The Cowlitz River Projects

Aerial view of Mossyrock reservoir (Riffe Lake) and the valley of the Cowlitz River; view to the northeast toward Mount Rainier. Photograph by R. W. Galster, July 1980.

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

The Cowlitz River Projects: Mayfield and Mossyrock Dams

HOWARD A. COOMBS
University of Washington

PROJECT DESCRIPTION

The Cowlitz River has its origin in the Cowlitz Glacier on the southeastern slope of Mount Rainier. The river flows southward, then turns toward the west and passes through the western margin of the Cascade Range in a broad, glaciated basin. It is in this stretch of the river that both Mossyrock and Mayfield dams are located. Finally, the Cowlitz River turns southward and enters the Columbia River at Longview.

Mayfield Dam, completed in 1963, is 13 mi downstream from Mossyrock Dam, constructed 5 yr later. Both are approximately 50 mi due south of Tacoma.

Mayfield Dam is a composite, thin wall, single curvature arch and gravity dam (Figures 1 and 2). The thin wall arch is founded on hard andesite for the lower half of the structure. The upper half rests against massive thrust blocks founded on andesite. The left, or south gravity structure contains a five-gate spillway. The powerhouse is located 1,300 ft downstream on the north bank and is connected to the dam by a horseshoe-shaped concrete-lined power tunnel leading to the forebay. The water is then led to the powerhouse through four 18-ft-diameter penstocks (Figure 1).

In 1926 the Bakus-Brooks Company of Minneapolis explored the site and did some core drilling. In 1948 they sold their rights to the project to the City of Tacoma. In 1936 and again in 1948 the U.S. Geological Survey conducted reconnaissance mapping in the general area (Erdman and Bateman, 1951).

During the initial investigation by the City of Tacoma in 1955, a core drilling program was started to augment the information gained earlier. The first construction contract was terminated in 1957 due to litigation with state agencies regarding fish migration. In 1960 another contract was awarded, and the dam completed 3 yr later.

Mossyrock Dam is the largest of the Tacoma dams. The size of the dam is not easily appreciated as only 365 ft shows above the river; the height above bedrock is 606 ft. The dam is a double curvature concrete arch 1,648 ft long, 27 ft wide at the top, and 116 ft wide at the base. A two-unit powerhouse (with room for a third unit) is at the toe of the dam on the north bank. The reservoir is 23.5 mi long. By impounding more than 1,600,000 acre-ft of water in the reservoir, the output of Mayfield Dam was greatly enhanced (Figures 3 and 4).

AREAL GEOLOGY

The southern Cascades of Washington are composed essentially of volcanic and sedimentary rocks that have been intruded by many dikes and sills and by small batholiths and stocks of dioritic composition, as well as plugs of andesite and basalt. Most of these rocks range in age from late Eocene to Miocene (Hammond, 1963; Hammond et al., 1977). The exceptions are the young stratovolcanoes of Mount Rainier, Mount Adams, and Mount St. Helens.

Most of the rocks along the Cowlitz River consist of pyroxene andesites, basalts, sandstones, shales, mudflows, and breccias. The majority of these units are of Eocene and Oligocene age (Figure 5). The total thickness of these units is approximately 2 mi, according to Fiske et al. (1963), who gave these rocks the name Ohanapecosh Formation (ΦEV on Figure 5). They consider the bulk of the formation to consist of volcaniclastic rocks and lava flows. Most of the lavas have been pervasively altered, and, as a result, a reddish or greenish color has been imparted to large areas of the formation. Vance and Naeser (1977) dated the Ohanapecosh as Oligocene by radiometric methods. Due to great variations in the Ohanapecosh Formation it is difficult to correlate extensions from the type locality in the southeast corner of Mount Rainier National Park.

Another formation in the Cowlitz Valley is the Stevens Ridge (included in unit ΦEV), named by Fiske et al. (1963) from the type locality along the southern boundary of Mount Rainier National Park. Within the park it varies from 450 to 3,000 ft in thickness but is much thicker elsewhere. It consists of ash flows and volcaniclastic rocks. Most of these rocks contain quartz and generally are light in color, characteristics that help to distinguish them from the Ohanapecosh below and the Fifes Peak Formation above. The Stevens Ridge Formation is in most places separated from stratigraphically adjacent units by a marked unconformity. Its age ranges

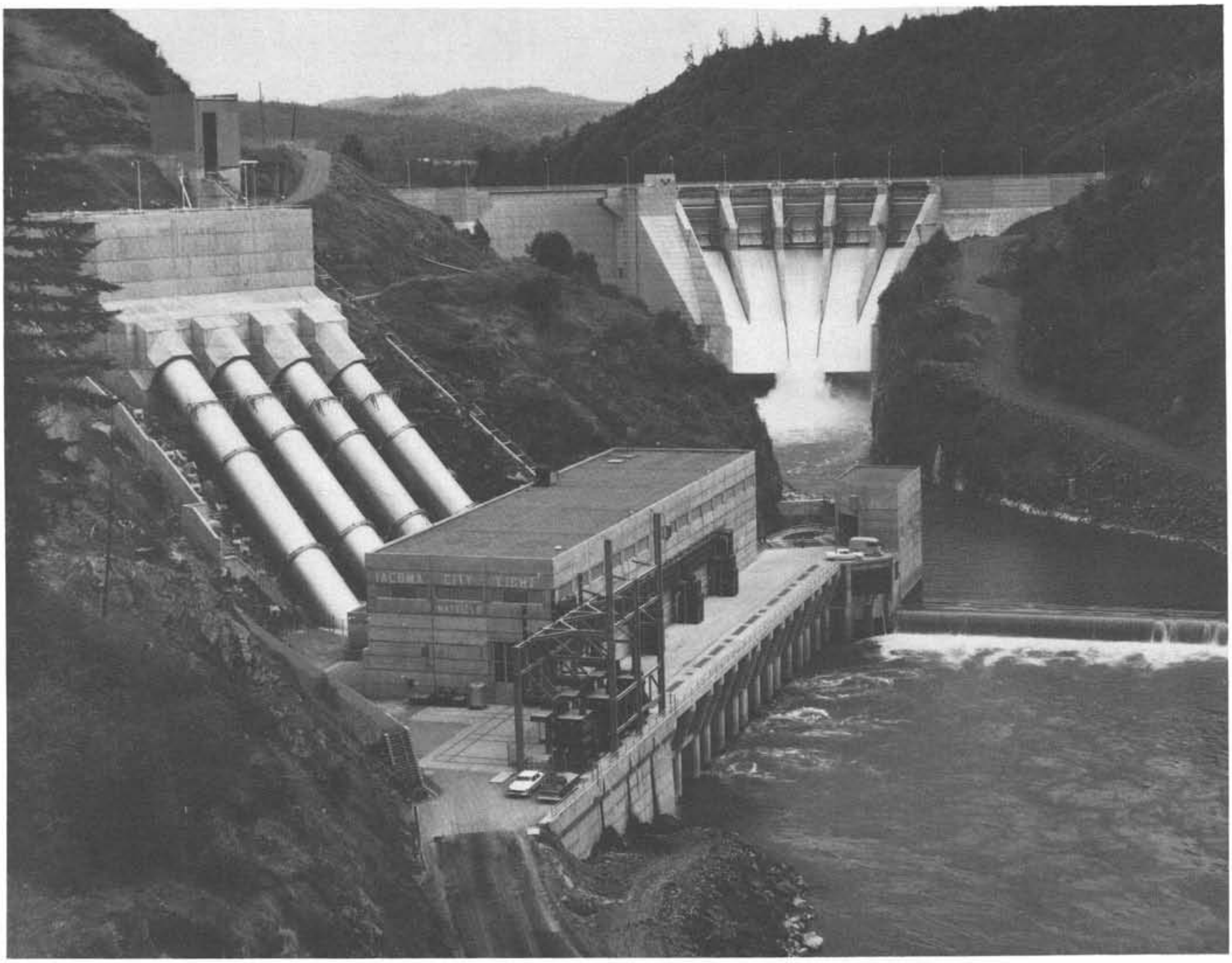


Figure 1. Mayfield Dam, spillway, forebay, penstock, and powerhouse. Photo by Tacoma City Light.

from middle Oligocene to early Miocene, according to Vance and Naeser (1977).

The third unit in this widespread volcanic sequence is the Fifes Peak Formation (unit Mv in Figure 5), named by Warren (1941) from outcrops east of Mount Rainier. It is composed mainly of dark brown to gray lava flows of platy jointed andesites and basalts with interbeds of tuff-breccia and volcanic-derived sandstones. The thickness reaches 1,500 ft. The age is regarded as early to middle Miocene (Vance and Naeser, 1977).

Intruding both the Ohanapecosh and Stevens Ridge formations are hundreds of basalt and diabase dikes and sills. In Mount Rainier National Park, Fiske et al. (1963) regarded these as a hypabyssal equivalent of the Fifes Peak rocks, based on mineralogical similarity. The intrusive bodies cut the Stevens Ridge Formation but are cut by offshoots of the dioritic plutons.

During the mid-Miocene this entire volcanic and sedimentary sequence was folded, with some faulting, into a series of anticlines and synclines trending from north-south to northwest-southeast.

During the Pleistocene several alpine glaciers moved down the Cowlitz River valley. The till left at the Mayfield site is coeval with brown till (probably Hayden Creek) encountered at the Mossyrock site. In addition to till there are outwash gravels and lacustrine sediments, all well exposed in the Mayfield switchyard area. Large, ice-rafted boulders can be seen in the powerhouse access road just downstream of the powerhouse.

A silt-loam loess overlying the Hayden Creek till (Crandell and Miller, 1974) blankets the entire area for tens of miles around the projects. It varies from a few feet to 20 ft in thickness. This silt can be seen overlying outwash gravels which, in turn, overlie a lava flow well exposed along the Mayfield dam access road.

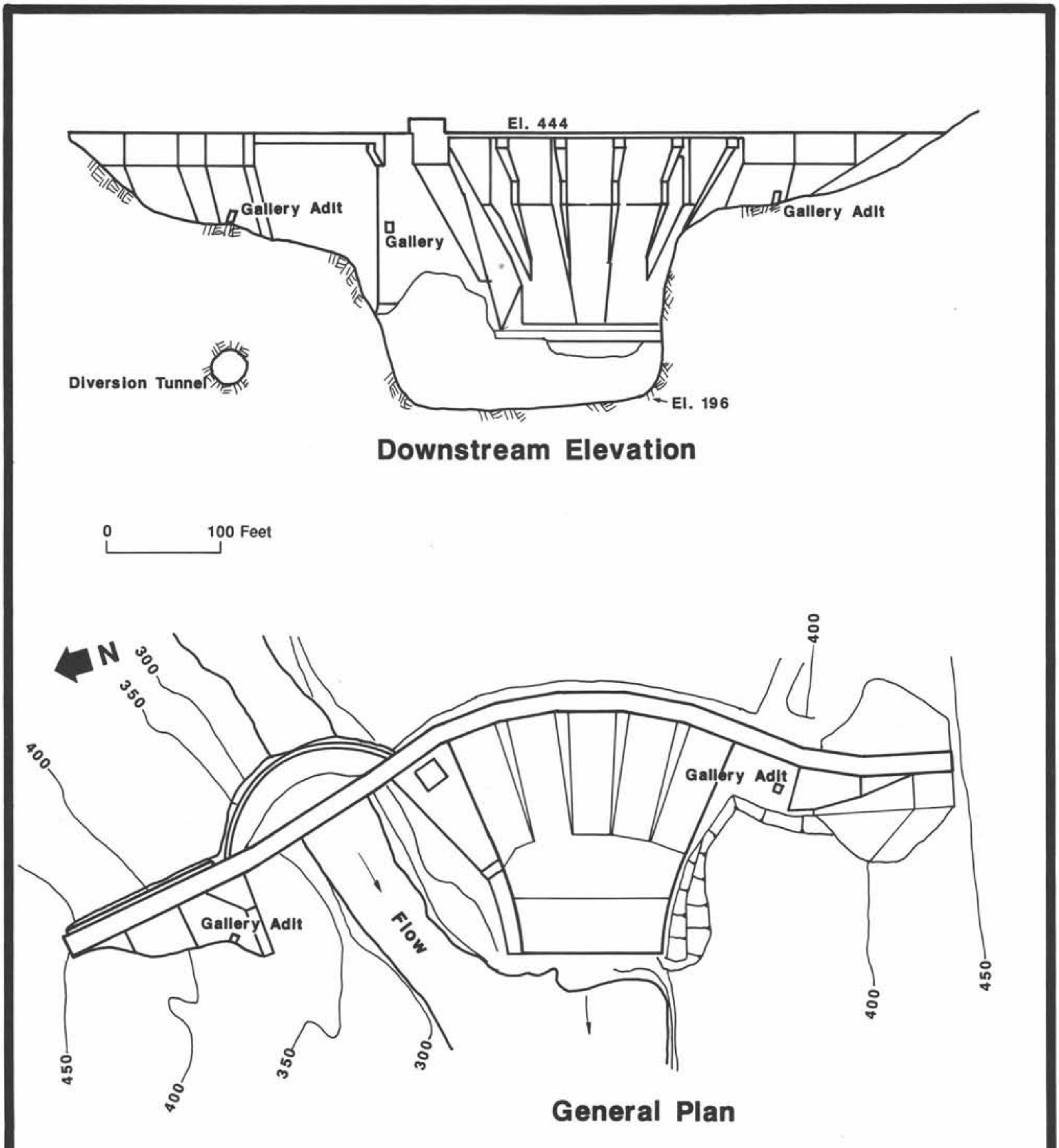


Figure 2. Mayfield Dam and spillway, plan and section; view upstream.

Figure 3. Mossyrock Dam and powerhouse. Note roughly horizontal layers of basalt, breccias, and sedimentary rocks along road cuts on the left side of the photo. Photo courtesy of Tacoma City Light.

MAYFIELD DAM

Dam Site Geology

In the vicinity of the Mayfield project the Tertiary rocks are folded into a series of anticlines and synclines. The Cowlitz River cuts across this structural pattern and forms a series of box canyons where the anticlines are breached and wide, alluvium-filled structural basins in the synclines.

The axis of Mayfield Dam is located approximately on the crest of a northwest-trending anticline to take advantage of the excellent rock exposed in the box canyon and to utilize the dip slope on the western flank of the anticline for the spillway. The arch, spillway, and abutments are situated on an anticlinal axis that plunges 30° to the southeast (Coombs, 1957). The powerhouse, forebay, and other appurtenant structures are located on the western, or downstream, flank of the anticline.

Bedrock units at the Mayfield Dam site consist of more than 900 ft of intercalated lava flows, breccia, sedimentary tuff, shale, and sandstone. The lower part of the arch is in the andesite, the power tunnel is in the shale and sandstone (Coombs, 1948, 1961; Butler, 1965). Overlying the shale and sandstone unit is a basalt sequence varying from 50 to 240 ft in thickness. Figure 6 shows the stratigraphic sequence at the forebay, penstock, and powerhouse.

Minor faults along the dam axis seem to be more of an accommodation along joints than definite shears. Downstream of the dam the fault zones are wider and have an effect on the stability of the canyon walls.

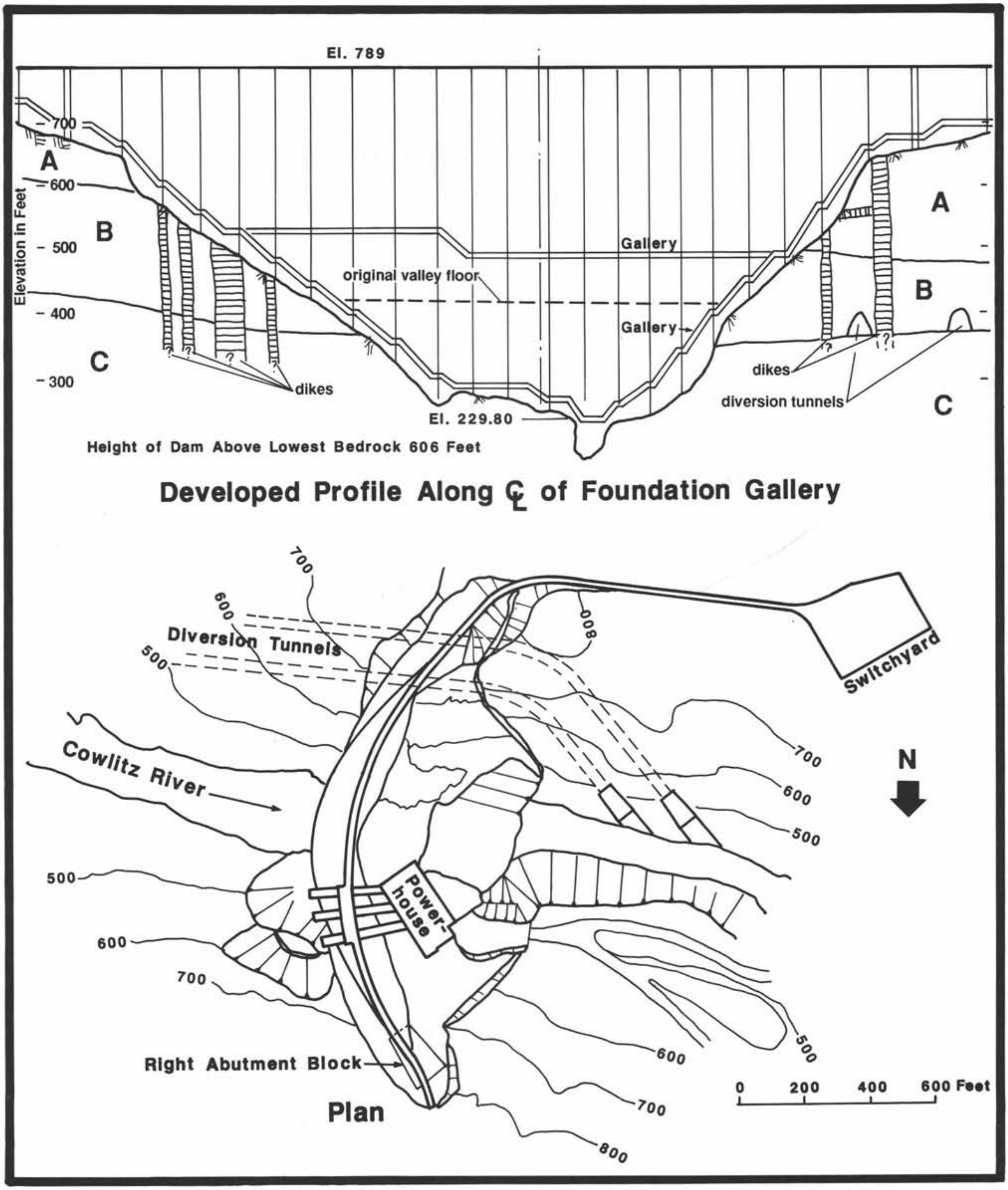


Figure 4. Mossyrock Dam plan and elevation; view upstream. The letters A, B, and C refer to lithologic units described in text.

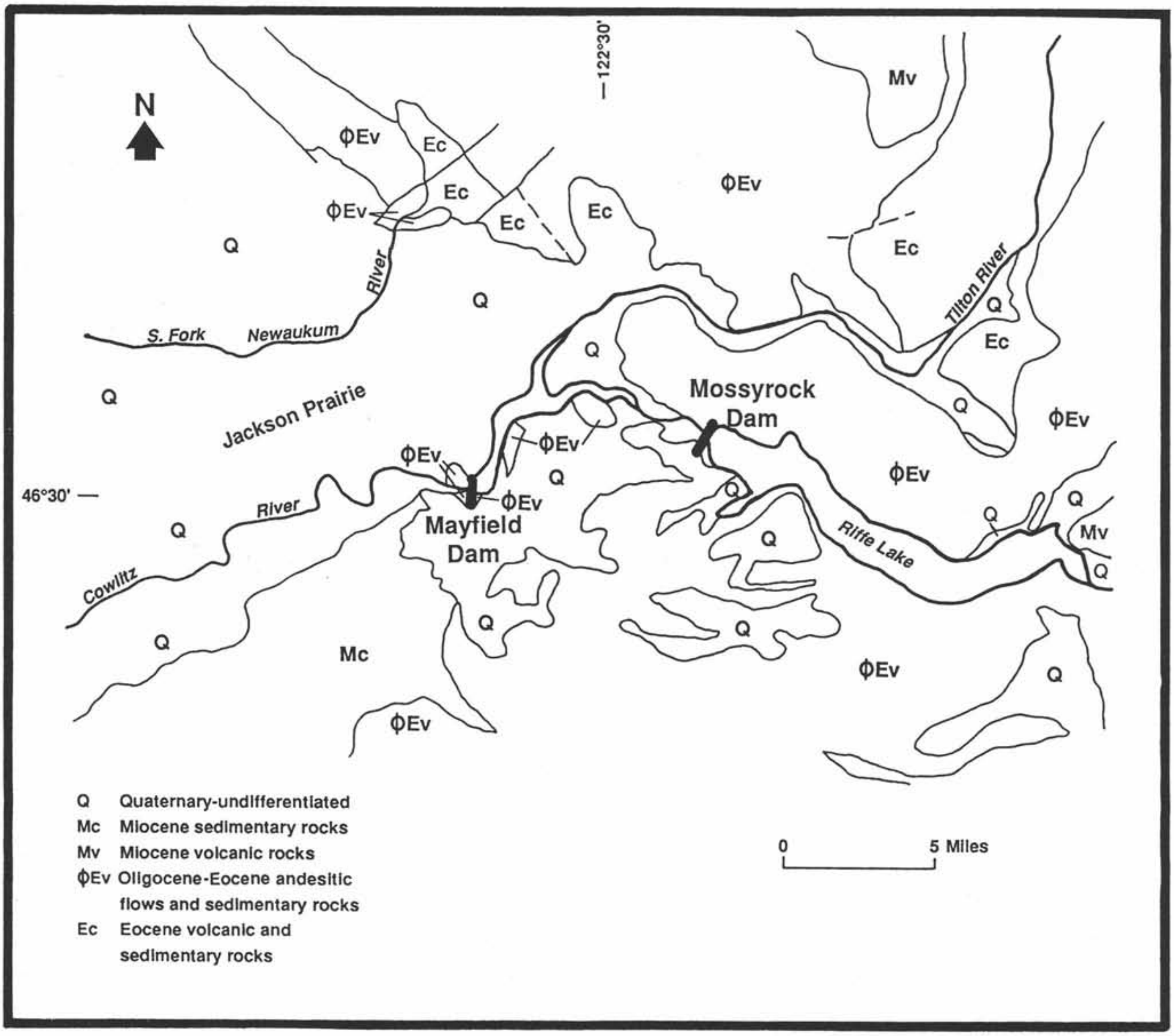


Figure 5. General geologic map of the Mossyrock and Mayfield Dam area. (After Walsh et al., 1987).

Slides

Slope stability was a problem during the construction of Mayfield Dam. Three slides took place between the dam and the powerhouse. It will suffice to describe one since the others are quite similar, though much smaller.

The trace of a fault extends from above the forebay down the penstock slope and bisects the powerhouse area. The fault is not large—offset is less than 10 ft, and the fault zone itself is only a few feet wide. The real culprit contributing to the slide is a thin tuffaceous shale overlying sandstone and covered by basalt layers. All dip 30° into the canyon and strike parallel to the river (Figure 7). The tuffaceous shale is soft and disintegrates when exposed. The thin basalt flow above the shale is highly fractured and quite permeable, permitting water to reach the shale below. When wet, the shales have a high degree of plasticity.

This slide area had a long history of instability, and a large volume of talus and debris material had accumulated at the base of the slide area. A cut was made in the slide approximately half way up the slope for an access road to the powerhouse (Figure 7). This road was also widened later and even filled below grade when the road slumped. Water collected in the ditch on the uphill side of the road and was not carried beyond the slide area. Another cut was made at the base of the slide by removing the talus and then cutting into bedrock for the powerhouse foundation (Figure 6). The powerhouse excavation removed support for both the sandstone-shale

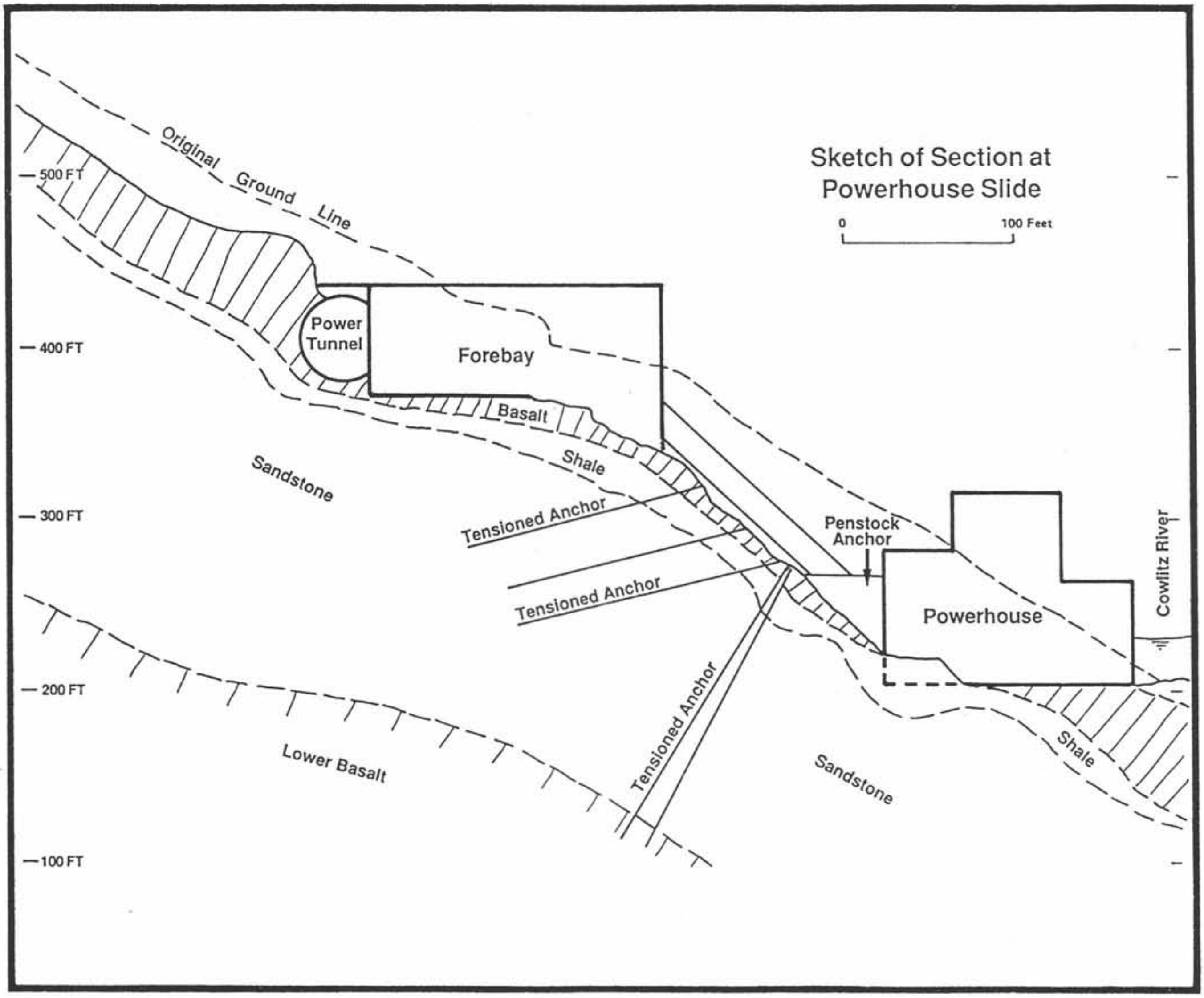


Figure 6. Generalized geologic section at Mayfield powerhouse. Photo by Tacoma City Light (Butler, 1965).

sequence and the upper basalt. This combination of events, together with heavy rains for several days, triggered a rock and mud slide on this 30° slope (Coombs, 1961). So obvious was the instability that just prior to the slide failure all construction was stopped in the penstock/powerhouse area and the head tower carrying the cable across the canyon was moved along its tracks to safer ground.

Remedial measures to prevent further slides included scaling back the basalt cliffs above the slide, removal of some of the basalt above the shale-sandstone layer, and construction of a high crib retaining wall to protect the powerhouse. Construction continued until the penstock, powerhouse, and forebay were completed. At this time there was concern that filling the forebay with water might exert too much of a load on the penstock slope. The decision was made to tie the rock mass in the penstock slope together with tendons.

The plan to stabilize the slope was conservative. A series of 75 holes was drilled into the slope at angles ranging from 0°-15° to the horizontal, dipping into the slope. The holes were 150 ft deep, sufficiently long to penetrate basalt and shale-sandstone; a few reached into the basalt below the sandstone. The tendons were 1/4-in. rods, 90 in a hole, with button head tops. The bottoms were firmly anchored into the rock with grout. At the surface, precast concrete jacking pads were placed over the holes and grouted to the rock surface. After the anchor grout hardened, the tendons were tensioned by hydraulic jacks to 650 kips. The steel rods were stretched 8 in. during the jacking. The hydraulic jack pads can be seen in the photograph of Mayfield Dam

Figure 7. General view of the landslide in the area of the Mayfield Dam, powerhouse, and penstock. The actual slide surface can be seen just to the left of the penstock excavation. The power tunnel portal emerges into the forebay area. Photo by Tacoma City Light.

and powerhouse (Figure 1) on the left side of the nearest penstock.

Since the installation of the tendons, the slope has been monitored, recording both horizontal and vertical components of motion. All drains have been kept open, and the piezometers have been maintained in good condition. Inclinometer installations are used to detect movement of the slope. To date there has been no indication of movement, and with this substantial treatment, none is expected.

MOSSYROCK DAM

Dam Site Geology

Overburden

For most of its length the Cowlitz River valley has a broad bottom typical of those modified by alpine glaciers (Erdman and Bateman, 1951; Coombs, 1948). These glaciers, 1,000 or more ft in thickness, reamed out the former river channel and overdeepened the bedrock bottom. At the axis of the dam the glacial sediments are 225 ft thick in the canyon bottom. The lower 170 ft of this is a sandy gravel, cobbles, and boulders as much as 14 ft in diameter. The upper 55 ft is a hard, impervious till. The glacial sequence at the site suggests at least two glacial advances down the valley. The older, harder till (probably Wingate Hill; Crandell and Miller, 1974) has been contorted by later ice advances. The younger, softer, brown till (Hayden Creek) rests on the older till high on the canyon walls. These tills and associated alluvial sediments were effective in contributing to the imperviousness of the reservoir basin.

Loess, deposited contemporaneously with the recession of the last (Hayden Creek) glaciation, formed beds as much as 25 ft in thickness over much of the Cowlitz and neighboring valleys. Alluvium in the form of stratified terrace sands and gravels was deposited on most of the broad valley floor near the dam.

Bedrock

The rocks upon which the arch dam, thrust blocks, and wing walls rest can be divided into three major units (Figure 4). The uppermost unit, 200 ft thick, is composed mainly of porphyritic andesite with thin beds of flow breccia. This unit (A of Figure 4) forms the foundation for the thrust block and wing walls only. The rock is good except for local spots where joints were filled with alteration products.

The 200-ft-thick middle unit (B) is mostly amygdaloidal basalt and porphyritic andesite with thin layers of flow breccia. Much of the basalt in this unit is susceptible to air slaking on exposure and thus posed a problem during construction. Approximately 20 percent of the arch dam rests on this middle unit.

The lowest rock unit (C) is 300 ft thick and is composed of dense to massive platy andesite with minor lenses of flow breccia. This is the best rock at the site and supports the high, central portion of the concrete arch. Dikes and sills of basalt cut through both abutments (Butler and Engstrom, 1968).

The lower portion of the terrace immediately upstream of the dam was used as aggregate for Mossyrock Dam concrete. Since the character of the aggregate varied considerably, frequent testing of the material was necessary to insure the high quality of the concrete in the dam.

Special Treatment of Foundation and Abutments

Concern for leakage through possible by-pass channels around the dam resulted in the construction of three drainage adits in the rock abutments just downstream of the dam. Two adits, 6 ft wide and 7 ft high, were driven into the right abutment, one 568 ft long and the other 397 ft long. The one on the left abutment was 2,117 ft long. The left abutment adit was in the uppermost rock unit described above (unit A). The right abutment adits were in the middle unit (unit B). No by-pass channels were found. It is interesting to note that two decades after construction of Mossyrock Dam, the abutments are regarded as exceptionally tight.

Exploratory drilling for the powerhouse showed a steep slot in the foundation surface. Upon excavation this slot was found to be controlled by platy andesite. The plates were approximately 1 in. thick, standing vertically, and parallel to the steep slope. The stability of this platy rock was enhanced by installation of 65 high-strength anchors, each 40 ft long, tensioned to 132.5 kips, and then backfilling the slot with concrete.

Extensive open joints in the foundation for the penstocks also required remedial treatment. Here, 25 high-strength anchors, 40 ft in length, were used to pin together the slabby rock and then tensioned to 132.5 kips.

The plunge pool is in the lowest rock unit, but the platy nature of the joints and some relief joints behind the left abutment indicated strengthening was needed against erosion during heavy spills. Rock faces were pinned normal to the major jointing system and held by 42 rock anchors tensioned to 132.5 kips. The face was covered by 8,000 cy of concrete, with a minimum of 15 in. of concrete over the surface. The armor has been satisfactory.

The actual height of the dam is more than 600 ft. Due to this height and the jointed nature of the lavas, it was necessary to provide an exceptionally complete grout curtain. The grout curtain, together with the incipient alteration of some of the lavas and volcaniclastic rocks, provide exceptionally tight foundation and abutments. The grout curtain was emplaced beneath the entire length of the dam to depths of 200 ft under the central arch section and progressively shallower on the abutments.

A small portion of the arch dam rests on the middle rock unit containing amygdaloidal basalt. During exploration for the site, the drill cores taken from this unit looked hard but after a few days in the core boxes they had slaked into small bits. In order to protect the rock in the abutments from slaking during excavation and before concrete was placed, specifications called for this material to be excavated 2 ft above grade and sprayed with a plastic material. Prior to pouring concrete the rock surface was sandblasted to remove all plastic spray material and dust and excavated the final 2 ft to grade.

ACKNOWLEDGMENTS

The writer acknowledges the cooperation of T. Coats, Director of Utilities, Department of Public Utilities, City of Tacoma. He and his staff, especially L. H. Larson, Chief Civil Engineer, have provided much of the background and illustrative material included in this discussion of Tacoma's dams.

C. D. Butler and J. Engstrom provided invaluable information on geology of Mossyrock Dam as they made their report at the conclusion of the project while all details were fresh in their minds.

REFERENCES

Butler, C. D., 1965, *Geology of Mayfield Dam*: Report to City of Tacoma, Department of Public Utilities, Light Division, Tacoma, WA, 57 p.

Butler, C. D. and Engstrom, J., 1968, *Final construction report of the Foundation Department*: Report submitted to the Resident Engineer for City of Tacoma, Department of Public Utilities, Light Division, Tacoma, WA, 75 p.

Crandell, D. R. and Miller, R. D., 1974, *Quaternary stratigraphy and extent of glaciation in the Mount Rainier Region, Washington*: U.S. Geological Survey Professional Paper 847, 59 p., 2 plates.

Coombs, H. A., 1948, *Geologic summary of Mayfield and Mossyrock Dam sites, Cowlitz River, Washington*: Report to City of Tacoma, Tacoma, WA, 5 p.

Coombs, H. A., 1957, *Geologic report on the foundation at the Mayfield hydroelectric project, Cowlitz River, Washington*: Report to City of Tacoma, Tacoma, WA, 19 p.

Coombs, H. A., 1961, *Geologic report on the rock slide conditions at the Mayfield hydroelectric project*: Report to City of Tacoma, Tacoma, WA, 10 p.

Erdman, C. E. and Bateman, A. F., Jr., 1951, *Geology of dam sites in southwestern Washington–Part II, Miscellaneous dams on the Cowlitz River above Castle Rock, and the Tilton River, Washington*: U.S. Geological Survey open-file report, 314 p., 20 plates.

