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AECOM

Centennial Reservoir Project
Roller Compacted Concrete Dam
Conceptual Design Criteria
Technical Memorandum

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List of Acronyms

ACI	American Concrete Institute
AEP	Annual exceedance probability
ASTM	American Society for Testing and Materials
c'	Cohesion strength parameter
DCTM	Design Criteria Technical Memorandum
DSHA	Deterministic seismic hazard analysis
DSOD	California Division of Safety of Dams
f'_c	Concrete compressive strength
FERC	Federal Energy Regulatory Commission
FS	Factor of safety
f'_t	Concrete tensile strength
HMR	Hydrometeorological Report
ICOLD	International Commission on Large dams
M	Moment magnitude
MCE	Maximum Credible Earthquake
NGA Relationship	Next Generation of Attenuation Relationship
NID	Nevada Irrigation District
NOAA	National Oceanic and Atmospheric Administration
NWS	National Weather Service
PEER Center	Pacific Earthquake Engineering Research Center
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RCC	Roller compacted concrete
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USCOLD	U.S. Committee on Large Dams (now U.S. Society on Dams)
ϕ	Angle of internal friction strength parameter

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1 Introduction

1.1 Background

The Nevada Irrigation District (NID) has initiated engineering and planning studies for a proposed reservoir project located on the Bear River at the site location shown in Figure 1-1. The site is between the existing Rollins and Combie Reservoirs, which are also owned and operated by NID. The NID has identified a reservoir storage capacity objective of 110,000 acre-feet for the project, which is known as the Centennial Reservoir.

A storage capacity of 110,000 acre-feet corresponds to a maximum normal reservoir water surface of approximately elevation 1855. The reservoir plan and reservoir storage-elevation curve are shown on Figure 1-2. A 275-foot-high dam above the Bear River will be required to store the reservoir.

The geotechnical information presented in the Preliminary Geotechnical Investigation, Phase II Report (AECOM, 2016), describes the initial geotechnical investigations performed to assess the site conditions for the proposed dam, and to evaluate the potential dam axis locations and dam types. AECOM is currently working on the Phase III geotechnical investigation that focused on dam foundation and rock borrow areas. The results of the combined Phase II and Phase III investigations will be presented in the Geotechnical Engineering Report, Phase III.

The work described in this technical memorandum was authorized under the agreement between AECOM and NID dated April 15, 2015, and in accordance with Task Order No. 7, executed on August 30, 2016.

1.2 Purpose and Scope

This Design Criteria Technical Memorandum (DCTM) documents the criteria and analyses to be used for conceptual design of a roller compacted concrete (RCC) dam for the Centennial Reservoir Project.

The DCTM is considered to be a "living document"; i.e., the discussions presented herein will be reviewed and 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 project performance requirements, spillway and diversion flood criteria, stability and seismic design criteria, and DSOD criteria. The DCTM includes known relevant constraints such as environmental restrictions and borrow area location constraints.

1.3 Organization of Technical Memorandum

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

- Section 2 describes the proposed project, including the dam foundation objective and surface treatment, foundation grouting and drainage, conceptual layout of the RCC dam and appurtenant structures, and construction materials.
- Section 3 discusses the RCC dam stability criteria including dam foundation evaluation, dam material properties for analysis, design load cases, and seismic design criteria.
- Section 4 discusses hydraulic and hydrologic criteria including handling floods during construction, spillway design storm and flood, freeboard, and reservoir evacuation requirements.
- Section 5 lists the references cited in this technical memorandum.

1.4 Limitations

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 technical memorandum.

2 RCC Dam Description

2.1 General

The conceptual plan and sections of the RCC dam are shown on Figures 2-1 and 2-2 that illustrate the general arrangement and the main features of the RCC dam. The description of Centennial Dam and Reservoir is summarized in Table 2-1.

Table 2-1. Description of Centennial Dam and Reservoir

Stream	Bear River
Location	Between Combie and Rollins Reservoirs
Purpose	Irrigation (main use), municipal and domestic use
Drainage Area	123 mi^2
Reservoir Storage	110,000 acre-feet
Reservoir Pool Elevations	
Streambed	1600
At Spillway Crest	1855
Reservoir Areas	
At Spillway Crest	1281 acres
At Top of Dam	1508 acres
Length of Reservoir	6¼ miles
Dam	
Type	Roller-compacted concrete (RCC) gravity
Elevation-Top of Dam	1875 approx. (to be confirmed based on PMF routing)
Freeboard-Spillway Crest to Top of Dam	20 feet (to be confirmed based on PMF routing)
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. 1800 feet
Spillway	
Type	Uncontrolled ogee crest overflow bay; stepped downstream face
Crest Length	Based on PMF routing
Energy Dissipater	Stilling basin
Outlet Works	
Type	Pipe through dam, upstream guard gate & downstream regulating valve
Location	Single–level intake at bottom of dam, with allowance for dead storage
Horizontal Datum	NAD 83 CA State Plane Zone 2
Vertical Datum	NAVD 88

2.2 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 (AECOM, 2016 and 2017). The rock is variably weathered and fractured. All soils and 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. 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 will be evaluated to confirm its acceptability to satisfy stability criteria. The degree of weathering, along with fracture intensity and strength, will be used to estimate dam foundation excavation level. The degree of rock weathering is defined in Table 2-2.

