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

Submitted by:
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September 18, 2017

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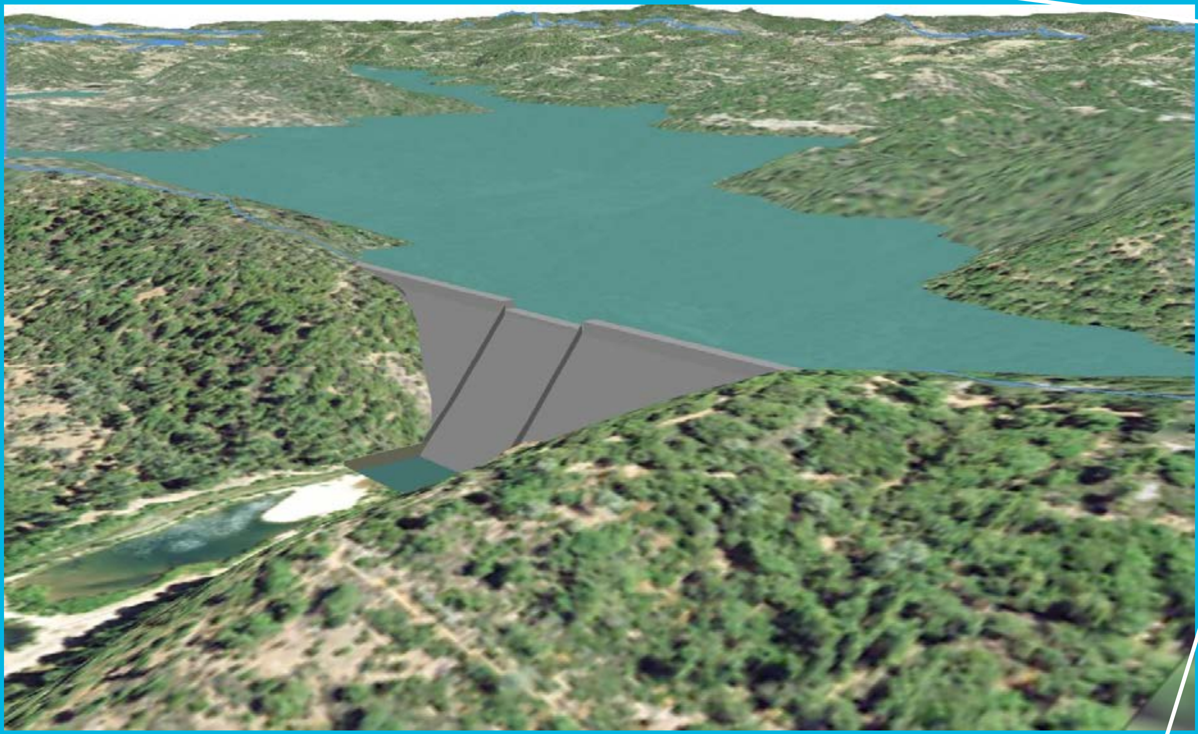
NID Centennial Reservoir Project Conceptual Engineering Report – Final



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September 18, 2017

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

Attention: Mr. Doug Roderick, P.E.

Subject: Centennial Reservoir Project
Conceptual Engineering Report – Final

Dear Mr. Roderick:

We are very pleased to submit this final Conceptual Engineering Report (CER) for the Centennial Reservoir Project located near Grass Valley, California. This final version addresses the comments from the California Division of Safety of Dams (DSOD) on the May 24, 2017, draft CER.

In accordance with the scope of work authorized under Task Order 7, this CER documents the alternatives analysis and conceptual engineering of the RCC dam. The conceptual design criteria for hydraulic, stability and seismic design, and DSOD criteria, were submitted to NID and DSOD in a technical memorandum dated February 16, 2017.

Conceptual design includes the following tasks:

- Alternatives Analysis – Evaluate roller compacted concrete (RCC) and concrete faced rockfill (CFR) dam alternatives and two dam sites to recommend a preferred alternative.
- Conceptual Engineering Analyses – Perform (a) routing of the probable maximum flood (PMF) through the spillway to size and configure the spillway and determine the required freeboard on the dam; (b) reservoir evacuation analysis through the outlet conduit; and (c) preliminary stability analyses of the dam for long-term, flood and seismic loading conditions.
- Conceptual Design of Dam and Appurtenant Works – Prepare layouts, profiles and cross sections of the dam, spillway, outlet works, diversion and cofferdam prepared in sufficient detail for general definition of the project features and for quantity and cost estimation.

The construction cost estimate and schedule are the subject of a separate technical memorandum.

Thank you for the continued opportunity to assist the NID on this very important project. We are available to discuss any questions or comments you may have on this report. Please contact me at (510) 874-3012 if you would like to schedule a time to meet.

Sincerely,
AECOM Technical Services, Inc.

M.P. Forrest

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

REGISTERED PROFESSIONAL ENGINEER
MICHAEL PAUL FORREST
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2/12/20

Enclosure:
Centennial Reservoir Project, Conceptual Engineering Report – Final

Cc: Noel Wong, Ted Feldsher, David Hughes, David Simpson (AECOM)

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Acronyms

ADAS	**automated data acquisition system**
AEP	**annual exceedance probability**
ASTM	**American Society for Testing and Materials**
CER	**Conceptual Engineering Report**
CFR	**concrete-faced rockfill**
cfs	**cubic feet per second**
CRP	**Centennial Reservoir Project**
DCTM	**Design Criteria Technical Memorandum**
DSHA	**deterministic seismic hazard analysis**
DSOD	**California Division of Safety of Dams**
FS	**factor of safety**
g	**acceleration of gravity**
GER	**Geotechnical Engineering Report**
H	**horizontal**
HMR	**Hydrometeorological Report**
lbs/cy	**pounds per cubic yard**
M	**moment magnitude**
m_b	**body-wave magnitude**
MCE	**Maximum Credible Earthquake**
M_L	**Richter local magnitude**
NID	**Nevada Irrigation District**
NOAA	**National Oceanic and Atmospheric Administration**
PGA	**peak ground acceleration**
PMF	**Probable Maximum Flood**
PMP	**Probable Maximum Precipitation**
psi	**pounds per square inch**
RCC	**roller-compacted concrete**
RQD	**rock quality designation**
RTS	**reservoir triggered seismicity**
SOP	**standard operating procedures**
UCS	**unconfined compressive strength**
USACE	**U.S. Army Corps of Engineers**
USBR	**U.S. Bureau of Reclamation**
USCOLD	**United States Committee on Large Dams (now United States Society on Dams)**
USGS	**United States Geological Survey**
V	**vertical**
WGNCEP	**Working Group on Northern California Earthquake Probabilities**

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Executive Summary

Background

The Nevada Irrigation District (NID) is undertaking engineering and planning studies for the proposed 110,000–acre-foot Centennial Reservoir, located on the Bear River between the existing Rollins and Combie Reservoirs, which are also owned and operated by NID. This storage corresponds to a maximum normal reservoir water surface of approximately Elevation 1,855 feet, which will require a 275-foot-high dam on the Bear River.

To advance the engineering for the proposed project, phased geotechnical investigations of the site were performed to identify the preferred dam axis locations and preferred dam types. The Phase I studies (2015) identified potential dam axis alignments and discussed the preferred axis locations and dam types considered most viable for the site.

Two potentially viable dam types were initially identified: roller-compacted concrete (RCC) and concrete faced rockfill (CFR). Two potential dam axis alignments were also identified, referred to as Axis 2 and Axis 6. The two dam types and two dam axis alignments were discussed in the 2016 Phase II Preliminary Geotechnical Investigation Report by AECOM. The Phase II investigation objectives were to 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. The Phase III investigations focused on the RCC dam at Axis 2, which are the preferred dam type and axis location. The geotechnical investigations carried out under Phases II and III are documented in the 2017 Geotechnical Engineering Report (GER). The GER is a companion report to this Conceptual Engineering Report (CER). Refer to the GER for geologic and geotechnical conditions at the dam site and rock borrow areas.

This CER documents the rationale for the recommendation of the preferred dam site and dam type. It also includes analyses and plans, sections, and main details of the recommended dam and appurtenant works.

Alternatives Evaluation

An alternatives comparison of the RCC and CFR dam types included (a) construction cost; (b) relative long-term operation and maintenance considerations; (c) materials availability; (d) constructability; and (e) areas of both permanent and temporary disturbance. The alternatives were ranked for each of these parameters to identify the preferred dam type. The RCC dam ranked higher than the CFR dam alternative in most categories. Based on the alternatives analysis, an RCC dam at Axis 2 was identified as the preferred alternative because it would (a) have a 3-foot lower reservoir elevation to store the same reservoir volume of 110,000 acre-feet; (b) have the lowest expected construction cost; (c) be constructed in less time than a CFR dam; (d) have a much smaller footprint area than a CFR dam, which is environmentally beneficial; and (e) would be much more capable of withstanding flood overtopping during construction than a CFR dam. The RCC dam at Axis 2 was therefore carried forward as the preferred dam type and axis location for the Centennial Reservoir Project.

Geologic Conditions

Geologic mapping performed at the site identified colluvial and residual soils on hillside slopes, and alluvium in the Bear River channel. Rock outcrops observed on both the northern and southern sides of the river canyon are comprised of basalt flow rock and volcaniclastic rock. The outcrops display widely spaced steep joints and gently inclined volcanic flow and depositional bedding surfaces.

In response to a question from the California Division of Safety of Dams (DSOD) on active faulting in the site area, a subconsultant specializing in seismic geology investigations was retained to perform an independent evaluation of the potential for active faulting at the Axis 2 site. Their report is contained as an appendix to the GER. In summary, they concluded that there is a lack of positive evidence to support the presence of active faulting at the proposed Axis 2 site; and that the potential for active faulting is low.

Seismic Design Parameters

Seismic sources, historical seismicity, and reservoir-triggered seismicity are discussed in the GER and summarized in this CER. 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 horizontal acceleration response spectra were developed for a maximum earthquake of moment magnitude (M) 6.5 on the Wolf Creek fault (Maximum Credible Earthquake, MCE). Based on DSOD guidelines, the 69th percentile deterministic ground motion, which has a peak horizontal ground acceleration of 0.31 g, was used for design of the dam.

Dam Foundation Conditions

The Phase II geotechnical investigation focused on cost-effectively obtaining the data needed to evaluate the technical feasibility of the potential dam sites at Axis 2 and Axis 6, and dam types. The primary emphasis was on identifying significant geologic flaws or other undesirable foundation conditions present in the areas investigated. The Phase III geotechnical investigation focused on filling in data gaps at the selected Axis 2 site and on exploring two potential rock borrow areas. The field investigation and laboratory testing included (a) geologic outcrop mapping; (b) seismic refraction surveys; (c) core borings; (d) water pressure (packer) testing and televiewer/caliper logging in borings; (e) seismic velocity measurements in selected borings; and (f) unconfined compressive strength tests on selected core samples.

The upper part of the rock foundation at Axis 2 is weathered and fractured, and the rock conditions improve with depth. The degree of fracturing and weathering decreases with depth, and hydraulic conductivities also generally tend to decrease with depth, with the exception of the upper part of the right abutment, where this trend does not occur. The depth of excavation is expected to extend to more than 100 feet in some locations of the foundation. The discontinuity analysis indicates that the more prominent features observed in borings and borehole televiewer surveys are not likely to persist as discrete, continuous foundation defects.

Rock Borrow Material Sources

Two rock borrow areas, termed the North and South Rock Borrow Areas, were investigated based on topographic conditions and proximity to the dam site area. Both of these potential rock borrow areas are on hills, on the northern side of the Bear River, north of the dam axis. Numerous basalt outcrops were found in both rock borrow areas. Subsequent to the geotechnical investigation, a future bridge over the Bear River was proposed near the North Rock Borrow Area. It is planned that the bridge will be constructed prior to rock borrow excavation operations; therefore, this area will be precluded from use. For this reason, only the South Rock Borrow Area is discussed in this CER. Another rock borrow source under consideration is the existing Bear River Quarry, which is located about a half-mile south of the dam site.

Stripping will be required to remove soil and weathered rock to expose slightly weathered to fresh rock suitable for RCC and concrete aggregate. Based on the core boring and seismic refraction data,

overburden stripping depths to expose suitable rock in the South Rock Borrow Area could range from 20 to 60 feet. These stripping depths will need to be confirmed by further geotechnical investigations.

Dam and Appurtenant Works

The design criteria for the RCC dam were based on DSOD criteria and guidelines from the U.S. Army Corps of Engineers (USACE) and the U.S. Bureau of Reclamation (USBR). The criteria are documented in the Design Criteria Technical Memorandum (DCTM) by AECOM that defines the project performance requirements, spillway and diversion flood criteria, stability and seismic design criteria. Stability analyses were performed for long-term static reservoir loading at full-pool, Probable Maximum Flood (PMF) condition and maximum credible earthquake (MCE) loading.

The dam foundation excavation will locally extend more than 100 feet below the existing ground surface to reach the foundation objective of slightly weathered to fresh, hard rock. Foundation treatment will consist of two grout curtains extending to as much as 135 feet below the foundation to control seepage through the foundation; consolidation grouting to strengthen the rock mass and increase the stiffness of the foundation; and drain holes to control uplift pressures beneath the RCC dam.

The RCC dam will have a structural height of 285 feet above the foundation, and a 1,600-foot-long crest at Elevation 1,878 feet. The RCC dam will include a spillway integral with the body of the dam with the capacity to pass the PMF to the Bear River channel. The conceptual design also includes a 10.5-foot-diameter, low–level, steel-lined outlet conduit cast into the body of the dam. Flows will be controlled by a sleeve valve at the downstream end of the pipe. A slide gate at the upstream end of the conduit will be closed for conduit inspections. The outlet conduit and valve were sized to meet DSOD reservoir evacuation criteria.

Performance monitoring instrumentation will be included in the design of the dam to monitor reservoir level, uplift pressures, seepage, crest movement, and earthquake accelerations.

Diversion

Diversion of river flow through the dam site could be accomplished by a diversion structure (e.g., a box culvert) constructed in the river channel, or through a tunnel excavated in an abutment. The contractor will be responsible for the design of the river diversion system. When diversion is no longer necessary, the culvert or tunnel would be plugged with concrete. The RCC dam would be placed on top of the culvert, if selected. A cofferdam would be needed to divert river flow through the culvert or tunnel.

With the RCC dam, flood flows over the dam during construction would not pose a dam safety issue due to a breaching failure, because the RCC would not be significantly erodible. The risk of controlling potential flood damage to the construction site will be the responsibility of the contractor. NID can operate Rollins Reservoir to reduce flood damage at the Centennial Dam site.

Construction Considerations

A conceptual site layout for RCC dam construction was prepared that shows the assumed rock borrow area, RCC and conventional concrete batch plant areas, disposal sites, and staging areas. The contractor will select its own construction site layout and plan. Construction considerations include dam site preparation and foundation excavation. Development of on-site rock borrow areas for RCC aggregate includes stripping roughly 1 million cubic yards of overburden and weathered rock. The underlying fresh rock would be drilled, blasted, crushed, screened, and washed to produce the RCC aggregate. An alternative source of rock material is available at the existing Bear River Quarry, south of the dam site. Approximately 60,000 tons of cement and 60,000 tons of fly ash would be imported to produce the required volume of RCC.

Operation, Maintenance, and Inspection

Operation, maintenance, and inspection of the dam and appurtenant structures will include the dam, spillway, and outlet works, including maintenance of the drain system. Inspection and monitoring results will need to be regularly collected and evaluated, and forwarded to DSOD annually. A Standard Operations and Procedures Manual (which includes operation, maintenance, and inspection), Emergency Action Plan, and Initial Reservoir Fill Plan would need to be prepared and submitted to DSOD during the final design phase.

Recommendations for Further Geotechnical Investigations and Preliminary Engineering

A fourth phase of geotechnical investigations is recommended to obtain additional data to develop the project design and reduce uncertainty in understanding of the dam foundation. If an on-site rock borrow area is considered further, additional investigation would also be needed to better characterize the subsurface conditions, and to locate the rock excavations to minimize stripping volume.

Preliminary engineering should be performed to advance the project design and should include the following activities: (a) confirming the dam axis alignment to improve foundation topographical conditions; (b) confirming the optimum spillway crest length; (c) developing the outlet works arrangement; (d) developing the river diversion approach; (e) routing the design flood through the spillway to determine the stilling basin size and height of the spillway training walls in the chute; (f) performing finite element stress and stability analyses of the dam, particularly to assess the seismic performance of the dam; (g) developing the details for design of the dam, spillway, outlet works, and mechanical equipment; and (h) updating the construction cost estimate and schedule to reflect the preliminary design of the project.

1. Introduction

1.1 Background

The Nevada Irrigation District (NID) is undertaking engineering and planning studies for a proposed water storage reservoir, to be located on the Bear River between the existing Rollins and Combie Reservoirs, which are also owned and operated by NID. 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 geotechnical investigations, to assist in identifying preferred dam axis locations and preferred dam types for further study. The study was carried out in several phases, as 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 1920s (Tibbetts, 1926). The dam site area is in Nevada County on the northern side of the Bear River, and in Placer County on the southern 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 1,855 feet (Figure 1-2). Retaining a reservoir at this elevation would require a dam height of approximately 275 feet above the Bear River.

Phased geotechnical investigations (Phases II and III) carried out in 2015 and 2016 are documented in the AECOM 2017 Geotechnical Engineering Report (GER) (AECOM, 2017a). The GER is a companion report to this Conceptual Engineering Report (CER).

1.2 Purpose and Scope

This CER documents the rationale for the recommendation of the preferred dam site and dam type. It also includes analyses and plans, sections, and main details of the recommended dam and appurtenant works. The scope of work described in this CER was authorized under Task Order No. 7, executed on August 30, 2016.

Conceptual design includes the following tasks:

- Design Criteria Technical Memorandum (DCTM) – Define the basic criteria for the project, including spillway, reservoir evacuation, stability and seismic design, and DSOD criteria. The DCTM was submitted to NID on February 17, 2017 (AECOM, 2017b).
- Alternatives Analysis – Evaluate roller-compacted concrete (RCC) and concrete-faced rockfill (CFR) dam alternatives and two dam sites to recommend a preferred alternative.
- Conceptual Engineering Analyses – Perform (a) routing of the probable maximum flood (PMF) through the spillway to size and configure the spillway, and determine the required freeboard on the dam; (b) reservoir evacuation analysis through the outlet conduit; and (c) simplified preliminary stability analyses of the dam for long-term flood and seismic loading conditions.
- Conceptual Design of Dam and Appurtenant Works – Prepare layouts, profiles, and cross-sections of the dam, spillway, outlet works, and diversion prepared in sufficient detail for general definition of the project features, and for quantity and cost estimation.
- CER – Prepare this CER, which documents the alternatives analysis and conceptual engineering of the RCC dam.

The construction cost estimate and schedule will be submitted separately in a technical memorandum.

1.3 Organization of CER

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

Section 2 – Discusses the alternatives analysis and rationale for selection of preferred dam site and dam type.

Section 3 – Summarizes the regional and dam site geology.

Section 4 – Summarizes the seismic source characterization and ground-motion parameters.

Section 5 – Summarizes the foundation conditions and construction materials.

Section 6 – Provides a description of the dam and appurtenant works, including foundation treatment, dam and spillway, construction materials, outlet works, river diversion, and dam performance instrumentation.

Section 7 – Presents the results of the stability analyses.

Section 8 – Presents the results of the design flood routing through the reservoir.

Section 9 – Discusses the main construction considerations.

Section 10 – Summarizes the key operation, maintenance, and inspection items.

Section 11 – Presents the conclusions and recommendations.

Section 12 – Lists the references cited in this CER.

Sections 3, 4, and 5 are summarized from the Phase III GER (AECOM, 2017a), and are included in the CER for completeness. Refer to the GER for geologic and geotechnical conditions at the dam site and rock borrow areas.

1.4 Acknowledgements

The following key AECOM personnel contributed to this CER:

- **Project Manager: Michael Forrest, P.E., G.E.**
- **Principal-in-Charge: Noel Wong, P.E.**
- **Project Engineer: David Hughes, P.E.**
- **Project Geologist: David Simpson, C.E.G.**
- **Civil Designer: Steven Tough, P.E.**
- **Hydrologic and Hydraulic Analyses: Phillip Mineart, P.E.**
- **Seismologic Investigation: Ivan Wong, Patricia Thomas, Ph.D., and Judith Zachariasen, Ph.D.**
- **Geotechnical Engineering: Josh Zupan, P.E., Ph.D.**
- **Independent Technical Review: Scott Jones, P.E., Ph.D.**

1.5 Limitations

The professional judgments presented in this report regarding the site conditions are based on information obtained from reference data review, geologic mapping, and phased geotechnical investigations.

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.

2. Alternatives Analysis and Selection of Preferred Alternative

2.1 Summary of Previous Studies

This section summarizes the *Preliminary Geotechnical Investigation, Phase II Report – Final* (AECOM, 2016a) and the *Conceptual-level Opinion of Probable Construction Cost* (AECOM, 2016b). Those two reports provide the basis for selection of the preferred dam site and preferred dam type.

2.1.1 Geotechnical Investigations, Phases II and III

The Phase II report concluded that both Axis 2 and Axis 6 were acceptable from a geotechnical standpoint for either an RCC dam or CFR dam (AECOM, 2016a). Fatal flaws were not identified at either site. Both dam types were judged to be suitable for the site based on the observed foundation conditions. Rock materials suitable for both RCC gravity dam aggregates and a CFR dam were judged likely to be available in the reservoir area and/or from the nearby Bear River Quarry in sufficient quantities for either dam type. The Phase III geotechnical investigation confirmed this conclusion (AECOM, 2017a). 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 1,855 feet, the reservoir capacity would be about 7,000 acre-feet less for a dam at Axis 6 than further downstream at Axis 2. Alternatively, a dam at Axis 6 would need to be about 3 feet higher to provide the same reservoir storage capacity as a dam at Axis 2.

2.1.2 Conceptual-Level Opinion of Probable Construction Cost (OPCC)

To assist in evaluation of potential cost differences between the dam types and dam sites, OPCCs were developed for RCC dam and CFR dam alternatives at each of the two site locations considered (Axis 2 and Axis 6) (AECOM, 2016b). The OPCCs and conceptual-level design layouts were developed based on the available geotechnical information presented in the Phase II Report (AECOM, 2016a).

As part of preparing the OPCCs, conceptual-level construction schedules were prepared for each dam type to provide a comparative assessment of the relative construction durations of the RCC and CFR alternatives. The schedules indicated that the RCC dam could potentially be constructed in about 2½ years, but the CFR dam would take about 4 years to construct.

The conclusion of the conceptual-level OPCC study was that the RCC dam at either axis is expected to have a lower estimated construction cost than the CFR dam type. The RCC dam at Axis 2 is expected to have the lowest construction cost of the alternatives considered. The estimated RCC dam cost is about 75 to 80 percent of the cost of the CFR dam, depending on whether it is at Axis 2 or Axis 6 (AECOM, 2016b).

2.2 Preferred Dam Site and Dam Type

The plans and typical sections of the RCC and CFR alternative dam types at Axes 2 and 6 are presented in Appendix A. An alternatives comparison matrix of these two dam types was prepared, and is presented in Table 2-1. The comparison includes evaluation of the following parameters:

1. Materials available on site
2. Imported cement and fly ash
3. Relative long-term operation and maintenance
4. Constructability and risk
5. Withstanding flood overtopping during construction

6. **Areas of both permanent and temporary disturbance**
7. **Potential for water quality degradation (turbidity) in Bear River**
8. **Estimated construction cost**
9. **Estimated schedule duration**

The alternatives were ranked for each of the above comparison parameters to identify the preferred dam type, as shown in Table 2-1.

As shown in Table 2-1, the RCC dam ranked higher than the CFR dam alternative in more categories. Based on the alternatives analysis, an RCC dam at Axis 2 was identified as the preferred alternative, for the following main reasons:

- **Axis 2 would have a 3-foot lower reservoir elevation than for Axis 6 to store the same reservoir volume of 110,000 acre-feet. This lower elevation would reduce the level of inundation around the reservoir rim.**
- **The RCC dam alternative at Axis 2 has the lowest expected construction cost of the alternatives. One reason for the lower cost is that the spillway and outlet works can be incorporated into the body of the RCC dam. These costs were significant for the CFR dam alternative.**
- **The RCC dam could be constructed in less time than a CFR dam.**
- **The RCC dam would have a much smaller footprint area than a CFR dam, which is beneficial from an environmental standpoint.**
- **The RCC dam would be much more capable of withstanding flood overtopping during construction than a CFR dam.**

Based on the above reasons, an RCC dam at Axis 2 was carried forward as the preferred dam type and axis location for the Centennial Reservoir Project. The remainder of this report focuses on the conceptual design of the RCC dam at Axis 2.

Table 2-1. Alternatives Comparison Matrix

Comparison Parameter	RCC Dam	CFR Dam	RCC Dam Ranking	CFR Dam Ranking
1. Materials available on site	· Rock material available on site for RCC and concrete aggregate. Bear River Quarry could also supply these materials. Less required volume than for CFR dam.	· Rock material available on site for rockfill, transition, filter, drain, and concrete aggregate. Bear River Quarry could also supply these materials.	1	1
2. Imported cement and fly ash	· Cement and fly ash would be imported. A greater quantity is required than for CFR dam, requiring more truck trips and traffic disruption.	· Cement and fly ash would be imported.	2	1
3. Relative long-term operation and maintenance	· NID personnel will periodically monitor the dam site facilities. · Dam, spillway, and intake – debris and vegetation removal. · Foundation drains and RCC body drains will need to be periodically cleaned out. · Mechanical and electrical equipment – periodically exercising the valves and checking the valve actuators. · Instrumentation – manual readings of dam performance instrumentation. · Site area – repair of erosion areas and removal of vegetation.	· NID personnel will periodically monitor the dam site facilities. · Dam, spillway, and intake – debris and vegetation removal. · Concrete facing may require repair of waterstops. · Mechanical and electrical equipment – periodically exercising the valves and checking the valve actuators. · Instrumentation – manual readings of dam performance instrumentation. · Site area – repair of erosion areas and removal of vegetation.	1	1
4. Constructability and risk	· Due to the simpler river diversion and spillway and outlet works in the body of the dam, relatively straightforward to construct. · Temperature control of RCC aggregates would be required. · Field quality control and foundation requirements would be more involved than for a CFR dam. · RCC construction can be affected by rainy and hot-weather conditions.	· In addition to the dam, spillway and diversion/outlet tunnel excavations would be subject to unknown geotechnical conditions. · Would require large cofferdam for diversion. · Field quality control and foundation requirements would be less involved than for an RCC dam. · A CFR dam would be less sensitive to adverse weather conditions during construction than an RCC dam.	1	2

Comparison Parameter	RCC Dam	CFR Dam	RCC Dam Ranking	CFR Dam Ranking
5. Withstanding flood overtopping during construction	· **Flood flows over the dam during construction would not pose a dam safety issue due to a breaching failure, because the RCC would not be significantly erodible.** · **RCC dam would be much more capable of withstanding flood overtopping during construction than would a CFR dam.**	· **Would require a cofferdam and diversion tunnel designed for at least the 100-year flood event.** · **Downstream slope of dam could require reinforcement to prevent erosion due to through-flow.**	**1**	**2**
6. Areas of both permanent and temporary disturbance	· **RCC dam would have a much smaller footprint area than a CFR dam. This is beneficial from an environmental standpoint.**	· **With the larger dam footprint and spillway on the right abutment, the footprint area would be significantly larger than for the RCC dam.**	**1**	**2**
7. Potential for water quality degradation (turbidity) in Bear River	· **Smaller potential for water quality degradation than for CFR dam because of smaller rock volume demand and smaller cofferdam.**	· **Greater potential for water quality degradation than for RCC dam because of larger rock volume demand and larger cofferdam.**	**1**	**2**
8. Estimated construction cost*	· **$260 million**	· **$330 million**	**1**	**2**
9. Estimated schedule duration*	· **2½ years**	· **4 years**	**1**	**2**

*** AECOM, 2016b.**

3. Regional and Dam Site Geology

This section is a summary of the regional geologic setting and dam site geology. For further detail, refer to the Phase III Geotechnical Engineering Report (AECOM, 2017a).

3.1 Regional Geologic Setting

The proposed site for the CRP is on the Bear River, Nevada and Placer Counties, California, in the Central Belt of the northern Sierra Nevada geomorphic province. An excerpt from the Geologic Map of the Grass Valley-Colfax Area (Tuminas, 1983) is included on Figure 3-1.

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 on the eastern limb of the Lake of the Pines Syncline, in 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 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. Field observations confirm that the site area is in 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 4). The Weimar Fault zone is approximately 1.25 miles (2 kilometers) due east of the site, while the Wolf Creek Fault Zone is approximately 3.75 miles (6 kilometers) due west. Both fault zones trend north-northwest; 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 easterly directed movement, then dip-slip reverse movement, followed by right-lateral strike-slip movement, and reverse or oblique-reverse movement (Tuminas, 1983). Although 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 4.

3.2 Dam Site Geology

3.2.1 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, increasing in frequency with depth. The soils represent a typical colluvial/residual weathering profile, and are a product of weathering of the underlying rock.

3.2.2 Alluvium

The sands and gravels currently in Lake Combie and the Bear River channel are primarily a result of gold mining in the early 1880s (Dupras, 1984). These operations used hydraulic mining procedures 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.

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 that is present upstream to the east in the Bear River drainage basin, but not in the project site. Though none were observed during the geologic mapping effort, the alluvial deposits may also contain chert, based on the mapping performed by Tuminas (1983). Alluvium is present primarily as gravel bars in the river channel, which also contains extensive rock outcrops.

3.2.3 Bedrock Conditions

The project site area is 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.

3.2.4 Rock Description

The observed outcrops on both the northern and southern 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 flow rock and volcaniclastic rock. The outcrops display widely spaced steep joints and gently inclined volcanic flow and depositional 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.

In the river channel, many of the observed outcrops have been smoothed by fluvial erosion. Surficial and structural geologic mapping performed for this investigation is shown on Figure 3-2. Bedrock mapping is depicted with two classes, based on continuity of outcrops, amount and interpreted depth of soil, and degree of weathering. Areas mapped as bedrock outcrop (Class 1) are characterized by extensive, continuous rock outcrop at the surface that is generally moderately weathered to fresh, with occasional small, localized deposits of talus, soil, and/or colluvium. Areas mapped as bedrock slope (Class 2) have fewer, isolated rock outcrops, typically with a greater degree of weathering. These areas may have locally thick deposits of residual soil where rock has weathered in place, and may have some thin (generally less than several feet) deposits of colluvial soil. The margins of areas mapped as bedrock slopes (Class 2) are typically diffuse and gradational, with adjacent bedrock outcrop or colluvial slopes. Also shown on Figure 3-2 are the locations of the observed rock outcrops where flow and clastic bedding were observed and recorded, as well as the locations of observed apparent landslides.

3.2.5 Rock Structure Observed in Outcrops

Rock structure orientations were measured in the Axis 2 study area. The discontinuity locations are shown on Figure 3-2, and the data are presented in AECOM (2017a). The principal discontinuity sets from the data are summarized in Table 3-1. The stereonets show consistent south-southwest dipping flow and clastic bedding orientations, with a concentration of strikes and dips centered at N55^{o}W (125^{o} azimuth), 12^{o}SW. Other discontinuity features plotted are clustered into distinct concentrations

representing two prominent, steeply dipping joint sets (joint sets 1 and 2 in Table 3-1) that trend roughly north-south and east-west.

Table 3-1. Discontinuity Sets from Geologic Mapping

Strike (Degrees Az.)	Dip (Degrees)	Discontinuity Type	No. of Data Points
120 to 130	12 SW	Bedding	25
8 to 20	80 E	Joint Set 1	37
277 to 292	85 N	Joint Set 2	29

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.

3.2.6 Landslides and Rockfalls

One active landslide was mapped on the southern canyon slope adjacent to a sharp turn in the river, as shown on Figure 3-2. This landslide extends upslope 140 feet from the left bank of the river channel to approximately Elevation 1,760 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. Two other possible landslide deposits were mapped in the slopes north of the Bear River upstream and downstream of the Axis 2 dam footprint, as shown on Figure 3-2. 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.

3.2.7 Faults

No faults considered active by either the California Geological Survey or U.S. Geological Survey (USGS) were mapped in the CRP dam site area. Conclusions from fault studies are summarized below (AECOM, 2017a):

- There is a lack of positive evidence to support active faulting at the proposed Centennial Dam Axis 2 site.
- The potential for active faulting at the Axis 2 site is low. The discontinuous nature of the fault observed in the Bear River Quarry, its association with late-stage mafic dike(s), and its lack of associated geomorphic expression all contra-indicate a potential for active faulting.
- Possible linear geologic structures identified in seismic refraction surveys at the site appear to correlate with lithologic contacts and mapped slope failures, rather than faulting.
- The meandering expression of the Bear River corresponds roughly to the north-south and east-west geomorphic lineaments that appear to be related to the regional orthogonal bedrock joint pattern, rather than faulting.
- Volcanic stratigraphy near the Axis 2 site appears to be relatively consistent, with a moderate to gentle southwest dip. The absence of vertical separation of lithologic contacts further supports the conclusion that faulting through Axis 2 is not present.

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4. Seismic Source Characterization and Ground Motion Parameters

This section presents the preliminary seismologic investigation completed for the project site, and includes (1) seismic source characterization; (2) historical seismicity, (3) evaluation of the potential for reservoir triggered seismicity, and (4) deterministic seismic ground motions. The seismologic investigation is presented in AECOM (2017a), and is summarized below.

4.1 Seismic Source Characterization

4.1.1 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-kilometer-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 kilometers west of the project site; and the Weimar fault, approximately 2 kilometers east of the project site.

4.1.2 Lineament Observations

The study included review of LiDAR data in the immediate vicinity of the project site, as well as review of 1975 and 1978 USGS black-and-white stereo aerial photography over a wider region, encompassing the breadth of the Foothills fault system and extending about 25 kilometers north and south of the project site.

Based on the analysis of the photographs and the LiDAR, a preliminary lineament map was developed (AECOM, 2017a). 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.

4.2 Historical Seismicity

The area of the proposed dam site has experienced very few historical earthquakes (Figure 4-1). The only reported events of magnitude M 5.0 or larger within 65 kilometers 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, mb, 5.9) Oroville earthquake that occurred about 60 kilometers to the northwest of the proposed dam site.
- **September 12, 1966**: M 5.9 earthquake occurred near Boca, California, a distance of 55 kilometers east-northeast of the proposed dam site.

- **March 3 and June 23, 1909: Two M ≥ 5 events occurred 41 and 44 kilometers northeast of the dam site, respectively. These events include an M_L 5 earthquake on March 3, and an M 5.5 event (unknown magnitude scale) on June 23.**

4.3 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 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. 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 an M 6.5 event, which is consistent with the maximum event assigned to faults in the Foothill fault system, as described in Section 4.4. This RTS event is also consistent with the background seismicity considered significant to the reservoir, and is therefore not expected to control the design.

4.4 Deterministic Seismic Ground Motions

4.4.1 Earthquake Magnitude

The maximum earthquake for any fault in the Foothills fault system is considered to be M 6.5, with a surface rupture length of less than 20 kilometers. 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).

4.4.2 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 percent-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 (Maximum Credible Earthquake, MCE).

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 Next Generation of Attenuation Relationship – West2 Project sponsored by Pacific Earthquake Engineering Research 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 AECOM (2017a). The median, 69th and 84th percentile geometric mean deterministic spectra are also compared in AECOM (2017a). The median,

69th- and 84th-percentile peak horizontal ground accelerations (PGAs) are 0.23, 0.31, and 0.42 g, respectively. The response spectra are shown on Figure 4-2.

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

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5. Foundation Conditions and Construction Materials

5.1 General

This section summarizes both the Phase II and Phase III geotechnical investigations. The Phase II geotechnical investigation focused on cost-effectively obtaining the data needed to evaluate the technical feasibility of the potential dam sites at Axis 2 and Axis 6 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 (AECOM, 2016a).

The Phase III geotechnical investigation focused on filling in data gaps at the selected Axis 2 site, and on exploring two potential rock borrow areas. No further investigations were carried out for the Axis 6 site. The Phase II and III geotechnical investigations performed to characterize the dam foundation are shown on Figure 3-2. The field investigation and laboratory testing included the following:

- **Geologic outcrop mapping**
- **Seismic refraction surveys**
- **Core borings**
- **Water pressure (packer) testing and televiewer/caliper logging in borings**
- **Seismic velocity measurements in selected borings**
- **Laboratory tests on selected core samples.**

5.2 Dam Site

5.2.1 Weathering and Fracturing

Table 5-1 summarizes the depths drilled, rock depths, and depths to slightly weathered to fresh rock in the borings at Axis 2. The depth to rock was generally less than 20 feet in most of the dam site borings. The core boring logs show that the degree of weathering is variable. The borings often encountered significant depths (more than 100 feet) of completely weathered to highly weathered rock, typically weak to very weak, and highly to intensely fractured, with rock quality designation (RQD) values from 0 to 30 percent. With increasing depths, all borings encountered slightly weathered to fresh rock, generally less fractured, and with higher RQDs (frequently 100 percent). In the slightly weathered rock zones, the fracture intensity typically decreased with increasing depth.

Table 5-1. Summary of Core Boring Results – Dam Foundation (Axis 2)

Abutment	Boring No.	Total Drilled Depth* (feet)	Approx. Drilled Depth to Rock* (feet)	Depth to Predominantly Slightly Weathered/ Fresh Rock* (feet)
Left	CB-1	199.7	23	133
	CB-2	178.0	4	4
	CB-10	202.8	7	49
	CB-11	150.4	2	17
	CB-12	100.3	5	32
	CB-17	152.5	5	6
	CB-20	168.3	13	16 to 24

Abutment	Boring No.	Total Drilled Depth* (feet)	Approx. Drilled Depth to Rock* (feet)	Depth to Predominantly Slightly Weathered/ Fresh Rock* (feet)
Right	CB-3	254.2	3	63
	CB-4	154.5	3	109
	CB-13	208.0	11	97 to 107
	CB-14	202.8	45	87
	CB-15**	209.9	9	20
	CB-16**	173.9	15	15
	CB-16A**	42.1	10	10
	CB-18	200.0	9	25 to 30
	CB-19**	117.3	3	17

*** Depths are measured along the length of the angled borings.**

**** CB-15, 16, 16A, and 19 were drilled on the right (northern) side of the river channel at the toe of the right abutment.**

5.2.2 Rock Strength

Unconfined compressive strength (UCS) tests were performed on selected testable core samples. Testable samples are likely to be the better-quality samples, so some degree of bias may be present in the UCS test results toward higher strength rock. Shorter cores were tested by a point-load device in the field (AECOM (2017a).

The UCS data (AECOM, 2017a) are summarized in Table 5-2 by degree of weathering.

Table 5-2. Summary of UCS Tests on Axis 2 Core Samples

Location	Predominant Degree of Weathering	Median UCS (psi)	Range of UCS (psi)	Number of Tests
Left Abutment	Moderately	2,700	N/A	1
	Slightly	9,900	1,800 to 24,100	7
	Fresh	10,950	9,350 to 12,550	2
Channel	Moderately	N/A	N/A	N/A
	Slightly	11,450	5,100 to 21,850	7
	Fresh	12,450	9,630 to 15,280	2
Right Abutment	Moderately	3,850	1,900 to 5,850	2
	Slightly	17,650	6,450 to 35,650	10
	Fresh	N/A	N/A	N/A

The point-load test data (AECOM, 2017a) are summarized in Table 5-3 by degree of weathering.

Table 5-3. Summary of Point Load Tests on Axis 2 Core Samples

Location	Predominant Degree of Weathering	Median UCS (psi)	Range of UCS (psi)	Number of Tests
Left Abutment	Moderately	2,150	950 to 2,200	3
	Slightly	20,250	14,850 to 24,350	8
	Fresh	24,700	24,500 to 32,150	3
Channel	Moderately	N/A	N/A	N/A
	Slightly	19,050	13,800 to 22,850	8
	Fresh	N/A	N/A	N/A
Right Abutment	Moderately	3,800	2,000 to 16,950	8
	Slightly	28,950	14,350 to 39,800	12
	Fresh	N/A	N/A	N/A

Notes:
N/A = not applicable
psi = pounds per square inch
UCS = unconfined compressive strength

As expected, both the UCS and point-load strengths generally increase with decreasing degrees of weathering. The slightly weathered to fresh rock has high strengths, with values up to about 40,000 pounds per square inch (psi). A substantial range of strengths was observed for a given degree of weathering.

5.2.3 Summary of Foundation Conditions

The upper part of the rock foundation at Axis 2 is weathered and fractured, and the rock conditions improve with depth. The degree of fracturing and weathering decreases with depth; generally, hydraulic conductivities also tend to decrease with depth, with the exception of the upper part of the right abutment (refer to Figure 5-1).

The depth of excavation is expected to extend to 100 feet in some locations of the foundation. The discontinuity analysis indicates that the more prominent features observed in borings and borehole televiewer surveys are not likely to persist as discrete, continuous foundation defects.

In the upper end of the proposed left abutment, predominantly slightly weathered rock was encountered at a depth of about 32 feet (approximately Elevation 1,865 feet). RQD values were typically 100 percent below Elevation 1,845 feet. Downhill, the thickness of residual soils and/or highly weathered/fractured rock generally decreases with decreasing elevation (i.e., towards the channel). A similar trend in the thickness of moderately weathered/fractured materials was also observed. In the flat bench area of the left abutment, a highly to completely weathered, intensely fractured, very weak to extremely weak rock was encountered to a depth of approximately 88 feet (Figure 5-1). Predominantly slightly weathered to fresh rock was encountered at a depth of about 133 feet (along the length of the inclined boring). A few hundred feet to the north toward the river channel, predominantly slightly weathered or fresh rock were encountered at depths of only 4 feet and 24 feet, respectively. In the left abutment, the Lugeon values generally decrease with depth and range from 1 to more than 100. Although there are several exceptions, the Lugeon values are generally low (about 1 to 2) in slightly weathered to fresh bedrock in the left abutment.

In the upper part of the right abutment, highly fractured rock conditions observed. RQD values were generally low (<30 percent). The depth to predominantly slightly weathered to fresh rock does not decrease with decreasing elevation in the depth explored in the upper right abutment. Further downhill,

the depth to predominantly slightly weathered rock is about 97 to 107 feet; and about 25 to 30 feet in CB-18 (about 220 feet downstream of Axis 2). Water pressure test data show that the hydraulic conductivities remain as high as 10 to 100 Lugeons, and do not decrease consistently with depth in the borings in the upper right abutment. This is likely due to the high degree of rock fracturing caused by stress relief of the abutment ridge. The potential for seepage through north abutment ridge and resulting shallow colluvial slides on downstream side of ridge will need to be evaluated during preliminary design.

In the valley bottom along Axis 2, along the Bear River, the depth to predominantly slightly weathered rock ranges from about 10 to 15 feet along the borings (about 8 to 10 feet vertical depth). The rock is highly fractured to a depth of 35 feet. The maximum measured hydraulic conductivity was about 10 Lugeons in the valley bottom.

5.3 Rock Material Sources

5.3.1 General

Two rock borrow areas, North and South Borrow Areas, were investigated based on topographic conditions and proximity to the dam site area (Figures 1-2 and 5-2). Both potential rock borrow areas that were investigated are on hills, on the northern side of the Bear River, north of the dam axis. The terrain consists of steep slopes with grass, brush, and scattered tree cover. Numerous basalt outcrops were found in both rock borrow areas.

Subsequent to the geotechnical investigation, a future bridge over the Bear River was proposed near the North Rock Borrow Area. It is planned that the bridge will be constructed prior to rock borrow excavation operations; therefore, this area will be precluded from use. For this reason, only the South Rock Borrow Area is discussed in this section. Another rock borrow source under consideration is the existing Bear River Quarry, about a half-mile south of the dam site.

5.3.2 Potential Borrow Area Rock Conditions

Weathering and Fracturing

The core boring logs show that the degree of weathering is variable. The borings typically encountered significant depths of completely weathered to highly weathered, weak to very weak, and highly to intensely fractured rock, with low RQD values, typically 0 to 20 percent. With increasing depth, the borings encountered slightly weathered to fresh rock, which was generally less fractured and had higher RQDs (frequently 100 percent). Table 5-4 summarizes the borings drilled in the South Rock Borrow Area.

Table 5-4. Summary of Core Boring Results – Potential South Rock Borrow Area

Boring No.	Total Drilled Depth* (feet)	Approx. Drilled Depth to Weathered Rock (feet)	Drilled Depth to Predominantly Slightly Weathered/Fresh Rock (feet)
CB-B1	200.9	8	20
CB-B2	100.0	9	45
CB-B3	101.3	7	45-51
CB-B4	103.5	6	45
CB-B5	79.0	4	60

* Depths are measured along the length of the angled borings.

