Rock Island Dam

RICHARD W. GALSTER
Consulting Engineering Geologist

PROJECT DESCRIPTION

Rock Island Dam was the first dam to be constructed on the main stem of the Columbia River in either the United States or Canada. The dam lies near the downstream end of Rock Island Rapids at river mile 453.5, where the river flows in two channels separated by Rock Island. As originally constructed between 1930 and 1933 (Figure 1), the dam consisted of a four-unit powerhouse in the north (left) channel of the river, tied to a concrete nonoverflow closure section on the left abutment, and provisions for a powerhouse extension. A 37-bay spillway was constructed across Rock Island and the west channel, terminating in a short concrete nonoverflow section with a short embankment section on the right (south) abutment. The embankment had a concrete core wall about 120 ft long and was contained on the river side by a concrete gravity wing wall. Fish passage facilities were provided on each bank as part of the original construction. Additional fish passage facilities were later built through the center of the spillway. In 1953 the powerhouse was expanded to 12 units and the reservoir raised to elevation 602 ft (R. W. Beck and Associates, 1982).

During the mid-1970s, the dam was modified by construction of a second powerhouse (eight units) adjacent to the south (right) bank (Figure 2). This required removal of the original right bank concrete nonoverflow section, right embankment, fish passage facilities, and six spillway bays. The remainder of the original dam was strengthened by installation of 163 cable tendons from the top of the dam into the bedrock foundation. A new fish passage facility was also constructed on the right bank.

The dam and its modifications were engineered by Stone & Webster Engineering Corporation of Boston. The original construction was done for Washington Electric Company (later Puget Sound Power & Light Company). Supplemental modifications were accomplished for the Chelan County Public Utility District (PUD), which currently owns and operates the dam as a low-head, run-of-the-river hydropower project. Normal reservoir elevation (since 1979) is 614 ft, and nominal tailwater elevation is 575 ft.

SITE GEOLOGY

Rock Island Rapids is positioned hard against the left (east) side of the Columbia River valley due to deposition of an extensive torrential glaciofluvial fill late in the Pleistocene. In addition to glacial meltwaters depositing detritus in a downvalley direction, high-velocity, high-volume glacial floods charging down Moses Coulee entered the Columbia valley and deposited large volumes of gravel, sand, boulders, and ice-rafted blocks in both upvalley and downvalley directions, partly filling the 6,000-ft-wide valley (Figure 3). Apparently, the low point on the fill surface was near the left valley wall, and the postglacial Columbia River was superposed on the higher bedrock surface near the valley side. A remnant of the glaciofluvial fill forms a hummocky terrace surface rising nearly 200 ft above the west bank of the river adjacent to the dam and crowds the river to one-sixth of the original valley width. The terrace surface and riverward slopes are littered with numerous ice-rafted basalt lag blocks as much as 20 ft in diameter. However, the internal composition of the terrace is largely gravel and sand with local lenses of cobbles and boulders and of thinly bedded fine sand, silt, and clay. A silty clay unit as much as 10 ft thick at about elevation 610 ft adjacent to the dam extends for a considerable distance up- and downstream. Such fine-grained materials directly overlie the bedrock surface at elevation 550 ft upstream from the dam (Stone & Webster Engineering Corp., 1974). The bedrock surface, which is at about elevation 580 ft at the right abutment of the dam, drops gently beneath the terrace and is presumably at its deepest level near the center of the old valley before rising to the west valley wall. Thus the bedrock surface at the dam is somewhat higher than to the west and south beneath the terrace.

The essentially flat-lying (2° south dip) basalt flows which form Rock Island Rapids are stratigraphically part of the R2 magnetostratigraphic section of the Grande Ronde Basalt (Tabor et al., 1982). The Hammond invasive flow (Hammond "sill" of Hoyt, 1961) is well exposed in the left (east) valley wall, and the foundation for the dam lies in unnamed units beneath the Hammond.

Figure 1. Rock Island Dam in about 1934 after completion of original construction. View to the northwest showing the four-unit powerhouse and blanks for future units (right) and partly undecked spillway (center and left). A glacial flood terrace is at the far left. Rock Island is at the center, upstream of the dam. High basalt cliffs are at the far right. Photo courtesy of W. L. Shannon.

Because of the proximity of the dam area to the margin of the lava field, many basalt units have invasive relations with Ellensburg sediments (Tabor et al., 1982), which were deposited more or less concurrently with the spread of lava. Thus much of the basalt at the site contains sedimentary rocks, principally tuffaceous siltstone and sandstone. (These were classified as "tuff" during construction of the second powerhouse.) Local sediment-rich lava breccias (hyaloclastites), as well as highly vesicular and scoriaceous zones, are common at flow contacts. The sedimentary rocks range in character from siltstone indurated by contact metamorphism to friable sandstone (Stone & Webster Engineering Corp., 1974).

Prior to dam construction, the Rock Island Rapids were dominated by the low, flat-topped mass of basalt called Rock Island. Normal channels were present on either side, each about 200 ft wide at low water (Figure 4) (U.S. Army Corp of Engineers, 1934, p. 776). At flood stage the entire area was under water. The bedrock surface away from the island was (and still is) highly irregular; potholes in the channel bottom downstream from the dam are as much as 100 ft deep (Stone & Webster Engineering Corp., 1974).

GEOLOGIC ASPECTS OF SITING AND DESIGN

Original selection of the dam site was based on the obvious availability of shallow bedrock at one of the five major Columbia River rapids within the United States. Selection of the north (left) channel for construction of the original powerhouse may have been influenced by the availability of rail and road transportation on that bank.

Figure 2. Vertical aerial photograph of Rock Island Dam. Powerhouse no. 1 is on the right, and powerhouse no. 2 is on the left. Large ice-rafted basalt blocks are on the surface and face of the bank terrace (at left). Three fish passage facilities are on each bank and through the center of the spillway section. Part of Rock Island remains above normal headwater upstream of the dam. The top of the photograph is to the northwest. Photograph courtesy Seattle District, U.S. Army Corps of Engineers.

The curved spillway section was sited on the high, though irregular, bedrock surface at the downstream end of Rock Island, just upstream from the deep potholes that characterize the toe of the rapids.

The right bank nonoverflow, embankment/core wall section was sited on a bedrock ledge at elevation 580 ft, and the original concrete dam abutted into the concrete gravity wing wall constructed normal to the dam axis (Figure 1). The siting of the second powerhouse on the right (south) portion of the dam minimized disruption of the original powerhouse and optimized excavation of rock and the gravel terrace face and construction accessibility. About 90 ft of the original 120-ft-long right bank core wall remains in place, tied to the new dam section.

CONSTRUCTION

Original Construction

Original construction was begun on the north (left) bank and progressed to the south bank (Stone & Webster Engineering Corp., 1934). Most of the left bank nonoverflow and concrete cut-off wall sections were completed prior to any diversion of water. The 900-ft-long gravity nonoverflow was keyed about 2 ft into the irregular bedrock surface. The 366-ft-long cut-off wall was not keyed. Reinforcing steel dowels were installed

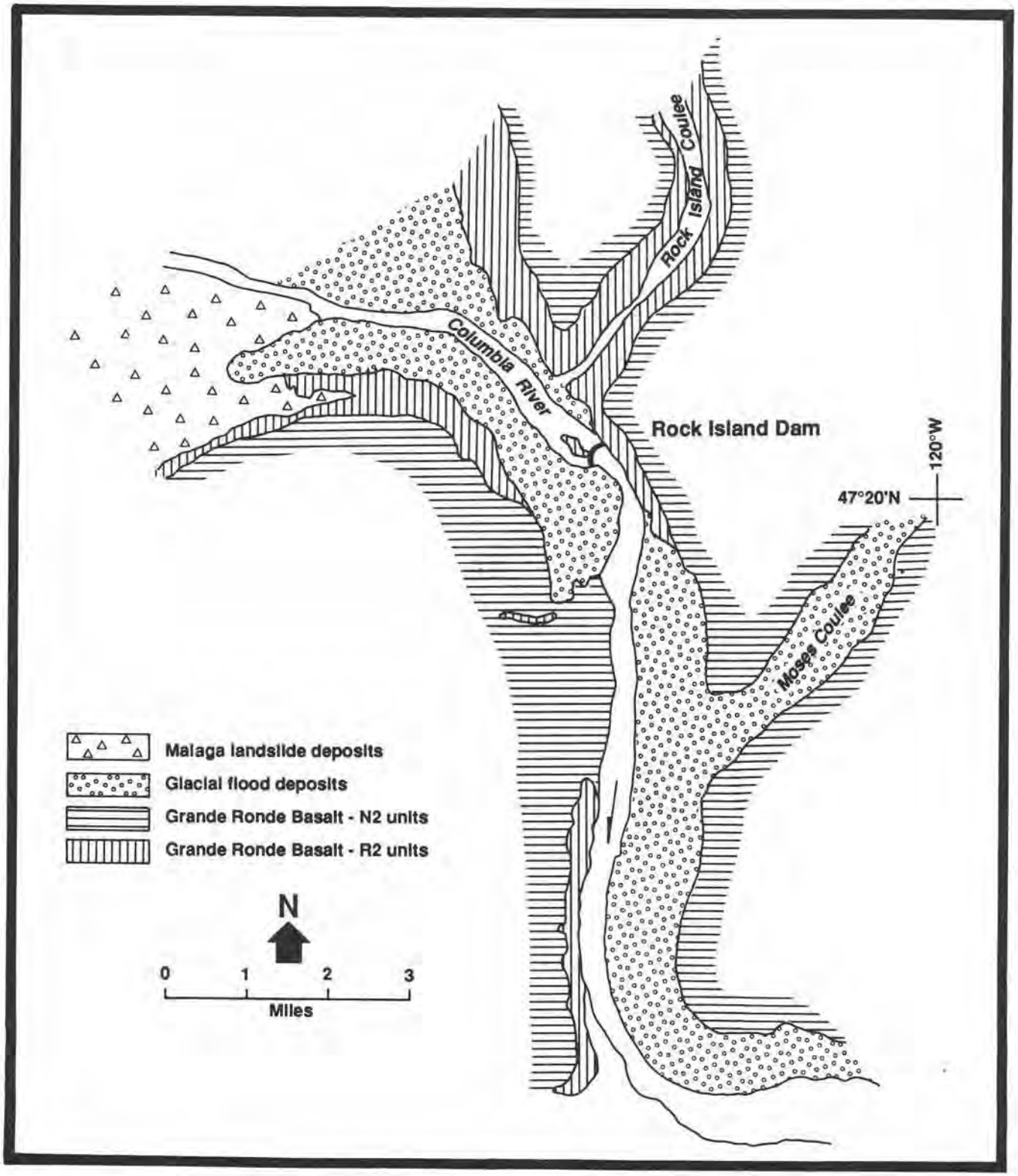


Figure 3. Generalized geologic map of the Rock Island area showing configuration of glacial-flood terrace deposits and bedrock canyon walls. Minor Holocene alluvial and landslide deposits are not shown. After Tabor et al., 1982.

Figure 4. Aerial view of Rock Island Rapids about 1929. View downstream. Photo from Shannon, 1929.

into the bedrock beneath the wall prior to placement of concrete and incorporated in the cut-off wall (Stone & Webster Engineering Corp., 1934). Diversion from the north channel was accomplished by using rock-filled timber crib cofferdams as much as 58 ft high. The original four-unit powerhouse, together with the intake dam for powerhouse expansion, and the eastern part of the east spillway including foundation grouting were completed during the first stage of construction in a 6-month period beginning September 1, 1930. Figure 5 shows the dam in mid-1931. Parts of the east and west spillway sections were constructed without cofferdams across the downstream end of Rock Island. The remaining south channel section required similar timber crib cofferdams as high as 72 ft and diversion through the partly completed spillway and intake sections of the dam (Stone & Webster Engineering Corp., 1934).

From data obtained during the period of tendon installation during the mid-1970s, powerhouse no. 1 and the left bank gravity and closure sections of the dam are founded on dense basalt, probably the colonnade of a single flow extending at least 60 ft beneath the powerhouse (Figure 6). Excavation for the powerhouse encountered excellent rock for the full depth. However, beginning at the south end of powerhouse no. 1 the basalt flows show invasive relations with Ellensburg sediments; thus, increased amounts of tuff, volcanic breccia, sandstone, siltstone, and locally greater amounts of scoria and vesicular basalt are found in the foundation to the south. The irregular bedrock surface included six channels that ranged from 30 to 60 ft wide and extended from 20 to 75 ft below the nominal river channel.

Rock Island appears to be capped by 10 to 35 ft of dense basalt, which is underlain by a complex assemblage of vesicular and scoriaceous basalt, tuffs, and breccias. Most of the section of the spillway across Rock Island was founded on this basalt cap. However, local irregularities in the rock surface, especially in the south channel, required excavation to as low as elevation 490 ft. Concrete backfill was brought up to between elevations 545 and 560 ft in order to provide adequate foundation for certain spillway sections (R. W. Beck and Associates and Chelan County Public Utility District, unpublished data). The excavation could not be planned because of the horizontal and vertical variation

Figure 5. Rock Island Dam under construction, June 18, 1931, showing concrete work completed for the four-unit powerhouse no. 1 and the east spillway. Openings for the future addition to powerhouse no. 1 (center) are ungated, as are the spillway bays (left). Rock Island, in the center of the river, is being used as a staging area. The right bank terrace is on the far bank. (Note large ice-rafted basalt blocks.) Jumpoff Ridge rises behind the terrace. View to the west-northwest. Photograph courtesy of H. A. Coombs.

of the rock (Stone & Webster Engineering Corp., 1934). At one deep channel a steam shovel was dismantled and lowered into the excavation to clean up the foundation.

The right nonoverflow section, right bank core wall, and wing wall were founded on an exposed ledge of sound basalt at about elevations 575 to 580 ft. The rock surface required little preparation. The core wall was keyed into bedrock with steel dowels installed across the foundation contact. Sandy clay was puddled against the core wall as it was raised (Stone & Webster Engineering Corp., 1934).

Grouting and Drainage

The requirement for grouting was recognized early in the original construction, although no grouting and drainage gallery was incorporated into the design of the original dam. Grout holes were drilled adjacent to the heel of the dam, generally in a double row, 8 ft on center. Depths ranged from 12 ft in the left abutment area to as great as 35 ft in the powerhouse/spillway area. Takes were highly variable, a function of jointing in the basalt and the characteristics of the adjacent sedimentary units beneath the spillway (Stone & Webster Engineering Corp., 1934). Although surface drains were installed on the foundation beneath concrete structures prior to placing concrete, no drainage system was installed below the foundation contact.

Construction of Powerhouse No. 2

Construction of the second powerhouse took place between 1975 and 1978 between upstream and downstream cofferdams. The former was largely an embankment structure crowded into the dewatered reservoir from the south bank and connected to a short cellular (steel pile) section that tied into the upstream side of the spillway dam. The downstream cofferdam was entirely steel sheet pile cells.

The powerhouse is founded primarily on "massive" basalt and vesicular basalt, although substantial portions are founded on tuff (siltstone) and invasive flow breccia. Most of the powerhouse foundation lies between elevations 499 and 510 ft; a keyway extends to elevation 486 ft for the entire length of the structure for sliding resistance. As ultimate bearing pressures were not to be greater than 500 psf, much of the tuff exposed in the foundation was left in place. However, the tendency of the tuff to air slake required a protective aerospray coating as temporary weather protection until concrete could be placed. Tuff zones caused considerable problems in setting and anchoring rock bolts during construction. However, vertical excavated rock walls 70 and 90 ft high stood well at each end of the excavation while the foundation was being prepared (Stone & Webster Engineering Corp., 1976).

A strong artesian zone was found beneath the powerhouse foundation. The top of the zone varies from elevation 480 ft to as low as 430 ft, probably due to the complex characteristics of the basalt-sediment interface. The aquifer heads rise to between elevations 500 and 515 ft beneath the main part of the powerhouse and to as high as 530 ft at the upstream edge (Stone & Webster Engineering Corp., unpublished data). The presence of the aquifer required installation of 48 shallow drains

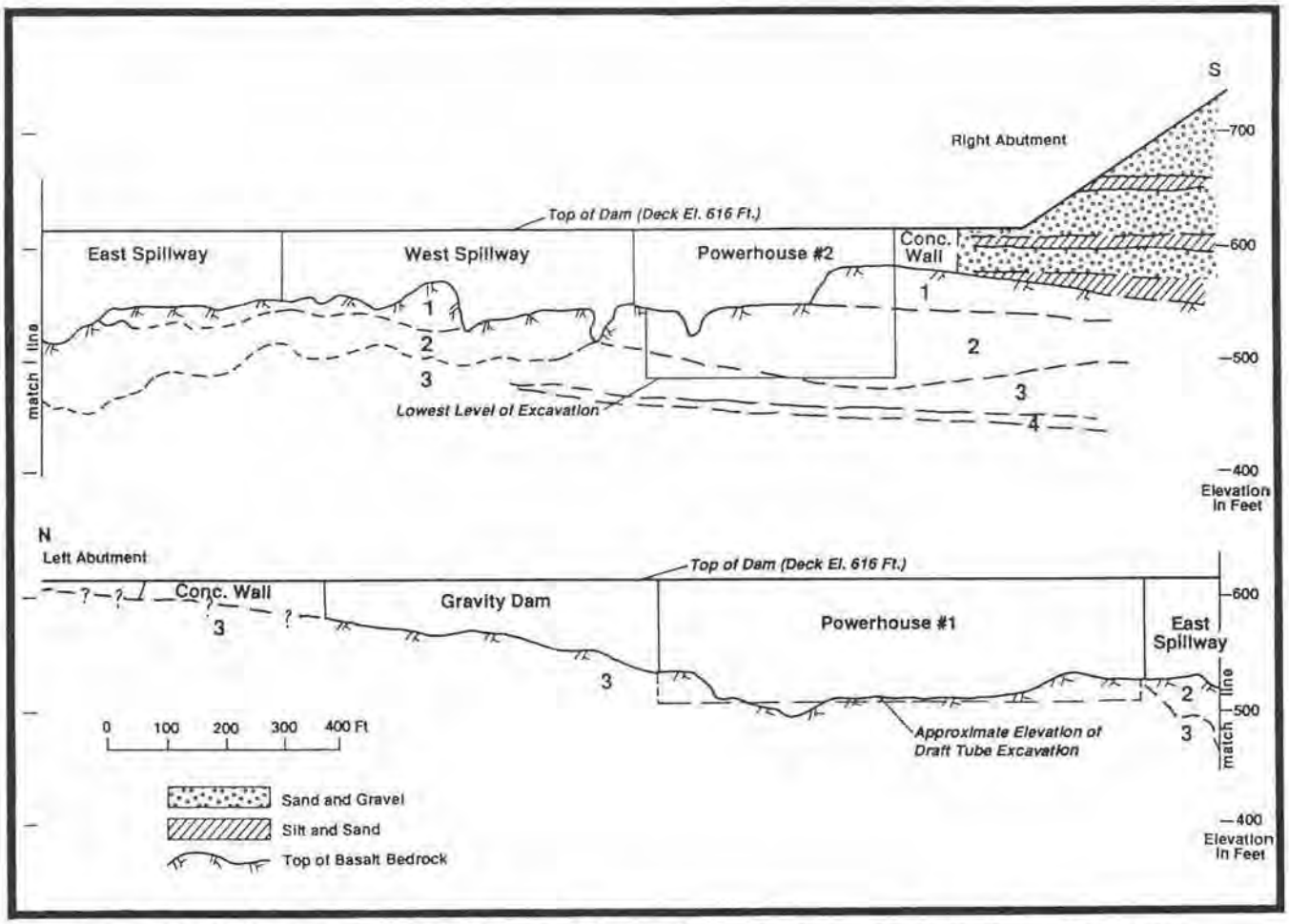


Figure 6. Geologic section at Rock Island Dam. Rock line based on tendon drilling data (R. W. Beck and Associates, unpublished data) except at powerhouse no. 2. View downstream. Numbered bedrock units are generalized as follows: 1, columnar basalt; 2, combination of invasive vesicular and scoriaceous basalt, tuffs, and breccia with highly irregular invasive characteristics; 3, generally columnar to "massive" basalt; and 4, tuff.

under the structure for pressure relief (R. W. Beck and Associates, 1987). Water from these drains is collected in a central drain and conducted to the dewatering sump from which it is pumped to tail water.

Post-Tension System

In order to increase the stability of the original concrete structures under conditions of a raised reservoir, 163 steel cable tendons were installed in the mid-1970s. All drilling for tendon anchorages was done from the roadway deck (elevation 618 ft). Because of the highly varied geologic conditions beneath the spillway, tendon anchor elevations range between 450 and 525 ft, depending on the position of sound basalt. A few tendons are anchored in tuff or scoriaceous basalt (R. W. Beck and Associates and Chelan County Public Utility District, unpublished data). Beneath powerhouse no. 1, anchor elevations range from 445 to 490 ft, all in "massive" basalt. Beneath the north abutment gravity wall, anchor elevations increase progressively toward the abutment from 490 to 540 ft; all are in "massive" basalt.

OPERATIONAL PROBLEMS

The combination of the invasive nature of the lava flows and sediments beneath part of the spillway and a general rapid drop of the bedrock surface at the toe of the rapids, produced some local erosion in the west spillway area during the late 1960s. Erosion downstream and immediately adjacent to several spillway bays had progressed to a point where stability of certain spillway monoliths might have eventually been compromised. The problem was mitigated by appropriate concrete backfill (Stone & Webster Engineering Corp., 1968).

Local high uplift pressures have been noted beneath certain sections of the dam from time to time (R. W. Beck and Associates, 1987). This may in part be a func-

tion of the type of drainage system installed and the extent of the grout curtain beneath the heel of the dam.

ACKNOWLEDGMENTS

I am indebted to W. L. Shannon (Shannon & Wilson, Inc.), whose father, W. D. Shannon, was project engineer on the original construction of Rock Island Dam, for providing me with much of the data on the original construction. The cooperation of M. J. Carney of R. W. Beck and Associates, Seattle, and E. Rickman of the Public Utility District of Chelan County, Wenatchee, is most appreciated. Numerous discussions with H. A. Coombs also assisted in my understanding of this project.

REFERENCES

Beck, R. W., and Associates, 1982, *Rock Island Hydroelectric Project, FERC Project No. 943, Periodic Safety Inspection Report*: Report for Chelan County Public Utility District No. 1, Washington, Wenatchee, WA: R. W. Beck and Associates, Seattle, WA, 20 p., 41 plates.

Beck, R. W. and Associates, 1987, *Rock Island Hydroelectric Project, FERC Project No. 943, Periodic Safety Inspection Report*: Report for Chelan County Public Utility District No. 1, Washington, Wenatchee, WA, R. W. Beck and Associates, Seattle, WA, 27 p., 51 plates.

Hoyt, C. L., 1961, The Hammond sill–An intrusion in the Yakima Basalt near Wenatchee, Washington: *Northwest Science*, Vol. 35, pp. 58-64.

Shannon, W. D., 1929, *J. O. 4260, Photographic and Geological Study of Rock Island Site, Columbia River* (unpublished report): Courtesy of W. L. Shannon, Shannon & Wilson, Inc., Seattle, WA, 69 p. [Includes geologic report by H. Landes]

Shannon, W. D., 1929, *J. O. 4260, Photographic and Geological Study of Rock Island Site, Columbia River* (unpublished report): Courtesy of W. L. Shannon, Shannon & Wilson, Inc., Seattle, WA, 69 p. [Includes geologic report by H. Landes]

Stone & Webster Engineering Corp., [1934?], *Construction Methods Report, Rock Island Hydro Electric Project of the Puget Sound Power and Light Co.*: Stone & Webster Engineering Corp., Boston, MA, 143 p.

Stone & Webster Engineering Corp., 1968, *Proposed Method of Erosion Repair in West Spillway Channel, Rock Island Hydropower Project*: Report for Public Utility District No. 1 of Chelan County, WA: Stone & Webster Engineering Corporation, Boston, MA, 12 p.

Stone & Webster Engineering Corp., 1974, *Report on Subsurface Investigations, Rock Island Project Second Powerhouse*: Report for Public Utility District No. 1 of Chelan County, WA: Stone & Webster Engineering Corporation, Boston, MA, 204 p., and plates.

Stone & Webster Engineering Corp., 1976, *Supplement to Report on Subsurface Investigations, Rock Island Project, Public Utility District No. 1 of Chelan County, Washington*: Stone & Webster Engineering Corporation, Boston, MA, 111 p., and plates.

Tabor, R. W.; Waitt, R. B.; Frizzell, V. A., Jr.; Swanson, D. A.; Byerly, G. R.; and Bentley, R. D., 1982, *Geologic Map of the Wenatchee 1:100,000 Quadrangle, Central Washington*: U.S. Geological Survey Miscellaneous Investigations Series Map I-1311, 26 p., 1 plate, scale 1:100,000.

U.S. Army Corp of Engineers, 1934, *Columbia River and Minor Tributaries*: 73rd Congress, 1st Session, House Document No. 103, Vol. II, Washington, DC, 1,845 p.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Rocky Reach Dam

HOWARD A. COOMBS
University of Washington

PROJECT DESCRIPTION

Rocky Reach Dam (Figure 1) is located in central Washington on the Columbia river, 7 mi upstream from the city of Wenatchee. The dam is a Z-shaped structure composed of a forebay wall, service bay, and 11-unit powerhouse, and a spillway composed of 12 regulating gates (Figure 2). The dam is essentially a gravity structure that rises 218 ft above lowest bedrock support. The head on the turbines is 92 ft. The length of the concrete dam is 2,860 ft. The cut-off on the left bank terrace is 1,950 ft long.

DAM SITE GEOLOGY

The mountains to the west and north of Rocky Reach Dam, as well as bedrock under the dam, are composed of the Swakane Biotite Gneiss (Waters, 1932). More than 85 percent of the Swakane gneiss is a biotite gneiss of remarkably uniform composition: almost equal amounts of quartz and feldspar, with approximately 13 percent biotite and a little muscovite (Waters, 1932). The rock is well foliated and fine grained, approaching the texture of a schist. Since the rock breaks along foliation planes, the parallel plates of biotite appear to make up a much larger percentage of the mineral content than is actually the case. According to Mattinson (1972), the gneiss has a Precambrian depositional age, based on analysis of zircons.

On the east bank the gneiss forms a cliff more than 2,000 ft above the river that is capped by the Columbia River basalts and marks the western edge of the Columbia Plateau. Also on this left bank is a terrace, 140 ft above the river at low water, that extends 3,500 ft from the river's edge to the base of the cliff.

The Rocky Reach site felt the effects of ice occupancy of the region in many ways. Perhaps the most striking feature of ice wasting is the development of enormous fill terraces in the Columbia River valley.

Concrete Structures

The 2,860-ft-long gravity section of Rocky Reach Dam rests directly on Swakane gneiss (Figure 3), and the 1,950-ft-long grouted cut-off on the east bank terrace has been carried down to Swakane gneiss. Railway and highway cuts on the right bank of the dam site provide excellent exposures of bedrock. The foliation of the Swakane gneiss forms an antiform very near the axis of Rocky Reach Dam. This antiform's axis coincides with the uplift of the Entiat Mountains (Waters, 1932). Elsewhere along the river, bedrock is covered by long talus slopes extending many hundreds of feet above the river. Preliminary exploration indicated bedrock at a very shallow depth under the river but at a greater depth beneath the terrace on the left bank.

During excavation many shallowly dipping planes of parting were seen parallel to the gneissic banding. Associated with these were several fault traces. One large fault, dipping at a moderate angle, passes under the wall of the powerhouse at a depth of 50 ft. Since this wall was to be heavily loaded with the crane that moves turbines and generators, engineers felt that the fault zone might yield under the load. A 36-in. calyx drill hole encountered the fault zone, and the writer was lowered in the hole in a bucket to make an examination. Because the zone was thought to be incompetent to support the load, it was mined out and backfilled with concrete.

The curved fault planes made it difficult to predict the dips and consequently the locations of the faults at depth. At the upstream apex of the dam, where the spillway and the powerhouse meet, a fault was seen on the surface when the area was dewatered. This fault dipped downstream and was thought to be shallow. With deeper excavation downstream the fault was found to dip more steeply. As a result, much more excavation than had been anticipated was necessary to remove incompetent rock from the fault zone.

The only other formational units involved in dam construction are the clays, silts, sands, and gravels of the east bank terrace. Prior to final design of the dam, the terrace on the left bank as well as the centerline of the concrete structure, were explored by seismic methods. This revealed a thin veneer of sediments in the river bottom and a thick covering of terrace deposits on the left bank where the highly irregular bedrock surface was as much as 130 ft lower than the rock surface beneath the river channel. This accurate location of bedrock surface was invaluable in designing the project.

Figure 1. Rocky Reach Dam, Columbia River; view upstream. The curved structure on the left is the fish ladder. Photo by Chelan County Public Utility District.

Left Bank Terrace

Stratigraphically, the left bank terrace can be divided into three units (Figure 4). The lowest unit, overlying bedrock, is composed of coarse gravels and varies in thickness from a few feet to more than a hundred feet.

Overlying the gravels is varved clay containing thin, scattered lenses of sand. The varved clays suggest a large lake once occupied this part of the Columbia River valley. Although the maximum thickness of the clays at the dam site is only 180 ft, at several places upstream the clay unit is several times thicker and extends much higher in elevation. Because of their pre-erosion load, the clays at the dam site exhibit greater than normal consolidation characteristics. Locally, the upper part of the varved deposit has been stripped away by river erosion. Overlying the clays are sands and gravels of the third unit.

The wide terraces along the Columbia River create problems for many of the Columbia River dams. The most difficult situation at Rocky Reach was controlling seepage through the left bank in the lower terrace gravel unit. Permeability tests were made on the gravel unit when the first stage cofferdam was dewatered. Dye travel time indicated the gravels were highly permeable. Upstream of the dam the bedrock surface in the river drops and the surface of the lower gravels rises. Therefore the thickness of these permeable gravels exposed to the river increases significantly upstream.

CONSTRUCTION OF THE LEFT BANK GROUT CURTAIN

The purpose of the curtain was to control seepage in the lower gravels so that (a) piping could not develop and (b) seepage would not cause slides on the downstream bank.

Since the remoteness of lower gravel unit presented the most difficult problem in controlling seepage, several methods of control were considered. It had been

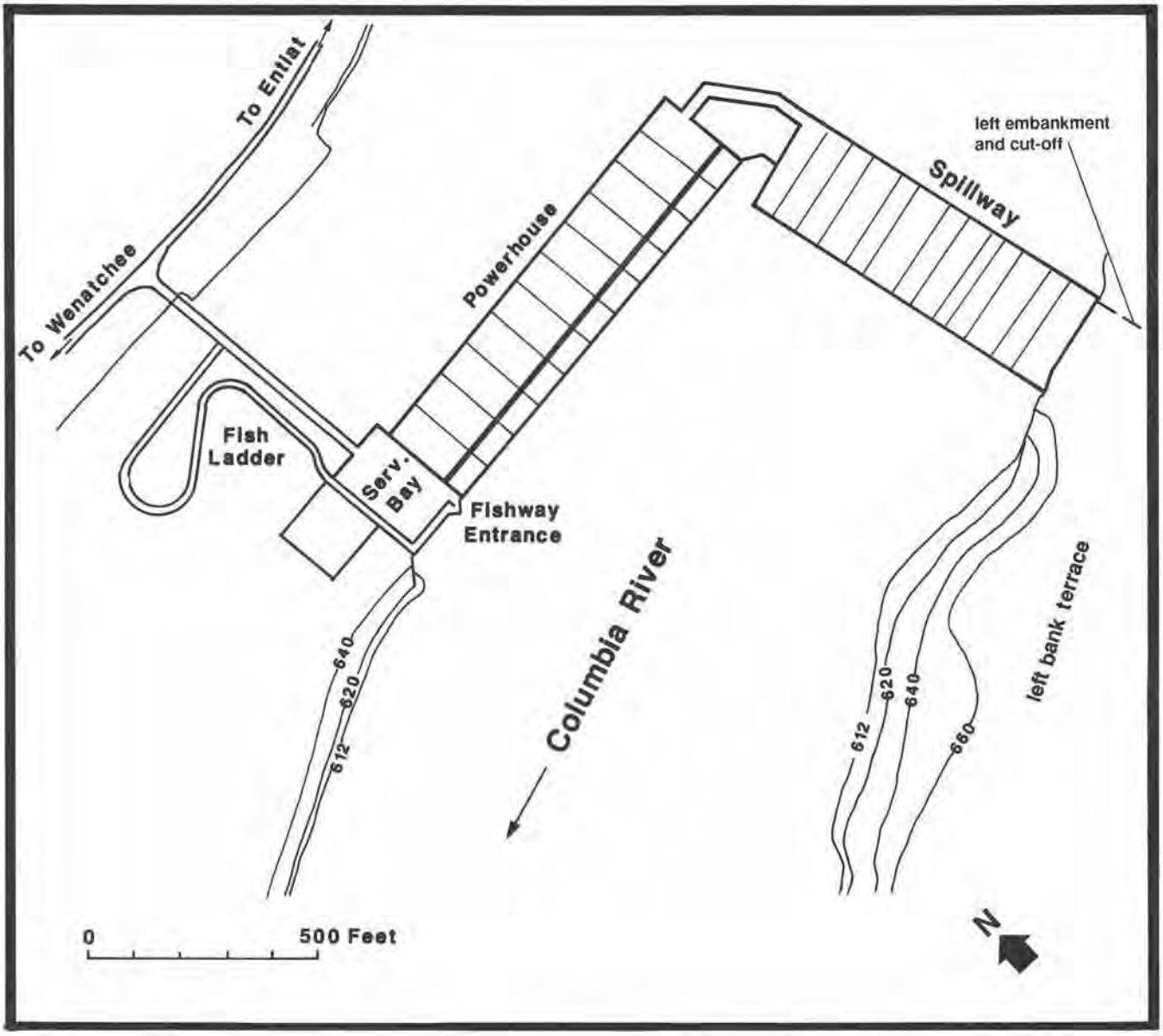


Figure 2. Plan of Rocky Reach Dam.

planned to use a relatively short upstream blanket and a downstream drainage system consisting of a series of relief wells extending into the east bank discharging through a collector pipe. Estimates based on actual observed coefficients of permeability indicated excessive seepage losses would be experienced with this design. After a study of several methods of control, it was decided to use a grouted cut-off. The plan included digging a wide trench through the upper gravels down to just above the top of the varved clays. Drills set on top of these clays would penetrate the clays and continue through the gravels to bedrock (Figures 4 and 5).

The basic design for the grout curtain was for three rows of holes on 10-ft centers. The exception was for five rows of holes for the first 200 ft east from the dam abutment. This curtain was carried out 1,500 ft from the dam. Beyond this, and to the end of the curtain (an additional 450 ft), a single row of grout holes was used. A chemical grout, having a viscosity near that of water, was used to seal some of the sands near the dam abutment.

As the cut-off was not conceived as a totally impervious structure, it was necessary to provide for discharg-

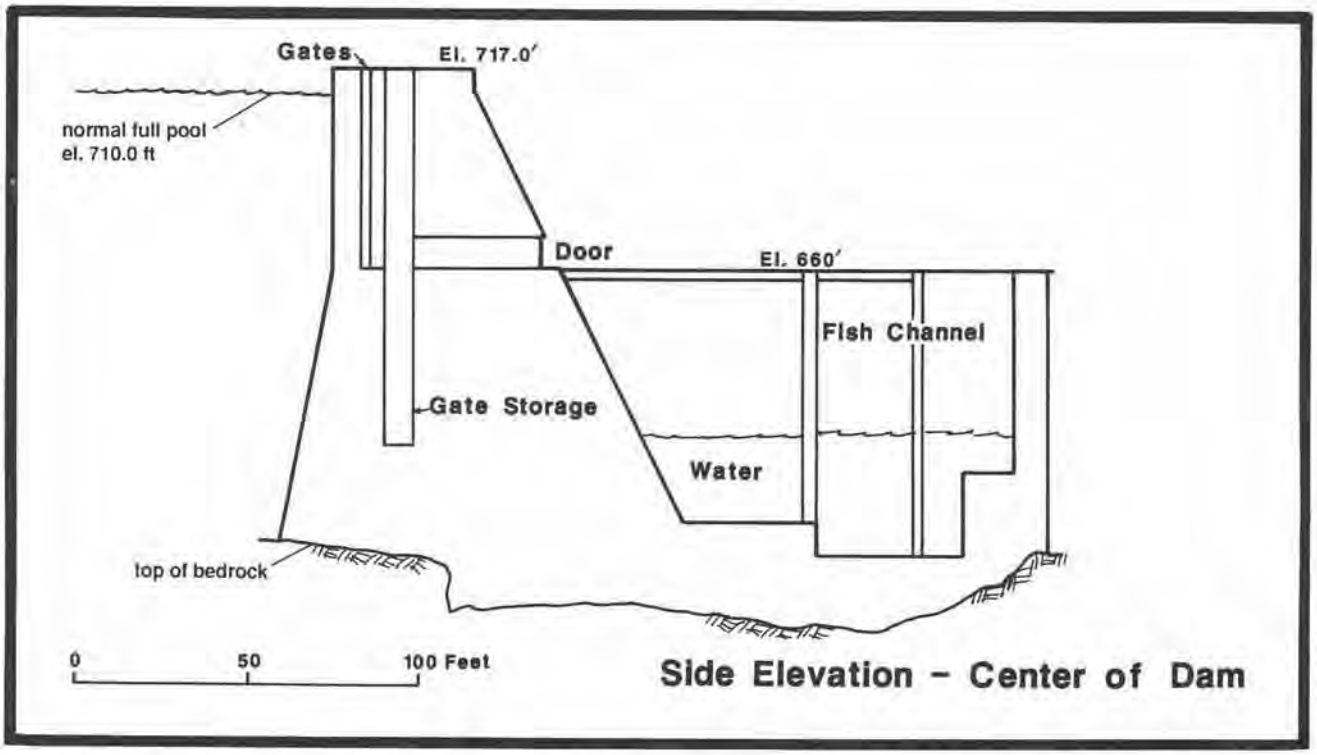


Figure 3. Side elevation of Rocky Reach Dam between the service bay and right bank in the area of the fish ladder.

ing seepage that would pass through or around it. To drain the lower gravel unit, a 60-in.-diameter open joint concrete pipe 450 ft long and with suitable filters was constructed immediately downstream of the left abutment and along the inner downstream face of the training wall. The drain discharges through the training wall below river level. For another 600 ft the river bank was excavated to bedrock under water and a graded drain placed along the face of the lower gravels.

After completion of the project, an independent review board pronounced the cut-off and drainage system an unqualified success as demonstrated by the influence in the upstream and downstream piezometer levels (Turnbull and Peck, 1959). Swiger (1959) prepared a detailed report on the grout curtain.

ACKNOWLEDGMENTS

The writer acknowledges the cooperation of W. F. Swiger, Vice President, Stone & Webster Engineering

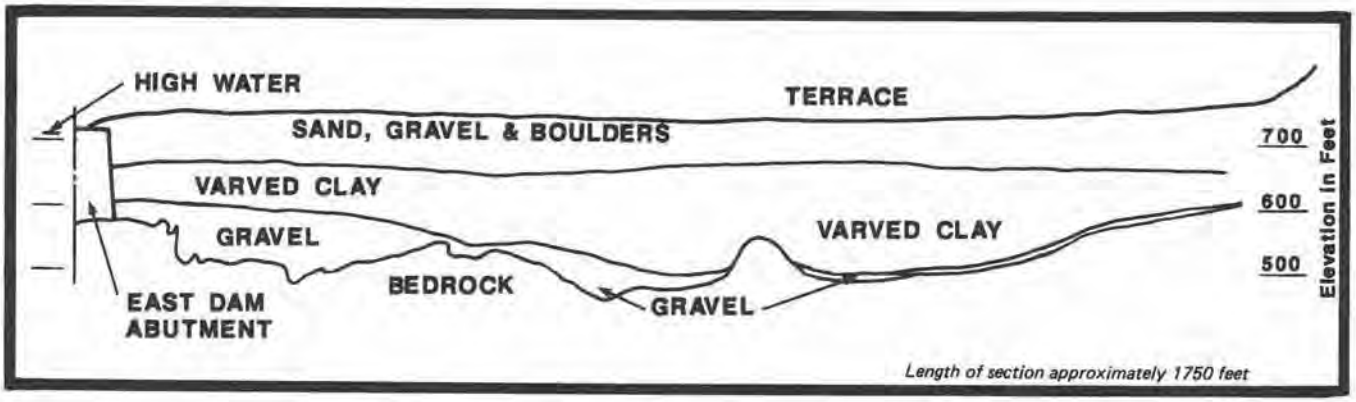


Figure 4. Profile along Rocky Reach Dam left bank cut-off; view upstream.

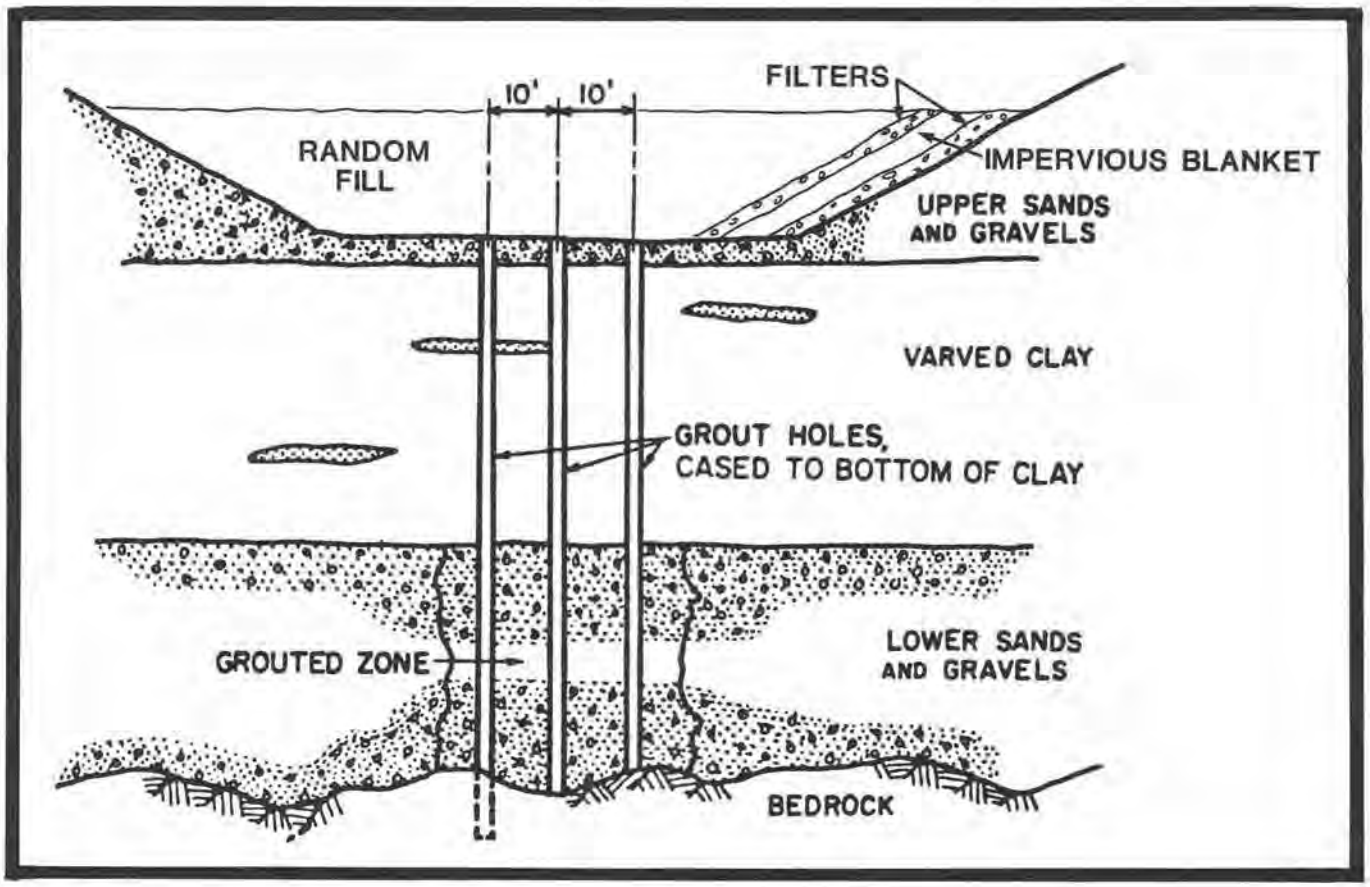


Figure 5. Section of Rocky Reach Dam cut-off. Downstream is to left.

Corporation, Boston, MA, for the many discussions both in the field and in the office during the construction of Rocky Reach Dam. Eldon Richman of Chelan County Public Utility District No. 1 was most helpful in supplying illustrative material.

REFERENCES

Mattinson, J. M., 1972, Age of zircons from the northern Cascade Mountains, Washington: *Geological Society of America Bulletin*, Vol. 83, pp. 3769-3783.

Swiger, W. F., 1959, *Report on east bank grouted cutoff, Rocky Reach hydroelectric power project*: Report to Public Utility District No. 1 of Chelan County, Washington, Wenatchee, WA, 40 p.

Turnbull, W. J. and Peck, R. B., 1959, *Report of results of grouting left abutment, Rocky Reach Dam*: Report to Public Utility District No. 1 of Chelan County, Wenatchee, WA, 14 p.

Waters, A. C., 1932, A petrologic and structural study of the Swakane gneiss, Entiat Mountains, Washington: *Journal of Geology*, Vol. 40, No. 6, pp. 604-633.

Rocky Reach project. This photograph, by the Public Utility District No. 1 of Chelan County, was taken in the summer of 1959 and shows the cofferdam needed to control the Columbia River. The downstream face of the powerhouse can be seen on the right.

Wells Dam

RICHARD W. GALSTER
Consulting Engineering Geologist

PROJECT DESCRIPTION

Wells Dam (Figure 1) was constructed between 1963 and 1967 at Columbia River mile 515.8, midway between Chelan and Pateros. The dam was engineered by the Bechtel Corporation and built by Douglas County Constructors (a consortium of Morrison Knudsen Co., Kaiser Engineers, Peter Kiewit Sons, and Perini Corp.) for the Public Utility District of Douglas County which operates the facility as a run-of-the-river hydroelectric project. It is the most recently completed project on the main stem of the Columbia within the United States.

The dam consists of a 1,130-ft-long central concrete hydrocombine section that includes a 10-unit powerhouse beneath the spillway, a design unique on the Columbia River. It has flanking zoned earth- and rockfill embankments. Concrete and embankment structures rise to elevation 797 ft, and normal reservoir level is elevation 779 ft. Fish passage and fish-rearing facilities are provided, the latter on the west bank just downstream from the right embankment. The hydrocombine unit is 196 ft high. Nearly half of this height is below the normal tailwater elevation of 712 ft.

SITE GEOLOGY

The valley floor at Wells Dam is about 4,000 ft wide. Prior to construction a 700-ft-wide river channel lay against the east valley wall. The east side of the valley rises sharply in a series of narrow terraces backed by granitic rock slopes to the basalt-mantled surface of the Waterville plateau at elevation 2,700 ft. The flood plain west of the river is about 1,000 ft wide at elevation 720 ft. This abruptly gives way to an intermediate terrace about 2,000 ft wide, which rises westward from elevation 750 ft to 775 ft to a steep rock face which serves as the west (right) abutment for the dam (Bechtel Corp., 1961). The west side of the valley rises to a fill terrace between elevations 875 and 890 ft. Behind this terrace a scabland is developed in the granitic valley wall. A major remnant of the "Great Terrace of the Columbia" (Flint, 1937) is present at elevation 1,200 ft on the west side of the valley where it is known as Gossman Flats (Figure 2).

The valley floor is underlain by an irregularly stratified sequence of glacial and fluvial deposits consisting of gravel and sand with local cobble and boulder units, silty sandy gravel, and lenticular units of fine sand and silt representing glacial lake deposits (Bechtel Corp., 1961). An extensive glacial lake deposit underlies the Columbia River gravels beneath the valley floor (Figure 3) but does not extend beneath the intermediate terrace. Till-like units of non-plastic silty sandy gravel dominate the eastern part of the overburden sequence and extensively overlie the bedrock surface. Permeability of the fluvial and glaciofluvial sands and gravels ranges from 2.08 to 5.58 ft/min (Bechtel Corp., 1961).

The configuration of the bedrock surface beneath and adjacent to the dam generally undulates between elevation 660 and 690 ft. Two major channels were identified below this range. The deeper is beneath the existing river channel and extends to below elevation 580 ft at the dam and somewhat deeper upstream (Figures 3 and 4). The other channel in the rock surface was beneath the right bank and extended to below elevation 630 ft.

The rock surface on both abutments rises steeply to elevations well above the dam. However, beneath the high terrace and beyond the rock face that forms the right (west) abutment, the rock surface was found to drop to at least elevation 354 ft, nearly 200 ft below the bottom of the contemporary river channel. This ancestral channel of the Columbia River is filled with glacial deposits to elevations well above the Wells reservoir (Bechtel Corp., 1961, 1973).

GEOLOGIC ASPECTS OF SITING AND DESIGN

The dam site was selected because of the availability of rock abutments in a valley reach midway between the head of the Rocky Reach pool and the tailrace of Chief Joseph Dam. The hydrocombine design was selected and the facility sited to take advantage of the highest and shallowest portion of the bedrock surface for foundation purposes. Embankment sections were designed to rest on the gravel overburden and span the two deep

Figure 1. Aerial view to the northwest of Wells Dam showing unique hydrocombine structure (center) combining spillway and powerhouse functions. Fish-rearing facilities are located downstream from the west embankment (left). Photo courtesy of Public Utility District No. 1 of Douglas County.

channels in the bedrock surface with appropriate cut-off walls extended to the bedrock. The west embankment is angled upstream to abut against a limited bedrock exposure, thus reducing the overall length of the dam and utilizing a bedrock abutment.

CONSTRUCTION

Stages

The dam was constructed in three stages, only one of which required major cofferdams. The hydrocombine was constructed between upstream and downstream embankment cofferdams connected on the east (river) side by a 14-unit cellular cofferdam. The embankment cofferdams included slurry trenches extending 15 ft into lake bed silt or to bedrock. A well point system was installed on the inside cofferdam slopes to improve stability. The cellular cofferdam was founded on bedrock. Simultaneously with construction of the hydrocombine, the west embankment was constructed "in the dry". Following completion of concrete work on the hydrocombine and a short section of the east embankment, both inside the cofferdams, the cofferdams were removed, and the river was diverted through the hydrocombine by crowding downstream and upstream "turning dikes" consisting of rock fill and gravel and built across the river channel. The remainder of the east embankment was then constructed without dewatering, partly by placement of uncompacted impervious fill under water.

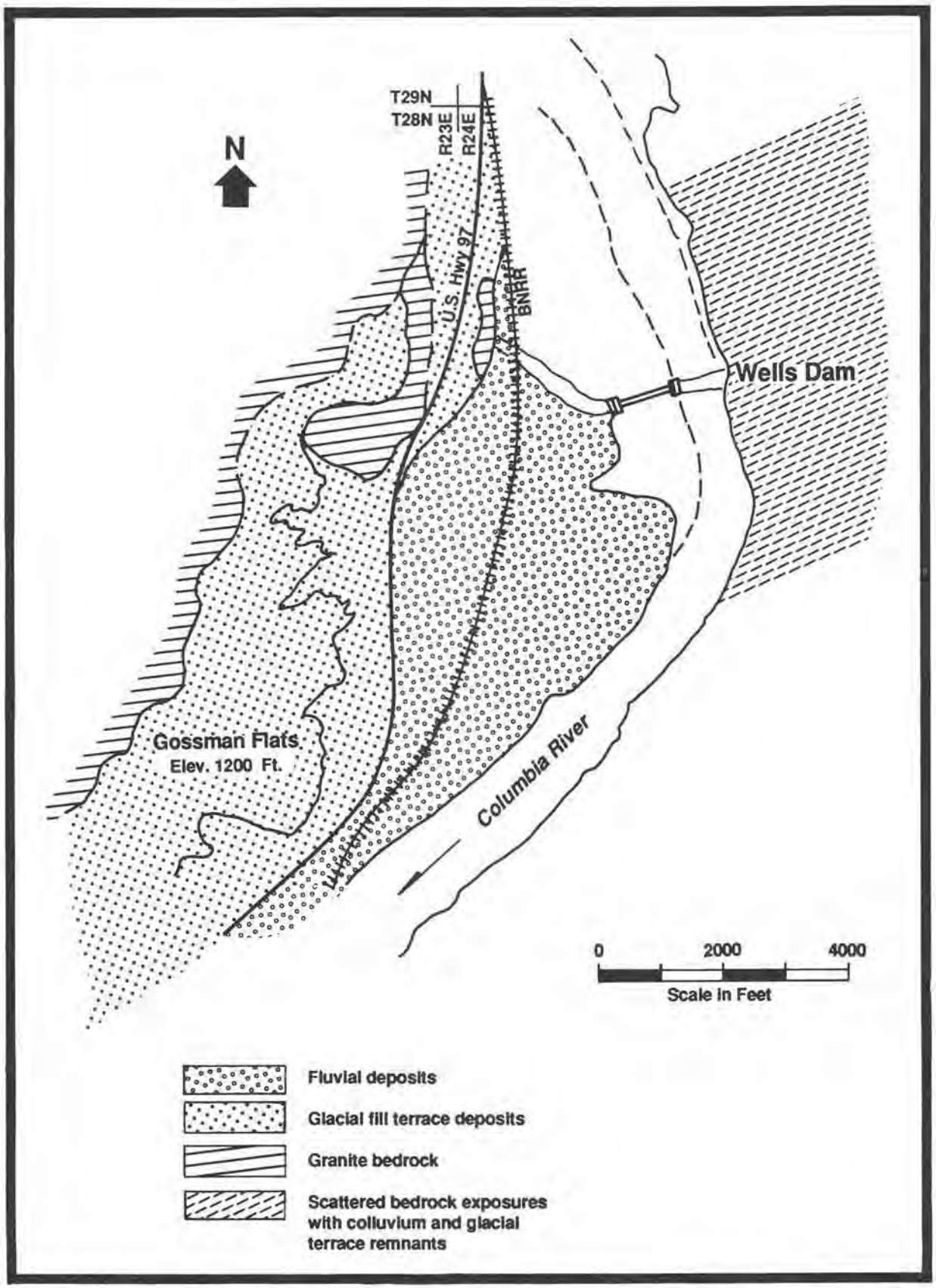


Figure 2. Generalized geologic map of the Wells Dam area. Based on U.S. Geological Survey Azwell and Wells Dam 7.5-minute topographic quadrangles. BNRR, Burlington Northern Railroad.

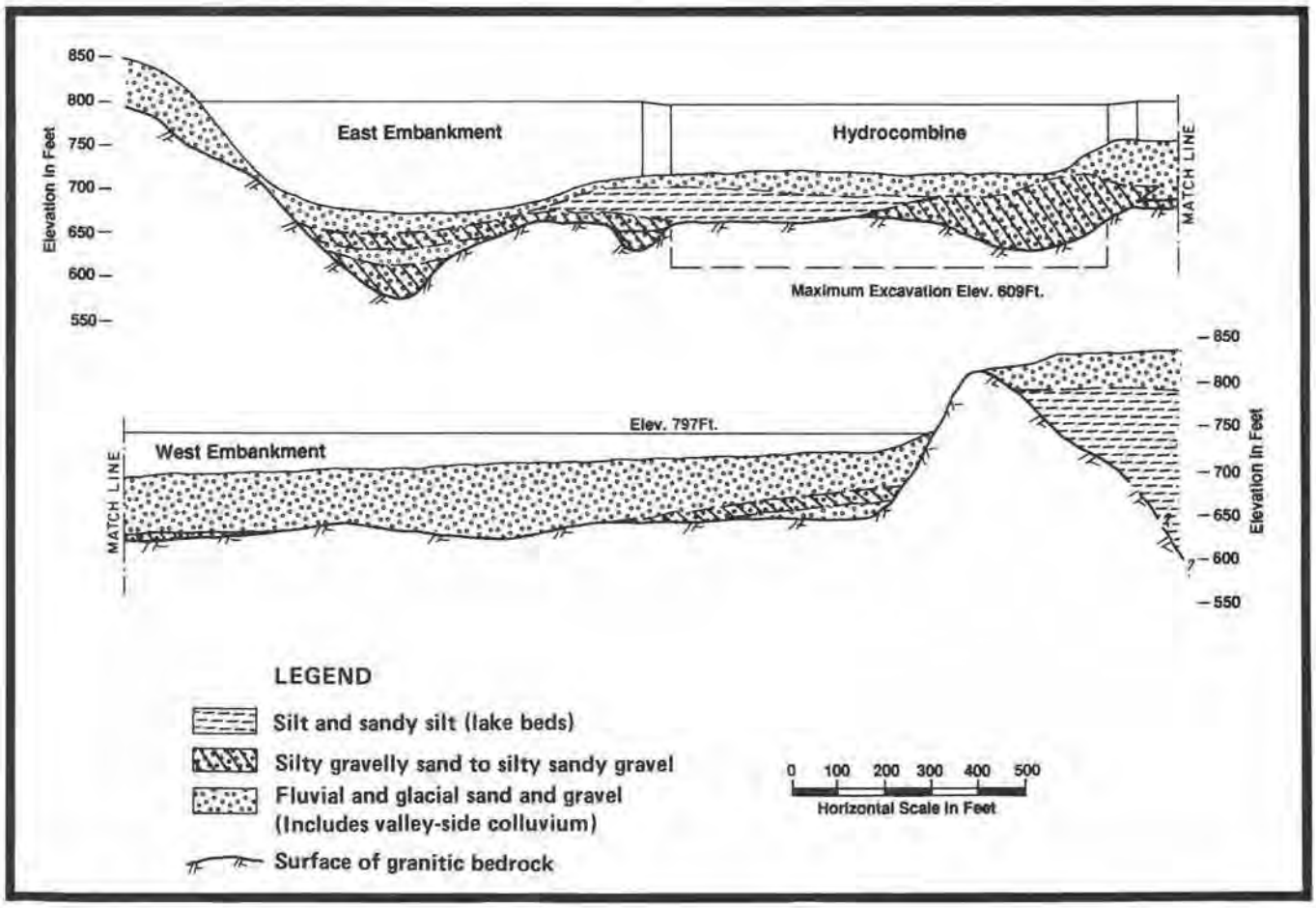


Figure 3. Generalized geologic section at Wells Dam. Surficial materials not shown. After Bechtel Corporation (1961).