Fiske, R. S.; Hopson, C. A.; and Waters, A. C., 1963, *Geology of Mount Rainier National Park, Washington*: U.S. Geological Survey Professional Paper 444, 93 p.

Hammond, P. E., 1963, *Structure and stratigraphy of the Keechelus volcanic group and associated Tertiary rocks of west-center Cascade Range, Washington* [Ph.D. thesis]: University of Washington, Seattle, WA, 264 p.

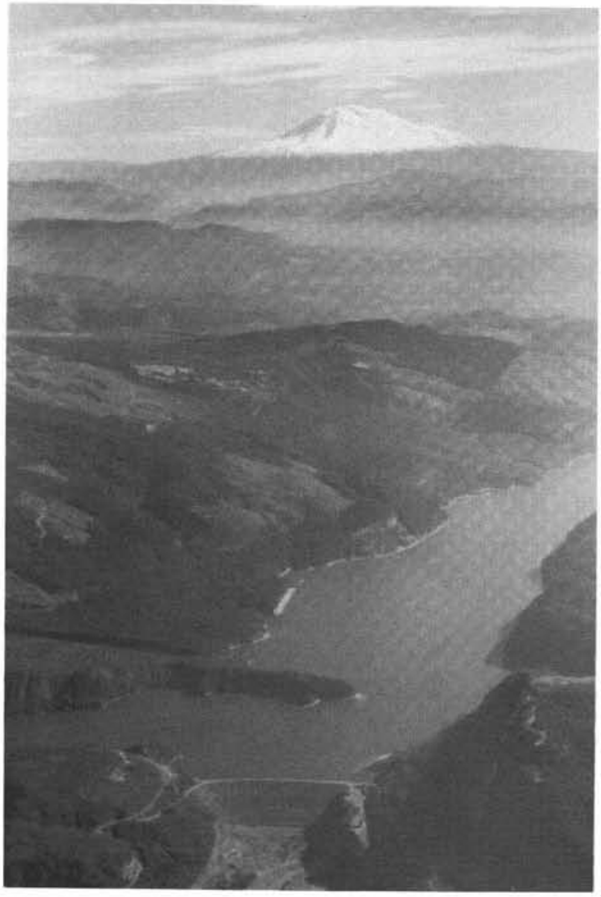
Hammond, P. E.; Bentley, R. D.; Brown, J. C.; Ellingson, J. A.; and Swanson, D. A., 1977, *Volcanic stratigraphy and structure of the central Cascade Range, Washington*. In Brown, E. H. and Ellis, R. C. (editors), Geological Excursions in the Pacific Northwest: Western Washington University, Bellingham, WA, 414 p.

Vance, J. A. and Naeser, C. W., 1977, *Fission track geochronology of the Tertiary volcanic rocks of the central Cascade Mountains, Washington* [abstract]: Geological Society of America Abstracts with Programs, Vol. 9, No. 4, p. 520.

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.

Warren, W. C., 1941, *Relation of the Yakima basalt to the Keechelus andesitic series*: Journal of Geology, Vol. 40, pp. 795-814.

Lewis River Projects



Swift Dam and reservoir; view northeast toward Mount Adams. The peninsula extending into the reservoir is underlain by part of the "Swift Creek Assemblage", lavas from an ancestral Mount St. Helens. Photograph by R. W. Galster, October 1981.

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

Lewis River Projects

WILLIAM S. BLITON
ATC Engineering Consultants, Inc.

INTRODUCTION

The Lewis River is a major drainage on the western slope of the Cascade Range in southwest Washington. The river flows along the southern flank of Mount St. Helens for 94 mi before emptying into the Columbia River on its way to the Pacific Ocean. The headwaters of the Lewis River are at Adams Glacier on the northwest flank of Mount Adams. The river has a fall of about 7,900 ft from its glacial source to its mouth, an average gradient of about 84 ft/mi. The total drainage area of the Lewis River and its tributaries, excluding the East Fork River, is about 860 sq mi. A location map and general map of the Lewis River basin are presented in Figure 1.

A large part of the tributary discharge is supplied by the tributaries that drain the eastern and southern slope of Mount St. Helens, the Muddy River, and Pine and Swift creeks. Most of the eastern slope of Mount St. Helens is drained by tributaries of the Muddy River, which join the Lewis River approximately 1 mi upstream of Swift Reservoir. Pine Creek drains the southeastern flank of Mount St. Helens and empties into the Lewis River 1/2 mi upstream of Swift Reservoir. Mount St. Helens' southern flank is drained by Swift Creek, which empties into Swift Reservoir near Swift Dam.

The development of hydroelectric facilities on the Lewis River is attributed to L. T. Merwin of Northwestern Electric Company, when that company was an affiliate of Pacific Power and Light Company, and Inland Power and Light Company. In 1919, Merwin initiated surveys for the comprehensive study of the Lewis River watershed and its hydroelectric possibilities for additional power sources to serve the Portland, Oregon, area. The study identified several potential dam sites, including Ariel (presently known as Merwin), Basket (presently known as Yale), Swift, and Muddy Creek. An application was filed with the Federal Power Commission in November 1928 by Inland Power and Light for a preliminary permit for an investigation of comprehensive development of the Lewis River, and studies were proposed for the previously mentioned four dam sites. The Federal Power Commission issued a preliminary permit for detailed investigation of the Ariel (Merwin) and Basket (Yale) sites in August 1929.

During the same month, Inland Power and Light filed an application with the Federal Power Commission for construction at the Merwin and Yale sites. The Federal Power Commission approved construction of the Merwin Project on October 16, 1929, and construction commenced the following day. The Merwin site was favored over the Yale site as it was approximately 12 mi closer to the rail head at Woodland. The Merwin Hydroelectric Project was completed on October 16, 1931, exactly 2 yr after startup, at a cost of about $8,600,000.

The next site to be developed was the Yale Project, where construction was started in early March 1951 under an accelerated schedule. The main dam was completed in early August 1952, and the entire project was completed in 1953.

The last of the Lewis River projects to be completed was the Swift Hydroelectric Project. Construction at the site commenced in May 1956 and was completed in December 1958 at an approximate cost of $55,491,000.

GEOLOGIC SETTING

Stratigraphy

The rocks of the Cascade Range in the Lewis River area can be divided into four groups. These are here termed, from oldest to youngest, Western Cascade Group (WCG), intrusive rocks, High Cascade Group (HCG), the Troutdale Formation and surficial deposits. No pre-Tertiary rocks have been mapped in the Lewis River area, and only a few intrusive bodies are present, mostly small granodiorite stocks. The geology of the Lewis River basin is presented in Figures 2a through 2d (from Tilford and Sullivan, 1981).

The WCG consists mostly of volcanic strata of Eocene to early Pliocene age. This group is divided stratigraphically into lower and upper parts on the basis of regional unconformities (Hammond, 1980).

The lower part of the WCG, or LWCG, underlies the entire Lewis River area and includes the oldest strata (middle Eocene to early Oligocene) exposed in the area.

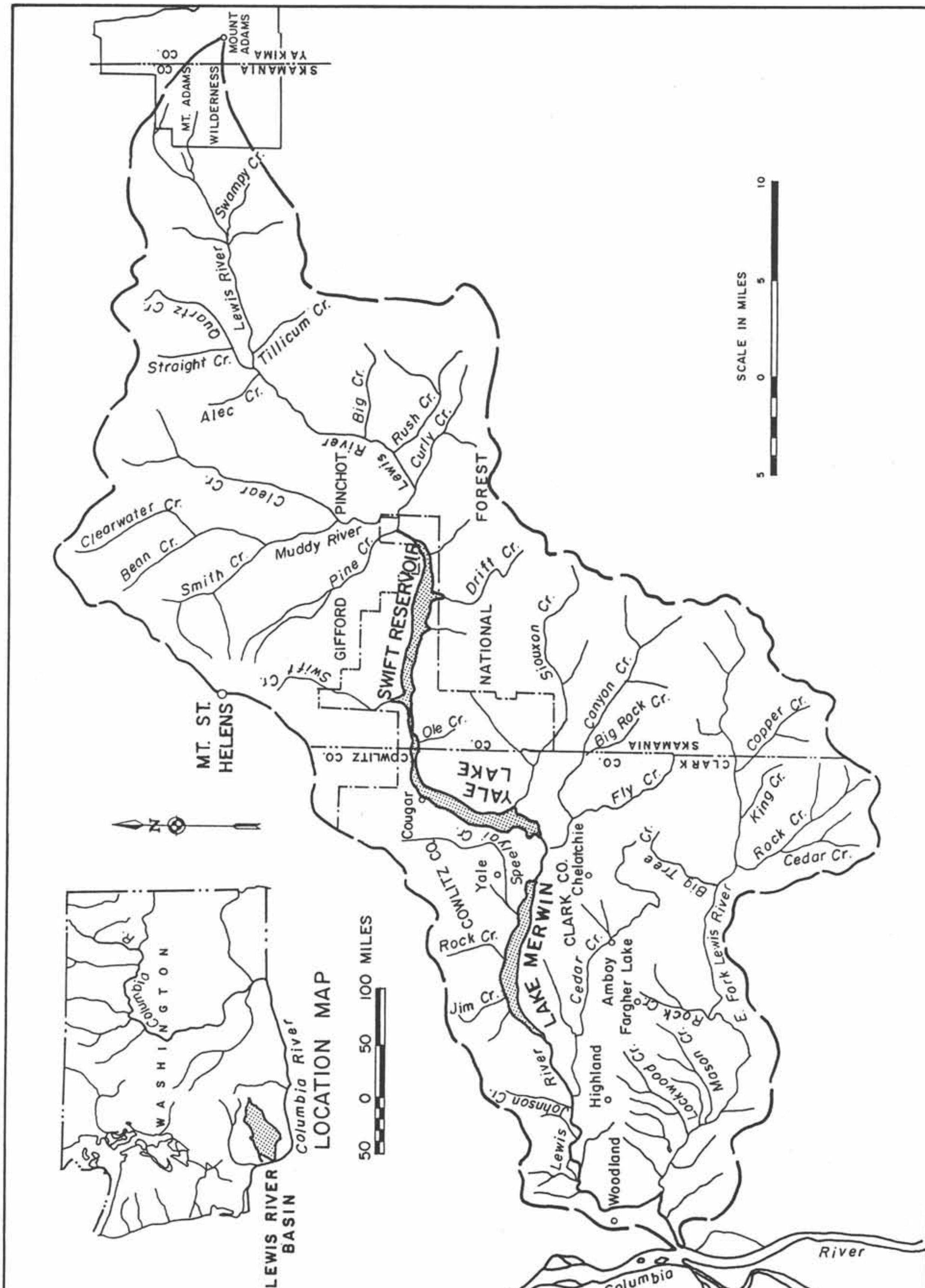


Figure 1. Lewis River Project area. From Tilford and Sullivan (1981).

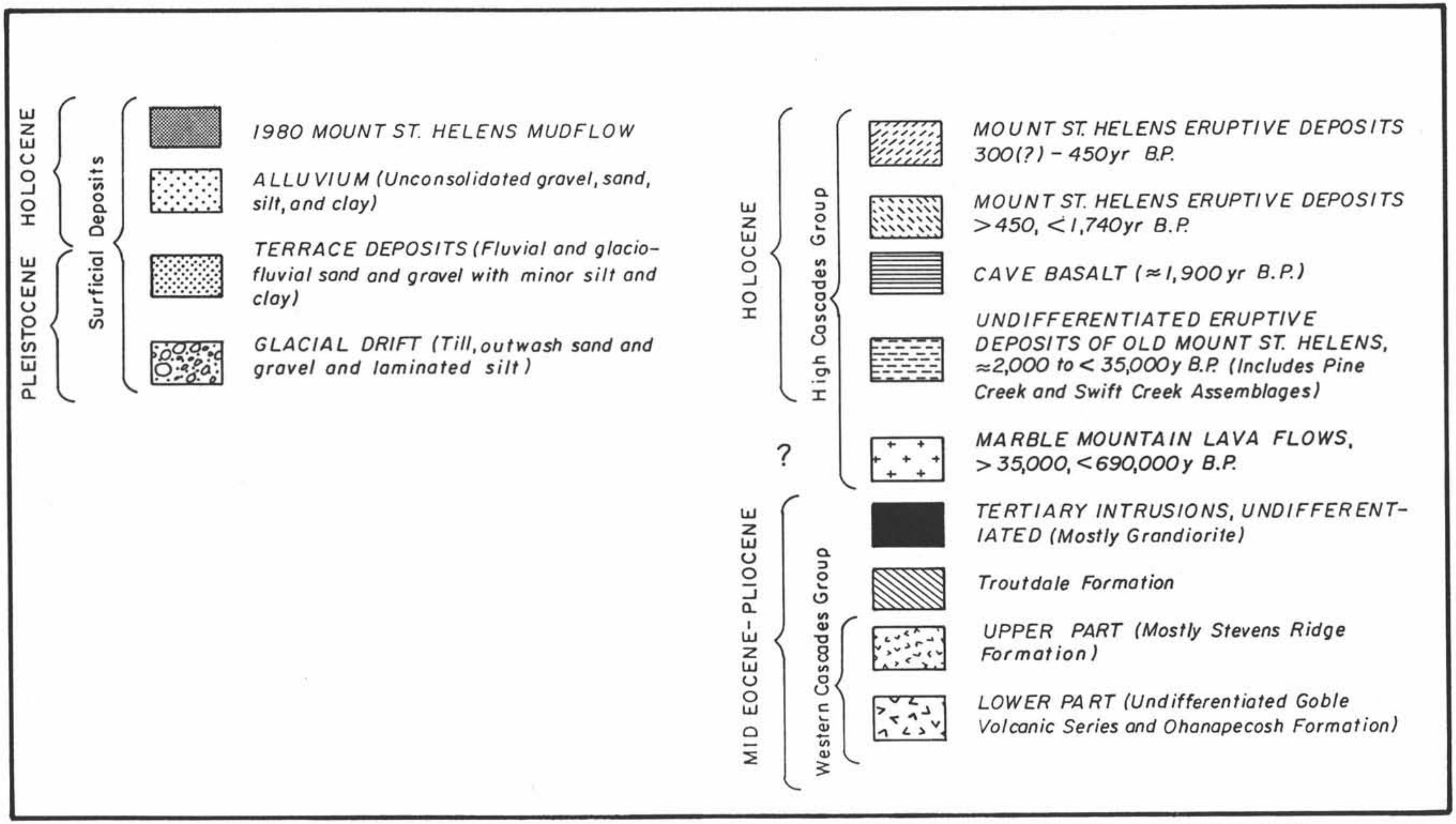


Figure 2a. Explanation for Figures 2b-d, geologic maps of the Lewis River Project area. Modified from Tilford and Sullivan (1981).

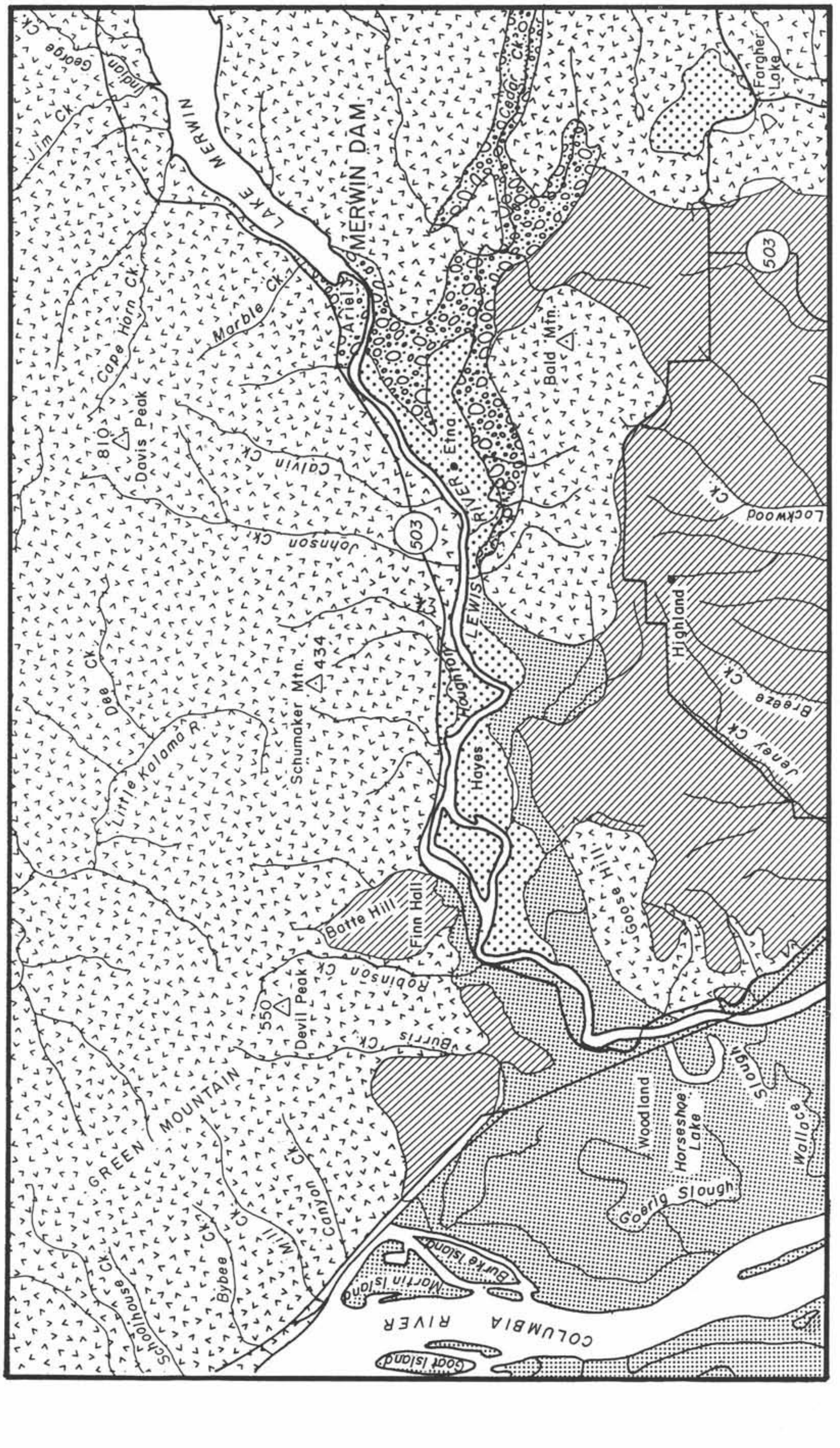


Figure 2b. Generalized geologic map of the Lewis River Project area, Columbia River to Merwin Dam. From Tilford and Sullivan (1981). See Figure 2a for explanation.

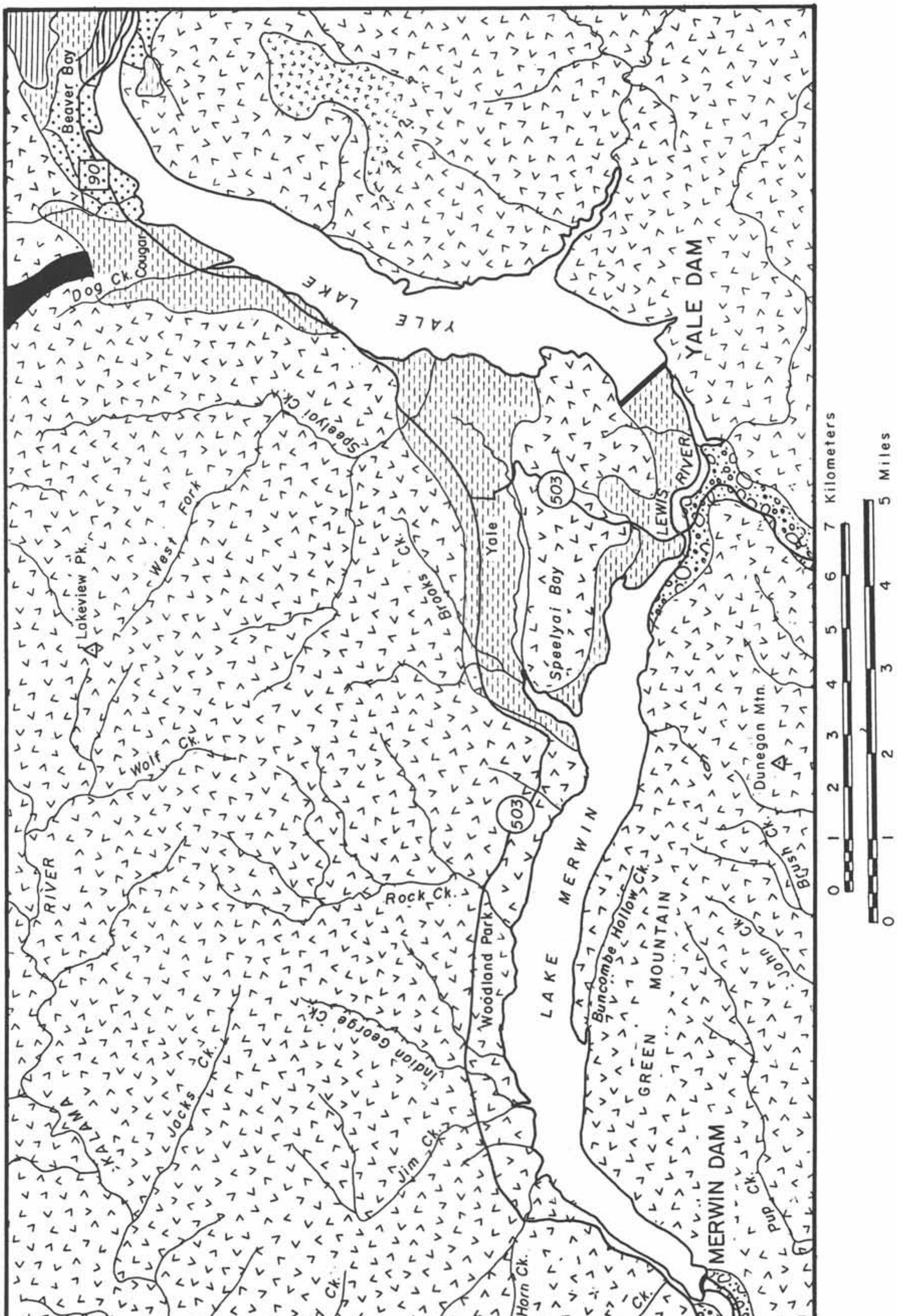


Figure 2c. Generalized geologic map of the Lewis River Project area, Merwin Dam to Yale Lake. From Tilford and Sullivan (1981). See Figure 2a for explanation.

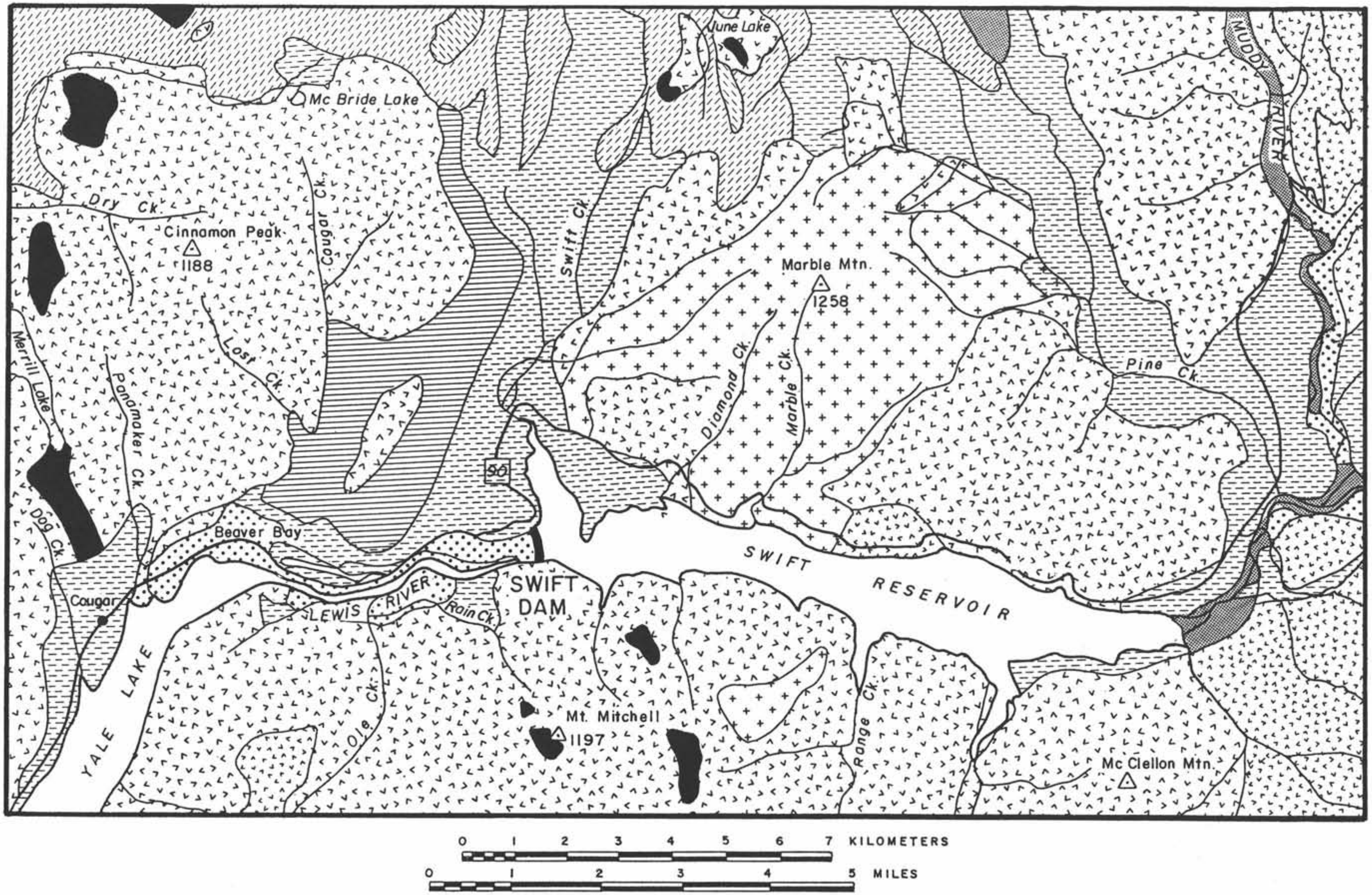


Figure 2d. Generalized geologic map of the Lewis River Project area, Swift Reservoir area. From Tilford and Sullivan (1981). See Figure 2a for explanation.

The unit consists primarily of lava flows and breccia of basaltic to andesitic composition interbedded with laharic breccia, tuffs, and volcaniclastic rocks. The type section for this series is at least 4,900 ft thick (Wilkinson et al., 1946). Individual lava flows range in thickness to as much as 100 ft. Near Mount St. Helens, the LWCG is covered in places by units of the HCG. On the south side of the Lewis River, the LWCG is exposed almost continuously from the lower end of Lake Merwin to the upper end of the Swift Reservoir.

The upper part of the WCG, or UWCG, is present only in the eastern part of the Lewis River drainage, where it underlies most of the area between Mount St. Helens and Mount Adams. Near Swift Reservoir, the UWCG is represented by a few scattered exposures of the Stevens Ridge Formation of late Oligocene to early Miocene age. This formation ranges between 160 and 4,900 ft in thickness and consists of interstratified rhyodacitic to dacitic pyroclastic flows, laharic breccia, volcaniclastic rocks including sandstone and conglomerate, and lava flows of dacitic to basaltic composition (Hammond, 1980).

Exposures of the dominantly Pliocene Troutdale Formation in the Lewis River area are mostly limited to the south side of the river between Woodland and Lake Merwin. The formation, which is 400 to 1,800 ft thick, is divisible into two members on the basis of lithology (Mundorff, 1964). The lower member, interpreted to be of possible lacustrine origin (Hammond, 1980), consists of well stratified thin beds of clay, silt, and micaceous sandstone. The upper member is interpreted to have been deposited as a great fluviatile piedmont fan along the western foot of the Cascade mountains (Mundorff, 1964). This member consists of cemented or semiconsolidated gravel containing cobbles and pebbles of basalt, andesite, and subordinate plutonic and metamorphic rocks, including white to pinkish quartzite, in a matrix of medium to coarse sand.

The HCG, of Pliocene to Quaternary age, consists predominantly of high-alumina, tholeiitic olivine basalt and basaltic andesite erupted from abundant monogenetic scoria cones and shield volcanoes and subordinate hornblende and/or pyroxene andesite and dacite erupted from widely scattered polygenetic composite cones (Hammond, 1980). In contrast to the rocks of the WCG, rocks from the HCG are generally unaltered and undeformed and retain their original positions of deposition (Hammond et al., 1977). In the Lewis River area, the HCG is chiefly represented by the lava flows of Marble Mountain, the Swift Creek volcanic assemblage (Hyde, 1975), the Pine Creek volcanic assemblage (Crandell and Mullineaux, 1973), and the Cave Basalt lava flow.

Marble Mountain, which borders the northwest shore of Swift Reservoir, is made up primarily of olivine basalt flows ranging up to 10 ft in thickness and consists of layers of clinkery scoria alternating with basalt. In addition, block lava flows of aphyric to microporphyritic hornblende andesite are present on the south flank of the mountain extending from approximately elevation 3,300 ft to the shore of Swift Reservoir. These flows range to as much as approximately 50 ft in thickness and have top and bottom breccia zones that have been dated as older than 35 ka and younger than 690 ka (Hammond, 1980).

The Swift Creek assemblage is an informal name for a stratigraphic sequence of pyroclastic flow deposits, lahars, tephra, and alluvium that forms a valley fill in Swift Creek and Lewis River valleys between the mouth of Swift Creek and the head of Lake Merwin. This assemblage was deposited during episodes of explosive andesitic and dacitic volcanism at an eruptive center which coincided with, but predated, the modern cone of Mount St. Helens. The Swift Creek assemblage has been divided into two parts, one older than glacial drift of Fraser age and the other younger. The fill deposits of the older part form high terraces on both sides of the Lewis River valley near Swift Dam. The thickness of the older fill may be as much as 660 ft near the mouth of Swift Creek. The younger part of the Swift Creek assemblage in the Lewis River valley is represented by a few pyroclastic flow deposits and by lahars and fluvial deposits derived from ancestral Mount St. Helens. These deposits accumulated in valleys cut in the older valley fill.

The Pine Creek assemblage is an informal term for a valley-fill assemblage of pyroclastic flow deposits, lahars, and alluvium that were formed during two or possibly three eruptive episodes of ancestral Mount St. Helens in late Pleistocene and Holocene time. This assemblage is younger than the Swift Creek assemblage. The Pine Creek assemblage extended from the base of the modern cone of Mount St. Helens southeastward down the valley of Pine Creek for a distance of 10 mi to the Lewis River valley and thence 43 mi downstream. This fill is now largely concealed by the three reservoirs in the Lewis River valley.

The Cave Basalt is a high-aluminum pahoehoe basalt flow that originated on the southeast flank of Mount St. Helens and flowed southward about 7 mi to the Lewis River (Greeley and Hyde, 1972). It is exposed on the north valley wall between Swift Dam and the head of Yale Lake. The basalt is vesicular to massive and porphyritic and has a radiocarbon age approximately 1.9 ka.

Surficial deposits in the Lewis River area consist primarily of Pleistocene glacial drift and terrace deposits and recent alluvium (Mundorff, 1964; Huntting et al., 1961). Deposits of Pleistocene age comprise many of the terraces that border the Lewis River downstream from Merwin Dam. There have been two major volcanic cones on the Mount St. Helens site, the present cone and an ancestral cone. The eroded base of the ancestral cone underlies the present cone and is visible on the flank up to an elevation of nearly 5,000 ft. Glacial deposits related to the present cone are limited to the general area of the mountain. Older glacial deposits (Amboy Drift [Mundorff, 1964]) are present in the Lewis River drainage. Deposits of recent alluvium are mostly restricted to the floodplain and low terraces along the river and are now chiefly covered by the reservoirs upstream of Merwin Dam.

Structure

Structural deformation along the western flank of the Cascade Range in southern Washington is mostly limited to the formations older than the HCG. The HCG and younger formations are generally undeformed and are either flat lying or gently inclined away from the volcanic centers (Hammond, 1979). Within these older formations, specifically the WCG, two periods of folding have been identified. The older period of deformation is characterized by a northwest-trending system of broad, open, irregularly spaced, double-plunging folds with limbs dipping between 10° and 45° (Hammond, 1979; Hammond et al., 1977). The younger system, which is superimposed on the older system, is a continuation of the Yakima fold system of the Columbia Plateau. This younger period of deformation appears dominant in the area of the Lewis River where the regional dip of the strata in the WCG is generally southeastward.

No faults of regional extent are known along the Lewis River in the immediate vicinity of the three hydroelectric projects, although several faults have been inferred in the area south of Lake Merwin on the basis of the topography and derangement of drainage. The amount of offset on these faults is unknown but may be on the order of several hundred feet. This faulting has been interpreted to have probably occurred during at least the latter part of the period of volcanism responsible for formation of the upper WCG (Mundorff, 1964). Faults are also present in the area between Mount St. Helens and Mount Adams, approximately 10 mi north of Swift Reservoir. These faults have a north-south to northeast trend and are interpreted as predating the formation of the HCG (Hammond, 1980).

Volcanic and Seismic Hazards

In addition to geologic hazards common to numerous other similar projects, the Lewis River projects have the hazard of being located close to the presently active Mount St. Helens volcano.

Studies of the past eruptive behavior of Mount St. Helens indicate that the Lewis River drainage has been affected by several periods of volcanic activity in the past and would likely be affected by future eruptions (Hopson, 1971; Crandell and Mullineaux, 1973, 1978; Crandell et al., 1975; Hyde, 1975). Some of these studies resulted in hazard zoning for potential eruptions. The area between Swift Dam and the head of Yale Lake, including the upper reaches of Swift Reservoir near the mouth of Pine Creek, was designated as a flowage-hazard zone that could be affected by lava flows, pyroclastic flows, mudflows, and floods. Swift Reservoir and Yale Lake were within a flowage-hazard zone likely to be affected only by mudflows and floods. The area downstream of Yale Dam including Lake Merwin was designated as a flowage-hazard zone likely to be affected only by floods. In the event of a major eruption of Mount St. Helens, one of the greatest potential hazards would be the sudden entry of a mudflow of large volume into Swift Reservoir, resulting in overtopping and/or failure of Swift Dam. It is estimated that the largest single mudflow expected would have a volume on the order of 100,000 acre-ft (Crandell and Mullineaux, 1978).

The potential for volcanic activity was suddenly brought to reality early on the morning of May 18, 1980, when Mount St. Helens violently erupted, resulting in the explosive removal of approximately a 1 cu mi of material, primarily on the summit and the north flank of the mountain. The debris from this explosive eruption was primarily directed northwest to northeast. The mudflow hazard was demonstrated when mud flows moved rapidly down Muddy River and Pine Creek, destroying the bridge near the mouth of Pine Creek and the Eagle Cliff bridge across the Lewis River before entering the upper end of Swift Reservoir. Records of water-level changes in Swift Reservoir indicate that the mudflow inflow into the reservoir on May 18 was about 60,000 cfs between 9 and 10 a.m., 55,000 cfs between 10 and 11 a.m., and 12,000 cfs between 11 a.m. and noon (Cummans, 1981). A total of about 11,000 acre-ft of material was deposited in the reservoir during this period, causing the water surface elevation to rise 2.61 ft. The instantaneous peak flow was estimated to be about 160,000 cfs. A series of waves was propagated; the highest of these was approximately 1.5 ft when it reached the water level recorder near Swift Dam approximately 11.5 mi downstream from the head of the reservoir (Schuster, 1981).

