
Crest Type	Ogee Weir
Crest Elevation*	5500.0 feet m.s.l.
Crest Length	500 feet
Stilling Basin	
Type	Conventional Hydraulic Jump
Width	390 feet
Length	154 feet
Floor Elevation*	5352.0 feet m.s.l.
End Sill Elevation*	5357.0 feet m.s.l.
Maximum Discharge	188,000 cfs @ Elev. 5521.6 feet m.s.l.*

* NGVD 1929

DOWNSTREAM CHANNEL

Capacity, maximum	5,000 c.f.s.
Width, minimum	100 feet
Length (approximate)	8 miles

WATER RIGHTS (Capacity)

	Total Decreed	**D/S Requirements**
Last Chance Ditch	43 c.f.s.	12 c.f.s.
Nevada Ditch	32 c.f.s.	32 c.f.s.
Denver Water Board		
City Ditch	86 c.f.s.	50 c.f.s.
Plum Creek Pump Station	34 c.f.s.	30 c.f.s.
Fish Hatchery	16 c.f.s.	16 c.f.s.

3.1 Embankment. Chatfield Dam is a rolled, zoned earthfill with a crest length of 13,136 feet, and a crest width of 30 feet. The maximum height of the embankment is 137 feet across the valley and 147 feet where it crosses the South Platte River. The embankment zoning consists of a symmetrical central impervious core with 1V on 3H side slopes; upstream and downstream random material shells; and a pervious inclined sand drain with continuous outlets adjacent to the downstream slope of the impervious core. The thickness of the pervious inclined drain is 20 feet in the valley sections and transitions to a 10 foot thickness in the abutments. An impervious cutoff trench excavated to bedrock through the pervious overburden materials joins the embankment core to provide a positive underseepage control. The outer portion of the downstream random zone includes a zone specifically for all Dawson Formation materials, which were excavated from the spillway, and outlet works excavations. The entire upstream face of the dam is protected with graded riprap. The grades (slopes) of the upstream and downstream slopes of the embankment are presented in Table Nos. 1 and 2.

Table No. 1 – Upstream Slope

Slopes	**Elevations***
1V on 2.5H	5527.75 to 5493
1V on 5H	5493 to 5431
1V on 15H	5431 to 5420
1V on 3H	5420 to 5408
1V on 10H	5408 to Ground Surface

* NGVD 1929

Table No. 2 – Downstream Slope

Slopes	**Elevations***
1V on 2.5H	5527.75 to 5493
1V on 5H	5493 to 5410
1V on 3H	5410 to Ground Surface

* NGVD 1929

Earthwork grading plans are presented on Plate Nos. A-3 thru A-7.

3.2 Outlet Works. The outlet works is located near the left abutment of Chatfield Dam just to the right (south east) of the spillway. The outlet works discharge channel joins the spillway discharge pilot channel approximately 1500 feet below the toe of the dam. The outlet works structures consist of an intake structure, intake structure service bridge, twin oblong outlet works conduit and an energy dissipating drop structure and stilling basin.

3.2.1 Approach Channel. The approach channel to the outlet works intake has a length of approximately 700 feet at elevation 5385.0 feet m.s.l., which is also the intake invert elevation. A U-frame structure, 31.5 feet wide with vertical walls varying in height from 24 feet to 5 feet, lines the channel for a distance of 63 feet upstream from the intake structure. The remainder of the channel is excavated with 1V on 3H side slopes and is 10-feet wide.

3.2.2 Intake Structure. The intake structure has three gated passageways that conduct water to the twin outlet works conduits. The two right passageways converge toward a conduit transition monolith in which the convergence to one passageway is completed. Each of these passageways has a service gate and emergency gate, which are controlled by hydraulic hoists. In each gate, a 2-foot by 2-foot auxiliary gate is provided to facilitate regulation of normal flows to the river. In the left passageway of the intake structure, a 6 foot diameter penstock, equipped with a butterfly valve near the upstream end, is provided to conduct releases to satisfy the downstream water rights. At the upstream end of the bellmouth entrance to each passageway are slots for bulkheads to facilitate maintenance of the gates and valves. Above the water passageways the intake is a rectangular, dry-well type structure with intermediate floors consisting of a hydraulic hoist chamber, bulkhead platform level, operating level and machine room level. An elevator and stairwells or embedded ladders furnish access between the floors. At the top of the intake structure, 142 feet above the invert, is a service deck that is accessible from the top of dam by a service bridge. A 10-foot high wall encloses the deck except for a 15 foot opening to provide access from the bridge.

3.2.3 Trash Control. Vertical concrete trash beams with horizontal circular struts are provided for trash control at the inlet of the two water passageways on the right side (looking downstream). The clear openings are approximately 5-feet by 5-feet in dimension. These trash fenders prevent trees and large floating objects from entering the water passageways but will allow passage of smaller debris, which will normally go through the intake structure and conduit without damage.

3.2.4 Conduit. Each opening in the twin oblong conduit has a width of 12.0 feet and a height of 16.0 feet. A 5.5 foot radius, semi-circular arc on top and bottom connected by 5.0 feet straight vertical side walls, forms each opening. The discharge through the right passageway is controlled by the service gates for high pools and can maintain a maximum discharge of 5,000 c.f.s. The left passageway provides ample space for the penstock and walkway. The conduit and transition is 1,280 feet long, with a slope of 0.0076 feet/feet and invert elevations of 5385.0 feet m.s.l. and 5375.0 feet m.s.l. at the upstream and downstream ends, respectively.

3.2.5 Penstock. Three irrigation ditches were blocked off by construction of Chatfield Dam. To continue the water supply to these ditches, the left barrel of the conduit contains a 72 inch penstock, which has an independent inlet at the left side of the intake tower. The inlet includes a steel trashrack, bulkhead slots, bellmouth entrance, and a butterfly valve, which is normally operated fully opened. All bulkhead slots are the same width, thus a single bulkhead may be used for either of the two outlet works water passageways or for the penstock inlet. A manifold structure at the downstream end of penstock contains the gates and branch pipes, which distribute water to the ditches.

3.2.6 Stilling Basin. The stilling basin is a U-frame structure consisting of a drop section, 70 feet long and a level basin 88 foot long. Walls are vertical and are a maximum of 27 feet high in the level basin. The stilling basin is designed for a maximum discharge of 8,300 c.f.s. This corresponds to a pool elevation of 5,500 feet m.s.l. with both gates wide open.

3.2.7 Discharge Channel. The discharge channel connects the stilling basin with the spillway discharge pilot channel and the improved river channel. The bottom width is 100 feet. The side slopes are 1V on 3H and are riprapped for a distance of 200-feet from the stilling basin. Riprap is placed on the side slopes upstream from the end sill to guard against erosion.

3.2.8 Service Bridge. The intake service bridge is a 5-span, pre-cast, pretensioned box girder type bridge, 514'-6" long between the centerlines of bearings at the abutment and the intake tower. The bridge deck is 12'-0" wide between the 1'-3" high cast-in-place curbs on which 2'-1/2" high aluminum guard rails are mounted. The three box girders which support the bridge deck are tied together laterally by 1-1/4" diameter tensioned steel bars. The deck is constructed of cast-in-place concrete. The intermediate bridge supports are reinforced concrete bents with spread footings, approximately equally spaced. The abutment is a closed reinforced concrete structure that contains a vault for a transformer and other electrical devices.

3.3 Spillway. The chute-type spillway is located in the left abutment of the dam. The spillway consists of an ungated ogee weir, 500 feet wide and 10 feet high above the top of the approach channel slab, a chute 838 feet in length and varying in width from 500 to 390 feet, and a stilling basin 390 feet wide and 154 feet long. The stilling basin floor is 148 feet below the crest of weir. The discharge channel has a bottom width of 550-feet with 1V on 5H side slopes. From the end sill, the channel floor gradually rises about 28-feet above the stilling basin floor and then slopes gently to the river. Sandstones, siltstones, and clay-shales of the Dawson Formation underlie the spillway area.

3.3.1 Weir. The weir consists of nineteen 25 foot monoliths. A 12.5 foot section of weir is constructed integrally as a part of each abutment. The total weir length, at the centerline of the crest is an arc with a radius of 860.34 feet. The width of the weir parallel to the direction of flow is 50 feet. The crest of the weir is at elevation 5500.0 feet m.s.l. There is an 8 feet wide by 8 feet high gallery under the weir structure which houses foundation drains in the form of deep intercepting holes (PVC pipe lined) under the weir structure. These drains discharge into the gallery.

3.3.2 Abutments. On both the north and south extremities of the spillway weir, there is an abutment monolith 39.5 feet long measured normal to the direction of flow and 50 feet wide to match the upstream-downstream width of the weir. The length of each abutment monolith includes a 12.5 foot section of the weir. The spillway side face of each abutment above the crest of the weir is a vertical surface. Both abutments have a gallery entrance complete with stairs as well as ventilating and lighting systems. The abutments are identical in dimension, except they are opposite hand.