Hydrocombine

The hydrocombine is founded on sound, generally unweathered granitic bedrock cut by numerous north-trending basic igneous dikes. Joint spacing ranges from 1 to 4 ft, although locally the rock is more closely jointed and fractured. Minor small, generally north- to northeast-trending shear zones are present and are locally characterized by as much as 2 in. of plastic gouge. Maximum rock excavation under the draft tubes was to elevation 609 ft. The rock elevation under the heel area varies between elevations 618 ft and 630 ft. A narrow shear zone under the central part of the structure was excavated to as much as 5 ft below grade and backfilled with concrete. A notch in the rock surface beneath the eastern part of the structure was similarly excavated and backfilled. Successful rock sculpting of the foundation was accomplished using controlled blasting methods. A single line grout curtain was installed near the heel of the structure to a depth of 40 ft below the foundation. Except in very isolated areas, the foundation accepted little grout (Bechtel Corp., 1973). Grout curtains were also emplaced around the peripheries of the fish ladder foundations.

Embankments

The west embankment is founded on Columbia River or glacial gravels. After fine-grained surficial material was stripped off, a core trench was excavated to elevations between 720 and 740 ft, and a backfilled slurry trench was constructed to the bedrock surface to cut off seepage. The impervious core rests directly on the bedrock surface toward the hydrocombine and above elevation 740 ft on the right abutment (Figure 3). The west rock abutment adjacent to the embankment core was grouted to depths of 40 to 60 ft behind the rock surface between elevations 800 and 690 ft.

Except for the short section constructed within the hydrocombine cofferdam, the east embankment was built without dewatering. Underwater excavation slopes were 1.75H to 1V between the turning dikes. The core

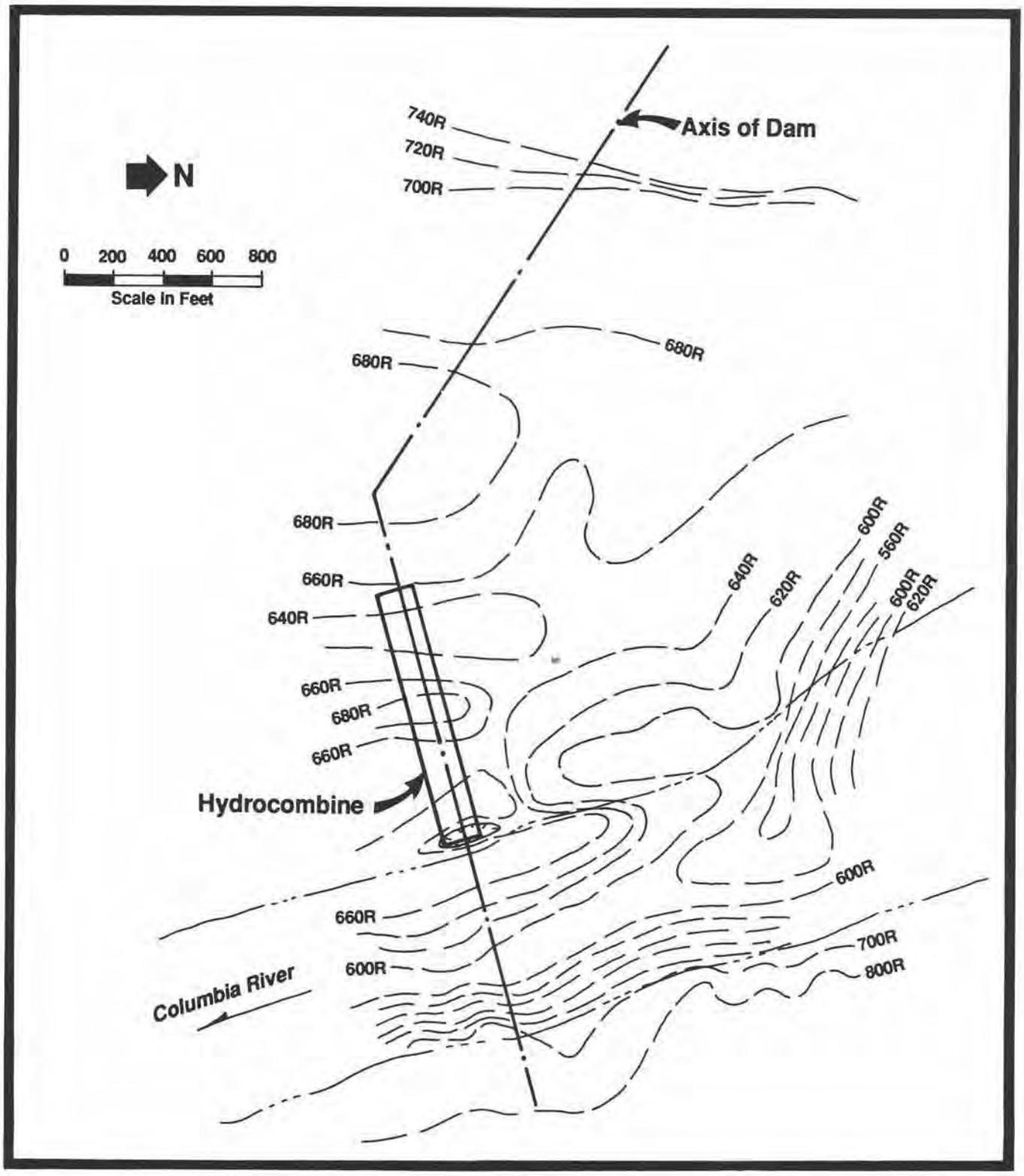


Figure 4. Bedrock configuration at Wells Dam. Rock surface contour interval 20 ft. After Bechtel Corporation (1961).

portion of the underwater fill was a special mixture of silt, sand, and gravel placed by a clam bucket to minimize segregation of material. The core was flanked by granular underwater fill. As no compaction was possible, significant settlement was anticipated and experienced during and following placement of overlying embankment materials above the water table. The east rock abutment adjacent to the embankment core was grouted to a depth of 50 ft behind the rock face between elevations 800 and 690 ft (Bechtel Corp., 1973).

Tailrace Channel

The channel connecting the tailrace with the original river channel downstream from the dam was underlain by fluvial sand and gravel to about elevation 695 ft. Below this elevation, glaciolacustrine silt and fine sand generally extended to bedrock. The gravel was removed by conventional excavating methods, and the side slopes were protected by appropriate riprap tied to the end walls of the concrete structure. The silt and fine sand were then removed by erosion during controlled spillway releases, first through the central gates and later by releases from the flanking gates. By this method a hydraulically efficient channel was brought to appropriate grade as much as 30 ft below the top of the silt. The addition of minor riprap along the channel sides was required during this process because of settlement, but no lateral erosion took place (Bechtel Corp., 1973).

Construction Materials

Except for cement and bentonite, all construction materials were obtained from required excavation or local borrow areas. Some riprap was derived from project-related highway relocation construction.

ACKNOWLEDGMENTS

I am indebted to A. B. Arnold of the Bechtel Corporation for providing much of the data upon which this summary is based. The cooperation of C. A. Cartmill, Chief Engineer, Public Utility District No. 1 of Douglas County, and C. G. Wolfe, formerly of Bechtel, is most appreciated.

REFERENCES

Bechtel Corporation, 1961, *Engineering Feasibility Report on the Wells Hydroelectric Project, Section II, Geology and Soils*: Bechtel Corporation, San Francisco, CA, 20 p., 6 plates.

Bechtel Corporation, 1973, *Construction Report for Wells Hydroelectric Project, Azwell, Washington:* Bechtel Corporation, San Francisco, CA, 393 p., 37 plates.

Flint, R. F., 1937, Pleistocene drift border in eastern Washington: *Geological Society of America Bulletin*, Vol. 48, pp. 203-232.

Chief Joseph Dam

Chief Joseph Dam, showing part of the intake dam and powerhouse. Blanks for future penstocks were provided in the intake dam during the original construction, and the project had a partly finished appearance between 1956 and 1978. Photograph by R. W. Galster, July 1974.

Chief Joseph Dam, showing excavation of two slots in the granite bedrock for installation of penstocks. Blanks for the future penstocks were provided during original construction. Photograph by R. W. Galster, January 1976.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Chief Joseph Dam

RICHARD D. ECKERLIN
U.S. Army Corps of Engineers
and
RICHARD W. GALSTER
Consulting Engineering Geologist

PROJECT DESCRIPTION

Chief Joseph Dam, named in honor of the famous Nez Perce Indian chief, is on the Columbia River in north-central Washington, 51 river miles below Grand Coulee Dam (Figure 1). The dam is a complex structure, 5,962 ft long, consisting of a 19-bay spillway dam that spans the river channel; intake structure with detached powerhouse; a gravity arch "buttonhook" section connecting the spillway and intake structure; a gravity "closure" structure connected to an earthfill section on the left (south) bank; and a wrap-around embankment at the right abutment (Figure 2). Construction was started in 1949, and the first of 16 generating units was placed on line in August 1955. Improved upstream basin storage permitted extension and completion of the powerhouse to 27 units together with raising the dam and reservoir an additional 10 ft between 1974 and 1980. Normal maximum reservoir elevation is 956 ft (U.S. Army Corps of Engineers, 1986). The reservoir (Rufus Woods Lake) backs water upstream to Grand Coulee Dam.

GEOLOGIC SETTING

The Chief Joseph project lies at the physiographic boundary between the Okanogan-Selkirk Highlands on the north and the Columbia Plateau on the south. No distinctive physiographic differences are to be seen on either side of the valley in this region because the characteristic flat-lying lava flows of the Waterville Plateau section of the Columbia Plateau are also present on the Omak Plateau north of the valley (Figure 3). Rocks in the Okanogan Highlands are a complex of metamorphic rocks intruded by Mesozoic granitic rocks. The Columbia valley has been modified by Pleistocene continental glaciers that moved south from the Okanogan-Selkirk Highlands across the trend of the valley onto the Columbia Plateau. The ice left a variety of glacial debris, including till, "dump moraine", outwash gravel and sand, and lacustrine silt, fine sand, and clay. The Columbia River has cut through the glacial sediments into granitic bedrock leaving a steep-walled, terraced inner valley within the broad, larger, older valley almost 1,000 ft deep and 12,000 ft wide. Fields of "lag" blocks litter many valley surfaces. Most of these are large basalt blocks plucked by the ice from the Omak Plateau, transported onto the glacial drift surface, and "let down" during the postglacial cutting of the valley. In the project area, the river is about 800 ft wide and flows along the base of the southern valley wall.

SITE GEOLOGY

The granitic bedrock surface exposed at the dam site gradually rises above the river channel north of the river. A glacial fill terrace extends from the river to the north valley wall. The terrace surface varies from elevation 1,030 ft adjacent to the river to more than 1,100 ft. A large bedrock hill protrudes above the terrace surface to elevation 1,200 ft, and a late-stage glacial outwash channel lies at the toe of the north valley wall (Figure 4). Till, morainal materials, and glaciofluvial and glaciolacustrine sediments fill abandoned channels of the Columbia River and many of the tributary canyons. On the south side of the valley the bedrock surface rises well above the dam.

The dam is founded on hard, competent crystalline rocks that include granodiorite, granodiorite gneiss, dark schistose granodiorite, hornblende granodiorite, and lamprophyre (U.S. Army Corps of Engineers, 1957). Joints, foliation, and faults influenced rock behavior during rock excavation. Joints are attributed to compression and shear coincident with and immediately following granodiorite emplacement. Many joint faces are slickensided and healed with chlorite, but they show no evidence of strain after chloritization. Faults and fault zones are common throughout the dam foundation and are attributable to late stages of the Mesozoic granodiorite intrusion and concurrent orogeny. The most common fault orientation strikes approximately N60°W and dips 70°NE. Fault gouge is a few inches or less in thickness.

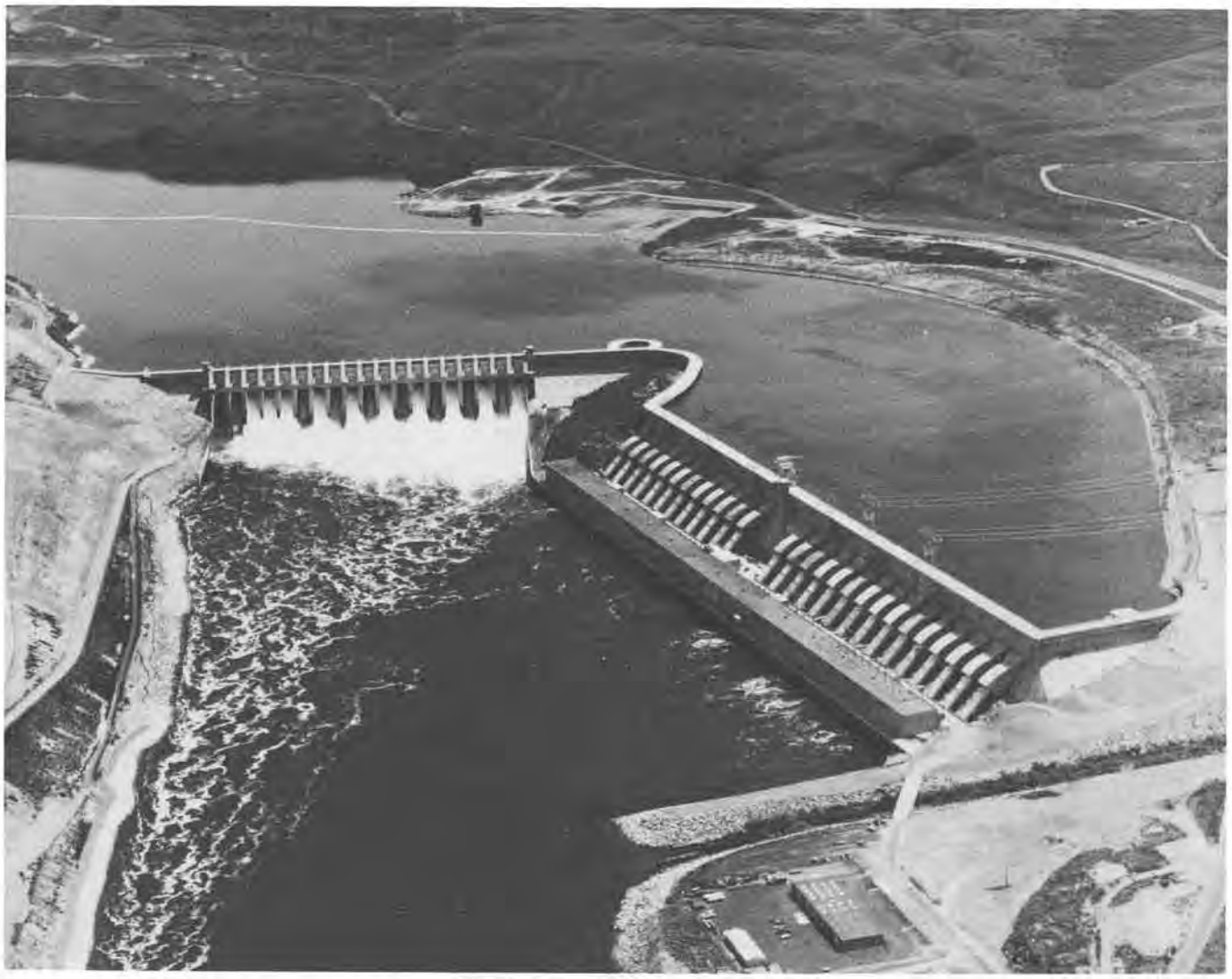


Figure 1. Chief Joseph Dam. View upstream showing the spillway dam abutting a glacial terrace (left) and the intake structure/powerhouse (right) connected by the gravity arch "bottonhook" section (center). Foster Creek enters the Columbia River in an artificial channel at the lower right. Pearl Hill Road (Douglas County Road 321) ascends across the Bridgeport slide beyond the forebay. Photo courtesy U.S. Army Corps of Engineers, May 1981.

GEOLOGIC ASPECTS OF SITING AND DESIGN

Originally referred to as the Foster Creek site in the "308 Report" and earlier studies (U.S. Congress, 1934), the site of Chief Joseph Dam was chosen for economic and geologic reasons. The inner canyon of the Columbia valley in the 51 somewhat sinuous miles between Grand Coulee Dam and Chief Joseph Dam is generally narrow and has little or no flood plain, yet it lacks good dam sites owing to the complexity of glacial deposits that occupy much of the inner valley. A site was considered about 14 mi upstream, but preliminary assessment determined it to be inferior from topographic and geologic standpoints. The Chief Joseph site lies at the downstream end of this remote segment of the valley where it enters the south end of the broad, glaciated Okanogan River valley (Okanogan Trench) (Figures 3 and 4). Sound granite was exposed on the left (south) bank and appeared to be at shallow depths beneath the river channel. Moreover, there was space for construction of extensive powerhouse facilities with a minimum of excavation. The 250-ft-high bluff rising to a broad glacial terrace on the right bank of the river was known to be composed of a heterogeneous mix of glacial drift, but it was believed to contain sufficient clay to prevent serious abutment leakage (U.S. Congress, 1934).

More detailed geologic investigations during the mid- to late 1940s revealed the true character of the gla-

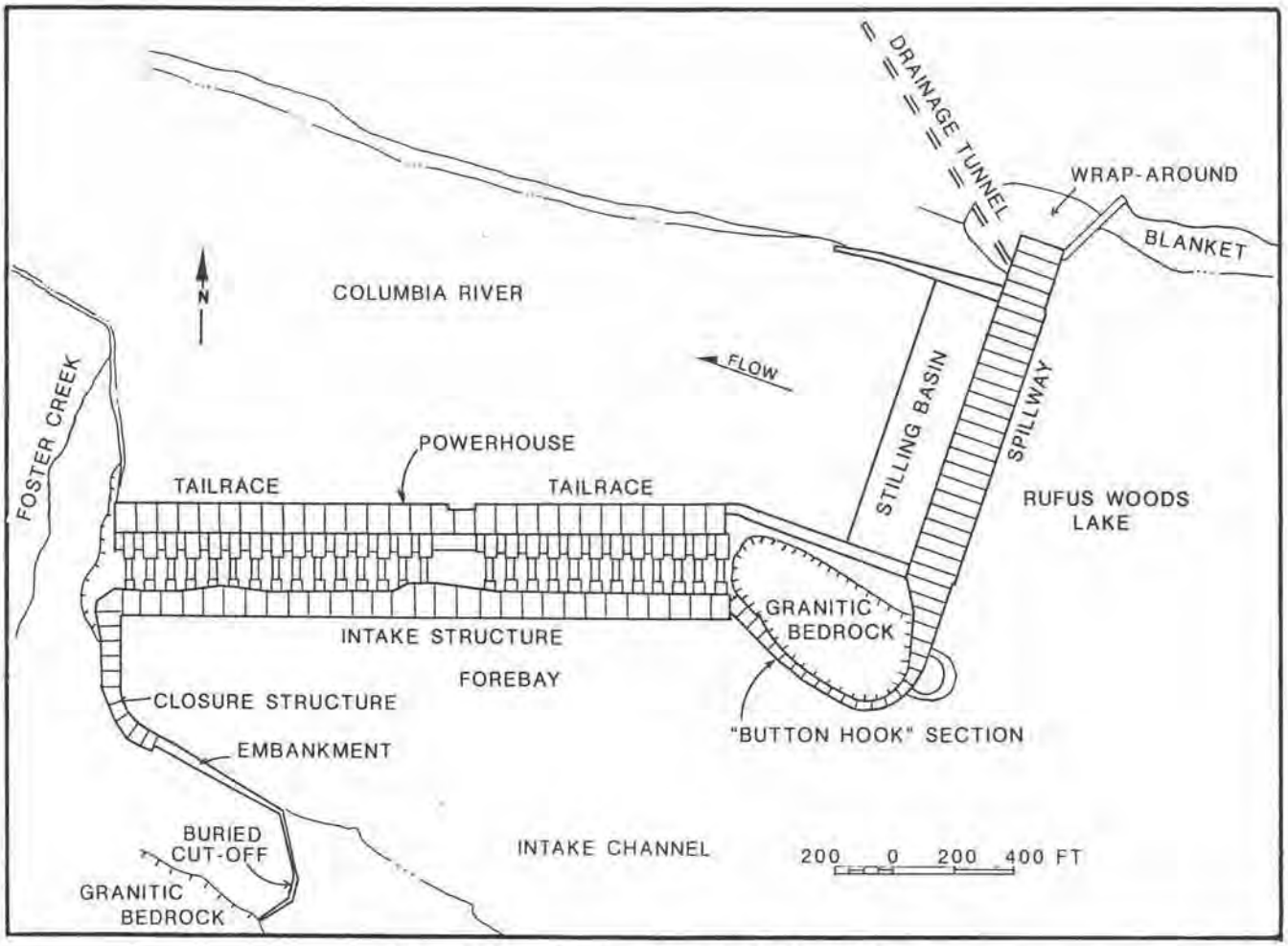


Figure 2. General project layout of Chief Joseph Dam.

cial sequence in the right bank. The sequence consists of an extensive deposit of torrentially cross-bedded, largely openwork gravel and cobbles that is 10 to 100 ft thick and directly overlies bedrock. This obviously highly pervious unit is overlain by a varied thickness of "dump moraine" and more than 100 ft of till. The dump moraine consists of large boulders and blocks of basalt and Latah sediments (siltstone and sandstone) in a varied matrix of locally bedded sand, gravel, silt, and clay. The till is a silt-rich, sandy, gravelly lodgement-ablation till. The design and construction of the right bank leakage control feature was, for the time, unique. It consisted of a 2,000-ft-long upstream blanket and a 1,000-ft-long relief tunnel (adit) located just downstream from the dam (Figure 2).

Because the project required a long (2,039 ft) powerhouse, advantage was taken of the bedrock configuration between the site of the spillway and the mouth of Foster Creek. An intake structure was designed to rest at the top of a 1H-to-1V rock slope, and surface penstocks would extend to the powerhouse at the toe of the rock slope. The great length of the powerhouse and intake structures required a complex closure of the forebay in order to tie the dam to a granite knob on the south side of the valley (Figure 2). The spillway and intake structures were connected by a curved gravity section, nicknamed the "buttonhook", that was sited on the large granite knob that forms the left abutment for the spillway. Jointing required design of grouting and drainage facilities beneath and in the spillway dam. Uplift drainage in the intake structure was provided for by a manifold system discharged through the penstocks.

ORIGINAL CONSTRUCTION

Stages

Two basic diversion stages were required for construction of the spillway dam and stilling basin that span the river channel. The first stage diversion allowed construction on the north half of the spillway. The river was then diverted through the low blocks in the partly constructed northern section, and the south half of the

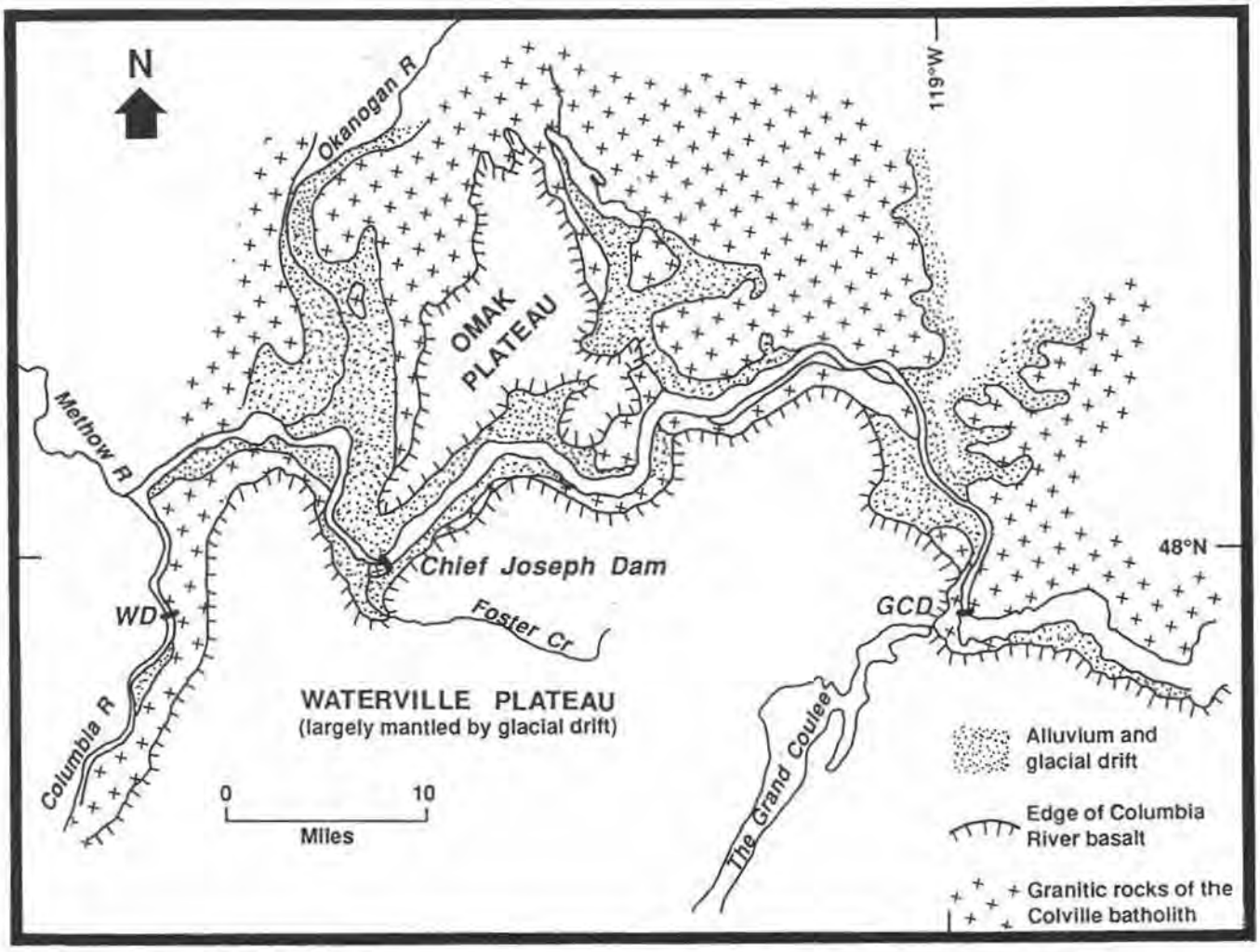


Figure 3. Regional geologic setting of Chief Joseph Dam. GCD, Grand Coulee Dam; WD, Wells Dam.

spillway was constructed behind cellular and earthen cofferdams. The right bank blanket and the powerhouse excavation proceeded simultaneously with spillway construction behind earthen cofferdams (Figure 5). Intake channel excavation and foundation preparation for the intake structure, closure structure, and buttonhook section were all above river flood levels and proceeded simultaneously with activities requiring diversion.

Excavation and Foundation Preparation

Spillway Dam

The bedrock surface beneath the river channel was found to be at a general elevation of 750 ft. The surface was smooth and waterworn and had relief of about 5 ft (Figure 6). Locally, river-bed erosion extended to depths of 20 ft along fault zones (U.S. Army Corps of Engineers, 1957). The river gravels varied in thickness by as much as 20 ft, and basalt lag blocks as much as 20 ft in diameter were found in the channel gravels. Minor rock excavation was needed beneath the spillway dam except where fault zones required treatment or excavation was necessary to provide minimum mass concrete between the foundation and the designed drainage gallery floor elevation. However, the rock surface in the stilling basin was excavated to elevation 738 ft. The left abutment of the spillway dam and the foundation for the buttonhook section required substantial rock excavation to produce a foundation in which the toe of the spillway dam was at a higher elevation than the heel. Foundation cleanup by air-water jets was normally adequate; however, some gouge was hand-mined from fault zones, which were then backfilled with concrete.

Intake Structure

The undulating bedrock surface in the area of the intake structure and the requirement for a foundation grade generally between elevations 840 and 864 ft

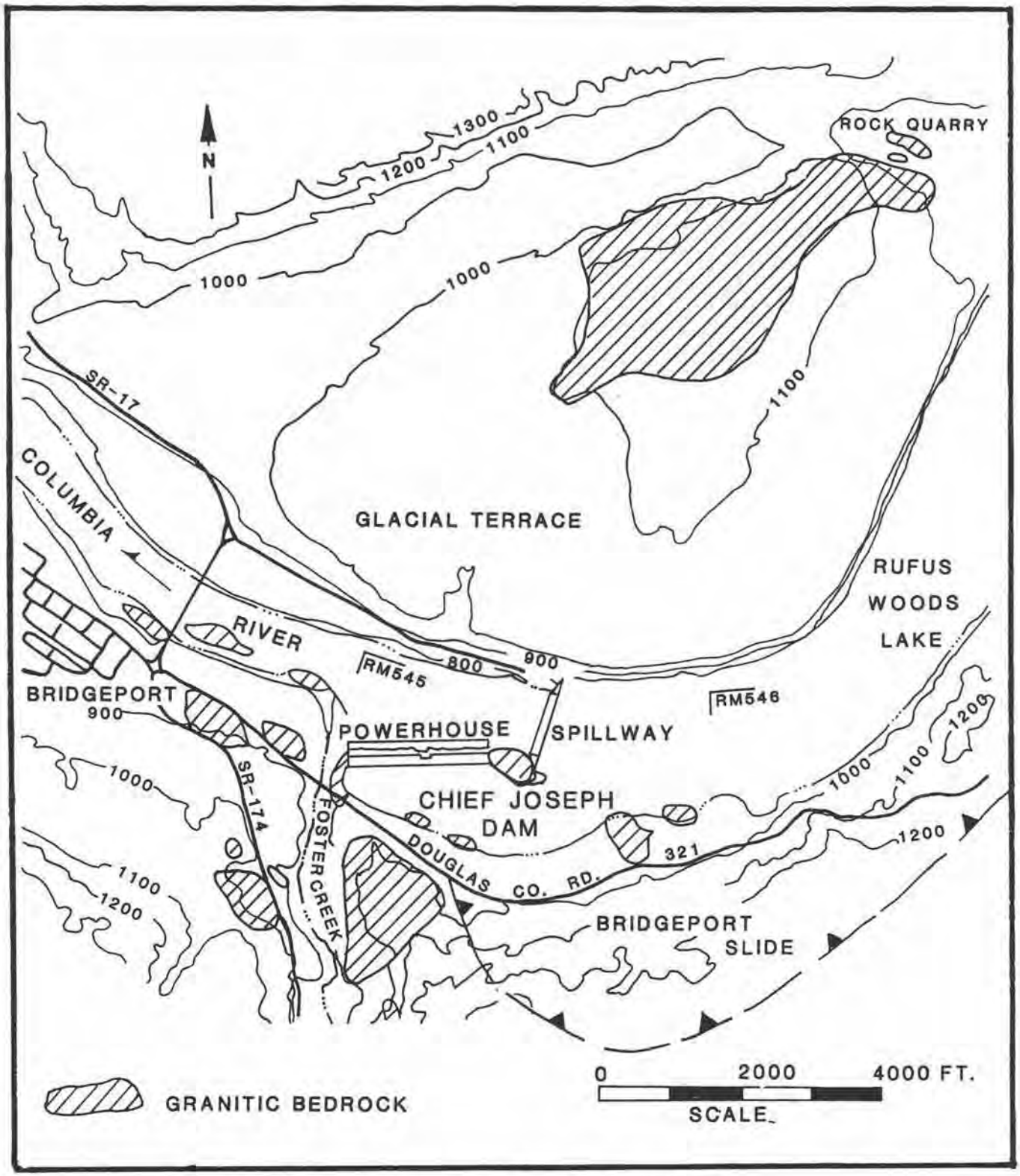


Figure 4. Generalized geology at the Chief Joseph Dam site.

Figure 5. Chief Joseph Dam. View upstream during construction (March 1952) at the beginning of stage 2 diversion. Monoliths for the north half of the spillway are partly constructed (left), and the river is being diverted through the low blocks. Preparation of the right abutment is under way (left), and construction of the right bank blanket can be seen behind dikes upstream from the partly completed spillway. Dewatering for stage 2 construction of the spillway is incomplete. Excavation for the powerhouse and intake structures is at the right. Rock ribs between penstock slots were removed prior to actual construction of the intake structure. Photo courtesy of U.S. Army Corps of Engineers.

necessitated considerable rock excavation. A bedrock low in the center section of the intake structure required the foundation to be as low as elevation 813 ft. Locally, till was present in depressions in the bedrock surface, and its removal commonly required blasting. Mass rock excavation methods were used in this excavation as well as in development of the penstock slots below the intake structure. The resulting rock ribs between the penstock slots (Figure 5) were judged to be unstable, however, and were removed prior to powerhouse construction.

Powerhouse

The powerhouse configuration required considerable rock excavation down to about elevation 720 ft (Figure 7). The mass rock method of excavation resulted in overbreak and locally caused opening of joints in the rock mass necessitating removal of additional rock. Although some controlled blasting methods (for that time) were tried, results were not effective. Several faults in the powerhouse foundation were treated by overexcavation and concrete backfill.

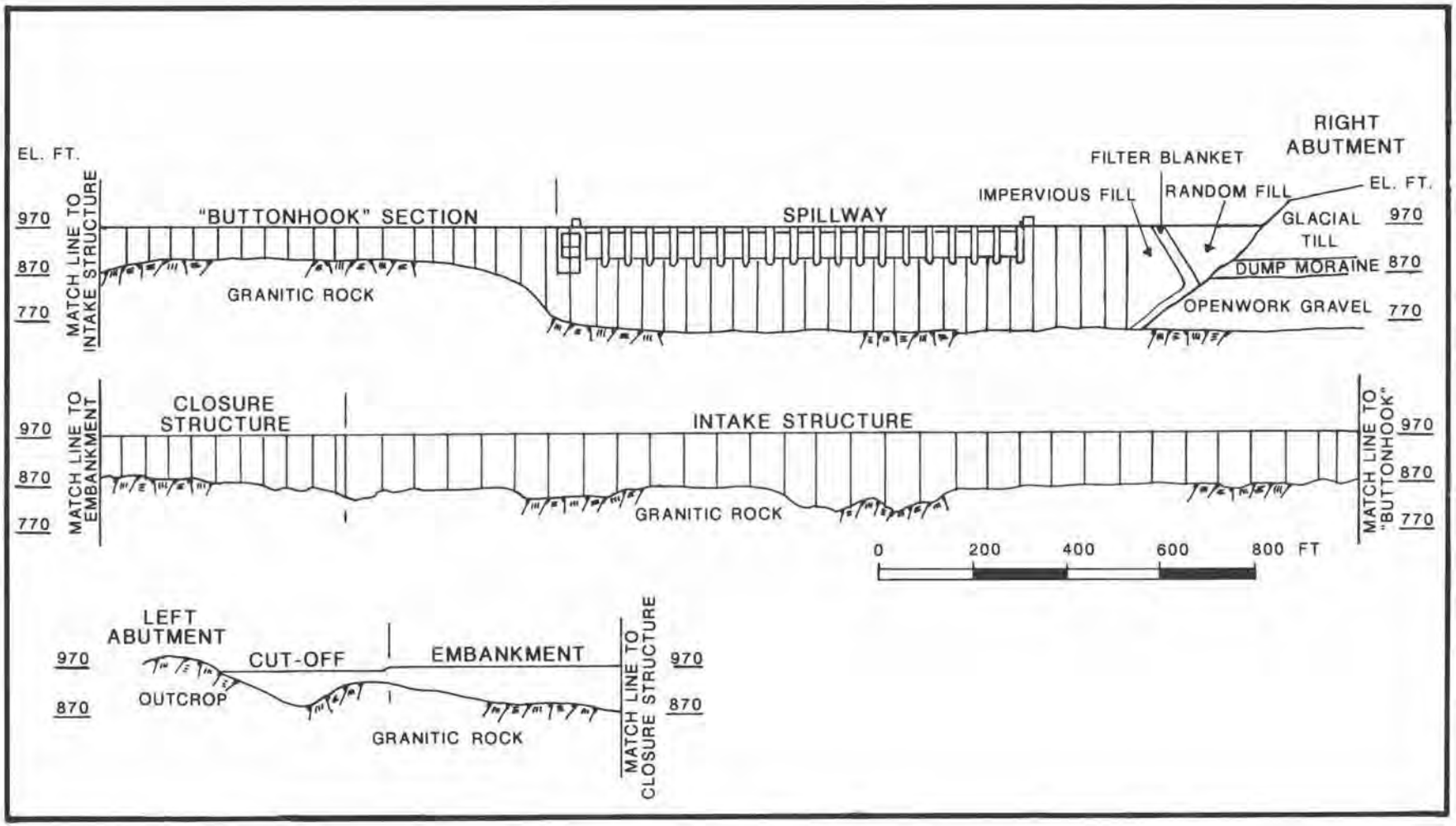


Figure 6. Geologic profile along the axis of Chief Joseph Dam; view downstream.

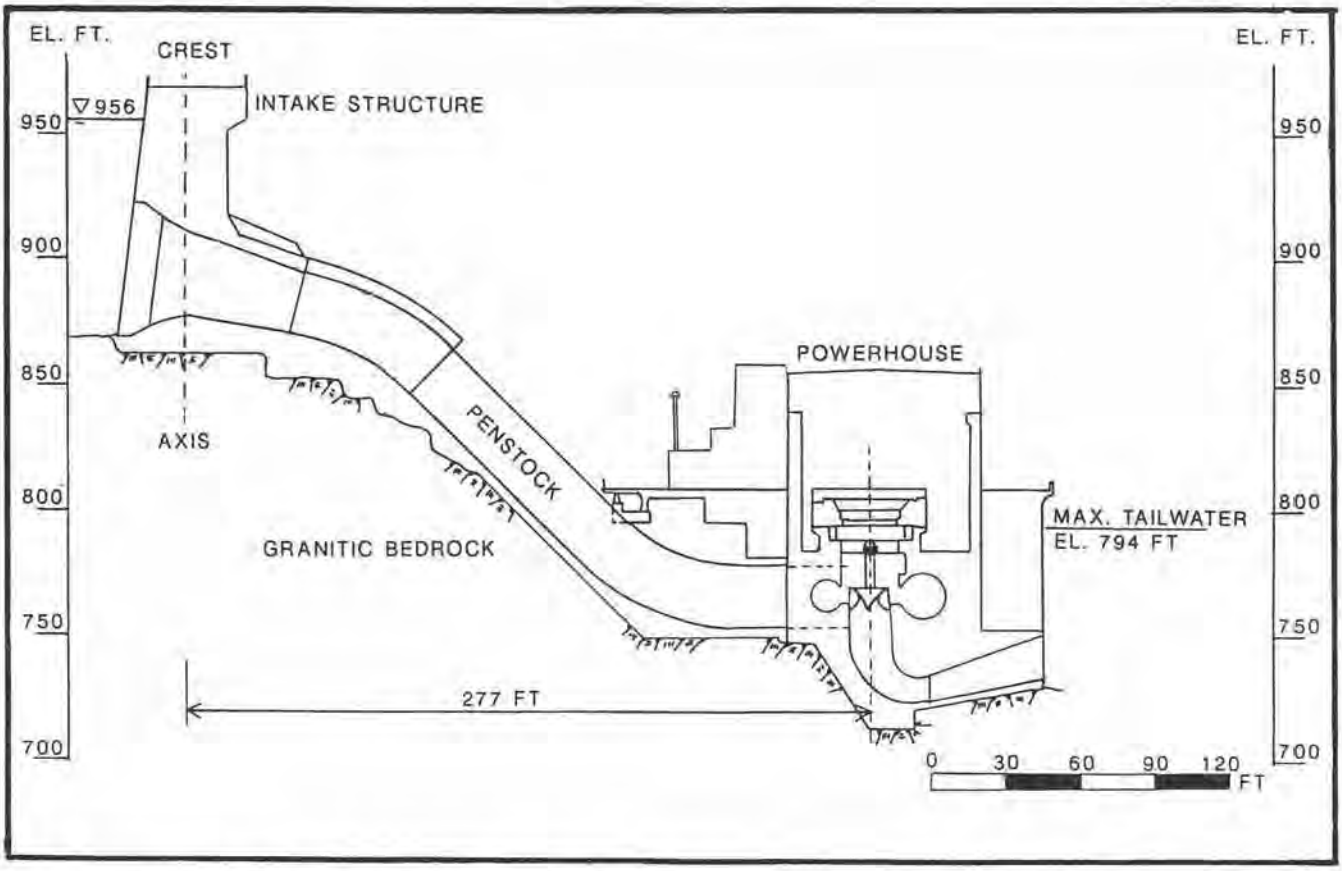


Figure 7. Typical geologic section through the intake structure and powerhouse at Chief Joseph Dam.

Closure Structure

The closure structure consists of a concrete section and a zoned embankment section, both founded on a somewhat irregular bedrock surface that was covered by glacial outwash to depths as great as 70 ft. The end of the embankment was tied to a granite knob, which forms the true abutment of the dam. The tie consists of a buried closure wall of zoned earth and rock fill with an impervious core resting, in part, on a concrete core wall extending to the bedrock surface. Several deeply eroded valley-parallel faults pass through this area; these required considerable overburden excavation and shaping of portions of the rock surfaces to create an appropriate foundation configuration.

Intake Channel

The intake channel (forebay) required excavation of sand and gravel (glacial outwash) and as much as 75 ft of rock excavation. Excavation was accomplished in 3- to 25-ft lifts. The material generated from this very large excavation and from the powerhouse and intake structure excavations was wasted in a fill across the lower mile of the Foster Creek valley. Engineers believed that the normal stream discharge would percolate through the waste rock fill and into the Columbia River.

Right Abutment and Blanket

A keyway was developed at the right abutment, sloped about 1.5H to 1V in the cobbles and gravel and about 0.5H to 1V in the overlying till and dump moraine (Figure 6). The till required blasting and was excavated in 24-ft-high lifts. The embankment section, which was constructed at the dam abutment, consisted of an impervious fill (composed of recompacted till) wrapped around the base of the three northernmost concrete monoliths of the spillway dam and tied to the upstream blanket. The impervious fill is separated from random fill in the keyway by a filter blanket (Figure 6). The impervious blanket extending 2,000 ft upstream was placed behind puddle dikes after the gravelly slope to the bedrock surface had been excavated (Figure 5). The bedrock surface was waterworn and had about 10 ft of relief. The blanket, composed of recompacted till from adjacent sources, was placed in the dry. Some moisture was added to attain the 10 percent optimum moisture content. Drainage of the gravelly slope was required prior to and during blanket placement. The top of the 10-ft-thick blanket was brought to elevations between 870 and 950 ft, depending on local geologic conditions (U.S. Army Corps of Engineers, 1957).

Relief Tunnel

The relief tunnel (adit) was driven as a 10-ft x 16-ft bore using a Conway mucker. A total of 900 ft of the tunnel length was underground; the outer 100 ft was constructed in open cut. Support was by 2-in. x 8-in. forepoles and 10-in. x 10-in. wooden timber supports. The wood supports were left in place. The final interior dimensions of the concrete-lined tunnel are 5 ft by 8 ft. The tunnel floor rises from elevation 778 ft at the portal to about 787 ft at the back. Relief wells, 3 ft in diameter, were hand excavated, generally to bedrock, at 50-ft intervals along the length of the tunnel; steel casing was used for support. A 12-in.-diameter slotted wood stave pipe was installed in each well as the casing was withdrawn, and the annular space was backfilled with gravel. The entire tunnel was driven in the lower gravel-cobble unit, although a sand unit in the usually openwork gravel was encountered in the back half of the tunnel and required constant bulkheading to prevent the working face from running. The relief tunnel contains 22 relief wells and has a discharge capacity of 100 cfs. The tunnel discharges into the spillway apron.

Grouting and Drainage

A continuous grout curtain was installed in the bedrock beneath the concrete sections of the dam. The depth of the curtain extends 80 ft beneath the spillway, 55 ft beneath the buttonhook, 50 to 75 ft beneath the intake structure, and 55 ft beneath the closure structure. Grout holes were angled upstream about 15° from vertical under the spillway, closure structure, and central part of the intake structure where grouting galleries were built into the structures. For most of the intake structure, however, the grout curtain was installed from outside the heel, angled downstream 5° from vertical. Similarly, drain holes were drilled to depths of 40 ft beneath the spillway and 28 ft beneath the buttonhook and closure structures. As no drainage gallery is present in most of the intake structure, eight drain holes per monolith were fanned from the penstock apertures in the upstream one-third of each monolith, each group of holes is connected to a collector pipe embedded in the concrete and drains to the downstream side. Only in the central section of the intake structure were drain holes drilled from galleries. Drain holes beneath the spillway were angled downstream 15° from vertical. Elsewhere, they were drilled in the plane of the dam axis. All grout and drain holes were angled in the plane of the dam axis to intercept an optimum number of joints.

Grouting beneath the spillway was done by zones; primary holes were on 20-ft spacing, and secondary and tertiary holes were as close as 2.5 ft. Stage grouting was employed throughout the intake and closure structures. Grout pressures varied from 150 to 200 psi, depending on zone and stage, and grout takes (sacks per lineal foot) varied from an average of 0.2 beneath the spillway to between 0.3 and 0.4 under the intake structure and 0.18 under the closure structure.

MODIFICATION AND COMPLETION

General

Eleven additional powerhouse units were added to the powerhouse and the entire dam was raised 10 ft between 1974 and 1980. The modification included raising the spillway, buttonhook, closure, and intake structures and removing and rebuilding the spillway piers. To accommodate the 11 additional units, the powerhouse was extended to a length of 2,039 ft and penstocks were added to the 11 spaces provided during the initial construction of the intake structure. Powerhouse skeleton bays 17 through 20 had been provided during the initial construction.

River Diversion

Powerhouse units 21 to 27 required excavation in the dry. This was done behind a combination cellular and embankment cofferdam between the original powerhouse and the right bank of Foster Creek (Figure 8). Foundation excavation for the 60-ft-diameter cellular cofferdam units was by dragline. Large boulders required blasting. The pile cut-off wall in the embankment cofferdam section did not extend to the rock surface. Thus, an extensive dewatering system and monitoring of phreatic levels within the cofferdam were required throughout the construction period. In December 1975 during a high spillway discharge, a leak occurred through the northwest corner of the embankment cofferdam section. Pre-dam topography indicated that an old channel of Foster Creek, about 50 ft east of the leak area, was apparently filled during original powerhouse construction. A seepage interceptor system was built to insure continued stability of the cofferdam.

Two watertight floating cofferdams, each 153 ft long and 74 ft high, were used to progressively dewater spillway bays to permit spillway pier modification. The cofferdams were built adjacent to the reservoir in such a way that they could be launched "on their backs" in the flat position. They were towed to the spillway area. Airtight chambers were selectively flooded to bring the cofferdams into the vertical position against a portion of the face of the spillway. As piers were progressively modified, the cofferdams were detached and moved to successive positions.

Treatment of Intake Slope

Prior to any excavation for the powerhouse addition, the 1H-to-1V rock slope between the intake structure and the powerhouse was strengthened by installation of deep rock bolts and drains. This was required by the inaccessibility of the grout and drainage system beneath the intake dam and by the need to enhance the stability

Figure 8. The powerhouse extension at Chief Joseph Dam; view across the cofferdam to the penstock slot excavation and intake structure showing penstock blanks provided during original construction. Completion of powerhouse skeleton bays is in progress (far left). Controlled blasting techniques allowed rock ribs to remain between penstock slots in which penstocks were later placed and encapsulated in concrete. Photo taken in December 1975; courtesy U.S. Army Corps of Engineers.

of the slope under conditions of a higher intake structure and reservoir. Bolts ranged in length from 40 to 75 ft; the upslope bolts extended beneath the downstream one-third of the intake structure foundation. The 2-in.-diameter bolts with double cone and shell anchors were tensioned to 110 kips. All bolts were grouted. Drain holes 90 ft deep, with the upslope drains extending beneath the middle of the intake structure foundation, completed the slope treatment.

Removal of Spillway Piers

The raising of the spillway required construction of wider, higher, and stronger spillway piers and narrower and higher spillway gates in order to operate with 10 ft of additional head. The weir crest elevations of the spillway were not changed (elevation 901.5 ft). For structural engineering reasons the upper 42 ft of each of 18 spillway piers and 2 end piers were removed to elevation 915 ft prior to the rebuilding to elevation 974 ft. The removal work was done behind floating cofferdams described earlier and using tightly controlled blasting techniques established and monitored by the Corps of Engineers' geologic staff, which was normally charged with blast design for construction projects. Demolition required zero damage to concrete remaining in the spillway, control of the blast debris, and cofferdam stability. A contractually established vibrational velocity limitation of 4 in./sec (vector sum) on concrete to remain in place, close review of blast plans, and instrumental monitoring of every blast allowed control of the demolition. Using a combination of pre-split and cushion blasting methods together with line drilling, most piers were demolished in a series of 8 or 10 blasts each; the lower 1 to 3 ft of a horizontal line-drilled/pre-split surface were mechanically removed. The removal was complicated by gate guides and the presence of a cage of steel reinforcing in the outer portion of each pier. The method used for demolition illustrates the suc-

cess in modifying a working dam employing restrictive and closely monitored blasting methods (U.S. Army Corps of Engineers, undated).

Powerhouse Extension

Approximately 340,000 cy of overburden and 97,000 cy of rock were removed from the excavation area. Powerhouse rock excavation was stringently controlled through contract specifications and blast monitoring. The proximity of the rock excavation to the intake structure and the necessity to maintain the structural integrity of the adjacent rock slope required limitations to be placed on ground vibrations generated during blasting. The maximum peak particle velocities (vector sum) allowed were 2 in./sec in rock at a minimum distance of 20 ft from the design rock face and 4 in./sec in concrete. During excavation work, the construction plan was modified to use natural rock piers to found the powerhouse service bridge in lieu of concrete (Figure 8). The rock piers were shaped using controlled blasting procedures; however, joints exercised considerable control of the excavation. Where the chlorite-bonded joints were broken by blasting, the joint surfaces offered little resistance to slippage. A complicated array of rock bolts and dowels was systematically installed as rock excavation progressed downward (U.S. Army Corps of Engineers, 1988).

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Foster Creek Flood

Foster Creek is an underfit stream that follows a Pleistocene glacial meltwater channel from the surface of the Waterville Plateau northward, entering the Columbia River immediately downstream from the Chief Joseph powerhouse. Although it has a large drainage basin, its normal flood flow probably does not exceed a few hundred cubic feet per second. For this reason, the lower part of the creek valley was used as a waste area for excess rock fill generated by excavation during original construction of the dam. This was done in the belief that the normal creek flows would percolate through the rock fill enroute to the river. On February 26, 1957, a concentrated storm dropped a heavy rain in the drainage basin, which was already snow covered. The result was an unusually large discharge down Foster Creek, which removed considerable amounts of rock fill to depths as great as 100 ft, deposited the debris in the river, raised the tailwater level at the powerhouse, and exposed portions of the ground mat. The erosion severed the Pearl Hill Road (Douglas County Road 321) and access to the dam and powerhouse from the south bank. This necessitated bridging Foster Creek in two locations and removing several thousand cubic yards of debris from the river in order to return tailwater to its former level (U.S. Army Corps of Engineers, unpublished data). Foster Creek has subsequently been provided with a channel through the project area.

Right Abutment

A zoned wrap-around embankment consisting of an impervious core, filter, and random fill serves to limit seepage and tie the concrete dam to the right abutment. Seepage control features consist of an impervious blanket, wells, and a relief tunnel. Since initial raising of the reservoir in 1955, flow from the tunnel has gradually diminished from a maximum of 93 cfs to the present average of about 25 cfs. Study of piezometer data between 1955 and 1972 showed a rising piezometric surface in the abutment just upstream from the dam, behind the impervious blanket. A study of ground-water temperatures in relation to cyclic reservoir temperatures indicated that the raised level was caused by a zone of high permeability near the top of the original impervious blanket about 1,000 ft upstream of the dam. Construction records note that the blanket was left low in this area because of the low elevation of apparently impervious glacial deposits. In 1976, the impervious blanket extending from the dam to 1,300 ft upstream, was raised from elevation 870 ft to 940 ft. The blanket was extended underwater by lowering buckets of silty gravel below the water to the working surface. Since raising and extending the blanket, piezometer data (through 1987) indicate that the piezometric surface has stabilized. Regular observation of existing piezometers in the abutment area serves to track the piezometric head within the aquifer (U.S. Army Corps Engineers, 1986).

Left Abutment

The left abutment embankment has remained stable since construction. A buried cut-off wall approximately 416 ft long connects the zoned embankment to the rock abutment. The buried cut-off wall is founded on bedrock and consists of an impervious core with upstream and downstream filters. Near the middle, where a depression in the rock surface is crossed, the bottom 20 to 30 ft of the core consists of concrete. During 1982, subsidence of the ground surface was reported near the cut-off wall. Studies indicated that the observed subsidence was probably the result of the settling of poorly compacted backfill both upstream and downstream from the buried cut-off wall. Continued analysis of data from adjacent piezometers indicates the seepage cutoff wall is functioning as designed (U.S. Army Corps of Engineers, 1986).

Uplift Pressures

Uplift pressure measuring devices have been installed in nearly all spillway and nonoverflow monoliths. Each monolith has a foundation drainage system to reduce uplift pressure. Drains in the foundation rock tend to become clogged with calcium carbonate. As foundation leakage is reduced, uplift pressures increase. The drainage systems throughout the

spillway dam are cleaned routinely; however, within the intake structure the system is inaccessible and cannot be monitored nor cleaned. Drainage is provided by an embedded pipe system for each monolith. To supplement the drainage system, subhorizontal drains were drilled in the exposed rock surface beneath the intake structure during modification of the dam.

Reservoir Erosion and Slides

Only minor progressive erosion has occurred since the reservoir was increased to elevation 956 ft in February 1981. Reservoir-related slumping in till has developed along the right bank for several miles upstream of the dam. Erosion by calving in the glaciolacustrine silts and clays and by ravelling in certain sand and gravel terraces is common in steep bank slopes around the reservoir. The Bridgeport slide (Figure 4) is a large ancient landslide located on the left bank just upstream of the dam. Portions of the slide mass are currently affected by the reservoir. Movement near the upstream end of the Bridgeport slide required abandonment and relocation of a portion of Douglas County Road 321. The slide poses no apparent threat to Chief Joseph Dam. Easements that restrict land use around the reservoir periphery have been obtained. Such easements include existing landslides that toe in or are adjacent to the reservoir and areas subject to erosion by the reservoir.

ACKNOWLEDGMENTS

The authors wish to acknowledge the many years of work on the Chief Joseph Project by K. D. Graybeal and E. T. Bailey of the Seattle District, U.S. Army Corps of Engineers. The numerous field investigations and discussions over the years and their review of this manuscript almost dictates that they be included as authors.

REFERENCES

U.S. Army Corps of Engineers, 1957, *Chief Joseph Project, Foundation Report*: U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 50 p., 17 plates.

U.S. Army Corps of Engineers, 1986, *Chief Joseph Dam, Periodic Inspection Report No. 8*: U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 66 p., 15 plates, appendix.

U.S. Army Corps of Engineers, 1988, *Chief Joseph Dam, Additional Units and Structural Modifications, Foundation Report*: U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 42 p., 23 plates, appendices.

U.S. Army Corps of Engineers (undated), *Chief Joseph Dam, Modification Spillway Pier Demolition by Explosives*: U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 91 p., 10 plates.

U.S. Congress, 1934, *Columbia River and Minor Tributaries*: 73rd Congress, 1st Session, House Document No. 103, Vol. II, [U.S. Government Printing Office, Washington, DC], 1,845 p.

Chief Joseph Dam, July 25, 1977, during modification and powerhouse addition activities. A steel trestle has been constructed across the face of the spillway dam, and the process of partial removal of spillway gate piers has begun (left center) behind a floating cofferdam. A second floating cofferdam is being prepared on the far shore behind the dam. Work on the powerhouse extension can be seen within a combination embankment and cellular cofferdam (lower right). Photograph by Seattle District, U.S. Army Corps of Engineers.