Renewed activity of Mount St. Helens has resulted in increased seismic activity. The general seismicity of the area had been low during the periods of historic record which extends back to the late 1820s. Prior to the recent activity at Mount St. Helens, the largest earthquake in the area was the magnitude 5.5 Portland earthquake of November 5, 1962, and had a maximum Modified Mercalli (MM) intensity of VII. Between October 1960 and September 1961 a series of earthquakes occurred adjacent to and south of Swift Reservoir. The series included two events with magnitudes of 4.4 and 5.1. These have been interpreted as events on the southern part of the St. Helens seismic zone (SHZ) (Grant and Weaver, 1986).

Earthquakes associated with the eruption of Mount St. Helens have occurred in three clusters. One cluster is near Marble Mountain just north of Swift Reservoir, another is located at the volcano itself, and the third is located along a north-south trend centered near Elk Lake to the north of the volcano. The maximum earthquake magnitudes associated with each cluster have been 5.5, 5.1, and 4.0, respectively. Since the May 1980 eruption of Mount St. Helens, the frequency of observations at the monitoring instruments has been increased, and strong motion accelerographs have been installed in the area.

Probably the most severe volcanic hazard that might be expected would be from a pyroclastic flow moving down Swift Creek and enveloping Swift Dam, resulting in damage to spillway gate equipment and making the gates inoperable. This would make it difficult to lower the reservoir level in the event of any subsequent mudflows triggered by pyroclastic flows and entering the reservoir. However, the possibility of a pyroclastic flow reaching Swift Dam seems unlikely, considering the present configuration of Mount St. Helens (breached in the north wall) and the volcano's eruption history (Tilford and Sullivan, 1981). The youngest pyroclastic flow deposits near the mouth of Swift Creek are more than 1,200 yr old (Hyde, 1975).

During 1982 and 1983, modifications were completed at Swift Dam to protect the spillway gates, operating equipment, controls, and power supply against thermal blasts and ashfalls. Design criteria included insured operation of the spillway gates for a minimum of 1 hr following elevated density wind blast forces of 100

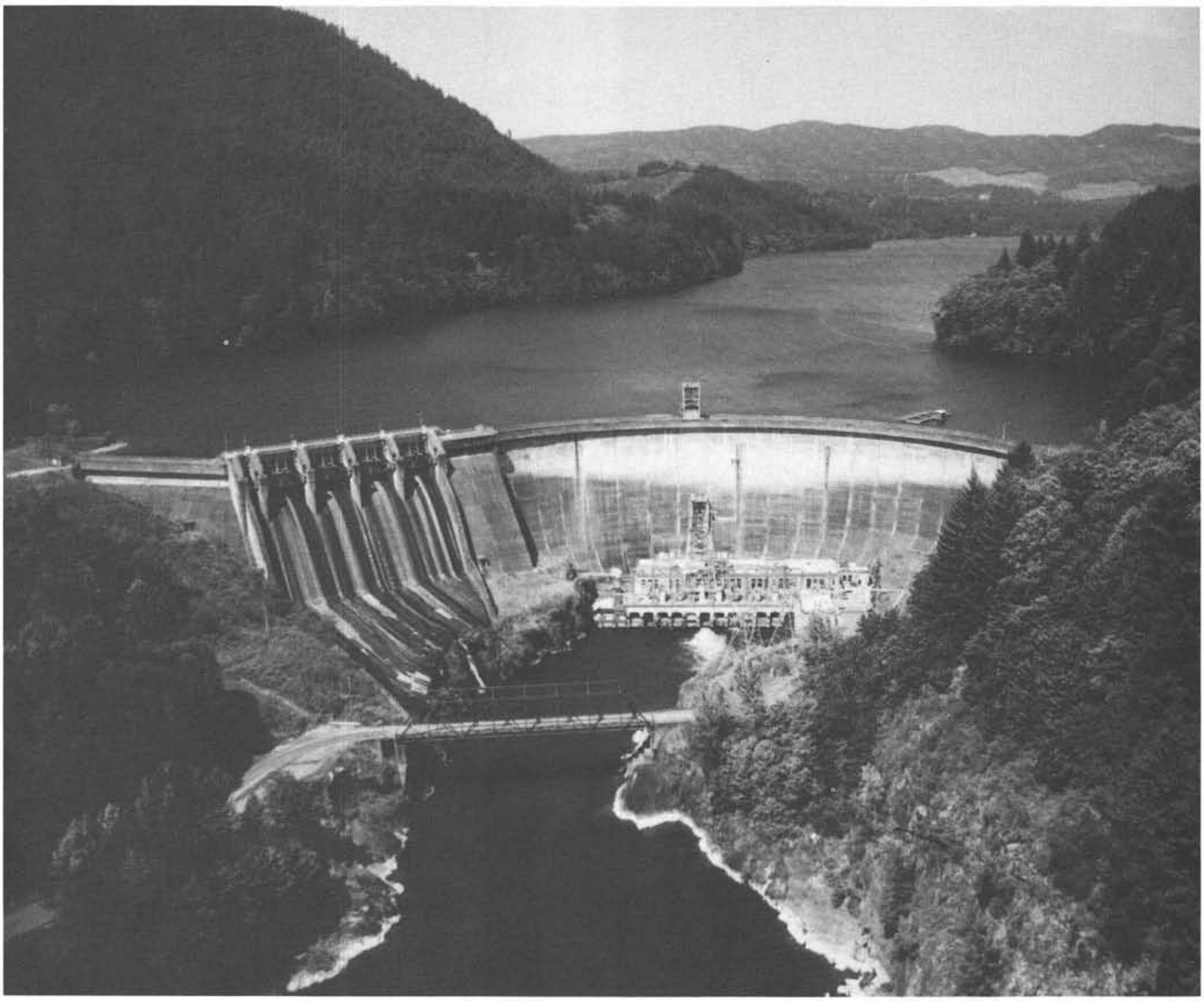


Figure 3. Merwin Dam and Lake Merwin. Photograph courtesy of Pacific Power and Light Company.

mph with a temperature of 200° C. In addition, instruments were installed to continuously monitor reservoir surface level and spillway opening during the 1-hr time frame (Pacific Power and Light Co., undated).

MERWIN HYDROELECTRIC PROJECT

Project Description

The Merwin Hydroelectric Project is located at river mile 19.6 and consists of a concrete dam and at-dam powerhouse (Figure 3). The dam consists of a variable radius concrete arch section, a gravity nonoverflow thrust-block section, a gravity ogee crest spillway section, and a nonoverflow section. The maximum height of the arch section above the river bed is 313 ft. The thickness of the arch is 93 ft at the base and 19.5 ft at the top. The spillway has five 30-ft-high tainter gates; four are 39 ft wide, and the other is 10 ft wide. The powerhouse is located immediately downstream of the dam. Water is conveyed through three 15.5-ft-diameter, 150-ft-long penstocks; a fourth penstock stub is in place for future expansion. The powerhouse contains three Francis turbines each rated at 55,000 hp at 170 ft of head. The dam impounds a reservoir, Lake Merwin, which is about 14.5 mi in length and has a total storage capacity of 422,800 acre-ft at normal maximum pool (Figure 4).

Site Geology

The Merwin Dam is located in the area where the Lewis River emerges from a narrow floodplain and enters a narrow rockbound gorge flanked by high ter-

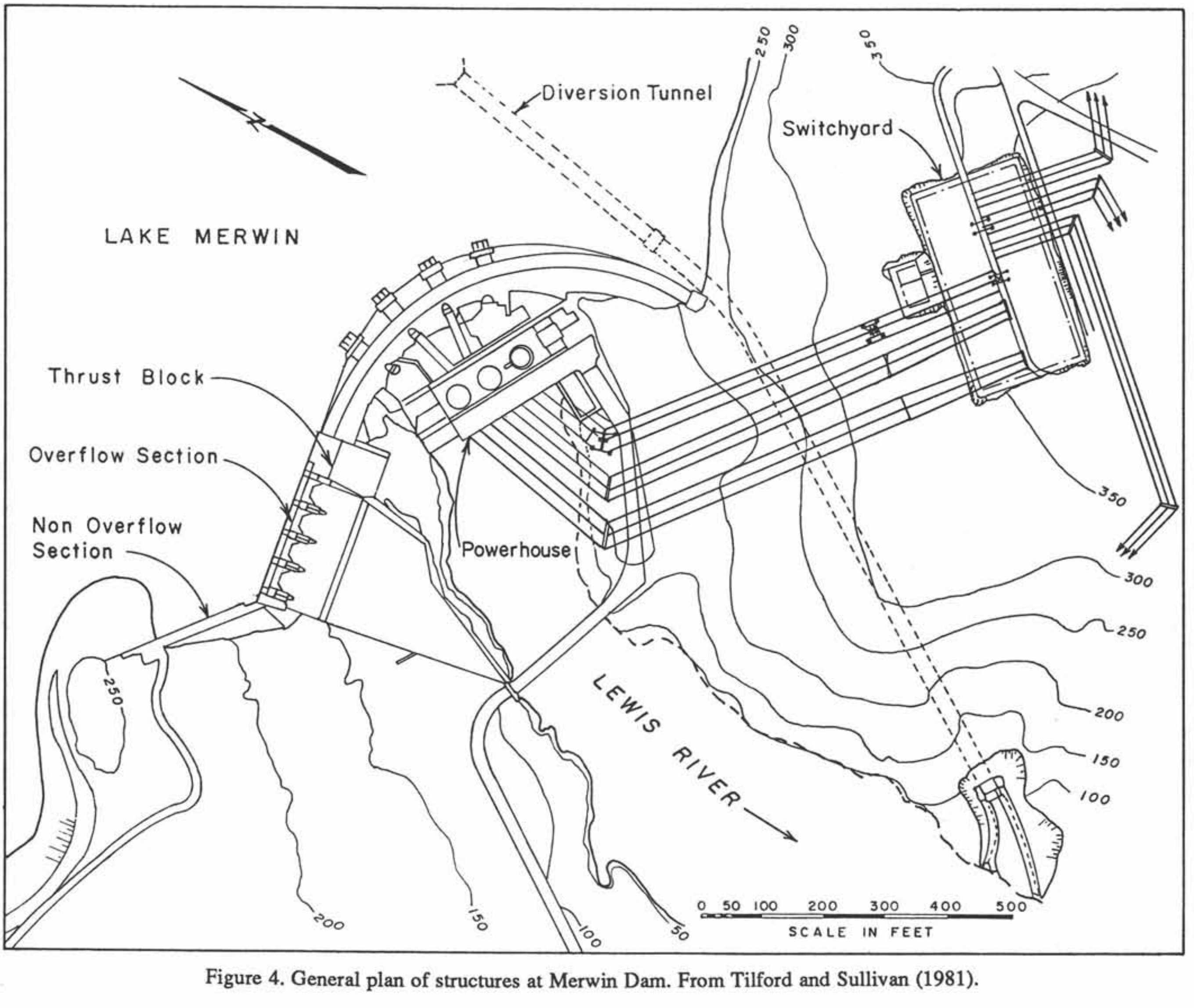


Figure 4. General plan of structures at Merwin Dam. From Tilford and Sullivan (1981).

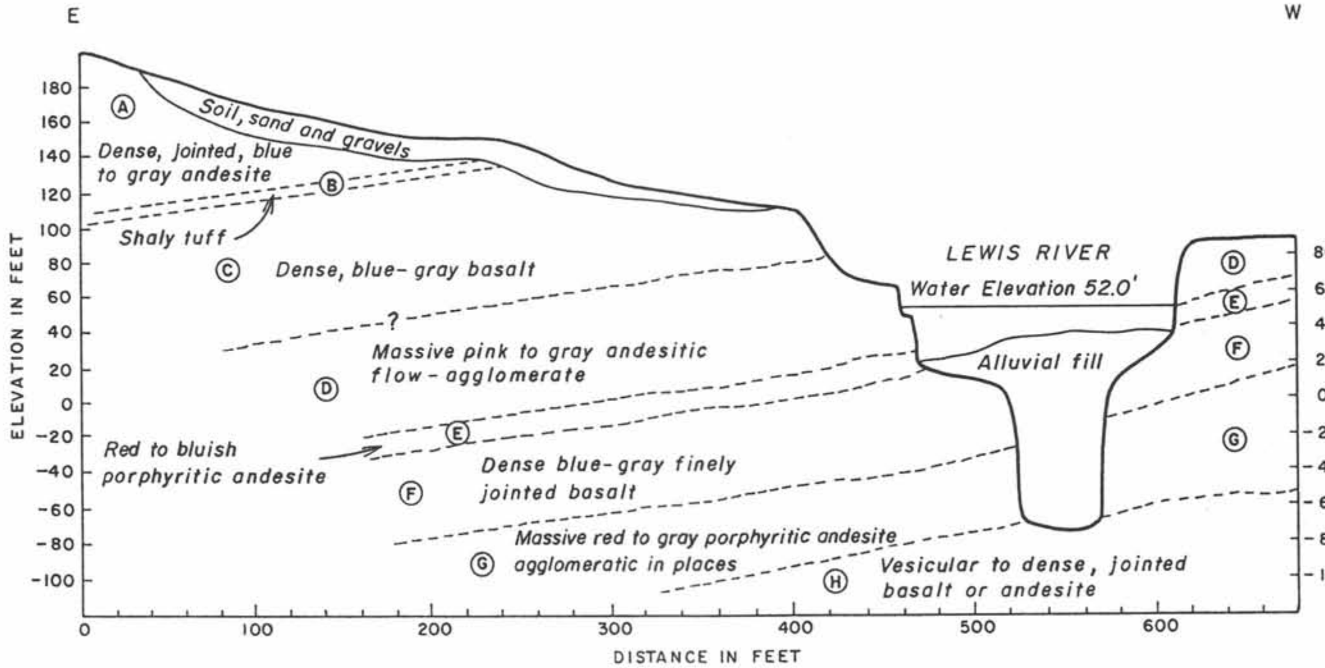


Figure 5. Schematic geologic section near the powerhouse of Merwin Dam; view downstream. From Tilford and Sullivan (1981).

races. The bedrock underlying the dam site consists primarily of basaltic or andesitic flows belonging to the LWCG. This bedrock is overlain by a discontinuous cover of unconsolidated soil and glacial drift that ranged up to a maximum thickness of about 60 ft in foundation excavations. Within the Lewis River channel proper, a channel had been eroded in the bedrock surface and subsequently buried by approximately 100 ft of alluvium.

The bedrock underlying the dam site consists of a series of basaltic or andesitic flows (Figure 5). The lowest rock unit encountered, beneath the channel alluvium, is a vesicular to dense, jointed basalt or andesite. This is overlain by a massive red to gray porphyritic andesite which is agglomeratic in places and ranges in thickness between 40 and 60 ft. The next overlying unit is an approximately 50-ft-thick unit of dense blue-gray finely jointed basalt, which is in turn overlain by an approximately 10-ft-thick unit of red to bluish porphyritic andesite. The above described units, designated as units H through E, were observed only in exploratory borings and foundation excavations because the units are all below the preconstruction level of the Lewis River and the channel infill deposits.

The next series of units was above the river level and was thus exposed for observation and examination. The lowest rock unit in this series consists of an approximately 60- to 70-ft-thick section of massive pink to gray andesite flow-agglomerate designated as rock unit D. Generally the unit consists of pebble- to boulder-size fragments of basalt and andesite in an andesitic matrix. In some areas, the agglomerate grades laterally and vertically into solid lava phases. The unweathered portions of the unit are highly indurated; the rock has the tendency to break through the fragments instead of through the matrix. In many areas, this unit forms vertical cliffs. Rock unit D forms the foundation for the gravity thrust block and the west end of the arch section of the dam. Rock unit D is overlain by unit C, which consists of an approximately 80-ft-thick section of primarily dense, blocky jointed, blue-gray basalt. The unit is characterized by an approximately 10-ft-thick, reddened, scoriaceous lava at the lower boundary and a scoriaceous flow-agglomerate at its upper boundary. The boundary between units C and D is marked by a thin, relatively soft, shale section, which has been interpreted as representing a thin soil horizon that had formed on the surface of unit D and that was baked by the molten lava that formed unit C. Unit C was exposed along the east valley wall and forms the steep cliffs approximately 600 ft downstream of the dam. The unit serves as the foundation for the eastern portion of the dam's arch section.

Unit C is overlain by rock unit B, which is a thin, compact bed of variegated tuff that ranges in thickness from a few feet to a maximum of approximately 25 ft. On a fresh surface, this unit has the appearance of a solid, massive, blocky shale, although in some areas it is soft and has a clayey consistency. When subjected to drying or to a wetting and drying cycle, the material rapidly weathers to a gritty, somewhat plastic clay.

The uppermost rock unit has been designated as unit A and consists of a hard, splintery, dark gray andesite, which is more than 200 ft thick. This unit provides the foundation for the east abutment of the arch section of the dam (Tilford and Sullivan, 1981).

Construction Problems Relating to Geology

Although the foundation preparation for Merwin Dam required the excavation of approximately 100 ft of alluvium beneath the central portion of the dam, no major construction problems were apparently related to this operation. One geologic structural feature that required special attention was a northeast-trending fracture zone in unit D that intersected the centerline of the dam near the river's edge on the west bank. The surface expression of this fracture zone was a steep-sided, gravel-filled ravine which varied in width between 20 and 30 ft. During foundation preparation, the feature was excavated, revealing the depth of alluvium to be on the order of 20 to 30 ft. At the bottom of the excavation, a fracture zone was revealed; it consisted of from one to four anastomosing fractures ranging up to several inches in width and filled by secondary minerals and/or fault gouge.

YALE HYDROELECTRIC PROJECT

Project Description

The Yale Hydroelectric Project is located upstream of the Merwin Project. The main dam is at river mile 34.2 (Figure 6). The development consists of a 323-ft-

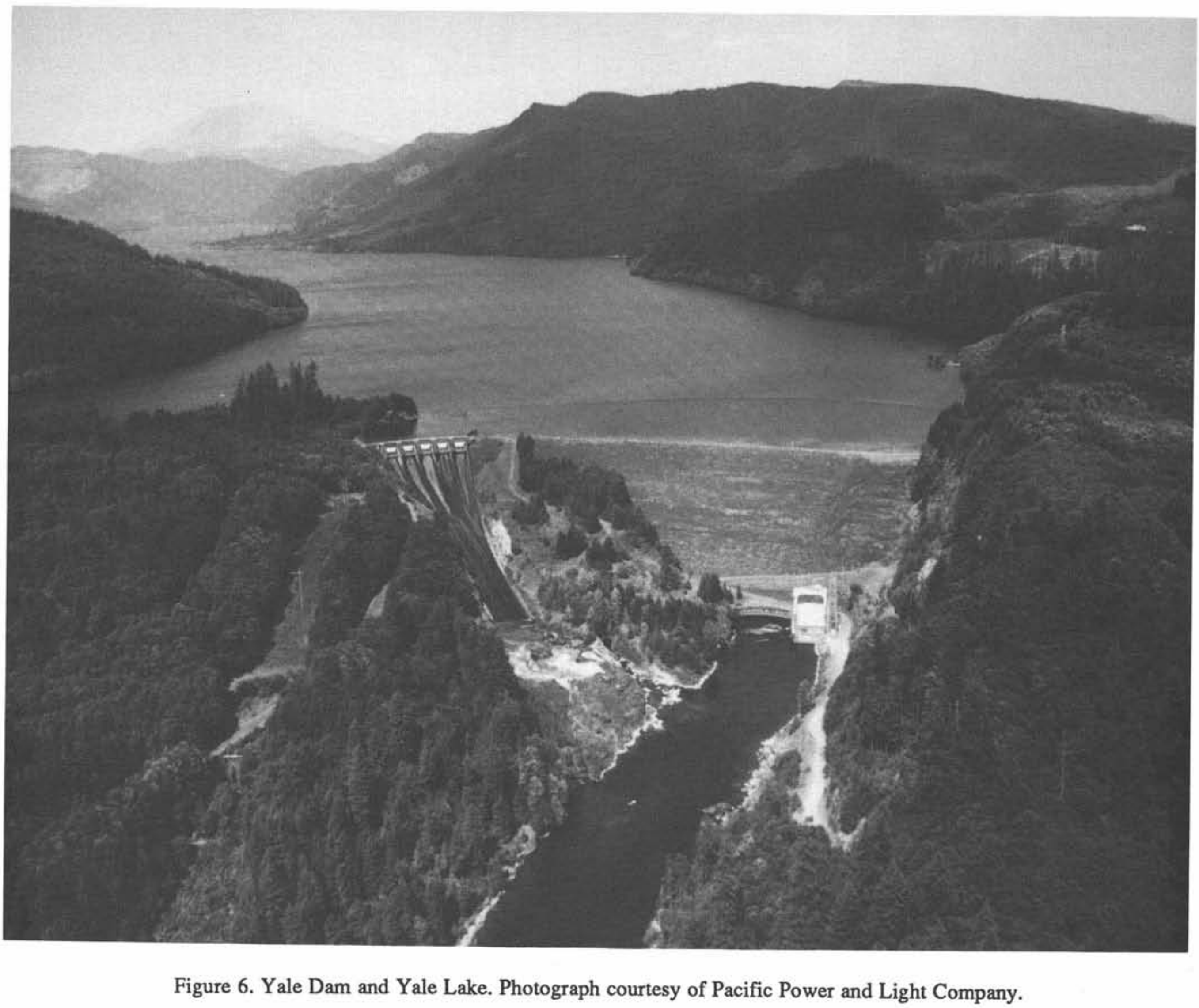


Figure 6. Yale Dam and Yale Lake. Photograph courtesy of Pacific Power and Light Company.

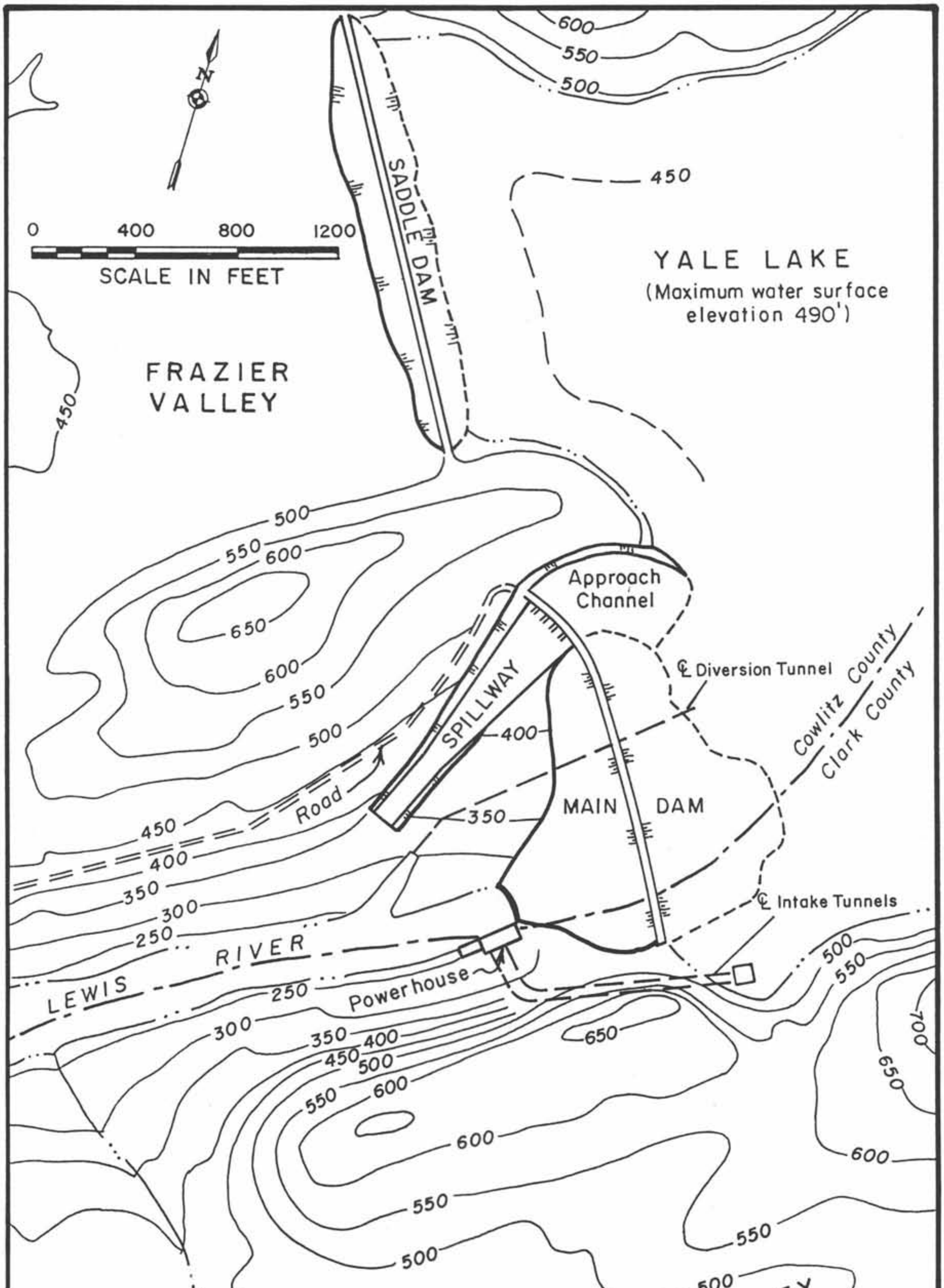


Figure 7. General plan of structures at Yale Dam. From Williams (1930).

high earthfill main dam, spillway, diversion tunnel, two penstock tunnels, and a powerhouse with two 54,000-kw generators. In addition, there is a 45-ft-high earth saddle dam located approximately 900 ft northeast of the main dam (Figure 7).

The main dam is a rolled-earth structure containing approximately 4.2 million cy of material. The dam has a crest length of 1,300 ft and is 323 ft high above the lowest point in the foundation. The dam is designed with a sloping central impervious core section supported by sandy gravel shells. The downstream toe of the dam embankment is contained by a 77-ft-high concrete arch retaining wall adjacent to the powerhouse discharge structure. The upstream slope of the dam is 2.5H to 1V and the downstream slope is 2H to 1V.

The spillway is a concrete chute equipped with five gates to control spilling. The saddle dam is also a rolled-earth embankment; it has a height of 45 ft and a crest length of 2,000 ft. The dam has a central impervious clay core with random fill outer slopes constructed at 3H to 1V. The reservoir impounded by the dam, Yale Lake, has a length of 10 mi and a surface area of 3,800 acres at the normal maximum pool elevation of 490 ft. The total storage capacity is 401,760 acre-ft, of which 190,000 acre-ft is available for power generation.

Site Geology

Geologic investigations at the Yale Dam site began as early as 1924. Additional investigations were completed in 1929 and 1930. The final period of investigation was completed in 1951 after the beginning of construction activities.

The dam abutments are comprised of two northeast-trending bedrock ridges with summit elevations on the order of 650 ft. The north, or right abutment ridge is separated from the main north valley wall by a high broad terrace locally known as Frazier valley. The maximum elevation of Frazier valley is approximately 460 ft, or 30 ft below the high reservoir elevation, which necessitated the construction of a saddle dam across the Frazier valley to enable reservoir impoundment to elevation 490 ft. At the dam site, the north abutment has a moderate slope upwards and reaches the dam crest elevation of 508 ft at a distance of approximately 950 ft from the original river location. The south abutment has a steeper slope and precipitous cliffs along the former river bank and between elevations 420 to 500 ft. On the south abutment, the dam crest elevation of 508 ft is reached at approximately 400 ft from the former river bank.

The bedrock which underlies the Yale Dam site consists of lavas and interbedded tuffs of the LWCG (Figure 8). These rocks have been divided into the stratigraphically lower older lava series and the overlying younger lava series. The contact between the series is an unconformity with at least 200 ft of relief developed on top of the older series. The surface between the older and younger series is marked by a fossil soil horizon consisting primarily of red clay and containing carbonized wood fragments.

The lower lava series, with the exception of the extreme right abutment, comprises the dam's foundation. The series consists of lava flows averaging approximately 50 ft in thickness and interlayered with bedded tuff beds that range in thickness from a few inches to 50 ft. The flows are andesitic to basaltic and commonly have brecciated tops and bottoms. In fresh exposures, the flows are dense and hard. However, in general the flows are weathered. Alteration has been accompanied by secondary mineralization, including calcite, chalcedony, and zeolites, which have effectively filled most of the voids in the rock. The interbedded tuffs consist primarily of angular fragments of ejecta which range in size from silt to approximately 4 in. In general the tuffs are extensively weathered to a pinkish to greenish gray. The coarser tuffs and tuff-breccias have been hardened by secondary cementation by silica and other minerals in the interfragment voids, while the finer tuffs have been primarily altered to clay minerals. The presence of slickensided surfaces within the tuff beds indicates movement has occurred in the past. The spillway and diversion tunnel, both located in the left abutment, are founded within the lower lava series.

The overlying younger lava series, which is exposed as precipitous cliffs along the south valley wall, consists of a series of columnar basalt flows, which range between 50 and 150 ft in thickness. Zones of breccia on the order of 40 ft in thickness are present at the surfaces of the flows. In a fresh exposure, the basalt is dense, hard, and generally porphyritic. In the original porous breccia zones, alteration is most intense, and in the most highly altered zones, the basalt has been converted to a moderately soft material which tends to disintegrate when exposed to the atmosphere. As with the coarser grained portion of the tuff beds, the breccia zones of the basalt have had secondary cementation by calcite and silica.

The bedrock units near the Yale Dam site generally strike northeast and dip between 17° and 22° SE, generally conforming to the regional dip. A gentle anticline trends approximately N 10° W across the dam site, and on the south valley wall it is located between the powerhouse and the centerline of the dam. Two sets of nearly vertical faults, one striking between N 11° W and N 44°E, the other approximately east-west, were exposed in foundation excavations. On the basis of nearly horizontal fluting on the fault planes, it has been assumed that the main displacement was horizontal; there is little evidence of vertical displacement. The faults contain between 1 and 3 ft of gouge. The faults appear to be the result of adjustment of internal stresses caused by the regional folding. Jointing in the older lava and

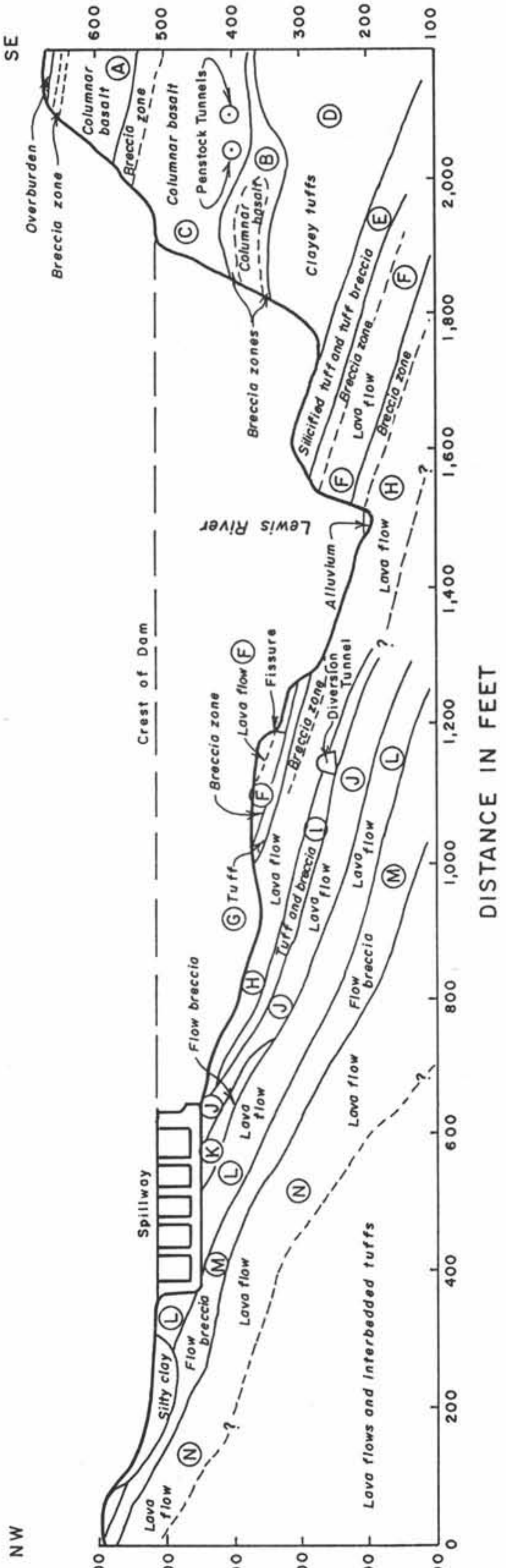


Figure 8. Schematic geologic section near Yale Dam; view upstream. From Tilford and Sullivan (1981).

tuff series is poorly developed, whereas in the younger basalts, columnar jointing is well developed. In the older lava and tuff series, two principal joint systems are normal to the bedding planes—one set is parallel to the strike and the other is normal to the strike. A third and minor joint set is parallel to the bedding. Most joints having any appreciable width are filled with calcite and chalcedony. In the younger lavas, in addition to the columnar joints, a secondary joint set parallels the bedding. In the upper approximately 100 ft of the younger lavas, the joint cracks have been widened by weathering, and some are filled with clay-like materials. Below this depth, the joints generally have linings and fillings of calcite.

Overburden deposits at the Yale Dam site consist of lahars and associated fluvial deposits, alluvial deposits, and colluvium. The lahars and associated fluvial deposits comprise at least the upper portion the surficial sediments in the Frazier valley and form the foundation for the saddle dam. These deposits contain several distinct lahar sequences, which consist primarily of multicolored deposits of angular to subangular andesite fragments ranging from silt to pebble size and scattered boulders. The coarser portion of the deposit appears to be concentrated in the lower beds. The fluvial portion of the deposits consists primarily of permeable lenses of well-rounded sand and gravel. This upper lahar unit has a thickness of at least 100 ft. The alluvial deposit consists primarily of channel fill of the Lewis River, sand- to cobble-size material of Mount St. Helens lava types. There are some boulders, which are derived from local rock types and in some areas probably represent rock falls from the steep cliffs. The alluvial deposits are generally less than 100 ft thick, but are locally thicker. The colluvial deposits are primarily concentrated below the steep slopes formed by the columnar basalts and consist of basalt and tuff debris in rock falls and slumps.

Construction Materials

The main Yale Dam was originally expected to be a rockfill structure with a clay core. Exploration revealed that potential quarry sources nearby would involve considerable waste because of the presence of the altered breccia zones and interbedded tuffs. Thus, the shell portion of the dam was changed to sand and gravel fill. This material was obtained from two borrow sources located 3/4 mi and 2-1/2 mi upstream of the dam on the north bank of the river. The original clay core was to have been constructed of sandy clay. However, exploration indicated that the proposed source deposit had a moisture content in excess of optimum and could not be satisfactorily placed during the rainy season. A substitute was found in a mudflow deposit in the upper lahar unit in the Frazier valley; this deposit was a silty sandy gravel containing approximately 12 to 20 percent silt. Filter and transition materials for the dam were obtained from beneath the closest upstream shell material borrow sources. Riprap for the dam faces was selected material obtained from required excavations and from a quarry located above the power tunnel intake.

Construction Problems Related to Geology

Most of the construction problems related to geology were caused by the clayey nature of some of the tuff layers or to the presence of faults or joints having adverse orientations relative to excavations. These factors resulted in slumping or sliding of large rock masses during excavation activities. During excavation of the north valley wall, a slide block developed where a nearly vertical, northeast-trending fault crossed behind the excavation; a large block of lava moved down-dip toward the river. Movement of the block was probably on or in the underlying tuff layer. Part of the block was removed by blasting, and the remainder became stabilized.

During excavations on the south valley slope, a previously designated colluvium deposit was determined to be a huge land slump involving a clayey tuff unit and the overlying columnar basalt. After excavation of the slump deposit, the clayey tuff beds responded to the unloading by expanding and sloughing, thereby leaving the overlying columnar basalt unsupported. Joints and faults resulted in several block movements of the columnar basalt, one involving approximately 10,000 cy of material.

