Yakima River Basin Study

Bumping Lake Enlargement
Planning Design Summary Update

U.S. Bureau of Reclamation
Contract No. 08CA10677A ID/IQ, Task 4.8

Prepared by

HDR Engineering, Inc

U.S. Department of the Interior
Bureau of Reclamation
Pacific Northwest Region
Columbia-Cascades Area Office

State of Washington
Department of Ecology
Office of Columbia River

March 2011

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Note:
Drawings 1407-D-1 and 1407-D-2 were developed as a part of the 1985 Planning Design Summary for the Bumping Lake Enlargement Dam by Reclamation. Drawing 443-D-3 was developed as a part of the 1976 Bumping Lake Enlargement Joint Feasibility Report by Reclamation.

The basic information presented in this technical memorandum is reproduced from the 1985 Bumping Lake Enlargement Planning Design Summary report. Text that has been significantly changed from the 1985 report has been highlighted by underlining.

1.0 Introduction

(Note: This section has been updated to reflect the changes required to maintain a maximum water surface elevation of 3,490 feet.)

This document updates the Bumping Lake Enlargement Dam Feasibility Design Summary that was completed in April 1985 (and revised in July 1985) by the U.S. Bureau of Reclamation's Engineering and Research Center in Denver, Colorado. This update by HDR Engineering, Inc. was requested by Reclamation to reflect conditions under the proposed enlarged reservoir at a maximum operating water surface elevation of 3,490 feet above sea level. These drawings have been revised to reflect operation at surface elevation 3,490 feet and are attached as figures 1 and 2.

The proposed dam site is about 40 miles northwest of Yakima, Washington, on the Bumping River, about 4,500 feet downstream of the existing Bumping Lake Dam. The dam site is within Wenatchee National Forest in Yakima County, Washington. The Bumping River is a tributary of the Naches River and thereby the Yakima River, as shown on Reclamation drawing 1407-D-1 (included in the previous summary report and attached to this report). Drawings from the 1985 report have been included in this technical memorandum using the best copy available.

The dam would rise about 163 feet above streambed and impound an enlarged reservoir of 198,300 acre-feet at elevation 3,490 (top of active conservation capacity) with a surface area of approximately 3,200 acres. The dam and reservoir would provide carryover storage against possible shortages of irrigation water for project lands and would provide incidental flood-control benefits.

The team developed quantities and estimates for two alternative embankment designs, which are described in this summary. The basic difference between the two alternatives is the foundation treatment used to control seepage. In the original study, Alternative I, a zoned rockfill, was the preferred design for the reasons discussed in this summary and presented on drawing 1407-D-1 and 1407-D-2. Alternative II, a zoned earthfill, is very similar to a 1963 feasibility design embankment (see drawing 443-D-3 from the 1976 Joint Feasibility Report) and is also discussed in this design summary. Drawings from the 1985 reports (drawing numbers 1407-D-1 and 1407-D-2) and a drawing from the 1976 Bumping Lake Enlargement, Joint Feasibility Report (drawing number 443-D-3) have been attached to this updated report for completeness of information. These drawings have been revised to reflect the changes described by this update and are attached to this report as Figures 1, 2, and 3. The figures generally reflect the zoned earthfill embankment that was shown as Alternative II in the 1985 summary report.

2.0 Principal Features

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been included verbatim from that report with only minor editorial changes.)

The principal features of the revised Bumping Lake Enlargement dam consist of the following:

- A zoned earthfill dam with a concrete cutoff wall into bedrock in the foundation.
- An uncontrolled overflow crest spillway with chute and stilling basin on the left abutment.
- An outlet works tunnel and gate chamber in the left abutment with an intake structure at the entrance and a stilling basin at the exit.

3.0 Hydrology

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been included verbatim from that report with only minor editorial changes.)

3.1 Inflow Design Flood

Three probable maximum floods (PMFs) were approved for use in feasibility designs and estimates by memorandum dated October 15, 1984 from the Chief, Division of Planning Technical Services, to the Regional Director, Boise, Idaho. Table 1 shows the characteristics of these three floods.

Table 1. Probable Maximum Floods

Event	Peak Discharge (ft^3/s)	Volume (acre-feet)
Early cold season general rain-on-snow with frozen ground	43,620	110,540 (7-day)
Spring-early summer general rain with spring snowmelt	34,650	101,820 (7-day)
Summer thunderstorm with 100-year antecedent storm	73,430	34,160 (2.5-day)

The flood routing criteria require that the reservoir be assumed full to the top of active conservation capacity, elevation 3,490.0, at the beginning of each PMF event. Both the spillway and the outlet works are assumed to be available to pass the PMF.

The inflow design flood (IDF) for Bumping Lake Enlargement dam is equivalent to the PMF, which produces the highest reservoir water-surface elevation. Based on the presently proposed capacities for the spillway and the outlet works, the current IDF is the cold-season event with rain on snow and frozen ground, which has a peak of 43,620 cubic feet per second (ft^3/s) and a 7-day volume of 110,540 acre-feet. (The previous IDF used for both the 1955 and 1963 feasibility designs had a peak of only 16,000 ft^3/s and a 15-day volume of 100,000 acre-feet.)

These floods should be carefully reviewed in the future stages of the design process to ensure that they are appropriate for the revised reservoir configuration.

3.2 Frequency Floods

A 100-year frequency flood was developed by Reclamation in 1985 for use as an antecedent flood for the summer thunderstorm PMF. This flood has a peak of 17,545 ft^3/s and a 24-hour volume of approximately 5,400 acre-feet.

3.3 Diversion Floods

Since the Bumping Lake Enlargement dam would be downstream from the existing Bumping Lake Dam, diversion during construction would be affected by operation of the existing spillway and outlet works. It is assumed that Bumping Lake could be drawn down to a level that could handle anticipated upstream floods without overtopping the existing spillway crest. The resulting outflow would then be limited to the safe discharge capacity of the outlet works, estimated to be 800 ft^3/s (based on the capacity of the concrete-lined downstream channel).

No diversion flood hydrographs are required for the planning design estimate.

3.4 Relocation of Meteorological Stations

Two existing hydrometeorological stations will require relocation: Bumping Lake Snow Course No. 21C8 and Bumping Lake Climate Station No. 450969 because they fall within the enlarged reservoir footprint.

4.0 Reservoir

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been updated to reflect the changes required to maintain a maximum water surface elevation of 3,490 feet.)

As proposed, Bumping Lake would be enlarged to a total active capacity of 198,300 acre feet at elevation 3,490, compared to the present reservoir capacity of only 33,700 acre-feet at elevation 3,425. The existing dam would be breached following construction to allow full use of the existing pool. The enlarged reservoir would inundate up to 3,200 acres of land, of which 1,300 acres are in the existing reservoir. The reservoir would extend approximately 5 miles upstream and create 14 miles of shoreline. All required right-of-way would be on Reclamation owned land within the Wenatchee National Forest.

Facilities that would be inundated by the proposed reservoir include a network of Forest Service roads and trails, a summer cabin and resort development built on land leased from the Forest Service, and a public campground and boat-launching complex serving the existing reservoir area.

5.0 Geology

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been included verbatim from that report with only minor editorial changes.)

5.1 Introduction

The site of the Bumping Lake Enlargement dam is on the Bumping River 10 miles upstream of the confluence of Bumping and American Rivers, 4,500 feet downstream from the present Bumping Lake Dam. The geology information in this section is based on the data contained in the references listed in Section XIII: the Dam Site and Structure Review Team Report, dated January 1982; Geologic Report, Bumping Lake Enlargement Dam Site, dated October 1984; Memorandum to Chief, Division of Dam and Waterway Design, from Head, Seismotectonic Section, dated November 6, 1984; Preliminary Geologic Report on the Bumping Lake Enlargement Dam Site, dated June 1953; Reconnaissance Geologic Report on the Bumping Lake Enlargement Dam Site, dated July 1952; and Design Request Data for the Feasibility Design of Bumping Lake Enlargement Dam, dated May 1963. The geologic topics discussed in the following paragraphs are described and illustrated in greater detail in the above-referenced documents.

5.2 Regional Geology

The Bumping Lake Enlargement dam site is on the eastern flank of the northern Cascade Range in the Cascade Range physiographic province. The oldest known rocks are a pre-Tertiary basement complex of sedimentary and igneous rocks. These rocks were slightly metamorphosed and folded into a series of north-trending folds. The basement complex was then overlain by tuff, breccia, and andesitic flows that form much of the bedrock in the dam site area.

Near the end of the Eocene Age, the formations were uplifted and gently folded in a broad anticlinorium parallel to and a few miles east of the Cascade Range. These older rocks were then eroded somewhat, prior to the deposition of sandstone volcaniclastics, basalt, and tuff of the Fifes Peak Formation of Lower Miocene Age, and were then intruded by plugs, dikes, and sills of andesite.

During the Miocene Age, the area was again uplifted and eroded. During the Pliocene, when the present drainage was partially developed, andesite flows were deposited in several of the drainage systems.

Subsequent erosion and deposition of thick glacial materials followed. Bumping River Valley is a typical erosional valley that has been modified by the action of alpine glaciers. The glacial deposits consist mostly of an unsorted mixture of boulders, cobbles, gravel, and sand with some silt. Exposures of hard sound volcanic flows occur at various places along the valley walls, however.

5.3 Site Investigations

In 1940, the U.S. Army Corps of Engineers drilled two foundation exploration holes in the valley section of the dam site to depths of 100 and 102 feet. Reclamation drilled one hole to a depth of 200 feet in the autumn of 1951 and eight holes in the summer and autumn of 1952. One test pit was dug at the site in 1951 and six pits were dug in prospective borrow areas in 1952. Since 1953, the geologic data identification of various construction borrow sites has been reviewed and updated. In 1963, R. 0. Birch explored the area within the reservoir basin for pervious and impervious materials. In 1973, 0. L. Tengesdal and G. I. Haskett reviewed the seepage loss estimates for the glacial materials at the dam site. In 1976, B. H. Carter conducted a reconnaissance examination for riprap sources.

In 1983, the Dam site and structure review team recommended additional work to include review of all existing geologic data, explorations to determine depth and characteristics of the deep valley fill, and explorations at the site of the spillway stilling basin. Due to lack of funding, all recommended explorations were dropped except a seismic refraction survey across the valley, which was performed to determine the depth of the alluvial material in the valley section of the dam site.

5.4 Dam Site Geology

At the dam site, the elevation of the valley floor is about elevation 3,350 feet near the left abutment and about 3,425 feet near the right abutment. From the valley floor, the right valley wall rises on a slope near 30 degrees to approximately elevation 5,500. The left valley wall rises on slopes from vertical to 55 degrees to approximately elevation 3,450, continues at a slope of about 14 degrees to elevation 3,475, and then steepens to a slope of about 26 degrees to elevation 3,600 where the slope begins to descend into a gully. After small gullies near elevation 3,585 to 3,550, the slope continues to rise at about 5 degrees to elevation 3,800 and then continues to elevation 6,000 at 16 degrees.

No surficial deposits exist on the left abutment below elevation 3,450. In this reach, a nearly vertical cliff is too steep to allow soil or rock debris to accumulate. Above elevation 3,450, the abutment is mantled with glacial till to about elevation 3,475, and with the slope wash and talus from elevation 3,475 to elevation 3,575. These surface deposits are estimated to be less than 15 feet thick.

The right abutment is covered with slope wash and talus accumulations of silt, clay, sand, and rock fragments. The major bedrock formation that underlies the dam site is the Ohanapecosh formation of the Eocene age, consisting mostly of tuff, breccia, and andesitic flows. Intruding the Ohanapecosh formation are dikes, plugs, stocks, and sills of rhyolite and rhyodacite composition. A large prominent rhyodacite intrusive is found in the right abutment area.

Hard, competent andesite is exposed in a high, steep slope that extends about 250 feet above the river on the left abutment. Between outcrops of andesite rock are thin deposits of talus and glacial till. This outcrop of andesite flow rock is part of a large (1-mile-square) erosional remnant of an intracanyon andesite flow that at one time extended for many miles in the Bumping River Valley. The thickness of the andesite flow is not exactly known but extends in depth to at least elevation 3,324. The relationship of this andesite flow to the underlying Ohanapecosh formation is not known in relation to the depth of the flow and the contact relationship with the underlying formation.

