Submitted to
Nevada Irrigation District
1036 W. Main Street
Grass Valley, CA 95945

Submitted by
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February 9, 2016

Nevada Irrigation District Centennial Reservoir Project Preliminary Geotechnical Investigation Phase II Report – Final

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February 9, 2016

Nevada Irrigation District
1036 W. Main Street
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Attention: Mr. Doug Roderick, P.E.

Subject: Centennial Reservoir Project, Preliminary Geotechnical Investigation
Phase II Report – Final

Dear Mr. Roderick:

We are very pleased to submit this final Phase II Report for the Centennial Reservoir Project located near Grass Valley, California. The purpose of the Phase II study was to perform a geotechnical investigation to provide an assessment of the site conditions for the proposed dam, and to evaluate the potential dam site locations and dam types.

This final report was prepared as part of the Phase II scope of work and builds upon the draft Phase I report submitted to NID on September 30, 2015. Conceptual designs of the preferred dam types are presented. This report also includes recommendations for further dam foundation and borrow area investigations and design studies that are necessary to support the next phase of the project and the EIR document.

This Phase II Report presents the following:

- Overview of the geologic setting and site conditions based on a data review;
- Seismic source characterization, historical seismicity, deterministic seismic ground motion parameters, and reservoir triggered seismicity;
- Geologic characterization of foundation soil and rock formations, soil conditions, rock conditions, and geologic hazards;
- Field geotechnical investigation (core drilling and geophysics) and laboratory testing program;
- Dam foundation conditions at the two preferred axis locations identified in Phase I;
- Potential on-site borrow materials and off-site commercial construction material sources;
- Conceptual design considerations of the preferred dam types; and
- Conclusions and recommendations on the selection of the preferred dam sites, dam types, and recommendations for future geotechnical investigations of the selected dam site and borrow areas.

We are available to discuss this report with you. Please contact me at (510) 874-3012 if you have any questions.

Sincerely,
AECOM Technical Services, Inc.

M.P. Forrest

M.P. Forrest, P.E., G.E.
Project Manager

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Enclosure:
Centennial Reservoir Project, Preliminary Geotechnical Investigation
Phase II Report – Final

Cc: Noel Wong, Ted Feldsher, Julien Cohen-Waeber, Dave Simpson (AECOM)

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List of Acronyms

ASTM	American Society for Testing and Materials
CFRD	concrete-faced rockfill dam
CGS	California Geologic Survey
CRP	Centennial Reservoir Project
DSHA	deterministic seismic hazard analysis
DSOD	California Division of Safety of Dams
ft	feet, foot
ft/s	feet per second
g	acceleration of gravity
GPS	global positioning system
ICOLD	International Committee on Large Dams
km	kilometer
M	moment magnitude
m_b	body-wave magnitude
M_L	Richter local magnitude
NID	Nevada Irrigation District
PEER	Pacific Earthquake Engineering Research
PGA	peak ground acceleration
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
P-wave	compressional wave
RCC	roller compacted concrete
RQD	rock quality designation
RTS	reservoir triggered seismicity
S-wave	shear wave
USBR	U.S. Bureau of Reclamation
UCS	unconfined compressive strength
USCOLD	United States Committee on Large Dams (now United States Society on Dams)
USGS	United States Geological Survey
WGNCEP	Working Group on Northern California Earthquake Probabilities

1 Introduction

1.1 Background

The Nevada Irrigation District (NID) is initiating engineering and planning studies for a proposed water storage reservoir located on the Bear River between the existing Rollins and Combie Reservoirs, which are also owned and operated by NID. In order to advance the engineering for the proposed project, called the Centennial Reservoir Project (CRP), the NID retained AECOM to perform a study of the site, including geologic mapping and preliminary geotechnical investigations, to assist in identifying preferred dam axis locations and preferred dam types for further study. The study, which is documented in this report, was authorized under the agreement between AECOM and NID dated April 15, 2015.

The proposed dam site on the Bear River was first identified and evaluated by NID in the 1920's (Tibbetts, 1926). The dam site area is located in Nevada County on the north side of the Bear River and in Placer County on the south side. The site area lies at the upstream end of Combie Reservoir and about 7 miles downstream from Rollins Dam (Figure 1-1). NID has identified a storage capacity objective of 110,000 acre-feet for the site. This corresponds to a maximum normal reservoir water surface of approximately elevation 1855. Retaining a reservoir at this elevation would require a dam height of approximately 275 feet above the Bear River, depending on the dam type, spillway design, and freeboard criteria.

The preliminary geotechnical investigation scope of work was divided into two phases. The Phase I work was described in a draft report dated September 30, 2015 (AECOM, 2015). Phase I included studies of the general geologic setting and site conditions, to identify factors considered most significant to dam site and dam type selection. The Phase I scope of work included the following main tasks:

- Task 1: Perform literature review
- Task 2: Conduct geologic reconnaissance
- Task 3: Conduct seismological investigation
- Task 4: Perform geologic mapping of the dam site area
- Task 5: Identify potential dam site(s) and dam type(s)
- Task 6: Prepare Phase I Report with recommendations for Phase II investigations

The Phase I Report included an assessment and characterization of the foundation soil and rock conditions along the identified potential dam axis alignments, along with a discussion of the preferred axis locations and dam types considered most viable for the site. The report also presented a recommended program of subsurface investigations.

The geotechnical site investigations were carried out under Phase II and are documented in this report. The Phase II investigation objectives are to further characterize and confirm the subsurface conditions along the most favorable axis locations and to assess the foundation suitability for construction of the most viable dam types.

This final report was prepared as part of the Phase II scope of work and incorporates the Phase I work. The additional geologic and geotechnical information obtained during Phase II field and laboratory investigation program is included, summarized and evaluated as it pertains to dam location and dam type selection. Conceptual layouts for the preferred dam types are also presented. This report also includes recommendations for further dam foundation and borrow area investigations that will be needed under subsequent phases of the project.

1.2 Purpose and Scope of Report

The overall purpose of the study was to perform a geotechnical investigation and to provide an assessment of the site conditions for the proposed dam, and to evaluate the potential dam axis locations and dam types. The Phase II scope of work included the following main tasks:

- Task 1: Conduct exploratory drilling
- Task 2: Excavate test pits (this task was later removed from the scope)
- Task 3: Conduct engineering geophysical surveys
- Task 4: Obtain field samples and perform testing (including borehole water pressure testing and downhole geophysics)
- Task 5: Perform laboratory testing
- Task 6: Prepare the Phase II Final Report

1.3 Organization of Report

After this introductory section, this report is organized into the following sections:

- Section 2 presents an overview of the geologic setting and site conditions based on a data review.
- Section 3 discusses seismic source characterization, historical seismicity, deterministic seismic ground motion parameters, and reservoir triggered seismicity.
- Section 4 discusses geologic characterization of soil and rock formations, soil conditions, rock conditions (e.g., rock units, weathering, and joints and fractures), and geologic hazards.
- Section 5 discusses the field geotechnical investigation (core drilling and geophysics) and laboratory testing program.
- Section 6 discusses the dam foundation conditions at the two preferred axis locations identified in Phase I.
- Section 7 discusses potential on-site and off-site construction materials sources.
- Section 8 presents conceptual design considerations for the preferred dam types.
- Section 9 presents conclusions and recommendations.
- Section 10 lists the references used to prepare this report.

1.4 Acknowledgements

The following key AECOM personnel performed the work for this Phase II Report:

- Project Manager: Michael Forrest, P.E., G.E.
- Geologic Mapping: Julien Cohen-Waeber, P.E., C.E.G.; David Simpson, C.E.G.; and Phil Respess, C.E.G.
- Logging of core borings: Sheri Janowski, C.E.G. and Ben Kozlowicz, C.E.G.
- Seismologic Investigation: Ivan Wong, Patricia Thomas, Ph.D., and Judith Zachariasen, Ph.D.
- Independent Technical Review: Theodore Feldsher, P.E., G.E.
- Advisory Review: Noel Wong, P.E., and Lelio Mejia, P.E., G.E., Ph.D.

Holdrege & Kull, of Nevada City, California, performed the reconnaissance for the clay borrow materials and conducted laboratory testing on rock cores.

Norcal Geophysical, of Cotati, California, performed the seismic refraction surveys and downhole geophysics (televiewer logging, caliper logging, and P- and S-wave measurements).

Ruen Drilling, of Modesto, California (main office in Clark Fork, Idaho), performed the core drilling.

The Nevada Irrigation District coordinated site access, constructed drill rig pads, cleared access routes to drill sites, and cleared the seismic refraction survey line areas.

1.5 Limitations

The professional judgments presented in this report regarding the site conditions are based on information obtained from reference data review, site reconnaissance, geologic mapping, and a limited preliminary geotechnical investigation.

AECOM represents that its services were conducted in a manner consistent with the standard of care ordinarily applied as the state of practice in the profession within the limits prescribed by our client. No other warranties, either expressed or implied, are included or intended in this report.

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2 Geologic Setting and Overview of Site Conditions

2.1 Reference Sources

This section describes the geologic setting and overview of site conditions and is based on a literature review. The results of this work formed the basis for the reconnaissance, mapping and geotechnical investigation that followed. Data was obtained for review from sources including NID, the California Geologic Survey (CGS), the United States Geological Survey (USGS), and others. The reviewed data included relevant mapping, published and unpublished documents, and available NID documents pertaining to the project site and other sites in the vicinity. Specific items reviewed included but were not limited to the following:

- Regional and local geologic and tectonic characterizations
- Data pertaining to material properties at and around the site
- Geologic and geotechnical evaluations of local dam sites within a 10 mile radius and their construction records
- Available ground surface imagery and topographic data for remote characterization of the site specific geomorphology
- Available subsurface data from other sites in the region for preliminary characterization of the site-specific rock mass properties.

The reference documents reviewed for this effort included the following:

- Berlogar Geotechnical Consultants, Seismic Deformation Analysis of Rollins, Dutch Flat Forebay and Dutch Flat Afterbay Dams, March 1997.
- Chandra, D., Geology and Mineral Deposits of the Colfax and Foresthill Quadrangles California, Special Report 67, California Division of Mines, 1961.
- Civil drawings and construction records for Combie Dam, beginning 1927.
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- Day, H.W., Moores E., Tuminas A.V., Structure and Tectonics of the Northern Sierra Nevada, Geologic Society of America Bulletin, v. 96, p. 436-450, April 1985.
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- Engeo, Combie Dam Geotechnical Report, 2013.
- Geomatrix Consultants, Review of Seismic Stability of Rollins and Dutch Flat Afterbay Dams, August 1999.
- Hamilton, D. H., Seismotectonic Review and Evaluation for Nevada Irrigation District Dams, March 2007.
- James, A., Time and the Persistence of Alluvium: River Engineering, Fluvial Geomorphology, and Mining Sediment in California, Geomorphology, 31, p. 265-290, 1999.
- LiDAR Data, prepared by ECorp for NID, November 2014.
- Mark Group, Preliminary Design of Modifications to Combie Dam, May 1990.
- Marliave, C., Rollins Dam Site, Letter Report 1933.
- Nevada Irrigation District, Bear River Soil Survey Data.
- Norcal Geophysical Consultants, Seismic Refraction Investigation Combie Dam, Report, April 2013.
- Norcal Geophysical Consultants, Geophysical Investigation Rollins Reservoir Bear River Arm, August 2014.
- PG&E, Earthquake Magnitude Evaluation of Potential Seismic Sources for Rock Creek (Drum) Dam, June 1991.
- QEST Consultants, Stability Analysis of Combie Dam FERC Project No. 2981, March 1987.
- Tibbetts, F., Bear River Diversion and Storage, study for Nevada Irrigation District, February 1926.
- Tuminas, A. V., Structural and Stratigraphic Relations in the Grass Valley-Colfax Area of the Northern Sierra Nevada Foothills California, PhD Dissertation, University of California Davis, 1983.

2.2 Regional Geologic Setting

The proposed site for the CRP is located on the Bear River, Nevada County, California, in the Central Belt of the northern Sierra Nevada geomorphic province. An excerpt from the Geologic Map of Grass Valley-Colfax Area (Tuminas, 1983) is included on Figure 2-1. The proposed reservoir plan and dam site area are shown on Figure 2-2. As shown in an inset on Figure 2-2, seven potential dam axis alignments were initially identified in the dam site area. As described in Section 6, these potential axis alignments were developed based primarily on topographic considerations, and were further screened, evaluated, and refined as part of this study.

In the dam site area, the Central Belt is described as being "composed of diverse ultramafic, plutonic, volcanic, and sedimentary rocks that have been variably metamorphosed at low or medium grade, affected by one or more periods of isoclinal folding, disrupted by numerous faults, and intruded and metamorphosed by granitic plutons of the Late Jurassic to Early Cretaceous age" (Day, et al., 1985). The site is located on the eastern limb of the Lake of the Pines Syncline, within the upper stratigraphic section of the Lake Combie Complex. The bedrock at the site is composed of the Lake Combie Upper and Middle volcanoclastic and epiclastic units which include massive flow rock, flow breccia and sandstones, with bedding dipping slightly to the west. Regional sub-vertical fracture planes dipping to the west have also been reported (Tuminas, 1983). Based on massive granitic intrusions located less than 3 miles southwest of the site, the bedrock is also expected to be metamorphosed to varying degrees with a potential for local plutonic intrusions. Preliminary field observations confirm that the site area is located within a massive meta-volcanic unit with bedding and fracture attitudes consistent with previous reports.

The project site area is also bounded to the east and west by the Weimar Fault Zone and the Wolf Creek Fault Zone respectively, which are both part of the greater Foothills Fault System (see Section 3). The Weimar Fault zone is approximately 1.25 miles (2 km) due east of the site, while the Wolf Creek Fault Zone is approximately 3.75 miles (6 km) due west. Both fault zones trend NNW, are steeply dipping both east and west and have varying thicknesses of 300 feet to 2.5 miles. Historically, the region is likely to have experienced multiple phases of faulting, beginning with an overthrust with east directed movement, then dip-slip reverse movement, followed by right lateral strike slip movement and reverse or oblique reverse movement (Tuminas, 1983). Though the Weimar Fault Zone is not believed to have been active during the Quaternary (1.8 million years ago), the Wolf Creek Fault Zone is believed to have been active within the Late Quaternary (700,000 years ago). A seismotectonic discussion of the project area is included in Section 3.

2.3 Local Geologic and Geomorphic Setting

2.3.1 Geologic Setting

In addition to the regional geologic setting described above (Tuminas, 1983; Day, 1985 and 2004), a site-specific geologic setting was developed based on additional review of local geotechnical and geological data from nearby dam sites (Combie and Rollins Dams) and rock mass data from a quarry adjacent to the site. A quarry located approximately ½ mile south-southwest of the site provides clear exposures of rock structure. Based on surface exposures and on limited subsurface data made available by the quarry, the local bedrock is generally characterized as hard to very hard massive greenstone or meta-basalt and meta-volcanic breccia, metamorphosed to varying degrees (predominantly lightly). Structurally, the bedrock generally dips to the west with prominent sub-vertical discontinuities trending generally north-south and east-west.

Combie and Rollins Dams are both situated within a 10 mile radius from the CRP site. Combie Dam is located approximately 3 miles downstream on the Bear River and Rollins Dam is located approximately 7 miles upstream. The investigation and construction records for each dam generally confirm the regional geologic setting and similar local geologic features to those described above.

2.3.2 Geomorphic Setting

Ortho-photographic imagery and a LiDAR produced digital elevation model for the project site were reviewed. The reviewed aerial photographs are summarized in Table 2-1. The purpose of this effort was to provide insight on accessibility to the site, as well as the existing geomorphic setting. The valley morphology presents generally steeper east- and south-facing slopes ranging from approximately 2 horizontal:1 vertical (2H:1V) to subvertical cliffs of rock outcrops in comparison to flatter north- and west-facing slopes which are generally flatter than 2H:1V with occasional vertical cliffs formed by outcropping rocks.

In the area north of the Bear River, roughness in the terrain made apparent by the LiDAR data is indicative of shallow bedrock along a majority of the slopes. The air photos also confirm steep rock outcrops at the southern end of the site defining a prominent ridge and extending approximately 200 feet above the Bear River. In the area south of the Bear River, smoother slopes in the LiDAR data suggest generally thicker residual soil deposits. The data also confirms a lower lying east-west trending outcrop of steep rock in the southern portion of the site extending approximately 100 feet above the Bear River.

The geologic setting at the site is generally reflected by the LiDAR and aerial photograph data. Steeper east facing slopes are indicative of west-dipping bedding where west facing slopes may be flatter due to dip-slope conditions. The Bear River also makes a series of clear and relatively orthogonal bends trending generally north-south and east-west, which are likely controlled by the bedrock structure and regional shears.

Several anomalous geomorphic features are evident from the LiDAR data. In the area south of the Bear River, a landslide deposit is apparent above the outside bend of the river where it sharply turns west through the site. Approximately 1000 feet downstream, a topographic swale is evident, suggesting a thick colluvial deposit at the top of a natural drainage path. On the north side of the Bear River, a combination of topographic depressions and bulges suggest the existence of at least two small landslides and one large debris deposit, which may have been man-made. Some evidence of historic mining activities exists in the site area, which may account for several of the observed geomorphic anomalies.

Table 2-1. Aerial Photographs Reviewed

Film ID	Line	Frame	Scale	Date
AV 4130	20	8/9	1:63360	09-12-91
AVP 4095	12	56/57	1:36000	07-30-91
AV 3600	20	1	1:63300	05-05-89
AV 3192	15	1	1:40000	11-15-87
GS-VFLL-C	1	4/5/6	1:24000	07-26-87
GS-VDXQ	1	9/10	1:80000	08-29-75
GS-CW	5	148/149/150	1:28400	02-22-47

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3 Seismologic Investigation

This section presents the preliminary seismologic investigation completed for the project site. The scope included the following elements: (1) seismic source characterization, (2) historical seismicity, (3) deterministic seismic ground motions, and (4) evaluation of the potential for reservoir triggered seismicity. Details of the seismologic investigation are presented in Appendix A and summarized below.

3.1 Seismic Source Characterization

3.1.1 Foothills Fault System

The west-central portion of the Sierra Nevada block, which includes the proposed CRP site, contains late Cenozoic faults that have reactivated portions of the 360-km long Mesozoic Foothills fault system (Page and Sawyer, 2001). The Foothills fault system is complex and its paleoseismic history is still not well known. The faults of the Foothills fault system nearest the project site are the Wolf Creek-Big Bend fault, approximately 6 km west of the project site, and the Weimar fault, is approximately 2 km east of the project site.