SWIFT CREEK HYDROELECTRIC PROJECT

Project Description

The Swift Creek Project is located upstream of the Yale Project at river mile 33.0 (Figure 9). The dam site is located a short distance downstream from the confluence of Swift Creek and the Lewis River. The development consists of a 512-ft-high earthfill dam, diversion and power tunnels, spillway, and powerhouse (Figure 10). When it was completed in 1958, Swift Dam was the highest earthfill dam in the world.

The dam is a rolled-earth structure having a crest elevation of 1,012 ft and a crest length of 2,100 ft. The crest of the dam is 412 ft above the original stream bed and 512 ft above the lowest point in the foundation. The dam is designed with a central vertical core supported by upstream and downstream shells of relatively pervious granular fill. A vertical granular chimney drain between the core and the downstream shell is connected with a horizontal rock drain blanket constructed on the river bed under the downstream shell. The chimney drain is protected both upstream and downstream by transition zones, and the rock drain is overlain by a transition zone. The upstream slope of the dam is 2.25 and 2.5H to 1V, and the downstream slope is 2H to 1V.

The dam foundation was constructed using a combination of open trench excavation, a cut-off wall, and

Figure 9. Swift Dam and Reservoir. Photograph courtesy of Pacific Power and Light Company.

a grout curtain. The open-cut excavation was made to a depth of approximately 100 ft below the stream bed. Parallel sheetpile cut-off walls extended from the open-cut excavation to bedrock; the area between the walls was excavated and filled with concrete. Above the sheetpile wall, a 30-ft-high concrete cut-off wall extends across the bottom of the trench and up the rock abutments. A grout curtain in the bedrock ranges in depth between 20 and 100 ft.

The spillway is an ogee-type overflow structure on the left abutment. It has a crest elevation of 950 ft. The spillway is divided into two sections, each controlled by a 50 x 51-ft tainter gate. The spillway and the concrete-lined discharge channel were excavated in the left abutment rock.

The powerhouse is located on the south side of the river a short distance from the toe of the dam embankment and contains three units, each powered by a 107,000-hp Francis turbine rated at 328-ft net head. The reservoir is connected to the powerhouse by a 25-ft-diameter power tunnel which has a flat gradient through about 1,230 ft of its 1,450-ft length and then is steeply inclined. It has a surge tank at the change of grade. Both the tunnel and the surge tank are steel and concrete lined throughout.

A second power plant is located 3-1/2 mi downstream of Swift Dam and makes use of the head differential to Yale Reservoir. Water from the main powerhouse is carried by an open canal and forebay structure constructed on the north terrace of the Lewis

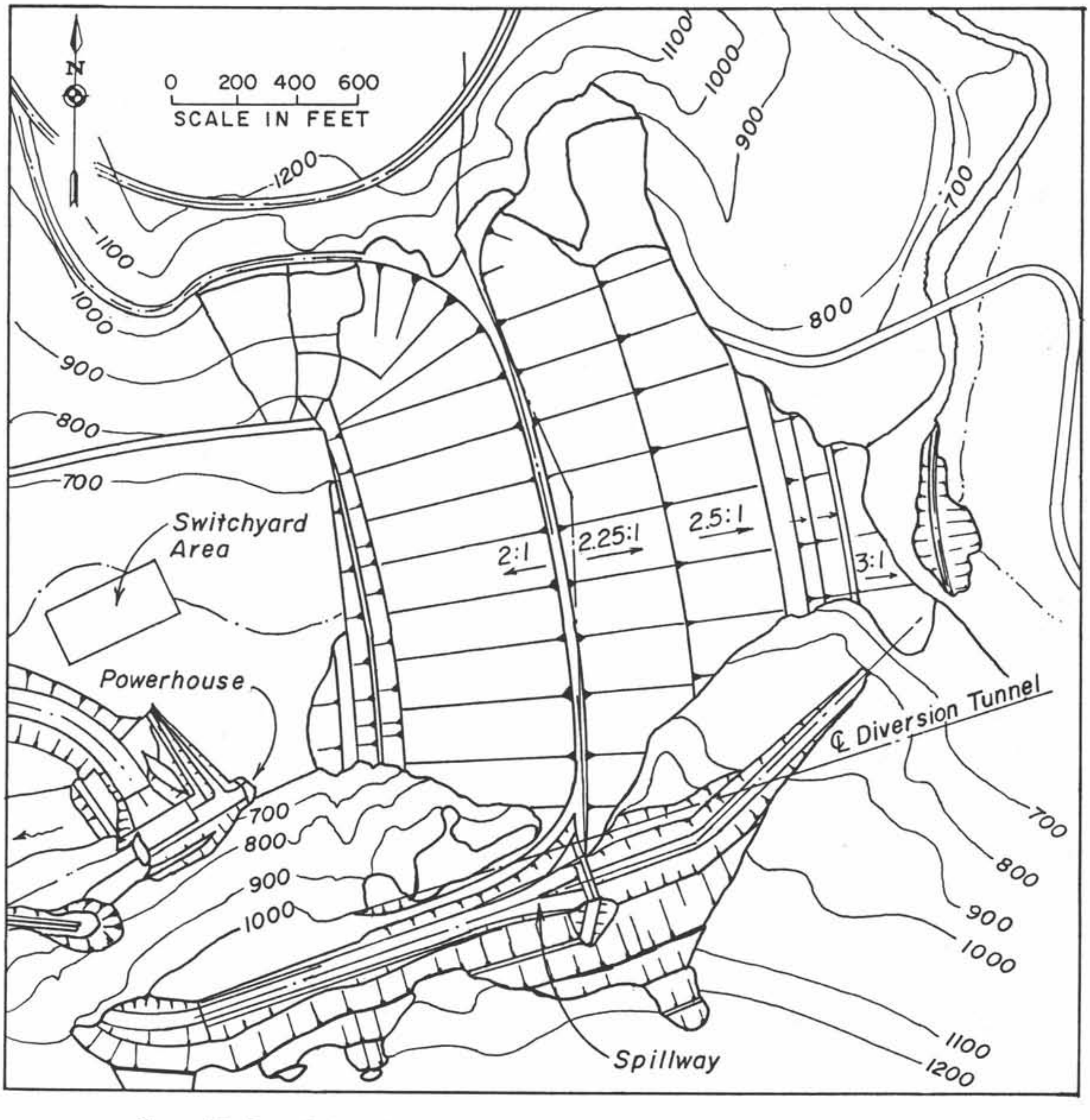


Figure 10. General plan of structures at Swift Dam. From Pacific Power & Light (1987).

River. The second powerhouse has two 35,000-kw generators operating under a 113-ft head.

Site Geology

Bedrock at the Swift Dam consists of andesitic tuff and tuff breccia, which are bedded, laminated, and fine grained. There is no consistent structural attitude. The rock is compact but only slightly cemented. On the left abutment, or south side of the river, bedrock extends upwards in excess of elevation 1,050 ft. The diversion tunnel, power tunnel, and spillway are all located in the left abutment. On the north, or right abutment, bedrock rises to an elevation of 818 ft and drops in a northerly direction to an unknown elevation. One drill hole located 1,400 ft north of the river penetrated to elevation 295 ft and did not encounter bedrock. The top of bedrock on the right abutment is below the dam crest elevation of 1,012 ft (Figure 11).

The overburden material at Swift Dam site consists entirely of volcanic mudflow deposits, which are as thick as 100 ft on the left abutment. On the right abutment, these deposits overlie the bedrock and reach thickness in excess of 400 ft. The mudflow deposits vary in thickness, individual layers having an average thickness on the order of 15 to 18 ft. The mudflows resemble glacial deposits and are composed of angular

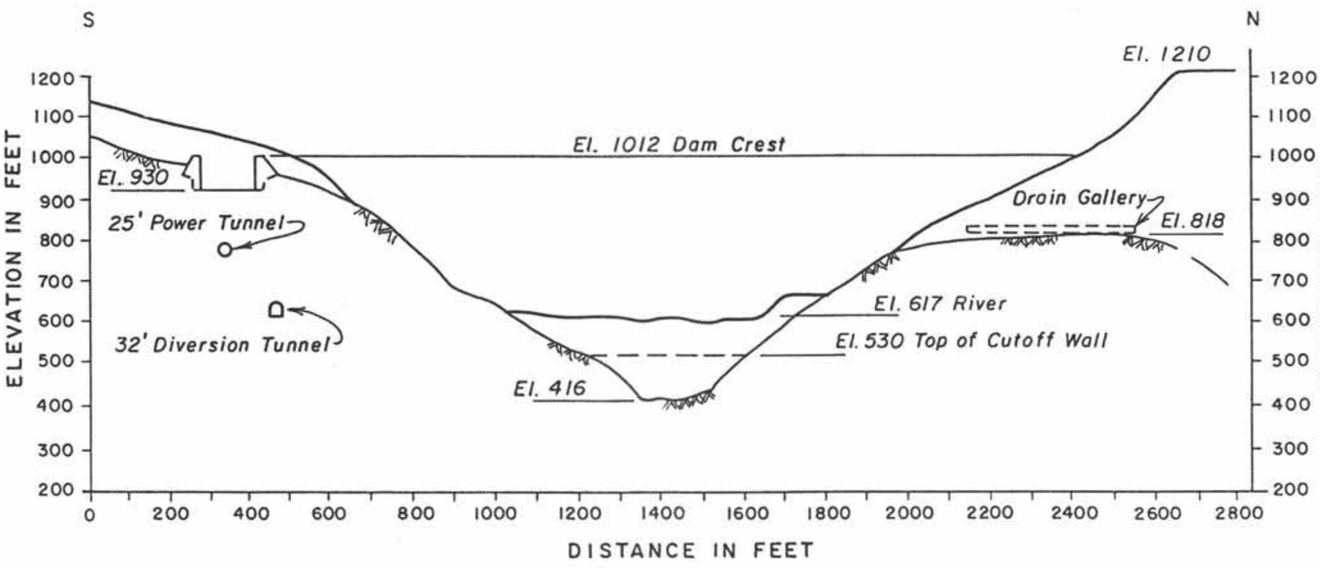


Figure 11. Schematic geologic section near Swift Dam; view downstream. From Jensen (1981).

to subangular fragments ranging to approximately 4 ft in maximum diameter in a coarse to fine sandy matrix. The channel fill beneath the river was also composed of mudflow deposits with a sporadic thin veneers of river gravel between flows.

Construction Materials

The zoned earthfill embankment required 15,400,000 cy of material. The borrow areas for the shell material were located upstream of the dam site in the Swift Creek valley. Impervious core material was obtained from an area on the upstream side of the north abutment in a deposit that contained a high percentage of fines. Sand and gravel utilized for filters and concrete aggregates were obtained from borrow areas approximately 1 mi upstream of the dam site in a broad part of the valley of the Lewis River. Since no suitable quarry rock was available near the project, rock slope protection was produced by using oversize rock from the shell material sources.

Construction Problems Related to Geology

No major construction problems related to geology were reported. The low elevation of the top of bedrock on the right abutment and the presence of the thick sequence of overlying mudflow deposits required the construction of a drainage gallery at the high point of rock elevation to intercept any seepage that might occur through the mudflow alluvium (Figure 11). Measured seepage volumes have been reported as being considerably less than anticipated.

Since the Swift Creek area normally receives 100 in. of rainfall annually, it was anticipated that it would be difficult to place fill materials because their natural moisture content was near optimum. A test fill was completed prior to invitations for bid; this indicated that the fills could be properly placed and compacted even during periods of heavy rain. Special precautions were observed in the borrow areas and consisted of working a high near-vertical face and sloping the area to drain, thus preventing ponding of surface water. In addition, all fill surfaces were kept sloped to drain and prevent ponding.

ACKNOWLEDGMENTS

Numerous articles and papers were reviewed during the preparation of this article. Two of these papers used as primary sources were authored by Norman R. Tilford and James G. Sullivan and by J. R. Jensen. These papers were prepared for and appear in the field trip guide for the Association of Engineering Geologists Annual Meeting held in Portland, OR, in 1981. The articles reflect extensive and time-consuming research by the authors for which this author is greatly indebted.

REFERENCES

Crandell, D. R. and Mullineaux, D. R., 1973, *Pine Creek Volcanic Assemblage at Mount St. Helens, Washington*: U.S. Geological Survey Bulletin 1383-A, 23 p.

Crandell, D. R. and Mullineaux, D. R., 1978, *Potential Hazards from Future Eruptions of Mount St. Helens Volcano, Washington*: U.S. Geological Survey Bulletin 1383-C, 26 p.

Crandell, D. R.; Mullineaux, D. R.; and Rubin, Meyer, 1975, *Mount St Helens Volcano; Recent and Future Behavior*: Science, Vol. 187, pp. 438-441.

Cummans, John, 1981, *Mudflows Resulting from the May 18, 1980 Eruption of Mount St. Helens, Washington*: U.S. Geological Survey Circular 850-B, 16 p.

Grant, W. C. and Weaver, C. S., 1986, Earthquakes near Swift Reservoir, Washington, 1958-1963 – Seismicity along the southern St. Helens seismic zone: *Bulletin of the Seismological Society of America*, Vol. 76, No. 6, pp. 1573-1587.

Greeley, R. and Hyde, J. H., 1972, Lava tubes of the Cave Basalt, Mount St. Helens, Washington: *Geological Society of America Bulletin*, Vol. 83, No. 8, pp. 2397-2418.

Hammond, P. E. 1979, A Tectonic Model for the Evolution of the Cascade Range. In Armentrout, J., et al. (editors), *Cenozoic Paleogeography of the Western United States*, Pacific Coast Paleogeography Symposium 3: Society of Economic Paleontologists and Mineralogists Pacific Section, pp. 219-237.

Hammond, P. E., 1980, *Reconnaissance Geologic Map and Cross Sections of Southern Washington Cascade Range, Latitude 45°30'–47°15' N., Longitude 120°45'–122°22.5' W.*: Portland State University Department of Earth Sciences, Portland, OR, 31 p., 2 sheets, scale 1:125,000.

Hammond, P. E.; Bentley, R. D.; Brown, J. C.; Ellingson, J. A.; and Swanson, D. A., 1977, Volcanic Stratigraphy and Structure of the Southern Cascade Range, Washington. In Brown, E. H. and Ellis, R. C. (editors), *Geological Excursions in the Pacific Northwest*: Western Washington University, Department of Geology, Bellingham, WA, pp. 127-169.

Hopson, L. A., 1971, *Eruptive Sequence at Mount St. Helens, Washington* [abstract]: Geological Society of America Abstracts with Programs, Vol. 3, No. 2, p. 138.

Huntting, M. T.; Bennett, W. A. G.; Livingston, V. E., Jr.; and Moen, W. S., 1961, *Geologic Map of Washington*: Washington Division of Mines and Geology, Olympia, WA, 1 sheet, scale 1:500,000.

Hyde, J. H., 1975, *Upper Pleistocene Pyroclastic-Flow Deposits and Lahars South of Mount St. Helens Volcano, Washington*: U.S. Geological Survey Bulletin 1383-B, 20 p.

Jensen, J. R., 1981, Swift Hydroelectric Project Lewis River, Washington. In *Field Trip Guidebook, Engineering Geology in the Pacific Northwest*: Association of Engineering Geologists Annual Meeting, 1981, Portland, OR, pp. 193-198.

McKee, B., 1972, *Cascadia*: McGraw-Hill Book Company, San Francisco, CA, 394 p.

Mundorff, M. J., 1964, *Geology and Ground-Water Conditions of Clark County, Washington, with a Description of a Major Alluvial Aquifer Along the Columbia River*: U.S. Geological Survey Water-Supply Paper 1600, 268 p.

Pacific Power and Light Company, 1987, *Swift Dam*: Pacific Power and Light Company, Portland, OR, 13 p.

Schuster, R. L., 1981, Effects of the eruptions on civil works and operations in the Pacific Northwest. In Lipman, P. W. and Mullineaux, D. R. (editors), *The 1980 Eruptions of Mount St. Helens, Washington*: U.S. Geological Survey Professional Paper 1250, pp. 701-718.

Tilford, N. R., and Sullivan, J. G., 1981, Lewis River Hydroelectric Development. In *Field Trip Guidebook, Engineering Geology in the Pacific Northwest*: Association of Engineering Annual Meeting, 1981, Portland, OR, pp. 156-185.

Wilkinson, W. D.; Lowry, W. D.; and Baldwin, E. M., 1946, *Geology of the St. Helens Quadrangle, Oregon*: Oregon Department of Geology and Mineral Industries Bulletin 31, Salem, OR, 39 p.

Williams, I. A., 1930, *Report on geologic conditions at Ariel dam site, North Fork of Lewis River, Washington*: Northwestern Electric Company, Portland, OR, 3 vol., 2 plates.

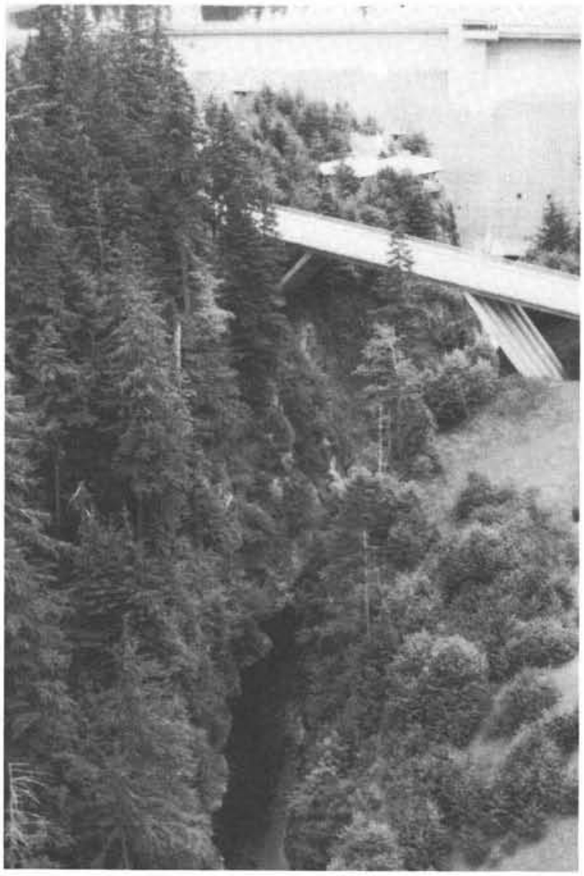
Dams of the Olympic Peninsula

Introduction and Geologic Setting

Elwha River Dams

Skokomish River Projects

Wynoochee Dam



Aerial view of Wynoochee Dam and canyon. Photograph by R. W. Galster, August 1980.

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

Dams of the Olympic Peninsula: Introduction and Geologic Setting

RICHARD D. ECKERLIN
U.S. Army Corps of Engineers

INTRODUCTION

The Olympic Peninsula is bounded by the Pacific Ocean on the west, Hood Canal and Puget Sound on the east, and the Strait of Juan de Fuca on the north (Figure 1). To the south, the peninsula has a less pronounced boundary, the Chehalis River. The Olympic Peninsula is part of the coast ranges within the Pacific Border Physiographic Province. Mount Olympus is the highest peak at elevation 7,965 ft. Most of the peaks, however, are less than 6,000 ft elevation. Pacific storms tend to contribute heavy precipitation to the mountains. The Olympics are drained by the many rivers radiating from the central part of the range as shown on Figure 1. The Elwha, Wynoochee, and North Fork Skokomish rivers have been harnessed by five dams, which are discussed in subsequent papers. These dams include both the oldest major dam in western Washington (Elwha) and the newest (Wynoochee), providing an interesting contrast in dam construction over a 75-yr period.

GEOLOGY

The Olympic Mountains are composed of a core of marine metasedimentary rocks of Eocene and Miocene age bounded on three sides by a horseshoe-shaped outcrop belt of volcanic and sedimentary rocks. These two bedrock terranes have been designated the core and peripheral rocks (Figure 2) (Tabor and Cady, 1978a, b). Core rocks are divided into the eastern and western terranes (Stewart, 1974). Ages of core rocks progress westward from oldest to youngest. Western core rocks are mostly sandstone, siltstone, and minor conglomerate with scattered volcanic rocks in major shear zones. Complex folds and faults are common. The eastern core rocks consist of sedimentary rocks locally metamorphosed to slate, semischist, and phyllite. Also present are minor quantities of conglomerate, basalt, diabase, and gabbro. The eastern core rocks are sheared and broken and resemble melanges in many places (Tabor and Cady, 1978a).

Core rocks are separated from the peripheral rocks by steeply dipping faults interpreted by Tabor and Cady (1978a, b) as thrust faults. The oldest peripheral rocks belong to the Blue Mountain unit, which consists of argillite, conglomerate, and sandstone. These rocks underlie and are interbedded with the Crescent Formation, which consists of early and middle Eocene volcanic rocks. The widespread development of pillow structure in the Crescent basalt flows suggests a submarine origin. Other rock types present are sill-like bodies of diabase, flow breccia, and volcaniclastic and associated sediments. Eocene to Miocene and minor Pliocene fossiliferous marine sedimentary rocks overlie the Crescent Formation. In the northern Olympic Peninsula, these units include the Clallam, Lyre, and Aldwell formations and Twin River Group; the southern peninsula post-Crescent units include the Montesano, Astoria, and Lincoln Creek formations. All the peripheral rocks are folded and faulted and are stratigraphically continuous. The major dams of the peninsula are in the peripheral rock terrane.

The Olympic Peninsula is extensively mantled by Pleistocene deposits. During the last major glaciation (Vashon) of the Puget Lowland, the Puget lobe of the Cordilleran ice sheet pushed southwestward to a point 20 mi west of Shelton (Crandell, 1964). Glacial debris derived from the continental ice sheet rims the mountains on the northwest, north, east, and southeast. Many alpine glaciers descended the eastern mountain valleys almost to sea level and coalesced with the continental sheet. Moraines preserve the outlines of the glaciers, and outwash deposits fill the lower parts of the valleys. Alpine glaciers still occupy areas around several peaks in the central Olympics.

TECTONIC DEVELOPMENT

The Olympic Mountains are a tectonic province exhibiting extremely complex deformation. The whole Pacific Northwest underwent significant stress change during the Tertiary due to interaction of the North American and Pacific plates. Volcanism and tectonics resulted from subduction of the Juan de Fuca plate under the North American plate and from related compressive stresses. Tabor and Cady (1978a) consider the deformation of the Olympics to have taken place in four episodes. Shortly after the Crescent basalts and as-

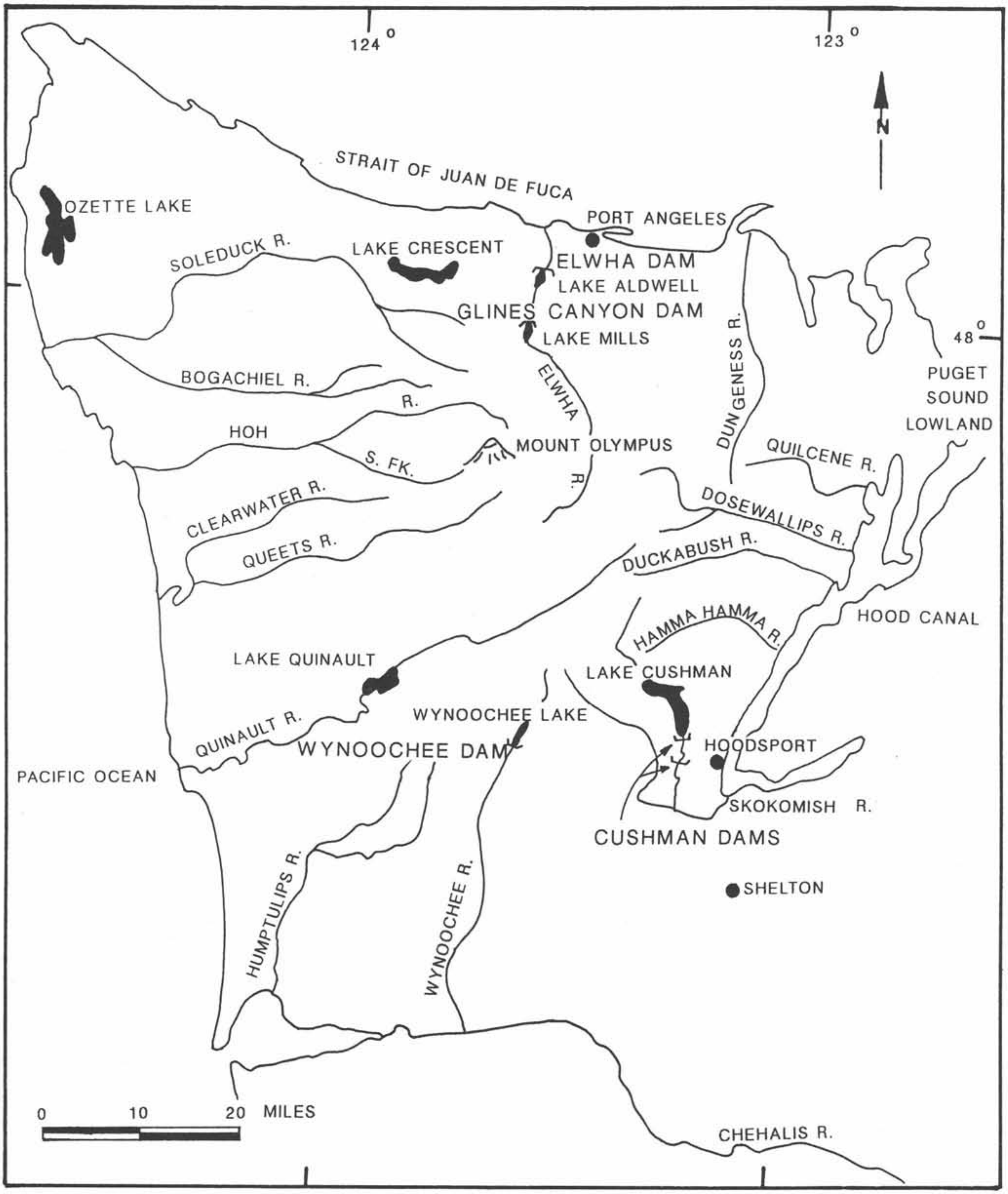


Figure 1. The Olympic Peninsula and locations of Elwha Dam, the two Cushman dams, Glines Canyon Dam, and Wynoochee Dam.

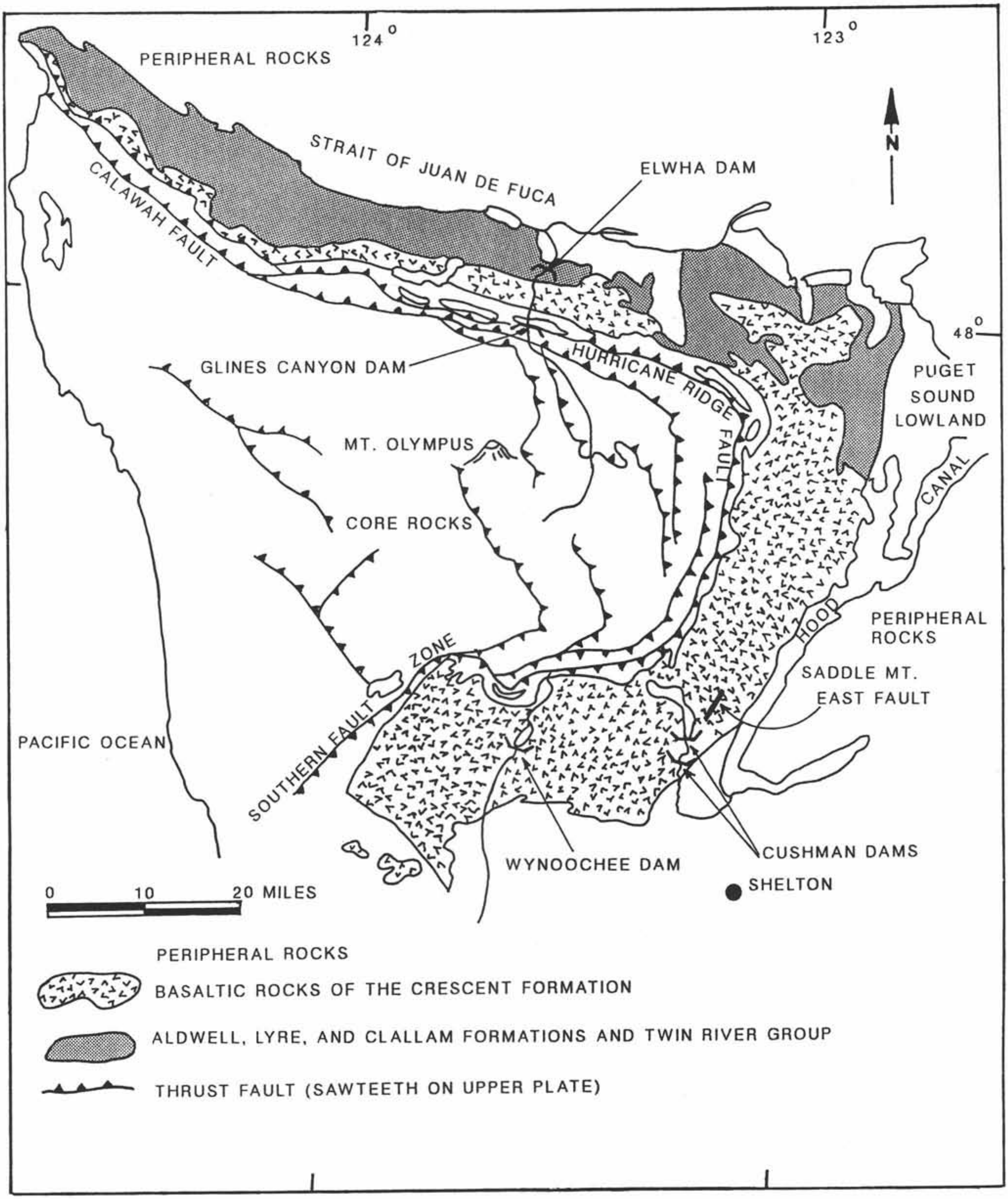


Figure 2. Generalized geologic map of the Olympic Peninsula showing geologic terranes and major thrust faults. Modified from Tabor and Cady (1978a, b).

sociated sediments were deposited, compression pushed these rocks on end along the eastern margin of the Olympics. Later the core rocks were moved eastward as a result of eastward compression in a series of episodes forming an accretionary prism under subduction-zone conditions.

Within the core bedrock terrane, succeedingly younger blocks are thrust under older units. In addition, core rocks are thrust under the peripheral rocks that wrap around the peninsula. On the northern side of the core, structures tend to curve around parallel to the peripheral Crescent Formation. The Calawah fault (Figure 2) and its probable westward extension separate the highly deformed core rocks from peripheral rocks. Eastward, the Calawah fault splays into several faults separating slaty units of the eastern core (Tabor and Cady, 1978a). West-northwest structural trends characterize most of the northern peninsula. The Hurricane Ridge fault separates rocks of contrasting lithologies in the north. The fault continues around the eastern side of the core into the southern Olympic Mountains and merges with the Southern fault zone, a zone of intense deformation, near Lake Quinault. The extension of faulting southwest of Lake Quinault is speculative because the area is covered by glacial outwash deposits.

Within the southern Olympics, the only known active fault is the Saddle Mountain East fault (Figure 2) located between Hood Canal and Lake Cushman (Wilson et al., 1979). It is a reverse fault, 1 mi long, that strikes N 26° E and dips 75° to the east. The fault displaces Pleistocene gravels 9 ft vertically. Last movement on the Saddle Mountain East fault appears to have occurred about 1,200 yr ago. This fault may be a surface branching of a deeper and more significant boundary fault manifest within Hood Canal.

REFERENCES

Crandell, D. R., 1964, *Pleistocene Glaciations of the Southwestern Olympic Peninsula, Washington*: U.S. Geological Survey Professional Paper 501-B, 5 p.

Stewart, R. J., 1974, Zeolite Facies Metamorphism of Sandstone in the Western Olympic Peninsula, Washington: *Geological Society of America Bulletin*, Vol. 85, pp. 1139-1142.

Tabor, R. W. and Cady, W. M., 1978a, *The Structure of the Olympic Mountains, Washington–Analysis of a Subduction Zone*: U.S. Geological Survey Professional Paper 1033, 38 p.

Tabor, R. W. and Cady, W. M., 1978b, *Geologic Map of the Olympic Peninsula, Washington:* U.S. Geological Survey Miscellaneous Investigations Series Map I-994, 2 sheets, scale 1:125,000.

Wilson, J. R.; Bartholomew, M. J.; and Carson, R. J., 1979, Late Quaternary Faults and Their Relationship to Tectonism in the Olympic Peninsula, Washington: *Geology*, Vol. 7, No. 5, pp. 235-239.

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

Elwha River Dams

WILLIAM S. BLITON
ATC Engineering Consultants, Inc.

INTRODUCTION

The Elwha River basin is the major drainage basin exiting on the northern flank of the Olympic Mountains. The basin consists primarily of a U-shaped glaciated valley trending north to northwest from the central Olympic Mountains to the Strait of Juan de Fuca. Peaks in the upper reaches of the mountain range rise to elevations in excess of 6,000 ft, and several glaciers are present in the higher elevations. Mean annual precipitation in the northern or lower elevations of the basin is on the order of 50 in. but ranges as high as 220 in. in the higher elevations. Two dams are located on the downstream portion of the river, the Glines Canyon and Elwha dams. The drainage basin above the upper dam, Glines Canyon Dam, is approximately 245 sq mi and above the lower dam, Elwha Dam, is 315 sq mi, including the Glines Canyon drainage area (U.S. Army Corps of Engineers, 1978).

AREAL GEOLOGY

The areal geology and the locations of the dams are shown on Figure 1. Geologic formations in the lower Elwha drainage consist of Eocene to Miocene Olympic core complex (Elwha Lithic Assemblage), the lower to middle Eocene Crescent Formation, the middle to upper Eocene Aldwell and Lyre formations, the upper Eocene to lower Miocene Twin River Group and the Miocene Clallam Formation (Figure 1) (Tabor and Cady, 1978).

The Olympic core complex is present in the core area of the Olympic Mountains and consists primarily of graywackes (poorly sorted sandstones) and volcanic rocks. These rocks originated as deposits of sand and silt that accumulated to thicknesses of many thousands of feet on a seafloor of an ancient ocean. Volcanic eruptions periodically interrupted the deposition of marine sediments; the eruptions are represented by interbedded volcanic flows. These core rocks have been intensely folded, faulted, and slightly metamorphosed. Some of the beds are now vertical or overturned.

The remaining formations are present downstream and stratigraphically overlie the Olympic core complex along the northern flank of the Olympic Mountains. The Crescent Formation ranges in thickness between 7,000 and 15,000 ft and consists primarily of marine basalt flows and flow breccia with interbedded sandstone, siltstone, shale, and tuff beds and minor amounts of chert and limestone. The Aldwell Formation is on the order of 2,900 ft thick and is composed of marine siltstone and sandstone with minor amounts of basalt and tuff in its lower part. The Lyre Formation is estimated to be 3,300 ft thick and is composed of marine conglomerate and sandstone. The Twin River Group, estimated to range between 6,000 and 12,000 ft thick, consists primarily of marine sandstone, siltstone, and shale and local thick beds of conglomerate. The Clallam Formation is estimated to be less than 2,500 ft thick and consists primarily of a sequence of marine and non-marine sandstone and siltstone with minor amounts of conglomerate and coal seams (McKee, 1972).

The regional topography has been greatly modified by several periods of glaciation during the Pleistocene. Continental glaciers advanced south out of Canada into the Puget Lowland area and west through the Strait of Juan de Fuca along the lower elevations of the Olympic Mountains. In addition, alpine glaciers in the Olympic Mountains, many times larger than the present-day glaciers, flowed down and shaped all of the major valleys by scour and deposition of glacial deposits. These valley glaciers may have reached the Strait of Juan de Fuca, but the evidence beyond the front of the range has been destroyed or greatly modified by the larger and younger continental glaciers.

ELWHA DAM

Project Description

Elwha Dam (Figure 2) is located between 4 and 5 mi upstream from the river mouth and approximately 7 mi west-southwest of the city of Port Angeles (Figure 1). The dam is located approximately 7.4 mi downstream from the Glines Canyon Dam. The Elwha Dam is operated as a run-of-the-river project for hydropower; there is no provision for flood storage.