Rock Strength

Unconfined compressive strengths were evaluated by performing laboratory and field-point load tests. The UCS data are summarized in Table 5-5 for cores selected from the South Rock Borrow Area.

Table 5-5. Summary of UCS Tests on Core Samples - South Rock Borrow Area

Test	Degree of Weathering	Median UCS (psi)	Range of UCS (psi)	Number of Tests
Lab Tests	Fresh to Slightly	10,400	2,200 to 22,200	8
Point Load Tests	Moderately	N/A	17,600	1
	Slightly to Moderately	N/A	24,800	1
	Fresh to Slightly	N/A	19,700	1

Notes:
N/A = not applicable
psi = pounds per square inch
UCS = unconfined compressive strength

The data are insufficient to show a clear correlation between weathering and strength; however, the slightly weathered to fresh rock would have greater strength than the more weathered rock.

Rock Durability

Durability tests were performed on rock core samples from the South Rock Borrow Area to evaluate suitability for RCC and concrete aggregate. These tests consisted of abrasion, sodium sulfate soundness, bulk-specific gravity, and absorption. The test results are summarized in Table 5-6, together with typical concrete aggregate acceptance criteria.

Table 5-6. Summary of Rock Durability Tests on Core Samples - South Rock Borrow Area

Rock Borrow Area	Test Results	Acceptance Criteria*
Abrasion – % weight loss at 100 revolutions	5.3%	10% max.
Abrasion – % weight loss at 500 revolutions	19.4%	40% max.
Sodium Sulfate Soundness –% weight loss at 5 cycles	1.7%	10% max.
Bulk Specific Gravity	2.80	2.60 min.
Absorption %	0.2%	2.0% max.

***American Society for Testing and Materials C 33**

The durability test data indicate that the slightly weathered to fresh basalt would satisfy American Society for Testing and Materials (ASTM) C33 concrete aggregate acceptance criteria.

5.3.3 Estimated Stripping Depths

Stripping will be required to remove soil and weathered rock to expose slightly weathered to fresh rock suitable for RCC and concrete aggregate. Based on the core boring and seismic refraction data, overburden stripping depths to expose suitable rock in the South Rock Borrow Area could range from 20 to 60 feet. These stripping depths will need to be confirmed by further geotechnical investigations.

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6. Description of Dam and Appurtenant Works

6.1 General

The description of Centennial Dam and Reservoir is summarized in Table 6-1. Exhibits showing the conceptual design of the dam and appurtenant works are included in this CER. Exhibit 1 lists the conceptual design exhibits.

Table 6-1. Description of Centennial Dam

Stream	Bear River
Location	Between Combie and Rollins Reservoirs
Purpose	Irrigation (main use), municipal and domestic use
Drainage Area	123 square miles
Reservoir Storage	110,000 acre-feet
Reservoir Pool Elevations:	
Streambed	1,600
At Spillway Crest	1,855
Reservoir Areas:	
At Spillway Crest	1,281 acres
At Top of Dam	1,530 acres
Length of Reservoir	6¼ miles
Dam:	
Type	Roller-compacted concrete (RCC) gravity
Elevation-Top of Dam	1,878
Freeboard-Spillway Crest to Top of Dam	23 feet
Structural Height, foundation to dam crest	285 approx.
Height, d/s toe to dam crest	275
Side Slopes	Upstream: vertical Downstream: 0.8H:1V
Length of Crest	Approx. 1,600 feet
Spillway:	
Type	Uncontrolled ogee-crest overflow bay; stepped downstream face
Crest Length	204 feet total (200-foot hydraulic width)
Energy Dissipater	Stilling basin
Outlet Works:	
Type	10.5-foot-diameter pipe through dam, upstream guard gate & downstream 6.5-foot-diameter sleeve-regulating valve
Location	Single–level intake at bottom of dam at Elevation 1,620 feet
Horizontal Datum	NAD 83 CA State Plane Zone 2
Vertical Datum	NAVD 88

6.2 Design Criteria

The Design Criteria Technical Memorandum (DCTM) documents the criteria that were used for conceptual design of the RCC dam for the Centennial Reservoir Project (AECOM, 2017b). The DCTM will be updated as the design is developed in succeeding work phases; new information becomes available; and decisions are made following discussions with NID and DSOD.

The DCTM defines the basic criteria for the project, including hydraulic, stability, and seismic design, and DSOD criteria, and addresses the following:

- **RCC dam stability criteria**
- **Dam foundation evaluation**
- **Dam material properties for analysis**
- **Design load cases**
- **Seismic design criteria**
- **Hydraulic and hydrologic criteria for spillway design storm and flood**
- **Handling floods during construction**
- **Freeboard**
- **Reservoir evacuation requirements.**

The dam must remain operable following the maximum credible earthquake (MCE) and design flood events. Refer to the DCTM for specifics on the conceptual design criteria.

6.3 Foundation

6.3.1 Dam Foundation Objective and Surface Treatment

The dam site area is underlain by hard to very hard massive greenstone or meta-basalt and meta-volcanic breccia (Section 3). The rock is variably weathered and fractured. All soils and any landslide debris will be removed from the dam foundation down to bedrock.

The foundation objective is to found the RCC dam mainly on slightly weathered to fresh, hard rock (AECOM, 2017b). It is expected that some localized areas of moderately weathered rock will be present in the foundation. In the upper abutments, where the dam will be low, slightly to moderately weathered rock was evaluated to confirm its acceptability to satisfy stability criteria. The estimated foundation surface for conceptual design is shown on Exhibit 2. The dam foundation surface will need to be further defined by additional geotechnical investigations at the dam site. The degree of weathering, along with fracture intensity and strength, were used to confirm the acceptability of the dam foundation levels (Section 7).

The rock characterization at Axis 2 described in the *Phase III Geotechnical Engineering Report* (AECOM, 2017a) was used to establish the configuration of the dam foundation. The rock characterization was also used to assess the strength properties of the dam foundation to confirm that necessary stability criteria are met. The profile of the dam foundation is shown on Exhibit 4A.

Foundation surface treatment will include cleaning for geologic mapping, final foundation cleaning prior to RCC placement, and surface preparation. Surface preparation entails dental excavation to remove soil, highly weathered or crushed material in shear zones and joints, and backfilling the cleaned discontinuities and other open discontinuities with dental concrete. The depth of dental excavation will depend on the width of the foundation feature, but could be about 3 to 6 feet deep. Due to the differential rock weathering, it is expected that the dam foundation surface will be irregular. Levelling concrete will be placed on the foundation to provide a platform to commence RCC placement. Bedding mortar, or facing concrete (on steep rock surfaces), will be placed on the abutments during RCC lift placement.

6.3.2 Foundation Grouting and Drainage

Grouting will be needed to control seepage through the foundation rock. The configuration of the grout curtain was based on the hydraulic conductivity data from the Phase II and Phase III geotechnical investigations, and on guidelines in USACE (1995) and USBR (1976), which relate grout curtain depth to reservoir head. The profile of the grout curtain is shown on Exhibit 4B.

The grout curtain layout will include two grout curtains, 10 feet apart. The grout holes in each curtain would be angled 20 degrees in opposing directions to more effectively intersect the near-vertical rock discontinuities that are common at the site. The grout holes will be fanned at the ends of the dam to control seepage around the dam. The grout curtain will be located along a concrete plinth, anchored into the rock foundation, at the upstream toe of the dam as shown on Exhibits 5 and 6. The plinth will act as a grout cap and will be sealed against the upstream face of the RCC dam with two waterstops (Exhibit 10). This grout curtain location removes grouting activities from the critical path and can be undertaken as the dam is constructed. The use of the grout plinth will be further evaluated during preliminary design.

The grout curtain profile on Exhibit 4A shows the hydraulic conductivities plotted for each boring. Following the USACE and USBR guidelines cited above, the grout curtain extends to more than 50 percent of the reservoir head. It also extends below the depth of about 5 Lugeons in the borings, except for the two right abutment borings (CB-3 and CB-4), where the hydraulic conductivities were measured at about 10 Lugeons (Exhibit 4A). The grout curtain depths, measured perpendicular to the foundation surface, are summarized below:

- **Left end of dam to Station 10+20: grout curtain depth is 65 feet**
- **Station 10+20 to Station 15+20: grout curtain depth is 100 feet**
- **Station 15+20 to Station 18+80: grout curtain depth is 135 feet**
- **Station 18+80 to right end of dam: grout curtain depth is 100 feet**

For conceptual engineering purposes, the maximum grout hole spacing will be 12 feet. Primary (P) holes will be spaced at 24 feet; secondary (S) holes will be split-spaced between the primary holes resulting in a 12-foot hole spacing (S holes will require grouting). Tertiary, quaternary, and higher-order holes will be drilled and grouted to achieve required grout closure criteria. Verification holes drilled between the two curtains will be water pressure–tested to confirm that specified hydraulic conductivity values have been achieved. The details of the grouting program will be the subject of future design engineering phases.

The foundation for the RCC dam may include areas of fractured rock that could require consolidation grouting. The purpose of consolidation grouting is to strengthen the rock mass and increase the stiffness of the foundation. Improvement of the foundation as a result of the consolidation grouting will not be considered in stability analyses. For conceptual design, the consolidation grouting would be done on a 10-foot by 10–foot grid, and the holes would be about 30 feet deep.

Drain holes to control uplift pressures beneath the RCC dam will also be required. The conceptual design includes 3-inch vertical drain holes drilled from a gallery (Section 6.4.2) within the dam, spaced on 10-foot centers (Exhibits 5 and 6). The depth of the drain holes, taken as two-thirds of the grout curtain depth, was based on the geotechnical investigation data, and on guidelines in USACE (1995) and USBR (1976), which relate drain hole depth to grout curtain depth. The hole depths range from 90 feet deep at the bottom of the valley to 45 feet on the left side of the dam and 65 feet on the right side of the dam. The drain holes will be accessible from the gallery for clean-out (by drain-hole reaming).

6.4 Conceptual Layout of Dam and Spillway

6.4.1 Dam and Spillway

The dam will have a maximum structural height of approximately 285 feet. Its 25-foot-wide, 1,600-foot-long crest will be at Elevation 1,878 feet. The estimated RCC dam volume is about 800,000 cubic yards. The conceptual plan of the RCC dam is shown on Exhibit 3, and the profile along the dam axis is shown on Exhibit 4B. The cross-section of the dam will have a vertical upstream face and a 0.8H:1V stepped downstream face (Exhibits 5 and 6). Cross-sections of the dam at 100-foot intervals are shown on Exhibits 7 and 8.

The RCC will be mixed in an on-site batch plant, transported to the dam with a conveyor system, placed in 12-inch-thick lifts, and compacted with 10-ton smooth drum vibratory rollers. For conceptual engineering purposes, the cement and fly ash content were estimated to be 150 pounds per cubic yard (lbs/cy) each, for 300 lbs/cy total cementitious material, to achieve an unconfined compressive strength of 2,500 psi at 1 year. Bedding mortar to provide for improved water-tightness and bond between RCC lifts will be placed as shown on Exhibit 10. The extent of the bedding mortar placement will be evaluated during preliminary design. On the upstream and downstream sides of the dam, the RCC will be faced with conventional concrete or grout-enriched RCC (GE-RCC) placed at the same time as the RCC (Exhibits 5, 6 and 10). The spillway chute will have a reinforced conventional concrete facing (Exhibits 5 and 10). These two facing types have been successfully used on many RCC dam projects.

Rock for RCC aggregate may be obtained from on-site rock borrow areas (see Section 5.3). An alternative source of material may be available at the existing Bear River Quarry, located within 0.5 mile south of the dam site (see Section 9). The RCC aggregate gradation could have a maximum size of about 1 to 1.5 inches and 4 to 10 percent silty fines. Cement (Type II/V, low alkali) and Class F fly ash will need to be imported to the RCC batch plant. The amount of cement and fly ash will be based on achieving the specified RCC strength and temperature control requirements of the RCC mix. The cement and fly ash will conform to specified ASTM standards, which will be certified by the manufacturer (through the supplier), and verified with supplied test data.

The PMF water surface will be at the top level of the RCC. A reinforced-concrete parapet wall on the upstream side of the dam crest will not impound water, but will be used to satisfy wave run-up and residual freeboard criteria. The reinforced-concrete parapet wall will be structurally tied to the concrete slab on the dam crest and anchored into the RCC (Exhibits 6 and 10).

The RCC dam will include a spillway integral with the body of the dam, aligned to discharge flows directly into the Bear River channel (Exhibit 3). The 204-foot-wide spillway bay with a crest at Elevation 1,855 feet was based on maximizing the available discharge width that can approximately match the river channel width immediately downstream. The spillway crest allows for inclusion of a 4-foot bridge pier, thereby providing for a 200-foot effective (hydraulic width) spillway crest. The pier will be hydraulically shaped to minimize head loss. The bridge across the spillway bay will be designed to allow for truck traffic (HS-20). The bridge will consist of two concrete box-girder spans resting on the pier (Exhibits 11 and 12).

A stilling basin will be situated at the toe of the spillway to dissipate energy prior to releasing flows back to the river channel (Exhibit 5). Reinforced-concrete training walls will be constructed on each side of the spillway bay and stilling basin to contain the discharge flows. The stilling basin slab will be anchored into the rock foundation.

6.4.2 Joint, Drain, and Gallery Details

The basic joint, drain, and gallery details are described below:

- Joints – Contraction joints will be formed within the RCC by "crack initiators" that extend transversely from the upstream dam face (Exhibit 10). Contraction joints typically will be located not more than 80 feet apart and at sharp breaks in the foundation topography.
- Contraction joint drains and RCC body drains near the upstream side of the dam will control potential seepage along lift lines and minimize uplift.
    - Contraction joint drains – Six-inch-diameter formed contraction-joint drains will be centered at the contraction joints in the upstream conventional concrete facing, and discharge into the gallery (Exhibits 4B and 10). Waterstops will be located just upstream of the contraction joint drains.
    - RCC Body drains – Three-inch-diameter body drains spaced on 10-foot centers will be drilled into the RCC from the dam crest and intersect the gallery (Exhibits 5 and 6).
- Gallery – A 6-foot-wide by 9-foot-high drain galley will be formed in the RCC dam to collect seepage water from the foundation drains, contraction joint drains, and body drains. The gallery will also be used to drill additional drain holes and to maintain them (Section 6.3.2), and to install additional foundation piezometers, if needed. The profile of the gallery follows the foundation surface, as shown on Exhibit 4B. For conceptual design, the criterion for the gallery invert elevation was selected to be above the PMF tailwater elevation.

6.5 Outlet Conduit

The conduit will include a single low-level intake, located near the base of the dam, in accordance with NID's requirements. The upstream end of the conduit will be at Elevation 1,620 feet (which is above the river bed at about Elevation 1,600 feet) to allow for some sediment accumulation. At the outlet, the conduit could be configured with a bifurcation and a blind flange for potential future addition of a power plant at the downstream toe of the dam (Exhibit 9).

DSOD "guidelines" for emergency drawdown rate for large reservoirs (AECOM, 2017b) are that outlet facilities be able to:

- Lower the reservoir elevation by an amount equal to 10 percent of the hydraulic head behind the dam in 7 days. Hydraulic head is defined as the elevation difference between the normal maximum water surface and the upstream toe elevation.
- Evacuate the reservoir to deadpool elevation within 90 days.

To meet the above criteria, the conceptual design includes a 10.5-foot-diameter low-level steel-lined outlet conduit cast into the body of the dam. Flows will be controlled by a 6.5-foot sleeve valve at the downstream end of the pipe. A slide gate at the upstream end of the conduit will be closed for conduit inspections. The maximum flow velocity in the pipe will be about 24 feet per second at full pool.

The reservoir elevation at the top 10 percent of the head is at 1,829.5 feet. The conduit can lower the reservoir to this elevation in 6.7 days, thereby satisfying the first requirement to lower the reservoir to this level in 7 days. The reservoir can be evacuated to the inlet Elevation 1,620 feet in about 35 days, satisfying the second requirement. The first requirement is the controlling reservoir evacuation requirement.

6.6 Instrumentation

Performance monitoring instrumentation will be included in the design of the dam. At a minimum, instrumentation will be included to monitor reservoir level, uplift, seepage, crest movement, and earthquake accelerations, as shown on Exhibit 13 and described below:

- **Reservoir level – A reservoir staff gauge, mounted on the upstream side of the dam, and an electronic sensor will be used to record reservoir levels.**
- **Uplift – Uplift pressures at the base of the dam will be recorded by vibrating wire piezometers installed in holes drilled from the gallery half way between drain holes at the foundation/RCC interface; and 30 feet below the base of the dam. In addition, vibrating wire piezometers will be installed in holes drilled from the downstream toe. The piezometers will be located at the maximum section, and at three additional sections (four sections total) (Exhibit 13). Two vibrating wire piezometers will be installed at each location as a back-up in case one fails.**
- **Seepage – V-notch weirs to measure seepage will be in the gallery gutters: one each for each abutment.**
- **Crest movement – Survey monuments on the parapet wall will be used to monitor horizontal and vertical crest movement. The monuments will be spaced at 100-foot intervals along the wall.**
- **Accelerographs – Accelerographs will be installed to monitor earthquake shaking. They will be installed (a) on the dam crest at the maximum section; (b) on rock at the downstream toe near the maximum section (two locations); and (c) on rock on both abutments, beyond the end of the dam. The right abutment accelerograph will monitor the effects that the right abutment ridge would have on accelerations.**

Data from the reservoir-level sensor, piezometers, and V-notch weirs will be transmitted by radio from the remote terminal units to the Automated Data Acquisition System (ADAS) central recording hub, where the data will be logged and processed according to programing. Strong motion recordings from the accelerographs can be downloaded using direct cable connection, or uploaded over cell or radio networks independent of the ADAS. The piezometer data can also be recorded manually at read-out terminals. V-notch weir and reservoir-level data can be visually recorded from staff gauges.

During construction, thermocouples will be used to record data on the RCC temperature to confirm heat rise is within design expectations.

6.7 Diversion

With the RCC dam, flood flows over the dam during construction would not pose a dam safety issue due to a breaching failure, because the RCC would not be significantly erodible. The risk of controlling potential flood damage to the construction site will be the responsibility of the contractor. NID can operate Rollins Reservoir to limit outflows to 150 cubic feet per second (cfs), and thereby reduce flood damage at the Centennial Dam site.

Diversion of river flow through the dam site could be accomplished by a diversion structure (e.g., a box culvert) constructed in the river channel, or through a tunnel excavated in an abutment. The contractor will be responsible for the design of the river diversion system. When diversion is no longer necessary, the culvert or tunnel would be plugged, or converted into a permanent auxiliary outlet. The RCC dam would be placed on top of the culvert, if selected. A cofferdam will be needed to divert river flow through the culvert or tunnel.

The concept for the Bear River diversion and flow management during construction is illustrated on Exhibit 14. This exhibit shows the profile of the RCC dam along the axis of the dam.

The concept involves construction of a temporary reinforced-concrete box culvert aligned across the dam footprint, and through which river flow will be diverted for the duration of construction. The culvert will be capped, and encased in mass concrete. The mass concrete will be constructed sequentially as described below. The goal of this concept is to complete construction of the culvert and both portions of mass concrete in the summer of Construction Year 1, when river flow can be controlled to not exceed 150 cfs. If flood flows occur during the following winter that cannot be fully routed through the culvert, water will back up, and safely flow over the mass concrete. An overtopping occurrence can be anticipated and accommodated without incurring much damage or delay to construction.

The conceptualized sequence of diversion shown on Exhibit 14 would be as follows:

- **Step 1 – A cofferdam is constructed by dumping fill onto the river channel, starting from an abutment and moving towards the other abutment. Preparation work for this operation would include excavation of the river channel boulders, gravel, and alluvial soils down to bedrock. The cofferdam fill will constrain river channel flows to a narrow corridor running up against the abutment so that construction can proceed along the opposite abutment. Turbidly control measures such as turbidity curtains would be implemented in the river downstream of the cofferdam.**
- **Step 2 – A trench will be excavated in which a box culvert will be constructed. A heavy sheetpile wall may be installed in the cofferdam between the re-routed river and excavation to cut off seepage flows into the excavation. Sump pumps at the toe of the excavation would intercept seepage that bypasses the cutoff.**
- **Step 3 –A reinforced-concrete box culvert is constructed.**
- **Step 4 – A reinforced-concrete slab is constructed as a roof to the culvert. Mass concrete is then placed up to a predetermined elevation, and will become the right side of the base of the dam. An outlet conduit is formed in the mass concrete, with a temporary bulkhead at its upstream end.**
- **Step 5 – A cofferdam is constructed upstream to divert the river to flow through the box culvert. The remaining dam foundation adjacent to the culvert is prepared by excavating the cofferdam, channel gravel, and unsuitable surficial bedrock.**
- **Step 6 – Mass concrete is then placed up to an elevation level with the right side containing the culvert.**
- **Step 7 – RCC/facing-concrete placement begins from the level surface of the mass concrete and continues until the dam is topped out. Once all mechanical/electrical/concrete work is completed for the inlet/outlet and spillway stilling basin, the box culvert will be closed off with a prefabricated bulkhead form and backfilled with mass concrete. River flows will be transferred to the permanent outlet, and the dam can begin to impound water.**

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7. Stability Analyses

7.1 Analysis Approach

The analyses were undertaken to demonstrate that the section configurations of the dam spillway and abutments meet minimum acceptable criteria for normal, flooding, and earthquake loading (AECOM, 2017b). The maximum sections of the spillway and abutment monoliths were evaluated for moment equilibrium, sliding stability, and overstressing using 2-dimensional, limit-equilibrium analyses. The method uses basic-limit equilibrium equations to resolve the forces and moments acting on the structure, and assumes that the normal stresses along any horizontal plane are linearly distributed.

The foundation strengths used for sliding stability analyses are presented in Appendix B. The stability analysis methodology and results are presented in Appendix C, and are summarized in the sections below.

7.2 Loads

The stability analyses considered the following loads: weight of RCC and concrete, reservoir water, tailwater, uplift, and seismic (inertial and hydrodynamic). Sediment loading against the dam was not considered as the upstream Rollins Reservoir would prevent most of the sediment from entering Centennial Reservoir. Ice loading in the reservoir also was not considered, because sustained freezing temperatures are not expected at the dam site. No reduction of uplift pressures was taken across the grout curtain used in the dam stability analyses (AECOM, 2017b).

7.3 Stability Criteria

The minimum allowable factors-of-safety for moment equilibrium and sliding stability follows USACE EM 1110-2-2100 (2005) guidelines for sliding, and resultant location for critical/high-hazard structures. The factors-of-safety and stability criteria are summarized in Table 7-1.

Table 7-1. Stability Criteria

Load Case	Comment	Reservoir Water Surface Elev.	Location of Resultant at Base	Factor-of-Safety for Sliding (1)
Usual	1:10 AEP	At spillway crest(3)	100% of Base in Compression	2.0
Unusual	1:300 AEP	At 1:300 flood level	75% of Base in Compression	1.5
Unusual	Drains inoperable	At spillway crest	75% of Base in Compression	1.5
Extreme	PMF	At PMF level	Resultant Within Base	1.1
Extreme	MCE (2)	At spillway crest	Resultant Within Base	1.1

Notes:
[1] Site information definition in USACE EM 1110-2-2100, Section 3-4. For a new dam, "Ordinary" Category applies.
[2] See USACE EM 1110-2-2100, subsection 3.11 b.
[3] For the Usual Load Case, the reservoir level will be taken at the spillway crest instead of the 1:10 AEP flood level.
AEP = annual exceedance probability
MCE = Maximum credible earthquake
PMF = Probable maximum flood

The 1:300 AEP flood is discussed in Appendix D.

7.4 Foundation Shear Strength

The sliding stability of spillway and abutment monoliths is controlled by the sliding potential on either a continuous or semi continuous sub-horizontal weak plane in the foundation, or through the rock mass itself.

Sliding along the foundation interface is resisted by the irregular profile of the prepared foundation surface that will force a potential sliding failure surface through the weaker of the two interface materials. The dam/foundation interface will be inspected during construction, and is not typically a critical failure scenario.

The foundation geologic investigations (AECOM, 2017a) found no evidence of persistent sub-horizontal weak planes in the foundation that would constitute a prescribed failure surface. On this basis, sliding resistance of the dams will be provided by the shearing through the jointed rock mass immediately below the base of the dam. Shearing strength will vary along the dam axis and with depth, reflecting the variability in rock conditions across the site.

Estimates for in situ shear strength of the jointed rock mass across the site were made using the Hoek-Brown criterion (Hoek and Corkum, 2002; Hoek et al., 1997) and expressed as equivalent Mohr Coulomb failure parameters. The basis for and recommended foundation strengths are presented in Appendix B and summarized in Table 7-2.

Table 7-2. Foundation Strength Parameters for Stability Analyses

Analysis Section	Degree of Weathering for Strength Estimates	Cohesion (psi)	Friction Angle (degrees)
15+50	Slightly Weathered to Fresh	100	60
17+00	Slightly Weathered	100	57
20+50	Moderately	40	55

Sensitivity analyses were also performed using reduced cohesion values (see Appendix C).

7.5 RCC Strength

For conceptual design purposes, the unconfined compressive strength of the RCC was assumed at 2,500 psi. This strength value has been found for several RCC dams (Hansen and Reinhardt, 1991). A unit weight of 150 pounds per cubic foot was used based on data from similar RCC dams (Hansen and Reinhardt, 1991).

7.6 Static Stability Results

The static stability analysis results indicate that the maximum section of the spillway and non-overflow sections of the dam satisfy overturning stability and sliding criteria for all normal and flooding configurations, up to and including the PMF. The resultant remains within the middle third, even under the PMF loading, indicating that the base of the dam remains in compression at all times. The body of the dam also remains in compression, and cracking along the lift lines is not predicted.

Summary of the stability results under the various static loading condition listed in Table 7-1 are presented in Table 7-3 (non-overflow), Table 7-4 (spillway) and Table 7-5 (representative non-overflow).

Table 7-3. Stability Results – Non-overflow Section, Station 15+50

Load Case	Comment	Reservoir Water Surface Elev.	Location of Resultant (percentage of base from toe)	Factor of Safety for Sliding
Usual	1:10 AEP	Spillway crest (El. 1855)	50.7 (> 33.3)	3.67
Unusual	1:300 AEP	1:300 flood level (El. 1873)	44.8 (> 12.5)	3.18
Unusual	Drains inoperable	Spillway crest, (El. 1855)	45.3 (> 12.5)	3.12
Extreme	PMF	PMF level, (El. 1877)	43.2 (> 0)	3.07

Table 7-4. Stability Results – Spillway Section, Station 17+00

Load Case	Comment	Reservoir Water Surface Elev.	Location of Resultant (percentage of base from toe)	Factor of Safety for Sliding
Usual	1:10 AEP	Spillway crest (El. 1855)	45.1 (> 33.3)	4.00
Unusual	1:300 AEP	1:300 flood level (El. 1873)	40.1 (> 12.5)	3.44
Unusual	Drains inoperable	Spillway crest, (El. 1855)	39.2 (> 12.5)	3.10
Extreme	PMF	PMF level, (El. 1877)	38.8 (> 0)	3.35

Table 7-5. Stability Results – Non-overflow Section, Station 20+50

Load Case	Comment	Reservoir Water Surface Elev.	Location of Resultant (percentage of base from toe)	Factor of Safety for Sliding
Usual	1:10 AEP	Spillway crest (El. 1855)	55.4 (> 33.3)	4.20
Unusual	1:300 AEP	1:300 flood level (El. 1873)	44.9 (> 12.5)	3.11
Unusual	Drains inoperable	Spillway crest, (El. 1855)	54.2 (> 12.5)	3.80
Extreme	PMF	PMF level, (El. 1877)	41.9 (> 0)	2.92

The section at Station 15+50 is at a localized downstream sloping foundation surface. Adjustments in the dam axis alignment and/or foundation shaping to minimize the localized downstream sloping foundation condition will need to be considered during preliminary design.

7.7 Dynamic/Earthquake Analyses

7.7.1 Analysis Approach

For concept design, a pseudo-dynamic analysis of the MCE loading of the spillway section at Station 17+00 was performed using the simplified method developed by Fenves and Chopra (1986). This method considers the effects of interaction between the dam, foundation, and impounded water, of water compressibility, and on the fundamental mode of vibration of the dam. Also, the Fenves and Chopra result was compared with the results from a pseudo-static analysis using hydrodynamic forces based on Westergaard's generalized theory for added mass (1933) (see AECOM, 2017b).

MCE load case was analyzed with reservoir at the spillway crest (Elevation 1,855 feet), and with no tailwater. The design MCE earthquake is the deterministic 69th percentile response spectra presented

in Section 4.4.2. The PGA for this earthquake is 0.31g, with a peak spectral acceleration (SAmax) of 0.74g at a period (T) of 0.15 second.

7.7.2 Dynamic Analysis Results

The results of the seismic analysis for the spillway section are presented in Appendix C. Resultant location and sliding factors of safety (FS) corresponding to the equivalent strength parameters for the foundation rock strength in Appendix B are shown in Table 7-6.

Table 7-6. Pseudo-Static Stability Results based on Fenves and Chopra (1986), Extreme (MCE)

Station	Comment	Reservoir Water Surface Elev.	Location of Resultant (percentage of base from toe)	Factor of Safety for Sliding
17+00	Fenves and Chopra	Spillway crest (El. 1855)	-18.1 (≵ 0)	1.49 (> 1.1)
17+00	Westergaard	Spillway crest (El. 1855)	-2.7 (≵ 0)	1.42 (> 1.1)
15+50	Westergaard	Spillway crest (El. 1855)	-15.5 (≵ 0)	1.32 (> 1.1)

In both analysis cases, sliding FS exceeds the minimum criteria of 1.1 for Extreme loading (Table 7-1). However, the resultant location for both cases is outside of the base by a small margin. Although this exceeds the criteria for the resultant to remain within the base, it is not considered to be an issue for the design. This conclusion is based on:

- **Any rocking of the section that might result will increase the period of the first mode thereby decreasing the inertia loading of the section (in other words, it is self-stabilizing), and**
- **The amount that the resultant is outside of the base is sufficiently small that any cracking/separation of the dam from the foundation will be of limited width and duration such that reservoir water pressure would not develop within the crack and result in increasing destabilizing loads on the dam section.**

A finite element model will be developed for the dynamic analyses under MCE loading during preliminary design to better define seismic performance of the dam for the seismic loading condition.

8. Flood Routing and Hydraulic Analysis

8.1 PMF Routing Approach

Based on the DSOD guidelines for dams in the high hazard classification, the design flood for Centennial Reservoir is the PMF. The PMF is derived from the Probable Maximum Precipitation (PMP).

The PMP was calculated using Hydrometeorological Reports 58 and 59 (NOAA, 1999). The calculated 72-hour cumulative precipitation for the 123-square-mile watershed varies from 30 inches near Centennial Reservoir to 43.5 inches in the watersheds above Rollins Reservoir. The PMF was calculated using the USACE HEC-HMS rainfall runoff model. The calculated PMF inflow to the proposed reservoir is 89,181 cfs. Variously sized spillways were evaluated from 150 to 220 feet of effective width.

The PMF analysis of inflow, routing, and results are presented in Appendix E, and are summarized in the sections that follow.

8.2 PMF Results

Inflow was routed through Rollins Reservoir using the stage-storage and storage-discharge curves for the reservoir and spillway (Appendix E). Rollins Reservoir was assumed to be full to its spillway crest for flood routing to the Centennial Dam site. The outflow from Rollins Reservoir was routed downstream to the proposed Centennial Reservoir, and combined with the additional runoff from the watershed between Rollins and Centennial Reservoirs.

The calculated peak PMF inflow to Rollins Reservoir is approximately 80,888 cfs. The maximum routed outflow from Rollins Reservoir was 78,700 cfs. The inflow hydrograph to Centennial Reservoir is shown in Appendix E. The peak inflow to Centennial Reservoir is 89,181 cfs.

8.3 Outflow Hydrograph Results

Using the stage-storage curve for Centennial Reservoir and the spillway rating curve (Appendix E), the PMF inflow was routed using HEC-HMS to generate the PMF outflow (USACE, 2010). For a reservoir storage capacity of 110,000 acre-feet, the maximum normal reservoir water surface (i.e., spillway crest) would be Elevation 1,855 feet, which is the starting elevation for spillway flood routing.

The peak outflow is dependent on the assumptions used in the spillway design. The spillway could range from 150 feet wide to 200 feet wide (these are hydraulic widths, and do not include bridge pier). Maximum routed PMF outflows and maximum PMF reservoir water surface elevations for these two spillway widths are summarized in Table 8-1.

Table 8-1. Results of PMF Spillway Routing

Spillway Hydraulic Width (ft)	Max. Routed PMF Outflow (cfs)	Max. PMF Reservoir W.S. Elev. (feet)
200 max	81,700	1,877.3
150 min	79,100	1,881.3

Note: cfs = cubic feet per second

Comparing the routed outflow through the spillway and PMF inflow, little flood peak attenuation would occur.

The final spillway width will be based on considerations of the outlet works arrangement and diversion within the Bear River channel topography. For conceptual design, the spillway hydraulic width was assumed to be 200 feet.

9. Construction Considerations

9.1 Conceptual Construction Site Layout

A conceptual site layout for RCC dam construction is shown on Exhibit 15 (Concept Plan 1) if an on-site rock borrow area is used; and on Exhibit 16 (Concept Plan 2) if the Bear River Quarry south of the dam site is used. These exhibits show the assumed rock borrow areas, RCC and conventional concrete batch plant areas, disposal sites, and staging areas. The selection of the rock source to produce RCC aggregate will be based on cost and environmental considerations. The contractor will select its own construction site layout and plan.

Exhibits 15 and 16 show conceptual locations for the main construction site features. These include the rock borrow area, aggregate crushing and screening plant, disposal sites, RCC batch plant, conventional concrete batch plant, and the staging area. The staging area would contain the contractor and construction management offices, site geotechnical and RCC/concrete laboratory, fuel depot, and equipment laydown and storage areas. The conceptual locations of the site features were developed based on access and proximity to the dam sites, and utilization of relatively flat topographical areas.

Conceptual layouts of the main on-site access routes are also shown on Exhibits 15 and 16. Two-lane all-weather road access will be needed from the aggregate crushing and screening plant area to the RCC batch plant site on the right abutment of the dam. This could consist of a haul road along the northern rim of the river canyon. Access from the rock borrow area to Disposal Site 1 would necessitate a temporary bridge or culvert crossing over the Bear River to the disposal site, similar to the existing crossing providing access to the Bear River Quarry downstream.

9.2 Dam Site Preparation and Foundation Excavation

The dam foundation area will be cleared and grubbed. All vegetation, including trees, will be removed and stumps grubbed. Standard Best Management Practices such as sedimentation ponds, straw wattles, and silt fences would be used to control sediment from entering the Bear River during construction. After clearing, the site will be excavated to the foundation level using bulldozers, front-end loaders, and excavators to load transport trucks that will haul the materials to disposal sites. Drilling and blasting are also expected to remove hard rock knobs. Approximately 600,000 cubic yards of materials will need to be excavated to reach foundation grade.

Groundwater seepage areas would be controlled by routing the water to sumps. Pumps would be used to convey the water from the sumps to sedimentation tanks, where it will be tested to confirm that all state and local water quality standards are met before releasing it to the Bear River.

9.3 Rock Borrow Materials

Clearing and grubbing operations for the rock borrow area would be the same as described above for the dam foundation. The rock borrow area would also need to be stripped of overburden and weathered rock. The underlying fresh rock would be drilled, blasted, crushed, screened, and washed to produce the RCC aggregate. The rock borrow area would be excavated to form slopes as steep as 0.25H to 1V, with 10-foot-wide benches at 25-foot to 50-foot vertical intervals. Overburden and rock waste material would be hauled by trucks to nearby on-site disposal sites, also shown on Exhibits 15 and 16. The useable rock would be truck-hauled to a processing plant adjacent to the borrow area and within the project area to produce aggregate for the estimated 800,000 cubic yards for the RCC dam. The stockpiled aggregate would then be hauled by truck to the RCC batch plant near the dam site.

Rock slopes would be scaled to remove loose rock as the excavation proceeds. The floor of the borrow area would be sloped to drain at about a 2 percent grade. At the completion of rock borrow operations, restoration of the rock borrow area would entail spreading rock on the borrow area floor. Drainage ditches would also be constructed, and would remain on site for the long term. The rock slopes of the rock borrow area will remain in their excavated condition, and no post-excavation work is envisioned.

An alternative source of rock material is available at the existing Bear River Quarry, south of the dam site (Exhibit 16). This material would also require quarrying and processing (e.g., crushing and mixing) to produce suitable aggregate. In this case, it is expected that the rock materials would be crushed and screened at the quarry to produce RCC aggregate. The aggregate would be hauled to the RCC batch plant on the southern abutment of the dam via a new haul road running northward from the quarry.

9.4 Imported Materials

Portland cement and fly ash would be trucked to site in bulk cement carriers from supply depots in Sacramento (approximately 100 miles round trip). The cement would be transported to Sacramento by rail from either Redding or Southern California. Fly ash would be sourced from either Wyoming or Arizona, and would be transported to Sacramento by rail. For the conceptual RCC dam layout, approximately 60,000 tons of cement and 60,000 tons of fly ash would be imported to produce the required volume of RCC. The cement and fly ash would be trucked in bulk-cement carriers to silos at the RCC batch plant.

9.5 Water

Water for RCC, concrete-batched on site, aggregate processing, dust control, soil compaction, and for other miscellaneous needs would be pumped from the Bear River to holding tanks.

9.6 Spoils Disposal

Exhibits 15 and 16 portray potential on-site disposal locations. Approximately 2.5 million cubic yards of excavated materials from the dam foundation, rock borrow area stripping, rock quarrying and processing waste, site development, and new road/bridge work would need to be wasted in the disposal sites (based on bulked volume in-place in the disposal sites). A summary of the disposal sites is presented in Table 9-1.

Table 9-1. Summary of Excess Material Disposal Sites

Disposal Site No.	Capacity (cy)	Top Area (acres)	Approx. Average Thickness (feet)
1	2.0 million	30	40
2	1.0 million	22	30

Storm runoff will be diverted around the disposal sites via rock-lined ditches. Erosion protection consisting of rock materials raked out by bulldozers to the exterior slopes of the disposal fills would be used for erosion protection during construction. Also, erosion control fabrics would be used to mitigate erosion.

10. Operation, Maintenance, and Inspection

10.1 General

Operation, maintenance, and inspection of the dam and appurtenant structures will include the dam, spillway, and outlet works, and maintenance of the drain system. Inspection and monitoring results will need to be regularly collected and evaluated, and forwarded to DSOD annually. A Standard Operating Procedures (SOP) Manual (which includes operation, maintenance, and inspection), Emergency Action Plan, and Initial Reservoir Fill Plan would need to be prepared and submitted to DSOD during the final design phase.

Operation, maintenance, and inspection of the dam and appurtenant structures will include the following activities:

- **Dam safety:**
    - **Overall dam safety management program**
    - **Scheduled (routine) inspections using a "Dam Safety Inspection Checklist":**
        - **First 2 years after reservoir is full for the first time: Weekly**
        - **After the first 2 years: Monthly**
    - **Special inspections following extraordinary events (earthquakes, major spillway operation, or abnormal seepage)**
    - **Maintenance of instrumentation system**
- **Dam and spillway:**
    - **Debris removal from the upstream side of the dam and spillway**
- **Outlet works:**
    - **Emergency power supply maintenance**
    - **Regular valve maintenance and exercise, and demonstration of release facilities operability**
- **Drainage system maintenance**
- **Access road maintenance**
- **Maintenance of environmental mitigation, including repair of erosion areas and removal of vegetation**

10.2 Instrumentation Data Recording

The performance instrumentation data will be recorded at regular intervals in accordance with an SOP plan. Instruments connected to the ADAS (Section 6.6) can be polled at any time to check performance. Table 10-1 shows the instrumentation data recording frequency.

Table 10-1. Instrumentation Data Recording Frequency

Instrumentation Type	Purpose	Frequency of Data Recording
Reservoir level sensor*	Reservoir levels	Daily or weekly
Vibrating wire piezometers*	Uplift pressures	Weekly for the first 2 years after reservoir is full for the first time; monthly thereafter
V-notch weirs*	Seepage rates	Weekly for the first 2 years after reservoir is full for the first time; monthly thereafter
Survey monuments	Horiz. and vert. crest movement	Quarterly for the first 2 years after reservoir is full for the first time; annually thereafter
Accelerographs	Earthquake acceleration	Triggered by earthquake above a threshold acceleration level (usually 0.05g)

***Connected to ADAS**

11. Conclusions and Recommendations

This Conceptual Engineering Report discussed the following topics:

- **Alternatives analysis and rationale for selection of preferred dam site and dam type**
- **Regional and dam site geology**
- **Seismic source characterization and ground motion parameters**
- **Foundation conditions and construction materials**
- **Dam and appurtenant works, including foundation treatment, dam and spillway, outlet works, construction materials, dam performance instrumentation, and river diversion**
- **Results of the stability analyses**
- **Results of the design flood routing through the reservoir**
- **Main construction considerations**
- **Key operation, maintenance and inspection items.**

The conceptual design of the RCC dam is based on the USACE and USBR guidelines, and satisfies DSOD criteria for high-hazard dams to handle the design flood, reservoir evacuation, and stability. The conceptual design described in this CER will be updated as the design is developed in succeeding work phases; new information becomes available; and decisions are made following discussions with NID and DSOD.

Subsurface geotechnical investigations to date have been carried out in three phases, in 2015 (Phases I and II) and 2016 (Phase III). A Phase IV geotechnical investigation is recommended to obtain additional data needed to develop the project design and reduce uncertainty in foundation and rock borrow material conditions that would be suitable for preliminary engineering (AECOM, 2017a). The dam foundation surface will need to be further defined by additional geotechnical investigations at the dam site.

As for the previous phases, the Phase IV investigation in the dam foundation should include seismic refraction surveys, core borings, water pressure (packer) testing, and televiewer/caliper logging to fill in data gaps. Laboratory testing should include strength of rock foundation materials. In addition, in situ testing of the rock mass strength and stiffness (using a downhole dilatometer device) is recommended in fractured zones of selected borings, to help establish and verify the minimum excavation depths.

If the on-site rock borrow is considered further, additional investigation would be needed to better characterize the subsurface conditions and to locate the rock excavations to minimize stripping volume. Borrow investigations should be carried out to confirm the depth to useable rock materials. The investigation should assess the amount of overburden that would need to be stripped and wasted. These investigations should include additional seismic refraction surveys, core borings, and laboratory testing. The testing should include strength of the rock materials, abrasion resistance, soundness, and bulk-specific gravity. Petrographic examination of the rock should also be performed to verify the minerology and the absence of potentially deleterious constituents. The potential for alkali reactivity of the rock to be used for RCC aggregate should be evaluated. Use of the Bear River Quarry south of the dam site needs to be determined based on availability and cost considerations.