The rock characterization at Axis 2 described in the Phase III Geotechnical Engineering Report (AECOM, 2017) will be used to inform decision making on the configuration of the dam foundation. The rock characterization will also be used to assess the strength properties of the dam foundation to confirm that the stability criteria are met. The recommended dam foundation configuration will be the subject of the forthcoming Conceptual Engineering Report.

Table 2-2. Rock Weathering

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Extremely Weathered/Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, although original fabric may be intact; material can be granulated by hand
Highly Weathered/Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 2-inch-diameter sample can be broken readily by hand across rock fabric
Moderately Weathered/Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 2-inch-diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered/Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering/alteration

Foundation surface treatment will include cleaning for geologic mapping, final foundation cleaning prior to RCC placement, surface preparation, and leveling concrete placed on the foundation to provide a platform to commence RCC placement. 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.

2.3 Foundation Grouting and Drainage

Grouting will be needed to control seepage through the foundation rock. The configuration of the grout curtain will be 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 grout curtain layout will include two grout curtains. The grout holes in each curtain would be angled in opposing directions to more effectively intersect the near-vertical rock discontinuities that are common at the site. No reduction of uplift pressures will be taken across the grout curtain used in the dam stability analyses described in this DCTM.

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.

Drain holes to control uplift pressures beneath the RCC dam will also be required. The conceptual design includes drain holes drilled from a gallery within the dam, spaced on 10-foot centers. The depth of the drain holes will be based on the geotechnical investigation data and on guidelines in USACE (1995) and USBR (1976), which relate drain hole depth to grout curtain depth. Besides foundation drains, body drains and contraction joint drains near the upstream side of the dam will control potential seepage along lift lines and minimize uplift.

As discussed in this DCTM, stability analyses will be based on both effective and ineffective drainage. The drain efficiency for the stability analyses is discussed in Section 3.8.

2.4 Conceptual Layout of Dam and Appurtenant Structures

The conceptual plan and section of the RCC dam are shown on Figures 2-1 and 2-2. It is expected that the cross section will have a vertical upstream face, a 0.8H:1V stepped downstream face, and a 30-foot wide crest. The dam would be constructed with one-foot thick lifts of RCC. Conventional concrete or grout-enriched facings will be used on the upstream and downstream sides of the dam. These two facing types have been successfully used on many RCC dam projects.

The RCC dam will include a spillway integral with the body of the dam, aligned to discharge flows directly into the Bear River channel. The width the spillway bay will be determined based on maximizing the available discharge width that can approximately match the river channel width immediately downstream. The spillway crest will allow for inclusion of bridge piers. The objective of the spillway layout will be to minimize the dam crest elevation and minimize the erosive power of the spillway discharge by reducing the required unit discharge (flow per foot of spillway width).

A stilling basin will be located at the toe of the spillway to dissipate energy prior to releasing flows back to the river channel. Reinforced concrete training walls will be constructed on each side of the spillway bay and stilling basin to contain the discharge flows.

The conceptual design also includes a low level outlet conduit that will be cast into the body of the dam. The conduit will include a single low-level intake, located near the base of the dam, in accordance with NID’s requirements. At the outlet, the conduit will be configured with a bifurcation and a blind flange for potential future addition of a power plant at the downstream toe of the dam. The outlet conduit will be designed with sufficient capacity to draw down the reservoir in accordance with the reservoir evacuation requirements described in Section 4.4.

Performance monitoring instrumentation will be included in the design of the dam. At a minimum, instrumentation will be included to monitor reservoir levels, uplift, seepage, and crest movement.

2.5 Construction Materials

Rock for RCC aggregate may be obtained from on-site rock borrow areas (see Figure 1-2). If possible, the on-site rock borrow areas will be located within the reservoir area to reduce environmental impacts. The rock borrow area would first need to be stripped of overburden and weathered rock. The underlying fresh rock would be drilled, blasted, crushed and screened to produce the RCC aggregate.

An alternative source of material may be available at the existing Bear River Quarry located within one-half mile south of the dam site (see Figure 1-2). This material would also require quarrying and processing to produce suitable aggregate.

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 flyash will conform to specified ASTM standards, which will be certified by the manufacturer (through the supplier) and verified with supplied test data.

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. The RCC will be faced with conventional concrete or grout enriched RCC placed at the same time as the RCC.