The project consists of a concrete gravity structure with north and south spillway sections and an at-dam powerhouse (Figure 3). The dam, which has a height of approximately 100 ft and a crest length of 417 ft including the spillway and intake section, was constructed

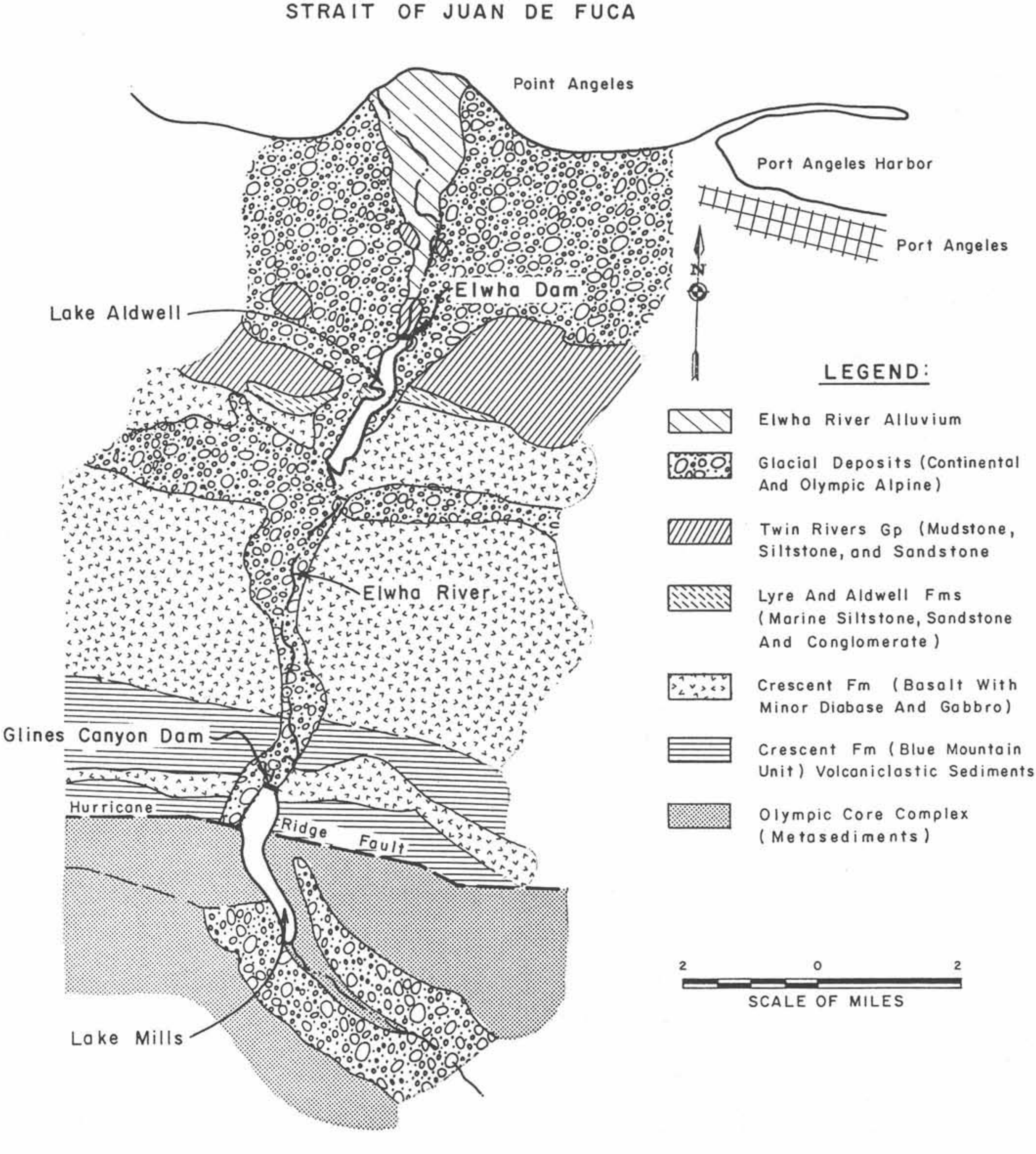


Figure 1. Generalized areal geologic map of the lower Elwha River showing locations of the Elwha and Glines Canyon dams. Adapted from Tabor and Cady (1978).

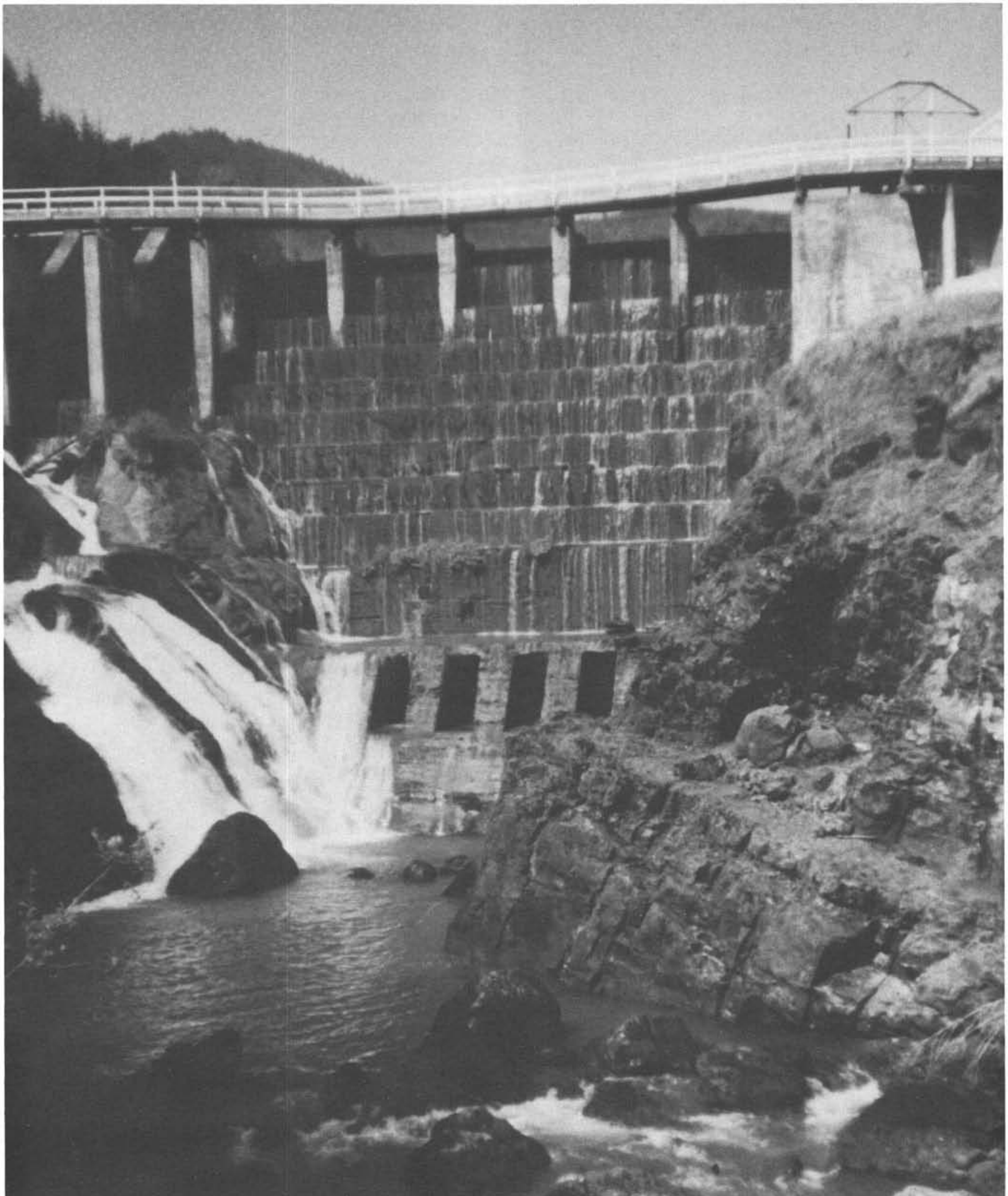


Figure 2. Elwha Dam; view upstream toward the main gravity section. Photograph courtesy of H. A. Coombs.

during 1911 and 1912. The original powerhouse, completed in 1913, contains two horizontal 3,000-KVA generators. In 1922 the powerhouse was expanded with the addition of two 3,300-KVA generators (U.S. Army Corps of Engineers, 1978). Reservoir impounding started in October 1912. The dam impounds approximately 7,600 acre-ft; the reservoir has a surface area of approximately 270 acres at the operating pool elevation of 187.0 ft. Lake Aldwell, the reservoir created by the Elwha Dam, is named after a former general manager of the Olympic Power Company, the original owner of the dam.

Site Geology

Elwha Dam is sited in the Twin River Group, and the reservoir is underlain by rocks of both the Twin River Group and Crescent Formation.

The dam is in a deep, narrow canyon eroded through a bedrock ridge where the strike is approximately normal to the trend of the river. Bedrock consists of pebble conglomerate with scattered clasts to 3 in. and mudballs as much as 6 in. in diameter; some shale is also present. In general the rock is weathered where exposed. The pebble conglomerate has widely spaced fractures and bedding planes. The thickness of this unit is estimated to be greater than 200 ft. Bedrock rises above the dam level on both abutments and forms the powerhouse foundation. The conglomerate beds strike approximately N 50° W and dip 20° N. Two joint sets are present; one set strikes N 30° E, dipping 75° NW and the other strikes approximately E-W and dips between 80° and 85° S. The combination of the high-angle fractures and the low-angle bedding planes gives the bedrock a blocky appearance.

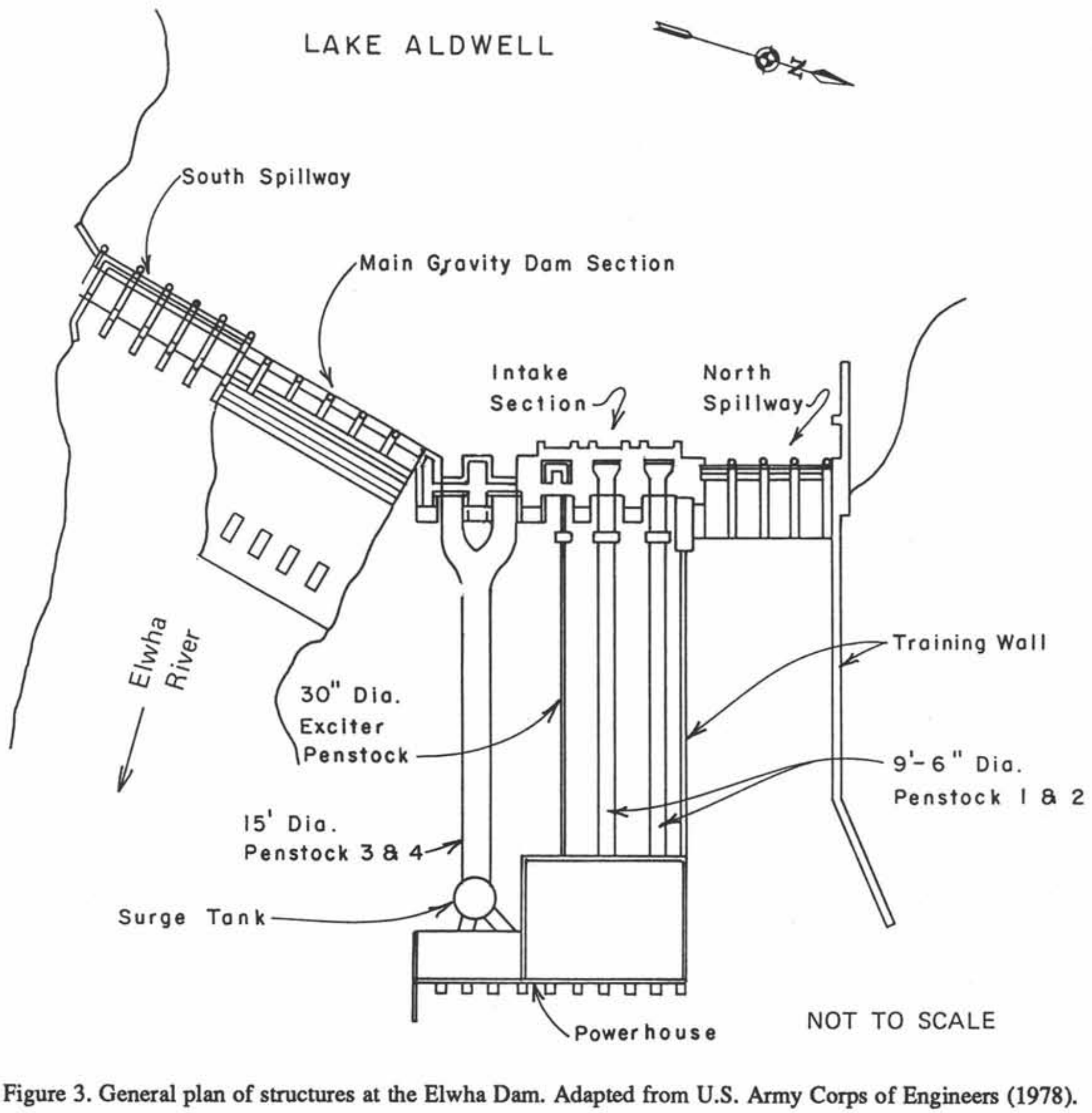


Figure 3. General plan of structures at the Elwha Dam. Adapted from U.S. Army Corps of Engineers (1978).

The river channel is underlain by alluvium estimated to extend to near sea level (U.S. Army Corps of Engineers, 1978).

Construction Problems Related to Geology

Construction began in 1911 after the completion of upstream and downstream cofferdams; diversion of river water was by a wooden flume over the top of the cofferdams. It was not possible to dewater the foundation area. This necessitated placement of concrete by bucket below water level. Above water level, the construction was completed by conventional methods. A caisson or rectangular opening approximately 26 ft by 9-1/2 ft was constructed in the central portion of the foundation with the intent to construct cut-off walls of reinforced concrete down to impervious material when deemed necessary. Prior to impounding of the reservoir, some leakage was noted at the downstream area of the dam, and approximately 3,000 cement bags filled with sand and gravel were placed on the river bed along the upstream face of the dam. This treatment reportedly had little effect on the leakage. Using the caisson structure in the foundation, upstream and downstream cut-off walls were constructed down to elevation 49.0 ft. During the same period, a row of sheet piling was driven across the river bed approximately 8 ft downstream of the toe of the dam. It was estimated the piling penetrated to elevations between 20 and 30 ft. On October 18, 1912, the sluice gates were closed, and the reservoir gradually rose to an elevation of approximately 184 ft, very near its maximum operating level.

On the evening of October 30, 1912, leakage beneath the dam resulted in erosion of the upper part of the alluvium. The reservoir is reported to have emptied in approximately 2 to 3 hr. Theory regarding the reason for the extensive erosion of the sand and gravel beneath the dam was that the obstruction formed by the downstream sheet pile wall resulted in a concentration of flow and caused an increase in the exit velocity between the sheet pile wall and the toe of the dam, resulting in the rapid erosion (Terzaghi and Peck, 1948).

Shortly after this failure, soundings completed at the face and toe areas of the dam indicated the extensive erosion of alluvial deposits. Soundings completed approximately 1 month later indicated that a considerable portion of the eroded area had been infilled by new river gravels (U.S. Army Corps of Engineers, 1978).

Remedial measures completed shortly after the failure consisted of additional sheet piling, a downstream concrete-backfilled caisson, and rock and earthfill blankets both upstream and downstream of the dam (Figure 4). Downstream of the dam, three rows of sheet piles were installed at distances of 10, 70, and 150 ft from the toe of the dam. These sheet piles were driven flush with and across the streambed; pile lengths ranged between 40 and 50 ft. Between the first two downstream rows of sheet piles, a rectangular caisson approximately 17 ft by 37 ft was formed by driving sheet piles with lengths between 40 and 50 ft. The caisson was excavated to a depth of 30 ft and backfilled with concrete. Upstream of the dam, two rows of sheet piles were driven flush with and across the river bed at distances of 15 and 94 ft. The pilings in the row 15 ft upstream of the dam were driven to refusal or bedrock at a depth of approximately 80 ft, whereas the upstream row of piling was driven to a depth of approximately 40 ft. Rock and earthfill upstream and downstream of the dam was placed by blasting rock from the steep canyon walls directly into the river bed. An estimated 20,000 cy of blanket materials were deposited in the downstream area, and an estimated 50,000 cy were placed in the upstream area. Large quantities of earth were placed over the rockfill in the upstream area to fill voids (U.S. Army Corps of Engineers, 1978).

The remedial measures described above resulted in the reservoir filling in 3 days. However, the project continued to experience considerable leakage. Additional measures to reduce the seepage consisted of the placement of mats constructed of fir boughs in an area extending 150 ft upstream of the dam. Individual mats measured 30 ft by 50 ft and 3 ft thick. These mats were in turn covered by fill consisting of loose rock and soil. The mats greatly reduced the amount of leakage (U.S. Army Corps of Engineers, 1978).

Leakage gradually increased between 1914 and 1919, and a hydraulic fill was placed in the area extending approximately 300 ft upstream of the dam. Subsequently, an approximately 3-in.-thick layer of reinforced gunite was placed over the hydraulic fill in that areas. This resulted in considerable reduction in the leakage, and no additional control measures have been required. These seepage reduction measures have resulted in essentially infilling the reservoir to an elevation of approximately 170 to 175 ft; water impoundment depth near the dam is only 10 to 15 ft (U.S. Army Corps of Engineers, 1978).

A subsurface investigation completed in 1962 by Shannon & Wilson, Inc. indicated that a void area still remains beneath at least part of the dam. A boring through the lower downstream portion of the gravity dam encountered a void approximately 14 ft deep beneath the dam (U.S. Army Corps of Engineers, 1978).

GLINES CANYON DAM

Project Description

Glines Canyon Dam is located approximately 7.4 mi upstream of the Elwha Dam and approximately 11.9 mi upstream of the mouth of the Elwha River. The purpose of Glines Canyon Dam is to impound water for hydroelectric generation.

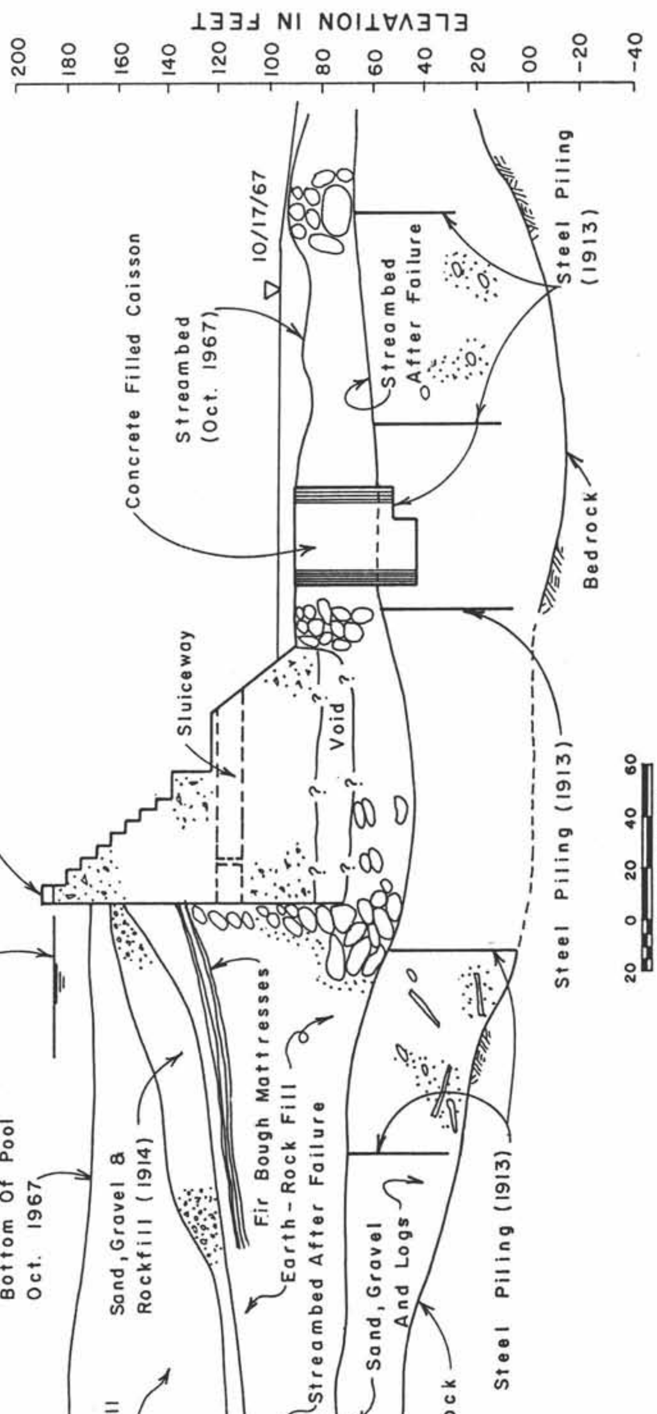


Figure 4. Transverse geologic section through the main gravity dam, Elwha Dam. Adapted from U.S. Army Corps of Engineers (1978).

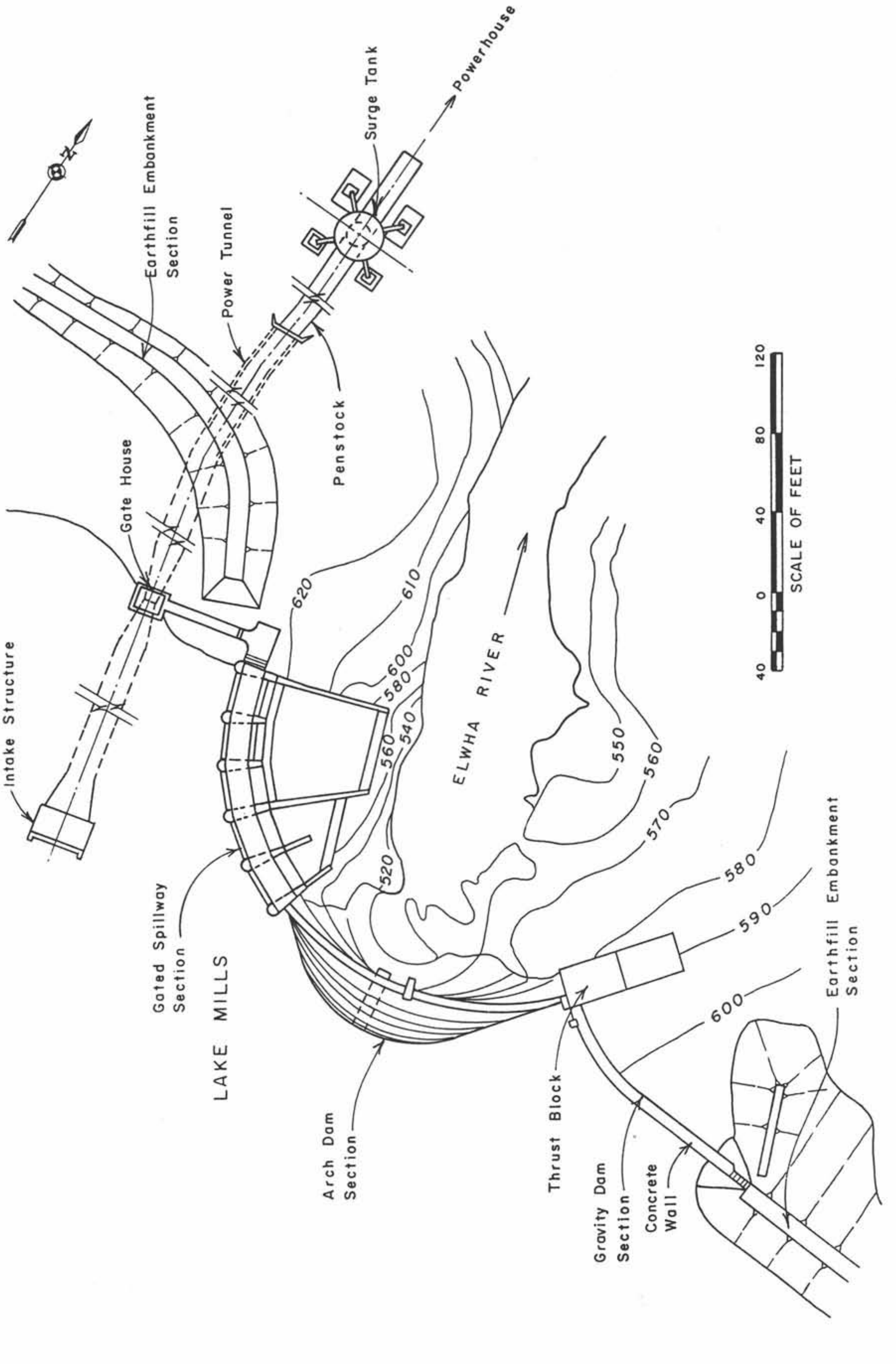


Figure 5. General plan of Glines Canyon Dam. Adapted from Coombs (1966).

The project was constructed in 1927 and consists of a 210-ft-high concrete-arch dam with a thrust block and gravity dam on the right abutment and a gated spillway section on the left abutment (Figure 5). The concrete-arch portion of the structure was designed to function as an ungated spillway. In addition, earthfill embankment sections are present on both abutments. The dam has a crest length of 510 ft, which includes the gravity section on the right abutment and the spillway section on the left abutment. The main dam has a thickness of approximately 30 ft near the base and 4 ft at the crest. The ungated spillway section of the dam has a crest elevation of 610 ft; this corresponds to the normal maximum reservoir elevation. The gated spillway is equipped with radial gates and has a spillway crest elevation of 590 ft. The gravity dam section has a crest elevation of 620 ft, while the embankment sections have a crest elevation of 632 ft. The embankment sections have concrete core walls adjacent to dam structures. An inlet structure, power tunnel and penstock, surface surge tank, and powerhouse are located on the right abutment area.

The reservoir created by Glines Canyon Dam is referred to as Lake Mills and is approximately 2-1/2 mi in length. The reservoir has a maximum capacity of 39,100 acre-ft. Glines Canyon Dam is generally operated as a run-of-the-river operation; however, storage in Lake Mills fluctuates.

Site Geology

Bedrock near Lake Mills and Glines Canyon Dam consists of well-cemented graywacke and lesser amounts of compacted shales of the Crescent Formation (Coombs, 1966). The bedrock in this section of the valley was deeply scoured by an alpine glacier that locally overdeepened the channel. The west side of the lake consists primarily of bedrock that has a thin veneer of till and outwash deposits. The east side of the lake has almost continuous bedrock exposures, generally as bold cliffs immediately above the shoreline.

The dam site is located in a narrow bedrock canyon. Jointing in the rock strikes approximately east-west at right angles to the river, and it dips steeply to the south (upstream).

Construction Problems Related to Geology

The major project features are all founded on bedrock, and no major construction problems were reported. The orientation of the joint system could have been a problem, but the dam arch is laid out on such a tight curve that the vector component of the major thrust is almost at right angles to the major joint system. The orientation of the thrust keeps the joints in compression so that they do not serve as translation planes for possible shear action.

A re-entrant on the right abutment near the thrust block was caused by a combination of jointing and erosion from spillway discharge. Analysis of the re-entrant showed that there is virtually no problem with structure stability in event of failure of rock (Coombs, 1966).

In 1938, a landslide occurred approximately 500 ft from the transformer yard. Evidence reviewed in 1966 indicated that the slide was probably triggered by an avalanche from the mountains high above the area. The slide did not encroach closer than approximately 500 ft to the transformer yard, and any future sliding in the area would not come any closer to the facility (Coombs, 1966).

REFERENCES

Coombs, H. A., 1966, *Geologic Report on the Glines Reservoir, Dam and Appurtenant Works*: Prepared for R. W. Beck and Associates, Seattle, WA, 2 p.

McKee, B., 1972, *Cascadia*: McGraw-Hill Book Company, San Fransisco, CA, 394 p.

Tabor, R. W. and Cady, W. M., 1978, *Geologic Map of the Olympic Peninsula, Washington*: U.S. Geological Survey Miscellaneous Investigations Series Map I-994, 2 sheets, scale 1:125,000.

Terzaghi, K. and Peck, R. B., 1948, *Soil Mechanics in Engineering Practice*: John Wiley & Sons, Inc., New York, NY, 566 p.

U.S. Army Corps of Engineers, 1978, *Phase 1 Inspection Report, National Dam Safety Program, Elwha River Basin, Elwha Dam, Port Angeles, Washington, WA-242*: U.S. Army Corps of Engineers, Seattle District, Seattle, WA, 41 p.

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

The Skokomish River Projects: Cushman Dam No. 1 and Cushman Dam No. 2

HOWARD A. COOMBS
University of Washington

INTRODUCTION

Both Cushman dams are on the lower stretch of the North Fork of the Skokomish River on the southeastern side of the Olympic Peninsula near the southern end of Hood Canal (Figure 1). The Skokomish River originates in the southeastern corner of the Olympic National Park and flows southeastward until it reaches the waters of Hood Canal. The City of Tacoma constructed these two dams. As part of this project, transmission lines had to cross southern Puget Sound at the Tacoma Narrows, a 6,240-ft span, then the longest in the world.

CUSHMAN DAM NO. 1

Project Description

Cushman No. 1, one of the first major dams in the west, was completed in 1926. The dam is a concrete structure rising 235 ft above the river bed (Figure 2). The central portion consists of a single curvature arch; its crest is 400 ft long. The entire dam measures 1,111 ft in length. A low-level valved by-pass is provided near the center of the arch dam. At each end of the arch are concrete gravity thrust blocks about 45 ft high. Gravity nonoverflow wing dams flank the arch on both sides. A low earthfill dam 180 ft long, with a concrete core wall, adjoins the gravity section on the left bank (Figure 3).

The remote spillway is located on the right bank approximately a quarter of a mile from the dam. The power tunnel intake is a separate structure upstream of the arch dam to the left of the river channel. The powerhouse is on the left bank downstream from the dam and is fed by a power tunnel 540 ft long and 17 ft in diameter (Figure 3).

Dam Site Geology

Rocks of the Crescent Formation form the foundation for both Cushman dams. Although the thickness of the Crescent Formation is not known because of faulting, it could be as much as 10 mi (Cady et al., 1972; Tabor and Cady, 1978a, b). In spite of the considerable deformation of these rocks, they retain their lithologic identity as the peripheral rock unit of the Olympic Peninsula. These basalts and associated sediments, including some limestones, are considered to be essentially oceanic in origin. Although the basalts are broken into small blocks because of the intense deformation, locally they retain good columnar jointing, as can be seen along the Hood Canal highway.

The Crescent Formation in the general area of the Cushman No. 1 Dam dips away from and downstream of the dam. Most of the units within the formation are highly fractured (Butler, 1964). The highest portion of the arch dam rests on a massive basaltic andesite flow that has pillow lavas at the base. Associated with the pillow lavas are black glass, calcite, and zeolites. Individual pillows are as much as 2 ft in width. Above the massive lavas on the left bank are agglomerates with blocks to 2 ft in diameter and showing glassy rims. Above this 125-ft sequence is a porphyritic and amygdaloidal andesitic flow that serves quite well as an abutment rock (Figure 4).

The right wing wall is founded on bedrock. The left wing wall is on an earthfill structure that has a concrete core. Several large protrusions of the rock surface into the concrete can be observed. Little information is available on foundation preparation for the arch dam.

Two major joint systems in the bedrock are exposed to the reservoir, lead under the structure, and emerge downstream of the dam. These joints, coupled with the fact that the dam has no known grout curtain, permit extensive leakage. This leakage, partially confined as hydrostatic pressure in the abutments downstream of the dam, could contribute to slumping and lack of support for the arch. Because of the small radius of the arch, there is very little rock on the outside of the concrete between the arch and river. E. M. Fusic of Harza Engineering Company (1964) recommended an abutment drainage system consisting of adits on each side of the dam a little above river elevation. These remedial measures were taken in 1966. From the adits NX holes were drilled into the abutments to relieve the pressure. The drain holes penetrate about 2 ft into the dam concrete. A grouting program also was recommended and carried out at this time to lessen the leakage through the joint system. A drainage adit was driven 240 ft into the right abutment to intercept seepage and relieve water pressure in the joint system. A total of 30 open and

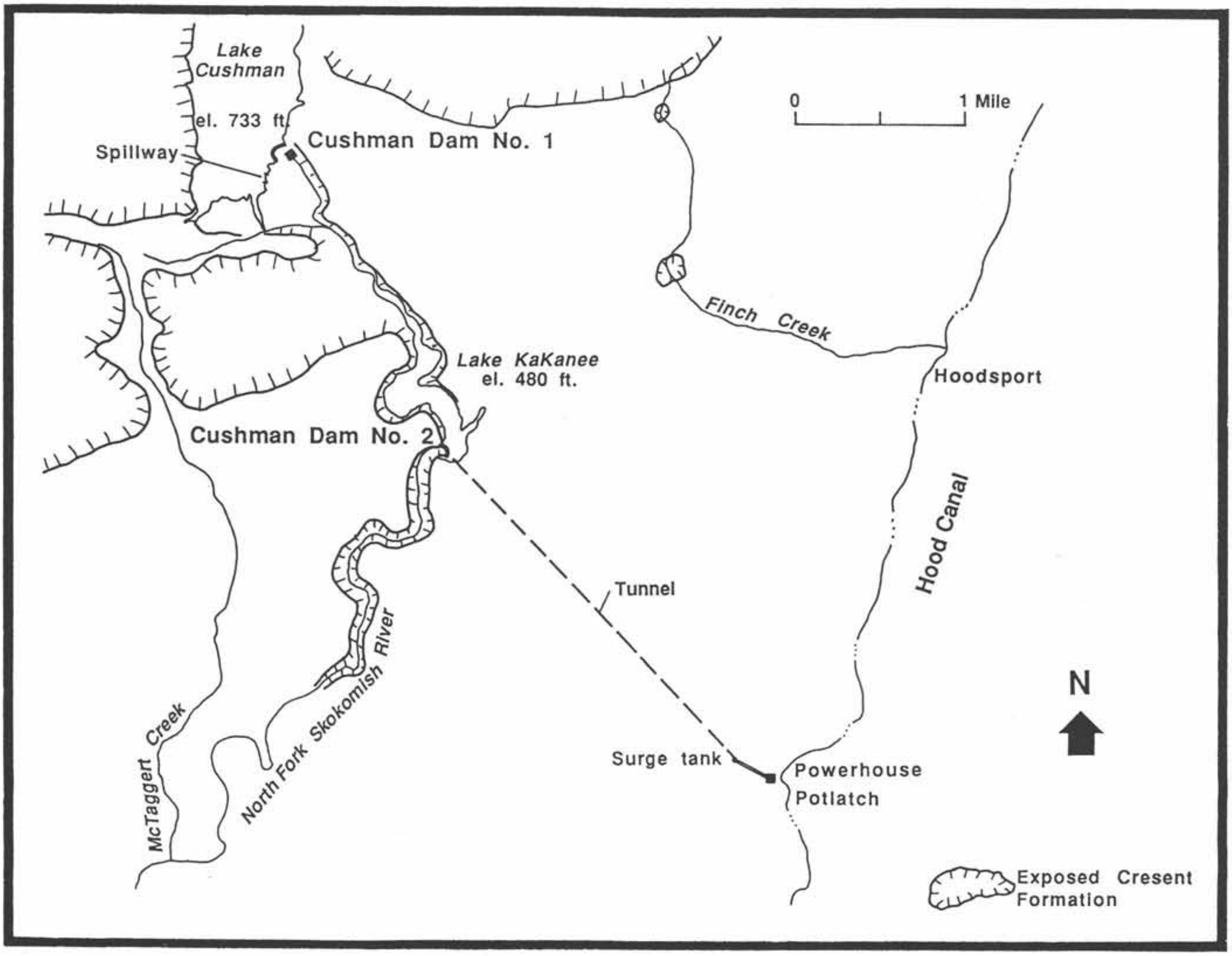


Figure 1. Location of Cushman Dams No. 1 and No. 2.

drainage joints were intercepted by both adits. Surface seepage on the right abutment nearly ceased after this treatment.