Grand Coulee Dam

Grand Coulee Dam, right forebay and Third Powerplant excavation showing the slots cut into granite for future penstocks. Photograph by R. W. Galster, October 1970.

Aerial view of Grand Coulee Dam showing the completed Third Powerplant behind the downstream cofferdam. Photograph by R. W. Galster, August 6, 1974.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Grand Coulee Dam

PHILLIP J. HANSEN
U.S. Bureau of Reclamation

PROJECT DESCRIPTION

Grand Coulee Dam is located on the main stem of the Columbia River approximately 90 mi west of Spokane. This dam is the uppermost dam on the main stem of the river in Washington. It is the key structure of the U.S. Bureau of Reclamation's Columbia Basin Project, a multipurpose development that provides irrigation water to agricultural lands in the central part of the state as well as flood control, power generation, and recreational opportunities.

The Grand Coulee Dam complex (Figure 1) includes the dam, Franklin D. Roosevelt Lake, Grand Coulee powerplant complex, switchyards, pump/generating plant, Feeder Canal, North Dam, and Banks Lake.

Power production facilities at Grand Coulee Dam are the largest in the continental United States. The total authorized generating capacity is rated at 6,480,000 kw. For years, Grand Coulee Dam reigned as the world's largest concrete structure. Today it remains the largest concrete structure on the North American continent. The dam is 5,223 ft long and 550 ft high and contains 11,975,500 cy of concrete. The original dam, authorized for construction by the Rivers and Harbors Act approved August 30, 1935, was essentially complete by 1941. The dam was later modified by construction of the Third Powerplant during the late 1960s and early 1970s.

Grand Coulee is a concrete gravity-type dam. The over-the-top spillway is controlled by 11 drum gates, each 135 ft long, and is capable of spilling 1,000,000 cfs. The dam also contains 40 outlet tubes, each 103 in. in diameter. Normal maximum reservoir elevation is 1,290 ft.

Power generating facilities consist of powerplants on the left and right sides of the spillway on the downstream face of the dam and the Third Powerplant below the downstream face of the Forebay Dam. An 11.95-kV switchyard and the 500-kV Third Powerplant cable spreading yard and switchyard are located high on the hills west of the dam. Water for irrigation is pumped from Lake Roosevelt to Banks Lake via the Pump/Generating Plant and Feeder Canal. The Pump/Generating Plant consists of six pumping units, each rated at 65,000 hp with a capacity to pump 1,600 cfs. The plant was designed to accommodate 12 such units. In the early 1960s it was determined feasible that the last six units be reversible—that is, that water be returned from Banks Lake back through these units to generate power during peak power demands. These units have since been added to the pumping plants, and the Feeder Canal has been modified to accommodate operation in the pump/generating mode (U.S. Bureau of Reclamation, 1981).

Franklin D. Roosevelt Lake, the reservoir behind the dam, extends 151 mi northeast to the Canadian border and up the Spokane River, a tributary of the Columbia, to within 37 mi of Spokane. The total storage capacity of the reservoir is 9,652,000 acre-ft, and the active capacity is 5,185,400 acre-ft. The lake has approximately 635 mi of shoreline, which is managed by the National Park Service in conjunction with the Colville Confederated Tribes and the Spokane Tribe.

SITE GEOLOGY

General

Grand Coulee Dam is located approximately on the physiographic boundary between the Okanogan Highlands to the north and the Columbia Plateau to the south. The Okanogan Highlands consist of north-south-trending mountains and valleys carved into granitic rocks of the Colville batholith. At the dam, these granitic rocks range in composition from granite to quartz monzonite. The Columbia Plateau consists of basaltic lavas that dramatically influenced the position of the Columbia River. During the Miocene, periodic eruptions from elongate fissures to the south released flows of highly fluid basaltic lava that ultimately inundated more than 100,000 sq mi in parts of Washington, Oregon, and Idaho. These flows pushed the Columbia River to its present course, which trends north at the site of the dam, around the Big Bend country to the west of the dam. The basalts cap the valley rim at the site of the dam and form portions of the reservoir rim, but they were not encountered in the dam foundation.

Sedimentary interbeds representing gaps in time between flows occur throughout the basalt sequence. These interbeds are referred to as Latah Formation, at

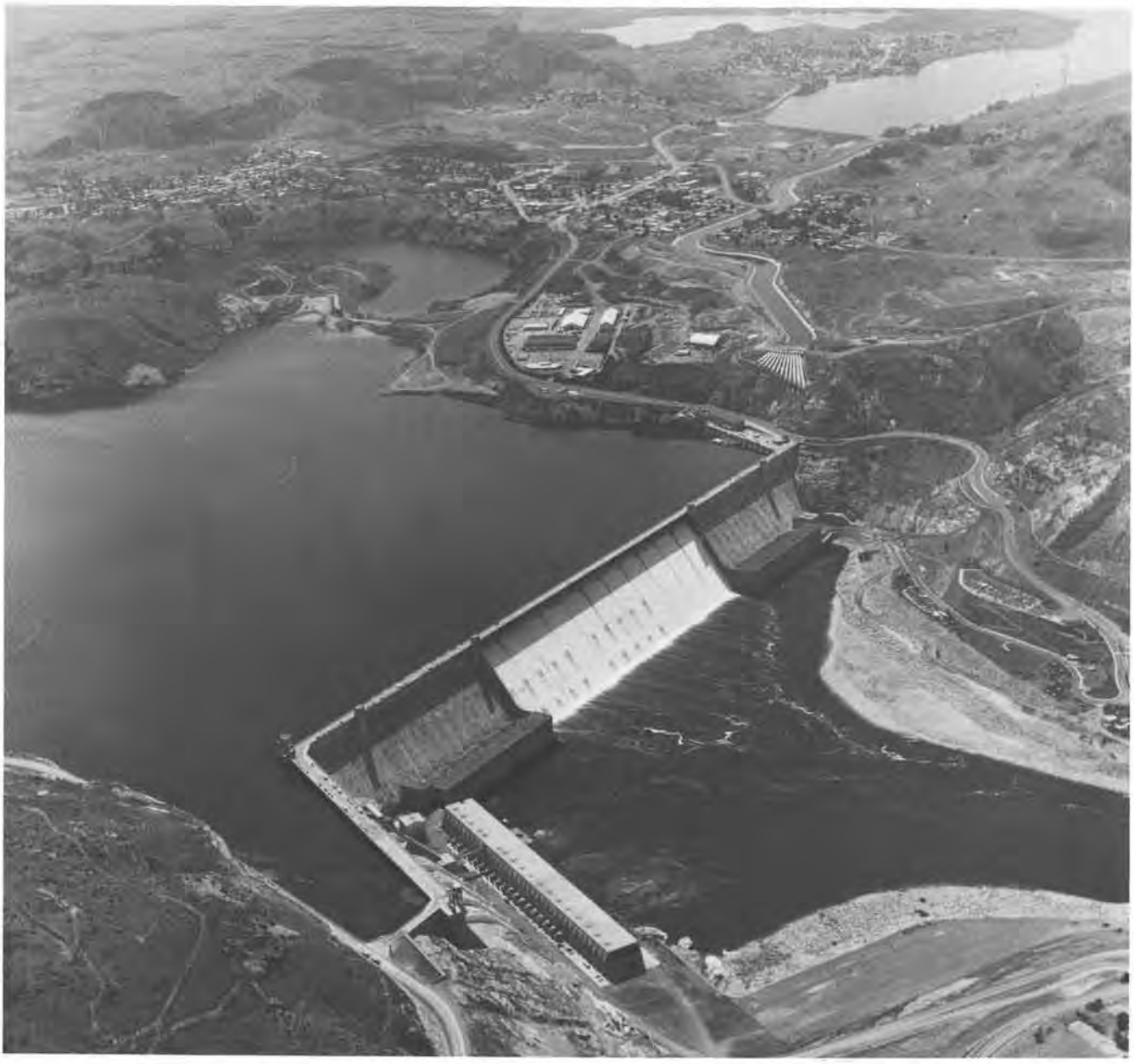


Figure 1. Grand Coulee Dam complex; view southwest, July 21, 1983. Franklin D. Roosevelt Lake is at nearly full pool elevation (1,290 ft). The Third Powerplant is in the center foreground, with the Forebay Dam to the left. The pumping plant is upstream of the far end of the dam, and the Feeder Canal and North Dam of the equalizing reservoir (Banks Lake) are in the right background. Columbia River basalt overlies Colville granitic rock at the upper left. The amphitheater-shaped valley downstream from the far side of the main dam is an area that was excavated to prevent landsliding into the foundation excavation. U.S. Bureau of Reclamation photograph by Hubert Holmes.

least in the project area, and consist of weakly indurated shale, siltstone, sandstone, and claystone. In many places these sedimentary units contain fossil leaf and stem material. These units contribute to the slope instability problems characteristic of the area.

Pleistocene continental glaciation played the final part in setting the stage for engineering geologic problems encountered at the dam. Lobes of the Cordilleran ice sheet pushed into the Columbia River valley from the north. The glacier carved a broad U-shaped valley. Most of the soil cover and weathered or altered rock in the path of the glacier was stripped, leaving a lightly weathered or fresh bedrock surface. Deposits of till and outwash locally cover the bedrock surface.

Glacier damming of the Columbia River downstream of the present dam formed glacial Lake Columbia (described below). Vast quantities of sediments were deposited in this lake as a sequence of varved clays and silts. These sediments, locally referred to as the Nespelem Formation (Flint and Irwin, 1939), filled and choked the preglacial drainages to elevations as high as 1,600 ft, more than 740 ft above the present granite floor of the valley. Periodic pulses and retreats of the glacier overrode the lake bed, forming override zones within the varved lake clays and locally depositing till and outwash on the varved sediments' surface.

The periodic floods from glacial Lake Missoula in western Montana resulted in cyclic deposition of coarse sands and/or gravel in the varved sequence. The Grand Coulee, one of the unusual features of the "channeled scablands" to the south of the dam, is a product of multiple floods of enormous size that issued from Lake Missoula. These floods encountered a pool of water referred to as glacial Lake Columbia, which was formed by glacial damming of the Columbia River near the site of the present Grand Coulee Dam. Lake Columbia attained a pool elevation of nearly 2,500 ft, 1,210 ft higher than the present elevation of Lake Roosevelt. As the on-rushing waters from Lake Missoula floods encountered the Lake Columbia pool, the lake rim was overtopped, and flood waters rushed over the Columbia Plateau to the south, catastrophically carving the features of the western channeled scabland. Estimated flow rates of the Lake Missoula flood waters through the narrow parts of the Clark Fork Canyon are 9-1/2 cu mi/hr, a rate of 386 million cfs, or about 10 times the combined flow of all the rivers of the world (U.S. Geological Survey, 1976). Recent work by Atwater (1986) on the Sanpoil Arm of Lake Roosevelt suggests that floods from Lake Missoula entered Lake Columbia periodically, perhaps as many as 100 times, during the Pleistocene.

Final retreat of the glaciers allowed the Columbia River to return to its preglacial course, generally around the northern and western margin of the lava plateau. The river cut down through the glacial sequence to its present level. In many places the entire sequence of lake sediments has been cut through, exposing bedrock in the channel. Numerous erosional and depositional terraces have been left along the valley walls. The weakness of the varved clays and silt and the rapid downcutting have resulted in an extremely active landslide history that is characteristic of most of the upper reaches of the Columbia River (Jones et al., 1961).

Bedrock

The valley walls and floor at Grand Coulee Dam consist of granitic rocks of the Colville batholith (Figure 2). These rocks are subdivided into an older, coarse-grained granite and a younger, fine-grained granitic rock that intrudes the older in a series of north-trending dikes. Basalt caps the valley but otherwise has little influence on the engineering geology at the dam.

Most of the eastern part of the foundation is the older gray biotite granite with roughly equigranular fabric. Some alteration of the feldspars and biotite is usually apparent. The western part of the foundation is the younger, finer grained granite with minor syenite and monzonite. It is light gray and porphyritic and has feldspar, quartz and chloritized biotite phenocrysts in a fine-grained groundmass of the same composition (Figure 3).

Several shear zones and faults were encountered in the foundation excavation of the main (Irwin, 1938) and forebay dams. The older granite is characteristically moderately to lightly jointed; typical joint spacing is 2 to 4 ft. This granite forms bold outcrops. The younger granite is typically moderately to intensely jointed (average joint spacing about 1 ft) and forms outcrops with more subdued blocky surfaces and conspicuous talus. As glaciation has stripped the surface of altered or weathered zones, the bedrock surface is typically only very lightly weathered or fresh.

The bedrock surface, the preglacial channel of the Columbia River, is relatively even beneath the western and central parts of the dam site and less regular in the eastern part (Irwin, 1938). Most of the depressions are linear trenches developed along faults or steeply dipping joints. Three sets of joints were recognized in the foundation rock: (a) vertical joints striking north to N20°E, (b) joints striking about N70°W dipping steeply northeast, and (c) joints striking N70°W and dipping 5°-15°SW. The steeply dipping joints are in places very closely spaced, forming zones as much as 4 ft wide of closely fractured rock resembling weakly defined shear zones. Most of the faults encountered in the foundation appear to be subparallel to the two steeply dipping joint sets. The fine-grained granite dikes that cut the coarse-grained granite have, for the most part, been intruded along structural lines parallel to the steeply dipping north-northeast joint set. The flat-lying joints are interpreted as being stress relief features. They appear to be more widely spaced at depth, and many show slight separations and tend to be discontinuous across short distances (Irwin, 1938; Nickell, 1942).

The sedimentary rocks of the Latah Formation played a key role in foundation excavation problems along portions of the Feeder Canal alignment. These sediments are characteristically gently dipping claystone, siltstone, sandstone, and shale. The sediments are very lightly indurated and behave essentially as soil. Ground water, traveling along and through the sandstone, tends to saturate the finer grained interbeds and lubricates failure surfaces within the formation. Where exposed to erosion by streams and small drainages, the sedimentary rock is quickly undercut, leading to destabilization of the overlying rock mass. Many block failures have occurred as

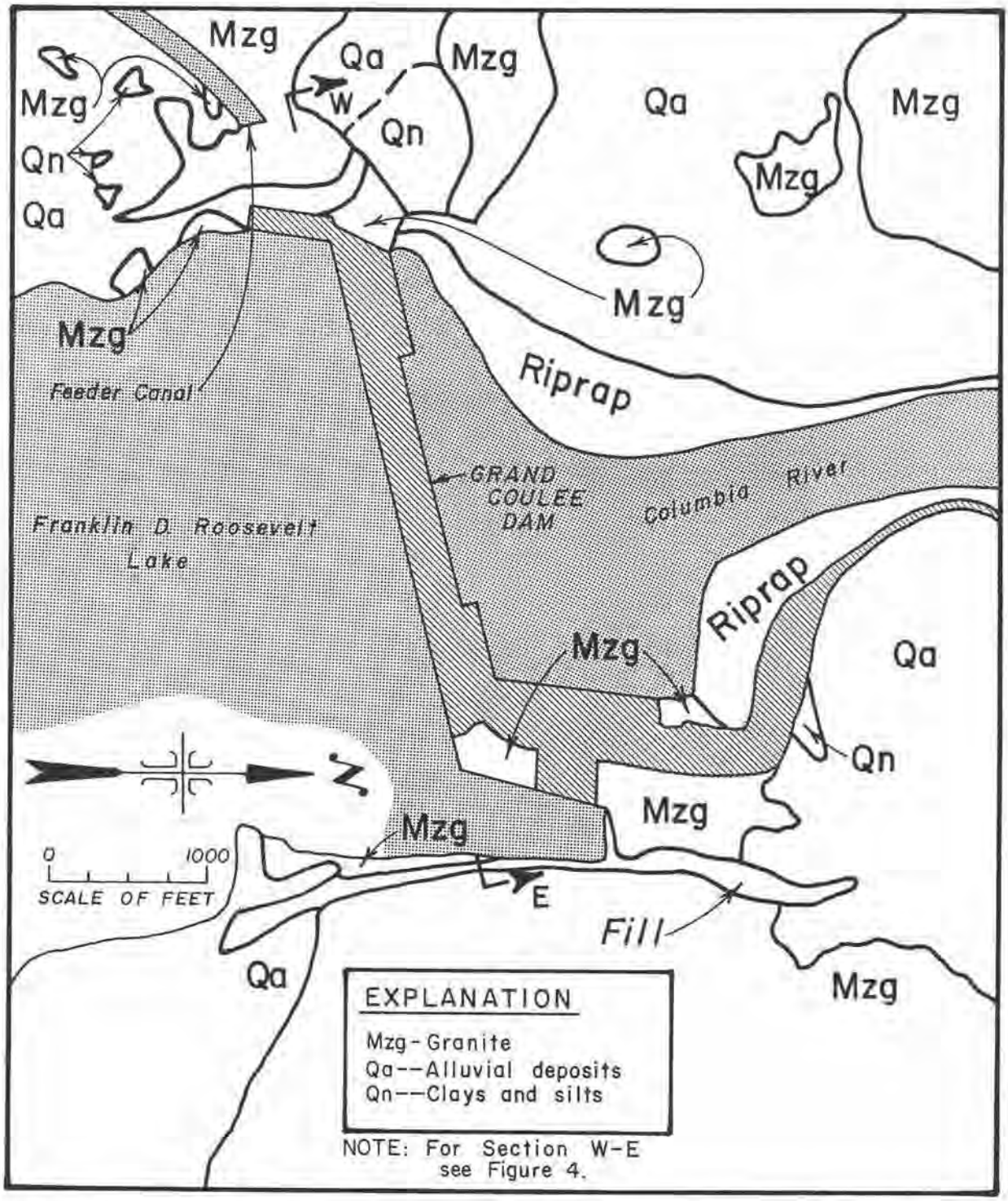


Figure 2. General geologic map of the Grand Coulee Dam area. Mzg, granite of the Colville batholith and related intrusive rocks; Qa, alluvial and colluvial deposits; Qn, clay and silt of the Nespelem Formation.

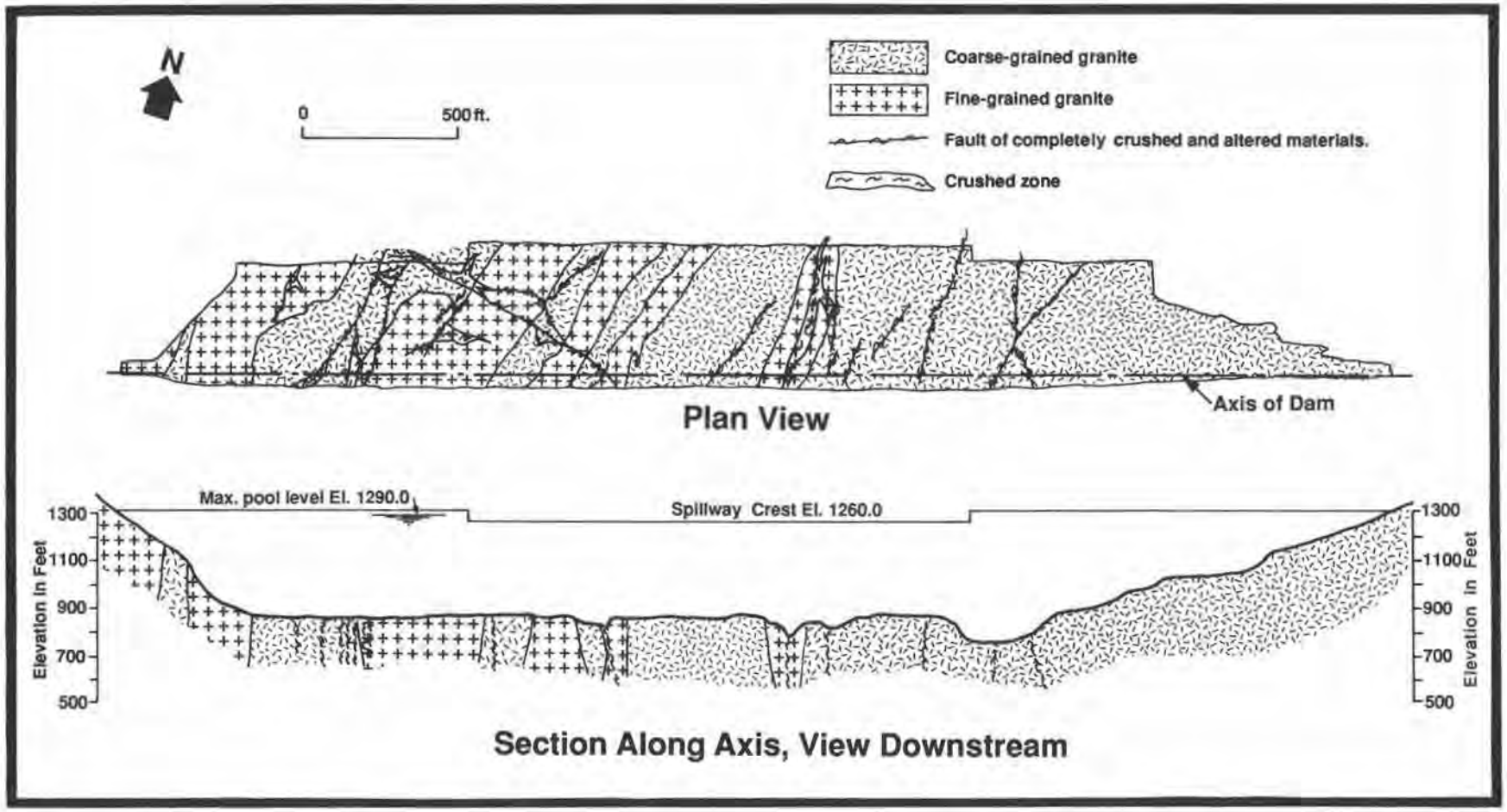


Figure 3. Grand Coulee Dam. Geologic plan and section of original dam prior to addition of Forebay Dam and Third Powerplant. Geology by W. H. Irwin, Grant Gordon, and R. L. Nichols. After Nickell (1942).

a result of this mechanism. Rotational slides also occur within this material. A segment of the Feeder Canal was founded on an ancient landslide that had occurred in this material, and excavation of another part of the Feeder Canal initiated a block slip movement along the top of a clay bed.

Sediments

In many places, the bedrock is overlain by outwash and till deposits consisting of basalt and granite clasts ranging in size from sand to blocks greater than 3 ft in diameter. Locally referred to as basal gravel, this material attains a thickness of nearly 100 ft; more commonly it is a few tens of feet thick. The outwash material is highly permeable and serves as a source of recharge to the overlying fine-grained materials (Figure 4). The till has a clay matrix and is relatively impermeable. The till is locally moderately indurated and has in places proven to be difficult to excavate.

Overlying either the basal gravel or bedrock are deposits of varved glacial lake clay and silt. These remnant deposits of glacial Lake Columbia appear predominantly as a wedge-shaped mass that is present for the most part on the east side of the river and thins or pinches out along or beneath the river. The varved sequence contains interbeds of sand and gravel, which vary from a few inches to a few feet in thickness and represent flood deposits. These coarse layers are much more permeable than the clay/silt deposits and also serve to recharge the finer grained aquifer materials. The clay and silt sequence typically has a low shear strength, which leads to the overall instability of the river banks. The clays vary from lean to fat, and the silt includes intervals of plastic silt. The glacial lake deposits are locally referred to as Nespelem Formation (Flint and Irwin, 1939).

Alluvial sand, gravel, cobbles, and boulders mantle the Nespelem unit, where present, and granite. These materials consist predominantly of clasts of granite and basalt and are relatively clean and highly permeable. The unit varies from tens of feet to more than 200 ft in thickness. This material provided a ready source of aggregate for construction of the dam.

GEOLOGIC ASPECTS OF SITING AND DESIGN

Investigations culminating in the construction of Grand Coulee Dam began as early as 1904. The need to provide water for irrigation to the arid but fertile lands that now comprise the Columbia Basin Project prompted investigation by the newly established U.S. Reclamation Service. In 1918, local interests initiated a proposal for irrigating the project area. The proposal was that a high dam be constructed on the Columbia at the head of the upper Grand Coulee. Such a dam would make use of the upper Coulee to convey water to the irrigable lands 50 mi to the south. An alternative proposal was to construct a canal from the Pend Oreille River in northern Idaho to convey water across the plateau to the central portions of the project area. In 1932, a report was submitted by the Bureau of Reclamation recommending construction of the main dam and the Left Powerplant. As originally conceived, power generation other than that required for pumping of the irrigation waters was not a significant consideration.

The unique geological characteristics of the area determined the configuration of the project as it now exists. The upper Grand Coulee provided natural conveyance for the irrigation water. Utilization of the upper Grand Coulee as a stabilizing reservoir made it feasible to pump water from the reservoir to Banks Lake with lower required pumping lifts. The generally fresh granite bedrock provided a suitable foundation for the dam. A reservoir elevation of 1,290 ft would create a pool that extended nearly to the Canadian border, providing optimum development of the river. Geologic investigations by the State of Washington and the U.S. Army Corps of Engineers in the early 1930s consisted of drilling 15 holes. The investigations indicated a bedrock surface beneath the river channel that varied in elevation from 780 ft to approximately 880 ft, and from 50 to more than 300 ft of alluvial sands and gravels and varved glacial lake deposits lay above the foundation rock (Figure 4). Two areas, each capable of containing 1.5 million cy of waste material, were identified as spoil areas. Subsequent design investigations included completion of numerous additional drill holes and test trench excavations to verify foundation conditions. These explorations confirmed the generally lightly weathered or fresh nature of the bedrock surface and revealed several deep crevices in the bedrock. Construction investigations included open trench excavation and excavation of drifts to further delineate the depth and extent of crevices and shear zones or faults in the foundation rock.

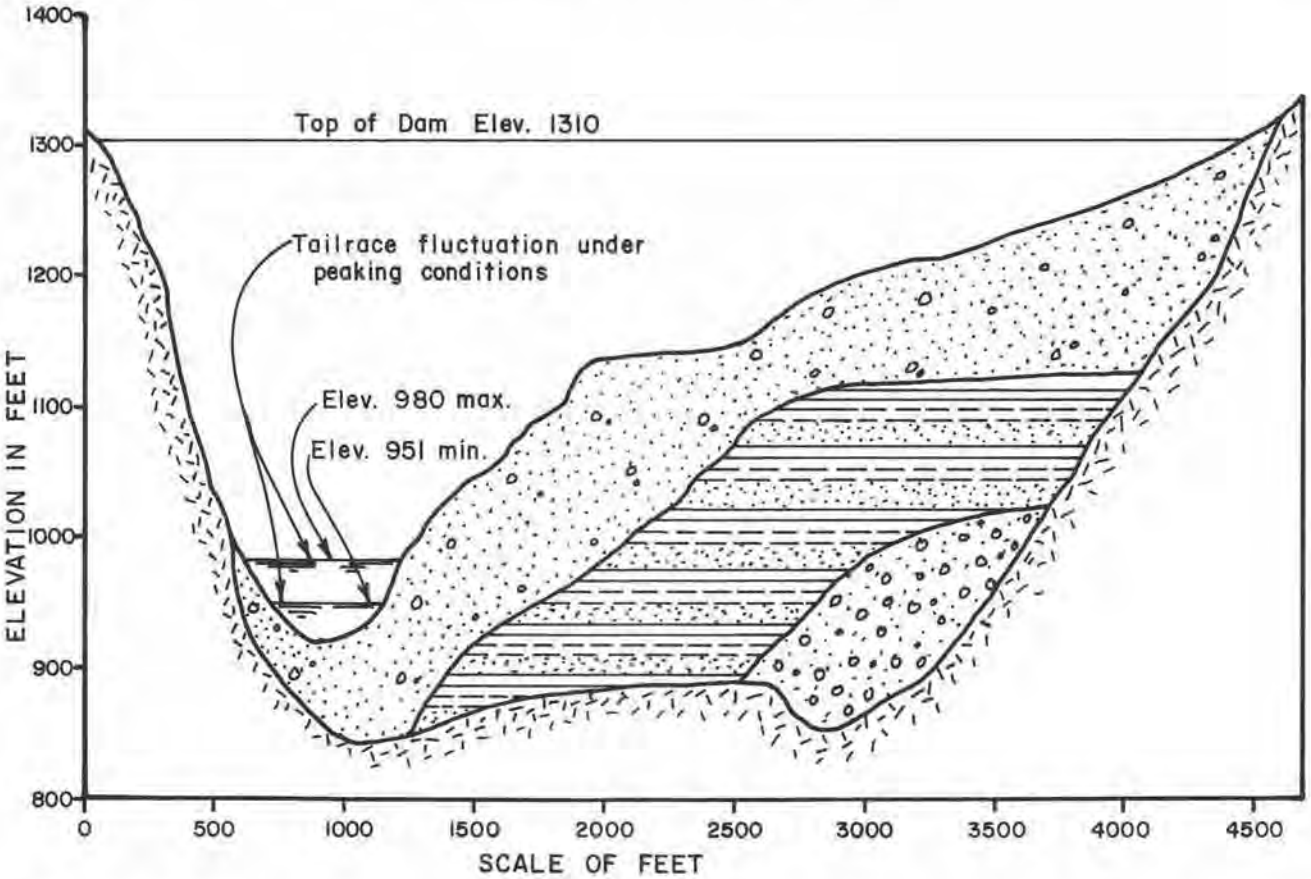
CONSTRUCTION PROBLEMS

Dam and Forebay Dam

Excavation

The overburden varied in thickness from about 40 ft in the river channel to almost 300 ft in parts of the right abutment area. Preparation of the foundations of the original dam involved excavation of 17.5 million cy of alluvial sands and gravels and varved clays and silts. Figure 4 depicts materials excavated and the general geologic conditions along the dam alignment. Additionally, faults and shear zones encountered within the granite required extensive treatment: overexcavation, scaling, and bolting.

The most challenging problems, then and now, resulted from the instability of the varved clays and silts. Initial excavation into the toe of the riverbank slopes resulted in landsliding. Between 1934 and 1937,



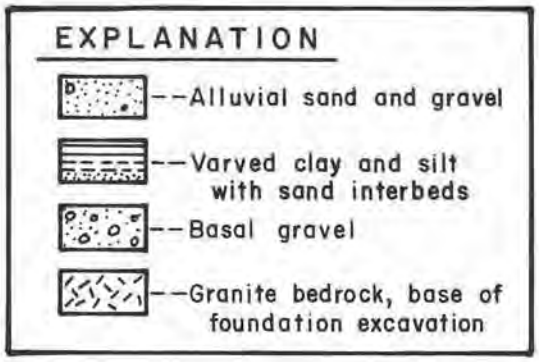


Figure 4. Generalized geologic section at Grand Coulee Dam; view downstream, west is on the left. All overburden sediments were removed from the foundation area during construction. Under peaking conditions the allowable daily maximum tailrace elevation fluctuation is 22 ft between the maximum and minimum elevations shown.

nearly 60 landslides occurred in the excavation area. Sliding into the left tailrace area began in March 1934, when a slide with nearly 100 ft of vertical displacement occurred. Treatment of this slide included extensive overexcavation, installation of drainage shafts, drifts, radiating drain holes, and construction of a rock-fill plug at the toe of the slope. Eventually, a riprap blanket was placed to protect the toe of the slope. Between 1934 and 1941, $2,726,000 in additional costs were incurred for treatment of this slide alone; placement of riprap between 1948 and 1953 brought the cost for treatment of this slide to more than $6 million (Jones et al., 1961).

Solutions developed to mitigate landslide problems encountered in excavations in these materials during construction included laying back the slopes, dewatering via drainage drifts and wells, and freezing the materials to form an arch dam to block flow of the slide materials into the excavation. This last approach was successfully used in the spring of 1936 to allow cleanup of a particularly deep crevice located near the axis of the dam, directly below the forebay cut, on the east bank. This crevice became known as "Ice Dam Gulch".

Preliminary exploration of "Ice Dam Gulch" revealed that this crevice extended deeper than any other similar

feature on the entire dam site. When the crevice was exposed, it proved to be a long, narrow gulch about 120 ft deeper than the average bedrock level.

Prior to emplacement of the ice dam, more conventional methods, such as laying back the slope, had proved inadequate. As described in the project history (U.S. Bureau of Reclamation, 1936, pp. 176-177), excavation slopes were originally laid out 1.5H to 1V, but evidence of sliding soon appeared on the slopes. The slopes were flattened to 2H to 1V.

> "In spite of the flattened slopes the sliding condition persisted and developed to menacing proportions in March 1936. The contractor then took steps to check the earth movement by constructing a concrete bulkhead at the toe of the slope. The bulkhead was designed as an arch dam spanning a 50-ft rock gulch through which liquid muck flowed like molten lava. The concrete arch was built 20 ft high, and above it a bulkhead was built up another 15 ft with timber cribbing. Its top was then at elevation 815, and it was intended to go higher but a sudden movement of earth overtopped the bulkhead on April 17, 1936. By this date the river had begun to rise, and it became apparent that the east pit would soon be flooded. Consequently, excavation equipment was moved to higher ground and efforts to clean out the crevice were abandoned until after high water.
>
> "In the meantime, slopes were staked as flat as 4[H]:1[V] in the critical area and retrimming progressed down to elevation 970 ft. The east pit was dewatered in July, and as the water level was lowered sliding again occurred. Solution for the problem seemed to lie in either flattening slopes drastically or in constructing an effective barrier at the toe of the slide. The latter method was adopted, and it was accomplished by freezing the liquid muck to form an arch dam on top of the concrete and timber bulkhead which the contractor had formerly built. Construction of the frozen arch was authorized under Extra Work Order No. 28."

Few data existed in 1936 on the mechanics and strength of frozen ground. Design calculations suggested a freeze zone thickness of 10 ft, but to provide an ample safety factor, the zone was made 20 ft thick. Freezing was accomplished by pumping brine through pipes that were driven into the ground. The freezing points were spaced 30 in. apart. Each point consisted of 3-in.-diameter pipe driven 40 ft into the ground. A 1-1/2-in.-diameter pipe was then installed inside the larger pipe to allow circulation of the brine in through the smaller pipe and out through the annulus between the smaller and larger pipes. The points were connected in series by means of rubber hose on the upper end. Construction of the ice dam required installation of 377 freezing points. Two ammonia-brine refrigerating machines with a combined capacity of 80 tons of ice per day were installed. Formation of the ice dam by freezing was accomplished in about 6 weeks. The ice dam performed satisfactorily, and clean-out of the crevice continued without further problem. The ice dam was a means of saving possibly several hundred thousand cubic yards of excavation at a cost of $1.00 per yard and several months' time in preparing foundations and concreting in the area (U.S. Burean of Reclamation, 1936).

Construction problems of this type, though on a smaller scale, were the rule rather than the exception during construction of the main and, later, the forebay dam. Landslides occurred frequently during construction of the Third Powerplant when slopes were oversteepened. The only slide that caused damage occurred during placement of the cellular cofferdam. This slide moved recurrently from November of 1968 to May 1969 as the contractor continued to undercut its toe. Its last movement damaged the partly erected cofferdam. The ultimate solution was laying back the slope.

Excavation for the Third Powerplant generated more than 10 million cy of waste material. This material was used in the construction of a staging area upstream of the dam for Third Powerplant construction contractors and also downstream, principally on the right (east) riverbank, to form the downstream stabilization embankment. Frequent failures occurred in the embankments where the material was placed on natural ground underlain by varved clays and silts.

Foundation Treatment

Preconstruction exploration for the original dam, including approximately 33,000 ft of drill holes, indicated that the granite foundation would have superior bearing capacity for a concrete gravity dam of sufficient size to fully utilize the potential of the site. Average unconfined compressive strength of the rock types at the site was found to be 15,000 psi (5,280 psi to 29,700 psi on 24 core samples tested). Joints and faults presented little difficulty during construction; some dental work was necessary locally.

Foundation Grouting

A comprehensive grouting program was used to seal the foundation and provide a cut-off curtain. Grouting was accomplished in three phases, using progressively higher injection pressures as succeeding lifts of concrete imposed higher structural loads on the foundation before placement of more concrete. Near-surface rock was sealed during phase B by employing grouting pressures less than 250 psi. "B" holes were 20 to 30 ft deep and at 2-ft centers in a staggered pattern of five parallel lines located along the dam axis and 20 and 40 ft

upstream and downstream. Grout take in the nearly 2,300 "B" holes averaged about 2 cu ft/linear foot. After concrete had been placed to at least 25 ft above bedrock, phase C grouting at pressures to 400 psi was done through a single line of 50- to 100-ft-deep holes inclined downstream at 20-ft centers from the upstream toe of the dam. Grout take in the approximately 300 "C" holes averaged 7.4 cu ft/linear foot.

When the dam was nearly at full height, the cut-off curtain was placed by phase A grouting at pressures as high as 1,000 psi. Located in a single line along the grouting and drainage gallery at the base of the dam, the more than 700 "A" holes were inclined upstream on 10-ft centers. Most of the holes were drilled alternately 150 and 200 ft deep, although some were as much as 500 ft deep where the rock was of questionable quality. Grout take averaged 4.1 cu ft/linear foot, although one 150-ft hole accepted 11,800 cu ft (Vehrs, 1968).

Feeder Canal Construction

The Feeder Canal and other features of the irrigation project were constructed during the 1950s. Initial construction was by the Bureau of Reclamation. Excavation of the first part of the canal prism was in granite bedrock. The excavation next penetrated bedded Latah Formation sedimentary rocks dipping 4° N and underlying remnants of a basalt flow (Figures 5 and 6). The alignment then crossed an ancient landslide of Latah Formation and basalt slide blocks. Excavation through the Latah Formation became a problem: undercutting immediately led to landslides, which threatened to fill the excavation. During a cessation of construction necessitated by transfer of work from the Bureau of Reclamation to contractors, this problem was solved by redesign of this part of the alignment to a cut-and-cover section. Twin 25-ft-diameter concrete conduits were placed in segments and immediately buried to minimize length of open trench.

The Feeder Canal was modified to allow reverse flow for pumping/generating by construction that began in the winter of 1979. The contract allowed 1,248 days to complete the construction and required allowance of pumping for irrigation during the spring and summer. Modification required widening the canal and replacing the cut-and-cover section with a flume wall section. Excavation for the flume section would have to pass through the landslide deposits. This geotechnical problem was solved in two ways: The old cut-and-cover conduits were used as part of an anchored retaining system; and automated instrumentation was installed to monitor and warn of impending instability. The monitoring system was at that time unique, but it has since become a "shelf item". It consists of a series of inclinometer wells installed upslope of the canal prism. In-place uniaxial inclinometers in the wells were monitored by a centrally located microprocessor. The microprocessor was set to record hourly data from more than 100 instruments and to alarm if movement of greater than 0.25 in. occurred. This system served as a model for the Riverbank Stabilization Monitor, which will be addressed later.

Anchoring of the twin cut-and-cover conduits was accomplished by drilling and installing tensioned soil anchor tendons into the hillside from within the conduits. The anchor tendons did not penetrate bedrock. Pairs of tendons were installed on 25-ft centers in each conduit to depths adequate to pass through known failure surfaces. These depths ranged to as much as 150 ft. Drilling of holes for the tendons ultimately required use of horizontal drills capable of advancing casing through the sandy material. Upon installation, the tendons were tensioned. This system proved effective, and the construction was completed with no major stability problems. Data from the automated inclinometers continues to be collected and evaluated.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Landslides upstream and downstream of the dam have been a recurrent geotechnical problem since construction of the dam was completed in 1941 and the reservoir was filled in 1942. During reservoir filling, 245 landslides occurred along the 635-mi shoreline of Lake Roosevelt. Though the frequency of sliding decreased on attaining full pool, landslides continued to be an important factor in engineering development and land use. Between 1943 and 1953, an additional 255 slides took place. The relation between increasing frequency and magnitude of landsliding and increasing severity of reservoir drawdown was established by an intensive geologic investigation conducted between 1948 and 1955. The goal of the investigation was developing criteria that could be used for predicting the probable amount of land impacted by sliding. These investigations are summarized in Jones et al. (1961), which provided a basis for procuring additional lands and serves as a guideline for performance of current investigations along the lake shore.

The Grand Coulee Project Office currently conducts an annual survey of landslides impacting the Lake Roosevelt shoreline. Data from this survey are included in the Bureau of Reclamation's Landslide Register. Currently, more than 130 active landslide areas are being monitored. Because of the increasing pressure for development along the shoreline, the Bureau of Reclamation is in the process of conducting further investigations of the shoreline. Detailed mapping has been completed on U.S. Geological Survey quadrangle sheets encompassing the entire shoreline. Additionally, detailed engineering geologic mapping has been accomplished for most of the shoreline, and zones of current and potential instability have been identified. The Bureau of Reclamation is acquiring additional freeboard

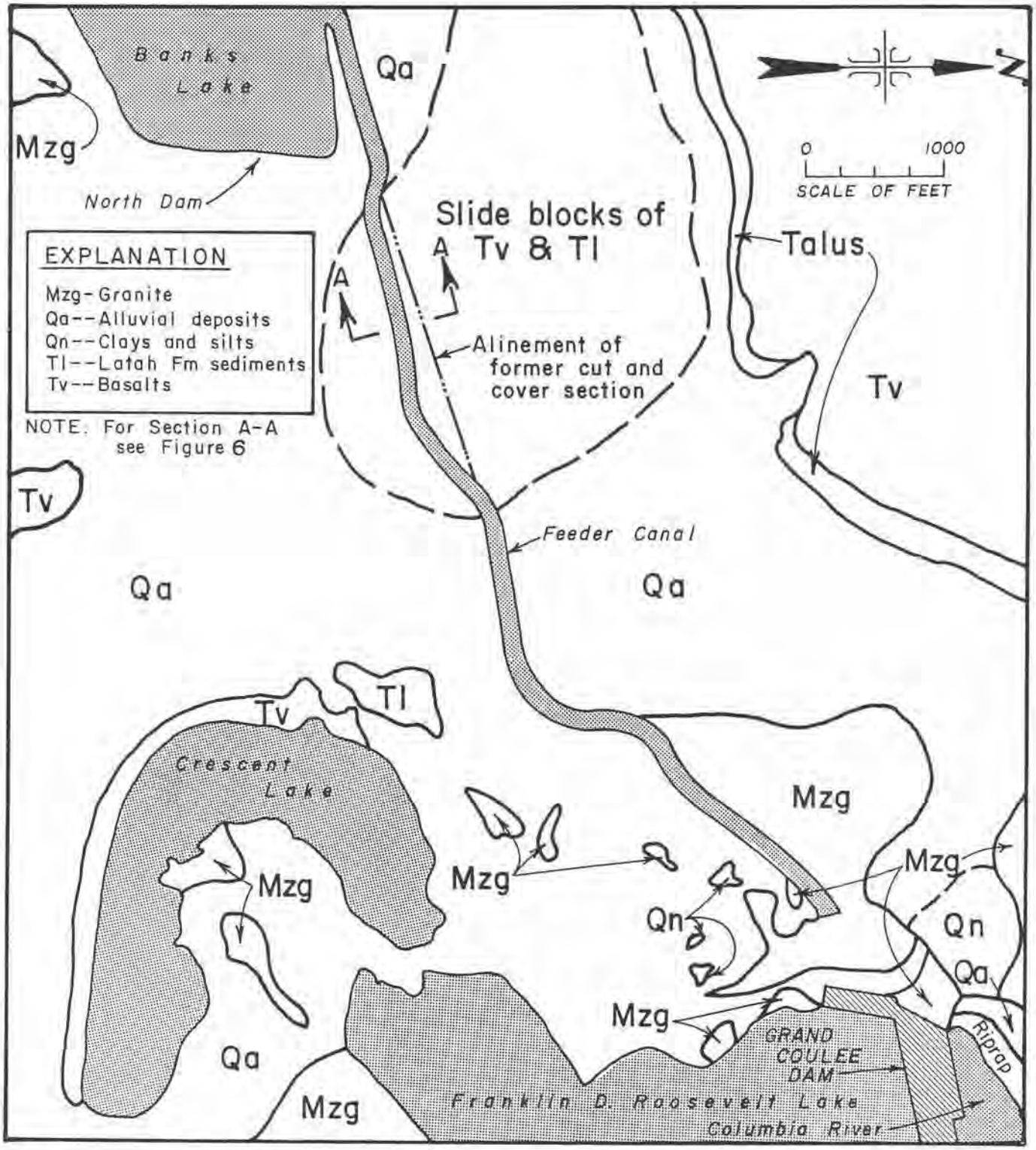


Figure 5. General geologic map of the Feeder Canal area adjacent to Grand Coulee Dam. Mzg, granitic rocks of the Colville batholith and associated intrusive rocks; Qa, alluvium and colluvium; Qn, silt and clay of the Nespelem Formation; Tl, sediments of the Latah Formation; Tv, Columbia River basalt. Cross section A-A is shown in Figure 6.

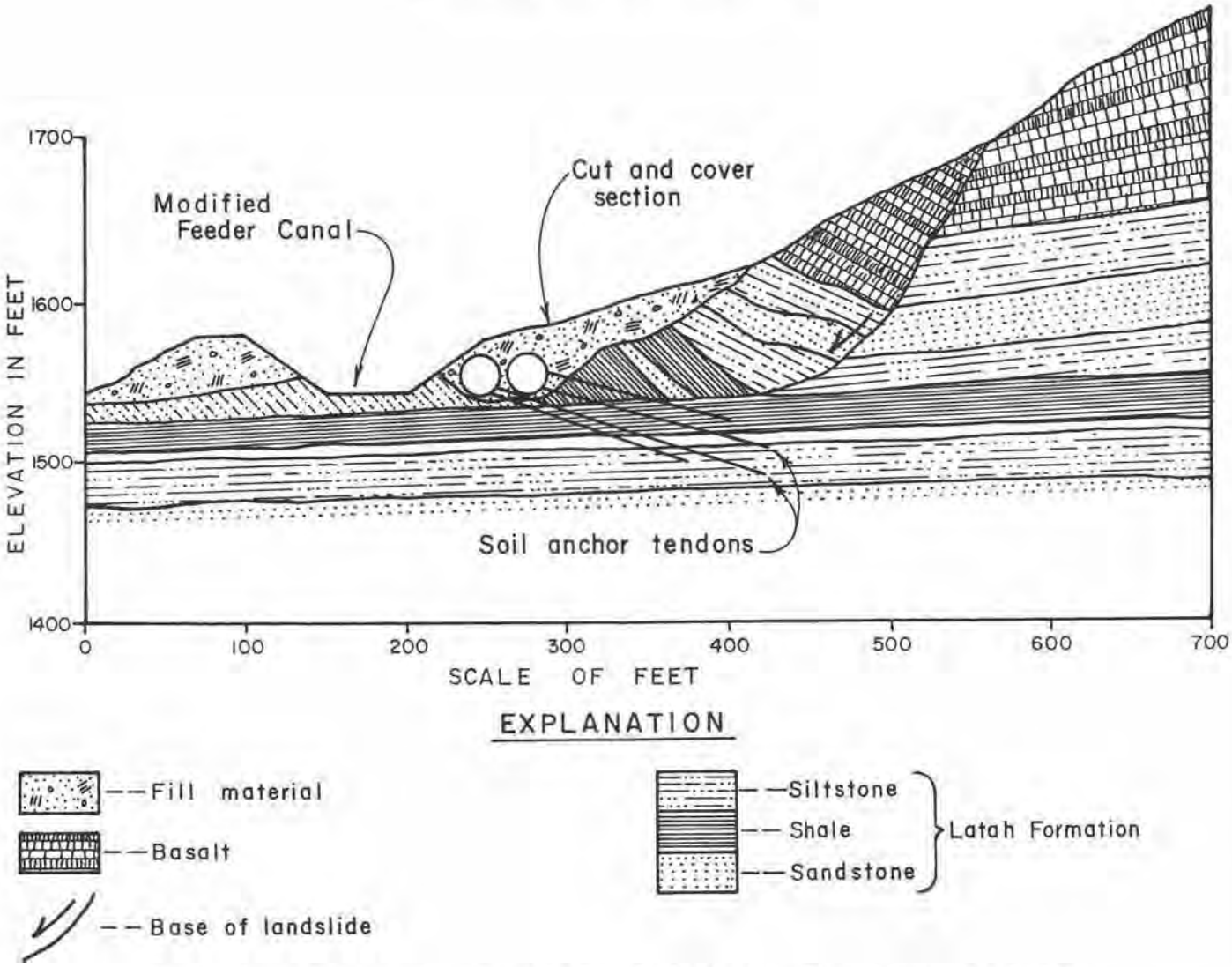


Figure 6. Generalized geologic section A-A of the Feeder Canal. See Figure 5 for location of section.

land in some areas and is providing shoreline protection in others.

Landslides downstream of the dam prompted the Bureau of Reclamation's most recent construction. These landslides occur mostly as a reactivation of ancient landslides that can be distinguished on maps and photographs predating the construction of the dam. Unusually high precipitation and seasonal runoff between 1948 and 1952 resulted in reactivation of these slides. Peaking operations of the Third Powerplant were recognized as having the potential of further destabilizing the area. Peaking operations will generate daily fluctuations in the tailrace elevations in excess of 20 ft. The added erosive force of this volume and velocity of water, coupled with fluctuation of pore pressures in the clays underlying the riverbanks, result in unacceptably low factors of safety for large rotational and planar failure surfaces downstream of the dam. At the time of construction of the Third Powerplant, the Bureau of Reclamation designers determined that placement of an embankment along the entire right (east) riverbank would be required to stabilize the area for peaking. Failures that occurred both during and after placement of the stabilization embankment indicated that the embankment was inadequate to stabilize the hillsides. In 1975 investigations were targeted at identifying and correcting additional factors lending to the overall instability of the downstream river banks. In June 1978, landslides occurred along the entire 6-mi river reach downstream from the dam. The most significant of these failures occurred just downstream of the State Route 155 bridge adjacent to the town of Coulee Dam. The slide, which had a maximum vertical displacement of approximately 4 ft and horizontal displacement of 0.5 ft, was triggered by a 13-ft drop in the mean tailrace elevation, which was caused by a failure of one of the

units in the Third Powerplant. This drop in the mean tailrace elevation was slightly more than fluctuations anticipated for Third Powerplant peaking operations. As a result of this renewed landslide activity, investigations ultimately leading to the riverbank stabilization program were intensified.

Numerous possible solutions to the riverbank stabilization problem were identified and evaluated. These included relocation of downstream communities and resloping of hillsides below the dam to reduce the head load placed on the failure surfaces. Ultimately it was determined that an engineered solution was feasible, and the decision was made to attack the problem with minimal disruption to the downstream residents. The Riverbank Stabilization Program addresses the stability of the downstream slopes in three ways:

(1) Reshaping and re-armoring of the previously placed stabilization embankment.

(2) Construction of drainage features to reduce pore pressure fluctuations in the clay/silt materials underlying the hillsides.

(3) Installation of automated instrumentation to monitor the effectiveness of the completed drainage features and to warn of adverse changes in the geotechnical environment and movement in the riverbanks. The automated system has real-time alarming capability and is operational at this writing.

Stabilization embankment modifications have been completed. This work involved placement of additional fill and armor rock in the river channel principally on the right bank. Final grade of the face of the embankment was reduced from an original 1.5H to 1V slope to slopes ranging to as flat as 5H to 1V, and the embankment is protected with graded armor stone quarried from granite exposed on the right bank and the staging area immediately upstream of the Third Powerplant on the Lake Roosevelt shoreline. Approximately 3 million cy of fill and 3 million cy of riprap were placed under the requirements of the contract. Production of quarried rock within the size gradation specified was one of the more difficult problems faced by the contractor.

The river banks downstream of the dam can be subdivided into two distinctly different geotechnical environments. Ground-water recharge in the initial 3 mi of riverbank downstream of the dam is controlled essentially by the river. Farther downstream the ground-water system is complicated by artesian conditions from upslope drainage. Features designed to control pore pressure fluctuations in the clay consist of traditional pumped wells and, on the right bank below the community of Coulee Dam, shafts with near-horizontal drains installed into the clay. The shaft concept was advanced as a method to more effectively collect and control water in the disturbed landslide and complexly interbedded clay and silt. Wells were selected for dewatering in the artesian areas and on the left bank immediately adjacent to the Left Powerhouse. Well sites were selected based on investigations and pump testing aimed at selecting points of recharge, such as bedrock anomalies and basal gravel. This approach has proven highly successful.

The riverbank stabilization monitoring system consists of: in-place uniaxial inclinometers installed in wells located so as to penetrate failure surfaces identified by stability analysis; electronic pore-pressure transducers in selected observation wells; river gaging stations at three locations downstream of the dam; and flowmeters installed on discharge piping and horizontal drains. The system now includes more than 600 monitored locations. The data from the instrumentation is collected at data scanner locations that are networked to a central computer. The system is capable of operation in a continuous scan mode, but will normally query the instruments on a 3-min cycle and record data hourly. Data will be automatically compared to alarms tables residing in the central computer. The alarms tables are based on the stability analysis work. Should alarm-triggering conditions occur, a warning will be generated at the powerplant dispatcher's station.

REFERENCES

Atwater, B. F., 1986, *Pleistocene Glacial-Lake Deposits of the Sanpoil River Valley, Northeastern Washington*: U.S. Geological Survey Bulletin 1661, 39 p.

Flint, R. F. and Irwin, W. H., 1939, *Glacial Geology of Grand Coulee Dam, Washington*: Geological Society of America Bulletin, Vol. 50, No. 5, pt. 2, pp. 661-680.

Irwin, W. H., 1938, *Geology of the rock foundation of Grand Coulee Dam, Washington*: Geological Society of America Bulletin, Vol. 49, No. 11, pp. 1627-1650.

Jones, F. O.; Embody, D. R.; and Peterson, W. L., 1961, *Landslides along the Columbia River Valley, Northeastern Washington*: U.S. Geological Survey Professional Paper 367, 98 p.

Nickell, F. A., 1942, *Development and use of engineering geology*: American Association of Petroleum Geologists Bulletin, Vol. 26, No. 12, pp. 1797-1826.

U.S. Bureau of Reclamation, 1936, *Grand Coulee Dam Project History, Vol. IV*: U.S. Bureau of Reclamation, Grand Coulee Project Office, Grand Coulee, WA, 360 p.

U.S. Bureau of Reclamation, 1981, *Project Data Book–Columbia Basin Project, Washington–Adams, Douglas, Franklin, Grant, Lincoln, and Walla Walla Counties*: U.S. Bureau of Reclamation, Pacific Northwest Region, Boise, ID, 20 p.

U.S. Geological Survey, 1976, *The Channeled Scablands of Eastern Washington–The Geologic Story of the Spokane Flood*: U.S. Geological Survey, 24 p.

Vehrs, R. A., 1968, A Summary of the Engineering Geology of Grand Coulee Dam and Related Features. In McKee, B., and Coombs, H. A., *Guidebook to Field Trips*: 1968 National Meeting, Seattle, WA, Association of Engineering Geologists, pp. 18-26.

Dams of the Pend Oreille River

Introduction and Geologic Setting

Boundary Dam

Box Canyon Dam

Boundary Dam site, Pend Oreille River; view up the river. Underground powerhouse tailrace tunnels at the right center.

Dams of the Pend Oreille River: Introduction and Geologic Setting

HOWARD A. COOMBS
University of Washington
and
WILLIAM S. BLITON
ATC Engineering Consultants, Inc.

The Pend Oreille River has an unusual course. Flowing west from Idaho, it crosses into the northeastern corner of the state of Washington, then turns due north until it crosses the international border into Canada. Just north of the border, it turns west and joins the south-flowing Columbia River (Figure 1).

Two dams have been constructed on this river in Washington, the Boundary Dam, 1 mi south of the Canadian border, and the Box Canyon Dam, 15 mi farther south. The two hydroelectric projects are located in in the southeastern part of the Okanogan-Selkirk Highlands. Though highland altitudes range between 4,000 and 8,000 ft, these mountains lack the sharpness of other mountain ranges in the state. Continental glaciation was so extensive that most of the peaks and ridges were subdued by glacial scour and the valleys exhibit the typically glacially rounded forms.

The rocks in this northeastern corner of the state range in age from PreCambrian to Devonian. There is scant evidence of deposition during the post-Devonian to late Tertiary hiatus.

Bedrock along the stretch of the Pend Oreille River in Washington is essentially limited to the following: the Middle Cambrian Metaline Limestone (and dolomite) that reaches a thickness of 3,000 ft; the Ordovician carbonaceous Ledbetter Slate with a maximum thickness of 2,500 ft; and a Devonian limestone as much as 700 ft in thickness. The late Tertiary semi-consolidated sands and gravels, known as the Tiger Formation, are present as scattered remnants along the river (Park and Cannnon, 1943).

The Pend Oreille River, for much of its length in the area of the two dams, is bounded on either side by long faults (Figure 2). Less significant high-angle faults cross the river at various angles. These will be discussed more fully in the project descriptions.

Prior to the Pleistocene, the general north-south valley system was well established and provided a ready avenue for the glaciers moving southward out of Canada. During the Pleistocene, continental ice moved down over the area, leaving erratics and striae at altitudes as high as 6,000 ft on either side of the Pend Oreille valley. Not only was the topography of the valley changed, but an enormous amount of debris was also left by ice wasting. Although the region has been subjected to at least two periods of continental glaciation, evidence indicates that only after the last period did the drainage in the Pend Oreille valley flow northward.

REFERENCE

Park, C. F. and Cannon, R. S., 1943, *Geology and ore deposits of the Metaline quadrangle, Washington*: U.S. Geological Survey Professional Paper 202, 81 p.