3 Dam Stability Criteria and Material Requirements

3.1 Stability Criteria

The criteria for moment equilibrium require that stability against overturning be maintained with an adequate margin of safety. Likewise, the criteria for sliding stability require that sliding stability of the dam be maintained with an adequate margin of safety. Both the non-overflow (abutment) and spillway sections of the dam will be analyzed for overturning and sliding conditions.

The stability analyses will consider the following loads: (a) weight of RCC and concrete, (b) reservoir water, (c) tailwater, (d) uplift, and (e) seismic (inertial and hydrodynamic). 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.

Sliding along RCC lift joints will be evaluated as described in USACE EM 1110-2-2006, 5-2.c and 4-2.c (2) (USACE, 2000). Uplift within the body of the dam will follow the recommendations of EM 1110-2-2006, 5-2.b. As stated in Section 2.3, body drains and contraction joint drains near the upstream side of the dam will control potential seepage along lift lines and minimize uplift.

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 Tables 3-1.

Table 3-1. 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 [4]	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.

The sliding stability factor of safety of the dam will be calculated using the following equation:

$$FS = \frac{c \times A + F_V \times Tan(\phi)}{F_H}$$

Where:
- FS = sliding factor of safety
- c = foundation cohesion
- A = area of uncracked base
- F_V = summation of vertical forces, including uplift
- Tan (ϕ) = coefficient of friction along sliding plane
- F_H = summation of horizontal forces at assumed contact.

The allowable bearing capacity in the rock foundation will be based on RQD, which considers rock fracturing (Peck, 1976).

3.2 Allowable Concrete Stress

The compressive stresses (σ_z) computed (without internal water pressure) at the upstream face of the dam will be compared with the minimum allowable stress (σ_{zu}) based on the following equation (USBR, 1977). This equation takes into account internal water pressures and concrete tensile strength.

$$\sigma_{zu} = pwh - \left(\frac{f_t}{s}\right)$$

Where:

- *p* = reduction factor to account for drains:
  - = 0.4 if drains are present and effective
  - = 1.0 if drains are not present or inoperable
- w = unit weight of water
- h = depth below water surface
- f_t = tensile strength of RCC at lift surfaces
- s = safety factor = 3.0 (usual), 2.0 (unusual)

Typically, no tension is permitted to develop across the dam-foundation interface or within the body of the concrete ($\sigma_{Zu} > 0$) for usual loads. For unusual loads, tensile stress is allowed, but cracking is not permitted ($\sigma_z > \sigma_{zu}$). The allowable compression in the RCC will be 0.33 f'c (180 day). These stresses will be increased 15% for unusual load conditions and 50% for extreme load conditions in accordance with EM 1110-2-2100, 3-10 (USACE, 2005).

3.3 Peak Ground Acceleration

This section summarizes the preliminary seismic design parameters for use in stability analyses (AECOM, 2016).

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

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

The 69th percentile deterministic spectra developed for each of the four ground motion prediction models along with the geometric mean are presented in AECOM (2016). The median, 69th and 84th percentile geometric mean deterministic spectra are also compared in AECOM (2016). The median, 69th and 84th percentile peak horizontal ground accelerations (PGAs) are 0.23, 0.31 and 0.42 g, respectively.

Based on DSOD guidelines (Fraser and Howard, 2002), the minimum earthquake peak ground acceleration (PGA) for new and existing dams should be 0.25 g. Considering this, the 69th percentile deterministic ground motions will be used for design of the dam (PGA of 0.31g). This is consistent with DSOD guidelines and recommendations by U.S. Committee on Large Dams (1985; 1998).

3.4 Methods of Analysis

The spillway and abutment monoliths will be 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 no 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 that state will not develop.

Limit-equilibrium analysis is adequate to evaluate static loading conditions for conceptual design. A finite element model (FEM) will be developed for the dynamic analyses (frequency or time domain). The FEM yields detailed static stress results that will be used to develop stress plots.

3.5 Seismic Stability

The sophistication of seismic stability analyses will progress as the level of design progresses in accordance with ER 1110-2-1806 (USACE, 2016). For preliminary design, a dam that meets sliding stability factor of safety requirements when evaluated by the seismic coefficient method (using a coefficient equal to ⅔ PGA) are considered to be safe and no additional seismic stability analyses are required at this stage.

In seismic coefficient methods, the hydrodynamic effects of the reservoir will be approximated by using the Westergaard method (Westergaard, 1933; in USACE, 2005).

$$P_E = (7/12)\, k_h \gamma_w h^2$$

Where:

P_E = hydrodynamic force per unit length
K_h = horizontal seismic coefficient
γ_w = unit weight of water
h = water depth.

The hydrodynamic force is added algebraically to the static water pressure force to get the total water force on the structure. The pressure distribution is parabolic and the line of action for the force P_E is 0.4h above the ground surface.

