R E V I S E D   D R A F T   R E P O R T

Feasibility Study
North Fork Cowiche Creek Reservoir

Yakima River Basin Integrated Plan
Yakima, Washington

Prepared for

State of Washington Department of Ecology
Yakima-Tieton Irrigation District

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ch2m

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Acronyms and Abbreviations

AID	Ahtanum Irrigation District
APE	area of potential effect
BGEPA	Bald and Golden Eagle Protection Act
CFRD	concrete face rockfill dam
cfs	cubic feet per second
CH2M	CH2M HILL Engineers, Inc.
CWA	Clean Water Act
cy	cubic yard
DAHP	Department of Archaeology and Historic Preservation
DOE	determination of eligibility
EA	environmental assessment
ECOS	Environmental Conservation Online System
ESI	Earthquake Severity Index
Estimate	conceptual design phase construction cost estimate
FCR	French Canyon Reservoir
FTE	full time equivalent
GERCC	grout-enriched roller compacted concrete
GIS	geographic information system
HMR	Hydrometeorological Report
ICOLD	International Commission on Large Dams
M_W	moment magnitude
MBTA	Migratory Bird Treaty Act
MCE	maximum credible earthquake
NEPA	National Environmental Policy Act
NFCCR	North Fork Cowiche Creek Reservoir
NHD	National Hydrography Dataset
NHPA	National Historic Preservation Act
NMFS	National Marine and Fisheries Service
NRHP	National Register of Historic Places
NWI	National Wetlands Inventory
NYCD	North Yakima Conservation District
O&M	operations and maintenance
PHS	Priority Habits and Species

PMF	probable maximum flood
PMP	probable maximum precipitation
RCC	roller-compacted concrete
SEPA	State Environmental Policy Act
SHPO	State Historic Preservation Office
TDH	total dynamic head
TICW	Tieton River Gauge Below YTID Diversion
TWSA	total water supply available
USACE	U.S. Army Corps of Engineers
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
WDFW	Washington Department of Fish and Wildlife
WISAARD	Washington Information System for Architectural and Archaeological Records Database
WNHP	Washington Natural Heritage Program
WSE	Water Surface Elevations
YTID	Yakima-Tieton Irrigation District

SECTION 1

Introduction

In 1984, The Yakima-Tieton Irrigation District (YTID) constructed the French Canyon Dam and Regulating Reservoir. The existing dam and reservoir are located approximately 2.0 miles west of Tieton, Washington, on the North Fork Cowiche Creek. The dam forms a small regulating reservoir (French Canyon Reservoir, FCR) with a total operational storage capacity of approximately 500 acre-feet. FCR is primarily filled using YTID's Tieton Canal, a 12-mile-long gravity aqueduct that diverts water from the Tieton River near the unincorporated community of Rimrock Retreat. The existing reservoir also receives inflow from the North Fork Cowiche Creek during the spring and early summer. The reservoir serves as a small storage buffer at the upper end of YTID's service area. Water from the reservoir is distributed throughout YTID's 28,000-acre service area via approximately 200 miles of buried pressure pipelines. The water is primarily used to irrigate high-value apples, cherries, and pears.

The North Fork Cowiche Creek Reservoir (NFCCR) is a proposed off-stream water storage reservoir located approximately 0.5 mile upstream of FCR on the North Fork Cowiche Creek (see Figure 1-1). This 35,000 acre-foot reservoir would increase available water supplies in the lower Yakima River Watershed and provide agricultural and environmental benefits consistent with the goals and objectives of the Yakima Basin Integrated Plan.

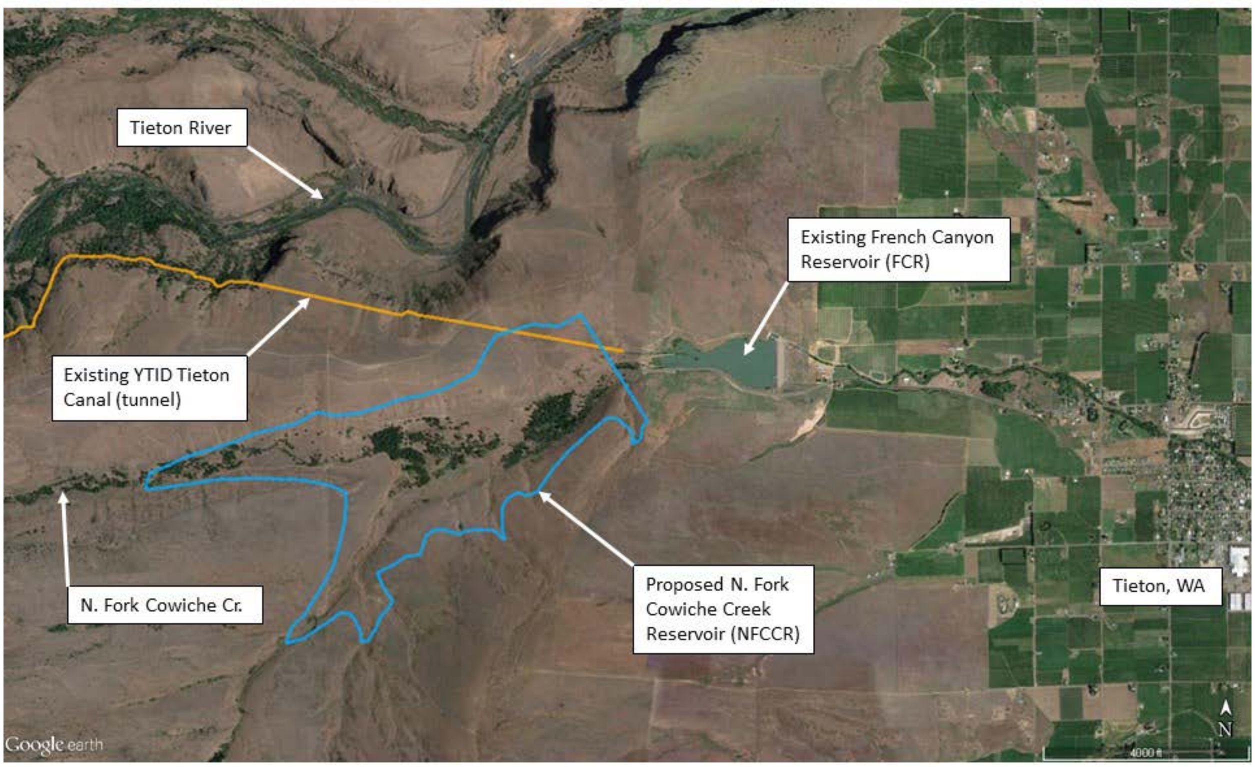


Figure 1-1. Project Area Map

1.1 Facilities and Operations

The proposed NFCCR would require construction of a dam approximately 240 feet high, a 15,000-horsepower pump station, and a 900- *to* 1,200-foot-long, 96-inch-diameter pipeline between the new and existing reservoirs (depending on the exact location of the new dam). Other major facilities would include a flow-control station, a bi-directional flow meter station, a spillway and stilling basin, and an intake/outlet structure where water enters and leaves the new reservoir.

The new reservoir would be operated in conjunction with the existing reservoir as a pump-storage project. During the spring months, when YTID is not using the full capacity of its canal, available water from the Tieton River would be diverted through the canal to the FCR. This water would be pumped from the FCR to the NFCCR. When the water is needed for agriculture or environmental purposes, the water would be released back to the lower reservoir and then delivered to end users through YTID's distribution system. The water could also be released to North Fork Cowiche Creek below the reservoirs. The additional storage volume created by the proposed reservoir would provide opportunities to increase beneficial uses of the basin's limited resources.

1.2 Cowiche Creek Water Exchange Project

The additional water storage could significantly benefit the environment. Cowiche Creek is a tributary to the Naches and Yakima Rivers. The creek passes through central portions of YTID's service area, and YTID's distribution system serves agricultural areas directly adjacent to the creek. For more than a decade, the creek has been a focal point for federal, state, and local efforts to restore anadromous salmon populations. Since 2004, YTID has been cooperating with the North Yakima Conservation District (NYCD) and their efforts to restore Cowiche Creek. Under an existing water exchange agreement, Cowiche Creek water users have agreed to discontinue direct withdrawals from the creek. Instead, they receive equivalent supplies of Tieton River water via YTID's distribution system. The water that remains in the creek provides higher fish passage and fish attraction flows that were not previously available in the summer and fall. In 2014, YTID completed two turnouts from their distribution system that serve approximately 400 acres within the Cowiche Creek Water Users Association. The initial efforts to restore Cowiche Creek have been very successful and have gained widespread acclaim from both agricultural and environmental interests. Construction of the new reservoir would directly support future phases of the Cowiche Creek Water Exchange Project. The new reservoir could eventually serve 1,250 acres of land near the creek and provide additional in-stream flow for fisheries restoration.

1.3 Other Potential Water Exchanges and Beneficial Uses

YTID is also exploring and cooperating with proposals for a similar water exchange agreement in the Ahtanum Creek Watershed. Water stored in the new reservoir could be made available for agricultural and environmental interests near Ahtanum Creek. Water supplies to Ahtanum Creek, the Wapato Irrigation Project, the Ahtanum Irrigation District, and other users could be considered after ongoing legal issues associated with the water rights adjudication within this sub-basin have been resolved. Water stored in this proposed reservoir could also be used in groundwater storage and recovery projects within these crucial tributary basins.

SECTION 2

Approach and Methodology

This section briefly describes the steps taken by CH2M HILL Engineers, Inc. (CH2M) to conduct the analysis and examine the feasibility of the project.

2.1 Collect and Review Existing Information

CH2M's initial step was to collect and review existing, relevant information related to the project. A wealth of relevant information is available from the design and 1984 construction of French Canyon Dam. Because the existing dam and reservoir are located immediately downstream of the proposed reservoir, topographic maps, geologic maps, geotechnical and seismic information, hydrology and spillway calculations, and as-built drawings from the existing dam are relevant for the new NFCCR. CH2M also collected and reviewed land ownership and land-use maps, archaeological and environmental studies, Tieton River flow data, and Rimrock Lake water storage and spill data. This initial step was vital in understanding conditions and data at the site, as required to evaluate dam and reservoir size, configuration, potential archaeological and environmental considerations, and reservoir operating criteria.

The project team also conducted a site visit during the spring of 2016 to make visual observations of existing facilities, topography, cultural and environmental resources, and other aspects of the project area that could affect project features and implementation.

2.2 Siting, Sizing, and Layout of Proposed Facilities

Based on review of existing information and field observation of topography, geology, seismicity, and hydrology, CH2M developed preliminary layouts for two types of dams in two alternate locations. This report presents preliminary sketches for both a roller-compacted concrete (RCC) gravity dam and a concrete face rockfill dam (CFRD). The analysis accounted for the types of materials available in the project area for dam construction and also provided configurations for an emergency spillway.

CH2M also developed preliminary layouts of the facilities needed to transfer water between the FCR and the NFCCR. These include an intake structure on FCR; a large pump station; large diameter pipeline; flow control and measurement vaults; and power transmission lines. Multiple sites and configurations were considered for the pump station, along with control schemes and operational flexibility. The analysis also included the potential for generating hydroelectric power during releases from the NFCCR. The pump station is sized for a nominal peak capacity of 370 cubic feet per second (cfs), consistent with an earlier study to replace the Tieton Canal. The large diameter pipeline transfers water in both directions: from the FCR to the NFCCR and from the NFCCR back to the FCR.

2.3 Assessing Project Operations

Project feasibility for a pumped storage project is a function not only of assessing the technical elements, costs, and impacts of the facilities, but also of assessing how the project would be operated. For the NFCCR project, CH2M evaluated the availability of water to fill the new reservoir, how stored water would be released and used downstream, and how some of the limiting factors would affect project yield. For example, the available capacity of the Tieton Canal limits the rate and volume at which water can be supplied to the FCR and NFCCR. A new reservoir is neither practical nor economical, if there is insufficient water to fill it.

2.4 Assessing Potential Project Impacts and Compliance Requirements

Understanding project feasibility also requires accounting for potential impacts on environmental and cultural resources. CH2M completed a preliminary assessment, based on existing information and field review, of resources present in the project area and permitting and environmental compliance requirements that could be expected should the project be implemented.

2.5 Estimating Project Costs

CH2M prepared construction cost estimates, as well as annual operations and maintenance cost estimates. Life cycle costs were also evaluated as a means of estimating the long-term value of the project relative to its expected long-term yield, and as a basis for comparison to other large water resources projects in the Yakima River Basin.

2.6 Conclusions and Recommendations

The final portion of the feasibility study summarizes findings, provides recommendations for the best apparent project concepts, and provides recommendations for next steps.

SECTION 3

Project Setting

3.1 Geology

Figure 17 in Appendix A shows a geologic map of the reservoir vicinity, based on existing geologic maps and site-specific geologic observations. Figure 18 in Appendix A shows a profile along the proposed dam centerline, with interpreted subsurface geology. Both figures also show proposed soil boring locations for a preliminary phase geotechnical exploration. The geologic symbols used on the map and associated geologic units are described in the following sub-sections.

3.1.1 Regional Geologic Setting

The site is located within the Columbia Basin physiographic province, which is underlain by the Columbia River Basalt Group—flood basalt flows ranging from 1,500 to 16,000 feet thick that issued from fissures and vents in southeastern Washington around 15 million years ago (Schuster and Moses, 2002; Lasmanis, 1991). The site is located in a subprovince that has been termed the Yakima Fold Belt, a series of anticlines, synclines, and thrust faults in the Columbia Basin of central Washington (Schuster and Moses, 2002; Lasmanis, 1991; Yeats et al., 1997; Reidel and Campbell, 1989; Campbell, 1989). The development of the folds began during the formation of the Columbia River Basalt Group, and has continued to the present, leading to the formation of numerous thrust faults within the Yakima Fold Belt (Schuster and Moses, 2002; Lasmanis, 1991; Yeats et al., 1997).

3.1.2 Bedrock Geology

Grande Ronde Basalt (Tgn2)

Bedrock within the proposed Cowiche Creek reservoir area consists primarily of basalt of the Grande Ronde Formation, described as non-porphyritic fine-grained, with normal magnetic polarity. These basalt flows form planar to stepped slopes with occasional cliffs and small benches between basalt flows. Based on site observations, the basalt is typically brown to gray, hard, and moderately to highly fractured. The fractured basalt breaks into angular talus deposits that mantle portions of the slopes. Based on observations along the road cuts and the French Canyon dam spillway and tunnel outlet, the uppermost several feet of basalt appeared to be highly weathered to rust-brown soil (saprolite) in some areas.

Tieton Andesite (Qvti)

Remnants of the Tieton Andesite were observed above the left (northern) abutment. This unit is described as several flows of andesite that erupted from vents in the southern Cascade Range. One of the flows extends down the Tieton River, and into the Naches and Cowiche creek valleys, and forms an intra-canyon flow. At the site, outcrops of the andesite typically manifest as hard, subrounded cobbles and boulders scattered on the upper slopes, overlying the basalt flows.

Ellensburg Formation (Tel)

A sedimentary interbed of the Ellensburg Formation is mapped in the hills just east of the proposed dam site. This formation overlies the Grande Ronde basalt and was originally deposited on the Grande Ronde as a sedimentary interbed; it consists of fluvial, laharic, and lacustrine facies. This unit is described as weakly-cemented, gray to white to light reddish brown, tuffaceous sandstone to siltstone that also contains clay and gravel. This formation may or may not be present near the dam abutments.

Surficial Geology

Surficial geologic units in the vicinity of the proposed site include stream alluvium, eolian sand dunes, eolian loess deposits, and colluvium. In the immediate vicinity of the proposed dam footprint, it appears that little or no surficial deposits cover the basalt outcrops:

- Ttg/Qal: Stream alluvium has been deposited by the North Fork Cowiche Creek and underlies the valley bottom. The alluvium is poorly exposed, but appears to consist primarily of stratified, interbedded, cross-bedded lenses of silty sand, sandy silt, and poorly graded rounded gravels. The thickness of the alluvium could not be determined by visually inspecting the surface, but is estimated to be relatively thin, probably less than 30 feet, based on the limited flow of the creek and because the creek appears to be down-cutting through basalt flows.
- Alluvial fans/colluvium/soil (af/c): A thin, discontinuous mantle of soil/loess deposits were observed in the vicinity; primarily on the lower-slopes on the north side of the proposed dam site. The colluvium/soil deposits consist primarily of tan brown fine silty sand to sandy silt. Based on visual observations, the thickness of the loess appears to be less than a few feet in the vicinity. Small alluvial fans have been deposited in steep intermittent side drainages.

3.1.3 Geologic Reconnaissance

An interpretation of the site geology was made based on visual observations during a brief geologic reconnaissance of the site. Other resources included the information from published geologic maps and the existing geotechnical information available for the French Canyon dam that is located about 3,000 feet downstream of the proposed site.

Based on this existing information, the proposed dam is underlain by the Grande Ronde basalt. At the abutments, it is assumed that the abutment rock is covered by a relatively thin (less than 10 feet) of soil, whereas the basalt in the valley floor is covered by a thicker alluvial layer. The depth to rock at the valley floor is expected to be generally less than approximately 30 feet and may vary along the dam axis.

Fractured zones may be present within the basalt flows that underlie the abutments and foundation, and weak interflow zones may be present under the proposed abutments. The northernmost part of the reservoir is bounded by a relatively narrow “saddle” that was interpreted as shallow surficial soils overlying potentially weak or fractured rock; this topographical feature may be attributable to a fault or shear zone. In addition, northeast-trending faults have been mapped southwest of the site. These faults project generally towards the right abutment of the proposed dam site. The faults are not interpreted to be active, but prehistoric faulting could have led to fractured zones and sheared zones in the basalt bedrock.

3.1.4 Potential Material Sources

The apparent material source for use as RCC aggregate as well as rockfill for the CFRD is the Grande Ronde basalt that underlies most of the dam and reservoir site. We have assumed that borrow quarries can be developed within the reservoir area and that haul distances will be limited to within 1 mile of the dam footprint.

Norman et al. discussed the properties of Grande Ronde basalt found within the Mount St. Helens 1:100,000 quadrangle, Washington, for use as a material source, and noted that the basalt generally indicates high-quality rock (Norman et al., 2001). The authors stated that the entablature and colonnade zones are most suitable for use as crushed aggregate, quarry stone, riprap, or decorative rock. They further noted that the rock is usually very hard and durable, with little weathering, and has many fractures and joints that developed during the cooling process.

Columns within the entablature zone are generally smaller than 1 foot in diameter and fractures, so that material from this zone is best for making crushed aggregate.

Norman et al. noted two potential issues with mining the colonnade: (1) some of the columns are too large (greater than 2 feet) to fit in a typical crusher; and (2) columns with platy joints can be excessively weathered where joints are closely spaced (Norman et al., 2001).

Norman et al. presented a few test results of Grande Ronde basalt with Los Angeles Abrasion values ranging from 19.7 to 27.6 and Specific Gravities ranging from 2.74 to 2.79 (Norman et al., 2001).

3.2 Seismicity

3.2.1 Regional Seismic Setting

The U.S. Geological Survey (USGS) maintains a database of quaternary (that is, the time period including the past 1.6 million years) faults and folds for the United States (USGS, 2006). Structures (faults and folds) with noted movement in the last 10 to 15 thousand years (late Quaternary or Holocene time period) are generally considered active for seismic hazard analyses. The quaternary faults documented by the USGS (2006) for the Yakima Fold Belt are shown on Figure 3-1 and listed in Table 3-1.

It is of importance to note that no identified faults are located on the project site, with the closest fault (Ahtanum Ridge Structures-Ahtanum Creek Fault) approximately 15 miles away.

3.2.2 Potential Seismic Sources

The principal tectonic feature of the Pacific Northwest is the active Cascadia Subduction Zone, where the Juan de Fuca Plate subducts beneath the North American Plate along the Cascadia margin. This subduction zone begins off the coast of Oregon and Washington and dips downward beneath western Oregon and Washington. Two primary seismic source mechanisms are associated with the subduction zone: (1) an interface source mechanism, and (2) an intraplate source mechanism.

In addition, there is potential for earthquakes from shallow crustal sources resulting from built-up tectonic stresses within the North American Plate.

The following subsections describe these three sources in more detail.

3.2.2.1 Cascadia Subduction Zone—Juan de Fuca-North American Plate Interface

Interface earthquakes occur at the interface boundary between the Juan de Fuca and North American Plate. This interface is a thrust fault and is located at depths of less than about 18 miles. Earthquakes generated from subduction zone interface sources are historically the largest earthquakes observed worldwide. According to Native American legends (Ludwin et al., 2005) and paleo-seismic and geologic evidence gathered from offshore and coastal regions of Washington and Oregon during the past several decades, very large earthquakes, with an estimated moment magnitude (M_w) of 8 to 9, have occurred. The USGS probabilistic seismic hazard study equally weighted magnitude 8.3 and 9.0 events for the interface source (Frankel et al., 2002) when generating the national seismic hazard maps.

Evidence indicates that these large earthquakes occur at intervals of 200 to 1,500 years, with an average return period of 500 to 600 years (Goldfinger, 2003). The last large earthquake occurred approximately 300 years ago, in the year 1700, and was estimated to be a magnitude of approximately 9.0 (Satake et al., 1996). Based on this information, the interface source is considered to be an active source.

The thrust fault dips at approximately 10 degrees to the east, and it has been suggested that the seismogenic portion of the fault is located just within the coastline in Washington (Hyndman and Wang, 1995; Wong et al., 2000), approximately 166 miles from the project site. The seismogenic portion of the subduction zone interface lies at a depth of 10 to 20 kilometers (km) (6 to 12 miles [mi]) beneath the earth's surface (Geomatrix Consultants, 1995).

3.2.2.2 Cascadia Subduction Zone—Intraplate Events

Intraplate earthquakes occur within the subducting Juan de Fuca plate, have a deeper focus than interface earthquakes, and typically occur along normal faults as a result of stress and physical changes in the subducting slab as it is pulled/pushed deeper into the asthenosphere. The events associated with this source are estimated to range from M_w 6 to 7.5, based on historical occurrences (Geomatrix Consultants, 1995).

Three earthquakes in recent history have been attributed to the intraplate source: the 1949, 1965, and 2001 earthquakes in the Puget Sound region, with M_w of 7.1, 6.5, and 6.8, respectively. No large intraplate earthquakes (M_w greater than 5.0) have occurred beneath the Yakima Fold Belt region, and it is anticipated that this source will not affect the project site given its distance to the site. The USGS (2006) considered the intraplate source active when generating the national seismic hazard maps. The intraplate source lies approximately 120 miles west and 28 to 31 miles beneath the ground surface (McCrory et al., 2004).

3.2.2.3 Crustal Sources

Crustal sources are shallow earthquakes occurring in the North American plate. The faults occur in the seismogenic portion of the crust, noted as approximately the upper 21 km (13 mi) of the crust in the Columbia Basin.

Crustal earthquakes are further categorized as occurring on *discrete fault sources,* where repeated earthquakes have occurred in the geologic past, or within *areal source zones* where earthquakes have been observed and will probably occur again but have not been associated with any specific geologic features. Table 3-1 and Figure 3-1 present the mapped crustal faults within the vicinity of the project site as identified by USGS (2006).

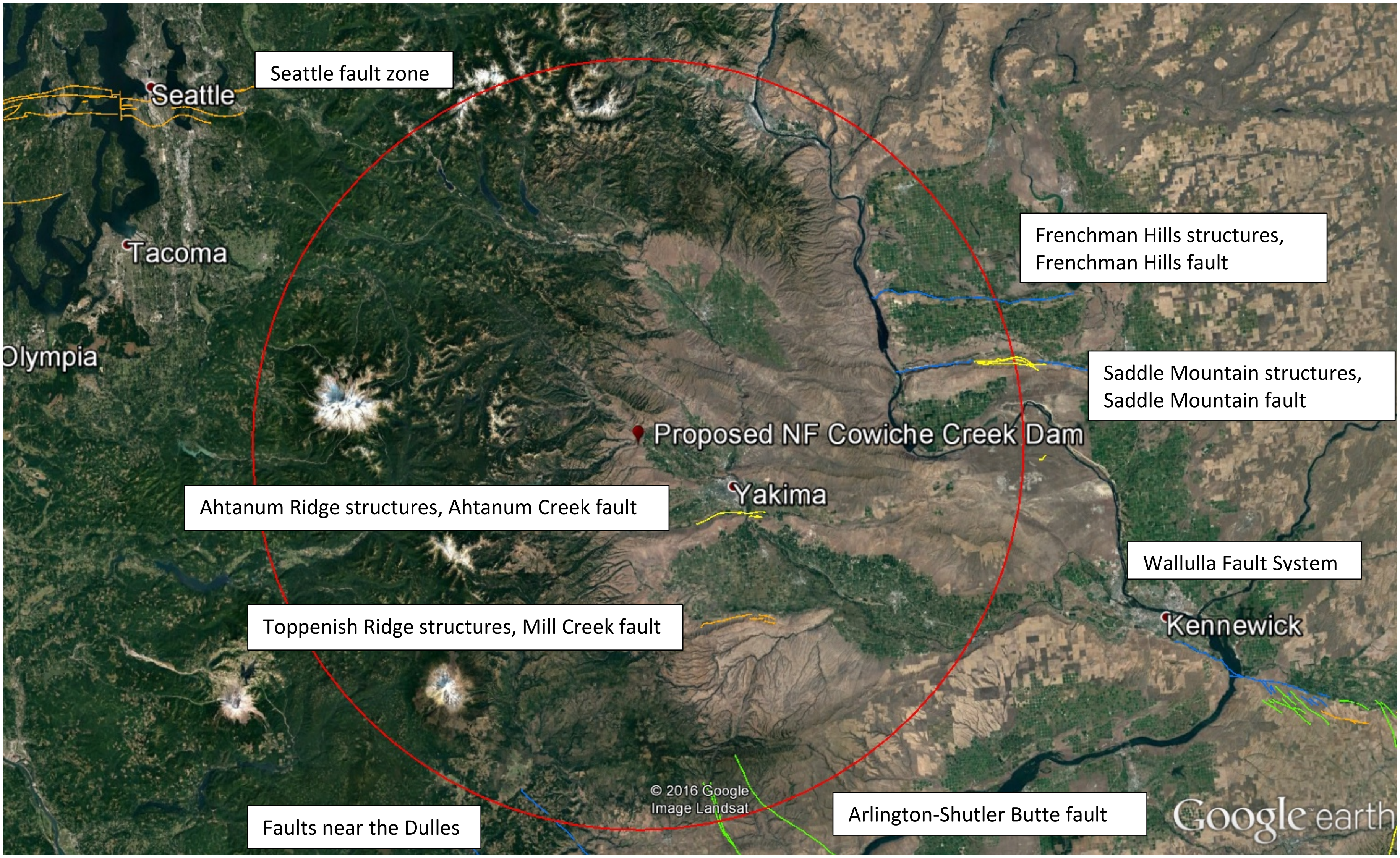


Figure 3-1. Regional Faults within 60-mile Radius of Site
North Fork Cowiche Creek Reservoir Feasibility Study

Table 3-1. Quaternary Faults Located within 60 Miles of the Site from the USGS Quaternary Fault and Fold Database
See Figure 3-1 for fault locations.

Name	Reference	USGS No.	USGS Class[a]	Closest Horizontal Distance to Site (mi)	Maximum Mapped Length (mi)	Fault Type	Time of Most Recent Deformation (thousands of years before present)	Slip Rate (mm/yr)
Toppenish Ridge Structures - Folds and Other Faults	Lidke and Bucknam, 2002	566b	B	5.5	52	Thrust	< 1,600	< 0.2
Ahtanum Ridge Structures - Folds and Other Faults	Lidke, 2003	564b	B	13	37	Thrust	< 1,600	< 0.2
Toppenish Ridge Structures - Mill Creek Fault	Lidke and Bucknam, 2002	566a	A	13	12	Thrust	< 15	< 0.2
Ahtanum Ridge Structures - Ahtanum Creek Fault	Lidke, 2003	564a	A	15	11	Reverse	< 130	< 0.2
Saddle Mountains Structures - Folds and Other Faults	Lidke, 2002	562b	B	31	65	Thrust	< 1,600	< 0.2
Arlington - Shutler Butte Fault	Personius and Lidke, 2003	847a	A	32	32	Strike-Slip	< 750	< 0.2
Saddle Mountains Structures - Saddle Mountains Fault	Lidke, 2002	562a	A	33	37	Thrust	< 130	0.2 - 1.0
Umtanum Ridge Structures - Central Gable Mountain Fault	Lidke, 2002	563a	A	36	1.2	Reverse	< 130	< 0.2
Frenchman Hills Structures - Folds and Other Faults	Lidke, 2003	561c	B	41	57	Thrust	< 1,600	< 0.2
Frenchman Hills Structures - Frenchman Hills Fault	Lidke, 2003	561a	A	45	32	Thrust	< 1,600	< 0.2

[a]USGS classes are categories of faults based on demonstrable evidence of tectonic movement during the Quaternary. Class A structures have geologic evidence demonstrating the existence of a Quaternary fault of tectonic origin, whereas Class B structures have the geologic evidence, but (1) the fault might not extend deep enough to be a potential source of earthquakes, or (2) there is not enough evidence to classify the structure as Class A.

Notes:
< = less than
mi = mile
mm/yr = millimeter per year

3.2.3 Local Seismic History

In general, the site is located in a region of relatively little historical seismic activity. No earthquakes of magnitude 5.0 or larger have occurred within 60 miles of the project site. There are three earthquakes of magnitude 5.0 or greater that have occurred within, or near, the Columbia Basin in recorded history that provide an indication of the potential for earthquakes at the project site, as follows:

- **1872 Lake Chelan Earthquake**. This earthquake, which occurred on December 14, 1872, was one of the largest recorded in Washington. Lake Chelan is located along the northwestern boundary of the Columbia Basin. The earthquake, with an estimated magnitude between 7.0 and 7.3, was felt from British Columbia to Montana to Oregon; it caused landslides, geysers, and ground fissures. The epicenter was in a wilderness region (at that time), with few human-made structures. Aftershocks followed the earthquake for several years.
- **1936 Walla Walla-Milton Freewater Earthquake**. This earthquake, with an estimated magnitude between 5.75 and 6.1, occurred on July 15, 1936, near the Washington-Oregon state line. The earthquake was felt in Washington, Oregon, and northern Idaho; it resulted in ground cracks, changes in the flow of well water, and many damaged chimneys and cracked plaster walls.
- **1943 Wenatchee Earthquake**. This earthquake occurred on April 24, 1943, and was centered about 15 miles southwest of Wenatchee, Washington, along the eastern boundary of the Columbia Basin. It had an estimated magnitude of 5.0 and was felt throughout central Washington.

3.3 Hydrology

The majority of the water entering the proposed NFCCR will be pumped from the existing French Canyon Regulating Reservoir, which is fed from the District’s Tieton Canal tunnel. Normal inflow from the North Fork Cowiche Creek drainage basin is relatively minor and ephemeral in nature.

Potential flood flow, however, could be significant in the North Fork Cowiche Creek drainage basin. The basin encompasses approximately 15.3 square miles upstream of the proposed dam site on the eastern slope of the Cascade Mountains in Washington state. The basin is roughly rectangular in shape, approximately 11.0 miles long and 1.5 miles wide. Stream elevations range from approximately 5,000 feet to approximately 2,200 feet and the location of the proposed dam, has a mean basin elevation of approximately 3,340 feet. The basin consists of grassland, sage brush, and tree-covered areas.

As with the existing French Canyon Reservoir, the proposed North Fork Cowiche Creek Dam would have a high downstream hazard potential, due to potential for loss of life and/or extreme economic loss in the case of a sudden failure. The proposed size and likely hazard classification lead to the recommendation of a design flood equal to the probable maximum flood (PMF) for the proposed North Fork Cowiche Creek Dam and storage reservoir.

The PMF is the flood that can be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible for a given location. Previous efforts to derive the PMF used the National Oceanic and Atmospheric Administration’s Hydrometeorological Report No. 43 (HMR 43) and the U.S. Army Corps of Engineers’ (USACE’s) rainfall-runoff hydrology software HEC-1, and calculated a peak inflow of 23,300 cfs. This calculated value for the PMF corresponds to the design spillway capacity for the existing FCR (CH2M, 1983).

A comprehensive update of hydrological calculations to match current Dam Safety guidance is beyond the scope of this feasibility study. With this in mind, and considering the size and other characteristics of the basin, as well as the proximity of the proposed facilities to the existing regulating reservoir, the previously calculated PMF of 23,300 cfs for the existing French Canyon Regulating Reservoir may also be deemed appropriate for sizing of the spillway for the proposed North Fork Cowiche Creek storage reservoir. This assumption is conservative, because it neglects any potential attenuation of the flood peak as it would pass through the reservoir.

3.4 Land Use

The project area lies within the Yakima Folds ecoregion of the Columbia Plateau (Clarke and Bryce, 1997). This ecoregion lies in the rain shadow of the Cascade Range, and receives 6 to 15 inches of precipitation annually. Shallow soils overtopping basalt flows support native shrub-bunchgrass communities, dominated by big sagebrush (*Artemisia tridentata*), antelope bitterbrush (*Purshia tridentata*), bluebunch wheatgrass (*Pseudoroegneria spicata*), and Idaho fescue (*Festuca idahoensis*).

The project area is dominated by several shrub species, including rubber rabbitbrush (*Ericameria nauseosa*) (see Figure 3-2), big sagebrush (*Artemisia tridentata*) (see Figure 3-3), and low sagebrush (*Artemisia arbuscula*). Tree species are relegated to the North Fork Cowiche Creek floodplain. Common species include ponderosa pine (*Pinus ponderosa*), black cottonwood (*Populus trichocarpa*), and Oregon white oak (*Quercus garryana*) (see Figures 3-4 and 3-5).

The primary land use in the project area is cattle grazing. Intensive grazing has resulted in low cover of native grasses and increased cover of the non-native crested wheatgrass (*Agropyron cristatum*) and medusahead (*Taeniatherum caput-medusae*). Areas close to the North Fork Cowiche Creek show evidence of off-highway vehicle use.

Figure 3-2. Rubber rabbitbrush (*Ericameria nauseosa*) shrubland

Figure 3-3. Big sagebrush (*Artemisia tridentata*) and Great Basin wildrye (*Leymus cinereus*) community

Figure 3-4. Oregon white oak (*Quercus garryana*) woodland

Figure 3-5. Rubber rabbitbrush shrubland with ponderosa pine (*Pinus ponderosa*) and black cottonwood (*Populus trichocarpa*) gallery along North Fork Cowiche Creek

3.5 Environmental Resources

An online literature review pertaining to state and federal environmental resources in the study area was conducted. The following databases were consulted:

- National Wetlands Inventory (NWI)
- National Hydrography Dataset (NHD)
- U.S. Fish and Wildlife Service (USFWS) Environmental Conservation Online System (ECOS)
- Washington Department of Fish and Wildlife (WDFW) Priority Habitats and Species
- Washington Natural Heritage Program (WNHP)

Geographic information system (GIS) data from these databases was overlain with the footprint of the proposed reservoir and dam alternatives. Figure 3-6 presents the results from the NWI, NHD, and WDFW Priority Habitats data. Figure 3-7 presents the WNHP and WDFW Priority Species data.

NWI and NHD: No wetlands identified by the NWI are located within the study area. The North Fork Cowiche Creek is the only surface water identified by the NHD in the study area.

USFWS ECOS: The USFWS ECOS system indicates that six federally listed threatened, endangered, and proposed threatened species could occur or be potentially impacted by activities in the project area. These species are the marbled murrelet (*Brachyramphus marmoratus*; Federally Threatened), yellow-billed cuckoo (*Coccyzus americanus*), bull trout (*Salvelinus confluentus*), Canada lynx (*Lynx canadensis*), gray wolf (*Canis lupus*), and North American wolverine (*Gulo gulo luscus*). There are no known occurrences of these species in the project area.