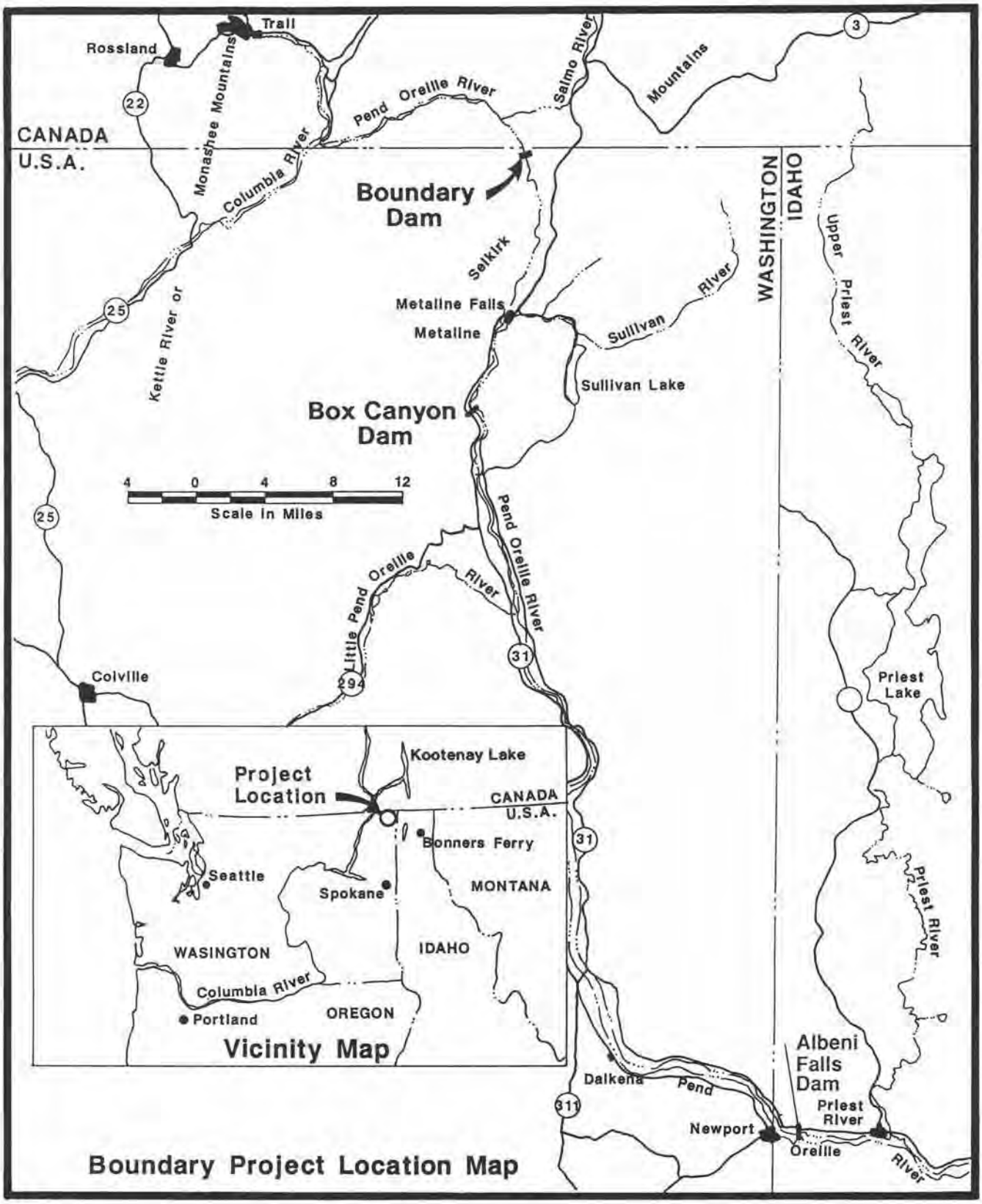


Figure 1. Location of dams and major topographic features of the Pend Oreille River area in northeastern Washington.

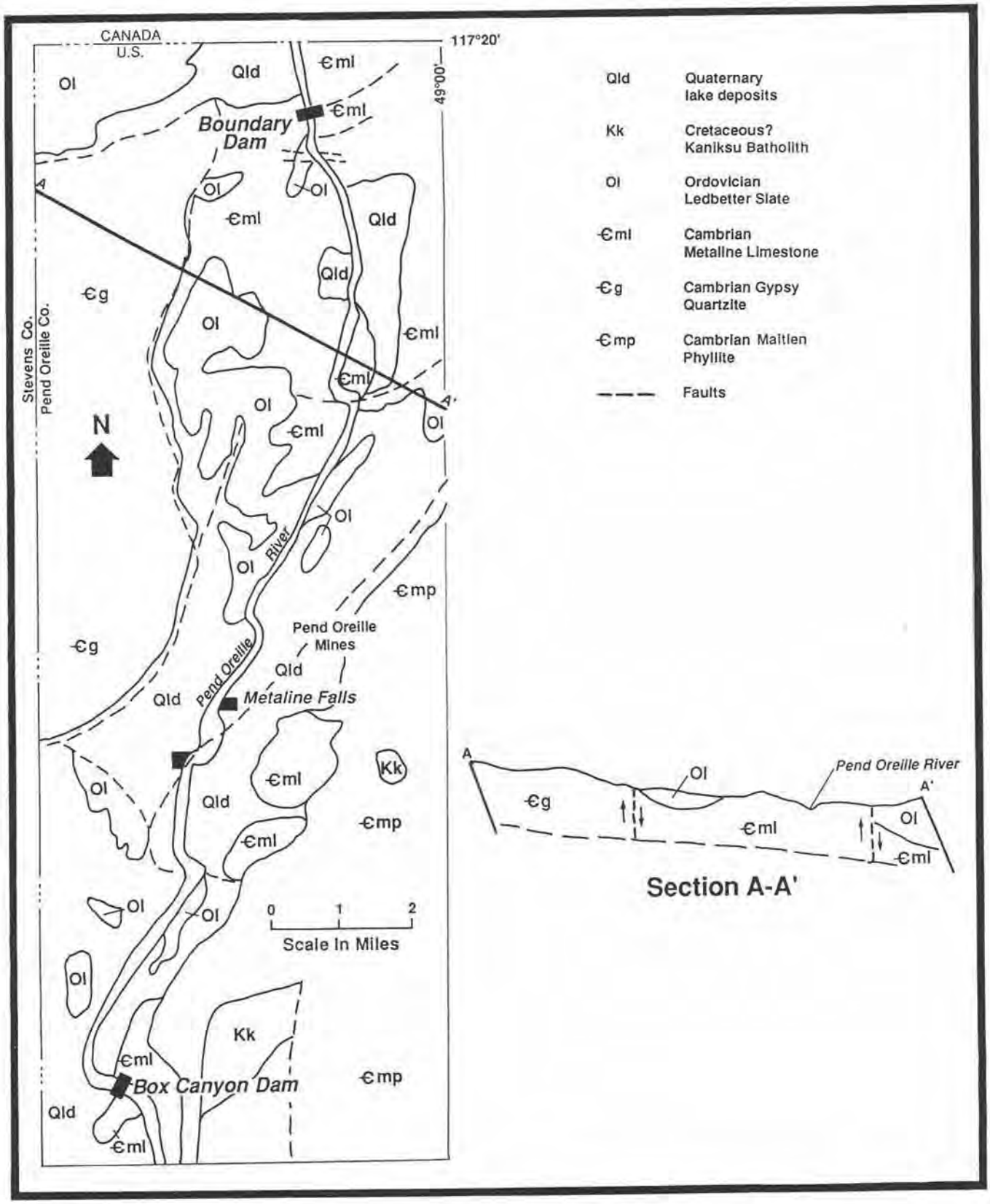


Figure 2. Generalized geologic map of the area of Boundary and Box Canyon dams. After Park and Cannon (1943).

Boundary Dam site, Pend Oreille River. Construction of transformer adits in Metaline Limestone. Photograph By H. A. Coombs, September 1965.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Boundary Dam

HOWARD A COOMBS
University of Washington

PROJECT DESCRIPTION AND HISTORY

Boundary Dam is located on the Pend Oreille River in the northeastern corner of the state 1 mi south of the Canadian border (Figure 1).

The main features of the project are an arch dam 340 ft high, a shallow forebay channel on the left bank with a skimmer gate structure, and an underground power plant with six 150-kw generating units (Strandberg, 1966). The dam is a double curvature, thin concrete arch; it varies in width from 8 ft at the crest to 32 ft at the base. The crest length between spillways is 508 ft (Figure 2).

The project is operated on a run-of-river development, using streamflow in the Pend Oreille River, which is regulated by Albeni Falls Dam in Idaho and other storage projects farther upstream. The minimum reservoir level is elevation 1,950 ft; however, the reservoir rarely fluctuates more than 10 ft. The reservoir extends 15 mi upstream.

As early as 1914 engineers had explored the Pend Oreille River for hydroelectric sites (Coombs, 1953; U.S. Army Corps of Engineers, 1945; Torpen, 1953). Studies were continued, and in 1953 Seattle City Light filed an application for a preliminary permit (Coombs, 1955). The permit was contested by another utility that wanted to build two dams along the same stretch of the river. It was also contested by mining companies that had operating mines on both sides of and under the proposed reservoir area. Geologic and hydrologic conditions played a major role in deciding the granting of a permit for a dam or dams along this stretch of the (Humphrey, 1955). The decision was made in 1954 that Seattle City Light made best use of the river. However, very stringent stipulations were made by the Federal Power Commission.

SITE GEOLOGY

Boundary Dam is constructed on, and the powerhouse in, Metaline Limestone of Cambrian age. The reservoir covers Metaline Limestone for several miles upstream of the dam and Ledbetter Slate in its upper reaches. The Pend Oreille River has cut a spectacular canyon in limestone upstream of the dam, but farther upstream, where Ledbetter Slate is exposed, the valley widens and has more gentle slopes (Park and Cannon, 1943; Dings and Whitebread, 1965).

Faults play a major role in shaping the Pend Oreille valley. High-angle faults lie on either side of and are roughly parallel to the Pend Oreille River. (See Figure 2 of the introductory paper to the Pend Oreille River dams.) Other high-angle faults cross the river at angles varying from 45° to 90°. This faulting brings the Metaline Limestone to the surface near Boundary Dam.

During the Pleistocene continental ice moved down over the Boundary project area, leaving erratics and striae at altitudes as high as 6,000 ft on either side of the Pend Oreille valley. Not only was the topography of the valley changed markedly, but an enormous amount of debris was also left by ice wasting.

The original layout for the project had the powerhouse on the right bank immediately beneath a vertical cliff that has a rock overhang 400 ft above. Bringing the potential dangers to the attention of the designers resulted in locating the powerhouse underground in its present position on the left bank. In deciding on an underground powerhouse, the engineers must have been influenced by the stability of the limestone in the large tunnels in the nearby mines (McConnell and Anderson, 1968). The rock lends itself well to line drilling; it also stands well without support.

The downstream face of the powerhouse is essentially a fault face. Just inside this face and parallel to it is a fault zone from which some of the fault breccia was removed by solution. Advantage was taken of this condition in establishing entrance haulways into and out of the powerhouse. The Metaline Limestone at the site dips 40° to the south and stands in near-vertical cliffs more than 500 ft high. A 6-ft x 8-ft exploratory tunnel was driven along the axis of the powerhouse to test the rock; flatjack, deformeter, and plate bearing tests were conducted *in situ* to determine the physical properties of the rock. Numerous calcite-filled joints and fractures were found in the machine hall rocks; however, they did not materially affect the roof and wall stability. The roof and walls of the machine hall are permanently reinforced by grouted rock bolts. Bolts 15 ft long and on 6-ft centers support the 75-ft-wide roof which is more than 175 ft above the bottom of the drainage tunnels. The

Figure 1. Aerial view south of Boundary dam. The underground powerhouse is contained in this bold mass of Metaline Limestone. The transformer bays emerging from the cliff will orient the diagram shown in Figure 3. Photograph by Seattle City Light.

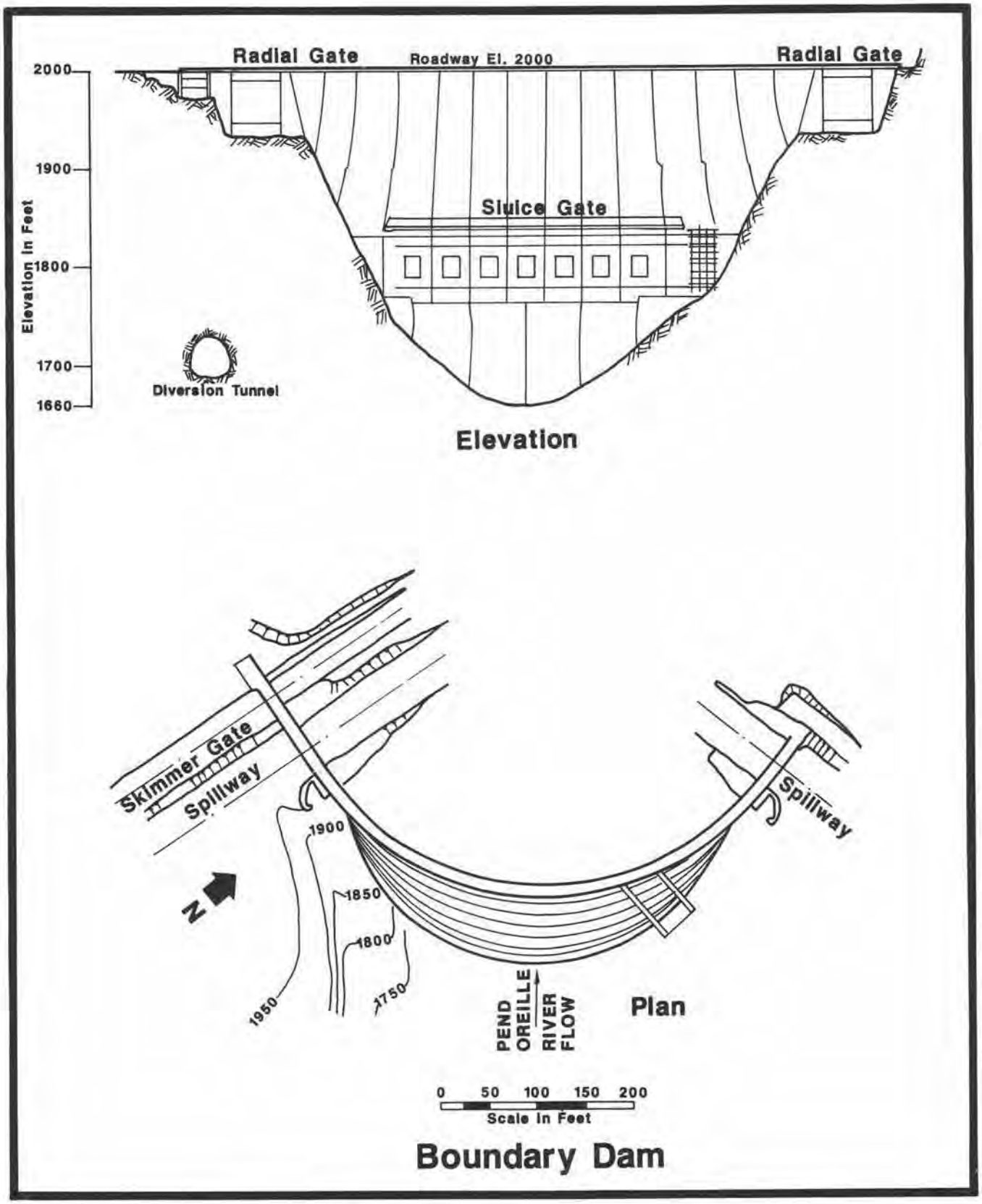


Figure 2. Plan and elevation of Boundary Dam; view downstream.

machine hall was excavated in limestone and dolomitic limestone, each of which has characterisitc physical properties. The limestone was reported by Strandberg (1966) to have a modulus of elasticity of 2.5 x 10^6 psi, and the dolomite a modulus of elasticity of 0.5 x 10^6 psi. Such values were also used in the foundation stability analyses. Because of these variations in physical properties a program was established to monitor deflections in the dam. To date the foundations and abutments have been quite stable. Instruments such as thermometers, strainmeters, deformeters, load cells, deflection targets, flatjack rock stressmeters, and joint movement pins are embedded in the dam and abutments. They are monitored on a regular basis.

High-strength, post-tensioned tendons were installed to stabilize the rock mass on the tailrace cliff against sliding along possible planes of weakness. These tendons were anchored at the lower end in concrete pillars between the draft tubes.

Figure 3 shows the complex layout of the underground powerhouse. The presence of joints and faults did not significantly affect the shaping of the power tunnels nor the stability of the machine hall roof and walls.

A small zone of rhyolite porphyry extends across the forebay area (Coombs and Sarkaria, 1972). Where exposed, the rhyolite has softened, and in the forebay floor the soft material has been replaced with concrete. In 1971 a settlement of 1-1/4 in. was observed on a parapet wall. This was over a contraction joint and above the rhyolite zone. No recurrent movement has been detected.

Mines and Ground Water

During the permit hearings before the Federal Power Commission in 1959, strenuous objections were raised to the dam project because of the presence of operating mines on both sides of and under the reaches of the proposed reservoir. It was felt that increasing the water

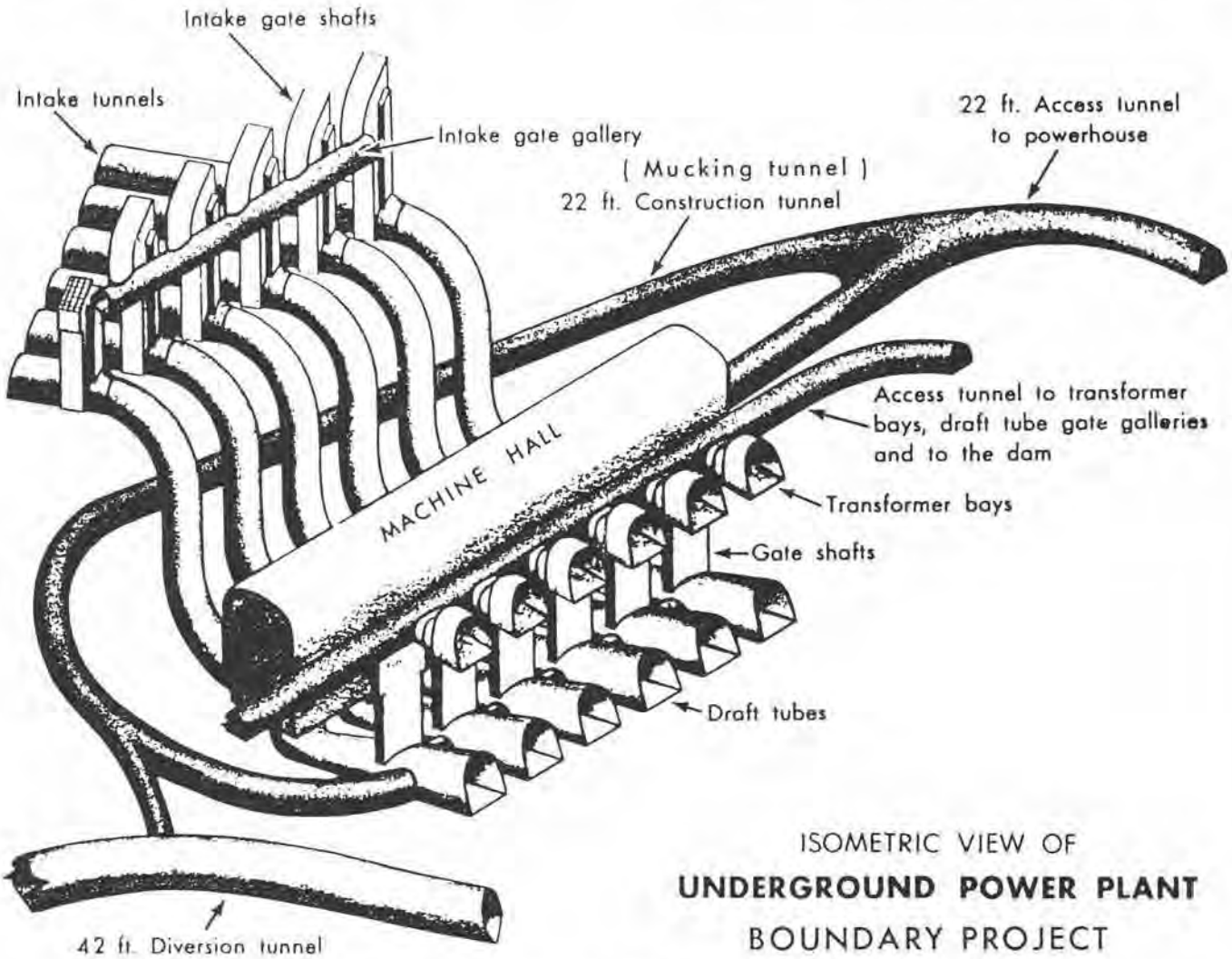


Figure 3. Isometric view of the underground power plant at Boundary dam. Courtesy of Seattle City Light.

level of the river over the mining area might flood the mines. The situation looked particularly hazardous since the mines were in limestone exhibiting numerous solution cavities and fault zones.

Since 1936 there has been considerable mining activity along the upper reaches of the proposed Boundary Dam reservoir. (See Figure 2 of Pend Oreille Dams: Introduction and Geologic Setting.) In 1955 two mines, the Grandview and the Pend Oreille, took out 2,000 tons of ore per day (McConnell and Anderson, 1968; Weissenborn, 1970). The main ore horizon for these lead-zinc ores is in the upper part of the Metaline Limestone just below the Ledbetter Slate. The main access to the mines is along the axis of a gently north-plunging anticline that dips north and roughly parallels the river. The bottom of the deepest mine shaft was approximately 400 ft below the bed of the Pend Oreille River at the time of dam construction.

The mines are unusual in that the unsupported stopes may be as much as 70 ft in height and up to 30 ft wide. Historically, precautions were taken to drill ahead of the working face to locate underground water. Pockets of water in the limestone encountered by such drilling were usually drained in a short time. Solution channels or open faults connected to the river could be disastrous if encountered by mining excavation. One of the mines passes under the river less than 100 ft below the river bed. In spite of these conditions, the mines are remarkably dry. A 400-gpm pump was adequate to remove mine water at the time of dam construction.

In order to establish some basis for evaluating the potential hazard of mine flooding, Seattle City Light engaged Leggett, Brashears and Graham to make a regional study of ground-water conditions, particularly in the area of the mines. To provide a background of conditions, 23 wells were drilled prior to filling the reservoir. Eventually 94 wells were drilled.

The study reported the principal water-bearing rocks to be the Metaline Limestone. By usual standards, the formation has very low permeability except in rare solution channels. The overlying Ledbetter Slate is essentially impervious. Where slate is present above river level, such as at the mines, the ground water in the limestone below is under artesian pressure. The impermeable slate prevents significant movement of water between the river and the underlying limestone. The hydraulic importance of the slate is also shown where major faults have placed blocks of slate against limestone, forming a barrier to the flow of water. The major fault zones between these two units are essentially impermeable. Where the slate is absent, the water table is unconfined, and water levels are responsive to seasonal variations in the recharge rate and changes in the level of the natural discharge point, the Pend Oreille River (Leggett, Brashears and Graham, 1969).

Due to confinement of the water under the slate in the mines area, the piezometric levels are higher than the original surface of the Pend Oreille River and now are higher than the Boundary reservoir level. The exception, of course, is downstream from the dam. These artesian conditions at the mines were influential in allowing a construction permit to be granted for the dam. Subsequent to the filling of the reservoir there has been no known encroachment of river or reservoir water into the mines that would suggest flooding.

Mapping the Reservoir Area

During the permitting process the Federal Power Commission stipulated that all solution channels and faults in the area of the proposed reservoir had to be mapped prior to receiving a construction permit for the dam. The manner of presenting such information in the canyon section of the reservoir was resolved by Coombs et al. (1966) by photographing the steep canyon walls from the opposite bank. A large polaroid camera was used, and geological data were plotted on plastic overlays. Credit must be given to geologist Gerald Hutterer for rope work on the walls and examining in detail the geologic conditions there.

OPERATIONAL PROBLEMS

Hydraulic model tests during the design stage were conducted to determine the operation sequence of sluices and spillways to minimize turbulence in the power-plant tailrace. In early June 1972, when spill exceeded 130,000 cfs for several days, considerable erosion occurred at the left bank parking lot near the entrance tunnel to the powerhouse. It is also possible that a sand bank may have been deposited in the tailrace channel, increasing tailwater elevation.

In October 1972, as part of the 5-yr inspection program, an underwater examination was made by Coombs (1972) of the downstream face of the dam, the draft tube area, and the spillway foundations. A small submarine was used to carry the chief civil engineer and the geologist to examine the areas indicated above. The submarine was electrically operated and could stay submerged for approximately 4 hr. It could carry two men and was equipped with a powerful, moveable, external light, an external 16-mm movie camera, large round windows, a telephone, and a cable to the surface boat. The depth range on this project was between 30 and 60 ft. Because of turbidity, visibility was limited to approximately 15 ft. The submarine moved along the surfaces to be examined at a distance of 5 to 10 ft. Photographs were taken and position fixes made by the boat at the surface. The submarine's position could be determined by those in the surface boat by bubbles emitted from the submarine. The buoyancy of the submarine was so delicately balanced that a 5-lb pull on the steel cable to the surface boat would raise the craft.

The inspection revealed the downstream rock and concrete faces are in excellent condition after a 130,000 cfs flood on a river with a mean annual flow of 25,000-cfs. The brecciated limestone under the right spillway was in excellent shape in spite of the flood during the previous 4 months. Between the dam and the right spillway there was a rock overhang of approximately 3 ft where a block had been plucked out but little or no rounding had been caused by scour. The large concrete plug at the base of the arch dam was fresh and unscarred. At the base of the arch the rocks 4 to 8 ft in diameter were angular, not rounded by scour. Under the left spillway the rock is undercut at a depth of 30 ft with 3 ft of overhang in the form of a steeply dipping face, not an abrupt break. This may be the original rock face.

Use of a submarine has many advantages over hard-hat diving. One can see the surfaces in question, pictures can be taken easily, notes can be jotted down or relayed to the surface via telephone, and there is no need to hurry the examination.

ACKNOWLEDGMENTS

Lawrence R. Whitney, Manager, Civil Engineering, Seattle City Light, provided much information on Boundary Dam. Raymond Hoidal, former Chief of Civil Engineering, Seattle City Light, and the writer had many valuable discussions, both in the field and in the office during the investigation and construction of Boundary Dam.

REFERENCES

Coombs, H. A., 1953, *Geologic report on the Pend Oreille power development and the effect on mining in the Metaline district, Washington*: Prepared for Seattle City Light, Seattle, WA, 17 p.

Coombs, H. A., 1955, *A geologic report on the spillway area at Z Canyon dam site, Pend Oreille River, Washington*: Report to Seattle City Light, Seattle, WA, 5 p.

Coombs, H. A., 1972, *Submarine inspection of Boundary Dam*: Report to Seattle City Light, Seattle, WA, 2 p.

Coombs, H. A.; Cox, W. M.; and Pifer, D., 1966, *Canyon geology*: Report to Seattle City Light, Seattle, WA, 13 p., 123 plates.

Coombs, H. A. and Sarkaria, G. S., 1972, *City of Seattle Department of Lighting, Boundary Dam and Powerplant, F. P. C. Project No. 2144 Safety Inspection*: Prepared for Seattle City Light, Seattle, WA, [chapters individually paginated, photo section and appendices].

Dings, M. G. and Whitebread, D. H., 1965, *Geology and ore deposits of the Metaline zinc-lead district, Pend Oreille County, Washington*: U.S. Geological Survey, Professional Paper 489, 109 p.

Humphrey, F. L., 1955, *Would water impounded by the dam jeopardize mining*: Report to Seattle City Light, Seattle, WA, 6 p.

Leggett, Brashears and Graham, 1969, *Effect of Boundary reservoir on groundwater conditions in the Pend Oreille Valley near Metaline Falls, Washington*: Report to Seattle City Light, Seattle, WA, 34 p.

McConnell, R. M. and Anderson, R. A., 1968, The Metaline district, Washington. In Ridge, J. D. (editor), *Ore Deposits of the United States, 1933-1967*: American Institute of Mining, Metallurgical and Petroleum Engineers, pp. 1460-1480.

Park, C. F. and Cannon, R. S., 1943, *Geology and ore deposits of the Metaline quadrangle, Washington*: U.S. Geological Survey Professional Paper 202, 81 p.

Strandberg, H. V., 1966, *Design and construction features, Boundary project*: American Society of Civil Engineers, Power Division, Vol. 4753, pp. 157-180.

Torpen, B. E., 1953, *Pend Oreille power for Seattle City Light*: Prepared for Seattle City Light, Seattle, WA, 19 p.

U.S. Army Corps of Engineers, 1945, *Preliminary report of the geology of the Boundary Dam site, Washington*: U.S. Army Corps of Engineers, Seattle District, Seattle, Wa, 34 p.

Weissenborn, A. E. (editor), 1970, *Lead-zinc deposits in the Kootenai Arc, northeastern Washington and adjacent British Columbia*: Washington Division of Mines and Geology Bulletin 61, Olympia, WA, 123 p.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Box Canyon Hydroelectric Project

WILLIAM S. BLITON
ATC Engineering Consultants, Inc.

PROJECT DESCRIPTION

The Box Canyon Hydroelectric Project is on the Pend Oreille River in the northeast corner of Washington, 15 mi south of the Canadian border and 90 mi north of the Spokane. The project is approximately 15 mi upstream of Boundary Dam and approximately 55 mi downstream of the Albeni Falls Dam, which is located a short distance upstream from the Idaho Washington border. The project is a "run-of-the-river" development, requiring that the design of the dam and spillway facilities not result in any increase in natural flood water levels upstream of the project.

The project consists of a main spillway dam, a 225 ft-long, 35-ft-diameter, horseshoe-shaped diversion tunnel leading to a forebay channel and auxiliary spillway, and semi-outdoor type four-unit powerhouse (Figure 1). Most of the project features are on the left bank of the river (Figure 2). The project has a maximum gross head of 46 ft and a minimum operating head of about 15 ft. A low concrete gravity section is located along the rock ridge between the forebay channel and the river (Geuss, 1958).

In general, the project's location was constrained by extensive mineral deposits and mining downstream and by the need to avoid encroaching on tailwater at the Albeni Falls powerhouse. The project layout was controlled by topography, geology and the proximity of a Chicago, Milwaukee, St. Paul & Pacific Railroad bridge.

Construction of the project started in September 1952 and was completed in the fall of 1955. The first generating unit began operation in June 1955.

SITE GEOLOGY

The project is located at the downstream end of a narrow canyon that has nearly vertical rock slopes. Bedrock consists primarily of sandstone, argillite, limestone, and dolomite. The foundation rock is marble and dolomitic limestone (part of the Metaline Limestone) of generally excellent quality.

Although bedrock was well exposed throughout most of the project area, seismic refraction and borings revealed a deep infilled channel beneath the existing river channel. This channel extended to approximate elevation 1,800 ft, or approximately 200 ft below the river level. The channel has been interpreted as representing either a cavern deroofed by the modern river (Geuss, 1958) or an overdeepened channel resulting from glacial scour or water scour in a plunge pool below an ancient falls in the river (Harza Engineering Co., 1951). The channel infill material consisted primarily of fine sand under an approximately 10-ft thick surficial layer of boulders. This boulder layer acted as natural riprap protecting the more easily eroded underlying sand. In general, bedrock was near the surface in the sites of the remaining major project structure sites.

GEOLOGIC CONDITIONS AFFECTING DESIGN AND CONSTRUCTION

A 3,000-ft stretch of the canyon extending upstream of the selected dam site was studied to determine the most economical layout for the project. Although congested by the presence of the railroad bridge, the selected site provided the best location considering topography and geology. The diversion tunnel, forebay channel, auxiliary spillway, and powerhouse were excavated primarily in bedrock along the left (south) bank (Figure 2). The auxiliary spillway was located in a bedrock low to minimize excavation. The powerhouse was situated at the bedrock exposure farthest downstream that allowed room for construction of a downstream cofferdam. The location of the diversion tunnel was dictated by the location of the south pier and abutment of the railroad bridge (Geuss, 1958).

Construction of the project was sequenced to use project features for diversion. Initial construction consisted of excavation of the forebay channel and diversion tunnel beneath the railroad alignment. A rock plug topped by a temporary arched concrete bulkhead formed the upstream cofferdam for this construction. Other construction during the period included the forebay gravity wall, the auxiliary spillway, and initial work on the powerhouse structure. After completion of the forebay channel and auxiliary spillway, the upstream plug was blown. Following construction of the main upstream and downstream cofferdams, the river was diverted through the diversion tunnel, forebay channel, and auxiliary spillway. During this period, the powerhouse

Figure 1. Main spillway of Box Canyon Dam. Chicago, Milawukee, St. Paul & Pacific Railroad bridge in the left center; auxiliary spillway in the right center. Photograph courtesy of Harza Engineering Co.

area was protected by an upstream fill cofferdam constructed in the forebay channel downstream of the auxiliary spillway and a natural rock plug on the river side of the powerhouse. After dewatering the area between the main cofferdams, construction of the main dam and spillway structure was completed (Geuss, 1958).

The major geologic feature affecting project design and construction was the deep infilled channel beneath the river and at the site of the main dam and spillway structure. Since the project was low head, it was not economical or necessary to construct a positive cut-off to bedrock in the channel area. The main spillway structure has four bays; the center two bays are located over the channel area. The design solution was to support the center two spillway bays on an arch founded on bedrock at the channel edges and spanning the fine sand infill material. The spillway crest was constructed as part of the arch structure. Three rows of sheet piling were driven and terminated at one-half the depth of the infilled channel. The depth of the sheet piles was determined to be sufficient to contain the foundation materials and increase the seepage path so that the exit velocity of the seepage was within safe limits (Geuss, 1958). Sections through the main dam and spillway structure are presented in Figures 3 and 4.

The upstream and downstream aprons were constructed on the channel infill materials after foundation improvement. A special watertight construction joint was installed between these aprons and the arch-supported crest so that settlement could take place without damage to the joint and thus prevent loss of foundation material. The foundation improvement consisted of dewatering the area and removing the surficial layer of boulders. However, seepage beneath the downstream cofferdam required that part of the new fill be placed by dumping in water. The original backfill had a relative density of about 25 percent. Vibroflotation equipment was used to compact the material to a depth of as much as 30 ft. The equipment consisted of a vibrator approximately 33 ft long, which was inserted to the desired depth by means of a self-contained jet and then slowly withdrawn in 1-ft increments while the vibrator was in operation. As compaction took place, additional backfill material was placed into the opening around the unit. Tests completed after the vibroflotation operation indicated that a minimum relative density of 70 percent was achieved. Additional rows of sheet pile cut-off walls were installed under the upstream and downstream aprons. A 5-ft-thick sand filter overlain by a 1-ft-thick gravel filter was installed beneath the downstream apron.

REFERENCES

Geuss, A. P., 1958, *Box Canyon Hydroelectric Project*: Journal of the Power Division, Proceeding of the American Society of Civil Engineers, Proceeding Paper 1672, p. 1672-1 1672-24.

Harza Engineering Company, 1951, *Project Planning Report-Box Canyon Power Project Pend Oreille River*: Prepared for Pend Oreille County Public Utility District No. 1 by Harza Engineering Company, 108 p, 51 plates.

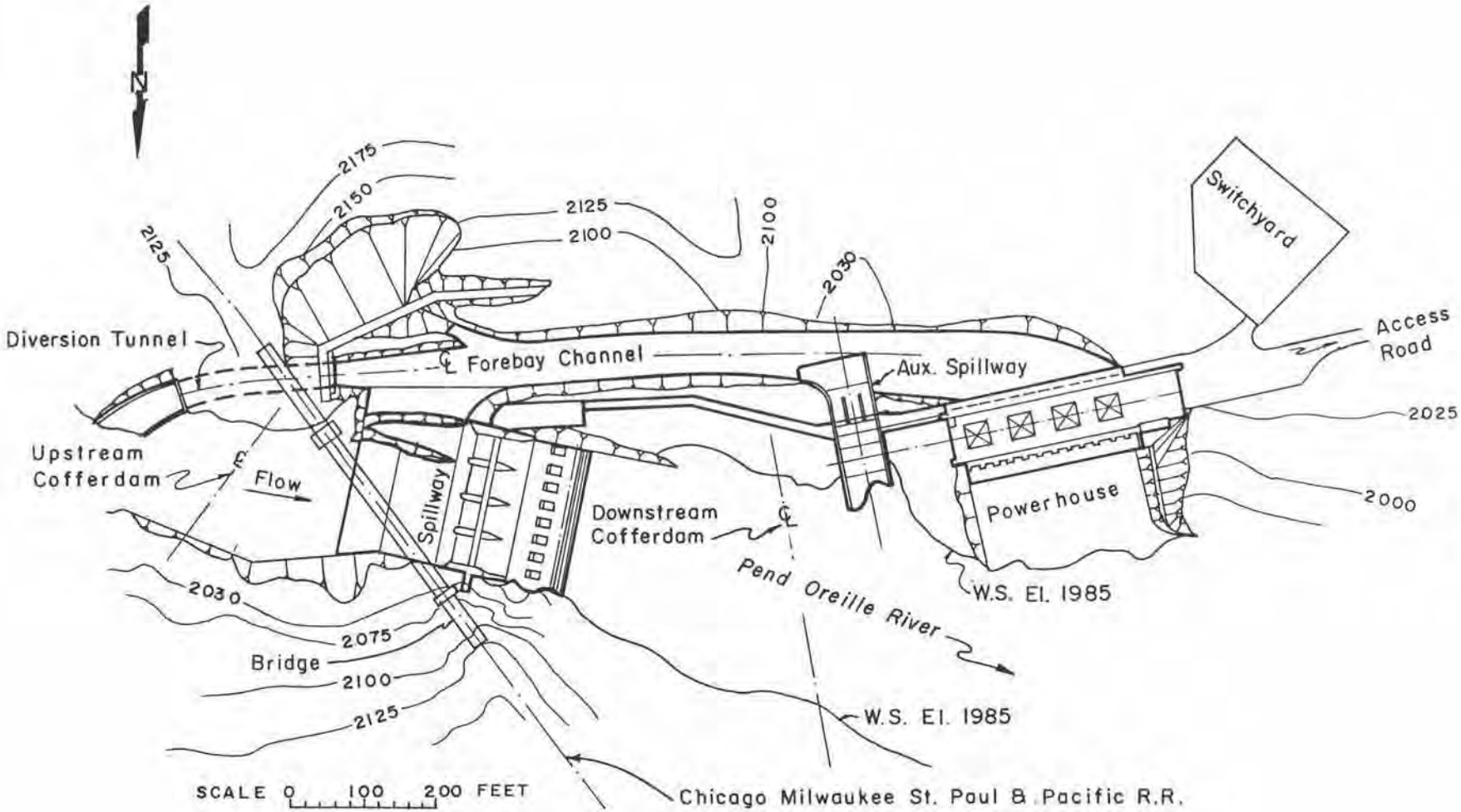


Figure 2. Plan of project works at Box Canyon Dam. Adapted from Geuss (1958).

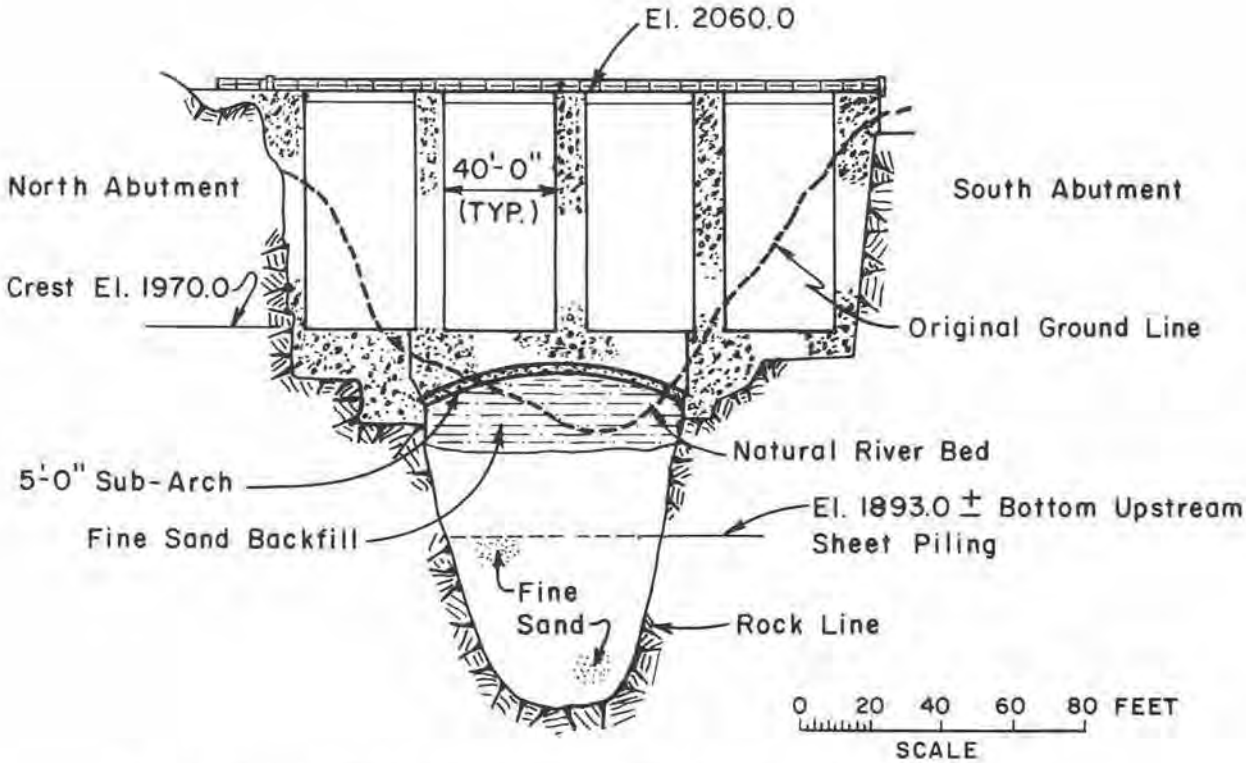


Figure 3. Geologic section at the main spillway of Box Canyon Dam. Adapted from Geuss (1958).

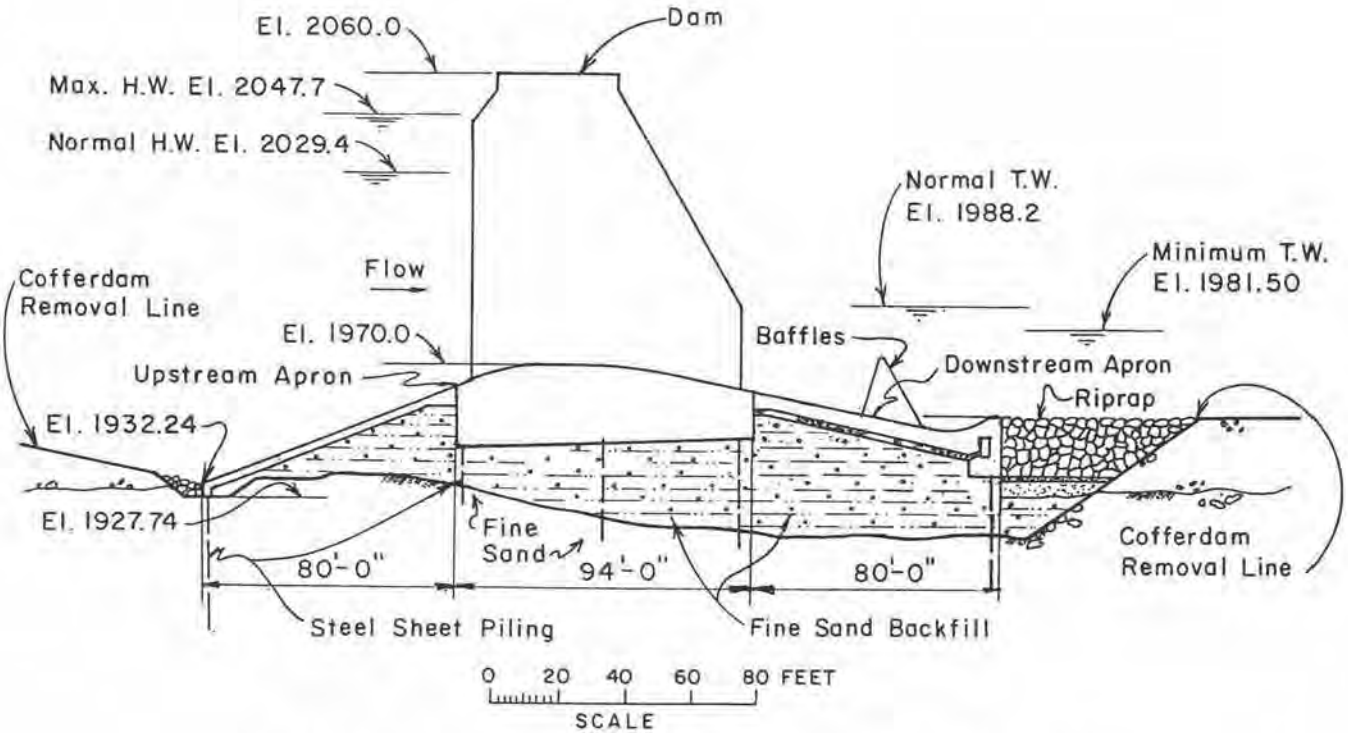


Figure 4. Section at the main spillway, Box Canyon Dam. Adapted from Geuss (1958).

Dams of the Lower Snake River

Introduction and Geologic Setting

Ice Harbor Dam

Lower Monumental Dam

Little Goose Dam

Lower Granite Dam

Rock cut on the relocated line for the Union Pacific Railroad at Lyons Ferry on the Snake River. Gunnite covers a weak volcanic breccia zone beneath columnar basalt. Photograph by R. W. Galster, June 1968.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Dams of the Lower Snake River: Introduction and Geologic Setting

FRED J. MIKLANCIC
U.S. Army Corps of Engineers

The lower reach of the Snake River, extending from near Lewiston, Idaho, and Clarkston, Washington, to its confluence with the Columbia River near Pasco, Washington, flows across the east-central part of the Columbia Plateau in a general westerly direction for approximately 139 mi (Figure 1). The U.S. Army Corps of Engineers constructed four locks and dams along this segment of the river from 1956 to 1975. The locks and dams, in ascending order, are Ice Harbor, Lower Monumental, Little Goose, and Lower Granite. These dams, in conjunction with those on the lower Columbia River, provide slackwater navigation from the Pacific Ocean to the Lewiston-Clarkston area, a distance of nearly 465 mi.

This part of the Columbia Plateau is drained by two principal streams, the Snake River in the south and the Columbia River to the north and west. The source of the Columbia River is in the Canadian Rocky Mountains. The source of the Snake River is in Yellowstone Park in northwestern Wyoming, from where it flows across the southern part of Idaho to the Oregon-Idaho border. Here it turns north (forming the state boundaries) in a mile-deep canyon cut through the Seven Devils Mountain Range. At Lewiston, where it is joined by the Clearwater River, it abruptly turns west along the base of a fault scarp. After following this scarp for approximately 10 mi, it swings northwestward on the first segment of a large-radius arc. The river has entrenched itself in the plateau surface to a depth of 2,000 ft. The larger tributary streams entering the Snake River between Lewiston and its mouth are the Tucannon River and the Palouse River. Both rivers enter the Snake River in the reservoir impounded by the Lower Monumental Lock and Dam (U.S. Army Corps of Engineers, 1969).

The predominant rock type of the Columbia Plateau is a thick sequence of Miocene flood basalts collectively named the Columbia River Basalt Group (CRBG). However, the Snake River drainage contains rocks of many ages, from Precambrian to Recent. Intrusive igneous rocks and extrusive lava flows of numerous types are found associated with sediments that range from loose soil through lakebed sediments, all having various degrees of compaction and consolidation. Limestones, shales, and metamorphic rocks are found underlying these sediments in many areas. Events prior to the Miocene have had little effect on geologic features along the lower Snake River, and at only one location does the river cut through the CRBG.

The flows of the CRBG were extruded through a series of north-trending fissures now preserved as dikes, principally in the southeast corner of Washington and the northeast corner of Oregon. The flows spread out across the plateau as a series of basalt sheets ranging in thickness from only a few feet to 300 ft or more. The flows are interbedded with, and are overlapped by Miocene to Pliocene terrigenous clastic and volcanic sediments along the plateau margins and in subbasins. While the Columbia Plateau is almost entirely underlain by basaltic lava flows, thus giving the impression of being surrounded by the Rocky Mountains, Blue Mountains, and Cascade Mountains, the original surface onto which the lava flows poured out was undoubtedly rough and irregular. To the east and north the flows abut against the flanks of the highlands; to the south they arch upward to the crest of the Blue Mountains. The cumulative thickness of the lava flows in the Pasco Basin is in excess of 10,000 ft (Hooper and Swanson, 1987).

The basalt of the CRBG is tholeiitic, characterized by silica oversaturation; olivine basalt is also present. Flow breccias and other contact characteristics such as pillow lavas, baked soil horizons, and interbed layers are used to distinguish units on a stratigraphic and lithologic basis. The CRBG in the lower Snake River region includes flows of the Imnaha, Grande Ronde, Wanapum, and Saddle Mountains basalts (Figure 2).

The underlying lower Tertiary basement rocks consist of sedimentary assemblages that were intruded and partially metamorphosed by a large igneous mass. Subsequent erosion and covering by the CRBG deeply buried them in the central plateau area, and they crop out only at the margins of the plateau and at isolated locations in deeply incised canyons within the plateau.

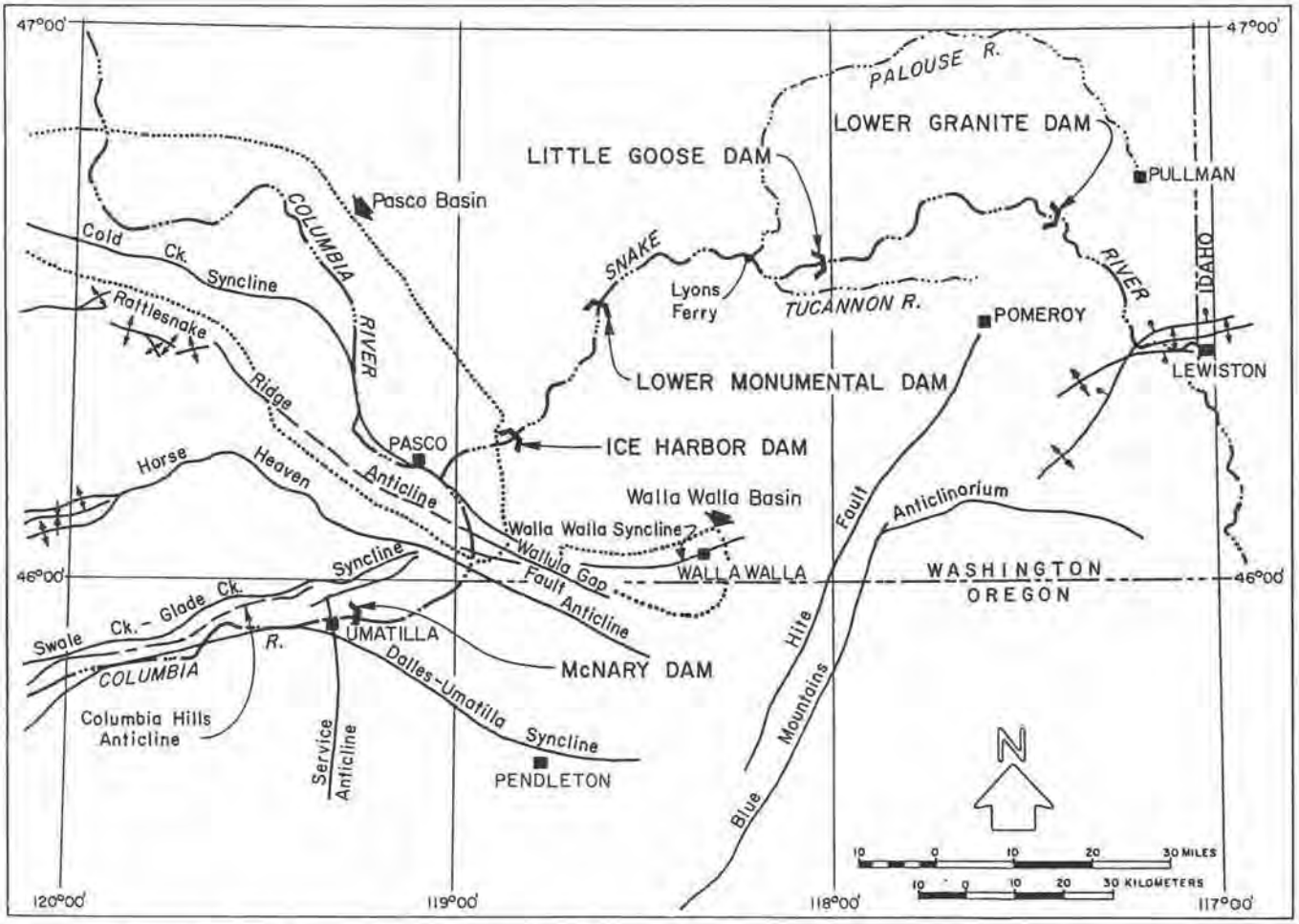


Figure 1. Regional tectonic setting of the lower Snake River and middle Columbia River basins. Modified from Kienle (1980).

Pliocene and Pleistocene sedimentary deposits overlie the CRBG in most of the basin areas. These deposits are correlative with, or contiguous with the Ellensburg Formation and the Ringold Formation in southern Washington and the Dalles Formation in Oregon. They range from coarse to fine, dirty gravels derived from weathering of the Blue Mountains to the south and east, to fine, silty clay lakebed sediments, which were deposited in local basins formed behind late Tertiary structures.

One of the major geologic events that had a significant influence on the shaping of the present landscape in the Snake River area of the plateau was the periodic breaching of ancient glacial Lake Missoula during the late Pleistocene. This lake was formed near the terminus of the continental ice sheets in northern Idaho and western Montana. During these catastrophic events, flood waters cascaded across the plateau surface, stripping soil and gouging large, linear grooves into the rock, forming coulees, and depositing large, bouldery gravel bars along major stream drainages and in basins.

Structurally, pre-CRBG topographic highs that surrounded the region guided the lava flows toward the central part of the plateau in the general vicinity of the Pasco Basin. The cumulative thickness of the lava flows is indicative of general subsidence. Associated with this subsidence were local folding and faulting (U.S. Army Corps of Engineers, 1969). Major structures that subsequently deformed the plateau lavas on a regional scale are the Yakima Fold Belt to the west and the Blue Mountain anticline to the south of the lower Snake River dams. The largest continuous uplift on the plateau is the Blue Mountain anticline, which extends approximately 200 mi from north-central Oregon to the Grande Ronde River in northeastern Oregon and southeastern Washington. The Snake River, near Lower Granite Lock and Dam, skirts the northern flank of the anticline. With the exception of the Lewiston syncline and scattered feeder dikes and northwest-trending faults, the the basalts along the lower Snake River drainage have had no major structural deformation (Kienle, 1980; U.S. Army Corps of Engineers, 1982).