3.3.3 Approach Walls. The approach walls, which are identical in design on each side of the spillway, are cantilever type structures extending 150 feet upstream of the abutments. The walls are elliptical in alignment and were constructed in four monoliths. The upstream monolith is 25.67 feet long measured on the elliptical working line. The next three monoliths are each 52 feet long. All vertical joints contain waterstops.

3.3.4 Approach Slab. The 8-inch thick spillway approach slab extends 50-feet upstream from the face of the weir. A seat is provided for the slab at the weir on the toe of the approach walls and on the upstream cutoff structure. The joint between the slab and these structures is an expansion joint provided with waterstops. A 7 feet deep cutoff structure is provided at the upstream end of the approach slab to prevent entry of water under the slab during operation of the spillway. Beneath the slab is a drainage system that discharges into a manhole at the upstream end of slab.

3.3.5 Chute Walls. The cantilever type chute walls, which are identical in design on each side, extend downstream from the abutments a distance of 801 feet. There are waterstops in all the vertical joints, extending from the top of the wall stem to the waterstop between the base of the wall and the chute slab. The stem heights vary from about 30 feet downstream of the abutments to 14.5 feet high on the upstream 3 percent chute slope. On the 25-percent slope portion of the chute, the wall height is 13.5 feet with the exception of the downstream 134 feet where the stem varies from 13.5 feet to 47 feet in height.

3.3.6 Stilling Basin Walls. The stilling basin walls are cantilever type walls extending 154 feet downstream from the chute walls. There are waterstops included in each vertical joint from the top of the wall to the waterstop between the base of the wall and the stilling basin slab. The top of the wall is at elevation 5399 feet m.s.l. and the stem height above the stilling basin is 47 feet. The top 3 feet of the channel face of the stem is vertical while the lower portion has a 1V on 12H batter. The chute and stilling basin walls have backfill drains and an embedded drain collector, which discharges into the wall manholes.

3.3.7 Chute and Stilling Basin Slabs. The chute slab just downstream of the weir is level, elevation 5490 feet m.s.l., and is a circular segment with a length of 34.63 feet at the slab centerline. A circular curve 8 foot long forms the transition between the level area and a 3 percent slope. The slab then extends downstream on a 3 percent slope for a distance of 283.33 feet to elevation 5481.5 feet m.s.l. In this area, spillway width converges from 500 feet at the crest centerline to 390 feet at the downstream end of the 3 percent slope. The slab slope then changes to 25 percent for a distance of 518 feet to the stilling basin, elevation 5352 feet m.s.l. The stilling basin slab extends an additional 123 foot downstream at elevation 5352 feet m.s.l. to the seat on the end sill. The width of the 25- percent portion of the chute is 390 feet, and the width of the stilling basin slab is 382.67 feet. The transition between the two widths is between the beginning of the stilling basin and a point 176 feet upstream measured horizontally on the 25 percent slope. The slab is 1.5 feet thick from the weir to Station 4+85, from which the thickness increases to 4.0 feet at Station 8+03. The remainder of the chute slab is 4.0-feet thick. In the stilling basin, the slab is 6.0 feet thick under the baffles and 9.0 feet thick at the end sill. Beneath the slabs is a system of slab and foundation drains which discharge into manholes constructed in the walls on each side of the spillway.

3.3.8 Baffles. Two rows of baffles 5-feet high extend across the width of the stilling basin. The bottom of each baffle is recessed six inches into the slab and anchored by means of No. 9 bars hooked into the slab.

4.0 Construction History. The Chatfield Dam and Reservoir Project is one unit in the comprehensive plan for flood control of the South Platte River and its tributaries within Colorado, Wyoming, and Nebraska. The project was authorized for construction by the Flood Control Act of 1950.

Construction on the Chatfield Project began in August 1967, under contract No. DACW45-68-C-0023, Earthwork - Stage I, Valley Cutoff. A list of the major construction contracts for Chatfield Dam is given below.

- **Earthwork - Stage I**, Valley Cutoff, contract number DACW45-68-C-0023 was awarded to Johnson Bros. Highway and Heavy Contractors, Inc., and D. H. Blattner and Sons, Inc., of Litchfield, Minnesota.
- **Earthwork - Stage II**, contract number DACW45-68-C-0131 was awarded to Johnson Bros. Highway and Heavy Contractors, Inc., of Litchfield, Minnesota.
- **Earthwork - Stage III**, contract number DACW45-70-C-0095 was awarded to Holloway Construction Company, of Wixom, Michigan.

- **Outlet Works**, contract number DACW45-71-C-0058 was awarded to Wietz Company Inc., of Des Moines, Iowa.
- **Spillway Structure and Intake Service Bridge**, contract number DACW45-72-C-0088 was awarded to Hensel Phelps Construction Co., of Greeley, Colorado.

Closure of Chatfield Dam was completed on 1 August 1973 under the Stage II earthwork contract. The Earthwork Stage III contract was completed in 1974. The contract for the outlet works was issued and completed during the time of the Stage III earthwork contract. All of the major facilities at Chatfield Dam were essentially completed by April 1977.

5.0 Seismic Evaluation.

5.1 Preliminary Seismic Evaluation – 1985. A Preliminary Reconnaissance Report addressing seismicity was published in June 1985. The analysis indicated that the saturated pervious overburden materials in the upstream and downstream right abutment and valley section of Chatfield Dam would be susceptible to liquefaction during postulated earthquake shaking. The preliminary evaluation concluded that in the event that these potentially unstable silty sands, silty gravelly sands and silts should liquefy during this earthquake, the seismic stability of the embankment would be questionable. It was recommended therefore to proceed with additional analysis including additional drilling, sampling and testing.

5.2 Seismic Evaluation – November 1986 (D.M. PC-44). As a result of the finding of the Preliminary Seismic Evaluation (1985), a more in-depth evaluation was conducted. The findings of this analysis concluded that the embankment materials including the sand drain would be safe against liquification. The analysis also concluded that the overburden materials (upstream and downstream) would be safe from liquefaction except for an area along the upstream toe between Station 60+00 and 95+00 and between 400 feet upstream of the embankment centerline and the embankment toe. It was also concluded that even in light of the identified liquefaction zone, the embankment would remain stable during the postulated earthquake.

5.3 Seismic Safety Review - 2002. The Omaha District performed a Seismic Safety Review (SSR) for Chatfield Dam in Fiscal Year (FY) 2002. The SSR, including a Policy Compliance and Criteria Review (PCCR), were completed in FY05. This evaluation was performed in accordance with ER 1110-2-1806 (31 July 1995) “Earthquake Design and Evaluations for Civil Works Projects” and ER 1110-2-1155 (12 September 1997) “Dam Safety Assurance Program”, Appendix B “Seismic Safety Evaluation Process for Embankment Dams and Foundations”.

This SSR evaluated the adequacy of the previous seismic design evaluations presented in Design Memorandum No. PC-44 “South Platte River Basin, Chatfield Dam and Lake, Colorado, Seismic Evaluation”, November 1986, to determine if changes in seismicity or analytical techniques would indicate that additional detailed evaluation was warranted.

Based on the findings of the SSR and the Independent Technical Review (ITR) comments, progression of this study to a Phase I Special Study was recommended to further evaluate the seismic hazards for Chatfield Dam. The SSR also recommended that the access bridge and the bridge piers be evaluated since the gates are dependent on the power feed that runs across the bridge.

5.4 Seismic Evaluation (Phase I). The scope of the Seismic Study was expanded to determine what impacts if any the higher pool elevations (up to elevation 5444 feet msl) have on seismic stability. The seismic analysis will be published under separate cover(s).

6.0 Slope Protection – The upstream slope protection material on the embankment consists of a 12" to 26" thick layer of dumped, quarried granitic gneiss riprap underlain by a 9" layer of spalls and a 6" layer of bedding. Typical cross sections of the dam along with slope protection sections and details are presented on the General Project Plates in Appendix A. The size and gradation of the material was designed using criteria developed by the Missouri River Division from wave tank tests.