Based on the recommended Phase IV investigations discussed above, preliminary engineering should be performed to advance the project design. Preliminary engineering should include the following activities:

1. **Confirming the dam axis alignment and/or foundation shaping to minimize the localized downstream sloping foundation just south of the spillway;**
2. **Confirming the optimum spillway crest length to fit within the Bear River channel topography;**
3. **Evaluating potential for seepage through north abutment ridge and mitigation of resulting shallow colluvial slides on downstream side of ridge, such as by using a drainage system;**
4. **Evaluating potential for reactivation of slides when reservoir is filled and potential mitigation measures if needed;**
5. **Developing the outlet works arrangement;**
6. **Developing the river diversion approach;**
7. **Routing the design flood through the spillway to determine the stilling basin size and height of the training walls in the chute;**
8. **Performing finite element stress and stability analyses of the dam, particularly to assess the seismic performance of the dam;**
9. **Developing the details for design of the dam, spillway, outlet works, and mechanical equipment; and**
10. **Updating construction sequencing, schedule and cost estimate to reflect the preliminary design of the project.**

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Figures

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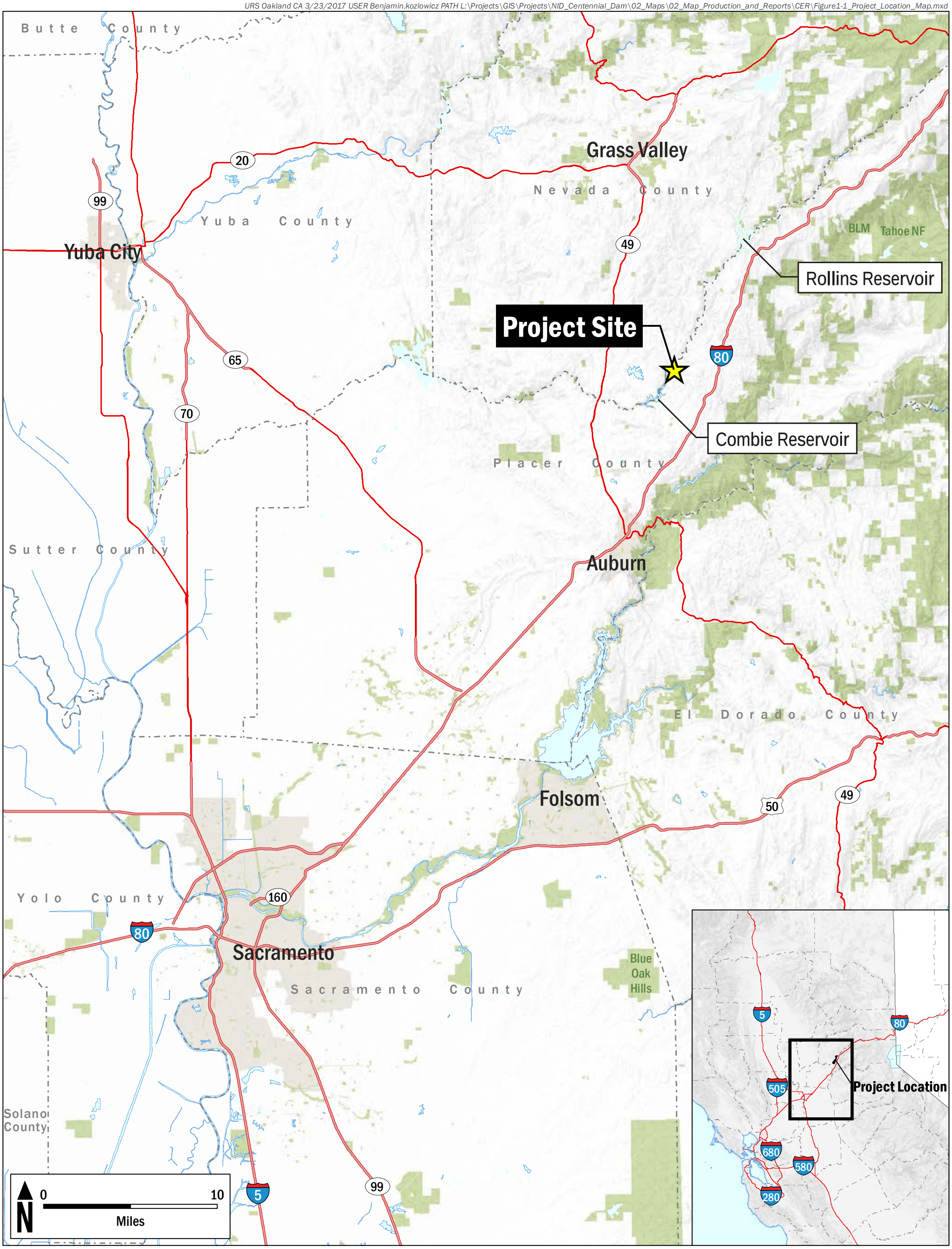
URS Oakland CA 3/23/2017 USER Benjamin.kozlowicz PATH L:\Projects\GIS\Projects\NID_Centennial_Dam\02_Maps\02_Map_Production_and_Reports\CER\Figure1-1_Project_Location_Map.mxd



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Centennial Reservoir Project

60519386

FIGURE 1-1

Project Location Map

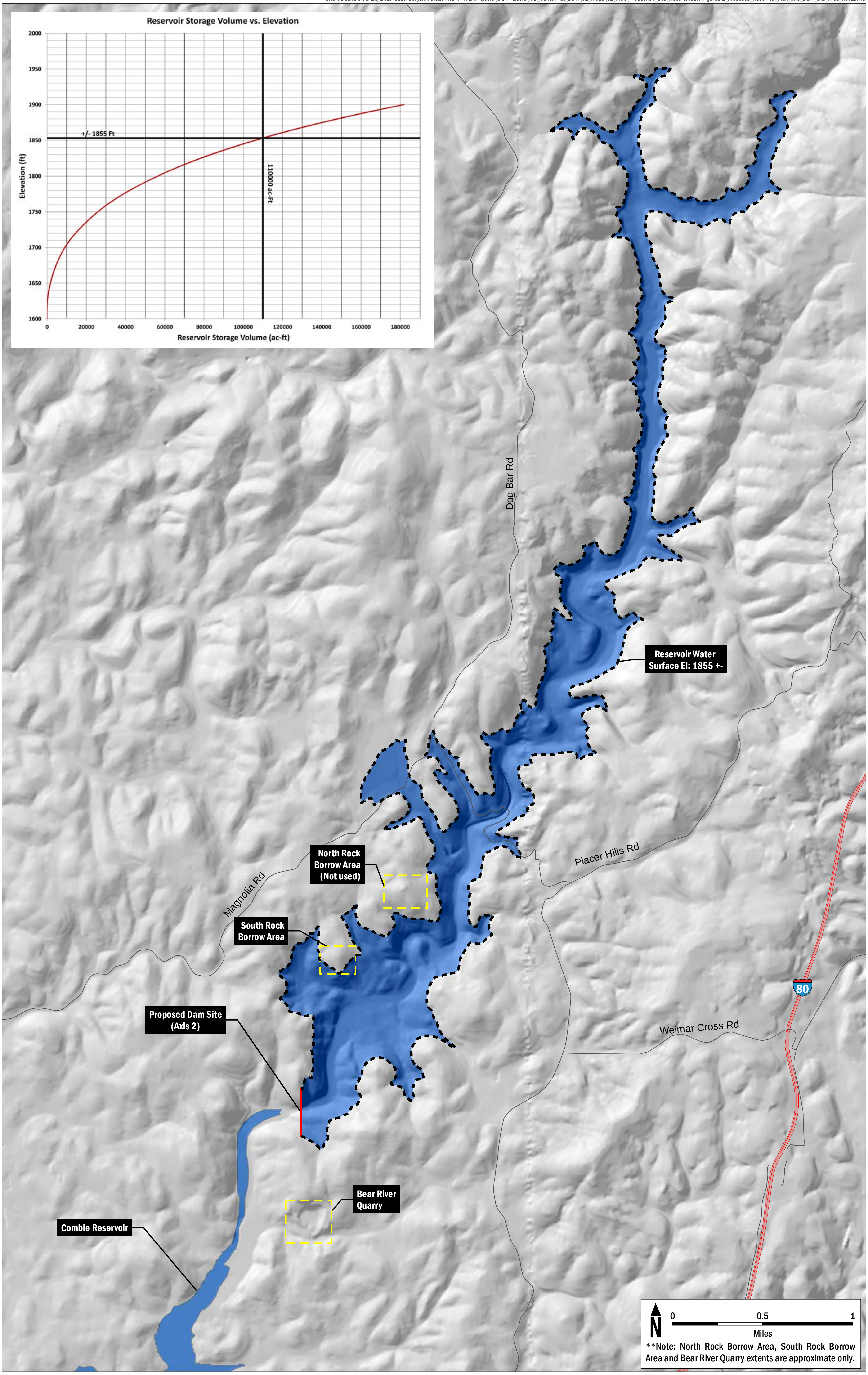

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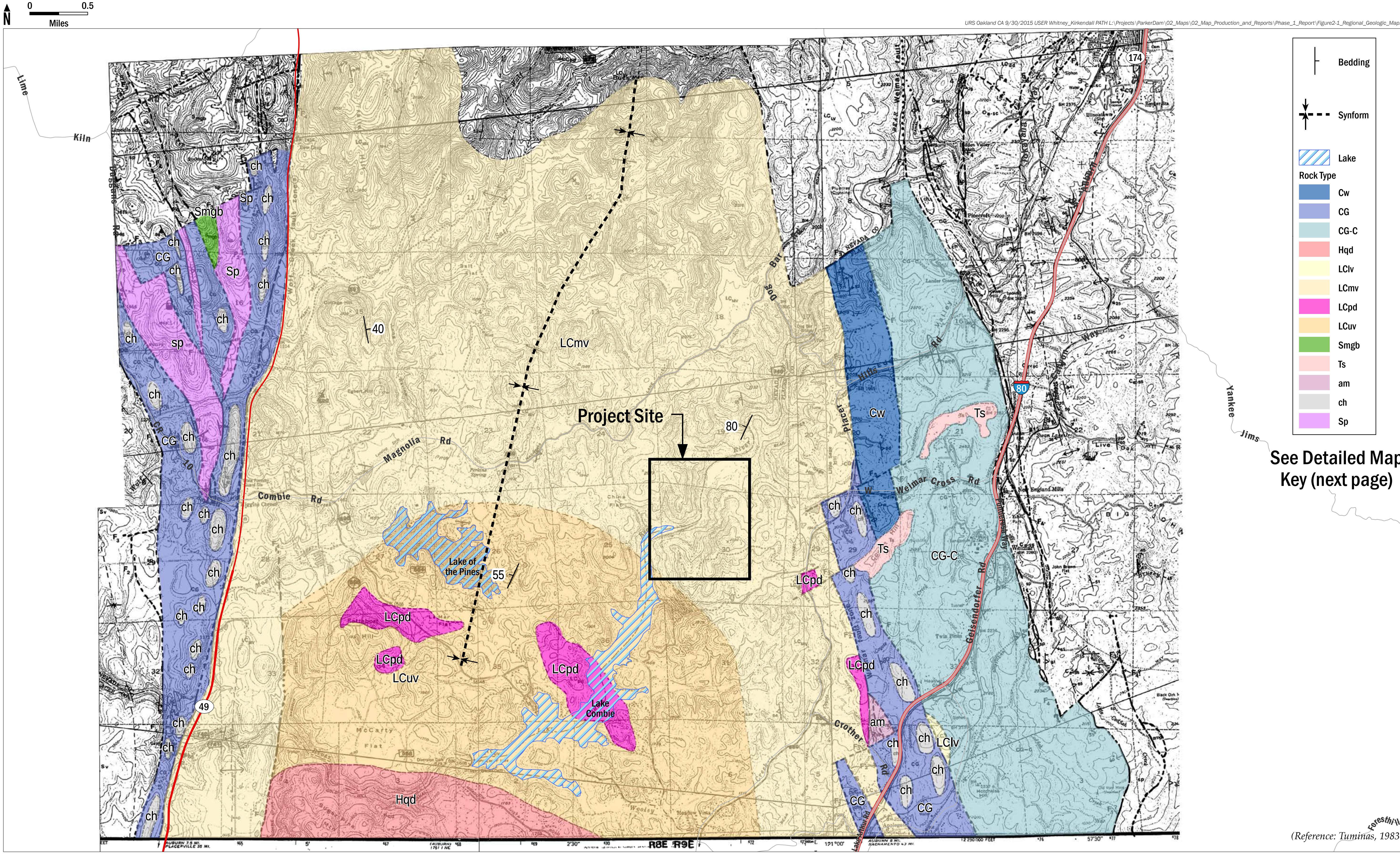
FIGURE 1-2

Centennial Reservoir Plan



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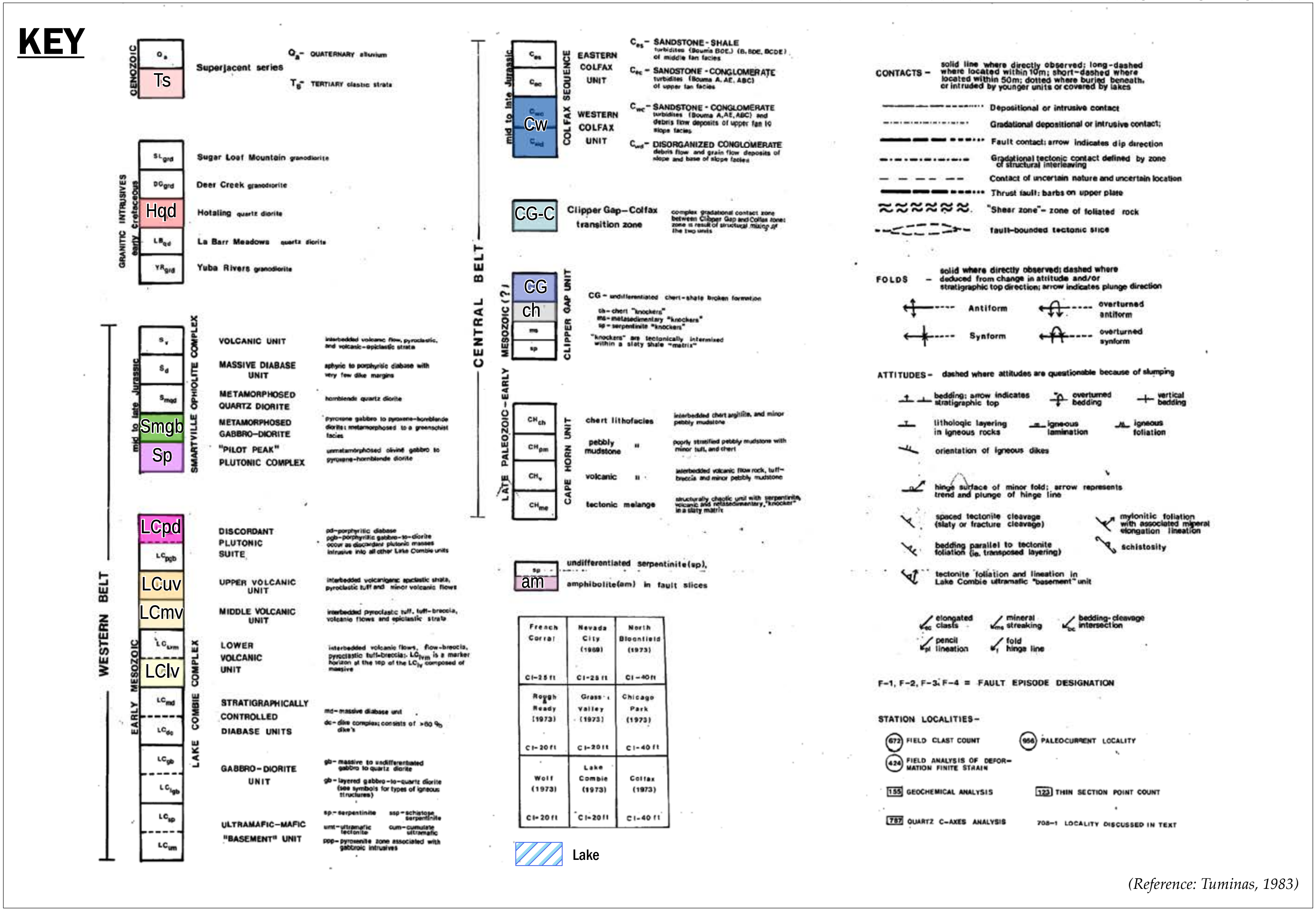
Centennial Reservoir Project

60503855

FIGURE 3-1

Regional Geologic Map



(Reference: Tuminas, 1983)

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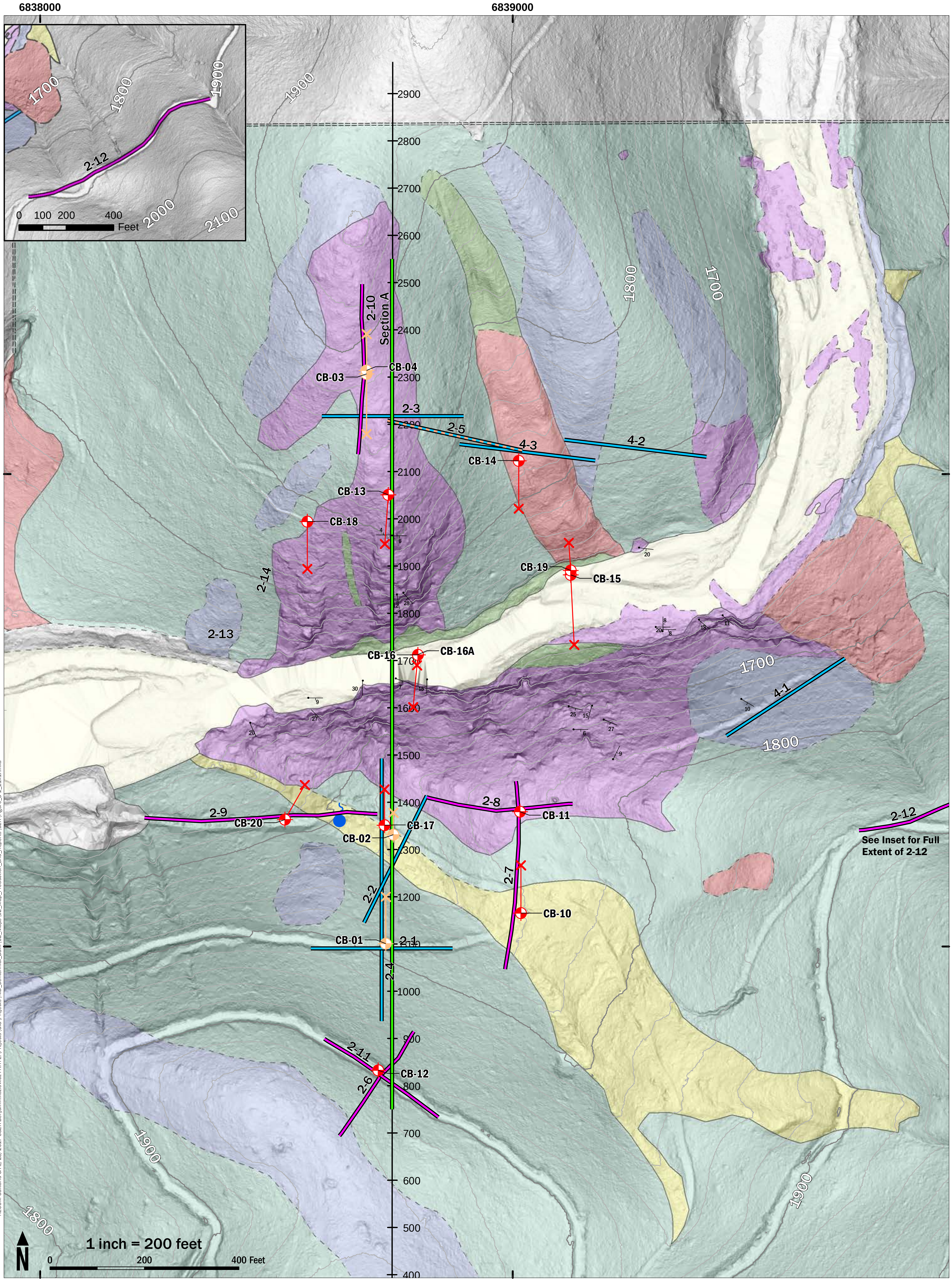
Nevada Irrigation District

Centennial Reservoir Project

60503855

FIGURE 3-1 (CONT'D)

Regional Geologic Map



Borings
- Borehole Top, Phase II
- Inclined Borehole Bottom, Phase II
- Borehole Top, Phase III
- Inclined Borehole Bottom, Phase III

- Proposed Dam Axis 2
- Geologic Cross Section Alignment
- Seismic Refraction Survey (Phase I)
- Seismic Refraction Survey (Phase II)
- Seismic Refraction Survey (Phase III)

- Index Contour (100ft)
- Contour (20ft)

- Bedding
- Spring

Surficial Geology Contacts
- contact, approximate or gradational
- contact
- Limit of Mapping

Surficial Geology Units
- Artificial Fill/Disturbed Ground
- Channel Alluvium
- Alluvial/Debris Fan Deposits
- Talus/Colluvial Deposits
- Landslide/Debris Flow Deposits
- Colluvial Slope
- Bedrock Outcrop-Class 1
- Bedrock Slope-Class 2

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

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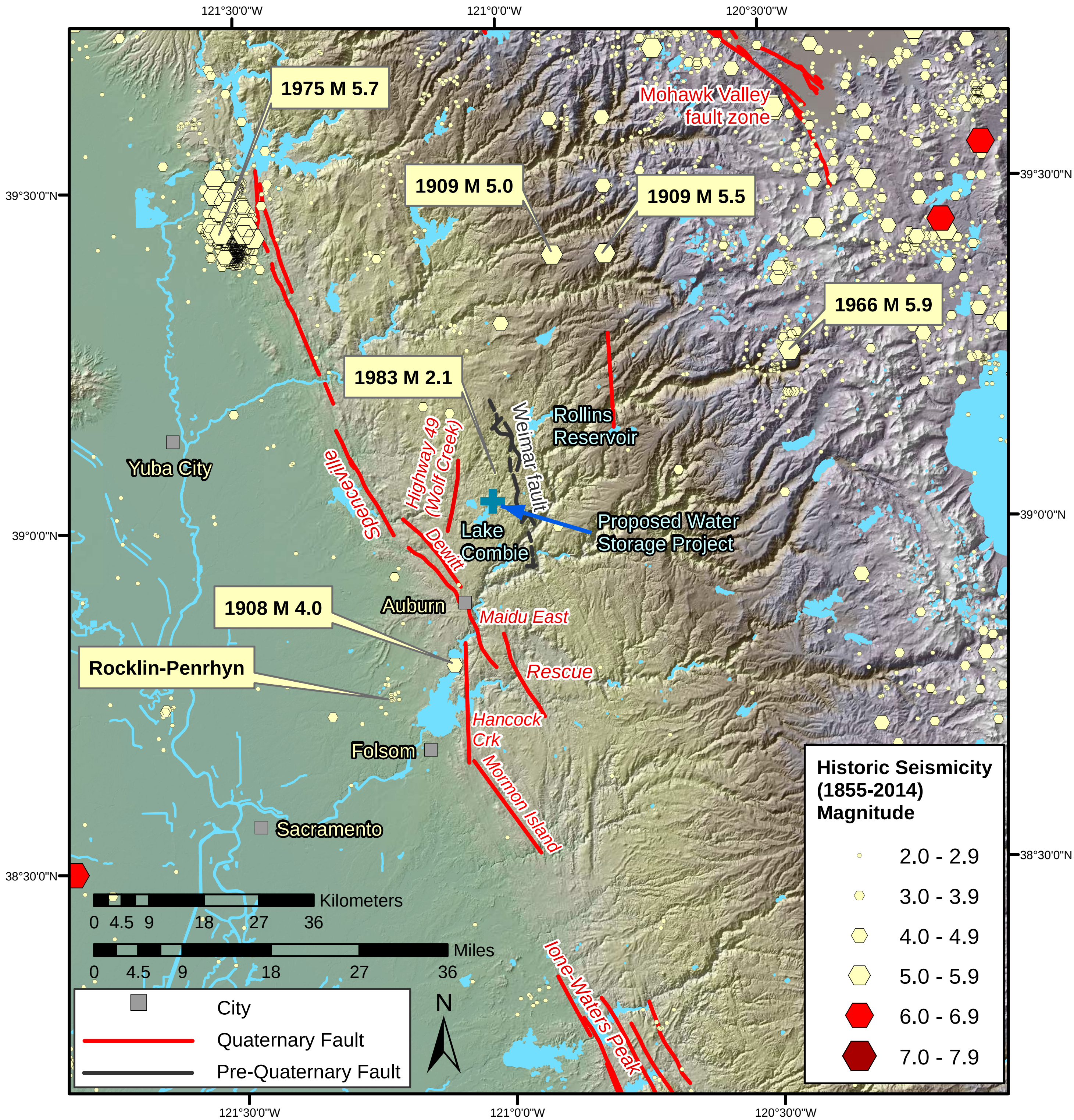
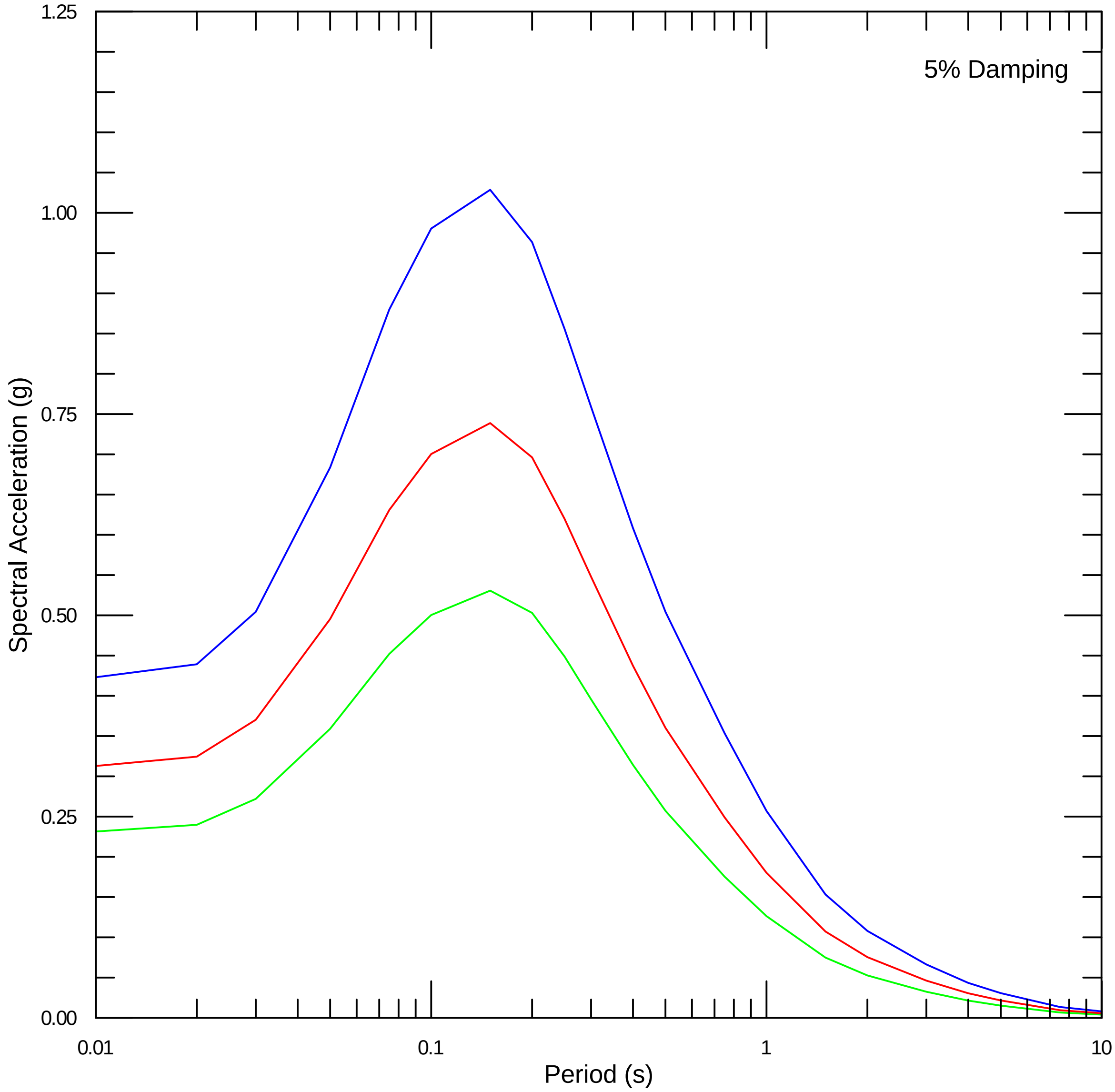
FIGURE 3-2
Surficial Geology and Geotechnical Exploration Plan



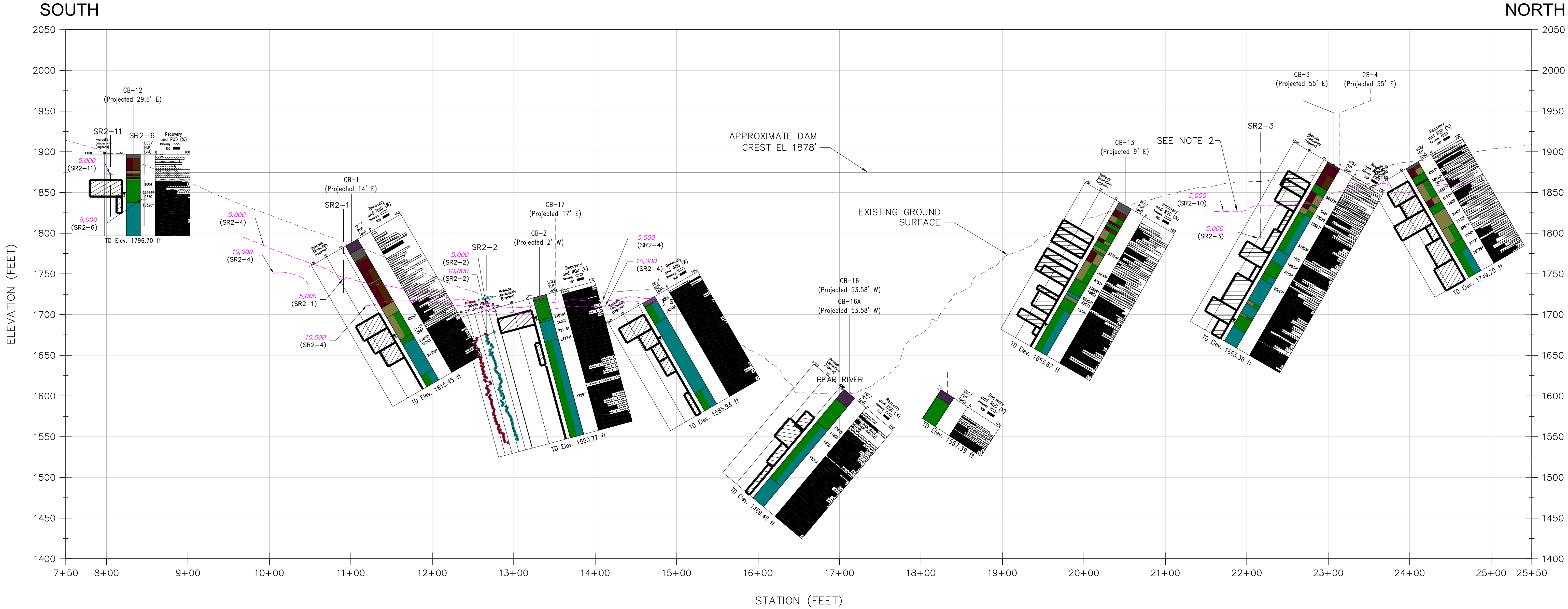
AECOM	Project No. 60519386 Nevada Irrigation District	HISTORICAL SEISMICITY (1855-2014)	Figure 4-1



Wolf Creek fault
M 6.5
R_{RUP} = 5.9 km
R_{JB} = 5.9 km
Normal Fault
Dip 80° west
V_S30 = 1000 m/s

Median
69th-Percentile
84th-Percentile

AECOM	Project No. 60519386 Nevada Irrigation District	DETERMINISTIC HORIZONTAL ACCELERATION RESPONSE SPECTRA FOR THE WOLF CREEK FAULT	Figure 4-2



SECTION A 3-2
SCALE: 1" = 60'

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)

TD — TOTAL DEPTH OF HOLE

UCS/PLI* — UNCONFINED COMPRESSIVE STRENGTH; * INDICATES RESULT FROM POINT LOAD TEST; SEE GER FOR DETAILS

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

SR2-4 — SEISMIC REFRACTION LINE 2-4

LOCATION OF SR LINE, AT CROSSING OF PROFILE (SEE GER)

NOTES:

1. ELEVATION DATUM IS NAVD (1988).
2. WHERE SEISMIC REFRACTION PROFILE DATA ARE PROJECTED ONTO THE SECTION FROM SURVEYS ACQUIRED AT DIFFERENT SURFACE ELEVATIONS, THE DEPTH TO THE PROFILE DATA IS MAINTAINED.

0 60 120 Feet

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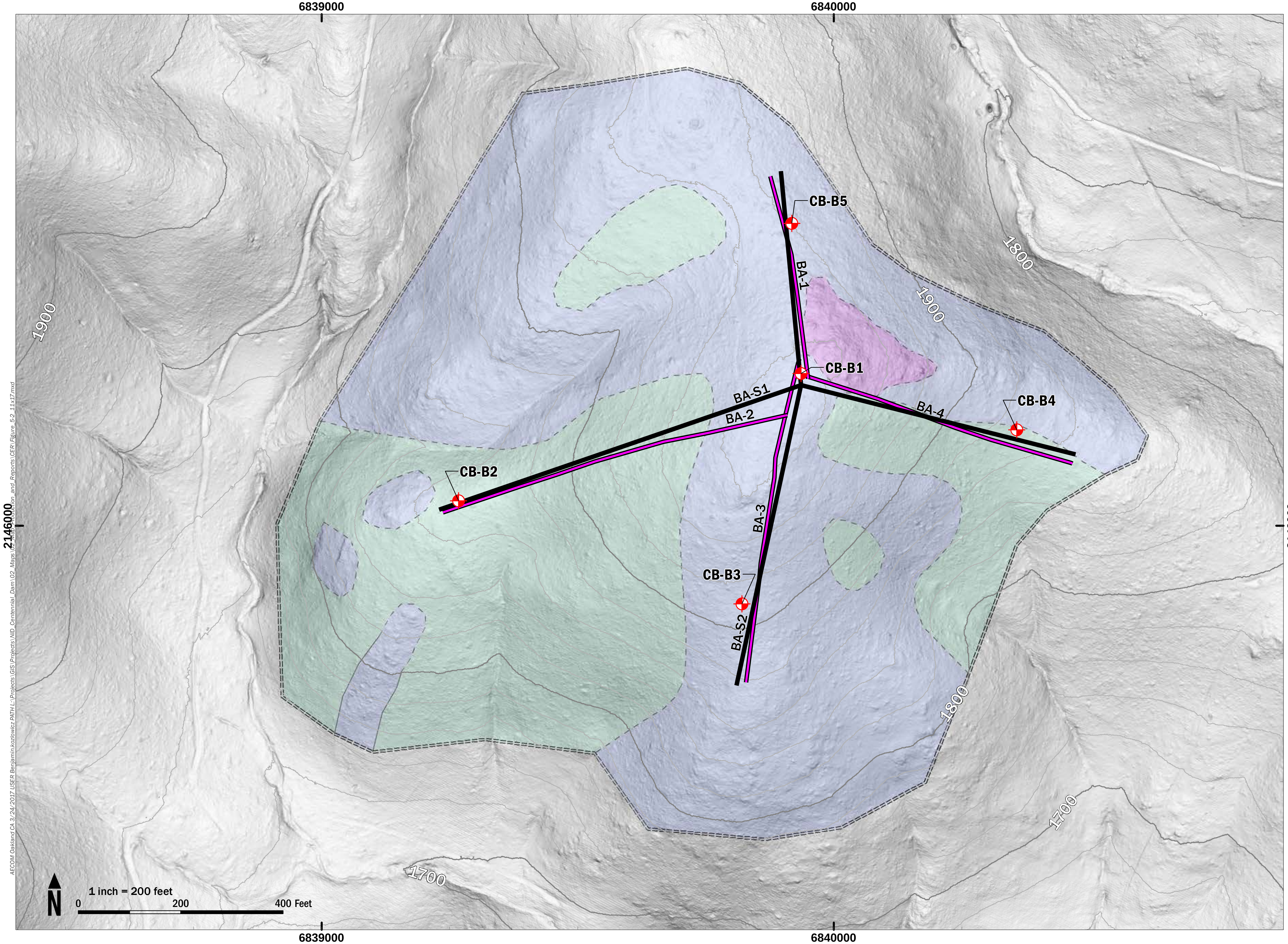
60519386

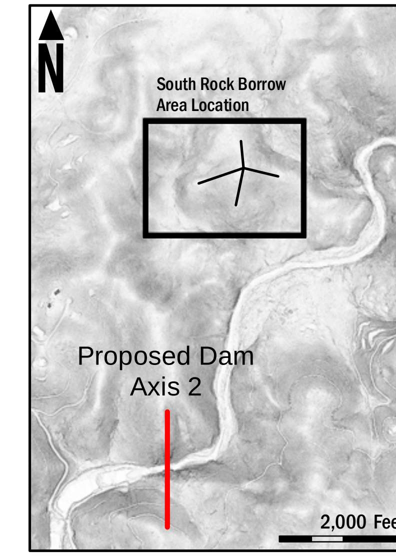
NEVADA IRRIGATION DISTRICT
CENTENNIAL RESERVOIR PROJECT

Geotechnical Profile
Axis 2

FIGURE
5-1



Map Location



Legend

Investigations

- Borehole, Phase III
- Seismic Refraction Survey (Phase III)
- Geologic Cross Section Alignment
- Index Contour (100ft)
- Contour (20ft)

Surficial Geology

- Contact, approximate or gradational
- Contact
- Limit of Mapping

Geologic Units

- Residual Soil/Colluvium
- Bedrock Outcrop-Class 1
- Bedrock Slope-Class 2

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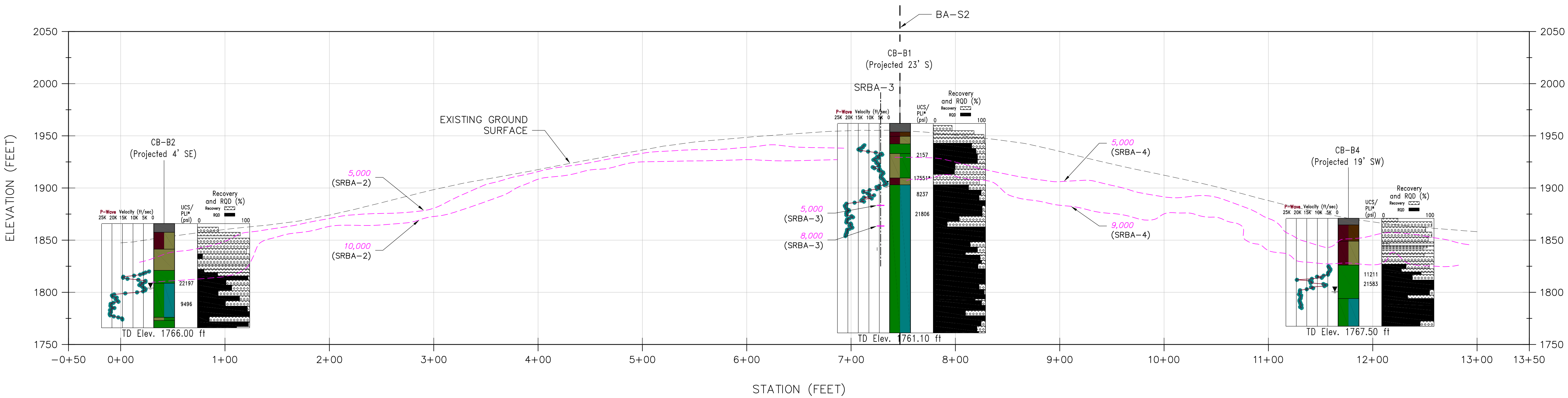
Nevada Irrigation District
Centennial Reservoir Project
60519386

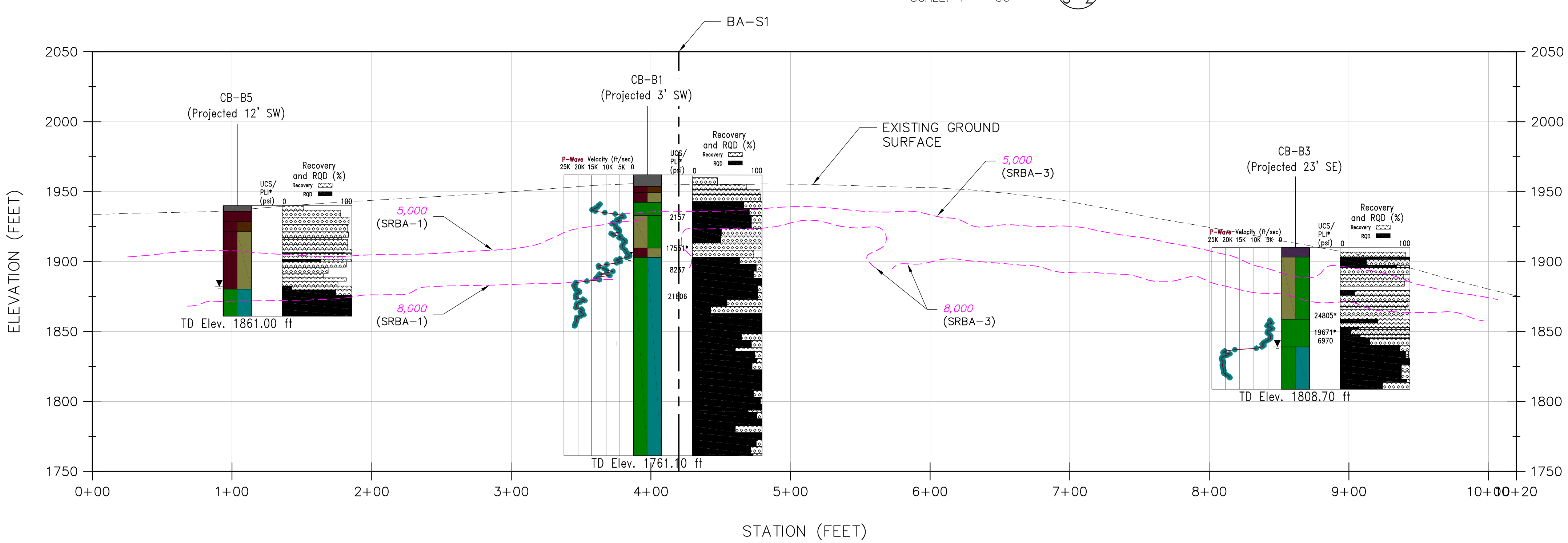
Reference:
NID Parker Dam Lidar,
Nevada Irrigation District, November 2014.

FIGURE 5-2

Geotechnical Exploration Plan - Potential South Rock Borrow Area



SECTION BA-S1 5-2
SCALE: 1" = 50'



SECTION BA-S2 5-2
SCALE: 1" = 50'

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)
TD	TOTAL DEPTH OF HOLE
UCS/PLI*	UNCONFINED COMPRESSIVE STRENGTH; * INDICATES RESULT FROM POINT LOAD TEST; SEE GER FOR DETAILS
5,000	P-WAVE VELOCITY (ft./sec.) FROM SEISMIC REFRACTION SURVEY; SEE GER FOR DETAILS
SRBA-1	SEISMIC REFRACTION LINE BA-1
⋮	LOCATION OF SR LINE, AT CROSSING OF PROFILE

0 50 100 Feet

NOTES:

1. ELEVATION DATUM IS NAVD (1988).
2. WHERE SEISMIC REFRACTION PROFILE DATA ARE PROJECTED ONTO THE SECTION FROM SURVEYS ACQUIRED AT DIFFERENT SURFACE ELEVATIONS, THE DEPTH TO THE PROFILE DATA IS MAINTAINED.