During final design, response spectrum or time-history finite element analyses will be used to assess the demands placed on the structure and foundation during the MCE. The results of these analyses will be used to demonstrate that displacements and stresses experienced by the dam and foundation are acceptable. If these more advanced analyses indicate zones of tension will be generated to the degree that cracking occurs within the dam or along the base, the limit equilibrium analyses will be run incorporating the cracking. This immediate post-earthquake analysis will be run with the reservoir at the spillway crest and evaluated against the Unusual Criteria (Table 3-1).

3.6 Thermal Considerations

Thermal mitigation and minimizing/controlling heat generation will be considered in the RCC mix design. Temperature control of the RCC mix during construction will be accomplished by shading and/or water spraying of the aggregate stockpiles, night-time RCC placement, and liquid nitrogen injection of the RCC mix at the batch plant.

RCC thermal studies will be the subject of a subsequent detailed design phase. The actual RCC mix design would be needed for the thermal analysis, along with the sequence of construction. Such studies are not needed for conceptual engineering.

3.7 Foundation Shear Strength

The sliding stability of a gravity dam is controlled either by the sliding potential on (a) a continuous or semi continuous sub-horizontal weak plane in the foundation or (b) through the rock mass itself. The dam/foundation interface will be inspected during construction and is not typically a critical failure scenario.

(a) Two failure modes are possible. One involves failure along the weak plane (directly under the structure) and along its extension until it daylights. The other mode involves slippage along the weak plane directly under the structure plus slippage along a plane through the foundation above the weak plane.

Evaluation of the shear capacity along the weak plane will be made using the Barton-Bandis (1990) criterion. The criterion is:

$$\tau = \sigma'_n \tan\left(\phi_b + JRC.log_{10}\left(\frac{JCS}{\sigma_n}\right)\right)$$

JRC = joint roughness coefficient,

JCS = joint compressive strength,

σ_n = normal stress across the joint

ϕ_b = basic friction angle (estimated from literature for basaltic rock)

Estimates of joint roughness and variability for the second order asperities will be modified to account for scale effects (Barton-Bandis, 1982) and the actual size, shape and geometry of the surfaces expected to have shear displacement.

Barton-Bandis is a non-linear relationship, whereas the limit equilibrium analyses use linear strength parameters expressed in terms of a Mohr-Coulomb cohesion (c) and internal friction angle (ϕ). For the limit equilibrium analyses, instantaneous cohesion (c_i) and instantaneous friction angle (φ_i) will calculated for the average σ_n from the analyses. The instantaneous parameters describe the tangent line to the Barton-Bandis relationship curve at the point of interest, in this case the average σ_n.

(b) An estimate for the in-situ shear strength of the jointed rock mass in the foundation will made using the Hoek-Brown criterion (Hoek, et. al., 2002 and 1997) to develop equivalent Mohr-Coulomb failure parameters (c′ and ϕ'). The uniaxial compressive strength of the rock (ci), Hoek-Brown constant (mi) based on the rock type, a disturbance index (D) and the Geologic Strength Index (GSI) form the basis of the Hoek-Brown criterion.

3.8 Foundation Drain Efficiency

Drain efficiency of 67% will be used and the sensitivity of stability results to an efficiency as low as 50% will be checked. FERC and USBR guidelines use foundation drain efficiency (E) of 66% for new designs. This recommendation is based on compilations of uplift profiles from many existing dams. USACE limits E to less than 50% in the absence of site-specific uplift data. In addition, the condition for inoperable drains will be evaluated against the Unusual Criteria (Table 3-1).

3.9 Uplift

Uplift pressure distribution beneath the dam, for a given 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), the reduced pressure at the line of pressure relief wells, and the full tailwater head at the dam toe (or downstream edge of the impact 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. However, full head dissipation would probably occur somewhere near the center of the base.

3.10 Tailwater

For the Unusual and Extreme flood cases, an effective tailwater depth equal to 60 percent of the full tailwater depth will be used for lateral load calculations in accordance with USACE and FERC guidelines.

The analyses will not include the stabilizing effect of water above the ogee crest or on the downstream face. The ogee will be designed for the PMF so that negative pressures are not created at the crest.

3.11 RCC Material Properties

The RCC mix design will be based on the properties of local aggregates, and varying proportions of cement and pozzolans. The design will account for static and dynamic requirements for tensile strength across and shear strength along lift surfaces, and will address thermal requirements to minimize heat generation. Aggregate durability will satisfy ASTM C-33, Standard Specification for Concrete Aggregates.

For conceptual design, estimates of the static direct tensile strength of the RCC and of the tensile strength across lift joints will be based on the relationships in USACE-EP 1110-2-12 (1995) wherein tensile strength (f'_t) is shown as a function of unconfined compressive strength (f'_c) for various maximum aggregate sizes, RCC lift joint preparation (with and without bedding mortar), and consolidation times (Vebe time). For conceptual design purposes, the unconfined compressive strength of the RCC will be assumed at 2500 pounds per square (psi). This strength value has been found for several RCC dams (Hansen, 1991).