Recognition of the left abutment rock as the remnant of an intra-canyon flow neutralizes an earlier concept that this rock may be part of a large block slide. This also nullifies the interpretation that a fault exists along the contact between the andesite of the left abutment and the rhyodacite of the right abutment.

In the left abutment, there may be older alluvial materials beneath the andesite flows that should be identified in any further studies at the dam site. The bedrock on the right abutment is known as a result of three holes drilled in 1952. Near the top of the proposed abutment, the talus in DH-6 is about 12 feet deep. Downslope this talus interfingers with the underlying glacial till, and at a point about 250 feet left these materials extend to a depth of 56 feet. About 500 feet left of OH-6, the glacial till was found at a depth of 136.2 feet. The 1953 geologic report identifies the bedrock in these three drill holes as porphyritic dacite. From these descriptions, it is probable that the right abutment bedrock is part of the rhyodacite porphyry stock or sill rather than the major bedrock unit, the Ohanapecosh Formation.

The logs of seven drill holes drilled in 1952 show that up to a depth of over 200 feet, the floor of the valley section consists of differing layers of silt, sand, gravel, cobbles, and boulders with the finer-grained materials occurring in greater thicknesses at depth. The continuity of any of these layers or lenses from drill hole to drill hole is not determined at this level of-exploration. The "core recovery" of the glacial materials in the drill holes was low; thus, the determination of specific engineering characteristics of the till is rather difficult. It appears, however, that there may be layers of fine, clean sand up to several tens of feet thick.

In August 1984, a seismic refraction survey was conducted across the valley floor. The survey alignment is about 100 to 200 feet downstream from the dam axis. The survey reflected distinct velocity change at a depth of about 250 feet, which can be construed as the top of bedrock (the interface between glacial till having a velocity of about 5,600 ft/s and bedrock having a velocity between 10,200 and 12,800 ft/s). The survey gives a 1,300-foot-long profile from DH-8 (where bedrock was found on the right abutment at a depth of 136.2 feet) across the valley floor to within 800 feet of the left abutment. The survey was not continued closer to the abutment because of its orientation.

The spillway and outlet works, in the present design concept, will be founded on the competent andesite flow of the left abutment, except for the stilling basins, which may be founded on glacial till. Even though the geologic section of the spillway and outlet works suggests a steep dropoff of the bedrock into the stilling basin area, the structure alignments should not be changed until explorations are conducted to ascertain the geology in this area.

Across the valley where earlier drilling bottomed in alluvial deposits, groundwater was encountered within a narrow band between elevations 3,324 and 3,332. Drill holes in rock within the abutments reflected water levels above elevation 3,350. Most water tests performed during the field exploration in

1952 suggested that glacial deposits were permeable. Considering that the depth of permeable deposits may approach 250 feet beneath the prospective dam, a large cross-sectional area of seepage may exist.

5.5 Reservoir Geology

Bumping Lake Enlargement reservoir is in a flat-floored steep-walled glaciated valley with a stream gradient of approximately 25 feet per mile. This is a typical erosional valley that has been greatly modified by the action of alpine glaciers. Glacial lakes occupy depressions in the cirques at the head of many tributary streams. Thick deposits of glacial till and alluvium extend for several miles along the valley floor. Since glacial times, the river has cut its flood channel about 75 feet into the glacial deposits, leaving wide alluvial terraces between the floodplain and the rock walls of the valley.

The reservoir site is rather narrow and flanked on both sides by steep canyon walls that rise more than 3,000 feet above the valley floor. The valley floor ranges between 2,500 and 4,500 feet wide. The floor has few exposures of bedrock, but outcrops of andesite, basalt, dacite, tuff, and granite have been noted on the valley walls. Flows appear to be horizontal, with some more than 100 feet thick. Exposures appear to be only lightly weathered, intensely jointed, and hard.

Nearly all of the valley floor contains alluvial and glacial deposits of varying thicknesses. Thickness at the dam site has been estimated to be 250 feet. Material exposed in cuts range from boulders several feet wide to cobbles, gravel, sand, silt, rock flour, and clay. Layers and lenses of stratified materials indicate that glacial melt waters sorted portions of the deposits dropped by the ice, leading to some degree of stratification.

5.6 Seismicity and Geologic Hazards

No site-specific seismotectonic study has been made at this site. However, a seismotectonic report for the existing Bumping Lake Dam 4,500 feet upstream from the enlargement site is currently in progress.

The conclusions in this section are based on analysis of readily available literature, previous limited-scope USBR reports, and a preliminary review of moderate-scale photography.

Existing geologic mapping has identified no significant faults in the vicinity of the proposed dam. However, a lineament named the Bumping Lake lineament has been identified from analysis of Landsat imagery and U-2 photography. The lineament is at least 25 km long, and corresponds topographically to the relatively straight, northeast-southwest trend of the Bumping River Valley. It has been suggested that the lineament may be fault related because older (pre-Miocene) rock units on opposite sides of the river appear to change strike and dip. This feature is presently being investigated as part of the seismotectonic study for the existing Bumping Lake Dam. The Bumping Lake lineament is not considered a potential earthquake source in this preliminary earthquake analysis.

Surface fault rupture is not considered a potential hazard to the new dam site. Although historic large earthquakes in Washington have been accompanied by surface fault rupture, there are no known late Quaternary faults in the vicinity of the dam site.

For planning design purposes, four potential earthquake sources should be considered for the proposed Bumping Lake Enlargement dam. Table 2 lists the maximum credible earthquakes (MCEs) and their estimated hypocentral distances.

Table 2. Estimated Preliminary Maximum Credible Earthquakes for Bumping Lake Enlargement Dam

Earthquake	MCE	Hypocentral
Subduction Zone	8.3[1]	105[2]
Puget Sound	7.1	120
Middle Cascade Province	7.0-7.5	20
Toppenish Ridge	7.4	90[3]

[1]The MCE 8.3 event is in the moment magnitude scale (Mw), and cannot be readily converted to other magnitude scales such as surface magnitude (Ms).In general, Ms will not exceed Mw for a given magnitude.
[2]The potential for, the magnitude, and especially the distance of the subduction zone event are based on theoretical considerations that involve many uncertainties. Future studies may resolve some of the uncertainties.
[3]Assumed focal depth for this event is 10 km. Epicentral distance is approximately 90 km.

The Bumping Lake Enlargement dam site is near the crest of the Cascade Range and about 23 miles east of Mt. Rainier, one of the largest volcanic cones in the range. Some observers have suggested that the potential for volcanic activity at Mt. Rainier may be increasing similar to that noted in the mid-1800s when one eruption at Mt. Rainier spread pumice and ash fall up to several inches thick at the Bumping Lake site. Volcanic activity also relates to increased seismic activity. Eruptions at Mt. Rainier would have limited influence on the Bumping Lake Enlargement dam site because it is situated on the east slope of the Cascade Range. The west slope drainage would expect to receive the major thrust of the volcanic activity. However, significant ashfall accumulations may be expected at the site.

5.7 Engineering Geology Considerations

The valley section of the dam site has been estimated to contain up to 250 feet of alluvial and glacial deposits overlying a dacite or andesite rock. The deposits consist of sand, gravel, cobbles, with scattered stringers of silt and clay. Lenses of sand and silt can be expected. The materials for the most part can be expected to be water sorted and possess a high permeability. In other words, this dam would be situated on a foundation of coarse-grained pervious material.

Design features for Alternative I include a concrete cutoff wall into rock and installation of relief wells along the downstream toe of the embankment. Seepage through the abutments is also an important consideration for which foundation grouting would be planned. Due to the location and depth of the concrete cutoff wall, curtain grouting would be required in the abutment rock adjacent to the cutoff wall to develop a compatible grout curtain. Special attention would be required to grout the contact between the rock and concrete cutoff wall. At this stage, no analysis of the permeability of the rock has been made.

Foundations consisting of cohesionless, low-density sand and/or silt are potentially liquefiable, which would require investigating the foundation for the amount and extent of liquefiable materials and design features to safeguard the dam. Excavation of the surface deposits of 10 to 20 feet would be done beneath the structure at this stage of the design to determine the possible presence of low-density surface materials.

All surficial deposits can be excavated by common methods. Temporary slopes in the alluvium of the valley section should be stable on 1:1 for a cut up to 10 feet, provided the banks are not saturated. For cuts deeper than 10 feet, slopes may require flattening to at least 2:1 unless intermediate benches are planned. Water control may be necessary during periods of heavy rainfall or high runoff. However, sumps, trenching and/or pipe drains should adequately handle the water.

Up to 15 feet of talus and slope wash can be expected in scattered areas of the abutments and along the proposed spillway alignment. Cuts in these deposits should be stable between 1:1 and 2:1. Cuts in rock should be stable at 1/4:1. At this stage of design it would be appropriate to bench all cuts over 30 feet to catch isolated rockfalls. Draped fencing should cover all cuts over 15 feet. Permanent cuts that will remain exposed for an indefinite period may require bolting. Regular examination and periodic scaling would be required on all exposed cuts in rock.

The spillway and the outlet works tunnel are expected to be in the andesite of the left abutment. Rock bolts may be more economically feasible than steel sets, depending on the diameter of the tunnel.

The spillway and outlet works stilling basins are currently proposed to be situated on glacial till described as reworked sand and gravel with silt filling the void spaces in the upper 2 or 3 feet of the deposit. Settling should not be a problem if excavation is done to a depth of 20 feet or more. Consideration for erosion control outside the stilling basins would be required in the design.

6.0 Construction Materials

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been included verbatim from that report with only minor editorial changes.)

6.1 Earthfill Materials

The embankment earthfill materials required for the Zone 1 core, filters, chimney drain, and blanket drain are available in borrow area C and from required excavation (borrow areas are shown in drawings attached to the 1986 planning study). Borrow area C is located on Deep Creek, on the right side of the valley about 2.5 miles upstream from the dam site. Explorations completed in 1963 indicated that

50,000,000 yd^3 of glacial materials lie within 50 feet of the surface above the existing reservoir and below the enlarged reservoir surface (C1) and another 20,000,000 yd^3 would be available if the existing reservoir was drawn down (C2). Borrow area D, a small hill near borrow area C, could produce about 18,000,000 yd^3 of similar material, but its use is not recommended because the hill's scarp would remain exposed above the reservoir water. In particular areas, silty sand and gravel may satisfy the physical properties for impervious fill. Selective borrowing and processing of materials would be required to produce the impervious and pervious gradations required, specific gradation band materials, and to remove oversize material.

The valley floor between the dam site and the existing Bumping Lake Dam contains sand and gravel glacial materials similar to borrow area C. If the dam contains a positive cutoff to bedrock beneath the dam, it may be possible to utilize these materials to construct the dam instead of those in borrow area C. The haul distance would thereby be shortened from 3 miles to about one-half mile for much of the required embankment material. Impervious materials in this valley area are not considered to be in sufficient quantities to use as a borrow source for zone 1 core materials that require use of borrow area C.

The groundwater table in the vicinity of the dam was 22 feet below the surface near the river and up to 87 feet deep further from the river when exploration holes were drilled in 1952. The groundwater table probably rises toward the surface closer to Bumping Lake Dam due to seepage from Bumping Lake. This situation would need to be investigated further to determine whether the area between the dam and dam site could be used as a borrow source. Alternative II would have to use the materials from borrow area C in order to leave the natural blanket materials in place beneath the reservoir between the dam site and the dam.

6.2 Rockfill Materials

Rockfill materials, defined as subangular or rounded fragments such as coarse gravel, cobbles, and boulders, are available for the upstream and downstream zone 2 shells for either embankment design alternative. Borrow area C with the required material separation would be able to produce such rockfill materials.

Four samples obtained from borrow area C in 1962 contained 51 to 72 percent larger than the U.S. No. 4 sieve. Oroville Dam zone 3 shell material contained 0 to 25 percent passing the U.S. No. 4 sieve and 100 percent smaller than 24 inches; a similar gradation could be produced from borrow area C. It can be assumed that the area between the existing dam and the dam site would also produce the required shell gradation.

6.3 Riprap

Based on a geologic reconnaissance made in August 1976, riprap for the upstream slope protection and the stilling basins is available from a large exposure of rock and talus about one-half mile upstream of the left abutment at or just above the proposed pool level. The talus contains an estimated 200,000 yd^3 of suitable andesite. The andesite ledge of rock above the talus could be quarried for any additional riprap. Additional talus slopes that may provide suitable rock are located along the left bank of Deep Creek near borrow area C. The spillway and outlet works excavations would also generate some andesite riprap. Particular riprap sizes and grades would depend on the joint structure and inherent characteristics of the rock. This evaluation would require further study prior to final design. The oversized boulders and cobbles from borrow area C could also be used as riprap.

