MILLSITE DAM REHABILITATION

80% DRAFT DESIGN REPORT

November 2015

Prepared for
Ferron Canal and Reservoir Company
United States Department of Agriculture: Natural Resources Conservation Service

Prepared by
Utah Division of Water Resources
MILLSITE DAM REHABILITATION

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List of Acronyms

ASDSO – Association of State Dam Safety Officials
BPT – Becker Penetration Test
DWRe – Utah Division of Water Resources
DWRi – Utah Division of Water Rights
F.S. – Factor of Safety
FCE – Franson Civil Engineers
FCR – Ferron Canal and Reservoir Company
GCI – Gerhart Cole Inc.
IDF – Inflow Design Flood
MCE – Maximum Credible Earthquake
MSL – Mean Sea Level
NRCS – Natural Resources Conservation Service
NWS – Normal Water Surface
PGA – Peak Ground Acceleration
PMF – Probably Maximum Flood
PMP – Probable Maximum Precipitation
SCS – Soil Conservation Service
SEF – Spillway Evaluation Flood
SPT – Standard Penetration Test
USBR – United States Bureau of Reclamation
USDA – United States Department of Agriculture
USU – Utah State University
Millsite Dam Rehabilitation

1. SUMMARY

The following is a summary of the information included in this design report for Millsite Dam Rehabilitation:

1. General: Millsite Dam is owned and operated by the Ferron Canal and Reservoir Company. The dam is located about 3 miles east of Ferron City on Ferron Creek, in Emery County, State of Utah. The dam is a zoned embankment dam that stands 115 feet high, has a crest length of 4,150 feet, and has a current reservoir storage capacity of 16,230 acre-feet of water at the normal high water elevation. The spillway is a 50-foot long by 60-foot wide duckbill reinforced concrete structure with a discharge chutes. The outlet works consist of a 54-inch main conveyance pipe and an 8-inch culinary pipeline that are enclosed in a reinforced concrete tunnel through the embankment.

2. Study History: Millsite Dam was designed by the Soil Conservation Service (SCS), now the Natural Resources Conservation Service (NRCS), in the late 1960s with construction completed in 1971. All design requirements and standards were met at that time. Design standards have changed since original construction, and preliminary evaluations indicated that modifications may be need to bring Millsite Dam into compliance with current NRCS and Utah State Division of Water Rights (DWRi), Dam Safety dam safety standards. The investigation and upgrade design are a cooperative effort between NRCS and the Utah Division of Water Resources (DWRi).

3. Geology: Millsite Dam is founded on alluvial sands, gravels, and cobble along the center portions of the dam, with the abutments being founded on shale and sandstone bedrock. These alluvial deposits are underlain by bedrock at depths of up to approximately 80 feet. Subsurface exploratory drilling was accomplished prior to original design and construction. Additional test holes and test pits were completed as part of this study associated with the present rehabilitation work.

4. Seismotectonic: Seismic evaluation were conducted to develop a Maximum Credible Earthquake (MCE) of 6.9 for the dam site. The peak ground acceleration (PGA) to be used for site evaluations was determined to be 0.39g in locations with deep alluvium deposits and 0.31g for areas directly on bedrock.
5. Seepage: Piezometric data collected from 12 piezometers indicate that the embankment downstream of the Zone I core is mostly dry with water present in the foundation. The water levels in the piezometers indicate that the phreatic water surface is generally below the internal drain system. This is most likely due to the effectiveness of the Zone I core, upstream slurry trench, upstream cutoff, and high permeability of the foundation materials. Due to the fact that the reservoir is only full for a few weeks during spring runoff before the reservoir begins to be lowered to supply irrigation water, the seepage observed has likely not established a steady-state condition, and the measured piezometers represent the annual fluctuations of the reservoir. For this study, a steady-state seepage analysis was completed and used in the slope stability and deformation analyses.

6. Liquefaction: Materials that might be susceptible to liquefaction were encountered during the recent subsurface studies in some foundation materials. The blow count data from the subsurface investigation was evaluated and zones of potentially liquefiable material were identified. These materials were generally found in the upper zone of the alluvial foundation with infrequent pockets of potentially liquefiable materials at deeper depths.

7. Slope Stability and Deformation: Soil strengths were defined for the embankment zones and foundation materials to complete slope stability analyses. Using steady state seepage conditions, both static and post-earthquake slope stabilities were performed on three separate cross sections. The stability analysis results were then incorporated into a deformation analysis to evaluate how much deformation would be likely to occur at the dam under seismic conditions. Results indicate that the proposed rehabilitation will meet slope stability and deformation minimum dam safety standards.

8. Internal Stability: Due to the coarse and broadly graded nature of the embankment, particularly Zone III drain materials; and considering current filter criteria, concern exists about the internal stability of these materials. Based on a review of all the available data, the existing zoned embankment does not meet all internal stability conditions.

9. Transverse Cracking: Due to the potential of transverse cracking of the crest during earthquake loading, a chimney drain is recommended for the downstream slope. The chimney drain will transition into a blanket drain with a collection pipe along the dam toe. The chimney drain will extend the entire length of the embankment.

10. Instrumentation: The existing piezometers will be extended to the new crest height. Additional piezometers will be installed to assist in evaluating the performance of the embankment and foundation. Survey monuments will be installed along the crest to replace
the existing monuments that will be removed with the proposed work. In addition, a new staff gauge will be installed to assist in field verification of the reservoir water level.

11. Sedimentation: A large portion of the reservoir capacity has been lost due to sedimentation in the reservoir. As part of the rehabilitation work, the dam crest will be raised to restore the original reservoir capacity.

12. Outlet Works: The existing outlet works will need to be extended as a result of the proposed work. This will include extending the tunnel, pipes, and other related utilities. New control valves will be installed at the downstream end of the outlet pipes.

13. Hydrology: Several probable maximum floods (PMF) were analyzed for the Millsite water shed area. The governing probable maximum precipitation (PMP) that created the largest inflow design flood (IDF) resulted in a maximum inflow of approximately 31,000 cfs. The existing spillway is undersized to route the critical PMF, and a new spillway will be required.

14. Spillway: The new spillway will consist of a reinforced concrete labyrinth spillway with a discharge chute similar to the existing chute. The labyrinth spillway will include three apexes that protrude into the reservoir with a height of 19.5 feet. The discharge chute will have a flip bucket on the downstream end to discharge flows away from the bedrock face on the downstream end of the spillway chute.

15. Cost Estimate: The estimated construction cost of the dam safety upgrade project is approximately $______________.
2. GENERAL

2.1 Authority

The Natural Resources Conservation Services (NRCS) Utah State Office has requested the services of the Utah Division of Water Resources (DWRe) in a cooperative technical effort to investigate and evaluate Millsite Dam and provide recommendations to rehabilitate the facility. Rehabilitation work will need to meet NRCS TR-60 (2005) and State of Utah Division of Water Rights, Dam Safety (2012) minimum standards. This will include providing engineering services for the preparation of a design report, plans, and specifications for the Millsite Dam Rehabilitation.

Contract Number XXXXXX, originally signed on MONTH DAY, 199X authorizes work to be accomplished. The description of work specifically calls for “Cooperative Agreement for Engineering and Planning for the Millsite Dam Rehabilitation Project (No.XXXXX)”. A number of amendments to the contract have been implemented through this process, primarily due to additional delays and costs.

This report presents the results of the investigation and engineering evaluation, and recommends remedial measures to rehabilitate the existing dam to meet required dam safety standards.

2.2 Plan EA Purpose and Need for Action

The purpose of this project is to rehabilitate the Millsite Dam (#UT00212) to meet current NRCS (2005) and DWRi Dam Safety regulations (2012) and current engineering standards.

The need for the project is to extend the life of the dam and to continue providing economic benefit through the primary use of water storage with incidental benefits to flood damage reduction, sediment retention, and recreation. The project would accomplish the following needs:

- Increase the stability of the dam by installing an improved internal drain system and stabilizing the downstream slope with a flattened slope and downstream berm
- Lengthen the outlet works as a result of the downstream slope improvements
- Construct a new spillway to pass the design flood event.
- Restore the original storage capacity in the reservoir by raising the dam crest and normal high water level
2.3 Project Description

2.3.1 General Notes

2.3.1.1 Geologic Report

As part of the rehabilitation design, an extensive geologic report was prepared by DWRe entitled Final Geologic Evaluation: Millsite Dam, Emery County, Utah (2015). The geologic report includes a large amount of general project description, geotechnical data, and other references that are referenced throughout this design report. Referenced documents in this design report will be included in the appendix unless they are already included in the appendix of the geologic report or unless they are published documents that are readily available.

2.3.1.2 Dam Crest Stationing

During review of the geologic report, it was discovered that the stationing along the dam crest that has been used during the rehabilitation design was not set to match the stationing of the original design documents. The difficulty in trying to make the stationing match the original design documents is that several recent documents have been prepared based on the stationing established for the geologic report and other preliminary design analyses. Because so many of the reports have already been finalized based on the recent stationing, it will be used as the stationing basis for all final design documents.

The difference between the current stationing and the original dam stationing is approximately 70 feet, with the current stationing being 70 feet greater than the original. For example, Station 10+00 in the original design will correlate to Station 10+70 in the current rehabilitation design.

2.3.1.3 Site Elevations

The original design was completed in the late 1960s. The original design survey was based on relative elevations. The proposed rehabilitation drawings and final report use an actual mean seal level (MSL) elevation datum. The correction factor between the original relative and MSL elevation is 5,160 feet. For example, adding 5,160 feet to the original spillway elevation of 1,055 feet will produce the actual MSL elevation of 6,215 feet.