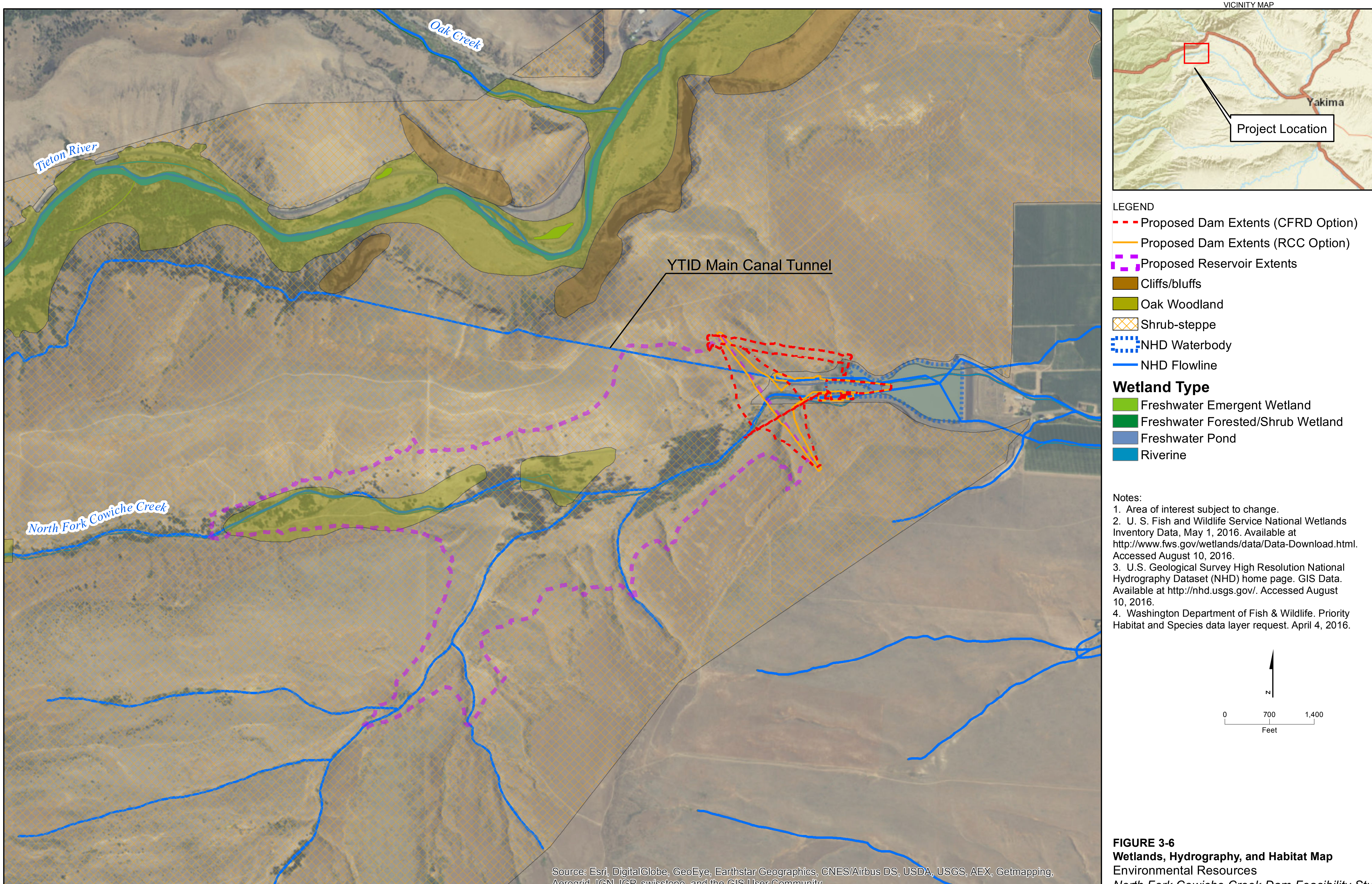
WDFW Priority Habitats and Species: WDFW maintains a list of habitats and species that are priorities for conservation and management. Priority species include State Endangered, Threatened, Sensitive, and Candidate Species; animal aggregations (for example, colonies) considered vulnerable; and vulnerable species with recreational, commercial, or tribal importance. Priority habitats have unique or significant value to a diverse group of species.

Two WDFW Priority Habitats, shrub-steppe and Oregon white oak woodland, were identified in the study area (Figure 3-7). The following descriptions are taken from WDFW's *Priority Habitats and Species List* (2008):

- Shrub-steppe is characterized as a shrubland with a conspicuous but discontinuous shrub layer, one or more perennial bunchgrass layers, and no trees. Dominant shrub species in this habitat type include big sagebrush, antelope bitterbrush, and low sagebrush. Common grass species include Idaho fescue, Sandberg bluegrass (*Poa secunda*), bluebunch wheatgrass, and needle-and-thread (*Hesperostipa comata*). Areas with higher precipitation may support a forb community.
- Oregon white oak woodland is described as oak or mixed oak/conifer stands where oak trees compose at least 25 percent of the canopy. Stands of oak greater than 5 acres are considered priority oak habitat east of the Cascades.

WDFW identifies utilization of the study area by mule deer and elk. Areas to the north have confirmed sightings of bighorn sheep and golden eagle nests. No state-listed threatened or endangered animal species are known to occur within the proposed dam and reservoir footprint.

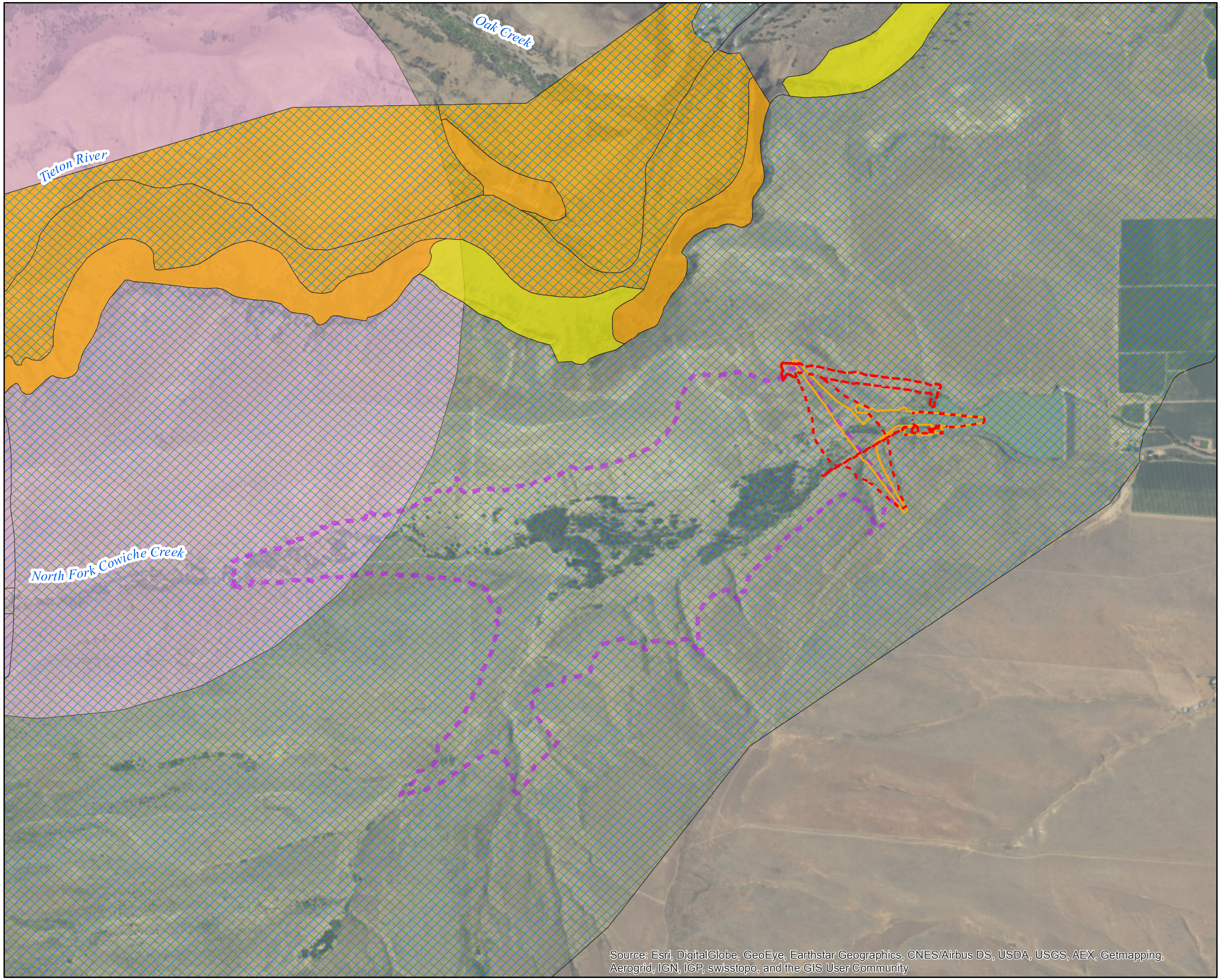
WNHP Rare Plants: To prevent collection of rare plants, WNHP displays occurrence information of these plants using 1- to 5-mile buffers. A documented occurrence of Oregon golden aster (*Heterotheca oregona;* State Threatened) with a 1-mile buffer intersects the northern arm of the proposed reservoir.



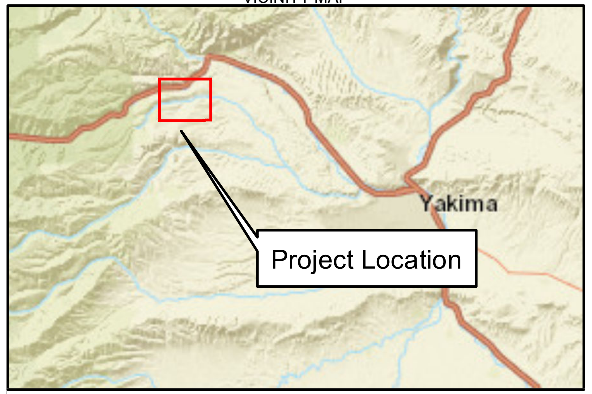
Notes:
1. Area of interest subject to change.
2. U. S. Fish and Wildlife Service National Wetlands Inventory Data, May 1, 2016. Available at http://www.fws.gov/wetlands/data/Data-Download.html. Accessed August 10, 2016.
3. U.S. Geological Survey High Resolution National Hydrography Dataset (NHD) home page. GIS Data. Available at http://nhd.usgs.gov/. Accessed August 10, 2016.
4. Washington Department of Fish & Wildlife. Priority Habitat and Species data layer request. April 4, 2016.

FIGURE 3-6
Wetlands, Hydrography, and Habitat Map
Environmental Resources
North Fork Cowiche Creek Dam Feasibility Study

ch2m



VICINITY MAP



LEGEND

- Proposed Dam Extents (CFRD Option)
- Proposed Dam Extents (RCC Option)
- Proposed Reservoir Extents
- Mule Deer
- Elk
- Golden Eagle
- Bighorn Sheep
- Oregon Golden Aster

Notes:
1. Area of interest subject to change.
2. Washington Department of Fish & Wildlife. Priority Habitat and Species data layer request. April 4, 2016.
3. Washington Natural Heritage Program Rare Plants and High Quality Ecosystems Data, August 2015. Available at: https://test-fortress.wa.gov/dnr/adminsaqa/dataweb/dmmatrix.html Accessed April 2016.

N

0 700 1,400
Feet

FIGURE 3-7
Plants and Wildlife Map
Environmental Resources
North Fork Cowiche Creek Dam Feasibility Study

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3.6 Cultural Resources

Cultural resources are considered any property valued (for example, monetarily, aesthetically, or religiously) by a group of people, and may include archaeological sites, built environment structures, human-altered landscapes, objects, and locations of traditional or ceremonial significance (Traditional Cultural Properties). These valued properties can be historical in character or date to the pre-contact past.

In recognition of the public's interest in cultural resources, and the benefit of preserving them, several federal, state, and local regulations have been developed for their protection. The National Historic Preservation Act (NHPA) of 1966 (as amended) is the primary law that guides management activities (36 Code of Federal Regulations [CFR] 800). Section 106 of the NHPA requires federal agencies to take into account the effects of undertakings that are federally funded, permitted, or take place on federally administered lands, if those undertakings have the potential to affect historic properties, defined as cultural resources that are eligible for listing in the National Register of Historic Places (NRHP). For this project, federal permits would likely trigger the need for compliance with the NHPA.

3.6.1 Site Investigation Methods

Several methods were used to investigate the cultural and archaeological significance of the proposed dam and reservoir site. A literature review was conducted using the Washington Information System for Architectural and Archaeological Records Database (WISAARD). This review identifies previous cultural resources inventory and evaluation efforts and documentation of previously identified cultural resources within and directly adjacent to the proposed reservoir.

A review of the Statewide Predictive Model produced by the Department of Archaeology and Historic Preservation (DAHP) was also conducted. DAHP uses a broad set of environmental criteria to generate a Statewide Predictive Model. The purpose of the model is to determine the probability for archaeological resources. Variables such as proximity to water and degree of slope are considered.

Fieldwork to support this feasibility study was completed March 31, 2016. Fieldwork consisted of a windshield survey of portions of the reservoir footprint and a site walk through areas of higher probability for archaeological resources. Part of this effort was the preliminary investigation of one previously unrecorded standing architectural structure and associated historic archaeological site, and successful identification of a previously recorded historic archaeological site that borders the project area of potential effect (APE).

3.6.2 Site Investigation Results

The literature review identified two cultural resource reports (Table 3-2) and two archaeological sites within a 0.5-mile (804-meter) radius of the APE (Table 3-3). Of these, one is a pre-contact site and one is an historic site. Neither site has been evaluated for eligibility for the NRHP, though both are considered potentially eligible. One of these archaeological sites was identified within the APE (45YA1283) and is described below. No historic properties were identified within the APE.

Table 3-2. Cultural Resource Reports within 0.5 Mile of the APE

Document Number	Title	Reference	Within APE
NADB#1352139	*Letter to Scott Tomren RE: Bear Canyon Repeater Location #348368, Yakima County, Washington Utility Line Survey*	Hannum, 2008	X
NADB#1686516	*WDFW Tieton River Relic Diversion Removal*	Kelly, 2015	

Table 3-3. Cultural Resources Recorded within One Half Mile of the APE

Site Trinomial #	Resource Type	Distance from the APE (miles)	Site Age		National Register Eligibility			Description
			Pre-contact	Historic	Eligible	Not Eligible	Uneval.	
45YA123	Site	0.5	x				x	Pictographs
45YA1283	Site	0		x			x	Historic Farmstead

Site 45YA1283 consists of a historic farmstead including foundations, outbuildings, corral, haying barn, rock terrace, and historic artifact concentrations. The westernmost features are up to 0.35 miles from the southwestern extent of the APE; the easternmost features, including building foundations, are within the APE. Analysis of artifacts and historic maps date the structures to between 1899 and 1971. This site has not been evaluated for eligibility.

Because the lands in the APE meet certain criteria, such as being close to water, the DAHP predictive model shows them as having high potential to contain archaeological resources. Most of the area within the project area is designated as high risk/probability for archaeological materials by the Statewide Predictive Model. Although the model indicates much of the APE is in an area with a “high risk” or “very high risk” for archaeological resources, the resolution of the model does not account for many site-specific, on the ground variables; thus, both surface and subsurface investigation are needed to verify. According to the model, archaeological survey is “highly advised” for both high risk and very high risk probability areas.

3.6.3 Site Investigation Methods

The affected area at the proposed North Fork Cowiche Creek Reservoir contains known cultural resource sites and has the potential to contain additional unidentified cultural resource sites. These sites would likely be directly and indirectly affected by the construction of dams, reservoirs, and support facilities associated with the project, as well as by subsequent public use. Impacts that can adversely affect cultural resources include anything that might significantly destroy or alter the important features of those resources. Direct and indirect effects to cultural resources can result from human activities or natural events. The probability that historical properties (cultural resources determined eligible for listing in the NRHP) are located within the North Fork Cowiche Creek Reservoir footprints is high:

- Native American and European American use of the valley is likely comparable to their use of other tributary valleys in the region.
- The geology and geomorphology of the valley floor and surrounding uplands are conducive to the preservation of cultural resources (comprised primarily of stable or depositional landforms).
- One previously recorded historical archaeological site and one known and unrecorded historical archaeological site are located within the proposed inundation area. Neither one has been evaluated for NRHP eligibility, but it is highly probable that eligible sites will be identified once survey is completed and determinations of eligibility (DOEs) are conducted.

3.6.4 Direct Impacts

Direct adverse effects to cultural resources may result from activities associated with the construction of the reservoir and dam (that is, ground disturbance); the inundation of water into the reservoir (that is, displacement of buoyant objects, coverage of resources, and loss of accessibility); increased sedimentation during inundation (that is, burial of cultural resources); and the erosion of shorelines from seasonal water fluctuations. Analysis of existing cultural resource data indicates a high potential for direct adverse impacts to archaeological sites within the reservoir footprint, assuming that NRHP eligible cultural resource sites are identified during inventory surveys or when DOEs are conducted on the known cultural resources during later stages of the project. The areas with the highest potential for adverse impacts to cultural resources are primarily along the valley slopes, where shoreline fluctuations will expose the ground surface to wave-induced erosion, and within the construction footprint for the dam and support facilities, where construction-related ground disturbance would occur. When DOEs are completed and effects are analyzed during later project stages, it may be feasible to redesign or move support facilities to avoid some sites or to minimize adverse effects. However, it is unlikely that the same opportunities are feasible within the reservoir. If avoidance of adverse effects is not possible, then an MOA or programmatic agreement and a cultural resources treatment plan would need to be developed and implemented.

3.6.5 Indirect Impacts

Indirect impacts may include disturbance, destruction, or increased damage to pre-contact and historic sites, or a combination of these impacts, because of increased public use or activities in the reservoir area. Increased public use may affect cultural resources in the following ways:

- Erosion from foot traffic or boat wake
- Vandalism of exposed cultural resources
- Destruction during the construction of public use structures

The uplands where cultural resources may be intermittently exposed during periods of shoreline fluctuation have the highest potential for indirect adverse effects to cultural resources.

3.6.6 Cultural Resources Summary

The WISAARD database lists two previous cultural resources inventory efforts at the proposed location. Both archaeological sites that have been identified within the APE are historic (one was previously recorded), which means that they were formed during the period in which European-American settlement occurred (after the mid-1800s). Historic site types include historic homesteads, agricultural features, cabin or residential structures, and cemetery or burials. These sites were likely formed as a result of ranching and farming during the twentieth century. One of the sites currently contains a standing structure that will require evaluation by an architectural historian. No previously documented pre-contact archaeological sites are located within the APE. None of the archaeological sites have been evaluated for their NRHP eligibility.

3.7 Groundwater

As mentioned previously, the North Fork Cowiche Creek is ephemeral in nature and dries up for a portion of the year. Little is known at this stage regarding groundwater presence or depths, though it is noted that there are multiple groundwater wells located to the east of the drainage, near Tieton, where agricultural lands are prominent. The depth of these irrigation wells and the aquifers from which they draw water is unknown at this time. Qualitatively, however, the terrain of the North Fork Cowiche Creek drainage basin is several hundred feet higher in elevation than the Tieton River drainage to the north.

Due to the topographical features in the vicinity of the area, there may be a need for a detailed reservoir seepage analysis. One area of particular concern may be along the northern boundary of the proposed reservoir, where there is a steep topographical drop into the Tieton River drainage and a relatively thin ridge between it and the North Fork Cowiche Creek drainage basin. Excessive seepage from the reservoir into the Tieton River drainage could cause erosion and slope stability problems on the canyon walls of the drainage where this seepage would daylight on the slopes.

Future investigational phases of the project should evaluate groundwater further as it relates to the feasibility of this project.

3.8 Existing Facilities and Operations

3.8.1 Yakima-Tieton Irrigation District Tieton Canal and Regulating Reservoir

In 1984, the YTID constructed the French Canyon Dam and Regulating Reservoir, located approximately 2.0 miles west of Tieton, Washington, on the North Fork Cowiche Creek. The dam forms a small regulating reservoir with a total operation storage capacity of approximately 500 acre-feet. The existing reservoir is primarily filled using YTID's Tieton Canal.

The existing reservoir also receives inflow from the North Fork Cowiche Creek during the spring and early summer. The reservoir serves to function as a small storage buffer at the upper end of YTID's service area during the normal irrigation season. Table 3-4 summarizes the typical regulating reservoir rule curve (CH2M, 1983). Water from the reservoir is distributed throughout YTID's 28,000-acre service area via approximately 200 miles of buried pressure pipelines. The water is primarily used to irrigate high-value crops such as apples, cherries, and pears. The reservoir also provides water storage that can be used for intermittent frost protection during the off-season.

Table 3-4. Regulating Reservoir Rule Curve

Approximate Time of Year	Elevation (feet)
March through May	2154.0
June through Mid-October	2151.0
Mid-October through February	2140.0

Source: CH2M, 1983

Current normal operations at the French Canyon Reservoir are to release water through the low-level outlet pipeline or main transmission lines; use of the spillway is only anticipated during large and rather infrequent flood inflow events.

The Tieton Canal was constructed between 1906 and 1909, beginning at a gravity diversion dam upstream from the unincorporated community of Rimrock Retreat. The 12-mile-long canal parallels U.S. Highway 12 and the Tieton River before crossing over into the North Fork Cowiche Creek drainage basin. The canal consists of approximately 9 miles of horseshoe-shaped precast concrete segments and six tunnels totaling approximately 3 miles in length. The Tieton Canal has a design capacity of 345 cfs. In recent years it has carried as much as 330 cfs but not for long periods of time. In its current condition, it can carry 320 cfs for peak periods of use if monitored closely. Because the grade of the Tieton River is much steeper than the canal, the canal is perched several hundred feet above the river prior to crossing over into the North Fork Cowiche Creek drainage and terminating at the French Canyon Reservoir.

3.8.2 Yakima-Tieton Irrigation District River Diversions and Water Deliveries

Under the adjudication settlement agreement YTID can divert up to 96,611 acre-feet form April 1 through October 31 each year. The District can also divert up to 3,881 acre-feet from November 1 to March 31 of each year, and up to 908 acre-feet from the North Fork Cowiche Creek from March 1-July 31 of each year. Table 3-5 summarizes total historical water volumes diverted from the Tieton River via the Tieton Canal annually by YTID from 1999 to 2015. Figure 3-8 shows the flow in the Tieton Canal during the 2015 irrigation season, which is reasonably representative of typical YTID system demand. Flow diversions typically begin in early March, peak in the mid-summer months, and end in early October.

Table 3-5. Total Annual Diversion

Year	Total Diversion (acre-feet)
1999	91,540
2000	84,483
2001	74,728
2002	73,727
2003	77,277
2004	72,547
2005	74,786
2006	66,529
2007	73,965
2008	74,431
2009	75,967
2010	67,553
2011	74,352
2012	76,732
2013	74,181
2014	81,462
2015	82,385

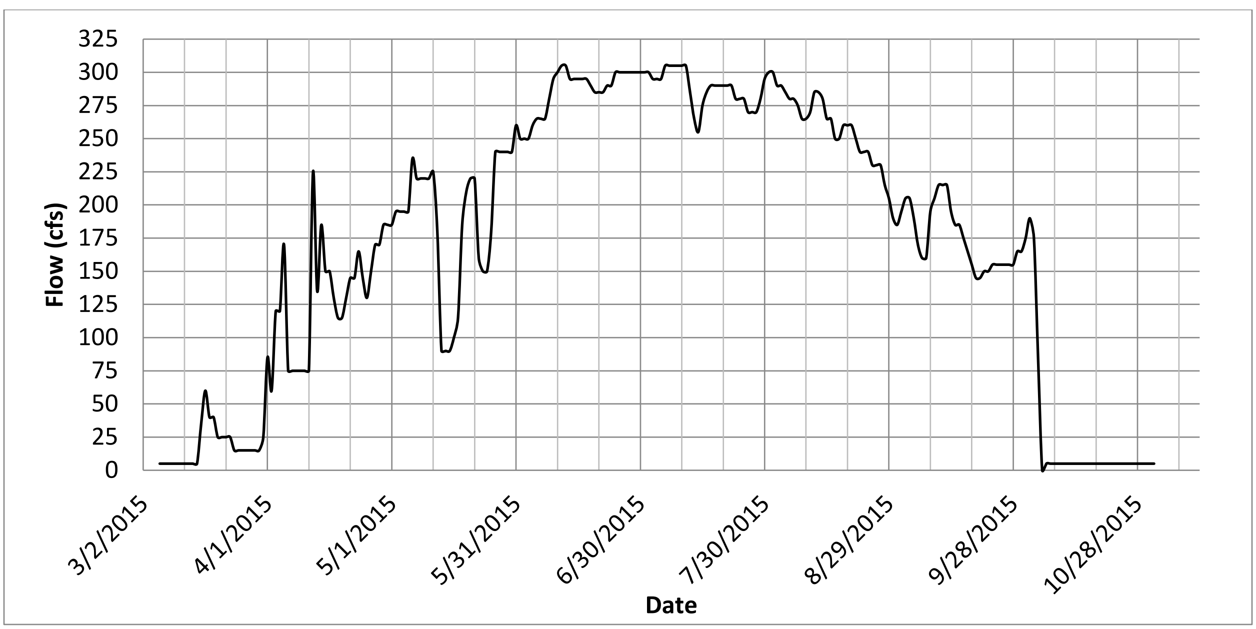

Figure 3-8. 2015 Tieton Canal Flows

The three largest water users and potential water users from the YTID Tieton Canal (YTID, Cowiche Creek Water Users, and Ahtanum Irrigation District [AID]) are all agricultural users with high demands in the middle of the summer. Demands from other uses (such as aquifer storage and recovery, municipal water, and instream supplementation) are smaller in comparison and ignored for the purposes of this feasibility study since they have not been quantified at this time. Due to the high peak summer demands and relatively small amount of existing system storage, the capacity of the Tieton Canal is more limiting than is the water availability (which is discussed in detail in Section 5).

SECTION 4

Project Scope

4.1 Proposed Dam and Reservoir

4.1.1 Assumed Subsurface Conditions

No geotechnical explorations were performed for this feasibility study. Our interpretation of assumed subsurface conditions is based on the geotechnical information available for the French Canyon dam that is located about 3,000 feet downstream of the proposed dam; visual observations of the terrain, including observations of rock outcrops, topography, and rock cut slopes (spillway and roads); and information from published geologic maps.

The proposed dam abutments are underlain by the Grande Ronde basalt. The relatively thin layer of soil that overlies the basalt at the abutments would be removed to construct the abutments directly on the rock. The bottom of the dam would be founded on basalt that underlies the valley floor, beneath the alluvium. The alluvium would be excavated so that the entire dam would be founded on basalt bedrock. The depth to rock at the valley floor is expected to be approximately 20 to 30 feet and may vary along the dam axis. Apart from removing the overburden soils, some of the upper rock layers that are weathered may also have to be removed. A concrete gravity dam will have to be founded on basalt that is relatively unweathered and hard, whereas the foundation requirements for a rockfill embankment dam could be less rigorous.

Fractured zones may be present within the basalt flows that underlie the abutments and foundation, and weak interflow zones may be present under the proposed abutments. The northernmost part of the reservoir is bounded by a relatively narrow “saddle” that was interpreted as shallow surficial soils overlying potentially weak or fractured rock; this topographical feature may be attributable to a fault or shear zone. Identifying and grouting potential fractured zones may be required. In addition, northeast-trending faults have been mapped southwest of the site. These faults project generally towards the right abutment of the proposed dam site. The faults are not interpreted to be active, but prehistoric faulting could have led to fractured zones and sheared zones in the basalt bedrock.

Actual subsurface conditions can have a considerable impact on construction cost, so it is important that a geotechnical investigation be performed in the next design phase to be able to make a more meaningful assessment of the feasibility of dam construction at this site.

4.1.2 Siting of Dam

The preliminary proposed location of the dam and reservoir is shown on the vicinity map on Figure 1 in Appendix A. The dam is to be located immediately upstream of the downstream portal of a tunnel which forms the downstream section of the District’s 12-mile-long Tieton Canal. The tunnel has a downstream portal invert elevation of 2153.0 feet and will form a penetration in the foundation of the dam. Note that the dam location and alignment are subject to change based on the results of future geotechnical investigations.

Two alternative dam alignments were considered in this study: an upstream alignment (Alignment A) and a downstream alignment (Alignment C). The upstream alignment had the smallest dam volume and largest overburden to the inlet tunnel, but appears to have weaker subsurface conditions. The downstream location resulted in high dam volumes and does not leave sufficient room for the downstream pumping station area.

The site map (Figure 1 in Appendix A) shows the topography of the project area. The rounded, rolling hills on the northern and southern sides of French Canyon have local relief of about 400 feet. The upper slopes are gentle, but steepen to more than 30 degrees at the base. An unnamed ephemeral stream forms a gully on the southern slope near the mouth of French Canyon. Two small gullies exist on the northern slope in the eastern half of the valley. The canyon width varies from 250 feet at the western end to more than 700 feet at the eastern end. The valley bottom is generally flat and slopes gently downward to the east.

Elevations of the valley floor range from about 2,140 feet at the existing diversion dam to 2,090 feet at the mouth of the canyon. North Fork Cowiche Creek flows intermittently through a channel along the northern half of the valley at the dam site location.

As shown on Figure 2 in Appendix A, the proposed dam with a height of about 240 feet would impound a reservoir with a storage capacity of about 35,100 acre-feet with a reservoir area of about 430 acres.

4.1.3 Hazard Classification

The degree of conservatism that is adopted for a dam project typically depends on the consequences of a dam breach. Where a dam breach could result in downstream flooding that may result in a loss of life, the design events or loadings adopted for the project are typically very conservative. On the other hand, if there is no or very low risk to the public, the design level is typically lower and is primarily based on economic considerations including project replacement and disruption of operations. The State regulating agency (Washington State Department of Ecology) prescribes the methods used to determine the hazard classification and the loads or events to be used in the design of the dam.

One challenge is to achieve a consistent design level amongst the various engineering disciplines, which can include hydrology, hydraulics, geotechnical, structural, mechanical, and electrical. Since dam failure will likely result from the failure of the weakest component, it is important to prevent dissimilar levels of protection from failure. Washington State has one of the most rigorous frameworks for assigning appropriate design events or loadings based on the consequence of dam failure. Their classification of design levels and steps are based on a decision framework which incorporates the concepts of Consequence Dependent Design Levels and Balanced Protection (Ecology, 1992, 2012).

No breach analyses and inundation mapping was performed for this feasibility study. However, an initial estimate of the hazard classification of the dam was made using Table 2 in the Dam Safety Guidelines from Washington State Department of Ecology (Ecology, 2012). According to this guidance, the proposed North Fork Cowiche Creek Dam will have a downstream hazard potential of “high” and a hazard classification designation of “1A”, based on the following; dam and reservoir size, the short distance between the dam and downstream development, and the potential for life loss of more than 300 people and impacts to more than 100 inhabited structures. The City of Tieton, Washington, is located approximately 2 miles east of the proposed dam and within the potential inundation area. This hazard classification implies that a design step of 8 will apply for the dam (Figure 4-1).

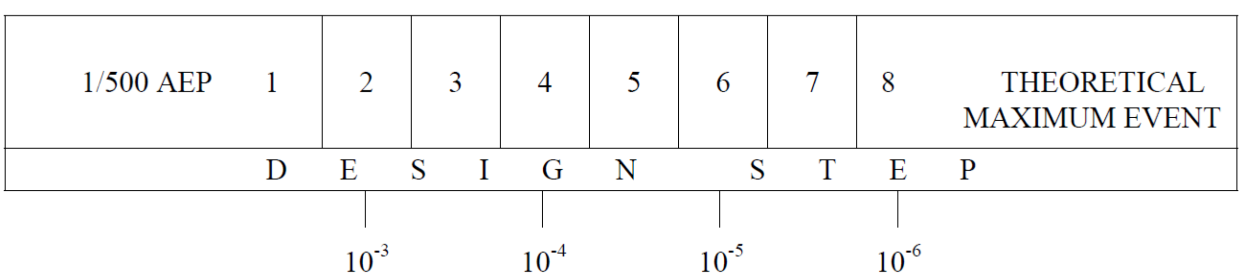

Figure 4-1. Design/Performance Goal – Annual Exceedance Probability
North Fork Cowiche Creek Dam Feasibility Study

4.1.4 Design Loads and Criteria

4.1.4.1 Design Earthquake

It is anticipated that the seismic design approach will generally follow the Washington State Department of Ecology, *Dam Safety Guidelines* (Ecology, 1993). The design step approach outlined in the guidelines results in a design step of 8 and an annual exceedance probability of 1×10^{-6}. This is a 1 in 1,000,000-year failure criteria. In previous discussions with the State of Washington Dam Safety Office (LaVasser, 2004) when working on the Sunnyside Canal regulating reservoirs, it was concluded that it is not practical to develop seismic design criteria for such long return period earthquakes. The main reason for this conclusion was the uncertainty involved in seismic hazard studies and the uncertainty in the seismic sources in central Washington.

Therefore, it is anticipated that the seismic design criteria will be to use a shorter return interval earthquake as the design earthquake. The U.S. Committee on Large Dams recommends using an earthquake with an annual probability of exceedance of 1/3,000 to 1/10,000 (USCOLD, 1999). For this project, it is recommended that the reservoir be designed for an event with a 1 percent probability of exceedance in 100 years (1%/100yr); representing an event with a nominal return interval of 10,000 years. The ground motions associated with the maximum credible earthquake (MCE) can provide an indication of the order of design motions that can be expected. The peak ground acceleration for the MCE is 0.24 g at this site, which represents moderate seismicity.

4.1.4.2 Design Flood

For design step 8, the design storm magnitude corresponds to the probable maximum precipitation (PMP) which is estimated to be the theoretical maximum amount possible for a given duration. Information on computing an applicable PMP for a specific site is contained in the National Weather Service HMR-57, which replaced HMR-43. HMR-57 is a study that provides all-season, general-storm PMP estimates for durations from 1 to 72 hours for the Columbia River basin, the Snake River basin, and drainages along the Pacific coast.

4.1.5 Dam Alternatives

Two dam types have been considered as part of this feasibility study: (1) an RCC Gravity Dam and (2) a CFRD. Given the lack of fine-grained materials in the vicinity of the site, neither an earth dam nor an earth core - rockfill dam was considered. Based on a desk study of geologic maps and the geotechnical report for the existing French Canyon dam and impressions from a site visit, it appears as if foundation conditions on the site are generally good and suitable for constructing either an RCC dam or a CFRD.

4.1.5.1 Roller Compacted Concrete Gravity Dam and Spillway

Dam Description

The RCC dam alternative is a concrete gravity dam that relies on the weight of the concrete to withstand the forces imposed on the dam. RCC is defined as a relatively dry, no-slump consistency concrete that is typically delivered by dump trucks or conveyors, spread by bulldozers in thin horizontal lifts (about 12 inches), and compacted by vibratory rollers similar to the placement methods used to construct earth dams (Bass, 1991), as shown on Figure 4-2. RCC has essentially the same ingredients as conventional concrete, but in different ratios and commonly with partial substitution of fly ash for Portland cement. It is placed at a much dryer moisture content compared to that of conventional concrete.

Figure 4-2. Typical RCC Dam Construction
North Fork Cowiche Creek Reservoir Feasibility Study

The biggest advantage of RCC versus conventional concrete is its much lower unit price per cubic yard. That is because RCC uses less cement, requires less formwork, and is placed with conventional earthwork techniques in essentially a continuous operation. The unit price for RCC is generally about $80 to $130, whereas conventional concrete can be double that amount. Compared to most conventional earth dams, an RCC dam can be constructed in less time. Another advantage is that little damage occurs if the dam is overtopped during construction.

Dam Foundation: An alluvial layer that is assumed to be about 15 to 30 feet thick overlies basalt bedrock in the valley floor. The RCC dam concept assumed that all the alluvium in the valley floor would be excavated, such that the entire dam is founded on hard, slightly weathered to fresh basalt rock at an assumed depth of about 30 feet below the existing ground surface. The excavation will include removal of the upper 10 to 15 feet of basalt rock, which is expected to be highly to completely weathered.

At the abutments, an excavation depth of 10 feet was assumed. This includes removal of a thin layer (5 feet) of soil that overlies the basalt, as well as about 5 feet of weathered basalt rock.

Note that the RCC dam will require a higher quality foundation rock than the conditions required for the CFRD. This is because the RCC dam is more rigid and less tolerable to deformations.

Dam Geometry: The RCC cross section assumed for this preliminary comparative analysis was determined based on review of existing RCC dams constructed on good quality rock foundations. The dam shown has a vertical upstream face, a crest that is 20 feet wide and a downstream slope of 0.8H:1V (see Figure 5 in Appendix A), which is similar to the photo of the dam shown on Figure 4-3. If sliding or seepage (and internal erosion of rock discontinuities) as a result of weathered and weak layers or seams in the foundation is a concern, the slopes will become flatter. For example, Duck River Dam in Alabama (Riker et al., 2016) was recently constructed with a 0.1H:1V upstream slope and a 1H:1V downstream slope because of the presence of shale layers and clay and coal seams in the foundation. As shown on Figure 5 in Appendix A, steps that are 4 feet high and 3.2 feet wide were assumed for the downstream slope, which is a typical step configuration.

Gallery: Figure 5 in Appendix A shows a drainage and inspection gallery that will be large enough (10 feet wide and 12 feet high) to allow installation of the grout curtain from the gallery. Various drains will extend to the gallery, including monolith joint drains, face drains, and foundation drains. The gallery will not be horizontal but will step up in the abutments to roughly follow the top of the excavated foundation surface as shown on Figure 4 of Appendix A.

Foundation Grouting: Although grouting from the gallery tends to be more difficult and expensive, it can be a useful option if the construction schedule is tight. The option will remove grouting from the critical path and will allow RCC placement to proceed, while concurrently grouting from the gallery without impacting RCC operations. Another benefit of grouting from the gallery is that the RCC placed below the gallery floor acts as a thick grout cap facilitating relatively high grouting pressures, and this a more effective grouting job in the upper foundation layers.

For the purpose of estimating the cost of the foundation grouting, a two-row grout curtain with maximum grout hole lengths of 200 feet was assumed (with shorter holes in the abutments). It was further assumed that the final pattern hole spacing is 10 feet, but with some areas of tertiary grouting with a final hole spacing of 5 feet. Two rows of blanket grout holes extending to a depth of 25 feet into the foundation and spaced at 7.5 feet were included in the concept.

Figure 4-3. Elkwater Fork Dam (180 feet high), West Virginia
North Fork Cowiche Creek Reservoir Feasibility Study

Facing Systems: Some early RCC dams experienced unacceptable seepage flows through the horizontal lift joints and vertical cracks. Since then, monoliths joints have been used to control or prevent vertical cracks and upstream facing systems consisting of conventional concrete, precast concrete, geomembranes, and grout-enriched roller compacted concrete (GERCC) have been used to reduce seepage along lift lines. Another, less robust, approach is to treat the upstream zone (approximately 8 feet wide) of lift joints with bedding mortar to reduce leakage along the joints. Facing systems have also been used to improve durability and appearance of the downstream RCC steps, especially in the spillway section. Facing systems have not been evaluated for this high-level feasibility study. In this study, for cost-estimating purposes, a GERCC facing system was assumed.