6.4 Concrete Aggregate

Sand and gravel for concrete aggregate could be obtained from river sandbars near Suicide Point, about 2.5 miles downstream of the dam site, or from either borrow area C or the area between the dam and the dam site. Separation may not be necessary for the Suicide Point material, but would be required for the latter two borrow areas to obtain the proper gradations.

6.5 Miscellaneous

All materials from required excavation should be usable within the embankment as zone 1, zone 2, or as a miscellaneous fill zone within the downstream shell zone.

7.0 Embankment Design Considerations

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been included verbadium from that report with only minor editorial changes.)

7.1 Alignment

The dam alignment is the same one considered for more than 50 years. At this site, the valley constricts somewhat compared to other locations along the river. Hard, competent andesite forms the left abutment and extends about 250 feet above the river, and a competent porphyritic dacite forms the right abutment. This alignment utilizes one of two low saddles on the left abutment for the spillway. The outlet works tunnel and gate chamber would be located in this same andesite flow on the left abutment, except at the stilling basin.

A total of 11 drill holes were completed at the site in 1940 and 1952 for evaluation of depth-to-bedrock, foundation permeability, depth-to-groundwater table, etc. One seismic refraction line was run in 1984 to confirm the previous estimates of depth to bedrock closer to the river. Additional exploration at the dam site would be required before final design to better determine the engineering characteristics of the bedrock and the glacial materials, especially the in-place surficial deposit density, which influences seismic concerns at the site.

7.2 Diversion and Unwatering

It will be necessary to release water through the dam site throughout the year. The existing Bumping Lake Dam should be able to control flows through the dam site during construction. River flows would be diverted around the embankment construction through the outlet works in the left abutment. Small upstream and downstream cofferdams would be required to dewater the dam site.

The groundwater table depth indicated by the drill holes in 1952 was 22 feet adjacent to the river and deeper toward the right abutment away from the river. Additional exploration required before final design would indicate whether the groundwater depths have changed. Alternative II uses a 30-foot-deep cutoff trench that would require dewatering along the trench with wells and wellpoints. Alternative I uses a 20-foot deep excavation beneath the zone 1 core and 10-foot-deep excavations beneath the shells. These excavations may not require dewatering if the groundwater table has remained unchanged.

7.3 Foundation

The dam foundation consists of rock abutments at each end and a 2,500-foot-wide valley floor of glacial deposits, layers of silt, sand, gravel, cobbles, and boulders, up to 250 feet in total thickness. Control of seepage through these glacial materials is of concern in the design of the dam. The permeability of these materials was measured in the 1952 field investigation of the dam site, indicating permeability values from 2,000 to 85,000 feet per year with an estimated average of 35,000 feet per year. A study for the feasibility design indicated total dam underseepage would be 80 to 100 ft^3/s for the upstream blanket, using a 30-foot-deep partial cutoff trench, blanket drain, and downstream relief wells seepage control system. This is the same seepage control provided by Alternative II as discussed in this design summary, except for an added toe drain in Alternative II.

The alternative to the above seepage control is a positive impermeable cutoff into bedrock beneath the dam. An open trench excavation to bedrock was not considered practical due to the groundwater table, the permeability of the glacial deposits, and the depth to bedrock. The best solution appears to be a concrete cutoff wall such as those constructed at La Villita Dam in Mexico and Manicougan 3 Dam in Canada. The most common location is beneath the central core of the dam, though some have been built just upstream of the embankment and were tied to the core by a blanket of core material beneath the upstream shell. The central cutoff wall location minimizes shear stresses from the embankment through the wall, but does experience loading from the embankment, producing negative shear stresses along the sides of the wall due to consolidation of the foundation.

Various design features can be used to minimize this loading of the wall, such as a zone of compressible material just above the wall. This compressible zone (not shown on the drawing) could be achieved by either constructing a zone of bentonitic material, constructing a zone of loose material, or constructing an overly wet zone of core material. Hydraulic fracturing across this zone would have to be prevented. This compressible zone would arch the loading from the embankment onto areas of the foundation further away from the wall, which should reduce the negative friction loading on the wall.

The central cutoff wall would be built before the embankment is constructed. For Alternative I, a working pad would be placed consisting of 20 feet of zone 1 in the excavation plus another 20 feet of zone 1 core. The trench for the wall would be excavated through this pad, creating a top-of-cutoff wall 40 feet above the foundation contact. The trench would be 2 to 3 feet wide supported by bentonite slurry until the concrete is tremied in and would penetrate 2 or 3 feet into bedrock. The trench would be excavated in panel or interlocking pile units depending on ease of excavation. The pile units would be advanced by percussion drilling used for greater depths and tougher foundation materials. The panel units would be advanced with a mechanical bucket. It may be possible to excavate all of the trench in panel units, which are faster and more economical. Some percussion drilling units should be assumed for the design and estimate, using a depth of 170 feet as the change from panels down to piles (170 feet was used at Manicougan 3 Dam).

Some foundation-related features such as the upstream blanket, the blanket drain, and the toe drain are discussed later. Both alternatives include downstream relief wells to control the foundation phreatic surface at the embankment toe. For Alternative I, relief wells are considered necessary in case the concrete cutoff wall leaks and produces a high foundation phreatic surface downstream. A berm to elevation 3,350 is indicated in the river area to provide a working pad for the relief wells. The relief wells would penetrate to bedrock or to a maximum depth of 200 feet with an average depth of 150 feet. They would be located about 100 feet apart (this assumes the Alternative I cutoff wall fails and the wells are therefore necessary). These design details would be refined during final design.

One foundation aspect that will require field investigation before final design is the potential for liquefaction or excessive deformation caused by earthquake loading. Three source areas, the Middle Cascade Province, Toppenish Ridge, and the Subduction Zone, are capable of producing maximum credible earthquakes that could cause liquefaction at the dam site depending on foundation conditions. It appears that layers of clean, fine sand up to tens of feet thick may exist in the foundation. The susceptibility of these layers to liquefaction would need to be determined for final design. Although liquefaction was not assumed to be a problem for the planning design, the top 20 feet of the foundation was excavated beneath the zone 1 core and the top 10 feet of the foundation was removed beneath the shells. This should remove the least dense materials beneath the dam. Other methods of improving the foundation density, such as compaction grouting and dynamic compaction, could be used, but were not included in the estimate.

The foundation glacial materials appear to contain layers of high and low permeability adjacent to one another. Assuming the low permeability layers are the silts and silty sands, the possibility of the finer materials piping out through any open gravels may exist. Control of this situation can be difficult if it exists. Alternative I with its concrete cutoff wall would control any potential piping in the foundation up to the wall, provided the wall did not fail due to shear or compression loading. Alternative II would not control piping through the foundation to the same degree.

Foundation excavation to remove surface materials beneath the shells and the core should be able to be done in the dry. Field investigation for final design should check this situation. The planning design assumes that some dewatering with wells and well points will be required.

The rock abutments at each end of the dam embankment would need grouting. The left abutment had very low water losses in packer tests in one drill hole in 1952. Water losses in the right abutment were much higher. Grout takes of one sack per foot of hole were assumed for the 1963 feasibility design and were assumed again for this planning design. The feasibility design used a grout cap in both abutments for grouting. This was deleted in the planning design because the rock is considered adequate if blanket grouting is used to tighten the surface rock. Blanket grouting adjacent to the grout nipples was therefore added to the design.

Abutment grouting beneath the Alternative I cutoff wall is considered necessary for a certain portion at the ends of the wall as shown on drawing 1407-D-1 and 1407-D-2. These holes would be drilled through the alluvium, and the rock would be grouted prior to construction of the concrete wall. Contact grouting would be performed at the bottom of the wall piles and panels.

Instrumentation would be required in the foundation at a minimum of two sections to provide information on the phreatic surface in the foundation. Alternative II would require more instrumentation to properly monitor foundation seepage effects. Alternative I would not experience the same 80 to 100 ft^3/s flow through the foundation estimated for Alternative II. Phreatic surface data along the line of relief wells at the downstream toe would be required for either alternative, but is considered more critical for Alternative I because of the underseepage anticipated. Information is needed on the integrity of the concrete cutoff wall, including horizontal deflection and vertical compression of the wall at various locations. This concern would need to be closely examined in final design of the monitoring system.

Some of the dam site foundation concerns that need to be resolved in the field explorations have already been mentioned in the preceding discussion. These include additional permeability testing of the glacial deposits at the dam site, evaluation of the groundwater table, and determination of the depth to bedrock between the left abutment and the left end of the seismic refraction line surveyed in 1984. This information would be required to proceed with final design.

7.4 Dam Structure

Two separate embankment dam designs are discussed in the following section. The recommended design, a rockfill dam with concrete cutoff wall in the foundation (Alternative I), is presented on drawing 1407-D-1 and 1407-D-2 (see revised drawings for the reduced-capacity reservoir attached to this report as figures 1, 2, and 3). The second design, an earthfill dam with upstream blanket (Alternative II), is almost the same design contained in the feasibility designs of 1955 and 1963. Both designs incorporate many of the same features such as blanket and chimney drains, filters, and downstream relief wells. The discussion below covers the various embankment design details and compares the two designs where necessary.

The principal reason for recommending the rockfill dam with concrete cutoff wall for the planning design involves serious concern over the seepage expected to pass beneath the earthfill dam of the feasibility design. The concrete cutoff wall greatly improves the ability to control seepage through the foundation and would, therefore, be highly desirable. Various factors such as the width, depth, and variability of the glacial deposits in the foundation, <u>the 143 feet of reservoir head to be controlled, and the 198,300 acre-feet of reservoir</u> are believed to be much better controlled by the Alternative I design. Another factor was the potential annual loss of up to 70,000 acre-feet of reservoir water through underseepage with the Alternative II design. A concrete cutoff wall approximately 500,000 cubic-feet in volume is expensive, but the wall would allow other changes in the design that would help offset its cost.

Alternative II uses a wide core tied to an upstream blanket to lengthen the seepage path and thereby reduce the amount of underseepage. Since the concrete cutoff wall would significantly, if not totally, reduce the amount of underseepage, the upstream blanket would not be needed with Alternative I. Also, the width of the core can be reduced for Alternative I to 0.5:1 slopes, upstream and downstream. Design analyses would be needed to check the core stresses with these slopes. Since the width of the Alternative I core is much narrower than the earthfill design and because the shell material should be very strong, the rockfill dam slopes can likewise be steepened. The outside slopes are thus reduced from 3:1 to 2.5:1 upstream and from a combination 2:1, 2.5:1, and 5:1 to a constant 2:1 slope downstream. The Alternative I volume is therefore about 1.6 million yd^3 less than the Alternative II design.

Another aspect of this change is the difference in the volumes of zone 1 and zone 2 materials in the dam, a reduction from 7.4 million yd^3 with Alternative II to 2.3 million yd^3 of zone 1 with Alternative I, and a corresponding increase from 3.9 million yd^3 to 7.7 million yd^3 of zone 2. Zone 2 material would require less processing than the zone 1 core material. Depending on the borrow area material gradations, pit-run material could be used for zone 2 in the dam.

One factor in the embankment and core slopes requiring more study is the earthquake loading concern. The rockfill and earthfill dam designs assume the foundation is adequately resistant to liquefaction and deformation induced by earthquake loading. If the field exploration for final design encounters a different situation, the dam and core slopes may need to be flattened.

The dam crest is 30 feet wide for both alternatives. The crest would be used only for dam maintenance and access, not for public traffic, which would require a wider crest. The crest would require guideposts and cable on 25-foot centers along the edges and 3 to 6 inches of gravel surfacing for vehicle traffic.

A transition upstream and a filter downstream of the core are required for both dam alternatives. Zone 1E is shown as the upstream transition material and zone 11 is shown as the core filter material downstream. The filter and transition gradations would need to be evaluated in final design. The 6-foot width of the filter is less than the usual 8- to 12-foot thickness based on ease of placement. The filter material would have to be processed in the borrow area and it is therefore desirable to minimize the volume of material while still requiring an adequate thickness. The transition is shown as 10 feet wide.

The same zone 1A filter is shown above and beneath the zone 3 blanket drain. The need to evaluate the different gradations involved would be required here also.