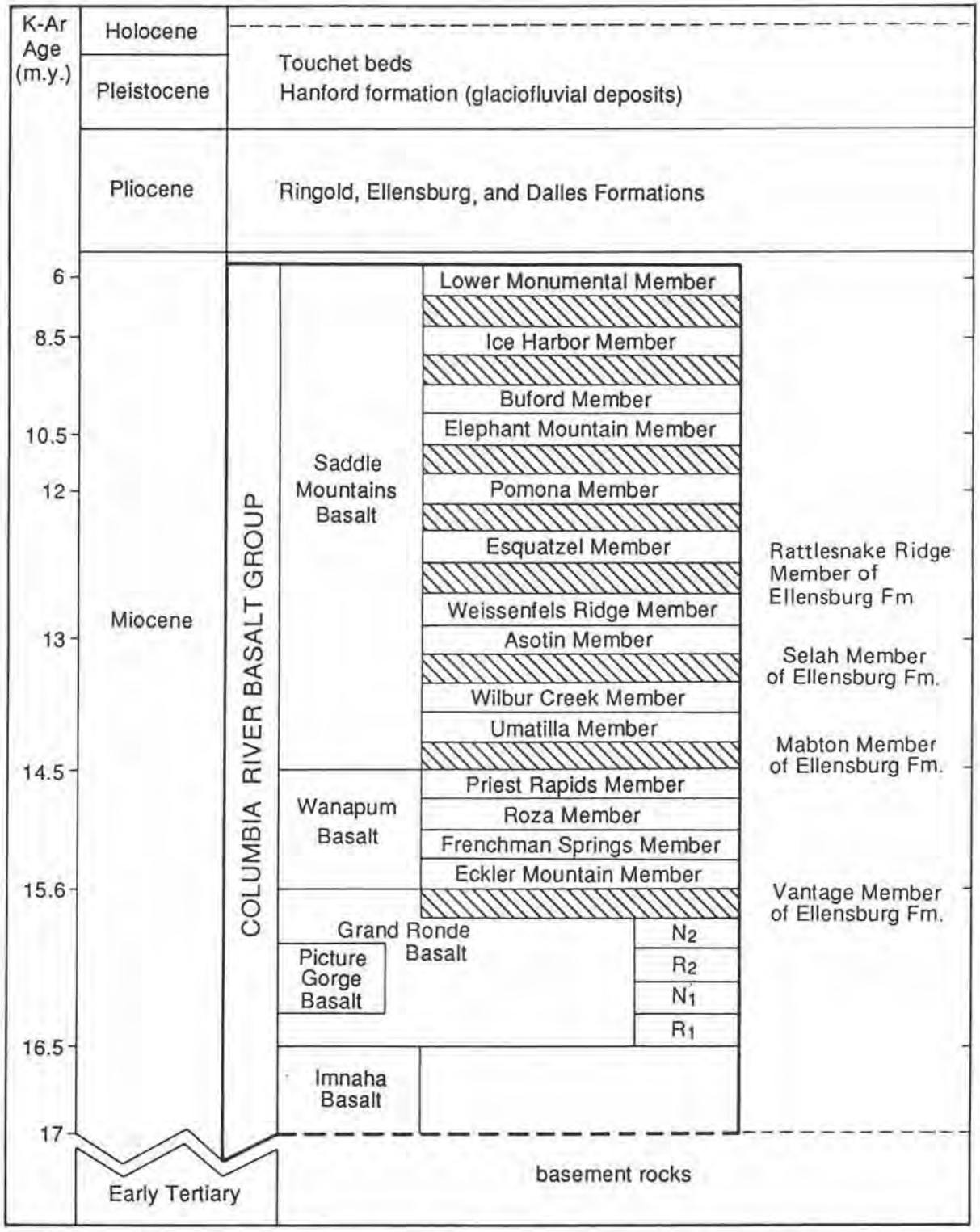


Figure 2. Stratigraphic column for the Columbia Plateau. Modified from Hooper and Swanson (1987) and Swanson (1979).

REFERENCES

Kienle, C. F., Jr., 1980, *Geologic Reconnaissance of parts of the Walla Walla and Pullman, Washington, and Pendleton, Oregon 1°X2° AMS Quadrangles*: Prepared for the U.S. Army Corps of Engineers, Seattle District by Foundation Sciences, Inc. Portland OR, 66 p., 3 plates.

Hooper, P. R. and Swanson, D. A., 1987, Evolution of the eastern part of the Columbia Plateau. In Schuster, J. E. (editor), *Selected Papers on the Geology of Washington*: Washington Division of Geology and Earth Resources Bulletin 77, Olympia, WA, pp. 197-217.

Swanson, D. A.; Anderson, J. L.; Bentley, R. D.; Byerly, G. R.; Camp, V. E.; Gardner, J. N.; and Wright, T. N., 1979, *Reconnaissance Geologic Map of the Columbia River Basalt Group in Eastern Washington and Northern Idaho*: U.S. Geological Survey Open-File Report 79-1363, 26 p., 12 plates.

U.S. Army Corps of Engineers, 1969, *Foundation Report, Lower Monumental Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 55 p., 46 plates.

U.S. Army Corps of Engineers, 1982, *Geological and Seismological Review, Lower Snake River Dams, McNary and Mill Creek*: Prepared for U.S. Army Corps of Engineers, Walla Walla District by U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 44 p., 3 plates.

Waters, A. C., 1961, *Stratigraphic and Lithologic Variations in the Columbia River Basalt*: American Journal of Science, Vol. 259, No. 8, pp. 583-611.

View upstream of the free-flowing Snake River prior to the impoundment of the reservoir behind Little Goose Dam. Photograph by U.S. Army Corps of Engineers.

Ice Harbor Dam

FRED J. MIKLANCIC
U.S. Army Corps of Engineers

PROJECT DESCRIPTION

Ice Harbor Lock and Dam (Figure 1) is a multiple-purpose project located on the Snake River approximately 10 mi above its confluence with the Columbia River. It is the fifth in a series of eight dams on the Columbia and Snake rivers to provide slackwater navigation to the cities of Clarkston, Washington, and Lewiston, Idaho, both approximately 464 mi inland from the Pacific Ocean.

The primary purposes of the Ice Harbor Dam are power generation and navigation, but other benefits include fisheries, recreation, irrigation, and water quality. The project provides a dam with an overall length of 2,822 ft and raises the water surface 100 ft. The dam creates a lake (Lake Sacajawea), which extends 32 mi upstream to the Lower Monumental project. The lake has a surface area of 8,375 acres. The dam consists of a 671-ft-long powerhouse containing six generating units that have a total capacity of 603,000 kw; a 590-ft-long concrete gravity-type spillway dam containing 10 bays; a 173-ft-wide navigation lock with a chamber 86 ft wide by 675 ft long; three sections of concrete non-overflow dams with a total length of 754 ft; and a 624-ft-long earth and rockfill embankment dam located on the right (north) abutment. The project also includes north and south shore fish-passage facilities for migrating fish. The maximum overall structural height of the concrete dam is 208 ft (U.S. Army Corps of Engineers, 1982a).

Construction of the Ice Harbor Project began in January 1956, and the project was placed in operation in January 1962. The final three powerhouse units were installed under subsequent contracts, and all units were producing power by January 1976.

SITE GEOLOGY

At Ice Harbor Dam, the Snake River Canyon is approximately 220 ft deep and 3,400 ft wide. The dam is located near the center of the Pasco Basin and is in an area where linear vent and dike systems transect the Snake River in a general northwesterly direction. The dam is founded on flows of the Elephant Mountain and Pomona members of the Saddle Mountains Basalt. Flows of the Ice Harbor Member are present in the left abutment of the dam, and Ice Harbor Member dikes and a series of small, complex folds are present along the valley walls upstream and downstream of the dam. The surface geology in the immediate vicinity of the dam is shown on Figure 2. A geologic cross section along the centerline of the dam is portrayed in Figure 3.

The present valley of the Snake River is an incised channel cut in glaciofluvial gravels which occupied the older Snake River valley. Modified erosional terraces of limited extent occur on one or both banks along its lower reaches. At the damsite, the right embankment dam rests against and on one of these terraces. Glaciofluvial gravels and loess deposits partially mantle both abutments and the uplands surrounding the dam.

The main foundation rocks are composed of the Elephant Mountain Member and Pomona Member. These units consist of a series of flows interbedded with sedimentary deposits. The basalt of individual flows is fine grained, hard, and structurally competent. Typical flows are characterized by columnar, blocky, and prismoidal jointing. The single flow upon which the dam rests below elevation 335 ft is nearly horizontal and varies in thickness from 105 ft beneath the south non-overflow dam to about 80 ft beneath the powerhouse and spillway. This flow rests irregularly on a tuffaceous silt interbed of varied thickness and also overlies other basalt flows ranging in thickness from 20 ft to 50 ft.

Above the flows of the Pomona Member, and exposed only in the left abutment, is a 40-ft-thick sedimentary interbed of lacustrine origin (Levey interbed). Overlying this interbed is a series of four basalt flows of the Elephant Mountain Member. The total thickness of these flows is 130 ft; individual flows range in thickness from 17 ft to 50 ft (Kienle, 1980; U.S. Army Corps of Engineers, 1982b).

The basalt flows in the site area have a general dip of less than 10° to the southwest. Regional fracture zones or faults having small displacements cut the basalt flows into a group of four blocks between points 1,000 ft upstream and 1,000 ft downstream of the dam. These zones appear to be vertical and form trench-like features cut 100 ft or more into the rock surface. One trench, upstream of the axis, has a northwest trend and dies out on both sides of the river. A second fracture zone

Figure 1. Ice Harbor Dam; view upstream (east). Components of the project, from left to right, are right (north) embankment dam, navigation lock, spillway dam, powerhouse, and south nonoverflow dam. Fish-passage facilities can be seen adjacent to navigation lock and downstream of south nonoverflow dam. Photograph by U.S.Army Corps of Engineers.

downstream of the axis trends almost due north across the valley. Drilling data collected during the exploration phase of the design imply the presence of a fault having about 20 ft of displacement. The dam rests on a downthrown fault block bounded upstream and downstream by high-angle faults. However, foundation excavation revealed only minor shear zones and fracturing within the block (U.S. Army Corps of Engineers, 1959-61).

GEOLOGICAL ASPECTS OF SITING

Subsequent to authorization, studies were carried out to determine the number of sites most economical and satisfactory for development of the lower Snake River. A report was submitted in March 1947 based on these studies and recommended that development be accomplished by the construction of four dams in the 140-mi reach of the river extending from the confluence of the Snake River with the Columbia River near Pasco, Washington, to the confluence of the Snake and Clearwater rivers at Lewiston, Idaho. The exact location of each project was then selected on the basis of more detailed studies to determine the most suitable foundation conditions and site layout for specific features of the dam, particularly those conditions most favorable for navigation. Initial investigations for the Ice Harbor Project were conducted approximately 0.5 mi upstream, but more detailed studies of site stratigraphy and foundation rock conditions indicated the present site to be the most favorable (U.S. Army Corps of Engineers, 1952).

CONSTRUCTION PROBLEMS

Most of the foundation for the concrete structures and the embankment dam consisted of hard, dense, and durable basalt. Excavation for the major components of the dam generally conformed to the original design intent. The only exceptions were for minor resloping of rock cuts or for special foundation treatment where shear zones transected the area. One such feature was a

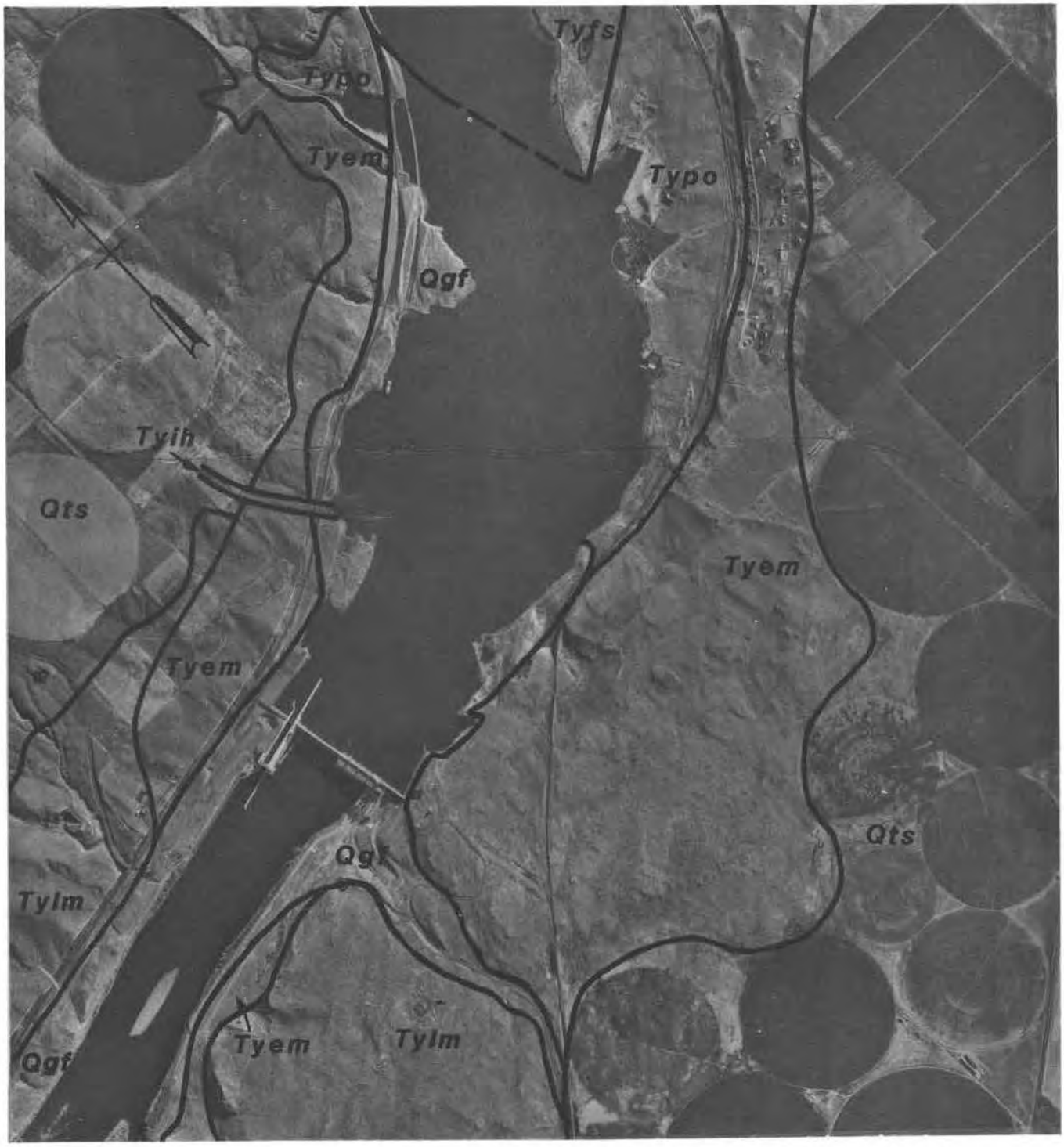


Figure 2. Photogeologic map in vicinity of Ice Harbor Dam. Heavy solid lines indicate approximate geologic contacts. Qts, sand; Qgf, glaciofluvial deposits; Tylm, Tyih, Tyem, and Typo, Lower Monumental, Ice Harbor, Elephant Mountain, and Pomona members of the Saddle Mountains Basalt; Tyfs, Frenchman Springs Member of Wanapum Basalt. Scale: 1 in. = approx. 2,000 ft.

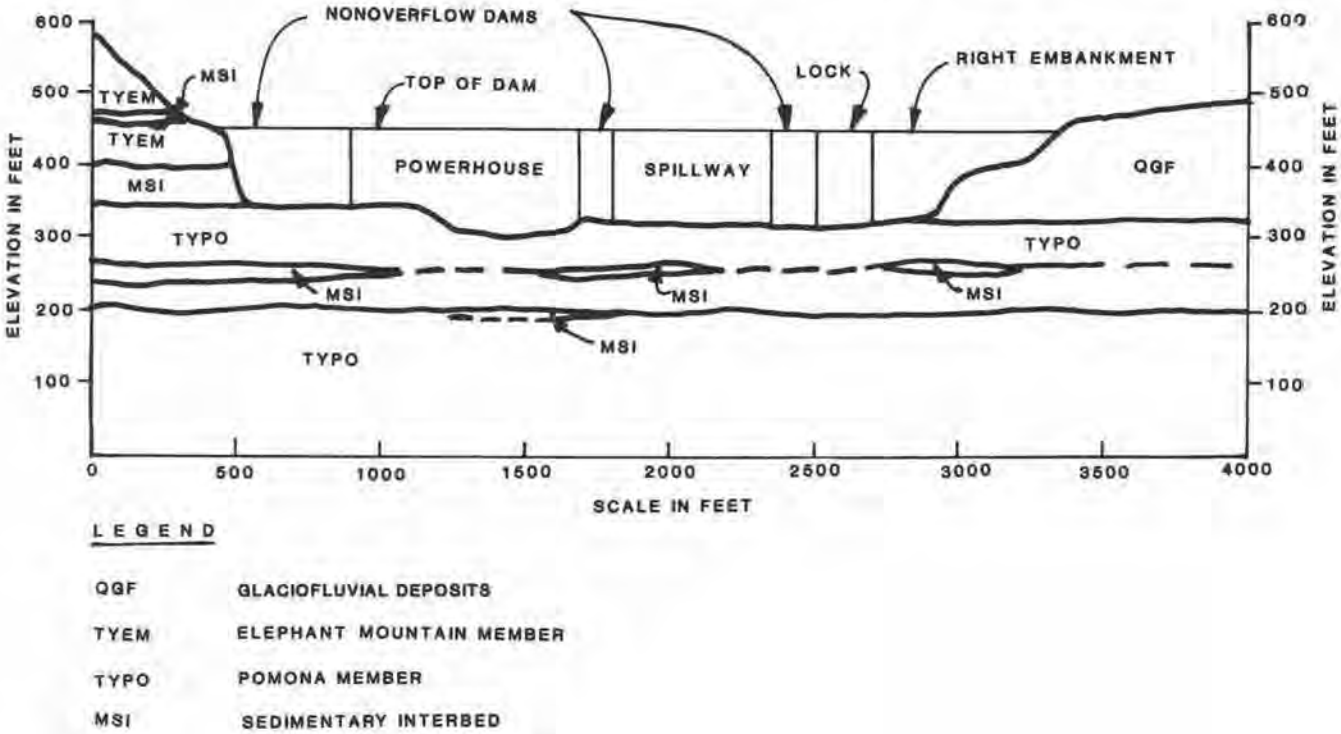


Figure 3. Geologic section of Ice Harbor Lock and Dam; view downstream.

high-angle shear zone located approximately 170 ft downstream of, and parallel to the dam axis. A deep gravel-filled fault zone transecting the river beneath the downstream guide walls to the navigation lock required wood piling support across several monoliths.

The 40-ft-thick interbed zone between two basalt flows near the base of the left abutment presented some problems in assuring stability of the abutment and prevention of end-around seepage. The rock slope and interbed were excavated at a steep angle so that the concrete structure could be founded on competent rock. Spalling of the interbed raised concerns that the overlying basalt was inducing excessive pressures. Therefore, the end monolith of the dam was designed as a "toe block" to provide support for the interbed zone as well as for the abutment rock above. A filter zone was placed from the abutment and extended along the downstream toe of the south nonoverflow monoliths to allow for drainage. Eight rows of holes were drilled and grouted along the concrete-to-rock contact and the upper contact of the interbed zone extending back into the abutment to fill voids and minimize seepage.

The right abutment embankment dam abuts a well graded terrace gravel deposit. To minimize end-around seepage through the gravel, an impervious blanket, consisting of a sandy silt core, a sandy gravel filter, and a gravel shell with riprap protection, extends 900 ft upstream from the dam. During construction, minor modifications to the blanket section were required to assure that highly pervious zones were covered.

A foundation grout drain curtain was placed along the full length of the concrete structures. No grouting was performed beneath the embankment section. The depth of the grout curtain varies from 110 ft at the south end of the dam to as little as 60 ft beneath the powerhouse intake structure. The grouting procedure used was the split-spacing, stage grouting method. The ultimate hole spacing was predicated upon grout takes in adjacent holes and varied between 5 ft and 10 ft. The foundation proved to be relatively impermeable in that the average grout take was approximately 0.1 sacks of cement per linear foot of grout hole (U.S. Army Corps of Engineers, 1959-61).

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Geologic problems at the Ice Harbor Project have been minimal, particularly following the first several years of operation. The principal problem involved the instability of the left abutment immediately upstream of the dam. Prior to pool raise, parts of a huge eolian sand deposit had been buttressed with a wastefill berm to provide stability to the deposit and to provide protection from wave erosion. Although wave erosion was expected on the exposed sand deposit above the berm, it

was a complete surprise to have a major landslide occur; this slide eventually involved an estimated 500,000 cy of material and severed a railroad spur track leading to the dam. The first slide, involving an estimated 50,000 cy, occurred in March 1962, approximately two months after pool raise. In July 1962, the slide became active again and in a matter of a few days extended landward to the basalt bluffs. A third slide involving a much smaller volume occurred in June 1962 about 1 mi above the dam. The sliding was initiated during periods of severe storms and wave action. The sliding progressed rapidly in a series of smaller slides for 2 or 3 days. The erosion developed almost vertical scarps, which caused overstressing of the saturated underlying sand mass and was followed by shear failure and underwater mass flow. The southward migration of the slide scarp continued until it reached the basalt cliffs. Soundings taken subsequent to the slide indicated that underwater mass movement of material had occurred on a nearly flat slope for a distance of approximately 1,200 ft into the reservoir. It was determined that once the slide had progressed to the basalt cliffs, it would become stabilized and would not pose a further threat to the dam (U.S. Army Corps of Engineers, 1966).

Interior drainage into the drainage and grout gallery initially was quite large. Measurements taken immediately following pool raise indicated approximately 7,600 gpm entering the gallery from all sources. In 1966, the flows were again measured and had been reduced about 50 percent to approximately 4,000 gpm. The dramatic decrease over the first several years of operation was attributed largely to the siltation of the reservoir bottom, which was accelerated by the massive landslide on the left abutment. Flows from the gallery have been monitored over the years and have been reduced to about 25 gpm at the present time. These reductions are consistent with experience at other dams where siltation and calcification of concrete cracks have resulted in reduced flows (U.S. Army Corps of Engineers, 1966).

A geological and seismological review was conducted in 1982 to assess the potential for seismic hazards in the vicinity of the Ice Harbor Dam. The review concluded that the Wallula fault zone, located approximately 14 mi to the south and along the Horse Heaven anticline, was capable of generating a magnitude 6.5 earthquake with a peak ground acceleration of 0.38 g (U.S. Army Corps of Engineers, 1982b). Seismic stability analyses (U.S. Army Corps of Engineers, 1988) recently completed indicate that the concrete structures and the right abutment embankment could withstand those motions without failure.

REFERENCES

Kienle, C. F., Jr., 1980, *Geologic Reconnaissance of Parts of the Walla Walla and Pullman, Washington, and Pendleton, Oregon 1°X2° AMS Quadrangles*: Prepared for the U.S. Army Corps of Engineers, Seattle District by Foundation Sciences, Inc., Portland, OR, 66 p., 3 plates.

U.S. Army Corps of Engineers, 1952, *General plan and Detailed Cost Estimate, Design Memorandum No. 1, Ice Harbor Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 132 p., 45 plates.

U.S. Army Corps of Engineers, 1959-61, *Foundation Report, Chapter 1-8–Ice Harbor Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 61 p., 46 plates.

U.S. Army Corps of Engineers, 1966, *Inspection Report No. 1, Ice Harbor Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 25 p., 5 plates.

U.S. Army Corps of Engineers, 1982a, *Project and Index Maps*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 52 p., 41 plates.

U.S. Army Corps of Engineers, 1982b, *Geological and Seismological Review, Lower Snake River Dams, McNary and Mill Creek*: Prepared for U.S. Army Corps of Engineers, Walla Walla District by U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 44 p., 3 plates.

U.S. Army Corps of Engineers, 1988, *Special Report–Dynamic Analysis of Concrete Structures and Embankment Dam, Ice Harbor Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 20 p.

Ice Harbor Dam. A general view of the foundation excavation and partially completed concrete structures contained within the second-step cofferdam on the left (south) shore. The Snake River is being diverted through the area of the future embankment dam. Photograph by U.S. Army Corps of Engineers, September 1957.

Lower Monumental Dam

FRED J. MIKLANCIC
U.S. Army Corps of Engineers

PROJECT DESCRIPTION

Lower Monumental (Figure 1) is a multiple-purpose project on the Snake River approximately 42 mi above the confluence of the river with the Columbia River. It is the sixth in a series of eight dams on the Columbia and Snake rivers to provide slackwater navigation to the cities of Clarkston, Washington, and Lewiston, Idaho, both approximately 464 mi inland from the Pacific Ocean.

The primary purposes of Lower Monumental are power generation and navigation, but other benefits include fish-passage facilities, recreation, irrigation, and water quality. The project provides a dam with an overall length of 3,800 ft and raises the water surface 100 ft from tailwater elevation. The dam creates a lake (Lake Herbert G. West) that extends 29 mi upstream to the Little Goose project. The lake has a surface area of approximately 6,590 acres. The dam is a concrete earth-gravity-type and has short earth- and rockfill embankments on each abutment. It includes a navigation lock with clear dimensions of 86 ft by 650 ft; an eight-bay spillway dam 508 ft long; a powerhouse with six generating units with a total capacity of 810,000 kw; and two fish passage facilities for migratory fish.

Construction of the project began in June 1961, and the project was placed in operation in January 1970. The final three power units were installed under subsequent contracts, and all units were producing power by April 1979 (U.S. Army Corps of Engineers, 1982a).

SITE GEOLOGY

The dam is located just downstream of the mouth of Devils Canyon, a spectacular gash in the basalt flows. At this location, the Snake River has cut a canyon 800 ft deep and 4,400 ft wide. The dam structure is founded on the upper flows of the Grande Ronde Basalt, although both embankments rest on thick glaciofluvial gravels. However, the core and filter zones of the embankments were placed on basalt. The geologic units exposed in the immediate vicinity of the dam are shown on Figure 2.

Silt and sands of the Touchet beds mantle the gravels downstream of the dam. At the mouth of Devils Canyon on the right abutment, a large alluvial fan composed of cobbles, boulders, and coarse black sand has developed. The erosion of the canyon and deposition of materials are probably due in large part to the outpouring of late glacial floods.

Upper flows of the Grande Ronde Basalt are exposed in the lower Snake Canyon walls; the upper slopes consist of intracanyon flows of Saddle Mountains Basalt and the Frenchman Springs Member of the Wanapum Basalt. As much as 200 ft of intracanyon basalt is exposed.

Early design studies inferred a major fault along Devils Canyon. However, more recent studies of the Columbia River Basalt Group (CRBG) stratigraphy and mapping refute the existence of the fault. The intracanyon flows have not been offset. Several small faults have been observed near the dam, but the nearest significant fault is the northwest-striking Burr Canyon fault approximately 5 mi downstream (U.S. Army Corps of Engineers, 1969, 1982b; Kienle, 1980).

The dips of the basalt flows beneath the dam vary somewhat, but are generally northwest at about 1°. Small local reversals of dip within the dam site are common, due in part to the initial attitude of the flows and in part to later minor deformation.

Foundation rock for all structures was either hard, dense basalt or contact-zone material consisting of flow breccia and baked soil horizons. The flows vary in thickness from a few tens of feet to 75 ft or more. Joints in the flows are typical for these types of basalt in that the lower parts of the flow exhibit well developed columnar structure grading upward to more irregular jointing, then to a brickbat structure higher in the flow. The upper surfaces of the flow are generally highly vesicular to scoriaceous. These contact zones generally presented the greatest problems in the foundation because of their permeability and low bearing strength and poor slope stability.

Soils overlying the bedrock are fairly thick on both sides of the river, but at the dam site only a thin mantle of river alluvium was present on the top of rock in the river section. Typical soils are sand, gravel, and silt, deposited either separately or in combination. Under the terraces near the south abutment embankment, gravel deposits are typified by open-work texture and well

Figure 1. Lower Monumental Dam; view upstream (east). Components of the project, from left to right, are right (north) embankment dam, powerhouse, spillway dam, navigation lock, and left (south) embankment dam. Fish-passage facilities can be seen to the left of the powerhouse and adjacent to the navigation lock. Photograph by the U.S. Army Corps of Engineers.

defined foreset bedding. Under the north embankment, the gravel deposit texture was similar except where modified by encroachment of the alluvial fan materials from Devils Canyon (U.S. Army Corps of Engineers, 1969).

GEOLOGICAL ASPECTS OF SITING

Several sites within a 3- or 4-mi reach of the Snake River were investigated to determine the most favorable location for Lower Monumental Dam. The general location was determined by studies subsequent to authorization (as at Ice Harbor and the two upstream locks and dams, Little Goose and Lower Granite). The investigations were part of a comprehensive plan for the development of the lower Snake River that would provide for slackwater navigation to the Lewiston, Idaho, area.

The initial site investigated for the Lower Monumental project was at about river mile 45. Later engineering studies indicated that power and navigation considerations dictated a location downstream of that site, where more favorable backwater conditions for the Ice Harbor pool would exist. General probing of the riverbed at various locations downstream showed that the bedrock surface dropped off rapidly below river mile 41.6. Accordingly, two sites that had similar foundation conditions were investigated, one at river mile 43 and one at river mile 41.6 (the present site). The present site was selected based on economics and engineering comparisons of the site layout. Later detailed investigations confirmed satisfactory foundation conditions for the main concrete structures and embankment, although somewhat erratic and unusual foundation conditions prevailed in the navigation lock area. It was determined, after concentrated investigations in the lock area, that the problems would not be insurmountable (U.S. Army Corps of Engineers, 1958).

CONSTRUCTION PROBLEMS

Three major construction contracts were awarded to complete the main dam and related facilities. The first contract work consisted of construction of a major portion of the north embankment dam and excavation of a

Figure 2. Photogeologic map in the vicinity of Lower Monumental Dam. Heavy solid lines indicate approximate geologic contacts. Ql, loess; Qal, recent alluvium; Qgf, glaciofluvial deposits; Tylm and Tyem, Lower Monumental and Elephant Mountain members of the Saddle Mountains Basalt; Typr, Tyro, and Tyfs, Priest Rapids, Roza, and Frenchman Springs members of the Wanapum Basalt; $TygrN_2$, Grande Ronde Basalt (upper flows of normal magnetic polarity). Scale: 1 in. = approx. 2,000 ft.

diversion channel along the right bank. The second contract was for construction of the south abutment embankment dam and the navigation lock structures. The third contract consisted of completion of the north abutment embankment dam and the concrete structures for the main dam.

Two major foundation problems were encountered during the construction of the project. One was unsuitable foundation rock in the navigation lock, and the second was higher than expected quantities of ground water in the excavation for the powerhouse.

Exploration during design revealed the irregularity of the contact zone on which the lock was to be constructed, and some design changes were expected during construction. The final grade was to be determined in the field by exploratory drilling and observation of excavated surfaces. However, what was not envisioned was the extent to which excavation had to progress to reach a suitable foundation grade. Virtually all of the unsuitable rock was found to be downstream of an undetected fault zone crossing the upstream monoliths. The unsuitable materials consisted of highly vesicular to scoriaceous basalt with phases of flow breccia and ashy material, which are generally at the contact zones. Rather than being a relatively stratified contact zone, it had numerous potholes and pockets, large irregular masses of broken rock, and linear channel-like deposits, all contained within a firm, dense basalt mass. A total of 1,350 probe holes and 27 NX-size core holes were drilled during construction to define the extent of unsuitable materials and to investigate the foundation rock below. The result of these investigations was the requirement to excavate approximately 50,000 cy of additional rock and to extend foundation grade to as much as 55 ft below original design grade. Ground water was not a problem during the deep excavation.

Excavation and foundation quality for the main concrete structures presented no problem in that final grades and design slopes were generally as anticipated. Some overexcavation was required locally, and overbreak was common, particularly on the steeper rock cuts and along contact zones. However, the primary problem in the excavation was the larger than expected volume of water that entered the excavation through highly permeable contact zones. The main source of water was contact 15 (Figure 3), which was intercepted by the powerhouse draft tube excavation. The contractor attempted unsuccessfully to grout this zone. The quantity of water pumped from the excavation during concrete placement varied from 15,000 to 18,000 gpm. The abundance of water greatly hampered concrete placement. Extensive dewatering systems, including french drains and standpipes, had to be used. In most instances, these drainage facilities were grouted when concrete placement reached a higher level. Dewatering was not as serious a problem in the excavation for those structures whose foundation grade was above contact 15.

Excavation for the north and south embankments consisted of removal of terrace gravel deposits down to rock in the areas of the core and filter zones and the removal of silt and other fine materials beneath the shells of the embankment. The exposed rock surface proved suitable and was cleaned of all loose rock and debris prior to placement of embankment materials. Ground water was not a problem.

Foundation grouting was performed as a single line of grout holes throughout the length of the dam, except for a short segment beneath the north embankment dam. The split-spacing, stage grouting method was used, with 10-ft spacing of primary holes inclined upstream on a slope of 18.5° from vertical. Holes beneath the embankment sections were vertical. The final hole spacing varied dependent upon grout takes in adjacent holes, but was generally 10 ft on centers. The depths of the grout holes also were varied depending upon subsurface conditions. Beneath the embankment sections, hole depths were generally less than 50 ft into rock. For the area beneath the concrete sections, the original plan was to grout to contact 15, but during excavation of the powerhouse, it was discovered that the next two lower flow contacts were interconnected and should also be grouted. Therefore, the decision was made to deepen the grout curtain an additional 40 ft across most of the powerhouse and spillway. The maximum depth of the curtain across the powerhouse area was on the order of 125 ft. Grouting was somewhat hampered where ground-water flow from grout holes was experienced. In these areas, special grouting procedures were required to prevent extrusion of the grout by the water pressure.

Approximately 22,000 bags of cement were pumped as grout into 437 grout holes, totaling 51,000 linear feet of hole for the entire project. The overall grout take averaged 0.21 bags of cement per foot of hole.

A drain curtain was installed along the downstream side of the drainage and grouting gallery to relieve any excessive water pressure that would result from the effects of the full reservoir. The specified depth of 60 ft in the contract was modified to preclude penetrating contact 15, which is the principal artesian aquifer zone. Penetration of this zone would have resulted in a volume of water greater than the design capacity of the drainage system; drainage of contact 15 was not determined necessary for required reduction of hydrostatic uplift pressures. The source of the large flows would be from tailwater, the assumption being that the grout curtain would adequately seal this zone from effects of the Lower Monumental Pool (U.S. Army Corps of Engineers, 1969).

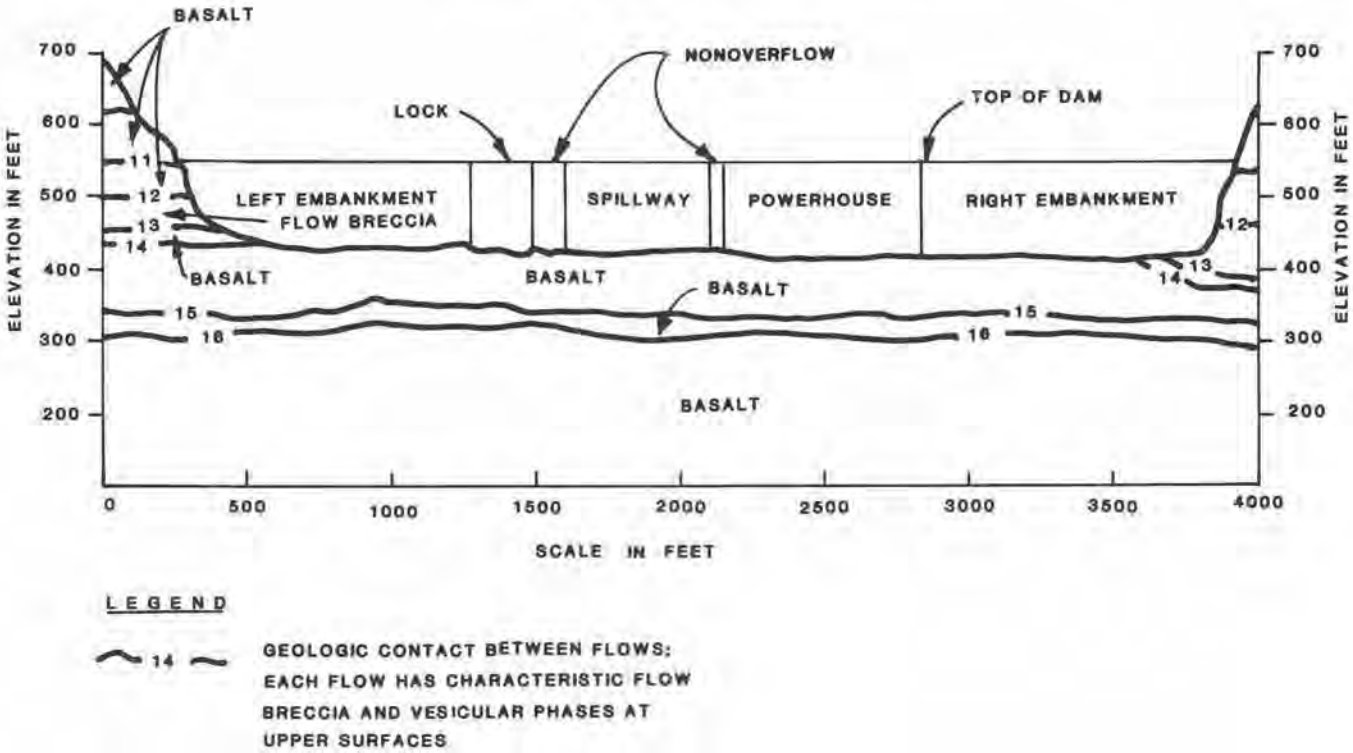


Figure 3. Geologic section of Lower Monumental Lock and Dam; view downstream.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

There have been no significant geology-related problems at the Lower Monumental project since its construction. Although there has been significant movement of the concrete structures, particularly in the navigation lock, causing spalling of concrete along the contraction joints, no movement has been attributed to compression of the rock foundation.

The grout and drain curtains have performed as expected. Total flow from all drain holes measured in February 1969 shortly after pool raise ranged between 60 and 200 gpm. The flows measured prior to the latest structural inspection indicate inflow from the drains to be of the same magnitude.

Shortly after pool raise in January 1969, uplift pressures at piezometers located beneath the intake slab of powerhouse unit 6 indicated a possibility of exceeding the design assumption pressures for full pool. Drain holes were drilled into the area to relieve the pressure. Subsequent analysis of the data indicated the high piezometric conditions were due to hydraulic conditions at the dam when the first three turbines were being installed and not due to effects of the reservoir.

A geological and seismological review completed in 1982 to assess the potential for seismic hazards in the vicinity of Lower Monumental Lock and Dam concluded that the active fault zone nearest the project was the Wallula fault zone, approximately 37 mi southwest. The predicted attenuated bedrock acceleration at the site for the maximum potential earthquake on this fault was 0.10 g. The original design assumption used this value. Therefore, no further seismic design analysis was required (U.S. Army Corps of Engineers, 1982b).

REFERENCES

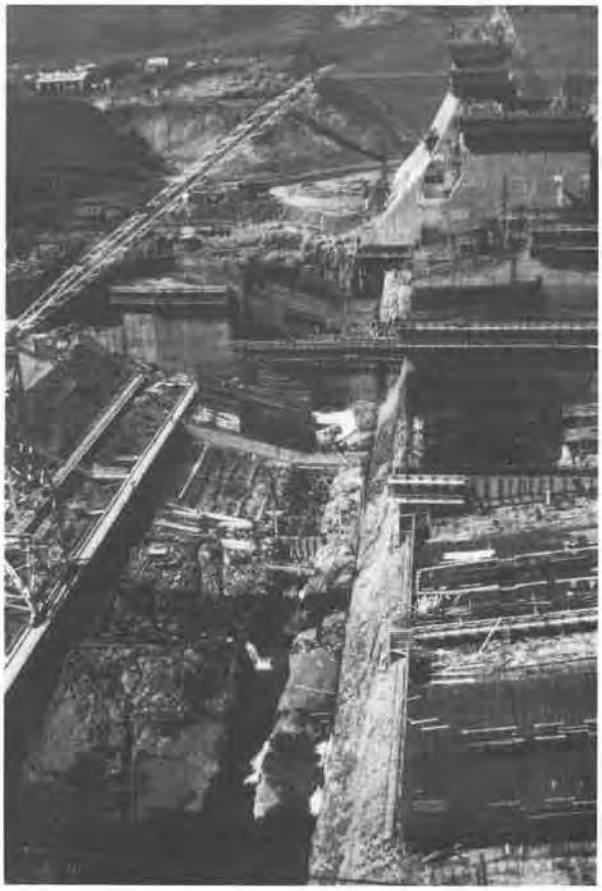
Kienle, C. F., Jr., 1980, *Geologic Reconnaissance of Parts of the Walla Walla and Pullman, Washington, and Pendleton, Oregon 1°X2° AMS Quadrangles*: Prepared for the U.S. Army Corps of Engineers Seattle District by Foundation Sciences, Inc., Portland, OR, 66 p., 3 plates.

U.S. Army Corps of Engineers, 1958, *Letter Report on Site Selection, Lower Monumental Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla, WA, 24 p., 11 plates.

U.S. Army Corps of Engineers, 1969, *Foundation Report, Lower Monumental Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 55 p., 46 plates.

U.S. Army Corps of Engineers, 1982a, *Project and Index Maps*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 5 p., 41 plates.

U.S. Army Corps of Engineers, 1982b, *Geological and Seismological Review, Lower Snake River Dams, McNary and Mill Creek*: Prepared for U.S. Army Corps of Engineers, Walla Walla District by U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 44 p., 3 plates.



Lower Monumental Dam. View to the north from the top of the completed spillway dam showing the powerhouse excavation on April 12, 1966. Photograph by R. W. Galster.

Little Goose Dam

FRED J. MIKLANCIC
U.S. Army Corps of Engineers

PROJECT DESCRIPTION

Little Goose Dam (Figure 1) is a multiple-purpose project located on the Snake River approximately 70 mi upstream of its confluence with the Columbia River. It is the seventh in a series of eight dams on the Columbia and Snake rivers that provide slackwater navigation to the cities of Clarkston, Washington, and Lewiston, Idaho, both approximately 464 mi inland from the Pacific Ocean.

The primary purposes of the Little Goose Dam are power generation and navigation, but other benefits include fish-passage facilities, recreation, irrigation, and water quality. The dam, located at the head of Lake Herbert G. West, is 2,655 ft long and has an effective height of 98 ft. The dam creates a lake (Lake Bryan), which extends up the Snake River approximately 37 mi to Lower Granite Dam and has a surface area of approximately 10,000 acres. The dam is a concrete gravity type and has an earth- and rockfill embankment on the right (north) abutment; a navigation lock with clear dimensions of 86 ft by 675 ft; an 8-bay spillway dam 512 ft long; a 6-unit powerhouse 656 ft long that has a total generating capacity of 810,000 kw and a fish-passage facility for migratory fish located adjacent to the navigation lock on the left abutment.

Construction of the project began in June 1963, and the project was opened for navigation in May 1970. Installation of the initial three generating units was complete in December 1970. The additional three units were constructed under subsequent contracts, and all units were producing power by July 1978 (U.S. Army Corps of Engineers, 1982a).

SITE GEOLOGY

The Little Goose Dam is located approximately 10 mi upstream of the confluence of the Palouse River with the Snake River at Lyons Ferry. At the dam site, the Snake River has cut a 1,100-ft-deep, 3,600-ft-wide canyon. The dam is founded on a sequence of basalt flows that form the upper part of the Grande Ronde Basalt. The surficial geology in the immediate vicinity of the project is shown on Figure 2. A geologic cross section along the centerline of the dam is shown on Figure 3.

The basalt flows at the dam site range in thickness from 30 ft to more than 100 ft. Each flow is composed of a lower, hard, dense basalt exhibiting columnar jointing that grades upward into a more irregular, brickbat structure. The top of each flow is generally characterized by highly vesicular to scoriaceous basalt to a cindery, ashy layer. These contact zones are the primary concern with regard to attaining a suitable foundation for the structures and are usually the primary route of ground-water migration.

The regional dip of the basalt flows is very gentle, 1°-3° southwest, although local variations and dip reversal are common due to faulting and minor deformation both upstream and downstream of the site. Excavation for the powerhouse exposed a major thrust fault that has a northeast strike and a 26° dip to the southeast. Other shear zones and faults with general northwest trends were exposed throughout the foundation area for the concrete structures. Generally, these reflect the regional trend on a local scale, and they presented no difficulties for foundation stability. A major artesian aquifer passes beneath the foundation at a depth of approximately 150 feet; this was one of the prime concerns during design (Figure 3).

Remnants of a large glaciofluvial gravel bar are present along both sides of the river extending from the dam site downstream toward Lyons Ferry. The canyon walls above the dam are composed of flow sequences assigned to the Frenchman Springs and Roza members of the Wanapum Basalt. The canyon walls also expose several remnant intracanyon basalt flows of the Saddle Mountains Basalt (Kienle, 1980; U.S. Army Corps of Engineers 1971, 1982b).

GEOLOGIC ASPECTS OF SITING

Three sites between river miles 70 and 76 on the Snake River were investigated for siting of the Little Goose project. The general location had been identified by earlier studies subsequent to authorization for the most comprehensive and economic plan for development of the river system.

The three specific sites studied, at river miles 70.3, 72.2, and 75.3, were judged to be the most favorable on the basis of topography and geology. An engineering

Figure 1. Little Goose Dam; view upstream (east). Components of the project from left to right are right (north) embankment dam, spillway dam, powerhouse, and navigation lock. Fish-passage facilities are adjacent to navigation lock. Photograph by U.S. Army Coprs of Engineers.

comparison of all factors, including explorations and surficial geology, optimum site layout of the navigation lock and other components of the dam, and economics, indicated that the most favorable site was at river mile 70.3. Subsequent foundation investigations confirmed this site to be adequate for the structures intended (U.S. Army Corps of Engineers, 1961).

CONSTRUCTION PROBLEMS

The construction of the Little Goose project was actually completed in two phases. The first phase consisted of building a cofferdam for river diversion to the right bank and partial excavation within the cofferdam. The final phase consisted of the major foundation excavation and construction of the main dam.

There were two main concerns with the foundation work during the construction phase of the project. One was the effects that the presence of the artesian aquifer zone might have on the open excavation, particularly in the powerhouse area where the excavation was to be at its deepest point. The other concern was the quality of rock in the powerhouse intake area where a low-angle thrust fault or shear zone was present.

The principal artesian aquifer occurs in the foundation at approximately elevation 300 ft, or about 150 ft below design grade. This aquifer zone was studied in great detail during design phases, first to determine if the high pressures (140 psi) calculated to be exerted on the foundation from below could rupture the overlying confining basalt flows that make up the foundations for the dam, and secondly, to determine uplift pressures and drainage requirements for the completed project. The study concluded that it would be safe to excavate to the design depths and that there would be no rupture of the basalt between the excavation and the aquifer. However, it was concluded that the artesian zone could be expected to transmit water vertically through faults, broken zones, or other open pathways; this water would be mixed with any water entering the two intervening contact zones (contacts 14 and 15, Figure 3). With the

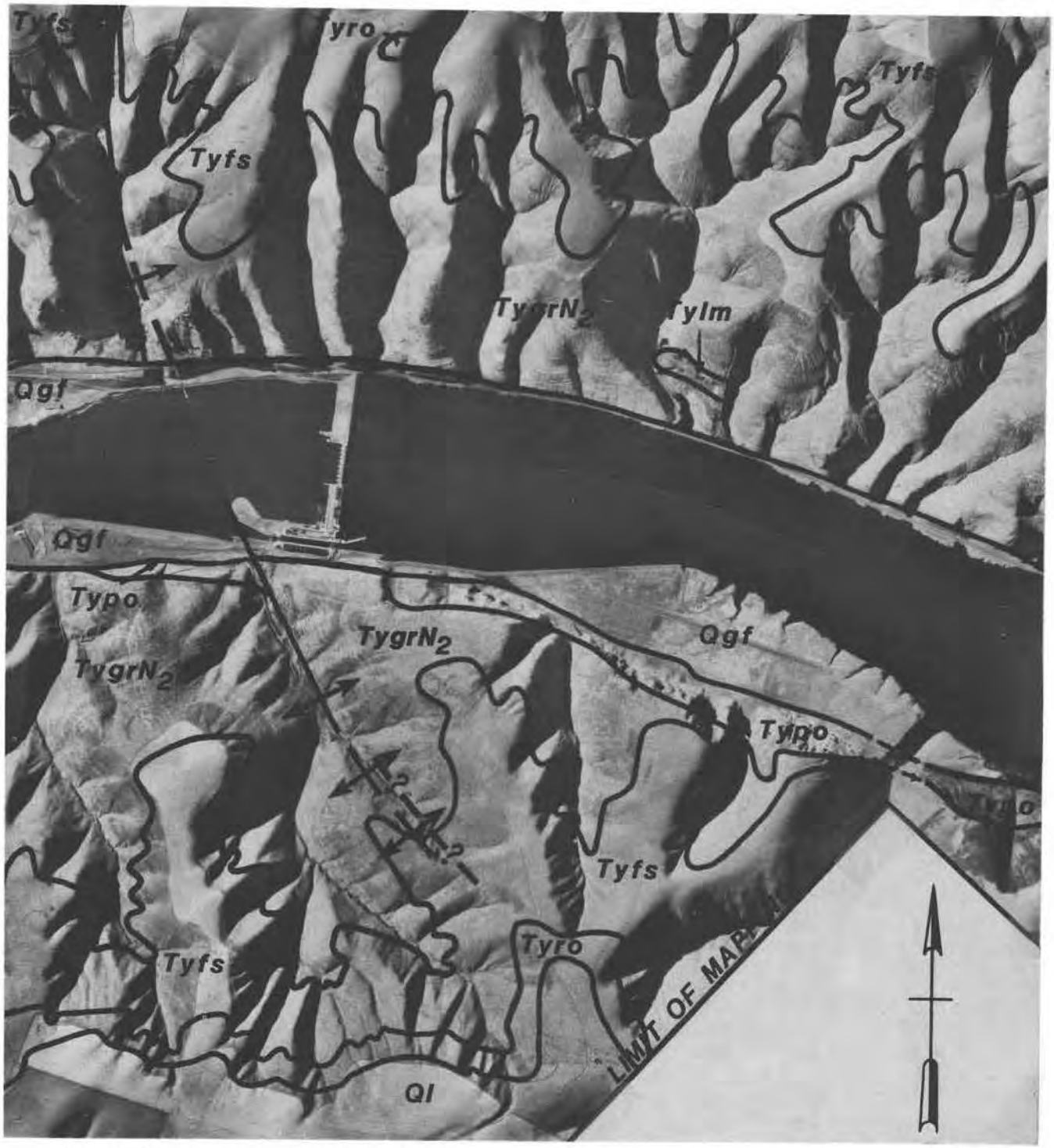


Figure 2. Photogeologic map of the Little Goose Dam area. Solid line indicates approximate geologic contacts. Ql, loess; Qgf, glaciofluvial deposits; Tylm and Typo, Lower Monumental and Pomona members of Saddle Mountains Basalt; Tyfs and Tyro, Frenchman Springs and Roza members of the Wanapum Basalt; $TygrN_2$, Grande Ronde Basalt (upper flows of normal magnetic polarity). Scale: 1 in. = approx. 2,000 ft.

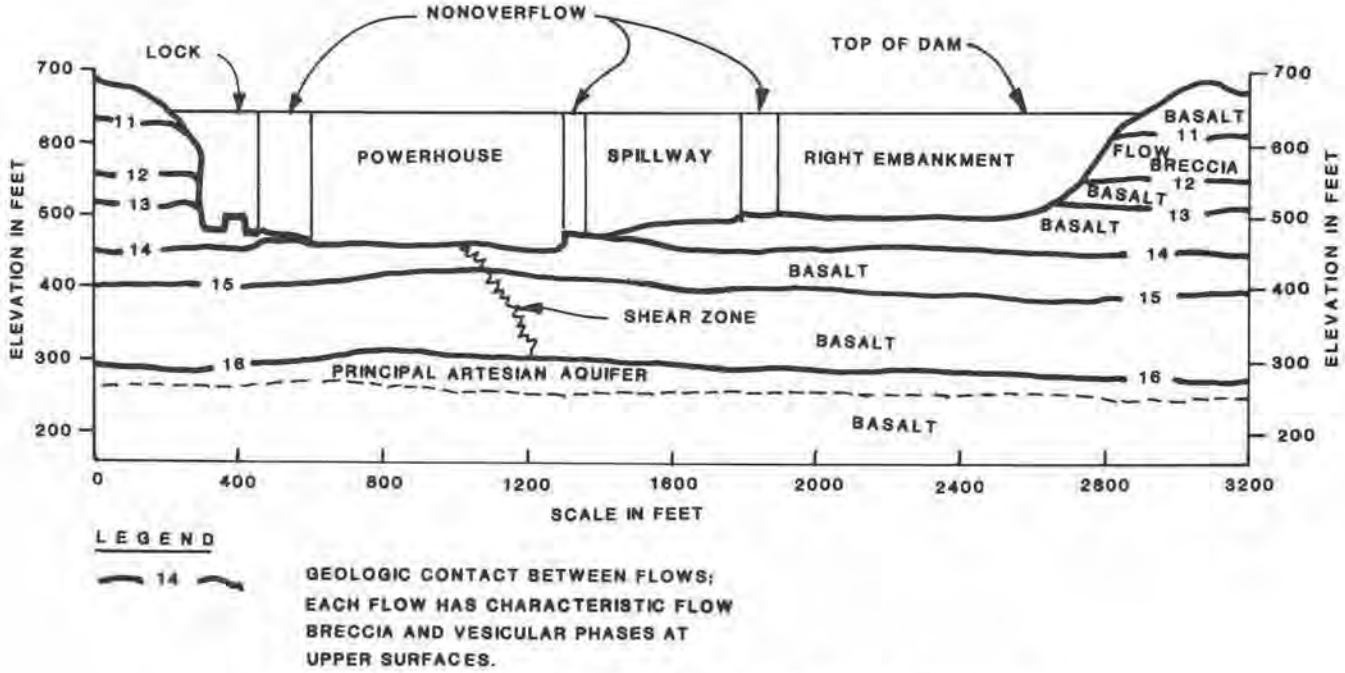


Figure 3. Geologic cross section at Little Goose Dam; view downstream.

effects of tailwater combined with uplift contributed by the artesian zone and a small addition from the forebay, excessive uplifts beneath the structures could be expected. Therefore, engineers decided to design the structures for uniform uplift pressures applied throughout the base of the structure; this amounts to tailwater pressure plus 50 percent of the difference between tailwater and forebay. In addition, each component of the dam was designed with its own intricate drainage system. These essentially consist of the normal upstream drainage and grouting gallery and a second gallery located toward the downstream portion of the structures. Drain holes were drilled in fan patterns from these galleries and extended through contacts 14 and 15 to provide the most uniform distribution of drainage possible (U.S. Army Corps of Engineers, 1964).

Ground-water control was not as great a problem during excavation as might have been expected, considering the underlying artesian zones. However, the deeper parts of the excavation did require extensive french drains and standpipes to control water inflow, particularly when the excavation intercepted the contact zones. Following placement of concrete, most of the temporary drainage systems were grouted closed.

The foundation rock conditions encountered during excavation were generally as predicted during the design. The low-angle fault zone in the powerhouse intake area contained numerous associated shear and fracture zones of varying orientations that splayed off the main fault. Analyses were performed to determine if additional rock excavation or addition of shear keys to increase shearing resistance should be constructed. The conclusion was that the existing foundation rock would provide an adequate foundation.

The primary foundation treatment was the placement of the grout curtain along the upstream heel of the dam in the drainage and grouting gallery. The depth of the grout curtain was designed to extend 15 ft below the flow breccia zone (contact 15) at approximate elevation 400 ft. A tailrace grout curtain was placed near the downstream toe of the powerhouse and extended through the same contact zone to seal it from the tailwater. This curtain was extended south to tie into the fish entrance facility on the south shore, and it was also connected to the upstream grout curtain with a segment along the north end of the powerhouse.

The split-spacing, stage grouting method was used with initial spacing of EX-grout holes 20 ft on center with a final spacing of 5 ft on center. The depth of holes varied depending upon the collar elevation of each hole, but all holes generally were extended to approximately elevation 400 ft. Approximately 53,000 linear feet of grout hole were drilled and more than 36,000 sacks of cement grout placed. The average injection rate for the main upstream grout curtain was approximately 0.63 sacks of cement per foot of drill hole. For the tailrace grout curtain, the average rate was approximately 1 sack per foot of hole (U.S. Army Corps of Engineers, 1971).

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Ground-water problems were recognized before completion of the structure. Early in 1969, the pool behind Lower Monumental Dam was raised, which raised the tailwater at the Little Goose project to the design level. There was an immediate flooding of the gallery system. The flooding was attributed to the internal drainage system and the numerous drain holes that had been extended into the foundation beneath the dam. Since the galleries sloped to the north, the galleries extending from bay 1 or the powerhouse northward were totally submerged. The installed temporary drainage pumps located in the main sump at the north end of the spillway could not handle the inflow. Provisions were immediately made to divert a portion of the water to the powerhouse sump by use of dewatering pumps placed in the galleries. Concurrently, caps were placed on drain holes beginning at the south end of the dam and continuing northward. As the drain holes were capped, total inflow gradually decreased. This process continued in a northward direction until nearly all drain holes in all galleries were capped and the inflow was finally controlled.

To monitor uplift and to assure the drain capping program was not detrimental to the safety of the project, each drain cap was adapted with a quick release pressure fitting. This allowed frequent pressure readings to monitor the uplift in each hole to assure that the design uplift was never exceeded. The maximum pressures recorded were approximately 30 psi, which reflected tailwater pressures.

The raising of the Little Goose pool took place a year later, in 1970. Inflows from joints, leakage from some of the drain holes, and water from other sources were closely monitored, as were the pressures on the individual drain holes. In some places, pressures showed slight increases, but overall indications were that the headwater had very little effect on the uplift beneath the structures.

The project has continued to operate in this condition since 1970; no problem has been associated with uplift. One current concern is the deterioration (by rusting) of many of the drain caps. Investigations are under way to attempt to permanently plug the drain holes near the collars to alleviate this problem.

The only other serious structural problem attributable to the foundation occurs in the navigation lock. Since the lock became operational, the cyclic emptying and filling of the lock has caused individual monoliths to deflect outward. These deflections were greater on the north (riverside) monoliths, but they also occurred on the south side.

The lock monoliths have a buttress stem design; the main portion of the riverside monoliths, containing the underlying emptying and filling culverts, are founded on a competent basalt flow. The buttress stems for these monoliths were founded on the next higher basalt flow, which has a thickness of approximately 25 ft and a thin flow breccia zone at about the same elevation as the bottom of the culvert section. The basalt beneath the stems is hard and dense, but the rock is highly fractured and contains several low-angle shear zones.