2.3.1.4 Design Specifications and Drawings

The design specifications are included as Attachment 1 to this design report and the design drawings are included as Attachment 2. In an effort to reduce potential errors from duplication of drawings and details, the design drawings are the figures of reference for any features in the proposed rehabilitation. Any items described in this report related to the proposed rehabilitation are shown in the design drawings.
2.3.2 Background Information

Millsite Dam and Reservoir are owned and operated by FCR. The dam and reservoir are located about 3 miles east of Ferron City, as shown in the design drawings. The dam and reservoir were built in the Ferron Creek drainage basin, which is a tributary to the San Rafael River. The stored water is used by the following main water users:

- FCR is the primary water user of water from the reservoir. FCR uses most of the water for irrigation in and around Ferron City. FCR also delivers stock water to its users throughout the year.
- PacifiCorp receives water from the reservoir for use at the Hunter Power Plant in Castle Dale, Utah.
- Castle Valley Water Conservancy District delivers water from the reservoir to Ferron City for culinary use after treatment. Millsite Reservoir is the only source for culinary water in Ferron City.
- Millsite Golf Course receives water for sprinkler irrigation of an 18-hole golf course located adjacent to the dam and reservoir
- Emery County Water Conservancy District assists in the overall water use and water distribution throughout the county.
- Millsite State Park is located at the southeast end of the reservoir. The park includes a boat ramp and several campsites with facilities.

The dam was originally designed to serve irrigation regulation purposes, but it also provides incidental benefits of flood prevention, sediment retention, and recreation. The hydrology study completed by NRCS for Millsite Dam also included a sedimentation study of the reservoir that showed how much of the reservoir capacity has been lost from sedimentation. As a result of the sediment accumulation, Millsite Reservoir is no longer capable of serving its initial irrigation retention purposes without raising the water level and restoring the irrigation storage rights.

2.3.3 Previous Studies and Reports

The following is a list of previous studies and reports that are discussed in the geology report, which are incorporated into the design information in this design report:

1) Millsite Geology Report, USDA NRCS, 1967a (Appendix 2a of geologic report)
2) Preliminary Design Report, USDA NRCS, 1967b (Appendix 2b of geologic report)
3) Supplement No. 1 to Preliminary Design Report, USDA NRCS, 1967c (Appendix 2b of geologic report)
4) Millsite Geology Report, USDA NRCS, 1968 (Appendix 3a of geologic report)
5) *Design Report*, Department of Agriculture Soil Conservation Service (USDA SCS), 1969a (Appendix 3b of geologic report)
6) Reports and comments from Dr. James Sherard regarding original dam design and construction, 1968-70 (Appendix 3c of geologic report)
7) As-Built Set of Drawings, USDA SCS, 1971a (Appendix 3d of geologic report)
8) *Impervious Cutoff by Means of Slurry Trench Construction*, USDA SCS, 1972 (Appendix 3e of geologic report)
9) *Millsite Dam Phase II Report*, RB&G Engineering Inc., 2006 (Appendix 8 of geologic report)
10) Several NRCS trip reports related to Millsite Dam, USDA NRCS, 1971, 2007, and 2008 (Appendix 9 of geologic report)
11) Information on test holes completed in 2007, Bureau of Reclamation, 2007 (Appendix 10a, 10b, and 10c of geologic report)
12) Information on test holes completed in 2009 and 2010, DWRe, 2009 and 2010 (Appendix 11a and 11b of geologic report)
13) Information on Becker hammer test holes completed in 2009, DWRe, 2009 (Appendix 12 of geologic report)
15) Geotechnical laboratory summary information, various, (Appendix 14, 15, and 16a through 16e of geologic report)
16) Piezometer information (Appendix 18a, 18b, 19a, and 19b of geologic report)
17) *Seismic Hazard Evaluation: Millsite Dam, Emery County, Utah*, USDA NRCS, 2013
18) *Site-Specific Probabilistic Seismic Hazard Analysis of Millsite Dam, Utah*, URS, 2013 (included in appendix of NRCS Seismic Hazard Evaluation report)
19) Millsite Dam – Seismic Ground Response (“SHAKE analysis”), GCI, 2013 (included in appendix of NRCS Seismic Hazard Evaluation report)
20) *Millsite Hydrology & Hydraulic Study: Millsite Watershed Rehabilitation Planning near Ferron, in Emery County, Utah*, NRCS, 2010 (Appendix A)

In addition to the reports in the geologic report, several other documents have been completed that impact the proposed rehabilitation design features. These documents include the following:

21) *Assessment Report, Ferron Creek Watershed, Millsite Dam, Emery County, Utah*, USDA NRCS, 2004 (Appendix B)
22) *Millsite Dam Evaluation Report*, USDA NRCS, 2006 (Appendix C)
23) Dive inspection of Millsite Dam intake tower, USBR, 2006 (Appendix D)
24) *Millsite Dam Seismic Assessment of Intake tower & Outlet Work, Ferron, Utah*, ABS Consulting, 2010 (Appendix E)
2.3.4 Existing Embankment and Hydraulic Structures

2.3.4.1 Embankment Configuration and Zoning

Millsite Dam is a zoned embankment dam with a dam crest at elevation 6222.5 feet and a top crest width of approximately 26 feet wide along the main dam embankment. The maximum height from the crest to the downstream toe is approximately 115 feet. The crest is approximately 4,150 feet long. The dam includes a sloping Zone I core and two shell zones (Zone II and Zone III) both upstream and downstream of the core (see original construction drawings in the geologic report). An internal drain was also constructed using Zone III materials. The upstream and downstream faces of the dam have a 1.75:1 slope and are lined with Zone III materials for slope protection and wave protection. (All slopes are given as horizontal to vertical, H:V, unless stated otherwise.) The upstream slope also includes a 60-foot wide by 25-foot high berm to elevation 6,130 feet at the upstream toe.

Section V of the geologic report gives a detailed description and evaluation of the foundation materials and the different zoned materials in the existing dam. The materials are summarized as follows:

1) Alluvial Foundation Materials – The existing foundation alluvial materials are sands, gravels, and cobbles with some fines, which are generally silts and clayey-silts.
2) Foundation Bedrock – The main formation at the dam foundation is the Cretaceous Mancos Formation, which includes shale and sandstone layers.
3) Zone I (Impervious Core) – This material was processed by mixing finer grained materials with borrow area materials. The average gradation is 39 percent gravels, 31 percent sands, and 30 percent fines.
4) Zone II (Outer Shell) – This material came from the nearby borrow areas and is coarser than Zone I materials with an average gradation of 47 percent gravels, 32 percent sands, and 21 percent fines.
5) Zone III (Filter/Drain and Riprap) – As Zone I and Zone II materials were screened, the remaining oversize materials were used for Zone III. Materials consist of cobbles and boulders with some gravel.
2.3.4.2 Slurry Trench and Cutoff Trench

A slurry trench and cutoff trench were included in the original construction in order to cutoff seepage flows through the alluvial foundation and highly weathered bedrock. The slurry trench was constructed across the alluvial channel approximately 220 feet upstream of the dam centerline. It was backfilled with bentonite slurry materials across the foundation section that was not excavated to bedrock. The cutoff trench was excavated through the highly weathered bedrock and was backfilled with Zone I core materials.

2.3.4.3 Toe Drain

A toe drain was constructed sometime within the first few years of operation because of seepage along the left abutment. No documentation has been able to be found on the design or construction of this toe drain. Based on information that FCR personnel have been able to get from local residents, the toe drain extends from the lower portion of the left groin and extends to the outlet works. The toe drain includes a perforated 4-inch collection pipe. It is thought that seepage along the abutment may have been from joints in the bedrock, but no documentation has been found to verify this. Since the drain was put in, there have not been any observations of high seepage at the left abutment.

2.3.4.4 Spillway

The spillway is located at the far right end of the dam and has a crest elevation of 6215.0 feet. The spillway is a 50-foot long by 60-foot wide duckbill reinforced concrete structure with a reinforced concrete discharge chute that is approximately 250 feet long. There is a flip bucket at the downstream end of the chute. Water is discharged from the spillway chute as it goes over the flip bucket, which is designed to project the spillway flows away from the bedrock cliff at the end of the chute. The water then falls approximately 70 feet into a stilling basin.

The embankment section at the spillway is referred to as the right abutment dike in original design documents. The spillway dike layout is different than the main embankment because it has a 3:1 upstream slope and 2:1 downstream slope. It also does not include any internal drain zones and the upstream slope protection is gravel rather than Zone III materials.

2.3.4.5 Outlet Works

The outlet works is located at approximately Station 17+70 of the dam crest. It consists of a 54-inch main conveyance pipe and an 8-inch culinary pipeline that are enclosed in a reinforced concrete tunnel through the embankment. Both pipes are mortar-lined steel pipe. The 54-inch outlet pipe has a butterfly valve on the upstream end for emergency shutoff and a Howell-Bunger valve on the downstream end to control releases. The 8-inch pipe has a gate valve on the upstream end for emergency shutoff and ties into the culinary delivery system, so there is not any
The outlet pipe discharges into a riprap lined plunge pool that is the start of FCR’s delivery through its system of canals.

Over the years, different connections have been made at the downstream end to facilitate delivery to the water users. This includes a 24-inch pipe to PacifiCorp, a 12-inch pipe to Ferron City, two 36-inch pipes to FCR’s pressurized irrigation system, and a 12-inch pipe for stock water. Franson Civil Engineers (FCE) prepared a design report for the outlet works that includes a description of all these additional connections. The report is included Appendix I.

2.4 Design Objectives

Millsite Dam was designed in the late 1960s with construction completed in 1971. All design requirements and standards were met at that time. Design standards have changed since original construction, and preliminary evaluations indicated that modifications may be need to bring Millsite Dam into compliance with current NRCS and DWRi dam safety standards. The investigation and upgrade design are a cooperative effort between NRCS and the Utah Division of Water Resources (DWRe).

The dam includes a number of technical deficiencies in meeting today’s design standards and also has other needs and maintenance issues, as described in the following list. The basis for determining and evaluating these deficiencies will be described throughout the design report.