3.1.2 Lineament Observations

The study included review of LiDAR data in the immediate vicinity of the project site and 1975 and 1978 U.S. Geological Survey (USGS) black and white stereo aerial photography in a wider region encompassing the breadth of the Foothills fault system and extending about 25 km north and south of the project site.

Based on the analysis of the photographs and the LiDAR, a preliminary lineament map was developed (see Appendix A). The mapped lineaments include topographic lineaments, along with vegetation and tonal lineaments. These are in places associated with linear erosion features, linear drainages, topographic steps, and range fronts. The mapped lineaments may be associated with faults, but lineaments can also be produced by other processes including fluvial and gravitational processes, differential erosion of different rock types, and jointing.

The analysis shows that many of the longer and more prominent lineaments are coincident with previously mapped faults of the Foothills fault system. In addition to these long lineaments, numerous shorter and less prominent lineaments were also observed. However, due to the short lengths of these features, and the lack of apparent continuity between them, they were concluded not to represent new (unmapped) tectonic faults in the study area. This analysis was conducted at a relatively small scale, and has a relatively high degree of confidence.

3.2 Historical Seismicity

The area of the proposed dam site has experienced very few historical earthquakes (see Figure 3-1). The only reported events of magnitude **M** 5.0 or larger within 65 km of the proposed dam site during the time period from 1855 to 2014 are the following:

- August 1, 1975: Richter local magnitude (M_L) 5.7 (body-wave magnitude, m_b, 5.9) Oroville earthquake that occurred about 60 km to the northwest of the proposed dam site.
- September 12, 1966: **M** 5.9 earthquake occurred near Boca, California, a distance of 55 km east-northeast of the proposed dam site.
- March 3 and June 23, 1909: Two **M** ≥ 5 events occurred 41 and 44 km northeast of the dam site. These events include a M_L 5 earthquake on March 3 and a M 5.5 event (unknown magnitude scale) on June 23.

3.3 Deterministic Seismic Ground Motions

3.3.1 Earthquake Magnitude

The maximum earthquake for any fault within the Foothills fault system is considered to be M 6.5 with a surface rupture length of less than 20 km. This is consistent with the maximum magnitude considered by the Working Group on Northern California Earthquake Probabilities (WGNCEP, 1996), Schwartz et al. (1996), Page and Sawyer (2001), and the 2008 USGS National Hazard Maps (Petersen et al., 2008).

3.3.2 Deterministic Seismic Hazard Analysis and Preliminary Seismic Design Parameters

As discussed above, the closest faults to the site are the Wolf Creek-Big Bend and Weimar faults of the Foothill fault system. A deterministic seismic hazard analysis (DSHA) was performed to develop preliminary design ground motions for the proposed dam site. To carry out the DSHA, site-specific 5%-damped median, 69th and 84th percentile horizontal acceleration response spectra were developed for a maximum earthquake of **M** 6.5 on the Wolf Creek fault.

To estimate the ground motions, recently developed ground motion prediction models appropriate for tectonically active crustal regions were used. The crustal models were developed as part of the NGA-West2 Project sponsored by Pacific Earthquake Engineering Research (PEER) Center Lifelines Program.

The 69th percentile deterministic spectra developed for each of the four ground motion prediction models along with the geometric mean are presented in Appendix A. The median, 69th and 84th percentile geometric mean deterministic spectra are also compared in Appendix A. The median, 69th and 84th percentile peak horizontal ground accelerations (PGAs) are 0.23, 0.31 and 0.42 g, respectively.

Based on DSOD guidelines (Fraser and Howard, 2002), the minimum earthquake peak ground acceleration (PGA) for new and existing dams should be 0.25 g. Considering this, AECOM recommends the use of 69th percentile deterministic ground motions for design of the proposed dam. This is consistent with DSOD guidelines and recommendations by U.S. Committee on Large Dams (1985; 1998).

At this phase of the project, a site-specific probabilistic seismic hazard analysis (PSHA) was not included in the scope of work. However, a site-specific PSHA should be considered when developing ground motions for final design. Guidelines from the International Committee on Large Dams (2010) recommend a range of design ground motion return periods from 3,000 to 10,000 years, with the appropriate return period depending on the risk rating of the dam. A PSHA would be needed to develop ground motions with these return periods.

3.4 Reservoir Triggered Seismicity

In California, at least eight reports exist of possible reservoir triggered seismicity (RTS) (Wong and Strandberg, 1996; Knudsen *et al.*, 2009). Perhaps the most notable of these cases is Lake Oroville, which may have triggered the occurrence of the 1975 M_L 5.7 Oroville earthquake (Toppozada and Morrison, 1982). Lake Oroville is located in a setting that is geologically, tectonically and seismically similar to NID's proposed CRP, so the risk of RTS needs to be considered.

For the purposes of evaluating the risk of RTS, the proposed CRP would be classified as a shallow and small reservoir (see Appendix A for classification criteria). Although lineaments have been mapped in the proposed reservoir area, including a possible continuation of the Weimar fault, no historical seismicity has been observed in the vicinity of the proposed reservoir. Based on these factors and on previous analyses for other sites, it appears that RTS has a low probability of occurrence at the proposed reservoir site, but should nonetheless be considered for design.

The RTS earthquake recommended for design is a **M** 6.5 event, which is consistent with the maximum event assigned to faults within the Foothill fault system as described above in Section 3.3.2. This RTS event is also consistent with the background seismicity considered significant to the reservoir, and is therefore not expected to control the design.

4 Geologic Characterization of Soil and Rock Formations

4.1 General

The CRP proposed dam site area is located on the Bear River with potential right abutment sites on the north bank of the river and left abutments on the south bank. The abutment slopes are moderately steep to sub-vertical, forested and drained by steep intermittent seasonal streams. Preliminary surface observations indicate that the site is primarily underlain by variably weathered and metamorphosed basaltic rock. Active faults have not been identified at the site (Section 3). Four landslide deposits were observed in the dam site area. Based on the general site conditions, topography, and field observations, seven potential axis alignments in the dam site area were initially identified for study (as shown in Figure 2-2).

Geologic field mapping of the project site was performed in June and July 2015. The results of the geologic mapping are presented on Figure 4-1, which shows geologic outcrops and geologic structural data overlain on a LiDAR-derived topographic map, along with the locations of the seismic refraction survey lines and core borings (discussed in Section 5).

4.2 Geologic Mapping

Geologic field mapping was performed to confirm and augment the findings from the background data review presented in Section 2. The method consisted of walking traverses across the site and visiting specific areas of interest identified during the preliminary desktop study. The field mapping effort was facilitated by NID, who cleared brush to allow access along key pathways. Along each traverse, bedrock outcrops were characterized based on the rock type, degree of weathering and discontinuity orientations. The collected data was compiled electronically using a hand-held GPS and was also recorded on a topographic base map. Compiled data are presented in Appendix B. Steep terrain, heavy brush, and the presence of water in the river at the time of the mapping work limited access to significant portions of the site area.

4.3 Surficial Conditions

4.3.1 Soils and Bedrock Weathering

During the geologic mapping effort, observations of surficial soil deposits were made primarily along road cuts and deeply incised runoff channels. The soil deposits appear to thicken with increasing elevation above the Bear River, but are generally thin throughout the site, as confirmed by the presence of many bedrock outcrops. Soils exposed in road cuts varied from sandy silt to sandy clay to silty sand, with gravel and bedrock fragments throughout and increasing frequency with depth. The soils represent a typical colluvial/residual weathering profile, and are a product of the underlying rock. Photograph 1 illustrates a typical section of bedrock and shallow residual soil observed within a road cut near the CRP site.

Based on observations of bedrock depths from the geotechnical data discussed in Section 6, the bedrock weathering profile at the CRP site is variable. Limited exploratory borings and seismic refraction studies at Combie Dam (Engeo, 2013) suggest regional residual soil thicknesses of 5 to 10 feet. Civil drawings for Combie Dam (NID, 1928) indicate that design excavation depths across the dam abutments were estimated to reach up to approximately 10 to 15 feet. Based on “As Built” drawings for Combie Dam (NID, 1932), the actual depth of excavation to foundation material was up to approximately 20 to 25 feet. Seismic refraction testing at Combie Dam yielded P-wave velocities greater than 11,000 ft/s in the “less weathered bedrock” starting at depths of approximately 15 feet (Norcal, 2013).

4.3.2 Alluvium

The sands and gravels currently in Lake Combie and the Bear River channel are primarily a result of gold mining in the early 1880’s (Dupras, 1984). These operations used hydraulic mining to process large amounts of channel sands and gravels within 20 miles upstream of the CRP site. The deposit is naturally sorted by fluvial activity, resulting in high quartz content, hard, well-rounded sand, gravel and cobbles. The original construction drawings for Combie Dam (NID, 1928) indicate a design depth of excavation in the river channel of approximately 10 to 15 feet. "As Built" drawings for Combie Dam (NID,1932) confirm channel deposit thicknesses of approximately 0 to 15 feet.

Alluvial deposits are present in the CRP dam site area between the abutment slopes and across the Bear River channel, which has a width of between 150 and 300 feet. The deposits consist of sandy gravel to gravelly sand and locally contain cobbles and boulders. The gravels and larger clasts include granite, quartzite, and vein quartz from rock which is not present on the project site. These materials were derived from higher up to the east in the Bear River drainage basin. Though none was observed during the geologic mapping effort, the alluvial deposits may also contain chert based on the mapping performed by Tuminas (1983). Alluvium is present variably within the river channel, which also contains extensive rock outcrops. The approximate limits of the alluvium in the site area are shown on Figure 4-1. Bedrock outcrops in the active river channel are also shown in Figure 4-1. Photographs 2 and 3 illustrate the Bear River alluvial deposits.

4.3.3 Springs

One groundwater spring was observed above the south bank of the river at about elevation 1700 near the western edge of the dam site area (Figure 4-1). This spring (seep) was present each time the site was visited from December 2014 through November 2015.

4.4 Bedrock Conditions

4.4.1 General

The project site area is located in the Central Belt of the northern Sierra Nevada geomorphic province, within the upper Lake Combie Complex. This geologic unit includes variably metamorphosed mafic volcanic formations. The more strongly metamorphosed portions are referred to as greenstone (metamorphosed basalt). Based on surficial geologic mapping, bedrock at the site is composed of basalt, some of which may be slightly metamorphosed.

4.4.2 Rock Description

The observed outcrops on both the north and south sides of the river canyon are comprised of similar rock. The rock is massive, dense, hard, strong, black to gray, fine grained, generally unweathered to slightly weathered basalt. The outcrops display widely spaced steep joints and gently inclined volcanic flow "bedding" surfaces. Rock outcrops are present in many places along the toe of the slopes near the river and in the active river channel. Near the river many of the outcrops are present as cliffs. A tall cliff on lower half of the north bank of the river near the western edge of the site area was too steep to safely access on foot. Within the river channel many of the observed outcrops have been smoothed by fluvial erosion. Figure 4-1 shows the locations of the observed rock outcrops with recorded strike and dip information as well as the locations of observed possible landslides. Photographs 4 and 5 illustrate the bedding and joint conditions of the bedrock. Photographs 6 and 7 illustrate the general morphology of the site.

4.4.3 Rock Structure Observed in Outcrops

Rock structure orientations were measured on a total of 76 joints and 30 volcanic flow "bedding" surfaces on rock outcrops in the study area. The discontinuity locations are shown on Figure 4-1 and the data are presented in Appendix B. The principal discontinuity sets from the data are summarized in Table 4-1 and plotted on Figure 4-2. Figure 4-2 shows stereonet plots displaying contoured orthogonal poles to the measured planar surfaces. Figure 4-2(a) shows consistent south-southwest dipping volcanic flow bedding orientations, with a concentration of strikes and dips centered at N68^{o}W (112^{o} azimuth), 10^{o}SW. Figure 4-2(b) shows three distinct pole concentrations representing three prominent steeply dipping joint planes (joint sets 1 to 3). Figure 4-2(b) also shows two secondary pole concentrations (joint sets 4 and 5).

Table 4-1. Discontinuity Sets from Geologic Mapping

Strike (Degrees Az.)	Dip (Degrees)	Discontinuity Type	No. of Data points
112	10 SW	Bedding	30
22	78 SE	Joint Set 1	33
84	87 S	Joint Set 2	15
296	81 NE	Joint Set 3	12
158	60 SW	Joint Set 4	5
327	72 NE	Joint Set 5	4

The mapped joint orientations were observed to be relatively persistent throughout the study area on both sides of the Bear River canyon. The joint surfaces observed in outcrops were generally slightly wavy, smooth to slightly rough, very narrow to tight, and with narrow bands of weathering along the joint surfaces. Geometry, roughness and weathering of the bedding and bedding-parallel joints were similar to the other joints.

4.5 Geologic Hazards

4.5.1 Landslides and Rockfalls

One active landslide was mapped on the south canyon slope adjacent to a sharp turn in the river, as shown in Figure 4-1. This landslide extends upslope 140 feet from the left bank of the river channel to approximately elevation 1760 feet. The west side scarp is prominent and clearly visible as is the hummocky nature of the ground surface and lack of large fir and pine trees. The head scarp and east side scarp are more subdued and not as obvious. The depth of this landslide appears to be about 30 to 40 feet from surficial observations. Three other possible landslide deposits were mapped within the slopes north of the Bear River, as shown on Figure 4-1. These possible landslide deposits are visible in the LiDAR hillshade maps but were not confirmed in the field. Small rockfalls have also occurred at some of the larger rock outcrops. Rocky rubble surrounding the steep cliffs at the right abutment near the downstream edge of the site area is interpreted as rockfall debris.

4.5.2 Faults

As stated in Section 3, no faults considered active by either the California Geological Survey or US Geological Survey are mapped within the CRP dam site area. During the mapping for this project, two minor inactive faults or shear zones were observed within the nearby rock quarry. Based on observations of the sheared bedrock zones, these features are about 5 to 15 feet wide in outcrop, and are both very steep to vertical. The first is oriented N55°E and the second between N10°E and N12°W. These features appear to coincide with a regional discontinuity fabric that follows local meanders of the Bear River. The second feature strikes directly toward the north-south stretch of the Bear River within the CRP dam site area. The rock observed within these features is highly fractured, sheared and discolored, but the fabric remains hard and strong. Slickensides present along shear surfaces indicate a strike slip sense of past movement. Rock surrounding the upper portion of the north-striking feature is exposed high on the north wall of the quarry as a zone of reddish brown discolored and possibly weathered rock that widens closer to the original ground surface. Photographs 8 and 9 illustrate the minor inactive fault or shear zone features observed in the quarry.

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5 Geotechnical Investigation

5.1 General

The geotechnical investigation focused on cost-effectively obtaining the data needed to evaluate the technical feasibility of the potential dam sites and dam types. The primary emphasis was on identifying significant geologic flaws or other undesirable foundation conditions present in the areas investigated. The investigations also obtained data to help evaluate foundation excavation depths, rock strengths, potential seepage conditions, and likely treatment requirements. The investigations obtained data to facilitate technical comparisons of potential dam axis locations and dam types.

5.2 Seismic Refraction Surveys

A total of 12 surface seismic refraction surveys were performed by Norcal Geophysical Consultants of Cotati, California, between August 24 and 28, 2015, and on November 4, 2015. The initial 10 seismic lines performed in August were located to obtain general site coverage. Seismic refraction line 2-5 was performed in November to obtain data between the east end of seismic line 2-3 and the west end of seismic line 4-3. Seismic line 6-4 was also performed in November to obtain data uphill of seismic line 6-3. The seismic refraction survey line locations are shown on Figure 4-1.

The surveys were performed with line lengths of 300, 600 and 900 feet. The results were used to characterize subsurface conditions in two potential dam axis areas. The survey locations were selected to help assess the depth and degree of weathering of bedrock, to evaluate the potential significance of local topographic features, and to aid in locating the core borings. Due to the steepness of the right abutment areas, the survey lines in these areas were oriented parallel to the topographic contours. However, the left abutment slopes are flatter, so two of the seismic lines (seismic line 2-4 at Axis 2 and seismic line 6-1 at Axis 6) were oriented perpendicular to the topographic contours. Seismic line 2-2 was located to check the depth of weathering in a surface drainage feature.

The seismic refraction survey methodology and results are presented in Appendix C.

5.3 Core Drilling and Water Pressure Testing

Between October 13 and November 13, 2015, eight HQ-size (2.4-inch diameter core), triple tube core barrel borings (CB-1 to CB-6, CB-8 and CB-9), were drilled in general accordance with ASTM D2113. Four borings each were drilled at Axis 2 (CB-1, -2, -3 and -4) and Axis 6 (CB-5, -6, -8 and -9) at the locations shown on Figure 4-1.

The borings were drilled by Ruen Drilling of Modesto, California. Two LF-70 track-mounted drill rigs were mobilized for this investigation. The borings ranged in length from 143 to 300 feet, for a total drilled length of 1612 linear feet. All borings were located by GPS. The summary of the core borings and in situ testing is presented in Table 5-1. The core boring logs are presented in Appendix D. The cores were photographed after they were placed in wooden core boxes and also individual runs were photographed in the inner split core barrels. The core box photographs are presented in Appendix D along with the core boring logs.

Due to economic constraints on this preliminary geotechnical investigation, the core boring locations were selected at sites suitable for track-mounted drill rigs and water truck access. The core borings were located with specific objectives as noted in Table 5-1. Inclined borings were used due to the presence of near-vertical joint sets (see Section 4.4.3). Inclined borings were necessary in order to intersect and characterize the frequency and hydraulic conductivity of the near-vertical joints. The borings were inclined at either 60 or 75 degrees from the horizontal. Two of the borings were inclined at 75 degrees to accommodate P- and S-wave downhole velocity measurements (see Section 5.4).

Water pressure (packer) testing was performed to assess the foundation hydraulic conductivities. The testing was performed in general accordance with USBR Method E-18, with tests in stages (generally 20 to 30 feet long) in the borings. The test results provide data for seepage analyses and to assess foundation grouting requirements. The water pressure test data is included in Appendix E. Photographs 10 and 11 show the typical core drilling operation.

After completion, each boring was tremie cement grouted from the bottom to the ground surface. The core boxes were temporarily stored in the Bear River Quarry south of the dam site while the core logs were reviewed and point load index testing was performed (see Section 5.5). The core boxes were then moved to a "Conex" container in NID's yard for long-term storage.

5.4 Downhole Geophysics

The investigation also included downhole in-situ testing to obtain additional data to characterize the rock mass conditions. The downhole geophysical testing was performed by Norcal Geophysical Consultants during the drilling program and included televiewer logging, caliper logging and downhole geophysical velocity measurements.