Maximum relative deflections at the tops of monoliths on the river side of the lock were as much as 0.7 in. This caused rupturing of nearly all the waterstops between monoliths and caused concrete spalling at many of the contraction joints. Considering the quality of the foundation rock, and attributing this problem to permanent deformation of the rock foundation beneath the buttress stems, a contract was awarded in 1978 to consolidate the foundation by grouting. Grout holes were spaced on an approximate 8-ft pattern and extended through the thin basalt flow immediately beneath the buttress stems, through the flow breccia contact zone, and into the underlying dense basalt. The grout take in the rock sections averaged approximately 1 sack of cement grout per foot of grout hole. Measurements of deflection subsequent to grouting indicated some improvement, but grouting has not entirely solved the problem. Instrumentation data are being collected and analyzed to determine if post-tensioning the structure to the foundation might be a solution. All ruptured waterstops have been replaced with 6-in.-diameter holes filled with a chemical grout; this has partially arrested the leakage.

A geological and seismological review of the project was completed in 1982 to assess the potential for seismic hazards. This review concluded that the controlling earthquake for the site was magnitude 6.2 in the Wallula fault zone approximately 37 mi southwest of the project. The attenuated site ground acceleration was calculated to be 0.1 g. Since the project was designed for an earthquake of this magnitude, no additional seismic analysis of the structures was considered necessary (U.S. Army Corps of Engineers, 1982b).

REFERENCES

Kienle, C. F., Jr., 1980, *Geologic Reconnaissance of Parts of the Walla Walla and Pullman, Washington, and Pendleton, Oregon 1°X2° AMS Quadrangles*: Prepared for the U.S. Army Corps of Engineers Seattle District by Foundation Sciences, Inc., Portland, OR, 66 p., 3 plates.

U.S. Army Corps of Engineers, 1961, *Site Selection and Upper Pool Determination, Design Memorandum No. 1, Little Goose Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 67 p., 27 plates.

U.S. Army Corps of Engineers, 1964, *Foundation Grouting and Drainage, Design Memorandum No. 20, Little Goose Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 30 p., 38 plates.

U.S. Army Corps of Engineers, 1971, *Foundation Report, Little Goose Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 67 p., 27 plates.

U.S. Army Corps of Engineers, 1982a, *Project and Index Maps*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 52 p., 41 plates.

U.S. Army Corps of Engineers, 1982b, *Geological and Seismological Review, Lower Snake River Dams, McNary and Mill Creek*: prepared for U.S. Army Corps of Engineers, Walla Walla District by U.S. Army Corps of Engineers, Seattle District, Seattle, WA., 44 p., 3 plates.

Little Goose Dam. A general view of the foundation excavation for the spillway, powerhouse, and navigation lock, all contained within the first-step cofferdam. The Snake River is being diverted through the area of the future embankment dam. Photograph by U.S. Army Corps of Engineers, March 1966.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Lower Granite Dam

FRED J. MIKLANCIC
U.S. Army Corps of Engineers

PROJECT DESCRIPTION

Lower Granite Dam (Figure 1) is a multiple-purpose project on the Snake River approximately 107 mi upstream from the confluence with the Columbia River. It is the last in a series of eight locks and dams to provide slackwater navigation to the cities of Clarkston, Washington, and Lewiston, Idaho, both approximately 464 mi inland from the Pacific Ocean.

The primary purposes of the Lower Granite Dam are power generation and navigation, but other benefits include fish passage, recreation, irrigation, and water quality. The dam, located at the head of Lake Bryan, is approximately 3,200 ft in length and has an effective height of 100 ft. The dam creates Lower Granite Lake, which extends 38 mi up the Snake River to the Lewiston-Clarkston area. The lake has a surface area of approximately 6,590 acres.

The dam is a concrete, gravity-type with an earth- and rockfill right embankment; a navigation lock with clear dimensions of 86 ft by 675 ft; an 8-bay spillway dam 512 ft long; a 656-ft-long 6-unit powerhouse that has a total generating capacity of 810,000 kw; and a fish-passage facility for migratory fish located on the left abutment.

The first stage construction of the project began in July 1965 with the construction of the downstream guidewall for the navigation lock and a combination cellular sheet pile and earthfill embankment cofferdam for river diversion. The project was delayed for 5 yr due to lack of congressional funding, but the main dam construction resumed in May 1970. The project was completed in November 1975. The final three generating units were installed under a separate contract, and all units were producing power in May 1978.

A major feature of the Lower Granite project, and constructed concurrently with the main dam, was 8 mi of levees to protect Lewiston, Idaho, from the backwater of the reservoir. Ground-water levels landward of the levees are controlled by drainage ditches and three pumping plants (U.S. Army Corps of Engineers, 1982a, 1984).

SITE GEOLOGY

At Lower Granite Dam, the Snake River has cut a canyon approximately 1,600 ft deep and 3,700 ft wide. The surface geology in the immediate vicinity of the dam is shown on Figure 2. A geologic section along the centerline of the dam is portrayed on Figure 3.

All concrete sections of the dam and the north embankment dam are constructed on basalt flows of the lower part of the Grande Ronde Basalt. The flows dip less than 1° west and northwest and contain several intervening flow contact breccia layers. The canyon walls above the dam are composed of upper flows of the Grande Ronde Basalt. Locally, caps of the Roza Member of the Wanapum Basalt overlie the Grande Ronde sequence. The uplands are mantled with loess deposits of varied thickness. Several feeder dikes for the Grande Ronde flows are present near the dam. One such dike, 80 ft wide and nearly vertical, constitutes the left abutment of the dam (Kienle, 1980; U.S. Army Corps of Engineers, 1982b).

The upstream and downstream shells of the embankment dam are founded partly on terrace gravels and Recent alluvium. The core and filter zones of the embankment dam are placed on a hard, dense basalt flow that is nearly 100 ft thick. This same flow extends across the entire dam site area and forms the foundation for most structures. The only area in which the excavation penetrates this flow is in the deepest part of the powerhouse draft tube excavation. A thin flow breccia layer forms a part of the foundation on each abutment; it is low in the stratigraphic section. Northwest-trending shear zones transect the foundation area of the navigation lock; dips vary from 30° northeast and southwest to vertical. These shear zones may have been created at the same time as the intrusion of the basalt dike that crosses the foundation at the left abutment tie-in and continues northwest to cross the river approximately 1 mi downstream of the dam.

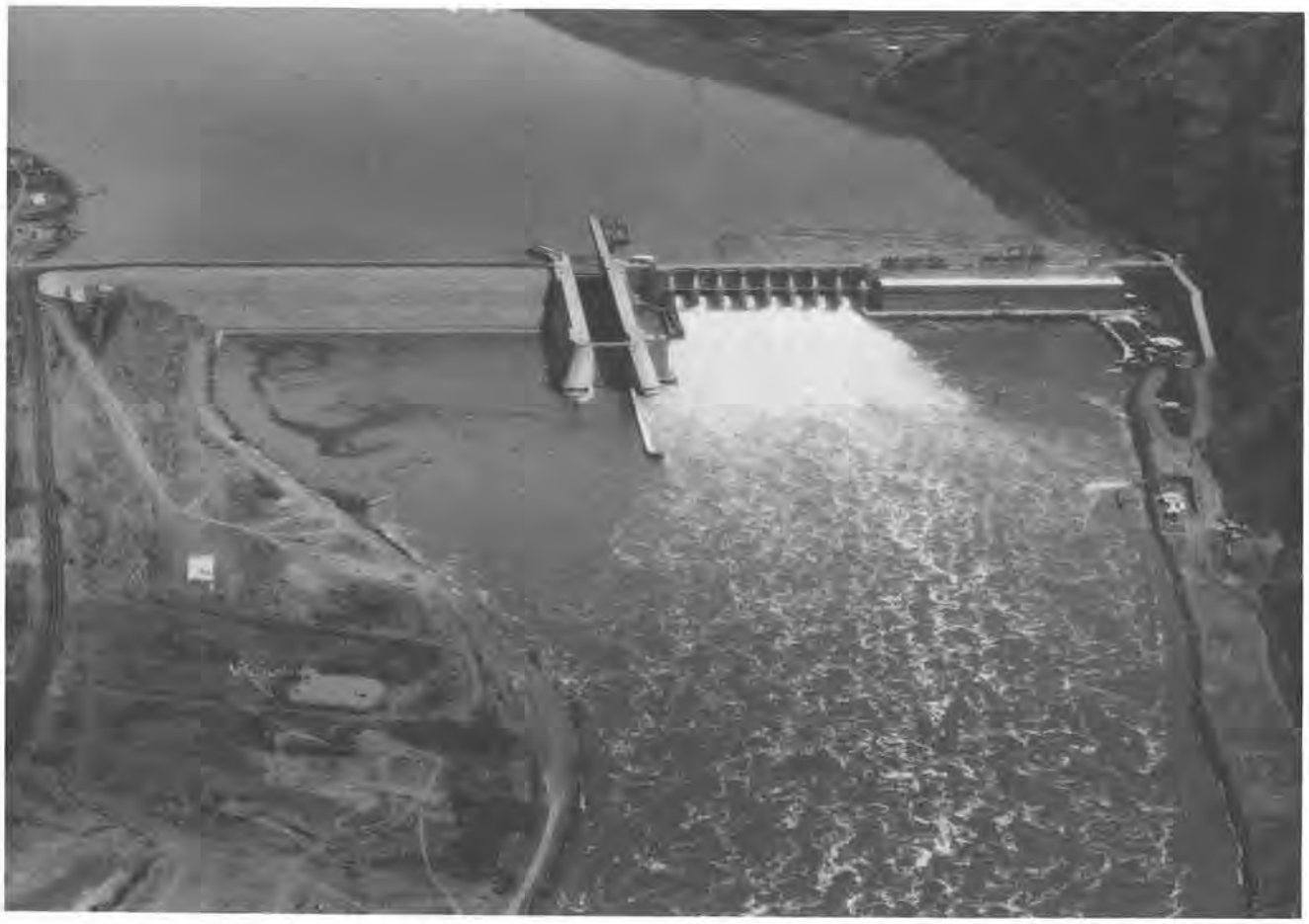


Figure 1. Lower Granite Dam; view upstream (southeast). Components of the project, from left to right, are the right (north) embankment dam, navigation lock, spillway dam, powerhouse, and fish-ladder facilities with overhead south shore access to top of dam. Photograph by U.S. Army Corps of Engineers.

GEOLOGIC ASPECTS OF SITING

Lower Granite Dam is the last of a four-dam plan of development for the lower Snake River that would provide for slackwater navigation to the Lewiston, Idaho, area. Five sites were considered in the reach of the Snake River from river mile 105 to 113.7. The selected site at river mile 107.5 was concentrated upon for detailed investigations. Eventually, this site was deemed the most economical to develop in terms of powerhead, navigation conditions, geology, and site layout. Foundation conditions were judged to be excellent, on the basis of exploratory investigations to that stage of the design (U.S. Army Corps of Engineers, 1961).

CONSTRUCTION PROBLEMS

The dam was constructed with relatively few foundation problems. Investigations conducted during design phases of the project identified nearly all areas of the foundation that might offer difficulties. The design was somewhat more conservative than for the previous three lower Snake River projects downstream in that rock cuts were placed on flatter slopes and attempts were made to avoid sharp breaks in the excavation that could lead to cracking of the concrete at a later time. Specific grade lines were established for each structure. The only concern was the irregularity of a flow-top breccia, which for the most part had been eroded by the action of the river. In two areas, one along the south shore near the erection bay for the powerhouse and the other in an area in the navigation lock, lobes of the flow breccia extended deep into the otherwise uniform basalt flow. Additional excavation was required in these areas.

All foundation excavation for the concrete structures was accomplished within the cofferdam that had been constructed 5 yr earlier. The first excavation removed Recent alluvial gravels, which overlay much of the foundation area. Once overburden was cleared, the rock excavation progressed rapidly. Decisions on final grade had to be made promptly, and in those areas requiring

Figure 2. Photogeologic map in vicinity of Lower Granite Dam. Heavy solid lines indicate approximate geologic contacts. Ql, loess; Qaf, recent alluvium; Qfg, glaciofluvial terrace deposits; Tyro, Roza Member, Wanapum Basalt; Tygr, Grande Ronde Basalt (includes lower and upper flows of normal magnetic polarity, N_1 and N_2, respectively, and upper flows of reversed polarity, R_2). Scale: 1 in. = approx. 2,000 ft.

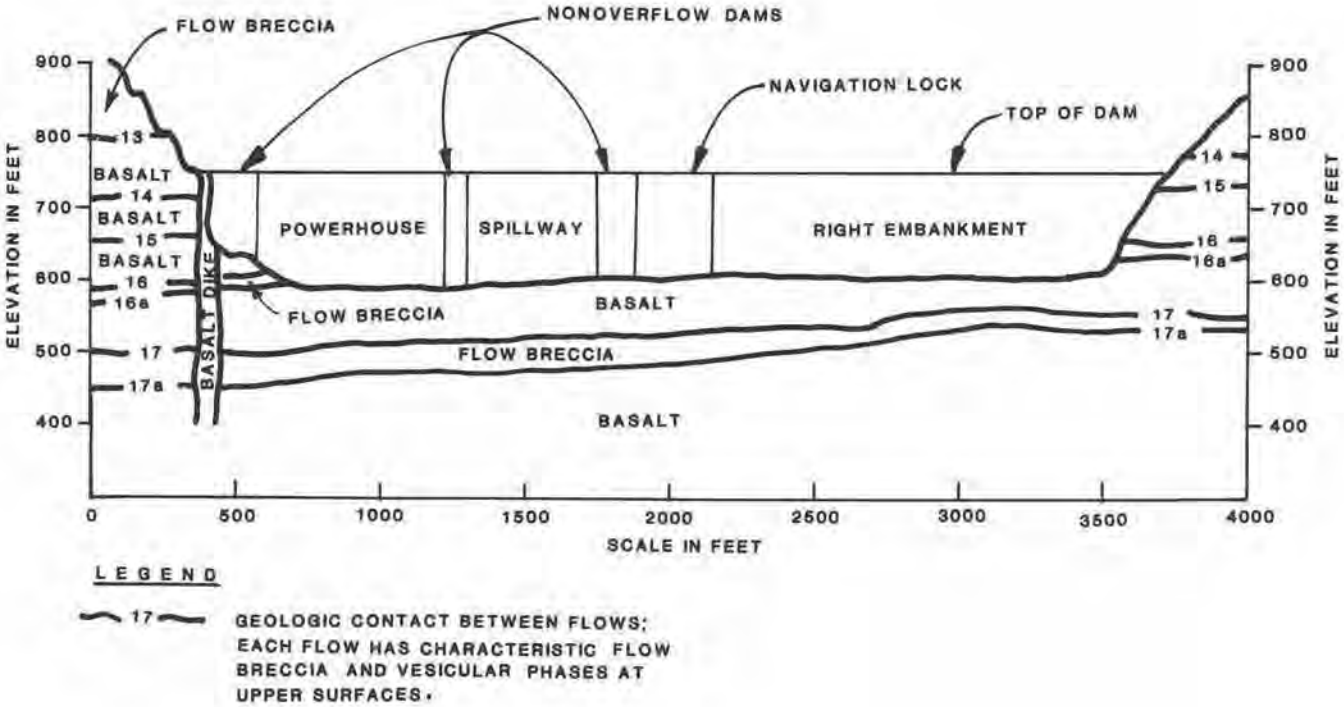


Figure 3. Geologic section at Lower Granite Dam; view downstream.

deeper excavation to reach an adequate foundation, this quick action prevented contract delays and possible changed conditions claims.

In three areas of the foundation it was necessary to remove substantial quantities of rock to achieve a suitable foundation. One area was in the erection bay where the flow breccia-basalt contact was extremely irregular; there, an additional 25 ft of rock had to be excavated. This zone extended northward into unit 1 of the powerhouse intake. Further, in the intake area across the remaining five units, an open, horizontal relief joint required removal of a 6-ft-thick slab of highly vesicular basalt.

A second area requiring substantial overexcavation was beneath the upstream gate monoliths of the navigation lock and extending downstream beneath the left (south) wall monoliths of the lock. Here, two low-angle shear zones intersected beneath the gate monoliths. One shear zone had a northwest strike and a 30° to 35° dip to the northeast. It contained a thin gouge zone but did not appreciably disturb the rock in the footwall or hanging wall. This shear transected two monoliths on the right side of the lock. Remedial treatment consisted of excavating a wedge of rock from the hanging wall until 10 ft of sound rock overlay the fault plane. The second shear zone had a southwest strike and a 35° dip to the southeast. It crossed beneath the left (south) gate monolith and continued downstream along the left wall monoliths and into the spillway right training wall. This zone was thicker and thus required extensive overexcavation.

The third area that required additional excavation was beneath lock monoliths 26 and 28 on the right side of the lock downstream. Here, it was necessary to remove a 5-ft-thick layer of highly vesicular basalt.

Experience showed that the foundation for the navigation lock was of the poorest quality of the entire project. Even though firm foundation was achieved, the rock contains numerous thin shear-like zones of highly weathered and mineralized rock. The attitude of these features is vertical, and all have a general northwest trend. Geologists postulated that these features were related to the disturbance of the nearly horizontal flows by the thick vertical dike that crosses the left abutment of the dam and which has a similar northwest trend.

Ground-water control during the excavation of the project was not a problem. There were localized areas of the excavation in which sump pumps and french drains were required to control the water until concrete was placed, but all were pressure grouted when concrete placement reached a higher level.

The construction of the north embankment dam began when the concrete structures were nearly complete. The river was diverted through the powerhouse, and cofferdams were placed across the previous diversion channel along the north shore to allow excavation and construction to proceed. The design was to excavate

to the rock foundation for the core and filter zones and to place the shells of the dam on the terrace gravels. The foundation rock for the embankment dam is an extension of the primary foundation rock for the concrete structures. It is generally of excellent quality and required only minor placement of dental concrete to fill potholes. No rock excavation was necessary. The only foundation grouting required beneath the embankment section was that accomplished on the right abutment tie-in in 1969 during the relocation of the Union Pacific Railroad. The grout curtain extended only 30 ft perpendicular to the rock face, and its intent was to extend the seepage path farther into the abutment.

The foundation drainage and grout curtains were placed from a gallery along the axis of the dam under separate contract while main dam construction was still progressing. The standard split-spacing, stage grouting methods were employed. The grout curtain extended from the left abutment and continued north to the navigation lock, then passed a short distance downstream along the north side of the lock. The grout curtain was extended through the main foundation basalt and a thick flow breccia (contacts 17 and 17a). The maximum depths of the curtain ranged from nearly 200 ft on the left abutment to between 120 ft and 150 ft below the powerhouse and spillway structures. Ultimate hole spacing was 5 ft on center. A total of 21,000 sacks of cement were placed in 31,000 linear feet of EX-grout holes; the average grout take was 0.68 sacks of cement per linear foot of grout hole. The complementary drain curtain on the downstream side of the gallery was extended to about 80 ft in depth, but precautions were taken not to enter the flow breccia zone because of the potential of encountering large quantities of water from tailwater as had occurred at the Little Goose project, the next dam downstream.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

There have been no significant geology-related problems at the Lower Granite Project since it became operational in 1975. Several cracks have developed in the concrete structures, but all are considered to be temperature-related cracks caused during the cooling of the concrete. Minor compressional changes have been recorded on deformation meters installed in the foundation of the powerhouse and navigation lock, but these are not of the magnitude that would cause any structural problem.

Drainage flow from the foundation drains has been minimal. Flow from the drains and other sources within the gallery peaked at approximately 2,400 gpm but has since declined to about 400 gpm. Uplift beneath the structures has not exceeded tailwater pressure, except at one location in unit 1 powerhouse intake. The instrument located at this point records forebay elevation, but because of its proximity to a contraction joint, the recordings are suspect. None of the several other instruments in the area give abnormal readings.

A geological and seismological review of the project was completed in 1982 to assess the potential for seismic hazards. This study concluded that the controlling earthquake for the site was a magnitude 6.2 event on the Wallula fault zone approximately 60 mi southwest. The attenuated ground accelerations at the site from this event were calculated to be 0.10 g. From this evaluation, no further seismic analyses were considered necessary (U.S. Army Corps of Engineers, 1982b).

REFERENCES

Kienle, C. F., Jr., 1980, *Geologic Reconnaissance of Parts of the Walla Walla and Pullman, Washington, and Pendleton, Oregon 1°X2° AMS Quadrangles*: Prepared for the U.S. Army Corps of Engineers, Seattle District by Foundations Sciences, Inc., Portland, OR, 66 p., 3 plates.

U.S. Army Corps of Engineers, 1961, *Site Selection and Upper Pool Determination, Design Memorandum No. 1, Little Goose Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA., 67 p., 27 plates.

U.S. Army Corps of Engineers, 1982a, *Project and Index Maps*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA., 52 p., 41 plates.

U.S. Army Corps of Engineers, 1982b, *Geological and Seismological Review, Lower Snake River Dams, McNary and Mill Creek*: Prepared for U.S. Army Corps of Engineers, Walla Walla District by U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 44 p., 3 plates.

U.S. Army Corps of Engineers, 1984, *Construction History Report, Lower Granite Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA., 91 p., 5 plates.

Lower Granite Dam. A general view of the foundation excavation and partially completed concrete structures contained within the first-step cofferdam on the left (south) shore. The Snake River is being diverted through the area of the future embankment dam. Photograph from U. S. Army Corps of Engineers, May 1971.

Dams of the Yakima Basin Irrigation Project

Introduction and Geologic Setting

Bumping Lake Dam

Cle Elum Dam

Clear Creek Dam

Easton Diversion Dam

Kachess Dam

Keechelus Dam

Roza Diversion Dam

Tieton Dam

Early construction of the Yakima Basin Irrigation Project at Keechelus Dam; view to the north toward the left abutment. The gravel fill shown here is nearly completed in the maximum section. Topping out of the earthfill and riprap is in progress; the dinkey train is using dump cars. Photograph by U.S. Bureau of Reclamation, July 24, 1916.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Dams of the Yakima Basin Irrigation Project: Introduction and Geologic Setting

BRENT H. CARTER
U.S. Bureau of Reclamation

INTRODUCTION

The Yakima Project of the U.S. Bureau of Reclamation (USBR) is coincident with the Yakima River basin of south-central Washington (Figure 1). It is the largest single river system entirely within the confines of the state. The waters are the most extensively utilized of all the rivers in Washington (Kinnison and Sceva, 1963).

The Yakima valley has a mild climate and rich soil, but rainfall is just a little more than 7 in./yr. A sustained water supply was needed to develop the area, and in 1903, citizens from Yakima County petitioned the Secretary of the Interior for assistance. The Reclamation Service (now the Bureau of Reclamation), created by Congress in 1902, was given the job of designing and building the project. The irrigation system was based on storage reservoirs that would catch and hold spring runoff, assuring delivery of adequate water supplies during the entire growing season (USBR, 1979).

Project facilities include 6 storage reservoirs (with a total capacity of 1.1 million acre-ft), 5 diversion dams, 2 power plants, 30 pumping plants, 416 mi of main canals, 1,702 mi of distribution canals, and 144 mi of drains. Construction began in 1906, and the first irrigation water was delivered the following year.

With a little water, almost anything will grow in the bottom lands of the Yakima valley. The project is a physical as well as an economic giant. Irrigation facilities serve 465,000 irrigable acres (Figure 1) (USBR, 1981). In recent years, the project has generated well over $200 million per year in crops.

GEOLOGIC SETTING

The diverse geologic terrain of the Yakima River basin can be divided into the Cascade Range highlands to the west and the valley lowlands to the east. The core rocks of the ancestral Cascade Range area are a structurally complicated assemblage of pre-Tertiary volcanic, metamorphic, mafic, ultramafic, and sedimentary rocks. Superimposed on the older rocks is a wide variety of Tertiary to Holocene volcanic rocks with some igneous intrusive rocks and marine and continental sediments (Figure 2).

Miocene Columbia River basalt underlies the eastern portion of the project area. Numerous catastrophic floods during the Pleistocene inundated the lowlands of the basin, modifying the topography and depositing related fine to coarse unconsolidated materials.

Recurring cycles of Pleistocene alpine glaciation occurred in the Cascades. The rugged topography was modified, and many valleys were widened and deepened. Large quantities of glacial drift were deposited in the form of moraines, outwash, and lacustrine materials.

Most of the structures in the Yakima basin trend northwest, the result of north-south regional compression that has acted on this area over Cenozoic time. The project area is characterized by a moderate to low rate of historical seismicity (Geomatrix Consultants, Inc., 1987).

The geologic setting of the area had a significant role in the siting of the storage dams. Natural lakes existed behind moraines at the Bumping Lake, Cle Elum, Kachess, and Keechelus sites. Glacial drift and outwash deposits that range from impervious to pervious were effectively utilized in construction of the dams to increase the water storage capacity of the natural dams. At the Tieton site, the channel section and right abutment foundation are in glacial materials, while the left abutment foundation is on rock. Hard rock exposed in narrow channels and canyons forms the foundations of most of the other storage and diversion structures.

Engineering geology considerations are discussed for each of the storage reservoirs (Bumping Lake, Cle Elum, Clear Lake, Kachess, Keechelus, and Tieton) and for the diversion dams (Easton and Roza) (Table 1).

REFERENCES

Geomatrix Consultants, Inc., 1987, *Seismotectonic Evaluation of the Northern Cascade Mountains Geomorphic Province for U.S. Bureau of Reclamation Dams*: Geomatrix Consultants, Inc., U.S. Bureau of Reclamation, Denver, CO, pp. 51-54.

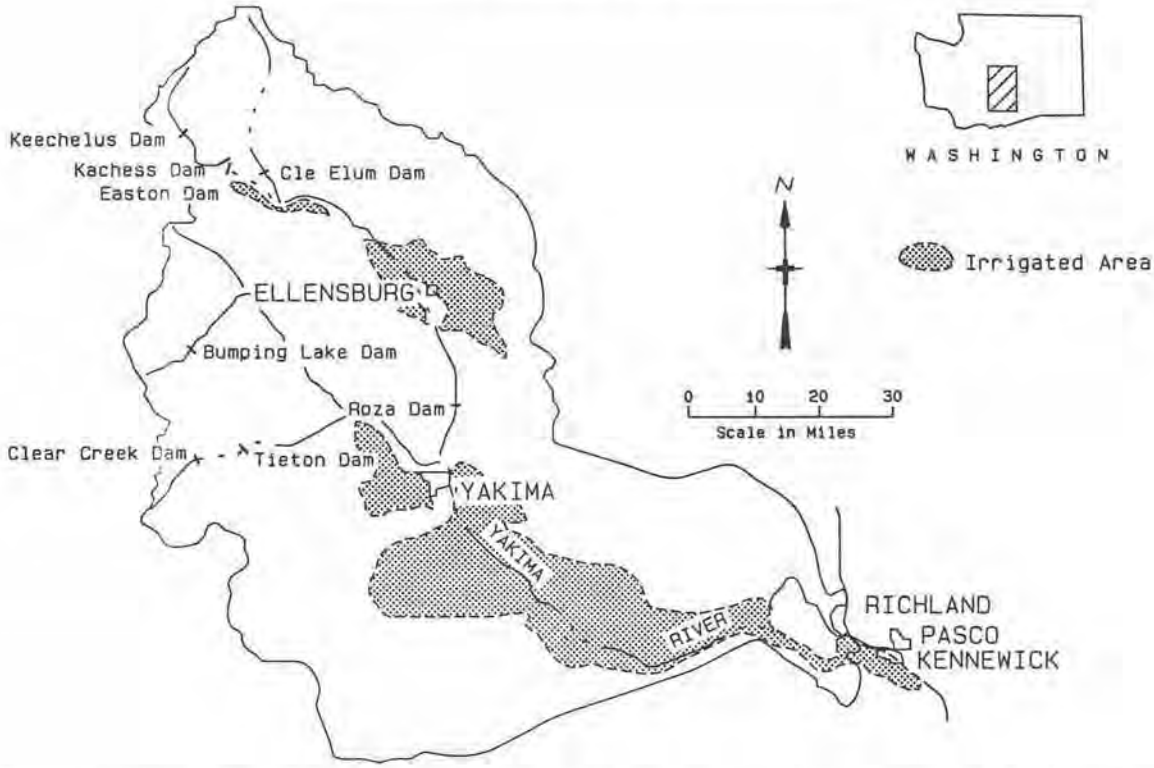


Figure 1. Location of Bureau of Reclamation dams and irrigated areas within the Yakima River basin, Washington.

Table 1. U.S. Bureau of Reclamation storage dams, Yakima Basin Irrigation Project

Dam	River	Type	Year completed	Structure height (ft)	Crest length (ft)	Maximum reservoir capacity (acre-ft x 1,000)	Foundation
Bumping Lake	Bumping River	Embankment	1910	61	2,925	33.7	glacial drift/outwash
Cle Elum	Cle Elum	Embankment	1933	165	1,801	437	glacial drift/outwash
Clear Creek	Clear Creek	Arch	1914	84	404	5.3	metavolcanic rocks
Easton Diversion	Yakima	Concrete Gravity	1929	66	248	2.5	schist
Kachess	Kachess	Embankment	1912	115	1,400	245	glacial drift/outwash
Keechelus	Yakima	Embankment	1917	128	6,550	171	glacial drift/outwash
Roza Diversion	Yakima	Concrete Gravity	1939	67	486	-	basalt
Tieton	Tieton	Embankment	1925	319	920	213.6	"microdiorite" (andesite), glacial drift/outwash

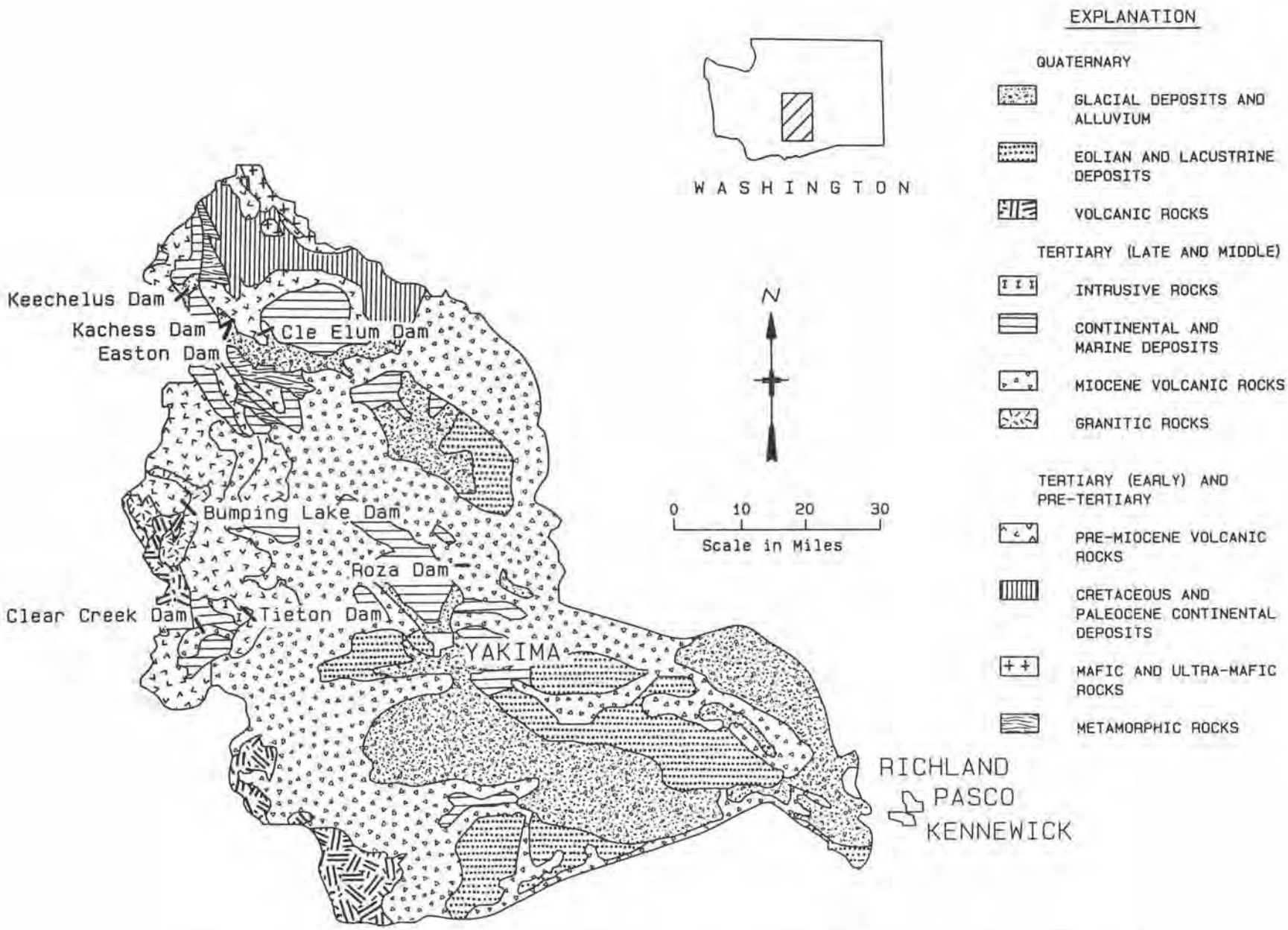


Figure 2. Generalized geologic map of the Yakima River basin, Yakima Project, Washington. Adapted from Weissenborn, 1969).

Kinnison, H. B. and Sceva, J. E., 1963, *Effects of Hydraulic and Geologic Factors on Stream Flow of the Yakima River Basin, Washington*: U.S. Geological Survey Water-Supply Paper 1595, 134 p.

U.S. Bureau of Reclamation, 1979, *Yakima Water Resource Project, Washington* (Booklet): U.S. Government Printing Office, pp. 1-16.

U.S. Bureau of Reclamation (Water and Power Resources Service), 1981, *Project Data*: U.S. Government Printing Office, pp. 1337-1356.

Weissenborn, A. E., 1969, Geologic Map of Washington: U.S. Geological Survey Miscellaneous Geological Investigations, Map I-583, 1 sheet., scale 1:2,000,000.

Typical panoramic view of Yakima Valley farm land that is supplied with irrigation water from the project facilities. Irrigation conveyances serve 465,000 irrigable acres. Crops include mint, apples, peaches, pears, hops, hay, and grains. U.S. Bureau of Reclamation photograph, June 22, 1978.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Bumping Lake Dam

J BRAD BUEHLER
U.S. Bureau of Reclamation

PROJECT DESCRIPTION

Bumping Lake Dam, constructed during 1909 and 1910, is located on the Bumping River approximately 43 mi northwest of Yakima and 12 mi east of Mount Rainier. Although the structure's primary purpose is to provide water storage for irrigation, it also supplies water for industrial, municipal, and recreational activities. The dam's embankment contains 253,000 cy of glacially derived earthfill materials. It has a crest length of 2,925 ft at an elevation of 3,435 ft, a maximum base width of 275 ft, and a top width of 20 ft (Figure 1). It has hydraulic and structural heights of 38 and 61 ft, respectively (U.S. Bureau of Reclamation [USBR], 1981). The dam was built at the northeast end of an existing glacial trough lake near its natural outlet. The lake level was raised approximately 38 ft after construction of the dam.

The embankment was constructed using hydraulic methods that were state-of-the-art during the early 1900s. It is composed of four zones with a riprap cover on the upstream face (Figure 2). These zones contain materials of differing size ranges and were emplaced by several methods. All of the embankment materials were borrowed from glacial drift deposits near the dam site. A 15-ft-deep cut-off trench, excavated just upstream of the dam's centerline, was filled with puddled (zone a) materials by dumping processed glacial drift on the trench edges and sluicing the fines into the trench. Embankment zone a, composed of silt and very fine sand, forms the impervious diaphragm core. Embankment zones b and c are composed of coarser materials remaining after the sluicing process. These materials were placed so that they are somewhat coarser from the inside of zone b toward the outside of zone c. Zone b is composed primarily of sand and fine gravel; zone c is chiefly sand with fine to coarse gravel. Zone d is composed of unprocessed glacial drift materials excavated from the borrow pit. All the glacial materials were excavated by hand and were transported by horse-drawn carriages and rail cars (USBR, 1983).

The appurtenant structures consist of a spillway and an outlet works conduit. The spillway is an open, uncontrolled, concrete weir structure with a concrete and wooden chute located on the left abutment of the dam. It has a capacity of 3,400 cfs at a reservoir elevation of 3,429 ft. The outlet works consists of a cut-and-cover concrete conduit through the base of the embankment and is controlled by two 5-ft-square service gates. This 10-ft-diameter outlet has a capacity of 1,500 cfs at a reservoir elevation of 3,426 ft. The reservoir has an active capacity of 33,700 acre-ft near elevation 3,426 ft (USBR, 1981).

Seeking additional storage space, a Bumping Lake Enlargement Program was investigated by the Bureau of Reclamation in 1953. The site was approximately 4,000 ft downstream from the existing dam. Several drill holes at the new site found glacial materials similar to those encountered at the existing structure (USBR, 1953). Construction of a new structure at this site was still under consideration in 1988.

SITE GEOLOGY

General

Mountainous areas adjacent to Bumping Lake Dam and Reservoir are chiefly underlain by extrusive and intrusive igneous rocks with some interbedded sedimentary rocks. The extrusive rocks are mostly andesitic in composition; the intrusive rocks are primarily granitic with some diorite. Once a volcanic and plutonic terrain, the area has since been modified by alpine glaciation creating classic geomorphic expressions that include cirques, tarns, and horns forming many of the mountain peaks and terminal and lateral moraines deposited in long, broad, U-shaped valleys (USBR, 1983).

Bedrock

The bedrock stratigraphy of the Bumping River valley, from oldest unit to youngest, consists of the Eocene Puget Group sandstone, Oligocene Ohanapecosh Formation, Miocene diorite and Bumping Lake granite, and Pleistocene Deep Creek andesite and basalt (Figure 3). Forming the most extensive bedrock unit in the Bumping River valley is the andesite and brecciated andesite of the Ohanapecosh Formation, which overlies the Puget sandstone, chiefly to the north. These volcanics have been intruded by the Bumping Lake pluton. Overlying the Ohanapecosh Formation is another fairly extensive rock unit, the Pleistocene Deep Creek andesite (Abbott, 1953; Clayton, 1983; Simmons et al., 1983).

Figure 1. Aerial view to the southeast of Bumping Lake Dam showing the exposed reservoir bottom during a drought year. The entire valley floor is covered with a thick accumulation of glacial drift, which supports a heavy cover of vegetation. U.S. Bureau of Reclamation photograph, October 9, 1979.

A small basalt flow is present near the upper end of the reservoir. This flow is apparently a distal valley flow from the Tumac shield volcano and is Pleistocene in age. The flow appears to predate the last major glaciation in the area (Clayton, 1983; Simmons et al., 1983; USBR, 1985).

The predominantly volcanic landscape was modified by east-flowing alpine glaciers. The resultant U shape of the Bumping River valley was controlled primarily by the hardness and structure of the various volcanic and plutonic rocks.

Unconsolidated Materials

The unconsolidated materials in the Bumping Lake area consist of glacial drift and alluvium. Throughout the Pleistocene and into the Holocene, glacial advances originating from the crest of the Cascade Range carved the deep, U-shaped Bumping River valley, which was partially infilled with drift. The glacial events that took place within the Bumping River valley and their resultant drift deposits have not been differentiated, although Hammond (1980) has correlated these deposits with Evans Creek alpine glaciation. The glacial advances probably began approximately 2 Ma, with the last major ice retreat occurring about 10,000 yr ago (USBR, 1985). The surficial geology of the Bumping Lake Dam area is shown on Figure 3 (Abbott, 1953; Simmons et al., 1983).

Bumping Lake Dam is founded on the drift deposited during the last major ice retreat (Figure 4). The drift is a heterogeneous mixture of gravel, sand, and silt with some cobbles and boulders. The drift in the area of the

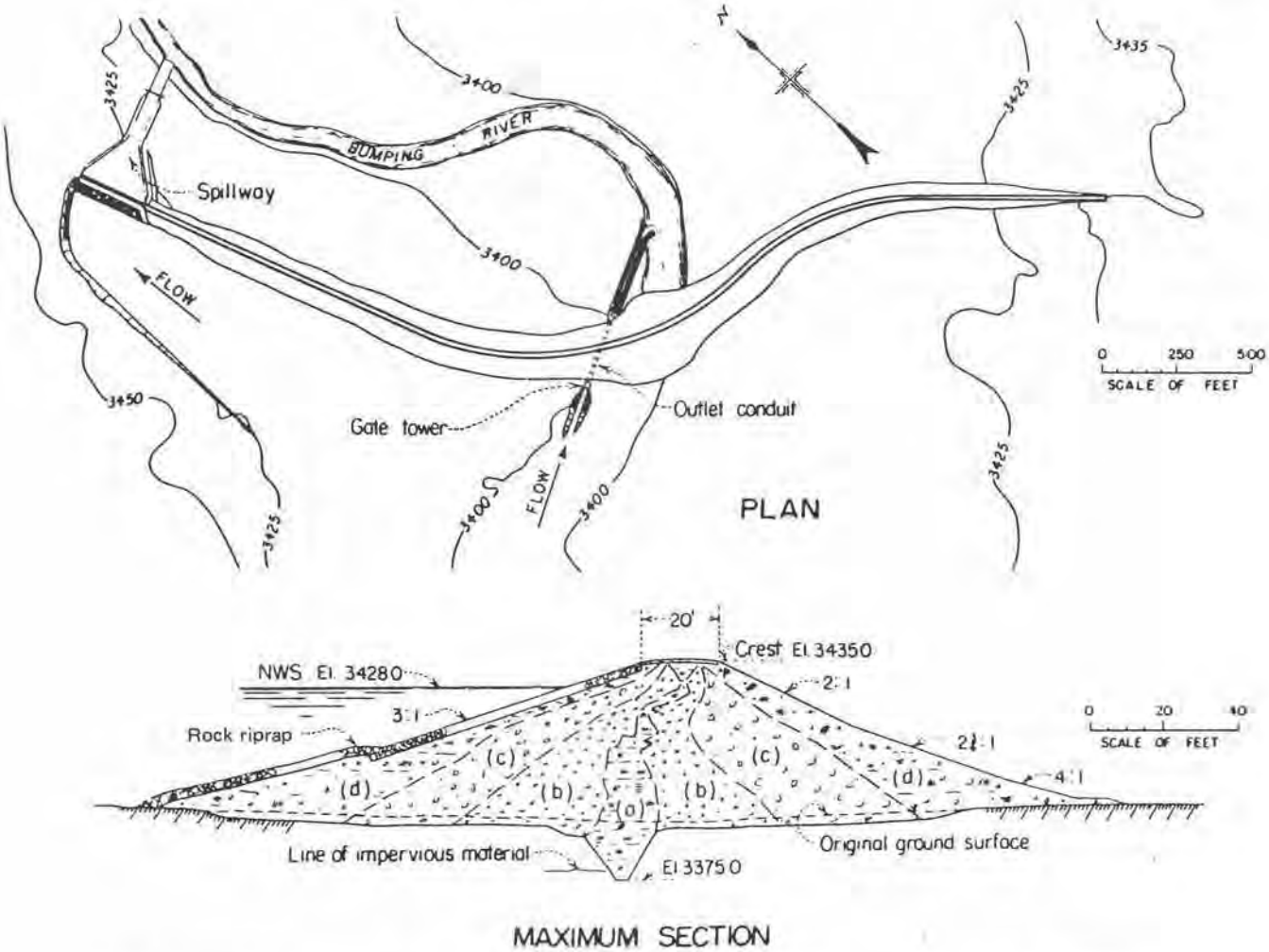


Figure 2. Plan and cross-section, Bumping Lake Dam. Adapted from U.S. Bureau of Reclamation (1981).

dam's foundation is believed to be approximately 200 ft thick near the center of the valley (Figure 4), and it pinches out abruptly at the edges (USBR, 1953). The ancestral Bumping River and the meltwaters of retreating glaciers reworked some of the drift, leaving lenses of stratified silt, sand, and gravel (USBR, 1983).

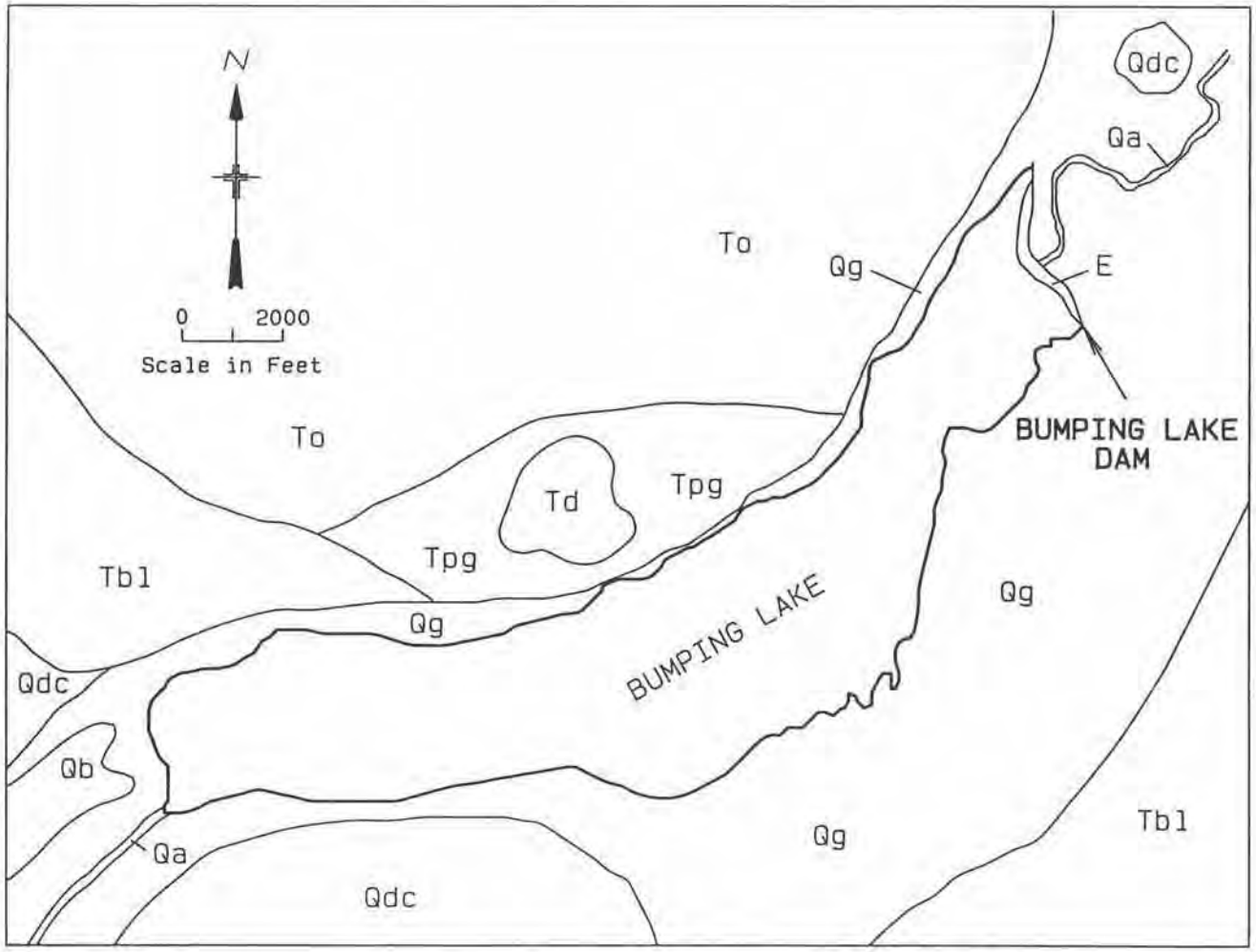
GEOLOGIC ASPECTS OF SITING AND DESIGN

Siting

Originally, two dam sites were investigated by a series of test pits and drill holes. One potential site was located at the northeast end of the natural Bumping Lake, where a large terminal moraine formerly impounded a larger glacial lake. Another site was located several thousand feet downstream, where bedrock crops out along the canyon walls. Preconstruction investigations included the excavation of 53 test pits and the drilling of 34 shallow borings. Investigations at both sites encountered extensive glacial drift deposits exhibiting a wide range of grain and clast sizes. Since a terminal moraine formed a sound natural barrier impounding the lake and a large amount of glacially derived construction material was readily available there, the location at the lake outlet was selected as the construction site.

Seepage

Seepage conditions were the primary concern during the siting and design of Bumping Lake Dam. This was due to the lack of clay in the glacial drift foundation and construction material. It was determined that the silt and very fine sand within the puddled core diaphragm would form an adequate, semi-impervious barrier within the structure and thus retard embankment seepage. The cutoff trench would control the foundation seepage to a certain depth; other than that, no further attempt was made to impede the deeper seepage (USBR, 1910).



EXPLANATION

E	Embankment
Qa	Alluvium
Qg	Glacial drift
Qb	Basalt
Qdc	andesite of Deep Creek
Tbl	Bumping Lake pluton
Td	Diorite
To	Ohanapecosh Fm
Tpg	Puget Group

Figure 3. Geology of Bumping Lake Dam area. After Abbott (1953).

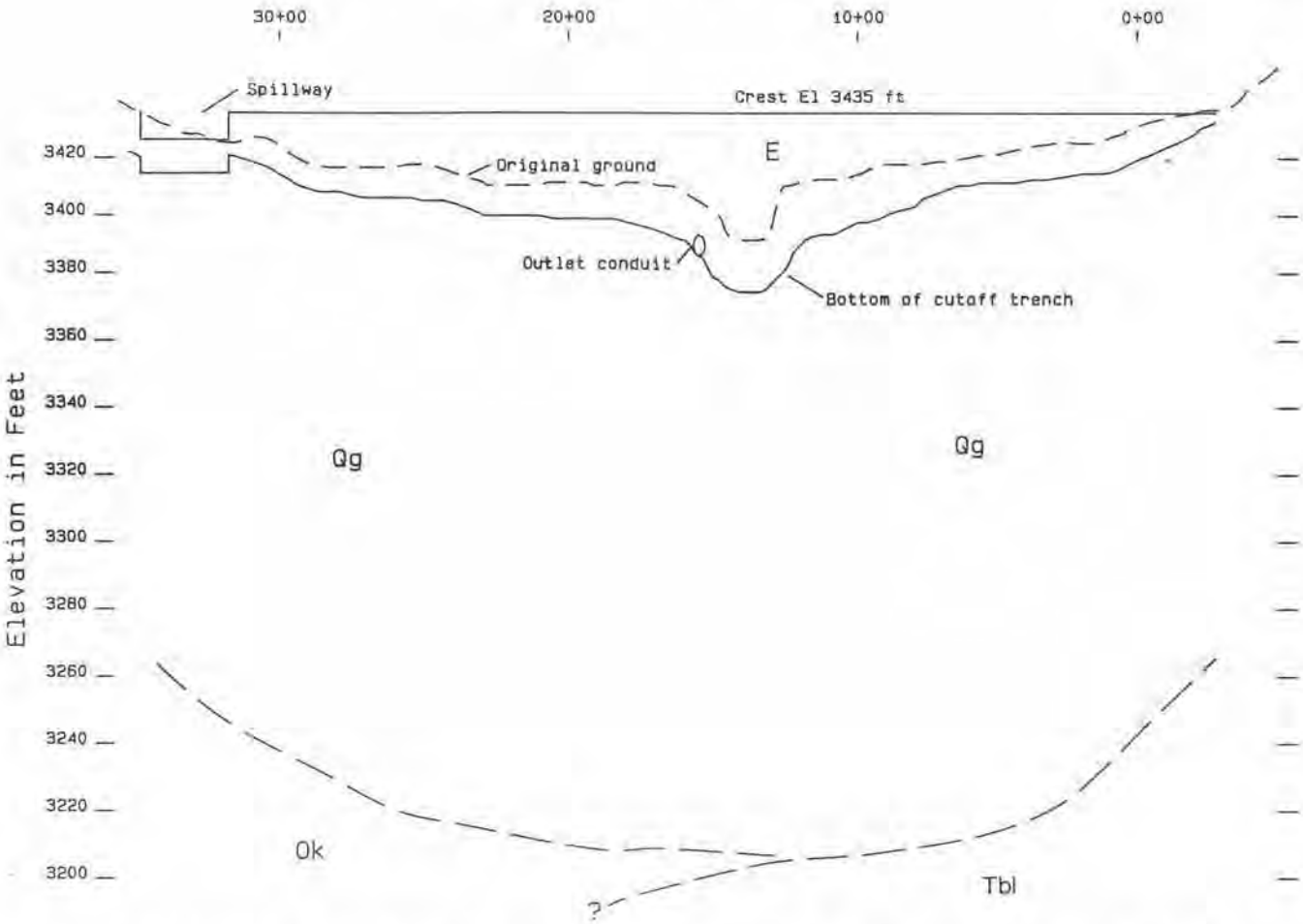


Figure 4. Longitudinal section of Bumping Lake Dam; view downstream. Longitudinal section adapted from unpublished U.S. Bureau of Reclamation sources. E, embankment, Qg, glacial drift; Ok, Ohanapecosh Formation; Tbl, Bumping Lake granite.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Reservoir Seepage

As anticipated, seepage near the downstream toe of the dam occurred during the initial filling and in each subsequent year. The seepage, totaling 2 cfs, was channeled into a single ditch and conducted back to the river. These seepage quantities were monitored by seven weirs located in various areas along the downstream toe of the structure. Presently, the weirs monitor reduced seepage of about 1 cfs (USBR, 1984).

During 1911 and 1912, six 2-in. well points were placed in the embankment near stations 14+00 and 23+00 (Figure 4). The instruments were positioned at depths ranging from 15 to 60 ft to determine whether the seepage was passing through the embankment or the foundation. Data indicated that the downstream seepage was passing through the foundation materials and was not jeopardizing the stability of the embankment. The instruments became inoperable at some time around 1919 (USBR, 1984).

Because monitoring with the well points was no longer possible, a Safety Evaluation of Existing Dams instrumentation program was conducted at Bumping Lake Dam during 1981. The program called for installation of piezometers to monitor the pore pressure within the embankment and foundation. Five single porous-stone piezometers were installed. One instrument monitors the embankment; the remaining four were placed within the foundation. The highest pore-pressure elevation monitored within the embankment during the 1982 season was on June 22. The phreatic water surface at this location was at about elevation 3,417 ft, approximately 18 ft below the crest of the embankment (elevation 3,435 ft) and about 9 ft below the reservoir water surface on that date (elevation 3,426 ft). This investigation concluded that the phreatic water surface within the embankment and the pore pressures within the foundation were adequately dissipated. En-

gineers attribute the current downstream seepage conditions to a permeable foundation rather than to an embankment seepage problem.

Assuming a consistent water surface within the entire embankment, the dam is stable under static conditions (USBR, 1983). A seismotectonic study currently under way will provide input in determining the structure's stability under dynamic conditions.

REFERENCES

Abbott, A. T., 1953, *The Geology of the Northwest Portion of the Mount Aix Quadrangle, Washington* [Ph.D. thesis]: University of Washington, Seattle, WA, 256 p.

Clayton, G. A., 1983, *Geology of the White Pass area, South-Central Cascade Range, Washington* [M.S. thesis]: University of Washington, Seattle, WA, 212 p., 1 plate

Hammond, P. E., 1980, *Reconnaissance Geologic Map and Cross Sections of Southern Washington Cascade Range, latitude 45 degrees 30 minutes-47 degrees 15 minutes N., longitude 120 degrees 45 minutes-122 degrees 22.5 minutes W.*: Portland State University Department of Earth Sciences, 31 p., 2 plates.

Simmons, G. C.; Van Noy, R. M.; and Zilka, N. T., 1983, *Mineral Resources of the Cougar Lakes–Mount Aix Study Area, Yakima and Lewis Counties, Washington*: U.S. Geological Survey Bulletin 1504, 81 p.

U.S. Bureau of Reclamation, 1910, *Yakima Project History, Washington Division*, Vol. 5, Part 1: U.S. Bureau of Reclamation, Yakima, WA, 87 p.

U.S. Bureau of Reclamation, 1953, *Preliminary Geologic Report on the Bumping Lake Enlargement Damsite, Yakima Project, Washington*: U.S. Bureau of Reclamation, Boise, ID, 12 p.

U.S. Bureau of Reclamation (Water and Power Resources Service), 1981, *Project Data*: U.S. Government Printing Office, pp. 1337-1356.

U.S. Bureau of Reclamation, 1983, *Safety Evaluation of Existing Dams, Geologic Report, Bumping Lake Dam, Yakima Project, Washington*: U.S. Bureau of Reclamation, Boise, ID, 20 p.

U.S. Bureau of Reclamation, 1984, *Structural Behavior Report, Bumping Lake Dam, Yakima Project, Washington*: U.S. Bureau of Reclamation, Denver, CO, 10 p.

U.S. Bureau of Reclamation, 1985, *Preliminary Seismotectonic Study for Bumping Lake Dam, Yakima Project, Washington*: U.S. Bureau of Reclamation, Denver, CO, 11 p.