2.4.1 Minimum Standards Deficiencies

1) The downstream slope does not meet minimum stability standards because of potentially liquefiable materials in the foundation.
2) The internal drain system does not meet filter criteria standards.
3) There is the potential of transverse cracking in the embankment during earthquake loading.
4) The outlet works will need to be extended downstream as a result of work needed to address the deficiencies listed above.
5) The existing upstream guard gate uses electric controls rather than hydraulic controls.
6) The existing spillway does not have the capacity to pass the design flood event.

2.4.2 Maintenance Issues

1) The reservoir capacity has decreased due to sedimentation.
2) The outlet works currently consists of several vaults that are not efficiently laid out.
3) Millsite State Park will require maintenance work to offset impacts to the park as a result of the raised reservoir water level.
4) Millsite Golf Course will need work to restore golf holes that will be impacted from the rehabilitation work.
5) The upstream riprap requires constant maintenance because of the steep upstream slope.

2.5 Methodology

In evaluating Millsite Dam, Utah Dam Safety (2012) requires the following:

1) Hydrologic Design: Design a spillway inlet, spillway chute, and stilling basin to pass the Spillway Evaluation Flood (SEF) and appropriate routing through the reservoir with adequate freeboard. Design the outlet works to meet existing hydrologic capacities and evacuate 90 percent of the reservoir in 30 days or less.
2) Seismic Design: Evaluate previously completed geological and seismological investigations and reports. Conduct deterministic and probabilistic studies to identify PGA and MCE values. Evaluate field and laboratory investigation and reports of the dam and foundation materials. Determine dam and foundation material properties. Undertake appropriate analyses for seismic events to predict factors of safety against slope failures, liquefaction, deformation, and structural deformations.
3) Seepage Analysis: Conduct appropriate seepage analyses to evaluate the phreatic surface in the embankment and foundation.
4) Embankment Requirements: All dams should meet required slope stability factors of safety for construction, steady state, instantaneous drawdown and post-earthquake conditions, as well as internal erosion control and appropriate internal drainage.
5) Spillway Requirements: The requirements include appropriate crest weir length, spillway chute alignment along the downstream dam and stability berm, suitable riprap armoring, spillway underdrain collection system, water tightness, concrete members and steel reinforcement designed to ACI 350 code, necessary log boom, and appropriate energy dissipation.
6) Outlet Requirements: Extend the existing outlet works as necessary to accommodate the downstream slope work. Replace the existing outlet discharge valve that is near the end of its life cycle.
7) Instrumentation: Install piezometers, seepage monitoring, staff gage, and needed survey monuments and benchmarks to assist in monitoring performance of the dam.

In evaluating Millsite Dam, NRCS design standards included in TR-60 (2005) require the following:

1) Hydrology: Comply with methods of flood routing hydrographs through reservoirs and spillways systems based on precipitation and runoff amounts from the National Weather Service (NWS) and National Engineering Manual (NEM).
2) Sedimentation: Allocate storage capacity for the calculated sediment accumulation during the design life of the reservoir.

3) Geologic Investigations: Evaluate previously completed geological, field, and laboratory investigations and seismological reports.

4) Seepage: Make analyses utilizing seepage rates and pressures through the embankment, foundation, and abutments.

5) Earth Embankments and Foundations: Design the earth embankment and its foundation to withstand anticipated loads, including all required slope stability conditions, seismic loads, and necessary provisions.

6) Principal Spillway (outlet) and Auxiliary Spillway: The structural design and detailing of principal and auxiliary spillway must conform to the NRCS standards, including capacity and routing for expected head and tail water conditions.

2.6 Proposed Rehabilitation Work

The following list includes the proposed design features that will address the current deficiencies of the dam and reservoir. These items will bring the dam and reservoir into compliance with current dam safety standards.

1) Loose sand and silt materials in the downstream foundation will be excavated and replaced with compacted materials to remove the top zone of potentially liquefiable materials and to help with stability concerns.

2) The downstream slope will be flattened and a berm placed at the downstream dam toe to improve stability of the dam.

3) As part of the downstream work, the downstream slope protection rock will be removed so that a chimney drain (filter and drain zones) can be constructed on the slope and in the foundation to collect seepage through the dam. The slope protection rock will then be replaced on the new downstream slope.

4) Pipes will be installed in conjunction with the drain materials to collect and monitor seepage through the dam.

5) Downstream work will necessitate the extension of the existing outlet works.

6) The existing concrete spillway will be removed and replaced with a new concrete labyrinth weir spillway with the capacity to pass the design flood event.

7) The dam crest will be raised four feet to restore the original reservoir capacity.

8) Piezometers, seepage flow weirs, survey monuments, and other necessary instrumentation will be installed to improve long-term monitoring of the dam.

9) Repairs will be made to Millsite State Park located near the reservoir, and Millsite Golf Course located downstream of the dam, to mitigate the construction impacts.
3. SITE SURVEY OF DAM AND RESERVOIR

3.1 Bathymetric Survey

NRCS completed a bathymetric survey of the reservoir in 2006. The reservoir was slightly above elevation 6200 feet when the survey was completed. This survey was used to estimate how much sedimentation had occurred at the reservoir. When this bathymetric survey was completed, several control points were also set at the project site. However, most of these control points have been destroyed or disturbed because they were set as temporary points and were not set deep into the soils.

3.2 Aerial Survey

A topographic survey was completed by Olympus Aerial Surveys in the fall of 2010. The reservoir level was at approximately elevation 6190 feet when the site was flown. Control points and survey targets were used to develop the site topography with 2-foot contour intervals. This information was imported into AutoCAD for use in design information.

3.3 Field Surveys

Several field surveys were completed by DWRe personnel during the design stages for the project. These field surveys were completed using a total station, with several control points or dam crest survey monuments being recorded in order to tie the surveys together. NRCS also completed a few field surveys using GPS survey equipment.

3.4 Combination of Surveys

As part of the design process, all of the surveys were combined and aligned to create a single surface for the dam site and reservoir basin. This process included adjusting some of the surveys so that they were all in the same coordinate system. Fortunately, the bathymetric and aerial surveys had several contours that overlapped. These contours were used to verify and adjust the contours so that the overlapping contours had similar surface areas within the reservoir. Appendix L includes a figure that shows the area correlations between the bathymetric and aerial survey. This figure shows that the overlapping 10 feet of contours in both surveys correlates well enough to indicate that the reservoir basin survey is accurate.
4. GEOLOGIC AND SEISMIC STUDIES

4.1 Geology

DWRe has prepared the geologic report which includes detailed descriptions of the site geology in Section II. The geologic setting for the general site area is described. The geology for the dam alignment is detailed, with information on the stratigraphy at and around the dam site. The geologic report includes regional and site geology, subsurface investigations, laboratory testing, piezometric levels, ground water conditions, foundation conditions, water quality, and geologic hazards. This information has been utilized in the design effort and report. See the referenced report for additional and detailed information.

4.2 Subsurface Investigation and Laboratory Testing

The geologic report also includes subsurface investigations and laboratory testing information in Sections III, IV, and V. These sections include work conducted by the NRCS prior to construction of the dam, subsurface investigation and laboratory testing conducted by RB&G Engineering in 1996, and subsurface investigations and laboratory testing conducted in cooperation with the NRCS and the DWRe in 2007 through 2015. See referenced report for additional and detailed information.

4.3 Seismotectonic Evaluation (NRCS)

NRCS prepared the Seismic Hazard Evaluation, Millsite Dam, Emery County, Utah (2013), including seismotectonic setting, historic earthquakes, potential seismic sources, deterministic ground motion, and probabilistic ground motion. Seismic evaluations were conducted to develop a Maximum Credible Earthquake (MCE) of 6.9 for the dam site. The peak ground acceleration (PGA) to be used for site evaluations was determined to be 0.39g in locations with deep alluvium deposits and 0.31g for areas directly on bedrock. See referenced report for additional and detailed information. The seismic report appendices include several other reports that were incorporated into the final analyses, such as a site-specific seismic probabilistic analysis prepared by URS and the resulting ground response prepared by GCI.
5. **FOUNDATION AND EMBANKMENT DESIGN**

5.1 **Embankment Zones**

There are several soil materials related to the original construction and proposed rehabilitation work for Millsite Dam. These material zones include both the embankment and foundation. The zones were identified based on original design drawings and geotechnical data that was collected from the original design, original construction, and geotechnical site investigations that were part of the rehabilitation design process. The geologic report includes a detailed description of the different site investigations that have occurred at the project site.

Appendix M includes a summary spreadsheet of geotechnical field and laboratory testing data from the drilled test holes at Millsite Dam. Following the summary spreadsheet are several plots of sieve analyses that were used to identify trends for the different embankment and foundation zones.

Based on the geotechnical data, the following zones and been identified for the existing conditions and proposed rehabilitation work. These zones are used throughout the different analyses.

5.1.1 **Existing Zones**

The following soil and bedrock descriptions are for the materials at the existing dam site.

**Zone I (Core)** – These materials are the core from the original construction. Materials were produced by mixing fine-graded materials with borrow area materials. The materials were intended to have a fines content of 25 to 40 percent. Based on 417 field tests results from construction, Zone I materials are 39 percent gravels, 31 percent sands, and 30 percent fines. The fines are generally clayey silts and silts.

**Zone II (Shell)** – These materials are the upstream and downstream shell from the original construction. Materials came from borrow areas within and near the reservoir basin. Based on 429 field tests results from construction, Zone I materials are 47 percent gravels, 32 percent sands, and 21 percent fines. The fines are generally clayey silts and silts.