A unit weight of 150 pounds per cubic foot (pcf) will be used based on data from similar RCC dams (Hansen, 1991). An instantaneous modulus of elasticity of the RCC concrete will be based on published data for similar RCC dams. In accordance with the USACE (1995) for long-term loadings, the effective modulus of elasticity will be assumed to be 2/3 of the static modulus of elasticity. This is consistent with the USBR (1977), which states that the sustained modulus should be taken as 60% to 70% of the instantaneous modulus of elasticity.

The density will be included as a target property during mix design. The RCC paste content will be sufficient to provide for an “impermeable” RCC mix.

3.12 Reinforced Concrete Design Requirements

Reinforced concrete design for the spillway and outlet works will be in accordance with the American Concrete Institute ACI 318, Building Code Requirements for Reinforced Concrete (2014). All reinforced concrete design will be in accordance with the ACI Strength Design Procedures and USACE EM 1110-2-2104, Strength Design for Reinforced Concrete Hydraulic Structures (1992). The minimum 28-day compressive strength of reinforced concrete structures will be 4000 psi.

The loads for hydraulic structures are summarized below:

– A unit weight of 150 pounds per cubic foot (pcf) will be used for concrete.

- Lateral loads on walls from soil, at-rest, active, passive, and seismic earth pressures will be developed based on backfill soil properties (unit weight of 130 pcf, friction angle of 35 degrees, and zero cohesion).

4 Hydraulic and Hydrologic Criteria

4.1 Handling Floods During Construction

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.

Diversion of river flow through the dam site will be accomplished by a diversion structure (e.g., a box culvert) constructed in the river channel (the RCC dam would be constructed on top of the culvert), or through a tunnel excavated in an abutment. When diversion is no longer necessary, the culvert or tunnel would be plugged or converted into a permanent auxiliary outlet. A small cofferdam will 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 after a short time following placement.

4.2 Design Storm and Spillway Design Flood

Based on DSOD criteria for dam height, reservoir size and downstream hazard potential, the spillway design flood will be the Probable Maximum Flood (PMF) resulting from the Probable Maximum Precipitation (PMP). The PMP will be estimated using the procedures and data presented in Hydrometeorological Reports (HMR) 58/59 (NOAA, 1999).

The design storms [1:300 annual exceedance probability (AEP), and PMF] will be developed with a HEC-HMS rainfall-runoff model of the watershed and reservoir tributary to the dam. The inflow hydrographs will consider spillway outflow from Rollins Reservoir upstream of Centennial Dam. Rollins Reservoir will be assumed to be full to its spillway crest for flood routing to the Centennial Dam site.

Design storm precipitation will be input into the model to calculate the reservoir inflow hydrograph. This hydrograph will be routed through the reservoir and spillway to demonstrate spillway adequacy, stability criteria, and to confirm freeboard requirements.

4.3 Freeboard

The PMF water surface will be contained to 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 that will be anchored into the RCC.

4.4 Reservoir Evacuation Requirements

The DSOD requires that dams be provided with outlet facilities with sufficient capacity to evacuate the reservoir quickly, should an unsafe condition develop at the dam. DSOD “guidelines” for emergency drawdown rate for large reservoirs are that outlet facilities be able to:

- Lower the reservoir elevation by an amount equal to ten percent of the hydraulic head behind the dam in seven 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.