Spillway Description

The unique benefit of an RCC dam is that a portion of it can serve as the spillway, by adding a spillway crest at the top and training walls at the crest and along the downstream side to form the chute. Given this, a relatively long spillway can be constructed economically with the main limitation being to transition the flow at the downstream side to a smaller channel. For this dam, the length proposed for the spillway is 250 feet. The transition to the French Canyon reservoir is a 250-foot-wide channel that reduces to a 150-foot-wide channel. The transition channel is excavated to rock, with RCC retaining walls on the side walls. Having a longer spillway provides the benefit of reduced surcharge on the spillway crest to pass the PMF flow, which reduces the freeboard and dam height requirements. The additional length will also reduce the unit discharge and reduce the size of the stilling basin required to dissipate the energy.

The spillway crest assumed is an ogee shape that is hydraulically efficient to pass flow. The shape of an ogee spillway is affected by the relative head, approach depth, approach velocity, and slope of upstream face. Determining the values for each of the parameters is dictated by both hydraulic design requirements and geometric requirements specific to the project. For this level of design, a constant discharge coefficient of 3.47 at the peak outflow for the PMF was assumed, which is similar to the French Canyon dam spillway. The result is a surcharge of 8.9 feet to pass the PMF. The height of the dam was then set with surcharge requirement of 9 feet with no residual freeboard, since the RCC structure can be overtopped for limited time periods without risk of breaching the dam.

The spillway chute is a “stepped chute” due to the construction methods for an RCC dam. The result is a flow regime over the steps that changes primarily with changes in the unit discharge or flow per unit length over the spillway. The length and drop of the individual steps and number of sequential steps also influence the flow regime. The flow regime varies from nappe flow at low unit discharges to skimming flow at high discharges. Nappe flow is characterized by a succession of free-fall jets from the vertical drop from a step that then impacts the horizontal run of the next step that is followed by a hydraulic jump before cascading off the end of the step to repeat the sequence. Most or all of the energy is dissipated before reaching the toe of the dam. As the unit discharge increases, the flow regime transitions to skimming flow where the water flows as a coherent stream skimming over the steps with residual energy reaching the bottom of the spillway chute requiring a stilling basin. The stilling basin was sized assuming skimming flow down the spillway chute for half the PMF. If a full PMF flow occurred, the hydraulic jump would likely sweep from the basin causing some downstream erosion, but would not compromise the safety of the dam. The design methods for the stilling basin follow guidelines for a USBR Type III basin.

4.1.5.2 Concrete Face Rockfill Dam

A CFRD (see example on Figure 4-4) is considered to be a more suitable dam type than an earth dam or a rockfill dam (with a clay core) for the following reasons:

- Adequate volumes of good-quality clay materials for a dam core are not available in the vicinity of the site.
- Grouting for a CFRD can be performed concurrent with rockfill embankment construction, compared to a clay core-rockfill dam or earth dam, where grouting needs to be completed prior to embankment construction.
- Weather: A CFRD can be constructed during wet and cold weather; risks due to weather delays, usually associated with clay core construction, are largely eliminated.
- CFRD does not have filter zones that are necessary for a dam with a clay core, which simplifies construction and reduces cost.
- The area underneath the plinth is the only area that requires a high level of foundation cleaning and treatment; this area is relatively small compared to the foundation areas that will require cleaning and treatment for an earth dam or clay-core rockfill dam.
- The total embankment volume is likely to be smaller, and the side slopes steeper, for a CFRD than for an earth dam or clay-core rockfill dam, resulting in lower costs for outlet conduit.
- Rockfill can be placed in large lifts (up to 4 feet thick), and construction is not negatively impacted by wet weather. This allows for a shorter construction duration than the more common clay core dams.

Figure 4-4. Teemburra Dam, Queensland, Australia
North Fork Cowiche Creek Reservoir Feasibility Study

Dam Description

The first rockfill dam to have a concrete face was constructed in California in 1895 (Fell et al., 2015). Many of the early CFRDs performed well, while others leaked excessively due to opening of the joints of the concrete face slab. The joint defects mainly resulted from compressibility of the dumped rockfill and the detailing of joints. The leakage did not endanger the dams, but in some cases was too high for operating reasons, resulting in unacceptable performance.

Construction practices for CFRDs were significantly improved over the period 1955 to 1965, by adopting the compaction of rockfill in layers instead of just dumping or sluicing the material in place. It was recognized that dumping and sluicing of rockfill can lead to significant segregation, resulting in voids and unacceptable settlements. Another improvement in CFRD design resulted from improvements in using a plinth (or toe slab) instead of a deep cutoff trench in the rock at the bottom of the concrete slab (Terzaghi, 1960).

For this feasibility design, we assumed a dam cross section that includes element and zoning consistent with modern practice (Fell et al., 2015) for a CFRD on a strong rock foundation. The cross section is shown on Figure 14 in Appendix A and includes the following elements:

Dam Foundation. The CFRD concept assumed that all the alluvium in the valley floor would be excavated such that the entire dam is founded on basalt rock. Whereas a foundation excavation depth of 30 feet was assumed for the entire RCC dam footprint, only the upstream portion below the concrete face slab is assumed to be excavated to that depth for the CFRD. An excavation depth of 15 feet is assumed under the dam downstream of the face slab. The reason for the different excavation depths is to reduce deflections of the face slab associated with loading when the reservoir is filled, whereas settlement of the downstream embankment is less critical and should largely occur during construction.

The excavation depth under the upstream embankment slope will include removal of the upper 5 to 10 feet of basalt rock, which is expected to be highly to moderately weathered. High quality rock that is preferably non-erodible is required below the plinth, as discussed in the next section. It is assumed that weathered rock could remain in the downstream portions, however, the material needs to be strong enough to support stable downstream slopes.

At the abutments, an excavation depth of 10 feet was assumed. This includes removal of a thin layer (5 feet) of soil that overlies the basalt, as well as about 5 feet of weathered basalt rock.

The foundation requirements for the CFRD are generally not as stringent as for the RCC dam. The foundation rock below the CFRD plinth must be sound, but the plinth area is relatively small compared to the RCC dam footprint.

Plinth. The plinth or toe slab connects the seepage barrier (grout curtain) in the foundation with the face slab on the upstream face of the dam. The plinth is anchored into rock with steel bars and serves as the grout cap for foundation consolidation and curtain grouting. The plinth runs across the bottom and up the abutment groins.

The plinth design, dimensions, stability, construction and foundation treatment are important design considerations. The plinth width is typically of the order of 1/15 to 1/25 of the water depth (Fell et al., 2015), but is dependent on the erodibility of the rock. The information in Table 4-1 can be used as a guide to establish an acceptable seepage gradient beneath the plinth as a function of foundation conditions (that is, to establish the required plinth width). Assuming that the foundation rock at the North Fork Cowiche Creek Dam would be slightly erodible to non-erodible, the acceptable plinth width/water depth ratio is 1/12 to 1/18 (that is, an acceptable hydraulic gradient of 12 to 18). Our concept cross section on Figure 14 in Appendix A shows a plinth width of 20 feet, which denotes a plinth width/water depth ratio of 1/10 (or hydraulic gradient of about 10) for a conservative water depth of 200 feet).

Table 4-1. Acceptable Seepage Gradient beneath the Plinth related to Foundation Conditions
North Fork Cowiche Creek Reservoir Feasibility Study

A	B	C	D	E	F	G	H
I	Non-erodible	1/18	>70	I to II	1 to 2	<1	1
II	Slightly erodible	1/12	50-70	II to III	2 to 3	1 to 2	2
III	Erodible	1/6	30-50	III to IV	3 to 5	2 to 4	3
IV	Highly Erodible	1/3	0-30	IV to VI	5 to 6	>4	4

Where:

A = Foundation Type
B = Foundation Class
C = Minimum Ratio: Plinth Width/Depth of Water, full reservoir
D = Rock Quality Designation, RQD, in %
E = Weathering Degree: I equals sound rock; VI equals residual soil
F = Consistency Degree: 1 equals very hard rock; 6 equals friable rock
G = Weathered Macro Discontinuities per 10 m
H = Excavation Classes:
- 1 = requires blasting
- 2 = requires heavy rippers; some blasting
- 3 = can be excavated with light rippers
- 4 = can be excavated with dozer blade

Source: International Commission on Large Dams (ICOLD), 2010, from Sierra, 1989

Face Slab. The preliminary concept for the face slab is a reinforced concrete slab that is 18 inches thick, as shown on Figure 14 in Appendix A. The slab will have vertical, and some horizontal and perimetric joints to accommodate deformation that occur during construction and when the reservoir is filled. The perimeter joint connects the concrete face slab and the plinth of the CFRD to form the upstream water barrier of the dam. The perimeter joint must preserve a watertight seal against full reservoir load while allowing for anticipated movements between the plinth and face slabs. The plinth is anchored to rock and not anticipated to move, whereas the face slab, which is supported on the rockfill, will deform as the rockfill undergoes settlement.

From examination of measured perimeter joint movements for several modern CFRDs of compacted rockfill, total perimeter joint movement can be expected to be less than 30 millimeters for well-constructed CFRDs less than about 100 meters (300 feet) in height (ICOLD 2004).

Perimeter joint designs typically incorporate a three-barrier waterstop system for CFRDs higher than 100m and a two-barrier system for lower dams. The system typically includes a metal (copper or stainless steel) waterstop at the bottom, a rubber waterstop in the middle and mastic at the joint surface. For the two-barrier system, the metal bottom waterstop is maintained with either the top or middle waterstop being typically eliminated.

Rockfill Embankment. The preliminary cross section shown on Figure 14 in Appendix A has upstream and downstream slopes of 1.5H:1V, which is relatively flat since slopes as steep as 1.3H:1V have been used on a sound rock foundation using strong rockfill. The flatter slopes were assumed since the soundness of the rockfill and foundation has not been investigated and the project is located in a region with moderate seismicity.

The zone designations of 1, 2, and 3 have become the standard (ICOLD 2004):

- Zones 1A, 1B – concrete face protection (upstream) zones, in increasing order of maximum particle size
- Zones 2D, 2E, 2F – concrete face supporting (downstream) zones, in increasing order of maximum particle size, these are processed granular materials
- Zones 3A, 3B – rockfill zones, in increasing order of maximum particle size

Zone 1B provides support for Zone 1A and in some cases also resists uplift of the face slab prior to reservoir filling. Zone 1A, a cohesionless silt or fine sand, is placed to a higher elevation on high dams so that it can act as a joint or crack healer over the perimeter joint and the lower part of the face slab. Compaction of Zones 1A and of the random Zone 1B is by hauling and spreading equipment.

Zone 2D is a processed fine filter with specific gradation limits, minus 1-inch or minus 3/4-inch material. It is to limit leakage in the event of waterstop failure and to self-heal with underwater placement of silt or silty fine sand. Zone 2E is typically fine selected rockfill that serves as a filter transition between Zones 2D and 3A. It has often been specified as crusher run minus 75 mm sound rock material. Alternatively, specific gradation limits are specified. Zone 2F is a filter zone downstream of the perimetric joint with two functions: (1) to serve as a high modulus zone to limit deformation of the slab at the perimetric joint, and (2) to serve as a filter to the Zone 1A (earthfill) upstream of the slab in the event the joint opens and leakage initiates.

Zones 3A and 2B are quarry run rockfill. The differences in 3A and 3B are principally in layer thickness and size and type of rock. Zone 3A resists the weight of the reservoir water and is designed to limit deflection of the upstream face. Zone 3A is typically quarry run rockfill (up to 18 in size) placed in 3-feet lifts to provide compatibility and to have material with smaller voids adjacent to Zone 2B.

Zone 3B receives little water loading, and settlement is essentially during construction as a result of the self-weight of the fill. Zone 3B is a coarse, free-draining rockfill, quarry run, placed in 4-feet layers. The thicker layer in Zone 3B accepts larger rock and is more economical to place.

Performance under Earthquake Loading. One issue of concern is the effect of earthquake-induced settlement on the face slab. The data presented on Figure 4-5 was used to make a rough estimate of anticipated settlement during an MCE event (which is not equal but about the same magnitude as the design earthquake). Figure 4-5 comprises data of measured earthquake-induced deformation of rockfill dams were compiled by Swaisgood and correlated to the Earthquake Severity Index, where the Earthquake Severity Index (ESI) is defined as follows:

$$ESI = PGA * (M-4.5)^3$$

Where:

PGA = Peak horizontal ground acceleration at the site
M = Earthquake event

The ESI estimated for the maximum design earthquake is about 4.0, assuming M of about 7.0 and PGA of about 0.25.

The estimated crest settlement as a percentage of the dam height from Figure 4-5 ranges from about 0.04 to about 0.2. Assuming dam height of 200 feet, the estimated crest settlement varies from 0.08 to 0.4 feet (or 1 to 5 inches). This amount of settlement is relatively small and should not cause major damage to the face slab.

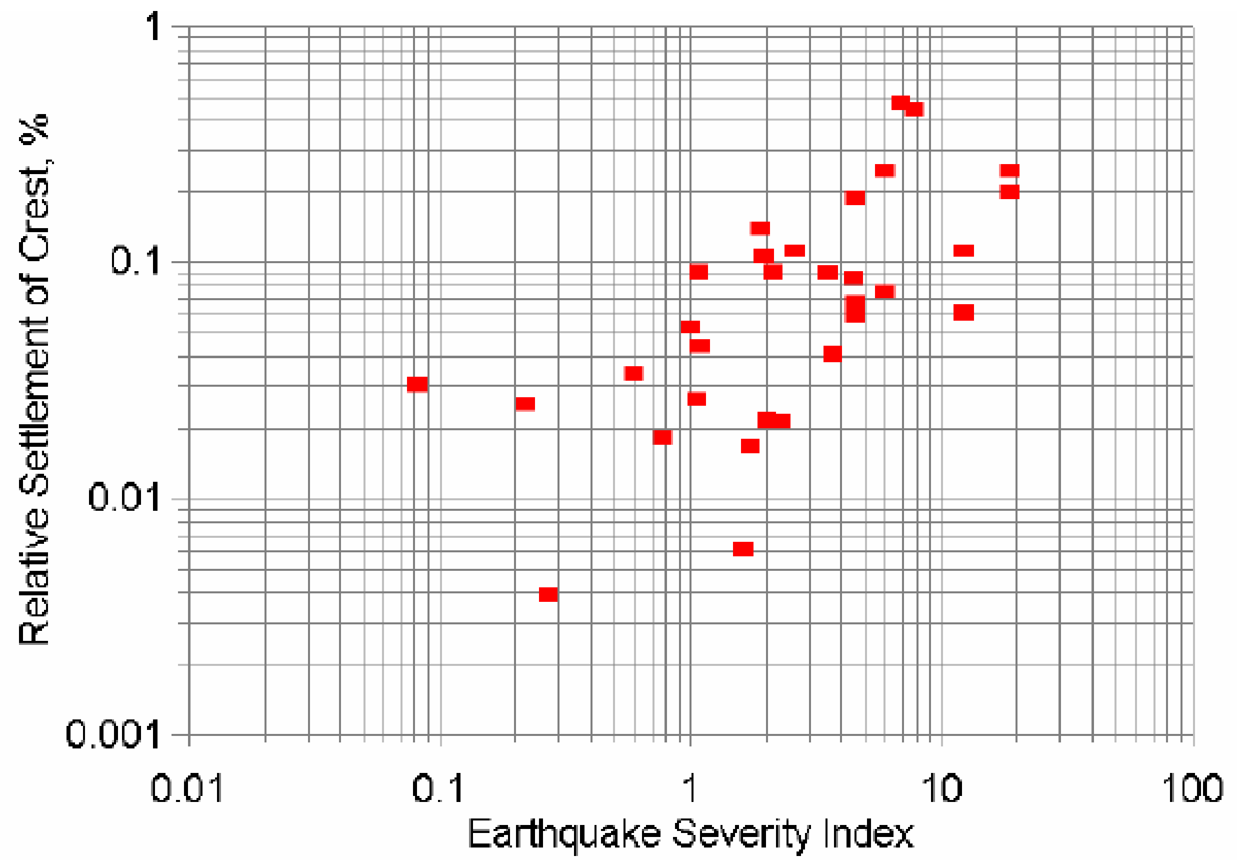

Figure 4-5. Performance during Earthquake, Crest Settlement for Rockfill Dams (Swaisgood, 1995)

Wieland and Brenner (2004) discussed the earthquake aspects of RCC dams and CFRDs and concluded that well-designed and properly compacted CFRDs on rock foundations are considered safe under the strongest earthquake loading. They noted that "this is because the crack width of the cracks in the face slab that may develop as a result of high dynamic stress and deformations, predicted from a traditional two-dimensional dynamic analysis of the highest dam section, are relatively small and thus the resulting leakage does not impair the safety of the dam." We have assumed that these conclusions will apply to the proposed North Fork Cowiche Creek Dam site, since the valley is relatively wide.

Wieland and Brenner (2004) noted that in *narrow canyons with steep abutments* (which is NOT the case for this site), the behavior of the concrete face during a strong earthquake is largely unknown but it is possible that "the large membrane forces in the slab generated by the cross-canyon excitation may cause local buckling at the joints, rupturing of water stops and/or movements of individual slab elements as rigid body."

Another advantage of the CFRD concerning earthquake loading is the anticipated absence of pore water pressures in the rockfill embankment because the rockfill is essentially dry.

With the absence of pore water pressures, the high shear strength of compacted rockfill, and the fact that the dam will be founded on rock, the performance of the CFRD embankment is anticipated to be excellent under seismic loading, with the effects of seismic shaking on the face slab the only real concern.

Spillway Description

For this feasibility study, we essentially duplicated the spillway and stilling basin design from the existing French Canyon Dam for the purpose of developing preliminary construction costs. If the CFRD is selected as the preferred alternative, additional hydraulic and structural evaluations will be performed in the next design phases to ensure the costs are better represented.

In general, the spillway is a side-channel structure constructed in a rock cut on the left abutment. The spillway crest is 150 feet long and is a hydraulically efficient ogee crest similar to the RCC dam. The spillway chute is shotcrete lined and reinforced with rock bolts. To match the site conditions, the slope of the spillway chute is steeper than French Canyon. It should perform equal or better than French Canyon Dam. The stilling basin length and width were duplicated from French Canyon. The stilling basin length may have to be increased to accommodate the additional energy from the higher dam.

Hydrology and PMF. A PMF inflow to the reservoir from the 16 square mile watershed was assumed to be the same as French Canyon Dam at 23,000 cfs. The methods for developing the PMF for French Canyon Dam have changed with the most significant being the development of the PMP from HMR 55 that has replaced HMR 43 since the French Canyon design. HMR 55 takes advantage of new storm data and developments in analytical procedures to revise previous studies to provide greater detail. HMR 55 provided some general comparisons between HMR 43 and HMR 55 for different locations. Comparing the smaller duration storm events (6 hours was the controlling event for French Canyon) and in the Washington region, it is anticipated that the PMF inflows will be within approximately 20 percent. If more represented by the Columbia River Plateau, the PMP duration went down by approximately 10 percent. If more represented by the Olympic Mountains, the PMP duration went up by 30 percent. It is also recognized that there will be some additional reservoir attenuation in the larger reservoir which will reduce the PMF outflow and spillway requirements. Using the PMF inflow for French Canyon was acceptable for this investigation. If the design advances, a new PMF should be computed and routed through the new reservoir to develop the spillway requirements.

4.1.6 Comparison of Alternatives

In this section the advantages and disadvantages of the alternatives are compared using some selected factors or criteria. The factors include foundation suitability, material availability, performance under earthquake and flood loading, contractor experience, diversion requirements and overtopping during construction, adverse weather during construction, and long-term maintenance.

Foundation Suitability

The requirements for foundation excavation and treatment are less stringent for the CFRD than for the RCC dam. The foundation rock below the CFRD plinth must be erosion-resistant and the foundation treatment thorough and complete. However, the plinth area is relatively small compared to the RCC dam footprint, resulting in an overall smaller effort.

Since a high concrete gravity dam (such as the RCC dam) requires a competent foundation of hard rock with low degree of weathering, discovery of poorer foundation conditions during the geotechnical explorations in next design phase would tend to motivate selection of a CFRD.

Conclusion: CFRD may have a slight advantage over RCC dam depending on the results from geotechnical investigations.

Material Availability

The materials that will need to be imported for constructing the RCC dam include Portland cement and fly ash as a minimum. It is assumed that all concrete aggregate would be produced on site using basalt rock, but the suitability of the rock for this purpose will need to be confirmed. Batching of the concrete will be on site.

Concrete for construction of the face slab and the plinth could be batched on site or brought in as ready mix concrete from a commercial concrete supplier. Some processing of rockfill to produce Zones 1 and 2, will be necessary, but the largest volumes of embankment material will be quarry run rockfill.

Because of the relatively small volumes of concrete and processed embankment materials associated with the CFRD, it seems that the risk that material on site may not be suitable or will take excessive effort to process/produce will be smaller for the CFRD than for the RCC dam.

Conclusion: CFRD has a slight advantage over RCC dam.

Performance under Earthquake Loading

Wieland and Brenner (2004) discussed the earthquake aspects of RCC dams and CFRDs and concluded that well-designed and properly compacted CFRDs on rock foundations are considered safe under the strongest earthquake loading (Wieland and Brenner, 2004).

They also noted that, based on a qualitative assessment, it can be assumed that the seismic safety of RCC dams under strong ground shaking is probably satisfactory as cracks in the highly stressed central upper portion of the dam will develop along the horizontal construction interfaces, and that this anticipated behavior is favorable for the dynamic stability of detached concrete blocks during strong ground shaking.

Conclusion: Both alternatives are considered to have similar good performance under seismic loading.

Performance under Flood Loading

Since the initial concept of the RCC dam shows it with a significantly wider spillway, the unit discharge and associated discharge energy for the RCC spillway will be smaller than for the CFRD spillway. In addition, the RCC will be able to resist erosion forces associated with overtopping better than the CFRD embankment.

Conclusion: RCC Dam has an advantage over the CFRD.

Contractor Experience

Relatively few CFRDs have been built in the United States (U.S.). It is estimated that there are about 10 CFRDs in the U.S. compared to more than 100 CFRDs in China, more than 50 CFRDs in South America, and about 25 CFRDs in Australia. Most of the CFRDs have been constructed decades ago, so that there are relatively few contractors with recent experience of CFRD construction in the U.S..

On the other hand, more than 50 RCC dam that are higher than 50 feet have been constructed in the USA, in addition to many others that are smaller than 50 feet. Numerous of these projects have been constructed over the last 10 to 15 years including Pine Brook (86 feet high) and Genesee No. 2 (103 feet high) in Colorado, Hickory Log Creek (188 feet high) in Georgia, New Big Cherry (85 feet high) in Virginia, Elkwater Fork (128 feet high) in West Virginia, and Duck River Dam (135 feet high). There are thus various contractors with recent RCC dam construction experience in the U.S..

Conclusion: Contractors in the U.S. have more experience with RCC Dam construction than with CFRD construction.

Diversion Requirements and Overtopping of Partially Complete Works

Diversion requirements for CFRDs can be significantly less than for an earth dam or an earth core rockfill dam, i.e., you can design the diversions for a flood event with a smaller recurrence interval than for those types of dams. This is because the rockfill layers have relatively high resistance against overtopping and/or flow through and the risk of damages and contamination between the clay, filter/drain and rockfill layers are eliminated. In addition, some rockfill could be placed on the abutments prior to river diversion and rockfill placement in the valley floor. This will decrease the volume of rockfill in the closure section, thus reducing overtopping risk during construction.

However, diversion requirements for an RCC dam can be even less than for a CFRD, because its resistance against overtopping is higher than that of a rockfill embankment.

Conclusion: Diversion requirements for both dam types could be reduced compared to the requirements typically used for construction of earth dams or earth core rockfill dams. RCC Dam has a slight advantage over the CFRD.

Weather Impacts

Rockfill embankment placement is largely independent of weather conditions. Compaction of the rockfill during rainfall is even advantageous as it reduced the wetting requirements.

Cold weather and rain will impact RCC placement, depending on the severity of the conditions.

Conclusion: CFRD has an advantage over RCC dam.

Long-term Maintenance

The concrete face in the CFRD will be affected by aging and degradation; for example, corrosion of steel reinforcing may occur at locations where cracks develop in the slab. It is, therefore, possible that the concrete face, the waterstops in the joints between the slabs, or both, will need to be repaired or replaced during the lifetime of the CFRD.

The elements in the RCC dam that typically need maintenance and repair are the internal drains, including face (lift joint) drains, and foundation drains. These drains may have to be cleaned by flushing accumulated materials with pressurized water using conventional pressure-washing equipment, or it may be necessary to use high-pressure water jetting to remove debris accumulations that are cemented. In a worst-case scenario, where drains get completely clogged with cemented accumulations, they may have to be replaced by drilling new drain holes from the gallery.

In the case that remedial grouting becomes necessary, it will be straightforward to perform such grouting from the gallery within the RCC dam, without lowering or emptying the reservoir. Remedial grouting for the CFRD will be more problematic, and will probably require that the reservoir be drained.

Conclusion: RCC Dam has a slight advantage over CFRD.

4.1.7 Study Limitations, Assumptions, and Exclusions

There are a number of limitations and assumptions associated with this preliminary feasibility study; these are described below.

4.1.7.1 Limitations

Dam Foundation Conditions: The preliminary concept for the proposed North Fork Cowiche Creek dam was basically developed from a brief site visit and a review of existing information, including geotechnical information for the French Canyon Dam, which is located a few thousand feet downstream of the proposed dam. No site-specific subsurface explorations were performed, so that the top of bedrock and the presence of any features such as faults and shear zones, weathered zones, permeable zones, and presence of sedimentary layers (that is, the Ellensburg Formation) in the foundation rock are unknown at this time.

4.1.7.2 Assumptions

The following assumptions were made for this preliminary feasibility study:

- The depth of foundation excavations for the RCC dam is 30 feet at the valley bottom and 10 feet at the abutments. For the CFRD, the depth of the foundation excavation beneath the upstream slope is 30 feet, and 15 feet below the downstream side of the embankment. The depth of excavation at the CFRD abutments is 10 feet.
- Bedrock at a depth of 30 feet along the valley floor consists of hard basalt rock that is moderately fractured, but only slightly weathered to unweathered. Along the abutments, hard competent rock would be encountered at the depth of excavation. For the downstream portion of the CFRD, alluvial soils would be 15 feet thick, i.e., weathered basalt rock would be encountered at a depth of 15 feet and this rock will be strong enough to support the relatively steep slopes of a rockfill dam.
- The basalt bedrock will be moderately to highly jointed or fractured, such that a rigorous grouting program will be required to improve the foundation. A two-row grout curtain, as well as two rows of blanket grout holes were assumed with a relatively high average grout take of about 1 sack of cement per lineal foot of grout hole.
- The foundation requirements for the CFRD will not be as stringent as for the RCC dam, as described in earlier sections. The different foundation treatment requirements for the two alternatives were reflected in the cost estimate with the estimated CFRD foundation treatment being about $700,000 less than the estimated foundation treatment for the RCC dam.
- All embankment zones for the CFRD can be produced on site, with the majority of the embankment, i.e., Zone 3, being quarry run. Zones 1 and 2 can be produced from the same quarries but will require some processing. For the RCC dam, all the RCC aggregate can be processed from a basalt quarry within 1 mile from the dam.
- Groundwater will be higher than the bottom of the excavations, but it can be controlled with typical drainage trenches, and sumps and pumps. No elaborate dewatering system was assumed for this feasibility study.

Other assumptions are described in the descriptions of the two alternatives in the previous sections.

4.1.7.3 Exclusions

The following elements have been omitted in this preliminary study, but represent important issues that must be assessed in subsequent design phases. Most significant is the existing YTID tunnel situated below the proposed dam and reservoir that could have a major impact on feasibility and construction cost of the dam.

Existing YTID Tunnel. As shown on Figure 4 in Appendix A, the rock cover between the bottom of the RCC dam and the YTID tunnel roof is nearly 100 feet thick at the centerline of the dam. However, the rock cover will be significantly less at the downstream toe.

For the CFRD, Figure 13 in Appendix A shows that the toe of the CFRD extends over the tunnel exit in the current concept. To rectify this condition, the tunnel must be extended further downstream or the dam alignment must be changed to miss the existing tunnel exit.

The following requirements will be developed in the next phase: (1) the minimum vertical distance between the existing tunnel and the dam and/or (2) lining the tunnel portions under the reservoir and the dam. These requirements and concepts will depend on the nature of the rock mass, most importantly its permeability and erodibility. Seepage (as well as internal erosion and piping) from the reservoir into the tunnel is a major concern, and the tunnel may have to be lined. A lining system will likely consist of either a large diameter welded steel pipe or a reinforced concrete lining. Where a welded steel pipe is used, the annulus between the pipe and the tunnel will be backfilled with concrete, and contact grouting will be used to seal the contact between the steel and the concrete backfill.

***Potential Seepage at Northern Saddle*.** The northernmost part of the reservoir is bounded by a relatively narrow “saddle” that was interpreted as shallow surficial soils overlying potentially weak or fractured rock; this topographical feature may be attributable to a fault or shear zone. Since the hydraulic gradient between the proposed reservoir and the bottom of the adjacent Tieton River canyon will be relatively large, there is a concern that seepage water could daylight on the canyon walls resulting in erosion and sloughing off the walls. During this preliminary feasibility study, this concern was disregarded, and no ground improvement was assumed at this location.

If permeable rock mass conditions are discovered at this location, they could be mitigated by installing a grout curtain, or other seepage barrier, along the northern ridge. An observational approach (whereby some minimum improvement is implemented, and additional improvements are applied only if unacceptable performance is observed) could be a cost-effective approach.

Other Subsurface Conditions. Northeast-trending faults have been mapped southwest of the site. These faults project generally towards the right abutment of the proposed dam site. The faults are not interpreted to be active; however, fractured and sheared zones may be present in the basalt bedrock. The extent to which these zones may impact the dam will have to be investigated by drilling borings in the right abutment, and some of these borings will need to be inclined.

4.1.7.4 Geotechnical Investigation Recommendations

To confirm feasibility of a dam at the proposed dam alignment, site-specific subsurface information is necessary. It is recommended that a geotechnical exploration be performed that comprises the following six minimum elements:

1. Six borings along the centerline of the dam and two borings along the proposed CFRD spillway alignment: The borings would represent a preliminary investigation that would act to verify or disprove the preliminary assumptions made for the dam foundation and to confirm the general geology. Parameters of interest would include the top of bedrock, strength of the rock, degree and orientation of jointing and fracturing, presence of other rock types besides the Grande Ronde basalt, and permeability of the rock mass. Figure 17 in Appendix A shows preliminary borehole locations. Most boreholes will be vertical, but some may be inclined. Proposed in-situ testing will include the following:

- Water pressure (packer) tests to estimate the permeability of the rock mass.
- Optical and acoustical televiewer logging in the holes to record continuous color images of the in-situ rock. The purpose of this logging will be to determine the nature of the rock in zones of potential rock core loss and to evaluate the nature and orientation of joints, shears, bedding, and other discontinuities in the rock.

2. One boring at the “saddle” at the northern shore of the reservoir to investigate the permeability of the rock and potential for excessive seepage through the ridge: Depending on the results, it may be necessary to incorporate grouting of the saddle and ridge.
3. Five borings at an identified quarry site: This site is tentatively shown as an area that is about 2,000 feet upstream of the right abutment, but this will have to be confirmed in the next phase prior to drilling.
4. A test quarry blasted to dimensions specified by the Engineer: The test quarry will only be performed at the proposed location if the rock cores recovered from the geotechnical borings confirm the suitability of the site. The purpose of the test quarry will be (1) to investigate the suitability of the rock as RCC aggregate, which is a major issue in determining the feasibility of constructing an RCC dam; and (2) to investigate the nature of the rockfill that will be produced as quarry run. Sound rock recovered from the quarry will be transported to the crusher where it will be processed. The processed material will be stockpiled and used for laboratory testing, including for RCC trial mixtures.
5. Visual inspection of the portion of the YTID tunnel below the proposed dam and reservoir: This inspection will occur at a time that the YTID canal is not in operation (that is, when it is dry). It is recommended that a remote camera inspection be performed first to establish safe entry conditions. Where the tunnel is lined, the purpose of the inspection would be to investigate the presence and condition of the lining. Where the lining is absent, the rock conditions in the tunnel will be mapped.
6. Assessment of reservoir seepage rates using any available French Canyon data: Consider the need to perform aquifer testing in the reservoir area to inform evaluation of seepage out of the reservoir.

Since a high concrete gravity dam (such as the RCC dam) requires a competent foundation of hard rock with low degree of weathering, the discovery of relatively poor rock conditions during subsurface investigations could promote selection of a CFRD.

4.2 Pump Station and Pipelines

The pump station and pipelines portion of the proposed project includes all of the facilities requires to transfer water from FCR to NFCCR, and vice versa. The pump station and pipeline facilities are intended to allow flow delivered to FCR from the YTID supply system (assumed to be the Tieton River Diversion) to be pumped to NFCCR during the filling cycle. During the draining cycle, these facilities would allow a controlled release of flows from NFCCR back into FCR for deliveries into the YTID distribution system below French Canyon Dam.

The following components of the pump station and pipeline facilities are described in this section:

- FCR Intake Facilities
- Transfer Pump Station
- Conveyance Pipeline
- Flow Control Station
- Flow Measuring Structure
- Power Transmission System

4.2.1 Facility Siting

Three alternative sites were considered for the pump station and pipeline facilities, as follows:

- **Site 1 – Below French Canyon Dam:** This site was considered since the pump station could use the existing inlet and main outlet pipeline through French Canyon Dam to withdraw water from FCR. There appears to be ample space available in the cleared area below the dam for all required facilities. The pump station would use a piped suction line leading from a new tie-in on the existing French Canyon Dam outlet pipe to the pump suction header; no forebay would be required. A flow control station would be required for pump control and for regulating flows returned to the system from NFCCR. During draining, the flow control station would discharge through a bypass line and back through the pump suction line to the existing outlet pipe. The main conveyance pipeline would extend from the pump station about 4,500 feet to the NFCCR inlet/outlet structure. A bi-directional flow meter would be included on the pipeline.

- **Site 2 – French Canyon Reservoir Platform Facility:** This site takes advantage of the existing reservoir depth and its relatively steep bank along the main access road on its southern side. The candidate site is about 1,000 feet upstream of French Canyon Dam. The pump station would be constructed on a platform over water of sufficient depth to allow proper pump suction conditions at the lowest planned reservoir operating level. One potential location for the platform for the pumps would be near the shore, with a concrete wet well excavated into the bank and extending out into the lake. Alternatively, a trestle arrangement could be used that would extend an access bridge to the pump platform located over the reservoir at a point where suitable depth is available. It was assumed that suitable depth is available about 200 feet into the reservoir from the southern access road. A flow control station would be required for pump control and regulating flows returned to FCR from NFCCR. During draining, the flow control station would discharge to a small outlet structure along the shoreline for return flows to FCR. The main conveyance pipeline would extend from the pump station about 3,000 feet to the NFCCR inlet/outlet structure. A bi-directional flow meter would be included on the pipeline. The site area is steep and narrow. An area for the flow control station, electrical room, and switchyard would likely have to be excavated into the slope south of the existing access road.

- **Site 3 – French Canyon Reservoir Upstream Facility:** This site takes advantage of available space along the southern access road near the upstream extent of the existing FCR. Sufficient space is available between the new NFCCR and the FCR for the pump station. The candidate site is about 2,000 feet upstream of French Canyon Dam and about 900 feet downstream of the proposed North Fork Cowiche Creek Dam. The pump station would be constructed at ground level over a concrete wet well. An approach apron with wing walls would provide the transition from the wet well to the spillway conveyance channel. A flow control station would be required for pump control and regulating flows returned to the FCR. During draining, the flow control station would discharge to a small outlet structure along one wing wall for return flows to FCR. The main conveyance pipeline would extend from the pump station about 1,450 feet to the NFCCR inlet/outlet structure. A bi-directional flow meter would be included on the pipeline. The site is generally flat and ample space is available for all required facilities, plus the spillway conveyance channel.

Site 3 was selected as the candidate site for this analysis for the following reasons:

- Site 3 provides the best combined use of the overall project site. It has the shortest pipeline and is expected to have the lowest cost.
- Site 2 does not have ample existing space. The cost associated with the longer pipeline, the platform pump station facility, and site development requirements is expected to result in the highest relative cost among the three alternative sites.
- Site 1 is a good site with mostly favorable characteristics. However, the cost of the additional pipeline length and tie-ins and piping in the area below the existing dam are expected to approximately offset the potential savings associated with eliminating the pump station wet well. Site 1 would have the highest operating cost due to the longer pipeline and slightly more complex operations associated with using the existing dam outlet works and piping. If the specific design criteria for the project change during more detailed project development (that is, lower flows and smaller pipelines, in particular), Site 1 should be reconsidered. However, Site 3 has slightly better characteristics at this time and is considered the most viable site to demonstrate the feasibility of the proposed project.

Figures 3 and 13 in Appendix A show the proposed site plan for the facilities.

4.2.2 Conceptual System Hydraulics

A conceptual hydraulic analysis was conducted to establish the performance requirements for development of the pump station and pipeline facilities. The key hydraulic criteria shown in Table 4-2 were used.

Table 4-2. Key Hydraulic Criteria

Flow Rate (filling and draining), cubic feet per second	maximum flow – 370	minimum flow - 70
FCR Operating Water Surface Elevations (WSE)	High: 2156	Low: 2151
NFCCR Operating Water Surface Elevations (WSE)	High: 2400	Low: 2200
Friction Loss (Hazen Williams “C” Value)	High: 150 (lowest friction-new pipe)	Low: 120 (highest friction-old pipe)
Discharge Pipe Length (feet)	1,450	
Discharge Pipe Diameter (inches)	96, nominally sized at less than 8 feet per second flow velocity	

The hydraulic criteria above were used to create the system-head curve operating envelope for the transfer pump station. System-head curves show the relationship between the flow rate and total dynamic head (TDH) for conditions specific to the proposed pumping and pipeline system. Figure 4-6 shows the upper and lower system head curves and the resulting operating envelope for the proposed pump station. The operating envelope is the area between the upper curve, which represents the worst case TDH condition for any given flow rate, and the lower curve, which represents the best case condition. The curves were developed as follows:

- Upper (worst case) curve—static lift from the lowest FCR WSE to the highest NFCCR WSE and the highest friction
- Lower (best case) curve—static lift from the highest FCR WSE to the lowest NFCCR WSE and the lowest friction

The system-head information was used to select candidate pumping equipment capable of performing at all points within the operating envelope.