AECOM | 60519386 | NEVADA IRRIGATION DISTRICT CENTENNIAL RESERVOIR PROJECT | Geotechnical Sections South Borrow Area | FIGURE 5-3

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L:\Projects\Legacy\IE_Xdrive\x_water\NID Parker Dam\16_Phase 3 Engineering\CER\CAD\BA Figure 5-3.dwg

Exhibits

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CENTENNIAL RESERVOIR PROJECT

Nevada Irrigation District

1036 W. Main Street
Grass Valley, CA 95945

September 2017

EXHIBITS

DRAFT
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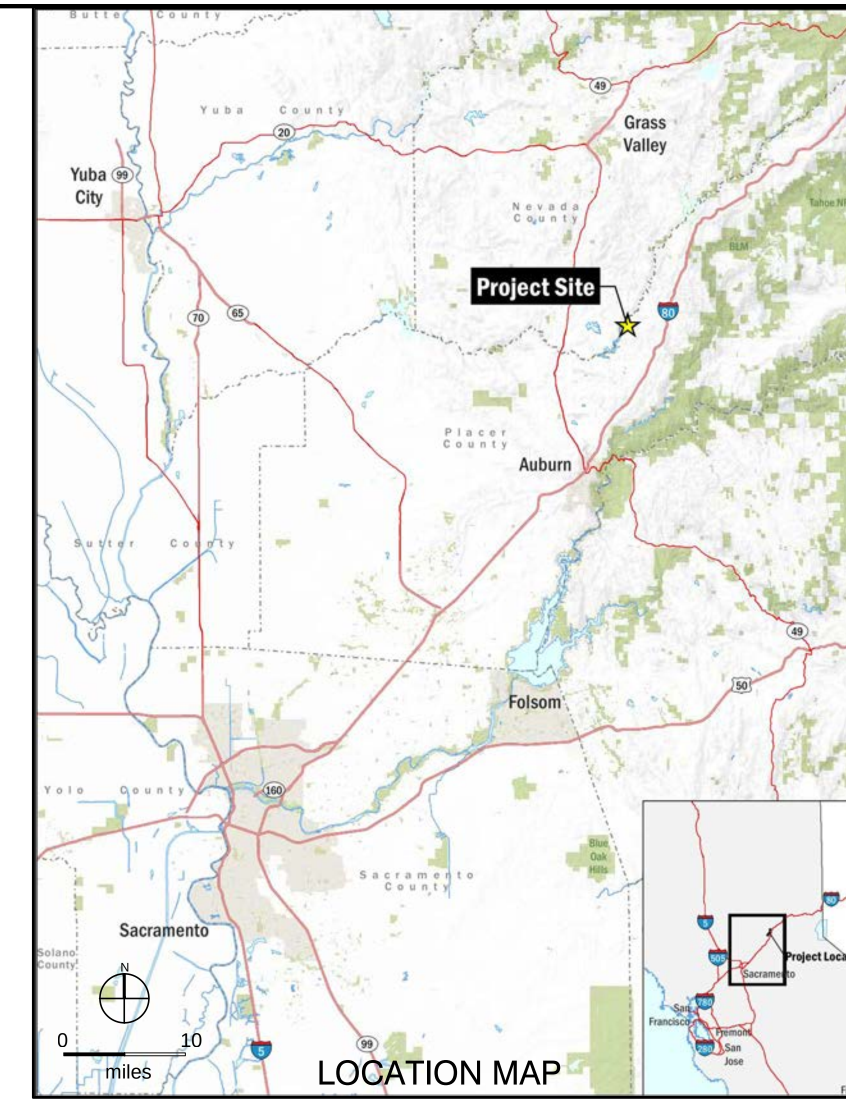
300 Lakeside Dr., Suite 400
Oakland, CA 94612



List of Exhibits

Exhibit No.	Title
1	COVER SHEET
2	EXCAVATION PLAN
3	PLAN OF DAM
4A	FOUNDATION PROFILE
4B	DAM PROFILE
5	MAXIMUM SPILLWAY SECTION AND CREST DETAIL
6	NON-OVERFLOW SECTIONS
7	DAM SECTIONS - 1
8	DAM SECTIONS - 2
9	OUTLET WORKS PLAN AND PROFILE
10	MAIN DETAILS
11	SPILLWAY BRIDGE PLAN AND ELEVATION
12	SPILLWAY BRIDGE SECTIONS
13	INSTRUMENTATION PLAN AND TYPICAL SECTION
14	DIVERSION CONCEPT
15	CONSTRUCTION SITE LAYOUT - CONCEPT PLAN 1
16	CONSTRUCTION SITE LAYOUT - CONCEPT PLAN 2

DRAWING NO. REV
SHEET NO.
EXHIBIT 1
PAGE NO. __ OF __

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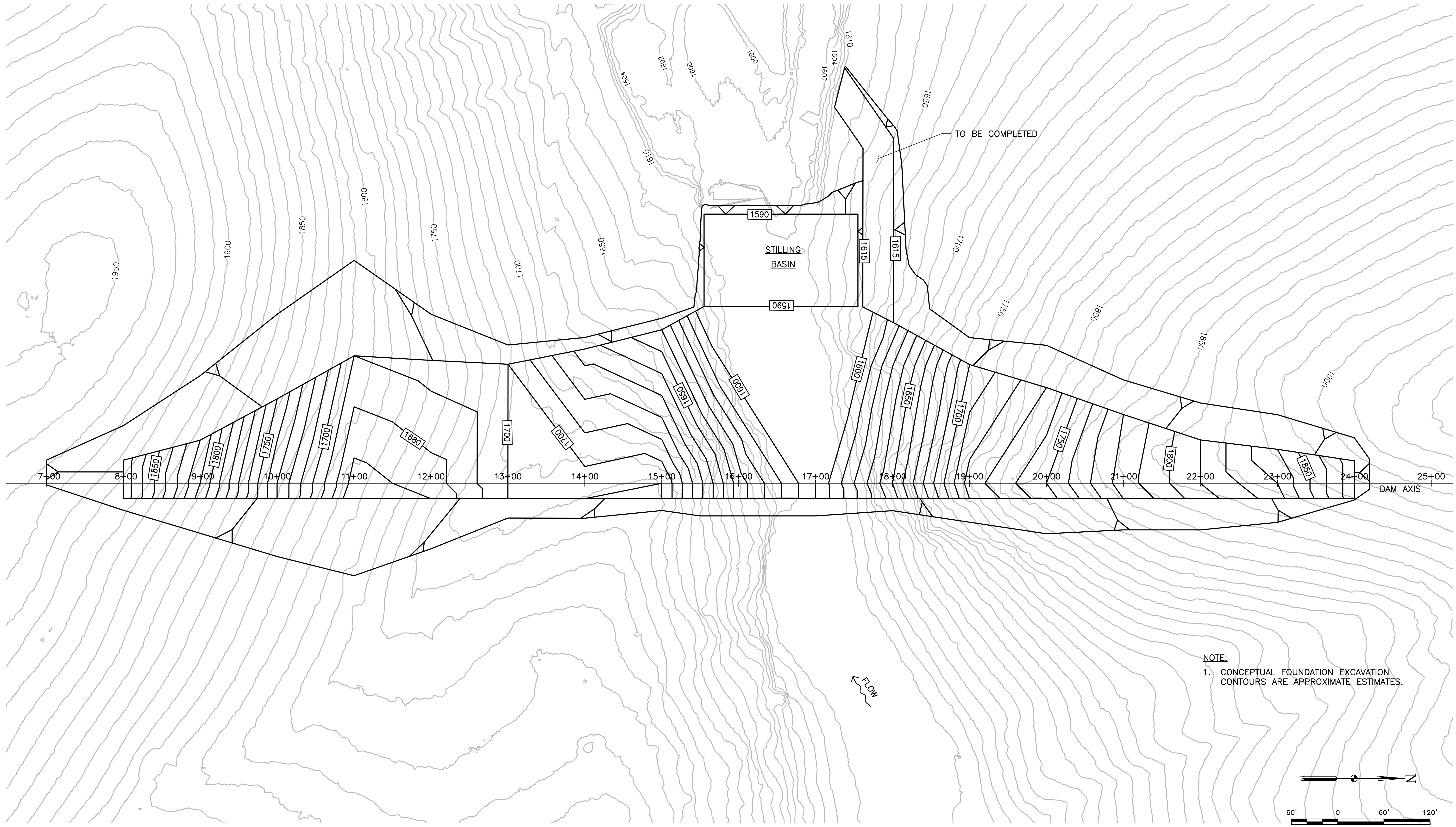


NOTE:

1. CONCEPTUAL FOUNDATION EXCAVATION CONTOURS ARE APPROXIMATE ESTIMATES.

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

AECOM
300 Lakeside Drive, Suite 400, Oakland, CA 94612
Tel: (510) 893-3600 Fax: (510) 874-3268

DESIGN	APPROVED
DRAWN	SCALE AS NOTED
CHECKED	CADFILE NID PLANS EXH.DWG

Nevada Irrigation District
Grass Valley, California

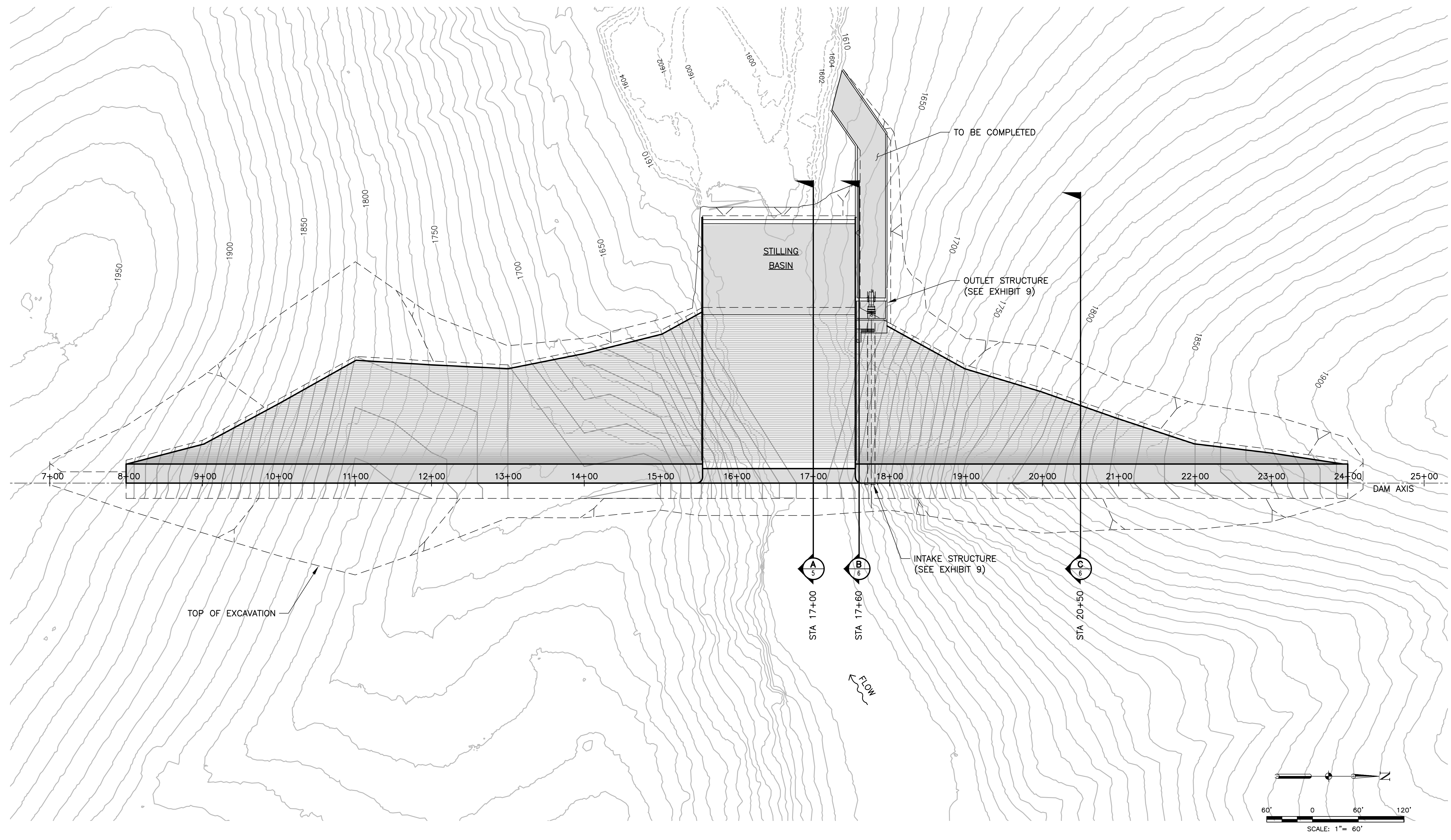
Centennial Reservoir Project

EXCAVATION PLAN

PROJECT NO 60519386
REV
SHEET EXHIBIT 2
_OF _

Jul 05, 2017 – 2:48pm C:\000\NID\Dam and Spillway\NID PLANS exh.dwg

THIS DRAWING TO BE READ IN CONJUNCTION WITH THE COMPLETE PROJECT DOCUMENTATION SET INCLUDING THE SPECIFICATIONS.



REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

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Tel: (510) 893-3600 Fax: (510) 874-3268

DESIGN	APPROVED
DRAWN	SCALE AS NOTED
CHECKED	CADFILE NID PLANS EXH.NO BRANCH PIPE.DWG

Nevada Irrigation District
Grass Valley, California

Centennial Reservoir Project

PLAN OF DAM

PROJECT NO 60519386
REV
SHEET EXHIBIT 3
_OF _

Jul 03, 2017 – 12:24pm
J:\NID\Dam and Spillway\NID PLANS exh.no branch pipe.dwg

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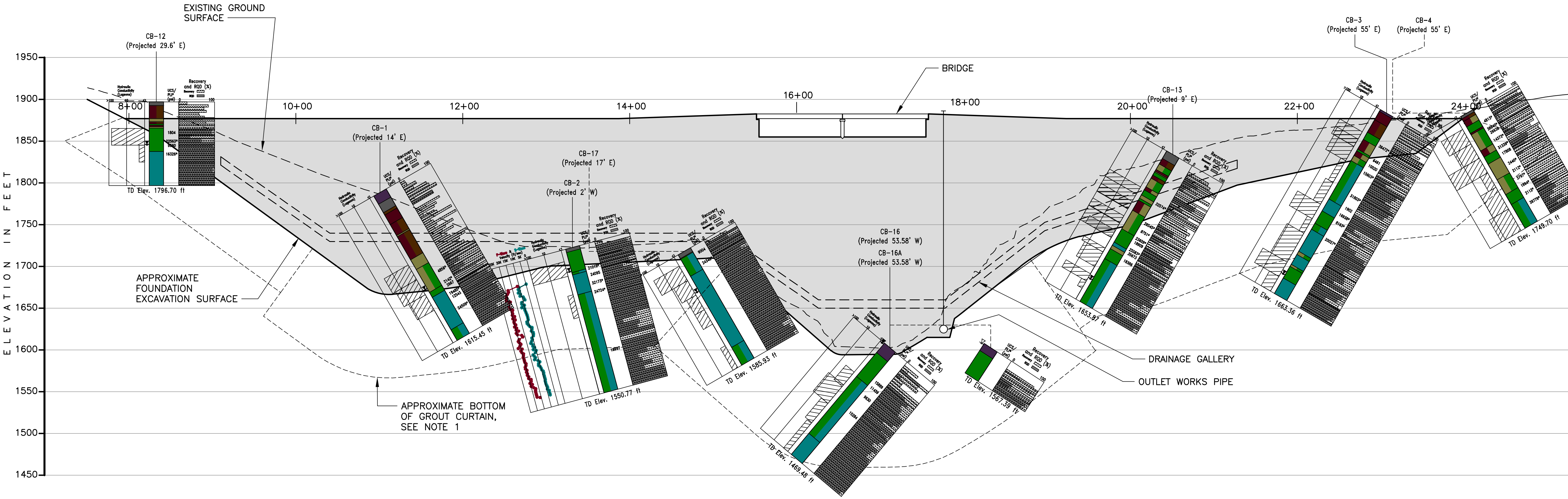

LEGEND:

WEATHERING PROFILE:

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

SECTION ALONG DAM AXIS

60' 0 60' 120'

SCALE: 1"= 60'

NOTE:

1. SEE EXHIBIT 4B FOR GROUT CURTAIN DIMENSIONS.

Apr 13, 2017 – 3:57pm
J:\NID\Dam and Spillway\Exhibit 4a.dwg

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES

0 1"

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DRAWN	SCALE AS NOTED
CHECKED	CADFILE EXHIBIT 4A.DWG

Nevada Irrigation District
Grass Valley, California

Centennial Reservoir Project

FOUNDATION PROFILE

PROJECT NO 60519386

REV

SHEET EXHIBIT 4A

_ OF _

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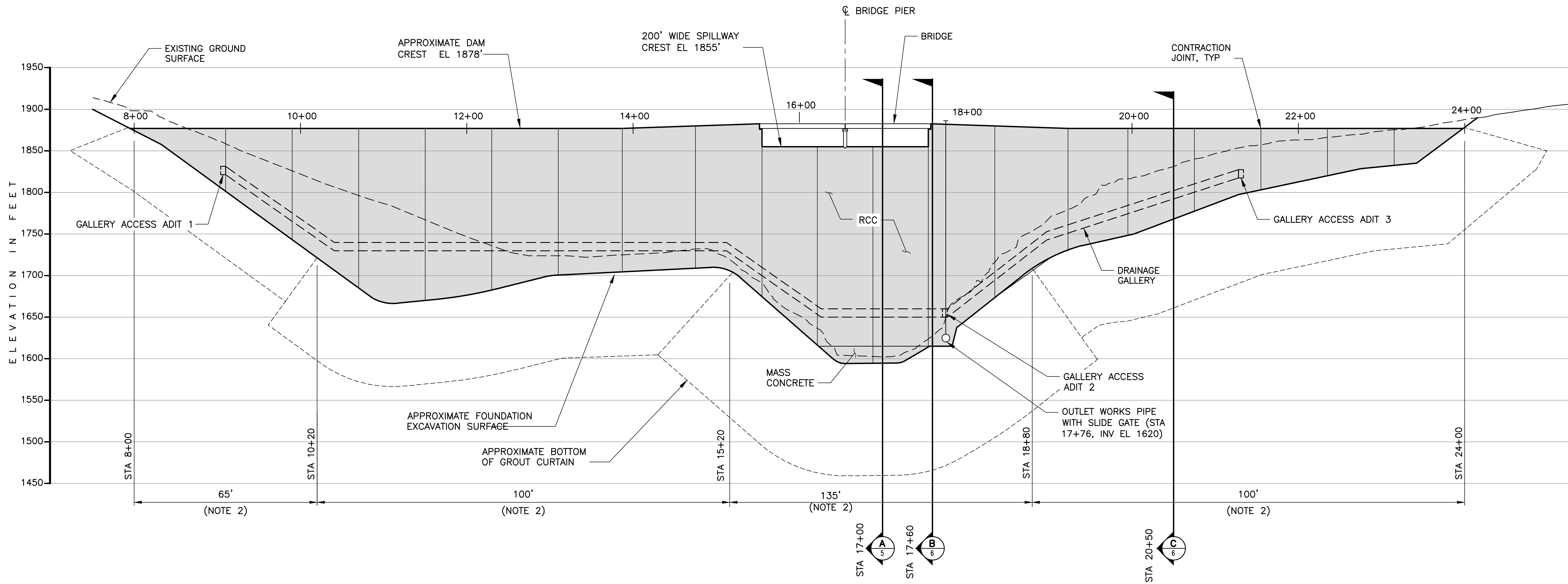

SECTION ALONG DAM AXIS

60' 0 60' 120'

SCALE: 1"= 60'

NOTES:

1. PARAPET WALL AND RAILINGS NOT SHOWN, SEE EXHIBIT 10.
2. MIN. LENGHTS OF PRIMARY & SECONDARY CURTAIN HOLES SHALL BE SUCH THAT THE DEPTH OF GROUT CURTAIN IS AS SHOWN IN PROFILE, MEASURED NORMAL TO FOUNDATION SURFACE.

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES

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CHECKED	CADFILE NID BASE EXH.DWG

Nevada Irrigation District
Grass Valley, California

Centennial Reservoir Project

DAM PROFILE

PROJECT NO 60519386

REV

SHEET

EXHIBIT 4B

_ OF_

Jul 07, 2017 – 8:48am
J:\NID\Dam and Spillway\NID BASE exh.dwg

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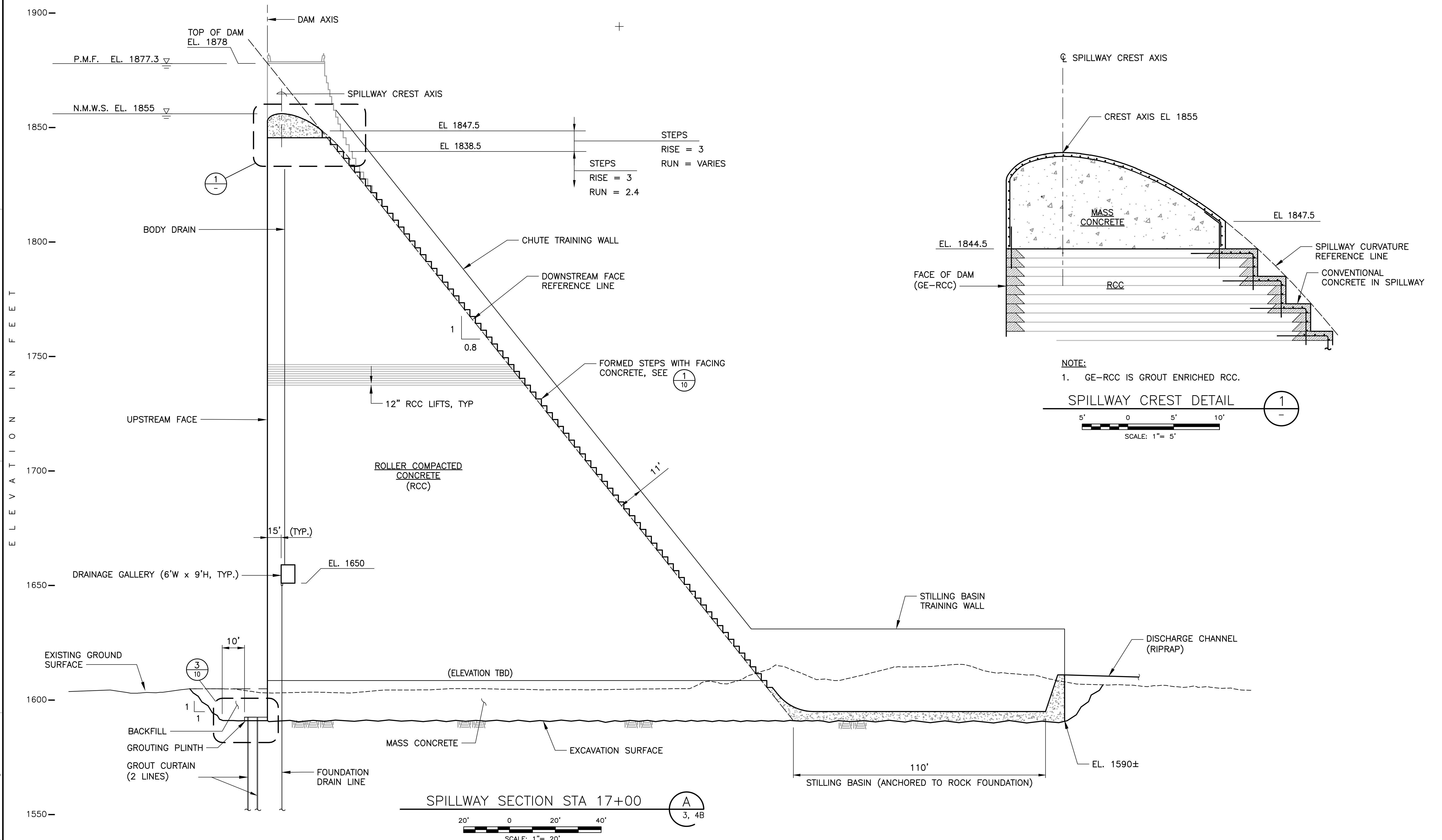

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES

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DRAWN	SCALE AS NOTED
CHECKED	CADFILE NID BASE EXH.DWG

Nevada Irrigation District

Grass Valley, California

Centennial Reservoir Project

MAXIMUM SPILLWAY SECTION AND CREST DETAIL

PROJECT NO 60519386

REV

SHEET

EXHIBIT 5

_ OF _

May 15, 2017 – 1:27pm
J:\NID\Dam and Spillway\NID BASE exh.dwg

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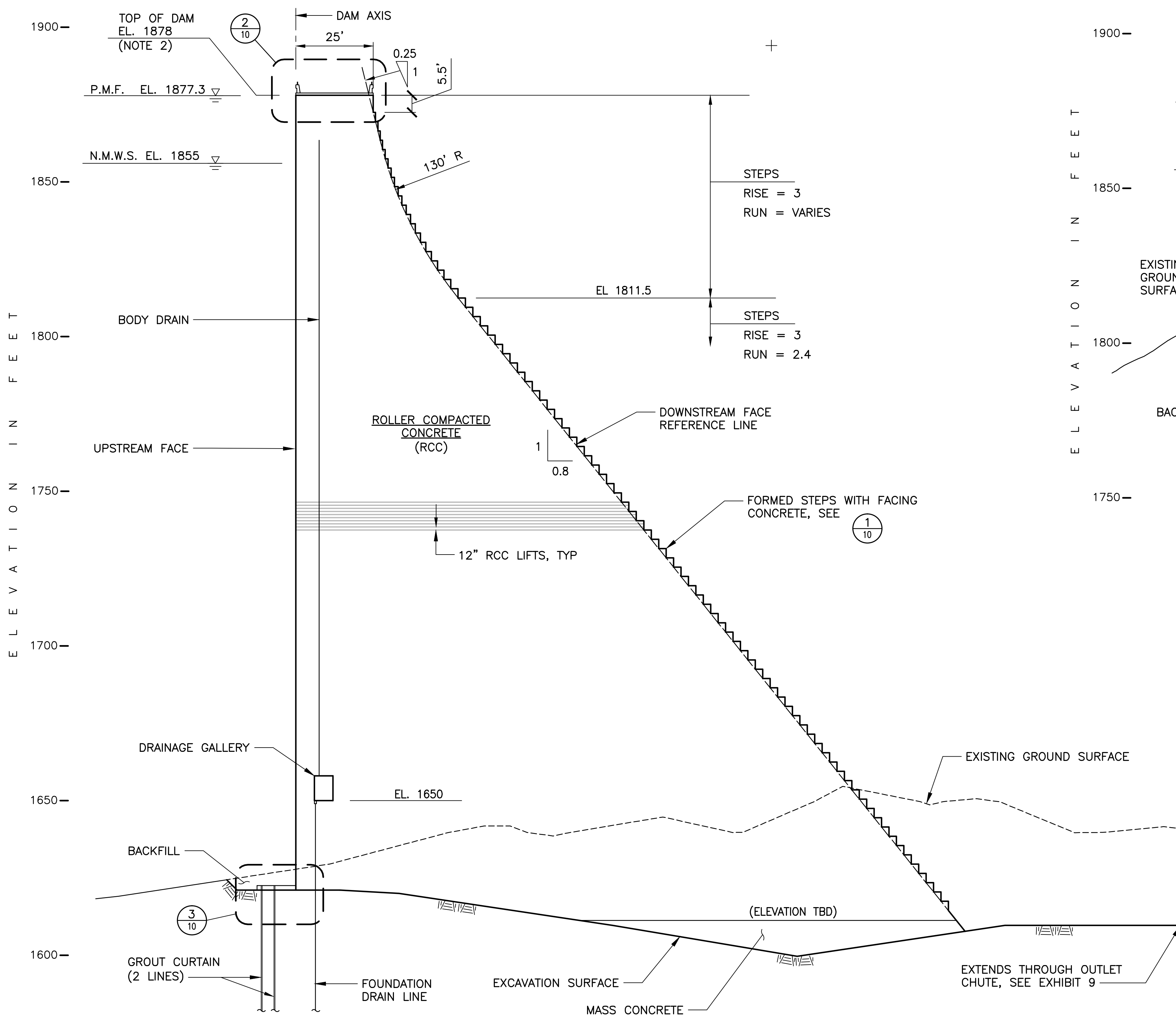
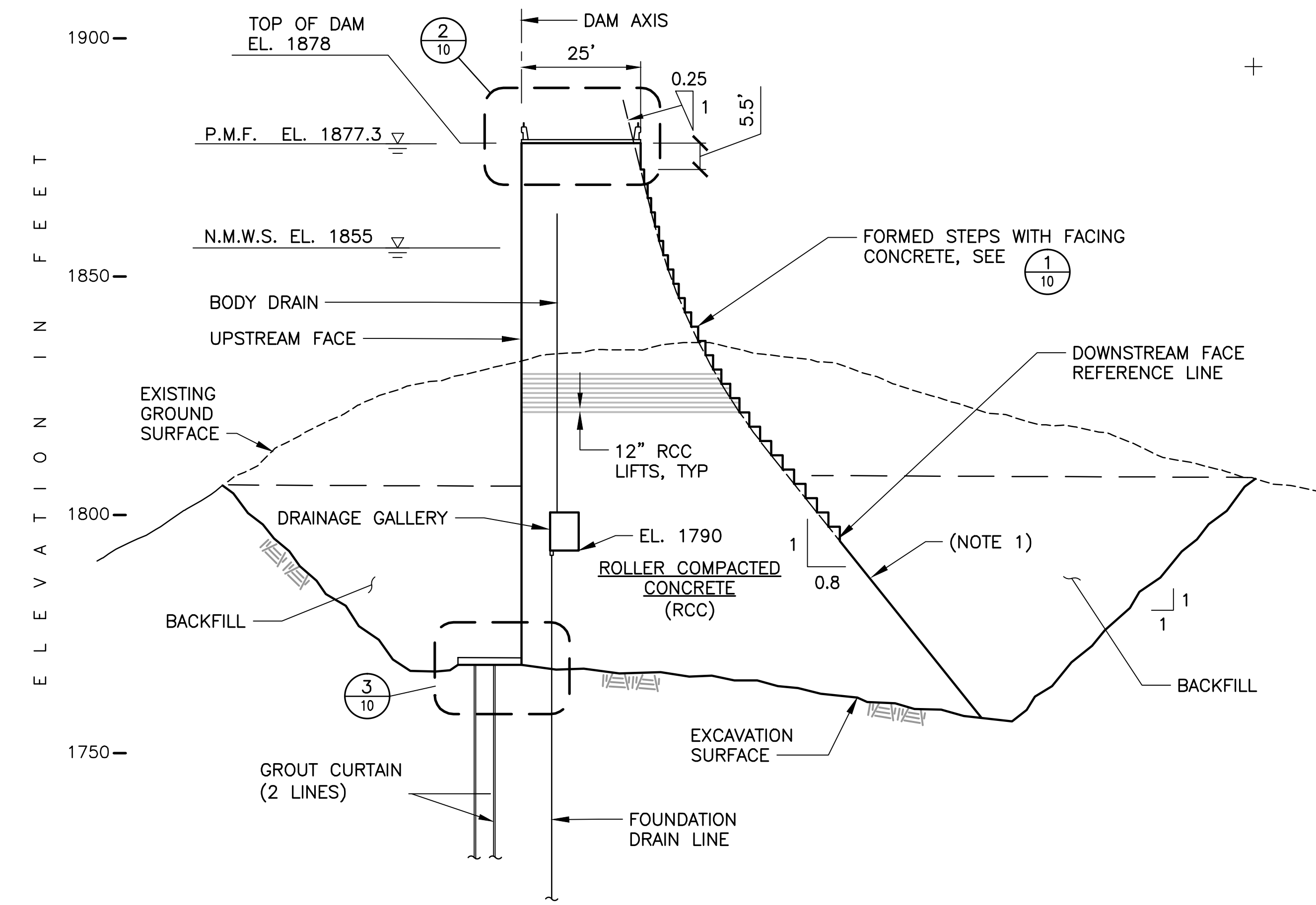


NON–OVERFLOW SECTION STA 17+60 (B, 3, 4B)

SCALE: 1"= 20'



NON–OVERFLOW SECTION STA 20+50 (C, 3, 4B)

SCALE: 1"= 20'

NOTES:

1. ELIMINATE STEPS BELOW BACKFILL.
2. RAMP TO SPILLWAY BRIDGE NOT SHOWN.

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES — BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

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DRAWN	SCALE: AS NOTED
CHECKED	CADFILE: NID BASE EXH.DWG

Nevada Irrigation District
Grass Valley, California

Centennial Reservoir Project

NON-OVERFLOW SECTIONS

PROJECT NO 60519386 | REV

SHEET: EXHIBIT 6
_ OF _

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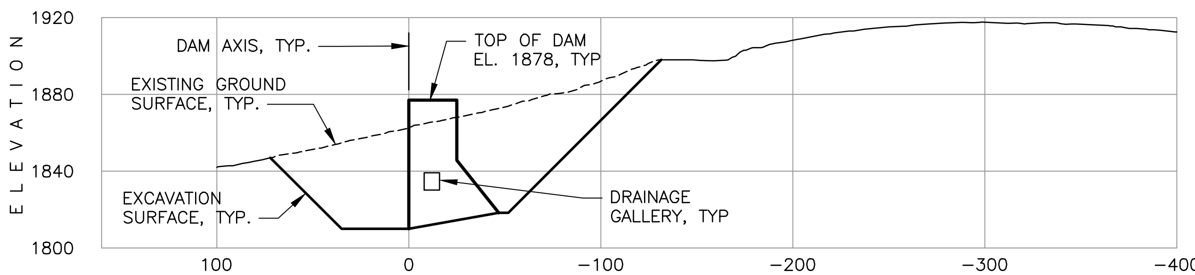
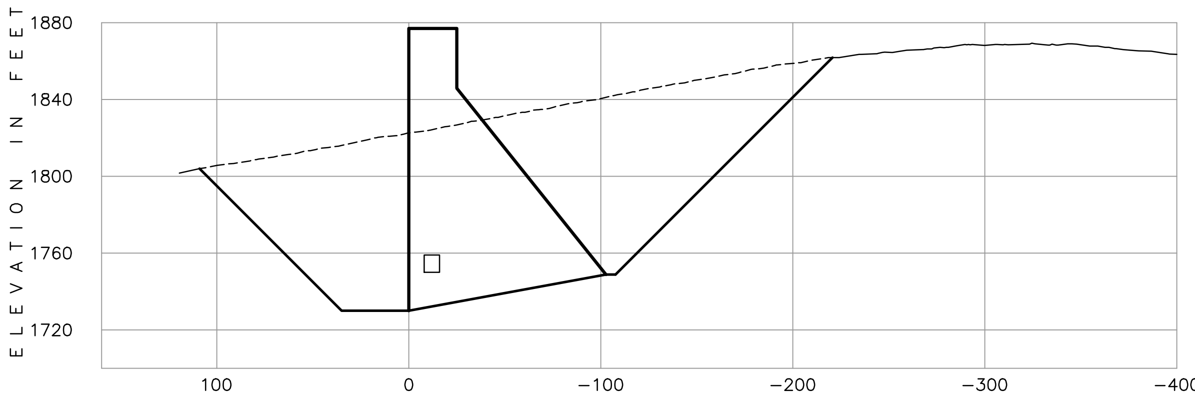
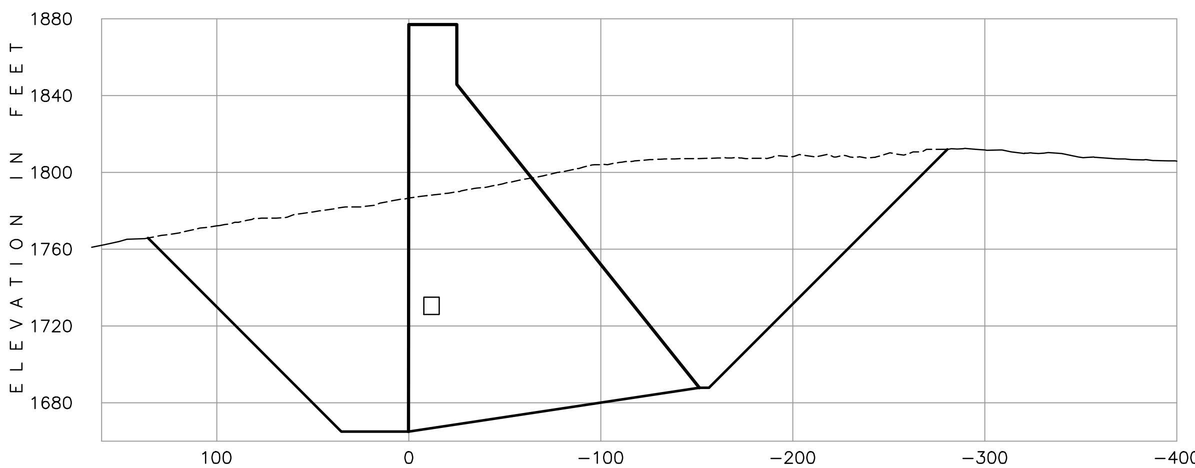


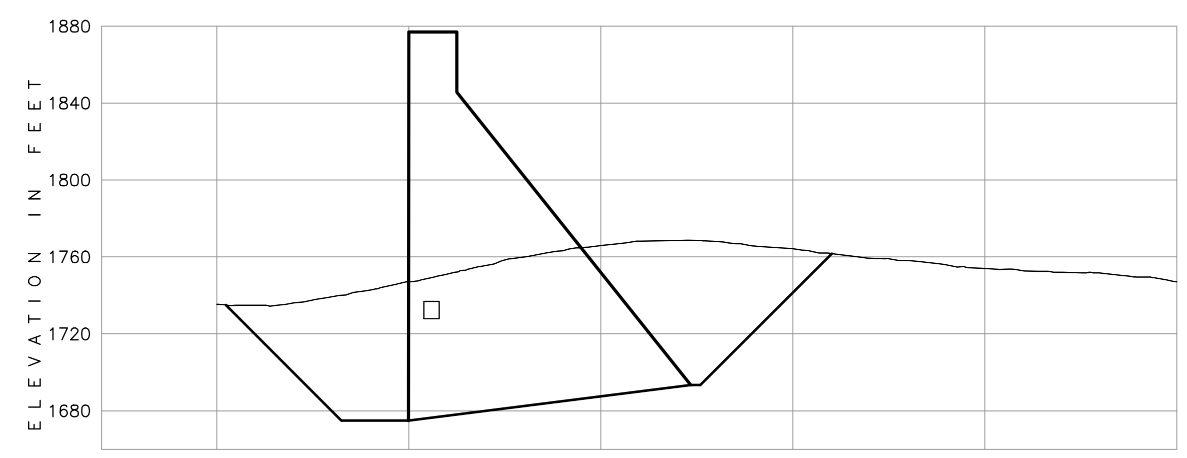
SECTION AT STA 9+00



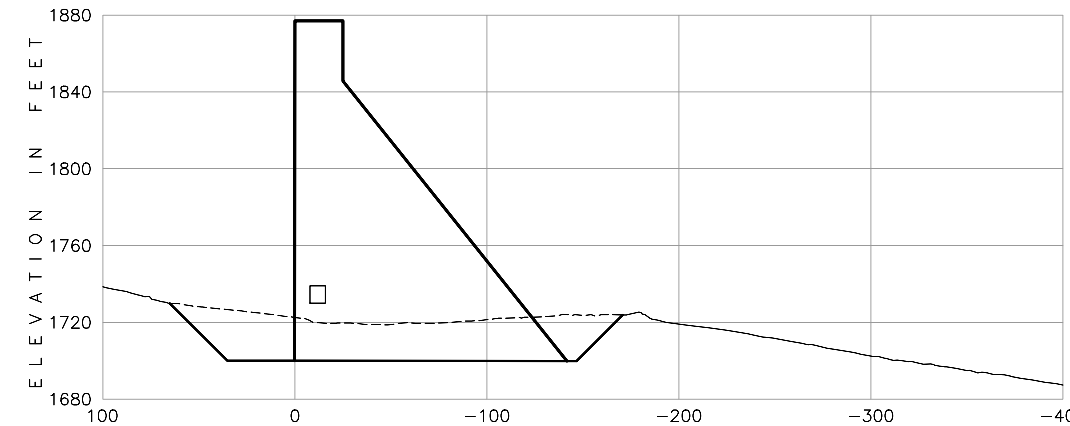
SECTION AT STA 10+00



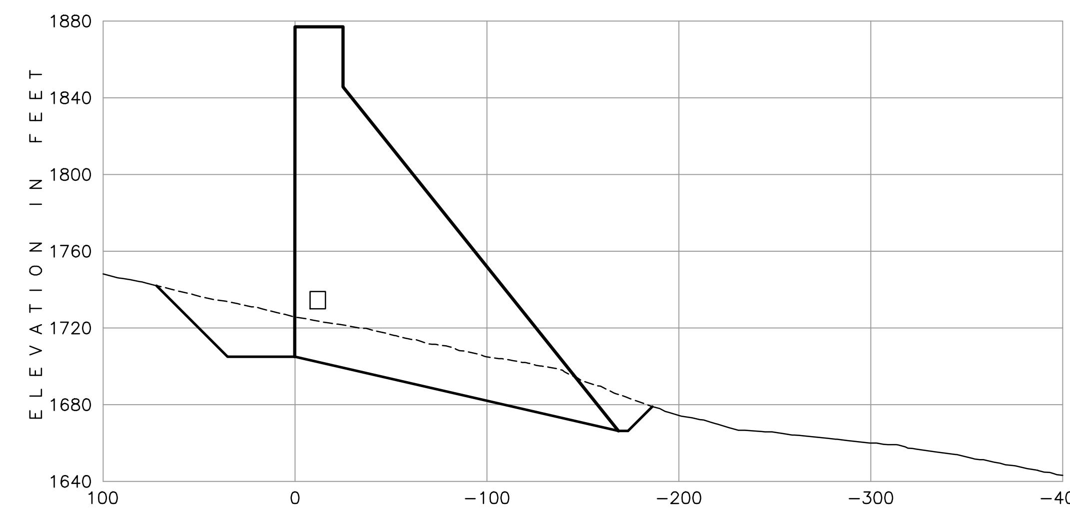
SECTION AT STA 11+00



SECTION AT STA 12+00



SECTION AT STA 13+00



SECTION AT STA 14+00

SECTION AT STA 15+00

50' 0 50' 100'
SCALE: 1"= 50'

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CHECKED	CADFILE NID SECTIONS.DWG

Nevada Irrigation District
Grass Valley, California

Centennial Reservoir Project

DAM SECTIONS - 1

PROJECT NO 60519386
REV
SHEET
EXHIBIT 7
_ OF _

Mar 29, 2017 – 12:45pm
J:\NID\Dam and Spillway\NID SECTIONS.dwg

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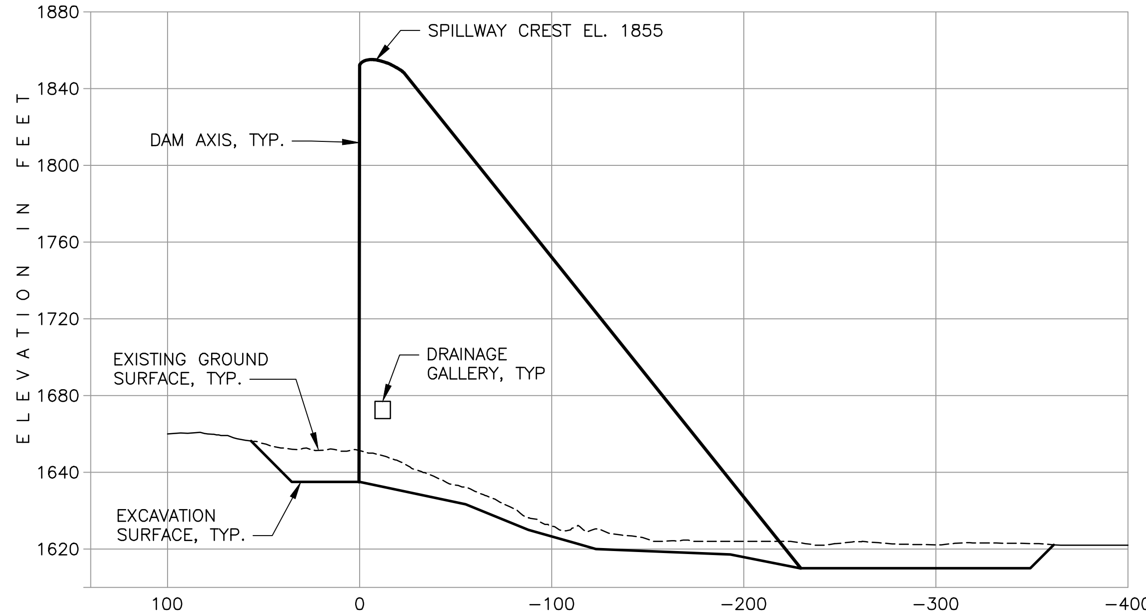
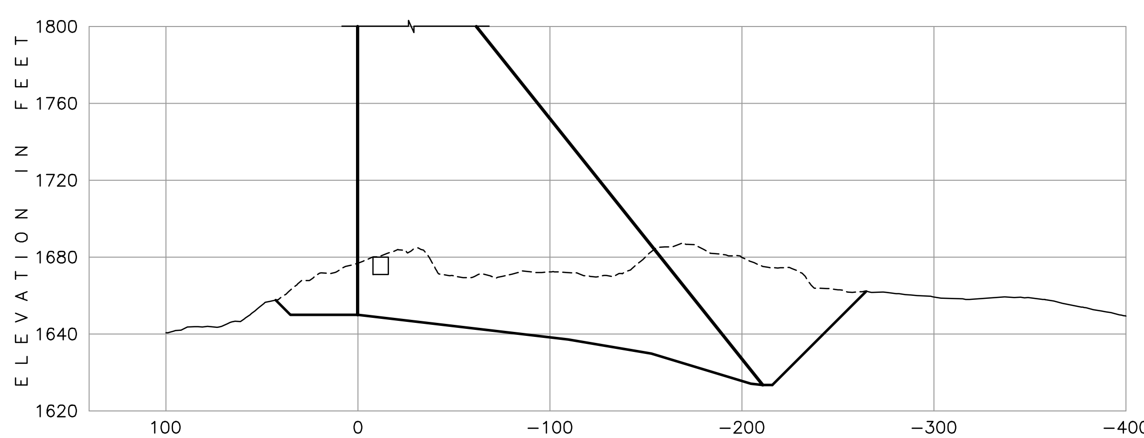
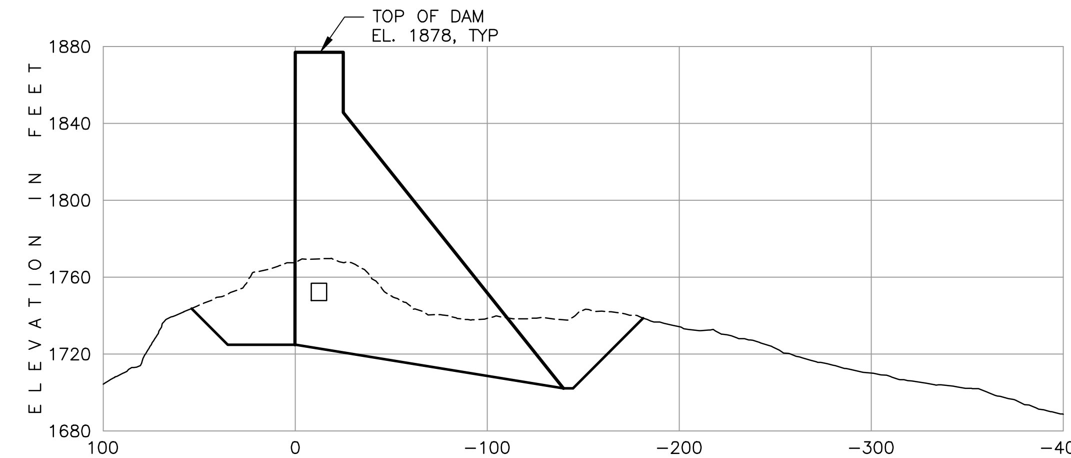