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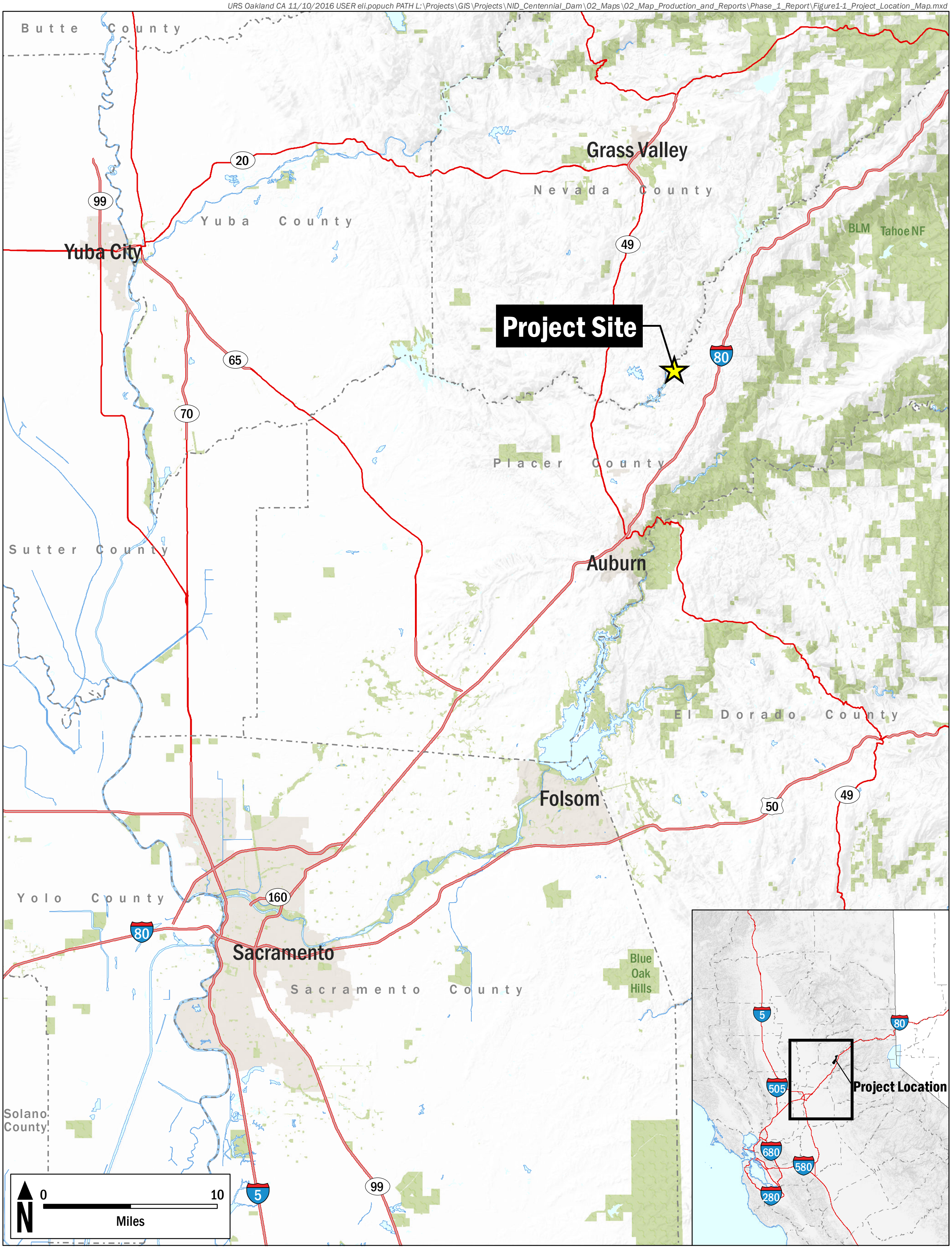

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FIGURE 1-1
Project Location Map