Televiewer logging [optical and acoustic (below water level within the borings)] was done to measure the orientation of discontinuities (e.g., joints and shears) to characterize the rock mass and foundation conditions and for use in stability analyses. Caliper logging that shows the diameter of the borehole walls was also obtained along with the televiewer logs. Televiewer and caliper logging was performed in all eight core borings and the data are presented in Appendix F.

Downhole geophysics (Oyo suspension logging) was done to measure compression (P) and shear (S) wave velocities to characterize bedrock weathering profiles with depth and to provide data for dynamic site response analyses. P- and S-wave velocities were measured in boring CB-2 (Axis 2) and in boring CB-5 (Axis 6). The P- and S-wave measurement methodology and test results are presented in Appendix G.

5.5 Rock Strength Testing

Selected rock cores were tested by Holdrege & Kull Consultants for unconfined compressive strength (UCS) (ASTM D7012) in the laboratory. The lengths of the cores that were selected for UCS testing were generally at least twice the diameter. Correction factors were used to correct for shorter length cores. Point load index tests (ASTM D5731) were performed on rock cores by AECOM to supplement the laboratory strength test data. These data are useful for rock strength characterization and evaluation of dam foundation suitability. The UCS test data and point load index test data are presented in Appendix H and Appendix I, respectively.

Table 5-1. Summary of Rock Core Borings

Boring No.	Location	Purpose	Approx. Surface Elev. (ft)	Boring Length (ft)	Boring Inclination		HC	TV/Caliper	P- & S-Wave Velocity Surveys	Nearby geophysical survey lines
					Dip (Degrees)	Direction				
CB-1	Axis 2 South	Upper abutment foundation rock quality, weathering zone depression and profile	1788	199.7	60	N	X	X		SR 2-1 and SR 2-4
CB-2	Axis 2 South	Lower abutment foundation rock quality and weathering profile	1723	178	75	N	X	X	X	SR 2-2 and SR 2-4
CB-3	Axis 2 North	Lower abutment foundation rock quality and weathering profile	1883	254.2	60	S	X	X		SR 2-3
CB-4	Axis 2 North	Upper abutment foundation rock quality, geomorphic saddle, weathering zone depression and profile	1883	154.5	60	N	X	X		SR 2-4
CB-5	Axis 6 East	Upper abutment foundation rock quality, geomorphic saddle, weathering zone depression and profile	1853	180	75	E	X	X	X	SR 6-1
CB-6	Axis 6 East	Middle abutment foundation rock quality, weathering zone depression and profile	1745	300	60	W	X	X		SR 6-1 and SR 6-2
CB-8	Axis 6 West	Middle abutment foundation rock quality and weathering profile	1778	203	60	E	X	X		SR 6-3
CB-9	Axis 6 West	Upper abutment foundation rock quality and weathering profile	1778	143	60	W	X	X		SR 6-3
Total				**1612.4**						

NOTES:
1. HC = hydraulic conductivity, TV = televiewer
2. All rock core borings are HQ size.
3. Boring CB-7 not drilled.

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6 Dam Foundation Conditions

6.1 Initial Selection of Dam Axes

Seven potential dam axis alignments were identified for this preliminary geotechnical study of the CRP dam site as shown in Figure 2-2. The initial axis location (Axis 2) was identified based on the Tibbetts 1926 report. Additional potential axis locations were laid out for study based on the site reconnaissance and data review. Axis 4 is located at a right angle bend in the Bear River channel. The right angle bend is likely structurally controlled by intersecting shears and/or other discontinuities, which could complicate this site. As such this axis was eliminated from further consideration. Axis 3 could have similar issues as at Axis 4. Axis 1 (just downstream of Axis 2), Axis 5, and Axis 7 (close to Axis 6) do not appear to offer significant topographical advantages compared to Axes 2 and 6. Therefore, Axes 1, 3, 5 and 7 were also eliminated from further consideration.

Based mainly on the topographic conditions, the geologic site reconnaissance and seismic refraction surveys performed in August 2015, the two most promising axis locations that were identified for further study are Axis 2 and Axis 6. These locations offer relatively favorable topographic configurations for dams of the size being considered.

6.2 General

This section covers the general foundation conditions based on the core borings, seismic velocities, rock strength, and discontinuity data. Section 6.3 and Section 6.4 discuss specific dam foundation conditions at Axis 2 and Axis 6, respectively.

6.2.1 Rock Cores - Summary

Core borings CB-1, 2, 3 and 4 were drilled at Axis 2 (see Figure 6-1) and core borings CB-5, 6, 8 and 9 were drilled at Axis 6 (see Figure 6-2). The boring logs show that the degree of weathering is variable. All borings except CB-2 encountered significant depths of completely weathered to highly weathered, weak to very weak, and highly to intensely fractured rock, with low RQD values, mostly 0 to 30%. Below a certain depth, all borings encountered slightly weathered to fresh rock, which is much less fractured, with higher RQDs (frequently 100%), and is hard and strong. Table 6-1 summarizes the borings at both axes.

Table 6-1. Summary of Core Boring Results

Axis No.	Abutment	Boring No.	Total Drilled Depth* (ft)	Approx. Drilled Depth to Rock (ft)	Drilled Depth to Consistently Slightly Weathered/Fresh Rock (ft)
2	Left	CB-1	199.7	23	140
		CB-2	178	4	4
	Right	CB-3	254.2	3	71
		CB-4	154.5	3	122
6	Left	CB-5	180	3	42
		CB-6	300	14	142
	Right	CB-8	203	16	99
		CB-9	143	0	78

* Drilled depth is measured along the length of the angled boring.

6.2.2 Seismic Velocities

The results of seismic refraction surveys were used to evaluate the depth of weathering in the dam foundation to supplement data from the borings. As detailed in Appendix C, the seismic refraction profiles indicate seismic velocities that gradually increase from less than 1,000 to 14,000 ft/s within the upper 70 feet.

The surveys indicate weak, highly weathered and fractured bedrock and residual soils with P-wave velocities of less than 5000 ft/s ranging between 5 and 30 feet in thickness and occasionally greater. This layer appears to be thickest within the upper elevation areas and thinnest at lower elevations towards the river. The transition from weak weathered materials to more competent bedrock generally occurs over a 5 to 10 foot depth interval.

Beneath this low velocity layer, a zone of intermediate velocities (5,000 to 9,000 ft/s) is interpreted as moderately weathered and/or fractured bedrock. This zone is relatively consistent throughout most of the survey area. However, it increases in thickness at the east end of line 4-2 and the center of line 6-1. The highest velocity material (over 9,000 ft/s) is indicative of rock that is slightly weathered to fresh (unweathered). It is generally defined at lower elevations (less that about 1720 feet) beneath lines 2-2, 4-1 through 4-3 and line 6-3, and at higher elevations (up to 1800 feet) beneath line 6-1 and line 6-4. The north end of line 6-4 shows increasing depths to higher P-wave velocities.

The results of downhole seismic velocity measurements in CB-2 at Axis 2 and CB-5 at Axis 6 typically show S-wave velocities in the range of 5000 to 10,000 ft/s and P-wave velocities in the range of 15,000 to 20,000 ft/s. The downhole P-wave velocities approximate those from the seismic refraction surveys for slightly weathered to fresh rock. These high velocities correspond with the slightly weathered to fresh rock (see Figures 6-1 and 6-2).

6.2.3 Rock Strength

UCS tests were performed on selected testable core samples. Ideal test samples need to have a length-to-diameter ratio of about 2. Due to the close fracture spacing, only a few samples of moderately weathered rock were suitable for testing in unconfined compression in the laboratory. Testable samples are likely to be the better quality samples, so some degree of bias in the UCS test results toward the higher values is judged likely. Shorter cores were tested by a point load device in the field (Appendix I).

The UCS data (Appendix H) are summarized below by degree of weathering:

- Moderately to slightly weathered: 1900 to 8100 psi (3 tests)
- Slightly weathered to fresh: 2700 to 39,000 psi (14 tests)

The point-load test data (Appendix I) (filtered out to remove results from cores that broke along existing fractures) is summarized below by degree of weathering:

- Highly to moderately weathered: generally 2100 to 5400 psi
- Moderately to slightly weathered: generally 2100 to 16,000 psi
- Slightly weathered to fresh: generally 14,000 to 40,000 psi

As expected, the strengths increase with decreasing weathering. The slightly weathered to fresh rock has high strengths, with values up to 40,000 psi. There is substantial range of strengths for a given weathering degree, which can result from variable weathering and micro-fractures within the core samples.

6.2.4 Discontinuity Data

Downhole televiewer logging in each of the eight core borings produced a total of 758 discontinuity measurements with depth, which are summarized in Appendix F. The data points were initially classified as significant (class 1) to minor (class 3). Lower hemisphere equal angle stereonet pole plots were then prepared for each of the abutments along Axis 2 and Axis 6 to look for significant trends in the discontinuity data and define apparent spatial variations. Discontinuity sets were identified based on significant pole concentrations as illustrated in Figures 6-3 and 6-4 and summarized in Table 6-2.

Evaluation of the data presented reveals prevalent north-south and east-west striking joint discontinuity sets along both abutments of Axis 2 with an additional south dipping bedding discontinuity set in the right abutment. In contrast, both abutments of Axis 6 reveal a persistent south to southwest dipping bedding discontinuity, with a second set of primarily north-south striking joint discontinuities in the right abutment, and southeast-northwest striking joint discontinuities in the left abutment.

Table 6-2. Discontinuity Sets from Downhole Televiewer Logging

Axis/ Abutment	Strike (Degrees Az.)	Dip (Degrees)	Discontinuity Type	No. of Data Points
2-left	106	81 S	Joint	37
2-left	16	73 E	Joint	7
2-left	344	50 E	Joint	6
2-right	253	90 N	Joint	102
2-right	90	12 S	Bedding	19
2-right	17	41 E	Joint	17
6-left	125	21 W	Bedding	74
6-left	310	49 N	Joint	51
6-right	108	18 S	Bedding	48
6-right	20	63 E	Joint	27
6-right	340	70 E	Joint	5

A general comparison of the discontinuity sets determined from investigation reveals discontinuity trends persistent throughout the entire study area. Figure 6-5 presents a lower hemisphere equal angle stereonet pole plot of the discontinuity sets from which five general discontinuity sets were identified based on significant pole concentrations. These five discontinuity sets, summarized in Table 6-3, are consistent with previous observations of north-south and east-west striking joint discontinuity sets and the southwest dipping bedding discontinuity set.

Figure 6-5 also shows six sets of regional discontinuity alignments observed from ridge, valley and saddle orientations, as well as geomorphic expressions such as drainage lineaments, abrupt changes in slope or pronounced weathering features. Two sets of generally north south alignments and four sets of east-west alignments are noted on Figure 6-5.

Table 6-3. Summary Discontinuity Sets from All Data

Strike (Degrees Az.)	Dip (Degrees)	Discontinuity Type	No. of Data Points
111	15 SW	Bedding	4
16	69 E	Joint	5
336	64 E	Joint	4
250	89 N	Joint	6
103	90 S	Joint	4

6.3 Dam Axis 2 Foundation Conditions

Figure 6-1 shows a geotechnical profile for dam Axis 2. Along this axis, the RQD values for slightly weathered to fresh rock generally range from about 40 to 100%. Notably in boring CB-4, RQD values are generally low for the entire boring. Also, significant amount of rock core was lost (indicated by partial core recoveries) in the upper 105 feet of boring CB-1. This indicates lower overall rock quality.

Seismic refraction lines 2-1 and 2-4 show depths to 10,000 ft/s rock [slightly weathered to fresh (unweathered)] of about 10 feet, which corresponds well with the depth to slightly weathered rock as encountered in boring CB-2 on the left abutment. At boring CB-1, which is about 230 feet uphill from CB-2, the depth to 10,000 ft/s rock is more than 65 feet. This corresponds with boring CB-1, which encountered a large depth of highly to completely weathered, intensely fractured, very weak to extremely weak rock, with many zones of no core recovery. On the right abutment, seismic line 2-3 and the west end of line 2-5 show no velocities as high as 10,000 ft/s. This corresponds with the highly fractured rock conditions in borings CB-3 and CB-4. The depth to 10,000 ft/s rock at the end of the left abutment is more than 65 feet and at the end of the right abutment, the depth to 5,000 ft/s rock is more than 65 feet (in seismic line 2-3). No rock with 10,000 ft/s velocities was reached in the seismic refraction surveys at the right end of the dam axis.

Water pressure test data in Axis 2 borings shows that the hydraulic conductivities (Lugeon values) remain as high as 10 to 100 Lugeons and do not decrease with depth in the right abutment borings CB-3 and CB-4. This is likely due to the high degree of rock fracturing in both borings throughout their full depths. Lugeon values typically range from 3 to more than 100, with predominant values in the range of 10 to 40 Lugeons. On the left abutment, the Lugeon values decrease with depth and range from 1 to more than 100. In boring CB-2, between 14 and 32 feet, the Lugeon value was high (>100). Otherwise, in the slightly weathered to fresh rock, the Lugeon values are typically low (about 1 to 2).

The groundwater level in boring CB-2 was at about elevation 1700 feet, about 27 feet below the ground surface, on October 29, 2015. This groundwater level corresponds with an observed spring west of CB-2 at about elevation 1700 feet (see Section 4.3.3 and Figure 4-1).

6.4 Dam Axis 6 Foundation Conditions

Figure 6-2 shows a geotechnical profile for dam Axis 6. Along this axis, the RQD values for slightly weathered to fresh rock are generally around 100%, with occasional lower values.

Seismic refraction line 6-1 shows depths to 10,000 ft/s rock of about 40 feet, which agrees well with core boring CB-5 on the left abutment. CB-6 shows a depth to highly weathered and fractured rock of 142 feet. Seismic refraction line 6-1 shows that the depth to 10,000 ft/s decreases to less than 20 feet at the Bear River. This agrees with observations of hard rock outcrops in the river channel. For seismic line 6-3, the depth to 10,000 ft/s rock is about 40 feet which approximates the zone of highly weathered and fractured rock in CB-8 on the right abutment. The depth to 10,000 ft/s rock at the end of the left abutment is less than 40 feet. At the end of the right abutment, the depth to 10,000 ft/s rock is less than 10 feet in seismic line 6-4.

At a depth of 231 to 232 feet in boring CB-6, a possible shear zone was encountered that included sandy highly plastic clay infilling. From the televiewer log of CB-6, this shear has a strike of about N62^{o}W and dips about 42^{o} SW. The extent of this shear zone could not be determined during this stage of investigation. This feature merits further investigation during a subsequent phase of the work.

Water pressure test data in Axis 6 borings shows that the Lugeon values decrease with depth and decreasing weathering. In the highly weathered and fractured rock, Lugeon values typically are about 100 or more in the right abutment borings. In CB-6, in the highly weathered and fractured rock, Lugeon values range from 10 to 100. The Lugeon values for slightly weathered to fresh rock are much less, typically less than 4.

6.5 Comparison of Foundation Conditions of the Dam Axes

As depicted on Figures 6-1 and 6-2, the rock conditions at both axes can be differentiated based on the depth of weathering, fracturing and hydraulic conductivities (Lugeon values).

Based on the results of the core borings and seismic refraction surveys, the depth of weathering and fracturing across Axis 2 is somewhat deeper than across Axis 6. The presence of a relatively narrow ridge at the right (north) abutment of Axis 2 may have resulted in increased weathering and fracturing depths. The maximum drilled depth of highly fractured rock was about 140 feet at both Axes 2 and 6 (see Table 6-1). However, the depth of highly fractured rock in the right abutment of Axis 2 is greater than at Axis 6. Boring CB-1, in the left abutment of Axis 2, had the most core loss of any of the eight borings drilled; about 50% of the core was lost in the first 90 feet of drilling. This indicates highly weathered and fractured rock conditions at that location. The depth of highly fractured rock in the left abutments at Axis 2 and Axis 6 are otherwise similar.

The Lugeon values mostly decrease with depth and with decreasing fracture intensity. The exception is the right abutment borings at Axis 2, where the Lugeon values remain as high as 10 to 100 Lugeons and do not decrease with depth for nearly the full drilled depths in the core borings. The upper approximately 100 feet of the right abutment at Axis 6 showed Lugeon values ranging from 60 to more than 100 in borings CB-8 and CB-9, which is greater than for the right abutment at Axis 2. However, below this depth, Axis 6 has lower Lugeon values than at Axis 2. The Lugeon values for the left abutments at Axis 2 and Axis 6 are similar.

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7 Potential Construction Material Sources

Various types of construction materials are required for the different dam types discussed in Section 8, which include both embankment and concrete dams. The types of construction materials considered include the following:

- Low permeability moderately plastic clay for the core zones of earthfill and earth-core rockfill embankment dams
- Earth and weathered rock for the shells of an earthfill dam
- Rockfill for free-draining shells of rockfill dams
- Higher quality rock suitable for producing concrete aggregate, roller compacted concrete (RCC) aggregate, and filter and drain zones.

The various construction materials available on- and off-site are discussed in the sections that follow.

7.1 Rock Materials from Dam Site and Reservoir Area

Although specific field investigations have not been completed to confirm this, it appears based on regional geology that rock materials from the reservoir area may be suitable for use as rockfill and potentially also as RCC aggregate. However, sampling and testing will be necessary to confirm this and to evaluate whether the available rock is of sufficiently high quality for crushing and screening for use as filter, drain and concrete aggregates. At this time, the specific locations of potential rock borrow areas have not been determined and no geotechnical investigations have been conducted. In general, it is preferable to locate borrow areas within the reservoir area to minimize environmental impacts. The recommended general area of study for a potential rock borrow site is shown on Figure 7-1, based on topographic conditions and proximity to the dam site area. The suitability of any potential borrow sites will need to be evaluated and confirmed with geologic mapping and geotechnical investigations including seismic refraction surveys, test pits or test excavations, exploratory core drilling, and laboratory testing.

It is possible that alluvium within the river channel may be suitable for processing to provide sand and gravel for filter and drain materials. However, the available quality and quantity of the materials would need to be confirmed. The minerology of the river alluvium would need to be tested to detect the presence of potentially reactive minerals, such as chert, if considered for use as RCC or concrete aggregates.

7.2 Offsite Commercial Sources

Although significant amounts of construction materials are likely to be available onsite, a limited search of available offsite commercial quarries was performed to confirm sand, gravel and aggregate resources within the project vicinity. An internet search was conducted to locate commercial sources within about 40 miles from the dam site. Six commercial quarries were located including facilities operated by Teichert, Hanson Bros., Nordic Industries, and Rock Ridge Quarry. The quarries were contacted to inquire about the types of materials produced, availability of quality control data, quantities in reserve, and whether they are expected to be operating during the period from 2021 – 2024, when the project may be under construction. The results of the contacts of offsite commercial sources are presented in Appendix J.