Spillway

The spillway is in a saddle about a quarter of a mile from the dam along the right rim of the reservoir. The spillway structure itself is a concrete gravity section with a concrete lined spill surface extending 900 ft down a sloping surface with a 103-ft drop in elevation. Six-in.-diameter drains were placed under the concrete slab and guide walls. The drains under the spillway have been functioning properly for many years (Coombs, 1972). The dam is periodically inspected as part of a regular maintenance program. The spillway is an ungated structure with 3 ft of flashboards added to increase storage in the reservoir from March to September. The structure is founded on glacial outwash, as are the adjoining short dikes. A concrete cut-off wall with a grout curtain extends through the dikes and under the spillway, but it is reported as not extending to bedrock, only to a depth of about 30 ft. The material below the cut-off wall is of such a nature that during construction the wall could not be deepened due to "flowing" sand. Drains beneath the chute slab empty into main drains, which discharge on the outside of the training walls at the downstream ends.

Although the concrete in Cushman Dam No. 1 is in good condition and the drains in the right abutment appear to be effective, the abutments are not capable of sustaining the structure during periods of prolonged or heavy overtopping of the dam. Not only is the rock cover thin between the arch abutment and the river, but the vertical jointing is roughly parallel to the river and would lend itself to stripping from overflow.

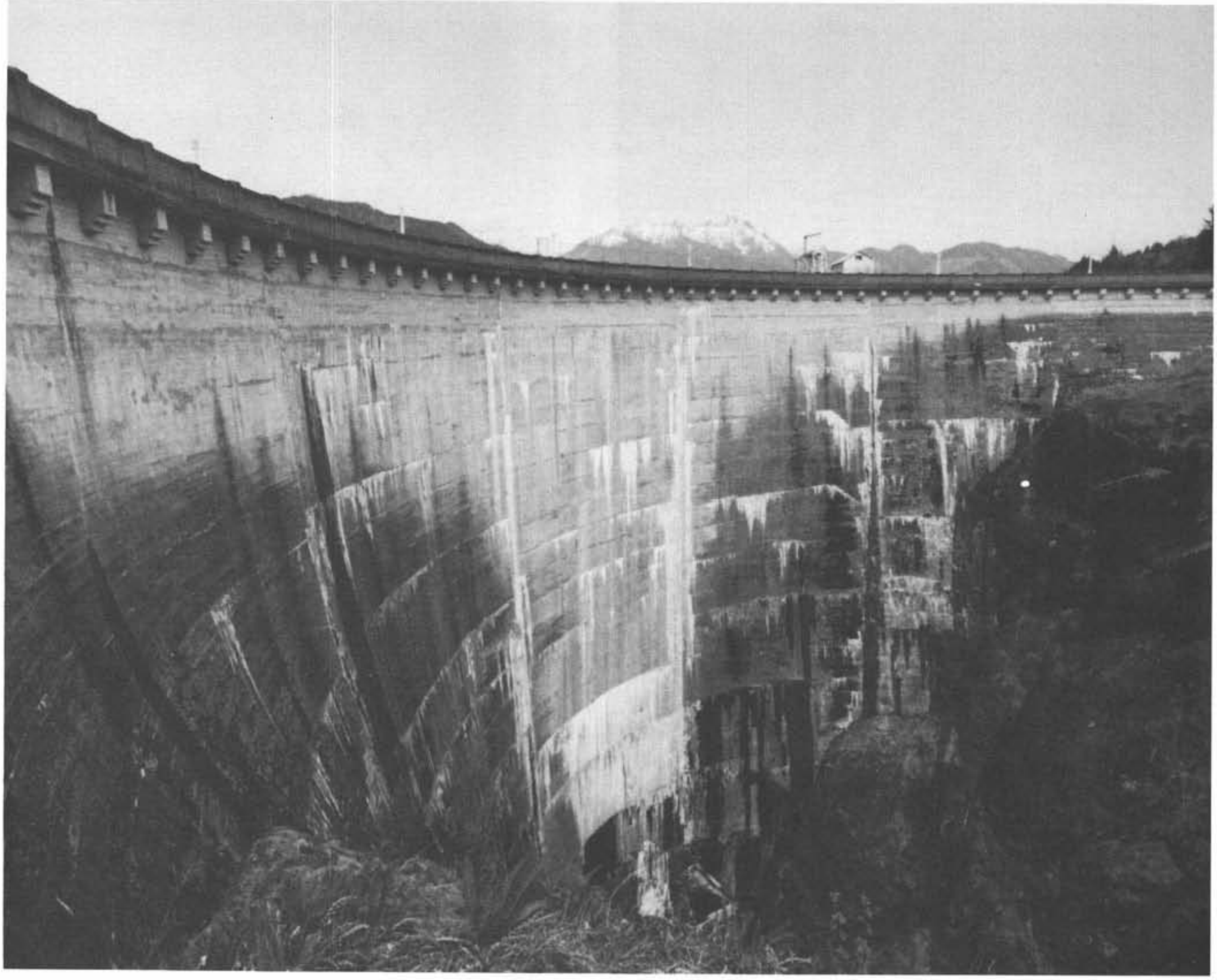


Figure 2. Cushman Dam No. 1. Photo by Tacoma City Light.

To insure against excessive overtopping, the flashboards that extend 3 ft above the spillway crest were set to trip with overtopping of 3 ft. A severe overtopping might go to the right of the spillway and cut a new channel in the sands and gravels. This new channel might be cut to a depth of 40 or 50 ft to bedrock. In this instance overtopping to the right might act as a "fuse plug". The safety provided by the "fuse plug" condition is quite important, considering the nature of the spillway foundation.

CUSHMAN DAM NO. 2

Project Description

Cushman Dam No. 2 (Figure 5), completed in 1930, is a concrete arch rising 175 ft above river level and is 400 ft in length (Figure 6). The spillway is located on the right abutment, and the power tunnel intake is located on the left abutment a short distance from the arch. The power tunnel is 13,000 ft long, 17 ft in diameter, and fully lined with concrete. Steel lining extends upstream from the powerhouse to just beyond the surge tank. The remote powerhouse can be seen from U.S. Highway 101 along Hood Canal between the towns of Hoodsport and Potlatch (Figure 1).

Dam Site Geology

Cushman Dam No. 2 is 2 mi downstream from Cushman Dam No. 1; there is little change in general rock structure and rock types. There are thin breccia zones between the massive andesites at Cushman Dam No. 2. All flows and interbeds are steeply dipping, and the rock is fractured owing to several periods of rather intensive deformation. A section along the dam axis is shown in Figure 7.

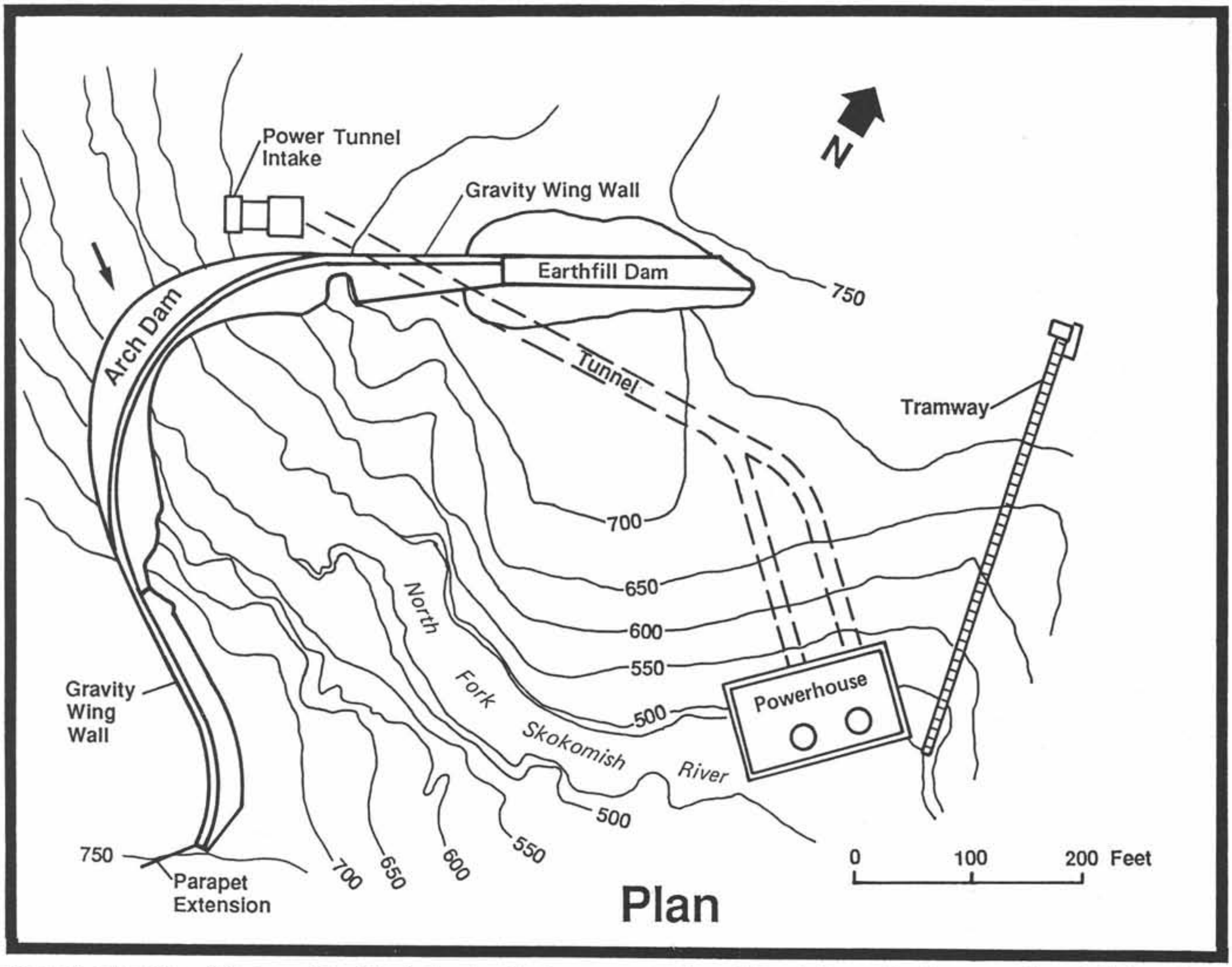


Figure 3. Plan view of Cushman Dam No. 1 showing the sharp curve to the arch in relation to the abutments. There is little rock exposed on the right abutment away from the concrete dam.

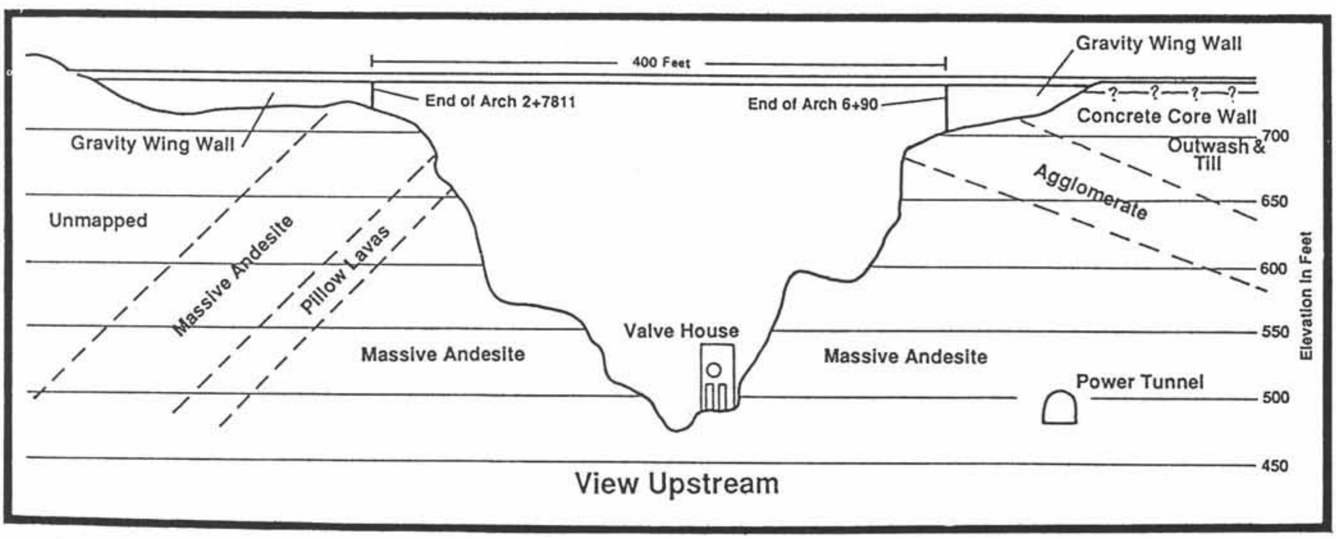


Figure 4. Profile of Cushman Dam No. 1; view upstream. Profile is along centerline of curving dam.

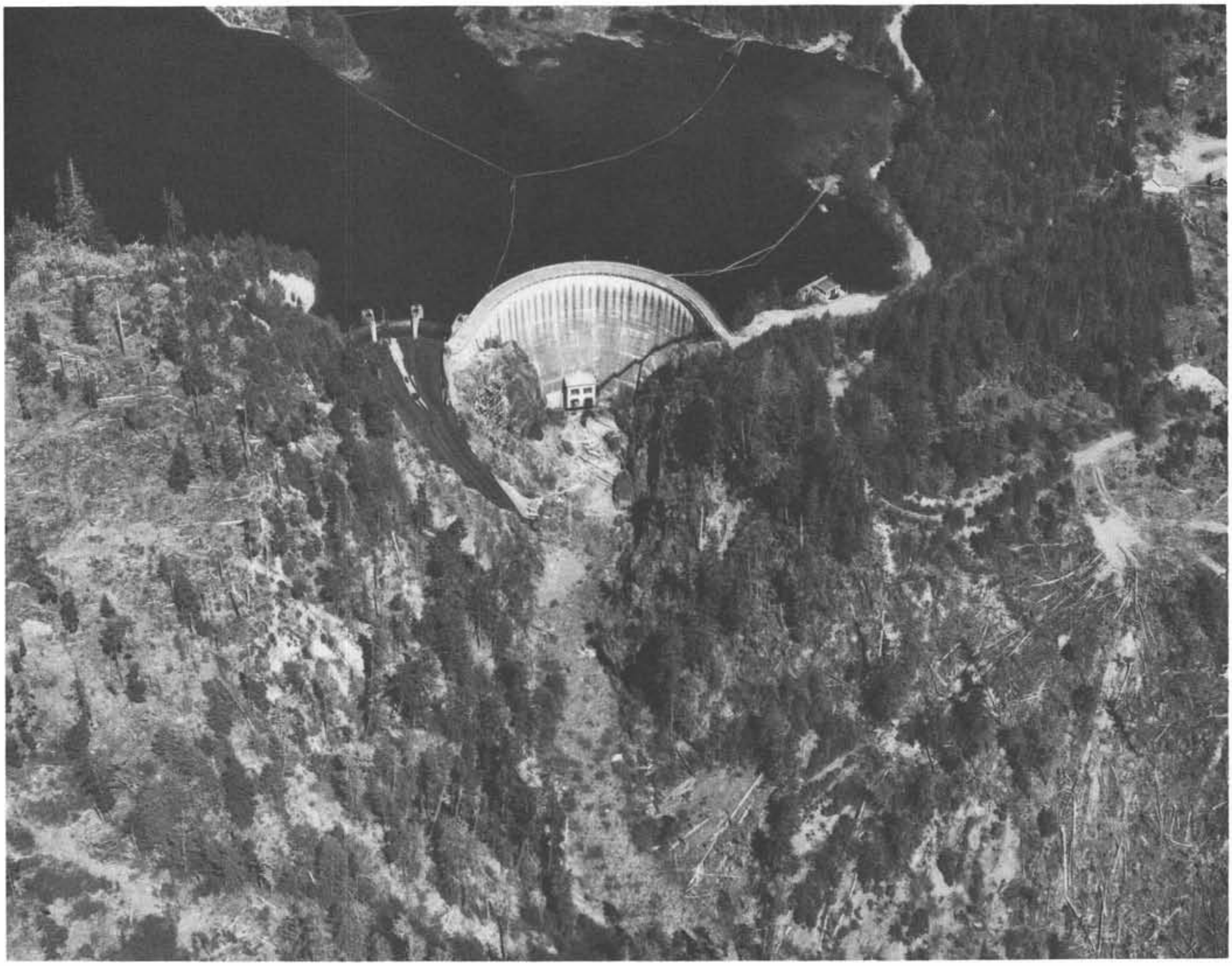


Figure 5. Cushman Dam No. 2. Note intake structure to the right of the dam and spillway on the left. Photo by Tacoma City Light.

The left abutment, though stronger than the right, has a vertical joint system parallel to the river. Originally it was thought that heavy overtopping of the dam might strip off rock from this abutment. However, many decades have passed with no overtopping.

Shortly after completion of the dam a large flood passed through the spillway, scouring a deep trench down the slope below the spillway chute. About this time a diagonal crack developed across the floor of the spillway. The crack, roughly parallel to the line of the arch thrust, extended down the face of the abutment rock. In 1947 L. F. Harza of Harza Engineering Company recommended grouting the crack, sealing it on the upstream side of the abutment, and drilling a number of drain holes to relieve the hydrostatic pressure in the crack and the joints in the rock mass. These recommendations were carried out. In 1960 the pool was raised to its maximum height with no significant changes in abutment conditions.

The original crack in the spillway floor follows the horizontal trace of a steeply dipping basaltic breccia layer between much more competent and solid basalt and andesite flows. In all probability the weaker breccia layer yielded slightly to the stresses imposed by the arch during times of maximum pool. The grouting and draining, suggested by Harza Engineering Company, have stabilized the right abutment so that normal operation of the reservoir can be accomplished. The 1972 inspection showed no renewed movement along this previous crack.

ACKNOWLEDGMENTS

The writer wishes to acknowledge the cooperation of T. Coates, Manager of the Department of Public Utilities, City of Tacoma, and L. H. Larson, Chief Civil Engineer, Light Division, Department of Public Utilities.

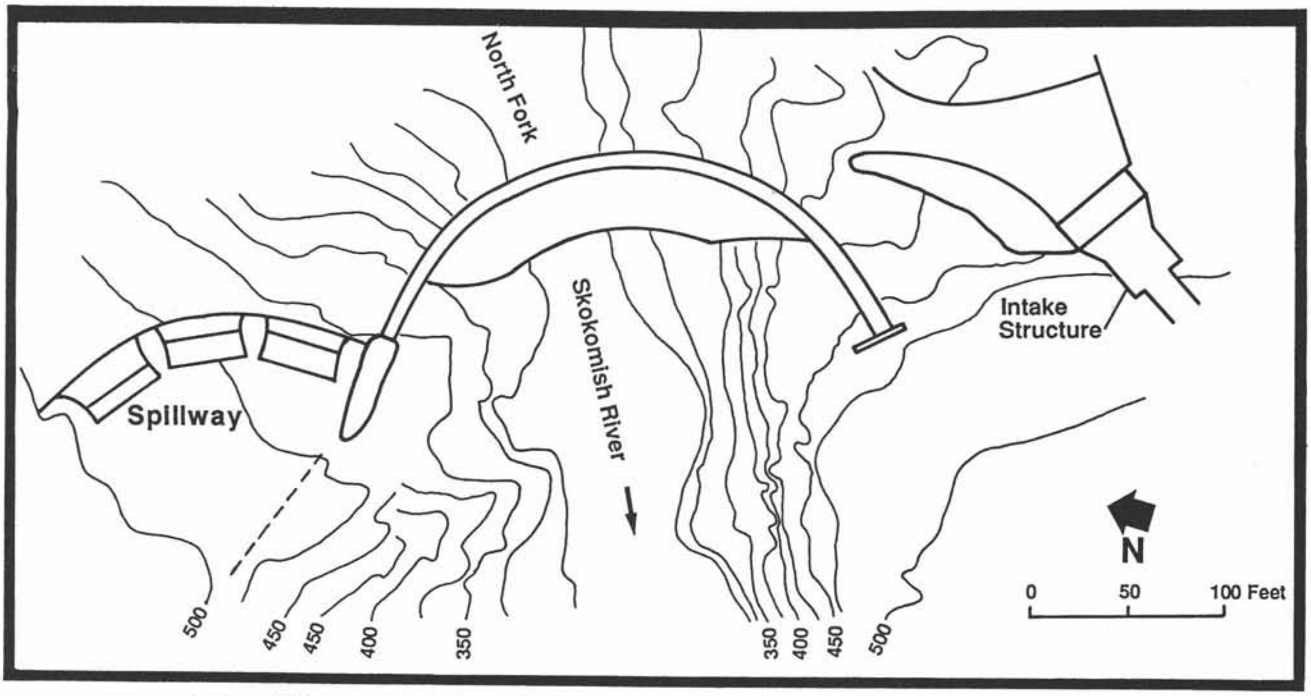


Figure 6. Plan of Cushman Dam No. 2. Dashed line across spillway is trace of small crack in spillway floor.

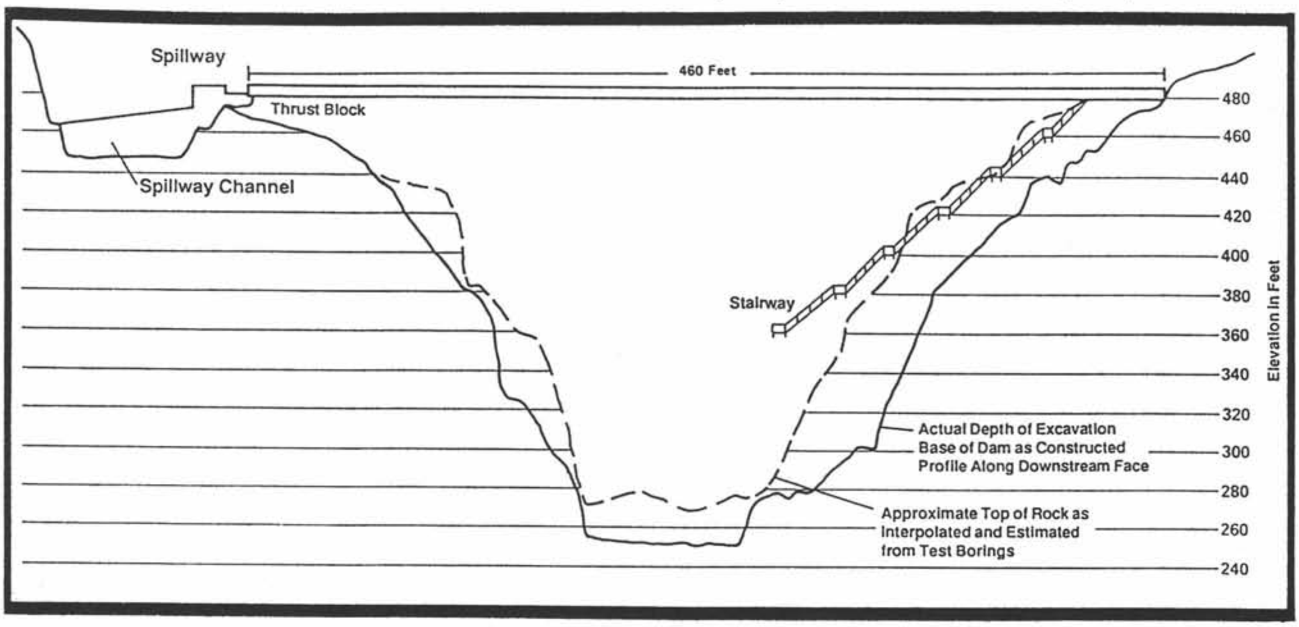


Figure 7. Section at Cushman Dam No. 2; view upstream.

REFERENCES

Butler, C. D., 1964, *Geology of Cushman Dam No. 1, Mason County, Washington*: Report to City of Tacoma, Department of Public Utilities, Light Division, Tacoma, WA, 20 p.

Cady, W. M.; Tabor, R. W.; MacLeod, N. S.; and Sorensen, M. L., 1972, *Geology of the Tyler Peak quadrangle, Washington*: U.S. Geological Survey Geologic Quadrangle Map GQ-970, 1 sheet, scale 1:62,500.

Coombs, H. A., 1972, *Report on Cushman Dam No. 1 and No. 2 projects*: Report to City of Tacoma, Department of Public Utilities, Light Division, Tacoma, WA, 5 p.

Fusic, E. M., 1964, *Abutment treatment Cushman Dam No. 1*: Report to City of Tacoma, Department of Public Utilities, Light Division, Tacoma, WA, 5 p.

Harza, L. F., 1947, *Report on Cushman Dam No. 2*: Report to City of Tacoma, Tacoma, WA, 5 p.

Tabor, R. W. and Cady, W. M., 1978a, *The structure of the Olympic Mountains, Washington; Analysis of a subduction zone*: U.S. Geological Survey Professional Paper 1033, 38 p.

Tabor, R. W. and Cady, W. M., 1978b, *Geologic map of the Olympic Peninsula*: U.S. Geological Survey, Miscellaneous Investigation Series Map I-994, 2 sheets, scale 1:125,000.

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

Wynoochee Dam

RICHARD D. ECKERLIN
U.S. Army Corps of Engineers

PROJECT DESCRIPTION

Wynoochee Dam is located on the Wynoochee River in the Weatherwax basin in the southern Olympic Mountains (Figure 1). The dam lies 51.8 river miles above the confluence with the Chehalis River. The project provides flood control, industrial water supply, and recreation and enhances the fishery.

Wynoochee Dam is a concrete gravity structure with zoned earth and rockfill embankments on both abutments. The 175-ft-high concrete gravity section is founded on basalt bedrock. The flanking embankments are founded on granular overburden materials, and their central semi-impervious cores are keyed to bedrock. Lengths of the right and left embankments, measured at the crest, are 211 ft and 817 ft, respectively. The entire dam is 1,691 ft long. A concrete-lined spillway is provided on the left (east) abutment. Water passage is normally through six low-flow outlets. Inlets are located at various levels within the concrete section for temperature control. The dam impounds a 4.4-mi-long reservoir in glacial sediments. Dam construction began in August 1969 and was completed in September 1972.

SITE GEOLOGY

General

Wynoochee River headwaters are in the core rocks bedrock terrane (Tabor and Cady, 1978). These rocks are mostly sandstone and conglomerate. Wynoochee River enters the peripheral rocks bedrock terrane about 8 mi upstream from the dam. The river continues south through peripheral rocks in the Wynoochee Valley. The valley peripheral rocks comprise a sequence of Tertiary sedimentary and volcanic rocks. Oldest are the middle Eocene Crescent Formation, consisting of basalt flows interbedded and overlain by thin beds of marine siltstone and sandstone (Rau, 1967). Wynoochee Dam lies on the Crescent volcanic rocks. From 8 mi upstream to 10 mi downstream from the dam, the Crescent rocks are mostly basaltic lava flows striking west to northwest and dipping to the south.

The Wynoochee Valley was repeatedly glaciated during the Pleistocene. Glacial deposition combined with interglacial stream erosion has produced a complex valley. Near the dam, Wynoochee Valley is 2 mi wide, U-shaped, and glaciated and bounded by rock ridges that rise 2,000 ft above the valley floor. The dam spans a 150-ft-wide canyon cut through the high point of a midvalley rock hill which is partly covered by glacial drift (Figure 2). The bedrock canyon extends several hundred feet upstream and more than 800 ft downstream from the dam.

Bedrock

Most of the bedrock in the area consists of south-dipping black to greenish gray basalt flows, including submarine pillow basalt. The rock is closely jointed and finely crystalline and has carbonate veinlets and zones of palagonite. Many joint surfaces are coated with unweathered dark chlorite. Locally, the basalt is cut by dark gray, moderately jointed diabase.

Bedrock beneath the dam and forming the canyon walls is characterized by numerous discontinuous, randomly oriented joints. Contraction joints, stress relief joints, and joints along flow contacts all complicate the rock structure and cause varied degrees of rock competency. Flow contacts are irregular, strike roughly northwest, and dip between 30° and 80° southwest. Some thin clay and fine sandy interbeds are present at flow contacts. Stress relief joints with uneven and rough surfaces dip toward the river in both canyon walls. Individual joints or flow contacts range in degree of openness from clean rock to rock surfaces to zones as much as 3 ft thick filled with weathered rock materials and clay.

Glacial Sediments

Wynoochee Valley is partly filled with till, morainal deposits, glaciofluvial deposits and with glaciolacustrine stratified deposits, including sand, gravel, silt, clay, and alluvium. Three glacial depositional units significant to the project occur in the reservoir area. The lowest unit, represented mostly by clays, occurs between elevations 700 and 750 ft. An intermediate unit composed of glacial till and compact sand and gravel is present between elevations 750 and 850 ft. The upper unit occurs between elevations 900 and 1,000 ft and is composed of a sequence of silt, sand, and gravel.

Figure 1. Aerial view to the northeast of Wynoochee Dam and Lake showing the broad glaciated valley (Weatherwax basin). The concrete section of the dam and the spillway are on the right. The spillway chute and both flanking embankments are concealed by the forest. Photograph courtesy of U.S. Army Corps of Engineers, Sept. 24, 1984.

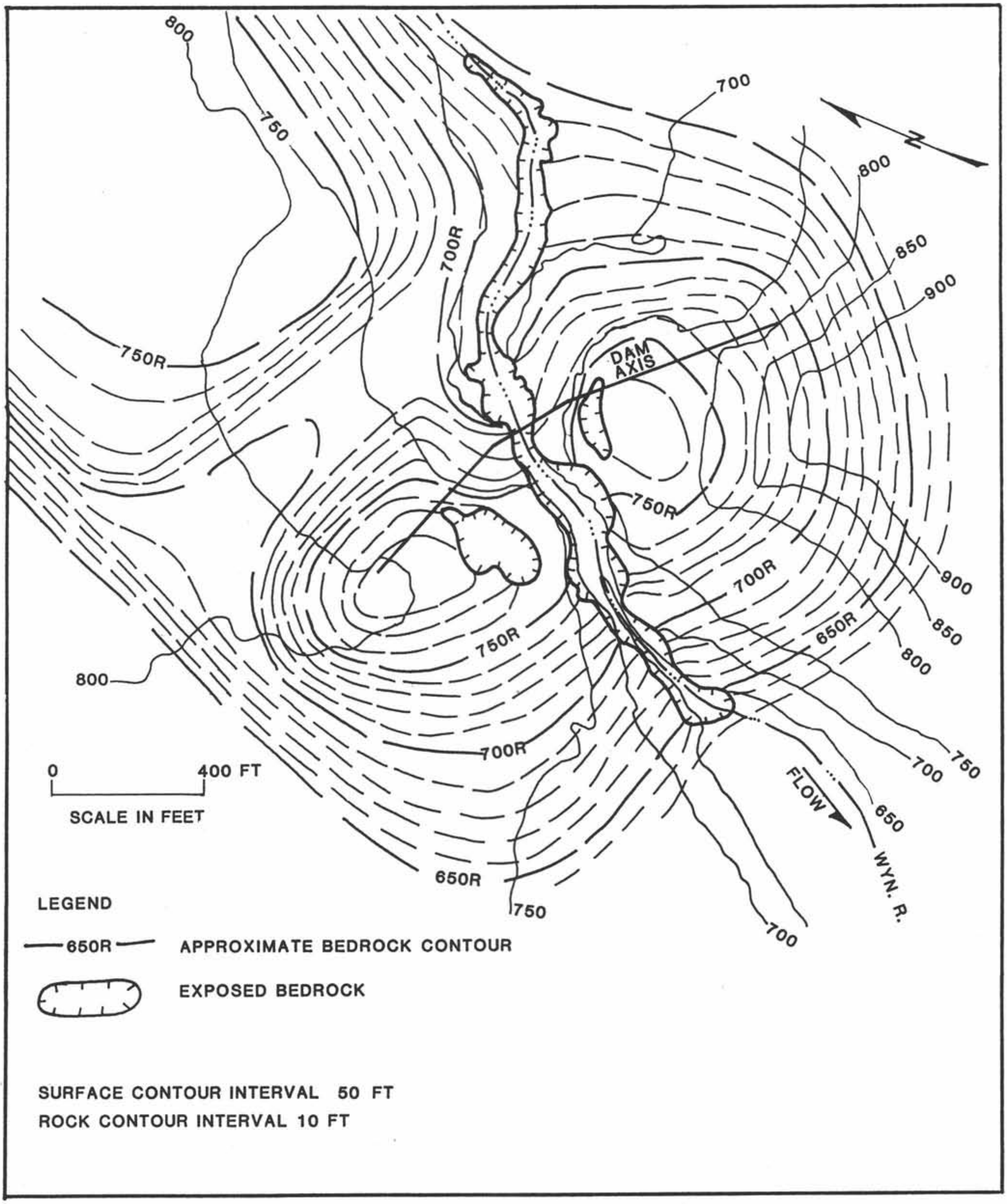


Figure 2. Bedrock contour map of the Crescent Formation - Quaternary sediment interface. The rock surface has been inferred from exploratory boring data. Dashed lines, designated by elevation followed by the letter R for rock, delineate the concealed rock surface. Solid lines are topographic contours. National Geodetic Vertical Datum.

The intermediate unit is responsible for numerous slides in the reservoir. All the slides are minor and are related to saturated glacial sediments. Reservoir slopes are periodically checked for potentially hazardous conditions.

Downstream from the dam, thick deposits of glacial sand and gravel overlie glaciolacustrine silt and clay. The fine-grained sediments are particularly susceptible to sliding. Landslides usually occur where the valley walls have been oversteepened, mainly along major bends of the river.

GEOLOGIC ASPECTS OF SITING AND DESIGN

Geologic reconnaissance mapping and exploratory drilling were completed during the general investigation in 1958 through 1961. Four possible dam sites at river mile 16, 42.5, 43.1, and 51.8 were examined. Two sites were found adaptable for construction of a project. The site at river mile 42.5 was determined suitable for a power dam; however, studies indicated that hydropower could not be produced economically at this location. Storage and other purposes could best be met by a dam at river mile 51.8.

The dam axis was located where the bedrock surface in the canyon wall is highest, thereby minimizing the thickness of the valley glacial deposits (Figure 3). Monoliths 1 through 5 on the right (west) abutment (Figure 4) were designed to be founded on the indurated till, but, after further exploration, the material was determined unsuitable and was removed down to the hard basalt bedrock surface. The right abutment of the dam is characterized by an east-dipping bedrock surface overlain by as much as 30 ft of compact sand and gravel (Figure 4). The right embankment central core is keyed entirely in bedrock to reduce seepage. A cut-off core trench extends from the right embankment upstream at a right angle to follow the reservoir shoreline for 1,000 ft. This trench is keyed either in glacial till or clay and is tied to an impervious clay blanket, which serves to control seepage around the right end of the dam. The left (east) abutment is characterized by a northeast-dipping bedrock surface (Figure 4). The left embankment transitions into a zoned upstream slope treatment section, which extends to a point approximately 700 ft beyond the end of the concrete dam. Approximately 400 ft of this section has a cut-off to rock, with the remainder bottoming in a cut-off on top of a clay layer.

A gravel filter blanket and seepage collection pipes are provided between 700 and 1,700 ft downstream from the dam on the left abutment (Figure 3). The pipes and blanket serve to control seepage, prevent erosion, and increase the stability in the seepage emergence area.

All construction material sources used for Wynoochee Dam were located within 2 mi of the project.

CONSTRUCTION PROBLEMS

No serious problems relating to foundation stability developed during construction. Examination of the geologic structure in the right canyon wall showed that the design rock excavation would be stable. However, as the rock slope was steepened to accommodate the contractor's diversion pipe, support for the rock mass in the upper slope was removed. The right canyon wall was redesigned, resulting in the monolith 5-6 joint line now lying on the slope instead of at the top of the cut. Solid core and groutable, 1-in.-diameter expansion shell rock bolts were installed in the right canyon wall to reinforce the rock mass. No progressive opening of major joints or evidence of mass instability has been observed in the exposed right bank rock excavation since bolt installation.