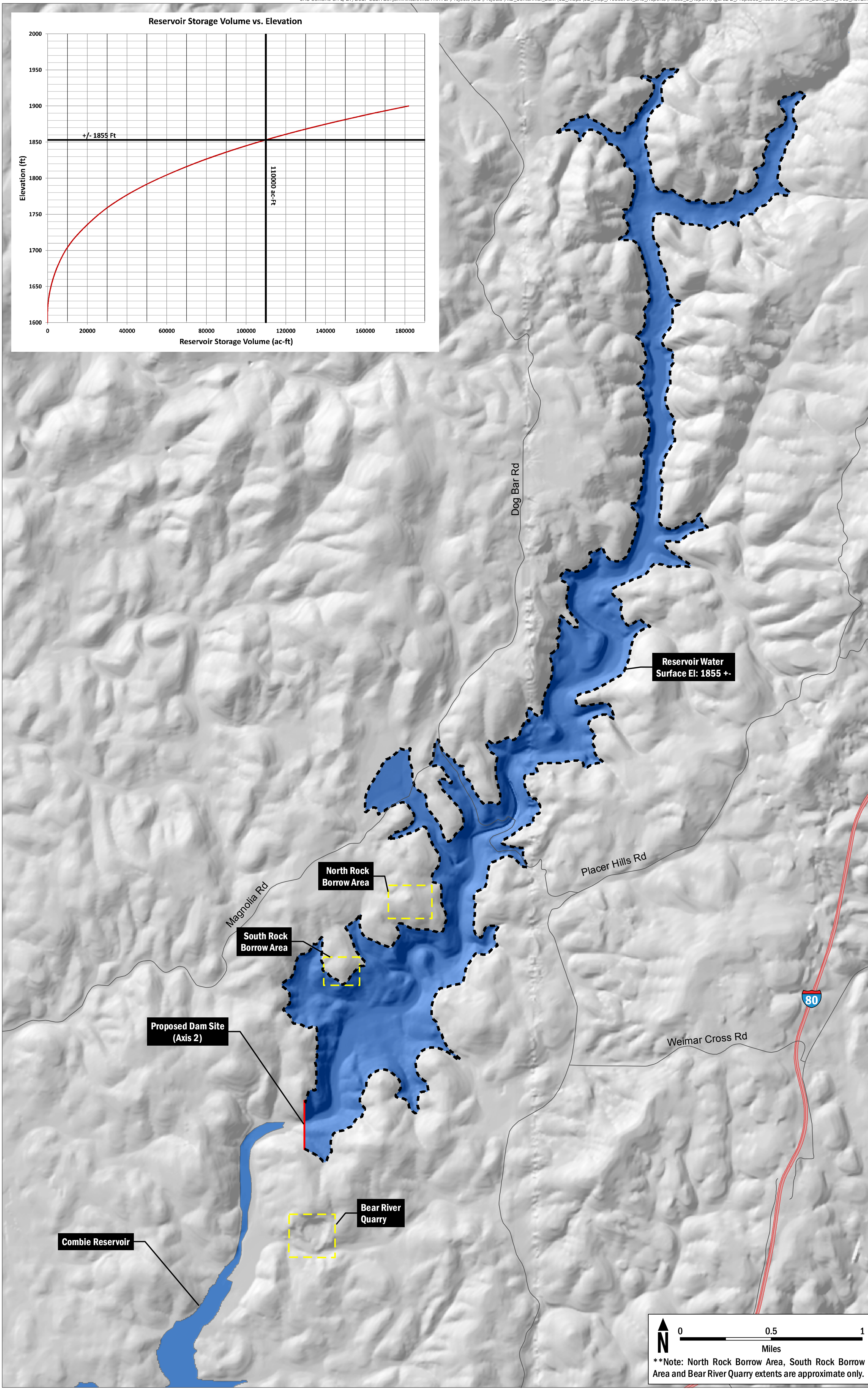

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FIGURE 1-2

Centennial Reservoir Plan

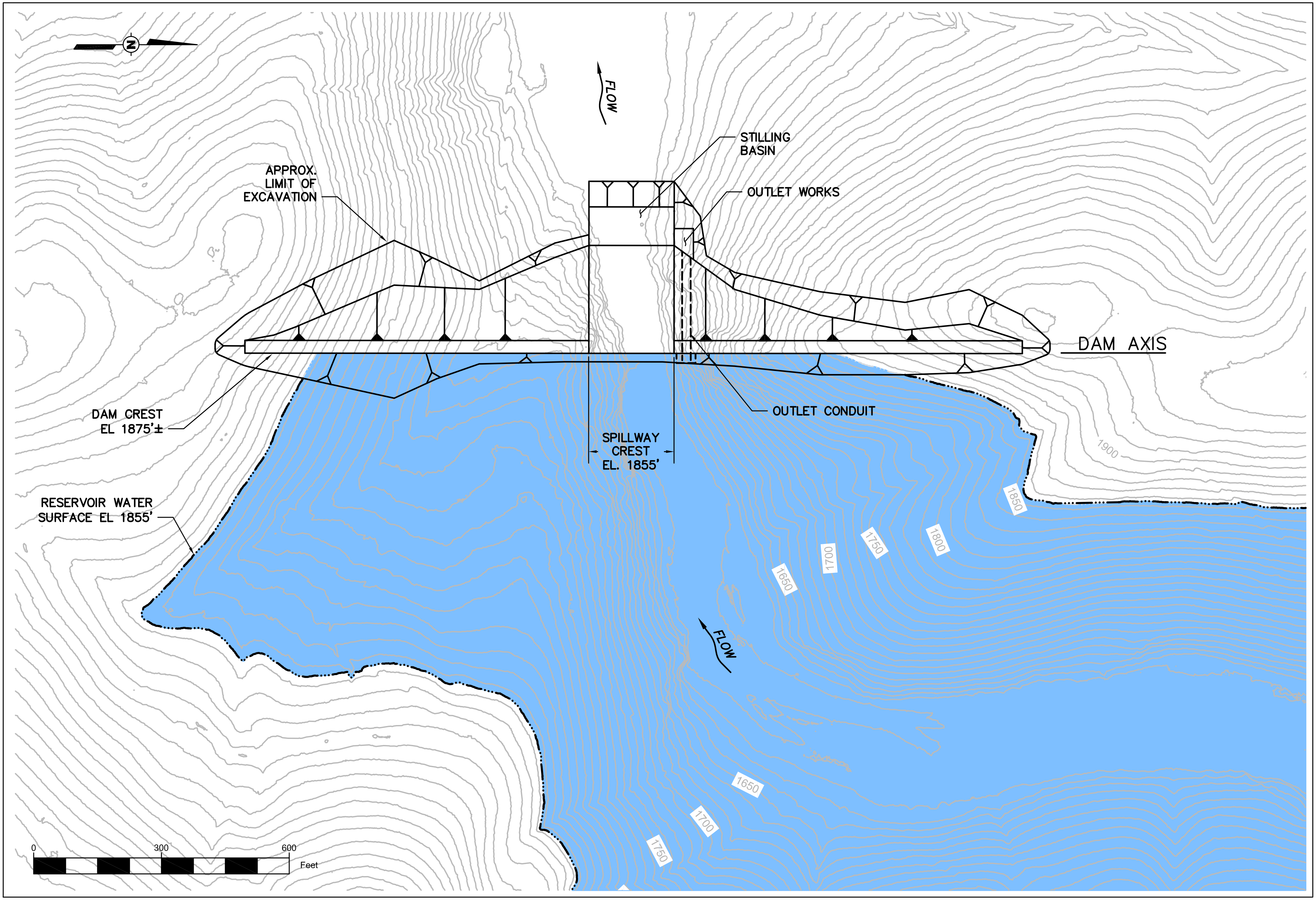

Figure 2-1. Dam Concept-Plan