7.3 Clayey Soil Materials

7.3.1 Results of Clay Borrow Material Study

A preliminary evaluation was performed to identify potential clayey materials sources in the site area, for possible use in the core of an embankment dam. The details of the evaluation are presented in Appendix K. The purpose of the study was to determine if borrow sources for low permeability soils are available in sufficient quantity for the construction of a core for an earthfill or rockfill embankment dam. As much as one million cubic yards of in-place borrow material may be needed for the core of an embankment dam at the project site. This estimated quantity would allow for removal of oversize particles and other unsuitable materials, shrinkage during compaction, and a reserve margin on the preliminary estimated core zone volume.

The study was based on review of the Nevada County and Placer County soil surveys in the area of the proposed dam, along with published reports of borrow soil analysis. A reconnaissance of potential sources within an approximate 10-mile radius of the dam site was also performed. Interviews were conducted of owners and others knowledgeable about potential areas of borrow, including Dark Horse Subdivision, Winchester Subdivision, Hansen Bros. quarry locations, and areas of current earthwork construction. In addition, data was reviewed on materials, borrow sources and construction of Rollins Reservoir including a map of the borrow areas used during construction of the dam.

As part of the borrow source study, the U.S. Department of Agriculture Natural Resources Conservation Service Online Web Soil Survey was reviewed for soil classifications within a 10-mile radius of the project site. The clay or clayey material present in local soil deposits is typically the result of chemical and physical weathering of the parent rock at shallow depths. In general, the residual soil of the Sierra foothills has relatively shallow depth to bedrock. Isolated areas may have up to 30 feet of soil and severely weathered rock overlying more competent rock.

7.3.2 Issues with Using Clayey Soils

The results of the study indicate that fine grained materials are present within three miles of the project site which would probably be suitable for the low permeability core of an embankment dam. However, the material is generally available in limited quantities due to the shallow depth of bedrock. Many potential borrow sites include an intermediate zone of severely weathered rock that would excavate as variable soil. Furthermore, from observations of residual soils in local road cuts, the thickness of the clayey soils varies considerably over short distances, with rock outcrops protruding into the soils. This could make the excavation relatively inefficient.

Because of the expected shallow depth of clayey soils in the site region, large areas approaching 200 acres would likely be required in order to provide an in-place volume of one million cubic yards. Such large areas would present the following impacts and challenges, which would likely increase costs:

- Environmental impact: Removal of shallow surface soils would create disturbance of large areas; such areas would require restoration.
- Material hauling: Transport of the materials to the project site could adversely affect adjacent communities and roadways, and would increase air emissions.
- Quality control: As stated above, the clay soils are the result of weathering of the parent rock. As the depth of excavation increases, the material will likely become less clayey and more granular and rocky, which would be unsuitable for use as core material. A high degree of borrow area and fill placement quality control would be required to separate out and exclude such materials from the core zone.

8 Conceptual Design of Dams

8.1 Dam Types Considered

Five potential dam types were considered for the CRP site. These dam types include (1) earthfill, (2) earth-core rockfill, (3) concrete-faced rockfill (CFRD), (4) concrete gravity/roller compacted concrete (RCC), and (5) concrete arch. The main features and considerations for each of these dam types are summarized in Table 8-1. The relative viability and economics of each dam type will depend mostly on the foundation conditions, site geometry, and construction materials availability.

For the reasons summarized below and in Table 8-1, the dam types recommended for further study are concrete-faced rockfill and concrete gravity/RCC dams. Both dam types are likely to be suitable for the site based on the apparent foundation conditions. Based on limited initial review of available information as discussed in Section 7, rock materials suitable for both an RCC gravity dam and a concrete-faced rockfill dam may be available within the reservoir area and/or from the nearby existing quarry in sufficient quantities for these dam types.

With regard to construction materials availability, some clay core materials exist in the site vicinity, but they appear to be relatively limited in quantity and inefficient to obtain due to the relatively shallow depth of bedrock as discussed in Section 7. Large areas would need to be stripped to obtain sufficient clay for an earthfill dam or an earth-core rockfill dam. For this reason, these dam types are considered less viable for this site.

Based on the investigation findings from this preliminary study, the potential viability of a concrete arch dam type at this site appears to be relatively low. Primarily, the dam site geometry is unsuited for an arch dam because the canyon is wide relative to the dam height. Ideal arch dam sites have narrower canyons relative to the dam height. Also, the canyon geometry is asymmetrical with steeper slopes on the right side of the river canyon. Because an arch dam would apply the most intensive foundation loading of any dam type, the rock quality requirements would be higher than for other dam types. The risks associated with discovering unacceptable foundation defects during construction would be the greatest with this dam type, and consequently the most intensive level of geotechnical investigation would be required. Regulatory approval would also be the most difficult to obtain for this dam type. For these reasons, further study of this dam type is not recommended.

8.2 Conceptual Design Considerations for Preferred Dam Types

Conceptual level designs were developed to illustrate the general arrangement and the main features of the RCC dam and concrete-faced rockfill dam (CFRD) types. Development of details is not warranted for this study and is beyond the scope of work. Significant geotechnical engineering and analyses will be needed in future phases of work to develop the design and size of the various project facilities.

For the concept designs discussed in the sections that follow, the dam crest elevation was assumed to be at elevation 1875 feet. The dam would provide a maximum normal water level of about 255 feet above the Bear River to provide for 110,000 acre-feet of storage. This assumes a 20-foot freeboard above the normal maximum reservoir surface (spillway crest).

8.2.1 Roller Compacted Concrete Dam

8.2.1.1 Foundation Treatment

The foundation for an RCC dam would require slightly weathered to fresh, hard rock. Excavation depths up 100 feet would be expected at Axis 2 and Axis 6; the excavation depths in the abutments could average about 60 to 70 feet. Shallower excavation depths are anticipated in the river channel areas. For a dam crest at approximately elevation 1875 feet, the length of Axis 2 could be around 1800 feet. Low seismic velocities are present at the ends of this axis as shown on Figure 6-1. The length of Axis 6 could be about 1600 feet.

Grouting would be needed to control seepage through the foundation rock. Considering that the maximum reservoir depth would be about 255 feet and the hydraulic conductivity data discussed in Section 6, the grout curtain depths could range from 100 feet to about 200 feet below the foundation level. At least two grout curtains are expected to be needed. The grout holes in each curtain would be angled in opposing directions to more efficiently intersect near vertical rock discontinuities. Due to the fractured nature of the rock, the foundation for an RCC dam at either axis would also require consolidation grouting of fractured rock areas within the footprint of the dam foundation to strengthen and increase the rock mass stiffness of the foundation.

Drain holes to control uplift pressures beneath the RCC dam would also be required. These holes would be drilled downstream of the grout curtain from a gallery within the dam.

8.2.1.2 Conceptual Layout of Dam and Appurtenant Structures
The conceptual plan and section of the RCC dam are shown on Figures 8-1 and Figure 8-2. The RCC dam concept has a vertical upstream face and a 0.8H:1V to 0.85H:1V stepped downstream face. The dam crest is 25 to 30 feet wide. As stated in Section 7.1, rock for RCC aggregate may be available from an on-site quarry. Such a quarry would need to be stripped of overburden and weathered rock and underlying rock would be drilled, blasted and crushed to produce the RCC aggregate. The RCC would be placed in 1-foot thick lifts and compacted with vibratory rollers. The RCC would be faced with grout enriched RCC and/or conventional concrete as the RCC is placed.

The spillway would be integral with the body of the RCC dam and would be located to discharge directly into the Bear River channel. An energy dissipater structure (e.g., stilling basin) would be located at the toe of the spillway. The outlet conduit would be cast into the body of the dam and could also serve as the river diversion during construction. NID has indicated that they would require only a single low-level intake, which would be located at the base of the dam and connect with the outlet conduit. The conduit could be fitted with a stub pipe with blind flange for a future power plant at the downstream toe of the dam.

8.2.2 Concrete Faced Rockfill Dam

8.2.2.1 Foundation Treatment
The concrete plinth (slab) foundation for a concrete faced rockfill dam (CFRD) would require groutable slightly weathered to fresh, hard rock. The plinth connects the concrete face to the foundation; there is a non-structural joint ("perimeter joint") between the plinth and the facing that includes waterstops. The plinth would generally be about 25 to 30 feet wide (about 10% of the hydraulic head) and would be founded on slightly weathered to fresh rock. The plinth would serve as the curtain grouting cap/platform and would be anchored into rock with grouted steel dowels. Excavation depths up to100 feet for the plinth in the abutments would be expected at Axis 2 and Axis 6; the excavation depths could average about 60 to 70 feet. Shallower excavation is anticipated in the river channel areas. The foundation for the rockfill zones would generally require moderately weathered rock conditions. Based on the core boring data, the excavation depths in the abutments would be up to about 75 feet at Axis 2 (averaging about 30 feet) and up to 30 feet at Axis 6 (averaging about 15 feet). River channel excavation depths would be less.

For a dam crest at approximately elevation 1875 feet, the length of Axis 2 could be around 1800 feet. Low seismic velocities are present at the ends of this axis (see Section 6.3). The full length of Axis 2 extends beyond the profile shown on Figure 6-1. The length of Axis 6 could be about 1600 feet.

Grouting would be needed to control seepage through the foundation rock. Considering the maximum reservoir depth of about 255 feet and the hydraulic conductivity data discussed in Section 6, the grout curtain depths could range from 100 feet to about 200 feet below the plinth level. At least two grout curtains are expected to be needed. The grout holes in each curtain would be angled in opposing directions to more efficiently intersect near vertical rock discontinuities.

8.2.2.2 Conceptual Layout of Dam and Appurtenant Structures
The conceptual plan and section of the concrete-faced rockfill dam (CFRD) are shown on Figures 8-3 and 8-4. The upstream slope is1.4H:1V to 1.5H:1V and the downstream slope is 1.5H:1V to 1.6H:1V. The dam crest is

Table 8-1. Evaluation of Dam Types

Consideration	Earthfill	Earth-core Rockfill	Concrete-faced Rockfill	Concrete Gravity/RCC	Concrete Arch
Precedent in California	Common	Common	Less common in CA. Only a few built in last 30 years.	Less common than earth- or rockfill dam types. Several RCC dam projects built in last 15 years.	A number exist in CA but very few built in last 40 years.
Footprint Size	Large, due to flat slopes	Smaller than earthfill dam type, due to steeper slopes	Smaller than rockfill dam type, due to steeper slopes	Smaller than concrete-faced rockfill dam type, due to steeper slopes	Smallest of any dam type
Foundation Considerations	Core should be founded on relatively competent rock Shell zones may be founded on highly weathered rock.	Core should be founded on relatively competent rock Shell zones may be founded on highly weathered rock.	Plinth (concrete foundation slab at the upstream toe of the concrete face) and transition zone should be founded on groutable slightly weathered to fresh competent rock. Rockfill body of dam can be founded on highly to moderately weathered rock.	Dam must be founded on slightly weathered to fresh competent rock. Sliding stability of foundation must consider potential presence of weak layers and discontinuities.	Dam must be founded on very competent rock Sliding stability of foundation must consider potential presence of weak layers and discontinuities. This issue is more critical than for concrete gravity/RCC dam type. The site is wide for the height of dam, thus making the site not optimal for an arch dam
Construction Materials Availability	Need sufficient clay for core zone; may need to source from off-site borrow On-site rock may be suitable for riprap and processed filter sands and drain gravels Shell zones from overburden and residual soils from on-site borrow excavations	Need sufficient clay for core zone; may need to source from off-site borrow On-site rock may be suitable for processed filter sands and drain gravels Rockfill shell zones likely available from on-site borrow areas and spillway excavation	On-site rock may be suitable for processed transition zone (sandy gravel), filter sands and drain gravels Rockfill body of dam likely available from on-site borrow areas and spillway excavation	On-site rock may be suitable for processed aggregate sands and gravels Significant amount of imported cement and fly ash required for RCC	On-site rock may be suitable for processed aggregate sands and gravels
Acceptability by DSOD	Well-accepted, large precedent	Well-accepted, large precedent	Likely would be acceptable, smaller precedent	Likely would be acceptable, smaller precedent	Most difficult dam type alternative to get accepted Intensive geotechnical investigation would be required
Potential Environmental Effects	Large area of excavation expected to obtain sufficient earth materials	Smaller area of excavation than earthfill dam type due to less reliance on earth materials	Smaller area of excavation than rockfill dam type due to less reliance on earth materials	Smaller area of excavation than concrete-faced rockfill dam type due its smaller volume	Smaller area of excavation than for concrete gravity/RCC dam type due its smaller volume
Appurtenant Facilities	Spillway on abutment and river diversion/ outlet works through tunnel in abutment	Spillway on abutment and river diversion/ outlet works through tunnel in abutment	Spillway on abutment and river diversion/ outlet works through tunnel in abutment	Spillway over dam and river diversion/ outlet works through dam	Spillway over dam and outlet works through dam River diversion through a tunnel in abutment may be required, rather than through dam
Construction Considerations	Earth materials are sensitive to adverse weather conditions Significant quality control would be required to verify that acceptable clay materials are placed in the core zone	Earth core materials are sensitive to adverse weather conditions Significant quality control would be required to verify that acceptable clay materials are placed in the core zone	Less sensitive to adverse weather conditions than earth dam types Foundation grouting not on critical path as this would be performed through plinth; advantageous to schedule	Sensitive to hot weather and rainy conditions Likely most rapid construction method	Sensitive to hot weather and rainy conditions Most complex construction of the alternative dam types
Conclusion	Not advisable for this site.	Not preferred for this site.	Further study justified.	Further study justified.	Not advisable for this site.

about 30 feet wide. Rockfill is likely to be available from an on-site quarry (see Section 7.1). Rockfill is typically placed in 3-foot-thick loose lifts and compacted by heavy vibratory rollers. Lesser quality (weathered) rock could be placed in a zone downstream of the dam axis as shown on Figure 8-4. A processed sand and gravel layer (from the quarry) would be placed under the concrete slab to help control any seepage through the joints in the concrete facing. A fine rockfill zone separates the transition zone from the rockfill zone. Filters would be required to prevent piping/erosion of the weathered rock foundation materials into the overlying rockfill.

The concrete face would range from about 1 to 1.5 feet thick. The face would be slip formed in vertical panels, 40 to 60 feet wide, from starter slabs above the plinth to the crest parapet wall. Details would include facing reinforcement and waterstops at the joints between the facing panels and along the perimeter joint with the plinth.

For both axis locations, the spillway would likely be located on the right abutment because of the shorter distance to the river channel than for a left abutment location. The presence of landslides shown on Figure 4-1 would need to be evaluated for spillway construction in either abutment. At either axis location, a right abutment spillway chute would be about 1000 feet long, not including the approach channel upstream of the dam. An energy dissipater structure (e.g., stilling basin) would be located at the toe of the spillway chute, which would discharge to the Bear River. The spillway excavation may yield usable quantities of rockfill materials suitable for a CFRD.

The outlet conduit could be located in a tunnel driven through either abutment. The tunnel would be excavated by drilling and blasting through the basalt rock. The required outlet tunnel for the two axes would be around 1500 feet long. For Axis 2, if the tunnel were to be located in the right abutment, the landslide shown on Figure 4-1 would need to be removed from the inlet end of the tunnel. An outlet tunnel in the right abutment at either dam axis could potentially discharge to the spillway stilling basin. As stated above, NID has indicated that they would require only a single low-level intake, which would be located at the upstream end of the tunnel. The conduit could be fitted with a stub pipe with blind flange for a future power plant at the downstream toe of the dam.

The outlet tunnel could also serve as the temporary river diversion conduit during construction. A cofferdam would be required downstream of the tunnel inlet to divert flows into the tunnel.

8.3 Estimated Dam Construction Quantities and Relative Construction Costs

For comparison purposes, approximate excavation and dam quantities were estimated based on the conceptual layouts show in Figures 8-1 to 8-4. The estimated quantities are summarized in Table 8-2. For the purposes of this study, order-of-magnitude total unit construction costs were estimated based on historical data from similar recent projects. Development of detailed line item cost estimates is beyond the scope of this initial study.

Table 8-2. Approximate Dam Quantities

Dam Type	Item	Axis 2*	Axis 6*
RCC Dam	Excavation (cy)	1,000,000	1,100,000
	RCC (cy)	1,250,000	1,500,000
Concrete–faced Rockfill Dam	Excavation (cy)	1,400,000	1,000,000
	Rockfill (cy)	3,300,000	3,300,000

*See Sections 8.2.1.1 and 8.2.2.1 for assumptions on excavation depths.

The excavation volumes for two dam types are generally comparable. For Axis 6, the excavation volume for the CFRD is less than at Axis 2 because the excavation depth for the rockfill shells is less. As expected, the RCC dam volume is less than for the CFRD volume. It is notable that the excavation volume for the RCC dam approximates the volume of the dam itself.

The amount of imported cement and fly ash for RCC will be a construction and environmental consideration. Approximately 200,000 to 250,000 tons of cement and fly ash may need to be imported for the RCC dam, which would be much more than for the CFRD alternative.

Based on the excavation and dam volumes indicated in Table 8-2, it appears that the CFRD could be less costly than the RCC dam. However, a conceptual-level construction cost estimate would be needed to confirm this conclusion.

9 Conclusions and Recommendations

The purpose of the Phase II study was to perform a geotechnical investigation to provide an assessment of the site conditions for the proposed dam, and to evaluate the potential dam axis locations and dam types. This final report was prepared as part of the Phase II scope of work and builds upon the draft Phase I report submitted to NID on September 30, 2015. Conceptual designs of the preferred dam types are presented. This report also includes recommendations for further dam foundation and borrow area investigations that are necessary to support the next phase of the project.

All conclusions regarding the foundation excavation and conceptual dam design in this report are preliminary and are subject to change based on the results of further studies described below.

9.1 Preferred Dam Site

Either the site at Axis 2 or Axis 6 is acceptable for either an RCC dam or CFRD. Fatal flaws were not identified at either site. The main geotechnical differences between the two sites are the extent of foundation excavation and treatment that would be required, which in turn would affect construction cost. For the same reservoir water surface elevation 1855 feet, the reservoir capacity would be about 7,000 acre-feet less for a dam at Axis 6 than further downstream at Axis 2. A dam at Axis 6 would need to be about 3 feet higher in order to offer the same reservoir storage capacity as a dam at Axis 2.