In addition to the above, the zone 1B chimney drain material and the zone 3 blanket drain material are probably sufficiently different to require a filter, named zone 1C.

The zone 13 chimney drain thickness was designed based on an assumed permeability of 1 foot per day for clean, washed concrete sand as the drain material. Calculations indicated a required drain thickness of less than 1 foot, which was increased to a 6-foot width for safety and for placement purposes.

The zone 3 blanket drain was designed based on an assumed permeability of 100,000 feet per day for screened gravel from 3/8 to 1.5 inches in size and on the assumption that the full 100 ft^3/s estimated underseepage for Alternative II flows into the drain. The designs of zone 3 for both alternatives were basically the same.

This may be overdesigned for Alternative I since the concrete cutoff wall should prevent seepage flow through the foundation, but if the wall was damaged by foundation or embankment consolidation or by earthquake loading, a blanket drain would be needed. The toe drain at the downstream end of the blanket drain contains a 36-inch-diameter perforated concrete pipe within the zone 3 gravel envelopes. At a 1 percent slope, this pipe should be able to carry 50 to 55 ft^3/s; this assumes about half the 100 ft^3/s total underseepage does not enter the blanket and toe drain system. The toe drain was included in both alternative designs.

Because of the earthquake loading concern and the lack of knowledge as to the density of the glacial foundation materials, the Alternative I design excavates 10 feet of surface foundation material beneath the shells and 20 feet of material beneath the zone 1 core. After 1 foot of stripping, much of the rest of the excavated material could probably be reused. The Alternative II design does not assume the same requirement for excavation of these materials because of its flatter slopes giving better stability under the earthquake loading condition.

Movement of zone 1 materials into the foundation at the contact surface between them is of concern. Both alternatives use a layer of zone 1 material with 3 to 5 percent bentonite (by weight) mixed in and placed against the foundation alluvium or bedrock to improve the resistance to piping erosion (not shown on drawing). Testing on this type of mixture for Sugar Pine Dam indicated no loss of shear strength. The Alternative II design would include a layer of this mixed zone 1 material beneath the upstream shell and for 50 feet beyond the embankment toe at the bottom of the upstream blanket. This should improve both the permeability and the erosion resistance at this critical location. This upstream layer would cause concern over its effect on the embankment's stability if the layer was of lower strength. Testing and further evaluation of this concept would be needed in final design of this alternative.

The Alternative I design, with the entire width of the core as the cutoff trench and with the concrete cutoff wall extending 40 feet up into the core, would experience high seepage gradients through the core over the top of the wall. Layers of the bentonitic zone material would be needed surrounding the concrete cutoff wall and extending both upstream and downstream from the cutoff wall over most of the width of the core (not shown on drawing). The purpose of these layers would be to significantly reduce the permeability of this core area and to improve the seepage gradient effect on the material.

As previously mentioned in the section on the foundation, a zone of compressible material would need to be constructed above the top of the concrete cutoff wall. The purpose of this material (not shown on the drawings), would be to arch the embankment load away from the top of the concrete wall and away from the material located adjacent to the wall. Different materials could accomplish the desired compressibility given the limited extent of the zone. A zone of highly bentonitic material, such as was used at Manicougan 3 Dam, would be both compressible and impermeable. A zone of loose, lightly compacted zone 1 would be compressible, but would be much more permeable. Zone 1 material placed 4 or 5 percent wet of optimum moisture would be similar to the lightly compacted zone 1 material. The first option is probably the best choice, but more study in final design would be required.

Alternative II contains an upstream blanket 10 feet thick, extending 500 feet upstream. It would be connected beneath the upstream zone 2 shell to the zone 1 core. Its purpose would be to lengthen the seepage path beneath the dam and, since the gradient is reduced, the seepage quantity is decreased. Alternative I does not contain an upstream blanket since the cutoff wall is more effective at reducing underseepage.

The material for the zone 2 shells can contain sizes from fines to 1- or 2-foot boulders, depending on the characteristics of the matrix the gradation produces. Placement layers of 1 to 2 feet thick should be possible. It may be possible to use pit-run material as zone 2, including material from the required foundation excavation. A miscellaneous zone within the downstream shell could be used for the less satisfactory material.

One aspect of the Alternative I design and its narrower core has to do with the impact of weather (rain and snow in particular) on construction of the embankment. A narrower core means less fill construction impact from rain or snow. The coarser zone 2 materials can be placed and compacted under rain or light snow conditions where the zone 1 core cannot, resulting in a longer construction season. The earthfill dam alternative, with its much wider core, would be much more affected by adverse weather, resulting in a shorter construction season.

Riprap is required for almost the entire upstream slope. The reservoir would have a gently curving fetch of about 5 miles, with the dam at the northeast end. A freeboard of 5.8 feet is provided above the maximum water surface at elevation 3,504.2. Due to the coarseness of the zone 2, no bedding is considered necessary beneath the riprap, but filter requirements would need to be checked.

Another aspect of the design involves the requirements for material separation in the borrow area. Alternative I and its narrower core would require less material separation to remove the plus 1- or 2-inch materials, which should reduce the cost compared to Alternative II.

8.0 Removal of Existing Dam

(Note: This section has been added and was not a part of previous study reports)

The existing dam at Bumping Reservoir would be inundated once the enlarged dam has been constructed. Initial discussions have begun surrounding the necessity, feasibility and practicality for removing the existing dam at Bumping Reservoir. These discussions have not been pursued formally and are ongoing. Recent general dialogue has suggested that simply breaching the existing dam once the new dam is built may be sufficient, but no determination has been made. There are a number of studies that would be required before such a decision including, the depth and quantity of sediment behind the dam, the presence of metals or other potential pollutants of concern in the sediment, downstream fisheries issues due to suspended sediment once the dam is breached, and an assessment of the cost of breaching or removing the dam. These studies are beyond the scope of this report, and should be included in future studies. (see also Section 11.2)

9.0 Spillway

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been updated to reflect the changes required to maintain a maximum water surface elevation of 3,490 feet.)

9.1 General

The spillway for the current planning design consists of a concrete overflow crest, open chute, and hydraulic jump stilling basin located on the left abutment of the dam. The spillway has the same alignment and similar profile as used in the 1963 feasibility design. The original design capacity was increased to handle the newly revised inflow design flood.

The left abutment was selected for the spillway for several reasons. The Bumping River flows along the left side of the valley at the dam site, therefore requiring a short outlet channel from both the spillway and the outlet works. Locating either structure on the right abutment would require a long outlet channel through a heavily wooded area, which would be opposed by the Forest Service and others. In addition, the topography of the left abutment is much better suited for a spillway than the topography of the right abutment. A long steep cut slope above any spillway on the right abutment would make construction and maintenance difficult due to snow and landslide hazards.

Although eliminated for the planning design, future consideration for locating the spillway and/or outlet works on the right abutment may be required after additional geologic investigations. The 1952 geologic report suggests that the stilling basins on the left abutment may be founded on glacial till, rather than on bedrock. Additional investigations would be required to verify this and to evaluate the need for design changes.

9.2 Hydraulic Design

The spillway selected for the planning design is similar to the 1953 feasibility design and consists of an uncontrolled overflow crest with a concrete chute and a hydraulic jump stilling basin. Alternative spillway types studied previously (for the 1955 and 1963 feasibility designs) include an unlined rock-cut spillway through a left abutment saddle and a tunnel spillway having either a gated or a side-channel entrance. The chute spillway was selected based on simplicity, economy, and energy dissipation capability.

The inflow design flood was routed through various spillway crest lengths, and preliminary cost estimates were developed to select an optimum combination of spillway width and dam height. Based on these studies, a 90-foot crest length was selected. The design spillway capacity of 17,562 ft^3/s, in combination with an outlet works capacity of 4,697 ft^3/s and a flood surcharge of 43,400 acre-feet, would be capable of accommodating the inflow design flood with a maximum water surface elevation of 3,490.9. Using the spillway and flood surcharge alone, the maximum water surface would rise 3.4 feet above the spillway crest, to elevation 3,493.4. This indicates the importance of the outlet works to the flood-routing results. However, both the spillway and the dam would be capable of handling this extreme condition as well.

The spillway inlet structure would consist of an approach apron at elevation 3,487.0, an ogee-shaped crest at elevation 3,490.0, and counterforted walls through the dam. The spillway crest would have a vertical upstream face and a design head equal to three-fourths of the maximum head. The resulting maximum discharge coefficient is 3.64.

The spillway chute would be about 900 feet long, 90 feet wide, and have a vertical drop of approximately 177 feet (elevation 3,485 to elevation 3,308). The profile would consist of three 100-foot-long vertical curves and would generally parallel the original ground surface, with a foundation on andesite bedrock. The walls would vary from 10 to 8 feet high, and were designed for adequate freeboard for the design discharge of 7,800 ft^3/s (assuming maximum losses), but would be capable of accommodating a 50 percent surcharge without overtopping during passage of the probable maximum flood through the spillway alone. No problems with cavitation in the chute would be expected for the range of discharges.

The spillway stilling basin was designed to produce a hydraulic jump for all discharges (assuming minimum losses) based on the corresponding tailwater conditions. A tailwater curve was prepared in 1953 and was provided in the 1953 design data. Section 11, located 1,100 feet downstream from the proposed dam axis and having a thalweg elevation of 3,335 feet, was selected to represent the tailwater conditions at the spillway and outlet works stilling basins. A degradation value of 1.5 feet for design was assumed in 1963, and was again assumed for this study.

For the spillway design discharge of 7,800 ft^3/s, the total downstream discharge with the outlet works operating would be approximately 12,500 ft^3/s and the resulting tailwater (adjusted for degradation) would be at elevation 3,340 or slightly less. The 90-foot-wide basin would require a floor elevation of

3,308.0 feet to accommodate the full conjugate depth for the design discharge and to prevent sweepout for higher discharges up to the extreme 50 percent surcharge condition (based on 80 percent of the corresponding conjugate depth). A length of 132 feet was selected for both the spillway and the outlet works stilling basins, based on the design conjugate depths. The tops of the basin walls were assumed at elevation 3,350.0 to provide adequate freeboard for the maximum tailwater condition (without degradation).

A combined outlet channel for both the spillway and the outlet works would be required to carry discharges to the Bumping River. A bottom width of 150 feet at elevation 3,335 would limit flow velocities to under 9 feet per second. Riprap and bedding would be provided for channel protection.

9.3 Structural Design

Design curves based on the working stress design method were used to proportion structural elements and to estimate concrete thicknesses. The inlet structure walls include counterforts, which may serve as cutoffs in the impervious embankment core to minimize potential seepage. All other walls were designed as cantilevers having a monolithic floor slab, including heels.

The inlet structure and most of the chute would be founded on a competent andesite bedrock. The stilling basin and the downstream end of the chute (below minimum tailwater) would be founded on glacial till, according to the 1952 geologic report.

To minimize the potential for differential settlement or undercutting of the stilling basin, the following features were included in the planning design:

- The spillway and outlet works stilling basin floors are combined to form a single raft foundation.
- The maximum total load on the glacial till foundation (static and dynamic) is less than the existing load due to overburden.
- The glacial till foundation is entirely below minimum tailwater and would be protected from freeze-thaw conditions.
- Riprap in the outlet channel and a concrete cutoff key beneath the basins would prevent scour and undercutting of the structure for the maximum anticipated flows.
- A series of steel H-piles on 8-foot centers beneath the basins were designed to carry loads in excels of those required for an average bearing pressure of 1 ton per foot on the glacial till.
- Waterstopped control joints would allow some rotation at the joints in the chutes and stilling basins without differential movement and detrimental seepage.

Chain link fence would be required along the spillway walls for public safety. A log boom would be provided upstream from the spillway to collect driftwood and debris. A two-lane bridge would be provided across the spillway inlet structure for access to the outlet works control house from the left abutment.

10.0 Outlet Works

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been updated to reflect the changes required to maintain a maximum water surface elevation of 3,490 feet.)

10.1 General

The outlet works for the planning design consists of a trash-racked intake structure, a concrete-lined tunnel with a gate chamber and access shaft, a chute, and a hydraulic jump stilling basin. The outlet works has the same alignment and similar profile as used in the 1953 feasibility design. The original design capacity was increased to meet current Reclamation guidelines for reservoir evacuation.