The existing multipurpose pool elevation (El. 5432 feet m.s.l.) is slightly above the transition point from the 1V:15H rocked slopes, or in some cases natural ground, to the 1V:5H slopes. The slope protection in this reach (1V:5H) consists of 20" of riprap placed on spalls and bedding. The new proposed elevation of the multipurpose pool (El. 5444 feet m.s.l.) will also be on the 1V:5H slope. Two distinct zones of horizontal riprap displacement have been identified at intermittent locations along the upstream slope during past inspections. The displacement is moderate (1 to 2 feet) and has been observed for several hundred feet along the shoreline. These displacement zones have been estimated to be between elevations 5434 and 5439 feet m.s.l. According to project personnel, the riprap was pushed up by ice-action during the winter of 1992-93. Although visual inspections indicate that 1 to 2 feet of displacement has occurred in some areas, no exposed spalls, bedding or embankment have been noted. A slight amount of displacement (1 foot or less) has also been noted near the existing normal pool line (approximate elevation 5430 to 5431 feet m.s.l.). The displaced stone has not been identified as a dam safety concern in recent Periodic or Annual Inspections. No remedial actions have been identified. These areas are monitored by project personnel on a routine basis. Raising the normal pool elevation by 5 to 12 feet should not have a direct bearing on the adequacy of the slope protection material; however, the slope protection material will continue to be monitored during routine dam safety inspections (monthly, annual, periodic,…). In addition to this, effort should be made to inspect existing areas of riprap displacement during low reservoir elevations.

7.0 Slope Stability.

7.1 General. Slope stability analyses were originally performed for three embankment sections during the design of Chatfield Dam: (1) embankment section at Station 95+00 where the embankment attains a maximum height of 137 feet and the depth of the alluvial material is approximately 55 feet; (2) the outlet works section, Station

104+34, where the embankment attains a height of approximately 117 and alluvial material is 25 feet deep; and (3) the right valley embankment section at Station 68+50 where the embankment is approximately 131 feet high but where the Dawson Formation is at the ground surface for the downstream portion of the section and 30 to 40 feet below the surface under the upstream portion of the section. The analyses consisted of investigating four cases which simulate conditions of stress during the life of the structure. The cases were: (1) end of construction; (2) sudden drawdown; (3) partial pool and (4) steady state seepage. Detailed information on the various cases analyzed during design is presented in Appendix B, Plate Nos. B-1 thru B-7.

7.2 Method of Analysis. The sliding wedge method was used for the stability analyses. The factors of safety in the analyses were defined as the ratio of the available shear strength to the average necessary to maintain equilibrium.

7.3 Summary of Results. The results of the original stability analyses performed during the design stage of Chatfield Dam are presented in Table No. 3. The results of the stability analysis (original design) based on seismic loading are presented for information only. Slope stability from seismic loadings is fully addressed under cover(s).

Table No. 3 - Slope Stability Analysis Results *

Case	Location	Critical Reservoir Elevation	Critical Factor of Safety			
			Normal		Earthquake	
			Actual	Req.**	Actual	Req.**
End of construction (u/s)	Sta. 95+00	NA	2.62	1.3	1.82	1.0
End of construction (d/s)	Sta. 95+00	NA	2.53	1.3	1.62	1.0
Partial pool (u/s)	Sta. 95+00	5460	1.49	1.5	1.04	1.0
Partial pool (u/s)	Sta. 104+35	5435-5450	1.46	1.5	0.90	1.0
Sudden drawdown (u/s)	Sta. 95+00	5500	1.33	1.2	0.92	1.0
Sudden drawdown (u/s)	Sta. 95+00	5521.6	1.23	1.0	na	na
Steady state seepage (d/s) - Conventional	Sta. 95+00	5500	1.43	1.5	.86	1.0
Steady state seepage (d/s) – At Rest pressures	Sta. 95+00	5335	1.13	-	-	-
Steady state seepage (d/s)	Sta. 104+35	5500	1.58	1.5	0.92	1.0
Steady state seepage (d/s)	Sta. 68+50	5500	1.62	1.5	0.94	1.0

* Documented in the Embankment Criteria and Performance Report, April 1980
** Based on requirements presented in EM 1110-2-1902

End of Construction, Station 95+00 - Plate No. B-1 presents the "end of construction case" (upstream & downstream) at Station 95+00. Adequate (above minimum required) factors of safety were obtained for these cases.

Partial Pool, Station 95+00 - Plate No. B-2 presents the upstream slope partial pool analysis at Station 95+00. The analysis produced a minimum (critical) factor of safety of 1.49 at pool elevation 5460 feet m.s.l. with the failure plane in the random fill. The minimum required factor of safety is 1.50. According to the analysis, the factor of safety increased both above and below this critical pool elevation (5460 feet m.s.l.) as shown on Plate No. B-2. According to the analysis, the factor of safety would be at or above the minimum required 1.5 value for a pool elevation of 5444 feet m.s.l.

Sudden Drawdown, Station 95+00 - Plate No. B-3 presents the results of the sudden drawdown analysis of the upstream slope at Station 95+00. Sudden drawdown stability (upstream slope) was evaluated for both the spillway pool (El. 5500 feet m.s.l.) and the maximum surcharge pool (El. 5521.6 feet m.s.l.). The minimum calculated factor of safety starting with a spillway pool (El.5500 feet m.s.l.) was 1.33 while the factor of safety at a surcharge pool (El. 5521.6 feet m.s.l.) was 1.23. The required factors of safety in these cases are 1.2 for pools starting at the spillway crest (5500 feet m.s.l.) and 1.0 for the maximum pool elevation (5521.6 feet m.s.l.). The analysis was performed using a conservation pool of El. 5426 feet m.s.l. whereas the actual conservation pool elevation (prior to a pool raise associated with the Re-Allocation Study) is El. 5432 feet m.s.l., six feet above what was used. This would make the analysis more conservative. The re-allocation study assumes an increase in the conservation pool elevation of up to 12 feet. This would be 18 feet above the water level used in the original design analysis (El. 5426 feet m.s.l.). Based on the conservative strengths assumed in design (as documented in the Embankment Criteria & Performance Report and discussed hereinafter), the factor of safety of 1.23 (1.0 required as minimum) at the lower conservation pool of El. 5426 feet m.s.l. and the increase in the conservation pool elevation, it is felt that the dam would have an adequate factor of safety for sudden drawdown.

Historic analysis (design stage) was based on USACE guidance available at that time and did not consider transient seepage conditions. Based on the conservative strengths assumed in design (as documented in the Embankment Criteria & Performance Report and other documents) higher conservation pool, and design calculated factor of safety of 1.23 (1.0 required as minimum), additional transient analysis is not considered necessary.

Steady State Seepage, Station 68+50, 95+00 & 104+35, - The downstream slopes of the embankment were analyzed under a steady state condition at Stations 95+00, 104+35 and 68+50 using the spillway crest pool elevation (El. 5500 feet m.s.l.). Plate No. B-4 presents the steady state seepage case at Station 95+00. Plate No. B-5 presents the steady state seepage case at Station 104+35. Plate No. B-6 presents the steady state seepage case for Station 68+50. These sections were considered to be the most critical of all the sections studied. Saturation through the embankment was assumed to be from the spillway crest pool (El. 5500 feet m.s.l.). The maximum pool elevation (El. 5521.6 feet m.s.l.) was not used in the analysis since it was considered that this pool elevation would not be maintained long enough to produce seepage equilibrium through the embankment. The downstream piezometric level was assumed to be at the top of the pervious blanket drain (elevation 5400 feet m.s.l. at Stations 95+00 & 68+50 and elevation 5420 feet m.s.l. at Station 104+35). There appears to be a conflict between various documents as to the exact elevation and thickness of the drain. It is documented in the Embankment Criteria and Performance Report that the thickness of the drain is 20 feet in the valley and 10 feet at the abutments. The drawings show 10 feet at Station 62+00 and 20 feet at Station 70+00 whereas the cross section used for the steady state seepage stability analysis (Embankment Criteria and Performance Report) at Station 68+50 shows a 10 foot drain with top elevation of 5405 feet m.s.l. It would be advantageous to verify the exact thickness of the drain.

Station 95+00 produced the lowest factor of safety for steady state seepage conditions. As shown in Table No. 3, the computed factor of safety at Station 95+00 was 1.43 for normal conditions with the critical failure plane in the Dawson Formation. This value is 0.07 below the required minimum factor of safety (1.50). The proposed Re-Allocation project would result in a maximum new normal pool elevation of 5444 feet m.s.l. This is 56 feet lower than the reservoir elevation (5500 feet m.s.l.) analyzed. The Re-Allocation will not increase the reservoir elevation analyzed in this case (5500 feet m.s.l.) and therefore the new normal pool elevation would not be expected to reduce the factor of safety calculated during design. The factors of safety at Stations 104+35 and 68+50 were 1.58 and 1.62 respectively as presented in Table No. 3 and on Plate Nos. B-5 and B-6.