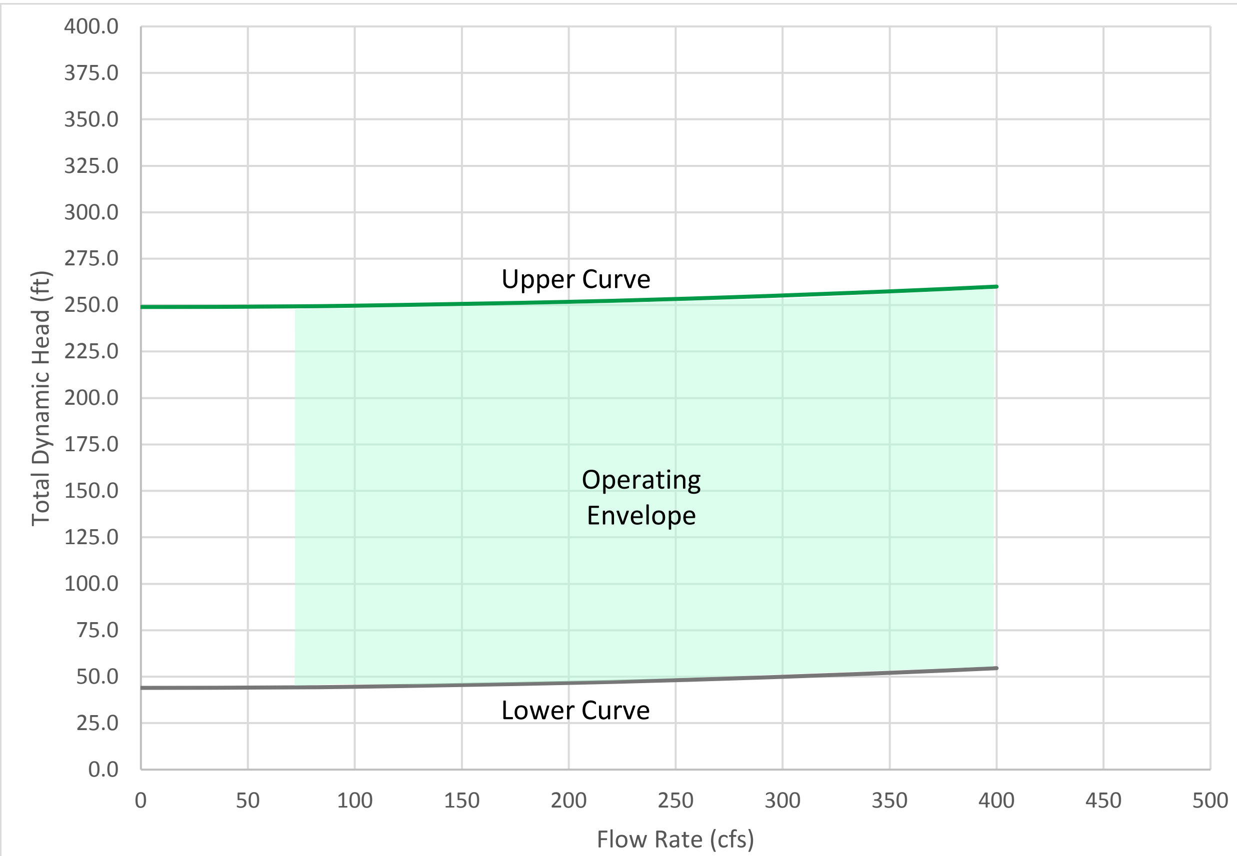

Figure 4-6. System-Head Curves and Operating Envelope for the Transfer Pump Station

4.2.3 Facility Descriptions

The development and configuration of each component of the overall pump station and pipeline facilities are described in the following sub-sections.

4.2.3.1 Intake Facilities

The intake for the proposed transfer pump station must accommodate conveying water from FCR into the wet well at flows up to the maximum rate. The intake is comprised of two main features described in this section.

Intake Transition

For an RCC dam, the intake transition is the transitional area between the spillway conveyance channel and the face of the pump station wet well (Figure 6 in Appendix A). The intake transition for a CFRF dam has a similar configuration, but spills from the NFCCR bypass to the pump station (Figure 13 in Appendix A). The transition includes a concrete apron on the bottom of the area and concrete wing walls flaring out along each side to provide a structural and hydraulic transition from the wet well to the channel excavation. The apron and wing walls are shown on the drawings.

The conceptual layout of the intake transition includes a 25-foot apron width and wing walls flaring at 45 degrees to the wet well face. This configuration provides a buffer between high velocity spillway flows and the face of the wet well. The bottom of the transition area was set at an elevation of 2,140 feet to provide suitable submergence for intake screens or racks at the low operating WSE in the FCR. These dimensions should be verified as the design is further developed.

Intake Screens or Racks

Intake screens or trash racks are the intake features installed on the outside of the upstream wall of the pump station wet well. For the proposed project, either fish screens or trash racks may be appropriate for the intake. The drawings were prepared showing six 60-inch-diameter by 18-foot-long stainless steel tee screens. Tee screens provide fish protection and are considered the most conservative approach for this intake. Since the Tieton River diversion already includes fish screening for all flows diverted to the FCR, the fish screens are not expected to be required on the transfer pump station intake. However, the screens were shown on the drawings to demonstrate their configuration on the structure if they are required.

Trash racks could also be used to cover the wet well opening for the intake. Six 8-foot-square stainless steel racks were assumed for this analysis. Other shapes and sizes should be considered during final design.

Fish screens require cleaning at a frequency significantly high than trash racks; normally several times per day. In some cases, the cleaning frequency is a regulatory requirement. For the conceptual development of this intake, tee screens with an integral hydraulically actuated rotating brush style cleaning system were assumed. Electrically actuated systems and screen cleaning using an air burst systems are also available.

The cost estimate for the project assumes that trash racks will be provided. If fish screens are required, the cost of the facility would increase by approximately $2.5 million.

Intake Operational Concept

The intake operational concept assumes that the fish screens or trash racks would be in place over the wet well inlet openings whenever the FCR WSE is above elevation 2140. in that case, all flow entering the wet well would pass through a rack or screen. The screens and racks would be installed on guide rails mounted to the face of the structure from the intake openings above to the top of the wet well. These guide rails allow the screens or racks to slide up and down for maintenance, cleaning, installation, or removal. Trash racks would be lifted to the top deck of the facility using a boom truck. Fish screens may be lifted in a similar manner or may have their own integral winch system that would allow each screen to be lifted to the top deck using a local control panel. The screen access deck is sized to allow a boom truck to operate between the intake wall and the pump building with outriggers in place.

4.2.3.2 Transfer Pump Station

The transfer pump station is intended to lift water from the FCR to the NFCCR under all operating WSE conditions and the full range of flows.

Pump Station Development

The primary evaluations conducted to assist in the development of the transfer pump station are discussed as follows:

Number of Pumps

The system-head curves and operating envelope described previously were reviewed with several major vertical turbine pump manufacturers. Each was requested to provide pump curves that they thought were the best fit for the performance requirements for this system. Generally speaking, the resulting offers for candidate pumping equipment ranged from four to seven pumps.

Figure 4-7 shows four pumps (from one manufacturer), operating at full speed and minimum required speed, superimposed on the system head curves. The use of full-speed pumps alone would not provide acceptable pump station performance, because full speed pumps are capable of operating only when the TDH is 180 feet or higher. The pumps would not perform properly (for example, recirculation or cavitation could occur) when the NFCCR is partially full and the TDH is less than 180 feet. Also, there are wide gaps in the available pumping rates. For example, a pumping rate of 250 cfs is not feasible using four, full-speed pumps without further pump control.

Two options are available for providing a wider range of pumping flexibility and performance. The first is throttling the pumps using flow control valves, and the second involves using variable speed pumps. Throttling would introduce head loss into the system, such that at least 180 feet of TDH is developed at all times. Since the single pump curve crosses the upper system-head curve at about 100 cfs, throttling would be required in excess of the upper curve head (full NFCCR) to achieve 70 cfs. Throttling could be used to achieve any combination of flow and head (reservoir level). For example, at 250 cfs, three pumps could be throttled at a TDH of about 275 feet to achieve that flow.

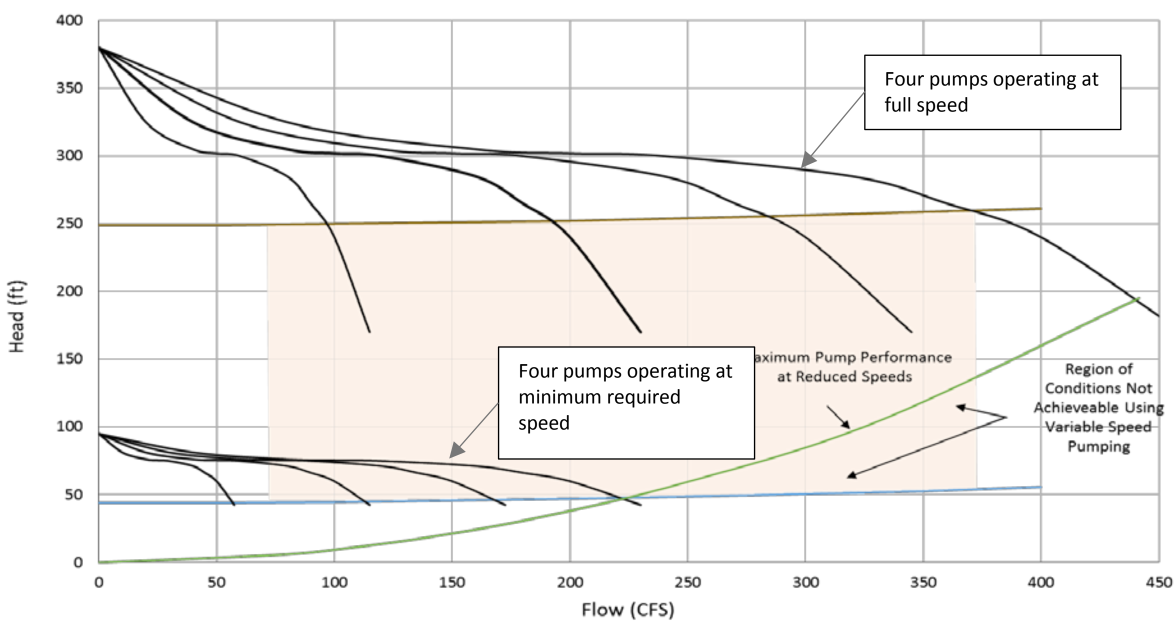

Figure 4-7. Four Pump Performance Relative to Conceptual System Head Curves and Operating Envelope

Figure 4-7 also illustrates the use of variable speed pumping. If variable speed drives are used to control flow, then Figure 4-7 shows a substantial region of the operating envelope, particularly at the higher flow rates, that cannot be accomplished at the lower TDH ranges of this system (that is, all areas to the right of the reduced speed coverage curve). This is especially problematic since the highest flow needs are expected to be at times when the TDH is the lowest (corresponding to times that the NFCCR is at or near empty and filling begins each year).

The use of additional pumps does not alleviate the inability to operate in the high flow and low TDH portion of the operating envelope using variable speed pumping. However, additional pumps may allow the minimum flow rate of 70 cfs to be accomplished with reasonable throttling, and potentially no throttling above the TDH required from high water levels in the NFCCR. As noted previously, Figure 4-7 shows that a TDH of about 290 feet would be required to achieve a flow rate of 70 cfs for one of the pumps in a four-pump system. This requires throttling to generate head far in excess of that required to pump to the reservoir at higher water levels. Conversely, Figure 4-8 shows a system with six pumps, and 70 cfs can be accomplished at a TDH of about 220 feet, which is less than the TDH from a full reservoir and is accomplished at a reasonable amount of throttling. Six pumps is the lowest number of pumps with which the minimum design flow can be accomplished without excessive throttling (above full lake level).

Figure 4-8 shows gaps where some flowrates still cannot be accomplished without excessive throttling. For example, at 100 cfs, two pumps would have to be throttled to about 285 feet of TDH, well above the full lake level TDH. Similarly, the flow gap between one and two operational pumps without excessive throttling is about 75 to 125 cfs. However, given the size of the existing FCR, staging pumps on and off (such that only flows within the available range are used) is expected to be feasible. Also, flow rates above about 250 cfs (four pumps operating) would not experience flow gaps.

The use of more than six pumps would reduce the flow gaps and be even more compatible with throttling, but would require a larger structure and probably increase the capital cost of the project. It is expected that the flow gaps can be managed and the additional cost of more pumps may not be warranted. Therefore, six pumps, as illustrated on Figure 4-8, were selected for this analysis, since six pumps provide a reasonable structure size, are compatible with reasonable throttling, and appear to have manageable flow gaps. However, the trade-offs between operational goals and cost should be reconsidered relative to the exact number of pumps during final design.

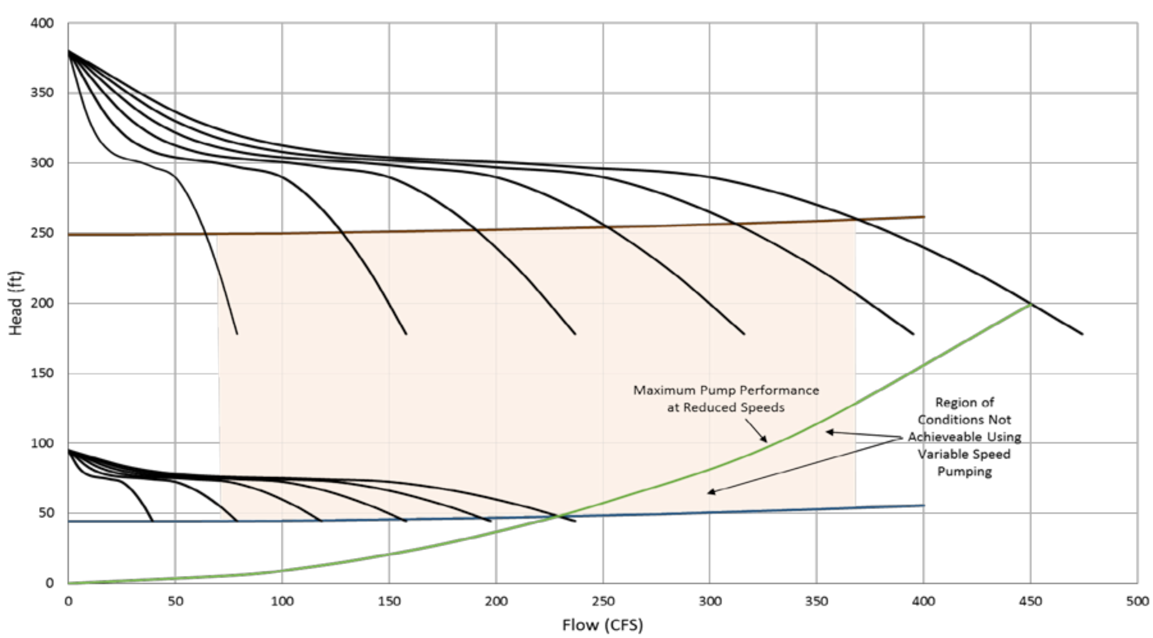

Figure 4-8. Six-pump Performance Relative to Conceptual System-Head Curves and Operating Envelope

Throttling Versus Variable Speed Operation

As described previously, the range in the operating envelope illustrated on Figure 4-6 cannot be covered by constant speed pumps. In order to achieve the flexibility implied by the wide operating envelope, either throttling or variable speed operation is required. Figures 4-7 and 4-8 also show that some of the higher flows in the lower TDH range will not be achievable using a variable speed operational scheme, even with six pumps. Therefore, throttling will provide better operability for the system, because it can be used to achieve any head condition within the operating envelope and most flows.

However, before throttling is selected as the flow control method by simple inspection, it is considered worthwhile to evaluate the cost impacts of throttling versus the use of variable speed pumping. This evaluation should determine if there is a compelling case to accept the reduced flexibility imposed by variable speed pumping as a more cost effective approach to the new facility design.

A life cycle cost analysis was conducted to compare the present worth cost of constructing, operating, and maintaining the system using a throttling or a reduced speed pumping approach. The following key parameters were used:

Analysis Period: 100 years - Overall facility expected to have a useful life of 100 years if properly maintained and with applicable equipment replacement

Discount Rate: 3 percent

Power Cost: $0.10 per kilowatt-hour

Capital Costs: Variable Speed Pumping - Variable frequency drives (VFDs), additional floor space for the electrical room for VFDs, and flow control facility to release flows from NFCCR to FCR

Throttling: Flow control station used for both throttling of pumped flow and control of flows released from NFCCR to FCR

Recurring Costs: Variable Speed Pumping - Replace VFDs every 20 years, replace flow control facility valves every 40 years, annual power consumption at average pumping head and pumping efficiency consistent with VFDs

Throttling - Replace flow control facility valves every 40 years, annual power consumption for average throttling heads and pumping efficiency consistent with constant speed motors

Capital costs were developed using cost estimates prepared for proposed system components where available (see Section 7). Other features were estimated using the same methodologies.

A net present value was developed for each alternative using the life cycle information described previously. The resulting values were a net present value of $34.5 million for the throttling alternative versus $34.3 million for the variable speed pumping Alternative. At the level of estimating used for the analysis, these values are identical. Initial capital costs are similar, but VFD replacement costs are higher for variable speed pumping, while power consumption is greater for throttling. If the actual cost of power is less than $0.10 per kilowatt-hour (kW-hr), this analysis would tip in favor of throttling. Conversely, if the cost of power is greater than $0.10 per kW-hr., this analysis would favor the use of variable speed pumping.

Since variable speed pumping does not show any long-term cost advantage over throttling, and throttling will allow the flexibility to pump over the full operating envelope, throttling was selected as the preferred configuration for the proposed transfer pumping system. Therefore, a flow control station capable of throttling flows in both directions (that is, from FCR to NFCCR, and from NFCCR to FCR) will be included in the pump station and pipeline facilities. Refer to Section 4.2.3.4 for a more detailed description of the proposed flow control station.

Pump Station Configuration

Vertical turbine pumps were used to conceptualize all configuration alternatives for the transfer pump station.

Horizontal pumps would require a deep dry well for proper hydraulic performance. A deep dry well configuration is normally used for a complex facility and is not expected to be cost effective for this project.

Submersible pumps could also be considered, but the basic facility configuration would not be substantially changed from that used for vertical turbine pumps. Submersible pump stations may not need the above-grade pump room building, which might provide some cost savings. Submersible turbine pumps and motors are typically more costly, less efficient, and have less performance curve choices compared to conventional vertical turbine pumps. However, submersible turbine pumps could be a viable option and a more detailed review of the long term cost and operational trade-offs should be conducted during final design. In any case, the cost of the transfer pump station is not expected to be materially lower if submersible pumps are used.

Therefore, vertical turbine pumps represent a good baseline for developing the feasibility of the proposed project and were used for the conceptual analysis.

The following three pump station configurations were considered:

- **Can-mounted Pumps**—This alternative includes mounting each vertical turbine pump in a pump can. A cross section depicting this configuration is shown on Figure 9 in Appendix A. The pump can is fed from a supply lateral connecting to a fish screen or trash rack installed on an intake headwall at the intake apron. The pump can extends down to the full depth required to provide suitable net positive suction head conditions (down to about elevation 2115). The pump discharge head, motor, and discharge piping are above grade in a building (identical to the other alternatives). This configuration is generally one of the lowest cost alternatives. However, the size and depth required for the proposed pump cans requires tight tolerance installation, and the cost difference relative to other alternatives is not expected to be substantial.
- **Conventional Wet Well**—This alternative includes mounting each vertical turbine pump over a concrete wet well that extends down to the full depth required to provide suitable net positive suction head conditions (down to about elevation 2115). The wet well is fed from openings in the upstream wall of the wet well fitted with fish screens or trash racks. The pump discharge head, motor, and discharge piping are above grade in a building (identical to the other alternatives). This configuration is generally one of the highest cost alternatives due to the depth of the large concrete wet well structure and need for intake bays for each pump. However, it is the most accessible and is often preferred for life-line pumping facilities.
- **Wet Well with Open Top Can**—This alternative includes mounting each vertical turbine pump over a concrete wet well that extends down only to the depth required to facilitate entry of flow into the wet well from the intake apron. A cross section depicting this configuration is shown on Figure 8 in Appendix A. An open top can is installed in the floor of the wet well to allow the pump inlet bell to extend down to the full depth required to provide suitable net positive suction head conditions (down to about elevation 2115). The wet well is fed from openings in the upstream wall of the wet well fitted with fish screens or trash racks. The pump discharge head, motor, and discharge piping are above grade in a building (identical to the other alternatives). This configuration is generally an efficient design, combining the accessible nature of the wet well with excellent hydraulic performance of the cans. It is usually cost competitive with large can-mounted facilities similar to the proposed transfer pump station.

Both the Can-mounted Pump and Wet Well with Open Top Can alternatives should be considered during final design. Site-specific cost details are needed to clearly differentiate between the two alternatives. The Wet Well with Open Top Can configuration was used as the basis for the cost estimate for this analysis, since it is easier to define at this stage of project development. Also, the Wet Well with Open Top Can arrangement has generally more favorable non-cost characteristics, so the Can-mounted Pump configuration would only be selected if it were lower cost. Therefore, the use of the Wet Well with Open Top Can configuration is expected to provide conservative cost estimates for this feasibility assessment.

Pump Station Facility Description

The Transfer Pump Station includes five main components, described in the following sub-sections. Refer also to the drawings for illustrations of the concepts and additional detail.

Wet Well

The wet well is a concrete structure used to allow flow from the intake transition area to be directed to the project pumps. The wet well is about 145 feet long to accommodate the intake screens and physical space for the pumps and motors. The wet well is about 50 feet wide to accommodate the screen access deck and provide space to transition flow from the intake openings to the pumps.

The top deck of the wet well is at elevation 2170 to approximately match the existing site grade and to provide freeboard above FCR WSEs and spillway flood flows. The bottom of the wet well is at elevation 2140 to match the intake apron and provide suitable depth to transition flow from FCR into the wet well.

There are six intake openings with fish screens or trash racks. Each is generally aligned with one of the six pumps.

Six, approximately 6-foot inside diameter, open top pump cans extend below the bottom of the wet well (that is, one beneath each pump). The cans are intended to be stainless steel inserts grouted into a can shaft drilled (or otherwise excavated) below the bottom of the wet well. The cans extend to a bottom elevation of about 2115 to provide suitable net position suction head for the pumps. The cans will be provided by the pump manufacturer, and the vertical turbine pumps will be installed from above into the cans with typical clearance dimensions consistent with Hydraulic Institute standards. The exact configuration of the cans is dependent on the pump manufacture's approach to the pump intake and will be developed in additional detail during final design.

Pumps, Motors, and Discharge Piping

Six vertical turbine pumps are proposed. The design point for each pump is 62 cfs at 258 feet TDH. Preliminary selections indicate a two-stage pump with rotational speed of 710 revolutions per minute. Each pump will be driven by a constant speed, 2,500-horsepower (nameplate), motor. Pump efficiency in the operating envelope is expected to range from 75 to 83 percent.

Preliminary pump sizing information indicates 36-inch steel column pipes, cast steel suction bells and bowl casings, and a fabricated steel discharge head with 32-inch diameter discharge nozzle. Specific details were not provided by manufacturers, but project experience indicates the pump and motor units will extend about 18 to 20 feet above the pump room floor.

Pump discharge piping includes a dismantling coupling at the pump discharge nozzle, a 32-inch by 42-inch reducer, a 42-inch check valve, a 42-inch isolation butterfly valve, and miscellaneous piping to connect to a 96-inch discharge header encased under the floor of the pump building.

A supplemental water supply is required to prelubricate the open pump line shafts before each pump is started. This system can be filtered raw water or potable water. Motor lubrication integral to the motor thrust bearing is required and associated lube-oil cooling may also be needed.

Motors are expected to be air-cooled using outside air supplied to the building enclosure.

Each pump and motor will include various instruments to monitor pressure, temperature, and vibration and provide control and protective functions.

Pump Building

A pump building will be provided for environmental protection for the pumps, motors, and discharge piping. The building will be about 145 feet long, 60 feet wide, and 45 feet tall. The building walls will either be concrete masonry unit (CMU) or precast concrete panels with a concrete or steel framing system. A joist supported, built-up elastomeric roofing system (or similar) will be used. The building will include a 50-ton bridge crane to install or remove all equipment. Large roll-up doors are positioned at either end of the building to support a drive-through concept to facilitate operations and maintenance.

Large roof-mounted exhaust fans will draw outside air through louvers at each motor to provide ventilation and cooling. Unit heaters will be provided to maintain a minimum temperature during the winter. A series of man-doors will facilitate ingress and egress to the building. Suitable space is available within the building for laydown and work areas and for locating miscellaneous appurtenant systems that may be defined in greater detail later in project development.

Electrical and Control Room Building

A 100-foot-long by 40-foot-wide electrical and control room building is proposed for the project. The building size was estimated from similar projects, but no specific layout was developed for this project. The electrical room is expected to house the main electrical switchgear, motor control centers (MCCs), reduced voltage pump starters, low voltage panel boards, and other related equipment. Additionally, the building includes space for a control room for local control workstations as well as a programmable logic controller (PLC), supervisory control and data acquisition, network, communications, security, and associated control equipment.

The electrical and control room building is expected to be air-conditioned for worker comfort (including both heating and cooling systems). Heating, ventilation, and air conditioning equipment is expected to be roof-mounted.

Site Development

The entire site around the pump station and pipeline facilities will be graded for vehicular access around all sides of each facility, including a traffic flow path onto and across the screen access deck at the intake. Pavement is planned for the main access road and the main access around the buildings and flow control station. Access past the flow control station up to the flow measurement structure and the dam would be all-weather gravel. Employee parking has not been sited, but suitable space is available at several locations adjacent to the main buildings and switchyard. Sidewalks and door stoops will be provided for the pump building as well as for the electrical and control building.

The site will be graded for drainage to FCR. Some drainage may need to be directed to the east to avoid the built-up berms at the site needed to contain the spillway flood flows.

Miscellaneous site features like protective guard posts and site lighting are included in cost allowances, but have not been specifically detailed at this time.

No fencing except for the power supply switchyard is planned. However, an access gate for entering the area should be considered along the access road east of the site.

Pump Station Operational Concept

The operational concept for the pump station will depend on the specific flow requirements needed to operate the system. In general, operations staff or the control system would need to select the correct number of pumps for the desired flow and head conditions. The flow control station would then be used to modulate the flow control valves to the selected flow rate. For this control scheme, the operators, or the control system, would need to understand the relationship of the desired flow rate with the pump performance curve and current head conditions to select the proper number of pumps, considering the fact that some flow gaps will occur under some conditions.

Also, the pump control scheme will need to consider the effect of pump operation on the water level in the lower reservoir. If minimal changes in the reservoir level are desired to help minimize changes in setting for flow releases below the reservoir, then the transfer pump operation will require close scrutiny to help avoid large mismatches between the incoming flow, the released flow, and the pumped flow. The combination of these control requirements could be a fairly complex control scheme to operate manually, but would be relatively simple to develop control logic to either automatically operate the pumps or provide the correct pumping configuration to an operator for selecting the number of pumps and notifications for manual starts and stops.

The actual control scheme will be developed as part of detailed design.

4.2.3.3 Conveyance Pipeline

The main conveyance pipeline extends from the transfer pump station to the inlet structure on the reservoir bottom upstream of the North Fork Cowiche Creek Dam. The pipeline starts at the downstream end of the pump discharge header buried beneath the floor of the pump building, passes through the flow control station and flow metering structure, and terminates at the intake structure above the dam. The pipeline will provide bi-directional flow capability to allow for both filling and draining of the NFCCR.

The pipeline is proposed to be 96 inches in diameter. It was nominally sized to limit velocity to less than 8 feet per second at the maximum flow rate. It is about 1,450 feet long and expected to be a cement mortar lined and coated welded steel pipe, generally in accordance with AWWA C200. A nominal wall thickness of 0.5 inch was assumed for this analysis. This wall thickness is thicker than required for pressure and loading conditions, but was considered appropriate given the critical nature of the pipeline and its position beneath the dam.

It is expected that the pipeline will be excavated into the native soil and rock for all areas at the site except directly beneath the dam. Under the dam, the pipeline will be excavated into the rock strata beneath the dam and concrete encased. To prevent seepage, special dam fills will be placed immediately above the encasement for the remainder of the pipe trench beneath the dam. For areas outside the dam foot print, a sand-cement slurry is assumed for the pipe zone, with native materials and a road structural section, as applicable, above the pipe zone.

Corrosion control for the pipeline will be evaluated in greater detail during final design. Cathodic protection as well as applicable linings and coating will be evaluated.

Appurtenant features for the pipeline are expected to be contained within other structures. As such, the only appurtenant features along the pipeline will be one or two buried access manways between the dam and the flow control station.

4.2.3.4 Flow Control Station

Figure 10 in Appendix A illustrates the proposed flow control station used for both filling and draining of the NFCCR. During filling, the flow control station will provide the throttling head to allow the main pumps to operate within their acceptable operating region for any flow and head situation. During draining, the flow control station will provide the needed head loss to regulate flows from NFCCR into FCR.

The flow control station was conceptualized with a bypass pipe and several isolation valves to regulate system pressure in both directions using the same set of flow control valves:

- During filling of the NFCCR, flow enters the station from the pump station, flows through the control valves, and discharge into the conveyance pipeline and ultimately into the NFCCR. The large isolation valve on the bypass pipe would be closed. The large isolation valve on the NFCCR side of the control valves (downstream left in the drawings) would be open to allow flow to the upper reservoir. The large isolation valve on the FCR side of control valves (downstream right in the drawings) would be closed to prevent flow to the lower reservoir.
- During draining of the NFCCR, the flow enters the station from the upper reservoir, flows through the bypass to the control valves, and discharges into the short discharge pipeline connecting the station to the FCR outlet structure at the intake apron wing wall. The large isolation valve on the NFCCR side of the control valves (downstream left in the drawings) would be closed, and the bypass isolation valve would be open. These valve positions force upper reservoir flow to the upstream side of the control valves. The isolation valve on the discharge side of each pump would also be closed, causing all flow to be directed through the control valves in the flow control station. The large isolation valve on the FCR side of control valves (downstream right in the drawings) would be open to allow flow to the lower reservoir.

The flow control station is a large concrete vault structure containing the isolation and control valves required for flow regulation. The structure is assumed to have a fully removable aluminum cover (insulated) to minimize the depth and associated cost of the facility. This approach eliminates the need for a bridge crane, but requires that a portable crane be used to install/remove equipment. Since these valves are not expected to require frequent removal and/or replacement, the removable cover system was included.

Flow control will be accomplished using three flow control trains, each using a 36-inch flow control valve. The flow control valves are shown on the drawings as sleeve valves. However, plunger valves are expected to provide the same service and may be slightly shorter and slightly lower cost. Each flow control train also includes a 48-inch isolation butterfly valve on the upstream end and a 36-inch isolation ball valve on the downstream end. Since the flow control valves result in turbulent flow at their discharge end, the full port ball valve is recommended on the downstream side. Both the upstream and downstream sides connect to header pipes to complete the flow control piping.

Flow control valves were preliminarily sized to provide both filling and draining flow regulation:

- For filling, the valves will modulate to maintain at least 190 feet TDH on operating pumps over the full range of flow (70 to 370 cfs). Once the upper reservoir is filled to the point where throttling head is no longer needed, the valves will open fully and provide only nominal head loss.
- For draining, the valves will modulate to provide whatever head loss is required to release the full range of flow (70 to 370 cfs) from the upper reservoir to the lower reservoir. For the draining condition, the flow control valves must have a small amount of back pressure on their discharge side to limit the cavitation potential. Therefore, a small outlet/overflow structure is provided at the west intake apron wing wall near the approach to the pump station. Flow released over the top of the outlet structure will maintain about 5 to 7 feet of head on the downstream side of the control valves.

A variety of instruments will be provided to monitor valve position and pressure in the flow control station. A PLC will be located at the flow control station to provide control logic required to modulate the flow control valves. For filling, the control logic will maintain a constant upstream pressure, and possibly a constant flow rate (operator selected), depending on operational constraints. For draining, the control logic will modulate the valves to provide a constant flow rate (operator selected) from the upper to the lower reservoir. The control system will use the flow rate signal from the flow measurement structure as a control parameter.

4.2.3.5 Flow Measurement Structure

A flow measurement structure is proposed to facilitate both filling and draining of the NFCCR. The flow measurement structure will be placed along the main conveyance pipeline a suitable distance downstream of the flow control station to facilitate measuring flow in a straight pipe.

A multi-path ultrasonic flowmeter was assumed for this application since it is relatively accurate and allows flow measurement in either direction. Maximum flow accuracy is not expected to be needed and other meter types could be considered during detailed design.

The flow measurement structure is a concrete vault structure containing the metering equipment. The vault will be fairly wide (about 20 to 30 feet), depending on meter characteristics. It must be wide enough to accommodate installation and removal of the ultrasonic probes. The structure is assumed to have a concrete cover since the ultrasonic meter components are small and can be removed through a roof hatch. It is expected that the flow signals will be communicated to the network via the PLC at the flow control station for control purposes, flow and volume documentation, and operator information.

4.2.3.6 Power Supply

The power supply will include transmission lines to the onsite switchyard from Bonneville Power Administration, Pacificorp, or Benton REA. The exact supply voltage and source of the power must be reviewed with the power supplier and is not yet known.

Incoming power will be connected to two 2 MVA transformers (nominal conceptual size) with appurtenant switching, power measurement, and protective devices. Each transformer string will be located in a fenced switchyard adjacent to the electrical building. Transformer strings will be sized for the full project loads (that is, all dam facilities plus pump station and pipeline facilities). Transformers will reduce supply voltage to 4160V station utilization voltage. Each string will be connected to opposite ends of a main-tie-main switchgear line-up in the electrical and control building. Normally, both transformers will be in operation, and the switchgear tie breaker will be open. In the event of equipment failure on one transformer string, the full station can be run from a single side by opening the associated main breaker and closing the tie breaker.

The switchgear is assumed to feed a reduced voltage starter for each pump. It will also feed associated MCCs for other facility loads. Step down transformers will be provided at the electrical and control building as needed for 480V and 110/220V loads. Power will be distributed to main panels at each major facility, as determined during detailed electrical design.

The total facility connected load is expected to be about 12 kW. However, peak load will probably not exceed 11 kW, since the primary load is for pumping and the actual peak load on the pump motors is lower than the nameplate load of 2,500 horsepower (depending on actual pump efficiency). The pump motors account for 15,000 horsepower (or about 11.22 kW) of connected load. None of the other facilities or appurtenant features have significant electrical load, so a total connected load of about 12 kW is expected.

4.2.4 Hydroelectric Power Considerations

Since the project includes pumping flows from the lower reservoir to the upper reservoir and later releasing it back to the lower reservoir, the potential exists to generate power when the flows are released.

Depending on the annual yield of the project, it is expected that power generation could result in about $275,000 to $315,000 of power revenue per year. This revenue generation assumes power can be sold at $.05/kW-hr, water stored above elevation 2300 in the upper reservoir can be used to run the turbines, and the annual average project yield ranges from 27,000 to 30,000 acre-feet per year. The annual power revenue results in a net present value for power generation ranging from $8.4 million to $9.7 million over a 100-year project life.

The net present value of the annual power revenue was conceptually compared to the net present value of the additional costs required to implement and maintain the project with a hydroelectric power generating component. Power generation would require that pump-turbine units be used in place of the proposed vertical turbine pumps and motors. Minor electrical system changes would also be required. Since the turbine units cannot be used at low upper reservoir water levels and throttling is still required for pumping, the flow control structure is still needed. Therefore, outfitting the facilities for hydroelectric power generation would not eliminate any equipment and would add to both the capital and replacement cost of the facility.

Since pump-turbine units would essentially run almost double the number of hours each year due to their role in both the filling and draining operations, the pump turbine units would be expected to require replacement more often than the pumping units. For this analysis, it was assumed that the pump-motor units would require replacement every 30 years, while the pump-turbine-motor units would require replacement every 20 years.

A present worth analysis was conducted where the initial capital and periodic replacement cost for the pumps was compared to the potential power revenue and the initial capital and periodic replacement cost for the pump-turbines. No additional cost was attributed to electrical components, and no cost penalty was assigned to the pump-turbine units for potential nominally lower pumping efficiency than the pumping units. Further, no additional operations and maintenance (O&M), legal, or administrative costs were assigned for operating the project to generate power and wheel it into the regional power grid. The present worth analysis revealed that the pump turbine units would need to be no more than 22 percent more costly than the pumping units to break even. This translates to about $2.5 million initially and at every 20-year increment.

The specific cost of the pump-turbine units was not developed for this analysis, but it is reasonable to expect that it may be possible to obtain these units within the additional $2.5 million incremental cost. However, given the limited operating season for the proposed project, using it for hydro-electric power generation does not appear to provide the potential for significant revenue. In fact, once administrative issues and the nominally higher cost of O&M and some of the appurtenant equipment are considered, it is possible that power generation may only provide something close to a break-even situation, at best. If it is determined that the administrative and legal burden associated with hydroelectric power generation is worthwhile, then a more detailed assessment of the cost trade-offs related to power generation should be revisited when the project is further refined during implementation.