10.2 Hydraulic Design

The outlet works was sized to meet evacuation requirements for a high-hazard, significant-risk facility, according to the guidelines of ACER Technical Memorandum No. 3. For a constant inflow of 400 ft^3/s and with the reservoir initially full, Table 3 shows the minimum evacuation periods that would result from operating the outlet works at capacity.

Table 3. Minimum Evacuation Periods

Evacuation Stage	Elevation	Period (days)	Guideline (days)
75% Hydraulic height	3,458	17	20-30
50% Hydraulic height	3,412	29	40-50
10% Reservoir storage	3,355	37	50-60
25% Hydraulic height	3,337	39	70-90

Downstream irrigation and fish enhancement require reservoir releases through the outlet works throughout the year. According to the 1963 design data, reservoir releases in June and July for irrigation purposes would represent 58,000 acre-feet in 30 days with 125,000 acre-feet of storage, or 74,000 acre-feet in 30 days with 242,000 acre-feet of storage. Diversion during construction would require an estimated discharge of 800 ft^3/s, based on the capacity of the outlet works for the existing dam upstream. All of these requirements are met by the current outlet works design, having a design capacity of 4,442ft^3/s at the top of active conservation capacity, elevation 3,490.0.

The outlet works intake would be a box-type structure having an invert at elevation 3,372.0, approximately 30 feet above the streambed. Although the top of the intake structure would be within the active conservation pool, a reservoir operation study included in the 1963 design data suggests the reservoir levels would normally be maintained well above this structure. No problems with ice, floating debris, or reservoir sedimentation are expected. A multilevel intake for this site is not required.

The upstream circular tunnel would have an 11-foot finished diameter and would be 636 feet long, including a 60-degree horizontal bend. The tunnel would normally be under pressure, but may be dewatered after placement of a bulkhead gate at the upstream end.

The gate chamber would contain two 5-by-7-foot outlet gates for regulating discharges, and two 5-by-7-foot outlet gates for emergency closure. A small bypass pipe containing a gate valve and a jet-flow gate would be required to pass small discharges to meet minimum streamflow requirements. Access to the gate chamber would be provided by a vertical shaft and a short adit. A 24-inch-diameter air vent pipe would be required downstream from the gates. Normal operation of the outlet works gates would be by remote control. The cost of required remote control operating equipment is included in the unlisted items allowance for the planning design estimate. Availability and adequacy of power and communications to the site will need to be investigated to verify whether remote operation is feasible.

The downstream modified horseshoe tunnel would have a finished diameter of 14 feet and would be 750 feet long. The tunnel was sized to ensure free-flow conditions for any discharge (assuming maximum losses) by limiting the flow area to 75 percent of the total cross- section. The downstream invert would be set above the maximum tailwater elevation. The tunnel bottom would-have a 2 percent slope and a crown on the centerline to improve drainage and flow conditions for single-gate operation.

Outlet works flows from the tunnel would enter a short chute section on a 100-foot-long vertical curve before reaching the stilling basin. The stilling basin was designed to produce a hydraulic jump for any discharge (assuming minimum losses) based on the corresponding tailwater conditions with or without operation of the spillway. The outlet works basin would be 23 feet wide, which requires a floor elevation of 3,308.0 feet and a length of 132 feet, the same as assumed for the spillway basin. (See discussion under Chapter VIII, Spillway.) The maximum outlet works discharge for design (which occurs during passage of the probable maximum flood) is 4,697ft^3/s at maximum water surface elevation 3,490.9.

10.3 Structural Design

Design curves and tables based on the working stress design method were used to proportion structural elements and to estimate concrete thicknesses. Tunnel-lining and shaft-lining thicknesses were based on standard criteria for spillways and outlet works (Bureau of Reclamation 1967) with allowances for over-excavation and tunnel supports. Chute and stilling basin walls were designed as cantilevers having a monolithic floor slab, including heels.

The tunnel and shaft would be excavated through hard andesite which, according to the 1952 geologic report "would stand well during tunneling operations and would not swell when exposed to atmospheric agencies." Structural steel tunnel supports on 4-foot centers were assumed at each tunnel portal extending five tunnel diameters, for a combined distance equal to 15 percent of the tunnel length. The remaining length of tunnel was assumed supported by light sets of rockbolts on 4-foot centers in the crown.

Ring grouting on 20-foot centers was assumed from near the upstream portal to approximately 100 feet downstream from the gate chamber. Crown grouting on 20-foot centers was assumed for the remainder of the downstream tunnel. A nominal grout take of one bag per lineal foot of hole was assumed. Drainage holes on 20-foot centers were assumed above the water surface in the downstream tunnel.

The intake structure and a portion of the chute would be founded on competent andesite bedrock. The stilling basin and the downstream end of the chute (below minimum tailwater) would be founded on glacial till. A discussion of design considerations for the outlet works basin foundation is included in Chapter VIII, Spillway.

Chain link fence would be required along the chute and basin walls for public safety. Rock bolts and wire mesh would probably be required for the steep cutslopes at the tunnel portals.

11.0 Existing Structures

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been updated to reflect the changes required to maintain a maximum water surface elevation of 3,490 feet.)

11.1 General

The existing Bumping Lake Dam is located 4,500 feet upstream from the proposed enlargement dam and was completed in 1910. The dam is a puddled core, earthfill structure having a structural height of 61 feet and a crest length of 2,925 feet at elevation 3,435.0. The outlet works consists of a concrete conduit through the base of the dam, and includes two 5-foot-square regulating gates and two 5-foot-square guard gates located in a concrete intake tower. A concrete chute spillway with a downstream timber flume is located at the left end of the dam. These structures would be inundated by the enlarged reservoir.

11.2 Required Modifications

The existing dam would be breached after construction of the downstream enlargement dam. Complete removal of the existing structure is not believed necessary because of expected infrequent exposure. A channel would be excavated through the embankment to the original streambed elevation of 3,389.0, with a bottom width of 200 feet and with 1.5:1 side slopes. Appurtenant features that might constitute a hazard at low-water stage in the new reservoir would be demolished. These features include the concrete spillway bridge and the timber flume; the concrete outlet works intake tower, including the steel footbridge and all gate equipment; and several hundred feet of chain link fence. The outlet works conduit would be permanently sealed at each end, since it would be within the active conservation pool.

Adequate posting would locate the remaining portions of the dam to alert reservoir users during periods of low water surface.

12.0 Preliminary Powerhouse Sizing and Energy Estimate

(Note: This section has been added and was not a part of previous study reports)

A potential hydropower project utilizing the discharge from the enlarged reservoir has been considered. The head water for the project was assumed to be the water surface of the enlarged Bumping Reservoir, and the tail water elevation was assumed to be the current elevation of the existing dam outlet. Two equally sized units were selected for this option with a plant capacity of approximately 6 MW.

An opinion of average annual energy was developed for the proposed power plant during a water use modeling study that utilized RiverWare. The powerhouse could produce as much as 89,000 MWhr annually, and the gross annual revenue for the hydropower plant could be as high as $ 1,300,000. The revenue was estimated at $0.06/kWH, and is considered to be gross revenue without accounting for renewable energy credits, capacity credits, wheeling charges or O & M costs.

It is important to note that the final configuration of the powerhouse is likely to change with further study, and the estimated project energy and annual revenue will be subject to change due to power sales agreements, water availability, and the ultimate project configuration. As stated above, flows through the powerhouse may not represent the final distribution of water.

13.0 Fish Passage

13.1 Downstream Fish Passage

The downstream fish passage concept is similar to that proposed at Cle Elum Dam. The proposed downstream passage facility would include a reinforced concrete intake structure and a conduit through the dam embankment. The intake structure would include two multilevel overshot, or tilting weir, gates set at different elevations to control passage of release flows. The gates would be raised or lowered as needed to match desired outflow and reservoir levels. Fish would pass over the gates into a 20-foot-long, 20-foot-wide stilling pool that would vary from 5 to 10 feet deep and then into a conduit. Fish would be conveyed downstream in an open channel flow through a 7 foot diameter reinforced concrete pipe from upstream of intake structure through the dam to the river near the dam outlet works. The downstream fish passage facilities would generally be operated from early April to late June.

13.2 Intake Structure

The reinforced concrete intake structure would include two overshot or tilting weir gates set at different elevations to control passage release flows. The gates would be raised or lowered as needed to match desired outflow and reservoir levels. Each gate would be 10 feet wide and 12 feet high. Flow over the gate would pass into a 20-foot-long, 20-foot-wide stilling pool section that would reduce energy to acceptable levels for juvenile fish. Water depth in the stilling pool would vary from 5 to 10 feet for flows from 100 to 300 cfs.

When the reservoir pool elevation is at the spillway crest, the maximum hydraulic drop over the fish passage gate to the stilling pool water surface would be 10 feet or less. Passage releases of 300 cfs could be made at reservoir pool elevation 3,440 or higher. The intake structure would butt up to the existing embankment with the structure deck at elevation 3,505. The structure includes a trashrack with 12-inch bar spacing that would be cleaned manually by raking from the top of the deck or from a trolley-mounted access platform on the front of the trashrack.

13.3 Fish Passage Conduit

A reinforced, cast in-place concrete conduit would carry passage flows from the upstream intake structure to be discharged downstream into the dam outlet works. The conduit would have an inside diameter of 7 feet, a minimum wall thickness of 18 inches, and would be formed in a horseshoe shape with a rounded top and open flume transition on the downstream end. The maximum open channel flow capacity would be 400 cfs, but normal releases would be from 100 to 300 cfs. The normal depth of flow in the conduit at a discharge of 300 cfs would be 4.5 feet with a velocity of about 12 fps.

A 10-foot transition would connect the conduit to a 5-foot-wide chute that would drop 7.7 feet in a distance of 20 feet and discharge to the receiving pool. The maximum velocity down the chute and discharging into the pool would vary between 24 and 21 fps and would be discharged horizontally just above the receiving pool tailwater elevation. A 6-foot-deep plunge pool would be excavated at the outfall structure.

13.4 Upstream Fish Passage/Adult Collection Facility

A trap and haul system is proposed to provide adult upstream passage at Bumping Lake Dam in lieu of a fish ladder. This system would be long enough to accommodate reservoir fluctuations exceeding 50 feet. Upstream migrating fish would be attracted to the fish ladder entrance using auxiliary attraction water and into the collection facility. From there, fis*h* would swim up the ladder into a holding pool. When adequate numbers of fish were collected in the facility, they would be placed into a transport truck to be hauled upstream for release into the reservoir and upstream tributaries. The adult collection facility would generally be operated from early April to late November.

Water to supply the adult collection facility would be delivered by a 16-inch-diameter pipeline from the reservoir at the downstream passage intake structure to the flume and holding pond at the adult collection facility. This pipe would be encased in the juvenile downstream passage conduit. Water will be delivered by gravity when reservoir pool elevations are suitable, and by pumping when low pool elevations prohibit gravity flow, which would typically occur in April and in September through November during normal water years.

14.0 Future Investigations and Design Considerations

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been included verbatim from that report with only minor editorial changes.)

A number of concerns and issues still remain to be resolved, most of which were mentioned in the previous discussion. The following summarizes these concerns and issues:

- Additional dam site foundation exploration would be required to resolve a number of questions. The profile of depth to bedrock between the left abutment and the end of the 1984 seismic refraction line needs to be surveyed, including the foundation for the spillway and outlet works stilling basins. In addition, at least two or three upstream- downstream refraction lines should be surveyed over the extent of the dam base length. Drill holes at the dam site are needed for additional permeability testing of the glacial deposits. Knowledge of the location of the current groundwater table is required from the drill holes. SPT testing in these drill holes is required to evaluate the foundation's resistance to earthquake loading. Data on the density of the foundation, possibly through cross-hole shear wave velocity measurements, are required. Drill holes are required at the stilling basins to determine the depth to bedrock and other foundation characteristics. Evaluation of the hardness of foundation gravels, cobbles, and boulders is required for the dam site to determine the proper cutoff wall trench excavation equipment. The earthfill design would need some study of the extent and continuity of any openwork gravels in contact with pipeable silts and sands in the foundation.
- The borrow areas for the dam embankment would have to be fully explored and tested. This would include a study on the use of the area between the existing dam and the dam site as a borrow area. The proper use of materials, the amount of separation, the gradations to be produced, and the depth to groundwater would all have to be determined for final design. One study required on the core material would be the shear strength and consolidation versus the percentage of bentonite added as was performed for Sugar Pine Dam. Another required study concerns the preferred material to be located over the top of the concrete cutoff wall.
- Instrumentation for the dam, foundation, and cutoff wall is required. The most difficult to design would be the system for evaluating the integrity of the concrete cutoff wall, whether or not excessive deformation or cracking has occurred.
- One design consideration that still remains is the possibility of using a concrete-faced rockfill design at the site. Terzaghi and Peck in the Soil Mechanics in Engineering Practice book on page 498 (1948 edition, August 1960 printing) shows a 160-foot-high concrete-faced rockfill dam with the facing tied to a 150-foot-deep concrete cutoff wall, 13 feet thick, containing an access passage at the top of the wall. The article indicates the upstream toe moved outward 11 inches and the downstream toe about 1 inch. These movements fractured the upper 40 feet of the cutoff wall, which required grouting the cracks. The possibility of constructing the rockfill before doing

the cutoff wall should significantly reduce such lateral movement, which probably resulted from construction of the rockfill after the wall had been built in the foundation. Construction of such a design may be possible at the site and should receive further consideration.