**Zone III (Internal Drain and Riprap)** – These materials are the cobbles and boulders that resulted from the screening process to produce Zone I and Zone II materials. The materials were to have less than five percent minus five-inch materials, with a maximum particle size of three feet. Construction documents indicate that gradation adjustments may have been made at some point during the construction process to use the smaller sized materials for portions of the internal drain. However, there was not enough documentation to clearly identify which areas, if any, actually used a finer graded material.
**Foundation Sands** – These materials are generally the upper portion of the native alluvial deposits in the stream channel, which consists of loose gravelly sands with silt. According to the construction documents, these loose materials in the top section of the alluvial deposits were removed below the footprint of the original dam. These loose materials still exist in the stream channel both upstream and downstream of the existing dam embankment.

**Foundation Gravels** – These materials are dense alluvial deposits underlying the loose foundation sands. There is a high percentage of cobble and sand in these materials.

**Foundation Bedrock** – Shale and sandstone provide the foundation for the abutments of the dam, and also underlay the alluvial deposits in the stream channel. Exposed surfaces of the bedrock are very weak, but the materials become more competent with depth.

### 5.1.2 New Zones for Rehabilitation Work

The following are descriptions of the soil materials to be used for the proposed rehabilitation work at the dam. Gradations and other general soil parameters can be found in the G Set (General Drawings) of the design drawings, with more detailed information regarding the placement and compaction in the project specifications.

**Zone 1 (Core)** – These materials are intended to be equivalent to Zone I materials from the original construction. They will be used to extend the height of the dam core with the proposed crest raise.

**Zone 2 (Shell)** – These materials are intended to be equivalent to Zone II materials from the original construction. They will be used to extend the height of the dam shell with the proposed crest raise and to cover the new chimney drain with the flattened downstream slope.

**Zone 3 (Upstream Riprap)** – These materials are intended to be equivalent to coarse-graded Zone III materials from the original construction. They will be used to extend the upstream riprap to the top of the proposed crest raise. These materials will also be used to fill in any voids or uneven areas of the existing upstream riprap.

**Zone 4 (Downstream Riprap)** – These materials are intended to be equivalent to finer graded Zone III materials from the original construction. They will be used to provide slope protection for the downstream face of the dam similar to the existing conditions.

**Zone 5 (Filter Sand)** – The materials will be the clean sand filter between the existing embankment or foundation materials and the Zone 6 (Drain Gravel) materials. Materials are based on meeting design filter criteria.
Zone 6 (Drain Gravel) – These materials will be the clean sandy gravels that surround the slotted drain pipes for the internal drain system. Materials are based on meeting design filter criteria for the Zone 5 (Filter Sand) materials.

Zone 7 (Transition Gravel) – These materials will be needed to provide a reverse filter where the new chimney drain will intercept the existing internal drain. This zone will inhibit migration of new filter or drain materials into the coarse existing Zone III materials if seepage flows down the chimney drain. The layout of these different filter and drain zones are detailed in the E Set (Embankment) of the new design drawings.

Zone 8 (Berm) – These materials will create the downstream stabilization berm for the proposed rehabilitation work. Because these materials are only needed to provide weight for the berm, they can vary in what type of materials are used.

The specifications also include requirements that any materials for any zone placed within one foot of structural concrete and pipelines be screened to remove the plus ¾-inch rock to reduce the likelihood of damage to the concrete or pipelines.

5.2 Seepage Analysis

A detailed seepage analysis was completed by GCI in June of 2014. The report for this seepage analysis is included in Appendix H. There are several references to the GCI report in the following seepage analysis description.

Millsite Reservoir has been primarily operated as a seasonal irrigation reservoir during its history of use. The reservoir fills with snowmelt runoff from the surrounding mountains during the spring, with the reservoir spilling during peak snowmelt. After this period, the water level is drawn down throughout the summer and fall for irrigation purposes and use by other water users. The reservoir is generally drawn down to near the conservation pool by late fall and remains at this level through the winter until the reservoir fills again in the spring. Due to possible uncertainties with future operation of the reservoir, where the water level could be held at the full pool elevation for extended periods of time, Utah Dam Safety requires a steady-state seepage condition analysis for Millsite Dam. This steady-state seepage is considered conservative for the current and future operating conditions of the reservoir.

Four cross sections were analyzed as part of this study. Seepage analyses were completed at these locations for both existing conditions and with the proposed rehabilitation layout. The locations of these cross sections are described below:

1) Station 16+00 – This cross section represents the maximum section of the embankment near the outlet works and through the original stream channel. At this section, the
foundation alluvium in the upstream portion is relatively shallow and the foundation alluvium in the downstream portion is deep.

2) Station 24+50 – This cross section represents portion of the dam where the upstream embankment is founded on bedrock and the downstream portion of the dam is founded on alluvial deposits.

3) Station 31+00 – This cross section represents the portion of the dam that is constructed on bedrock.

4) Station 10+50 – This cross section represents the maximum section of the embankment similar to Station 16+00 with the difference being that this is the portion of the dam where the upstream foundation alluvium is deep and the downstream foundation is bedrock.

5.2.1 Piezometric Data

There are currently 12 piezometers at Millsite Dam. An evaluation of the piezometer data collected from the Millsite Dam was completed to evaluate the piezometric water surface under the current operating conditions. Section VII of the geologic report includes detailed information on instrumentation, piezometric levels, and ground water conditions.

In summary, results of the piezometric data analysis indicate that the reservoir has never been full long enough to establish a steady-state condition of seepage through the dam embankment and foundation. It also appears that the phreatic surface has never been high enough to intercept the internal drain system. One of the piezometers at the downstream toe, near the outlet works, indicates that the groundwater level generally fluctuates from four to six feet below the ground surface in the vicinity of the outlet works.

5.2.2 Toe Drain

As described in Section 2 of this report, the only existing toe drain is located in the left groin. Because no information has been found regarding the original construction or extents of the toe drain, it has been neglected in all seepage analyses.

5.2.3 Hydraulic Conductivity of Soils

The GCI seepage analysis report describes how the hydraulic conductivity values were developed for the seepage analyses. The downstream work that will occur as part of the rehabilitation will all be beyond the extents of the phreatic surface that would develop because the main existing internal drain system will be left in place. The only change to the phreatic surface is that the downstream phreatic surface elevation drops a couple of feet because of the internal drain system and collection pipe that will be installed as part of the rehabilitation work.
Therefore, the seepage analysis does not include hydraulic conductivity values for the new embankment materials.

5.2.4 Seepage Analysis

The GCI seepage analysis report includes both saturated seepage analyses and partially saturated seepage analyses. In addition, analyses for each section were completed using varying ratios of the horizontal and vertical hydraulic conductivity to evaluate the sensitivity of the seepage dependence on this ratio.

The following text is the conclusions section from the completed seepage analysis report:

To assess potential impacts to the existing chimney drain (Zone III), seepage modeling was performed for eight (8) different cross-sections incorporating proposed reservoir changes and dam modifications together with a range of hydraulic conductivity values for embankment Zones I and II. The ranges of hydraulic conductivity (k) values (minimum, maximum, and average) were estimated using grain-size distribution data, which we judged to be representative of in-place materials when compared with all other field and laboratory data we have been provided and understand to have been collected to date. Phreatic elevations computed from seepage models were then compared to bottom of the chimney drain elevations and expressed as relative head on the chimney drain (RHCD). Additional modeling was performed to help assess the impacts of saturated-unsaturated behavior on the previously determined, saturated model results.

Results from this study suggest the following:

1. Relative head on the chimney drain (RHCD) values range from 2.5 feet to 14.5 feet above the bottom chimney drain elevation and average values range from 5 to 10 feet.
2. Changes in RHCD were computed for proposed reservoir level increases and show increases ranging from 3.0 feet below the bottom chimney drain elevation to 1.0 feet above, suggesting increasing reservoir levels 4 feet accompanying geometric and toe drainage improvement will have minimal influence on the chimney drain.
3. Inclusion of the effects of saturated-unsaturated behavior do not materially alter the conclusions reached previously regarding the water head in the chimney using saturated models with various hydraulic conductivities or water impoundment elevations.

Results of the seepage analysis were used to establish piezometric lines for the slope stability analyses, discussed later in this report.
5.2.5 Seepage Areas

Based on original design information, plans and specifications, and construction information from the original dam, significant effort was made to try and extend the Zone I core and the slurry trench down to the bedrock interface during construction. The left abutment groin toe drain was installed within a few years of original construction because there was seepage along the abutment, according to local residents.

The only other seepage that has been identified at the dam is through the bedrock at the far right end of the dam and on the face of the cliff below the spillway chute. At the far right end of the dam, seepage water appears in the bedrock just downstream of the dam toe when the reservoir is near full. The water appears to be conveyed through weathered and fractured bedrock. The seepage amounts are small enough that no visible flow can be seen, but there is enough seepage to sustain some vegetation. There is also seepage in the cliff face below the spillway chute when the reservoir level is high.

As part of the rehabilitation work, the existing left abutment toe drain will be removed. The new internal drain system will extend over both the left abutment area and the right embankment seepage areas discussed above. The work along the right embankment will include a new toe drain trench with filtered drain materials to intercept and collect the visible seepage areas. Work on the spillway will include lowering the spillway chute floor and excavating a trench through the seeping bedrock that will be backfilled with a drain collection system so that the seepage water in the spillway bedrock foundation can be collected and monitored.

5.3 Liquefaction

Liquefaction is the condition where saturated, loose, granular soils lose strength due to pore pressure buildup during a seismic event. A liquefaction triggering analysis of the embankment and foundation materials was conducted from standard penetration test (SPT) and Becker penetration test (BPT) field work that has occurred at the dam site.