Project Operations

5.1 Available Water Supply

The available water supply for this project is dependent on several factors, including but not limited to Rimrock Reservoir operations, instream flow requirements, hydrology and watershed yield, and capacity of YTID's Tieton Canal:

- Rimrock Reservoir storage and operations. The data available for Rimrock Reservoir shows that, in most years, the reservoir fills to capacity, usually reaching full pool between late May and early July. By early springtime, the U.S. Bureau of Reclamation (which operates releases from the reservoir) has a relatively accurate assessment of the available snowpack in the watershed above the reservoir, and excess water is released from the reservoir in order to make room for spring runoff, snowmelt, and flood control storage. Water releases from Rimrock will often occur early in the spring, before the water is needed for irrigation purposes. Spring releases from Rimrock constitute the majority of the available water supply for the NFCCR. (U.S. Bureau of Reclamation, 2011)

  The date when the U.S. Bureau of Reclamation begins releasing water from reservoirs to meet irrigation demands is known as the storage control date. Historically, storage control occurs on June 24. After the storage control date, the U.S. Bureau of Reclamation releases only the amount of water required to meet agricultural and instream flow demands downstream. This study assumes that any water released from Rimrock to the Tieton River, prior to the storage control date and in excess of minimum instream flow requirements, currently flows downstream into the Yakima and Columbia Rivers and eventually to the sea. This volume of water is assumed to be available to the North Fork Cowiche Creek Reservoir for storage and later use.

- Instream target flows and project operations. Instream target flows for fish are relevant to this project and consist of several components. With some minor exceptions, current minimum instream flow targets in the Tieton River below YTID's point of diversion (TICW) must meet or exceed 75, 100, and 120 cfs in dry, normal, and wet years, respectively. In addition, there are also minimum target flow requirements of 450 cfs or the natural flow (whichever is less) at the stream gage located on the Naches River near Naches, which is downstream of the confluence of the Tieton and Naches Rivers. In addition to these target flows, there is a volume that is typically released from Rimrock Reservoir in the April and May timeframe that must pass downstream in the form of a pulse flow. This experimental pulse flow is largely intended to mirror the timing and general magnitude of unregulated flow and assist with the outmigration of salmonids. It is dependent on estimates of total water supply available (TWSA) and other factors that influence The Yakima Project operations. TWSA is the total water supply available for the Yakima River Basin above the USGS gage at Parker (PARW), located below Union Gap and the Sunnyside Diversion Dam), for the period April through September," expressed in a mathematical formula, reading as follows:

  April 1 through July 31 forecast of runoff

  \+ August 1 through September 30 projected runoff

  \+ April 1 reservoir storage contents in the five major reservoirs that serve the Yakima River Basin

  \+ Usable return flow upstream from Parker gage

  = TWSA

The total demand to be placed against this TWSA for irrigation, regulation, and flows passing Parker gage averages 2.7 million acre-feet in a normal year. (U.S. Bureau of Reclamation, 2011)

It is noted that, prior to 2011, instream target flows were significantly lower than those currently established. For the purposes of this feasibility study, availability of flow from the Tieton River is principally based on maintaining 100 cfs in the river at the TICW gauge located below the YTID diversion. Other instream flow requirements are highly dependent on operations and major reservoir releases throughout the Yakima River Basin, and these requirements can be addressed during subsequent studies or design phases of NFCCR.

- <u>Hydrology and watershed yield.</u> A complete and thorough hydrologic analysis to determine potential inflow or watershed yield into Rimrock Reservoir is complex and beyond the scope of this feasibility study. Generally speaking, however, historical flow released from Rimrock to the Tieton River usually exceeds the combination of instream flow requirements and downstream agricultural demands. In other words, water is intentionally spilled because the inflow exceeds the Rimrock storage capacity in most water years, indicating that there is sufficient water available for diversion and storage in the NFCCR. Estimates of the available water supply for NFCCR are presented in Section 5.3.

5.2 Water Demand and Water Use

In 2013, CH2M completed a study to evaluate alternatives to address the YTID Tieton Canal. The study included rehabilitation alternatives and cost estimates (CH2M, 2013). In order to achieve YTID's long-term goals for a reliable water supply and flexibility to support other water users, the preferred project alternative identified in that study consisted of expanding the canal capacity to 370 cfs. A canal capacity of 370 cfs would provide sufficient capacity to accommodate both YTID's peak demand (345 cfs) and the peak demand for the Cowiche Creek Water Users (25 cfs).

For the purposes of this feasibility study, it was assumed that demands would increase to a peak of 370 cfs, would also be agricultural in nature, and would follow the general timing and demand pattern of the existing demand curve shown previously in Section 3 (refer to Figure 3-8).

5.3 Typical Project Operations and Project Yield

Typical project operations and annual project yield for NFCCR depend on available capacity in YTID's Tieton Canal. During the spring and early summer months, when YTID is not using the full capacity of its canal for YTID agricultural demands, available water in the Tieton River could be diverted into the Tieton Canal and conveyed to the FCR. From there, the excess water could be pumped to the NFCCR.

The design capacity of the Tieton Canal is 345 cfs. However, today, the canal is over 100 years old, and due to the increased efficiency of the pipeline distribution system completed in 1987 the District rarely exceeds 320 cfs. For the purposes of this study 300 cfs was chosen as the flow rate for the existing canal. Using available annual historical flow and diversion data dating from 2000 to 2015, CH2M created a spreadsheet water model to simulate project operations for two potential scenarios:

Existing YTID Tieton Canal capacity with a flow rate of (300 cfs)

For the purposes of this study the existing Tieton Canal capacity is limited to approximately 300 cfs. Also, prior to March 1, ice and snow accumulation in the canal restricts the possibility of diversion. Therefore, beginning on March 1 and using a 7-day ramp-up period, up to 300 cfs is diverted from the Tieton River *only* when historical flow records indicate that flows in the Tieton River are sufficient to maintain the instream target flow of 100 cfs at the TICW gage (downstream of YTID's diversion point).[1] For years prior to 2011, the previous instream target flow of 40 cfs was used. This condition is held through the historical storage control date on June 24. After the storage control date, the model assumes that YTID canal diversions are equal to historical demands through the remainder of the irrigation season (October 31), and no flow is available to NFCCR.

YTID Tieton Canal is reconstructed and capacity expanded (370 cfs)

This scenario assumes that the existing Tieton Canal is reconstructed as a buried pipeline and capacity is expanded to 370 cfs. Because the newly-constructed canal is now enclosed, year-round diversion is possible. Beginning on November 1 and using a 7-day ramp-up period, flow up to the maximum capacity of the pipeline (370 cfs) is diverted from the Tieton River *only* when historical flow records indicate that flows in the Tieton River are sufficient to maintain the instream target flow of 100 cfs at the TICW gage (downstream of YTID's diversion point). For years prior to 2011, the previous instream target flow of 40 cfs was used. This condition is held through the historical storage control date on June 24. After the storage control date, the model assumes that YTID canal diversions are equal to historical demands through the remainder of the irrigation season (October 31) and no flow is available to NFCCR.

Based on the two operating scenarios described previously, estimates of potential water deliveries to FCR were compared to actual historical water deliveries to FCR for the same time period in order to determine potential project yield. Table 5-1 shows the volume of water for each operating scenario that could theoretically be made available for storage in the NFCCR by year. Minimum, maximum, and average water volumes are also given for the same time period.

Table 5-1. Volumes of Water Theoretically Available for NFCCR Storage

Year	Historical YTID Delivery to FCR (AF)	Potential Water Delivery to FCR (300 cfs canal capacity) (AF)	Water Available for NFCCR (300 cfs canal capacity) (AF)	Potential Water Delivery to FCR (370 cfs canal capacity) (AF)	Water Available for NFCCR (370 cfs canal capacity (AF)
2015	85,692	113,825	**28,133**	158,326	**72,634**
2014	82,863	113,506	**30,643**	137,520	**54,656**
2013	81,078	103,418	**22,339**	122,288	**41,209**
2012	76,924	116,326	**39,401**	151,147	**74,222**
2011	75,863	118,429	**42,566**	166,218	**90,356**
2010	71,821	99,021	**27,199**	112,348	**40,526**
2009	77,466	103,158	**25,692**	125,960	**48,494**
2008	74,749	106,896	**32,147**	128,945	**54,196**
2007	79,511	115,847	**36,335**	147,140	**67,628**
2006	73,023	99,159	**26,135**	116,789	**43,765**
2005	75,302	93,179	**17,876**	103,640	**28,338**

[1] For example, data from April 30, 2015, indicates that flow out of Rimrock Reservoir was equal to 381 cfs, and with reach gains, flow immediately upstream from the TICW gage was 421 cfs. YTID diverted an average of 191 cfs, leaving 230 cfs in the Tieton River. Under this operating scenario, YTID's diversion would increase from 191 to 300 cfs (an increase of 261 acre-feet for the day).

Data from May 1, 2015 indicates that flow out of Rimrock Reservoir was equal to 313 cfs, and with reach gains, flow immediately upstream from the TICW gage was 336 cfs. YTID diverted an average of 190 cfs, leaving 146 cfs in the Tieton River. Under this operating scenario, YTID's diversion would increase from 190 to 236 cfs (an increase of 91 acre-feet for the day), while continuing to maintain the 100 cfs in-stream flow requirement.

Table 5-1. Volumes of Water Theoretically Available for NFCCR Storage

Year	Historical YTID Delivery to FCR (AF)	Potential Water Delivery to FCR (300 cfs canal capacity) (AF)	Water Available for NFCCR (300 cfs canal capacity) (AF)	Potential Water Delivery to FCR (370 cfs canal capacity) (AF)	Water Available for NFCCR (370 cfs canal capacity (AF)
2004	72,178	93,311	**21,132**	100,330	**28,151**
2003	76,524	105,861	**29,337**	124,300	**47,776**
2002	72,930	98,697	**25,767**	105,396	**32,465**
2001	75,544	95,386	**19,842**	95,105	**19,561**
2000	84,182	117,577	**33,395**	153,828	**69,646**
Minimum	71,821	93,179	**17,876**	95,105	**19,561**
Average	77,228	105,850	**28,621**	128,080	**50,852**
Maximum	85,692	118,429	**42,566**	166,218	**90,356**

Notes:
AF = annual flows
cfs = cubic feet per second

As shown in Figure 2 in Appendix A, the capacity of the proposed NFCCR at pool elevation of 2,400 feet is approximately 35,101 acre-feet. With an assumed dead pool storage of 500 acre-feet, the figures in Table 5-1 indicate that NFCCR, with an active storage of 34,600 acre-feet, is within the realm of feasibility, even without expanding the capacity of the existing Tieton Canal.

6 Environmental Impacts

6.1 Preliminary Environmental Impacts

A preliminary assessment of environmental impacts was made using available GIS data. Acreages of impacted habitat and species utilization are presented in Table 6-1. Approximately 9,696 feet (1.8 miles) of North Fork Cowiche Creek would be inundated by the proposed reservoir. At full pool, the proposed reservoir would submerge approximately 427.8 acres. This area is composed entirely of mule deer and elk habitat and shrub-steppe. Approximately 63.4 acres of Oregon white oak woodland would be submerged, as would 29.5 acres of potentially occupied Oregon golden aster habitat.

The proposed dam would impact additional habitat. The CFRD option would impact 40.6 acres of mule deer and elk habitat and 30.0 acres of shrub-steppe habitat. The RCC option would impact 20.3 acres of mule deer and elk habitat and 7.7 acres of shrub-steppe.

Table 6-1. Summary of Impacts to Priority Habitats and Priority/State-listed Species

Environmental Resource	Full Pool Reservoir Footprint (acres)	CFRD Option Footprint (acres)	RCC Option Footprint (acres)
Total Footprint	427.8	40.6	20.3
Priority Habitats			
Oregon white oak woodland	63.4	0	0
Shrub-steppe	427	30.0	7.7
Priority/State-listed Species			
Mule deer	427.8	40.6	20.3
Elk	427.8	40.6	20.3
Oregon golden aster	29.5	0	0

6.2 Environmental Compliance and Permits

The construction of the proposed project may include funding from state and federals sources. As such, the proposed project would need to comply with the stipulations set forth by various federal and state acts before construction could proceed. A list of these federal and state acts is provided below:

- Clean Water Act (CWA) (33 U.S.C. §§ 1251-1387)
- National Environmental Policy Act (NEPA) (42 U.S.C. § 4321)
- State Environmental Policy Act (SEPA) (Chapter 43.21C RCW)
- Endangered Species Act (16 U.S.C. §§ 1531-1544)
- Bald and Golden Eagle Protection Act (BGEPA) (16 U.S.C. § 668)
- Migratory Bird Treaty Act (MBTA) (16 U.S.C. §§ 703–712)

Initial steps towards compliance with the CWA would involve a wetland delineation of the project area. This would determine the presence of any potentially jurisdictional wetlands and Waters of the United States. After the receipt of a preliminary jurisdictional determination from USACE, a Section 401/404 permit application would be filed to allow the discharge of dredged or fill material into jurisdictional wetlands and waters. Mitigation and avoidance measures would be developed as part of the permitting and design process.

An environmental assessment (EA) would be initiated to ensure compliance with the NEPA. The EA would determine the significance of any impacts from the proposed project and would satisfy requirements of the SEPA. SEPA follows similar protocol as NEPA and permits the issuance of joint documents.

Consultation with the National Marine and Fisheries Service (NMFS) and U.S. Fish and Wildlife Service would be recommended to assess any impacts to protected fish downstream of the YTID Tieton Canal diversion.

The EA development and NMFS consultation could begin immediately. A wetland delineation could be performed at any time during the growing season. As the first step in a months-long process of USACE permitting, it is recommended that the delineation occur once funding for the project is secured. Compliance with the BGEPA and the MBTA would require timing construction to avoid nesting periods.

6.2.1 Limitations of this Investigation

This investigation of environmental resources is based on a brief site visit and available online resources. Field biological surveys were not conducted. Formal protocol-level surveys for any threatened or endangered species with potential habitat in the project area would need to be performed at appropriate times for each species (for example, nesting period for birds). A wetland delineation performed during the growing season would be required to accurately assess any impacts to wetlands.

7 Cost Estimate

7.1 Capital Cost

A Class IV cost estimate as defined by AACE International was developed for the proposed project. The cost estimates presented in this section include capital costs for the dam and reservoir, as well as all of the components of the pump station and pipeline facilities described in this report and depicted on the associated drawings. Costs for power transmission from the source of power to the switchyard at the pump station site are not included at this time.

The purpose of this conceptual design phase construction cost estimate (Estimate) is to aid strategic planning, project screening, alternative scheme analysis, confirmation of economic and or technical feasibility, and preliminary budgeting for proposed projects. This Estimate is prepared based on limited field and design-specific information where the conceptual engineering is less than 15 percent complete. Examples of estimating methods used are equipment and or system process factors, scale-up factors, and parametric techniques. The expected accuracy ranges for this class of estimate are – 15 percent to – 30 percent on the low range side and + 20 percent to + 50 percent on the high range side.

The costs presented include general conditions and contractor overhead and profit. Also included are a 20 percent contingency for the dam and reservoir and 30 percent contingency for the pump station and pipeline facilities. The contingency allowance is intended to account for changes in the project scope and items that have not been defined at the conceptual level of project design. A smaller contingency was used for the dams than for the other facilities because (1) the construction cost for the dam is largely controlled by the volume of the CFRD embankment (or RCC gravity dam); (2) the dam volumes have been estimated using 3D Inroads modeling and are considered reasonably accurate; (3) adequate investigations has been performed into contractor pricing for the major items including rockfill and RCC unit prices; and (4) all major items have been included in the dam cost. No costs are included for land acquisition, administrative, legal, financing, engineering, construction management, environmental analyses and mitigation, or permitting. Cost are presented in second quarter 2016 dollars and no escalation has been provided.

The overall estimated capital cost for the project is summarized in Table 7-1. More detailed information regarding the cost for the various components is provided in Appendix B. Dam and reservoir costs reflect the cost for the CFRD. An RCC dam was also considered, but appears to be more costly.

Table 7-1. Summary of Estimated Capital Cost for North Fork Cowiche Creek Reservoir Project

Facility	Estimated Capital Cost ($1000s)
Concrete Faced Rockfill Dam	139,374
Transfer Pump Station	37,293
Flow Control Station	8,035
Conveyance Pipeline	2,722
Flow Measuring Structure	563
Project Total	**187,987**

7.1.1 Dam and Reservoir

Material quantity and cost estimates were performed for an RCC Gravity dam and a CFRD, which were the two alternatives considered in this study. The same dam alignment was assumed for both alternatives.

The Inroads civil engineering module in the Bentley Microstation CAD software was used to create a base map of the available topographic information. Using this base map and the available information of the geology and subsurface conditions at the nearby French Canyon dam site, assumptions regarding foundation excavation were made and modeled in 3D with the Inroads software. Following excavation, the dam structures were modeled, resulting in the respective 3D models of the two dam alternatives. These 3D models were then used to extract relatively accurate estimates of the following: reservoir volume, reservoir area, excavation volume, and dam volumes.

The maximum normal reservoir water surface elevation for both dam alternatives is 2,400 feet. Because of the longer spillway length of 250 feet for the RCC dam compared to a length of 150 feet for the CFRD, the RCC dam has a smaller freeboard of 9 feet compared to the freeboard of 12 feet for the CFRD. This results in nominal crest elevations of 2,409 feet and 2,412 feet for the RCC dam and CFRD, respectively.

The estimate did not include any cost associated with modifications that will likely be needed for the existing YTID inlet tunnel that traverses beneath the dam and the northeastern portion of the reservoir. Neither did the estimate include any ground improvement at the "saddle" at the northern rim of the reservoir to mitigate the potential for excessive seepage.

The approaches for estimating quantities and unit prices for the main dam elements used in the construction cost estimates are detailed as follows:

Quantities

- *RCC Volume*: For the RCC dams, the two main cost components are the RCC (including the material, placement, compaction, formwork, and drain installation) and the foundation excavation. An RCC volume of 1,048,170 cubic yards and excavation quantity of 322,000 cubic yards (cy) was computed.
- *Rockfill Volumes*: For the CFRD, the main cost components are the rockfill embankment and the reinforced concrete face slab. A total embankment volume of 3,897,000 cy was computed from the 3D Inroads model, with a breakdown of quantities for the different zones as follows:
  - Zone 1A – 36,353 cy
  - Zone 1B – 72,710 cy
  - Zone 2D – 120,673 cy
  - Zone 2E - 121,718 cy
  - Zone 2F - 9,023 cy
  - Zone 3A – 2,107,512 cy
  - Zone 3B - 1,426,519 cy

  For the purpose of performing order-of-magnitude cost estimates, the subzones were combined to yield 110,000 cy, 252,000 cy, and 3,535,000 cy for Zones 1, 2, and 3, respectively.

- *Foundation Grouting Quantities*: This is the same foundation grouting concept: that is, the same quantities and unit prices were assumed for both dam alternatives. Note, however, that if grouting is performed from within the RCC dam gallery, it will be more expensive than grouting performed in the open. The schedule advantages associated with taking grouting off the critical path is likely to offset the disadvantages, including extra cost, of grouting from the gallery. The following assumptions were made in estimating the quantities associated with the foundation grouting:
  - Foundation grouting will include a two-row grout curtain and two rows of blanket grout holes, with a blanket grout row on each side of the curtain.
  - Average spacing of the grout curtain holes will be 7.5 feet for total of 644 holes. Assume final pattern hole spacing of 10 feet with some areas of tertiary grouting with final hole spacing of 5 feet. Assume that average depth of holes in center 60 percent of curtain will be 200 feet, and average depth of the holes at the abutments will be 100 feet. Note that final hole depth and spacing will depend on the geologic conditions and estimates of these parameters are typically refined after a test grouting program is performed on site.
  - Spacing of blanket grout holes of 7.5 feet for total of 644 blanket grout holes. Assumed depth of the holes is 25 feet.
  - Grout take is 1 sack (94 pounds) per 1 lineal foot, on average. Microsilica content is 10 percent of cement.
  - Perform upstage grouting with average grout stage length of 15 feet. Assume two extra hook-ups per grout curtain hole. Assume one hook-up for each of the blanket grout holes.
  - Water pressure testing. Assume 500 hours of testing.
  - Verification boring every 100 feet along curtain for total of 24 borings. Assume average length of verification boring at 150 feet.

Unit Prices

Since the unit prices for RCC and rockfill are the two factors that largely control the construction costs, additional effort was spent to develop unit prices for these elements. Pricing assumptions do not include Washington State taxes.

- *RCC Construction Unit Price*: A review of the cost of existing RCC dams and consultation with well-known RCC contractors was undertaken to obtain an opinion on the unit price of RCC. For example, the RCC cost was about $107 per cubic yard for a recent CH2M project (Duck River Dam in Cullman Alabama), which had a much smaller RCC volume. Based on this information and given the relatively large scale of this project, a unit price of $90 per cubic yard was assumed for RCC construction.
- *Rockfill Embankment Construction Unit Price*: To obtain an estimate for rockfill unit prices, CH2M reviewed the cost of existing rockfill dams and also consulted with contractors. Specifically, two contractors were consulted to obtain an opinion on the unit price of rockfill. The first was a specialized dam contractor not familiar with the local conditions and the basalt rock characteristics, whereas the other contractor is more local and has experience with the rock conditions (including its quarrying qualities, and placement and compaction behavior). The local contractor's estimate was significantly less than the estimate from the other contractor. Considering this information, a CH2M cost estimator then developed a unit price for rockfill embankment construction (Zone 3) by considering specific labor and equipment unit costs to perform the various aspects of quarrying, transporting, placing, and compacting the rockfill, assuming typical production rates. A unit price of $15 per cubic yard for Zone 3 construction was estimated (including all contractor markups) by making the following assumptions:
  - The construction and maintenance of haul roads (dust control) is included in the price.

- Development of the quarry is included in price. Rock can be blasted for $3 per cubic yard (prior to contractor markups).
- Blasted rock will be loaded directly onto trucks for placement in the dam.
- The quarry will be less than 1 mile from the dam.
- Rockfill will be spread by a large bulldozer and compacted in relatively thick layers of 3 to 4 feet.
- Rock can be readily accessed in ample quantities without having to remove and spoil significant quantities of unsuitable materials.
- Weather delays will be minimal with no overtopping or washout during construction.

Since some sorting and processing will be required to produce Zone 2 rockfill materials, a unit price of $25 per cubic yard was assumed for that zone. A unit price of $20 per cubic yard was assumed for Zone 1 embankment placement.

7.2 Annual Operations and Maintenance Costs

O&M costs were developed to help estimate the order-of-magnitude for annual operating and overall life cycle cost of the facility.

Typical O&M costs, excluding the cost of power, were established as a percent of the capital cost for each facility as summarized in Table 7-2. These costs are expected to account for general upkeep, repair and replacement of minor items, and normal efforts required to inspect and monitor the facility as well as keep it in good condition and functional (including such tasks as periodic painting and cleaning, or lubrication). Factors were developed from past project experience and are expected to be conservatively high.

Table 7-2. Operations and Maintenance Cost Factors by Facility

Item	Annual O&M Cost Factor (Percent of Initial Capital Cost)
CFRD	0.1
Transfer Pump Station	0.5
Flow Control Station	0.5
Conveyance Pipeline	1.0
Flow Measurement Structure	1.0

Specific estimates are provided for annual power cost assuming $0.1/kW-hr, 80 percent pumping efficiency, 30,000 acre feet pumped, and 5 percent administrative power for non-pumping equipment. Also included is a specific operational estimate at 2 full time equivalent (FTE) employees for the full year. It is recognized that the facility will not be functional the entire year, but there will be 24-hour operation required for portions of the year by more than one staff member.

Table 7-3 provides a summary of the total estimate annual O&M cost.

Table 7-3. Estimated Annual Operations and Maintenance Cost Summary

Item	Annual O&M ($1000s)
General O&M	
CFRD	140
Transfer Pump Station	186
Flow Control Station	40
Conveyance Pipeline	27
Flow Measurement Structure	6
Operations (2 FTE)	300
Power	757
Total	**1,456**

7.3 Life Cycle Cost

An estimate of life cycle cost was developed to help determine the long term value of the project relative to the long-term expected yield. The following parameters were used to estimate the life cycle cost.

- Facility Life (assuming good O&M practices are maintained):
  - Dam: 100 years
  - Transfer Pump Station:
    - Structure and site (40 Percent of Facility): 100 years
    - Mechanical Electrical Components (60 Percent of Facility): 30 years
  - Flow Control Station:
    - Structure and site (40 Percent of Facility): 100 years
    - Mechanical Electrical Components (60 Percent of Facility): 40 years
  - Conveyance Pipeline: 100 years
  - Flow Measurement Structure:
    - Structure and site (70 Percent of Facility): 100 years
    - Mechanical Electrical Components (30 Percent of Facility): 20 years
- Analysis Period: 100 years
- Discount Rate: 3 percent
- Cost Basis:
  - Capital Costs: 2016 capital costs from Section 7
  - O&M Costs: Table 7-3
  - Replacement Costs: Frequency and percentage of initial capital cost as indicated above

A net present value analysis was conducted using the information describe above. The resulting 100-year net present value of the project is $251 million. Table 7-4 shows a summary of the unit net present value water cost per acre-foot for the 100-year analysis period at several annual yield values

Table 7-4. Unit Water Cost Summary

Annual Yield (Acre-feet/yr)	Net Present Value Water Cost[a] ($/acre-feet)
20,000	125
22,000	114
24,000	105
26,000	97
28,000	90
30,000	84

[a]Unit water cost does not include all costs and may not represent the full cost of financing the project.

7.4 Opportunities for Enhancing Project and Reducing Cost

The project facilities described in this report are presented in accordance with the various criteria and design parameters described in each section. The following list includes ideas and opportunities for improving the project, or reducing project cost during subsequent design phases, or both. These concepts would result in a slightly different project configuration, without sacrificing fundamental project performance and goals:

- It is recommended that one of the first activities associated with moving the project forward is to conduct a formal facilitated Value Engineering session. The Value Engineering session would bring applicable subject-matter experts together to review the conceptual project and suggest ideas that the project team can consider to improve the functionality or reduce the cost of the project.
- The maximum design flow rate of 370 cfs drives the sizing of several of the pump station and pipeline facilities. A reduced design flow rate may substantially reduce cost without changing the overall yield and benefit of the project. For example, using 300 cfs as the design flowrate would be expected to reduce the pump station by one pump and allow smaller pipelines and flow control facilities. Further, a reduction to 300 cfs may not materially affect the ability to transfer 30,000 acre-feet per year to the upper reservoir. It is recommended that an analysis of the lowest design flow rate that would statistically provide the best project yield be evaluated. Then, the conceptual design could be refined for a lower flow rate.
- This study assumes that all water delivered to the NFCCR is diverted from the Tieton River and conveyed through the YTID Tieton Canal. A separate study is recommended to develop concepts and costs for diverting water from the Naches River near the existing Wapatox Diversion Dam. Diverting and supplying water from the Naches River would greatly increase the available water supply and average project yield, because flow from both the Naches River and Tieton River would be available to the project. This concept would require a new pump station near the Naches River and a large-diameter pipeline from the Naches River to the FCR.
- As noted in this report, the use of a can-mounted pump station configuration versus the wet well with open-top can configuration for the pump station could result in lower pump station costs. It is recommended that a more detailed evaluation of the alternative configurations be conducted to better assess whether cost saving can be achieved.

- The power supply switchyard assumes the use of two 2 MVA transformers, where each transformer train is capable of supplying the full station power requirement. The use of dual transformer trains with a small capacity, perhaps 75 percent of full load should be considered to save cost without a substantial portion of the long-term project yield. Full load is only required for pumping the maximum design flow rate when the upper reservoir is approaching its full condition. A reduced load capacity in the rare event that a transformer train is off-line for a short period of time is expected to have minimal impact on the overall project yield.
- Fish screens are contemplated on the transfer pump station intake. These screens have not been included in the cost estimate, but were included in the layout of the facility. The need for fish screens should be verified. If they are not needed, the length of the wet well and pump station should be reconsidered to help reduce costs further.
- Alternative transfer pump station configurations, especially the can-mounted pump alternative and the possibility to use submersible pumps, should be evaluated in detail to determine if they can provide the same functionality at a lower cost than the pump and wet well configuration included in the conceptual design.

8 Conclusions and Recommendations

8.1 Dam and Spillway

The proposed 240-foot dam will impound a 35,100-acre-foot reservoir with a surface area of about 430 acres. The dam site is underlain by Grande Ronde basalt that is considered a sound rock formation, slightly weathered to unweathered hard rock, which should provide a suitable foundation for supporting a high concrete gravity dam or rockfill dam.

Two dam alternatives were considered and compared for this study: an RCC dam and a CFRD. An earth core dam was not considered feasible, because adequate volumes of good-quality clay materials for a dam core are not available in the vicinity of the site.

Based on the available information, both the RCC and CFRD alternatives appear to be feasible for this site. The CFRD is more adaptable to poorer rock conditions and construction under adverse weather conditions. However, one disadvantage of the CFRD is that there may be few contractors in the United States that have experience with constructing a CFRD of this size.

The RCC dam requires good rock conditions, and discovery of less than ideal foundation conditions will tend to promote selection of a CFRD. Numerous large RCC dams have been constructed over the last 10 to 15 years, and there are several qualified contractors with recent RCC dam construction experience in the United States.

Based on the assumptions adopted for this study, the estimated construction cost for a CFRD is about $12 million less than for an RCC dam. The Estimate included a contingency of 20 percent. The Estimate did not include any cost associated with modifications that will likely be needed for the existing YTID tunnel crossing traverse beneath the dam and reservoir.

Based on available information, technical considerations, and preliminary rough, order-of-magnitude construction cost estimates performed for this study, it is premature to dismiss the RCC dam concept at this stage. Both an RCC dam and a CFRD should be considered feasible and equivalent alternatives, pending the outcome and analysis of additional geotechnical investigations.

If this project is to move forward, a geotechnical exploration program is recommended so that further meaningful comparisons between the two alternatives can be performed and a final dam type and configuration can be selected. The exploration program is detailed in Section 4.1.6.4 and comprises borings at the dam foundation, spillway, borrow area, and saddle at the northern rim of the reservoir. It is also recommended that the program include a test quarry, RCC aggregate production, and RCC trial mixes for laboratory testing. An inspection of the YTID tunnel that crosses under the dam and reservoir should also be completed to assess the condition of the tunnel and to inform design of the modifications that will be required for the tunnel. The cost associated with the modifications can then be included in an updated project cost estimate.

8.2 Pump Station and Pipeline

The pump station and pipelines portion of the project includes all of the facilities required to transfer water from the FCR to the NFCCR and vice versa.

The following components are included in the pump station and pipeline facilities:

- FCR Intake Facilities
  - FCR intake (including optional fish screens)
  - Pump station
  - Conveyance pipeline
  - Flow control station
  - Flow measurement structure
  - Power transmission system

Three sites were considered for the pump station: below the FCR dam, at a platform facility near FCR, and at a location upstream of FCR. Ultimately, the upstream location was selected, because it provides the best combined use of the overall project site and requires the shortest, lowest cost pipeline.

8.2.1 French Creek Reservoir Intake Facility

The FCR Intake Facility will be located on the upstream (western) side of the FCR. The intake transition configuration for an RCC dam will be slightly different than for a CFRD and an RCC dam and will be further developed after the type of dam is chosen. It is recommended that the intake include either fish screens or a trash rack. Once further direction is provided by regulatory agencies, the design will be refined. The cost Estimate for the project assumes that trash racks are provided. Fish screens would add approximately $2.5 million to the project cost.

8.2.2 Pump Station

The pump station will lift water from the FCR to the NFCCR under the full range of reservoir levels and the full range of flows. Two pump station alternatives were analyzed: a four-pump configuration and a six-pump configuration. For the purpose of the feasibility study, six 2,500-horsepower pumps is the preferred configuration, because a six-pump configuration provides a reasonable structure size, is compatible with throttling requirements, and appears to have manageable flow gaps. However, the trade-offs between operational goals and cost should be reconsidered relative to the exact number of pumps during final design.

Neither pump station alternative can adequately cover the full operating range using constant speed pumps. Therefore, to provide flexibility in the operating envelope, flow throttling or variable speed operation is required. Throttling will provide better operability for the system, because it can be used to achieve most flow and head combinations within the operating envelope.

Three pump station wet well configurations were analyzed, including can-mounted pumps, a conventional wet well, and a wet well with an open top can. The conventional wet well is the most expensive option. Therefore, the can-mounted pump and wet well with open top can alternatives are preferred and should be considered during final design. Site-specific cost details are needed to clearly differentiate between the two alternatives. The wet well with open top can configuration was selected as the basis for the cost estimate for this analysis, since it is easier to define at this stage of project development. Also, the wet well with open top can arrangement has generally more favorable non-cost characteristics, so the can-mounted pump configuration would be selected only if it lowers the cost.

A pump building is proposed to provide environmental protection for the pumps, motors, discharge piping, and electrical equipment. The building is about 145 feet long, 60 feet wide, and 45 feet tall. The building walls could be CMU or precast concrete panels with a concrete or steel framing system. As part of the pump building, a 100-foot-long by 40-foot-wide electrical and control room building is proposed for the project. The electrical room is expected to house the main electrical switchgear, MCCs, reduced voltage pump starters, low voltage panel boards, and other related equipment. Depending on project funding and YTID's preference, other options could be considered during final design.

8.2.3 Conveyance Pipeline

The main conveyance pipeline extends from the transfer pump station to the inlet structure on the reservoir bottom upstream of the North Fork Cowiche Creek Dam. The pipeline starts at the downstream end of the pump discharge header buried beneath the floor of the pump building; passes through the flow control station and flow metering structure; and terminates at the intake structure above the dam. The pipeline is proposed to be 96 inches in diameter and about 1,450 feet long. The pipe material is expected to be a cement mortar lined and coated welded steel pipe, generally in accordance with AWWA C200. The pipeline will provide bi-directional flow capability to allow for both filling and draining of the NFCCR.

8.2.4 Flow Control Station

The flow control station will be used for both filling and draining the NFCCR. During filling, the flow control station will provide the throttling head to allow the main pumps to operate within their acceptable operating region for any flow and head situation. During draining, the flow control station will provide the needed head loss to regulate flows from NFCCR into FCR. The flow control station was conceptualized with a bypass pipe and several isolation valves to regulate system pressure in both directions using the same set of flow control valves. A variety of instruments will be provided to monitor valve position and pressure in the flow control station. A PLC will be located at the flow control station to provide control logic required to modulate the flow control valves.

8.2.5 Flow Measurement Structure

A flow measurement structure is proposed to facilitate both filling and draining of the NFCCR. The flow measurement structure will be placed along the main conveyance pipeline a suitable distance downstream of the flow control station to facilitate measuring flow in a straight pipe. A multi-path ultrasonic flowmeter was assumed for this application, since it is relatively accurate and allows flow measurement in either direction.

8.2.6 Power Transmission System

The power supply will include transmission lines to the onsite switchyard. The exact supply voltage and source of the power is currently being reviewed with the power supplier and is not yet known. Incoming power will be connected to two 2 MVA transformers (nominal conceptual size) with appurtenant switching, power measurement, and protective devices. Each transformer string will be located in a fenced switchyard adjacent to the electrical building.

8.2.7 Hydroelectric Power Considerations

Since the project includes pumping flows from the lower reservoir to the upper reservoir and later releasing it back to the lower reservoir, the potential exists to generate power when the flows are released. A present worth analysis was conducted where the initial capital and periodic replacement cost for the pumps was compared to the potential power revenue and the initial capital and periodic replacement cost for the pump-turbines. The specific cost of the pump-turbine units was not developed for this analysis, but it is reasonable to expect that it may be possible to obtain these units within the additional $2.5 million incremental cost.

Given the limited operating season for the proposed project, using it for hydro-electric power generation does not appear to provide the potential for significant revenue. In fact, once administrative issues and the nominally higher cost of O&M and some of the appurtenant equipment are considered, it is possible that power generation may only provide something close to a break-even situation, at best. If it is determined that the administrative and legal burden associated with hydroelectric power generation is worthwhile, then a more detailed assessment of the cost trade-offs related to power generation should be revisited when the project is further refined.

8.3 Project Costs and Funding

The Class IV construction cost estimate for this project is $188 million. However, the total cost could be expected to range from $175 million to $200 million, depending on the type of dam selected and options for the configuration of the pump station, intake, and associated building. This study also provided a $1.5 million annual O&M cost estimate, and a 100-year net present value of $251 million This translates to approximately $84 per acre-foot of yield assuming 30,000 acre-feet average annual yield. The cost per acre-foot does not include the cost of financing the project.

8.4 Cultural and Environmental Analyses

This feasibility study included a brief assessment of environmental and cultural resources that could potentially be affected by the project, and what actions would need to be taken during future phases of the project to comply with applicable state and federal requirements. The assessment was limited to what could be observed from a review of existing literature and from a brief site visit.

8.4.1 Environmental Resources

No wetlands identified by the NWI are located within the project area. The USFWS ECOS system indicates six federally listed threatened, endangered, and proposed threatened species (birds, fish, and mammals) that could occur or be potentially impacted by the project. However, there are no known occurrences of these species in the project area. Two WDFW Priority Habitats, shrub-steppe and Oregon white oak woodland, were identified in the project area. A documented occurrence of a WNHP rare plant, the Oregon golden aster (*Heterotheca oregona*; State Threatened) with a 1-mile buffer intersects the northern arm of the proposed reservoir. Potential impacts to these priority habitats and priority/state-listed species, in terms of acreages inundated by the reservoir, are tabulated in Section 6.

The construction of the proposed project may include funding from state and federals sources. As such, the proposed project would need to comply with the stipulations set forth by various federal and state acts before construction could proceed. Initial steps towards compliance with the CWA would involve a wetland delineation of the project area. After the receipt of a preliminary jurisdictional determination from the USACE, a Section 401/404 permit application would be filed to allow the discharge of dredged or fill material into jurisdictional wetlands and waters. Mitigation and avoidance measures would be developed as part of the permitting and design process.