Non-circular, block failure surfaces were analyzed at Station 95+00 and 68+50 with Spencer's method utilizing the 2007 version of SLOPE/W, developed by Geo-Slope International, Ltd. For the re-analysis of the steady state seepage cases, the critical slide plane and peak effective strengths were used as presented in the Embankment Criteria and Performance Report (CENWO, 1980). The intent of these re-analyses was to compare the current methodology (Spencer's method) with the method used for the original design. The results of the re-analysis of the steady seepage cases at Station 95+00 and Station 68+50 are presented in Table No. 4. These results indicate the factors of safety determined with Spencer's method exceed those factors of safety determined during the original design.

Table No. 4			
Station	Factor of Safety		
	Original	Re-Analysis	
		Janbu's Method	Spencer's Method
95+00	1.43	1.46	1.74
68+50	1.62	1.57	1.84

The difference in the Factor of Safety between the original analyses and the re-analyses using Spencer's method is due to the side force assumptions and statics of each method.

Upstream Slope Partial Pool, Station 104+35 - The upstream slope partial pool analysis (normal loading) at Station 104+35 produced a minimum factor of safety of 1.456 @ pool elevation 5435 feet m.s.l. with the failure plane in the Dawson Formation, as presented on Plate B-5. The required minimum factor of safety is 1.50. The factor of safety increased slightly both above and below this critical elevation (5435 feet m.s.l.) as shown on Plate No. B-5. According to the information presented on Plate No. B-5, the factor of safety at the proposed new normal pool elevation (5444 feet m.s.l.) would also be approximately 1.46. It is pointed out that the current normal pool elevation is 5432 feet m.s.l. and the reservoir has historically operated approximately between elevations 5428 feet m.s.l. and 5447 feet m.s.l.
Although factors of safety slightly lower than that required by Corps criteria were obtained during the original design, the analyses were considered "as adequate" because of the exceptionally conservative adopted strengths and other assumptions in the analyses. The adopted design strength of the embankment material was based on primarily the lowest strength material of all the types placed in the embankment. The bedrock was assumed to be homogeneous in strength with no allowances made for cross bed shear and the sandy and silty phases of the Dawson Formation. The value of the adopted strength of the Dawson Formation was taken as an average between the residual shear strength test values (approximately 8 degrees) and the average peak strength of 24 degrees. The resulting adopted phi value of 15 degrees for the Dawson Shale, as shown on Plate No. B-7, was considered to be very conservative.

Additional slope stability analyses including soil testing were performed for the section of embankment at the outlet works as presented in "Supplement to Design Memorandum No. PC-24, Embankment and Excavation, December 1970." The general consensus in the August 1968 Board of Consultants meeting was that the conventional wedge analysis may not be applicable where failure is assumed in the Dawson Formation. The Board discussed the potential for strain incompatibility between the Dawson Formation and the embankment materials. The new test results fell within the range of the previous test data. The additional analyses involved no change in shear strength but took into account some or all of the following: (1) apparent soft seams in the Dawson Formation and their depth relative to the slide planes; (2) earth pressures at rest; (3) seepage pressures in the foundation shale; (4) pore pressures recorded at Fort Peck Dam which were translated to these studies; and (5) excavations of the intake and stilling basin included in some analyses. Specific cases analyzed are as follows:

End of Construction – Sta. 103+34: These studies were made assuming the following.

- Pore pressure responses in the Dawson Formation varying from 0 to 100%.
- Driving forces computed from earth pressure theory using an at-rest pressure coefficient of 0.5.
- Strengths assumed along failure plane: phi = 15 degrees & cohesion = 0

Depending on the specific assumptions in the analysis, the computed upstream slope factors of safety ranged from 1.77 to 0.81. The downstream factors of safety were approximately 0.2 higher than calculated for the upstream slope.

Steady Seepage Case – Sta. 103+34: All design strengths were the same as used in the original analysis presented in the basic design memorandum. The factors of safety computed ranged from 1.22 to 1.24. In this case the factors of safety were considered ….”satisfactory due to the relatively low adopted shear strength and other maximum conditions assumed in the analysis”[1].

Partial Pool Case – Sta. 103+34: All design strengths were the same as used in the original analysis presented in the basic design memorandum. For three pool levels analyzed, the critical pool was at elevation 5450 feet m.s.l. with a factor of safety of 1.34. In the original stability analysis, a factor of safety of 1.46 was computed for pool elevations 5435 feet m.s.l. and 5450 feet m.s.l. “A factor of safety of 1.5 usually is required for conventional type analyses; however, the lower factor of safety appears justified here due to the use of the conservative strength assumptions”[1].

Sudden Drawdown Analysis – Sta. 103+34: For the case of drawdown from pool elevation 5500 feet m.s.l. to the minimum pool elevation of 5426 feet m.s.l. using various assumptions, a factor of safety of from 0.84 to 1.0 was obtained. It was concluded that .. ”It is extremely unlikely that all of the above conditions assumed in the analyses would ever be met, that is, the assumptions used have been conservatively chosen, and as such the resulting factors of safety reflect those assumptions”.[1]

Steady Seepage – Sta. 95+00: A revised analysis of the downstream slopes of the embankment at Station 95+00 was performed using at-rest pressures. The resulting factor of safety for the “At-Rest Pressure Analysis” was 1.13. “For this condition, with all of the maximum conditions imposed in the analysis, maintaining equilibrium is considered satisfactory”.[2]

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[1] Supplement to Design Memorandum No. PC-24, Embankment and Excavation, December 1970.

[2] Design Memorandum No. PC-24, Embankment & Excavation, Dec 1968

Results of the additional studies gave factors of safety lower than those previously obtained; in some cases below equilibrium conditions (less than 1); however, it was recognized that the many different assumptions were, as a whole, extremely conservative and as a result, the factors of safety so obtained reflected those assumptions. No definite conclusions/recommendations were identified in the Supplement to Design Memorandum PC-24 other than the use of conservative assumptions; however, it was stated that this subject would be further addressed in the Board of Consultants meeting scheduled for the fall of 1971. There was very little information documented about embankment stability and/or test results in the minutes of this meeting; however, no stability concerns were identified. Author Casagrande stated in part…"On this stability analysis, I have the impression that the assumptions are certainly on the safe side…."[3]

A discussion of the current and projected piezometric levels as compared to what was assumed in the design phase (stability analysis) is presented hereinafter.

7.5 Instrumentation Review. A piezometer location plan is presented in Appendic C, Plate No. C-1. The Chatfield Dam piezometer data is presented on Plate Nos. C-2 thru C-10. Plate Nos. C-11 thru C-14 presents various cross sections exhibiting piezometeric data. Plate Nos. C-15 thru C-35 present specific detailed piezometer plots. These plots were developed to identify any potential reflection of fluctuations in the reservoir level and ultimately the effect of an increased pool loading condition (piezometeric data) if any.

The upstream piezometric levels including data from piezometers located in the core are fairly responsive to the reservoir elevation and in some cases there is minimal headloss. Based on this piezometer data along with data projections as discussed hereinafter, it appears that the piezometric levels in both the upstream area and the core are/will be very similar to those assumed in the design (stability analysis).

The downstream piezometers are affected by the pool to a much lesser degree than the upstream and core piezometers as expected. The piezometer levels in the downstream bedrock (Dawson Formation) are currently at or below the elevation assumed in the steady state stability analysis at Stations 68+50, 95+00 and 104+35. Based on the relationship with pool fluctuations, it appears that there are potentially three critical downstream piezometers that include instruments numbers 504A, 505B, and 561. Piezometer 504A is a pneumatic pressure cell located at Station 68+90, 195 feet downstream. Piezometer 505B is a pneumatic pressure cell located at Station 68+90, approximately 400 feet downstream. Piezometer 561 is an open tube device located at Station 93+00, approximately 300 feet downstream. Data plots for these instruments are presented on Plate Nos. C-23, C-23A, C-25, C-25A, C-25 and C-28A.

These three piezometers monitor pressures in the downstream Dawson Formation and currently exhibit the highest piezometric levels of the downstream bedrock instruments, approximately elevation 5400 feet m.s.l. (piezometric level used in

[3] Meeting of Board of Consultants, Chatfield Dam, 16 Nov 1971

stability analysis). These instruments also exhibit minor fluctuations which are assumed to be related to the reservoir level.

A review of the data from Piezometer 504A reveals a slight decrease in level from approximately elevation 5408 feet m.s.l. in 1979 to the current level of approximately 5400 feet m.s.l. as shown on Plate Nos. C-23 and C-23A. There does appear to be a slight reflection of the reservoir elevation in the piezometer data, in particular in June 1995 during a high pool elevation. The tip elevation of Piezometer 504A; however, is 5338.3 feet m.s.l. This is approximately 18 feet above the elevation of the critical "failure plane" as documented on Plate No. B-6.