SECTION AT STA 16+00

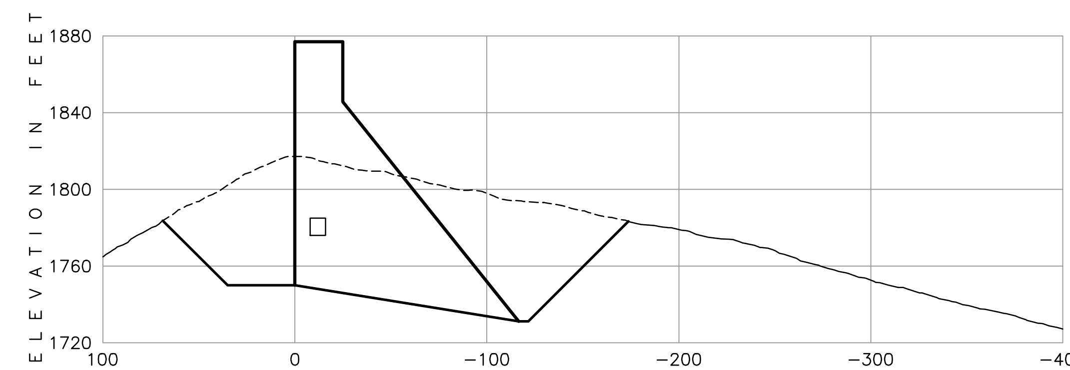
SECTION AT STA 17+00



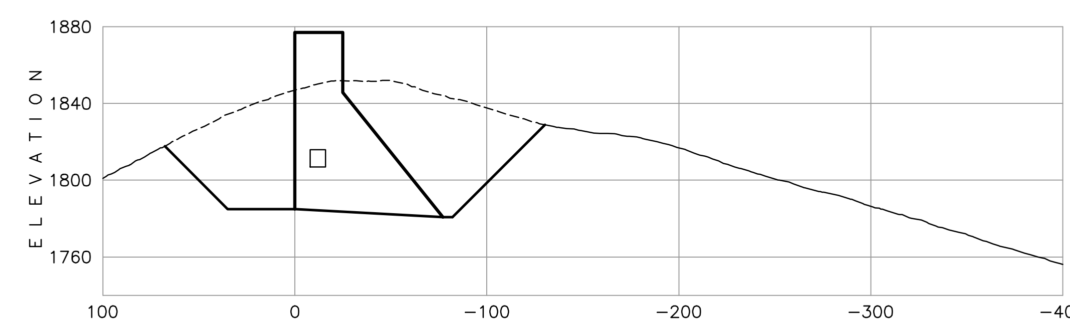
SECTION AT STA 18+00



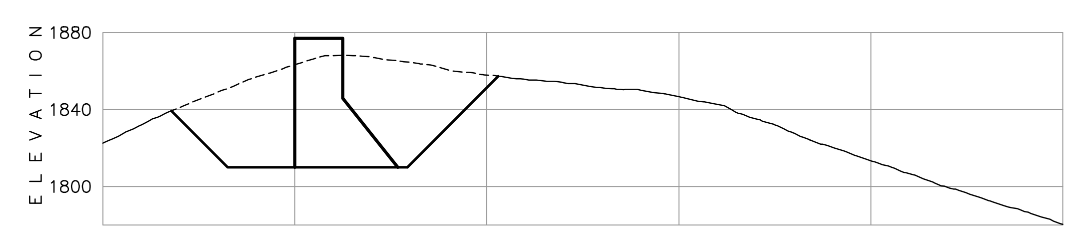
SECTION AT STA 19+00



SECTION AT STA 20+00



SECTION AT STA 21+00



SECTION AT STA 22+00

50' 0 50' 100'
SCALE: 1"= 50'

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES
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CHECKED	CADFILE NID SECTIONS.DWG

Nevada Irrigation District
Grass Valley, California

Centennial Reservoir Project

DAM SECTIONS - 2

PROJECT NO 60519386
REV
SHEET
EXHIBIT 8
_ OF _

Mar 29, 2017 – 12:45pm
J:\NID\Dam and Spillway\NID SECTIONS.dwg

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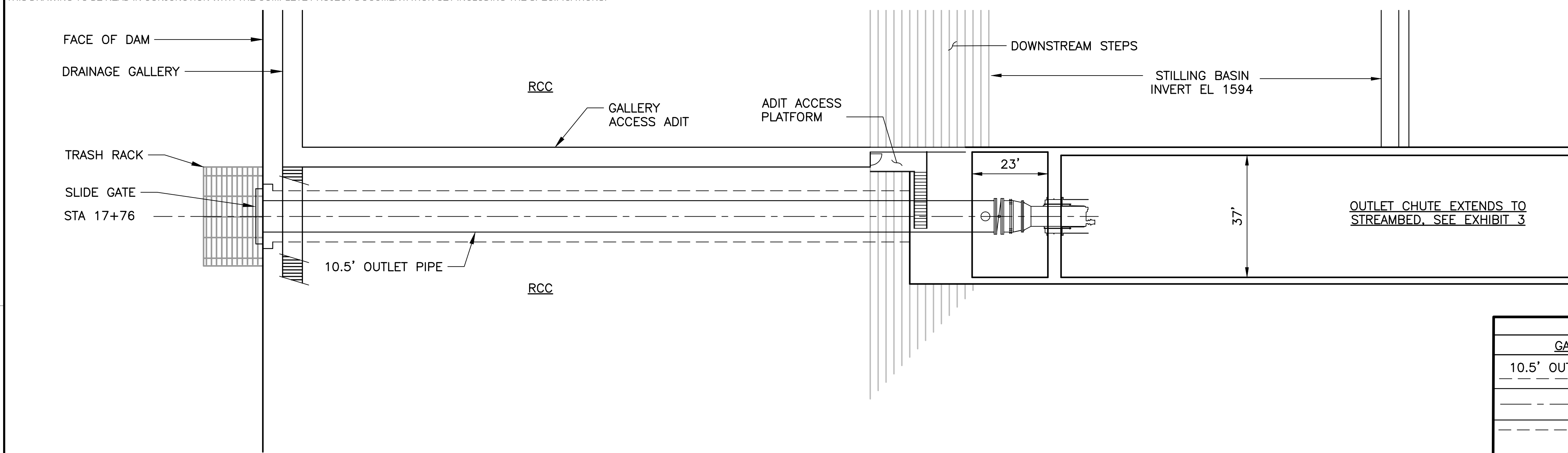
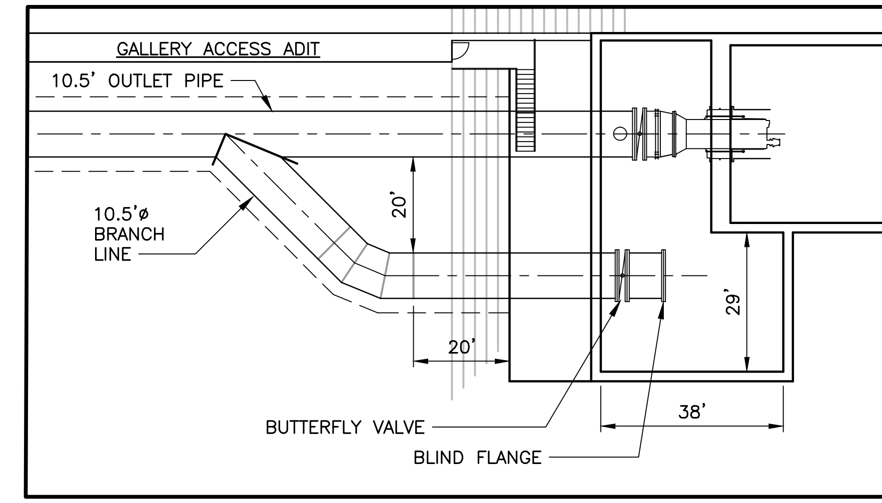


OUTLET WORKS PLAN

20' 0 20' 40'

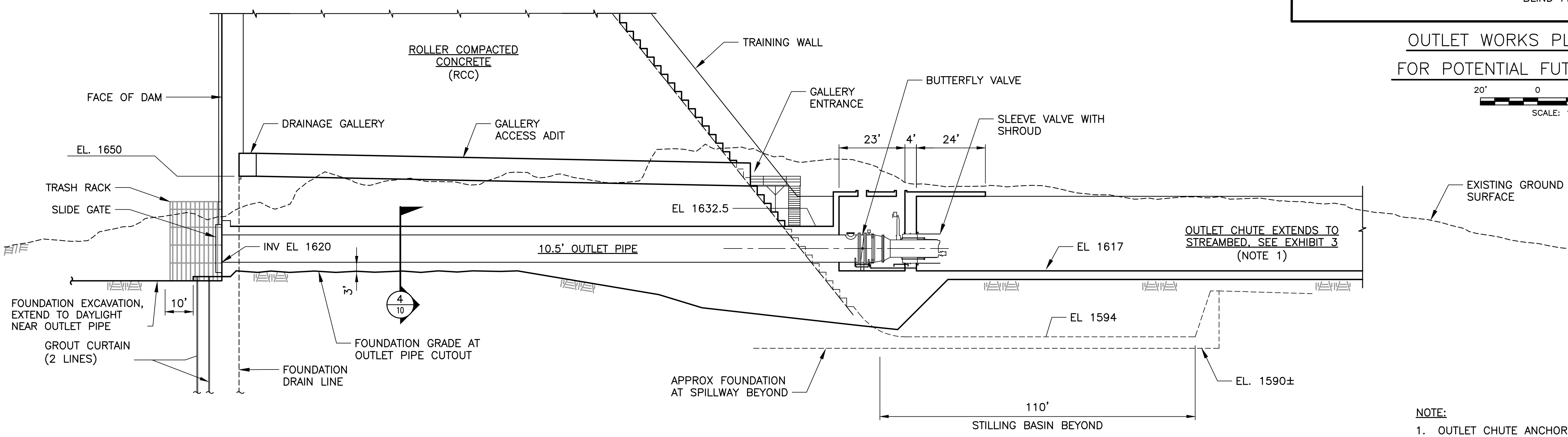
SCALE: 1"= 20'



OUTLET WORKS PLAN WITH CONDUIT FOR POTENTIAL FUTURE POWER PLANT

20' 0 20' 40'

SCALE: 1"= 20'



NOTE:

1. OUTLET CHUTE ANCHORED TO ROCK FOUNDATION.

OUTLET WORKS PROFILE

20' 0 20' 40'

SCALE: 1"= 20'

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES

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DRAWN	SCALE AS NOTED
CHECKED	CADFILE

Nevada Irrigation District

Grass Valley, California

Centennial Reservoir Project

OUTLET WORKS PLAN AND PROFILE

PROJECT NO 60519386

REV

SHEET

EXHIBIT 9

_ OF _

Jul 13, 2017 - 11:19am
C:\000\NID\Dam and Spillway\NID BASE exh no branch pipe.dwg

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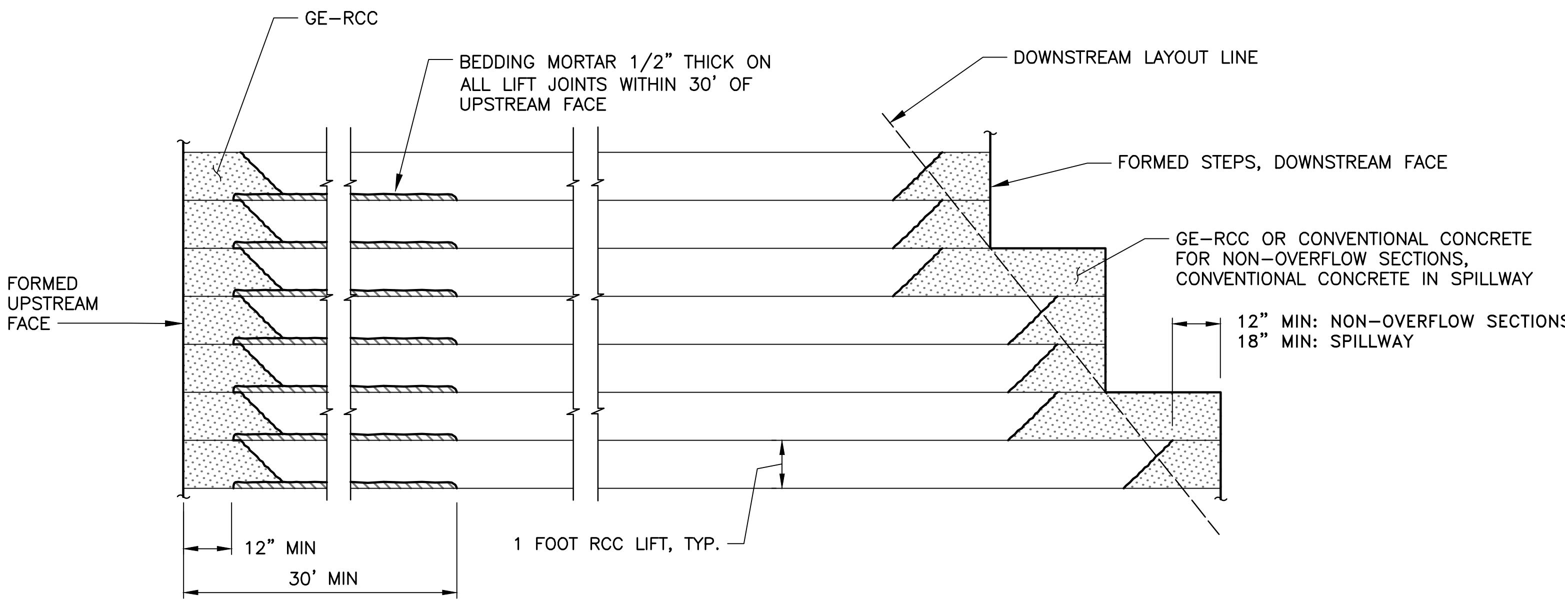
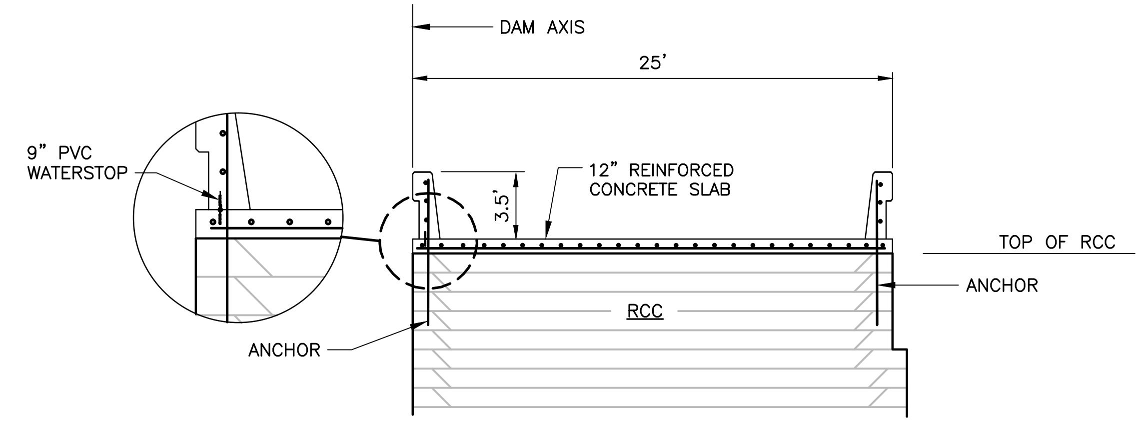
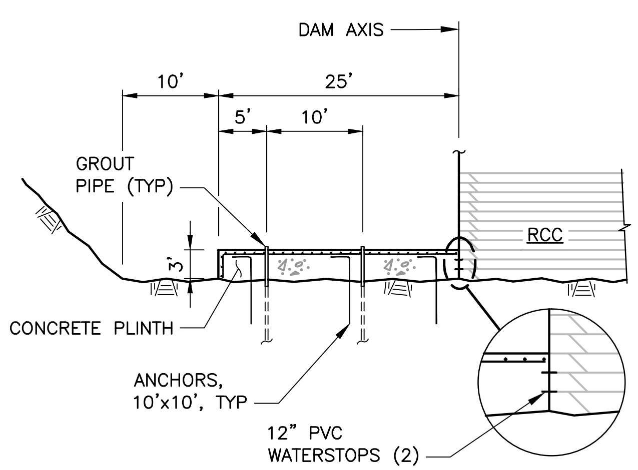


RCC LIFT JOINT AND FACING 1 (5, 6)
NOT TO SCALE

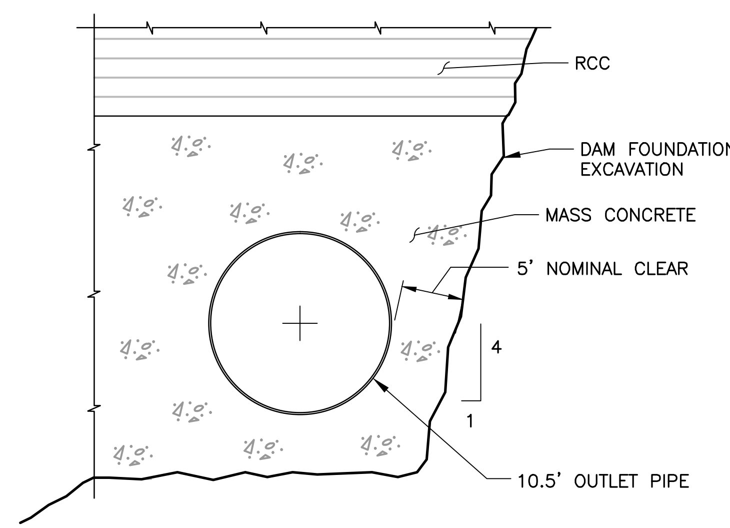


NOTE: SEE SPILLWAY ACCESS BRIDGE PLAN AND ELEVATION (EXHIBIT 11) FOR TRANSITION ACCESS RAMP AND BRIDGE DETAILS.

TOP OF DAM @ NON–OVERFLOW 2 (6)
5' 0 5' 10'
SCALE: 1"= 5'

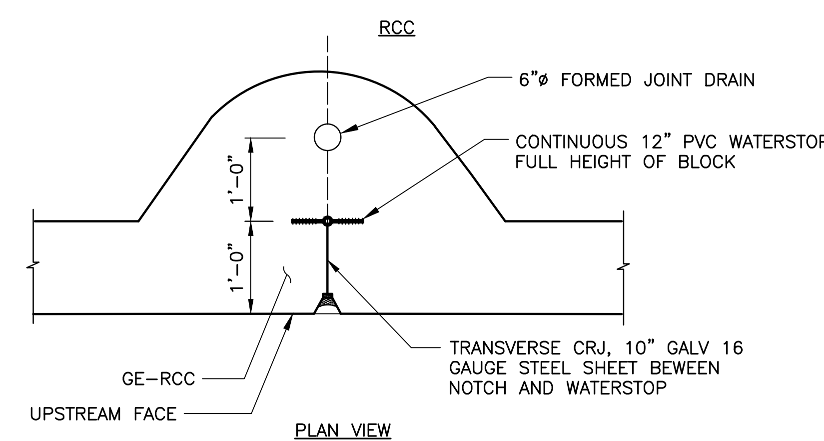


GROUT PLINTH DETAIL 3 (5, 6)
10' 0 10' 20'
SCALE: 1"= 10'



(SEE ALSO EXHIBIT 14)

OUTLET PIPE SECTION 4 (9)
NOT TO SCALE



WATERSTOP DETAIL AT CONTRACTION JOINTS 5 (–)
NOT TO SCALE

REV	DESCRIPTION	APPR	DATE
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CHECKED	CADFILE

Nevada Irrigation District
Grass Valley, California

Centennial Reservoir Project

MAIN DETAILS

PROJECT NO 60519386
REV
SHEET
EXHIBIT 10
_ OF _

Jul 17, 2017 – 4:21pm
J:\NID\Dam and Spillway\NID BASE exh.dwg

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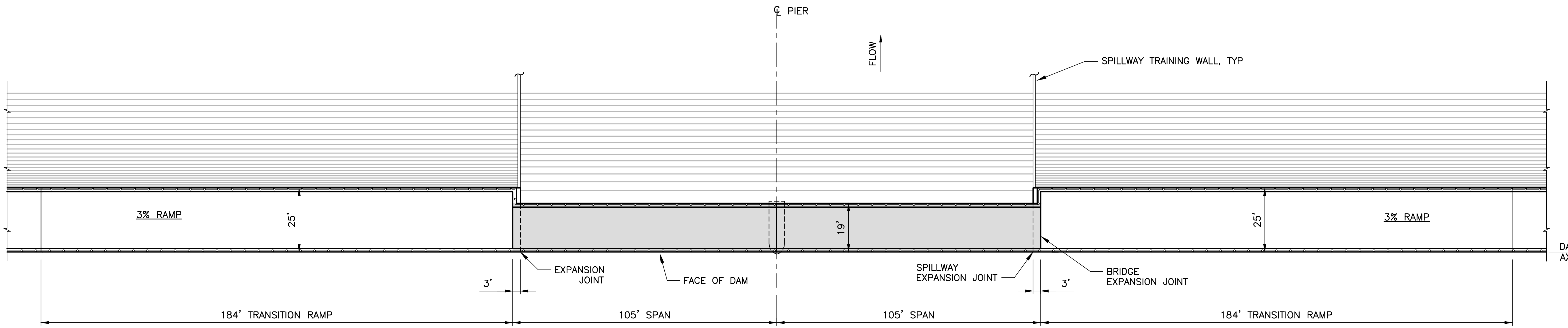
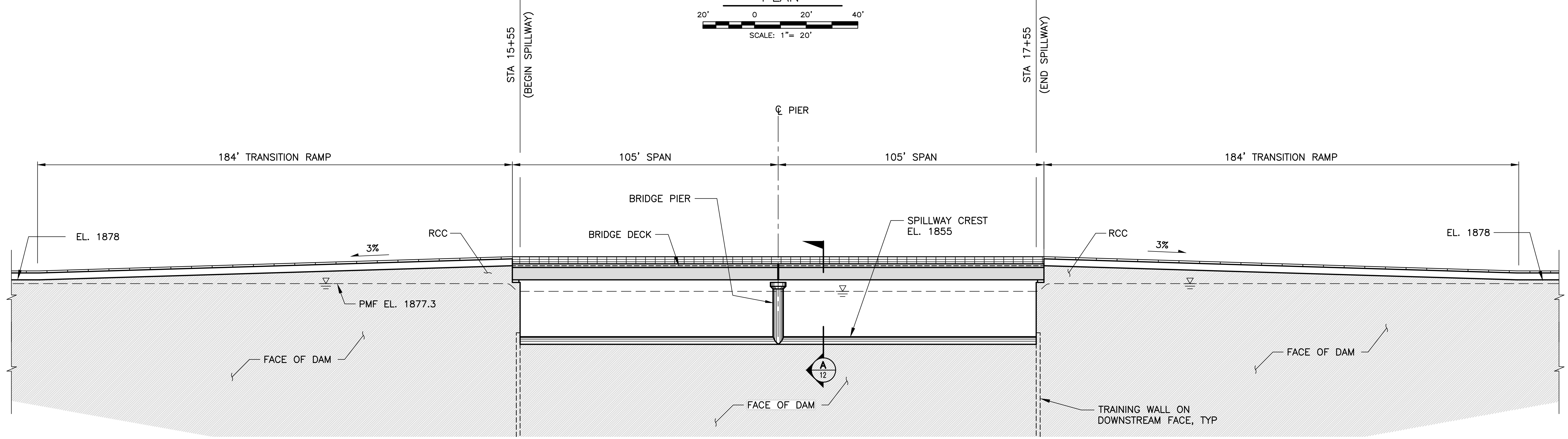


PLAN

20' 0 20' 40'

SCALE: 1"= 20'



ELEVATION

20' 0 20' 40'

SCALE: 1"= 20'

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

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0 1"

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DRAWN	SCALE
	AS NOTED
CHECKED	CADFILE
	NID BASE EXH.DWG

Nevada Irrigation District

Grass Valley, California

Centennial Reservoir Project

SPILLWAY BRIDGE
PLAN AND ELEVATION

PROJECT NO 60519386

REV

SHEET

EXHIBIT 11

_ OF _

Apr 11, 2017 – 11:39am C:\000\NID\Dam and Spillway\NID BASE exh.dwg

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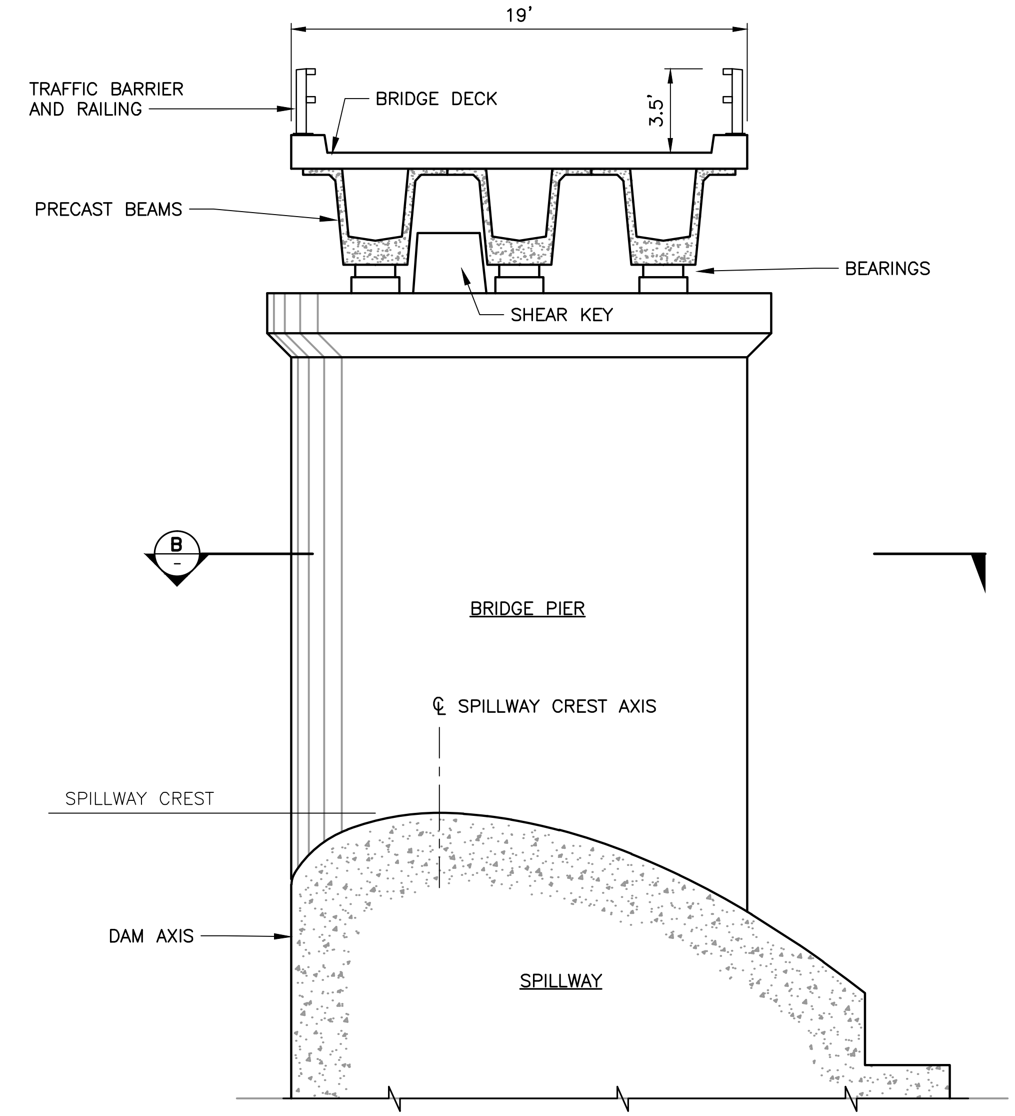
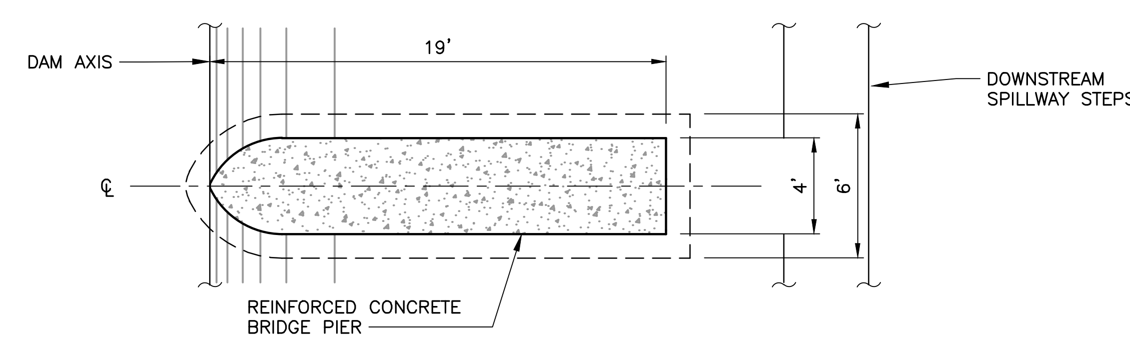


BRIDGE PIER ELEVATION A 11

1' 0 2' 4' 6' 8' 10'

SCALE: 1/4"= 1'-0"



BRIDGE PIER SECTION B –

1' 0 2' 4' 6' 8' 10'

SCALE: 1/4"= 1'-0"

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

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CHECKED	CADFILE NID BASE EXH.DWG

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Grass Valley, California

Centennial Reservoir Project

SPILLWAY BRIDGE SECTIONS

PROJECT NO 60519386

REV

SHEET

EXHIBIT 12

_OF_

Mar 29, 2017 – 3:04pm
J:\NID\Dam and Spillway\NID BASE exh.dwg

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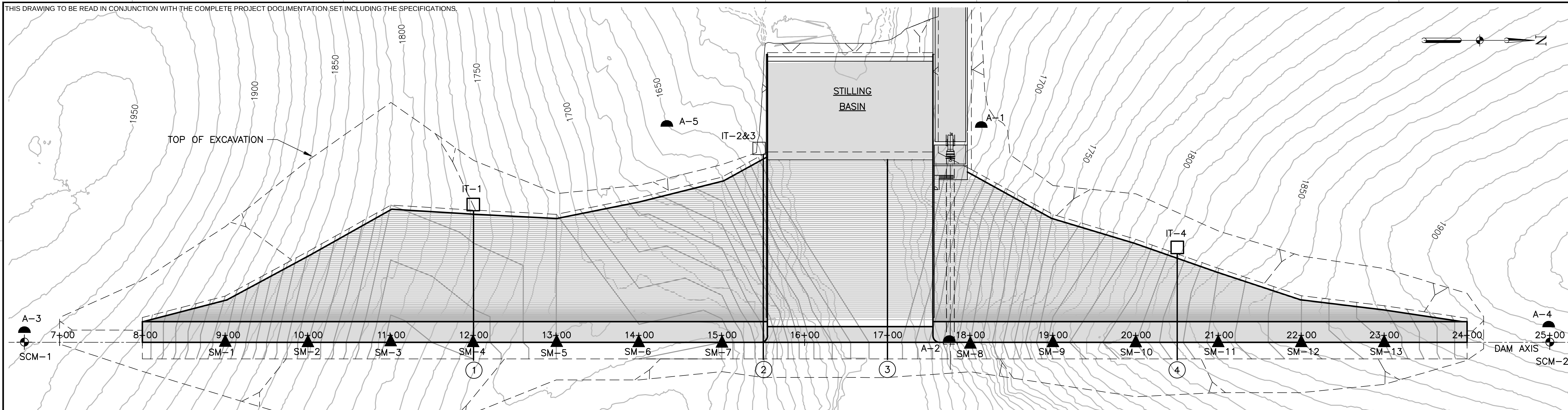
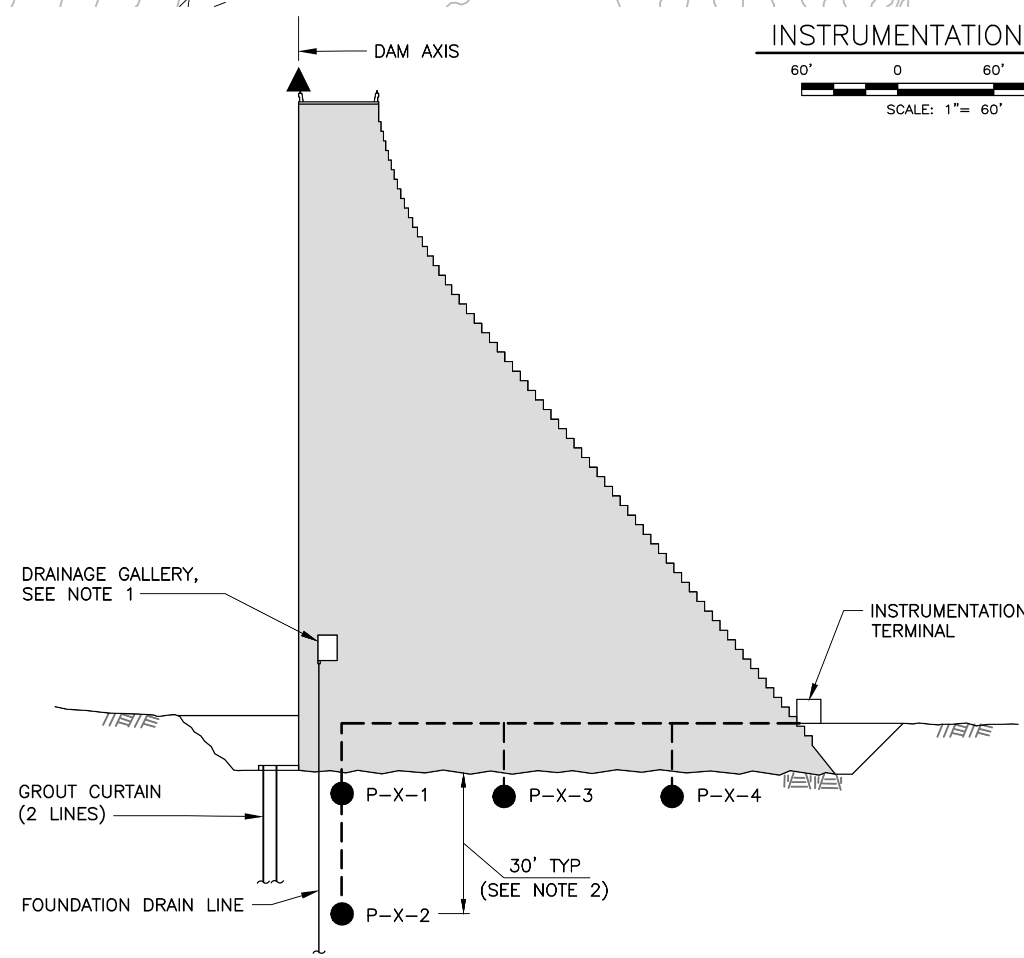


INSTRUMENTATION PLAN

60' 0 60' 120'

SCALE: 1"= 60'



TYPICAL INSTRUMENTATION SECTION

NO SCALE

NOTES:

1. SEEPAGE V-NOTCH WEIRS TO BE LOCATED IN GALLERY GUTTERS.
2. PIEZOMETERS INSTALLED IN DRILLED HOLES.

LEGEND

#	INSTRUMENTATION SECTION No.
SCM-1	SURVEY CONTROL MONUMENT (4 TOTAL, 2 SHOWN)
SM-1	SURVEY MONUMENT
P-X-1	VIBRATING WIRE PIEZOMETER (2 AT EACH LOCATION)
	PIEZOMETER CABLE
A-1	ACCELEROGRAPH
IT-1	INSTRUMENTATION TERMINAL

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

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CHECKED	CADFILE

NID PLANS EXH.NO BRANCH PIPE.DWG

Nevada Irrigation District

Grass Valley, California

Centennial Reservoir Project

INSTRUMENTATION PLAN AND TYPICAL SECTION

PROJECT NO 60519386

REV

SHEET

EXHIBIT 13

_OF _

Jul 05, 2017 – 9:35am
J:\NID\Dam and Spillway\NID PLANS exh.no branch pipe.dwg

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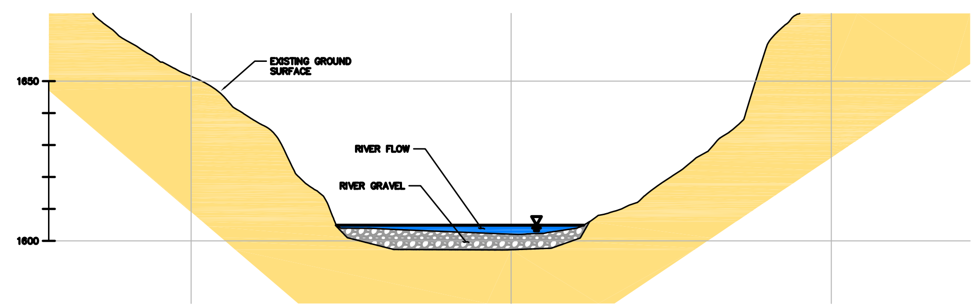
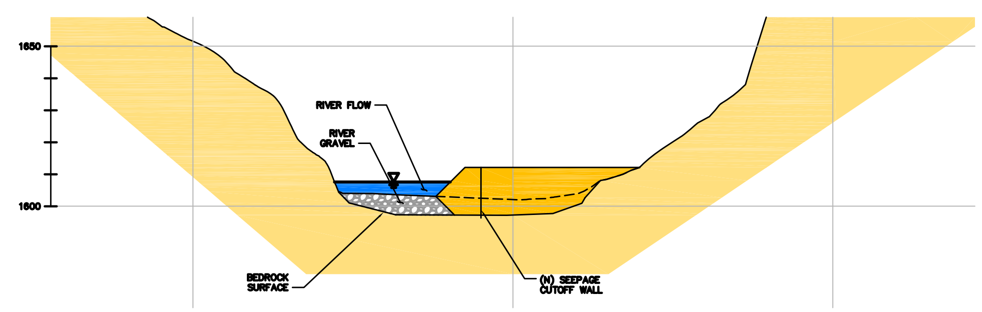
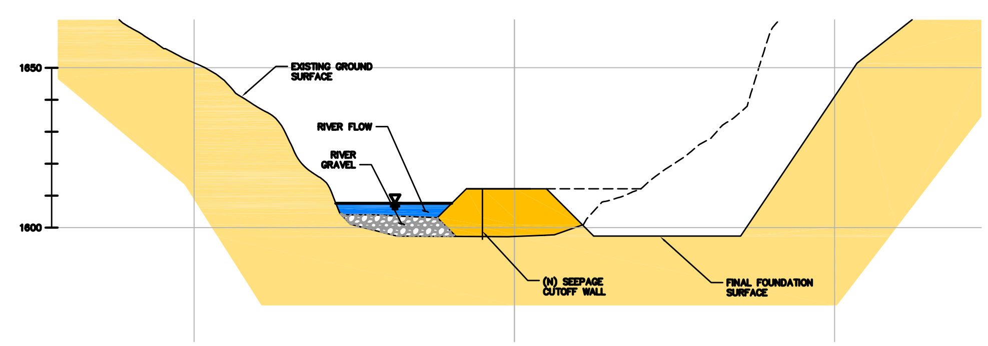


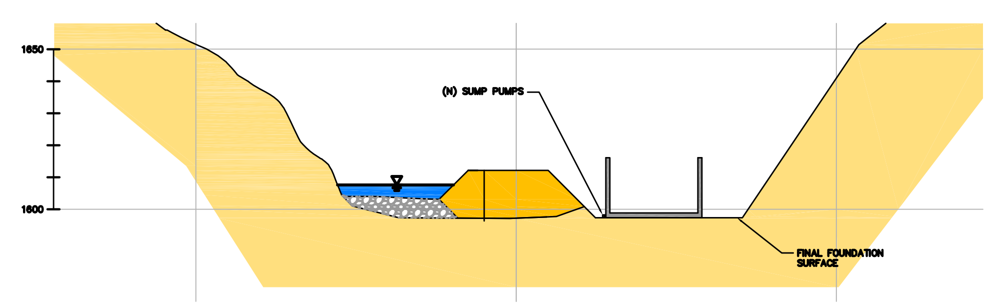
EXISTING CONDITIONS



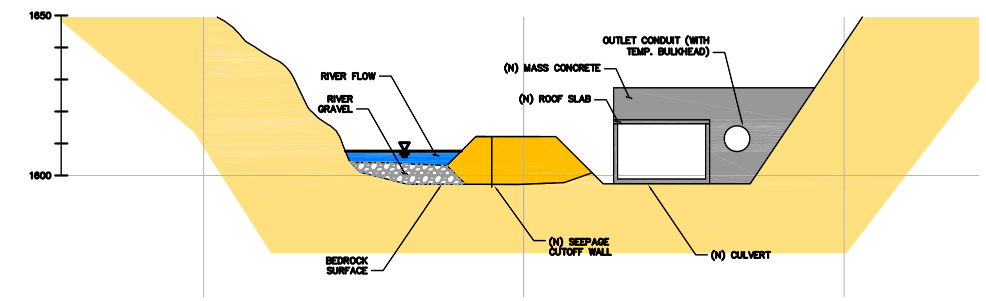
STEP 1 – COFFERDAM CONSTRUCTION/RIVER DIVERSION



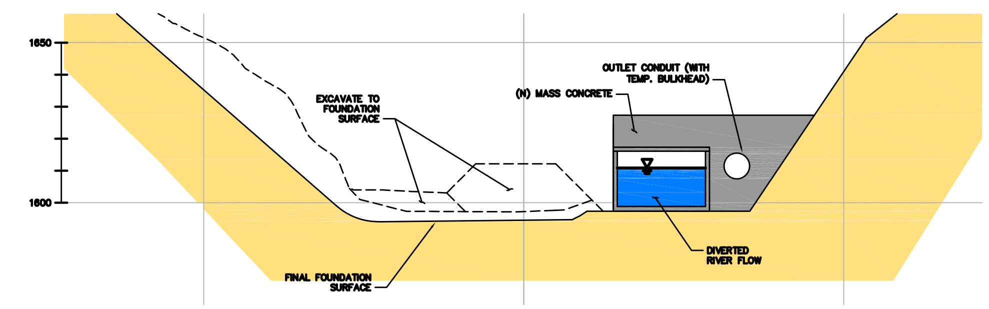
STEP 2 – EXCAVATION FOR BOX CULVERT



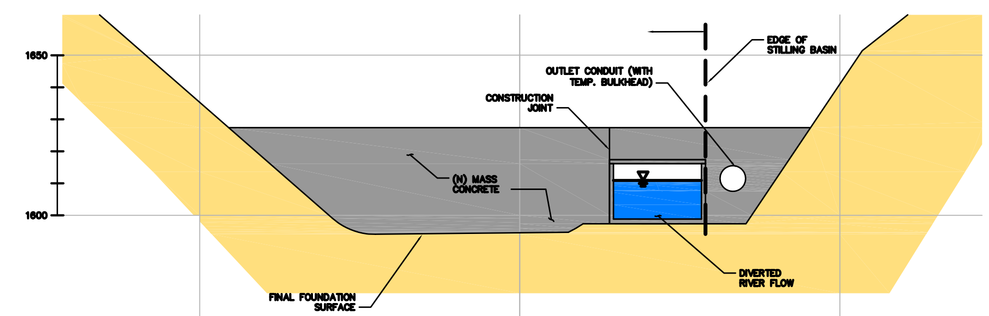
STEP 3 – BOX CULVERT CONSTRUCTION



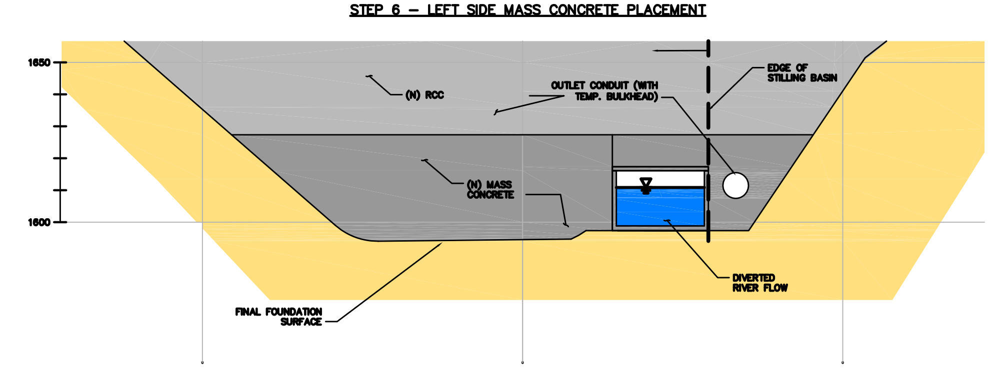
STEP 4 – ROOF SLAB & MASS CONCRETE ENCASEMENT



STEP 5 – RIVER RE-DIVERSION AND FOUNDATION EXCAVATION (COFFERDAM UPSTREAM)



STEP 6 – LEFT SIDE MASS CONCRETE PLACEMENT



STEP 7 – RCC CONSTRUCTION

NOTES:
1. NOT TO SCALE.
2. SECTIONS ARE SHOWN LOOKING DOWNSTREAM.
3. FOUNDATION GROUTING NOT SHOWN.