Should the project continue to advance with anticipation of state/federal funding, an EA should be initiated to ensure compliance with the NEPA. The EA would determine the significance of any impacts from the proposed project, and would satisfy requirements of the SEPA. SEPA follows similar protocol as NEPA, and permits the issuance of joint documents.

Consultation with the NMFS would be recommended to assess any impacts to protected fish downstream of the YTID Tieton Canal Tunnel intake.

The EA development and NMFS consultation could begin immediately. A wetland delineation could be performed at any time during the growing season. As the first step in a months-long process of USACE permitting, it is recommended that the delineation occur once funding for the project is secured.

8.4.2 Cultural Resources Summary

The WISAARD database lists two previous cultural resources inventory efforts at the proposed project location. Both archaeological sites that have been identified within the APE are historic, which refers to historic homesteads, agricultural features, cabin/residential structures, and cemetery/burials. These sites were likely formed as a result of ranching and farming during the twentieth century. One of the sites currently contains a standing structure that will require evaluation by an architectural historian. No previously documented pre-contact archaeological sites are located within the APE. None of the archaeological sites have been evaluated for their NRHP eligibility.

Given the lack of previous inventory coverage, if project implementation moves forward, cultural resources inventory surveys (both surface and subsurface) are recommended to determine whether NRHP eligible properties are located within the footprint/APE. According to the DAHP predictive model, such surveys are highly likely to result in the identification of additional resources. To determine whether project implementation will result in adverse effects to historic properties as defined by Section 106 of the NHPA, a DOE for the NRHP would need to occur for each of the documented sites, including those that may be identified during survey. Doing so would be a necessary step towards meeting the requirements for federal undertakings outlined in Section 106 of the NHPA. To assess whether previously undocumented Traditional Cultural Properties are located within the site, consultation with the affected Tribes and SHPO should occur.

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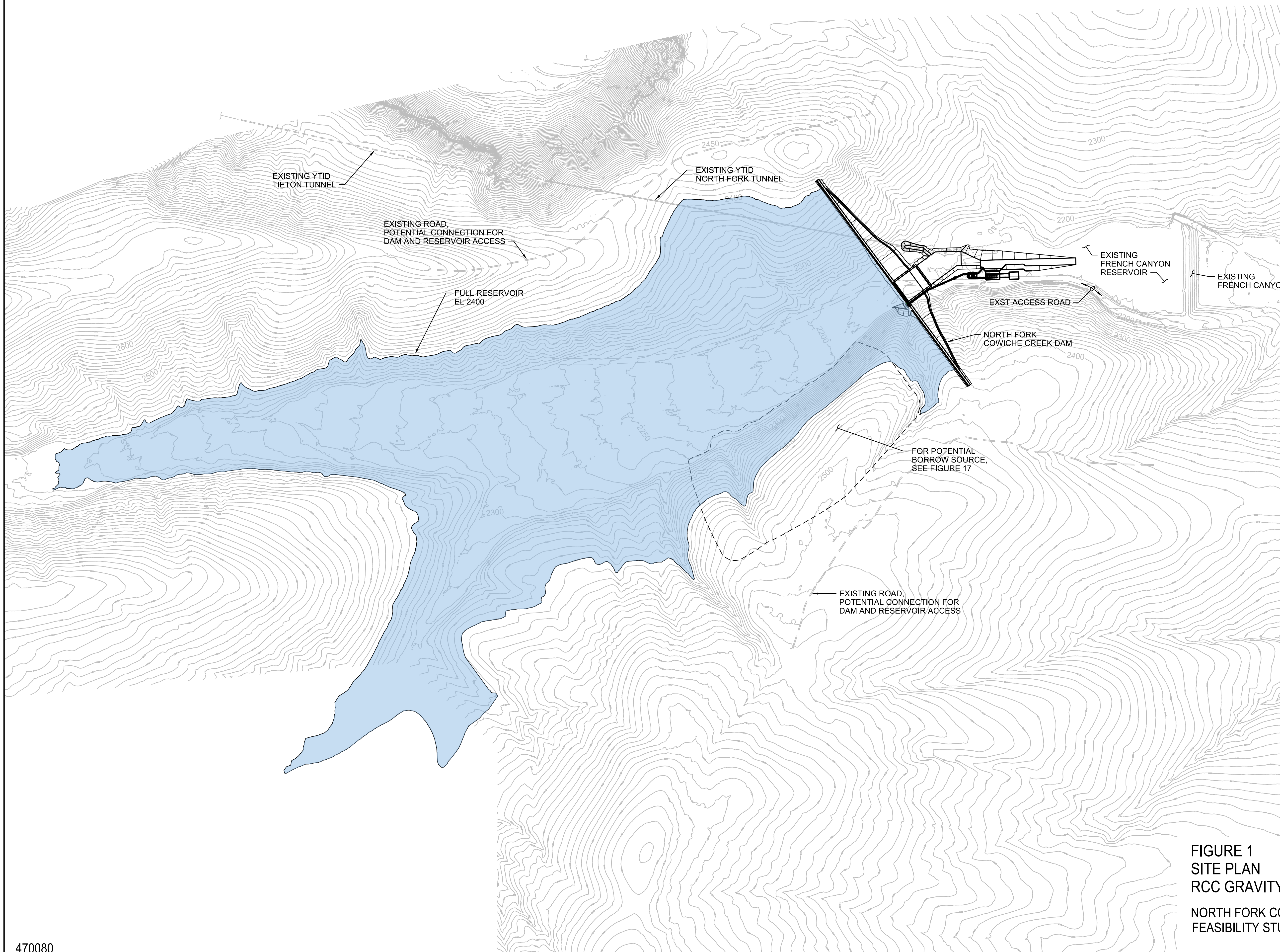
Appendix A
Conceptual Design Drawings



N

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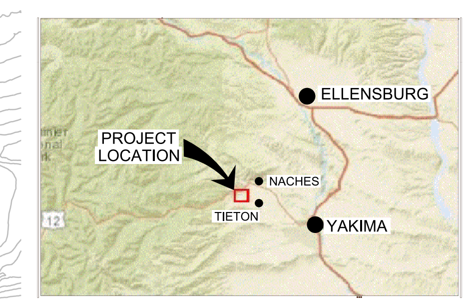
Scale In Feet



VICINITY MAP

FIGURE 1
SITE PLAN
RCC GRAVITY DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

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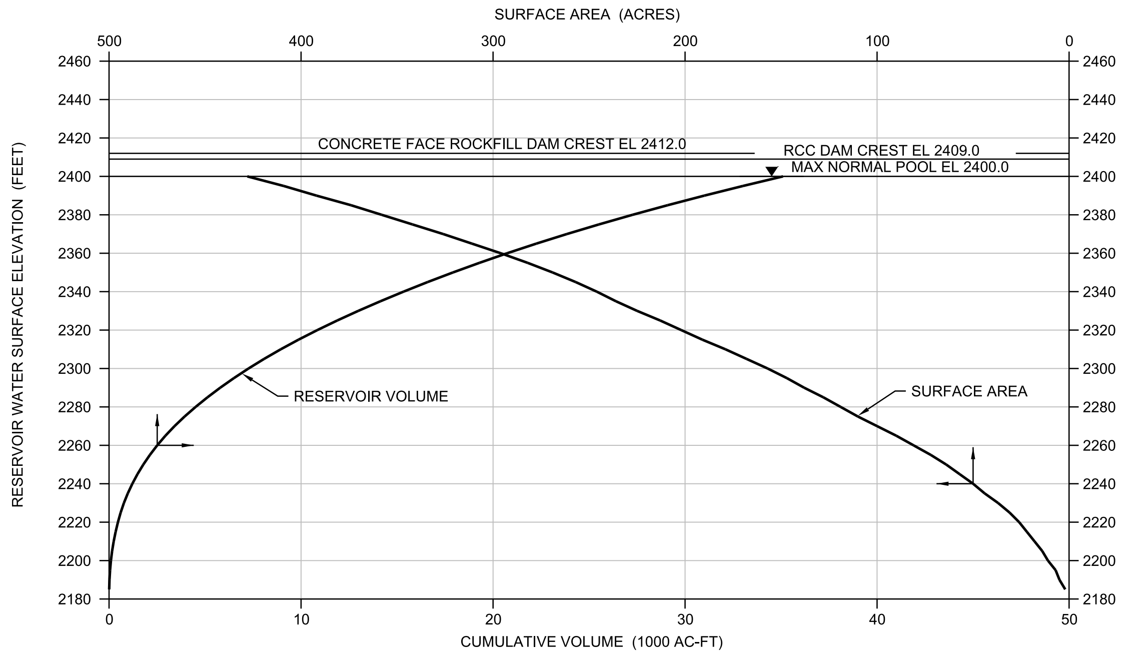


ELEVATION (FEET)	CUMULATIVE VOLUME (ACRE-FEET)	SURFACE AREA (ACRES)
2185	6	2
2200	97	11
2220	462	26
2240	1212	50
2260	2507	81
2280	4508	119
2300	7262	157
2320	10848	202
2340	15338	246
2360	20734	296
2380	27263	358
2400	35101	428

FIGURE 2
DAM AND RESERVOIR
CAPACITY DATA

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

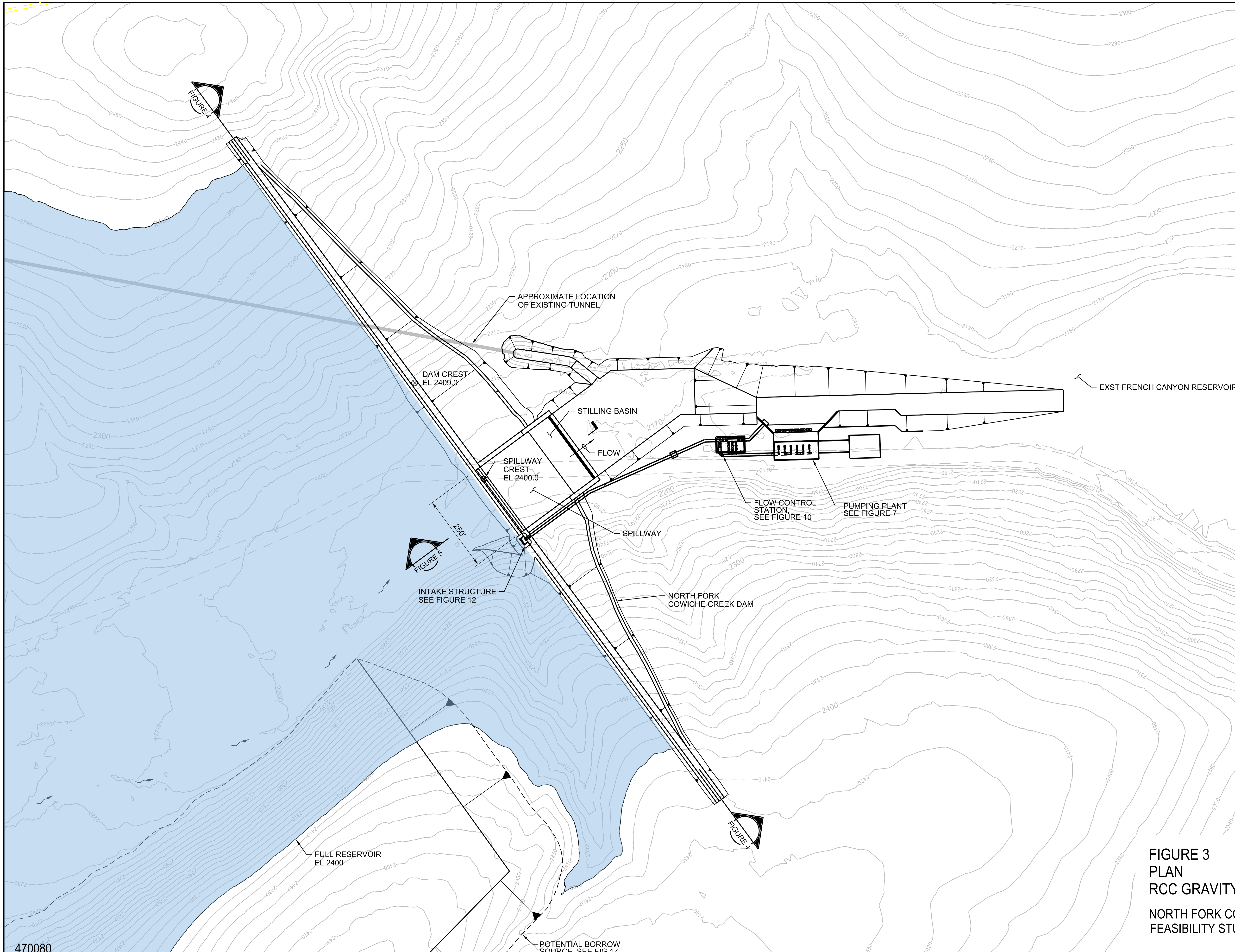


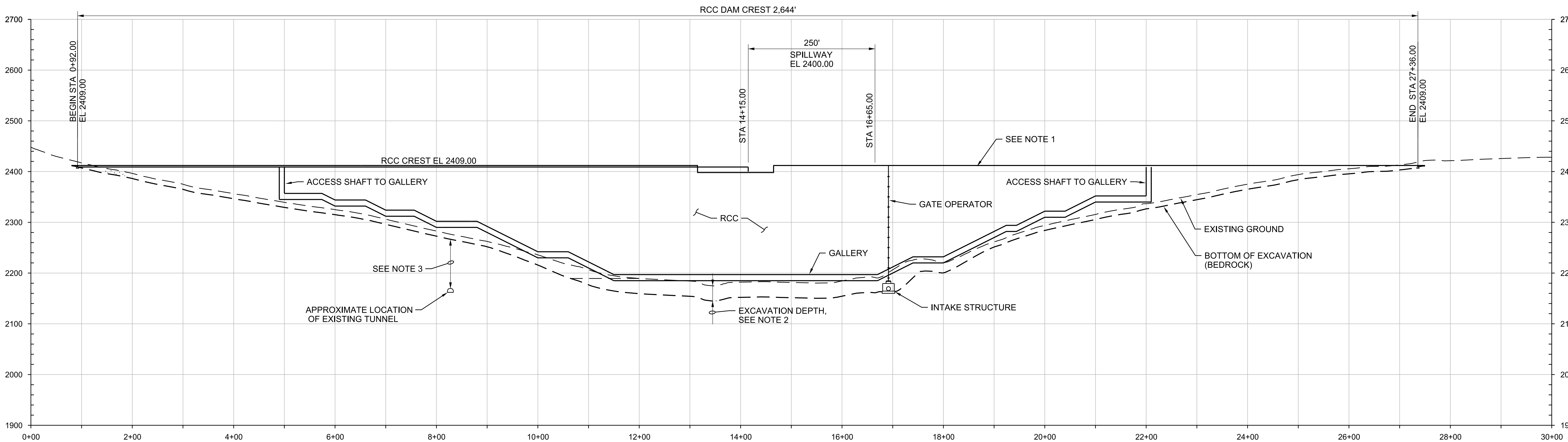
FIGURE 3
PLAN
RCC GRAVITY DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

ch2m

FILENAME: FS-Fig03_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:52:00 PM

NOTES:
1. CENTERLINE PROFILE FOR CONCRETE FACE ROCKFILL DAM (CFRD) OPTION IS SIMILAR TO THIS RCC DAM PROFILE. FOR CONCRETE FACE ROCKFILL DAM PLAN, SEE FIGURE 13.
2. DEPTH TO ROCK IS UNKNOWN. ESTIMATED EXCAVATION DEPTH SHOWN IS BASED ON GEOTECHNICAL INFORMATION FOR FRENCH CANYON DAM.
3. REQUIREMENTS FOR MINIMUM VERTICAL SEPARATION BETWEEN EXISTING TUNNEL AND DAM AND TUNNEL IMPROVEMENTS WILL BE EVALUATED IN NEXT DESIGN PHASE.



RCC GRAVITY DAM CENTERLINE PROFILE

1"=100'

FIGURE 4
EMBANKMENT PROFILE
RCC GRAVITY DAM

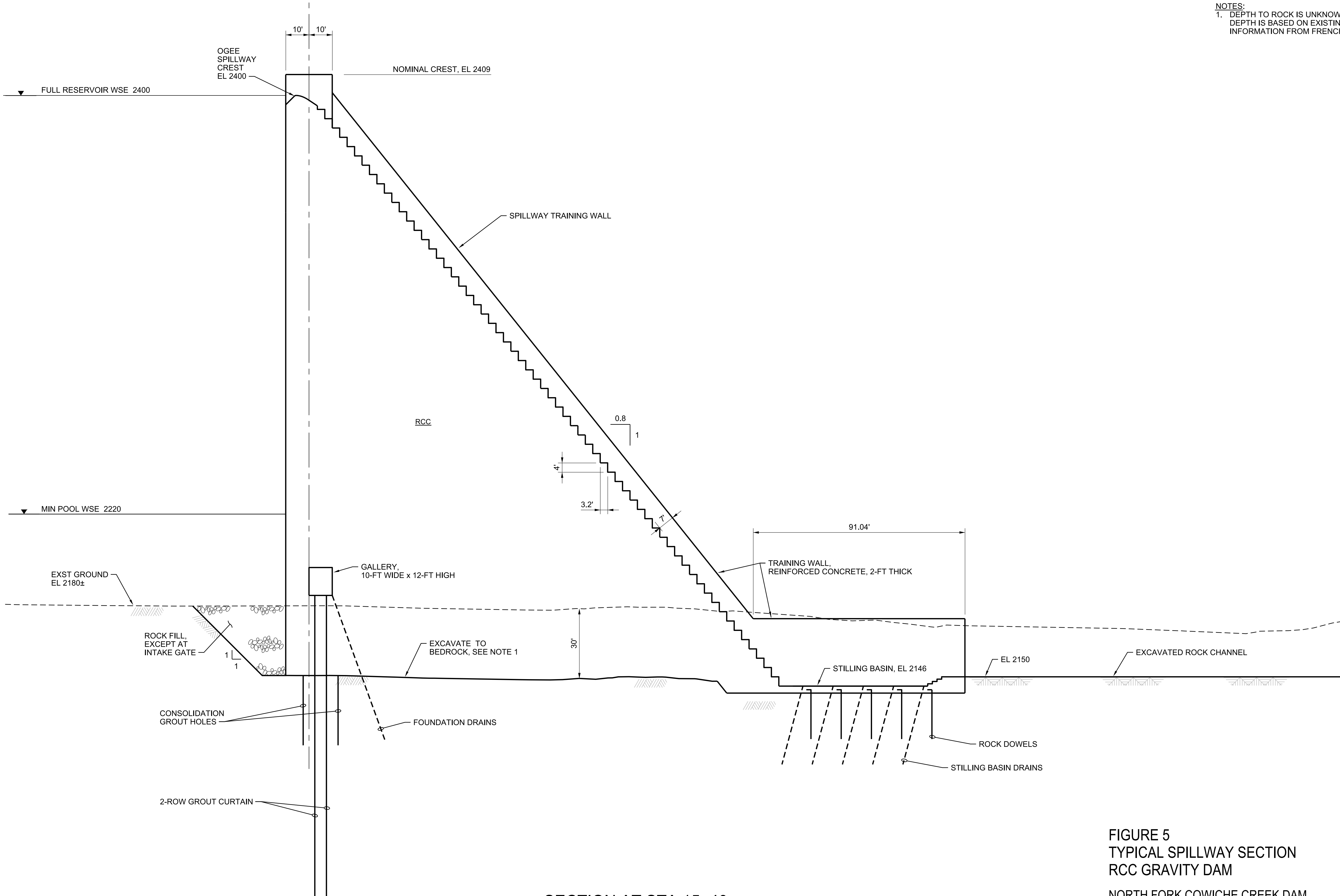
NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m:

FILENAME: FS-Fig04_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:52:02 PM

NOTES:
1. DEPTH TO ROCK IS UNKNOWN. ESTIMATED DEPTH IS BASED ON EXISTING GEOTECHNICAL INFORMATION FROM FRENCH CANYON DAM.



SECTION AT STA 15+40
1"=20'

FIGURE 5
TYPICAL SPILLWAY SECTION
RCC GRAVITY DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

FILENAME: FS-Fig05_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:51:21 PM

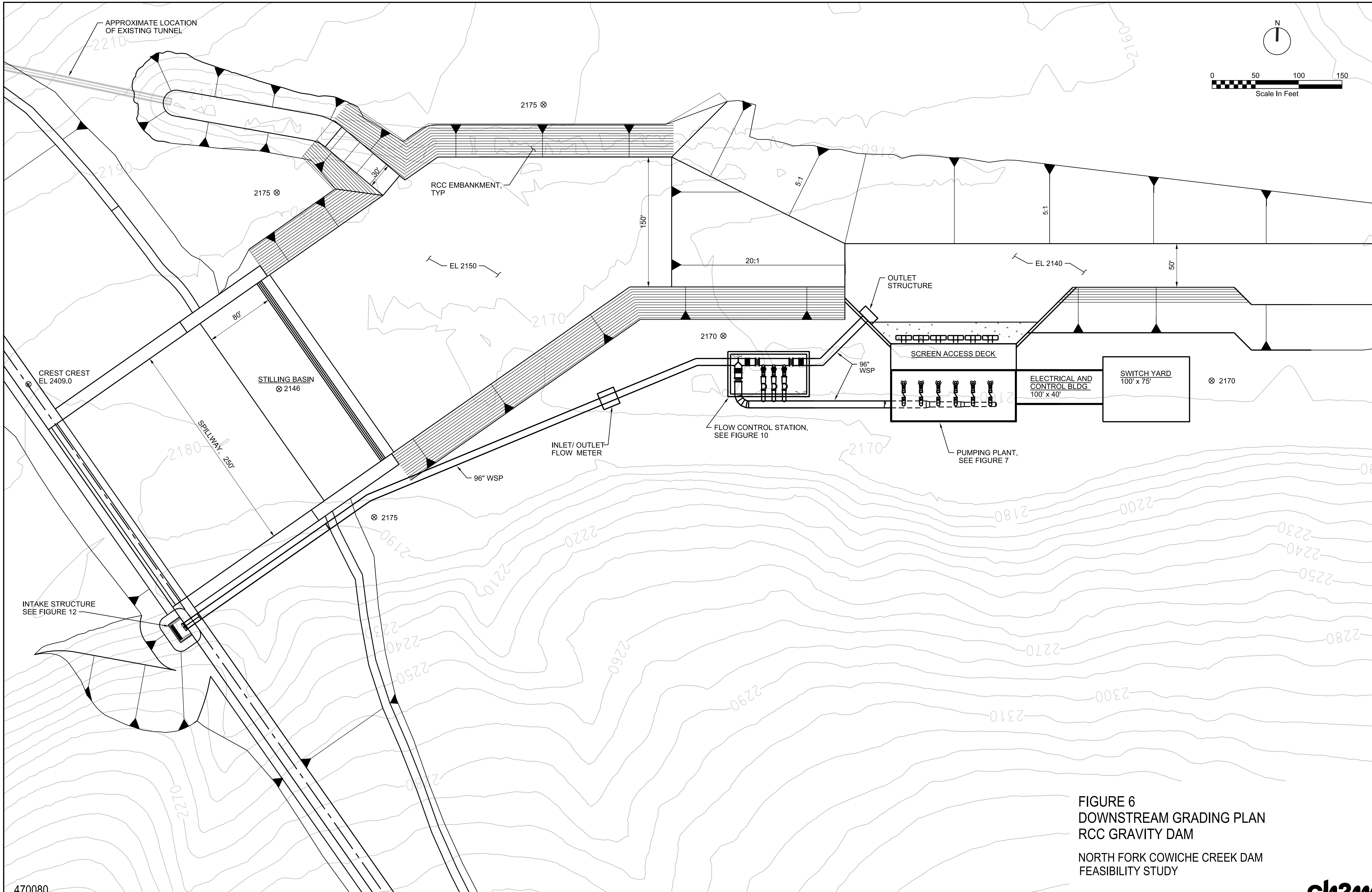


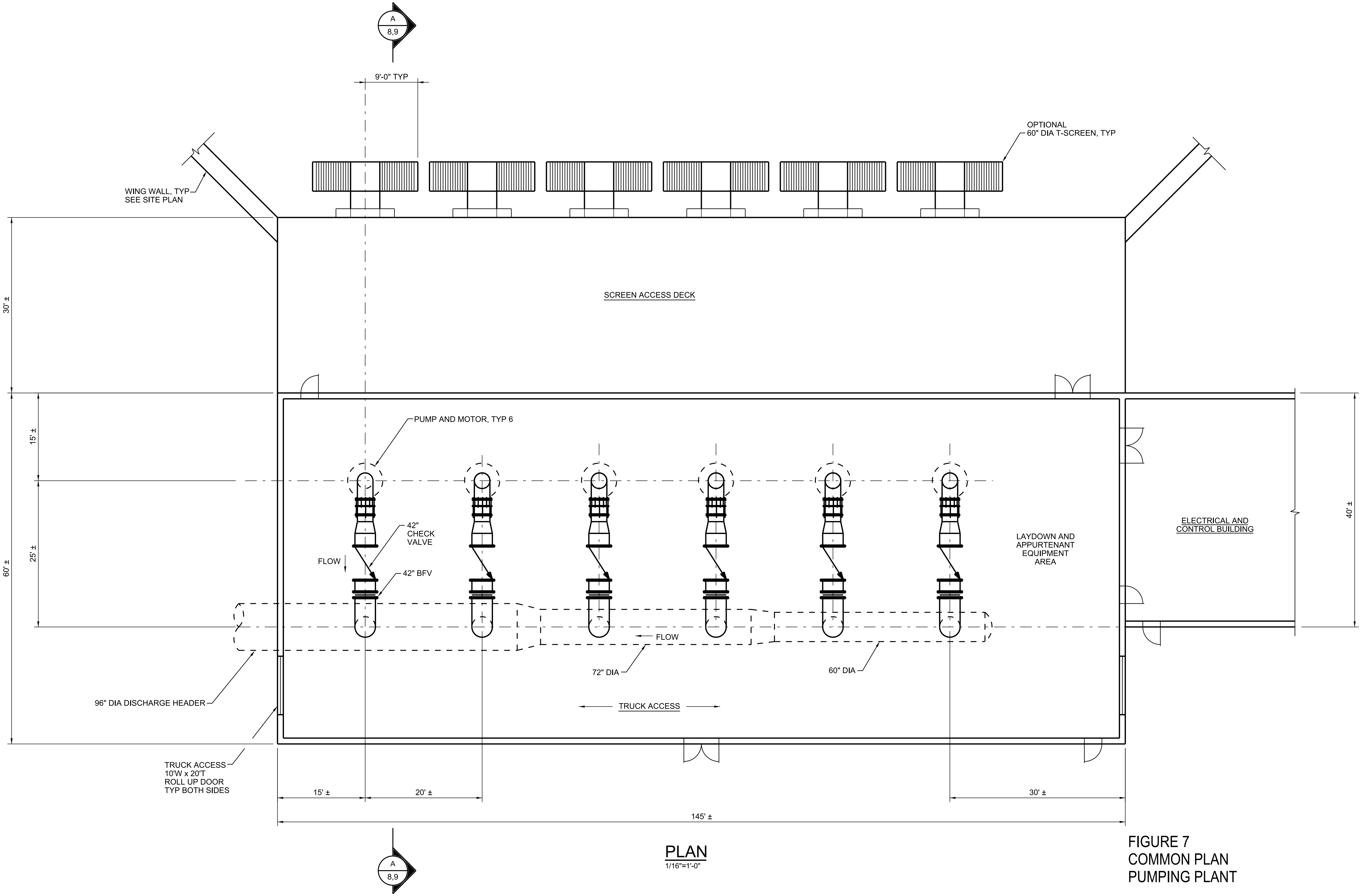
FIGURE 6
DOWNSTREAM GRADING PLAN
RCC GRAVITY DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

FILENAME: FS-Fig06_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:52:40 PM



PLAN

1/16"=1'-0"

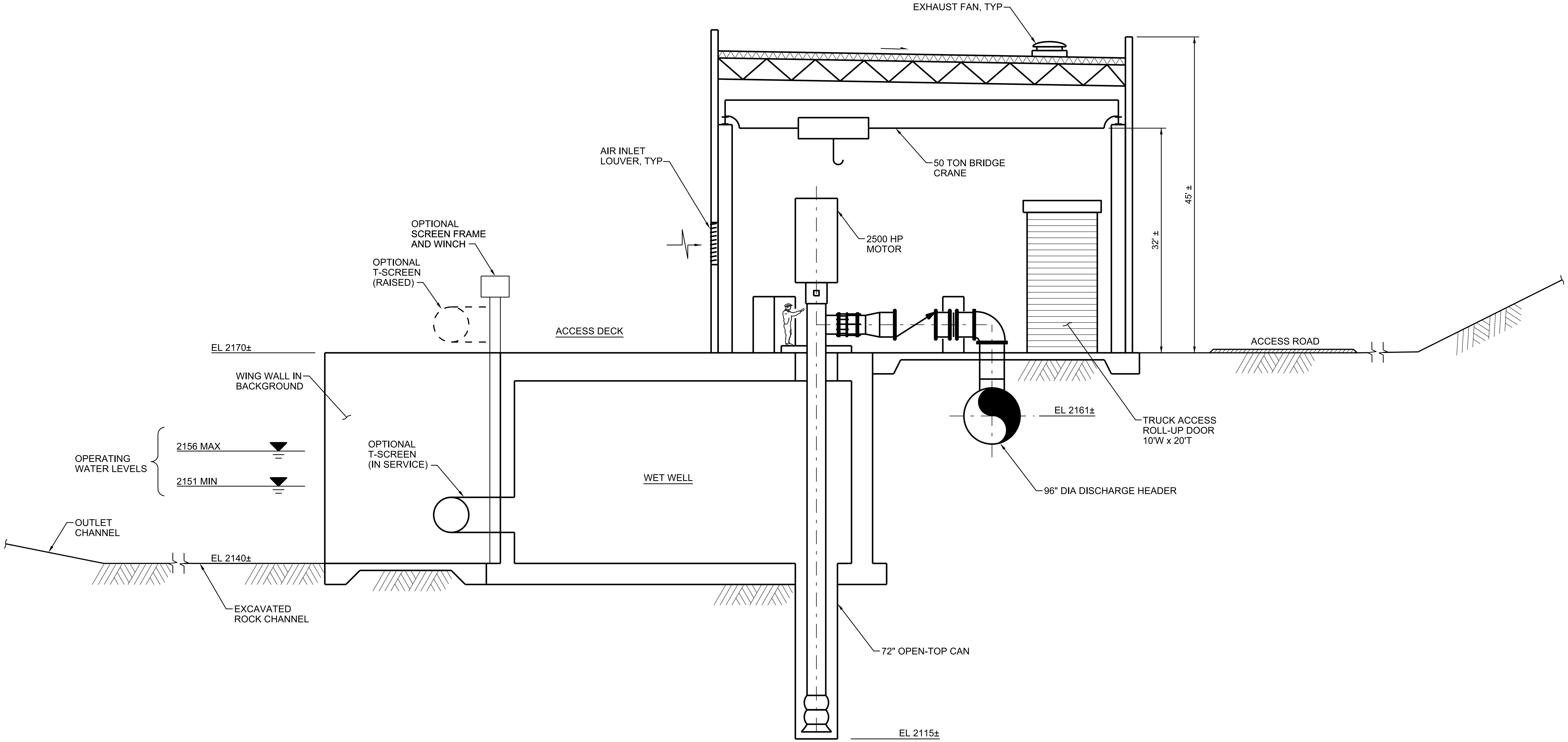
FIGURE 7
COMMON PLAN
PUMPING PLANT

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

FILENAME: FS-Fig07_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:51:27 PM



A SECTION
1/16"=1'-0"
FIG 7

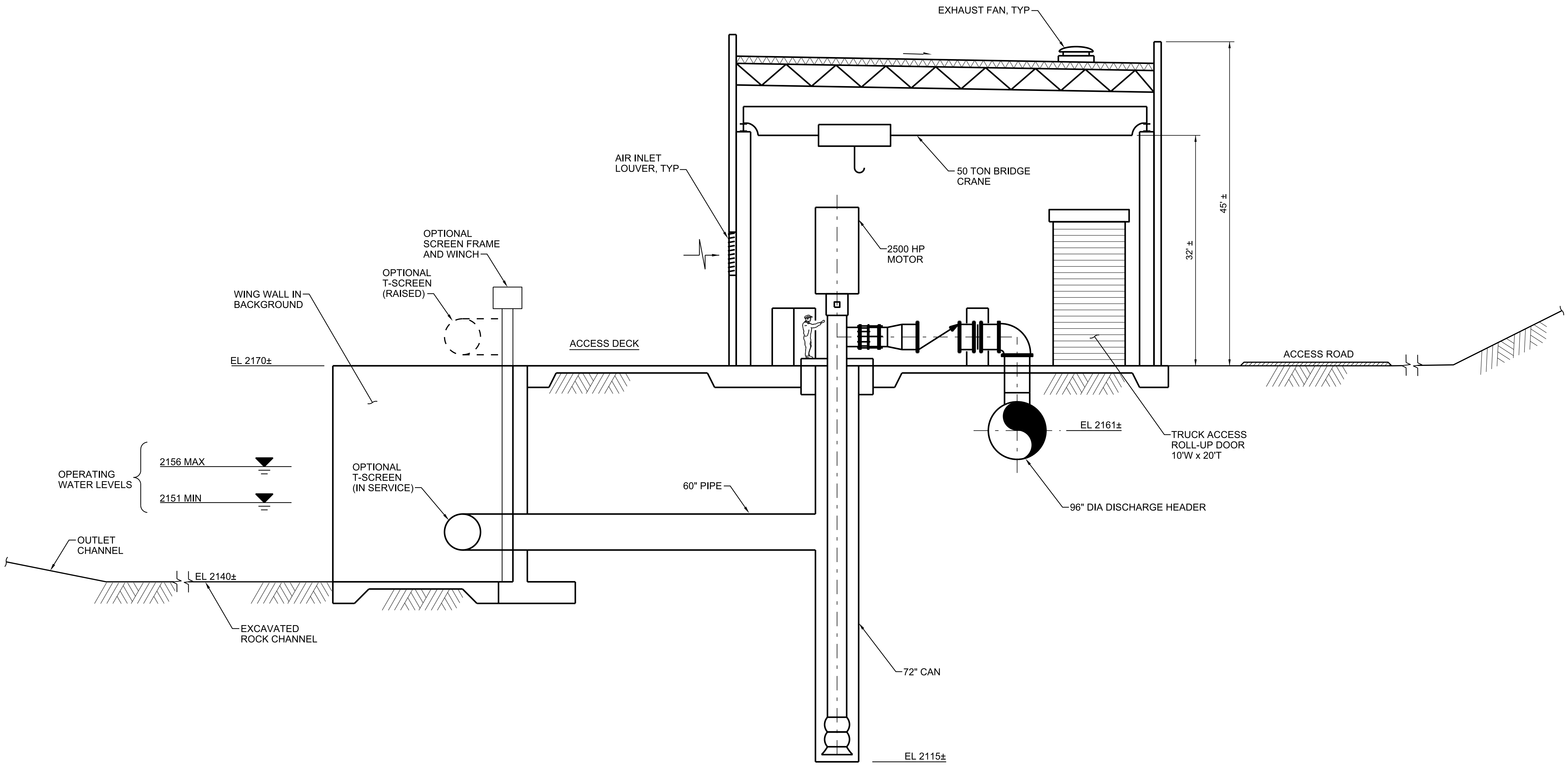
FIGURE 8
WET WELL WITH OPEN TOP CAN ALTERNATIVE SECTION

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

FILENAME: FS-Fig08_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:51:38 PM