Aerial view to the west of Bumping Lake. The crest of the Cascade Range lies between the lake in the foreground and Mount Rainier in the distance. U.S. Bureau of Reclamation photograph, August 31, 1975.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Cle Elum Dam

RICHARD A. LINK
U.S. Bureau of Reclamation

PROJECT DESCRIPTION

Cle Elum Dam (Figure 1) is located on the Cle Elum River about 8 mi northwest of the town of Cle Elum on the eastern flank of the northern Cascade Range. The dam was constructed between 1931 and 1933 and was one of the first dams to be built by the U.S. Bureau of Reclamation (USBR) using modern construction control methods (USBR, 1983). The dam creates a storage reservoir with an active capacity of 437,000 acre-ft to supply irrigation water for the Yakima valley. It also provides for flood control, fish and wildlife enhancement, and recreation.

Cle Elum Dam is a zoned earthen embankment with a large, impervious central core flanked on the downstream side by a thick gravel and cobble section (Figures 2 and 3). Extensive gravel fills blanket both the upstream and downstream toes of the structure. The dam has a structural height of 165 ft and a crest length of 1,801 ft, which includes a main dike extension of the embankment and three smaller saddle dikes. The total volume of materials placed in the dam is 1,411,000 cy.

A concrete-lined spillway channel excavated into the right abutment is controlled by five 37-ft x 17-ft radial gates with a top elevation of 2,240 ft (USBR, 1981). A 14-ft-diameter outlet tunnel excavated through the right abutment has a length of about 1,663 ft and is controlled by two 5-ft x 6.5-ft slide gates at the base of a gate shaft located on the crest of the dam.

Continuing problems involving operation of the cylinder gates resulted from surging and pulsations during high flow, which destroyed gate parts and steel liners and caused cavitation of the gate portals (USBR, 1983). The original cylinder gates were replaced with the present slide gates during a rehabilitation of the structure in 1979 and 1980. The gooseneck section of the outlet works tunnel was also replaced with a straight section constructed on the same grade as the rest of the tunnel.

SITE GEOLOGY

General

The valley of the Cle Elum River drains a large basin on the east flank of the northern Cascades that is underlain by a sequence of intrusive and strongly deformed metamorphic rocks of probable Late Jurassic to Cretaceous age. Unconformably overlying the older rocks is a thick sequence of Eocene sedimentary and volcanic rocks, which crop out along the reservoir. Successive multiple advances of alpine glaciers during the Pleistocene have formed the U-shaped valley in which the dam and reservoir are sited. The youngest of several terminal moraine/outwash complexes deposited during the most recent Pleistocene glacial advance essentially blocked drainage of the valley. Following deglaciation, a lake formed behind this natural dam. The natural dam was subsequently breached, and a deep channel was incised through moraine and outwash deposits, forming the modern outlet of Lake Cle Elum.

The modern dam, main dike, and three smaller saddle dikes were sited along the crest of the youngest moraine. A rock-filled, timber crib dam was constructed across the channel outlet in 1905 and was later replaced by the present structure in 1931 to 1933. The modern reservoir surface very closely approximates the highstand water level of the glacial Lake Cle Elum.

Bedrock

Exposures of phyllite, greenschist, and blue amphibole schist crop out west and north of the reservoir along Kachess Ridge (Figure 4). These rocks are part of the Easton Schist and form the base of the stratigraphic section in the Cle Elum Reservoir area. The Easton Schist has a probable age of Late Jurassic and has subsequently undergone metamorphism and strong deformation which continued into the Early Cretaceous (Frizzell et al., 1984).

Resting unconformably upon the Easton Schist is a sequence of Eocene sedimentary and volcanic rocks (Tabor et al., 1984; Frizzell et al., 1984) that form the floor of the reservoir and probably underlie the dam at depth. The oldest of these units is the Swauk Formation, which is composed of fluvial feldspathic sandstone interbedded with conglomerate, carbonaceous siltstone, and shale. Included within the Swauk Formation are intercalated volcanic rocks of dacitic to andesitic compositions belonging to the Silver Pass Volcanics. The tightly folded rocks of the Swauk Formation and Silver

Figure 1. Aerial view to the northwest (upstream) of Cle Elum Dam. The dam is sited on the upstream edge of a large, tree-covered outwash terrace known as Domerie Flats, in the foreground. Outcrops of silty, stony till occur along the rim of the terrace and margin of the reservoir; they represent remnants of the terminal moraine marking the last advance of the Pleistocene glaciers in the Cle Elum River valley. U.S. Bureau of Reclamation photograph, October 5, 1979.

Pass Volcanics crop out along the upper reaches of the reservoir.

The Teanaway Formation consists of gently dipping basaltic to andesitic tuffs and flows that unconformably overlie the Swauk Formation. The Teanaway Formation crops out as a wide belt of exposures bordering the central portion of Lake Cle Elum.

A sequence of dominantly fluvial feldspathic sandstone known as the Roslyn Formation conformably overlies the Teanaway Formation. Interbedded within the sandstone are conglomerate and minor siltstone layers. Coal beds are extensive in the upper portion of the Roslyn Formation. The Roslyn Formation crops out near the lower end of Lake Cle Elum along either side of the valley.

No bedrock exposures are known in the immediate vicinity of Cle Elum Dam. Explorations conducted at the site have failed to penetrate through the thick sequence of glacial and alluvial deposits forming the foundation of the dam.

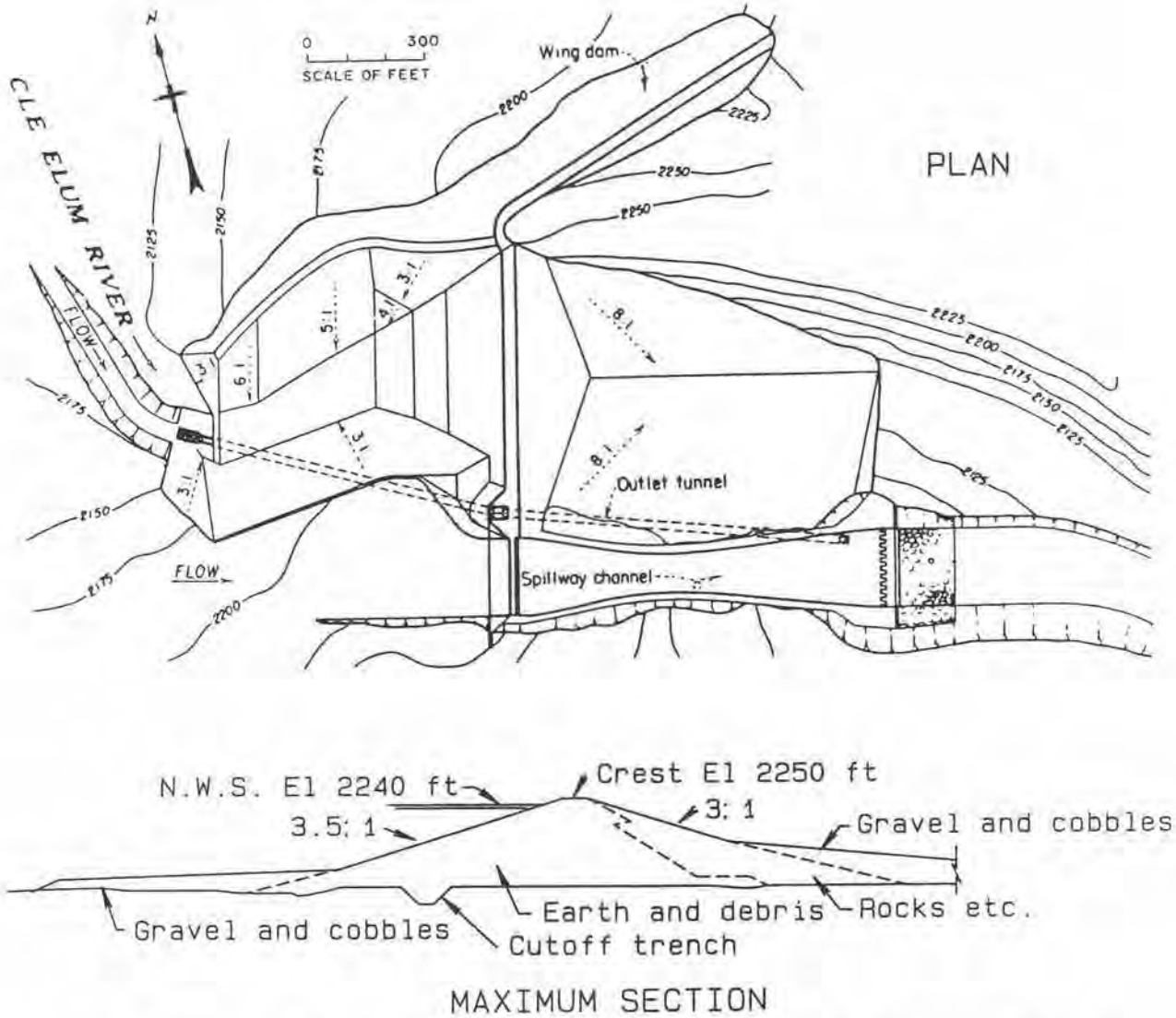


Figure 2. Plan (a) and section (b) of Cle Elum Dam. Adapted from U.S. Bureau of Recamation (1981).

Lakedale Drift

Porter (1976) discusses the Pleistocene glaciation of the upper Yakima River valley and describes three major advances of alpine glaciers down the Yakima and its tributaries. The youngest of these advances, the Lakedale, is coeval with the maximum advance of the Cordilleran ice sheet's Puget lobe during Fraser glaciation (Vashon advance) about 15,000 to 13,500 yr ago. Lakedale glaciation in the Cle Elum River valley consisted of three successive, but progressively less extensive, ice advances which produced three distinct sets of terminal moraines and corresponding outwash terraces. Cle Elum Dam is located along the crest of the youngest Lakedale moraine complex, which Porter (1976) named the Domerie Member after the extensive outwash terrace, known as Domerie Flats, immediately downstream from the dam.

The bulk of the Lakedale Drift forming the foundation of Cle Elum Dam consists of glacial outwash deposited along the margin and downstream of the Domerie ice front. The outwash forms a large apron below the dam and has a relatively uniform slope downstream to the Yakima River (Bryan, 1927; Ransome, 1930). The materials in the outwash range from large boulders to rock flour but generally consist of pervious, crudely stratified sand and gravel with interbeds of variously compact clay and silt. Till forms the innermost morainal crest and caps the steeply sloping upstream edge of the outwash apron along the southern margin of Lake Cle Elum. Till is absent along and adjacent to the former river channel, suggesting erosion of the till during breaching of the morainal dam and subsequent incising of the outlet channel by the Cle Elum River. The moraine includes considerable deformed

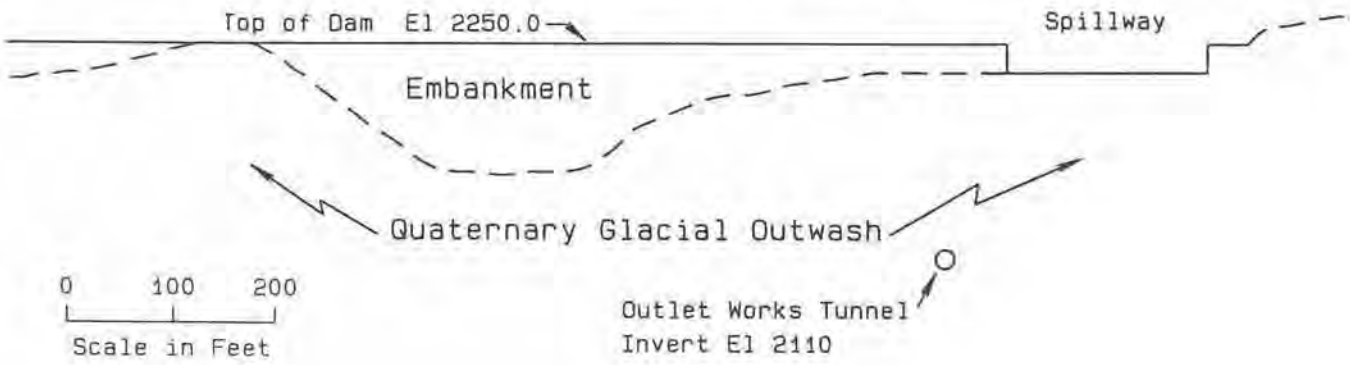


Figure 3. Longitudinal section of Cle Elum Dam; view downstream.

lake sediments scoured from the lake bottom by the last ice advance. Large erratic boulders are present on and within the till and are scattered across the surface of the outwash terrace downstream from the dam. Fine-grained lacustrine sediments ranging in thickness from 25 to 45 ft also blanket the floor of the reservoir upstream from the dam, effectively preventing excessive reservoir leakage through the pervious outwash materials (USBR, 1983). Alluvial deposits consisting primarily of reworked glacial drift line the present river channel.

The thickness of the Lakedale Drift at Cle Elum Dam has not been established; explorations at the dam site as deep as 216 ft have not penetrated into the underlying bedrock units. A water well drilled about 1 mi northeast of the dam intercepted the Roslyn Formation at a depth of 173 ft. A second well located about 7 mi from the dam near the town of Cle Elum was drilled through about 650 ft of unconsolidated materials before encountering bedrock reported to be part of the Roslyn Formation (Kinnison and Sceva, 1963).

GEOLOGIC ASPECTS OF SITING AND DESIGN

The outlet of glacial Lake Cle Elum provided an excellent site for a storage dam as the Cle Elum River had incised a steep-walled channel about 140 ft deep through the Lakedale Drift which impounded the natural lake. The lack of bedrock at a reasonable depth led to the selection of an earth-fill dam, which was sited to take advantage of the local topography and geologic conditions. The dam was sited far enough upstream to place the upstream toe of the dam over the natural impervious blanket lining the lake bottom. The pervious character of the outwash materials present in the foundation of the dam was recognized as a result of numerous explorations at the site, and pump-out tests were conducted to determine permeability of the materials.

Cle Elum Dam was designed with a very conservative cross section with greatly extended slopes that included extensive blankets both upstream and downstream of the structure to lengthen seepage paths. The importance of the naturally occurring impervious blanket on the lake bottom was also recognized and incorporated into the design. Additional impervious material ranging from 5 to 40 ft thick was placed wherever windows of gravel were exposed in the natural blanket in both the river channel and along the abutments. A cut-off trench was excavated through the river alluvium and into an underlying layer of blue clay to a maximum depth of 25 ft; the depth of the trench tapered to 8 ft near the maximum pool elevation.

A further design concern for the embankment was the thickness of the ridge line forming the left abutment of the dam. A reach of the ridge about 850 ft long was strengthened by placement of the main dike embankment. Three smaller saddle dikes were constructed at low points along the crest of the moraine/outwash complex to the north of the dam and main dike.

The design of Cle Elum Dam included a provision to tap the upper 10 ft of the natural lake by means of the outlet works tunnel to provide additional storage. An open approach channel was excavated through the lake sediments upstream of the dam to the trashrack located at the intake of the outlet works tunnel (Figure 2). The river channel downstream from the dam was enlarged and deepened for a distance of about 5,168 ft where the natural grade of the river was intercepted. The spillway was excavated through the right abutment; the bulk of the structure is founded on outwash, although the inlet is constructed in part on till deposits. A drain envelope was installed beneath both the chute section and the reinforced stilling basin at the toe of the slope. The outlet works were constructed through outwash materials through much of the length of the structure; the intake and part of the outlet tunnel are founded on a blue clay exposed underneath the channel alluvium during excavation of the inlet channel.

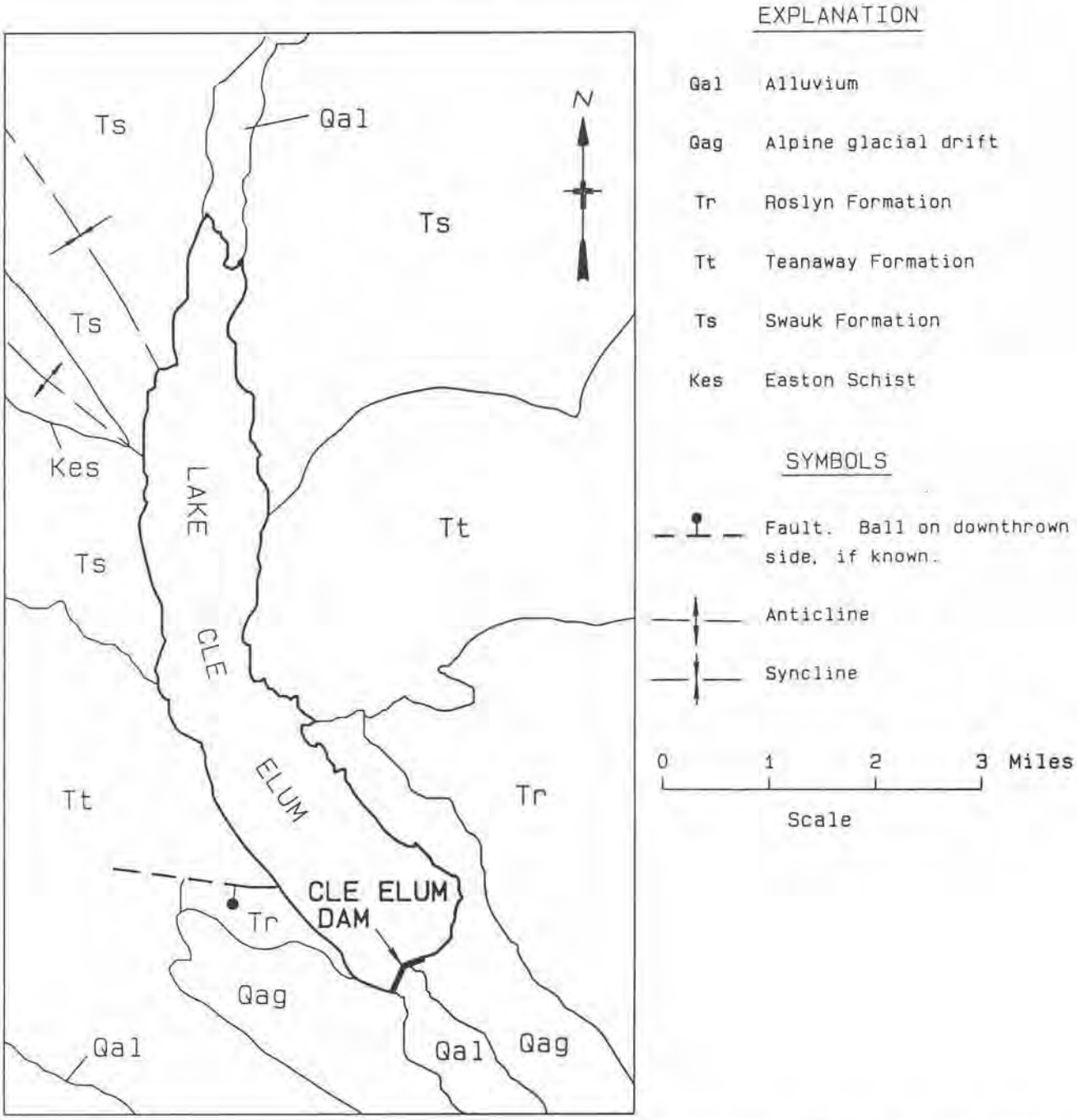


Figure 4. Generalized geologic map of the Cle Elum Dam area. After Frizzell et al. (1984). Kachess Ridge is located about 5 mi west of the reservoir, beyond the area shown in this map.

CONSTRUCTION PROBLEMS

The surface of the foundation was cleared and stripped as a preliminary to foundation treatment (USBR, undated). Following excavation and leveling to grade, shallow bonding trenches ranging from 2 to 3 ft deep were excavated across the floor of the foundation and along the sides of the abutments on 50-ft intervals upstream of the cut-off trench. The materials used in the impervious portion of the dam were borrowed from till deposited on the morainal crest west of the dam, where a series of pits lowered the top of the crest to elevation 2,250 ft, the approximate crest elevation of the dam. The main dike and saddle dikes were constructed with materials borrowed from till deposits east of the dam. The embankments were constructed in thin lifts as much as 8 in. thick, which were then compacted using

sheepsfoot rollers. The gravel and cobble toe blankets were constructed using material excavated from the stilling basin and the upper 1,000 ft of the channel enlargement and from the old rock-filled timber crib dam that was removed early in construction. A rock source located about three-fourths of a mile west of the dam provided riprap facing on the upstream flank of the embankment. Concrete aggregate was hauled by rail from a commercial pit near Steilacoom, Washington.

Construction problems related to driving of the outlet works tunnel, gate shaft, and ventilator shaft through the pervious outwash deposits in the right abutment were anticipated prior to construction (USBR, undated). The outlet tunnel was excavated to an outside diameter of 19 ft with hand picks and air spades using full face excavation and steel liner plates, which were installed at 1-ft intervals to minimize overbreak. The portion of the outlet tunnel upstream of the gate shaft proved to be relatively dry, and only minimal pumping at the heading was required to keep the excavation dry. The tunnel section downstream from the gate shaft experienced considerable inflow through old channel deposits composed of clean gravel and cobbles which were buried within the outwash. Inflow into the excavation was controlled by constant operation of two 6-in. pumps located in a sump at the outlet portal of the tunnel and a third pump situated at the tunnel heading. Use of the steel liner plates proved very effective in controlling overbreak and caving in the outwash materials; only a few scattered incidents of excessive overbreak were noted. The gate shaft was excavated from the top of the embankment down to the grade of the outlet tunnel, a depth of 140 ft. The walls of the shaft were stabilized by first driving interlocking sheet piling and followers to the base of the gate shaft prior to excavation. The gate shaft excavation proved to be relatively dry until the invert of the outlet tunnel was reached, at which point a rapid increase of seepage was noted. Overbreak areas outside both the outlet tunnel liner plates and the gate shaft sheet piling were backfilled using grout pumped at pressures as high as 100 psi.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Cle Elum Dam has experienced very few operational problems related to the site geology since it was placed into service in 1933. Reservoir leakage appears to have been controlled effectively by the natural impervious blanket lining the lake bottom; no seepage has been observed downstream from the structure. Landslides have not been noted within the reservoir area.

REFERENCES

Bryan, Kirk, 1927, *Memorandum in Regard to the Geology of Damsites at Cle Elum Lake, Washington*: Prepared for U.S. Geological Survey, Washington, DC, 7 p.

Frizzell, V. A., Jr.; Tabor, R. W.; Booth, D. B.; Ort, K. M.; and Waitt, R. B., 1984, *Preliminary Geologic Map of the Snoqualmie Pass 1:100,000 Quadrangle, Washington*: U.S. Geological Survey Open-File Report 84-693, 43 p., 1 sheet, scale 1:100,000.

Kinnison, H. B. and Sceva, J. E., 1963, *Effects of Hydraulic and Geologic Factors on Streamflow of the Yakima River Basin, Washington*: U.S. Geological Survey Water-Supply Paper 1595, 134 p.

Porter, S. C., 1976, Pleistocene Glaciation in the Southern Part of the North Cascade Range, Washington: *Geological Society of America Bulletin*, Vol. 87, pp. 61-75.

Ransome, F. L., 1930, *Cle Elum Dam Site*: Prepared for Chief Engineer, U.S. Bureau of Reclamation, Denver, CO, 12 p.

Tabor, R. W.; Frizzell, V. A., Jr.; Vance, J. A.; and Naeser, C. W., 1984, Age and Stratigraphy of Lower and Middle Tertiary Sedimentary and Volcanic Rocks of the Central Cascades, Washington–Application to the Tectonic History of the Straight Creek Fault: *Geological Society of America Bulletin*, Vol. 95, pp. 26-44.

U.S. Bureau of Reclamation, undated, *History of the Construction of the Cle Elum Dam, Yakima Project, Washington*: U.S. Bureau of Reclamation, Yakima, WA, 242 p.

U.S. Bureau of Reclamation (Water and Power Resources Service), 1981, *Project Data*: U.S. Government Printing Office, pp. 1337-1356.

U.S. Bureau of Reclamation, 1983, *Technical Data Workbook, Cle Elum Dam, Yakima Project, Pacific Northwest Region*: U.S. Bureau of Reclamation, Denver, CO, 749 p.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Clear Creek Dam

VIKKI L. McQUEEN
U.S. Bureau of Reclamation

PROJECT DESCRIPTION

Clear Creek Dam was one of the first dams built in conjunction with the Yakima River Project. The dam is located on the North Fork of the Tieton River 1/2 mi downstream from the confluence of the South Fork of Clear Creek and the North Fork of the Tieton River, approximately 48 mi west of Yakima and 25 mi southeast of Mount Rainier. Clear Creek Dam (Figure 1), originally constructed in 1914, is a concrete, thin-arch structure with gravity abutments (U.S. Bureau of Reclamation [USBR], 1985a). In 1918, an additional 21 ft were added to the crest of the dam, raising the structural height to 84 ft and the hydraulic height to 57 ft (USBR, 1985a). Other modifications constructed in 1918 included a spillway and two small earthen dikes placed to close topographic saddles about 155 and 325 ft southeast of the dam (USBR, 1983). The dikes each have structural heights of 6 ft (USBR, 1982). Because of progressively increasing deterioration of the concrete, complete rehabilitation of the structure was performed in 1964. This rehabilitation consisted of removal and replacement of the upper 23 ft of the arch system and part of the gravity sections. The spillway is located 600 ft northwest of the left abutment and is an open-rock cut channel with a crest 2 ft below the top of the dam (USBR, 1983). Designed for overtopping, the arch of the dam is also capable of handling extreme flows of water (Birch, 1961). The outlet works consists of two 36-in.-diameter cast-iron pipes through the base of the dam near the left abutment; discharges are controlled by two slide gates (USBR, 1983).

With the maximum water surface at 3,013 ft, Clear Lake has an active capacity of 5,300 acre-ft and is used as a storage reservoir to supply irrigation water for the Yakima valley. It also supplies water for industrial, municipal, and recreational purposes (USBR, 1985a).

SITE GEOLOGY

General

The topography and geology of the area near Clear Creek Dam are the result of late Cenozoic volcanism, tectonic uplift, and multiple glaciations. At the dam, the Tieton River valley is about 150 ft wide. Cliffs rise vertically for 30 ft and then more gently to elevation 3,015 ft, the elevation of the top of the dam (USBR, 1982).

The oldest strata exposed in the Clear Creek Dam area are pre-Tertiary and possibly early Tertiary shales and sandstones which, with interbeds of volcanic tuff and lava flows, were deposited in a coastal trough. During mid-Tertiary time, many large bodies of acidic igneous rock were intruded into the sediments (Walsh et al., 1987). Subsequent eruption of the Miocene Columbia River basalts (Tgrb on Figure 3) buried these sediments. Basic sills and dikes were also locally intruded (Birch, 1961). Regionally, large volcanoes formed and sent lava flows into the Clear Creek area. The upper ends of the stream valleys were completely covered with lava and ash, while andesitic flows partially filled sections of the stream canyons.

The region was uplifted in the late Tertiary and tilted east. Uplift and tilting accelerated stream erosion in the area, and the streams carved deep canyons into the basalts and underlying rock.

Pleistocene glaciers have eroded much of the andesitic rocks, exposing the older underlying rock and depositing glacial drift in the Clear Creek area. Some of this drift has since been reworked by the Tieton River, which has redeposited these materials as stratified silts, sands, and gravels (Birch, 1961).

Bedrock

The foundation of Clear Creek Dam (Figure 2) is composed mostly of a basic porphyry which has been moderately to intensely altered to a greenstone. This rock was probably an andesitic porphyry or basalt before propylitization (hydrothermal alteration). The basic porphyry is reddish purple to green and moderately jointed and intensely fractured. The rock is cross-cut by numerous thin quartz stringers. A marine shale, which has been partly metamorphosed, is also present in the damsite area. Though none of this metashale is exposed in the dam-site area, inclusions were excavated in the cut-off trench under the center of the dam. The metashale is fissile, black, and greasy in appearance (Birch, 1961; Miller, 1985).

Figure 1. Aerial view to the west (upstream) of Clear Creek Dam showing the crest of the concrete thin-arch dam. The exposure on the left abutment is a basic porphyry that has been altered to serpentine and epidote. U.S. Bureau of Reclamation photograph, October 5, 1979.

The general area of Clear Lake reservoir consists of Cenozoic volcanic rocks and strongly deformed and metamorphosed, pre-Tertiary rocks of the Russell Ranch Formation (Figure 3). Argillite and feldspathic graywacke that represents a turbidite assemblage comprise the bulk of the formation. Also common are pre-Tertiary greenstone and lesser amounts of chert, chert-pebble conglomerate, and metatuff, all part of the Rimrock Lake inlier (Miller, 1985). The bedding is graded and, where undeformed, planar and continuous. Younger sediments unconformably overlie the Russell Ranch Formation (Swanson, 1978).

Lava flows and volcaniclastic rocks are the most abundant Tertiary rocks occurring near Clear Lake. Lava flows exposed east of the dam are tholeiitic basalts that belong to the Grande Ronde Basalt, part of the Columbia River Basalt Group. The Grande Ronde Basalt is generally less than 1,000 ft thick, and flow directions indicate that the lava entered the area from the east and southeast (Swanson, 1978). Several andesitic lava flows of Quaternary age are exposed near the dam and lake. These flows are local to the area and are known as the Tieton Andesite (Swanson, 1978).

GEOLOGIC ASPECTS OF SITING AND DESIGN

Feasibility investigations for Clear Creek Dam were conducted in 1913 and 1914. Seven wash drill holes

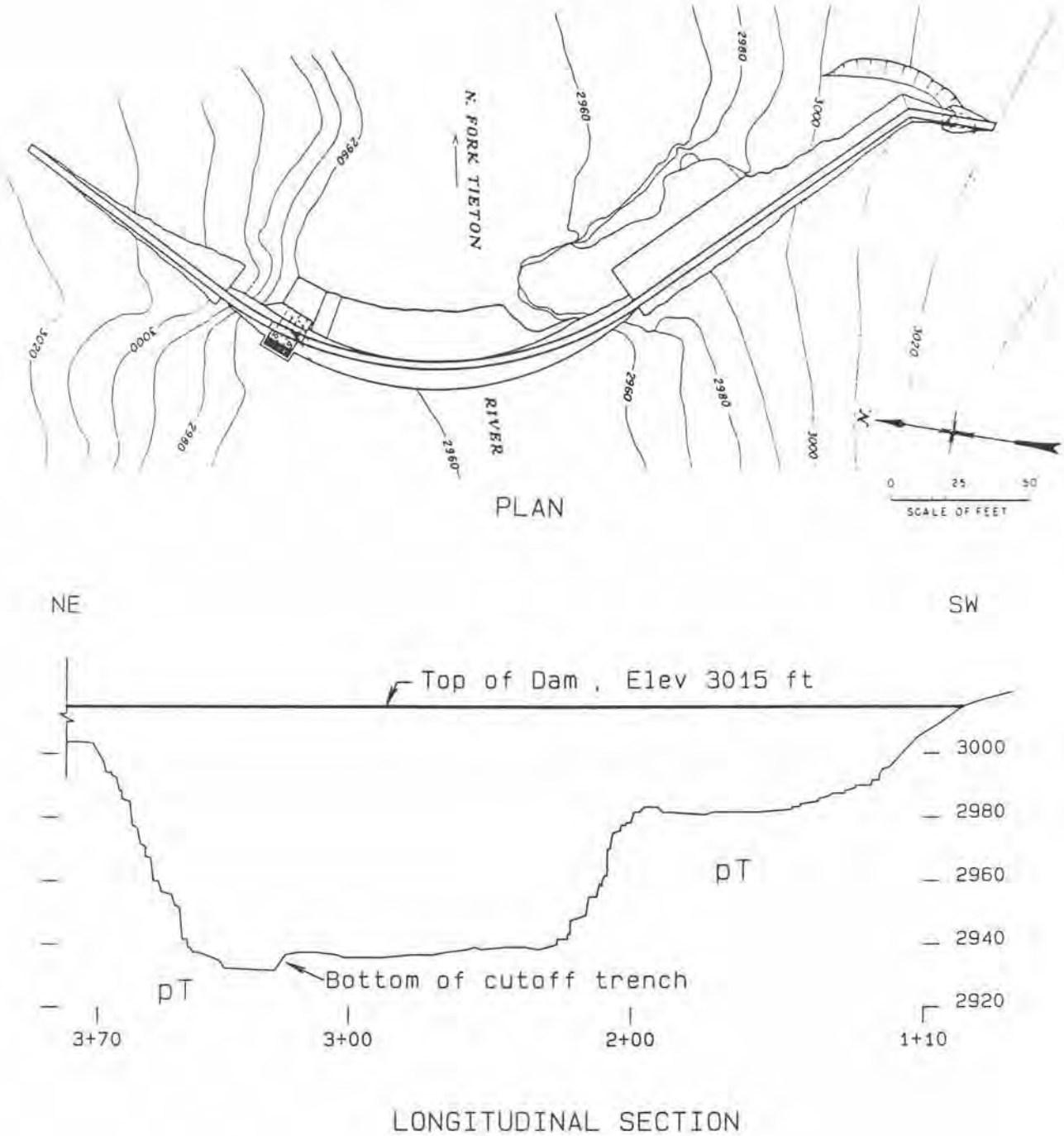


Figure 2. Plan and longitudinal section of Clear Creek Dam. Plan view adapted from U.S. Bureau of Reclamation (1983); longitudinal section adapted from unpublished Bureau sources.

were excavated in the lower abutment areas of the proposed dam and reached bedrock at depths of 6 to 19.5 ft. In addition, four diamond drill holes were drilled within the river channel. Bedrock was encountered in these holes at depths of 21 to 27 ft, and drilling continued another 14 to 30 ft into hard rock. The information gained from these investigations indicated that the rock was a hard, tight basalt and would make a competent foundation. With this knowledge, the decision was then made to build Clear Creek Dam at its present location (USBR, 1982).

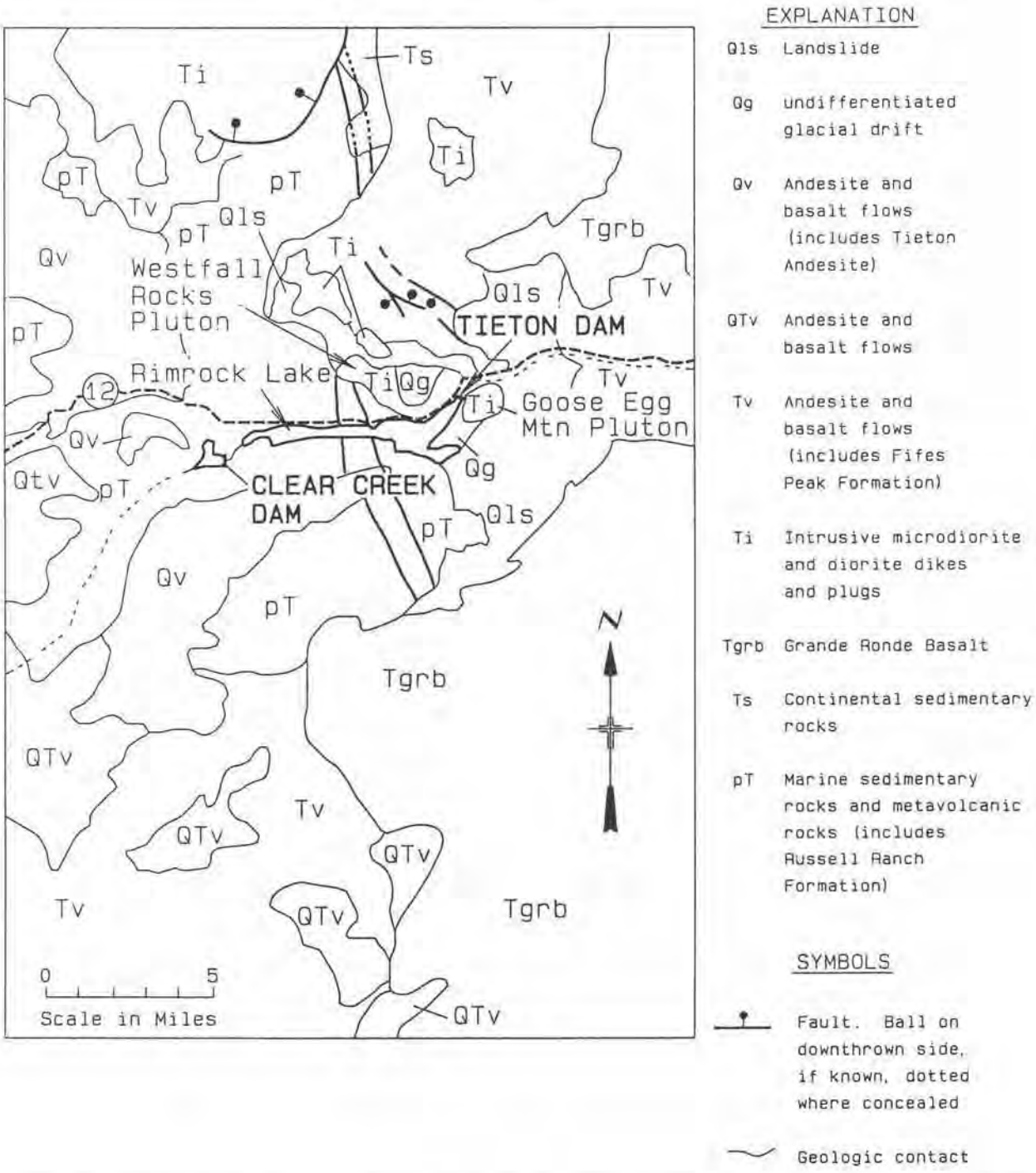


Figure 3. Generalized geologic map of the Clear Creek Dam and Clear Lake area. Adapted from Walsh et al. (1987).

CONSTRUCTION PROBLEMS

There is little information available concerning construction methods, materials, testing, or controls used during the original construction or during the 1918 enlargement. Seepage problems through the dam following the enlargement appear to be due to poor quality control of the concrete and poor methods of placing the concrete (USBR, 1982).

In 1961, an examination was begun to determine the structural stability of Clear Creek Dam and the need for possible rehabilitation of the structure. The investigations consisted of diamond drilling cores of the concrete

and foundation rock and a crack and deterioration survey of the concrete on both faces of the dam (USBR, 1961). Altogether, seven holes were drilled: two in concrete, two in rock, and three which began in concrete and ended in rock. The recovered core showed that all contacts between the concrete and rock were bonded, tight, and clean (Birch, 1961). A small amount of water (≈1 gpm) leaked into one hole at the downstream toe of the dam. Slow leaks were also noted in two other holes on the crest of the dam through cold joints in the concrete (Birch, 1961).

Grouting was not done when the dam was constructed in 1914 or in 1918 during enlargement, but was done during the 1964 rehabilitation. The plans called for a grout curtain located at the upstream heel of each abutment. Grout holes were located on 10-ft centers and drilled from 20 to 90 ft deep. Twenty grout holes were drilled on the right abutment and 13 on the left abutment. A total of 110 sacks of cement was injected into about 1,600 ft of grout holes (USBR, 1964). These minor grout takes indicate that the rock was tight (USBR, 1982). During the past several Safety Evaluation of Existing Dams examinations, the abutments of Clear Creek Dam have been free of flowing seepage (USBR, 1985b). Several small, nonflowing, damp areas exist on the left abutment and farther downstream, but they pose no danger to the structure (USBR, 1982).

In addition to the grouting, concrete was removed and replaced from the top 22 ft of the arch, the top 6 ft or more of the right thrust block, and various depths to elevation 2,994 ft of the left thrust block depending upon the degree of deterioration. Epoxy mortar was then applied to the upstream and downstream faces and a neoprene coating was applied to the upstream face and all cracks (USBR, 1964).

In 1987 and 1988, concrete core samples were obtained to aid in determining the extent of any further deterioration of the structure at Clear Creek Dam. Approximately 216 ft of concrete core samples from five drill holes were examined. Core evidence indicates that bonding between the 1964 concrete and 1914-1918 concrete is excellent. Some seepage was noted in an unbonded lift line in the 1964 concrete about 10 ft below the crest of the dam, as well as below about elevation 2,998 ft in the central arch, where localized fracturing of the 1914-1918 concrete is apparent. Although observations of the downstream face of the arch section and the 1987-1988 core indicate that the 1914-1918 concrete is locally fractured, the dam has performed well in the 70 yr since it was constructed (Gilbert, 1988).

REFERENCES

Birch, R. O., 1961, *Geologic Field Investigation, Clear Creek Dam*: U.S. Bureau of Reclamation, Yakima, WA, 12 p.

Gilbert, J. D., 1988, *1987 Concrete Core Sampling, Clear Creek Dam*: U.S. Bureau of Reclamation, Yakima, WA, 9 p.

Miller, R. B., 1985, *The Pre-Tertiary Rimrock Lake Inlier, Southern Cascades, Washington*: Washington Division of Geology and Earth Resources Open File Report 85-2, Olympia, WA, 16 p., 1 plate.

Swanson, D. A., 1978, *Geologic Map of the Tieton River Area, Yakima County, South-Central Washington*: U.S. Geological Survey Miscellaneous Field Studies Map MF-968, 1 sheet, scale 1:48,000.

U.S. Bureau of Reclamation, 1961, *Annual Project History for 1961–Yakima Project*: U.S. Bureau of Reclamation, Yakima, WA, p. 31.

U.S. Bureau of Reclamation, 1964, *Annual Project History for 1964–Yakima Project*: U.S. Bureau of Reclamation, Yakima, WA, pp. 34-37.

U.S. Bureau of Reclamation, 1982, *Report on Safety of Dam and Appurtenant Works, Clear Creek Dam*: U.S. Bureau of Reclamation, Denver, CO, 63 p.

U.S. Bureau of Reclamation, 1983, *Project Data Book, Yakima Project, Washington: Benton, Franklin, Yakima, and Kittitas Counties*: U.S. Bureau of Reclamation, Denver, CO, 20 p.

U.S. Bureau of Reclamation, 1985a, *Standing Operating Procdures, Clear Creek Dam and Clear Lake, Yakima Project, Washington*: U.S. Bureau of Reclamation, Yakima, WA, 151 p.

U.S. Bureau of Reclamation, 1985b, *Review of Operation and Maintenance Program and Safety Evaluation of Existing Dams Programs, Examination Report, Clear Creek Dam*: U.S. Bureau of Reclamation, Yakima, WA, 16 p.

Walsh, T. J.; Korosec, M. A.; Phillips, W. M.; Logan, R. L.; and Schasse, H. W., 1987, *Geologic Map of Washington–Southwest Quadrant*: Washington Division of Geology and Earth Resources Geologic Map GM-34, Olympia, WA, 28 p., 2 sheets, scale 1:250,000.

Diamond core hole drilling in September 1988 on the channel section of Clear Creek Dam. Drillers were using a portable drill motor to obtain NX-size samples for laboratory testing to determine the extent of concrete deterioration. The foundation rock exposed is greenstone. U.S. Bureau of Reclamation photograph.

Engineering Geology in Washington, Volume I
Washington Division of Geology and Earth Resources Bulletin 78

Easton Diversion Dam

DAN N. MAGLEBY
U.S. Bureau of Reclamation

PROJECT DESCRIPTION

Easton Diversion Dam (originally called the Kittitas Diversion Dam) is on the upper reach of the Yakima River about 1/2 mi west of Easton (Figure 1). The dam is used mostly for diversion to the Kittitas Main Canal, which provides irrigation water to more than 50,000 acres in the Kittitas Valley near Ellensburg. The reservoir, whose water level fluctuates very little, has a capacity of about 2,500 acre-ft and provides a prime recreation facility at Lake Easton State Park. Easton Diversion Dam was built by the U.S. Bureau of Reclamation in 1929 (U.S. Bureau of Reclamation [USBR], 1928, 1981).

The dam is a small, concrete, gravity diversion dam with a structural height of 66 ft and a crest length of 248 ft; elevation at top of dam is 2,184 ft (Figure 2). The spillway is an ogee-overflow-weir section across the central portion of the dam and is controlled by one 64-ft x 14.5-ft drum gate with a design capacity of 13,000 cfs; the top of drum gate is at elevation 2,180.3 ft. The outlet works consists of two sluiceways, one in each side of the dam below the spillway crest. Each of these outlets is controlled by a 4.8-ft x 6.0-ft gate. Located in the right abutment of the dam is the Kittitas Main Canal headworks (capacity 1,320 cfs) from which flow is diverted through the trash screens and fish screens to a horseshoe-shaped tunnel 12.25 ft in diameter. These flows are controlled by two 12-ft x 11-ft radial gates. The weir-type fish ladder is located on the left abutment of the dam.

The Bureau of Reclamation's planned improvement of the fish passage facilities at Easton Dam will include a new, enlarged fish ladder on the left abutment and a new rotating drum screen in the Main Canal just downstream from the right abutment of the dam. Associated with the fish screen will be the fish bypass pipe and the juvenile-fish counting facility.

SITE GEOLOGY

General

Easton Dam is high on the eastern flank of the northern Cascade Range. Geologic units at the site range in age from pre-Tertiary (probably Jurassic) through Quaternary. The north-south-trending Straight Creek fault system passes near the town of Easton and divides the area into two distinct structural blocks—the Teanaway River block on the east and the Cabin Creek block on the west (Tabor et al., 1984).

The pre-Tertiary rocks (Easton Schist) are classified mostly as schists and related rock types. Unconformably overlying the oldest rocks are Tertiary sandstones, basalt and andesite flows, and other volcanic and sedimentary rocks. The on-lapping Miocene Columbia River basalt flows are just to the east of the area (Smith and Calkins, 1906; Frizzell et al., 1984). Quaternary glacial debris masks much of the valley sides, and Holocene alluvium is present along the major drainageways.

The Easton Schist is exposed in anticlinal cores in the Easton area. It occurs in a northwest-trending narrow outcrop along the Yakima River from Easton Reservoir downstream for several miles beyond the town of Easton (Figure 3). The Easton Schist has been variously described as quartzite, amphibolite, blueschist, greenschist, and greenstone. The schist is mostly hard and competent but locally has softer zones related to schistosity, shear zones, and weathering along prominent joints.

In the Cabin Creek block to the west of Easton, the rocks overlying the schist are mostly Eocene volcanic rocks of the Naches Formation. In the Teanaway River block to the east, the rocks overlying the schist are mostly sedimentary rocks of the Eocene Teanaway Formation.

Glacial deposits occur mostly as bench deposits above the river. Slopewash covers some of the bedrock above the glacial benches (USBR, 1985a).

No faulting has been identified in the dam foundation. However, a USBR underwater inspection report indicates a possible localized shear zone at the submerged downstream toe of the dam (USBR, 1983).

Bedrock

The bedrock for the dam and all the structure sites for the fish passage facilities is the Easton Schist (USBR, 1985a, b). The schist is exposed on both abutments of the dam and along the canyon walls for several hundred feet downstream from the dam.

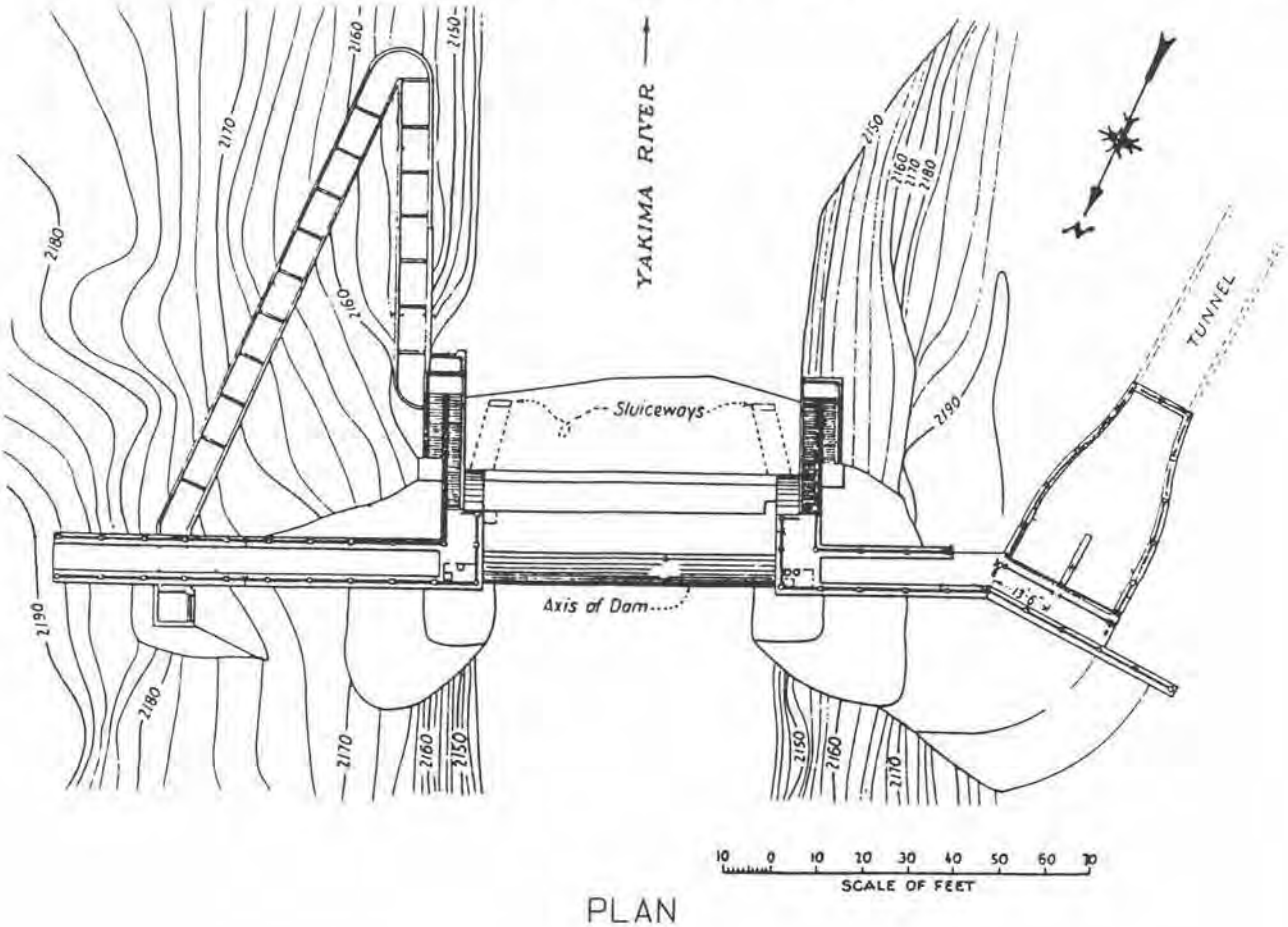
Figure 1. Aerial view upstream of the Easton Dam and Reservoir. Interstate Highway 90 is on right side of photo. The Kittitas Main Canal diverts from the dam through a tunnel beneath the railroad tracks (left). The rock exposed at the left and right abutments of dam is the Easton Schist. U.S. Bureau of Reclamation photograph, July 20, 1979.

The schist in the dam area, as described from outcrops and drill cores, is mainly a fine-grained, green to blue-green to gray, moderately to intensely fractured greenschist which has a distinctive, very closely spaced schistosity. The fracture surfaces are mainly planar; some are stained with limonite, a few are filled with as much as 1/4 in. of silty and clayey alteration products. The planar features due to the schistosity may parallel the fractures. The foliation surfaces feel slick, probably due to the presence of sericite mica. The greenschist is characterized by quartz lenses and blebs that mostly parallel the foliation. (The logs of the preconstruction drill holes at the dam site showed the schist as quartzite.) The schist is mostly hard but locally can be easily broken, especially adjacent to prominent fractures. The engineering properties of the schist in the dam area are highly varied, due mainly to weathering, fracturing, and foliation (USBR, 1985b).

Overburden Materials

Glacial outwash forms the "bench" on the right canyon side just downstream from the dam. It is mostly a compact, unsorted material consisting of hard, rounded to subrounded sand, gravel, cobbles, and some boulders primarily of quartzites and igneous rocks. In many places, a silty to clayey matrix holds the coarse-grained fractions together.

Recent alluvium is present in the bottom of the Yakima River canyon to depths of about 15 ft.



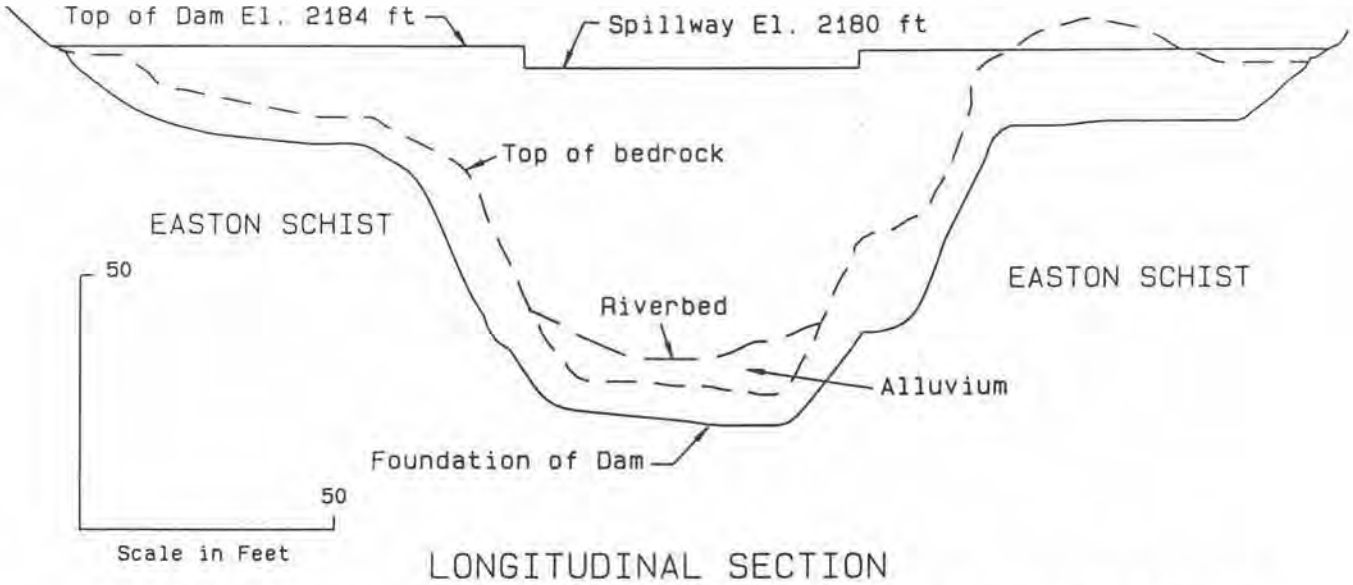


Figure 2. Plan and longitudinal section of Easton Dam; view downstream. Plan view adapted from U.S. Bureau of Reclamation (1981).

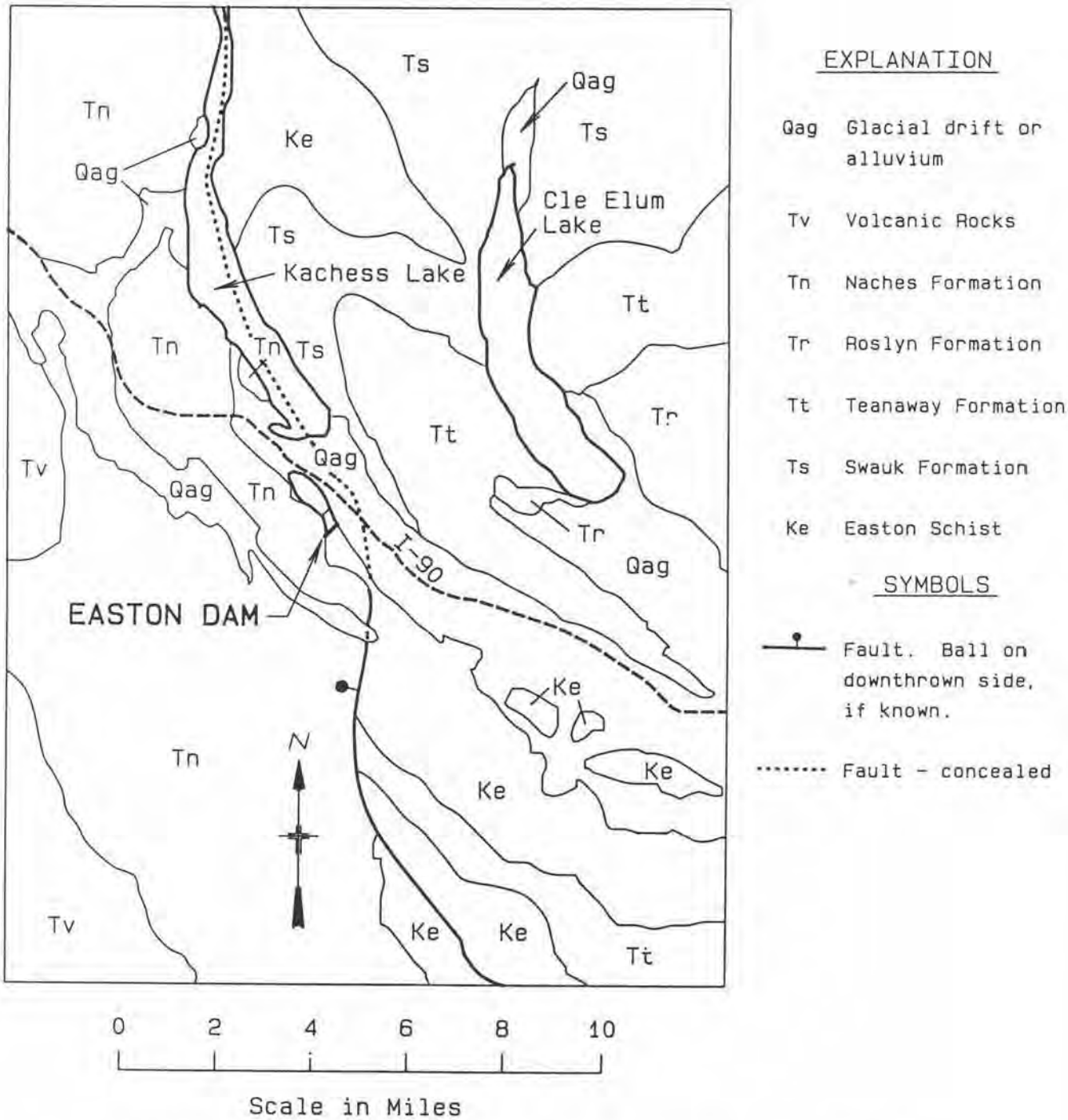


Figure 3. Generalized geologic map of the Easton Dam area. Adapted from Frizzell et al. (1984).