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Mar 27, 2017 – 10:11am

REV	DESCRIPTION	APPR	DATE
	PRELIMINARY DRAFT NOT FOR CONSTRUCTION		

VERIFY SCALES
0 1"
BAR IS ONE INCH ON ORIGINAL DRAWING IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY

AECOM
300 Lakeside Drive, Suite 400, Oakland, CA 94612
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DESIGN	APPROVED
DRAWN	SCALE AS NOTED
CHECKED	CADFILE EXHIBIT 14.DWG

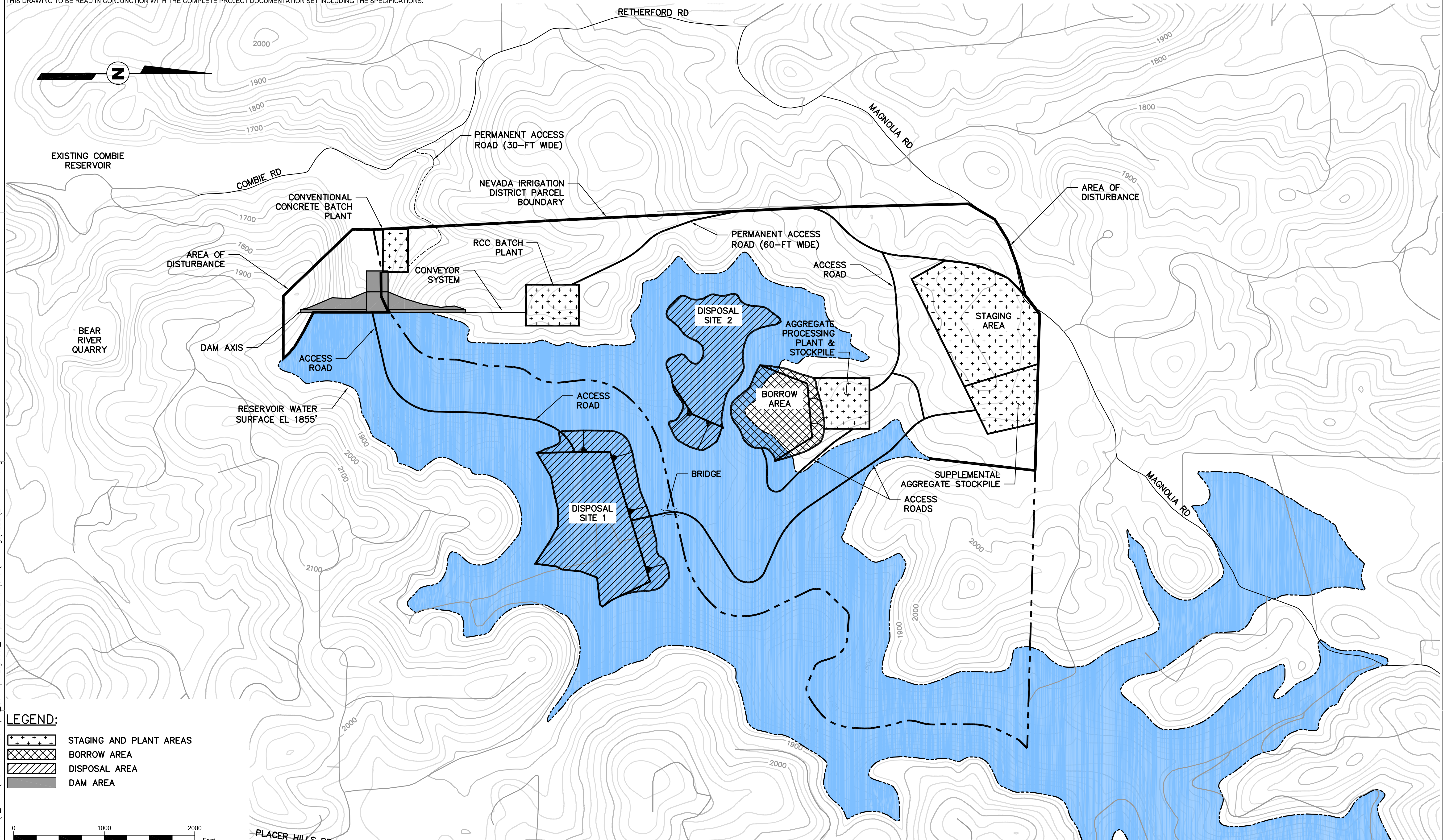
Nevada Irrigation District
Grass Valley, California

Centennial Reservoir Project

DIVERSION CONCEPT

PROJECT NO 60519386
REV
SHEET EXHIBIT 14
_ OF _

THIS DRAWING TO BE READ IN CONJUNCTION WITH THE COMPLETE PROJECT DOCUMENTATION SET INCLUDING THE SPECIFICATIONS.



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CHECKED	CADFILE EXHIBIT 15.DWG

Nevada Irrigation District

Grass Valley, California

Centennial Reservoir Project

CONSTRUCTION SITE LAYOUT
CONCEPT PLAN 1

PROJECT NO 60519386

REV

SHEET

EXHIBIT 15

_ OF_

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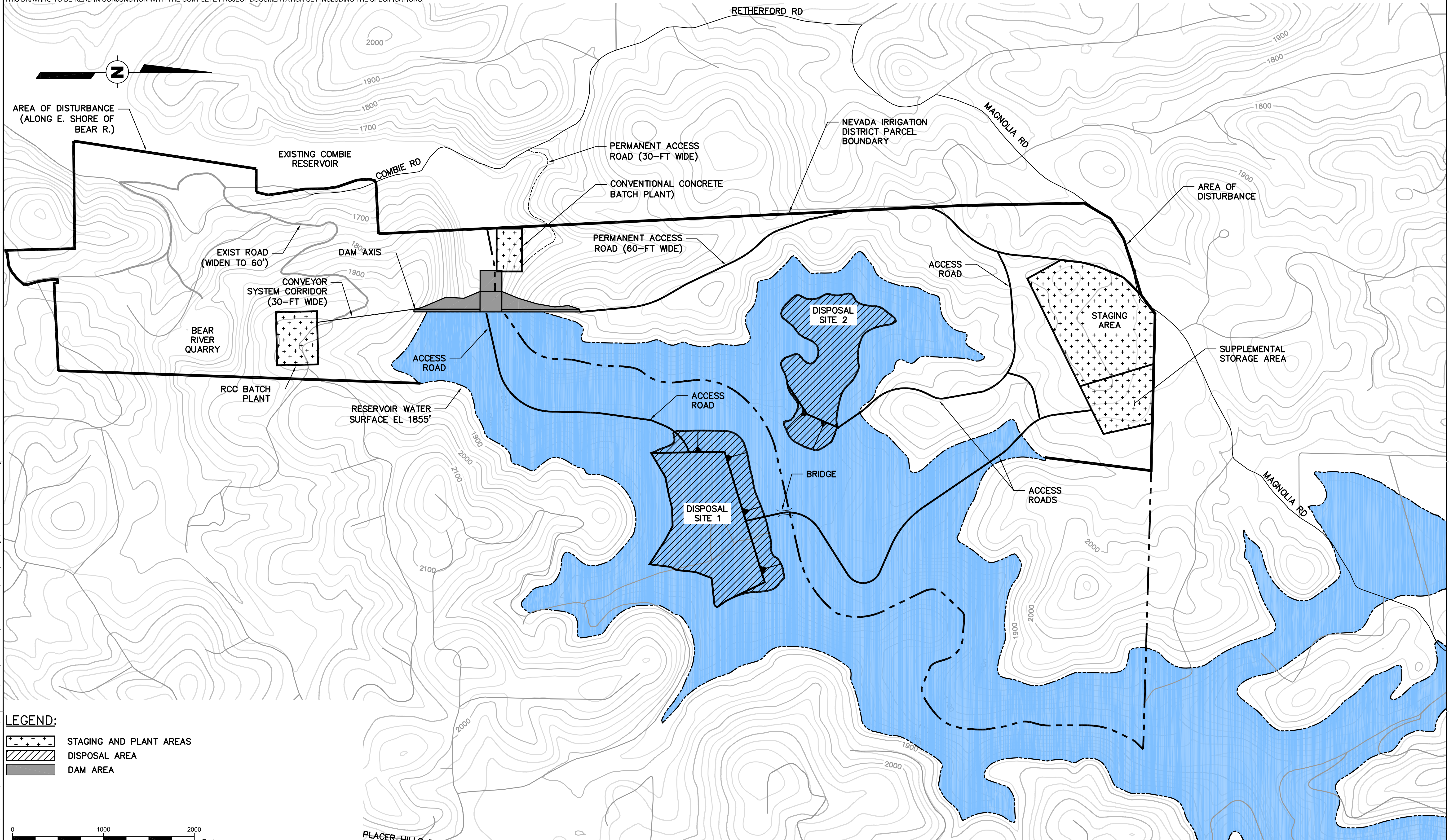

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Tel: (510) 893-3600 Fax: (510) 874-3268

DESIGN	APPROVED
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CHECKED	CADFILE EXHIBIT 16.DWG

Nevada Irrigation District

Grass Valley, California

Centennial Reservoir Project

CONSTRUCTION SITE LAYOUT CONCEPT PLAN 2

PROJECT NO 60519386

REV

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EXHIBIT 16

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Appendix A RCC and CFR Dam Alternatives – Plans and Sections

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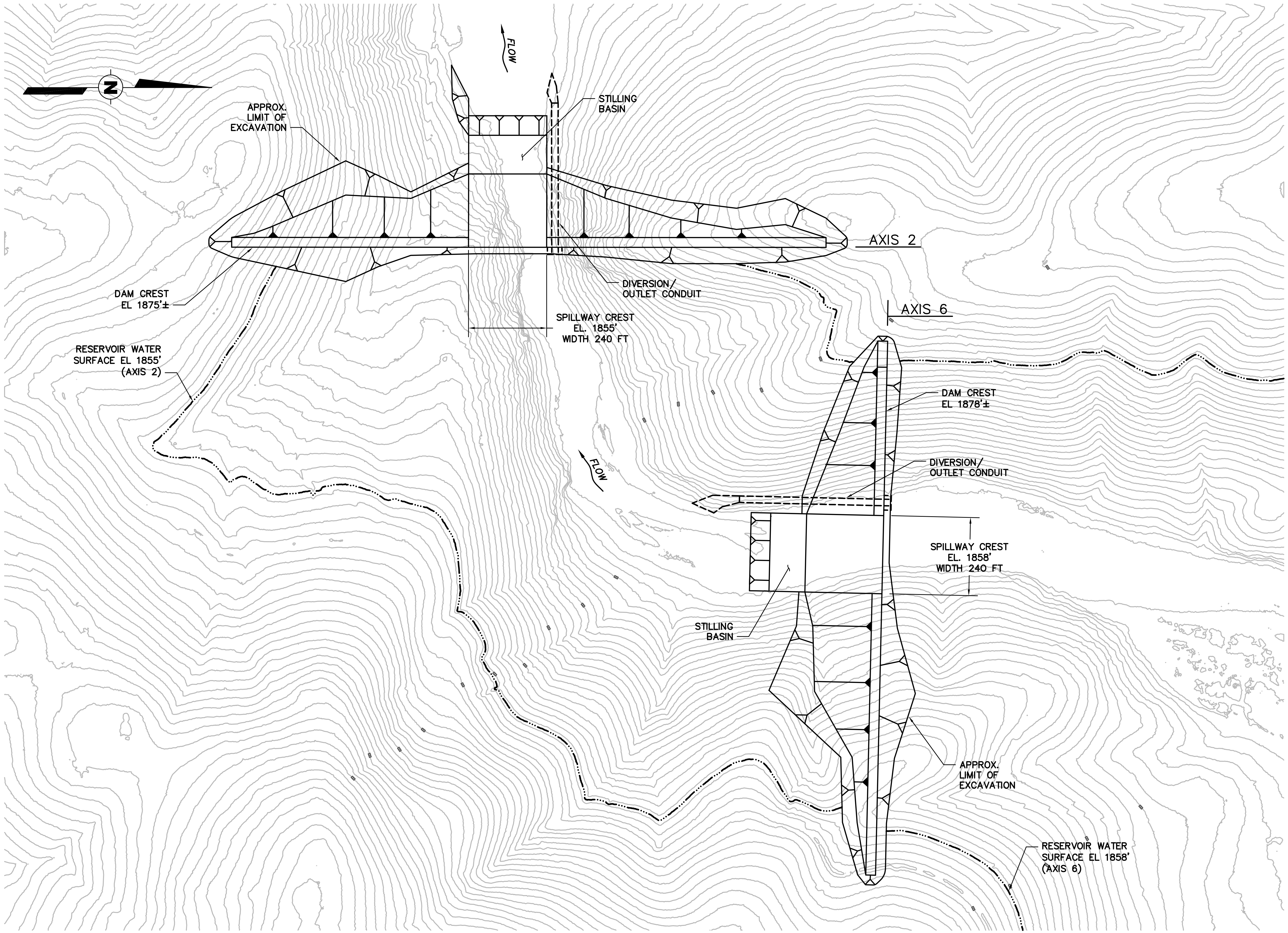

0 300 600 Feet

AECOM

60488086

NEVADA IRRIGATION DISTRICT
CENTENNIAL RESERVOIR PROJECT

Roller Compacted Concrete Dam Concept Plan

FIGURE
A-1

Apr 14, 2017 - 9:54am
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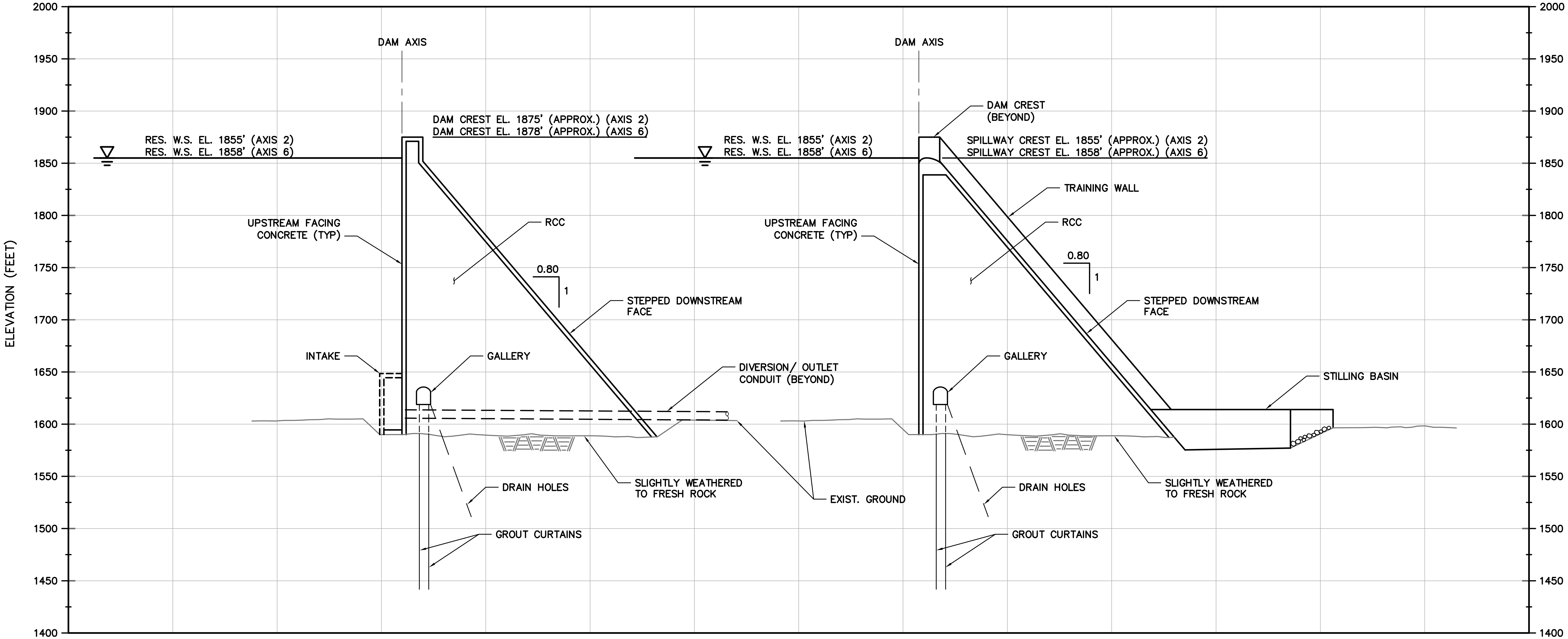

NON–OVERFLOW SECTION

NOT TO SCALE

SPILLWAY SECTION

NOT TO SCALE

Apr 14, 2017 - 10:12am
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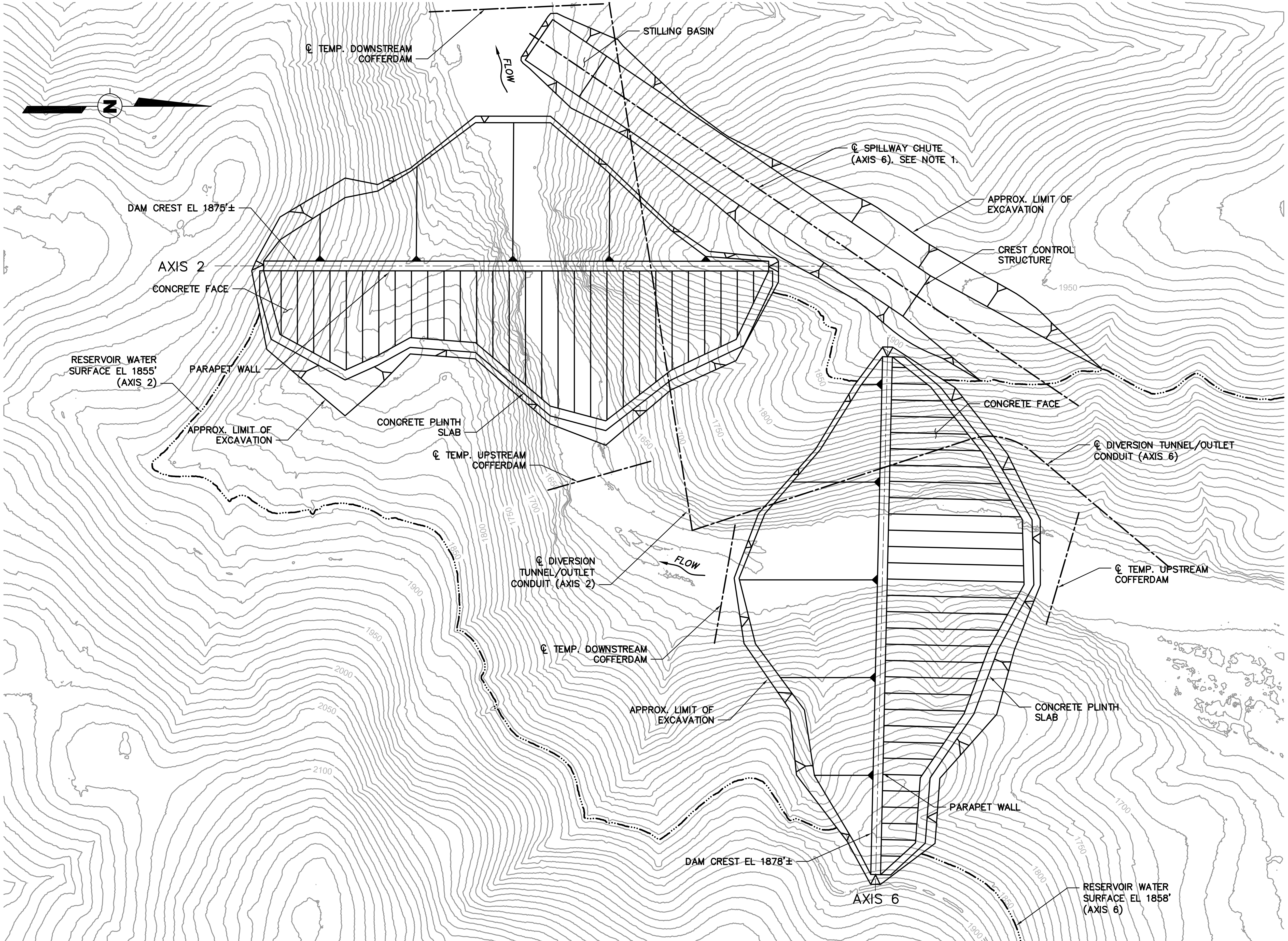
AECOM	60488086 NEVADA IRRIGATION DISTRICT CENTENNIAL RESERVOIR PROJECT	Roller Compacted Concrete Dam Concept Maximum Sections	FIGURE A-2



NOTE:

1. SPILLWAY SIMILAR FOR AXIS 2.

Apr 14, 2017 - 9:56am
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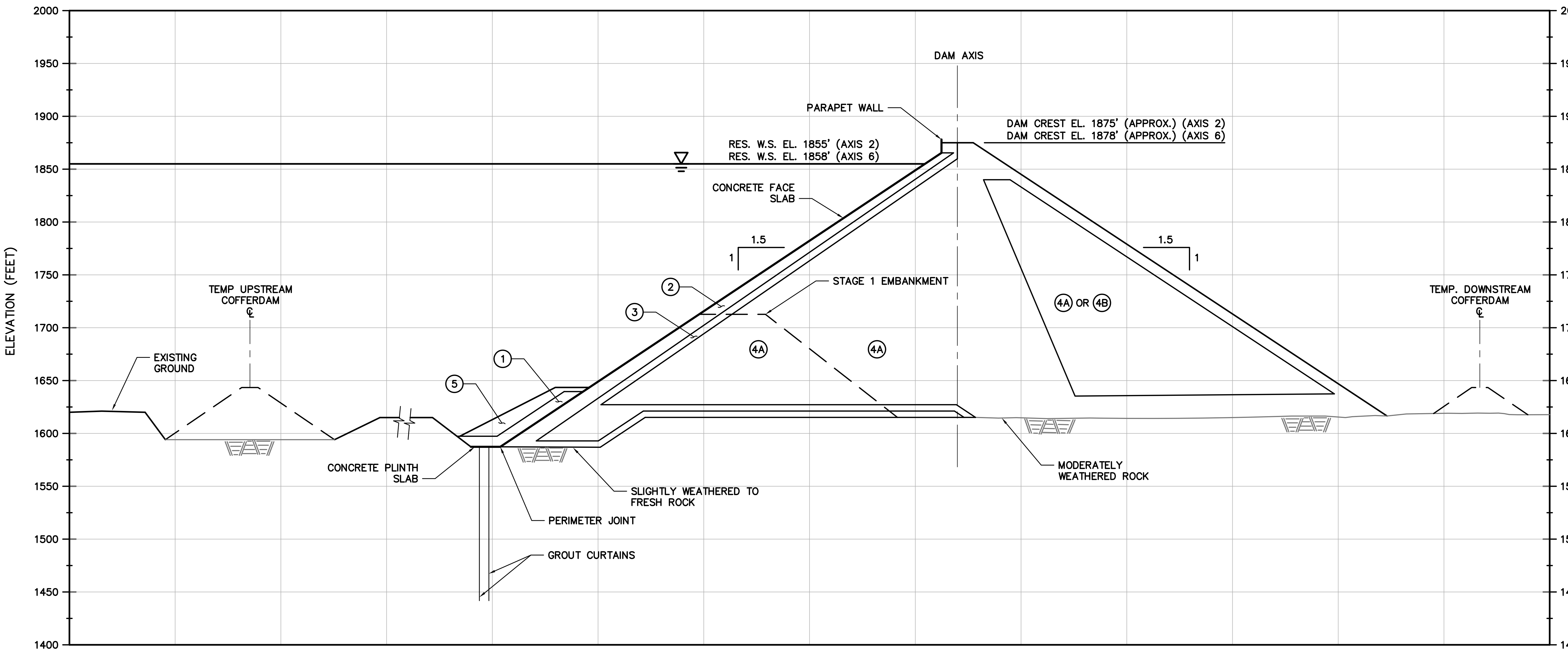
AECOM | 60488086 | NEVADA IRRIGATION DISTRICT CENTENNIAL RESERVOIR PROJECT

Concrete-Faced Rockfill Dam Concept Plan

FIGURE A-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

AECOM	60488086 NEVADA IRRIGATION DISTRICT CENTENNIAL RESERVOIR PROJECT	Concrete Faced Rockfill Dam Concept Maximum Section	FIGURE A-4

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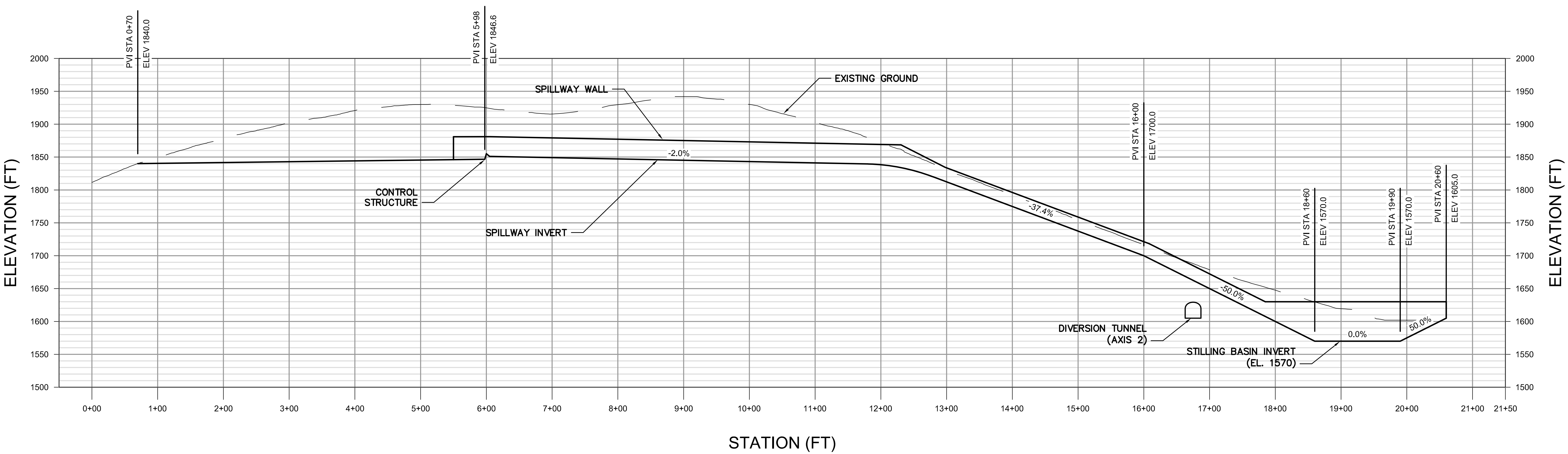

SPILLWAY PROFILE

SCALE 1"=150'

0 150 300 Feet

AECOM

60488086

NEVADA IRRIGATION DISTRICT
CENTENNIAL RESERVOIR PROJECT

CONCRETE FACED ROCKFILL DAM
CONCEPT - SPILLWAY PROFILE

FIGURE
A-5

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Appendix B Foundation Strength and Deformation Characterization

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Appendix B

Centennial Reservoir Project
Foundation Strength and Deformation Characterization

1.0 Introduction

This technical memorandum summarizes the work performed to characterize foundation bedrock strength and small-strain deformation modulus parameters required for conceptual-level analysis and design of a roller-compacted concrete (RCC) dam at Axis 2.

The approach for this characterization consisted of estimating the Geological Strength Index (GSI) for bedrock materials encountered in the vicinity of the estimated foundation depth of the proposed RCC dam at borings CB-1 (left abutment), CB-16/16A (valley bottom), and CB-13 (right abutment). The GSI was estimated for each core run over the depth range of interest, based on the descriptions of the materials provided on the boring logs and core photographs.

Using representative estimates of the GSI, the strength of the bedrock was characterized in accordance with the generalized Hoek-Brown failure criterion (2002). The Hoek-Brown criterion relates the strength of intact rock, quantified by the unconfined compressive strength, to the strength of the jointed rock mass. This relationship is a function of the estimated GSI, disturbance caused by construction activities (e.g., blasting or mechanical excavation) and/or stress relaxation, and material constants. Equivalent Mohr-Coulomb strength parameters were then estimated from the Hoek-Brown failure criterion over the stress range of interest for analysis using the computer program RocLab (Rocscience Inc., 2013), which incorporates the Mohr-Coulomb fitting equations described by Hoek et al. (2002).

The small-strain dynamic modulus was estimated using seismic velocity measurements and elasticity theory.

2.0 Rock Description

Surface and subsurface geological and geotechnical investigations were performed by AECOM at Axis 2 as part of the Phase II and Phase III investigations. Findings from these investigations, along with descriptions of the regional and local geologic setting, are summarized in the Phase III Geotechnical Report (AECOM, 2017).

Rock encountered in the core borings in the vicinity of Axis 2 primarily consists of basalt and basalt breccia with varying degrees of weathering and fracturing. The foundation objective is to found the RCC dam on slightly weathered to fresh, hard rock. It is expected, however, that there will be some localized areas of moderately weathered rock in the foundation. Therefore, the characterization summarized in this technical memorandum was focused on the moderately weathered and slightly weathered basalt and basalt breccia encountered in the vicinity of Axis 2. Fresh bedrock would have higher strength and experience less deformation, for a given load, than moderately or slightly weathered rock.

3.0 Unit Weight

Laboratory unit weights were measured on core samples selected for unconfined compressive strength testing. Results of these measurements are presented in Table 1, organized by degree of weathering and location along the dam axis.

Table 1. Summary of Moist Unit Weight Measurements

Location	Predominant Degree of Weathering	Median Moist Unit Weight (pcf)	Range (pcf)	Number of Tests
Left Abutment	Moderately	146	N/A	1
	Slightly	163	148 – 167	6
	Fresh	167	167 – 167	2
Channel	Moderately	N/A	N/A	N/A
	Slightly	170	147 – 173	3
	Fresh	N/A	N/A	N/A
Right Abutment	Moderately	149	147 – 150	2
	Slightly	164	140 – 171	10
	Fresh	N/A	N/A	N/A

For preliminary analyses, unit weights of 147 pcf and 165 pcf are considered appropriate for moderately and slightly weathered bedrock, respectively.

4.0 Summary of Unconfined Compressive Strength Data

Unconfined compressive strength (UCS) laboratory tests were performed on selected core samples. The test results are summarized by degree of weathering and location along the dam axis in Table 2.

Table 2. Summary of UCS Tests on Axis 2 Core Samples

Location	Predominant Degree of Weathering	Median UCS (psi)	Range of UCS (psi)	Number of Tests
Left Abutment	Moderately	2,700	N/A	1
	Slightly	9,900	1,800 – 24,100	7
	Fresh	10,950	9,350 – 12,550	2
Channel	Moderately	N/A	N/A	N/A
	Slightly	11,450	5,100 – 21,850	7
	Fresh	12,450	9,630 – 15,280	2
Right Abutment	Moderately	3,850	1,900 – 5,850	2
	Slightly	17,650	6,450 – 35,650	10
	Fresh	N/A	N/A	N/A

Point load tests were also performed on selected samples from the core borings. Estimated UCS from the point load tests are summarized by degree of weathering and location along the dam axis in Table 3.

Table 3. Estimated UCS from Point Load Tests on Axis 2 Core Samples

Location	Predominant Degree of Weathering	Median UCS (psi)	Range of UCS (psi)	Number of Tests
Left Abutment	Moderately	2,150	950 – 2,200	3
	Slightly	20,250	14,850 – 24,350	8
	Fresh	24,700	24,500 – 32,150	3
Channel	Moderately	N/A	N/A	N/A
	Slightly	19,050	13,800 – 22,850	8
	Fresh	N/A	N/A	N/A
Right Abutment	Moderately	3,800	2,000 – 16,950	8
	Slightly	28,950	14,350 – 39,800	12
	Fresh	N/A	N/A	N/A

The trends in results from the UCS and point load tests with degree of weathering are generally consistent. Results from the UCS lab tests were used for the strength characterization summarized in this technical memorandum.

5.0 Geological Strength Index (GSI) Characterization

The GSI was estimated for bedrock materials encountered in the vicinity of the estimated RCC dam foundation depth at borings CB-1, CB-16/16A, and CB-13. Boring locations are indicated on Figure 3-2 in the CER. Results from the GSI characterization are summarized in Table 4 and Figures B-1 through B-3.

Table 4. Estimated GSIs at borings CB-1, CB-16/16A, and CB-13

Boring ID	Location	Foundation Depth*	Depth Range of Interest* (ft)	Predominant Degree of Weathering	Median GSI	25th Percentile GSI	Range of GSI	Number of Occurrences
CB-1	Left Abutment	133	99 - 153	Moderately	35	N/A	N/A	1
				Slightly	50	36	30 – 55	13
				Fresh	78	N/A	75 – 80	2
CB-16/16A	Channel	13	11 - 53	Moderately	N/A	N/A	N/A	0
				Slightly	53	40	15 – 70	14
				Fresh	N/A	N/A	N/A	0
CB-13	Right Abutment	97	83 – 143	Moderately	48	40	40 – 50	6
				Slightly	60	49	40 – 70	10
				Fresh	78	N/A	75 – 80	2

*Depths are measured along the lengths of angled borings.

6.0 Equivalent Mohr-Coulomb Strength Parameters

Using the computer program RocLab, the shear strength of the rock mass was characterized in accordance with the Hoek-Brown failure criterion. Equivalent Mohr-Coulomb shear strength parameters were then estimated from the Hoek-Brown failure criterion over the stress ranges of interest for analysis. For these calculations, the intact material constant, m_i, was assumed to be 25 (Marinos and Hoek, 2003) and the disturbance factor, D, was assumed to be zero (indicating no disturbance due to foundation excavation).

Conceptual-level engineering analyses will be performed to evaluate sliding and overturning stability of the proposed RCC dam at stations 15+50, 17+00, and 20+50, along the Axis 2 alignment. Table 5 summarizes the input parameters and the estimated Mohr-Coulomb shear strength parameters for analyses at these locations. Confining stresses for the estimation of the equivalent Mohr-Coulomb parameters were estimated using an assumed unit weight for the RCC of 150 pcf, the estimated foundation elevations, and the assumption of a full reservoir (i.e., lake level at elev. 1855 feet).

The GSI values selected to estimate strength parameters at Stations 17+00 and 20+50 correspond to the 25th percentile GSI for the corresponding representative borings. This is considered appropriately conservative for conceptual-level analyses.

GSI analyses have not been performed for borings CB-2 and CB-17, but the rock encountered in these borings was generally good quality, slightly weathered to fresh. Therefore, the median GSI for the slightly weathered to fresh rock encountered at boring CB-1 is judged to be a conservative estimate of the GSI for the estimate of foundation rock strength parameters at Station 15+50.

Table 5. Conceptual – Level Estimates of Equivalent Mohr-Coulomb Shear Strength Parameters for Foundation Rock at Stations 15+50, 17+00, and 20+50 with D=0.

Station	Degree of Weathering for Strength Estimates	Representative Boring	Median Unconfined Compressive Strength (psi)	Selected GSI*	D	Confining Stress (psi)	Cohesion (psi)	Friction Angle (degrees)
15+50	Slightly Weathered to Fresh	CB-1	9,900	50	0	130	100	60
17+00	Slightly Weathered	CB-16/16A	11,450	40	0	180	100	57
20+50	Moderately	CB-13	3,850	40	0	80	40	55

* The GSI values selected for stations 17+00 and 20+50 correspond to the 25th percentile values for the indicated degree of weathering (Table 4). The median GSI from boring CB-1 has been applied for conceptual-level estimate of foundation rock strength at Station 15+50.

7.0 Deformation Modulus

Based on measured P-wave and S-wave velocities using seismic refraction and downhole suspension logging techniques, the dynamic, small-strain modulus was estimated. This is summarized in Table 6.

Table 6. Dynamic, Small-Strain Modulus Estimates

P-Wave Velocity (ft/s)	Poisson's Ratio	ED (psi)	Comment
9,000	0.33	1.9×10^6	Based on seismic refraction measurements and corresponding boring logs, NorCal estimated V_p = 9,000 ft/s as the approx. boundary between moderately weathered/fractured rock and slightly weathered/fractured rock.
20,000	0.33	9.2×10^6	Downhole suspension logging measurements indicated consistent V_p = 20,000 ft/s below depths of about 35 feet at boring CB-2. Shallow slightly-weathered to fresh rock with relatively high RQD values were observed at this location.

Based on the calculated values and comments presented in Table 6, the small-strain modulus for a representative thickness of foundation bedrock beneath the dam (on the order of one to two times the width of the dam) is estimated to be on the order of 9×10^6 psi. This estimate is considered reasonable for conceptual-level analyses.

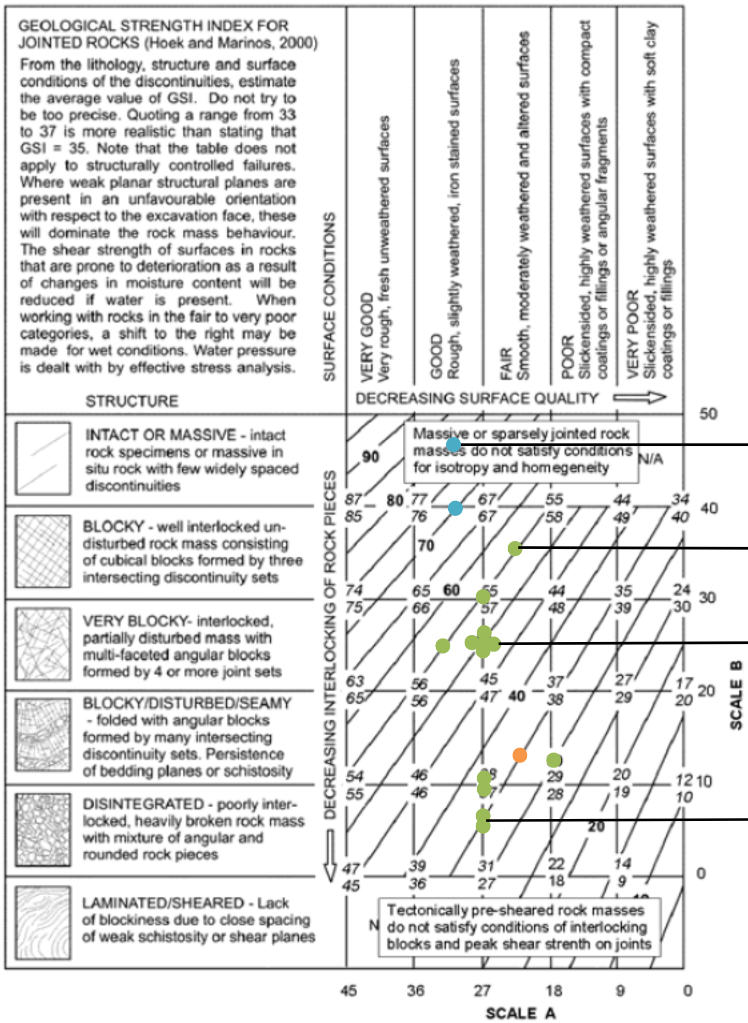
8.0 References

AECOM, 2017. Geotechnical Engineering Report, Phase III – Final, August 30.

Hoek, E., Carranza-Torres, C.T., and Corkum, B., 2002. Hoek-Brown Failure Criterion – 2002 Edition. Proceedings of the Fifth North American Rock Mechanics Symposium, Toronto, Canada, Vol. 1, Pages 267-273.

Marinos, P., and Hoek, E., 2002. Estimating the Geotechnical Properties of Heterogeneous Rock Masses such as Flysch. Bulletin of Engineering Geology and the Environment (IAEG). Vol. 60, Pages 85-92.

Rocscience Inc., 2013. RocLab, Version 1.033 – Rock Mass Strength Analysis Using the Hoek-Brown Failure Criterion, User's Guide. Toronto, Canada.