The depth of weathering and fracturing across Axis 2 is somewhat deeper than across Axis 6. The narrow ridge in the right (north) abutment of Axis 2 likely caused increased weathering and fracturing depths. The depth of highly fractured rock in the right abutment of Axis 2 is greater than for Axis 6. The hydraulic conductivities mostly decrease with depth and with decreasing fracture intensity. The exception is for right abutment borings at Axis 2, where the hydraulic conductivities do not show this trend due to the fractured nature of the rock for nearly the full drilled depths in the core borings. It is expected that the foundation grouting requirements in terms of depth and lateral extent of grouting would be similar for both dam sites, although the amount and duration of the grouting effort would probably be greater at Axis 2 due to the relatively higher hydraulic conductivity at depth.

A factor that needs to be considered for Axis 2 is the Bear River Quarry located 1700 feet south of the south end of this axis. The current quarry floor elevation of 1710 feet is lower than the proposed reservoir water surface at elevation 1855 feet. A dam at Axis 2 would likely result in increased seepage into the quarry (some groundwater is already being pumped from the quarry). If the face of this quarry advances towards the dam site, the distance to the reservoir would decrease and the potential seepage could increase. A dam at Axis 2 could require seepage control measures, such as a grout curtain beyond the left end of the dam, to mitigate seepage into the quarry.

Based on the results of the preliminary geotechnical investigation discussed in this report, we conclude that both Axis 2 and Axis 6 appear to be acceptable for dam construction. However, this conclusion is based on limited data on the foundation conditions at the two sites and should be confirmed with further investigations before selecting a preferred site.

9.2 Preferred Dam Type

Based on the investigation results presented in this report, either an RCC dam or CFRD could be constructed for the proposed Centennial Reservoir Project. Both dam types appear likely to be suitable for the site based on the observed foundation conditions. Rock materials suitable for both RCC gravity dam aggregates and a CFRD appear likely to be available within the reservoir area and/or from the nearby quarry in sufficient quantities for these dam types.

Although initial evaluations suggest that a CFRD could be less costly than an RCC dam, further investigation and a formal alternatives analysis are needed to verify this. In addition to further geotechnical investigations, preliminary engineering studies, conceptual designs, quantity and construction cost estimates, and environmental reviews are needed.

The following are some key considerations that differentiate between an RCC dam and a CFRD:

- An RCC dam would need to be founded on slightly weathered to fresh competent rock. Sliding stability of the foundation must consider the potential presence of weak layers and discontinuities.
- The potential impacts of finding unexpected adverse foundation conditions would be greater for an RCC dam.
- Field quality control requirements would be more intense for an RCC dam than for a CFRD.
- An RCC dam would not require a separate spillway or outlet tunnel.
- Cement and fly ash import would be significantly greater for an RCC dam than for a CFRD.
- An RCC dam would have a smaller excavation footprint area than a CFRD.
- Construction of a CFRD would require a temporary cofferdam for diversion of the Bear River into an outlet/diversion tunnel.
- An RCC dam would likely be constructed more rapidly than a CFRD.
- A CFRD would be less sensitive to adverse weather conditions during construction than an RCC dam. RCC construction can be affected by rainy and hot weather conditions.
- An RCC dam would be more capable of withstanding floods during construction.

9.3 Recommendations for Design Development

Design development to assist in selecting a preferred dam site and dam type should include further geotechnical investigations. A phased investigation approach is recommended focusing on the dam site and rock borrow materials.

The next phase of geotechnical investigation should be sufficient to identify a preferred dam site and dam type, and to support development of the EIR document. The investigations should be performed in the areas of both Axes 2 and 6, and they should cover the potential dam footprint areas upstream and downstream of both of these dam axes. The investigations should include borings along the plinth alignment of a potential CFRD dam type to provide data for excavation and grout curtain design. These investigations should include seismic refraction surveys, core borings, water pressure (packer) testing, and televiewer/caliper logging. Laboratory testing should include strength of the rock foundation materials.

Borrow investigations should be carried out to confirm the nature and depth of the available rock materials. The investigation should assess the amount of overburden that would need to be stripped and wasted. These investigations should include geologic reconnaissance and mapping, seismic refraction surveys, and core borings. Laboratory testing should include strength of the rock materials, abrasion resistance, soundness and bulk specific gravity.

The CFRD and RCC dam alternatives should be evaluated at both Axes 2 and 6. A formal alternatives analysis should be carried out based on the geotechnical investigations discussed above, along with preliminary engineering studies, conceptual layouts and designs, assessment of environmental considerations and constraints, and development of construction cost and schedule estimates.

Based on the results of the above design development studies, sufficient data should be available to confirm the selection of a preferred dam site and dam type for the project.

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Figures

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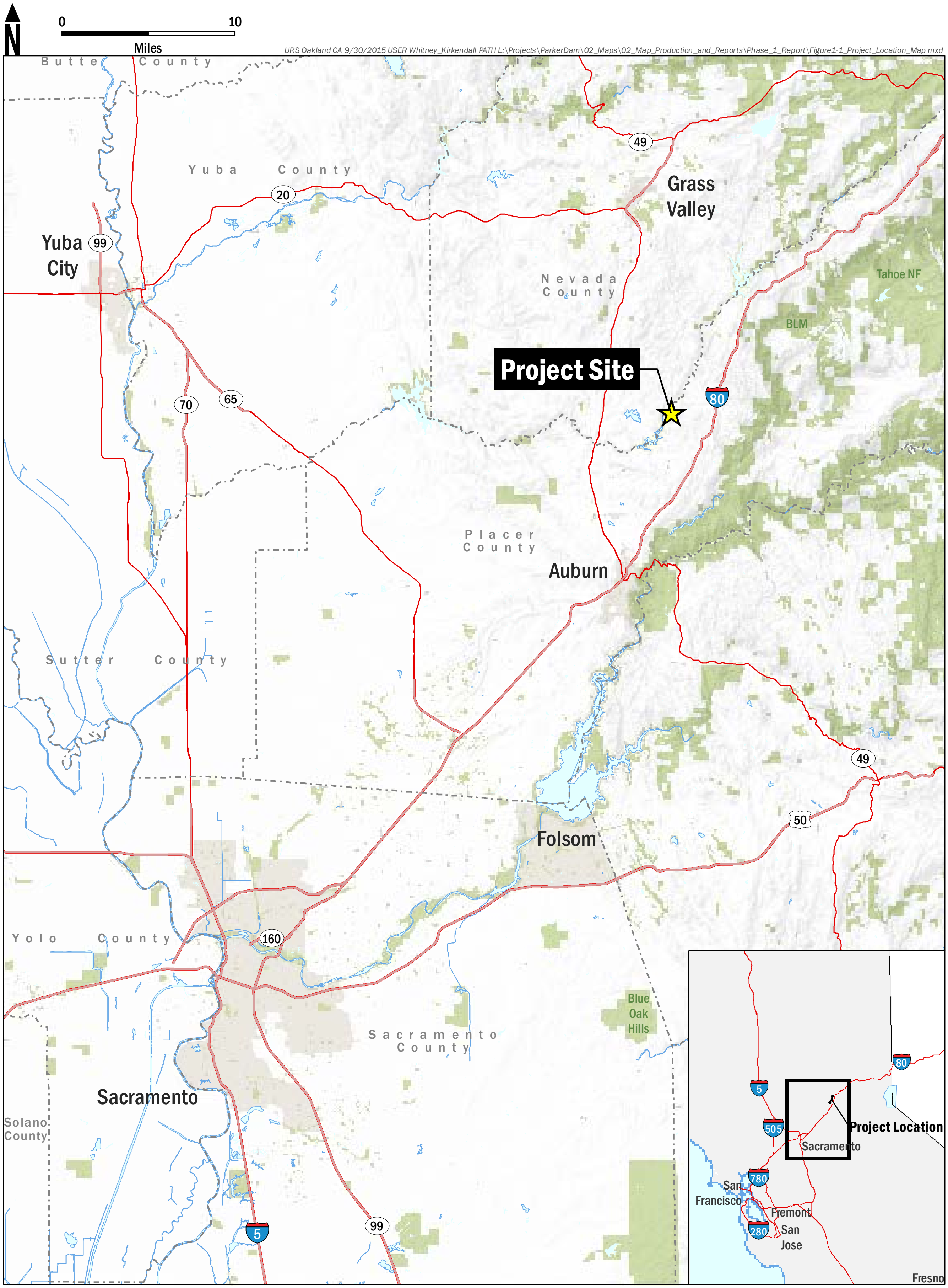

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Centennial Reservoir Project

PRELIMINARY GEOTECHNICAL INVESTIGATION

60393322

FIGURE 1-1

Project Location Map

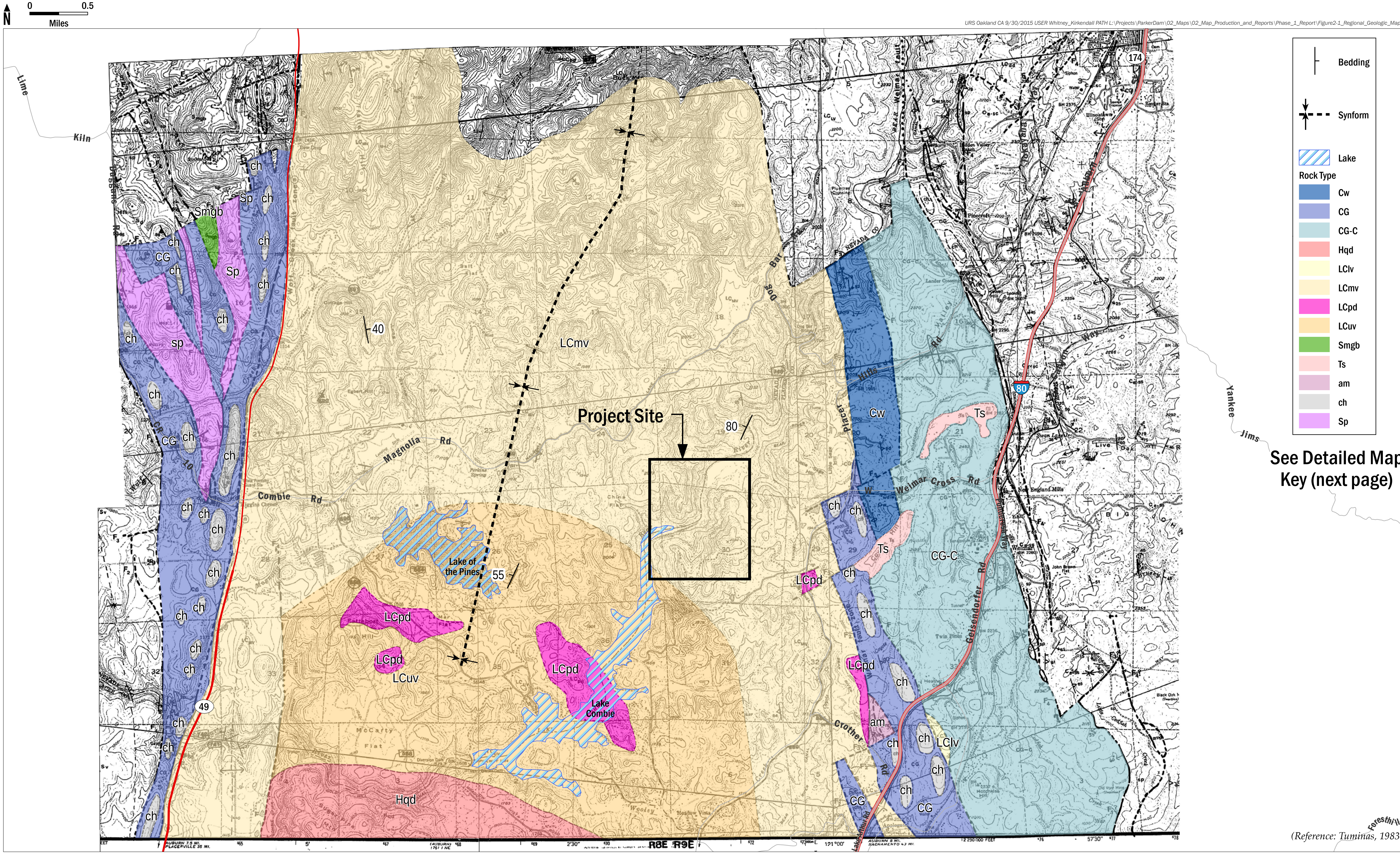

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FIGURE 2-1

Regional Geologic Map

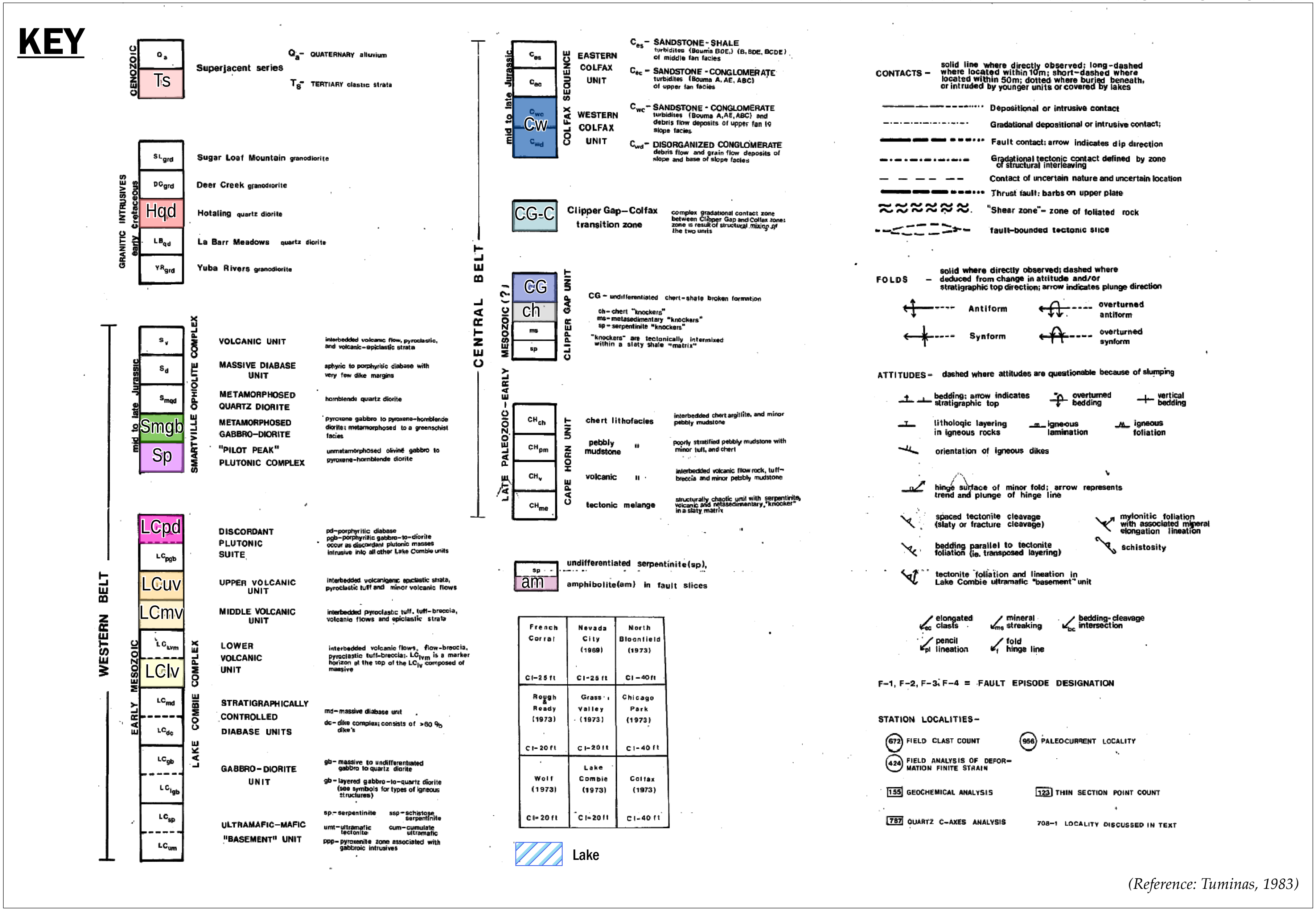

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FIGURE 2-1 (CONT'D)