Liquefaction analyses were conducted in accordance with the procedures establish by Youd et al., (2001) in the NCEER/NSF workshop. This method correlates the cyclic stress ratio (CSR) causing liquefaction with the cyclic resistance ratio (CRR). The CSR is computed based on earthquake magnitude, maximum acceleration, effective stress, and the depth below ground surface. The CRR is computed using corrected SPT blow count and fines content. For these analyses, fines content from laboratory data was used when available. For those samples that did not have any laboratory data, fines content and plasticity index (PI) were estimated based on material type and known laboratory data of the samples located nearby. The factor of safety against liquefaction is defined as the ratio of the CRR causing liquefaction to the CSR seismic...
demand. Factors of safety with a ratio of less than 1.1 are assumed to liquefy. For this study, a maximum credible earthquake (MCE) with a moment magnitude of 6.9 and peak ground acceleration (PGA) of 0.39 were used in the analysis.

The analyses were also computed using an age correction for the foundation materials. Dr. Les Youd was contacted to discuss if the age correction is applicable for under dams and it was his recommendation that the age correction be used. Appendix 7 of the geologic report includes information for developing the age of the alluvial deposits.

5.3.1 SPT Test Holes

All SPT test holes with blow counts were used in evaluating the potential liquefaction of the existing foundation materials. As noted in the geologic report for Millsite Dam, there is a significant percentage of gravel and cobble in the foundation materials. Gravel and cobble tend to increase the blow count values during the SPT sampling procedures.

In an effort to remove the bias from the larger graded materials, one-inch blow counts were collected during the more recent test hole drilling. These one-inch blow counts were then evaluated to identify if less dense materials are being penetrated during the sampling as the sampler passed by a larger piece of gravel or cobble. (As is standard for the SPT procedures, the top six-inch interval of the 18-inch sampling depth is neglected in the blow count due to potential slough materials.) An analysis spreadsheet was set up that takes the three-inch interval with the lowest blow counts and applies them to create a 12-inch interval with a corresponding blow count value. For example, if a SPT sample has a high blow count of 60 for the first nine inches and then breaks through the denser materials and has a low blow count of eight for the last three inches, the blow count of eight would be multiplied by four to create an equivalent 12-inch blow count value of 32. These corrections were only made if the corrected blow count value was less than 50 blows. Appendix N includes all of the blow count corrections for the test holes that recorded one-inch blow counts. The appendix includes a spreadsheet for each test hole followed by the plots for the corrected samples. The plots provided a visual confirmation that the process was providing reasonable correction values.

The corrected blow counts were then used in the liquefaction analyses. Appendix O includes the liquefaction analysis results for the SPT holes.

5.3.2 BPT Test Holes

There were concerns that the SPT sampling procedures were being overly influenced by the cobbles and gravels in the foundation. In an effort to address this concern, BPT test holes were completed at the site, with several test holes located adjacent to SPT test holes. The BPT blow counts were then used to run similar liquefaction analyses.
The steps for analyzing the BPT samples are as follows:

1) Collect blow counts and PDA results – The field sampling data information for all the BPT test holes is included in Appendix P.
2) Develop a skin friction value – The skin friction was developed by creating an average skin friction value that varies with depth based on a combined average of the pullback and CAPWAP readings that were taken during field testing. The plot and data for this are included in Appendix P.
3) Calculate blow counts – The skin friction values are then used with the BPT blow counts to calculate a blow count value that is equivalent to an SPT blow count. Two methods were used; the Harder & Seed method and the Sy & Campanella method, for comparison of the results. The Sy & Campanella results were used because they are considered the more accurate results if the pullback and CAPWAP corrections are available. These blow count calculations are included in Appendix P.
4) Run liquefaction analyses – These blow counts can then be used in the liquefaction analyses. The liquefaction analysis results for the BPT test holes are included in Appendix Q.

5.3.3 Potentially Liquefiable Zones

The general recommendation is to use caution in comparing SPT and BPT analyses because there is a high likelihood that the different procedures and varying soils can produce results that don’t correlate very well. However, the liquefaction results for Millsite Dam using SPT and BPT methods had very similar results. These results also correlate will with documentation from the original construction that identified loose materials in the foundation. The liquefaction analyses indicate that only 6.3 percent of the SPT samples and 4.6 percent of the BPT samples are considered potentially liquefiable. Of the potentially liquefiable samples, approximately 70 percent of both methods are above elevation 6095 feet, which is within the top 15 to 25 feet of the alluvial deposits.

The proposed rehabilitation plan is to excavate the native foundation materials down to elevation 6095 feet in the area where potentially liquefiable materials were identified to remove the loose materials, similar to what was done during the original construction. (Rehabilitation will also include proof rolling which generally penetrates several feet into the foundation. Using an elevation of 6090 feet to account for materials that will likely be densified from proof rolling, approximately 75 percent of the potentially liquefiable materials will be removed.) By removing materials to elevation 6095 feet, only 25 to 30 percent of the potentially liquefiable samples would be left in the foundation. These remaining potentially liquefiable are only 2.3 percent of the total SPT samples and 1.2 percent of the total BPT samples.
Below elevation 6095 feet, there is not an elevation that has a high number of potentially liquefiable samples. The samples are spread out through the test hole depths, which indicate that there are small pockets of potentially liquefiable materials. In addition, the proposed rehabilitation includes constructing a berm on the downstream toe to elevation 6145 feet, resulting in a new compacted embankment depth of 50 feet where the foundation excavation will take place. Although a general consensus has not been made at what depth liquefaction is less likely to occur, 50 feet has been commonly discussed as a depth where liquefaction is less likely.

There have been concerns raised that the gravel and cobble in the foundation have affected SPT and BPT blow counts enough that the results are not reliable. However, review of the analyses indicate that the methods have been able to identify potentially liquefiable materials. The SPT one-inch blow count corrections clearly show that loose materials have been identified in samples that would generally be considered dense if the initial total blow counts were used. The BPT test holes were able to identify softer layers of soils at different depths, which indicate that the Becker hammer was able to penetrate through the denser materials and then have lower blow counts in the loose materials. The geologic report also indicates that during construction the foundation materials were dense at the final foundation excavation depth and the slurry trench excavation was extremely difficult. Recent personal communication with one of the local residents that helped with the construction indicate that the only way the construction contractor was able to excavate the slurry trench was by adding several large steel plates to the excavation drag line in order to make the bucket heavy enough to penetrate through the dense foundation materials.

In summary, removing foundation materials down to elevation 6095 in the portion of the stream channel that has potentially liquefiable materials will remove the majority of the loose materials. The remaining two percent of loose materials are in pockets and thin zones scattered throughout the generally dense foundation materials through the depth of the alluvial deposits. In order to be conservative, the foundation will be modeled in stability and deformation analyses using ten-foot thick by 50-foot foot wide pockets of liquefiable gravel foundation materials just below the excavation limits. These pockets will be extended through the areas of looser materials to account for approximately 50 percent of the materials in the top ten feet of the deeper alluvial deposits. These pockets of liquefiable materials can be seen in the slope stability analyses. Also, the top 10 to 15 feet of the foundation materials left in place upstream and downstream of the dam will be considered to be liquefiable sand materials.

5.4 Static and Dynamic Slope Stability

5.4.1 Static Material Strength Properties

The material properties are based on field descriptions, blow count data, laboratory testing, and correlation tables. If laboratory triaxial test results were available for a specified zone, they were
used for the soil strength parameters. As verification, three correlations were performed to correlate the SPT and BPT blow counts to friction angle. The correlations used were based on studies by Meyerhof (1956), Schmertmann (1975), and Peck, Hansen, and Thornburn (1974). The equations that were used to calculate the friction angle from SPT blow counts these studies detail were taken from Hettiarachchi and Brown (2009). These three methodologies were used due to their more conservative results when compared to other correlation methods. The results of the blows per inch analysis were used in determining the strengths due to the possible influence that the coarse-grained materials would have on the blow counts. These friction angle calculations for the existing embankment and foundation materials are summarized in Appendix R.

Table 1 shows the parameters for each soil type and zone used in the stability analyses. Appendix S includes these same parameters with a brief description of the basis for each parameter of each soil type.

Materials on the upstream and downstream slopes were given a nominal effective cohesion value of 50 psf (0.35 psi) to prevent thin surface failures on the slope. To verify that the cohesion was not altering the safety factor, a couple of the circular failure surfaces were run without the cohesion value and the change to the safety factor was insignificant enough that it did not impact the results.

The static material properties were also used for the analysis of the rapid drawdown condition for the slope stability analysis. For this analysis, it was assumed that the embankment and foundation materials remain in a saturated condition associated with the steady state phreatic surface, with the exception that the upstream slope is partially drained because of the drain finger extensions that were constructed on the upstream side of the dam.
Table 1 – Soil Parameters for Slope Stability Analyses

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<td>34</td>
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</tr>
<tr>
<td>Zone 7 - Transition Gravel</td>
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<td>34</td>
<td>0</td>
</tr>
<tr>
<td>Zone 8 - Berm</td>
<td>125</td>
<td>50</td>
<td>40</td>
<td>50</td>
</tr>
</tbody>
</table>

5.4.2 Dynamic Material Strength Properties

For material strength due to seismic conditions in materials that do not experience liquefaction, the effects of pore water pressure build up were accounted for by relating the excess pore pressure to a reduction in strength. This was done using information as shown in Marcuson et al. (1990) as referenced in the NRCS Seismic Analysis Manual (2014). From this, the factor of safety against liquefaction is related to an excess pore pressure ratio. A review of the liquefaction analysis performed found that the materials that do not show a liquefaction potential at Millsite Dam generally have factors of safety against liquefaction well in excess of 2.5. The limits of the study by Marcuson (1990) only extend to a factor of safety of approximately 2.7. Due to unknown properties beyond this limit, the excess pore pressure ratio was determined based on a factor of safety of 2.5. For embankment materials that were placed in engineered lifts and the densities of the material are well compacted and uniform, an excess pore pressure ratio of 0.1 was used, resulting in a ten percent reduction of the effective friction angle. For the heterogeneous foundation materials, the upper limit of the curve was used resulting in an excess
pore pressure ratio of 0.14 for foundation materials, for a 14 percent reduction of the effective friction angle.