15.0 Revised Cost Estimates

(Note: This section has been completely revised from the 1985 Bumping Reservoir Planning Study.)

Updated cost estimates for Bumping Lake Enlargement Dam are presented in a separate technical memorandum: *Costs of the Integrated Water Resource Management Plan.*

16.0 References

(Note: The basic information presented in this section is from the 1985 Bumping Reservoir Planning Study. This section has been included verbatim from that report with only minor editorial changes.)

1. Planning Design Summary, Bumping Lake Enlargement Dam, Yakima River Basin Water Enhancement Project, Bureau of Reclamation, April 1985

2. Planning Design Summary, Bumping Lake Enlargement Dam, Yakima River Basin Water Enhancement Project, Bureau of Reclamation, July 1985 Revision

3. ACER Technical Memorandum No. 3, "Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low-Level Outlet Works," Bureau of Reclamation, Denver, Colorado, January 1990.

4. Dam Site and Structure Review Team Report - Yakima River Basin Water Enhance Merit Project, Washington, Bureau of Reclamation, January 1984.

5. Design Request Data for the Feasibility Design of Bumping Lake Enlargement Dam - Supplemental Storage Division - Yakima Project, Washington, Upper Columbia Development Office, Spokane, Washington, May 1963.

6. Earth Embankment Materials - Preliminary Report - Bumping Lake Enlargement Yakima Project, Washington, Upper Columbia Development Office, June 1963.

7. Geologic Report - Bumping Lake Enlargement Dam Site - Yakima River Basin Water Enhancement Project, Washington, Addendum, PN Region, October 1984.

8. Memorandum to Chief, Division of Dam and Waterway Design, from Head, Seismotectonic Section, Subject: "Seismotectonic Evaluation for Feasibility Design of Bumping Lake (Enlargement) Dam Site, Yakima Project, Washington," November 6, 1984.

9. Memorandum to Chief, Division of Planning Technical Services, from Regional Director, Boise, Idaho, Subject: "Request for Reevaluation and Updating of the Designs and Cost Estimates to

Current State of the Art for Bumping Lake Dam Enlargement, Yakima River Basin Water Enhancement Project, Washington," July 18, 1984.

10. Memorandum to Regional Director, Boise, Idaho from Chief, Division of Planning Technical Services, Subject: "Notes of September 19-21, 1984 Meetings on Design Requests for Features of the Yakima River Basin Water Enhancement Project, Washington," December 7, 1984.

11. Memorandum to Regional Director, Boise, Idaho, from Chief, Division of Planning Technical Services, Subject: "Probable Maximum Flood for Bumping Lake Enlargement Dam - Yakima Project, Washington," October 15, 1984.

12. Reconnaissance Geologic Report on the Bumping Lake Enlargement Dam Site - Yakima Project, Washington, July 1952.

13. Terzaghi, Karl and Ralph B. Peck, "Soil Mechanics in Engineering Practice," John Wiley and Sons, Inc., 1960 edition, 1948.

14. Tunnel Supports - Provisions for Payment, Report of the Task Force on 7717-W1 Supports, Bureau of Reclamation, Denver, Colorado, Revised June 1967.

17.0 List of Preparers

NAME	BACKGROUND	RESPONSIBILITY
HDR ENGINEERING, INC.		
Brian Grogan	Staff Engineer	Document Preparation
Leanne Greisen	Staff Engineer	Document Preparation
Blaine Graff	Senior Engineer	QA/QC
Stan Schweissing	Senior Engineer	QA/QC and Coordination

Attachments

Figure 1 – Embankment Plan and Profile (revised 1985 Planning Design Summary Drawings)

Figure 2 – Outlet Works and Spillway Sections and Details (revised 1985 Planning Design Summary Drawings)

Figure 3 – Maximum Section and Details (revised 1985 Planning Design Summary Drawings)

Drawing 1407-D-1, Bumping Reservoir Planning Design, Sheet 1 of 2

Drawing 1407-D-2, Bumping Reservoir Planning Design, Sheet 2 of 2

Drawing 443-D-3, Bumping Reservoir Enlargement

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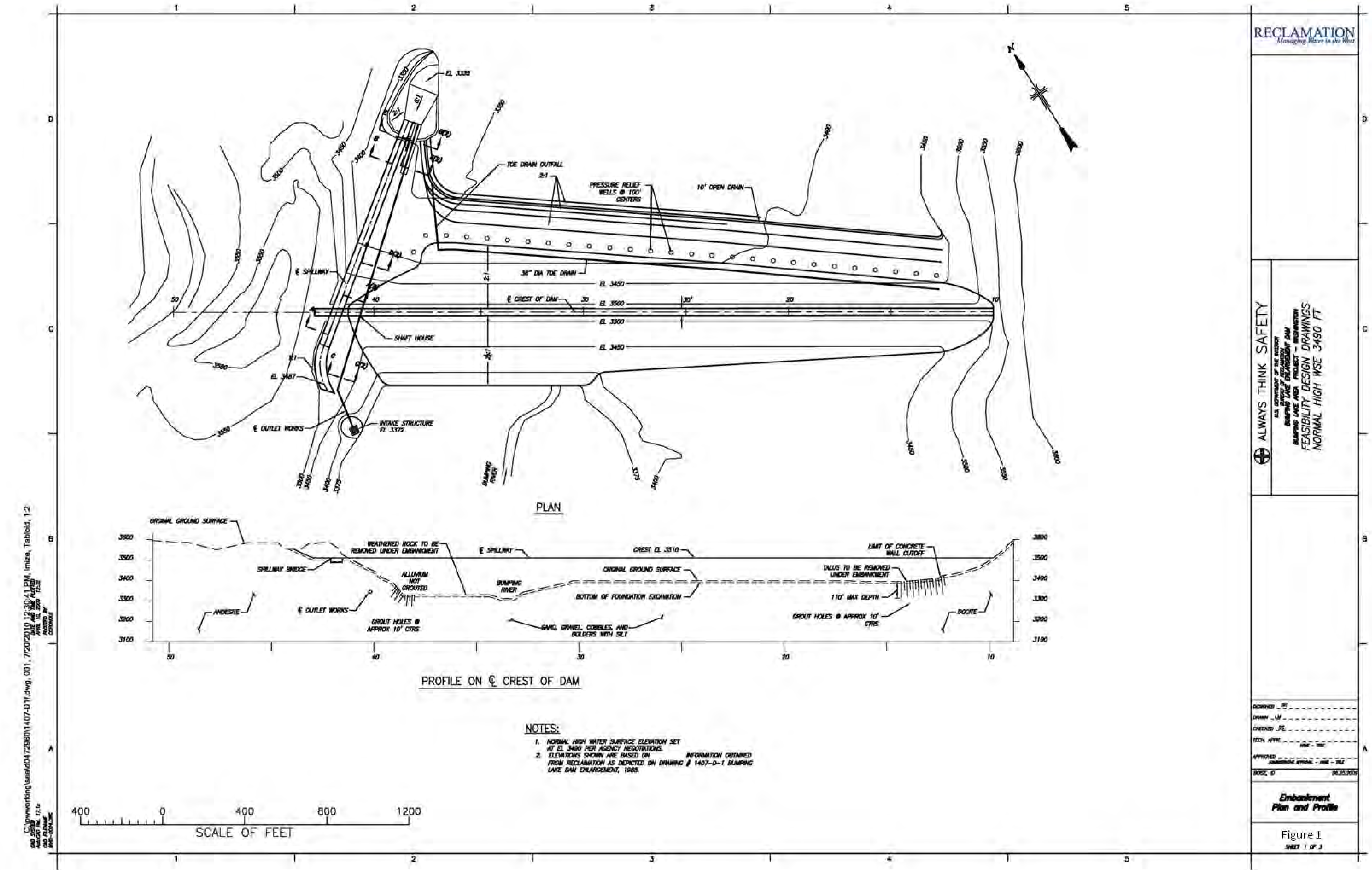
RECLAMATION
Managing Water in the West
ALWAYS THINK SAFETY
FEASIBILITY DESIGN DRAWINGS
NORMAL HIGH WSE 3490 FT
TOE DRAIN OUTFALL
PRESSURE RELIEF WELLS @ 100' CENTERS
10' OPEN DRAIN
36" DIA TOE DRAIN
℄ CREST OF DAM
EL 3500
EL 3450
SHAFT HOUSE
INTAKE STRUCTURE EL 3372
℄ OUTLET WORKS
℄ SPILLWAY
BUMPING RIVER
PLAN
ORIGINAL GROUND SURFACE
WEATHERED ROCK TO BE REMOVED UNDER EMBANKMENT
CREST EL 3510
LIMIT OF CONCRETE WALL CUTOFF
SPILLWAY BRIDGE
ALLUVIUM NOT GROUTED
TALUS TO BE REMOVED UNDER EMBANKMENT
BOTTOM OF FOUNDATION EXCAVATION
110' MAX DEPTH
ANDESITE
GROUT HOLES @ APPROX 10' CTRS
SAND, GRAVEL, COBBLES, AND BOULDERS WITH SILT
DACITE
PROFILE ON ℄ CREST OF DAM
NOTES:
1. NORMAL HIGH WATER SURFACE ELEVATION SET AT EL 3490 PER AGENCY NEGOTIATIONS.
2. ELEVATIONS SHOWN ARE BASED ON INFORMATION OBTAINED FROM RECLAMATION AS DEPICTED ON DRAWING # 1407-D-1 BUMPING LAKE DAM ENLARGEMENT, 1985.
400 0 400 800 1200
SCALE OF FEET
Embankment
Plan and Profile
Figure 1