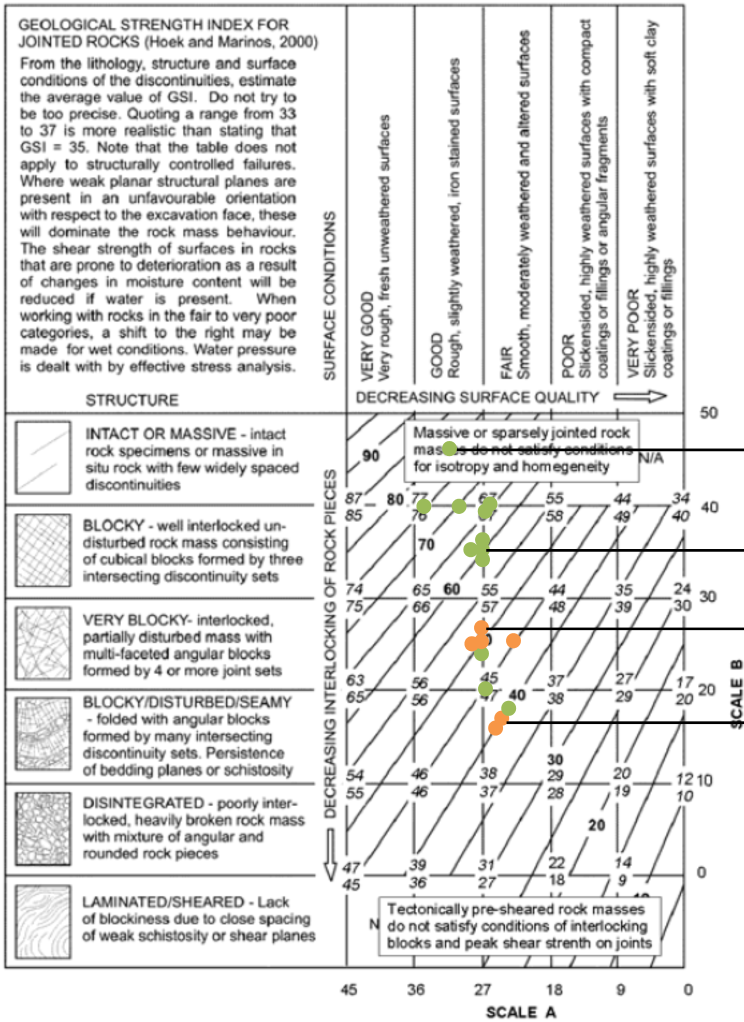

Run 53: 147.5 - 152.5 feet


Run 39: 105 – 109 feet


Run 42: 117.5 – 120.5 feet


Run 46: 131 – 133 feet

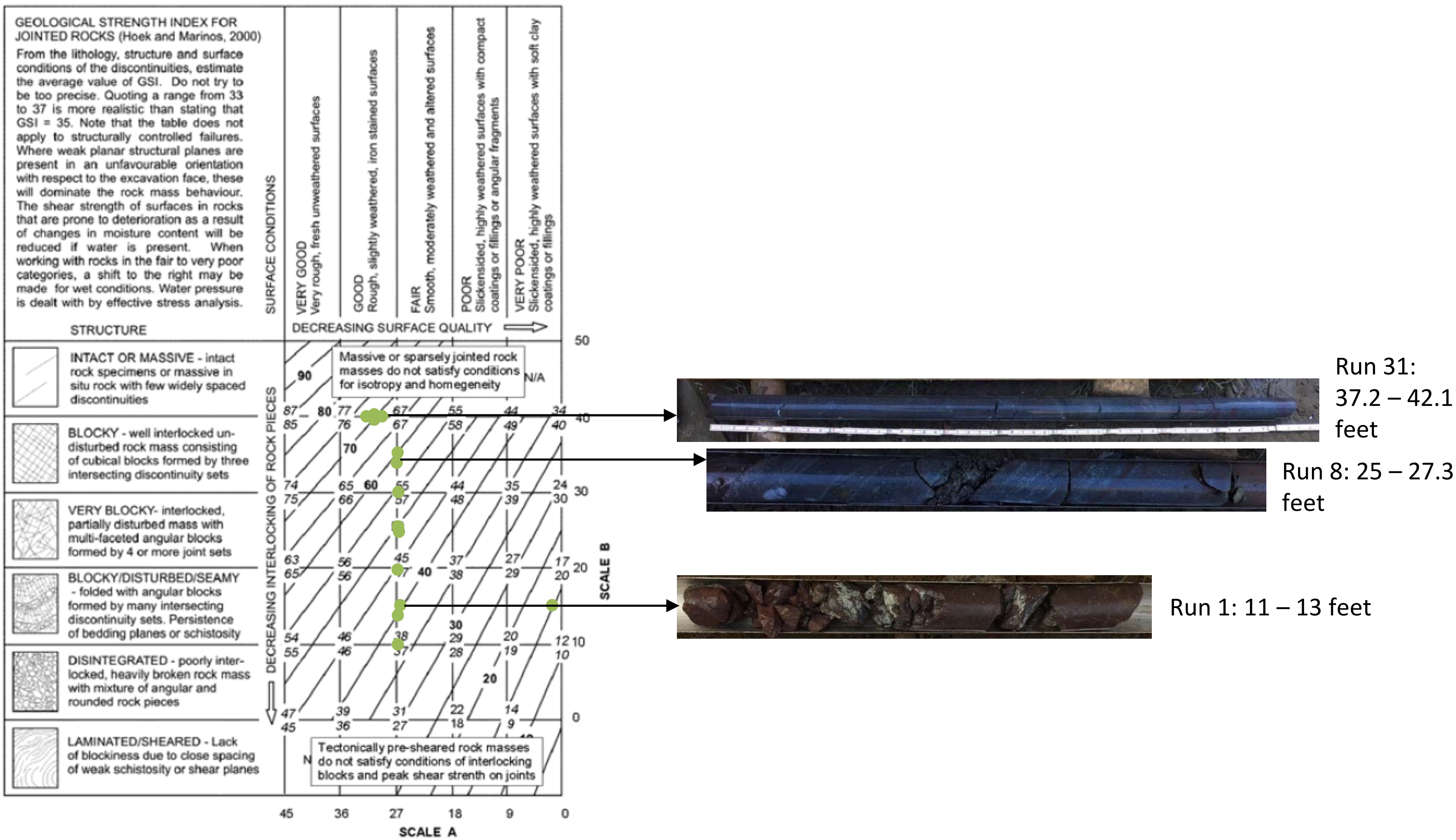
Project No. 60503855	NID Centennial Reservoir Project	GSI Characterization Boring CB-1 Depth Range: 99 – 153 feet	Figure B-1



Run 44: 127.5 – 131 feet

Run 31: 91.5 – 95.5 feet

Project No. 60503855	NID Centennial Reservoir Project	GSI Characterization Boring CB-13 Depth Range: 83 – 143 feet	Figure B-2
AECOM			



Project No. 60503855	NID Centennial Reservoir Project	GSI Characterization Boring CB-16/16A Depth Range: 11 – 53 feet	Figure B-3
	AECOM		

Appendix C Dam Stability Analyses

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Appendix C

Centennial Reservoir Project
Dam Stability Analyses

1.0 Introduction

This technical memorandum summarizes the results of the preliminary static and pseudo-static (dynamic) analyses of the conceptual configuration of the RCC dam as shown in Exhibits 5 and 6 in the CER. The analyses were undertaken to demonstrate that the configuration of the dam spillway and abutment sections meets minimum criteria for normal (usual), flooding and earthquake loading (unusual and extreme). The analyses also allow the stability implication of the variability in foundation rock conditions both with respect to depth of excavation and laterally across the site.

The maximum spillway section (Sta 17+00, Exhibit 5) and two abutment monoliths (Sta 15+50 and 20+50, Exhibit 6) were evaluated for moment equilibrium, sliding stability, and overstressing using 2-dimensional, limit-equilibrium analyses. The method uses basic limit equilibrium equations to resolve the forces and moments acting on the structure and assumes that the normal stresses along any horizontal plane are linearly distributed.

Limit equilibrium analyses do not account for the deformations required to mobilize various types of resisting forces; they only consider balancing forces to maintain equilibrium. The method therefore has to assume the resisting shear is at its limit state and applies a FS to these strengths to show that that state will not develop. For preliminary design, limit equilibrium analyses are adequate to evaluate static loading conditions and for pseudo-static earthquake loading. During preliminary design, a finite element model will be developed for the dynamic analyses (frequency or time domain) under MCE loading. From these analyses, detailed static stress plots will be developed to confirm the limit equilibrium analysis results.

The minimum allowable factors of safety for moment equilibrium (resultant location) and sliding are presented in Table 1 (AECOM, 2017a).

Table 1. Stability Criteria

Load Case	Comment	Reservoir Water Surface Elev.	Location of Resultant at Base	Factor of Safety for Sliding [1]
Usual	1:10 AEP [3]	At spillway crest[4]	100% of Base in Compression	2.0
Unusual	1:300 AEP	At 1:300 flood level	75% of Base in Compression	1.5
Unusual	Drains inoperable	At spillway crest	75% of Base in Compression	1.5
Extreme	PMF	At PMF level	Resultant Within Base	1.1
Extreme	MCE [2]	At spillway crest	Resultant Within Base	1.1

Notes:
[1] Site information definition in USACE EM 1110-2-2100, Section 3-4. For a new dam, "Ordinary" Category applies.
[2] See USACE EM 1110-2-2100, subsection 3.11 b.
[3] AEP = annual exceedance probability.
[4] For the Usual Load Case, the reservoir level will be taken at the spillway crest instead of the 1:10 AEP flood level.

2.0 Loading Conditions and Foundation Shear Strength

The stability analyses considered the following loads: (a) weight of RCC and concrete, (b) reservoir water, (c) uplift, (d) tailwater, and (e) seismic (inertial and hydrodynamic) (AECOM, 2017a). Sediment loading against the dam will not be considered as the upstream Rollins Reservoir would prevent most of the sediment from entering Centennial Reservoir. Ice loading in the reservoir also will not be considered as sustained freezing temperatures are not expected at the dam site.

2.1 Uplift

Uplift pressure distribution beneath the dam, for a given drain efficiency (E), determined using the USACE procedure, assumes that uplift pressures vary linearly between the reservoir head at the heel (or tip of the tensile zone in a cracked base analysis), to a reduced pressure at the line of pressure relief wells, and the full tailwater head at the dam toe (or downstream edge of the stilling basin slab). If tensile stresses normal to the base are predicted, a crack is assumed to form and a cracked base analysis is run. In a cracked base analysis, the full reservoir head is applied along the length of the crack and the limit equilibrium analysis repeated until the calculated crack length stabilizes. If the crack reaches the drain line, the procedure conservatively assumes that the drain efficiency is lost; i.e. E = 0. For monoliths where no foundation drainage is provided, uplift pressures will be based on a linear dissipation between the reservoir and the downstream toe. Drain efficiency (E) of 67% was used for the analyses and sensitivity of results checked for an E of 50%.

2.2 Tailwater

The PMF surcharge is 22 feet (8% of the structural height of the spillway) due to the effective width of the spillway (200 feet). With the preliminary stepped design of the spillway it is anticipated that the majority of energy dissipation will occur within the spillway chute. As such, the depth of tailwater in the stilling basin will be similar to the spillway surcharge depth. Such a small tailwater depth (relative to the spillway height) will have minimal influence on sliding stability.

A PMF tailwater depth of 30 feet was assumed; this will be verified during a later phase of design. The effective tailwater depth for lateral load calculations was set equal to 60 percent of the full tailwater depth in accordance with USACE and FERC guidelines. There is no tailwater for usual load conditions.

2.3 Foundation Shear Strength

The sliding stability of spillway and abutment monoliths is controlled either by the sliding potential on either:

1. A continuous or semi-continuous sub-horizontal weak plane in the foundation, or
2. Through the rock mass itself.

Sliding along the foundation interface is resisted by the roughness of the prepared foundation that will force a potential sliding failure surface through the weaker of the two interface materials. The dam/foundation interface will be inspected during construction and is not typically a critical failure scenario.

The foundation geotechnical investigations (AECOM, 2017b) found no evidence of continuous or semi-continuous sub-horizontal weak planes in the foundation that would constitute a prescribed failure surface. The discontinuity analysis concluded that the more prominent features observed in borings and borehole televiewer surveys were not likely to persist as discrete, continuous foundation defects. On this basis, sliding resistance of the dams will be controlled by the shearing through the jointed rock mass immediately below the base of the dam. Shearing strength will vary along the dam axis and with depth reflecting the variability in rock conditions across the site.

Estimates for in-situ shear strength of the jointed rock mass across the site were made using the Hoek-Brown criterion (Hoek, et. al., 2002 and 1997) and expressed as equivalent Mohr-Coulomb failure parameters. The basis for and recommended foundation strengths at three core boring locations (CB-1, CB-16/16A and CB-13) at the preliminary estimated foundation elevations are presented in Appendix B and repeated below in Table 2.

2.4 RCC Properties

For conceptual design purposes, the unconfined compressive strength of the RCC was assumed at 2,500 psi (at one year). A unit weight of 150 pcf was also assumed. See also Section 4.3 for properties used in the seismic stability analyses.

Table 2. Equivalent Mohr-Coulomb Foundation Strength Parameters

Analysis Section	Degree of Weathering for Strength Estimates	Representative Boring	Median Unconfined Compressive Strength (psi)	Selected GSI*	D**	Confining Stress (psi)	Cohesion (psi)	Friction Angle (degrees)
15+50	Slightly Weathered to Fresh	CB-1	9,900	50	0	130	100	60
17+00	Slightly Weathered	CB-16/16A	11,450	40	0	180	100	57
20+50	Moderately	CB-13	3,850	40	0	80	40	55

* The GSI values selected for stations 17+00 and 20+50 correspond to the 25th percentile values for the indicated degree of weathering (Appendix B). The median GSI from boring CB-1 has been applied for conceptual-level estimate of foundation rock strength at Station 15+50.

** Disturbance factor (D); disturbance caused by construction activities. See Appendix B.

The sensitivity of the analysis results was checked by halving the cohesion component of the strength parameters, and also by eliminating it entirely.

3.0 Static Results

The static stability analysis results indicate that the maximum section of the spillway and non-overflow sections of the dam satisfy overturning stability and sliding criteria for all normal and flooding configurations up to and including the PMF. The resultant remains within the middle third even under the PMF loading indicating that the base of the dam remains in compression. The body of the dam also remains in compression and cracking along the lift lines is not predicted.

Summary of the stability results under the various static loading condition listed in Table 1 are presented in Table 3 (non-overflow), Table 4 (spillway) and Table 5 (non-overflow).

Table 3. Stability Results – Non-overflow Section, Station 15+50

Load Case	Comment	Reservoir Water Surface Elev.	Location of Resultant (percentage of base from toe)	Factor of Safety for Sliding
Usual	1:10 AEP	Spillway crest(1) (El. 1855)	50.7 (> 33.3)	3.67
Unusual	1:300 AEP	1:300 flood level (El. 1873)	44.8 (> 12.5)	3.18
Unusual	Drains inoperable	Spillway crest, (El. 1855)	45.3 (> 12.5)	3.12
Extreme	PMF	PMF level, (El. 1877)	43.2 (> 0)	3.07

[1] For the Usual Load Case, the reservoir level was taken at the spillway crest instead of the 1:10 AEP flood level

The section at Station15+50 is at a localized downstream dipping foundation surface.

Table 4. Stability Results – Spillway Section, Station 17+00

Load Case	Comment	Reservoir Water Surface Elev.	Location of Resultant (percentage of base from toe)	Factor of Safety for Sliding
Usual	1:10 AEP	Spillway crest(1) (El. 1855)	45.1 (> 33.3)	4.00
Unusual	1:300 AEP	1:300 flood level (El. 1873)	40.1 (> 12.5)	3.44
Unusual	Drains inoperable	Spillway crest, (El. 1855)	39.2 (> 12.5)	3.10
Extreme	PMF	PMF level, (El. 1877)	38.8 (> 0)	3.35

[1] For the Usual Load Case, the reservoir level was taken at the spillway crest instead of the 1:10 AEP flood level

Table 5. Stability Results – Non-overflow Section, Station 20+50

Load Case	Comment	Reservoir Water Surface Elev.	Location of Resultant (percentage of base from toe)	Factor of Safety for Sliding
Usual	1:10 AEP	Spillway crest(1) (El. 1855)	55.4 (> 33.3)	4.20
Unusual	1:300 AEP	1:300 flood level (El. 1873)	44.9 (> 12.5)	3.11
Unusual	Drains inoperable	Spillway crest, (El. 1855)	54.2 (> 12.5)	3.80
Extreme	PMF	PMF level, (El. 1877)	41.9 (> 0)	2.92

[1] For the Usual Load Case, the reservoir level was taken at the spillway crest instead of the 1:10 AEP flood level

Sensitivity analyses showed that each of the three sections generally met sliding stability criteria using only the respective friction component of the strength parameters listed in Table 2. The governing load case (lowest FS) for all three sections was the Unusual load case with inoperable drains (E=0). For this load case, the sliding factors of safety are presented in Table 6. The controlling section for sliding is the overflow section at Station 15+50. This is the result of the adverse downstream dipping foundation surface. However, this degree of downstream slope in the dam foundation is confined to a relatively limited area immediately to the left of the spillway, and is not typical of the majority of the foundation.

Table 6. Sliding Stability Results – Sensitivity Analyses (Cohesion = 0), Inoperable Drains (E=0), Water Surface Elevation 1855 feet

Station	Factor of Safety for Sliding
15+50	1.38
17+00	1.60
20+50	2.18

Figures C-1, C-2 and C-3 show the free-body diagrams for each of the three sections with the linear distribution of normal stress calculated along the foundation interface and at five equally spaced horizontal surfaces (lift surfaces) up the height of the dam. The figures are for the Usual load case but the same diagrams were also produced for each of the Unusual and Extreme loads cases. While the distribution of normal stress within the section is more complex than assumed by limit equilibrium analyses, such assumptions are adequate for preliminary analyses to assess the performance of the dam. The free-body diagrams showed that the body of the dam will remain entirely in compression for the Usual (Figures C-2 and C-3), Unusual and PMF Extreme loads cases and, therefore, cracking along the lift lines is not predicted. The MCE Extreme load case is discussed separately in Section 4.

Compressive stresses in the RCC were checked against allowable stress recommendations outlined in EM1110-2-2100, namely $f_c < 0.33\ f'_c$ (at 180 days) for usual load with a 15% increase for unusual loads and 50% increase for extreme load cases. In all cases, compressive stresses were substantially below the allowable stresses. Bearing stresses in the foundation were checked against allowable capacity derived from Peck (1976) using an RQD range of 25% to 50% (allowable capacity of 400 to 900 psi). Capacity exceeds the induced loads on the foundation and, therefore, compressive failure would not be a factor in the design.

Potential sliding along lift joints is typically not an issue, but this was checked as described in EM 1110-2-2006, 5-2.c and 4-2.c (2). As a preliminary assessment, strength parameters along a lift joint were conservatively assigned values of zero cohesion and 45 degree angle of internal friction (typically ranges between 40 and 60 degrees).

Closely spaced full height body drains extending upward from the gallery into the RCC together with bedding mortar placed on the upstream 30 feet of each lift will control and intercept seepage along lift surfaces such that there will be minimal uplift generated within the body of the dam. For all load cases, a sliding factor of safety along each of the horizontal surfaces shown in Figures C-1, C-2 and C-3 was calculated as the ratio of the ratio of the normal force on the surface (weight of RCC above) and the hydrostatic driving force of the reservoir above the surfaces. Factors of safety were all above 2.0, validating that failure along RCC lift lines is not an issue.

4.0 Dynamic/Earthquake Analyses

4.1 Analysis Approach

For concept design, a pseudo-dynamic analysis of the MCE loading of the spillway section at Station 17+00 was performed using the simplified method developed by Fenves and Chopra (1986). This method considers the effects of interaction between the dam, foundation and impounded water, of water compressibility and on the fundamental mode of vibration of the dam. It has been demonstrated that the traditional design procedures (Westergaard, 1933) for short vibration period structures, such

as concrete gravity dams, have limitations because they are based on unrealistic assumptions: rigid dam and incompressible water. The simplified method of Fenves and Chopra considers only the fundamental mode of vibration since this captures the majority of the dynamic response of a short vibration period structure.

The Fenves and Chopra result was compared with the results from a pseudo-static analysis using hydrodynamic forces based on Westergaard's generalized theory for added mass (1933). This was done for both the spillway section analyzed by the Fenves and Chopra approach and the non-overflow dam section (Station 15+50).

The seismic coefficient for both the hydrodynamic and inertial dam mass loading was taken as the spectral acceleration ($k_h = S_a$) for the fundamental mode of vibration (T_1) of the spillway section as calculated by Fenves and Chopra. No reduction was applied to the spectral acceleration for the pseudo-static application. Vertical acceleration was not included in the inertial loading of the dam mass.

4.2 Loading

MCE load case was analyzed with reservoir at the spillway crest (El. 1855) and with no tailwater.

Uplift pressures were assumed to be unchanged by earthquake loads and a cracked base analysis was not used. This is a standard assumption that is based on studies that have shown that the rapidly cycling nature of opening and closing of a crack does not allow reservoir water, and associated pressure, to penetrate the crack.

The design MCE earthquake is the deterministic 69th percentile response spectra presented in AECOM (2017b) and in Figure 4-2 in the CER. The peak ground acceleration (PGA) for this earthquake is 0.31g with a peak spectral acceleration (S_{Amax}) of 0.74g at a period (T) of 0.15 seconds.

4.3 Dam/Foundation System Properties

A sustained (static) modulus of elasticity (E_c) for the RCC was based on the equation in ACI 318 that relates E_c to unconfined compressive strength (f'_c).

$$E_c = 57{,}000\sqrt{f'_c} \qquad \text{(for normal weight concrete)}$$

$$E_c = 57{,}000\sqrt{2{,}500\text{ psi}} \qquad (f'_c \text{ at 365 days})$$

$$E_c = 2.85 \times 10^6 \text{ psi}$$

Instantaneous (dynamic) modulus of elasticity (E_s) was set at 1.25 E_c based on Section 3.4 and Table 3.5 of ACI 207.5R

$$E_s = 1.25(2.85 \times 10^6) = 3.5 \times 10^6 \text{ psi}$$

RCC unit weight and viscous damping ratio were set at 150 pcf and 5%, respectively.

Allowable dynamic direct tensile capacity across the RCC lift joints (f_{tlj}) is assumed to be $0.05\, f'_c$ **based on Section 3.2.2 of ACI 207.5R.**

$$f_{tlj} = 0.05(2{,}500\text{ psi}) = 125\text{ psi}$$

Unit weight for the foundation rock was taken as165 pcf.

An instantaneous (dynamic) modulus of elasticity was taken as the small-strain modulus for a representative thickness of foundation bedrock beneath the dam (on the order of one to two times the width of the dam). This was estimated to be 9×10^6 psi **based on P-wave and S-wave velocities measured from the seismic refraction and downhole suspension logging (Appendix B).**

4.4 Dynamic Analysis Results

The period of fundamental mode of vibration for the maximum spillway section (17+00) is approximately 0..28seconds (3.6 Hz), which corresponds to a spectral acceleration of 0.57 g from the design spectrum (CER Figure 4-2).

The results of the Fenves and Chopra analysis for the spillway section are presented in Figure C-4. Resultant location and sliding factors of safety (FS) corresponding to the equivalent strength parameters for the foundation rock strength in Table 2 are shown in Table 7.

Table 7. Pseudo-Static Stability Results based on Fenves and Chopra – E = 67%, Extreme (MCE)

Analysis Section	Comment	Reservoir Water Surface Elev.	Location of Resultant (percentage of base from toe)	Factor of Safety for Sliding
17+00	Fenves and Chopra	Spillway crest[(1)] (El. 1855)	-18.1 (≵ 0)	1.49 (> 1.1)
17+00	Westergaard	Spillway crest[(1)] (El. 1855)	-2.7 (≵ 0)	1.42 (> 1.1)
15+50	Westergaard	Spillway crest[(1)] (El. 1855)	-15.5 (≵ 0)	1.32 (> 1.1)

[1] For the Usual Load Case, the reservoir level was taken at the spillway crest instead of the 1:10 AEP flood level

In both analysis cases, sliding FS exceeds the minimum criteria of 1.1 for Extreme loading (Table 1). However, the resultant location (R) for both cases is outside of the base by a small margin. Although this exceeds the criteria for the resultant to remain within the base, it is not considered to be an issue for design. This conclusion is based on:

1. Any rocking of the section that might result will increase the period (T) of the first mode thereby decreasing the inertia loading of the section (in other words, it is self-stabilizing), and
2. The amount that R is outside of the base is sufficiently small that any cracking/separation of the dam from the foundation will be of limited width and duration such that reservoir water pressure would not develop within the crack and result in increasing destabilizing loads on the dam section.

As mentioned in Section 1, a finite element model will be developed for the dynamic analyses under MCE loading during preliminary design to better define seismic performance of the dam for this loading condition.

Linear stress distributions were calculated at various elevations within the dam monolith for the spillway section to check the approximate extent of RCC subject to tension. The zone of the dam subject to tension is shown on Figure C-4. The maximum tensile stress (f_t) of about 230 psi exceeds the estimated tensile capacity across the lift joints of 125 psi (f_{tlj}) indicating that some cracking of the lift joints initiating at the upstream face may occur. However, the length of potential cracking ($f_t > f_{tlj}$) is limited to the upstream quarter of the dam section. This degree of lift joint cracking is considered acceptable during an MCE and is expected to result in only minor (inconsequential) damage the dam.

4.5 Immediate Post-Earthquake Stability Results

The immediate post-earthquake stability of the dam was checked by assuming (very conservatively) that the foundation drains have been compromised and all drain efficiency has been lost. The results of this analysis for the non-overflow and spillway sections are shown in Tables 3 and 4, respectively as the “Unusual – Drains Inoperable” load case. In both sections, the FS exceed the minimum criteria for Unusual loading of 1.5 (Table 1).

5.0 References

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Figure C-1

Non-overflow, Sta 15+50

E = 67%, USUAL load (NMWS = El. 1855)

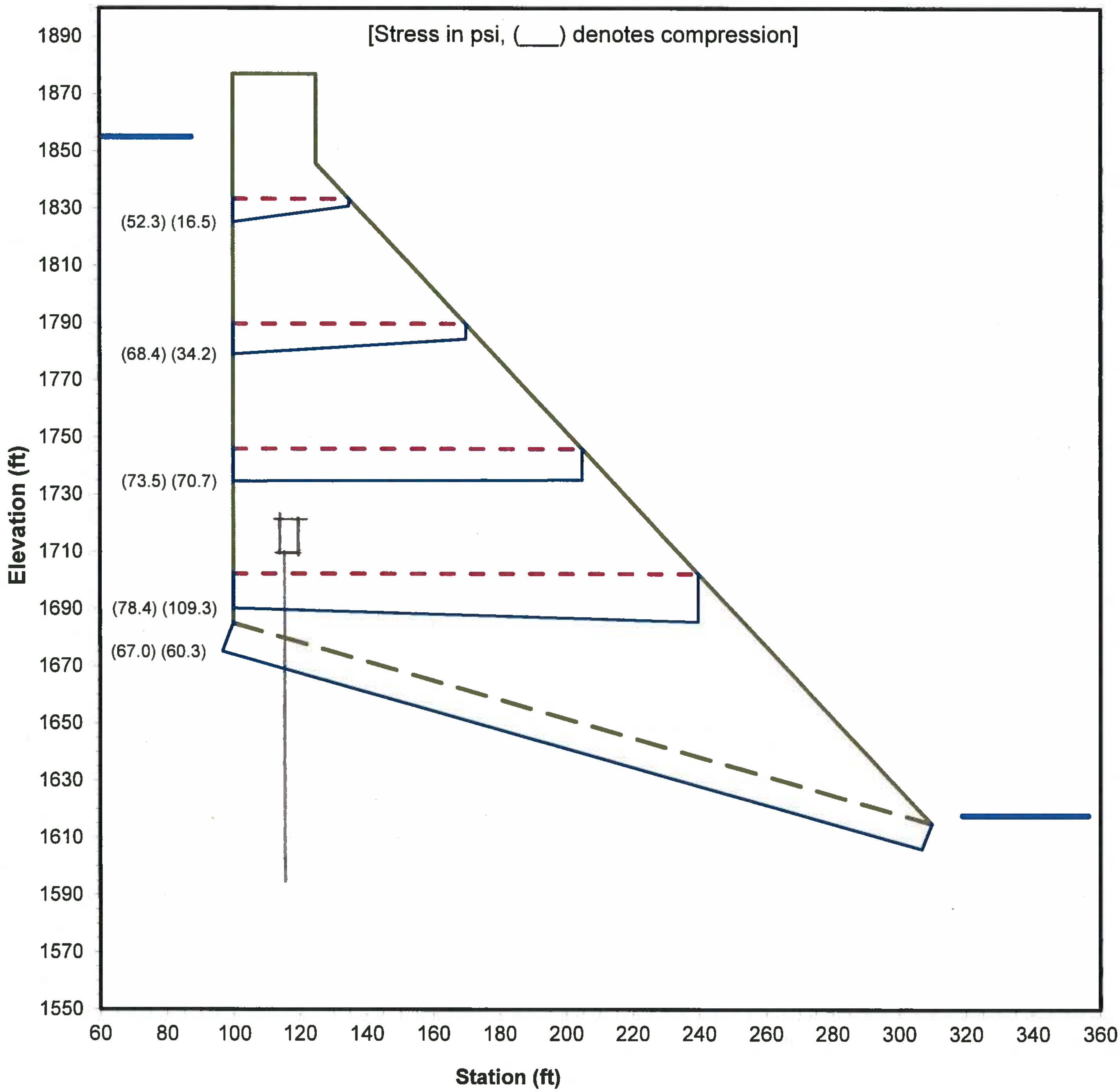


Figure C-2
Spillway, Maximum Section, Sta 17+00
E = 67%, USUAL load (NMWS = El. 1855)

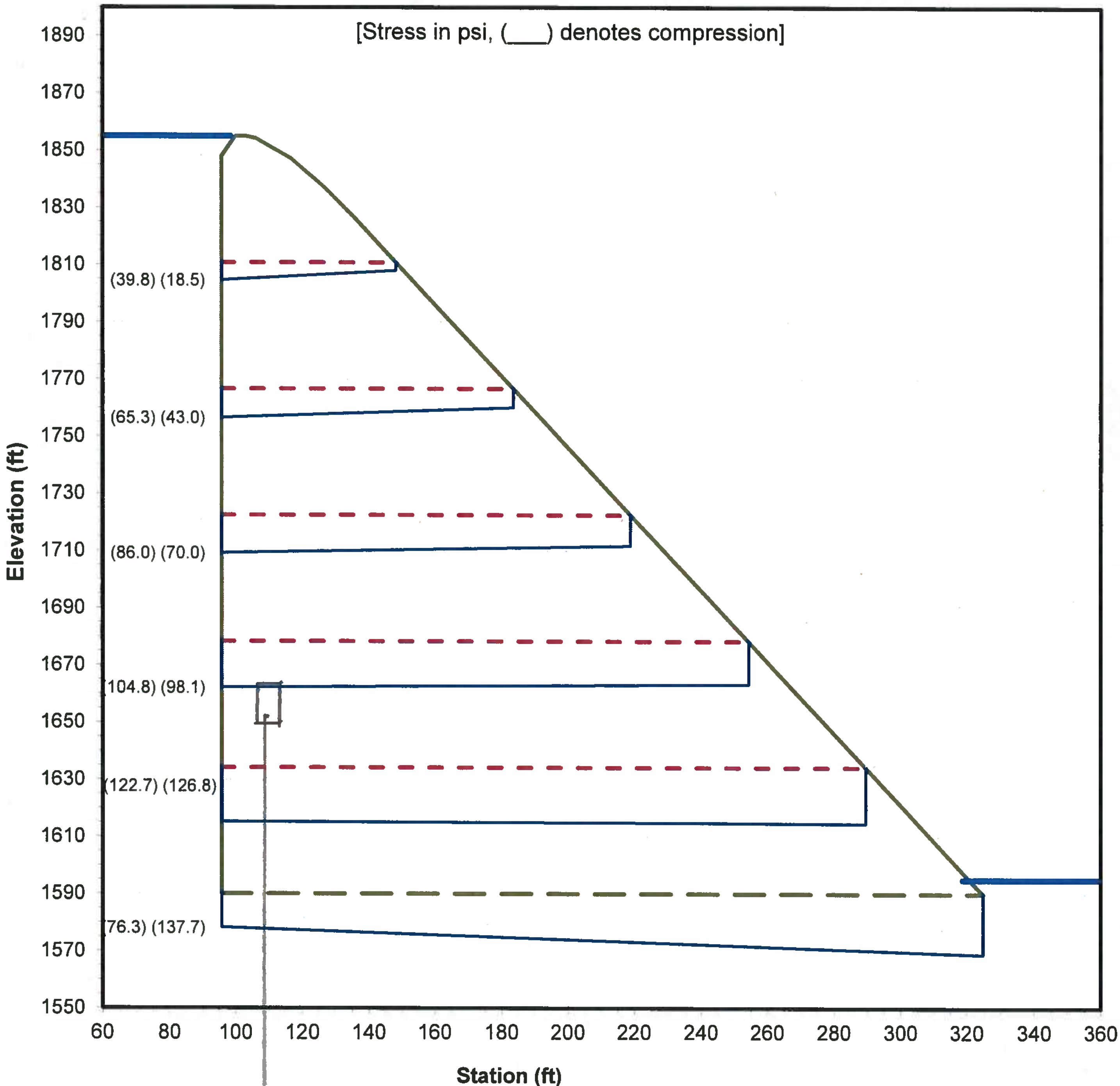


Figure C-3
Non-overflow, Sta 20+50,
E = 67%, USUAL load (NMWS = El. 1855)

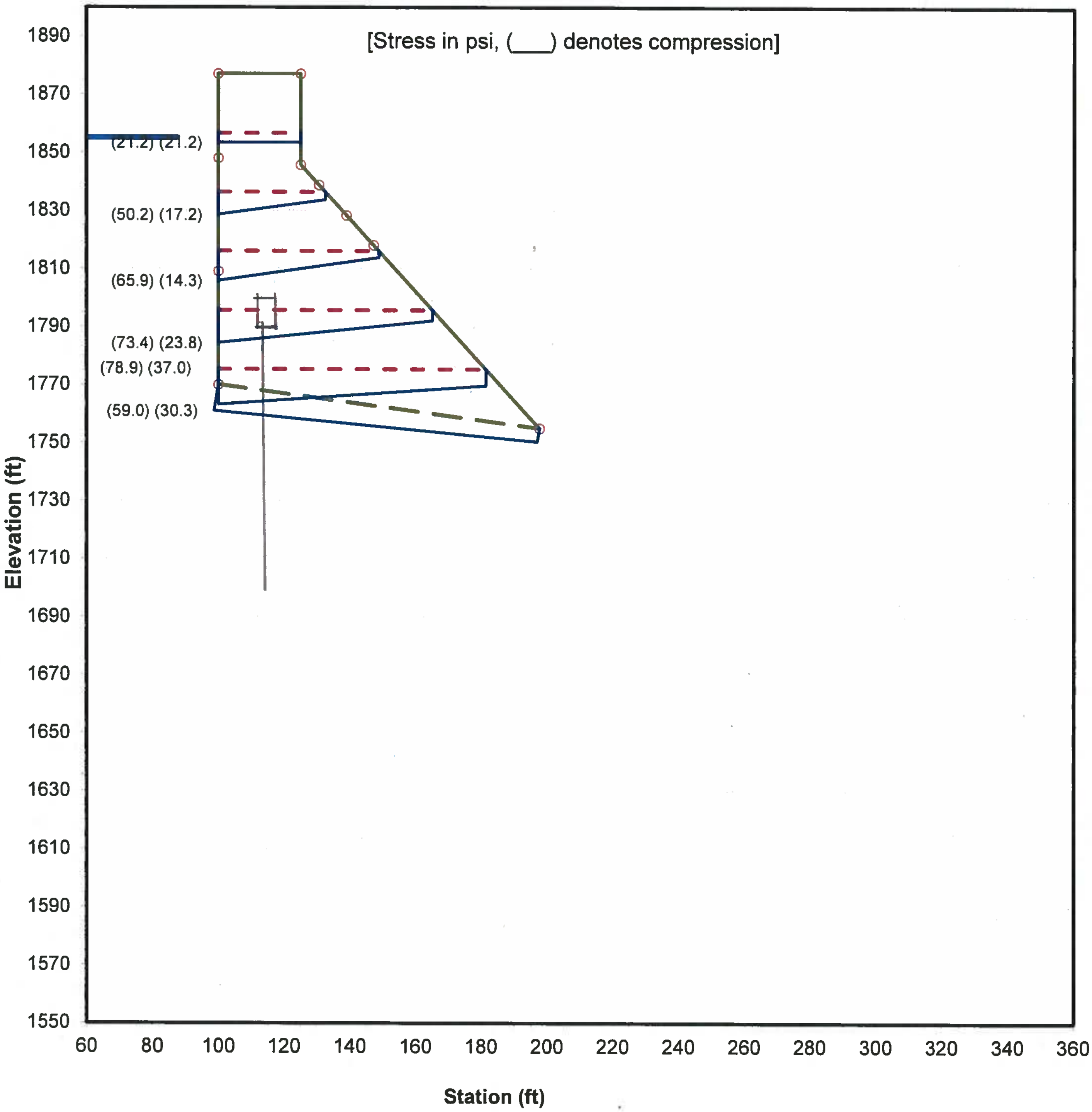
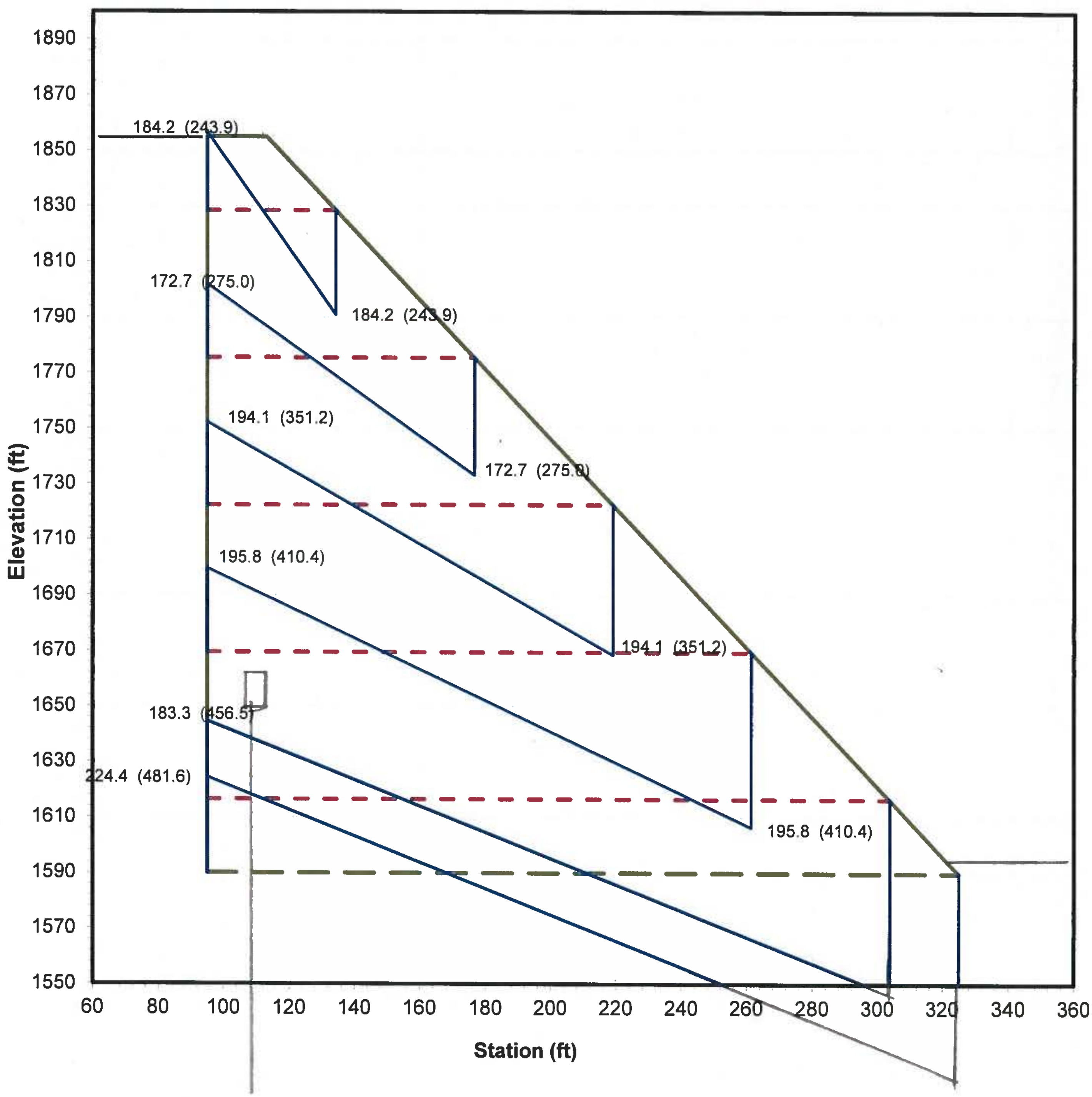


Figure C-4
Spillway, Maximum Section, Sta 17+00
E = 67%, Seismic - Chopra (NMWS = El. 1855)



Appendix D 300-year Storm Event Model

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Appendix D

Centennial Reservoir Project
300-year Storm Event Model

1.0 Introduction

This technical memorandum provides an estimate of the peak water levels in the proposed Centennial Reservoir due to a 300-year runoff event in the Bear River above Centennial Reservoir. The Army Corps of Engineers HEC-HMS model was used for the evaluation. A description of the model used is provided AECOM's PMP-PMF analysis report (AECOM, 2016). The model used in this analysis is the same with the exception of the input hyetograph which was changed from a PMP to an event that produces a 300-year flood event in the Bear River above Centennial Reservoir.

2.0 Development of 300-year Hyetograph

To determine the peak water levels in the Centennial Reservoir for a given event it is necessary to route a hydrograph through the reservoir. This requires that either an upstream hydrograph for that event be available or a hydrograph needs to be developed from precipitation or flow data. No detailed flow data (more frequent than daily) are available; therefore, a hydrograph was developed from precipitation data. A 300-year precipitation event was developed from precipitation data as described below. Since annual peak flow data are available in the Bear River downstream from Rollins Reservoir that data was used to develop a 300-year peak flow event. A given return period precipitation event does not necessarily produce the same return period flow event (e.g., a 100-year rainstorm in a watershed does not necessarily produce a 100-year flow, especially if there are reservoirs on the stream). The resulting hyetograph was adjusted to produce the 300-year flow at the gage location. Though the adjustment will just increase the precipitation volume it also is accounting for antecedent moisture and existing flow in the River. For example, a day with a 300-year annual peak flow was likely preceded by days that also had large flows. The adjustment is to provide a 300-year flow event into Centennial Reservoir.

2.1 Precipitation

The precipitation used to develop the 300-year hyetograph for this analysis was obtained from the California Department of Water Resources (DWR) engineering meteorology website for Rainfall Depth-Duration-Frequency Data (http://ferix.water.ca.gov/webapp/precipitation/). DWR provides a depth-duration-frequency data for precipitation gages throughout the state. For most of the stations data are provided for durations from 1 day to 60 days (e.g., 1-day. 2-day, 10-day, etc.). For a small subset of stations data are provided for durations from 30 minutes to 24 hours. The data for several stations within the vicinity of the Bear Creek watershed were reviewed and two stations were selected as appropriate for this analysis. The Drum Power House gage located on Bear River about 12 miles above Rollins Reservoir was used for the watershed above Rollins Reservoir. The Grass Valley Station, located 13 miles north of the proposed location of Centennial Reservoir, was used for the watershed between Rollins Reservoir and Centennial Reservoir.

Each of these gages has precipitation volumes for durations of 30 minutes, 1-hour, 2-hour, 3-hour, 6-hour, 12-hour and 24-hours for 200-year and 500-year return periods. The 300-year precipitation values were obtained by interpolation between the 200 and 500-year values. Interpolation was in natural ln-space (i.e., the natural logarithms of the values were interpolated then converted back to arithmetic space).

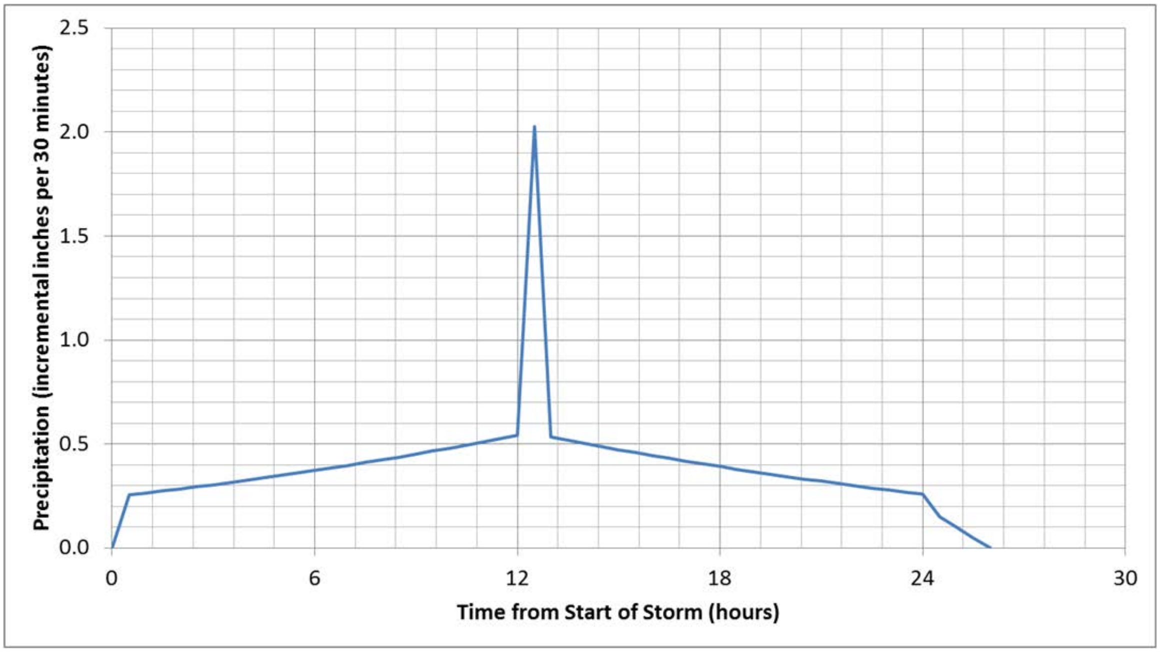

Figure 1. Hyetograph used in HEC-HMS Model

A 24-hour hyetograph was developed with 30 minute increments. The 300-year 24-hour precipitation volume was 10.8 inches. The peak of the storm was centered in the hyetograph. Each sub-interval in the hyetograph (e.g., 30-min, 1-hour, 6-hour, etc.) also had a 300-year return period. Figure 1 shows the hyetograph for the watershed above Rollins Reservoir. A similar hyetograph was developed for the area between Rollins Reservoir and Centennial Reservoir.