For materials that show a liquefaction potential based on the results of the liquefaction analysis that was performed, residual strengths were estimated based on SPT blow counts corrected for overburden pressure, hammer energy, and liner effects. The blow counts were also converted to “clean sand” values using the adjustments for fines content recommended by Seed (1987). Based on the liquefaction analysis, sand materials within the upper alluvium and gravel materials at deeper depths showed a liquefaction potential. The blow counts of those materials that show liquefaction were averaged and corrected as previously described resulting in an average clean sand blow count of 8.5 for the liquefiable materials above elevation 6095 feet, and 12.7 for the liquefiable materials below elevation 6095 feet.

The ratio of residual strength was then evaluated using methods described in Seed and Harder (1990), Idriss and Boulanger (2008), and also Ledezma and Bray (2008). The selected method for determining the residual strength was the curve from Figure 88 in the Idriss and Boulanger method, which is slightly more conservative that the other methods. The residual strength ratio from the Idriss and Boulanger method was also used to compare with the residual strength, but the change in the slope stability analyses were insignificant. Spreadsheets showing the residual strength calculations are included in Appendix T. The spreadsheets include several methods that were evaluated for the alluvial materials.

5.4.3 Slope Stability Analysis

Slope stability analyses were performed using the computer program Slope/W. The analysis satisfied general limit equilibrium requirements, and used Spencer’s method to compute factors of safety, which satisfies both force and moment equilibrium. Circular trial failure surfaces were used for the analyses. The three cross sections analyzed were Stations 10+50, 16+00, and 28+50. Stations 10+50 and 16+00 were also modeled in the seepage analyses. Station 24+50 was modeled in the seepage analyses, but Station 28+50 was used for the slope stability analysis because the new downstream berm was extended farther than was planned during the preliminary design. It was originally planned to end the berm before Station 24+50, but the current design is to extend the berm until it blends into the embankment after the bend in the dam. The seepage analysis results were similar enough between the three analyzed cross sections that a similar piezometric surface was able to be created in the model for Station 28+50.
For these three cross sections, the existing and proposed configurations of the embankment were analyzed with the normal high water level at elevation 6215 feet for the existing dam and elevation 6219 feet for the proposed rehabilitation. All analyses were conducted using steady-state seepage conditions.

The liquefaction analysis described previously shows that the alluvial material beneath the embankment indicates areas of liquefaction. The areas of potential liquefaction in the foundation are generally considered to be confined to the materials above elevation 6095 feet, with pockets of potentially liquefiable materials below that elevation. As described previously, pockets of liquefiable materials below elevation 6095 feet were modeled using 10 feet deep by 50 feet long zones in the areas of deep alluvium. This was done to conservatively model the presence of liquefiable pockets at a location that would have the most influence on the slope stability.

The critical factors of safety calculated for each of the conditions analyzed at stations 10+50, 16+00, and 28+50 are shown in Table 2. For slopes that include more than one slope stability analysis for the same general condition, such as a deep and a shallow slope failure for the downstream slope, only the lowest factor of safety is shown in the table. As shown in the table, the existing embankment does not meet all minimum factors of safety (highlighted in light red) while the proposed rehabilitation does meet all minimum factors of safety. Instantaneous drawdown was only run for the existing embankment using the maximum section at Station 16+00. Figures showing the failure surfaces for each cross section are included in Appendix U.
Table 2 - Slope Stability Factors of Safety for the Existing and Proposed Embankment

<table>
<thead>
<tr>
<th>Condition</th>
<th>Slope</th>
<th>Calculated Factor of Safety</th>
<th>Required Min. Factor of Safety</th>
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</thead>
<tbody>
<tr>
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<td></td>
<td>Sta. 16+00</td>
<td>Sta. 10+50</td>
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<tr>
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<td>1.6</td>
</tr>
<tr>
<td></td>
<td>Upstream</td>
<td>1.5</td>
<td>1.6</td>
</tr>
<tr>
<td>Instantaneous Drawdown</td>
<td>Upstream</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>Post-Earthquake:</td>
<td>Downstream</td>
<td>1.0</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>Upstream</td>
<td>1.3</td>
<td>1.4</td>
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<th>Required Min. Factor of Safety</th>
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<td>Post-Earthquake:</td>
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<td></td>
<td>Upstream</td>
<td>1.4</td>
<td>1.4</td>
</tr>
</tbody>
</table>

\(^1\) DWRi (2012)  
\(^2\) NRCS (2005)

### 5.5 Earthquake Deformation Analyses

Concern exists regarding the stability of the dam during a major earthquake event. A deformation analysis was conducted to determine appropriate levels of deformation for the embankment during the design earthquake event. This was accomplished using coupled methodology, including Newmark sliding block approach and QUAD4M, and was completed by GCI. The full deformation analysis information is included in Appendix K.

The design peak ground acceleration specified in the NRCS seismic analysis report is 0.31g on bedrock and 0.39g at the ground surface where there is deep alluvium. For the deformation analysis, the target response spectrum was developed for bedrock using the same NGA ground motion predictive equations and input parameters as developed in the seismic report, and the peak bedrock acceleration was determined to be equal to 0.31g.
Five time histories were selected based on magnitude, faulting, distance, etc. Yield accelerations for the upstream and downstream slopes at 1/3, 2/3, and the full height of the embankment were determined for both static and post-earthquake strengths. These yield accelerations were calculated for each of the cross sections at 10+50, 16+00, and 28+50. The dynamic analysis using QUAD4M was implemented and acceleration time histories for each potential failure mass were completed for each of the failure surfaces detailed above. In addition, the computed maximum acceleration of the mass ($K_{\text{max}}$) for each earthquake record was also completed for each potential failure mass. The time histories for each failure mass were then processed using Newmark methodology to compute the respective displacements. From the total displacement calculated, the vertical displacement of each failure mass was estimated using an approximate angle of seismic thrust to develop a vertical-to-horizontal ratio.

For each of these scenarios, the deformation was calculated using the full post-earthquake yield acceleration and stepped yield acceleration based on the static and post-earthquake yield accelerations. Through this analysis, it was found that the stepped yield acceleration had very little effect on the overall calculated displacement because the number of cycles to induce liquefaction happens very rapidly. As a result, only the calculated displacements for the post-earthquake condition are shown.

Based on the deformation analysis completed by GCI, the critical calculated mean displacements for both the total and vertical deformations are shown in the following table for each of the three cross sections analyzed.

With regard to deformation, Utah Dam Safety requires a minimum factor of safety against overtopping of the combined upstream and downstream slopes of 3.0 for embankments and foundation subject to liquefaction, and 2.5 for embankments and foundations not subject to liquefaction. The proposed freeboard for Millsite dam is 7.6 feet with the increased water surface. Due to the liquefiable nature of the foundation materials at sections 10+50 and 16+00, a factor of safety of 3.0 is required, with a factor of safety of 2.5 required at station 28+50.

From GCI report, it can be seen that the calculated deformation is 0.74 feet for Station 16+00 and 0.70 feet for Station 10+50. These provide respective factors of safety of 10.3 and 10.9, which are well above the minimum standards. (Results for Station 28+50 are still being completed by GCI and will be included in the next draft of their report and this design report.) These deformation values are also conservative because they are based on the assumption that the vertical deformation is equivalent to the horizontal deformation, when the vertical deformation is generally less than half of the horizontal deformation. The GCI report Table 3 also includes estimated vertical displacement values.
5.6 Embankment Zones and Internal Stability

This section addresses the material type, consistency, and internal stability of the zoned embankment materials. An evaluation of the embankment materials follows; based on specifications, borrow area investigations, construction laboratory testing, recent embankment exploratory drilling, and laboratory testing. The geologic report contains more detailed and complete data for the site conditions. The following is a description of embankment materials and a discussion of internal stability.

5.6.1 Embankment Materials, Compatibility and Internal Stability

The original embankment materials consist of Zone I (core) and Zone II (shell) materials that are broadly graded. Zone III materials, which consist of cobbles and boulders, were used for the internal drain and for the upstream and downstream slope protection. Review of the construction specifications and documentation indicate that the Zone III materials were not designed to meet current filter criteria. The geologic report has more discussion regarding this issue.

No gradations were provided for the drain materials during construction, but photographs clearly show that the internal drain system is coarse enough that adjacent embankment materials would most likely migrate into the drain materials if the phreatic surface were to ever reach the internal drain elevation. As part of the rehabilitation process, several alternatives were evaluated in 2010 to address the concern regarding the cobbles and boulders that were used as the internal drain without any transition or filter zones between the other embankment zones.

5.6.1.1 Alternatives

The following alternatives were evaluated and cost estimates were prepared in deciding how to address concerns with the existing internal drain zone.

5.6.1.1.1 Downstream Filtering of the Internal Drain

This option includes the general layout described in this design report. It has been estimated that the existing Zone III internal drain materials has a void ratio of approximately 30 percent. The current proposed alternative includes filtering the downstream side of the internal drain so that any materials that migrate into the internal drain would then be filtered and contained in the internal drain zone. This would allow a large volume of finer grained materials to be washed into the internal drain, but the materials would remain in the dam. There is a likelihood that some settlement would occur on the upstream face if this were to happen.
The cost estimate for this alternative in 2010 was approximately 13 million dollars. The cost estimate has increased significantly as more detailed analyses have been completed and more accurate proposed work using more current construction estimates have been incorporated. However, the 2010 cost estimate is being used as a reference to show the relative difference between the cost estimates. Similar cost comparison ratios would be applicable using current cost estimating procedures and more detailed information for the following alternative comparisons. This option includes a thicker chimney filter in the location of the existing internal drain than would be required in the other alternatives.