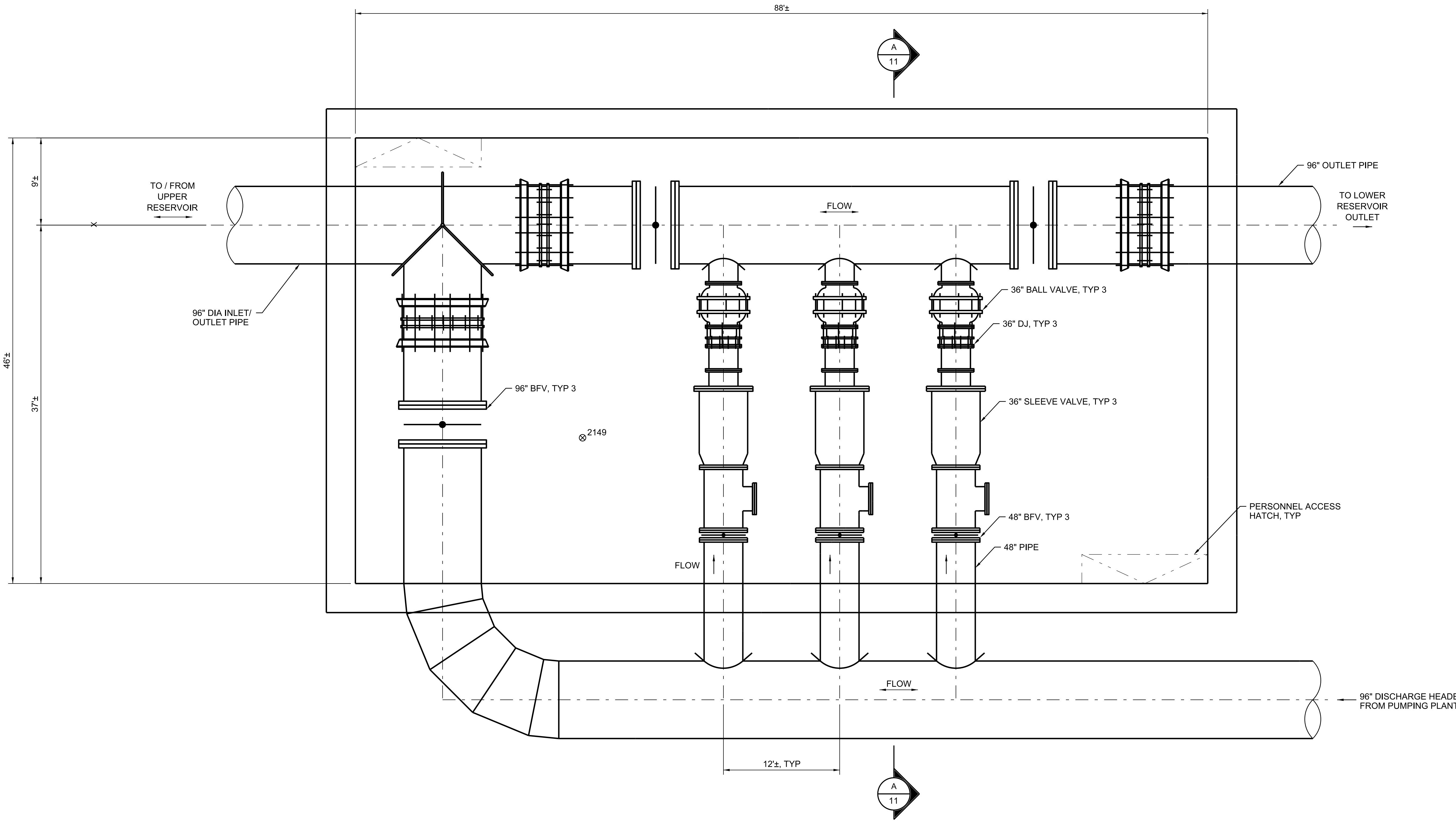
A SECTION
1/16"=1'-0"
FIG 7

FIGURE 9
CAN-MOUNTED PUMP ALTERNATIVE
SECTION

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m



FLOW CONTROL STATION

PLAN

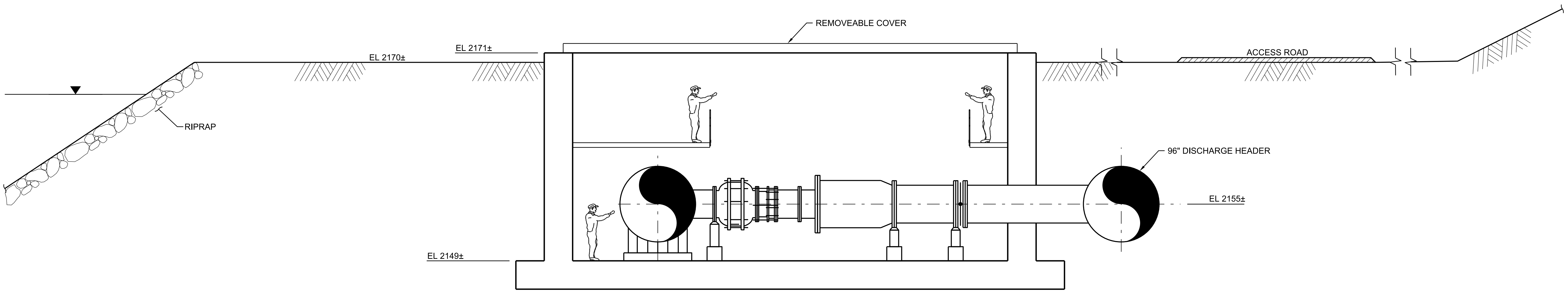
3/16"=1'-0"

FIGURE 10
FLOW CONTROL STATION PLAN
RCC GRAVITY DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m



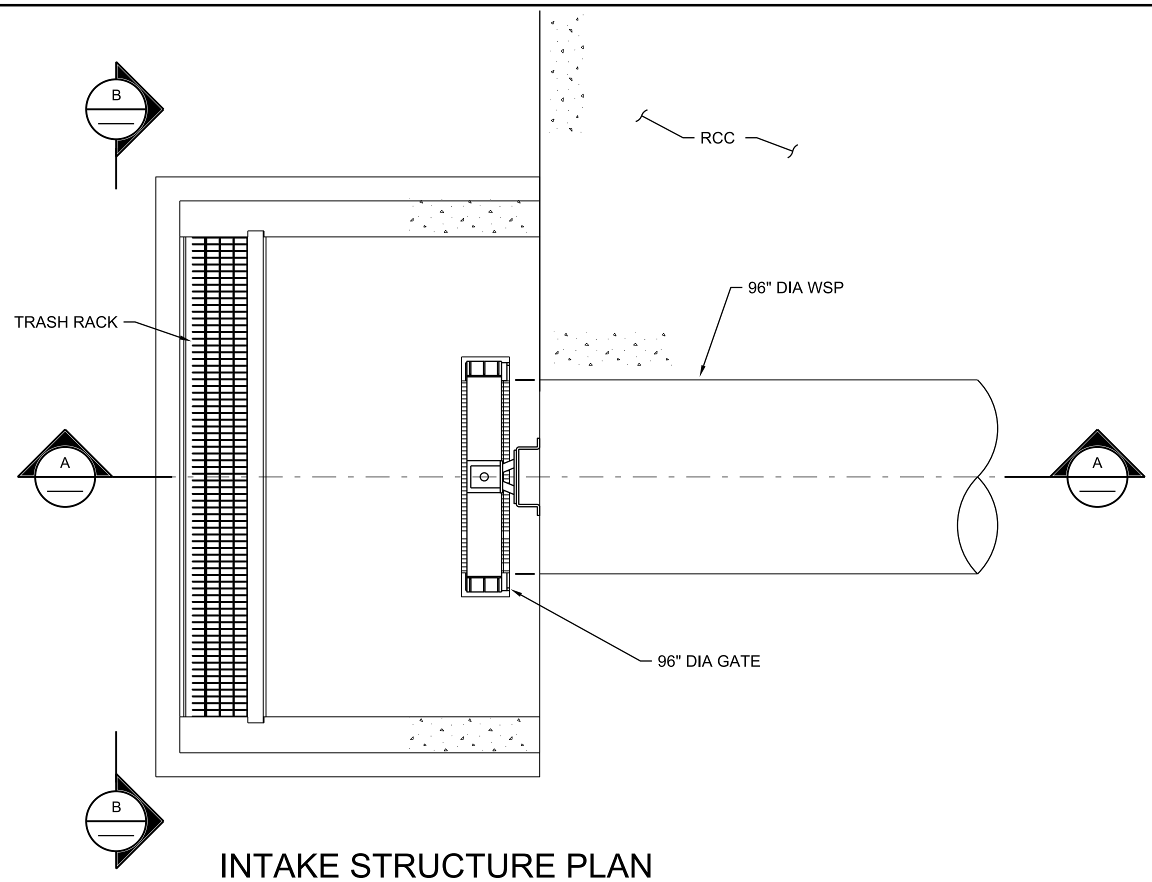
A
FIG 10
SECTION
3/16"=1'-0"

FIGURE 11
FLOW CONTROL STATION
SECTION

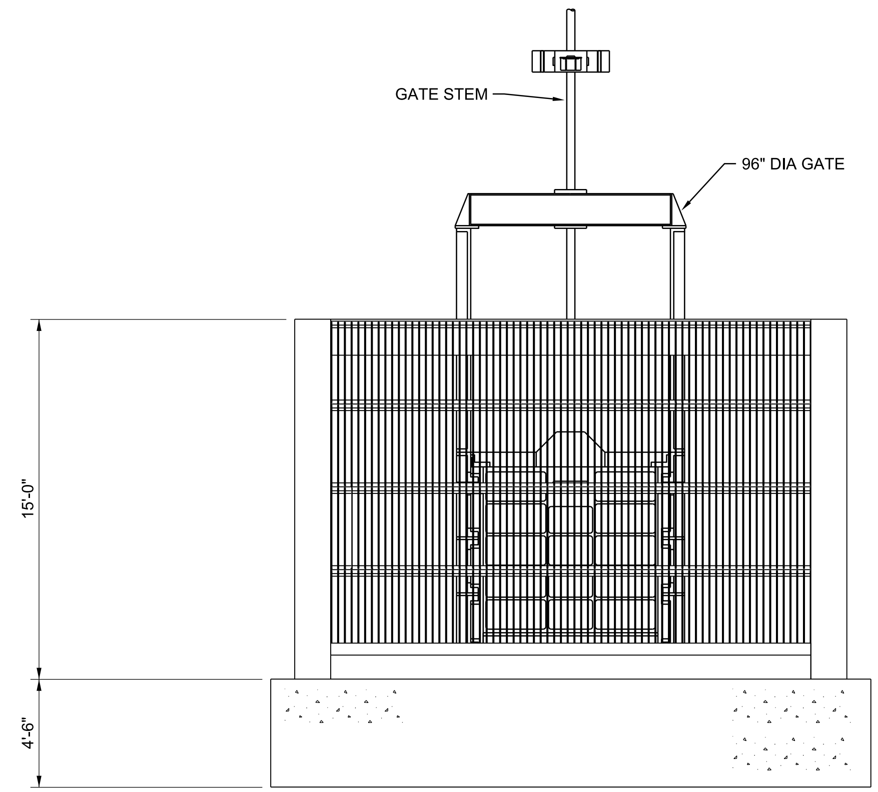
NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

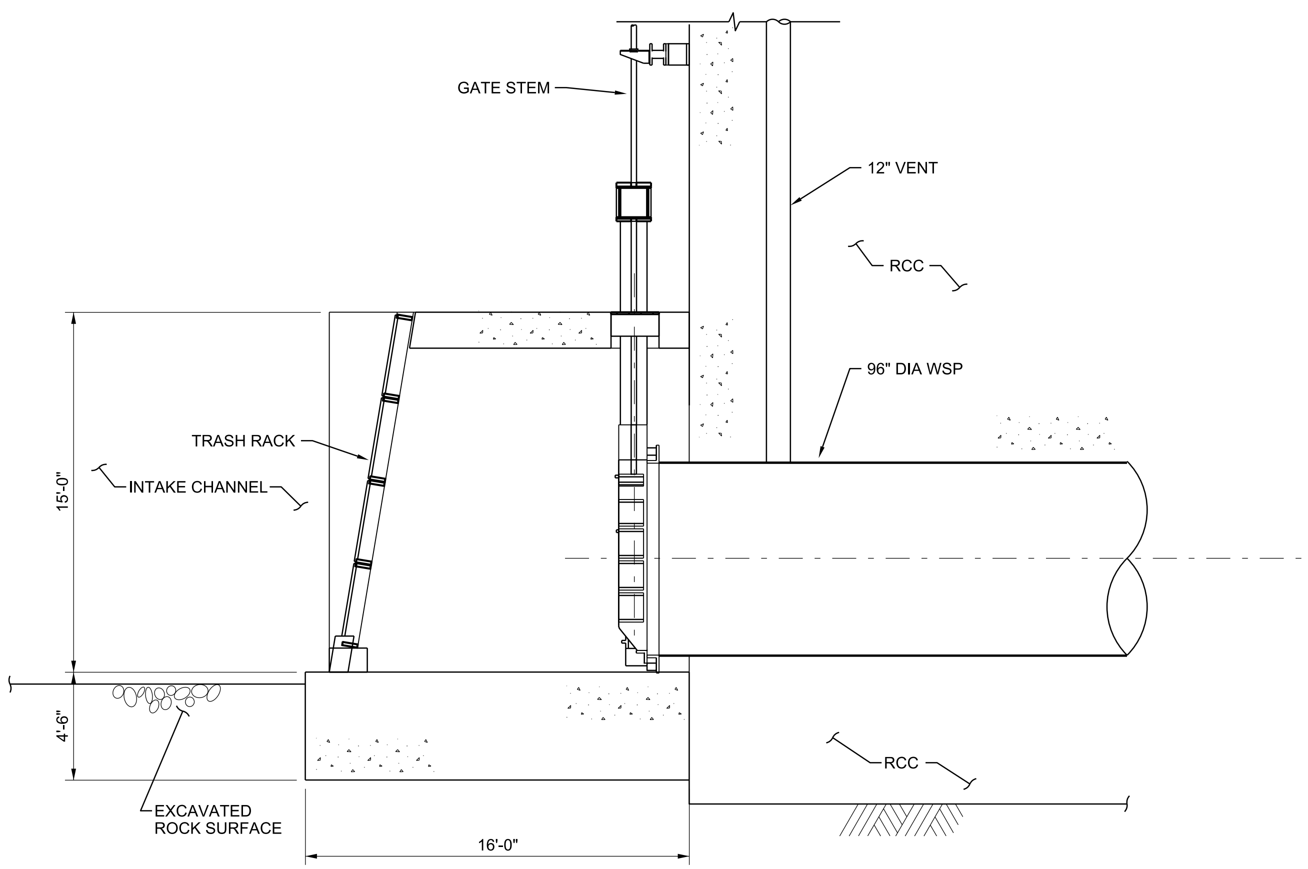
ch2m



INTAKE STRUCTURE PLAN
1/4"=1'-0"



B SECTION
1/4"=1'-0"



A SECTION
1/4"=1'-0"

FIGURE 12
INTAKE STRUCTURE
RCC GRAVITY DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

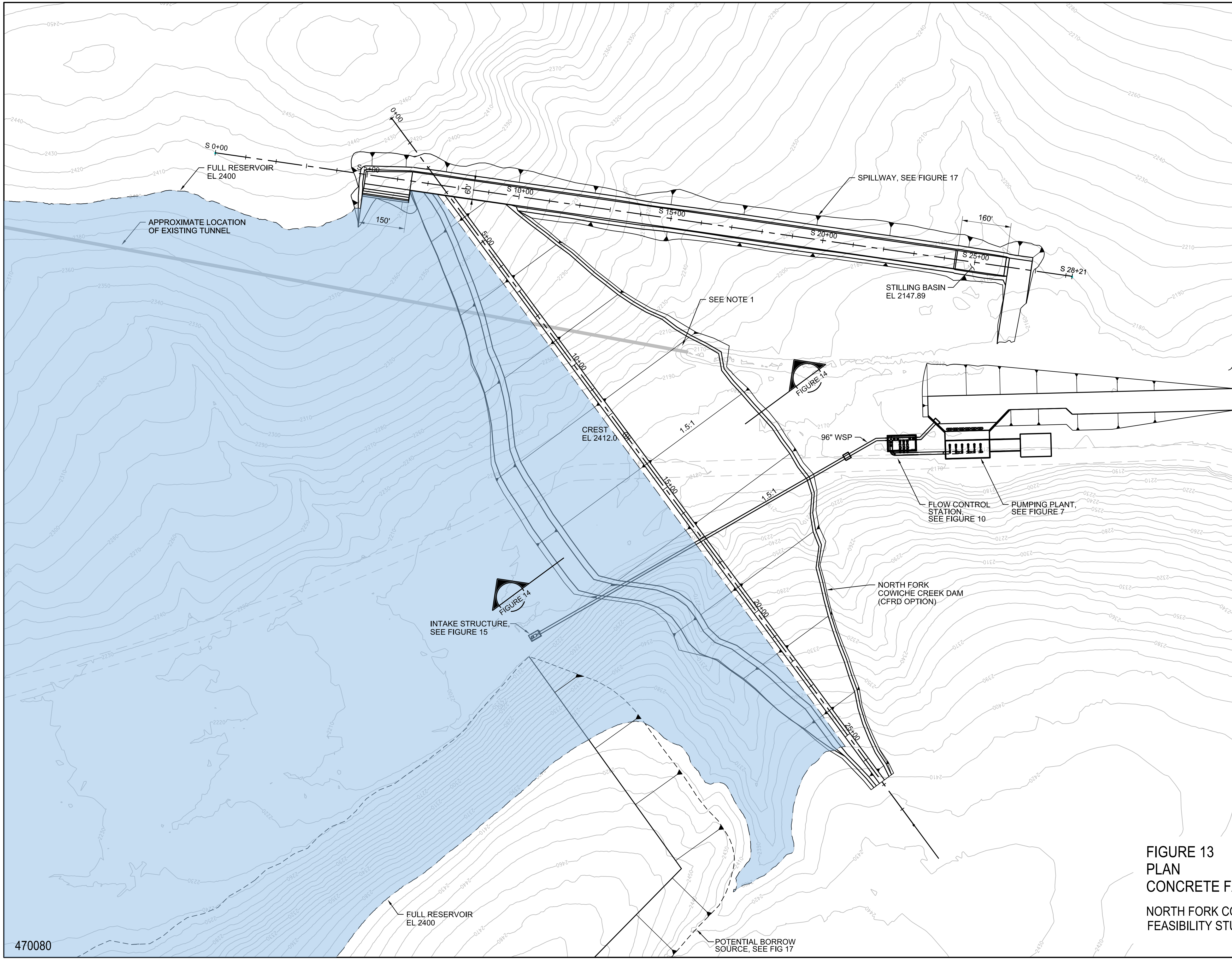


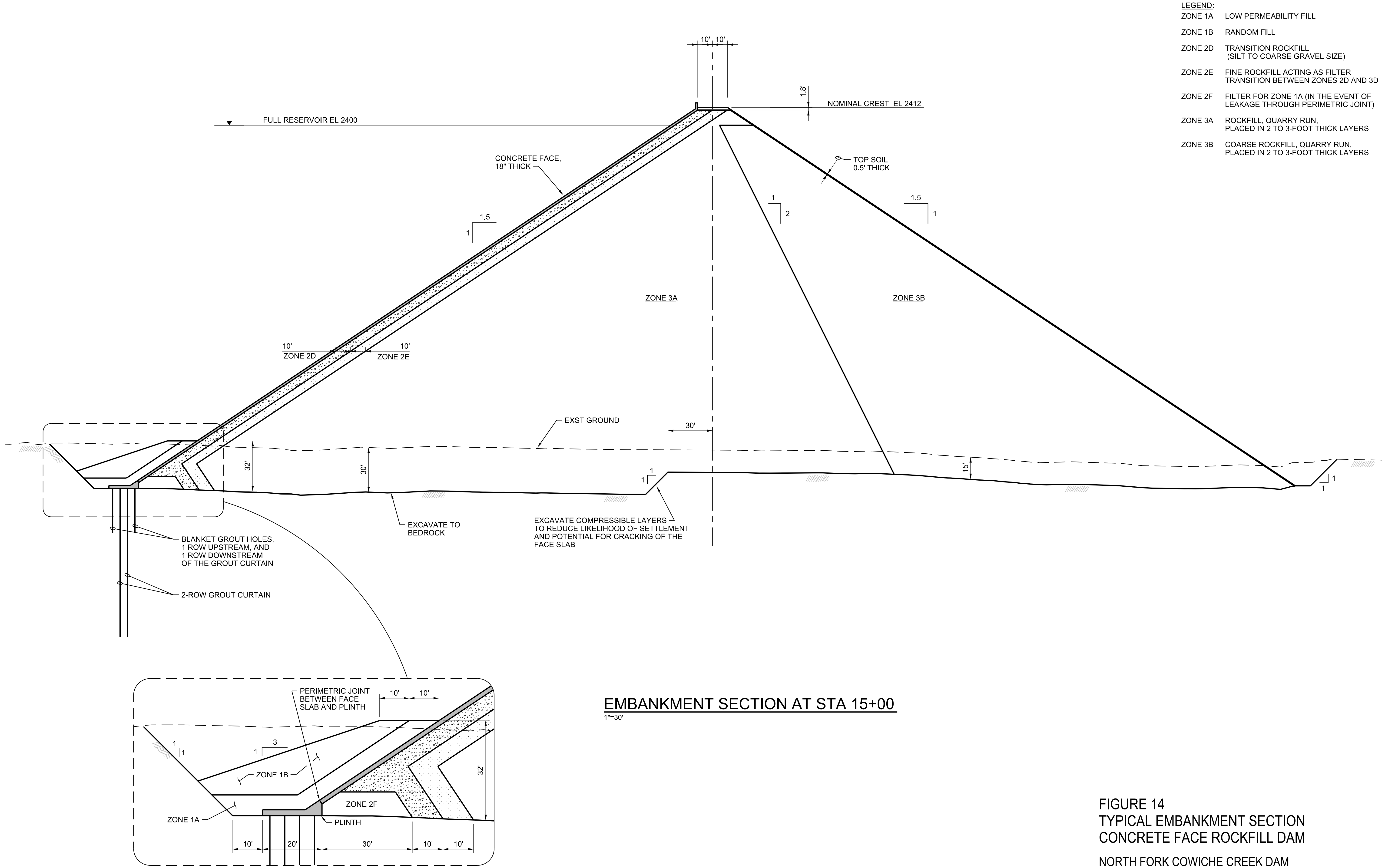
FIGURE 13
PLAN
CONCRETE FACE ROCKFILL DAM (CFRD)

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

FILENAME: FS-Fig13_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:52:15 PM



EMBANKMENT SECTION AT STA 15+00

1"=30'

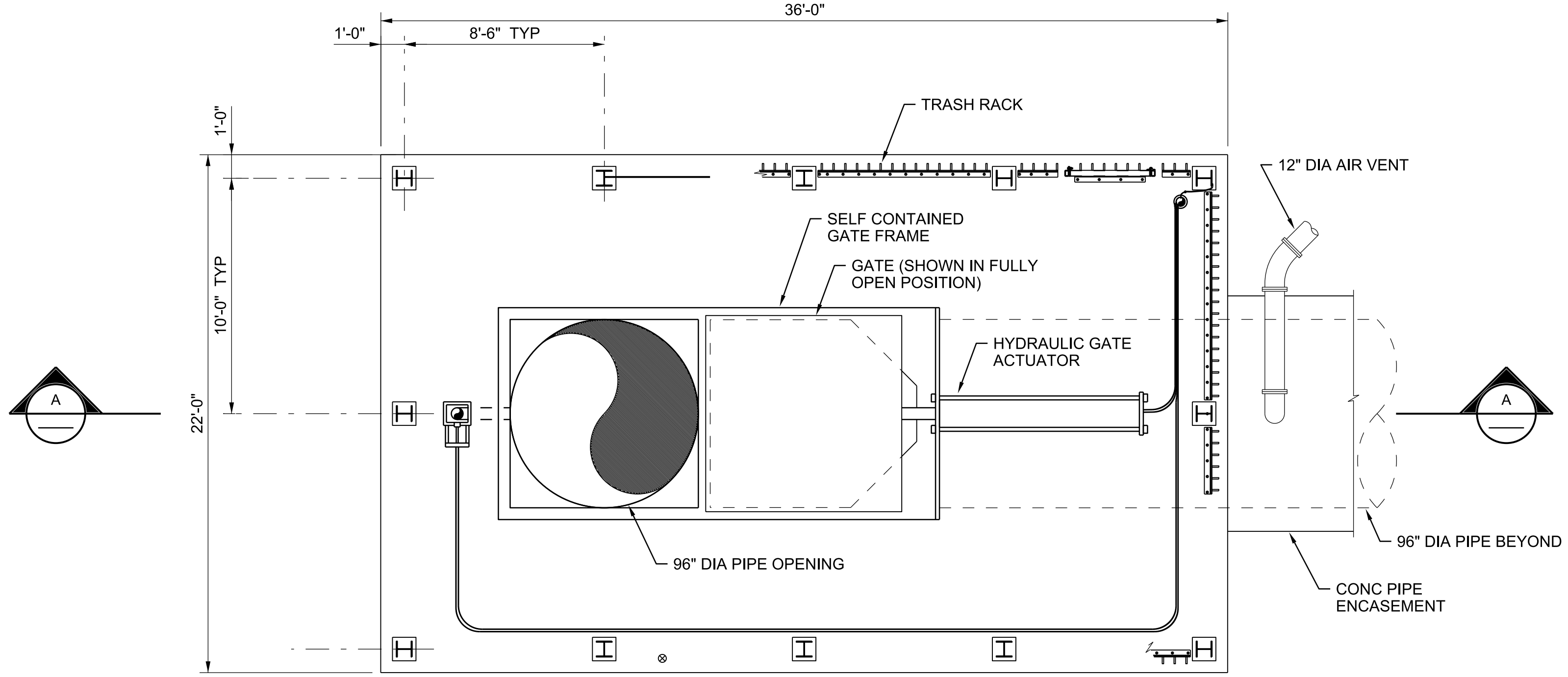
FIGURE 14
TYPICAL EMBANKMENT SECTION
CONCRETE FACE ROCKFILL DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

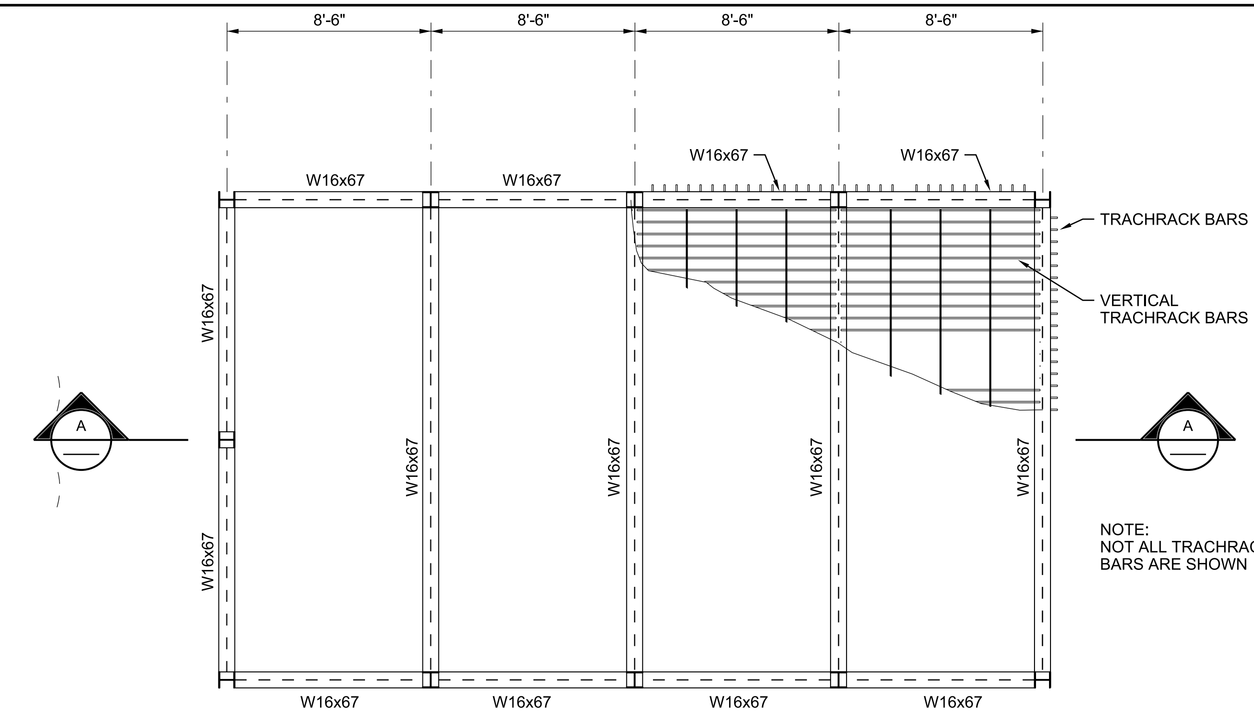
470080

ch2m

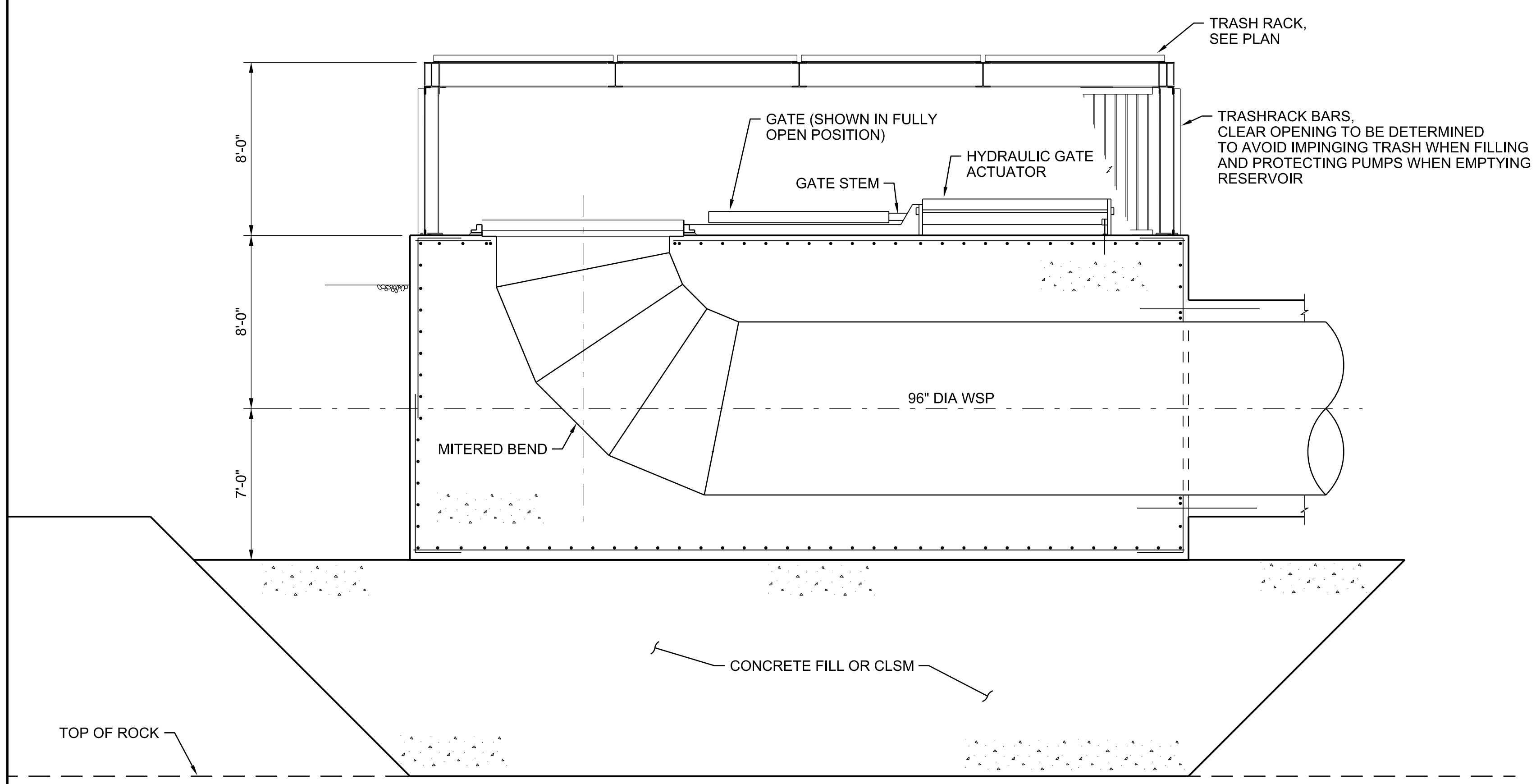
FILENAME: FS-Fig14_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:51:41 PM



INTAKE STRUCTURE PLAN
1/4"=1'-0"



INTAKE STRUCTURE TRASHRACK PLAN
1/4"=1'-0"



A SECTION
1/4"=1'-0"

SIMILAR STRUCTURE AT BIG SAND WASH DAM
NTS

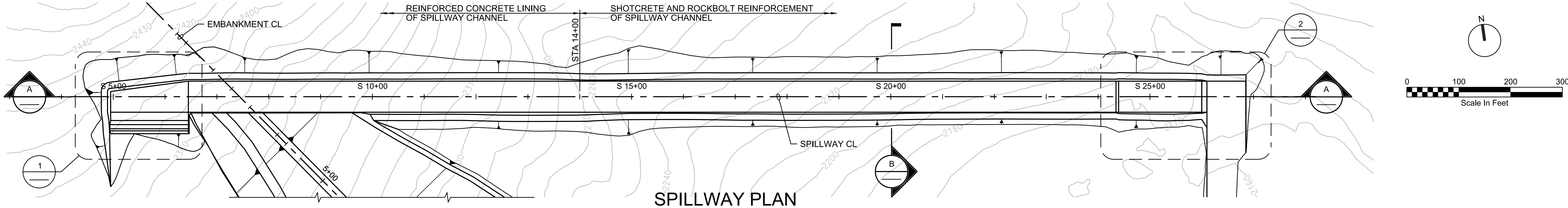
FIGURE 15
INTAKE STRUCTURE
CONCRETE FACE ROCKFILL DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

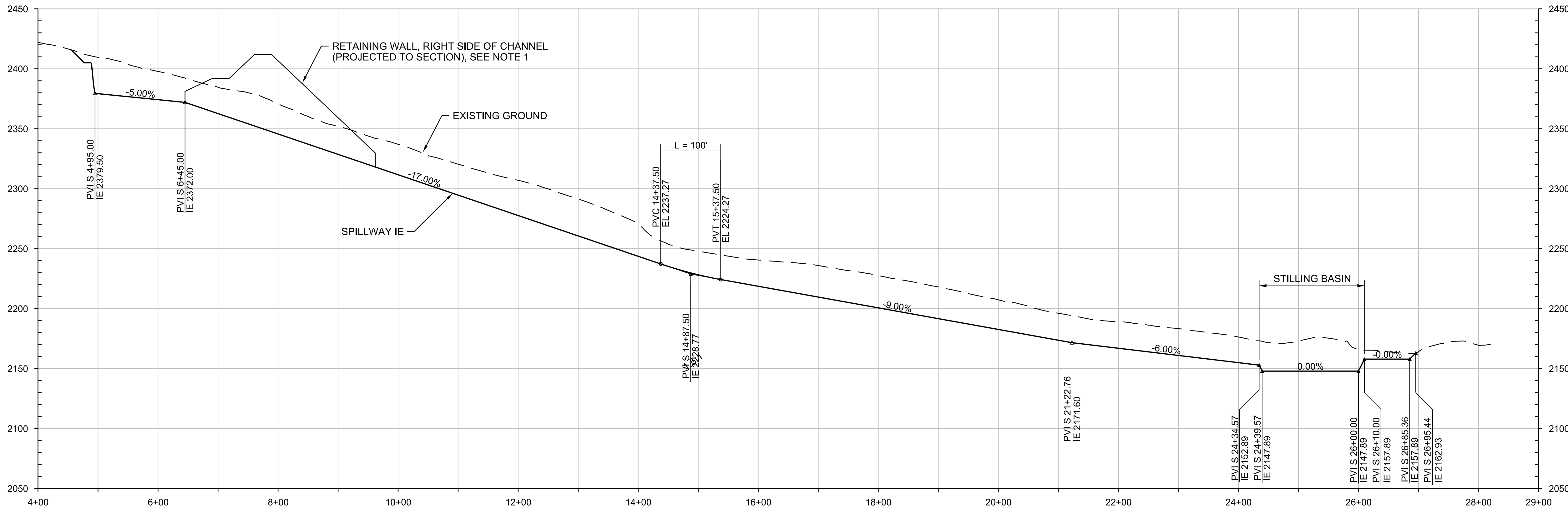
ch2m

FILENAME: FS-Fig15_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:51:47 PM



SPILLWAY PLAN

1"=100'

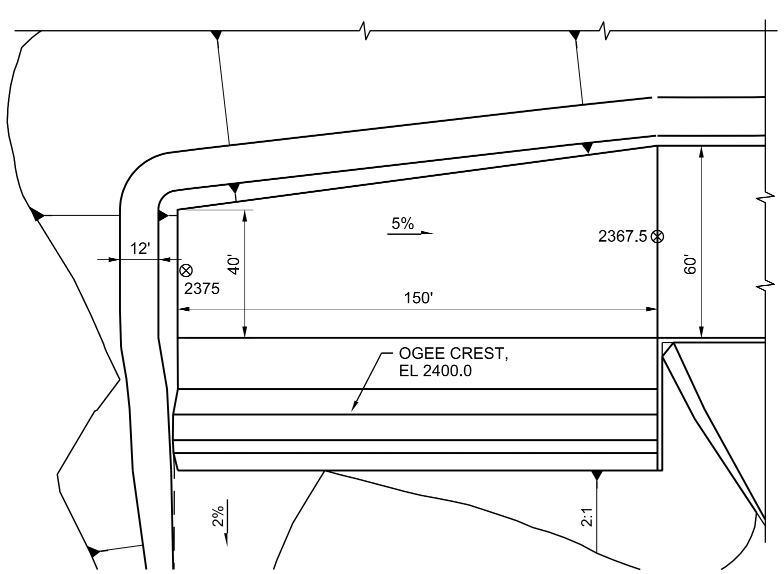


NOTE:

1. THE SPILLWAY ALIGNMENT SHALL BE OPTIMIZED IN NEXT DESIGN PHASES TO BALANCE ROCK CUT VERSUS THE SIZE OF THE RETAINING WALL.