A review of the data from Piezometer 505B reveals a slight increase in level from approximately elevation 5396 feet m.s.l. in 1975 to the level of approximately 5400 feet m.s.l.in 1985 as shown on Plate Nos. C-25 and C-25A. From 1985 to the present the piezometric level has remained fairly constant at the approximate elevation 5400 feet m.s.l.. There does not appear to be a substantial reflection of the reservoir elevation in the piezometer data. The tip elevation of Piezometer 505B; however, is 5386.6 feet m.s.l. This is approximately 66 feet above the elevation of the critical "failure plane" as documented on Plate No. B-6.

A review of the data from Piezometer 561 reveals a definite decrease in level from approximately elevation 5415 feet m.s.l. in 1979 to the current level of approximately 5400 feet m.s.l. as shown on Plate Nos. C-28 and C-28A. There does appear to be a substantial reflection of the reservoir elevation in the piezometer data, primarily in 1995. The tip elevation of Piezometer 561 is 5297 feet m.s.l. The sensing zone is one foot below the tip to 3 feet above the tip. This is approximately the elevation of the assumed "failure plane" used in the stability analysis at Station 95+00.

The piezometric levels in these three instruments are currently at or below the piezometric level assumed in design (5400 feet m.s.l.); however, minor reflections of reservoir fluctuations have been observed primarily in Piezometer 561 in 1995. Additional bedrock piezometers with tip elevations in the "critical failure plane" would be warranted to better monitor the effects of higher pools. This would also enable the determination of the elevation and depth of the drain.

There are no piezometers located in the drain and very few in the downstream overburden. Additional piezometers in both the drain and downstream overburden would be warranted if a pool raise were to be implemented. Preliminary locations of proposed new piezometers are Stations 69+00, 81+00, 93+00 and 102+00. Additional piezometers would not only provide needed piezometric data during high pool elevations but would also enable the determination of the exact elevation and thickness of the drain.

The existing piezometer data is at or below that which was assumed in design (slope stability); however, two piezometers (504A & 561) in the Dawson Formation have exhibited a reflection to fluctuation in the reservoir elevation. Based on historic records, primarily during the record pool in 1995 these piezometers reflected the pool change by 15 to 20%. Piezometer 504A is a pneumatic cell located at Station 68+90, Range 195 d/s. Piezometer 561 is an open tube device located at Station 92+90, Range 300 d/s. Both of these instruments monitor pressure in the Dawson Formation. A piezometric level of 5405 feet m.s.l. was assumed in the stability analysis at Station 68+50. Piezometer 504A is currently reading approximately 5401 feet m.s.l. If the normal pool elevation was increased by 12 feet, this instrument could, based on historic fluctuations, increase to the approximate elevation of 5403 feet m.s.l. This is still below what was assumed in the design. It is also stressed that the factor of safety in the original analysis at Station 68+50 was 1.62. Piezometer 561 located at Station 92+90 is currently reading the approximate elevation of 5398 feet m.s.l. A piezometric level of 5400 feet m.s.l. was assumed in the stability analysis at Station 95+00. The current piezometric level observed by Piezometer 561 is approximately 5398 feet. m.s.l. If the normal pool elevation was increased by 12 feet, this instrument could, based on historic fluctuations, increase to the approximate elevation of 5400 feet m.s.l., the elevation used in the design. A potential concern is for development of pressures that exceed those used in the design analysis at a reservoir elevation of 5500 feet m.s.l. The piezometric level in both Piezometer 504A and 561 would be expected to exceed the level used in design based on past performance. Piezometeric levels exhibited by these instruments do not pose a concern related to the Re-Allocation Project; however, this concern should be pursued as part of the routine dam safety program.

A review of the embankment movement data (inclinometers and survey points) does not reveal any areas of instability or potential instability. Movement data (surveys) along with inclinometer data is presented in Appendix D as discussed hereinafter. No relationship has been identified between movement (potential instability) and thrust of the pool; however, the maximum pool elevation to date has been approximately 5447.58 feet m.s.l. and this elevation was maintained for a relatively short time period.

8.0 Seepage Control.

8.1 Embankment Seepage.

8.1.1 General. The embankment was designed with a central symmetrical impervious core with 3V on 1H side slopes to provide an effective barrier against through seepage. A pervious inclined sand drain with a blanket outlet was placed adjacent to the downstream slope of the impervious core to intercept and dissipate any seepage through the embankment. According to the Embankment Performance and Criteria Report the thickness of the drain in the valley is 20 feet and transitions to a 10 foot thickness in the abutments.

8.1.2 Instrumentation Review. A review of the embankment (core) piezometer data, as presented in Appendix C, revealed a slight influence of the reservoir on piezometric levels for some of the instruments located in the impervious core, in particular the following instruments.

- **Piezometer No. 502A.** Piezometer 502A is a hydrostatic pressure cell located at Station 68+90. It is not know for sure if the cell is located in the bedrock, at the interface of the bedrock and the core or completely in the core. The observed piezometric level has fluctuated only slightly over the years, plotting at the approximate elevation of 5417. Since 1980 there has been; however, a very slight but steady overall decline in the piezometric level exhibited by this instrument.

- **Piezometer No. 502B.** Piezometer 502B is a hydrostatic pressure cell located in the core at Station 68+90. The observed piezometric level has fluctuated only slightly over the years, plotting at the approximate elevation of the downstream blanket drain; however, the piezometric level exhibited by this instrument increased approximately 4 feet in early 2005. The reason for this is unknown; however, this increase is not consistent with reservoir fluctuations. Pool influence is also not apparent in historical data. See Plate No.C-15, Appendix C for a detailed plot.

- **Piezometer No. 507C**. Piezometer 507C is a hydrostatic pressure cell located in the core at Station 81+20. The readings have been somewhat erratic; however, there is a general slight relationship with fluctuations in the pool elevation. See Plate No.C-16, Appendix C for a detailed plot.
- **Piezometer No. PZ95-03.** Piezometer No. PZ95-03 is an open tube located in the core at Station 81+20. The readings are approximately 10 feet below the reservoir elevation and 10 feet above the elevation of the downstream blanket drain. There is a definite relationship with fluctuations in the pool elevation. See Plate No.C-17, Appendix C for a detailed plot.

- **Piezometer No. 83+00/CTR**. Piezometer 83+00/CTR is an open tube device located in the core at Station 83+00. This instrument fluctuates only very slightly if at all with the elevation of the reservoir with the readings at or below the bottom of the blanket drain located downstream of the core. See Plate No.C-18, Appendix C for a detailed plot.

- **Piezometer No. 102+00/CTR.** Piezometer No. 102+00/CTR is an open tube device located in the core at Station 102+54. The readings fluctuate very slightly with a general decline over the past 10 years. The elevation of the readings is at or slightly above the top of the downstream blanket drain. No definite relationship with fluctuations in pool elevation is apparent. See Plate No. C-19, Appendix C for a detailed plot.

- **Piezometer No. 519C**. Piezometer No. 519C is a hydrostatic pressure cell located in the core at Station 102+54. Up to approximately 7 feet of fluctuation has been observed in this instrument since 1998. There has been a general increase (3'-5') over the past 7 years. The elevation of the readings is approximately 15' above the top of the downstream blanket drain. A slight relationship with fluctuations in pool elevation is apparent; however, a current investigation has revealed that one of the lines is plugged making the data extremely questionable. See Plate No. C-20, Appendix C for a detailed plot.

Piezometer 512D (pneumatic cell) is the only core instrument with a piezometric level that approaches the level used in the stability analysis (5450 feet m.s.l.). This instrument has produced erratic data over the years. A recent investigation revealed that there is gas flow in one direction; however, there is no flow in the other direction. This makes the data questionable.

There are no piezometers located in either the upstream fill or downstream fill to evaluate; however, Piezometer 563 located adjacent to the fill in the abutment material (downstream) has been historically dry. Additional piezometers installed in the downstream fill are warranted if a pool raise were to be implemented.

8.2 Foundation Seepage.

8.2.1 General. The overburden materials, in the valley and abutments, are of such high permeability as to require a positive form of underseepage control so that excessive losses of stored water and/or seepage concerns do not occur. The final adopted and approved form of underseepage was a backfilled trench located at the centerline of the embankment alignment. Numerous piezometers/pressure cells are located in the upstream overburden. These instruments reflect a direct influence of the reservoir which would be as expected. All piezometric levels are within normal trend and range.

Studies of types and associated costs of underseepage controls were reported in Design Memorandum No. PC-9, Initial Earthwork -Valley Cutoff. The final adopted and approved form of underseepage control was a backfilled impervious trench located at the centerline of the embankment alignment. The limits of the cutoff trench are from station 40+00 in the right abutment to its interception with the spillway structure in the left abutment at approximate station 123+00. The bottom width of the trench was designed to be a minimum of 35 feet in the valley and 25 feet in the abutments.