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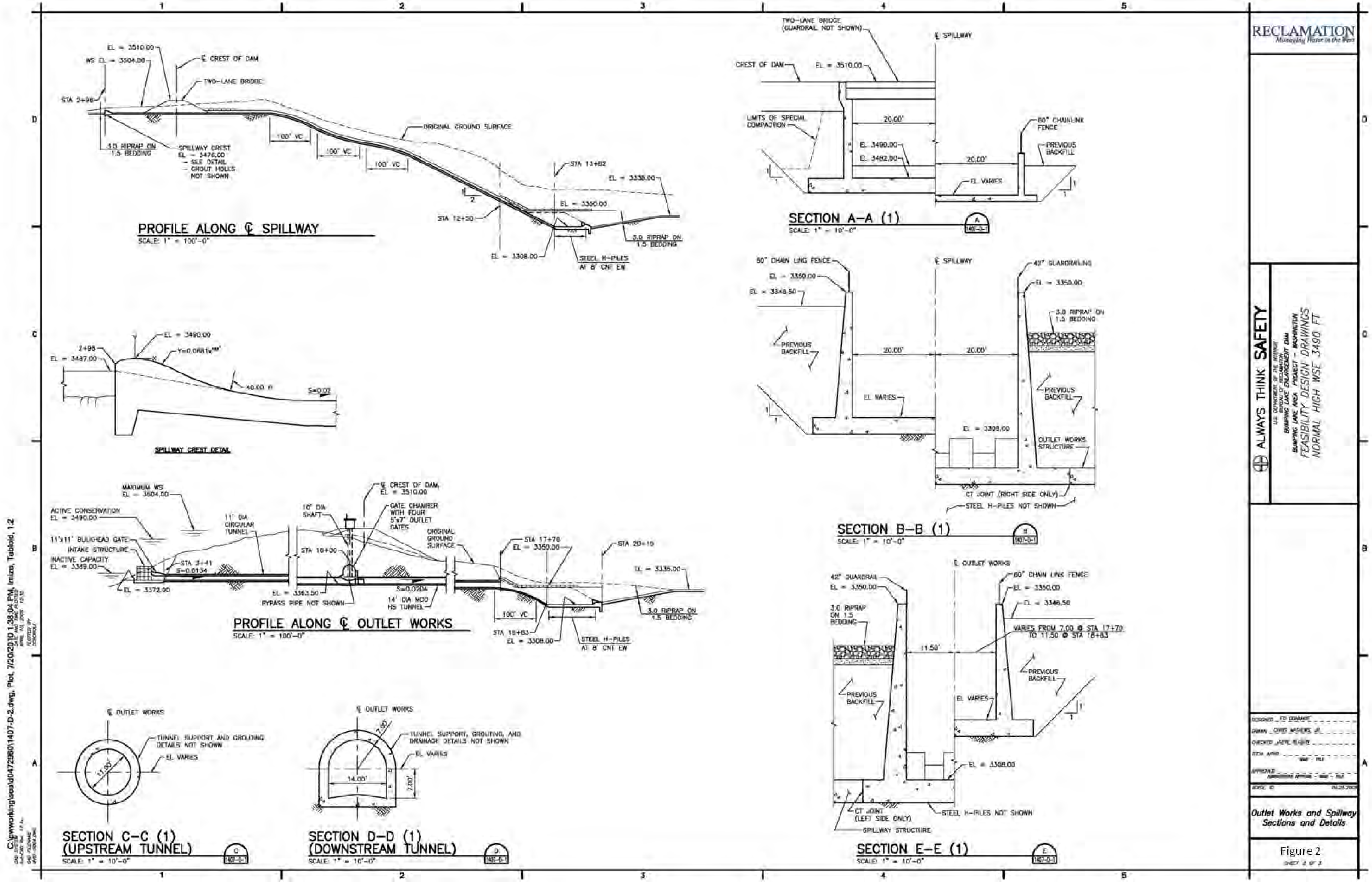
PROFILE ALONG ℄ SPILLWAY
SCALE: 1" = 100'-0"
SECTION A–A (1)
SCALE: 1" = 10'-0"
SPILLWAY CREST DETAIL
SECTION B–B (1)
SCALE: 1" = 10'-0"
PROFILE ALONG ℄ OUTLET WORKS
SCALE: 1" = 100'-0"
SECTION C–C (1)
(UPSTREAM TUNNEL)
SCALE: 1" = 10'-0"
SECTION D–D (1)
(DOWNSTREAM TUNNEL)
SCALE: 1" = 10'-0"
SECTION E–E (1)
SCALE: 1" = 10'-0"
RECLAMATION
Managing Water in the West
ALWAYS THINK SAFETY
FEASIBILITY DESIGN DRAWINGS
NORMAL HIGH WSE 3490 FT
Outlet Works and Spillway Sections and Details
Figure 2

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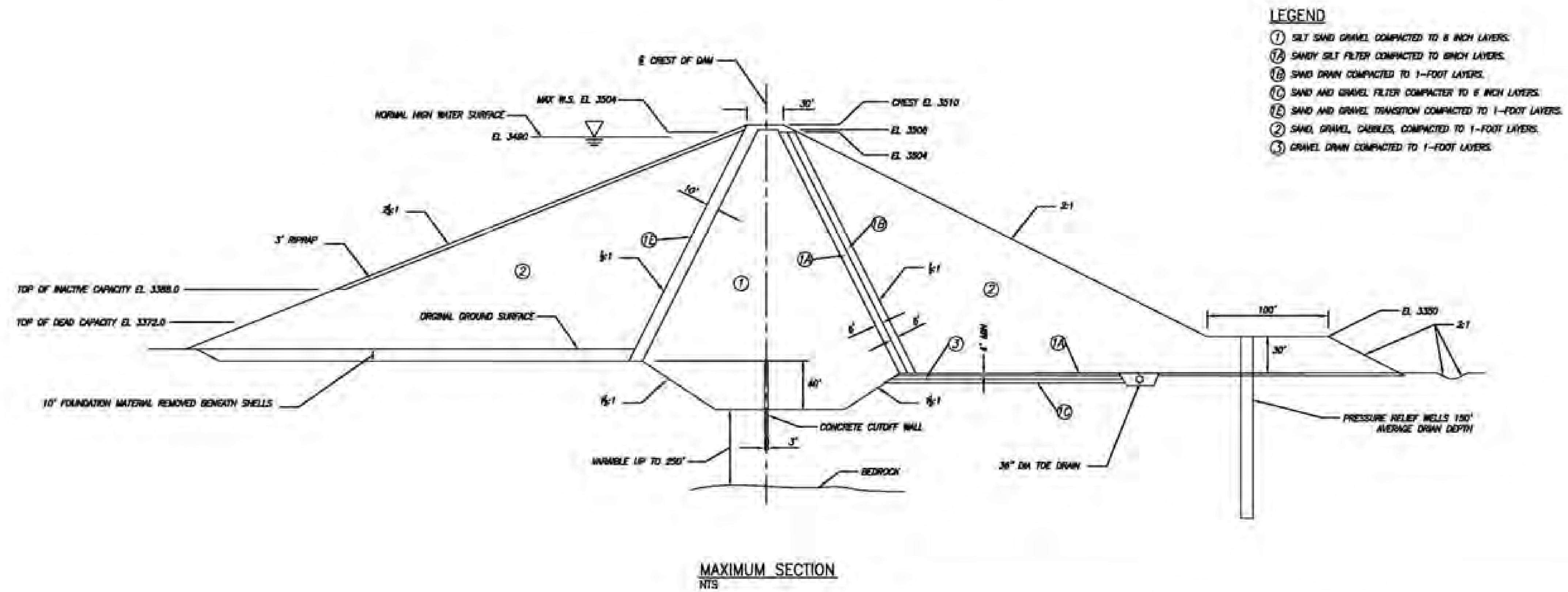
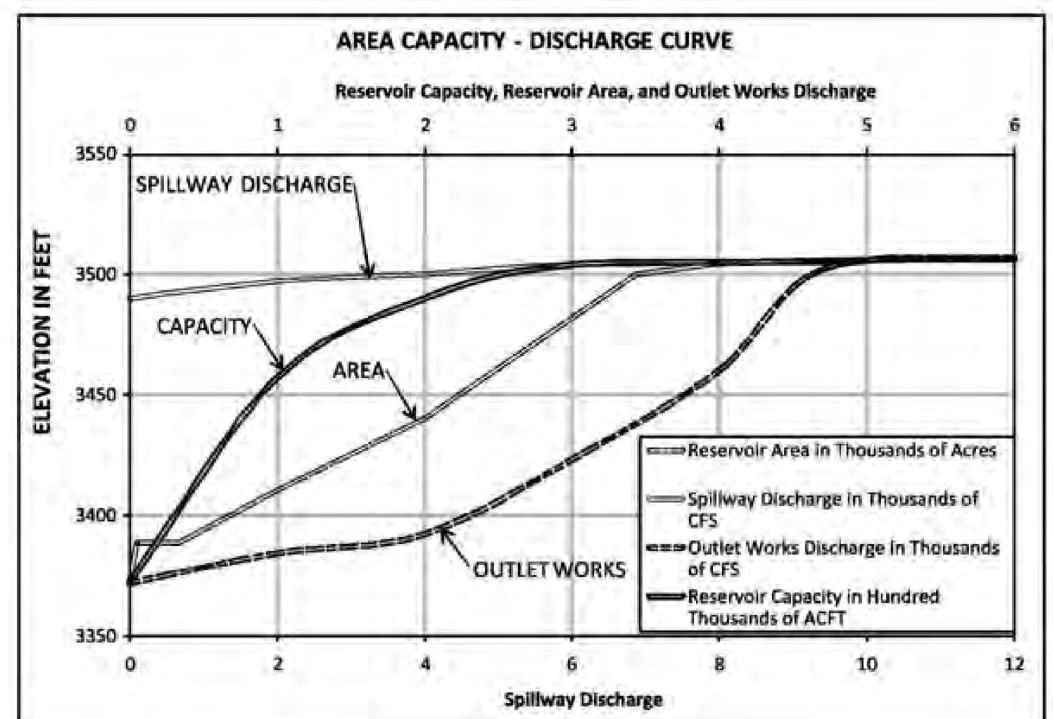


MAXIMUM SECTION
NTS

RESERVOIR CAPACITY ALLOCATIONS		
PURPOSE	ELEVATIONS	CAPACITY ACRE-FEET
ACTIVE CONSERVATION	3389.0 to 3490.0	174,100
INACTIVE	3372.0 to 3389.0	8,800
DEAD	STREAMBED TO 3372.0	15,300
TOTAL RESERVOIR CAPACITY		198,300

A SURCHARGE OF 57,800 AF (EL 3500.0 TO EL 3574.2) IN COMBINATION WITH A SPILLWAY DISCHARGE OF 7,800 CFS AND AN OUTLET WORKS DISCHARGE OF 5,800 CFS IS TO PROVIDED TO PROTECT AGAINST INFLOW DESIGN FLOOD HAVING A PEAK OF 43,020 CFS AND A 7-DAY VOLUME OF 110,540 AF.



RECLAMATION
Managing Water in the West

ALWAYS THINK SAFETY

FEASIBILITY DESIGN DRAWINGS
NORMAL HIGH WSE 3490 FT

Maximum Section
Sections and Details

Figure 3

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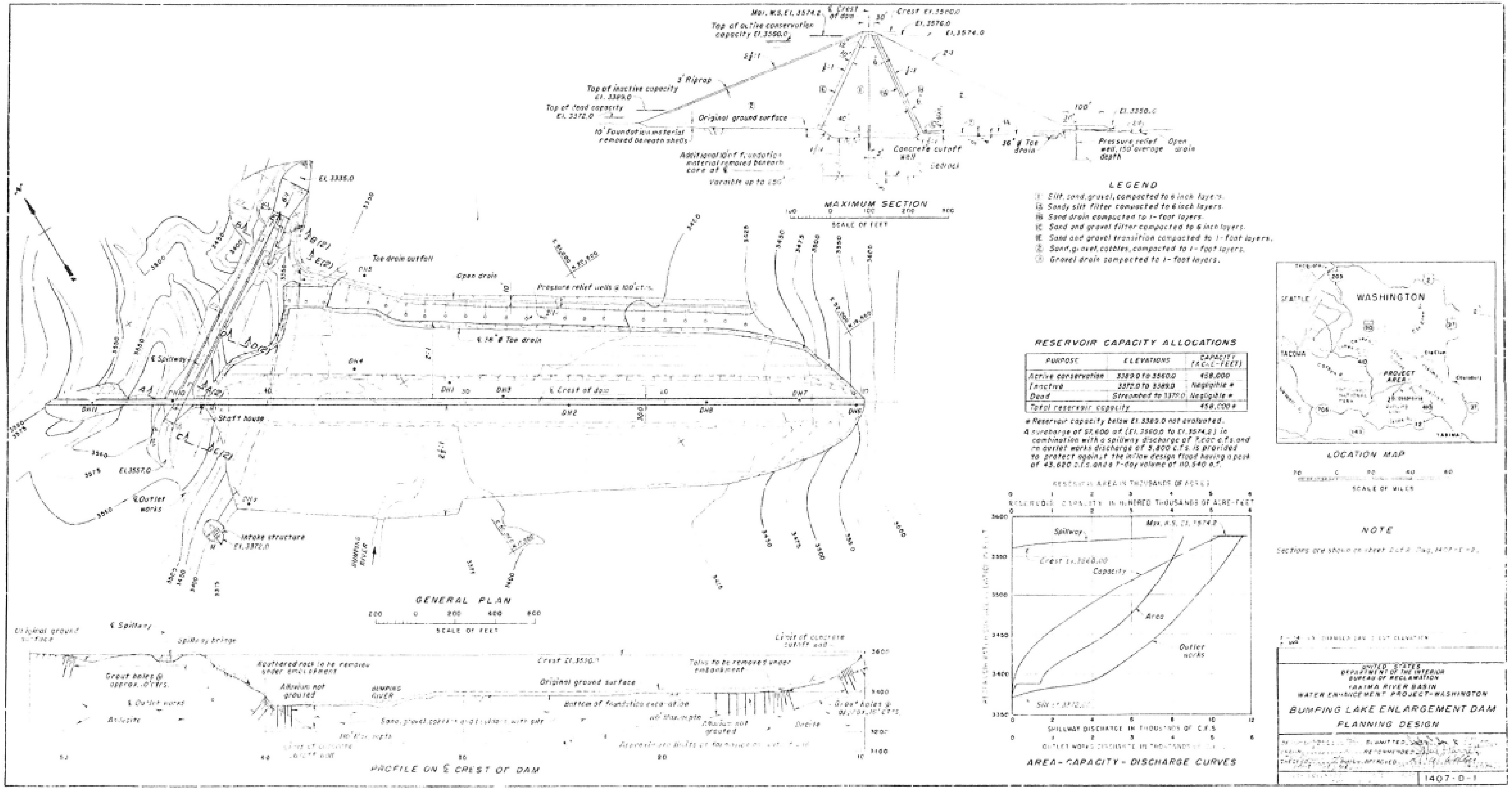
MAXIMUM SECTION
SCALE OF FEET
LEGEND
Silt, sand, gravel, compacted to 6 inch layers.
Sandy silt filter compacted to 6 inch layers.
Sand drain compacted to 1-foot layers.
Sand and gravel filter compacted to 6 inch layers.
Sand and gravel transition compacted to 1-foot layers.
Sand, gravel, cobbles, compacted to 1-foot layers.
Gravel drain compacted to 1-foot layers.
RESERVOIR CAPACITY ALLOCATIONS
PURPOSE | ELEVATIONS | CAPACITY (ACRE-FEET)
Active conservation | 3389.0 to 3560.0 | 458,000
Inactive | 3372.0 to 3389.0 | Negligible *
Dead | Streambed to 3372.0 | Negligible *
Total reservoir capacity | 458,000 *
* Reservoir capacity below El. 3389.0 not evaluated.
WASHINGTON
LOCATION MAP
SCALE OF MILES
NOTE
GENERAL PLAN
SCALE OF FEET
PROFILE ON ℄ CREST OF DAM
AREA-CAPACITY-DISCHARGE CURVES
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA RIVER BASIN
WATER ENHANCEMENT PROJECT-WASHINGTON
BUMPING LAKE ENLARGEMENT DAM
PLANNING DESIGN
1407-D-1