GEOLOGIC ASPECTS OF SITING AND DESIGN

Easton Dam is located in the gorge of the Yakima River where the outcrops of the Easton Schist form a narrow, steep-walled section of canyon. This site was also topographically favorable for the elevation of the canal headworks to serve, by gravity, the land intended for irrigation. The narrow canyon cut in competent bedrock provided a favorable site for a concrete gravity dam (Bryan, 1924).

CONSTRUCTION

Preparation of the dam foundation included removal of as much as 15 ft of alluvial deposits in the canyon bottom. The foundation excavation extended into solid rock for a depth of 3 to 4 ft over the entire dam base, and a keyway trench was then excavated to a depth of 5 ft. Grout holes were drilled on 5-ft centers in the keyway for the entire length of the dam. These holes were drilled to a depth of 25 ft in the river channel and 10 ft on the side slopes. The average grout take was 4.75 cu ft per hole. The concrete aggregate materials were hauled in by rail from the Tacoma area. All loose material was removed from the foundation of the dam before placement of the concrete was started. New placements on previous lifts were made by removing 3/4 in. of existing concrete, cleaning the surface, and placing two layers consisting of a 1-in. layer of the grout mortar followed by a 6-in. layer of concrete before starting a new placement (USBR, 1983).

OPERATIONAL PROBLEMS

The purpose of the sluiceways is to pass low flows and to sluice accumulated silt from upstream of the dam. The gates to the sluiceways are operated numerous times during the year to flush this silt.

Minor seepage flows from a horizontal construction joint in the left nonoverflow section of the concrete dam at approximate elevation 2,163 ft. Seepage has also been noted in the left abutment just below the area of a leaky construction joint. Some undercutting of the spillway section at the toe of the dam has been reported in underwater inspections. The amount of undercutting does not presently affect the safety of the dam (USBR, 1983).

Although much of the reservoir abuts glacial and alluvial deposits, neither landslides nor unstable ground have been reported around the reservoir shoreline during the operation of the project.

REFERENCES

Bryan, Kirk, 1924, *Geology of the Diversion Dam Site and of the Tunnel Sites, Kittitas Division, Yakima Project, Washington*: U.S. Geological Survey unpublished report, 22 p.

Frizzell, V. A., Jr.; Tabor, R. W.; Booth, D. B.; Ort, K. M.; and Waitt, R. B., 1984, *Preliminary Geologic Map of the Snoqualmie Pass 1:100,000 Quadrangle, Washington*: U.S. Geological Survey Open-File Report 84-693, 42 p., 1 plate, scale 1:100,000.

Smith, G. O. and Calkins, F. C., 1906, *Snoqualmie Washington*: U.S. Geological Survey Folio 139 of the Geologic Atlas of the United States, 14 p., 3 maps.

Tabor, R. W.; Frizzell, V. A., Jr.; Vance, J. A.; and Naeser, C. W., 1984, *Ages and Stratigraphy of Lower and Middle Tertiary Sedimentary and Volcanic Rocks of the Central Cascades, Washington–Application to the Tectonic History of the Straight Creek Fault*: Geological Society of America Bulletin, Vol. 95, pp. 26-44.

U.S. Bureau of Reclamation, 1928, *Kittitas Diversion Dam and Main Canal to Sta. 34+37.9, Yakima Project, Washington*: Specification No. 468, U.S. Bureau of Reclamation, Boise, ID, 21 p.

U.S. Bureau of Reclamation (Water and Power Resources Service), 1981, *Project Data*: U.S. Government Printing Office, pp. 1337-1356.

U.S. Bureau of Reclamation, 1983, *Examination Report of Easton Diversion Dam for SEED Program, Yakima Project, Washington*: Memorandum from H. W. Sandhaus to Chief, Inspections Branch, dated August 24, 1983, U.S. Bureau of Reclamation, E&R Center, Denver, CO, 10 p.

U.S. Bureau of Reclamation, 1985a, *Reconnaissance Geologic Report, Easton Diversion Dam Fish Screen, Yakima Fish Passage Facilities, Yakima Project, Washington*: U.S. Bureau of Reclamation, Boise, ID, 12 p., drawings and photos.

U.S. Bureau of Reclamation, 1985b, *Geologic Report for Final Design, Kittitas Main Canal Fish Screen and Easton Diversion Dam Fish Ladder, Yakima Fish Passage Facilities, Yakima Project, Washington*: U.S. Bureau of Reclamation, Boise, ID, 22 p., photos and drawings.

Aerial view downstream showing the construction of a new fish ladder on the left abutment of the Easton Diversion Dam. U.S. Bureau of Reclamation photograph, October 7, 1988.

Aerial view showing the construction of the fish screen structure in Kittitas Canal just downstream of the right abutment of the Easton Diversion Dam. U.S. Bureau of Reclamation photograph, October 7, 1988.

Kachess Dam

ALLEN C. LOCKHART
U.S. Bureau of Reclamation

PROJECT DESCRIPTION

Kachess Dam is located 14 mi northwest of Cle Elum in Kittitas County on the Kachess River (Figure 1). The dam was constructed about 1,500 ft downstream from the outlet of Kachess Lake, an existing natural lake, as a multi-purpose project providing flood control, recreation, and storage for irrigation needs downstream. Construction of this zoned, rolled earthfill structure was completed in 1912; storage of water started in the summer of 1911.

Kachess Dam has a structural height of 115 ft and a hydraulic height of 59 ft (U.S. Bureau of Reclamation [USBR], 1981). The length of the dam at the crest elevation 2,268 ft is 1,400 ft, including a 250-ft-long dike filling the original spillway on the left abutment; the crest width is 20 ft (USBR, 1981). The upstream zone is a rolled impervious material consisting of clay, sand, and gravel; the downstream zone consists mostly of gravel. The embankment was compacted in 6-in. layers with a 3H to 1V upstream slope and 2H to 1V downstream slope and is contained in a rock shell (USBR, 1981). The rock shell consists of 3 ft of small rock overlain by 2 ft of larger riprap rock. A cut-off trench was excavated along the main portion of the dam (Figure 2).

In 1936, the dam was rehabilitated with a new reinforced concrete spillway with a 50-ft-wide crest at elevation 2,254 ft. The new spillway, located on the right abutment, was constructed 4 ft lower than the original spillway crest. The original spillway, located on the left abutment about 1/2 mi from the dam, was abandoned and an earth dike placed over it. This dike embankment consists of compacted fine-grained soil with a maximum height of 18 ft (USBR, 1987).

When the reservoir is below elevation 2,230 ft, water passage from the reservoir is through an open channel and tunnel to the outlet works structure. A 12-ft horseshoe-shaped concrete conduit with three tandem pairs of 4-ft x 10-ft slide gates controls the downstream water passage (USBR, 1987). The open channel and tunnel outlet works were excavated through the upstream part of a moraine. Kachess Lake has an active storage capacity of 239,000 acre-ft at the normal reservoir water surface elevation of 2,262 ft.

SITE GEOLOGY

Kachess Dam is situated in a glaciated valley cut into pre-Tertiary and Tertiary metamorphic, volcanic, and sedimentary rocks. Pre-Tertiary metamorphic rocks of the Easton Schist and the Tertiary Swauk Formation, an arkosic sandstone including the Silver Pass Volcanic Member, are the oldest exposed rocks in the vicinity of the dam (Frizzell et al., 1984). The reservoir and dam site are underlain and contained by these two formations, as well as by volcanic rocks of the Tertiary Teanaway and Naches formations (Figure 3). The Teanaway Formation consists predominantly of basaltic lava flows, whereas the Naches Formation consists of basaltic and andesitic and rhyolitic volcanic materials (Frizzell et al., 1984).

Structures within the area trend primarily north and northwest, although some less well expressed features trend west. Rocks in the area of Kachess Dam and Reservoir are folded into a series of anticlines and synclines (Figure 3). The northwesterly structural trend is expressed by folds and faults, which formed concurrently with the deposition of the above formations.

Several normal faults are known in the immediate area of Kachess Dam and Reservoir. The largest, most significant of these is the Kachess Lake fault, which trends north-northwest through the Kachess River valley (Figure 3). The Kachess Lake fault is thought to be the southernmost segment of the Straight Creek fault zone, a major structure extending south from Canada into Washington State (Piety, 1986). Two northeast-trending faults, the Swan Lake and Lodge Creek, have been mapped on Kachess Ridge east of the reservoir (USBR, 1987). Each is less than 5 mi. long, and both offset the Swauk and Teanaway formations (Figure 3).

Valley glaciers were extensive in the area of the dam during the Pleistocene and have greatly modified the topography. Glacial materials including moraines, till, and outwash deposits mantle much of the area to various thicknesses (Frizzell et al., 1984). These materials are on the ridges above the dam and reservoir, and fill the Kachess River valley to more than 100 ft at the site of Kachess Dam. The terminal moraine on and out of which Kachess Dam was constructed consists of a

Figure 1. View to the northwest of Kachess Dam and Reservoir. The entire dam is situated on a terminal moraine that extends across the Kachess River valley. The dam restored the water level behind the moraine to near its pre-breach level. Rocks of the Easton Schist are exposed in the upper reservoir area with the Swauk Formation on the east of the reservoir and the Naches Formation along the west side. I-90, Interstate Highway 90. U.S. Bureau of Reclamation photograph, July 1968.

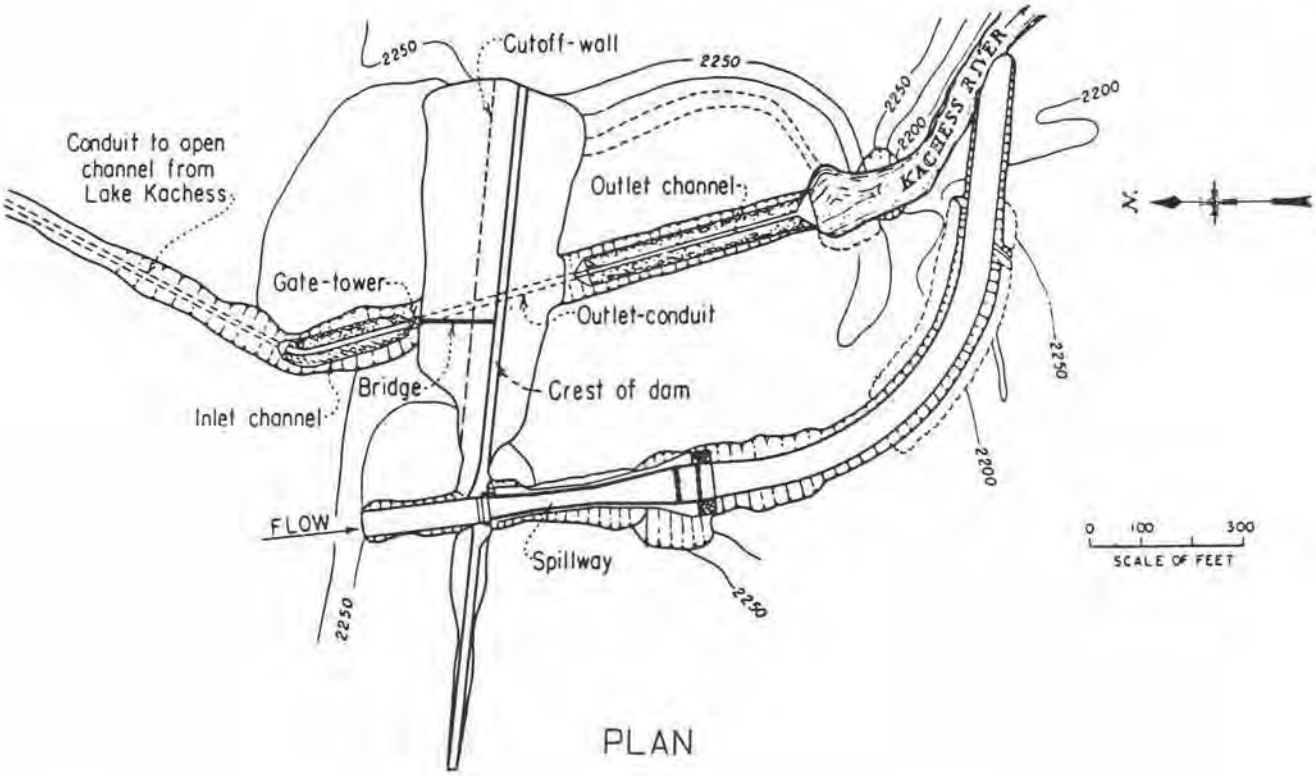
heterogeneous mixture of clay, sand, gravel, cobbles, and boulders. Pleistocene stream terrace deposits and Holocene slopewash and alluvium mask areas of the older rock on the valley floor. Holocene alluvial deposits and reworked glacial debris mantle the rocks in most of the surrounding major stream drainages, as well as the Kachess River channel downstream of the dam (Porter, 1976). The Pleistocene units which overlie large areas near the dam are generally undeformed, except for some minor normal faulting (Piety, 1986).

GEOLOGIC ASPECTS OF SITING AND DESIGN

The site for Kachess Dam was selected solely because an existing natural lake and morainal dam were present and the moraine would provide readily available construction materials. In the early 1900s, project personnel determined that a dam could be placed across the natural outlet of Kachess Lake. The dam was constructed in the breach of the moraine. By restoring the natural glacial dam to its pre-breach height, a lake similar in size to the original natural lake was impounded (USBR, 1985).

Prior to construction of the dam, the maximum depth of the river channel below the top of the moraine was approximately 59 ft.

The entire foundation of the dam, including the abutments, is formed by the terminal moraine. Maximum thickness of the moraine at the dam can only be estimated, as most of the early test borings were placed in areas other than along the centerline of the dam on the crest of the moraine. Borings 500 to 700 ft upstream of the dam on the lower portion of the moraine and on both sides of the intake conduit encountered bedrock at depths ranging from 50 to 87 ft. Bedrock was not encountered at either abutment or at any location throughout the length of the dam. Excavations for the cut-off trench and the core wall trench, as well as exploratory test pits, indicated that much of the moraine contains sufficient fine-grained material to be relatively



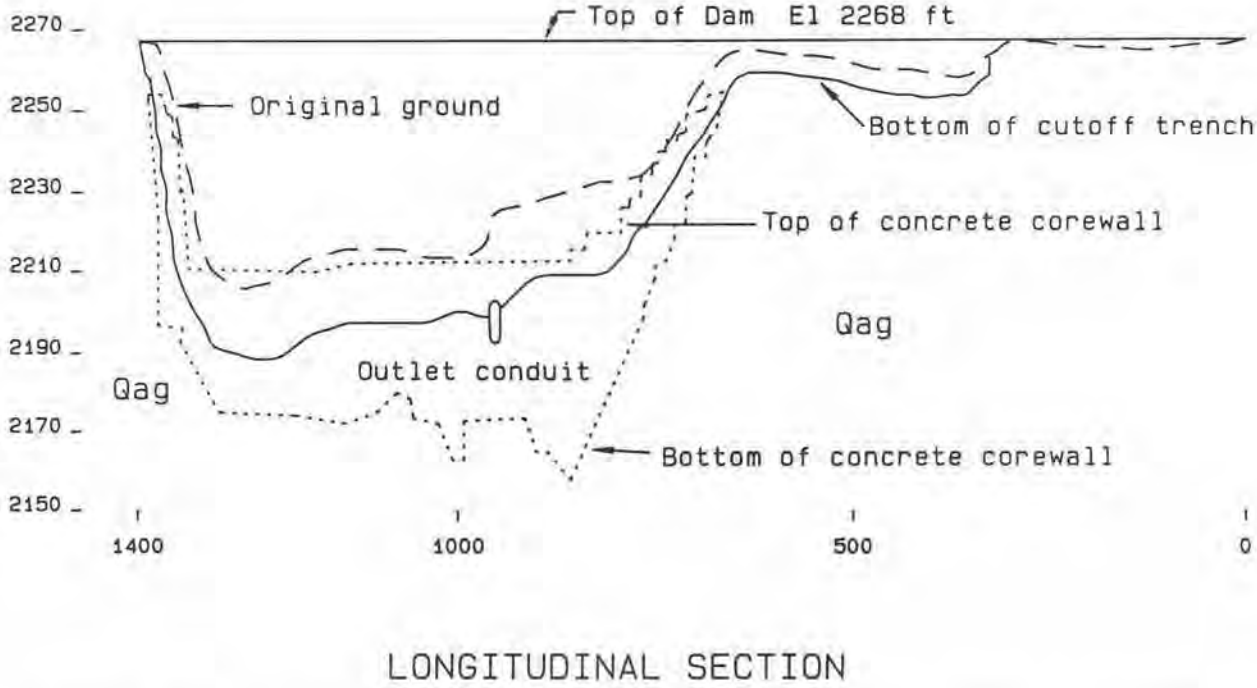


Figure 2. Plan and longitudinal section, Kachess Dam; plan view adapted from U.S. Bureau of Reclamation (1981). See Figure 3 for explanation of symbols.

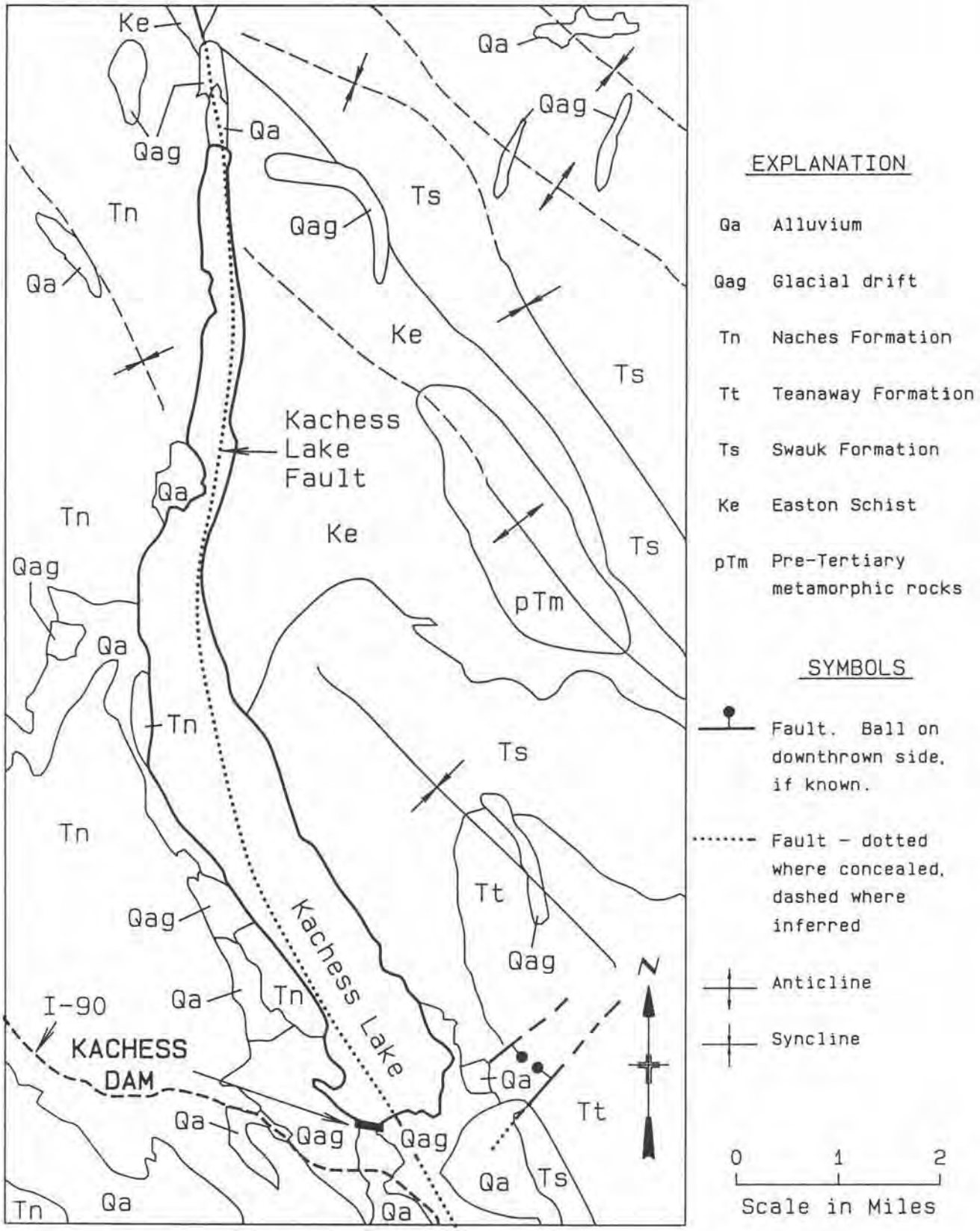


Figure 3. Generalized geologic map of the Kachess Dam area. Adapted from Frizzell et al. (1984).

impervious (USBR, 1987). Some of the borings and excavations encountered a hardened layer, termed hard pan or cemented gravel, in the foundation at the dam; however, little is known about this material other than that it is shown on construction drawings (USBR, 1987). The material is probably a well compacted gravel with a silty and clayey matrix and not a cemented layer. This competent material forms the core wall foundation under the dam (Figure 2). The cut-off trench and core wall were constructed to prevent any under seepage in the immediate foundation at the dam. Embankment materials were borrowed from a site on the crest of the moraine adjacent to the dam.

Owing to limited drilling capabilities at the time of construction and the rationalization that seepage through the moraine was not a significant problem, no attempt was made to determine the depth to bedrock across the valley floor at the dam. On the basis of the reported depth of water in the lake and in conjunction with the topography, the moraine is probably more than 100 ft thick in the middle of the valley (USBR, 1985).

CONSTRUCTION

Stripping

Stripping at the site was performed with teams of horses and drag scrapers. The total volume of stripped material was reported to have been 10,000 cy. The depth of stripping was not recorded; however, records indicate that from 5 to 25 ft of the alluvium and outwash deposits, consisting of coarse sand and gravel, were removed from the dam site (USBR, 1987). Records did not indicate what was done with this material, but it is believed that the material containing organic debris was wasted and the remainder of the material was used as miscellaneous fill.

Cut-off Trench

The cut-off trench extends longitudinally under the main portion of the dam for a distance of 860 ft (Figure 2). Excavation depths ranged from 3 to 4 ft in the peripheral areas on either side of the channel to a maximum of about 28 ft at the center of the river; the excavation was approximately 20 ft wide at the base. The core wall trench was extended below the base of the cut-off trench approximately 75 ft below the original ground surface, to elevation 2,153 ft at the lowest point (USBR, 1987). The core wall is approximately 640 ft long and extends along the center portion of the dam. Records indicate that the original intention was to extend the core wall to bedrock; however, a single 13-ft drill hole in the bottom of the core wall trench did not encounter bedrock (USBR, 1987). It is believed that the well compacted layer of gravel was encountered in the drill hole. Owing to the impervious nature of the materials in the core wall trench, excavation operations were suspended and the core wall poured. The core wall is a non-reinforced concrete wall except at its connection with the outlet works conduit. Backfilling around the core wall was completed by the hydraulic fill method (puddling) of placement (USBR, 1985).

Materials which form the foundations for the gate house tower and appurtenant features, as well as the outlet tunnel, are typical of the moraine deposits. On the basis of records of exploration test pits and borings and construction excavations, it is believed that the tunnel and outlet channel were excavated with little difficulty.

No grouting is known to have been performed at Kachess Dam (USBR, 1985). Records indicate that two drains were installed at the base of the dam to collect any seepage that might come through the foundation and dam and around the abutments (USBR, 1983). A rockfill toe drain 6 to 10 ft wide at the bottom and 6 to 8 ft deep extends the entire length of the dam. Two outlets were provided for the toe drain; one is an open trench just west of the outlet works conduit, and the other consists of a rock drain about 4 ft by 10 ft in cross section and 100 ft long at the old river channel. An additional rockfill toe drain was constructed of 12-in. tile laid with open joints in a trench and surrounded with small stones, with closely spaced outlets to the main toe drain. This drain was installed 30 to 60 ft upstream from the downstream toe of the dam and was extended the entire length of the dam. The system may also serve to drain the foundation as well as the embankment (USBR, 1983).

Only a general description of the materials or criteria used in constructing the embankment is available. The majority of the impervious zone of the dam was constructed of a glacial material taken from a borrow area located just east of the dam (USBR, 1987). Materials for the embankment were hauled to the site by a steam locomotive riding on a trestle system built along the centerline. The plus-4-in.-size material was removed and used as either riprap or riprap bedding (USBR, 1985). Based on the average of the grain and clast sizes from the tests on the borrow material, the impervious zone of the embankment had approximately 46 percent gravel, 46 percent sand, and 7 percent fines. The materials were placed in a loose, 8-in. layer, then compacted to approximately 6 in. with a "grooved roller". Placement of the downstream gravel zone was accomplished in a similar manner, except that more water was used and less attention was paid to removing the plus-4-in.-size material (USBR, 1985).

OPERATING PROBLEMS RELATING TO GEOLOGY

Reservoir Leakage

Project histories (USBR, 1987) indicated that, upon initial filling of the reservoir, springs or boils occurred along the left bank of the Kachess River approximately 500 and 1,300 ft downstream from the centerline of the dam. Some springs flows were as much as 225 gpm.

Conclusions from the first observations of the springs were that the seepage was not a hazard to the structure. Since the initial filling, the spring flows have diminished from 250 gpm from several springs to 210 gpm for a combined flow of all leakage at the dam. Sedimentation may have stemmed the flows by filling intergranular spaces in the dam and foundation materials.

At the present time, most of the seepage at Kachess Dam appears to be coming through the left abutment. These seeps have been reported in the annual operation and maintenance reports since the construction of the dam. Two weirs located approximately 300 and 600 ft downstream from the toe of the dam measure the present seepage. The weirs are located in a natural drainage which is the approximate location of the old river channel. The seepage was reported to be clear (USBR, 1985).

A small amount of seepage (1 to 2 gpm) was reported in 1914 above the lower end of the retaining wall on the right side of the original spillway. According to Project personnel (U.S. Reclamation Service, 1914), this seepage causes the slope above the spillway, consisting of glacial debris, to slump and slide, requiring periodic repair. However, the slides have been only minor since the construction of the dam. At the present, the slope is boggy, and distortion of trees growing in the area indicates continued movement (USBR, 1987).

Ground and Embankment Stability

There are no known landslide areas in the vicinity of the dam and reservoir, and no landslides are known to have been recorded in the history of Kachess Reservoir. The materials in the reservoir rim suggest that there is little potential for landslides. Most of the west side of the reservoir has relatively gentle topography. Although the topography is considerably steeper on the east side of the reservoir, the rock is quite competent and has little tendency for sliding.

Several small (approximately 2 ft across) sinkholes developed in the embankment in the early 1970s. Following an investigation, the holes were judged not to indicate a safety problem. The settlement was attributed to localized rotting of timber trestles used in the original construction. Another possibility is that settlement occurred in poorly compacted material beneath the trestle. The sinkholes were repaired following their evaluation, and none have been noted since (USBR, 1985).

Parts of the slopes along the open channel extending upstream into the reservoir from the conduit to the outlet works have, from time to time, been unstable. The problem is created by bank storage and the slope's inability to be rapidly relieved of such storage during drawdown. Elevated pore pressure causes localized failures in the slopes which are composed of fine-grained glacial lake sediments. The problem is not significant, and the channel has not been restricted.

REFERENCES

Frizzell, V. A., Jr.; Tabor, R. W.; Booth, D. B.; Ort, K. M.; and Waitt, R. B., 1984, *Preliminary Geologic Map of the Snoqualmie Pass 1:100,000 Quadrangle, Washington*: U.S. Geological Survey Open-File Report 84-693, 42 p., 1 plate, scale 1:100,000.

Piety, L., 1986, *Seismotectonic Analysis for SEED Report, Kachess Dam, Draft Technical Memorandum No. SAR-1632-109*: U.S. Bureau of Reclamation, Denver, CO, 14 p.

Porter, S. C., 1976, Pleistocene Glaciation in the Southern Part of the North Cascade Range, Washington: *Geological Society of America Bulletin*, Vol. 87, pp. 61-75.

U.S. Reclamation Service, 1914, *Yakima Project History, Annual Inspection, Washington*: U.S. Reclamation Service, Yakima, WA, pp. 2-3. [Available from U.S. Bureau of Reclamation, Yakima, WA]

U.S. Bureau of Reclamation (Water and Power Resources Service), 1981, *Project Data*: U.S. Government Printing Office, pp. 1337-1356.

U.S. Bureau of Reclamation, 1983, *Examination Report of Kachess Dam for SEED (Safety Evaluation of Existing Dams) Program–Yakima Project, Washington*: U.S. Bureau of Reclamation, Denver, CO, 11 p.

U.S. Bureau of Reclamation, 1985, *Examination Report of Kachess Dam for SEED (Safety Evaluation of Existing Dams) Program–Yakima Project, Washington*: U.S. Bureau of Reclamation, Denver, CO, 9 p.

U.S. Bureau of Reclamation, 1987, *Preliminary SEED Report for Kachess Dam–Yakima Project, Washington*: U.S. Bureau of Reclamation, Denver, CO, 44 p.

Keechelus Dam

BRENT H. CARTER
U.S. Bureau of Reclamation

PROJECT DESCRIPTION

Keechelus Dam is a zoned, rolled, earthfill embankment at the outlet of a natural glacial lake (Lake Keechelus) a few miles east of the crest of the Cascade Range. The dam is at the head of the Yakima River, about 10 mi northeast of Easton. By spanning the river 300 ft downstream of the lake, the 128-ft-high (structural height) dam raised the natural lake level 97 ft (Figure 1). The dam was constructed from 1913 to 1917 for the primary purpose of irrigation water storage. Flood control is provided above and beyond contractual and decreed flows.

Constructed of essentially two zones using local materials, the 6,550-ft-long dam has a total volume of 684,000 cy (U.S. Bureau of Reclamation [USBR], 1981). As shown on Figure 2, the zones within the embankment consist of rolled impervious materials upstream and rolled gravel fill downstream. The cut-off trench utilizes a concrete and wooden sheet pile core wall backfilled with a fine-grained material placed by hydraulic methods (USBR, 1982). The downstream face of the dam slopes 2H to 1V, whereas the upstream embankment slope is 3H to 1V (USBR, 1981).

Keechelus Lake has a reservoir capacity of 158,800 acre-ft at maximum pool elevation of 2,517 ft. The spillway, which has a crest length of 302 ft, is a concrete-lined channel on the left abutment and has an uncontrolled overflow concrete weir. At elevation 2,521 ft, the spillway discharge capacity is 10,000 cfs.

Initially, downstream water was released through a multi-gated concrete-lined tunnel. Rehabilitation of the outlet works in 1978 replaced the antiquated cylinder gates with a single hydraulic slide gate. Maximum capacity of the outlet works is 3,750 cfs at reservoir elevation 2,521 ft (USBR, 1981). During the rehabilitation, the concrete tunnel was lined with reinforced concrete and a 22-in.-diameter conduit was installed to bypass minimum flows for fishery and stream enhancement when the regulating gate is closed.

SITE GEOLOGY

General

Exposed bedrock in the area of Keechelus Dam and Reservoir is a wide variety of early Tertiary units, principally volcanic flows, tuffs, and breccias with some interbedded sedimentary rocks. These geologic units were deformed by regional north-south tectonic compressional movements during several events in early and mid-Tertiary time. Evidence of this tectonic compression is the prominent Keechelus Ridge anticline to the northeast and other less prominent synclinal and anticlinal structures surrounding the area (Figure 3). Several faults have been mapped adjacent to the reservoir.

During Pleistocene time, recurring cycles of alpine glaciation modified the valley and mantled the bedrock surface with glacial drift. Topography was subdued, valleys were widened and deepened, and appreciable glacial drift was deposited in the form of moraines and outwash deposits. Holocene units consist of unconsolidated alluvium concentrated along drainages and local deposits of reworked glacial material, talus concentrations, and landslides.

Bedrock

The basement rock deeply underlying the Keechelus Dam area is the Easton Schist of pre-Tertiary age. Where exposed to the northeast and east of the reservoir, this unit consists of strongly metamorphosed greenschist and blueschist with local interbedded phyllite.

Essentially, two major bedrock assemblages bound the reservoir. The oldest is the Naches Formation (Frizzell et al., 1984) of Eocene age, consisting predominantly of rhyolite, andesite, and basalt flows, tuffs, and breccias with interbedded sandstone, siltstone, shale, conglomerate, and some coal seams. This unit occurs predominantly along the east and northwest rims of the reservoir (Figure 3). Lightly to moderately jointed vol-

Figure 1. Aerial view to the north of Keechelus Dam and Reservoir. The dam was constructed on a prominent glacial moraine of Lakedale Drift. Much of the embankment was placed as sliver sections mantled against the upstream side of the moraine. Interstate Highway 90 parallels the east rim of Lake Keechelus. U.S. Bureau of Reclamation photograph, July 30, 1968.

canic and some sedimentary rocks showing predominantly westerly dips are well exposed in road cuts along Interstate Highway 90 which traverses the eastern edge of the reservoir. A varied mixture of Naches Formation volcanic rocks underlies the left abutment and spillway structure. Bold exposures of rhyolite and andesite crop out immediately along the left side and downstream of the concrete-lined spillway channel.

The volcanic rocks of Huckleberry Mountain unconformably overlie the Naches Formation (Frizzell et al., 1984). This is the second major rock unit in the area and is well exposed along the southwestern shore of the reservoir. Andesite and basalt breccia and tuff predominate; andesite and basalt flows are abundant in basal parts. Within the Huckleberry Mountain volcanic unit, a thick tuff, called the tuff of Lake Keechelus (Frizzell et al., 1984) occurs as a massive interbed. Prominent exposures of this tuff are along both sides of the central portion of the reservoir. Both the Huckleberry Mountain volcanic rocks and tuff of Lake Keechelus are Oligocene and Miocene in age.

Within the thick assemblage of Tertiary volcanic, volcaniclastic, and sedimentary rocks around Keechelus Reservoir, several faults have been noted. One extends east-west, upstream of the dam, from Keechelus Ridge across the reservoir to Roaring Creek on the west side of the reservoir (Figure 3). Another fault is believed to trend northeast-southwest along the Gold Creek drainage at the upper end of the reservoir. This fault could be associated with the major Straight Creek fault zone to the northeast. Past reconnaissance studies have revealed no Holocene or Pleistocene movement on these structures.

Unconsolidated Materials

Major deposits of glacial drift occur at Keechelus Dam as well as downstream and within the reservoir area. Pleistocene ice advances greatly modified the upper Yakima River valley. Extensive quantities of glacial debris in the form of moraines, outwash, and lacustrine deposits mantle the lowland areas.

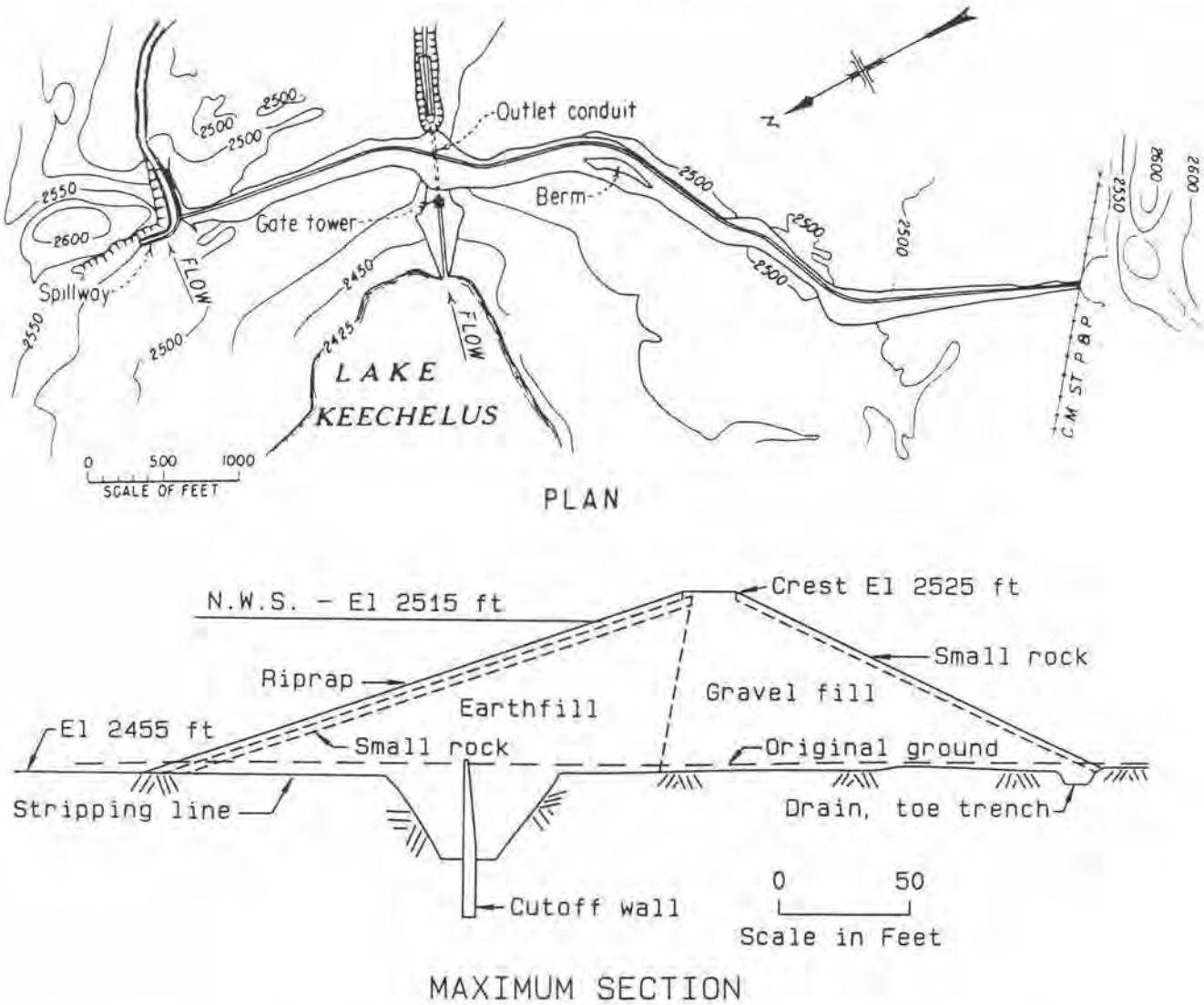


Figure 2. Plan and section of Keechelus Dam. Adapted from U.S. Bureau of Reclamation (1981).

The natural Lake Keechelus was formed behind a prominent moraine system composed of Lakedale Drift which is believed to correlate with Vashon Drift that was deposited during the last major Puget lobe advance west of the Cascades (Porter, 1976). The dam is founded on the highest moraine ridge within a complex of lower ridges that define a terminal moraine system. Heterogeneous mixtures of gravels, cobbles, and boulders within a matrix of clay, silt, and sand make up most of the moraine deposits at the dam. Localized interbeds of poorly stratified, gravelly to cobbly outwash and fine-grained lake deposits are also present. Nonglacial Holocene deposits include alluvium, reworked glacial drift, landslide deposits, and colluvium.

GEOLOGIC ASPECTS OF SITING AND DESIGN

Keechelus Dam is founded on a glacial moraine with the maximum section filling the breached area eroded by Yakima River downcutting. The dam was constructed to take advantage of the moraine ridge crest, with much of the embankment placed as sliver sections against the upstream side of the moraine. The subsurface conditions were explored by numerous test pits and drill holes. Records indicate that more than 46 hand-dug test pits were excavated to depths of 75 ft, and at least 32 exploratory holes were advanced to depths of 90 ft at the proposed site (U.S. Reclamation Service [USRS], 1906, 1912).

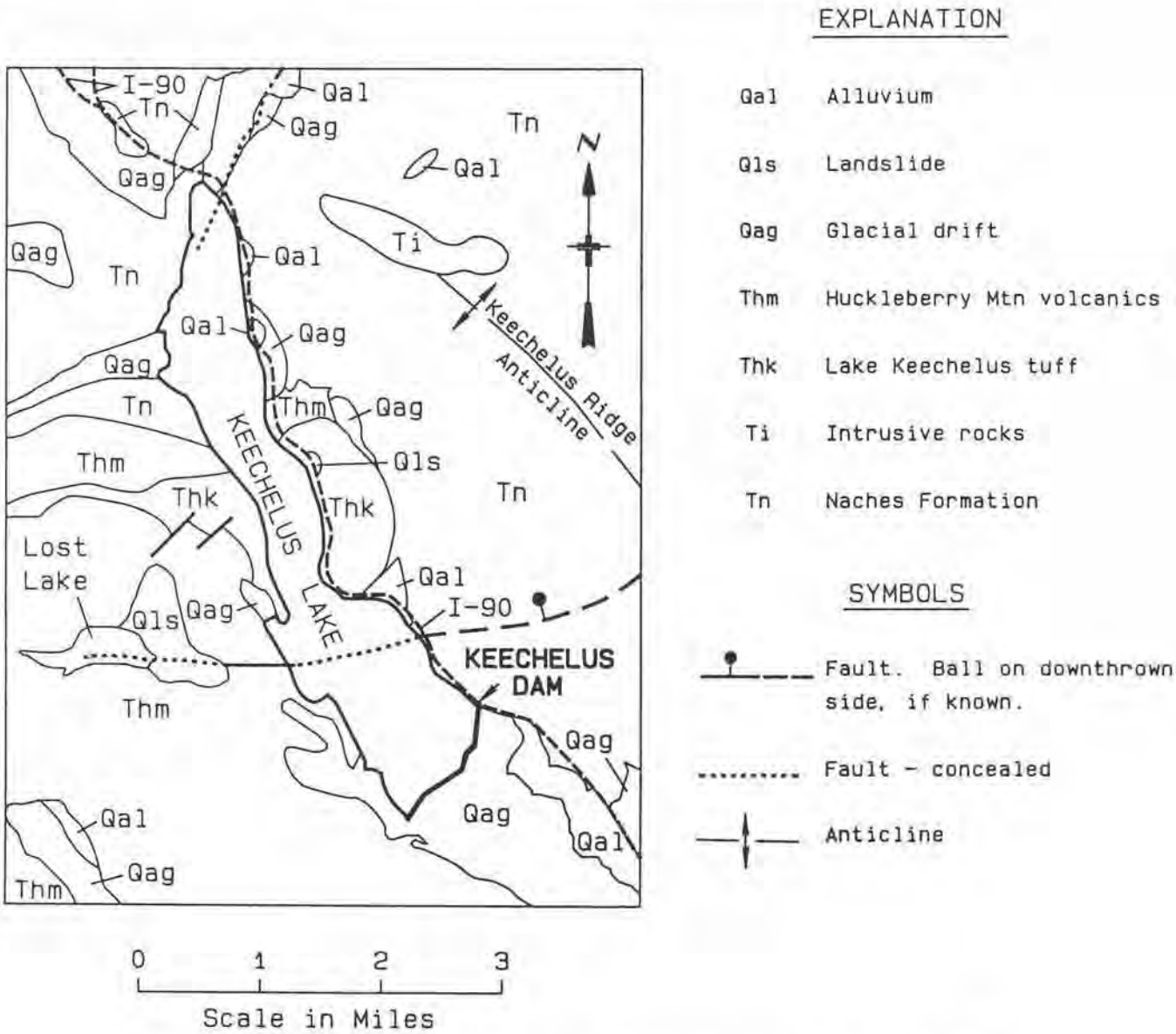


Figure 3. Generalized geologic map of the Keechelus Dam area. Adapted from Frizzell et al. (1984).

The north end of the moraine (left abutment area) abuts against a small hill exhibiting scattered exposures of hard, lightly jointed, moderately to lightly weathered volcanic rocks of the Naches Formation. Twenty-eight test pits whose depths range from 3 to 24 ft defined the buried bedrock surface in this area and assisted in the location for the spillway structure. Reworked gravelly and bouldery drift mantle the volcanic rocks in this area.

Test pit and boring records along the dam axis show varied depths of glacial materials consisting of heterogeneous, fairly impervious drift units and reworked outwash interbeds of semi-pervious to pervious sand and/or gravels with varying percentages of cobbles and boulders. From the left abutment, the *in situ* rock/glacial material contact drops rapidly to the south. No attempt was made to determine the depth of bedrock across the valley floor. The results of the explorations showed the moraine to be suitable for the foundation of Keechelus Dam. Apparently, it was determined that the drift was largely impervious, and major, potentially pervious interbeds would be cut off with the cut-off trench and planned core wall (Figure 2).

Initially, a timber crib structure was built to regulate flows from Lake Keechelus (USBR, 1982). In the area of the crib dam, excessive depths of "slimy ooze" and "muck" were noted on old drawings (USRS, 1906, 1912). The reservoir area was blanketed by various thicknesses of sediments which had been derived from

both glaciation and normal flood-plain deposition (USBR, 1983). This material was determined to be clay/silt-size fractions. It was rationalized at the time the dam was constructed that the soft ooze would effectively prevent excessive seepage though pervious portions of the moraine.

Explorations on the outer abutments located suitable borrow materials in the moraine. The North Borrow Pit was adjacent to the State Highway relocation, and the South Borrow Pit was situated next to the Chicago Milwaukee St. Paul & Pacific Railroad line on the right abutment.

Some seepage was anticipated downstream of the dam. Incomplete records indicate a toe drain was designed to be constructed under the toe of the embankment. Drawings show a trapezoidal trench 3 to 8 ft deep that was backfilled with free-draining gravel (USRS, 1915). Transverse tile drain pipe was installed on various centers 40 to 50 ft upstream of the trench in the foundation. These drains directed seepage into the toe drain. A toe drain outlet would control the seepage water downstream of the dam and monitoring would be at centrally placed weirs.

CONSTRUCTION PROBLEMS

Foundations

Preconstruction explorations indicated zones of pervious materials below the bottom of the cutoff trench excavation. A report by the USBR (1982) states:

> "Mention is made in project memorandums about founding the core wall 57 to 60 ft below the original river bed which is about 28 ft below the bottom of the outlet tunnel. Other records (Engineering News, Vol. 74, No. 18, October 28, 1916) indicate that the core wall was bottomed 31 ft below the outlet tunnel grade. It is reported that test holes drilled from the bottom of the core wall trench excavation at the level 3 ft below outlet tunnel grade encountered pervious sand under pressure at a depth of 6 ft. Additional test holes were drilled as much as 35 ft below the bottom of the core wall trench excavation and did not penetrate the pervious sand. It was considered impractical to extend the core wall through the pervious sand so the core wall excavation was thus stopped in the overlying gravelly clay."

Construction details of the core wall are lacking. Design drawings indicate that, for its entire length, it is founded on practically impervious gravelly clay (Figure 4). Whether the core wall was constructed beneath the total length of the dam is unknown. Also, the record is unclear as to the total depth or lowest elevation to which the core wall was extended (USBR, 1982).

It appears that during construction the pervious sand bed in the glacial foundation materials was believed to be of limited lateral extent or not in upstream contact with the lake water because of the protective layer of glacial "ooze" or "muck" known to cover the lake bottom.

During construction downstream of the sheet piling and concrete core wall, it was necessary to control inflow from several locations because of heavy springs and wet conditions (USRS, 1916). Rock-filled drain trenches, transverse to the dam centerline, were placed to carry the water to the longitudinal toe drain. To further insure adequate drainage during embankment placement, a rock drain was built at the intersection of the earthfill and gravel fill sections and was connected to the transverse drains (USRS, 1916).

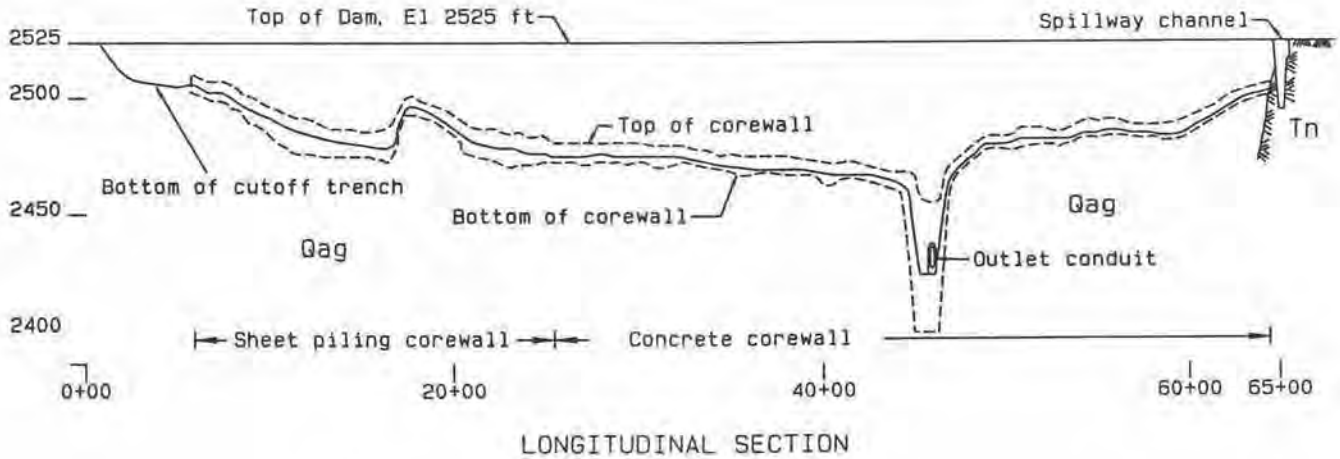


Figure 4. Longitudinal section of Keechelus Dam. Adapted from USRS (1915)

Spillway

Most of the spillway construction required rock excavation. Two 3-in. Woods air drills and two jack-hammer drills were used on two- and three-shift schedules. The more portable jack hammers were used for trimming rock highs and breaking up large, lightly jointed masses of rock. Most of the shot holes were from 8 to 12 ft in depth and were spaced about one-half their depth apart. Usually from 60 to 120 holes were shot at once, as it was found that the more holes were shot the less danger of losing holes near the edges of the blasting areas (USRS, 1916). Most of the small rock facing on the dam and the upstream riprap was obtained from the spillway excavation.

During construction of the spillway overflow weir, grout holes were drilled on 6- to 10-ft centers down to the grade of the downstream channel. The pipes were left projecting through the weir concrete. These holes were grouted at 80 psi with 116 batches of 1:1 grout (1 sack of cement and 1 cf of sand) to seal the foundation of the weir, which was in "very seamy rock" (USRS, 1916).

Borrow Materials

Adequate pervious and semipervious materials were obtained from the north and south borrow pits on the glacial moraine. Also, supplemental sand and gravel were obtained from the downstream river channel and outlet works channel excavations. Because of the extremely complex interlayering of suitable impervious materials in the initial borrow pits, great care was required to acquire satisfactory materials.

The desired quantity of fine-grained materials was not readily available in the North and South borrow areas, so another area downstream of the dam was found which had an appreciable amount of clay and silt. This area was called the East borrow pit and was opened up in the fall of 1915. Most of the excavation was done with a Marion 35-ton steam shovel loading horse-drawn dump wagons (USRS, 1916). The pit required shooting before shovel excavation because of the numerous boulders present. As noted in the Annual Project Report (USRS, 1916):

> "It was necessary to have the ground well shaken up or else the break-downs on the shovel became too frequent."

Test pit records indicate that high water-table conditions occurred within the borrow pits (USRS, 1912, 1916). The high moisture content of the material and the presence of free water required special handling and excavation techniques. For example, at the East borrow pit, the ground was so wet that it was impossible to load the drill holes with black powder, and the low-grade dynamite used was not nearly so efficient. "Coyote" holes were utilized effectively. Irregular holes were driven by hand, and it was possible to line the pockets with tar paper and load the black powder without removing it from the kegs. The coyote hole blasting, while incurring higher labor costs, has required less powder and produced much better breakage of the ground (USRS, 1916).

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Seepage

Several areas of seepage occur downstream of the dam. Most of the seepage is believed to originate through the foundation materials. The largest area of seepage occurs at the toe of the dam about 2,000 to 3,000 ft north of the right abutment and extends 2,000 to 3,000 ft beyond the toe of the dam. This area is characterized by bogs, standing water, and growths of water lilies and other swamp-type vegetation. Some of this water is believed to originate from Meadow Creek. Meadow Creek enters Keechelus Lake just above the right abutment. Yakima Project personnel believe water from the creek fan probably flows through and possibly around the right abutment (USBR, 1982).

A system of weirs installed below the dam many years ago has generally deteriorated. Seepage has not been a safety problem; periodic measurements and observations indicate that, overall, seepage has lessened over the years (USBR, 1983).

Reservoir Area

The only geologic problem within the reservoir basin was a landslide that occurred in 1957. This landslide is located on the east shore, about 2 mi north of the dam. The cause of the slide is unknown. Exposures along Interstate Highway 90 (I-90) show the slide to be 250 to 350 yd wide. The slide material consists of large, angular blocks and boulders of rock within a matrix of soil and sand-to-cobble-size fragments. This material is concentrated at the bottom of a deep bedrock ravine on the southwest flank of Keechelus Ridge. There is no evidence of recent instability; however, USBR (1982) reported that I- 90 requires periodic repair where it crosses the area.

Adjacent to the slide area (to the south), the Washington State Department of Transportation constructed a concrete snowshed over the west lanes of traffic for protection from snow and rock slides. Across the major part of the slide, a grid work of bolts, cabling, and wire mesh keeps large debris from falling onto the highway. Upslope of I-90, slope protection methods appear to have been effective in containing and controlling the slide mass. Downslope of the highway, as the toe of the landslide differentially settles into the soft sediments in Lake Keechelus, some movements could continue to impact the I-90 roadbed.

REFERENCES

Frizzell, V. A., Jr.; Tabor, R. W.; Booth, D. B.; Ort, K. M.; and Waitt, R. B., 1984, *Preliminary Geologic Map of the Snoqualmie Pass 1:100,000 Quadrangle, Washington*: U.S. Geological Survey Open-File Report 84-693, 42 p., 1 plate, scale 1:100,000.

Porter, S. C., 1976, Pleistocene Glaciation in the Southern Part of the Northern Cascade Range, Washington: *Geological Society of America Bulletin*, Vol. 87, pp. 61-75.

U.S. Bureau of Reclamation (Water and Power Resources Service), 1981, *Project Data*: U.S. Government Printing Office, pp. 1337-1356

U.S. Bureau of Reclamation, 1982, *Safety Evaluation of Existing Dams (SEED)–Report on Keechelus Dam*: U.S. Bureau of Reclamation, Denver, CO, pp. 27-31.

U.S. Bureau of Reclamation, 1983, *Technical Data Workbook, Keechelus Dam*: U.S. Bureau of Reclamation, Denver, CO, Vol. 1, 288 p.; Vol. 2, 527 p.

U.S. Reclamation Service, 1906, *Lake Keechelus Dam Site, Details of Drill Holes and Test Pits*: U.S. Reclamation Service Project Drawing No. 33-109-3000, January 3, 1906 (Available in the U.S. Bureau of Reclamation Yakima, WA, Office)].

U.S. Reclamation Service, 1912, *Keechelus Dam, Detail Map and Record of Test Pits*: U.S. Reclamation Service Project Drawing No. 33-109-3044, October 1912 [Available in the U.S. Bureau of Reclamation Yakima, WA, Office].

U.S. Reclamation Service, 1915, *Keechelus Dam, Plan, Profile, and Sections of Dam*: U.S. Reclamation Service, Project Drawing No. 33-109-3072, January 1915 [Available in the U.S. Bureau of Reclamation Yakima, WA, Office].

U.S. Reclamation Service, 1916, *Annual Project Report for Season 1916*: U.S. Reclamation Service, pp. 28-66 [Available in the U.S. Bureau of Reclamation Yakima, WA, Office].

A Marion 35-ton shovel at work excavating the east borrow pit at Keechelus Dam. Materials were loaded on horse-drawn dump wagons. The pit was shot ahead of the shovel because of the presence of boulders. A cyclone well drill was used to put down the holes; later, coyote holes were used. Because most of the material in the area was wet, corduroy or planked roads were commonly used. U.S.Bureau of Reclamation photograph, July 24, 1916.

Steam shovel loading riprap-size shot rock into rail cars. This view, to the west, shows the base-level excavation in the spillway channel at Keechelus Dam. U.S. Bureau of Reclamation photograph, September 29, 1916.