The cutoff trench was intended to extend a minimum of 3 feet into the Dawson bedrock formation. After the cutoff trench was excavated to the required depth, auger holes were then drilled at a minimum of 100 feet on centers to a depth of 30 feet to explore possible continuous layers of sandstone beneath the trench. Where layers were found, they were then excavated and replaced with compacted impervious material. The approximate maximum depth considered practical to excavate was 10 feet. Due to the deeper excavations below the regular bottom of the trench, it was necessary to reduce the width of the trench to a width slightly less than the original planned width; however, in no instance was the width allowed to be less than 25 feet. During drilling when sand layers were encountered, laboratory testing was used to assist in the decision to remove or leave the sand layer. If the sand had less than 20 percent passing the number 200 sieve size and a plasticity index of less than 5, it was removed and replaced with impervious material.

Pump tests were performed on selected piezometers to further investigate the permeability and susceptibility of the Dawson Formation to seepage. Bailing and recharge measurements were made on five (5) piezometers which had been installed in the Dawson Formation. Permeabilities at each piezometer were computed from the recharge tests by the Jacobs Modification to the Theis Recovery Method of Analysis. The thickness of the aquifer was taken as the depth of water in each piezometer which in most cases was 50 feet. The derived permeabilities were considered as semi-impervious or very low. From these tests and the pressure tests, it was concluded that the Dawson Formation had a relatively low permeability and would be relatively free of under-seepage problems.

Investigations were made to determine whether seepage could be a problem either under or around the cutoff trench. It was found that any seepage that might occur under the cutoff in the sandstones or around the end of the cutoff in the right abutment would be of such a small quantity that it would not be particularly noticeable. As an added precaution to intercept seepage and/or prevent piping of impervious material, a pervious section was placed on the downstream side of the cutoff trench which was then tied to the blanket drain.

8.2.2. Instrumentation Review. A review of the bedrock piezometer data, as presented in Appendix C, revealed considerable headloss across the cutoff trench. Bedrock piezometers located at or upstream of the cut-off trench are responsive to pool fluctuations with minimal headloss while those downstream do not exhibit a definite relationship with the reservoir and are in many cases exhibiting a downward trend, with up to 60 feet of headloss (Piezometer No. 560 @ Station 68+90). All of the downstream bedrock piezometers exhibit water/pressure levels within or below the elevation of the downstream blanket drain. All piezometric levels are within (historic) normal trend and range. The following bedrock piezometers are discussed in more detail.

Cross sections of Chatfield Dam exhibiting bedrock piezometric gradients are presented on Plate Nos. C-11 thru C-14 in Appendix C.

- **Piezometer No. 536.** Piezometer No, 536 is a hydrostatic pressure cell located approximately 200 feet upstream at Station 68+85. The instrument appears to be slightly responsive to pool fluctuations; however, there has been an overall downward tend over the past 10 years indicating possible siltation upstream and/or decreased permeability of the bedrock. See Plate No. C-21, Appendix C for a detailed plot.

- **Piezometer No. 41.** Piezometer No. 41 is an open tube located approximately 25 feet upstream from the dam centerline at Station 68+90. The tip is located in the bedrock slightly upstream of the cutoff trench. This instrument is highly responsive to pool fluctuations with only 1-2 feet of headloss. See Plate No. C-22, Appendix C for a detailed plot.

- **Piezometer No. 504A.** Piezometer No. 504A is a hydrostatic pressure cell located approximately 195 feet downstream at Station 68+90. There has been an overall downward tend over the past 10 years indicating possible siltation upstream and/or decreased permeability of the bedrock. The headloss between Piezometer No. 536 located in the foundation upstream of the cut off and Piezometer No. 504A located in the foundation downstream of the cut off is approximately 10 feet. An apparent or definitive relationship with pool fluctuations has been identified as discussed in Paragraph 7.5. See Plate No. C-23 & C-23A, Appendix C for detailed plots.

- **Piezometer 505A.** Piezometer No. 505A is a hydrostatic pressure cell located approximately 400 feet downstream at Station 68+90. This instrument appears to be slightly responsive (less than 504A). There has been no trend over the past 10 years. The headloss between Piezometer No. 536 and Piezometer 505A is approximately 15 feet. No apparent or definitive relationship with pool fluctuations has been identified. See Plate No. C-24, Appendix C for a detailed plot.

- **Piezometer 505B.** Piezometer No. 505B is a hydrostatic pressure cell located approximately 400 feet downstream at Station 68+90. This instrument appears to be slightly responsive (less than 504A). The headloss between Piezometer No. 536 and Piezometer No. 505B is approximately 10 feet. No apparent or definitive relationship with pool fluctuations has been identified except for as discussed in Paragraph 7.5. See Plate No. C-25 & C-25A, Appendix C for detailed plots.

- **Piezometer 560.** This instrument is located approximately 950 feet downstream in the Dawson Formation at Station 68+90. There is approximately 60 feet of headloss as compared to the pool elevation. There has been a very slight overall downward trend over the past 10 years indicating possible siltation upstream and/or decreased permeability of the bedrock. No apparent or definitive relationship with pool fluctuations has been identified. See Plate No. C-26, Appendix C for a detailed plot.

- **Piezometer 79+00.** This instrument is an open tube device located approximately 25 feet upstream from the dam centerline at Station 79+00. The tip is located in the bedrock slightly upstream of the cutoff trench. This instrument is highly responsive to pool fluctuations with only 1-2 feet of headloss. See Plate No. C-27 for a detailed plot.

- **Piezometer 551D & 551S.** These instruments are closed tube piezometer systems equipped with a pressure gages. They are located 750 feet downstream at Station 87+50. Prior to 2005, Piezometer 551D was experiencing a definite rise in piezometric level. In May of 2005 both gages were replaced. Since that time Piezometer 551D has read approximately elevation 5400 feet m.s.l., while 551S has read approximately 5390 feet m.s.l.

- **Piezometer No. 561.** Piezometer No. 561 is an open tube device located approximately 300 feet downstream in the bedrock at Station 93+00. The instrument appears to be responsive. There has been an overall downward tend (approximately 5 feet) over the past 10 years indicating possible siltation upstream and/or decreased permeability of the bedrock. An apparent or definitive relationship with pool fluctuations has been identified as discussed in Paragraph 7.5.See Plate No. C-28 & C-28A, Appendix C for detailed plots.

- **Piezometer 102+00/25US.** This instrument is located in the bedrock, 25 feet upstream at Station 102+54. The tip is located slightly upstream of the cutoff trench. The tip is located in the bedrock slightly upstream of the cutoff trench. This instrument is highly responsive to pool fluctuations with approximately 6 feet of headloss. See Plate No. C-29, Appendix C for a detailed plot.

- **Piezometer 486.** This instrument is an open tube piezometer located approximately 25 feet upstream from the dam centerline at Station 102+54. The tip is located in the bedrock slightly upstream of the cutoff trench. This instrument is highly responsive to pool fluctuations with approximately 4 feet of headloss. See Plate No. C-30, Appendix C for a detailed plot.

- **Piezometer 520A.** This instrument is a hydrostatic pressure cell located approximately 125 feet downstream in the Dawson Formation at Station 102+54. The instrument appears to be responsive but with no definite relationship with pool fluctuations. There has been an overall downward tend (approximately 3 feet) over the past 10 years indicating possible siltation upstream, decreased permeability of the bedrock and/or dissipation of pore pressure. The average headloss is approximately 45 feet as compared to the reservoir. No apparent or definitive relationship with pool fluctuations has been identified. See Plate No. C-31, Appendix C for a detailed plot.

Based on a review of the foundation piezometeric data, seepage within the Dawson Formation would not pose a concern under the proposed reallocation.

<u>**8.3 Overburden.**</u> The downstream overburden piezometeric data, as presented in Appendix C, revealed considerable headloss. All piezometric levels are all within historic (normal) trend and range. There has been a general slight decreasing trend observed in many of the instruments. All of the downstream overburden piezometers exhibit water/pressure level within or below the elevation of the blanket drain.

A “wet area” has been identified just upstream of the outlet works stilling basin area at various times since construction of the dam. Normally the area is dry but on rare occasions water is observed emerging the slope. Historic information points towards precipitation as the source of the wet area. Additional piezometers would be beneficial in determining the exact source(s) of the water in this area.

The following downstream overburden piezometers are discussed in more detail.

- **Piezometer 522.** This instrument is located in the overburden approximately 550 feet downstream at Station 102+54. There is no definitive relation with pool fluctuations. The average headloss is approximately 50 feet as compared to the reservoir. See Plate No. C-32, Appendix C for a detailed plot.

- **Piezometer 558.** This instrument is located in the overburden approximately 725 feet downstream at Station 102+54, which is downstream of the toe drain. There is no definitive relationship with pool fluctuations. The average headloss is approximately 45 feet as compared to the reservoir. See Plate No. C-33, Appendix C for a detailed plot.