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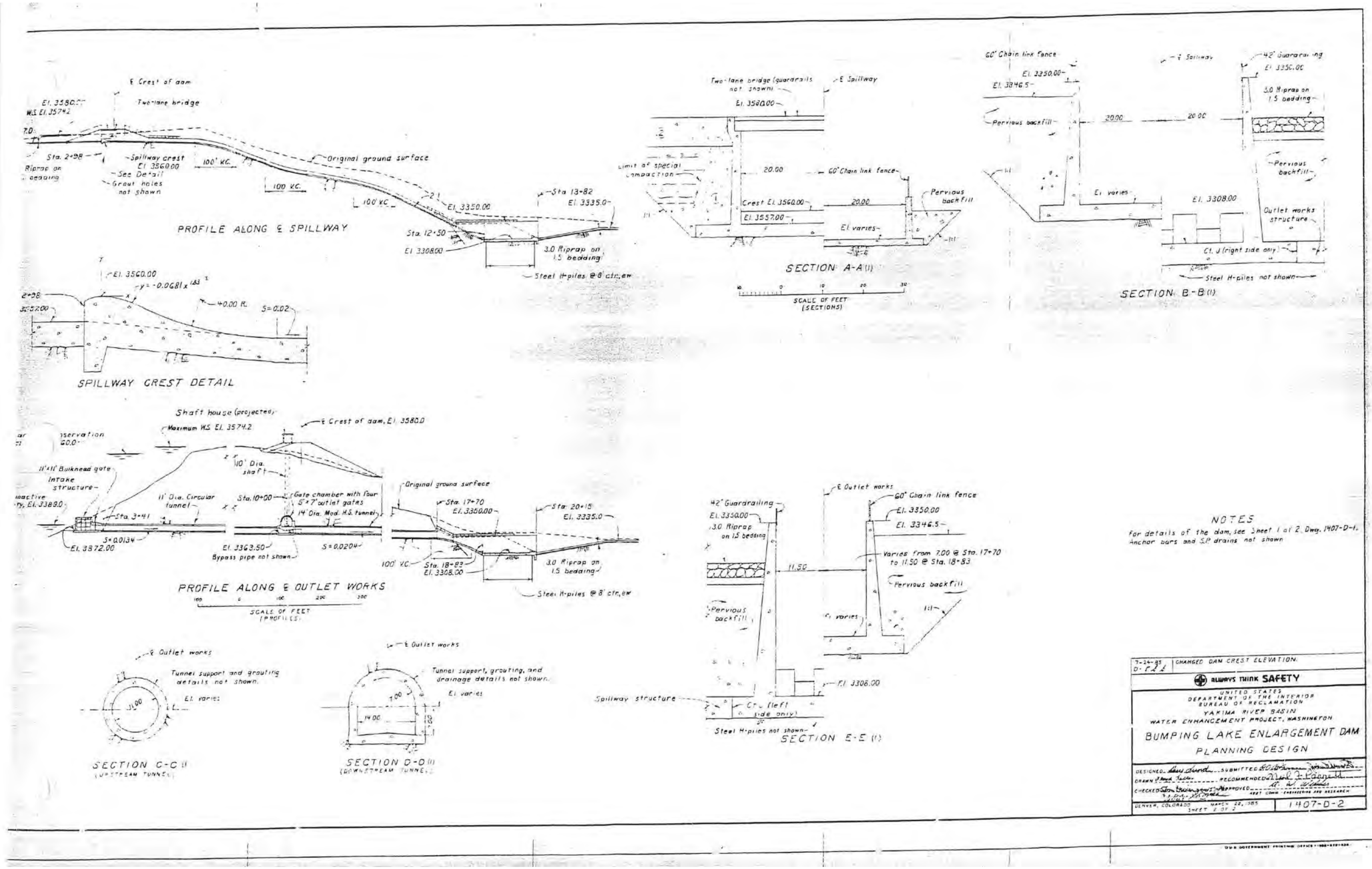
PROFILE ALONG ℄ SPILLWAY
SPILLWAY CREST DETAIL
PROFILE ALONG ℄ OUTLET WORKS
SECTION A-A
SECTION B-B
SECTION C-C (UPSTREAM TUNNEL)
SECTION D-D (DOWNSTREAM TUNNEL)
SECTION E-E
NOTES
For details of the dam, see Sheet 1 of 2, Dwg. 1407-D-1.
Anchor bars and S.P. drains not shown
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA RIVER BASIN
WATER ENHANCEMENT PROJECT, WASHINGTON
BUMPING LAKE ENLARGEMENT DAM
PLANNING DESIGN
1407-D-2

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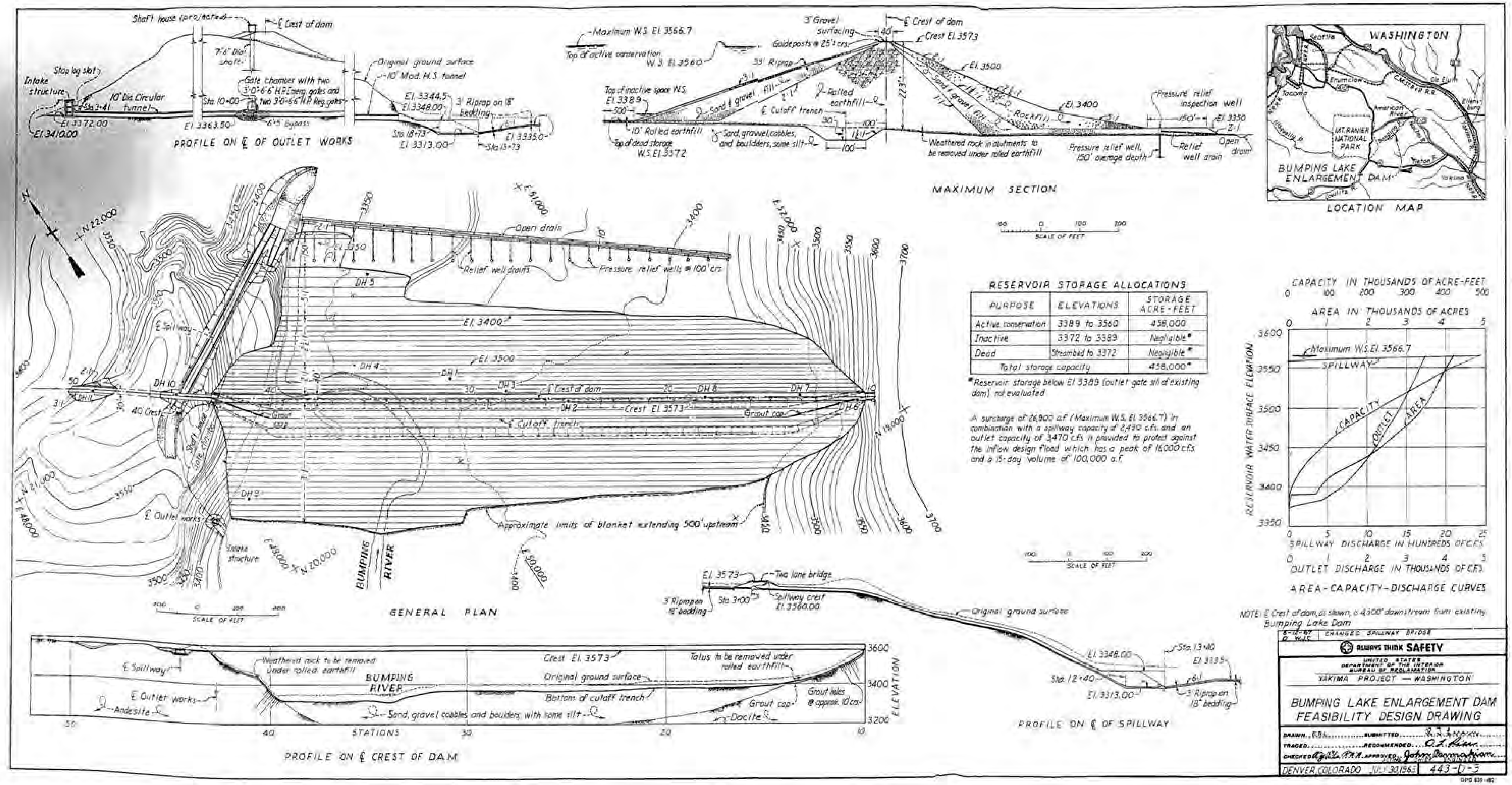
PROFILE ON ℄ OF OUTLET WORKS
MAXIMUM SECTION
WASHINGTON
BUMPING LAKE ENLARGEMENT DAM
MT. RAINIER NATIONAL PARK
LOCATION MAP
RESERVOIR STORAGE ALLOCATIONS
PURPOSE | ELEVATIONS | STORAGE ACRE-FEET
Active conservation | 3389 to 3560 | 458,000
Inactive | 3372 to 3389 | Negligible*
Dead | Streambed to 3372 | Negligible*
Total storage capacity | 458,000*
*Reservoir storage below El 3389 (outlet gate sill of existing dam) not evaluated
A surcharge of 26,900 a.f. (Maximum W.S. El 3566.7) in combination with a spillway capacity of 2,430 c.f.s. and an outlet capacity of 3,470 c.f.s. is provided to protect against the inflow design flood which has a peak of 16,000 c.f.s. and a 15-day volume of 100,000 a.f.
CAPACITY IN THOUSANDS OF ACRE-FEET
AREA IN THOUSANDS OF ACRES
RESERVOIR WATER SURFACE ELEVATION
SPILLWAY DISCHARGE IN HUNDREDS OF C.F.S.
OUTLET DISCHARGE IN THOUSANDS OF C.F.S.
AREA-CAPACITY-DISCHARGE CURVES
GENERAL PLAN
BUMPING RIVER
PROFILE ON ℄ CREST OF DAM
PROFILE ON ℄ OF SPILLWAY
NOTE: ℄ Crest of dam, as shown, is 4300' downstream from existing Bumping Lake Dam
ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT — WASHINGTON
BUMPING LAKE ENLARGEMENT DAM
FEASIBILITY DESIGN DRAWING
DENVER, COLORADO JULY 30, 1963
443-D-3

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