Regional Geologic Map

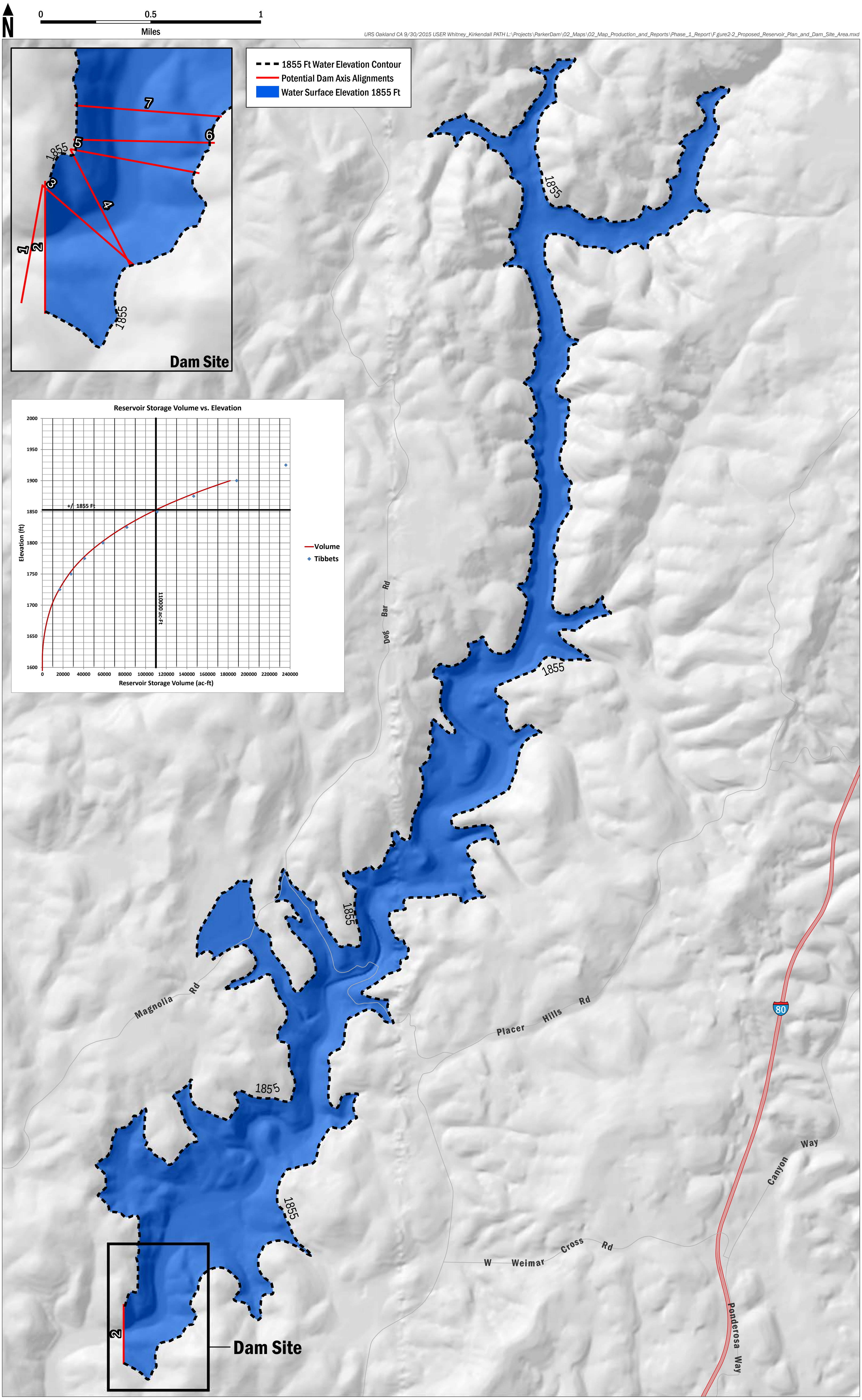

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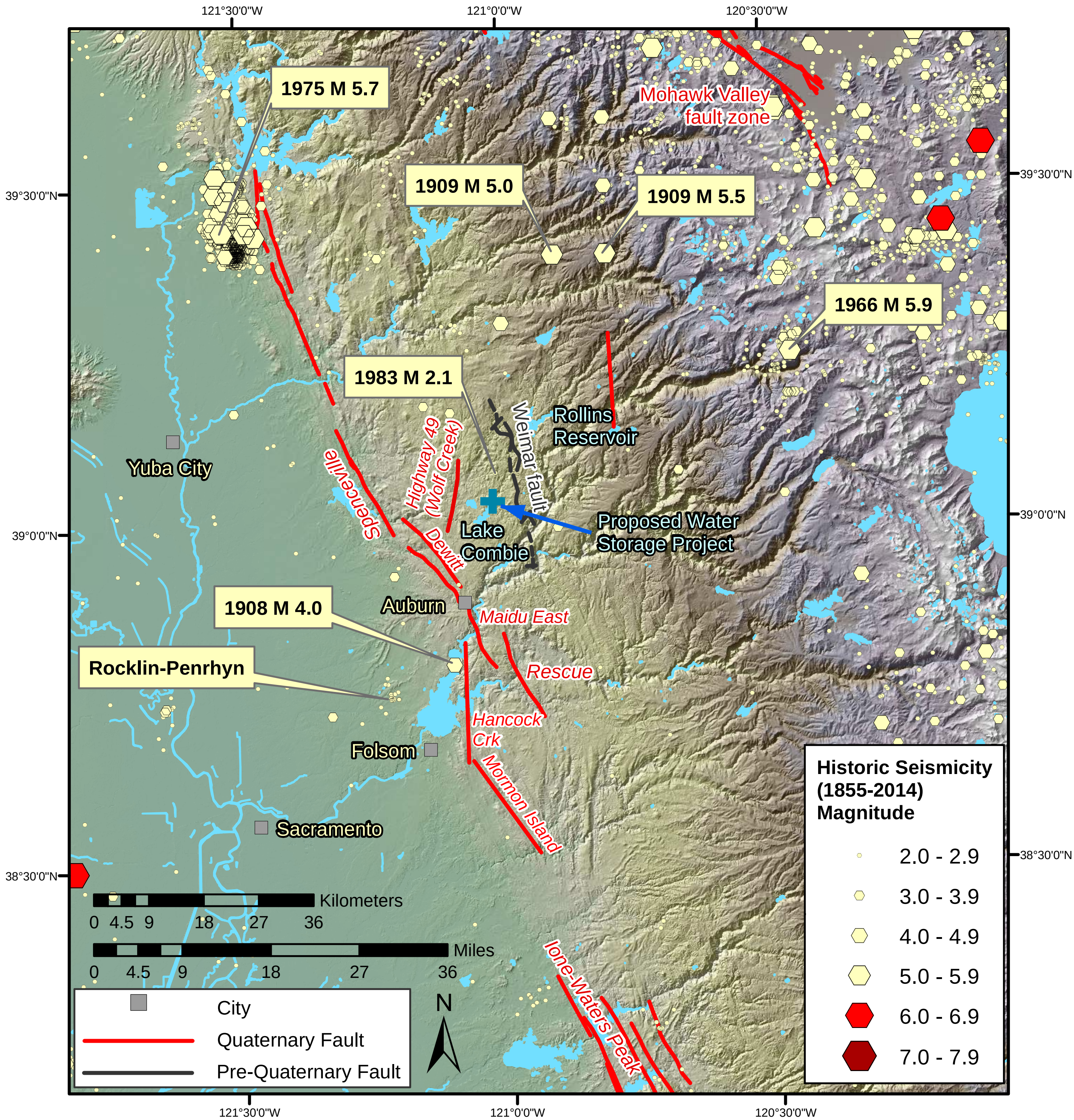
FIGURE 2-2

Proposed Reservoir Plan and Dam Site Area

110,000 Acre Ft Reservoir



AECOM	Project No. 03150578 Nevada Irrigation District	HISTORICAL SEISMICITY (1855-2014)	Figure 3-1

AECOM
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Joint
Bedding
Spring
Drilled Incline Boring (Top)
Drilled Incline Boring (Bottom)
Geologic Cross Section Alignment
NorCal Seismic Refraction Survey (Phase 1)
NorCal Seismic Refraction Survey (Phase 2)
Basalt Outcrop
Volcanic Breccia Outcrop
Quaternary Alluvium
Landslide Deposit
Area of Interest, *approximate*
Index Contour (100ft)
Contour (20ft)

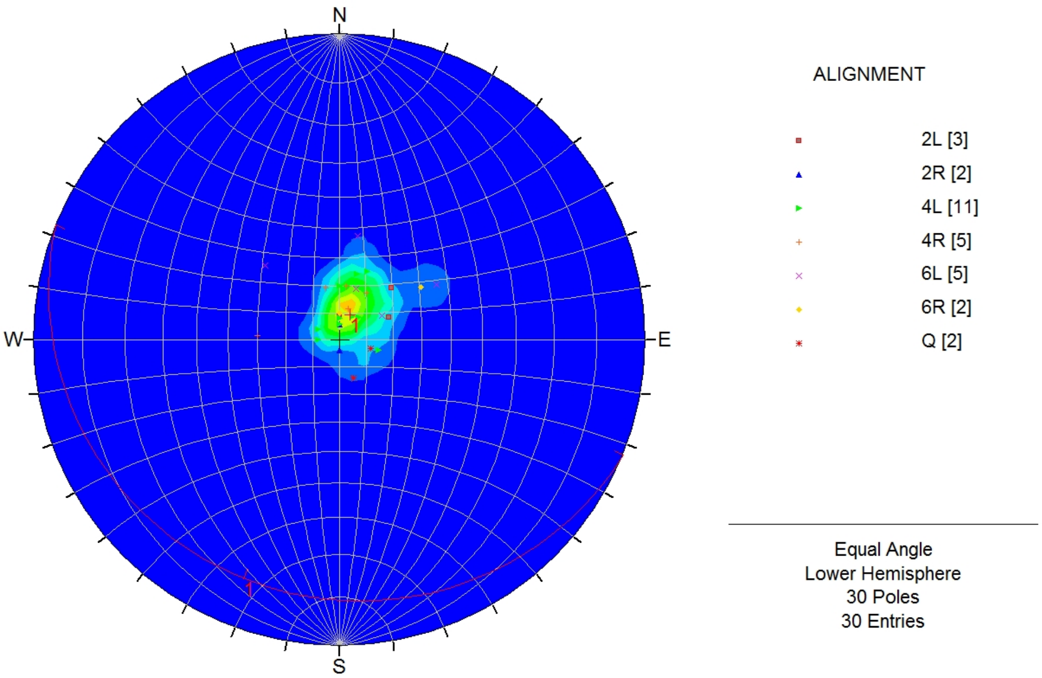
Note:
-Outcrop limits shown are approximate and are based on limited geologic mapping and air photo review. Not all outcrops are labeled. Areas with shallow soil cover over bedrock may not be labeled.

Reference:
-Elevation, NID Parker Dam Lidar, Nevada Irrigation District, November 2014.

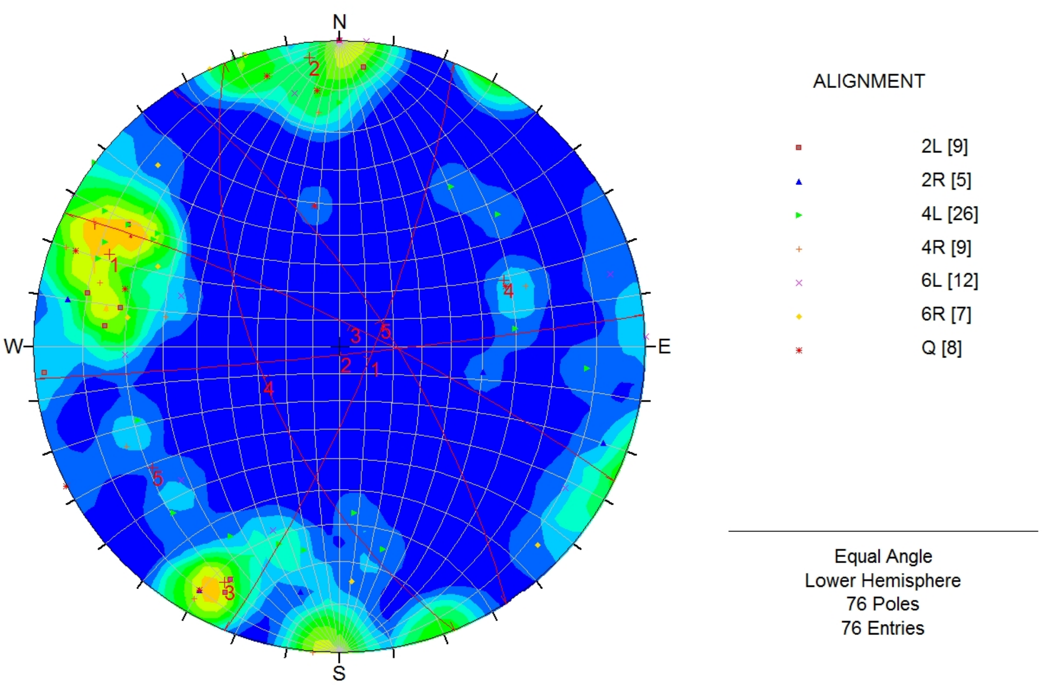
N
0 100 200 300 400 500 Feet

FIGURE 4-1

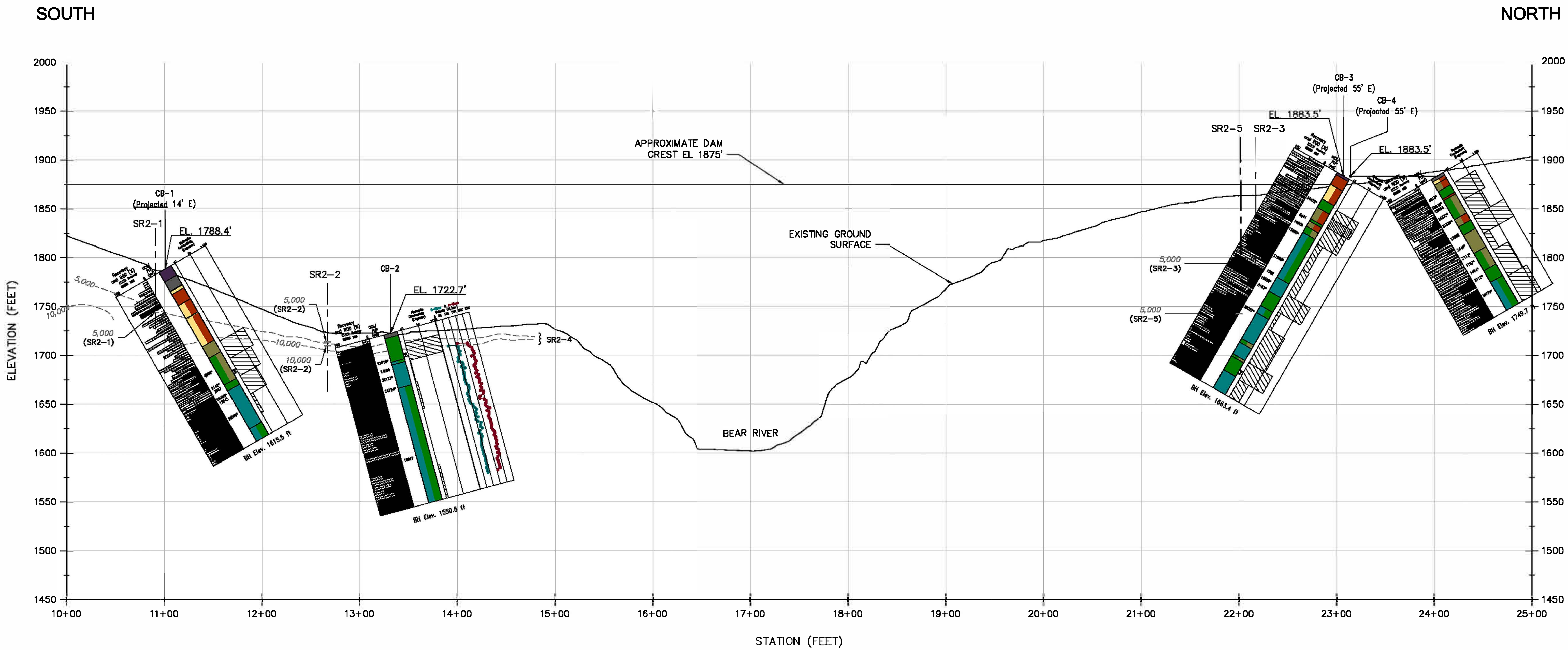
Geotechnical Exploration Plan



a. Bedding poles and Fischer concentration plot on lower hemisphere stereonet showing typical bedding orientation.



b. Joint and fracture poles and Fischer concentration plot on lower hemisphere stereonet with typical discontinuity set orientations.



LEGEND:

WEATHERING PROFILE:

- COLLUVIUM
- RESIDUAL SOIL
- COMPLETELY WEATHERED
- HIGHLY WEATHERED
- MODERATELY WEATHERED
- SLIGHTLY WEATHERED
- FRESH

GROUNDWATER LEVEL DURING DRILLING (MAY NOT REFLECT NATURAL GROUNDWATER LEVEL)

BH — BOTTOM OF HOLE

UCS/PLI* — UNCONFINED COMPRESSIVE STRENGTH; * INDICATES RESULT FROM POINT LOAD TEST; SEE APPENDICES H AND I FOR DETAILS

5,000 — P-WAVE VELOCITY (ft./sec.) FROM SEISMIC REFRACTION SURVEY; SEE APPENDIX C FOR DETAILS

SR6-4 — SEISMIC REFRACTION LINE 6-4

LOCATION OF SR LINE, AT CROSSING OF PROFILE

NOTES:

1. ELEVATION DATUM IS NAVD (1988).
2. PROFILE IS LOOKING DOWNSTREAM.

0 50 100 Feet

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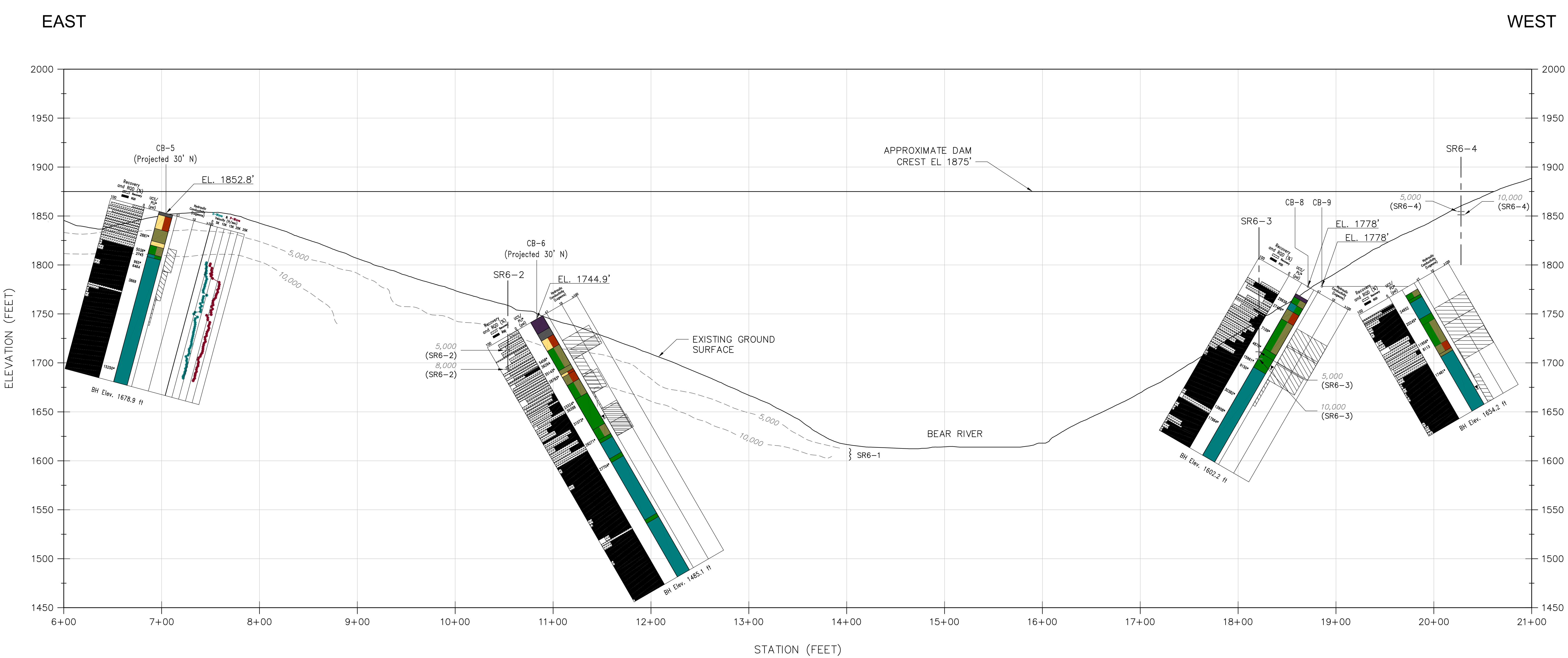
NEVADA IRRIGATION DISTRICT CENTENNIAL RESERVOIR PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION

Geotechnical Profile Axis 2

FIGURE 6-1

Dec 21, 2015 - 11:31am L:\Projects\Legacy\E_Xdrive\x_water\NID Parker Dam\13_Phase 2\Cross Sections\Sections working.dwg



LEGEND:

WEATHERING PROFILE:

- COLLUVIUM
- RESIDUAL SOIL
- COMPLETELY WEATHERED
- HIGHLY WEATHERED
- MODERATELY WEATHERED
- SLIGHTLY WEATHERED
- FRESH

GROUNDWATER LEVEL DURING DRILLING (MAY NOT REFLECT NATURAL GROUNDWATER LEVEL)

BH — BOTTOM OF HOLE

UCS/PLI* — UNCONFINED COMPRESSIVE STRENGTH; * INDICATES RESULT FROM POINT LOAD TEST; SEE APPENDICES H AND I FOR DETAILS

5,000 — P-WAVE VELOCITY (ft./sec.) FROM SEISMIC REFRACTION SURVEY; SEE APPENDIX C FOR DETAILS

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2. PROFILE IS LOOKING DOWNSTREAM.