- **Piezometer 555.** This instrument is located in the overburden approximately 625 feet downstream at Station 85+50, which is downstream of the toe drain. There is no definitive relationship with pool fluctuations. There is; however, a very slight downward trend in the piezometric level. The average headloss is approximately 60 feet as compared to the reservoir. See Plate No. C-34, Appendix C for a detailed plot.

- **Piezometer 562.** Piezometer 562 is located 443 feet downstream at Station 102+92. This instrument was abandoned in 1980.

- **Piezometer 563.** Piezometer 563 is located approximately 350 feet downstream at Station 90+00. This instrument has been historically dry. See Plate No.C-35, Appendix C, for a detailed plot.

A review of the toe drain data revealed that the drain has always been dry indicating evidence of a positive cutoff; however, higher reservoir elevations have not been experienced to assess the overall effectiveness of the cutoff.

Based on a review of the overburden piezometeric data, seepage within the overburden would not pose a concern under the proposed reallocation.

9.0 Movement Review

9.1 General. A settlement analysis was performed (during design) on the embankment impervious core and impervious cutoff trench at station 95+00 where the maximum height of impervious core and cutoff trench was attained. Since the surface foundation clays beneath the embankment were removed, it was determined that settlement of the pervious foundation would take place during construction and that the critical materials for determining residual settlement or settlement after completion of construction would be the embankment core and cutoff trench materials.

Consolidation tests of the impervious cutoff trench material were used in the settlement analyses. The total height of material analyzed was 187 feet. The total settlement was computed to be 5.6 feet of which 3.2 feet (56%) would occur during construction and 2.4 feet (44%) would occur after completion of the embankment. Only 9 inches of overbuild were actually provided for residual settlement of the embankment. This was an arbitrary decision which recognized that a gravel road on top of the dam would provide additional buildup of the crest and that there usually is some conservancy in the computations and procedures of analyses, due to inaccuracies of testing and differences in rate of load application to that assumed.

9.2 Instrumentation Analysis. Typical movement data (survey) is presented in Appendix D for information. Plate Nos. D-1 through D-4 presents vertical movement of crest, slope and toe movement markers. Vertical movement plots of the crest movement markers indicate a maximum consolidation of the embankment and cutoff trench of approximately 0.75 feet (Point C-15) from November 1975 to 2007.

Horizontal movement data for the crest, slope and toe markers is presented on Plate Nos. D-5 through D-9. Up to approximately 0.2 feet of horizontal movement has been observed with the exception of Points S-1 and S-2 as of the 2007 survey. Point S-1 has experienced approximately 0.30 feet of movement in the south direction while Point S-2 has experienced approximately 0.7 feet of horizontal movement also in the south direction. These points are located on the downstream (north) side of the overbuild section. Movement of these points to the south would not be consistent with their locations. Point S-1 has exhibited erratic data from the early 1980's. Possibly these points have been disturbed/damaged.

Foundation settlement data at Stations 70+00 and 90+00 is presented on Plate Nos. D-10 and D-11. Foundation settlement from 1971 to 2007 was measured at approximately 2.5 feet with the majority (1.5 feet) occurring between 1971 and 1975. Currently, the crest markers are exhibiting a movement rate of approximately 0.01 feet per year while the settlement gage movement was essentially zero from 2001 to 2003. Settlement since 1980 has been approximately 0.5 feet or 0.02 feet per year. Current (2007) top of dam centerline surveys indicates a minimum elevation of 5526.729 at Station 65+00 as shown on Plate No. D-12. The design top of dam elevation is elevation 5527 feet m.s.l. This results in a low area approximately 3.25" below the design elevation.

Survey movement data of the outlet works including the intake structure bridge is presented on Plate Nos. D-13 through D-16. As can be observed, no excessive or out of trend data has been identified.

There are 12 slope inclinometers (tiltmeters) located at Chatfield Dam as shown on Plate No. D-17. An inclinometer is a metal tube approximately 3 inches in diameter that is placed in a drill hole through the embankment/abutment/foundation. The purpose of this instrument is to measure active subsurface horizontal movement at various depths and identify areas of active instability or potential instability. Inclinometer data is presented for information in Appendix D, Plate Nos. D-18 thru D-31. A review of the inclinometer data indicates no active movement zones and no apparent movement related to pool thrust. Inclinometer 496 located at Station 104+54, Range167 D/S (Plate D-23) indicates what could be perceived as "movement zone" in the east direction. A review of the cumulative deviation plot (shape of pipe) on Plate D-23A revealed a sharp bend at this location resulting in poor quality data. This bend probably occurred during installation and/or settlement of the pipe. Inclinometer 545 located at Station 81+10, Range 17 D/S (Plate D-29) displays somewhat erratic data. The data does not appear to be related to movement of the embankment/foundation. The cumulative deviation (shape of pipe) plot for Inclinometer 545, presented on Plate No. D-30, indicates considerable deviation (approximately 6.5 feet) of the pipe from vertical. This normally affects the quality of the readings because just a slight change in depth produces a big change in the slope of the pipe. The checksums presented on Plate D-31 indicates relatively poor data below the depth of approximately 140 feet. This corresponds to the substantial change in the deviation of the pipe as shown on Plate D-30.

Based on a review of survey and inclinometer data, movement (settlement/instability) would not pose a concern under the proposed re-allocation.

10.0 Structural Evaluation.

10.1 General. The purpose of the structural evaluation was to determine if it is feasible to raise the normal and maximum pool elevations at Chatfield Dam without requiring modification to the existing outlet works structure and/or spillway structure. Pool elevations used in the original design calculations were compared with the proposed new pool elevations and the outlet works structure was evaluated to project how these new pool elevations may affect the existing structures. A brief description of the structures located at Chatfield Dam is presented in Paragraph 3.0.

10.2 Critical Structures. Critical structures are defined as those who's failure during or immediately after an earthquake could result in loss of life. The ability to lower the reservoir pool following an earthquake may be required to relieve pressure head on a damaged embankment or to inspect and repair the embankment in order to prevent loss of pool.

The existing guidance available on whether the intake structure should be classified as critical or not states that the intake structure is considered critical if any of the following four scenarios is likely to occur. (I) The intake structure can no longer discharge water and the embankment is damaged, but not breached right away leading to long-term use of the spillway. This could cause severe erosion and failure of the spillway, the abutment, or the embankment dam. (II) The embankment does not fail right away but is damaged such that the hydraulic capacity of the spillway is reduced because of the need to restrict the pool level to prevent overtopping. In this scenario, the outlet works must be used to prevent overtopping the dam for the design flood event. (III) The outlet works must be used to draw down the pool to a level below the spillway crest to prevent piping/internal erosion in the embankment. This assumes that the intake structure is capable of draw down rates in the range of 3 to 5 feet per day. (IV) Failure of the intake structure could cause the outlet conduit to rupture leading to piping and eventual failure of the embankment. Determination of a critical/non-critical rating will be pursued as part of the Seismic Phase I Study.

10.3 Study Data.

10.3.1 Intake Structure. The intake structure is the primary outlet structure at Chatfield Dam. In the event of a large flood event, it is imperative that the intake structure sustains functionality; however, this may not be the case depending on the results of the Phase I Seismic Evaluation. In the original design, the stability of the intake structure was designed for 4 different cases as shown below:

Case I. Construction Condition.

Reservoir Empty
Dead Load of Structure (Including Bridge Reaction)
Earth Loads
Wind Load (In Direction to produce most severe foundation pressures)

Case II. Normal Operating Condition.

Reservoir at elevation 5426 (Conservation Pool)
All Gates Open
Dead Load of Structure
Earth Loads
Wind Loads
Full Uplift on Base

Case III. Full Flood Condition.

Reservoir at Elevation 5500 (Crest of Ungated Spillway)
Emergency Gates Closed
Dead Load of Structure
Earth Loads
Wind Load
Full Uplift on Base

Case IV. Maximum Flood Condition.

Reservoir at Elevation 5521.6 (Max Reservoir Elevation)
Emergency Gates Closed
Dead Load of Structure
Earth Loads

Wind Load
Full Uplift on base

Note: In addition to these load conditions. Case IA, IIA, and IIIA, as presented on Plate E-1, were analyzed in the same way, except with an earthquake load of 0.10g substituted for the wind load.

The original stability calculations for Cases I through IV as well as IA through IIIA can be seen on attached Plate No. E-1 located in Appendix E. For the purposes of this study, Case I, the construction condition, does not apply and does not need to be looked at for the new pool volumes. Also, Case III, the Full Flood Condition, remains the same as it was in the original design of the intake structure. Case I originally yielded a shear-friction sliding safety factor of 7.56 (or 6.36 for earthquake), and Case III yielded a safety factor of 13.06 (or 6.33 for Seismic). Both cases yielded high safety factors which provide assurance that the structure would perform well in a large flood event.