The lack of control over contractor blasting procedures resulted in extra excavation on the right canyon wall and in the left abutment spillway chute; in addition, a concrete wall on the right side of the spillway chute had to be designed and constructed. During the initial spillway chute excavation, several blast holes were overloaded, causing considerable damage outside the excavation limits. Overbreak occurred along the top left wall area, and a rib of rock along the right wall area broke out along an open joint. Original design called for using the rib of rock as lateral support for the right spillway wall. The rib was removed, necessitating construction of a free-standing wall on the right side of the spillway. Radial cracks surrounding a number of holes in spillway monoliths 10 and 11 indicated the holes were loaded and detonated below grade. This loosened rock required removal during foundation preparation. Only minor structural defects were found in the foundation, and these were readily corrected through standard bedrock foundation preparation and reinforcement techniques.

Grout injection quantities and current drainhole seepage records indicate that the foundation is generally tight, and overall condition of the dam foundation is excellent.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Uplift Pressure Development and Relief

A drainage curtain is used to intercept seepage and relieve possible hydraulic pressures downstream from the grout curtain. Instruments have been placed in the dam and abutments to measure structural behavior (including uplift pressure, joint and crack movement, and response to earthquake activity) and internal drainage and abutment seepage and to insure safety.

Gradually increasing uplift pressures due to calcification of the foundation drainholes were noted from 1975

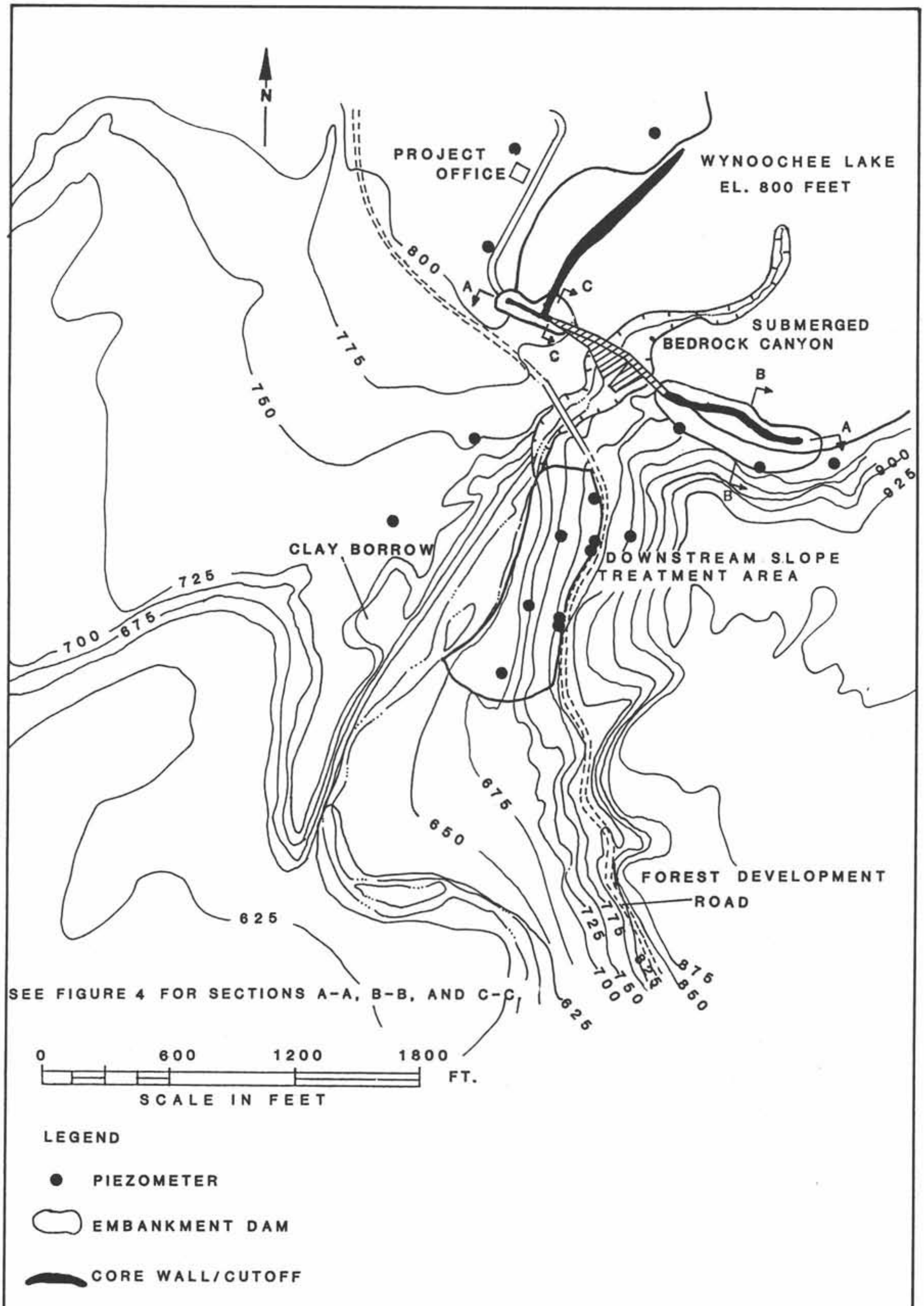


Figure 3. Topographic contour elevations from Wynochee Dam to approximately 3,000 ft downstream. National Geodetic Vertical Datum.

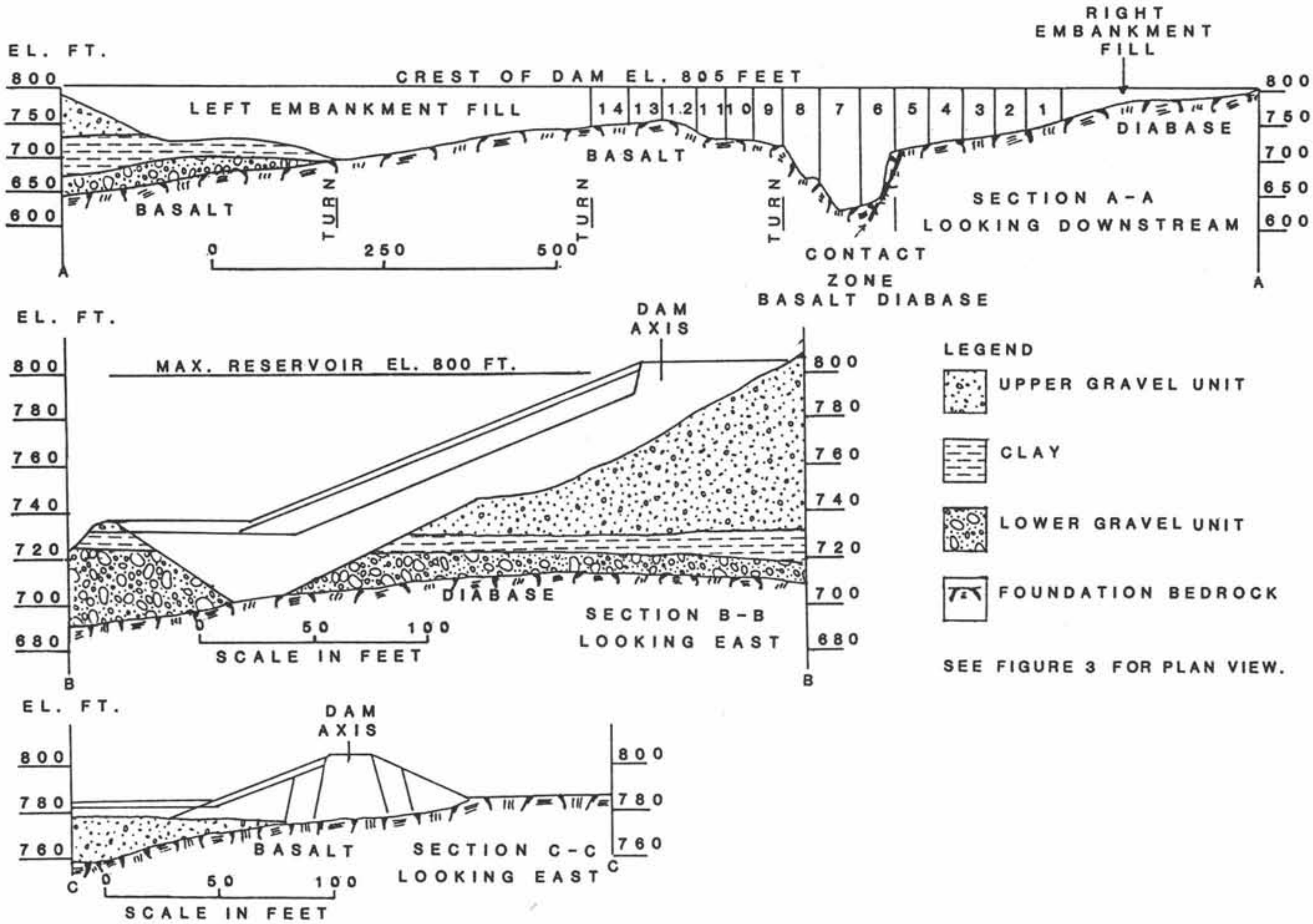


Figure 4. Geologic sections of Wynoochee Dam. A-A, section along dam axis; view downstream. B-B, section normal to axis through the left embankment and cut-off trench. C-C, section normal to the axis through the right embankment. See Figure 3 for section locations.

through 1977. One uplift pressure cell in monolith 7 approached the design hydraulic gradient. When the drainholes were cleaned in January 1978 the uplift pressures decreased.

Reservoir Leakage

Abutment leakage and downstream spring discharges are monitored by measurement in weirs, piezometers, and staff gages. No appreciable problems have been associated with abutment leakage.

REFERENCES

Rau, W. W., 1967, *Geology of the Wynoochee Valley Quadrangle, Grays Harbor County, Washington*: Washington Division of Mines and Geology Bulletin 56, 51 p.

Tabor, R. W. and Cady, W. M., 1978, *Geologic Map of the Olympic Peninsula, Washington*: U.S. Geological Survey Miscellaneous Investigations Series Map I-994, 2 sheets, scale 1:125,000.

Dams of the Columbia River and Tributaries

Richard W. Galster and Howard A. Coombs, Chapter Editors

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

Dams of the Columbia River and Tributaries: Introduction – Early Projects

RICHARD W. GALSTER
Consulting Engineering Geologist

The Columbia River and its major tributaries, the Snake, Yakima, Wenatchee, Methow, Okanogan-Similkameen, Spokane-Coeur d'Alene and Pend Oreille-Clark Fork, have provided the major impetus for settlement and commercial development in the eastern half of Washington as well as the major share of the hydropower developed in the Pacific Northwest. In the 20 counties lying wholly or partly east of the Cascade crest are 333 dams of various sizes, shapes, and vintages (Washington Department of Ecology, 1981). Many of these are constructed on minor tributaries, and some are off-stream storage structures. They range in height from a few feet to 550 ft (Grand Coulee) and in engineering complexity from simple, low, earth embankments to large combinations of concrete and steel with the highest navigation locks and fish ladders in the world. The main stem and major tributary dams are listed on Table 1 and their locations shown on Figure 1.

As dams and reservoirs do not respect political boundaries, Washington shares four of the large projects on the lower Columbia with Oregon; part of Lower Granite reservoir extends up the Snake River a short distance into Idaho.

Probably the first dam constructed on a major Columbia tributary is the still-operating Horn Rapids Dam. This is a low irrigation diversion weir built in 1893 across the Yakima River northwest of what is now Richland. The site of this facility is significant. Horn Rapids is at the apex of the bend in the Yakima River where it was geologically diverted some 9 mi north from its general southeasterly trend by late Pliocene uplift along the Olympic-Wallowa lineament. The river here crosses a basalt ledge (Elephant Mountain Member of the Saddle Mountains Basalt) that produces the rapids (Rockwell Hanford Operations, 1979).

Though irrigation began in the Yakima valley as early as 1867, the first corporate effort was by the Northern Pacific Railway Co. in 1890, when the Sunnyside project was begun. This was the first irrigation project of any magnitude in the United States (U.S. Army Corps of Engineers, 1934). The Yakima basin work was assumed by the U.S. Reclamation Service (now the Bureau of Reclamation) in 1905, shortly after passage of the Reclamation Act of 1902. Between 1907 and 1914 that agency built several structures on the Yakima River and its tributaries. During the same period the Reclamation Service was working in the Okanogan Valley constructing a 63-ft-high embankment, Conconully Dam, on Salmon Creek in 1910.

The development of hydropower proceeded apace. Prior to 1901 hydropower was being generated at Spokane Falls (Washington Geological Survey, 1902), and between 1908 and 1914 the Washington Water Power Company constructed two dams, Little Falls, and Long Lake, on the Spokane River below Spokane Falls and below Nine Mile Falls Dam, which had been constructed in 1908 by the Spokane and Eastern Railway. All these were "natural" dam sites where the Spokane River, skirting the edge of the Columbia Plateau, had cut narrow canyons into spurs of granitic rock of the Okanogan-Selkirk Highlands (Griggs, 1973). In 1907 a low-head hydropower diversion structure was constructed on the Wenatchee River in Tumwater Canyon and another was built downstream near Dryden.

The first concrete dam of significant size built for hydropower, however, was Condit Dam, which was completed in 1913 on the White Salmon River, a tributary entering the Columbia between Bonneville and The Dalles. The structure was designed and constructed by Stone & Webster Engineering Corporation of Boston for Pacific Power and Light Company. The Reclamation Service Clear Creek project, completed the following year, was the earliest arch dam in the Northwest. About 1920 the Okanogan Valley Power Company constructed a 54-ft-high gravity arch (Enloe Dam) in the canyon of the Similkameen River 7 mi northwest of Oroville. Enloe Dam, founded on coarse conglomerate, featured a remote powerhouse and produced hydropower for the Okanogan region from 1923 until 1959 when the project was abandoned largely because of complete siltation of the reservoir.

The only early hydropower generated on the main stem of the Columbia was at Priest Rapids where a power canal (called the Hanford Canal) was constructed

Table 1. Major onstream dams, Columbia River and tributaries.

Dam type: CG, concrete gravity; E, embankment; GA, gravity arch.

Owners: COE, U.S. Army Corps of Engineers; USBR, U.S. Bureau of Reclamation; PUD, Public Utility District of named county; WWP, Washington Water Power Co.; PP&L, Pacific Power and Light Co.

Dam	River	Type	Year completed	Structure height (ft)	Crest length (ft)	Maximum reservoir capacity (x 1000 acre-ft)	Foundation	Engineer	Owner
Bonneville	Columbia	CG	1937	197	2,477	537	siltstone	COE	COE
The Dalles	Columbia	CG/E	1957	200	9,735	330	basalt	COE	COE
John Day	Columbia	G/E	1968	230	5,900	253	basalt	COE	COE
McNary	Columbia	CG/E	1957	220	7,365	1,350	basalt	COE	COE
Priest Rapids	Columbia	CG/E	1959	196	8,412	250.2	basalt	Harza Engineering	Grant PUD
Wanapum	Columbia	CG/E	1963	213	8,537	748.5	basalt	Harza Engineering	Grant PUD
Rock Island	Columbia	CG	1933	1082	2,657	130	basalt	Stone & Webster	Chelan PUD[1]
Rocky Reach	Columbia	CG/E	1962	197	2,690	412	granite/gneiss	Stone & Webster	Chelan PUD
Wells	Columbia	CG/E	1967	196	4,380	361.2	granite/gneiss	Bechtel	Douglas PUD
Chief Joseph	Columbia	CG/E	1955	230	4,300	593	granite/gneiss	COE	COE
Grand Coulee	Columbia	CG	1942	550	4,173	9,562	granite	USBR	USBR
Ice Harbor	Snake	CG/E	1962	213	2,790	376	basalt	COE	COE
Lower Monumental	Snake	CG/E	1969	242	3,800	376	basalt	COE	COE
Little Goose	Snake	CG/E	1970	226	2,655	556	basalt	COE	COE
Lower Granite	Snake	CG	1975	228	3,200	483.8	basalt	COE	COE
Tieton	Tieton	E	1925	319	920	203.5	phyllite-metasandstone	USBR	USBR
Keechelus	Yakima	E	1917	128	6,550	171	glacial till/outwash	USBR	USBR
Kachess	Kachess	E	1912	115	1,400	710	glacial till/outwash	USBR	USBR
Cle Elum	Cle Elum	E	1933	165	1,801	710	glacial till/outwash	USBR	USBR
Little Falls	Spokane	CG	1910	72	1,634	4.3	granite	WWP	WWP
Long Lake	Spokane	GA	1914	248	539	253	granite	WWP	WWP
Nine Mile	Spokane	CG	1908	68	364	5.8	granite	Sanderson & Porter	WWP[2]
Boundary	Pend Oreille	Arch	1967	340	740	122	limestone	Bechtel/Leedshill	Seattle
Box Canyon	Pend Oreille	CG	1955	125	260	100	limestone	Harza Engineering	Pend Oreille PUD
Condit	White Salmon	CG	1913	125	471	2.1	basalt	Stone & Webster	PP&L

[1] Original owner was Puget Sound Power and Light Co. [2] Original owner was Spokane and Eastern Railway

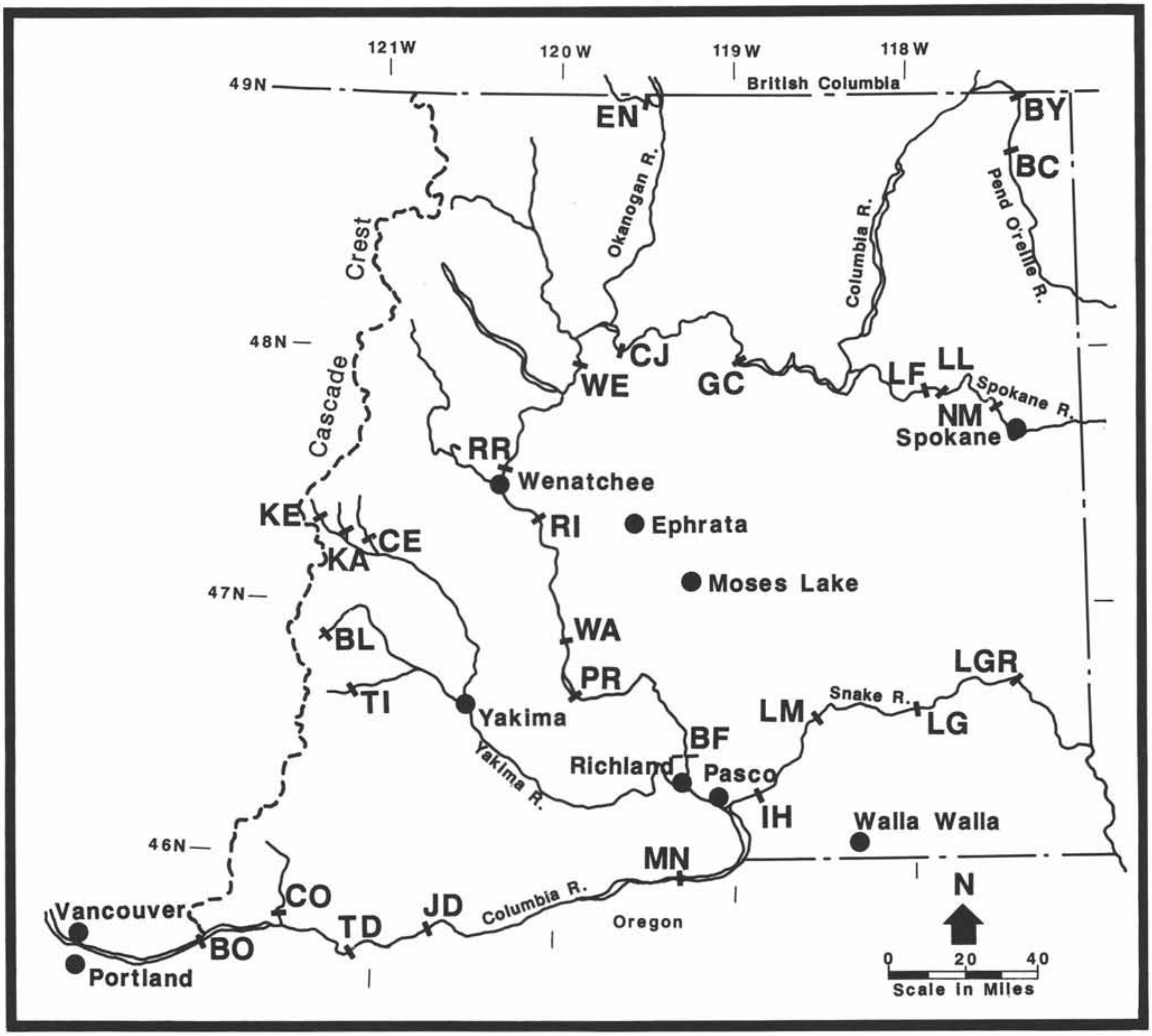


Figure 1. Major dams on the Columbia River and tributaries in Washington.

BC, Box Canyon Dam; BF, Ben Franklin dam site; BL, Bumping Lake Dam; BO, Bonneville Dam; BY, Boundary Dam; CE, Cle Elum Dam; CJ, Chief Joseph Dam; CO, Condit Dam; EN, Enloe (Similkameen) Dam; GC, Grand Coulee Dam; IC, Ice Harbor Dam; JD, John Day Dam; KA, Kachess Dam; KE, Keechelus Dam; LF, Little Falls Dam; LG, Little Goose Dam; LGD, Lower Granite Dam; LL, Long Lake Dam; LM, Lower Monumental Dam; MN, McNary Dam; NM, Nine Mile Dam; PR, Priest Rapids Dam; RI, Rock Island Dam; RR, Rocky Reach Dam; TD, The Dalles Dam; TI, Tieton Dam; WA, Wanapum Dam.

by the Hanford Irrigation and Power Company in 1907-1908 (Rice, 1986). It fed a powerhouse at the foot of the rapids, which has a head of 26 ft. Most of the power produced was used for pumping irrigation water to the Hanford Tract from Coyote Rapids downstream, although some surplus power was sold to Pacific Power and Light Company (U.S. Army Corps of Engineers, 1934).

In 1926 Washington Water Power Company began development of the Chelan Falls Project, tapping water from the Chelan River a short distance downstream from the lake at a low headworks dam and conducting it by a 11,000-ft-long tunnel penstock to a powerhouse adjacent to the Columbia River. The tunnel was driven in about 5 months through locally migmatitic granitic gneiss from several headings accessed by adits and shafts. Apparently little or no support was used, though a concrete lining was later installed (Washington Water Power Co., unpublished data).

In 1927 Congress directed the U.S. Army Corps of Engineers to survey the Columbia River and its tributaries with respect to power development and other improvements. Work began in April 1928 and was completed in 1931 and included geological reconnaissance and site investigations at scores of sites at the astronomical cost of less than $550,000 (U.S. Army Corps of Engineers, 1934). The results were published in the now famous "308 Report", so named for the Congressional authorizing document. While this study was under way, the Stone & Webster Engineering Corporation began construction in 1930 of the first barrier across the Columbia at Rock Island Rapids south of Wenatchee. Shortly following submittal of the "308 Report", construction of Bonneville Dam was begun by the U.S. Army Corps of Engineers and of Grand Coulee by the Bureau of Reclamation. Over the following 40 yr, the Columbia, along with the lower Snake and Pend Oreille, was the scene of dam construction unparalleled on any North American river. Most of the activity took place during the 1950s and 1960s, spurred by the disastrous flooding of the Columbia in the spring of 1948, by the post-war requirements for additional hydropower and by the establishment of improved navigation on the lower Columbia and Snake rivers.

The papers in this chapter describe the general geology of most of these projects and the major problems in engineering geology which had to be solved during site development.

REFERENCES

Griggs, A. B., 1973, *Geologic Map of the Spokane Quadrangle, Washington, Idaho and Montana*: U.S. Geological Survey Miscellaneous Investigations Series Map I-768, 1 sheet, scale 1:250,000.

Rice, D. G., 1986, Personal communication, U.S. Army Corps of Engineers, Seattle, WA.

Rockwell Hanford Operations, 1979, Compilation Geologic Map of the Pasco Basin South Central Washington, in Myers, C. W.; Price, C. M., et al., 1979, *Geologic Studies of the Columbia Plateau—A Status Report*: Rockwell Hanford Operations, RHO-BWI-ST-4, Richland, WA, 12 sheets, scale 1: 62,500.

U.S. Army Corps of Engineers, 1934, *Columbia River and Minor Tributaries*: 73rd Congress, 1st Session, House Document No. 103, Vol. II, Washington, DC, 1,845 p. [Also known as the "308 Report"]

Washington Department of Ecology, 1981, *Inventory of Dams in the State of Washington*: Washington Department of Ecology WDOE-81-18, Olympia, WA, 142 p.

Washington Geological Survey, 1902, *Annual Report for 1901*: Washington Geological Survey, Olympia, WA, 344 p.

Enloe Dam on the Similkameen River, 7 mi northwest of Oroville in north-central Washington. The surface penstock can be seen on the far bank. This fed to a remote powerhouse about a quarter mile downstream. The powerhouse and gravity arch dam are no longer in use, and the reservoir is largely filled with flood detritus. Photograph by R. W. Galster, November 1982.

Dams of the Lower Columbia River

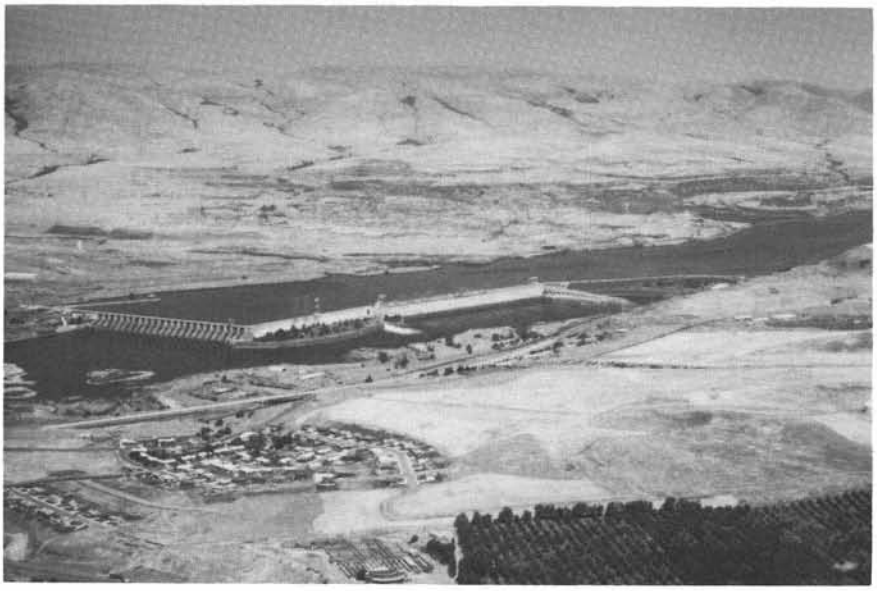
Geologic Setting

Bonneville Dam

The Dalles Dam

John Day Dam

McNary Dam



The Dalles Dam. Aerial view to the northeast toward the Columbia Hills anticline in Washington. Photograph by R. W. Galster, July 1980.

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

Dams of the Lower Columbia River: Geologic Setting

RICHARD W. GALSTER
Consulting Engineering Geologist

and

JOHN W. SAGER
U.S. Army Corps of Engineers

The lower Columbia River flows generally west for 324 mi from its confluence with the Snake River in the southern Pasco Basin to its entrance into the Pacific Ocean. In so doing, it passes through four geologic provinces, provides an unparalleled geologic view through the southern Cascade Range and forms a substantial segment of the political boundary between Washington and Oregon (Figure 1).

Within the eastern one-third of its lower segment the river passes out of the Pasco Basin through the Horse Heaven anticline at Wallula Gap and into the Umatilla Basin section of the Columbia Plateau. Its pre-dam gradient was 185 ft in 124 mi (U.S. Congress, 1934). The river then swings west, generally following the structural low of the Dalles-Umatilla syncline for about 100 mi, close to and parallel with the southern toe of the Columbia Hills anticline, the southernmost structure of the Yakima Fold Belt. Throughout much of this reach, the river has cut well into the lava flows of the Miocene Columbia River Basalt Group (CRBG) (Figure 2). Two of the four dams on the lower Columbia (John Day and McNary) lie within this reach and are founded on the Columbia River basalt. The CRBG is overlain by Pliocene epiclastic and volcaniclastic sediments of the Dalles Formation (Hodge, 1938) (or Dalles Group of Farooqui et al., 1981) over large areas of the Umatilla Basin, principally in two basins of deposition: the Arlington Basin to the east and the Dalles Basin to the west (Farooqui et al., 1981). Although much of the Dalles Formation has been eroded from the Columbia River valley proper, exposures testifying to its former extent are found in a large part of the western Umatilla Basin and form the valley sides near The Dalles.

Just downstream from the mouth of the Deschutes River, which enters the Columbia from the Oregon side, the pre-dam Columbia River entered an 11-mi-long series of rapids formed on the eroded surface of the CRBG beginning at Celilo Falls and ending at The Dalles. Much of this section is now inundated by the reservoir behind The Dalles Dam, which spans the river 9 mi downstream from Celilo Falls. Immediately downstream from The Dalles the river passes through the Ortley anticline, an extension of the Columbia Hills anticline, and into the 50-mi-long, 3,000-4,000-ft-deep Columbia River Gorge, where the complexities of the southern Cascade Range geology are well illustrated. The Paleogene volcaniclastic sediments beneath the CRBG are exposed together with the complex of Pliocene and Pleistocene intrusive rocks and lavas, Holocene landslides, and various paleochannels of the ancestral Columbia River dating from the late Miocene (Allen, 1979; Hodge, 1938; Waters, 1973; Palmer, 1977; Tolan and Beeson, 1984). Bonneville Dam, the first dam constructed on the lower Columbia, lies within this gorge. The pre-dam fall of the river from the head of Celilo Falls through the gorge was 123 ft in 60 mi (U.S. Congress, 1934).

The gross structure in the Columbia River Gorge is dominated by the broad north-trending arch of the Cascade Range. Uplift of the range may have begun as late as 2 Ma (Beeson and Tolan, 1987); the onset of incision of the present gorge dates from that time. Lowered sea levels during the Pleistocene permitted cutting of the gorge to elevations 100 ft below modern sea level near Bonneville Dam. The present gorge is the latest in a series of river valley positions dating back to the Miocene when the ancestral Columbia, passing through a structural low in the Cascade volcanic pile, was diverted on several occasions by the flows of the Columbia River basalt filling former channels (Tolan and Beeson, 1984; Beeson and Tolan, 1987).

In addition to cutting the broad Cascade arch, the eastern part of the gorge cuts through several northeast- to east-northeast-trending folds that appear to be western manifestations of the Yakima Fold Belt. The youngest structural features in the Columbia Gorge are

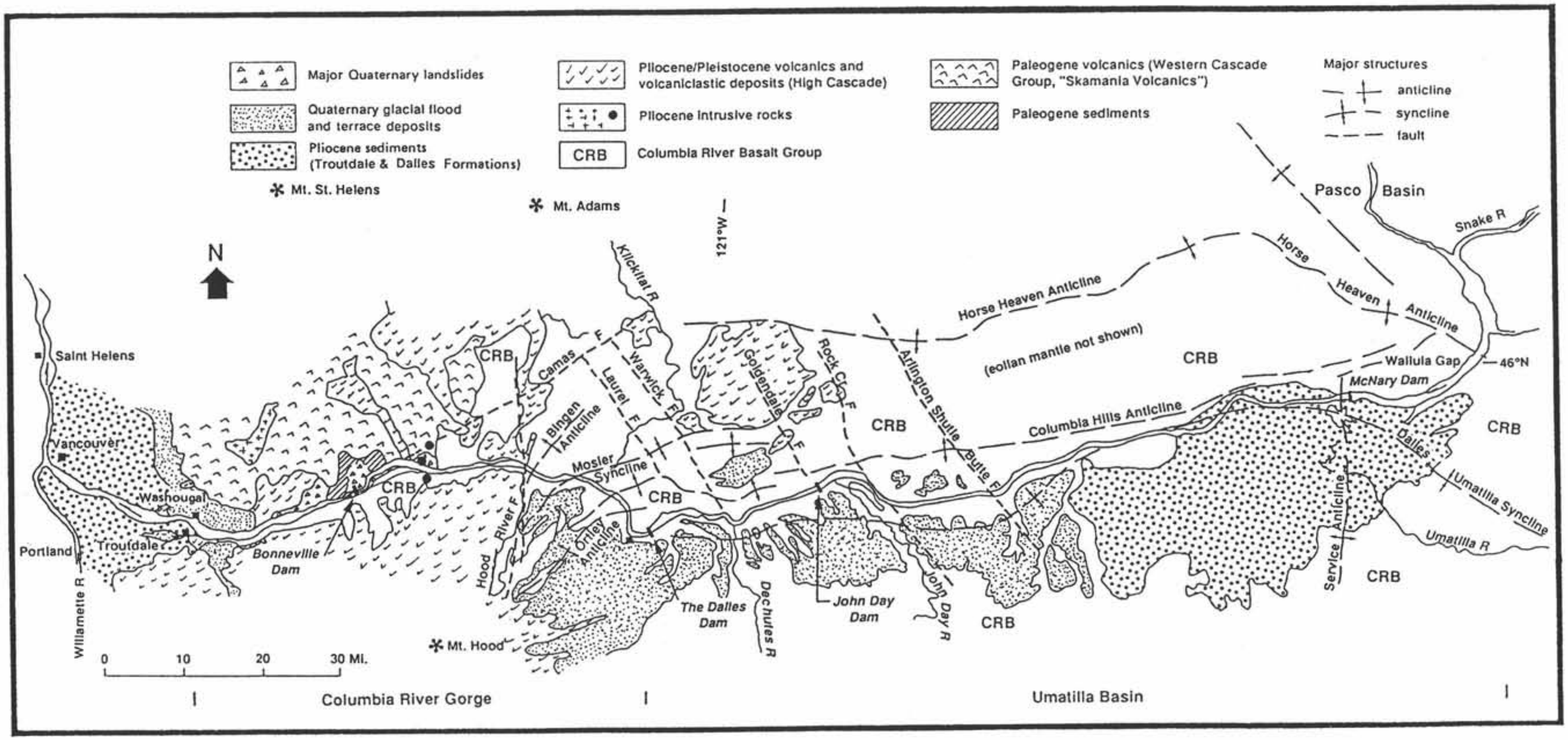


Figure 1. Generalized geologic map of the area along the lower Columbia River. After Huntting et al. (1961); Peck (1961); Newcomb (1970); Walker (1973); and Walsh et al. (1987).