0 50 100
Feet

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NEVADA IRRIGATION DISTRICT
CENTENNIAL RESERVOIR PROJECT
PRELIMINARY GEOTECHNICAL
INVESTIGATION

Geotechnical Profile
Axis 6

FIGURE
6-2

Dec 21, 2015 - 11:33am
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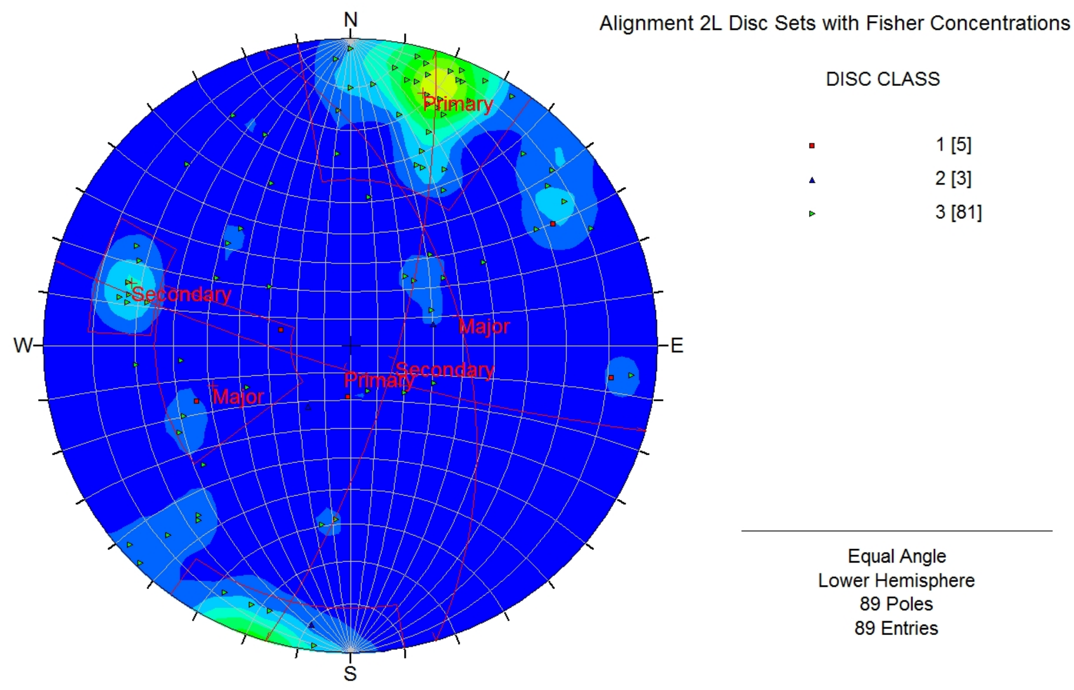
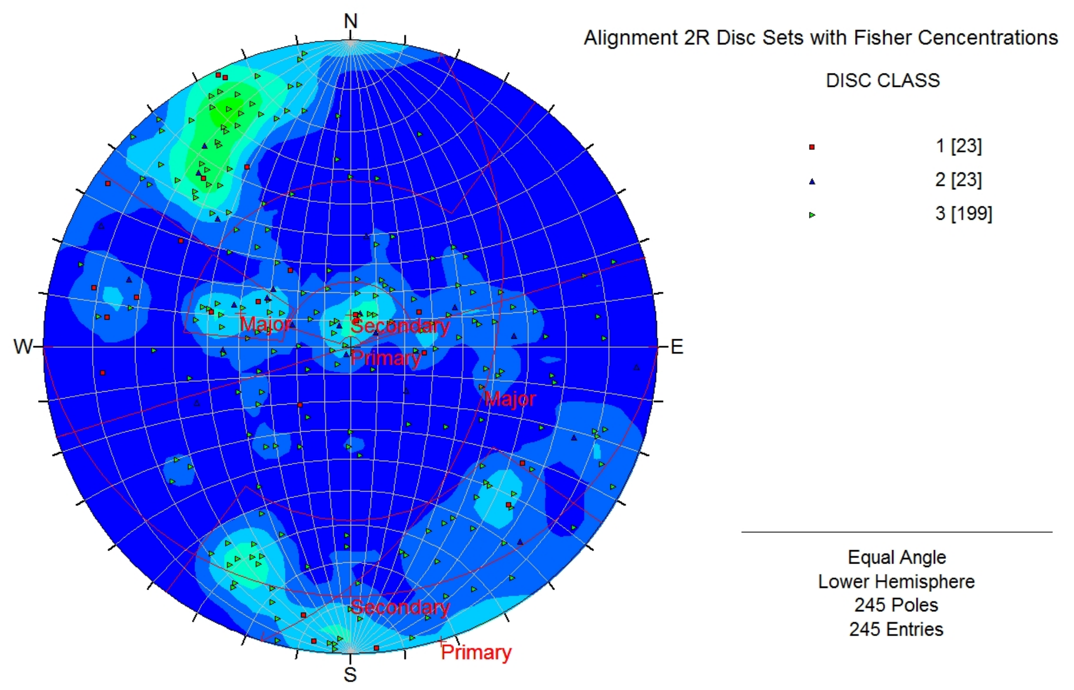


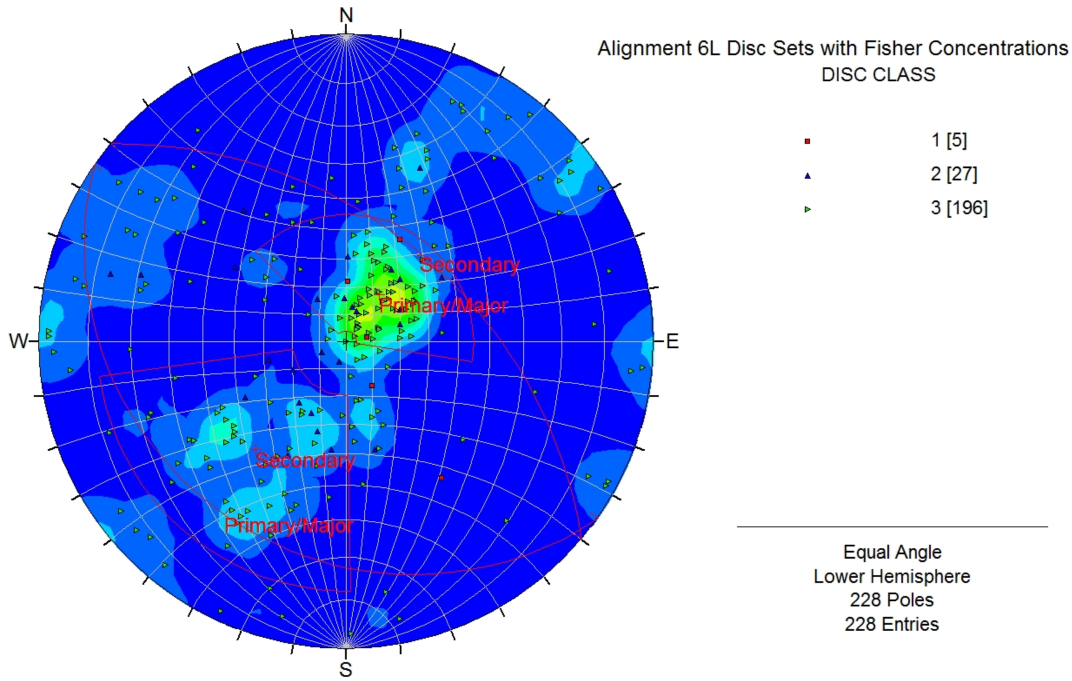
a. Axis 2 - Left Abutment: Lower hemisphere equal angle stereonet pole plot for discontinuity data, CB-1 and CB-2.



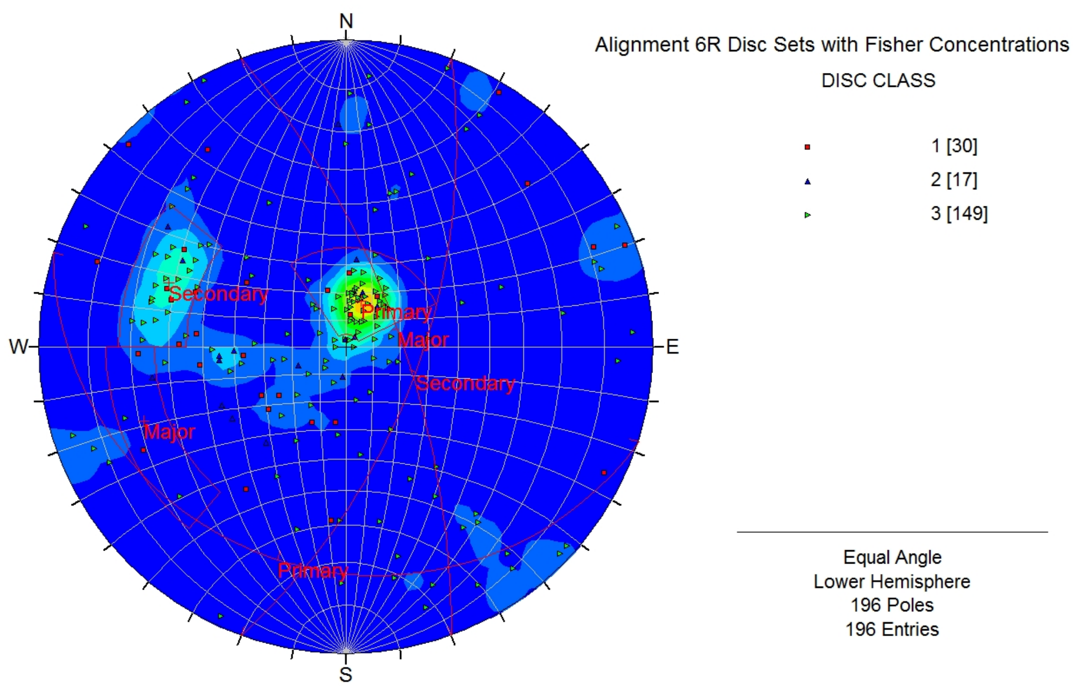
b.Axis 2 - Right Abutment: Lower hemisphere equal angle stereonet pole plot for discontinuity data, CB-3 and CB-4.

FIGURE 6-3

Stereonet Plots of Borehole Discontinuities-Axis 2



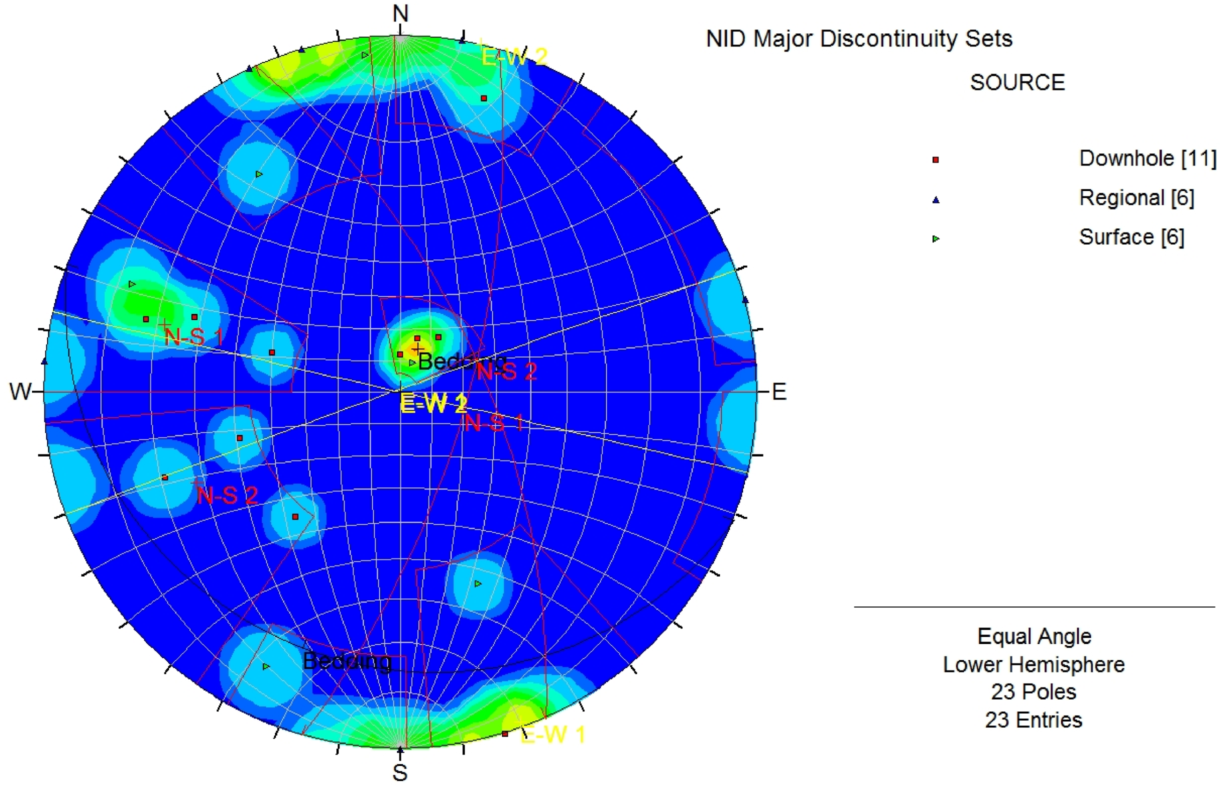
a.Axis 6 - Left Abutment: Lower hemisphere equal angle stereonet pole plot for discontinuity data, CB-5 and CB-6.



b. Axis 6 - Right Abutment: Lower hemisphere equal angle stereonet pole plot for discontinuity data, CB-8 and CB-9.

FIGURE 6-4

Stereonet Plots of Borehole Discontinuities-Axis 6



Lower hemisphere equal angle stereonet pole plot of discontinuity sets determined from all data.