Cases II and IV are the most important calculations to investigate to check if raising the normal pool elevation at Chatfield Dam is a feasible option. Case II designates a reservoir pool elevation at 5426.0 feet m.s.l. In the original design, Case II calculations yielded a shear-friction sliding safety factor of 13.64 under normal operating conditions and a safety factor of 7.73 when seismic loading was considered. The structural stability of the intake structure is marginally more stable under the Case II loading conditions with the new pool elevation of 5444.0 feet m.s.l. The increased stability of the intake structure can be attributed to the increase in vertical water loads on the intake structure when raising the pool versus slightly increased horizontal loads improving the stability of the structure. This case will be looked at as a part of the Phase I Seismic Evaluation.

Case IV specifies the reservoir elevation at max elevation of 5521.6 ft. In the original design calculations, Case IV yielded a sliding stability safety factor of 13.0. Case IV causes no major concerns for the Re-Allocation; however Case IV will be reviewed in the seismic review since seismic forces were not originally considered in this case.

Due to the possibility of high hydrostatic head, the intake structure was originally designed with walls and components with adequate internal strength to satisfy the allowable shear and moment requirements. All structures were designed with enough steel reinforcement to meet minimum temperature and shrinkage requirements. Also, the working stresses for Cases I and IV and all cases with earthquake loads were increased by 1/3rd for conservatism. In the most recent Periodic Inspection Report (2008), it was noted that the intake structure's concrete surfaces were in excellent condition with only minor shrinkage and map cracking which ensures that the structure should function as designed.

The bulkheads, gates, and valves in the intake structure were all originally designed to handle max flood conditions as well as the max pool of 5521.6 feet MSL. In fact, the design of the intake structure was based on the loading cases used for stability or a combination of loadings which produced the most severe stresses. These structures are expected to perform just as well as they currently do in a max pool event.

The oblong conduit was originally chosen over circular conduits because it was much more economical to build and was comparable to the circular conduit in strength. This conduit also was designed for the most severe loading conditions and will not be affected by the change in pool elevations entailed in this study.

The stilling basin was originally designed for a worst case scenario flood discharge condition of 8150 c.f.s. which is equivalent to the max discharge possible to pass through the outlet works. This structure also should not be of any concern with the newly proposed pool elevations.

The intake structure service bridge was designed for the maximum pool loading condition. In addition to these loading conditions the structure was designed to bear the AASHTO H-20 live load or a 25-ton mobile truck crane (37.5 tons total load), plus a low boy parked simultaneously on the same span as the crane. The service bridge is not a concern under the original loading criteria with a new normal pool elevation of 5444 Ft. MSL.

10.3.2 Spillway Structure. The spillway structure at Chatfield Dam was originally designed for similar loading conditions as the outlet works structures. The original maximum flood that the spillway was designed for is the only condition that would change due to raising the pool elevation according to this study. However, similar to the outlet works structures, there are no major concerns regarding the structural integrity of the spillway structures.

11.0 Conclusions. No immediate dam safety concerns have been identified based on either the existing normal reservoir elevation of 5432 feet m.s.l. or on a projected reservoir elevation of 5444 feet m.s.l considering static loading.

No indications of instability have been identified either by field inspections or by the instrumentation program after approximately 35 years of service.

Based on fluctuation of piezometer data as discussed hereinbefore, there is the potential concern for development of excess pressures in the foundation (Dawson Formation) at the reservoir elevation of 5500 feet m.s.l (spillway crest pool). Based on piezometer projections, the piezometric levels in both Piezometer 504A and 561 may exceed the levels used in the original slope stability analyses for pool elevation 5500 feet m.s.l. This does not pose a concern for the re-allocation project since the spillway pool elevation does not change as a result of the re-allocation project. This concern should be evaluated as a part of the routine dam safety program.

Due to the relatively responsive nature of some of the piezometers located in the core of the embankment, close monitoring of these instruments would be warranted during a reservoir level approaching the spillway crest (El. 5500 Ft.). This does not pose a concern for the re-allocation project since the spillway pool elevation does not change as a result of the re-allocation project. This concern should be evaluated as a part of the routine dam safety program.

No evidence of seepage concerns have been identified since construction of the project (35 years). All piezometric levels are considered to be within normal trend and range. The downstream toe drain has always remained dry. Minimal fluctuation except for a general overall decline in pressure has been observed in the bedrock piezometers located near the downstream toe (Piezometers 560, 557, 564, and 522). The most responsive downstream bedrock instrument is Piezometer 561. Piezometer 561 is an open tube devices located at Station 93+00, approximately 300 feet downstream. The piezometric level exhibited by this device initially reflected pore pressure during construction and then was affected by the reservoir impoundment. The piezometric level in this instrument has decreased approximately 15 feet since impoundment of the reservoir; however, small spikes in the piezometric level can be seen during periods of increased reservoir level (1979). This instrument has fluctuated approximately 2 feet over the past 10 years during “normal” fluctuations of the reservoir (approximately 10 feet). Currently the piezometric level in this instrument is at the approximate elevation of the bottom of the blanket drain.

All of the structures at the site have been designed to withstand the small increase in loading caused by the proposed pool elevations. In addition to this, the most recent periodic inspection report (2008) found these structures to be in very good condition which provides confidence that these structures are still in a condition to function as designed. A review of the instrumentation data (piezometers, inclinometers and survey points) did not reveal a relationship between movement (potential instability) and thrust of the pool; however, the maximum pool elevation to date has been approximately 5447.58 and this was for a relatively short time period.

The primary concern for the structures at Chatfield Dam stems from the most recent Seismic Safety Review which recommended a Phase I study.

Although continued monitoring will be required, raising the normal pool elevation by up to 12 feet should not have a direct bearing on the adequacy of the slope protection material; however, the slope protection material will continue to be monitored during routine dam safety inspections (monthly, annual, periodic,…). In addition to this, effort will be made to inspect existing areas of riprap displacement during low reservoir elevations.

Based on a review of design assumptions, instrumentation data and performance since completion of Chatfield Dam, it is concluded that the new “normal” pool elevation (El. 5444) proposed in this reallocation study will not adversely impact the integrity of the embankment or structures. It is emphasized that this conclusion is based strictly on static loading scenarios. Although no dam safety concerns have been identified for the proposed reservoir loading, based on project performance and the instrumentation program, increased monitoring of the project would be pursued as part of the routine dam safety program to assure continued safe operation of the dam. This would include the development and implementation of a Reservoir Raise Monitoring Plan which would include additional inspection efforts, instrumentation data acquisition and data analysis. The Project Surveillance Plan and Emergency Action Plan should also be updated as appropriate.

Installation of additional instrumentation prior to the pool raise along with an increase in instrumentation readings and inspection frequencies during and following the pool raise would be warranted.

12.0 Recommendations. Specific recommendations are as follows:

- Add new piezometers in the overburden near the outlet works at Station 105+00.
- Add new piezometers in the downstream drain at Stations 69+00, 81+00, 93+00 and 102+00 to establish elevation and depth of the drain and to monitor effectiveness of the drain.
- Add new piezometers in the downstream fill at Stations 69+00, 81+00, 93+00 and 102+00 to monitor an historic “wet area”.
- Add new piezometers in the downstream Dawson Formation at Stations 69+00, 81+00, 93+00 and 102+00 to monitor potential excessive pressures.
- Develop a reservoir raise monitoring plan addressing frequencies of inspection, instrumentation data acquisition including the toe drain and data analysis.
- Revise the Emergency Action Plan and Surveillance Plan as appropriate.
- Maintain routine (monthly, Annual, Periodic) inspection of the upstream slope protection. Effort should be made to inspect existing areas of riprap displacement during low reservoir elevations.

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Meeting of Board of Consultants, Chatfield Dam, 16 Nov 1971

ACRONYMS

approx.:	approximate
cfs:	cubic feet per second
Cfs:	Cubic feet per second
COE:	Corps of Engineers
D/S:	Downstream
Elev. (el):	Elevation
ER:	Engineering Regulation
Ft, FT, ft:	feet
H	Horizontal
in.:	inch
lb/cft:	pounds per cubic foot
max.:	maximum
min.:	minimum
msl	mean sea level
N:	North
Nos.:	Numbers
P:	Piezometer
psf:	pounds per square foot
psi:	pounds per square inch
Pz:	Piezometer
rt	right
SC:	Slope and Crest Movement Marker
SG:	Settlement Gage
sq in.:	square inch
Sta.:	station
T:	Inclinometer designation
U/S:	Upstream
USACE:	United States Army Corps of Engineers
V:	Vertical
W:	West
wt:	weight
yrs:	years