5.6.1.1.2 Alternative 2 – Grout in the internal drain

This option is similar to Alternative 1 with the exception that the existing internal drain would be grouted rather than filtered. Analyses were based on the assumption that the existing internal drain has a void ratio of approximately 30 percent, which would be filled with cement grout to inhibit migration of materials into the drain system. Grouting options included sluice grouting from the top of the internal drain or injection grouting at lower elevations.

This had a cost estimate of 8 to 16 million dollars more than Alternative 1, depending on the grouting method used. An additional concern would be verification that grouting of the entire internal drain was obtained. A downstream filter system might still be needed even if this alternative was selected.

5.6.1.1.3 Alternative 3 – Remove the Entire internal drain

This option included excavating and removing the entire internal drain. The embankment would then be rebuilt with a zoned embankment using current conventional methods.

This alternative had a 2010 cost estimate of over 36 million dollars. Although this alternative would best address the concerns with the internal drain system, the alternative is not cost effective and the construction impacts would be much greater than the selected alternative, too.

5.6.1.2 Incorporation of Design into Selected Alternative

As part of the proposed rehabilitation, the downstream slope needs to be flattened and a berm added at the downstream toe to meet slope stability requirements. In addition, raising the crest four feet to restore the lost reservoir capacity also moves the crest and downstream slope further downstream. By implementing these measures, a chimney drain can easily be added on the downstream slope of the dam. The benefits and reason for the chimney drain are described later in this report. These proposed improvements provide an ideal situation to incorporate Alternative
1 to filter the internal drain system. Excavation of the downstream slope protection will be used to identify the areas where the internal drain collection zone was installed at the downstream toe. The internal drain at the downstream toe will then be exposed so that filtering materials can be placed that will isolate the internal drain if materials begin migrating into the Zone III materials.

5.7 New Chimney Drain and Transverse Cracking

The proposed work will include a chimney drain on the downstream slope of the dam. As shown in the design drawings, removal of the existing downstream slope protection will provide beneficial conditions to construct a chimney drain. The chimney drain will provide the following benefits:

1) Intercept any seepage through the embankment materials
2) Prevent migration of materials beyond the Zone III internal drain
3) Provide protection against seepage along potential transverse cracking

The first two items listed above were addressed in the previous section. The following description addresses transverse cracking.

Transverse cracking at Millsite Dam could potentially occur during an earthquake. An open crack in the embankment could lead to seepage through the dam with the potential to wash out embankment materials if a chimney drain is not in place. The severity of transverse cracking due to an earthquake is dependent upon the deformation of the embankment caused by the earthquake ground motions. In a study conducted by the State of California, Division of Safety of Dams (Fong & Bennett, 1995), the evaluation of dams in California following three major earthquakes provided important information for the implementation of needful provisions to prevent internal instability due to transverse cracking caused by earthquakes (see also, Pells and Fell (2003). The amount of deformation, width and depth of cracking, and effect of abrupt foundation grade changes were presented.

The vertical deformations values were presented in the deformation analysis section of this report. Based on this, Pells and Fell (2003) classify Millsite in a Damage Classification type 3 (Major). From this classification, crack width at the crest of the dam is expected to be 3.2 to 5.9 inches, and the maximum crest settlement is estimated to be 0.5 to 1.5%. The maximum embankment height of Millsite Dam is 115 feet, resulting in a maximum estimated crest settlement of 0.58 to 1.73 feet. The maximum vertical deformation of 0.74 from the GCI deformation analysis lies within these estimated bounds.

Based on the above assessments, an embankment provision is recommended to prevent internal erosion failure of the dam, due to transverse cracking caused by seismic ground motions. It is
recommended that a transverse crack filter zone be installed to prevent internal erosion due to seismic cracking. This filter zone would be installed on the exposed downstream face. The chimney drain would run the entire length of the dam, and it would extend from the crest to the downstream toe. This would include a filter sand zone to prevent migration of Zone I and Zone II existing embankment materials into the drain system. The gravel drain would be immediately downstream of the filter sand. Because the downstream excavated slope is relatively flat at 1.75:1, there is a potential that seepage from the downstream face could migrate materials into the drain from the back side. In order to inhibit this potential, a thin zone of filter sand is also need on the downstream side of the drain gravel.

Because of the existing Zone III internal drain materials, a transition gravel zone will be needed to provide a reverse filter to inhibit movement of new drain materials into the Zone III materials. The embankment design drawings include details for how this transition zone will be incorporated into the chimney drain.

The design drawings also include thicknesses of the varying internal drain materials. The chimney drain will have a minimum horizontal width of 4.0 feet, resulting in a vertical thickness of 2.3 feet. This thickness will provide a chimney drain that is thick enough to maintain its lateral connection even if one section of the embankment were to settle the entire deformation amount and an adjacent section of the embankment did not settle at all.

5.8 Blanket and Toe Drain

The proposed rehabilitation includes flattening the slope and constructing a stability berm at the maximum section. These two features result in extending the new toe farther downstream. In order to convey any seepage from the chimney drain to the new downstream toe, a blanket drain and toe drain will be utilized. The blanket and toe drain layout are shown in the design drawings. A slotted pipe will be included in the toe drain to collect and convey any seepage from the internal drain system to a manhole near the outlet works where the flows can be monitored.

5.9 Filter Criteria Analysis

The filter sand, drain gravel, and transition gravel zones were all designed to meet critical filter criteria. The analysis calculations are included in Appendix V.

Part of the filter criteria analysis include selecting a base material that is representative of the materials against which the filter sand will be placed to inhibit migration of materials. In selecting a base material for the existing embankment and foundation materials, the laboratory test results were reviewed to decide what would be an appropriate base material. Of all the tested samples, only one sample (SPT10-10 @ 9') had a fines content greater than 70 percent. The vast
majority of the remaining gradations indicate that the embankment and foundation materials would be classified as category 2 or 3 base materials. The gradation for SPT10-11 @ 4.0' was selected as the base material because it is a finer graded category 2 base material. This will provide a filter that would prevent migration of the finer grained materials that would be present in category 2 materials. It will also filter category 3 materials, but the permeability may not be as high as is generally preferred for a category 3 soil filter. However, because the embankment and foundation materials vary quite a bit, it will be beneficial to have one filter sand based on the finer graded category 2 base materials.

Another factor for the filter design is whether the materials being filtered are dispersive or not. No dispersivity test were found from the original design and construction records. In addition, no samples from the site drilling were ever tested for dispersivity. As part of the final borrow area evaluations, a couple of samples were tested for dispersivity. However, because the percentage of fines were relatively low and the fine materials are generally silts at the site, the dispersivity test results may not be representative of the actual site materials. The dispersivity tests indicated that the materials could be slightly dispersive. A quick comparison of the filter design shows that the proposed gradation meets critical filter criteria whether the base material is dispersive or not.

The fine limits of the resulting filter sand were then used to develop the drain gravel gradation. The drain gravel needs to meet filter criteria for the transition gravel materials, so the fine limits of the drain gravel were used to develop the transition gravel gradation. Because it is possible that the Zone III internal drain may have very large cobble and boulders, the transition gravel was graded toward the coarse side to provide a relatively coarse-grained gravel and cobble material for this zone.

### 5.10 Instrumentation

Piezometers were installed during the drilling in 1996 and 2010. The drawings and specifications give the construction contractor the option to protect and extend the existing piezometers to the new final ground surfaces, or grout in the existing piezometers and install new ones at the same locations. A few new additional piezometers will be installed at the sections that already have piezometers in order to better monitor the phreatic surface through the dam. The design drawings show the locations and information regarding these piezometers.

The design also includes survey monument markers installed at 200 foot intervals along the crest of the dam. Permanent control points will also be established at the dam site. These survey features will be used to monitor any settlement or movement of the dam over time and after seismic events.
Instrumentation monitoring and reporting will meet the minimum standards of Utah Dam Safety. Such standards include all piezometers and drains monitored at least monthly when the reservoir level exceeds 50% of the hydraulic height. Readings will be obtained on a weekly basis when the reservoir exceeds 90% of the hydraulic height. Instrumentation should be monitored at the beginning of the reservoir filling season, at peak elevation, and at the maximum reservoir drawdown. Copies of all instrumentation monitoring data collected should be forwarded to the State Engineer on a monthly basis.

Monitoring the proposed drain discharge systems is a critical component of operating the dam and reservoir. Manholes will be used to provide access to monitor the collected seepage. There will be manholes at the spillway to monitor seepage around the spillway and through the embankment near the spillway. Manholes will be near the outlet works to monitor seepage in the toe drain collection pipe. Because the toe drain is so long, manholes will be spaced at intervals that will allow camera inspection of the toe drain collection pipe. These manholes can be seen in the design drawings.

On the first filling of the reservoir after the rehabilitation work is finished an initial filling plan shall be followed, as approved by Utah Division of Water Rights, Dam Safety Section.
6. **SEDIMENTATION**

The reservoir basin and water shed above the dam were evaluated for sedimentation as part of the dam rehabilitation plan. The NRCS hydrology and hydraulic report includes an evaluation of sedimentation at Millsite Dam and Reservoir. The report can be found in Appendix A, and includes the following:

- Original documentation
- Calculated and derived sedimentation yields
- Trap efficiency
- Sediment storage in Millsite Reservoir
- Discussion/conclusion

With a 4-foot increase in water level, the original dam service life will be restored. The 4-foot water level increase will also meet the storage right of 18,000 acre-feet for irrigation. The total reservoir capacity at the new spillway crest will be 18,000 acre-feet.
80% DRAFT

7. HYDROLOGY AND HYDRAULICS

7.1 Outlet Works Design

7.1.1 Existing Outlet Works

The existing outlet works consist of a reinforced concrete tunnel and two outlet pipes, with appurtenant utilities. The original record drawings and new design drawings include drawings of the existing outlet works layout.