FIGURE 6-5

Stereonet Plots of Discontinuities-All Data

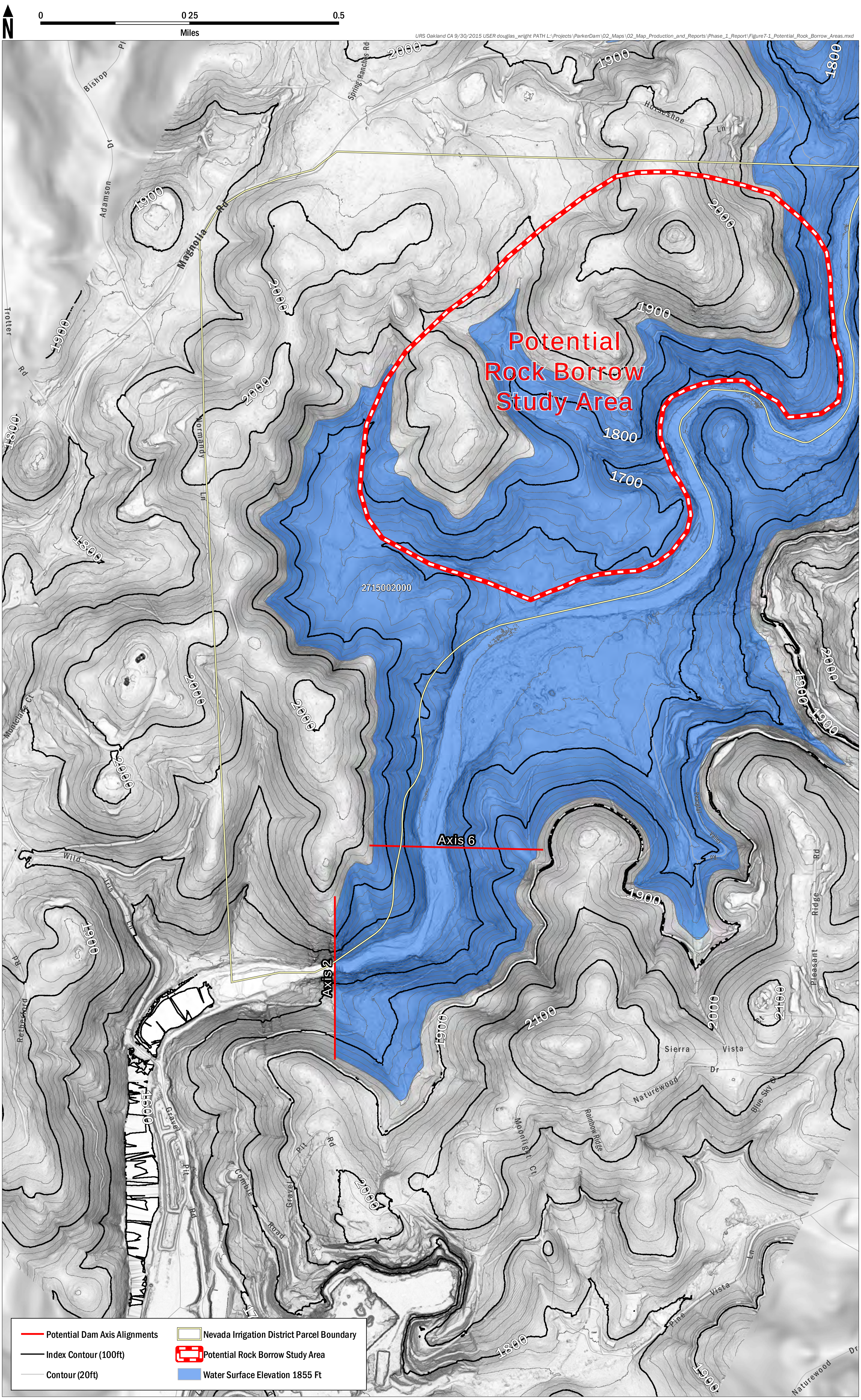


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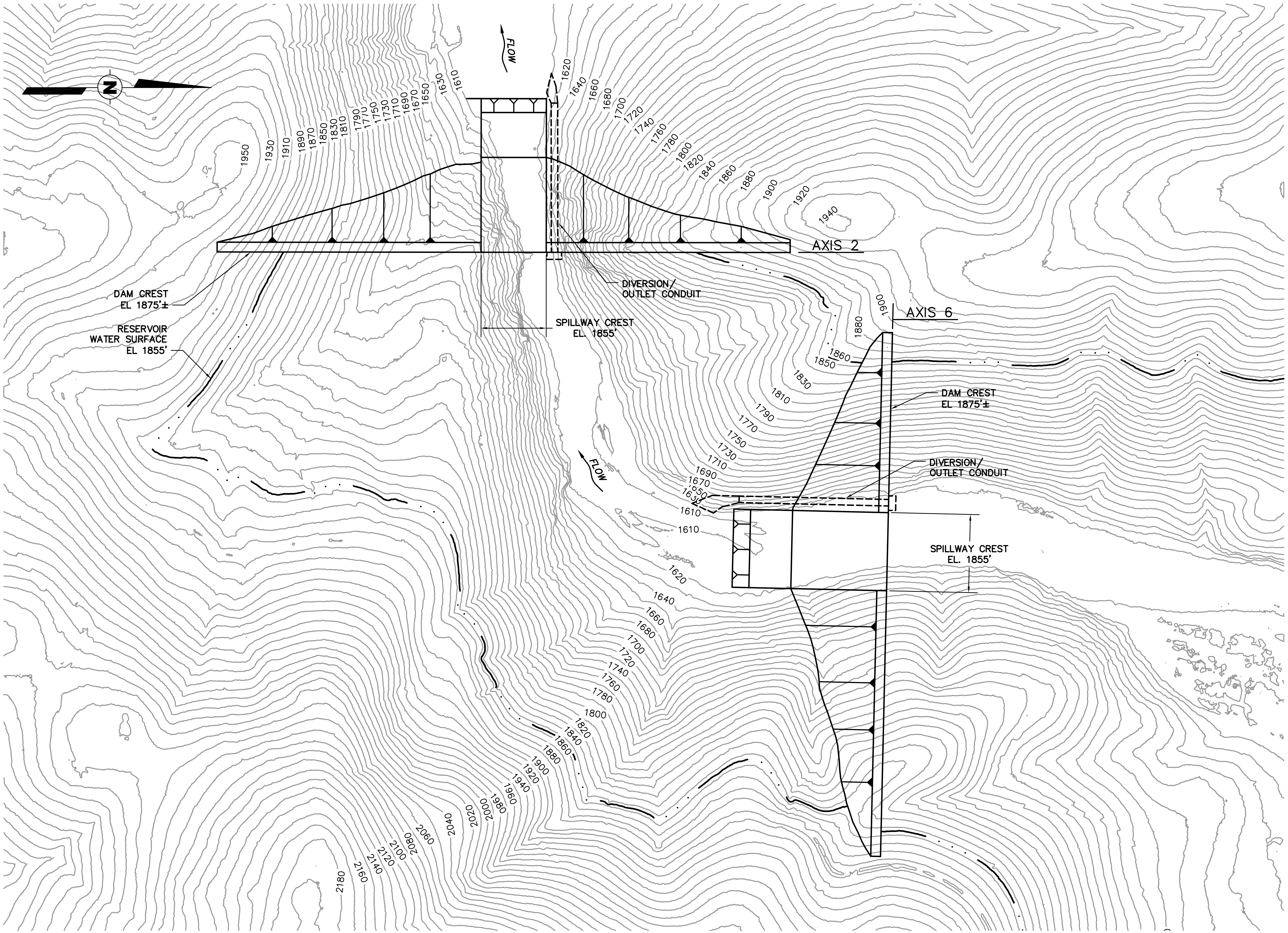
FIGURE 7-1
Potential Rock Borrow Area
110,000 Acre Ft Reservoir



NOTE:

1. EXCAVATION LINES NOT SHOWN.

0 300 600
Feet

AECOM	60393322 NEVADA IRRIGATION DISTRICT CENTENNIAL RESERVOIR PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION	Roller Compacted Concrete Dam Concept Plan	FIGURE 8-1

Dec 18, 2015 - 1:51pm
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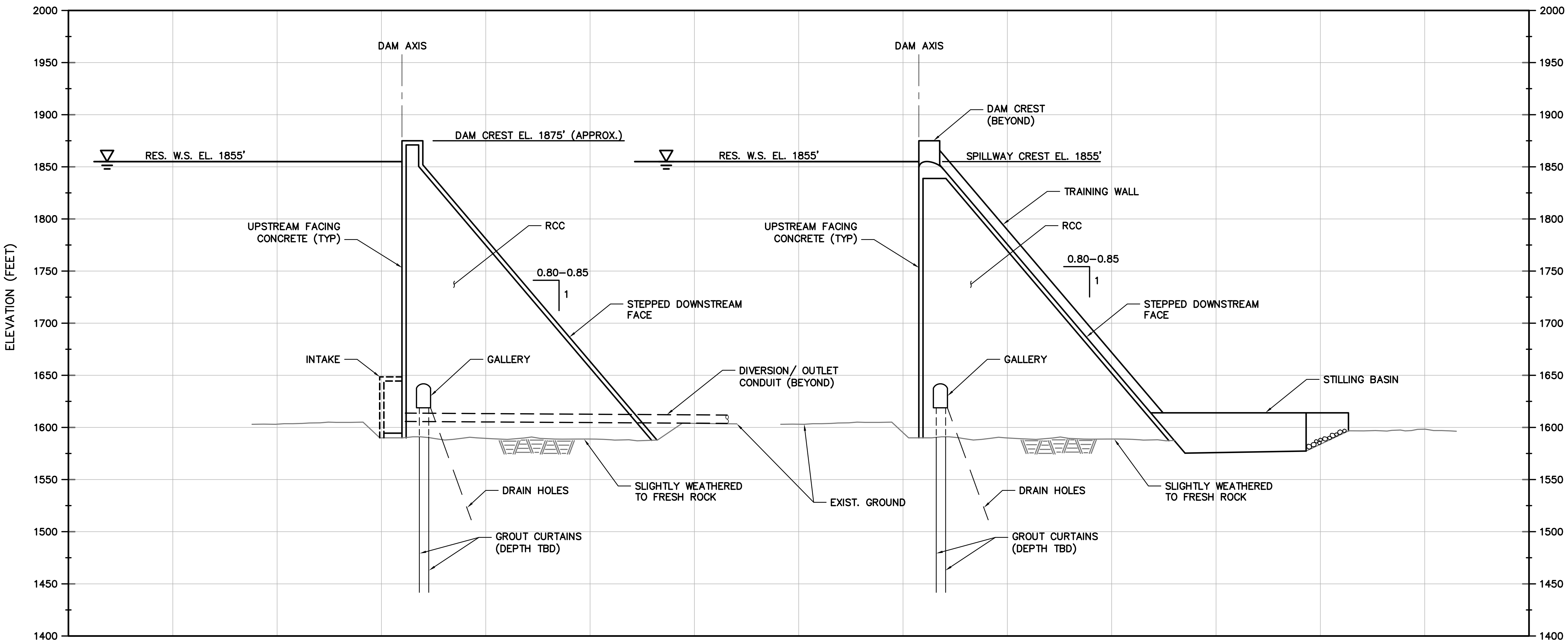
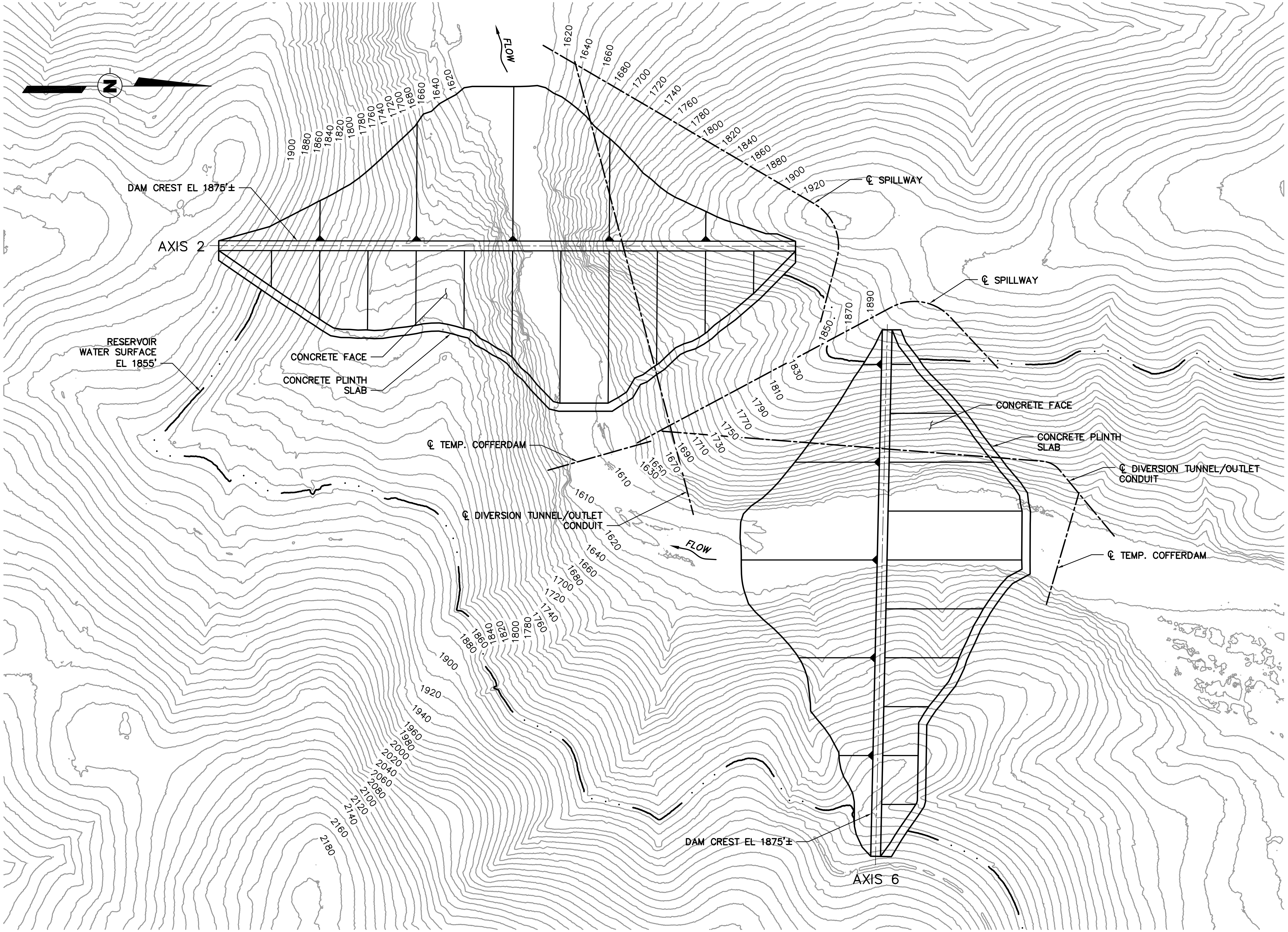


NON–OVERFLOW SECTION
NOT TO SCALE

SPILLWAY SECTION
NOT TO SCALE

Dec 18, 2015 - 1:57pm
L:\Projects\Legacy\IE_Xdrive\x_water\NID Parker Dam\13_Phase 2\Cross Sections\Figure 8-2 and 8-4.dwg

AECOM	60393322 NEVADA IRRIGATION DISTRICT CENTENNIAL RESERVOIR PROJECT PRELIMINARY GEOTECHNICAL INVESTIGATION	Roller Compacted Concrete Dam Concept Maximum Sections	FIGURE 8-2



NOTE:

1. EXCAVATION LINES NOT SHOWN.

Dec 18, 2015 - 1:22pm
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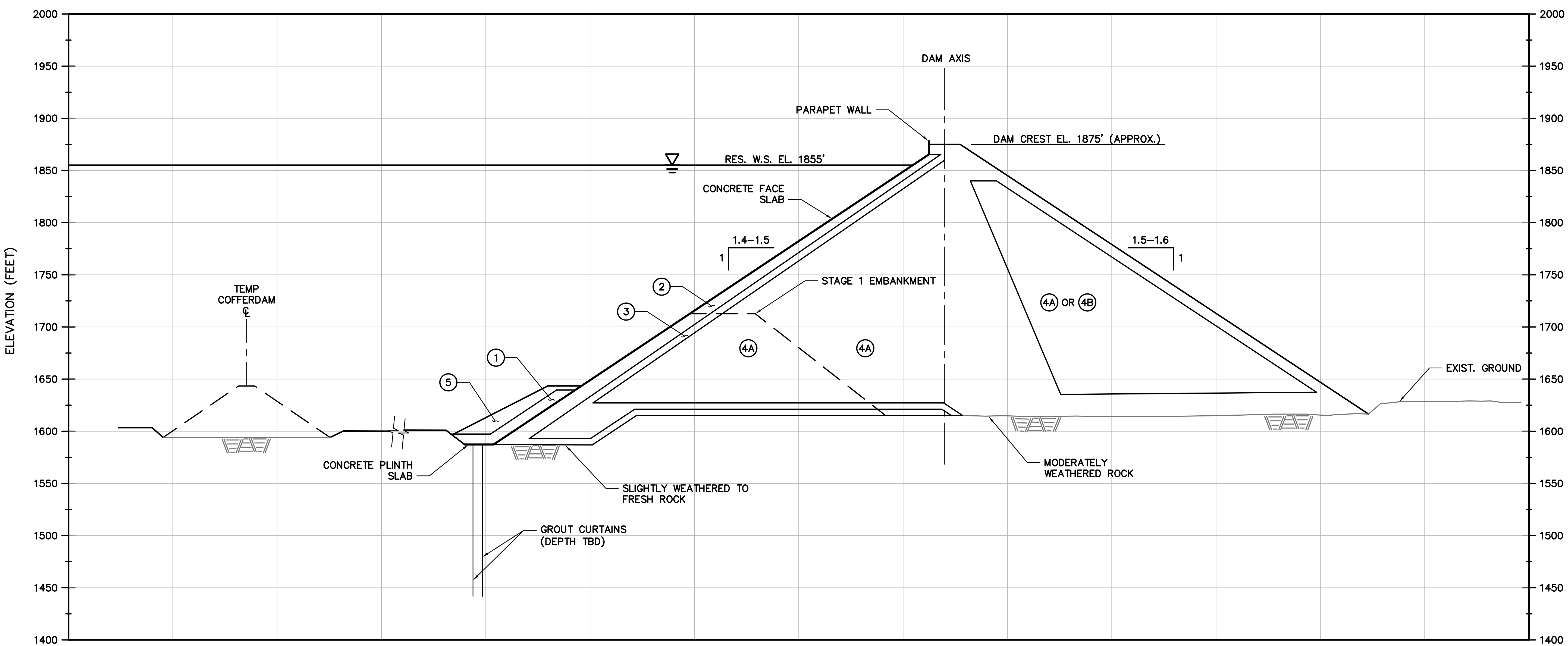
NEVADA IRRIGATION DISTRICT
CENTENNIAL RESERVOIR PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

Concrete-Faced Rockfill Dam Concept Plan

FIGURE 8-3



NOT TO SCALE

ZONE DESCRIPTIONS:

1. Clayey Soil
2. Bedding Layer (Processed Sand & Gravel)
3. Fine Rockfill
4A. Rockfill
4B. Rockfill (Weathered Rock)
5. Random Soil/Rock

NOTE:

1. FOR SPILLWAY AND DIVERSION TUNNEL/OUTLET CONDUIT LOCATIONS, SEE FIGURE 8–3.

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NEVADA IRRIGATION DISTRICT
CENTENNIAL RESERVOIR PROJECT
PRELIMINARY GEOTECHNICAL INVESTIGATION

Concrete Faced Rockfill Dam Concept
Maximum Section

FIGURE 8-4

Dec 18, 2015 - 1:55pm
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Photographs

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Photograph 1. Typical section of residual soil observed within a road cut southwest of the CRP site

Photograph 2. Northwest facing view of the Bear River from the southeast end of the site
[Illustrates distribution of alluvial deposits within the active channel (Left: point bar; Right: mid-channel bar)].

Photograph 3. West facing view of outcropping bedrock and mid-channel bar deposits within the Bear River at Axis 6

Photograph 4. West facing view of bedding in outcropping bedrock (massive basalt) at the toe of slope along north bank of Bear River, near Axis 4

[Well rounded cobble and gravel sized point bar deposits are also visible (Left).]

Photograph 5. East facing view of typical discontinuities in outcropping bedrock (massive basalt) at the toe of slope along the south bank of Bear River, near Axis 4

Photograph 6. North facing view the Bear River channel at Axis 6

Photograph 7. North facing view of the right abutment at Axis 2
[Illustrates steep (up to sub-vertical) slopes of outcropping bedrock (massive basalt).]

Photograph 8. North facing view of quarry shear zone in Teichert quarry
[Orientation between N10ºE and N12ºW (arrow points to shear zone).]

Photograph 9. North facing close-up of quarry shear zone

Photograph 10. Drilling Boring CB-3 (Axis 2, right abutment)

Photograph 11. Drill rig with televiewer logging at Boring CB-6 (Axis 6, left abutment)

Appendix A.
Seismologic Investigation

Appendix A.
Seismologic Investigation

Appendix B.
Surface Geologic Data Points and Stereonets

Appendix B.
Surface Geologic Data Points and Stereonets

Appendix C.
Seismic Refraction Survey Report

Appendix C.
Seismic Refraction Survey Report

Appendix D.
Core Boring Logs and Core Photographs

Appendix D.
Core Boring Logs and Core Photographs

Appendix E.
Water Pressure Test Results

Appendix E.
Water Pressure Test Results

Appendix F.
Televiewer and Caliper Data

Appendix F.
Televiewer and Caliper Data

Appendix G.
Downhole Geophysics
(P- and S-wave Velocities) Report

Appendix G.
Downhole Geophysics (P- and S-wave Velocities) Report

Appendix H.
Laboratory Testing Results

Appendix H.
Unconfined Compressive Strength Test Data

Appendix I.
Point Load Index Test Data

Appendix I.
Point Load Index Test Data

Appendix J. Commercial Sand and Gravel Sources

Appendix J.
Commercial Sand and Gravel Sources

Appendix K.
Potential Clay Borrow Sources

Appendix K.
Potential Clay Borrow Sources

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