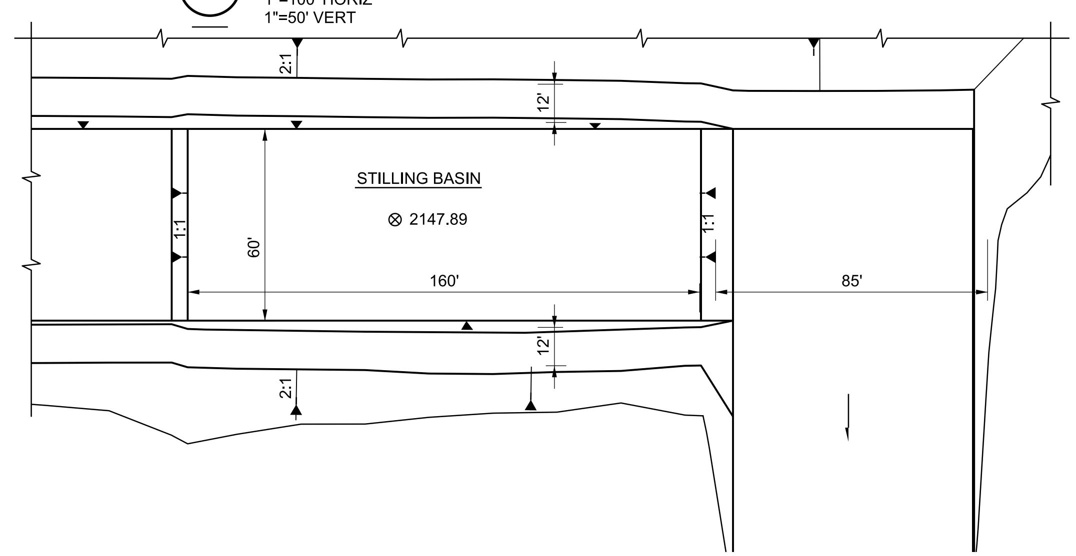
A SECTION

1"=100' HORIZ
1"=50' VERT



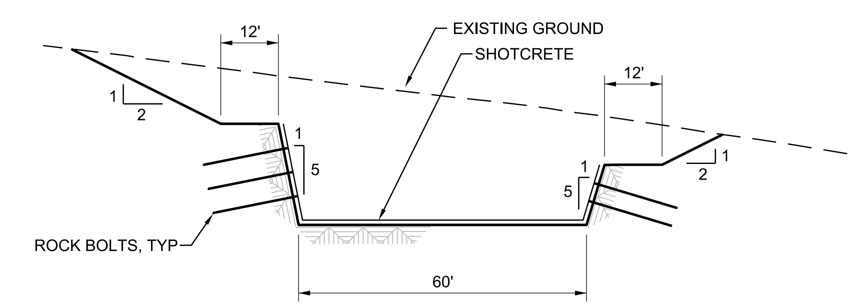
1 DETAIL

1"=30'



2 DETAIL

1"=30'



B SECTION

1"=20'

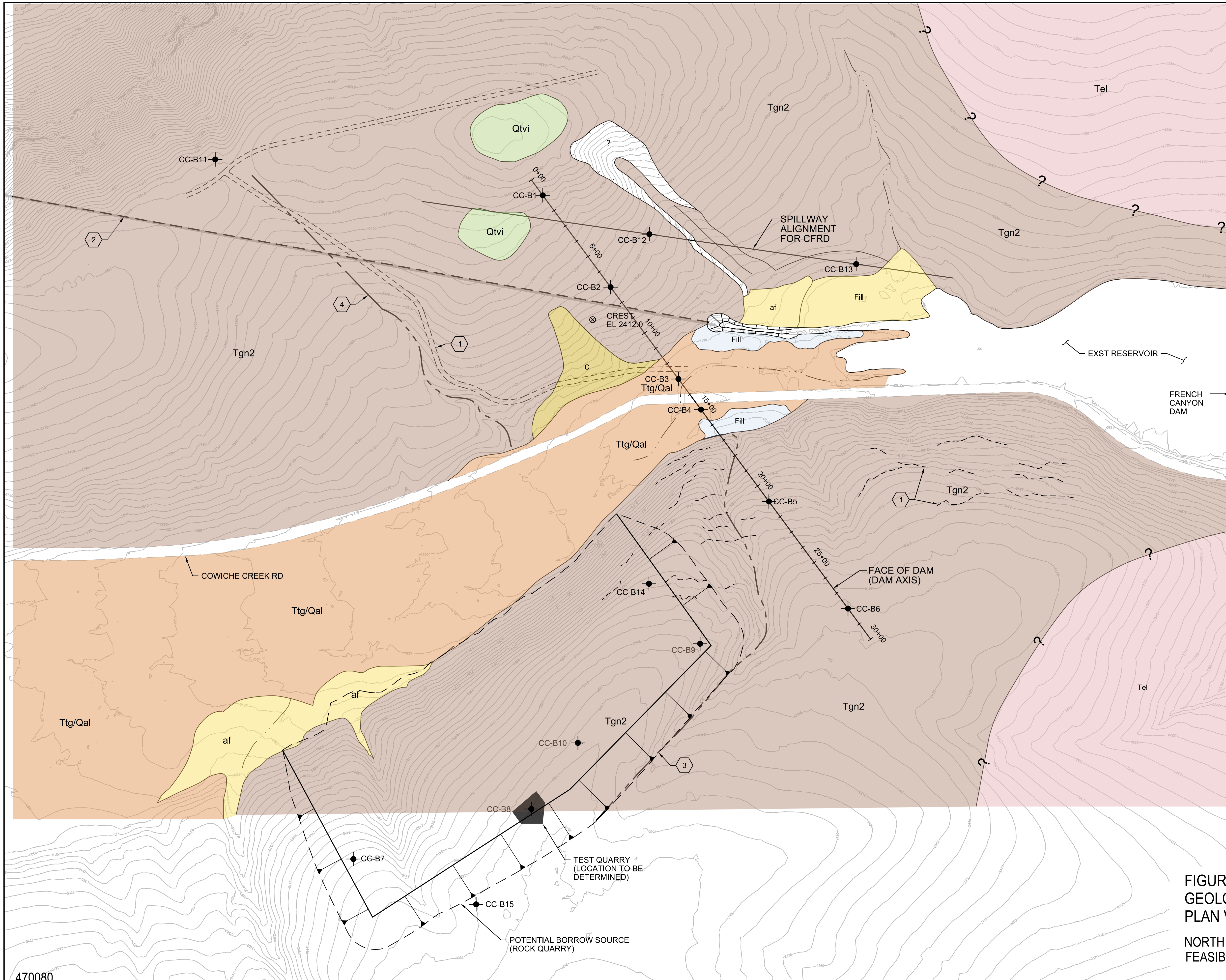
FIGURE 16
SPILLWAY PLAN, SECTIONS AND DETAILS
CONCRETE FACE ROCKFILL DAM

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

FILENAME: FS-Fig16_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 6:53:15 PM



N

0 250 500 750
Scale In Feet

NOTES:

1. BASALT EXPOSURES
2. TUNNEL
3. POSSIBLE SHEAR ZONE
4. POSSIBLE SHEAR ZONE

LEGEND:

Fill	FILL
af	ALLUVIAL FAN
c	COLUVIUM/ SOIL
Ttg/Qal	THORP GRAVEL/ ALLUVIUM
Qtvi	TIETON ANDESITE
Tgn2	GRANDE RONDE BASALT
Tel	ELLENSBURG FORMATION

SEE REPORT FOR A DESCRIPTION OF GEOLOGIC UNITS

CC-B1 PROPOSED EXPLORATORY BORING FOR PRELIMINARY DESIGN

PROPOSED DAM ALIGNMENT

GRAVEL ROAD

NOTE:

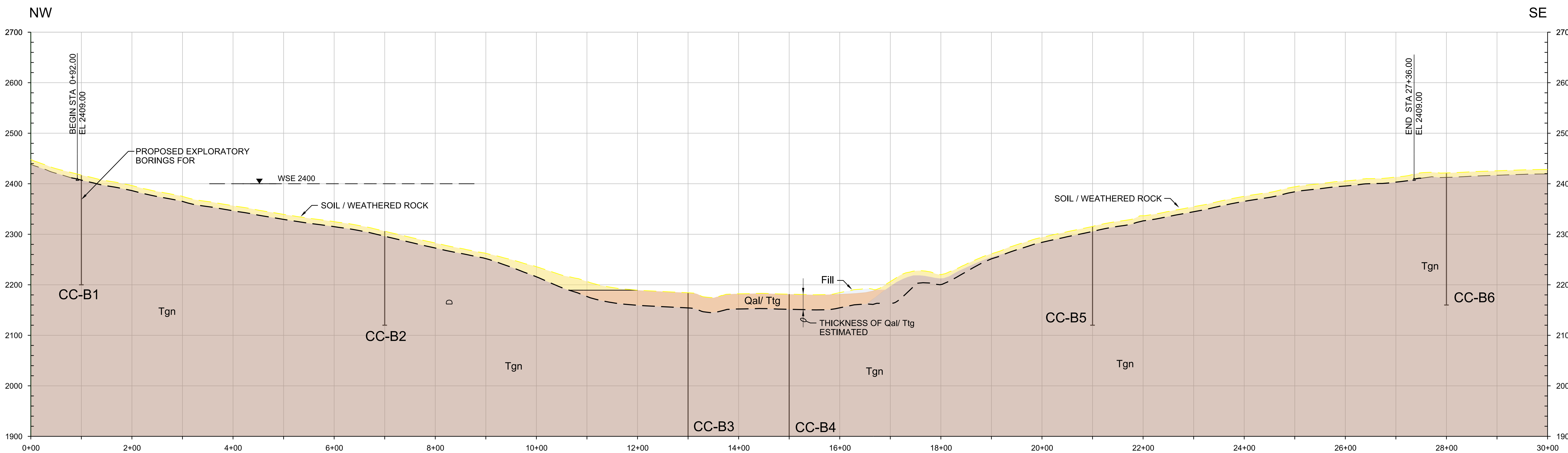
1. THE PROPOSED EXPLORATORY BORINGS SHOWN WILL BE USED FOR PRELIMINARY DESIGN AND FEASIBILITY PURPOSES, AND WILL NOT BE SUFFICIENT FOR FINAL DESIGN. A MORE COMPREHENSIVE ADDITIONAL DRILLING PROGRAM WILL BE NECESSARY FOR FINAL DESIGN.

FIGURE 17
GEOLOGIC MAP
PLAN VIEW

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

ch2m

FILENAME: FS-Fig17_470080.dgn PLOT DATE: 10/6/2016 PLOT TIME: 7:12:38 PM



GEOLOGIC SECTION AT CENTERLINE PROFILE

1"=100' (NO VERTICAL EXAGERATION)

FIGURE 18
GEOLOGIC PROFILE

NORTH FORK COWICHE CREEK DAM
FEASIBILITY STUDY

470080

ch2m

Appendix B
Construction Cost Estimates

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
October, 2016

Facility		2016 Cost
CFRF Dam	$	139,374,000
Flow Meter Vault	$	563,000
Main Pipeline	$	2,722,000
Flow Control Station	$	8,035,000
Pump Station	$	37,293,000
	$	**187,987,000**

Notes:

1. See individual cost sheets for breakdown
2. Costs include contractor's General Conditions, overhead and profit
3. Costs include 20% contingency for dam and 30% contingency for pump station, mechanical, and electrical
4. No cost included for land acquisition, admin, legal, engingeering, construction management, environmental mitigation, or permitting
5. Cost are 2nd qtr 2016, no escalation is included
6. Class 4 cost estimates are based on conceptual designs and limited field information. Probable accuracy of cost estimate is 30% below to 50% above bid price, at the time the estimate was completed.

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
CFRF Dam

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
General						
	Mobilization	1	LS	$ 3,485,000	$ 3,485,000	Approximately 3% of Total
	SWPPP	1	LS	$ 200,000	$ 200,000	Allowance
	SWPPP Best Practices	1	LS	$ 100,000	$ 100,000	Allowance
	Builders Risk Policy	1	LS	$ 500,000	$ 500,000	Allowance
	Demobilization	1	LS	$ 2,325,000	$ 2,325,000	Approximately 2% of Total
				Subtotal	**$ 6,610,000**	
CFRF Dam						
	Clearing and Grubbing	45	AC	$ 5,000	$ 225,000	About 20 acre under footprint of dam and downstream channel Assume additional 15 acres for access roads, stockpiling, staging, etc..
	Surface Restoration	15	AC	$ 5,000	$ 75,000	
	Excavation	735,000	CY	$ 12	$ 8,820,000	About 735,000 cy excavation for dam Assume 1/4 rock, 3/4 overburden soil; unit cost is $3 less than for RCC Dam
	Large Rockfill (Zone 3)	3,535,000	CY	$ 15	$ 53,025,000	
	Gravel/small rockfill (Zone 2)	252,000	CY	$ 25	$ 6,300,000	Unit cost of $25/cy provided by Barnard
	Fine-grained/random (Zone 1)	110,000	CY	$ 20	$ 2,200,000	
	Concrete Face (including plinth)	36,000	CY	$ 500	$ 18,000,000	
	Foundation Preparation (cleaning, dental concrete, etc..)	1	LS	$ 1,900,000	$ 1,900,000	High-quality prep below plinth est 200,000 sf @ $4/sf; low-quality prep rest of dam est. 1.1 million sf @ $1/sf
	Tunnel Extension to beyond Dam Toe	1	LS	$ 500,000	$ 500,000	Allowance
	Geotechnical Instrumentation	1	LS	$ 300,000	$ 300,000	Allowance
				Subtotal	**$ 91,345,000**	
Foundation Grouting						
	Mobilization/Demobilization	1	LS	$ 400,000	$ 400,000	Approximately 5% of Total
	Drilling	117,900	LF	$ 25	$ 2,947,500	Grouting from Sta 0+00 to Sta 2410+00, i.e., total distance of 2410 ft; **Curtain Grouting**: - Assume 2-row grout curtain with final average hole spacing of 7.5 feet for total of 964 grout curtain holes, i.e, assume final pattern hole spacing of 10 feet with some areas of tertiary grouting with final hole spacing of 5 ft. - Three hundred and seventy four (374) 200-foot holes from Sta 7+00 to Sta 21+00 (total drilling length = 74,800 ft) - Two hundred and seventy (270) 100-ft holes from Sta 0+00 to Sta 7+00 and from Sta 21+00 to Sta 2410 (total drilling length = 27,000 ft) **Blanket Grouting:** - Assume 2 rows of blanket grout holes with final hole spacing of 7.5 feet for total of 644 blanket grout holes. - Assume blanket grout hole length = 25 ft - Total drilling length = 16,100 ft (644 holes x 25/ft/hole)
	Grout Nipples	1,288	EA	$ 100	$ 128,800	Total number of holes = 1,288 (374 + 270 + 644)

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
CFRF Dam

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
	Hookups	9,058	EA	$ 300	$ 2,717,400	Assume average 16 hookups per hole (200-ft long hole assuming 15-ft stages, plus 2 extra hook-ups per hole) for total of 5984 hook-ups. Assume average 9 hookups per hole (100-ft long hole assuming 15-ft stages, plus 2 extra hook-ups per hole) for total of 2430 hook-ups. Assume 1 hookup for 25-ft long blanket grout holes for total of 644 hook-ups
	Type 3 Cement	117,900	SACKS	$ 15	$ 1,768,500	Assume grout take of 1 sack (94 lbs) per 1 lineal foot
	Microsilica	11,790	SACKS	$ 8	$ 94,320	Assume microsilica content is 10% of cement
	Water Pressure Testing	500	Hrs	$ 160	$ 80,000	Estimated No of hours = 500 hrs
	Verification Borings	3,600	LF	$ 60	$ 216,000	Assume verification boring every 100 ft along curtain for total of 24 borings Assume average length of verification boring = 150 ft
				Subtotal	**$ 8,352,520**	
Spillway						
	Excavation	168,000	CY	$ 17	$ 2,856,000	About 168,000 cy excavation for downstream channel; 3/4 rock, 1/4 overburden
	Ogee Crest	150	CY	$ 3,000	$ 450,000	Unit price for ogee crest based on Duck River Dam cost
	Shotcrete	1,800	CY	$ 300	$ 540,000	Length of spillway channel est 1,800 ft. Est avg shotcrete vol = 1 cy per lin ft
	Rock Bolts/Dowels	1,800	EA	$ 300	$ 540,000	Length of spillway channel est 1,800 ft. Assume 1 rock dowel per lin ft
	Retaining Wall between rockfill and spillway channel	1,900	CY	$ 500	$ 950,000	Reinforced concrete volume of 1100 cy for wall based on wall area of 14300 sq ft and wall thickness of 2 ft; Assume additional reinforced concrete volume of 800 cy (300 cy for base and 500 cy for counterfort walls)
	Stilling Basin	1	EA	$ 300,000	$ 300,000	Allowance
				Subtotal	**$ 5,636,000**	
Intake Structure						
	CLSM/Concrete Backfill Below Structur	500	CY	$ 100	$ 50,000	
	Trash Rack	1	EA	$ 200,000	$ 200,000	Allowance
	Reinforced Concrete	440	CY	$ 500	$ 220,000	Approximately 440 cy
	Slide Gate including frame and actuato	1	EA	$ 300,000	$ 300,000	Allowance (horizontal slide gate including hydraulic actuator)
	Gate Control House	1	EA	$ 200,000	$ 200,000	Include building, hydraulic pump, air vent piping, carrier pipe for hydraulic lines
	Extra 96-in Pipeline for CFRF Dam	470	LF	$ 2,620	$ 1,231,400	Assumed unit price includes excavation ($200/lf), concrete backfill ($500/lf) and pipe ($1,920/lf)
				Subtotal	**$ 2,201,400**	
Diversion and Control of Water						
	Cofferdam/Diversion of Creek/Water c	1	LS	$ 2,000,000	$ 2,000,000	Allowance ($500k greater than for RCC Dam because overtopping has more severe consequences)
				Subtotal	**$ 2,000,000**	
Totals						
				Subtotal	**$ 116,144,920**	
	Contingency including Miscellaneous Items (20%)				$ 23,228,980	
				Subtotal	**$ 139,373,900**	
				Total Facility Cost	**$ 139,374,000**	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Flow Meter Vault

Facility	Description	Qty	Unit	Unit Price		Extended Total		Comments
General								
	Mobilization	1	LS	$	13,000	$	13,000	Approximately 3% of Total
	Start-up/Testing	1	LS	$	2,500	$	2,500	Allowance
	Demobilization	1	LS	$	8,700	$	8,700	Approximately 2% of Total
					Subtotal	**$**	**24,200**	
Site Work								
	Clearing and Grubbing	0.00	AC	$	5,000	$	-	Included in FC Station
	Site Grading	0	CY	$	10	$	-	Included in FC Station
	Gravel Surfacing	0	SQFT	$	1.5	$	-	Included in FC Station
	Bollards	8	EA	$	500	$	4,000	
					Subtotal	**$**	**4,000**	
Vault Structural								
	Excavation	1210	CY	$	17	$	20,570	
	Backfill	1000	CY	$	25	$	25,000	
	Concrete-Slab	40	CY	$	750	$	30,000	
	Concrete-Walls	140	CY	$	1,000	$	140,000	
	Top Hatch	1	EA	$	15,000	$	15,000	
	Removable Roof Panels	290	SF	$	50	$	14,500	
	Interior Metals	1	LS	$	2,500	$	2,500	Allowance
	Misc Structural	1	LS	$	5,000	$	5,000	Allowance
					Subtotal	**$**	**252,570**	
Vault Mechnical								
	Ultrasonic Flowmeter	1	EA	$	100,000	$	100,000	
	Sump Pump and Piping	1	EA	$	15,000	$	15,000	
	HVAC/ducting	1	LS	$	10,000	$	10,000	
	Misc Mech	1	LS	$	5,000	$	5,000	Allowance at vault
					Subtotal	**$**	**130,000**	
Electrical and I&C								
	Site Electrical	1	LS	$	7,500	$	7,500	Power/signal run and ground
	110/200V Panel	1	EA	$	5,000	$	5,000	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Flow Meter Vault

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
	Vault Wiring	1	LS	$ 2,500	$ 2,500	
	Misc electrical	1	LS	$ 2,500	$ 2,500	Allowance for lighting, conduit, cable, etc
	Misc instrumentation and control	1	LS	$ 5,000	$ 5,000	Allowance for instruments, network switches, wiring, etc
				Subtotal	**$ 22,500**	
Totals						
				Subtotal	**$ 433,270**	
				Contingency (30%)	$ 129,980	
				Subtotal	**$ 563,250**	
				Total Facility Cost	**$ 563,000**	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Main Pipeline

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
General						
	Mobilization	1	LS	$ 65,000	$ 65,000	Approximately 3% of Total
	SWPPP	1	LS	$ 3,000	$ 3,000	Allowance
	SWPPP Best Practices	1	LS	$ 5,000	$ 5,000	Allowance
	Start-up/Testing	1	LS	$ 5,000	$ 5,000	Allowance
	Demobilization	1	LS	$ 40,000	$ 40,000	Approximately 2% of Total
				Subtotal	**$ 118,000**	
Site Work and Pipeline						
	Clearing and Grubbing	0	AC	$ 5,000	$ -	Included in dam and FC Station
	Surface Restoration	0	AC	$ 5,000	$ -	Included in FC Station
	Excavation	900	LF	$ 200	$ 180,000	
	Backfill	900	LF	$ 500	$ 450,000	
	96" Pipe	900	LF	$ 1,440	$ 1,296,000	
	Corrosion Protection	1	LS	$ 25,000	$ 25,000	Allowance
	Manways	3	EA	$ 5,000	$ 15,000	
	Misc Civil/Pipeline	1	LS	$ 10,000	$ 10,000	Allowance
				Subtotal	**$ 1,976,000**	
Totals						
				Subtotal	**$ 2,094,000**	
				Contingency (30%)	$ 628,200	
				Subtotal	**$ 2,722,200**	
				Total Facility Cost	**$ 2,722,000**	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Flow Control Station

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
General						
	Mobilization	1	LS	$ 185,000	$ 185,000	Approximately 3% of Total
	SWPPP	0	LS	$ 3,000	$ -	Included in Pump Station
	SWPPP Best Practices	0	LS	$ 5,000	$ -	Included in Pump Station
	Start-up/Testing	1	LS	$ 25,000	$ 25,000	Allowance
	Demobilization	1	LS	$ 125,000	$ 125,000	Approximately 2% of Total
				Subtotal	**$ 335,000**	
Site Work						
	Clearing and Grubbing	0.8	AC	$ 5,000	$ 4,247	
	Site Grading	1370	CY	$ 10	$ 13,704	Assume 1 foot over cleared area
	Paved Surfacing (incl AB)	25000	SQFT	$ 5	$ 125,000	
	Gravel Surfacing	12000	SQFT	$ 1.50	$ 18,000	
	Bollards	30	EA	$ 500	$ 15,000	
				Subtotal	**$ 175,951**	
Vault Structural						
	Excavation	7390	CY	$ 17	$ 125,630	Assume 1/2 rock
	Backfill	3350	CY	$ 25	$ 83,750	
	Concrete-Slab	500	CY	$ 750	$ 375,000	
	Concrete-Walls	570	CY	$ 1,000	$ 570,000	
	Top Hatch	2	EA	$ 15,000	$ 30,000	
	Removable Roof Panels	4850	SF	$ 50	$ 242,500	
	Interior Walkways	900	SF	$ 50	$ 45,000	
	Misc Structural	1	LS	$ 150,000	$ 150,000	Allowance
				Subtotal	**$ 1,621,880**	
Vault Mechnical						
	96" BFV	3	EA	$ 150,000	$ 450,000	
	96" Harness Set w/Ins Coupling	3	EA	$ 35,000	$ 105,000	
	96" Headers and Piping	1	LS	$ 1,530,000	$ 1,530,000	
	48/36" Piping	1	LS	$ 625,000	$ 625,000	
	48" BFV	3	EA	$ 50,000	$ 150,000	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Flow Control Station

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
	36" Sleeve/Plunger Valve	3	EA	$ 225,000	$ 675,000	
	36" Ball Valve	3	EA	$ 110,000	$ 330,000	
	Sump Pump and Piping	1	EA	$ 15,000	$ 15,000	
	HVAC/ducting	1	LS	$ 10,000	$ 10,000	
	Misc Mech	1	LS	$ 10,000	$ 10,000	Allowance at vault
				Subtotal	**$ 3,900,000**	
Electrical and I&C						
	Site Lighting	1	LS	$ 3,000	$ 3,000	Allowance
	Site Electrical	1	LS	$ 10,000	$ 10,000	Power/signal run and ground
	480V Panel	1	EA	$ 15,000	$ 15,000	Panel and Main Disconnect
	110/200V Panel w/XFMR	1	EA	$ 10,000	$ 10,000	
	Vault Wiring	1	LS	$ 50,000	$ 50,000	
	Misc electrical	1	LS	$ 30,000	$ 30,000	Allowance for lighting, conduit, cable, etc
	Misc instrumentation and control	1	LS	$ 30,000	$ 30,000	Allowance for instruments, network switches, wiring, etc
				Subtotal	**$ 148,000**	
Totals						
				Subtotal	**$ 6,180,831**	
				Contingency (30%)	$ 1,854,250	
				Subtotal	**$ 8,035,081**	
				Total Facility Cost	**$ 8,035,000**	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Pump Station

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
General						
	Mobilization	1	LS	$ 860,000	$ 860,000	Approximately 3% of Total
	SWPPP	1	LS	$ 3,000	$ 3,000	Allowance
	SWPPP Best Practices	1	LS	$ 5,000	$ 5,000	Allowance
	Start-up/Testing	1	LS	$ 150,000	$ 150,000	Allowance
	Demobilization	1	LS	$ 575,000	$ 575,000	Approximately 2% of Total
				Subtotal	**$ 1,593,000**	
Site Work						
	Clearing and Grubbing	4.8	AC	$ 5,000	$ 24,000	
	Cofferdam/water control	1	LS	$ 1,000,000	$ 1,000,000	Wet well is constructed during off-irrigation season
	Site Grading	7740	CY	$ 10	$ 77,400	Assume 1 foot over entire site
	Site Drainage	1	LS	$ 10,000	$ 10,000	Allowance
	Excavation	24970	CY	$ 17	$ 424,490	Assume 1/2 rock
	Backfill	8566	CY	$ 25	$ 214,140	
	Gravel Surfacing	7500	SQFT	$ 1.50	$ 11,250	Switchyard only
	Paved Surfacing (incl AB)	31400	SQFT	$ 5	$ 157,000	
	Site Lighting	1	LS	$ 5,000	$ 5,000	Allowance
	Site Electrical	1	LS	$ 100,000	$ 100,000	
	Bollards	4	EA	$ 500	$ 2,000	
				Subtotal	**$ 2,025,280**	
Pump Station (not including Electrical Room)						
	Approach Slab	1117	CY	$ 750	$ 837,500	
	Wet well slab	940	CY	$ 750	$ 704,667	
	Pump room floor	361	CY	$ 750	$ 270,542	
	Wing wall base slab	157	CY	$ 750	$ 117,833	
	Wing walls	518	CY	$ 1,000	$ 518,467	
	Wet well walls	1474	CY	$ 1,000	$ 1,474,000	
	Outlet Box Walls	74	CY	$ 1,000	$ 74,074	
	Wet well top slab	940	CY	$ 1,000	$ 939,556	
	Exterior PS walls	18450	SF	$ 40	$ 738,000	Includes finishes

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Pump Station

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
	Roof	8700	SF	$ 50	$ 435,000	
	Doors	1	LS	$ 35,000	$ 35,000	Includes all man doors and roll-up door
	Sidewalks	23	CY	$ 250	$ 5,754	
	Crane support structure	1	LS	$ 250,000	$ 250,000	Allowance
	Misc structural	1	LS	$ 100,000	$ 100,000	Allowance for undefined structural features
	Misc metals	1	LS	$ 50,000	$ 50,000	Allowance-ladders, embeds, wall sleeves, etc.
	Pump Cans	6	EA	$ 160,000	$ 960,000	
	Pump bases	6	EA	$ 20,000	$ 120,000	Base, base plate, and bolts
	Fish screen/trash rack assemblies	6	EA	$ 10,000	$ 60,000	Price includes trash rack only
	Bridge crane	1	LS	$ 100,000	$ 100,000	Includes controls
	Pumps and motors	6	EA	$ 1,500,000	$ 9,000,000	
	Pump discharge piping	6	EA	$ 40,000	$ 240,000	
	Discharge header	1	LS	$ 450,000	$ 450,000	
	Check valves	6	EA	$ 140,000	$ 840,000	
	Isolation valves	6	EA	$ 50,000	$ 300,000	
	Exhaust Fans	6	EA	$ 5,000	$ 30,000	Includes control panel
	HVAC ducting	1	LS	$ 25,000	$ 25,000	
	Louvers	6	EA	$ 5,000	$ 30,000	
	Misc mechanical	1	LS	$ 100,000	$ 100,000	Allowance for piping etc.
	Misc plumbing	1	LS	$ 25,000	$ 25,000	Allowance for drains/process water
	Lighting	1	LS	$ 100,000	$ 100,000	
	Misc electrical	1	LS	$ 500,000	$ 500,000	Allowance for conduit, cable, etc
	Misc instrumentation and control	1	LS	$ 86,000	$ 86,000	Allowance for instruments, network switches, wiring, etc
				Subtotal	**$ 19,516,392**	
Electrical Room						
	Floor Slab	150	CY	$ 750	$ 112,500	
	Exterior Walls	8400	SF	$ 40	$ 336,000	Included finishes
	Roof	4000	SF	$ 50	$ 200,000	
	Doors	1	LS	$ 10,000	$ 10,000	Includes all man doors
	Sidewalks	17	CY	$ 250	$ 4,343	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Pump Station

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
	Misc structural	1	LS	$ 25,000	$ 25,000	Allowance for undefined structural features
	Misc metals	1	LS	$ 10,000	$ 10,000	Allowance-ladders, embeds, wall sleeves, etc.
	Equipment bases	20	CY	$ 400	$ 7,822	
	Main switchgear	1	EA	$ 750,000	$ 750,000	
	RVSS	6	EA	$ 100,000	$ 600,000	
	MCC	2	EA	$ 150,000	$ 300,000	
	480V Panelboard	2	EA	$ 25,000	$ 50,000	
	110/220 Panelboard w/xfmr	4	EA	$ 15,000	$ 60,000	
	HVAC	1	EA	$ 150,000	$ 150,000	Allowance for A/C unit and ducting
	Lighting	1	LS	$ 25,000	$ 25,000	
	Misc electrical	1	LS	$ 250,000	$ 250,000	Allowance for conduit, cable, etc
	Main PLC Panel	1	EA	$ 100,000	$ 100,000	
	Switchyard controls	1	LS	$ 100,000	$ 100,000	
	Control/SCADA workstation	1	EA	$ 25,000	$ 25,000	Included desk/chair and computer equipment
	Network/Comms panel/racks	1	EA	$ 50,000	$ 50,000	
	UPS	2	EA	$ 20,000	$ 40,000	
	Fire Detection/Alarm	1	LS	$ 100,000	$ 100,000	
	Security	1	LS	$ 100,000	$ 100,000	
	Misc instrumentation and control	1	LS	$ 150,000	$ 150,000	Allowance for instruments, network interface, wiring, etc
				Subtotal	**$ 3,555,665**	
Switchyard						
	Perimeter Fence	310	LF	$ 15	$ 4,650	
	Gate	1	LS	$ 5,000	$ 5,000	
	Control Equipment	1	LS	$ 25,000	$ 25,000	Switch, relays, comms, etc.
	Incoming bus and connections	1	LS	$ 150,000	$ 150,000	
	Air Switch	2	EA	$ 100,000	$ 200,000	
	Metering	2	EA	$ 50,000	$ 100,000	
	Transformer	2	EA	$ 500,000	$ 1,000,000	
	Ougoing bus to E bldg	1	LS	$ 150,000	$ 150,000	
	Equipment bases	54	CY	$ 500	$ 26,963	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
Pump Station

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
	Misc electrical	1	LS	$ 200,000	$ 200,000	Allowance for lighting, conduit, cable, etc
	Misc instrumentation and control	1	LS	$ 100,000	$ 100,000	Allowance for instruments, network interface, wiring, etc
	Containment	1	LS	$ 25,000	$ 25,000	
	Security	1	LS	$ 10,000	$ 10,000	
				Subtotal	**$ 1,996,613**	
Totals						
				Subtotal	**$ 28,686,950**	
				Contingency (30%)	$ 8,606,080	
				Subtotal	**$ 37,293,030**	
				Total Facility Cost	**$ 37,293,000**	

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
RCC Dam

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
General						
	Mobilization	1	LS	$ 3,790,000	$ 3,790,000	Approximately 3% of Total
	SWPPP	1	LS	$ 200,000	$ 200,000	Allowance
	SWPPP Best Practices	1	LS	$ 100,000	$ 100,000	Allowance
	Builders Risk Policy	1	LS	$ 500,000	$ 500,000	Allowance
	Demobilization	1	LS	$ 2,530,000	$ 2,530,000	Approximately 2% of Total
				Subtotal	**$ 7,120,000**	
RCC Dam						
	Clearing and Grubbing	25	AC	$ 5,000	$ 125,000	About 15 acre under footprint of dam and downstream channel Assume additional 10 acres for access roads, stockpiling, staging, etc..
	Surface Restoration	10	AC	$ 5,000	$ 50,000	Assume 10 acres restoration
	Excavation	322,000	CY	$ 15	$ 4,830,000	About 322,000 cy excavation for dam Assume 1/2 rock, 1/2 overburden soil
	Roller Compacted Concrete (RCC)	1,048,170	CY	$ 90	$ 94,335,300	***Assumed unit cost is 10% less than estimate provided by Barnard***
	Gallery	1	LS	$ 500,000	$ 500,000	Allowance
	Ogee Crest	250	LF	$ 3,000	$ 750,000	Unit price for ogee crest based on Duck River Dam cost
	Foundation Drains	5,650	LF	$ 30	$ 169,500	Assume 15 ft drain spacing (1,700 ft from Sta 5+00 to Sta 22+00) and 50 ft long holes; total no. of holes = 113
	Foundation Preparation (cleaning, dental concrete, etc..)	1	LS	$ 2,600,000	$ 2,600,000	Estimate approx 650,000 sq ft; $4 per sq ft
	Geotechnical Instrumentation	1	LS	$ 300,000	$ 300,000	Allowance
				Subtotal	**$ 103,659,800**	
Foundation Grouting						
	Mobilization/Demobilization	1	LS	$ 400,000	$ 400,000	Approximately 5% of Total
	Drilling	117,900	LF	$ 25	$ 2,947,500	Grouting from Sta 0+00 to Sta 2410+00, i.e., total distance of 2410 ft; **Curtain Grouting**: - Assume 2-row grout curtain with final average hole spacing of 7.5 feet for total of 964 grout curtain holes, i.e, assume final pattern hole spacing of 10 feet with some areas of tertiary grouting with final hole spacing of 5 ft. - Three hundred and seventy four (374) 200-foot holes from Sta 7+00 to Sta 21+00 (total drilling length = 74,800 ft) - Two hundred and seventy (270) 100-ft holes from Sta 0+00 to Sta 7+00 and from Sta 21+00 to Sta 2410 (total drilling length = 27,000 ft) **Blanket Grouting:** - Assume 2 rows of blanket grout holes with final hole spacing of 7.5 feet for total of 644 blanket grout holes. - Assume blanket grout hole length = 25 ft - Total drilling length = 16,100 ft (644 holes x 25/ft/hole)
	Grout Nipples	1,288	EA	$ 100	$ 128,800	Total number of holes = 1,288 (374 + 270 + 644)

North Fork Cowiche Creek Reservoir Feasibility Study
Class 4 Construction Cost Estimate
RCC Dam

Facility	Description	Qty	Unit	Unit Price	Extended Total	Comments
		9,058				Assume average 16 hookups per hole (200-ft long hole assuming 15-ft stages, plus 2 extra hook-ups per hole) for total of 5984 hook-ups. Assume average 9 hookups per hole (100-ft long hole assuming 15-ft stages, plus 2 extra hook-ups per hole) for total of 2430 hook-ups.
	Hookups		EA	$ 300	$ 2,717,400	Assume 1 hookup for 25-ft long blanket grout holes for total of 644 hook-ups
	Type 3 Cement	117,900	SACKS	$ 15	$ 1,768,500	Assume grout take of 1 sack (94 lbs) per 1 lineal foot
	Microsilica	11,790	SACKS	$ 8	$ 94,320	Assume microsilica content is 10% of cement
	Water Pressure Testing	500	Hrs	$ 160	$ 80,000	Estimated No of hours = 500 hrs
	Verification Borings	3,600	LF	$ 60	$ 216,000	Assume verification boring every 100 ft along curtain for total of 24 borings Assume average length of verification boring = 150 ft
				Subtotal	**$ 8,352,520**	
Spillway						
	Training Walls	830	CY	$ 500	$ 415,000	Estimated training wall surface = 2 x 5600 sq ft Assume 2-ft thick walls
	Stilling Basin Slab	2,400	CY	$ 500	$ 1,200,000	About 85 ft x 250 ft in area and 3 ft thick
	Stilling Basin Anchors	225	EA	$ 300	$ 67,500	Assume 225 rock dowels at 10 ft spacing and 15 ft deep
	Stilling Basin Drains	2,000	LF	$ 30	$ 60,000	Assume 100 drains (15 ft spacing) and 20 ft deep
	Excavation - Channel downstream of Spillway at El 2150	120,000	CY	$ 15	$ 1,800,000	About 120,000 cy excavation for downstream channel; 1/2 rock
	RCC Retaining Walls	1,230	LF	$ 1,500	$ 1,845,000	Assume avg 20 ft high walls , 15 ft wide, about 10 cy of RCC per lin ft of wall Assume RCC cost = $150/cy (50% cost increase for working on slope and formwork)
				Subtotal	**$ 5,387,500**	
Intake Structure						
	CLSM/Concrete Backfill Below Structure	300	CY	$ 100	$ 30,000	Included in dam and FC Station
	Reinforced Concrete	100	CY	$ 500	$ 50,000	approximately 100 cy
	Trash Rack	1	EA	$ 50,000	$ 50,000	
	Slide Gate including frame and actuator	1	EA	$ 250,000	$ 250,000	
				Subtotal	**$ 380,000**	
Diversion and Control of Water						
	Cofferdam/Diversion of Creek/Water contr	1	LS	$ 1,500,000	$ 1,500,000	Allowance ($500k smaller than for CFRF Dam because overtopping has lesser consequences)
				Subtotal	**$ 1,500,000**	
Totals						
				Subtotal	**$ 126,399,820**	
	Contingency including Miscellaneous Items (20%)				$ 25,279,960	
				Subtotal	**$ 151,679,780**	
				Total Facility Cost	**$ 151,680,000**	