One of the first steps was to evaluate if the existing outlet pipes need to be removed and replaced because of wear and tear. DWRe contracted with 5 Star to complete an evaluation of the two existing outlet pipes. The report is included in Appendix F. In summary, the report indicates that the 8-inch culinary line has lost enough wall thickness that it needs to be replaced, but the 54-inch irrigation line still has enough wall thickness that it does not need to be replaced.

Both outlet pipes have valves on the upstream end that are only used as guard gates. The 8-inch line has a hand-operated gate valve and the 54-inch line has an electric-operated butterfly valve. The 8-inch line flows are controlled by demands from the culinary system. The 54-inch also has a Howell-Buger valve on the downstream end to control releases into the irrigation canal. There are several other connections into the outlet works that are described in the outlet works design report that was prepared by FCE and is included in Appendix I. The FCE report also includes structural analysis results for the outlet works.

7.1.2 Analysis of Riser Towers and Outlet Piping

The riser towers and outlet piping were evaluated for seismic and structural performance by ABS Consulting and summarized in a report that is included in Appendix E. ABS Consulting’s scope of work included:

- Review all available existing data on the riser towers, and outlet piping, including existing drawings, site photos, aerial photos, dam construction documentation, and site specific geological data.
- Perform a structural analysis of the riser towers and spillway walls.
- Evaluate the outlet pipe for seismic ground motion and liquefaction effects.
- Develop conceptual retrofit measures to mitigate any discovered deficiencies.

Based on this assessment, the intake tower and outlet works are considered to be structurally sufficient to meet the strength requirements for the design seismic event, including settlement of the outlet from potential liquefaction during a seismic event. The only modifications, besides a few maintenance items listed in the ABS Consulting report, include improving the straps and neoprene gaskets at the 54-inch outlet pipe support blocks.
7.1.3 Outlet Deformation

Some concern exists about the potential impacts to the outlet conduit due to embankment deformation from seismic loading. Embankment deformation (5.6 Embankment Deformation Analysis) and factors of safety based on 7.0 feet of freeboard are shown in Table 3.

The slope stability analysis indicate that the most critical failure surface under seismic loadings exits the slope above both the upstream and downstream berms at the maximum embankment section. As a result, it is expected that the largest deformations would occur in the embankment above the outlet tunnel.

In conclusion, the possibility of damage to the outlet works due to deformation following an earthquake is possible, but considered nominal. However, a close inspection of the outlet works following the earthquake is recommended and will be included as part of the O&M requirements.

7.1.4 Modifications to the Outlet Works

Modifications to the outlet works will include lengthening the tunnel and outlet pipes approximately 90 feet to the new downstream toe and constructing a new vault for all of the pipe connections to the outlet pipes. The FCE outlet works design report includes details on how the new vault will be laid out. The existing Howell-Bunger control valve will be replaced with a plunger valve. A new plunge pool will also be constructed as a result of the dam toe being pushed farther downstream.

7.2 Hydrology

The NRCS hydrology analysis in Appendix A includes the design flood that the spillway has to pass. The design flood has a maximum inflow of approximately 31,000 cfs. The spillway must be able to pass the design flood without overtopping the dam.

The inflow hydrograph from the 100-year and USU Local PMP storm were routed through the existing spillway. The routing was started with water at the spillway crest (6219.0 feet). The dam crest height was required to be at elevation 6226.6 feet in order to pass the design flood without overtopping.

The drawings include the updated area-capacity curves and tables for the proposed new reservoir water level. The area-capacity curves were developed using the spreadsheet information included in Appendix L. The appendix information also shows the drain time for the new reservoir elevations, which show that 90 percent of the reservoir can be drained within 30 days by only using the outlet valves.
7.3 Spillway Design

The existing spillway does not have the capacity to pass the design flood. One of the critical reports used in designing the outlet works is a paper presentation that was prepared by DWRe with details about the design of the new spillway for Millsite Dam. The paper is included in Appendix G. The paper describes the alternatives that were evaluated and why a labyrinth weir was selected for the proposed new spillway. The following information is summarized from the design information presented in the paper.

7.3.1 Criteria and Procedures

Design of the structural reinforced concrete auxiliary spillway is in accordance with criteria specified in TR-60, Dam Safety publications, and USBR publications. A summary of key design criteria followed is presented below:

1. The discharge capacity of the auxiliary spillway will be the maximum design discharge as determined in the Hydrology Section. The spillway will be proportioned to have sufficient capacity to pass the freeboard hydrograph with the water surface in the reservoir at or below the elevation of the top of dam.

2. The crest of the auxiliary spillway will be increased 4.0 feet above the original crest elevation of 6215.0 feet, resulting in a new spillway crest elevation of 6219.0 feet.

3. The structural spillway must be designed so that passage of the freeboard hydrograph does not cause serious damage to the embankment or the structure itself.

4. The structural spillway must be compatible with the foundation conditions, the channel stability downstream of the spillway, the possible range of tail water depth, and the proximity of the spillway to the embankment.

5. The inlet portion of the spillway chute must be equipped with a control section that will produce critical flow at the crest, and result in a determinate stage discharge relationship.

6. Consideration must be given to effects of air entrainment (air bulking) by water traveling at supercritical velocities, if applicable.

7. The design discharge for hydraulic proportioning of the structural spillway must not be less than 2/3 of the planned structure capacity during passage of the routed freeboard hydrograph, except that all headwalls and sidewalls shall be designed to prevent overtopping during passage of the full maximum freeboard discharge.
8. The outlet section of the concrete chute spillway will consist of a hydraulic jump basin which will dissipate the energy of the high velocity discharge, if necessary.

7.3.2 Concrete Spillway Design

The spillway concrete is designed to have a 28-day strength of 5,000 psi. FCE prepared a spillway structural design report that includes the concrete design for the spillway, which is included in Appendix J. The report includes analyses for several different loading conditions, and design calculations for appurtenant features of the spillway, such as the downstream flip bucket, seepage cutoffs, drains, and a new golf cart bridge that spans the spillway discharge chute.

7.3.3 Spillway Sliding Analysis

The spillway chute only has a nine percent grade and the entire base of the spillway floor is anchored into the bedrock to prevent uplift, as detailed in the FCE spillway structural design report. Because of these features, a detailed analysis for potential sliding was not completed.

7.3.4 Proposed Spillway Crest Geometry and Stage-Discharge Relationship

The spillway design paper in Appendix G describes the detailed layout of the spillway, which has been incorporated into the design drawings. The labyrinth spillway includes three apexes that protrude into the reservoir. The weir section has a wall height of 19.5 feet. The spillway rating curve from the design paper was incorporated into the spillway discharge curve shown in the drawings.

7.3.5 Spillway Cavitation Analysis

The discharge chute is not steep enough to create conditions that would create cavitation, so a specific analysis was not completed.

7.3.6 Spillway Plunge Pool

The spillway chute discharges spillway flows that drop over 60 feet to a plunge pool at the bottom of the cliff. The existing plunge pool will be used as the plunge pool for the new spillway without making any modifications.
8. CONSTRUCTION NARRATIVE

8.1 Possible Construction Schedule

A possible construction schedule is included in Appendix W. This schedule shows that the proposed rehabilitation work will take approximately one and a half years. To begin construction, the water level in Millsite Reservoir will be drawn down at a quicker rate than normal in the summer of 2016 to facilitate work at the downstream toe. Dewatering measures will be needed in order to complete the foundation excavation and backfill before freezing conditions arrive in late 2016. Concrete work on the outlet works and spillway would then take place during the winter. The reservoir will be filled to a predetermined lower elevation than full during the spring of 2017 because the new dam crest will most likely not be completed. The downstream berm and backfill would then occur during the spring through end of 2017.

8.2 Quality Assurance

Quality Assurance (QA) refers to those actions, procedures, and methods employed at the management and senior technical levels to observe and ensure that prudent quality procedures are in place and are being carried out and that the desired result of a quality product is achieved. The QA team and responsibilities for each party include the following:

- DWRe – Provide a full time Resident Project Representative for construction observation, documentation, submittal review, and directly working with the Contractor’s project manager and on-site superintendent. Also, DWRe in conjunction with FCR will contract with a geotechnical testing lab that will supply technicians for sampling and testing on-site materials as needed to verify that materials are meeting specifications. Frequency of testing and sampling will be determined by the Engineer and the QA team. DWRe geologists will also make several site visits to observe and document excavation of the foundation, toe drain, spillway, and other features that will need a geologic evaluation.
- FCR – The canal company will contract with an outside engineering firm to provide additional construction observation and QA as needed to assist the Engineer.
- NRCS – Provide quality assurance site visits as needed.

It is anticipated that the bid opening and awarding the contract will be in June of 2016, and construction starting by the end of June 2016.
9. **COST ESTIMATE**

**Millsite Dam Rehabilitation - Cost Estimate**

<table>
<thead>
<tr>
<th>Item No.</th>
<th>Item Description</th>
<th>Bid Quantity</th>
<th>Unit</th>
<th>Unit Price</th>
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<td>General Site Work</td>
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<td>Embankment</td>
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<td>3</td>
<td>Outlet Works</td>
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<td>4</td>
<td>Spillway</td>
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<td>5</td>
<td>Miscellaneous</td>
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</table>

Construction Contingency @ 10%  
**Total**
10. REFERENCES (These need to be reviewed and updated)


Fong, F.C., and Bennett, W.J., 1995, Transverse cracking on embankment dams due to earthquakes, presented at the 1995 ASDSO Western Regional Conference, Red Lodge, Montana,


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Utah Division of Water Rights (DWRi), 2012c, Utah Administrative Code Rule R655-10 (Dam Safety Classification, Approval Procedures and Independent Reviews and Rule R655-11 (Requirements for the Design, Construction and Abandonment of Dams)

Utah State University, 1995. Probable maximum precipitation estimates for short duration, small-area storms in Utah, October 1995, Utah Climate Center, Utah State University

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