2.2 Flow

A USGS gage station is located on the Bear River downstream of Rollins Reservoir (USGS Station 11422500, Bear River below Rollins Reservoir, Near Colfax). The station is located downstream of the Bear River Canal Diversion. During winter months the canal diverts approximately 200-300 cfs, which is less than 1% of the 300-year peak flow so was ignored in this analysis. The largest flow of record is 34,300 cfs which occurred on January 2, 1997. This corresponds to about a 50-year event. This 34,300 cfs value is an estimate based on extending the rating curve above 11,600 cfs (USGS 1998). Note that Rollins Reservoir had been spilling for several weeks before the flow of record occurred. The peak reservoir level was reached one day before the peak flow in the river.

The Army Corps of Engineers HEC-SSP V2.0 program (USACE 2010) was used to analyze the annual peak flow data from the USGS gage. HEC-SSP fits a log-Pearson Type III distribution to the data using procedures described in Bulletin 17B (USGS 1982). Data are available from 1966 to 2016. Table 1 below provides the results of the analysis. The 300-year event, natural-logarithmically interpolated between the 200-year and 500-year events, is 55,600 cfs. The uncertainty in the flow is between 60% and 200% of the predicted value, or between 33,360 and 111,200 cfs.

Table 1. Flood Frequency for USGS Flow Gage below Rollins Reservoir

Flow Rate (cfs)	Percent Change Exceedance	Return Period (years)	Confidence Limits (flow, cfs)	
			0.05	0.95
64,998.40	0.2	500	135,500.30	37,533.70
51,505.70	0.5	200	103,045.20	30,566.80
41,938.70	1	100	80,967.00	25,479.90
33,025.60	2	50	61,230.00	20,599.90
22,414.90	5	20	39,021.90	14,551.80
15,408.50	10	10	25,344.60	10,354.80
9,404.60	20	5	14,478.50	6,557.70
3,219.40	50	2	4,526.60	2,306.70
923.8	80	1.25	1,319.10	605.3
446.9	90	1.111111	674.6	265.4
235.9	95	1.052632	379.7	126.5
64.5	99	1.010101	121.1	27.5

3.0 Results

The hyetograph developed from the data provided on the DWR Engineering Meteorology website was adjusted until the peak flow in the Bear River downstream of Rollins Reservoir was equal to the 300-year flood event developed from the USGS measured data below Rollins Reservoir. This required that the original estimate be doubled for a 24 hour rainfall of 20 inches for the entire watershed (16 inches for the area to the below Rollins gage). Note, the hyetograph should not be considered as a "real" rainfall volume but rather the volume of rainfall that is needed to produce a 300-year 24-hour flow event given minimal flow in the river initially. For a 300-year event, the flow in the river may be a significant fraction of the 300-year event at the start of the day. Instead of picking an arbitrary initial flow an artificially high rainfall was used to generate the needed runoff. The peak inflow to Centennial Reservoir was 63,480 cfs due to the additional drainage area between the gage station below Rollins Dam and the Centennial Reservoir.

The peak water surface elevations for different spillway widths are provided in Table 2 below.

Table 2. Peak Water Surface Elevation in Centennial Reservoir for 300-year Inflow Event

Spillway Width (ft)	Peak Reservoir Outflow (cfs)	Peak Water Surface Elevation (ft)	Peak Surcharge (ft)
210	56,690	1872.3	17.3
200	56,300	1872.7	17.7
180	55,470	1873.8	18.8
170	54,960	1874.4	19.4

4.0 Comparison to PMP/PMF

The 300-year flood event below Rollins Reservoir is estimated to be 55,660 cfs. This compares to 78,700 cfs predicted as the outflow from Rollins Reservoir in the AECOM PMP-PMF Study (2016). This indicates that the 300-year event is 71% of the PMF. This implies that the PMF estimate could be too low or the 300-year event could be too high.

The PMF was also estimated by in two other studies. The Montgomery Group (2006) using HMR 59 estimated the PMF at Rollins Reservoir to be 73,150 cfs. The Sierra Hydrotech Report (1986) using HMR 36 predicted a PMF outflow of 63,900 cfs from Rollins Reservoir. The AECOM and Montgomery Group PMF estimates are larger than the Sierra Hydrotech estimate likely because of the switch from HMR 36 to HMR 59. The AECOM and Montgomery Group estimates are similar. Therefore, the PMF estimate of 78,700 cfs seems reasonable.

The peak flow of record for the below Rollins Reservoir gage occurred on January 2, 1997. It was equal to 34,300 cfs or 44% of the predicted PMF. This corresponds to about a 50-year event according to the analysis results shown in Table 1. The 300-year event was also calculated for other gages on the Bear River; two gages below the gage below Rollins Reservoir (nr Wheatland [#11424000, 1929-2015], and nr Auburn [#11423000, 1940-1967]) and one gage upstream (bl Dutch Flat after bay [#11421790, 1965-2015]). The 300-year values calculated for those stations are shown in Table 3. The uncertainty in the values (0.05% and 0.95% confidence limits) is on the order of 50% to 200% of the values shown in the table. The flow per area shown in the last column should decrease with an increase in drainage area, which is the general trend in Table 3. The 300-year event below Rollins of 55,660 cfs seems reasonable compared to other gages on the river.

Table 3. Estimated 300-year Event for Several USGS Gage Stations on the Bear River

USGS Station Name	Station No.	Drainage Area (mi^2)	Flow (cfs)	Flow per area (cfs/mi^2)
Bear River bl Dutch Flat Afterbay nr Dutch Flat	11421790	21.5	12,000	558
Bear River bl Rollins Dam nr Colfax	11422500	105	55,660	530
Bear River nr Auburn	11423000	140	33,200	237
Bear River nr Wheatland	11424000	292	78,200	268

A review of the rainfall data for the Bear River watershed showed large volumes of precipitation preceding the largest events on record. Table 4 shows the rainfall data for the 3 largest events of record. The PMP above Rollins Reservoir is 43 inches in 3 days for comparison. The rainfall in the 3 days preceding the largest flows on record was about 20% to 30% of the PMP and 35% to 40% of the PMP in the preceding 7 days.

The 300-year peak flow of 55,600 cfs seems reasonable given the available data. The uncertainty in the estimate is large (from 33,360 to 111,200 cfs) but the best estimate is consistent with other data from the watershed (other flow gages and precipitation).

Table 4. Precipitation Volumes Preceding Three Largest Flow Events at USGS Gage Stations on the Bear River Below Rollins

Date	Flow (cfs)	Precipitation Volumes (inches)		
		30 days	7 days	3 days
January 2, 1997	34,300	32	16[1]	9
December 31, 2005	25,800	28	15	9
February 17, 1986	22,500	23	17	13
8 days				

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Appendix E Probable Maximum Flood and Probable Maximum Precipitation Study

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Appendix E

Centennial Reservoir Project
Probable Maximum Flood and Probable Maximum Precipitation Study

1.0 Introduction

1.1 Background

This technical memorandum documents the preliminary analysis performed to calculate the Probable Maximum Precipitation and Probable Maximum Flood for Centennial Dam, a new facility proposed by the Nevada Irrigation District (NID). This study will be updated during the design phase with refined improvements and hydraulic modeling.

The Probable Maximum Precipitation was calculated using Hydrometeorological Reports 58 and 59. The calculated 72-hour cumulative precipitation for the 123-square mile watershed varies from 30 inches near Centennial Reservoir to 43.5 inches in the watersheds above Rollins Reservoir.

The Probable Maximum Flood was calculated using the Army Corps of Engineers HEC-HMS rainfall runoff model. The calculated Probable Maximum Flood inflow to the proposed reservoir is 89,181 cfs. Various sized spillways were evaluated from 180 to 210 feet of effective width. The surcharge on the spillway weir varied from 20.8 to almost 24 feet.

1.2 Purpose

The purpose of this technical memorandum is to document the planning phase calculation of the Probable Maximum Flood (PMF) for the proposed Centennial Dam based on the Probable Maximum Precipitation (PMP) developed using NOAA Hydrometeorological Report 58/59 (NOAA 1999). This PMF study will need to be updated during the design phase following further design refinements of the spillway and dam.

2.0 Probable Maximum Precipitation

The PMP approach involves calculating an area-weighted index PMP for the watershed of interest, applying a depth-duration ratio based on the storm duration of interest, and then applying an area reduction factor based on the watershed size. The index PMP is calculated from the PMP index map, a precipitation depth contour (isohyetal) map provided in the Hydrometeorological Report (e.g., HMR 59 (NOAA 1999)).

The Montgomery Water Group included snowmelt in their 2006 study. The precipitation intensity contributed by snowmelt was less than 0.05 in/hr in their results (see Figure 5-2 in Montgomery Water Group 2006). This compares to their peak precipitation intensity of about 2.5 inches per hour. Given that most of the watershed is less than 5,000 feet in elevation, the uncertainty in estimating snowpack and snowmelt, and the small volume contributed by snowmelt, snowmelt was not included in the analysis.

The procedure for calculating the PMP for an all-season general storm can be found in HMR 59 (Chapter 13, page 233) and can be applied to areas from 10 to 10,000 mi^2 for durations from 1 to 72 hours. The following steps were used to calculate the PMP for Centennial Dam.

1. Calculate the 10-mi² 24-hour PMP Index

The drainage area to Centennial Dam was divided into 4 sub-basins: Bear River above Rollins Reservoir, Greenhorn Creek above Rollins Reservoir, Rollins Reservoir and the small drainages surrounding it, and Bear River between Rollins Reservoir and Centennial Dam. The Centennial Dam watershed and sub-basins were delineated using the USGS StreamStats program (http://water.usgs.gov/osw/streamstats/). Figure 1 shows the depth contours of the HMR 59 index PMP (10-mi^2, 24-hour) superimposed on the outline of the watersheds. The watershed lies between contours of 18 to 27 inches. Table 1 lists the area and the area-weighted PMP Index values for each of the sub-basins used in the analysis. The total area-weighted average index PMP (10-mi^2, 24-hour) estimated to be about 25.4 inches.

Table 1. PMP Index Values for Centennial Dam Watershed

Watershed_Name	10-mi^2 24-hour PMP index (in.)	Area (mi^2)
Bear River drainage between Centennial and Rollins Dams	19.0	19.6
Rollins Reservoir and Surrounding area	21.8	12.2
Bear River above Rollins Reservoir	27.3	52.9
Greenhorn Creek	27.1	38.2
Total	25.4	123

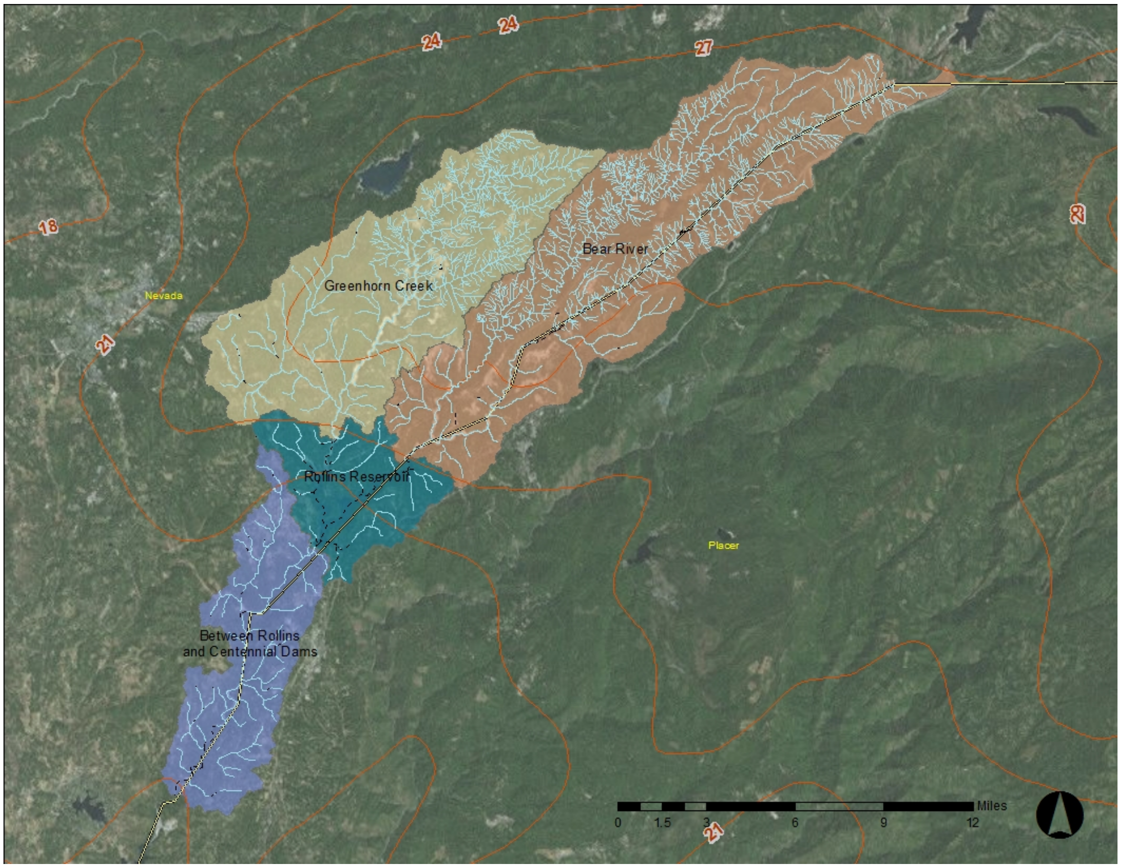

Figure 1. Centennial Dam and PMP Index Map from HMR 59 (inches)

2. <u>Depth-Duration Ratios</u>

Depth-duration ratios are provided in Table 13.1 of HMR 59 (or Table 2.1 in HMR 58) for different subregions of California. The Centennial Dam is located on the border between the Central Valley and Sierra subregions. The Sierra subregion was assumed. The all-season depth duration values for the Sierra subregion are between 0 and 7% greater than the Central Valley subregion so the use of the Sierra subregional is slightly conservative. The depth-duration ratios for the Sierra subregion were multiplied by the 24-hour index values shown in Table 2 from the first step to obtain 1, 6, 12, 24, 48, and 72-hour PMP depths. Depth-duration ratios for the Sierra Region are provided in Table 2.

Table 2. All Season Depth-Duration Ratios for Sierra Region

Duration (hrs)	1	6	12	24	48	72
Ratio	0.14	0.42	0.65	1.00	1.56	1.76

3. Areal Reduction Factors

For watershed areas greater than 10 mi^2, reduction factors are applied to the index PMP. In order to obtain the PMP for the 123-mi^2 watershed, the applicable reduction factors were multiplied by the corresponding 10-mi^2 values from the previous step. Table3 shows the areal reduction factors for the Centennial Dam watershed.

Table 3. All Season Depth Area Reduction Factors for Centennial Dam Watershed

Area (mi^2)	1 hr	6 hr	12 hr	24 hr	48 hr	72 hr
100	82.5	84.0	85.5	87.0	89.25	91.25
123 (Centennial Dam)[1]	81.2	82.8	84.4	86.0	88.4	90.6
200	76.75	78.75	80.75	82.75	85.5	88.25

1 Interpolated values

4. 6-hour Incremental Estimates

The cumulative 6-hour precipitation values for the 72-hour duration storm were obtained by plotting a smooth curve of the PMP depths calculated for the 1, 6, 12, 24, 48, and 72-hour durations. The 6-hour increments were determined from successive subtraction of the values from the smooth curve as shown in Figure 2. The PMP values for each watershed are shown inTable 4.

Table 4. Cumulative PMP Depths for Variable Durations form 1 to 72 Hours

		Duration (hrs)	1	6	12	24	48	72
		Depth Duration Ratio	0.14	0.42	0.65	1	1.56	1.76
		Area Reduction Factor	81.2	82.8	84.4	86	88.4	90.6
Watershed	Area	10-mi^2 24-hour PMP index						
Bear River drainage between Centennial and Rollins Dams	19.6	19	2.16	6.6	10.4	16.3	26.2	30.3
Rollins Reservoir and surrounding area	12.2	21.8	2.48	7.6	12.0	18.7	30.1	34.8
Bear River above Rollins Reservoir	52.9	27.3	3.10	9.5	15.0	23.5	37.6	43.5
Greenhorn Creek	38.2	27.1	3.08	9.4	14.9	23.3	37.4	43.2

The cumulative 72-hour PMP varies from 30 to 43 inches among the subbasins.

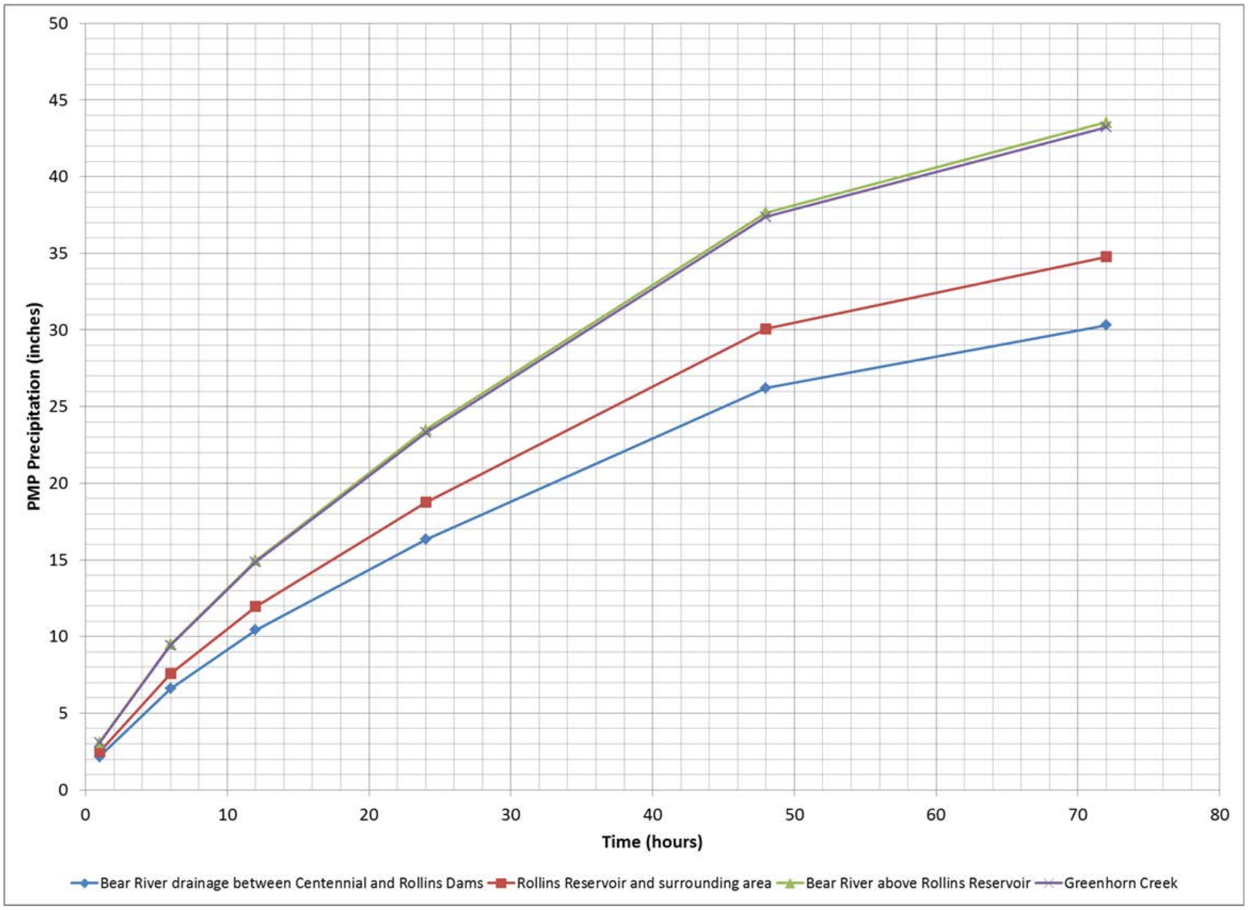

Figure 2. Centennial Dam Watershed 72-hour 123-mi^2 PMP Depths

2.2 Watershed Model

Hydrologic modeling of the Centennial Dam watershed was performed using the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) Version 4.1 (USACE, 2010). The model includes three main components: basin model, meteorologic model and control specifications. The basin model allows the user to define the characteristics of the stream network, which could include the network and characteristics of sub-basins, reaches, junctions, reservoirs, etc. The meteorologic model describes precipitation. The control specifications indicate the time period and time step for a simulation run. The following sections describe the input parameters used to develop the HEC-HMS model.

2.2.1 Sub-basin Definition

The Centennial Dam watershed has an area of 123 square miles. The watershed was divided into four sub-watersheds (see Figure 1):

1. There are two major tributaries to Rollins Reservoir: Greenhorn Creek and Bear River.

2. There are several small tributaries that contribute to Rollins Reservoir surrounding the reservoir.

3. There are additional flows into the proposed reservoir from the canyon between Rollins Reservoir and the proposed dam site.

2.2.2 Rainfall-Runoff Loss and Transform Methods

The rainfall interception and infiltration losses were calculated using the SCS curve number method. It is a widely used method for determining the amount of runoff from a rainfall event. Two parameters need to be specified for this loss method: curve number and percent impervious ground.

1. The curve number was obtained from Table 9-1 in the National Engineering Handbook (USDA, 2004) which involves the vegetation cover type and the hydrologic soil group. The hydrologic soil groups are A, B, C, and D and are classified based on the soil permeability. The highest runoff is associated with soils in Group D.

   - Land cover is primarily forest.
   - Soil data including the hydrologic soil group by sub-basin was obtained from the Web Soil Survey maintained by the US Department of Agriculture (http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx). The average hydrologic soil group for each subbasin was calculated as the soil type weighted average value.

2. The watershed was estimated to be zero percent impervious[1].

The transform method converts excess precipitation into runoff at the basin outlet. The Clark Unit Hydrograph method was used which requires a time of concentration and a storage coefficient. The time of concentration was estimated using Equation 3-1 below from Chapter 15 of the National Engineering Handbook.

$$T_C = \frac{l^{0.8}(S+1)^{0.7}}{1{,}140Y^{0.5}} \qquad 3\text{-}1$$

where:

Tc = time of concentration, hrs

l = longest flow path length, ft

Y = average watershed land slope, %

S = maximum potential retention, in

= (1000/CN) -10

CN = curve number

Data on the flow length and average watershed slope were obtained from the USGS StreamStats program (http://water.usgs.gov/osw/streamstats/).

Table 5 shows the data input into the HEC-HMS model.

[1] Note that the reservoirs themselves have a land cover classification of "water", and these areas are assigned a CN of 100.

Table 5. Input Data used in the HEC-HMS Model

Watershed	Area	Curve Number	Watershed slope (%)	Longest Flow Path (mi)	Time of Concentration (hrs)	Clark Storage Coefficient
Bear River drainage between Centennial and Rollins Dams	19.6	68	26	14	4.8	4.8
Rollins Reservoir and surrounding area	12.2	72	20.0	2.5	1.2	1.2
Bear River above Rollins Reservoir	52.9	69	30.2	22	6.0	6.0
Greenhorn Creek	38.2	69	22.3	14	4.8	4.8

The outflow from Rollins Reservoir was routed downstream to Centennial Reservoir using the Muskingum-Cunge routing method. The length of the channel is 49,000 feet with a slope of 0.00276. The channel was assumed to have a trapezoidal shape with an 80 foot bottom width and 3:1 side slopes. The Muskinggum-Cunge method does not significantly attenuate the flow so the results are not sensitive to the routing parameters. The peak flow was attenuated by about 1%.

2.2.3 Rainfall Hyetograph

The time of concentration is less than 6 hours so the runoff would quickly reach a steady state with the 6-hour rainfall increment. Therefore, a finer time increment of 15 minutes was developed and used in the model to better capture the runoff process[2]. The 15-minute cumulative rainfall values were estimated by reading off the values from 6-hour cumulative rainfall curve shown in Figure 2. The 15-minute incremental values were then calculated from successive subtraction of the cumulative rainfall values. Incremental values were then rearranged and centered at 2/3 of the time from the beginning of the storm. Linear interpolation was used in short portions of the hyetograph to transition the curve smoothly when the incremental rainfall values had a sharp increase or decrease. HMR 59 does not have a specified method for determining the temporal distribution of the incremental rainfall; however, it does recommend to group the four heaviest 6-hour values into front-, middle-, or end-loaded temporal distributions and select the most critical distribution for a particular basin. For this prelimianry analysis, the peak of the storm was located at 45 hours into the 72 hour storm. Figure 3 shows the selected end-loaded 72-hour rainfall hyetograph with 15-minute increment.

[2] HEC-HMS recommends a time step less than 0.39 times the lag time, where the lag time is 0.6 times the time of concentration. The shortest lag time in the watershed model is a little less than 0.72 hours (1.2 hours × 0.6), which would suggest a time step less than 17 minutes (0.72 hours × 60 minutes/hour × 0.39).

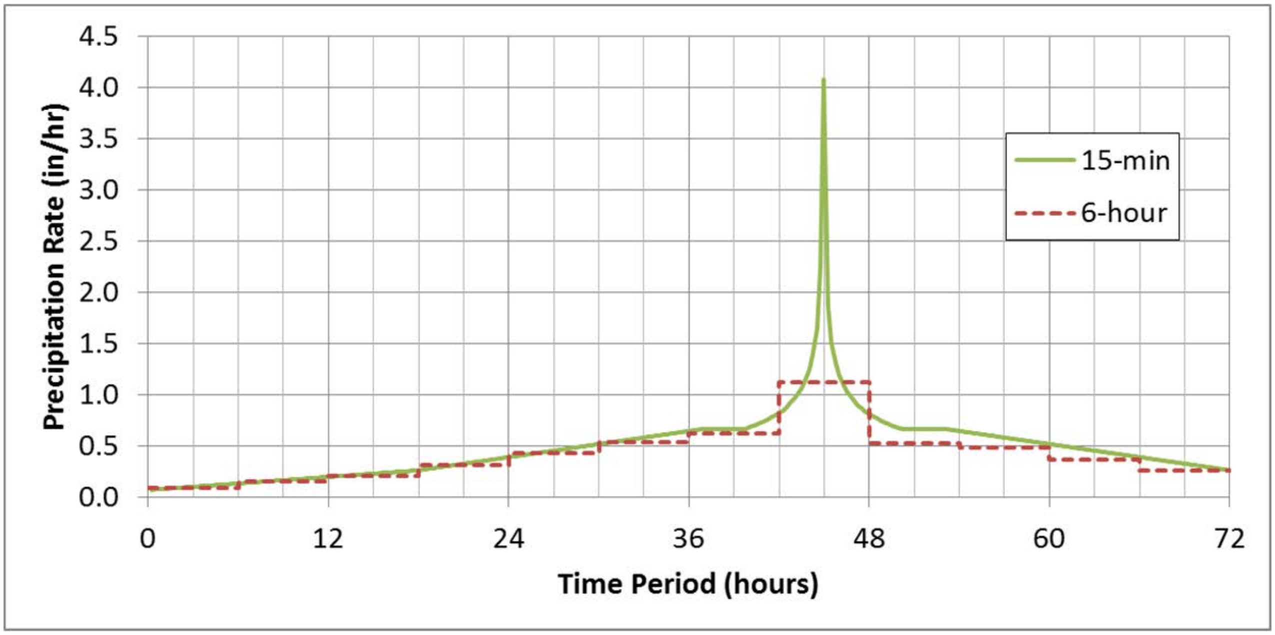

Figure 3. Hyetograph for Precipitation near Rollins Reservoir

2.2.4 Stage-Storage Curves

The stage storage curve for Rollins Reservoir was obtained from drawing G-173803, Nevada Irrigation District Yuba-Bear River Development Rollins Dam, Reservoir Plan SH 1 (Figure 4). The reservoir elevation discharge curve for Rollins Reservoir was obtained from Rollins Reservoir Dam and Details Exhibit L-13a (Figure 5).

Figure 6 shows the stage-storage curve for Centennial Reservoir. The spillway has not yet been designed. For this analysis an ogee spillway was assumed with a width of between 150 and 210 feet and a design head of 20 and 25 feet. The approach depth will be deep so no loses were assumed. Figure 6 shows the stage-discharge curve for Centennial Reservoir.

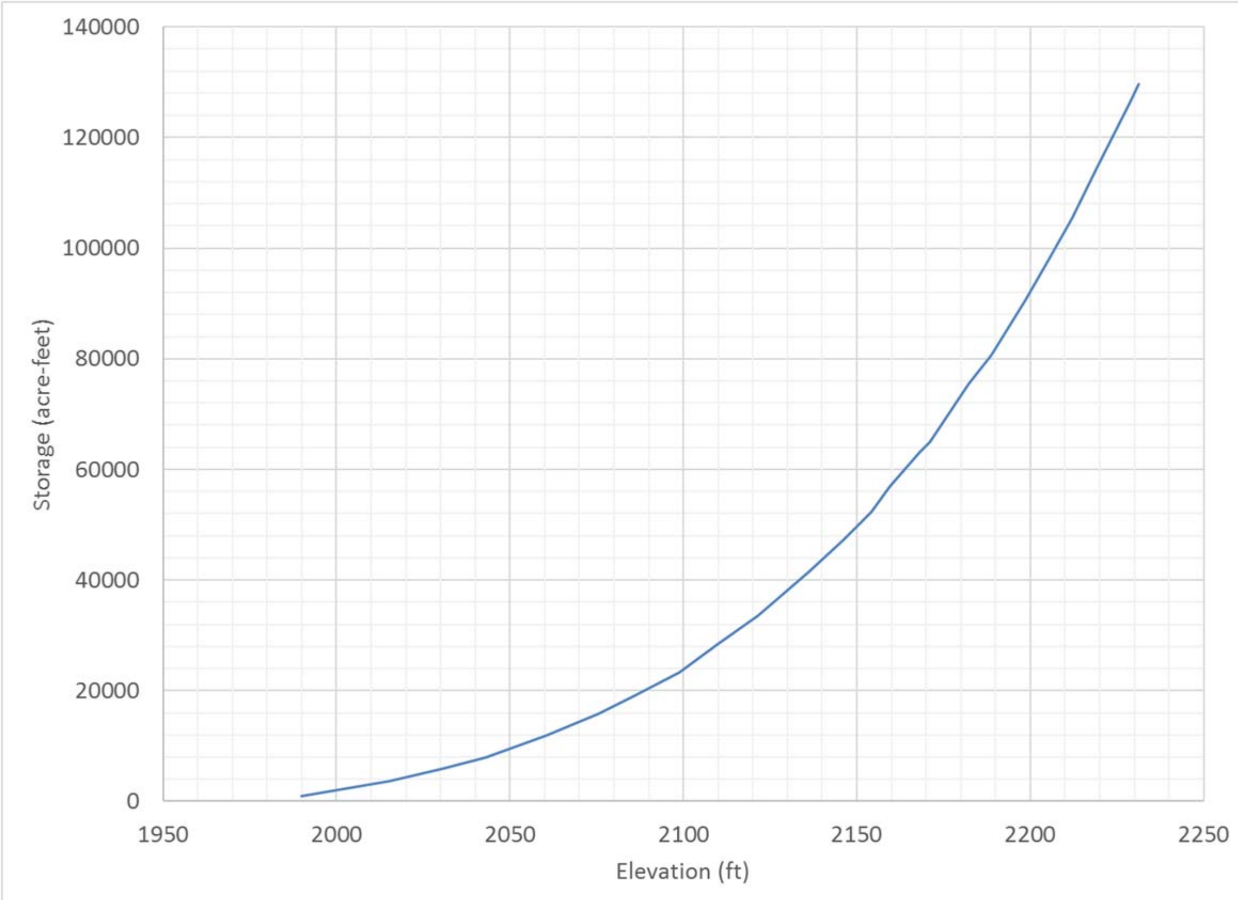

Figure 4. Stage-Storage Curve for Rollins Reservoir

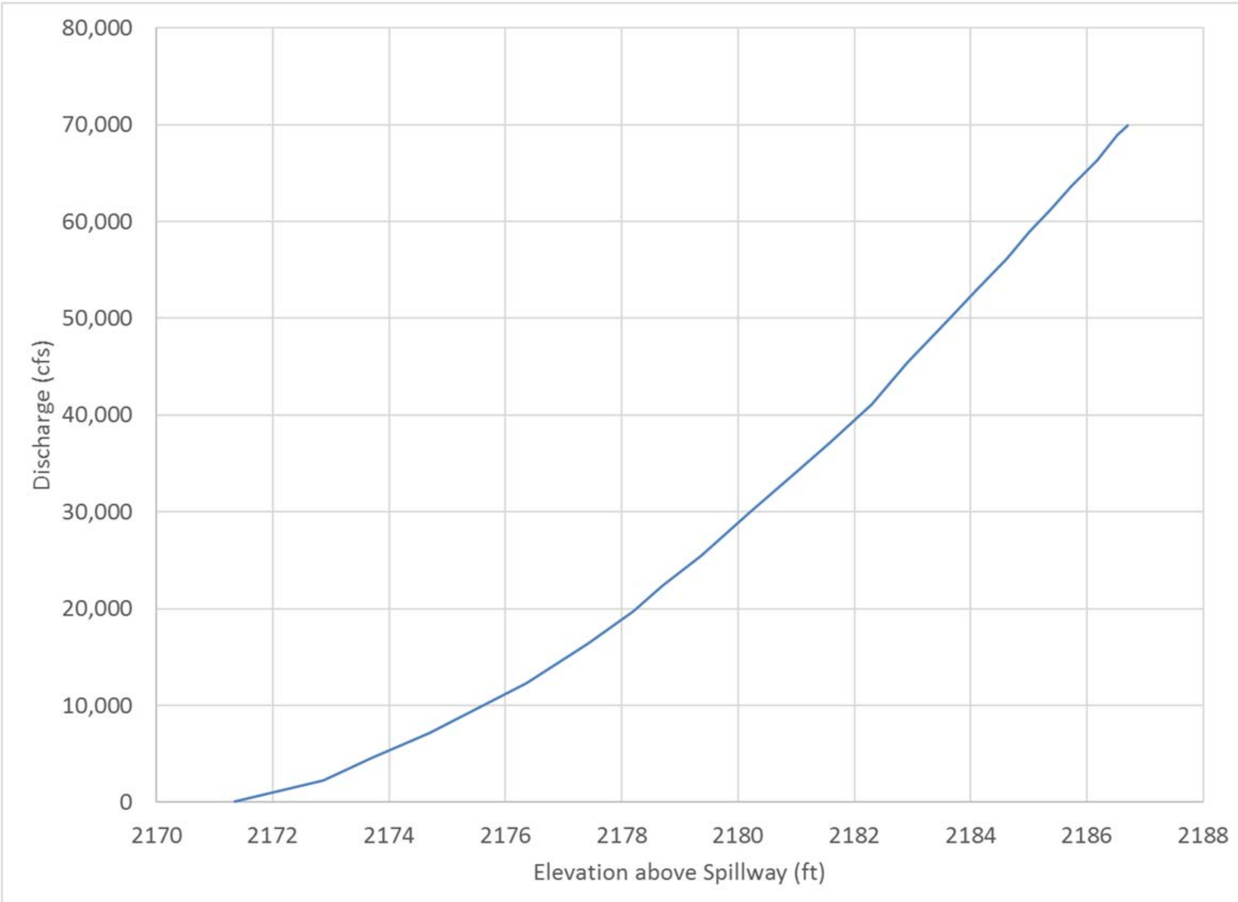

Figure 5. Stage-Discharge Curve for Rollins Reservoir

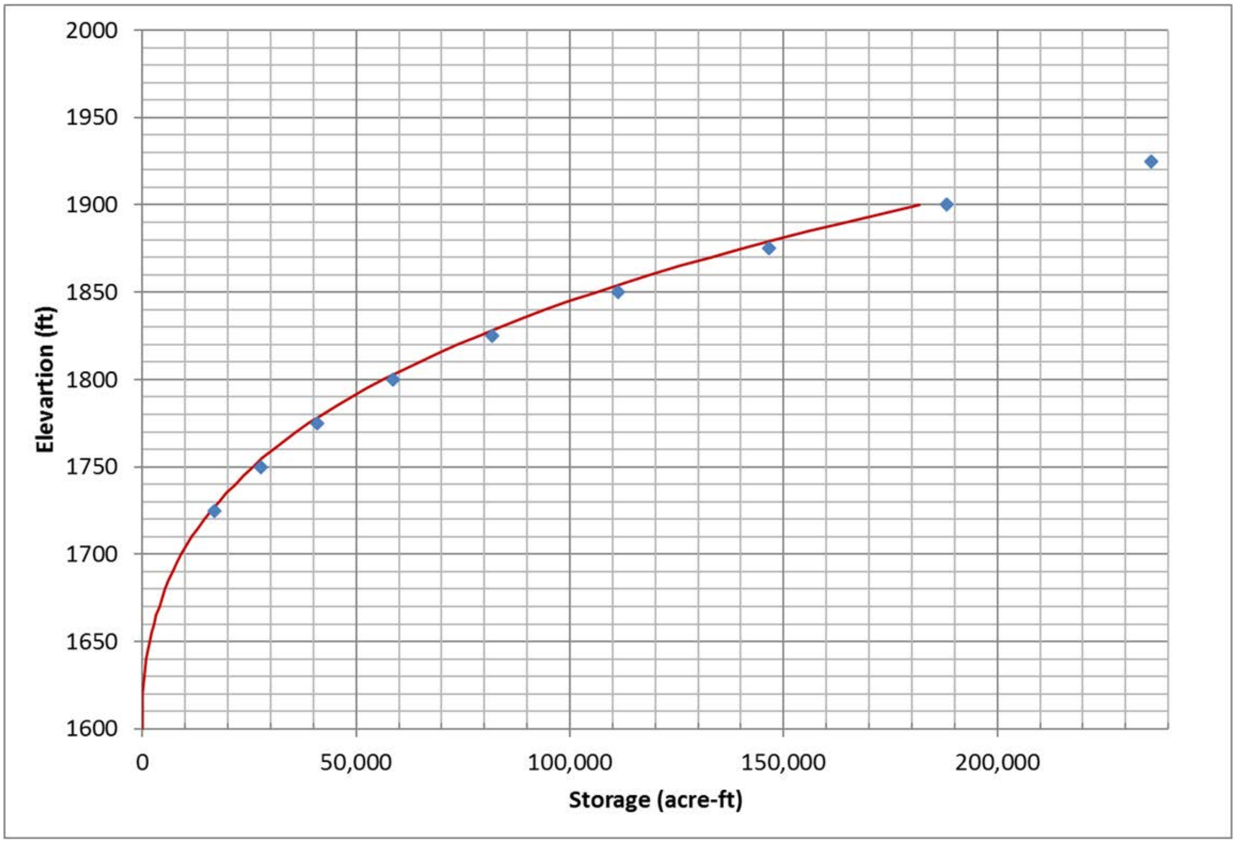

Figure 6. Stage-Storage Curve for Centennial Reservoir

2.3 PMF Results

The hydrologic routing was performed using the HEC-HMS model with the 15-minute PMP hyetograph as precipitation input, the sub-watershed basin characteristics (e.g. area, curve numbers, times of concentration), and reservoir stage-storage and stage-discharge curves . The model was run over a 72-hour period with a 15-minute time step. Greenhorn, Bear River above Rollins and Rollins Reservoir sub-watersheds discharge directly to Rollins Reservoir. These inflows were routed through Rollins Reservoir using the stage-storage and storage-discharge curves for the reservoir and spillway shown above. This outflow was routed through the reach between the reservoirs downstream to the proposed Centennial Reservoir and combined with the additional runoff from the watershed between Rollins and Centennial Reservoirs.

2.3.1 Rollins Reservoir

The resulting PMF inflow to Rollins Reservoir is approximately 80,888 cfs. The outflow from Rollins Reservoir was 78,700 cfs. This compares to 74,928 cfs inflow and 73,149 cfs outflow reported in Montgomery Group 2006 study.

2.3.2 Centennial Reservoir

The inflow hydrograph to Centennial Reservoir is shown in Figure 7. The maximum inflow to the reservoir is 89,181 cfs.

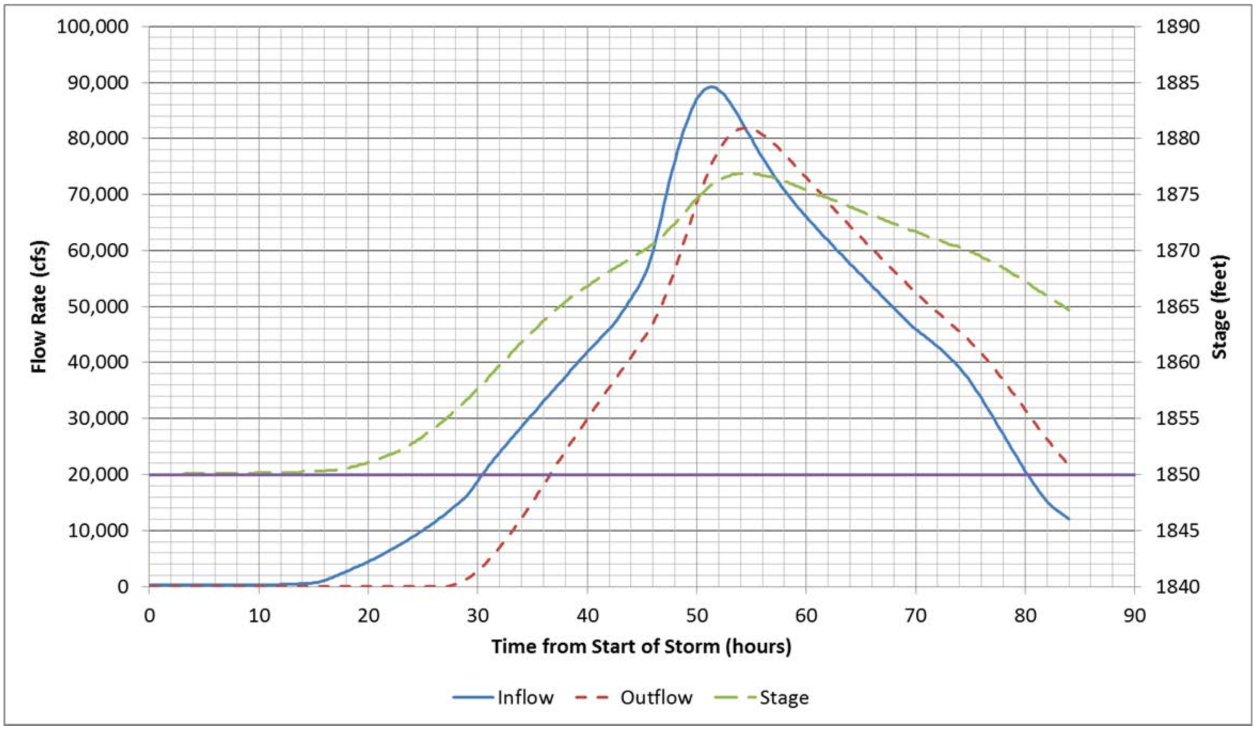

Figure 7. Inflow Hydrograph and Outflow Hydrograph for Centennial Reservoir PMF with 200 foot Wide Spillway

Outflow Hydrograph

Using the stage-storage curve for the reservoir and the spillway rating curve above, the PMF inflow was routed in HEC-HMS to generate the PMF outflow. For a reservoir storage capacity of 110,000 acre-feet, the maximum normal reservoir water surface (i.e., spillway crest) would be elevation 1855 feet for flood routing purposes.

The peak outflow is dependent upon the assumptions used in the spillway design. An ogee spillway was assumed using two different design heads, 20 and 25 feet. This results in a spillway coefficient that varies with head on the weir. In HEC-HMS the discharge coefficient in the weir flow equation is automatically adjusted when the upstream energy head is above or below the design head. The range in discharge coefficients is provided in Table 6. Spillway lengths from 150 to 220 were simulated. The results are summarized in Table 6.

The surcharge on the spillway varies from 20.8 to almost 24 feet. The peak outflow varied from 80,800 cfs to 82,600 cfs.

Table 6. Results of HEC-HMS model for Centennial Reservoir PMF Study

Spillway Width (ft)	Design Head (ft)	Peak Outflow cfs)	Max Reservoir Elevation (ft)	Peak Surcharge over Spillway* (ft)	Range in Weir Coefficient
180	20	80,883	1878.3	23.3	2.7-3.8
190	20	81,368	1877.6	22.6	2.8-4.0
200	20	81,837	1876.9	21.9	2.8-4.0
210	20	82,207	1876.3	21.3	2.8-4.0
220	20	82,568	1875.8	20.8	2.7-3.8
150	25	79,069	1881.3	26.3	3.0–3.9
180	25	80,843	1878.7	23.7	2.8-3.9
190	25	81274	1878	23.0	2.8-3.9
200	25	81,684	1877.3	22.3	2.8-3.9
210	25	82,043	1876.7	21.7	2.8-3.9

*Spillway crest at elevation 1855 feet.

3.0 References

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