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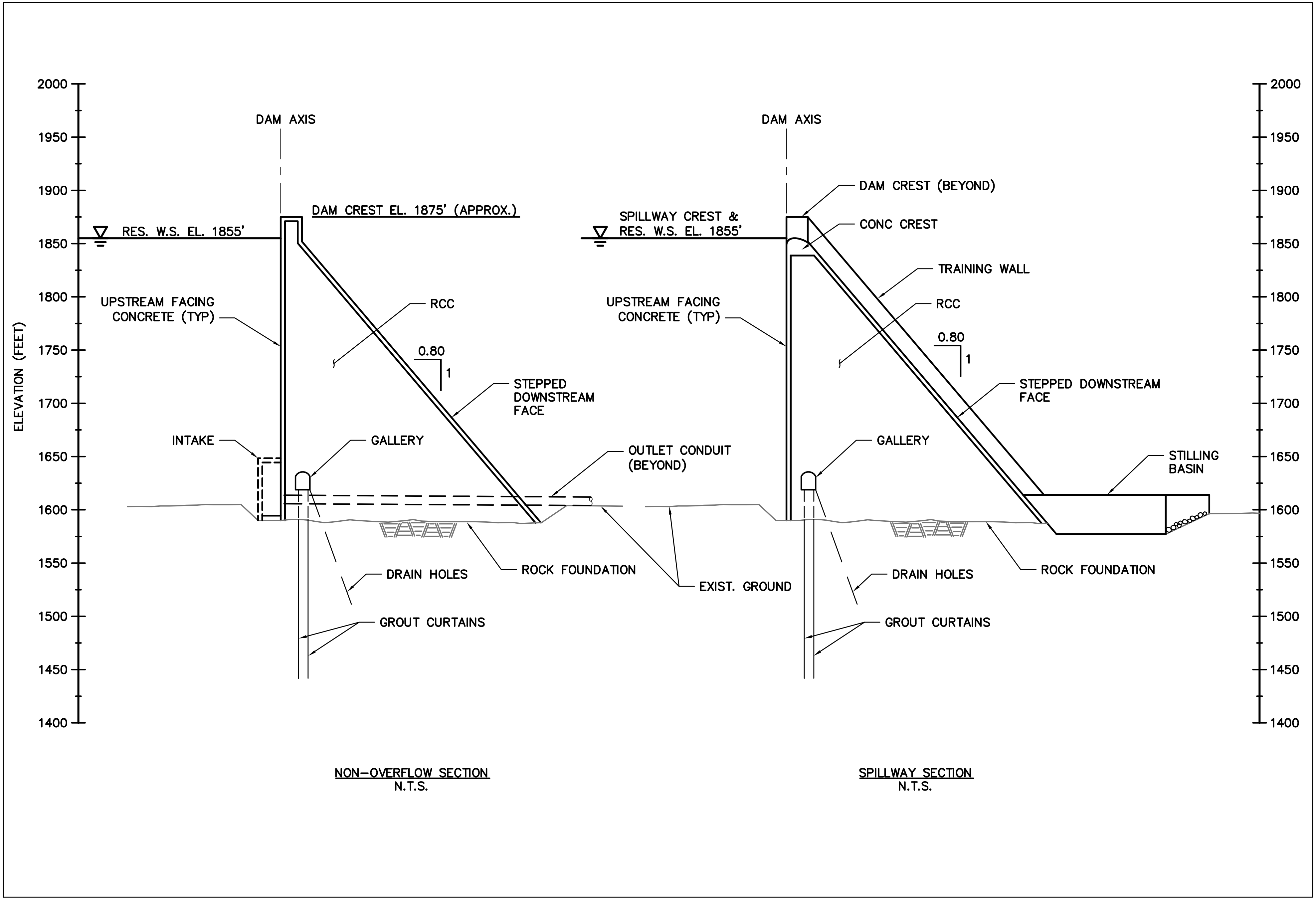

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Figure 2-2. Dam - Maximum Sections

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