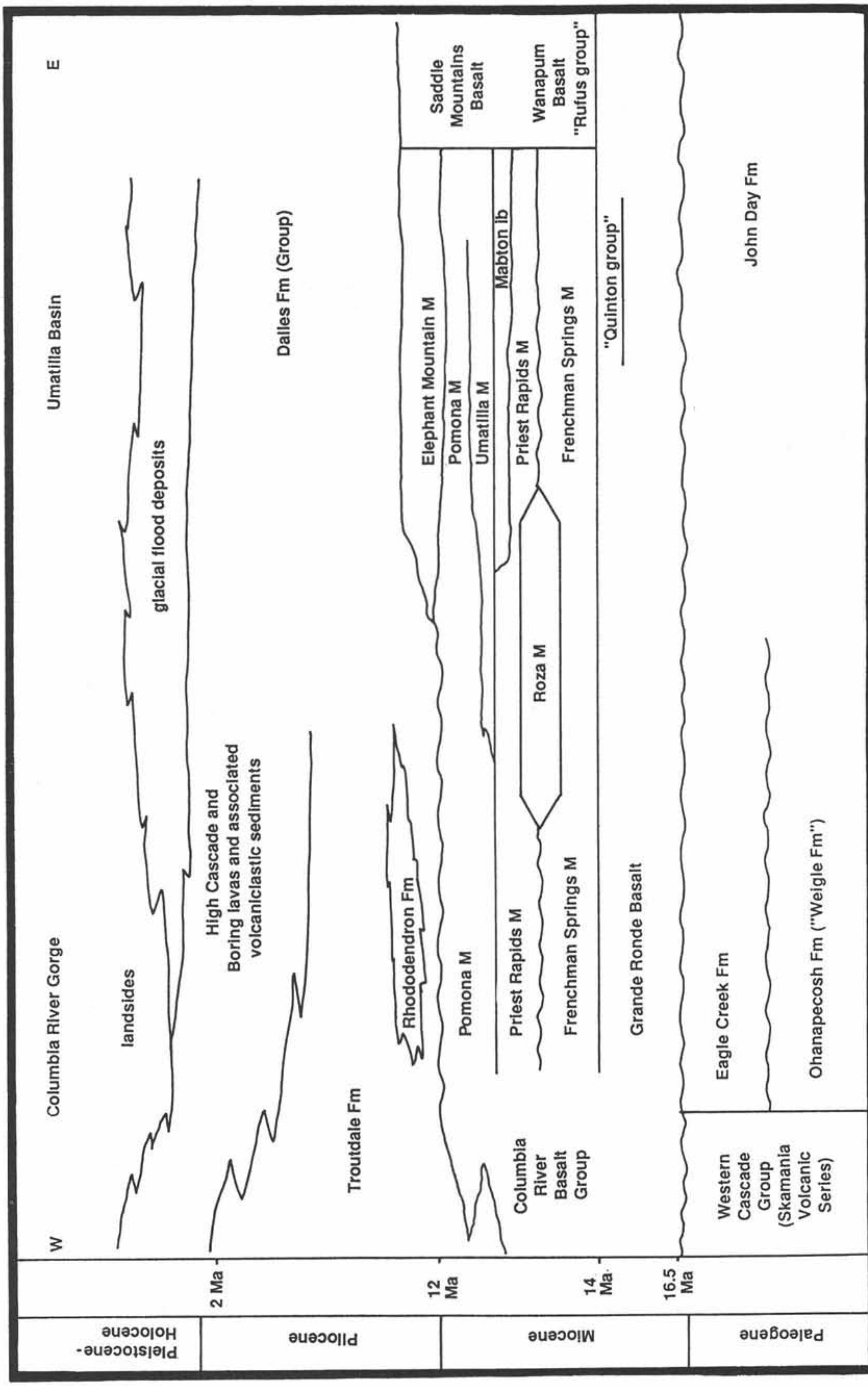


Figure 2. Generalized stratigraphic column for the lower reach of the Columbia River. M, Member; ib, interbed.

north-trending normal faults (such as the Hood River fault) that appear to be related to development of the High Cascade graben and the Pliocene to Holocene volcanism associated with it (U.S. Army Corps of Engineers, 1982). Northwest-trending faults dominate the structure of the western Umatilla Basin near the Columbia River valley, and the north-trending Service anticline crosses the eastern part of the basin.

Basalt and related hyaloclastic debris of the CRBG predominate in the gorge (Figure 2). In the deepest part of the gorge the river has cut through the basalt into the underlying Paleogene volcanic rock and volcaniclastic sediments of the Ohanapecosh and Eagle Creek formations (Waters, 1973; Allen, 1979). Overlying the CRBG in the western part of the gorge and filling ancestral river's paleochannels in the basalt is a sequence of largely quartzitic conglomerates of the Troutdale Formation, locally intercalated with lahars of the Rhododendron Formation (Tolan and Beeson, 1984). Rock units that are age equivalents of the Troutdale in the eastern part of the gorge and in the Umatilla Basin are the epiclastic and volcaniclastic sediments of the Dalles Formation.

Capping the stratigraphic sequence in and near the Columbia Gorge are the Boring and High Cascade lavas and associated sediments of Pliocene to Holocene age. Flows and clasts of these units in the upper part of the Troutdale Formation attest to the onset of basaltic and andesitic volcanism in Pliocene time. Lava flows blocked the present gorge on several occasions during the Pleistocene (Waters, 1973).

The effects of cataclysmic Pleistocene flooding by impounded glacial meltwaters are well documented along the segment of the river in and upstream of the gorge. Maximum flood crests at Wallula Gap are estimated to have been at elevation 1,200 ft, at about elevation 1,000 ft in the eastern part of the gorge, and at 600 ft in the western part (Allen, 1979). The flow through Wallula Gap is estimated at 1.66 cu mi/hr for 2 to 3 weeks (Allen, 1979). These floods stripped much of the Dalles Formation from the Umatilla Basin, created scablands in the CRBG, and scoured channels and depressions in the bedrock valley bottom as great as 225 ft below present sea level.

Just east of the Washougal, Washington–Troutdale, Oregon, area, the Columbia enters the Willamette Lowland. Here it becomes influenced by ocean tides, although current reversal is not experienced above Saint Helens, Oregon (U.S. Congress, 1934). At Portland/Vancouver the river swings north-northwest along the east side of the Portland Hills-Clackamas River alignment (Schmela and Palmer, 1972) or "fault zone" (Tolan et al., 1984) to the Kelso-Longview area. The river again swings west cutting a broad valley through the basalts and sedimentary rocks that make up the Coast Range. Within these two western reaches the Columbia drops only 12 ft in 140 mi (U.S. Congress, 1934). Thus the section of the river below the gorge is inappropriate for hydroelectric development, and navigation improvements are accomplished by selective dredging.

The four dam sites within this segment were originally studied and selected during the early 1930s, a study documented in the "308 Report" (U.S. Congress, 1934). All four dams were designed and constructed and are operated by the U.S. Army Corps of Engineers principally for navigation and hydropower generation.

REFERENCES

Allen, J. E., 1979, *The Magnificent Gateway*: Timber Press, Forest Grove, OR, 144 p.

Beeson, M. H. and Tolan, T. L, 1987, Columbia River Gorge–The geologic evolution of the Columbia River in northwestern Oregon and southwestern Washington. In Hill, M. L. (editor), *Centennial Field Guide, Volume 1, Cordilleran Section of the Geological Society of America*: Geological Society of America, pp. 321-326.

Farooqui, S. M.; Beaulieu, J. D.; Bunker, R. C.; Stensland, D. E.; and Thoms, R. E., 1981, Dalles Group–Neogene formations overlying the Columbia River Basalt Group in north-central Oregon: *Oregon Geology*, Vol. 43, No. 10, pp. 131-140.

Hodge, E. T., 1938, Geology of the lower Columbia River: *Geological Society of America Bulletin*, Vol. 49, pp. 831-930.

Huntting, M. T.; Bennett, W. A. G.; Livingston, V. E.; and Moen, W. S., 1961, *Geologic Map of Washington*: Washington Division of Mines and Geology, Olympia, WA, 1 sheet, scale 1:500,000.

Newcomb, R. C., 1970, *Tectonic Structure of the Main Part of the Basalt of the Columbia River Group, Washington, Oregon, and Idaho*: U.S. Geological Survey Miscellaneous Geologic Investigations Map I-587, 1 sheet, scale 1:500,000.

Palmer, L., 1977, Large landslides of the Columbia River Gorge, Oregon and Washington. In Coates, D. R., (editor), *Landslides*: Reviews in Engineering Geology, Vol. III, Geological Society of America, pp. 69-83.

Peck, D. L., 1961, *Geologic Map of Oregon West of the 121st Meridian*: U.S. Geological Survey Miscellaneous Geologic Investigations Map I-325, 2 sheets, scale 1:500,000.

Schmela, R. J. and Palmer, L. A., 1972, Geologic analysis of the Portland Hills-Clackamas River alignment: *The Ore-Bin*, Vol. 34, No. 6, pp. 93-102.

Tolan, T. L.; Beeson, M. H.; and Vogt, B. F., 1984, Exploring the Neogene history of the Columbia River–Discussion and geologic field trip guide to the Columbia River Gorge: *Oregon Geology*, Vol. 46, No. 8, pp. 87-97.

Tolan, T. L. and Beeson, M. H., 1984, Intercanyon flows of the Columbia River Basalt Group in the lower Columbia River Gorge and their relationship to the Troutdale Formation: *Geological Society of America Bulletin*, Vol. 95, pp. 463-477.

U.S. Army Corps of Engineers, 1982, *Bonneville Lake Earthquake and Fault Study, Design Memorandum No. 38*: U.S. Army Corps of Engineers, Portland District, Portland, OR, 69 p, 16 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.

Walker, G. W., 1973, *Preliminary Geologic and Tectonic Maps of Oregon East of the 121st Meridian*: U.S. Geological Survey Miscellaneous Field Studies Map MF-495, 2 sheets, scale 1:1,000,000.

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.

Waters, A. C., 1973, The Columbia River Gorge: Basalt stratigraphy, ancient lava dams and landslide dams. In *Geologic Field Trips in Northern Oregon and Southern Washington*: Oregon Department of Geology and Mineral Industries Bulletin 77, Portland, OR, pp. 133-162.

Celilo Canal at The Dalles prior to construction of The Dalles Dam. Photograph by R. W. Galster, 1949.

Special clam bucket use in implacement of ICOS cut-off wall prior to Bonneville Dam, north powerhouse construction. Concrete cut-off wall was required in order to dewater excavation in Cascade landslide materials. Photo by R. W. Galster, October 1976.

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

Bonneville Dam

JOHN W. SAGER
U.S. Army Corps of Engineers

PROJECT DESCRIPTION

The Bonneville Project (Figure 1) is on the Columbia River near river mile 146 and is 42 highway miles east of Portland. The Bonneville Project consists of a spillway dam, two powerhouses, a navigation lock, and fish ladders. The northern half of the spillway dam and the second powerhouse are in Washington. The southern half of the spillway dam, the first powerhouse, and the navigation lock are in Oregon. Primary functions are hydropower generation and the provision of slack water to improve navigation.

The concrete spillway dam is 197 ft high and 1,450 ft long and has a design discharge capacity of 1,600,000 cfs at a pool elevation of 87.5 ft. The original powerhouse, on the south side of the river, is 190 ft high, 1,027 ft long, and has a total rated capacity of 518,400 kw generated from 10 power units. The second powerhouse, on the river's north side, is 953 ft long, 235 ft wide, and 200 ft high and has a total rated capacity of 540,000 kw generated from eight power units. The navigation lock is 500 ft long and 76 ft wide and has a 50- to 70-ft single lift. A new navigation lock is being constructed south of the existing lock. That lock will have a width of 86 ft and a length of 675 ft.

Lake Bonneville, impounded by the dam, is 47 mi long and provides a normal operating head of 52 ft. Bonneville is a "run of the river" dam; storage capacity is limited to 537,000 acre-ft at normal maximum operating pool. The original project was completed in 1937 and the second powerhouse was completed in 1982.

SITE GEOLOGY

General

Bonneville Dam is located in the Columbia River Gorge, a 50-mi-long canyon cut through the Cascade Range. During the Pleistocene, when sea level was lower, the Columbia River cut a channel 1 mi north of the modern channel near Bonneville Dam to a depth of at least 100 ft below present-day sea level. The river channel was subsequently pushed south by the 700-yr-old Bonneville (Cascade) landslide. Gorge slopes are mostly steep, range up to more than 3,000 ft in height, and include numerous vertical rock cliffs with associated talus accumulations. Many of these slopes were oversteepened by the catastrophic Pleistocene floods. Average valley width is nearly 1 mi and is locally wider where extensive landslides have left 3° to 9° slopes. Lateral tributary drainage gradients are steep, except in areas of low slopes. Some of the smaller streams on the southern side of the gorge enter from hanging valleys and produce spectacular waterfalls.

The stratigraphic section at Bonneville Dam is more than 3,000 ft thick and consists primarily of volcanic and volcaniclastic rocks overlain by landslide and alluvial materials. Several formations of the Western Cascade Group, Columbia River Basalt Group, and High Cascade Group are exposed in the Columbia River Gorge (Hammond, 1980; Williams et al., 1982). Bedrock units exposed near the dam and reservoir area are (oldest to youngest): Ohanapecosh (Weigle) Formation; Eagle Creek Formation; Columbia River Basalt Group; Rhododendron, Dalles, and Troutdale formations; and High Cascade Group (Figure 2). During investigations for the second powerhouse, the U.S. Army Corps of Engineers used the informal name Weigle for the oldest rock unit found in the Columbia River Gorge; the same rocks are called Ohanapecosh by Waters (1973) and Wise (1970). The older name is retained for this discussion. Most of the valley floor in the Bonneville area is underlain by the Weigle formation, which consists of siltstone, claystone, sandstone, and conglomerate and dips less than 30° southwest. The Bonney Rock diabase cuts the Weigle and younger units, underlies the southern two-thirds of the first powerhouse and the existing navigation lock, and forms the rock knob directly south of the existing navigation lock. Exposed in the lower valley slopes and unconformably overlying the Weigle formation is the Eagle Creek Formation, which dips less than 15°. The Eagle Creek Formation is composed primarily of volcanic conglomerate and/or agglomerate that contain both rounded and angular fragments. Columbia River basalt flows overlie the Eagle Creek Formation and form the distinct near-vertical cliff faces in the middle and upper canyon walls. The Rhododendron Formation, which consists of ash, tuff, and volcanic conglomerate or agglomerate, overlies the basalt flows in the upper canyon area. Olivine basalt flows (High Cascade or Boring lavas)

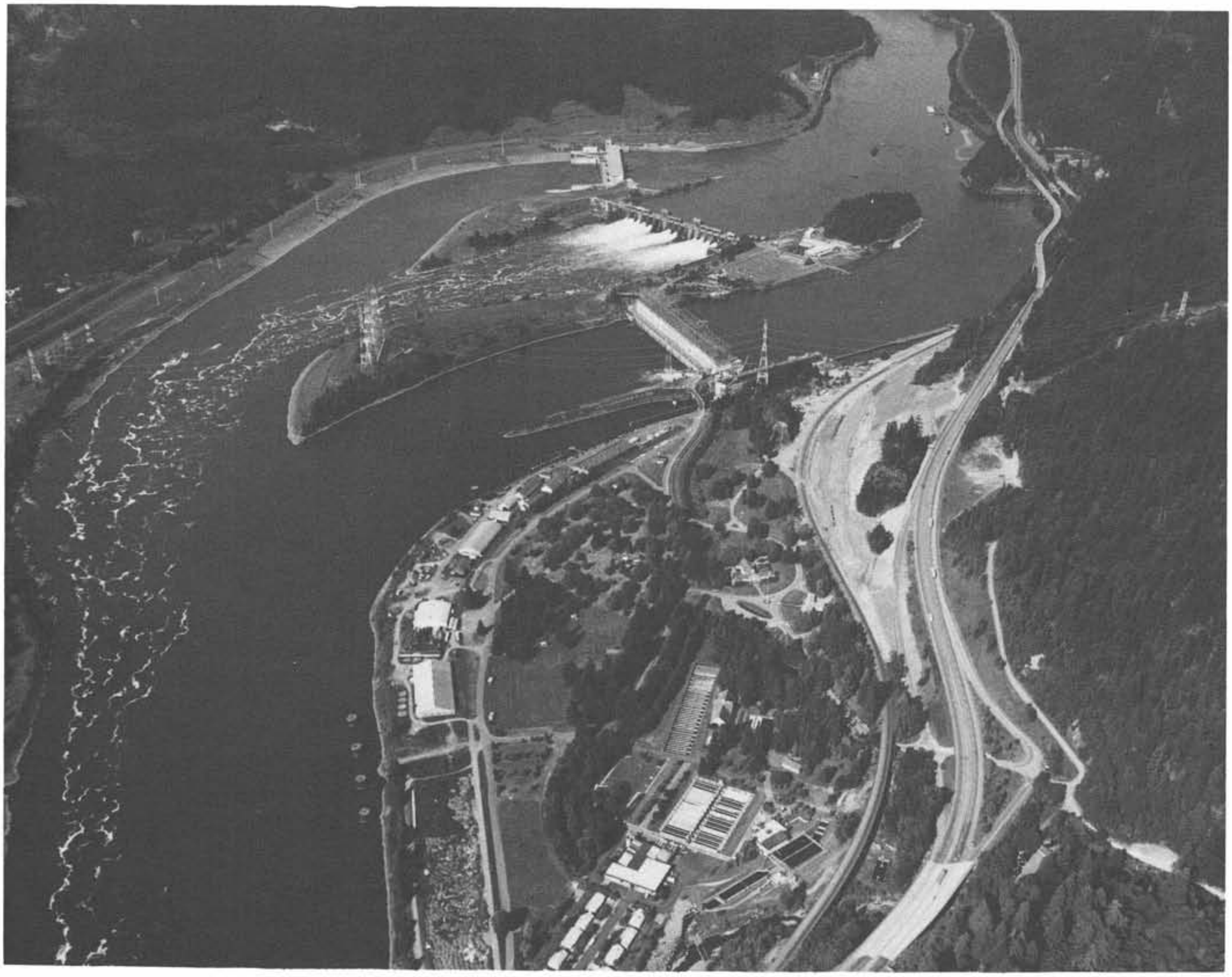


Figure 1. Aerial view to the east (upstream) of the Bonneville Project. The original powerhouse and navigation lock are in the center of the photograph between the Oregon shore (right) and Bradford Island. The spillway dam lies behind, spanning the river between Bradford Island and the original Washington shore. The second powerhouse and intake and tailrace channels are at the upper center. The forested Bonneville (Cascade) landslide has crowded the river against the south wall of the gorge. The fish rearing facilities are at the lower center. Interstate Highway 84 and the Union Pacific Railroad dominate the Oregon shore, and the Burlington Northern Railroad and State Route 14 occupy the Washington shore (upper left and center). Photograph courtesy U.S. Army Corps of Engineers.

overlie the Rhododendron Formation and form some of the prominent cliffs near the top of the south gorge slopes.

Major Landslides

Extensive landslides have occurred on the steepened undercut slopes since the Pleistocene floods. The northern shore of the Columbia River is nearly 50 percent landslide deposits from Camas to Hood River, a distance of 34 mi (U.S. Army Corps of Engineers, 1982). Individual slides range up to a maximum of 14 sq mi. The southern shore also has numerous, but significantly smaller, landslides. Major factors contributing to the concentration of landslides in the gorge are believed to be clayey saprolites on an impervious tuff and low-grade metamorphism of the tuff to zeolites and clay (Waters, 1973). The landslides are more extensively developed along the north bank of the Columbia River due to the southward regional dip of the clay layers. Pleistocene floods, which oversteepened the gorge slopes, combined with loading from overlying Columbia River basalts, precipitated downslope failures where clays were exposed on lower slopes. Some of the significant landslides near Bonneville Dam are the Bonneville (Cascade) landslide, Wind Mountain-Collins Point landslide, Tooth Rock landslide, and Ruckel landslide.

The Bonneville (Cascade) landslide (Figure 2) is the largest and youngest, covering nearly 14 sq mi. Its most recent major failure is radiocarbon dated at 700 yr. The slide originated on the Washington side of the valley, temporarily dammed the Columbia River, and forced the river to the south. The slide deposit slopes about 6° and is composed of fractured slide blocks that are as much as 300 ft across. The toe of the landslide deposit forms the right abutment of the Bonneville second powerhouse, and the Burlington Northern Railroad tunnel traverses part of the slide toe. The slide is currently stable, except for minor creep and rockfalls near the headwall. The Wind Mountain-Collins Point landslide, also on the Washington side of the valley, covers 10 sq mi and extends to the Columbia River. The landslide is divided into two lobes by the Wind Mountain intrusive body. The western lobe is currently stable, but the eastern lobe, Collins Point, is active. The active portion of the slide covers 2.7 sq mi and involves about 250 million cy of material. The surface of the slide deposit slopes about 9°. The rate of movement varies and correlates with annual changes in precipitation. Although the toe of the slide is in the Bonneville pool, toe movement is not affected by fluctuations in pool level. Maximum rate of movement between 1953 and 1956 was 35 ft/yr at the upper end. Between 1960 and 1970, the maximum rate was 1.5 ft/yr at the lower end.

Tooth Rock and Ruckel landslides form a series of coalescing slides on the Oregon shore near Bonneville Dam. Ruckel landslide was active prior to Bonneville Dam construction, but it was stabilized between 1918 and 1924 by construction of eight drainage tunnels. The Tooth Rock landslide is currently stable. The estimated age of the Tooth Rock landslide is 20 ka, and Cascade landslide deposits overlie Tooth Rock deposits on Bradford Island.

Several other small landslides, some of which are active, are known in the gorge area.

Overburden

Overburden in the Bonneville area includes silt, sand, and gravel layers which are most abundant in the downstream terrace areas. They are overlain on the Oregon shore by the Tooth Rock slide deposits and on the Washington shore by the Cascade slide deposits (Figure 2). Interspersed between the landslide deposits and on valley floor surfaces are scattered outcrops of alluvium and/or slopewash. Overburden depths within the project area range from about 65 ft to more than 220 ft on the Washington shore and from a few feet to more than 200 ft on the Oregon shore. Maximum slide debris depth may exceed 1,000 ft in the middle of Tooth Rock landslide; no overburden overlies Bonney Rock where it is exposed on the Oregon shore as a rock knob.

GEOLOGICAL ASPECTS OF SITING, DESIGN, AND CONSTRUCTION

First Powerhouse

The first powerhouse at Bonneville is founded almost entirely on the Bonney Rock diabase (Figure 3). All site selection and design issues relative to geology center around this hard, columnar intrusive. The primary geologic factors that affected design and construction were the orientation, spacing, and nature of the sheeting and columnar jointing. A grout curtain was constructed at the upstream edge of the structure to minimize seepage through the fractured diabase. The original navigation lock was also excavated in and founded on the Bonney Rock diabase.

Spillway Dam

The spillway dam is founded entirely on sedimentary beds that were originally identified as part of the Eagle Creek Formation (Hodge, 1932)(Figure 4). However, these beds were later considered to be the Weigle (Ohanapecosh) formation, which also forms the foundation for the second powerhouse. Both abutments of the spillway dam are tied into overburden and landslide materials. It was known from the beginning that neither the foundation nor abutment materials were as sound as might be desired. As a result, a great deal of study was devoted to both. Foundation bedrock consists of soft, weak beds of sandstone, agglomerate, plastic bentonite, and hard conglomerate. Highly sheared and fractured areas were common in the foundation, and several minor faults were noted. Three distinct faults cross the foundation from northeast to southwest, and generally dip 20° to 45° NW and SE. These faults, especially where they intersected in soft rock, were overexcavated and replaced with concrete, then grouted to assure adequate foundation bearing. Under the south abutment and the center of the structure, bedrock is hard conglomerate with interbeds of finer grained, softer sandstone and bentonite. Between the center conglomerate and the north abutment was another area of sandstone and bentonite with local thin beds of conglomerate; these materials also extended beneath the north abutment. Weak, clay-like strata were found beneath both the north and south abutments. These weak layers ranged from 2 ft to as much as 15 ft in thickness and, like other beds beneath the dam, dip southeast (essentially upstream) at about 15°. Beneath the south abutment are overburden deposits of landslide debris, terrace, and buried river deposits of coarse sand, gravel, and boulders. To prevent seepage around the end of the spillway dam through these materials, a cut-off trench was excavated through them and backfilled with impervious material. Similar conditions at the north abutment

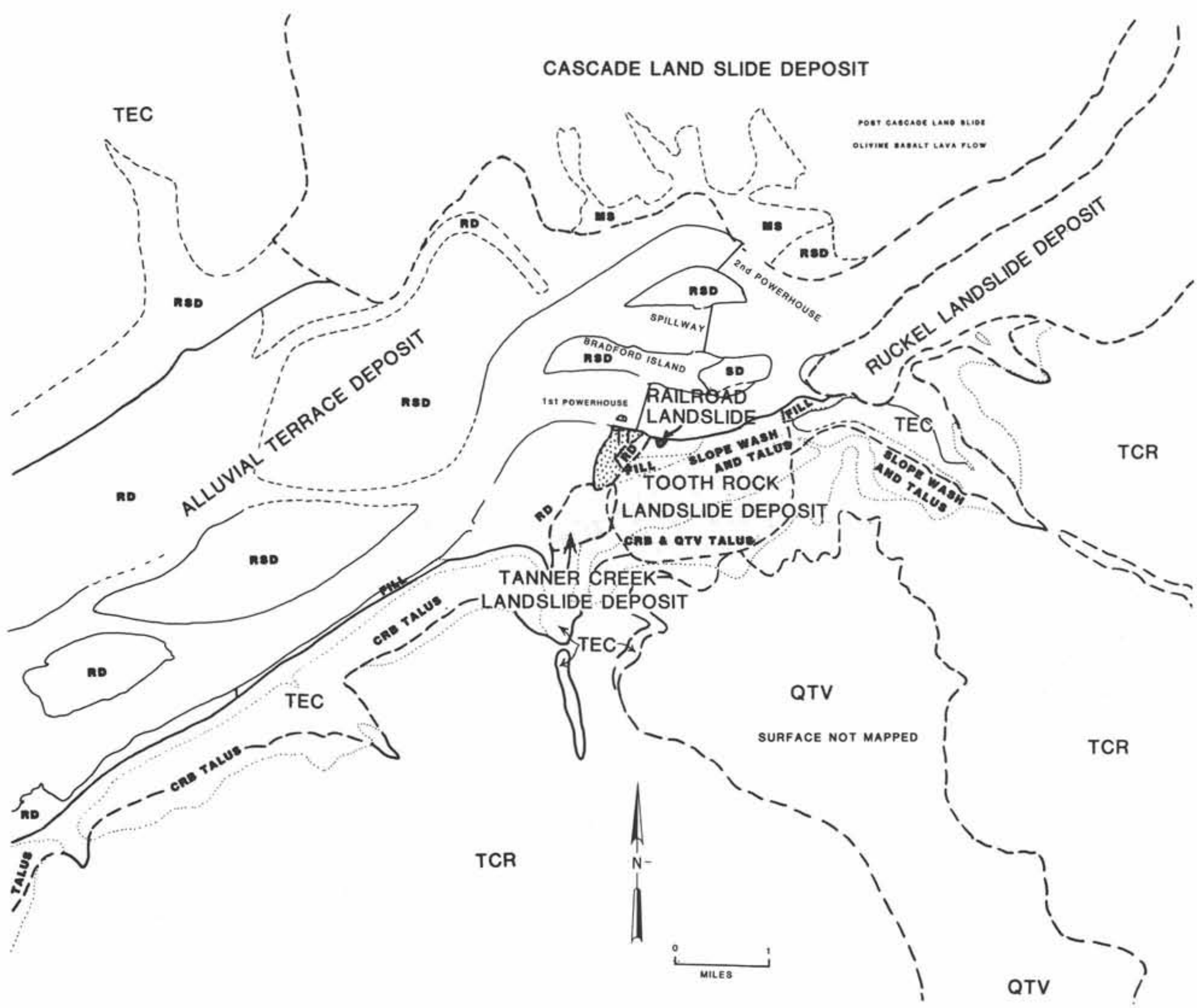


Figure 2. Generalized geologic map of the Bonneville Project area. Explanation on facing page. Figure courtesy of U.S. Army Corps of Engineers; explanation modified from this source.

required construction of upstream and downstream wing walls, a cut-off wall, and a dike of impervious material on top of the landslide deposits.

Second Powerhouse

The most significant geological aspect affecting the site selection and design of the second powerhouse was the massive Cascade landslide. The entire foundation area lies below the deposits of this ancient landslide. Landslide materials had to be excavated to depths as great as 180 ft before founding the powerhouse on the soft, weak beds of the Weigle formation. The pervious landslide debris deposits, particularly a continuous buried river deposit known as the "Pre-Slide Alluvium" or PSA, required construction of a mile-long concrete cut-off wall extending to in-place bedrock around the entire perimeter of the excavation. The powerhouse structure was constructed within the "bathtub" formed by the concrete cut-off wall. Dewatering of the "bathtub" during excavation required construction of an extensive system of deep wells, well points, and surface drainage ditches and pipes. Foundation rock consisted of siltstone, sandstone, claystone, and conglomerate. The relatively low shear strength and deformation moduli of the soft, weak, and highly fractured and sheared sedimentary beds controlled the foundation design. Extensive overexcavation and "dental treatment" of soft, weak, shear zone areas were necessary at several places in the foundation. The highly slakable nature of the predominately fine-grained beds also made

EXPLANATION

Unconsolidated Materials - Soil Units are Delineated by light lines:

RD -- Recent Columbia River alluvium includes stratified sand, gravel, cobble and boulder deposits with occasional silt and clay layers. Coarse fraction typically rounded consisting of Columbia River Basalt and lesser amounts of olivine basalt. Also contains metamorphic and granitic "exotics" and white mica derived from the Idaho batholith area. RD unit is generally present below el. 100 ft. However, a silty gravelly sand deposit with exotics is present on the el. 400-ft bench west of Tanner Creek. This deposit may be related to the Missoula Floods.

RSD -- Reworked Slide Debris. Forms unconsolidated "outwash" deposits downstream of the slide areas. Incorporates varying amounts of both alluvial and slide derived materials.

CRB Talus -- Angular rock fragments of predominately Columbia River Basalt with local minor olivine basalt forms deposits of GP to GM at the base of steep TCR cliffs. Talus deposits cover the TEC-TCR contact throughout most of the area.

Slope Wash and Talus -- Brown to red-brown silty sand to sandy silt deposits. Unstratified, forms a thin cover over much of the area below the talus deposits. May contain varying amounts of angular rock fragments. This transported soil unit is probably derived from the chemical weathering of the olivine basalt on the cliffs above.

This unit also includes a possible residual soil unit covering the el. 500-ft bench east of Tanner Creek. This deposit contains the brown mica, phlogopite, which is a common alteration product of the olivine basalt boulders possibly deposited by the Missoula floods or a spheroidally weathered olivine basalt lava flow.

MS -- Unconsolidated fine to medium fine sand deposits with white mica. Forms unstratified deposits in depressions and channels in the Cascade landslide. MS unit is present in lesser amounts in the Tooth Rock landslide area.

SD -- Slide Debris. Unconsolidated slide materials ranging to 10 ft in size.

Landslide Deposits -- Landslides are delineated by textured areas.

Railroad Landslide -- Last movement in mid-1930s during construction of existing lock.

Tanner Creek Landslide -- Last movement in 1960 during construction of I-84.

Ruckel Landslide -- Last movement during construction of railroad.

Cascade Landslide -- No known recent activity. Wood fragments from slide dated at 700 yr.

Tooth Rock Landslide -- No known recent activity. Slide estimated to be 10,000-20,000 yr old. Local unstable materials at toe failed in 1936 during construction of Bonneville Dam.

Bedrock Units -- Heavy lines indicate approximate and inferred contacts between major rock groups; light lines delineate areas of in-place outcrops for each rock group.

QTV -- Boring Formation consists of olivine basalt lava flows, flow breccias and related clastic rocks.

TCR -- Columbia River Basalt Group consists of basalt lava flows and flow breccias with local soil interbeds between flows. Flows are horizontal.

TEC - Eagle Creek Formation consists of volcanic conglomerates and volcanic sandstones poorly bedded local siltstone and sandstone interbeds.

TW -- Weigle Formation (Ohanapecosh Formation). Consists of massive mudstones with interbedded sandstones and siltstones with local conglomerates. Materials are generally well bedded. Matrix is altered to clay minerals. Failure planes for major slides are seated in the unit. Maximum in-place bedding dip 30°.

TI -- Bonney Rock Intrusive (stippled area). Consists of diabase, columnar jointed fine to medium grained. May be same age as Boring Formation.

---

early protection of final grades and slopes with shotcrete and concrete a crucial part of the foundation preparation specifications. Sliding stability investigations, analyses, and calculations on potential failure surfaces involving pre-existing shear planes within the foundation were also crucial in the design analysis.

New Navigation Lock

Several aspects of major foundation construction were considered in analyzing the optimum location for the new lock. The primary consideration was to construct as much of the lock as possible upon a sound rock foundation. A second siting consideration was to minimize to the greatest extent possible construction effects on the slide-prone areas in the upstream approach channel. The farther south the approach alignment is moved, the more it would impinge on landslide deposits and the greater the potential for sliding stability problems.

The selected site was located to permit the lock foundation to be constructed directly upon the Bonney Rock

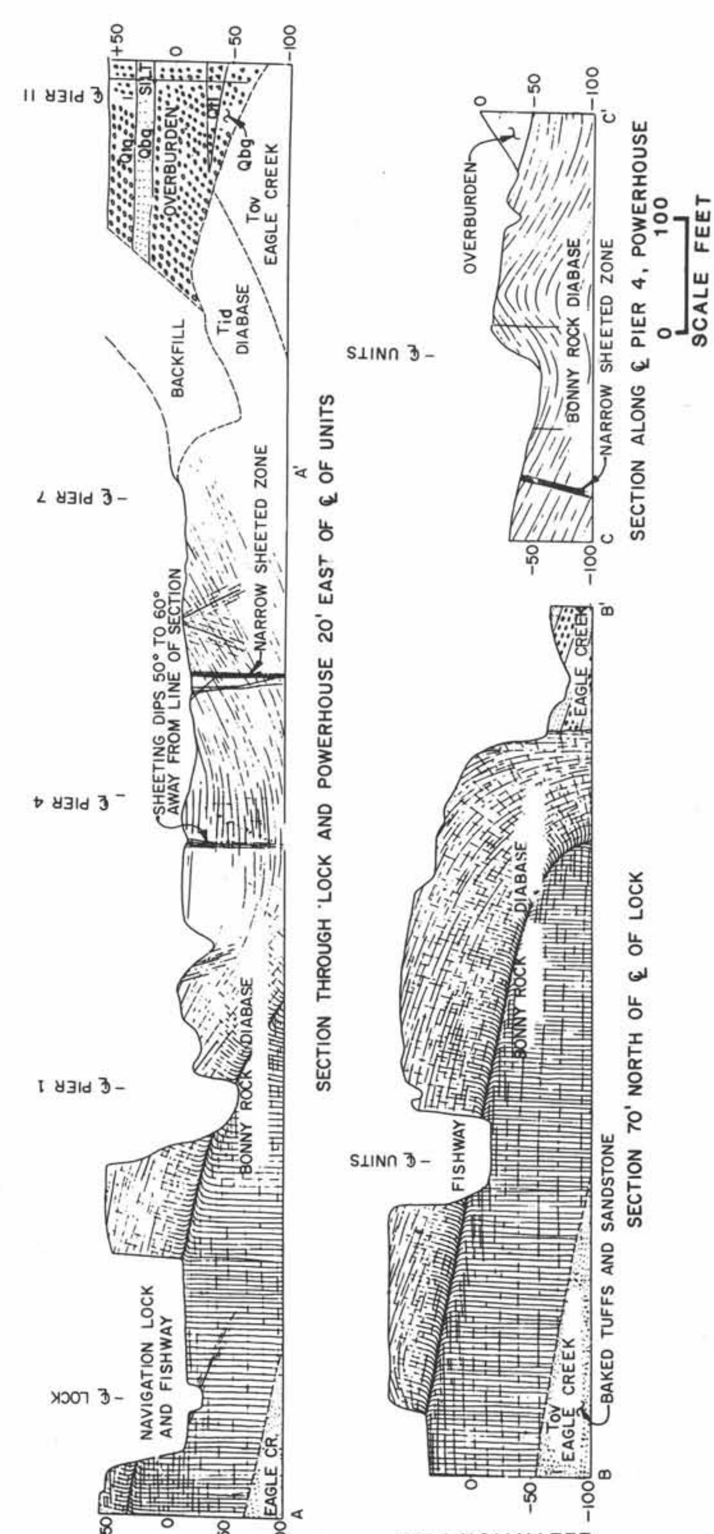


Figure 3. Geologic sections through the first powerhouse and original navigation lock, Bonneville Dam. Upper section is 20 ft east of the center line and is a view downstream. The lower section is a view north to the Washington shore. Strata labelled Eagle Creek on this figure were later termed Weigle formation. From Holdredge (1937).

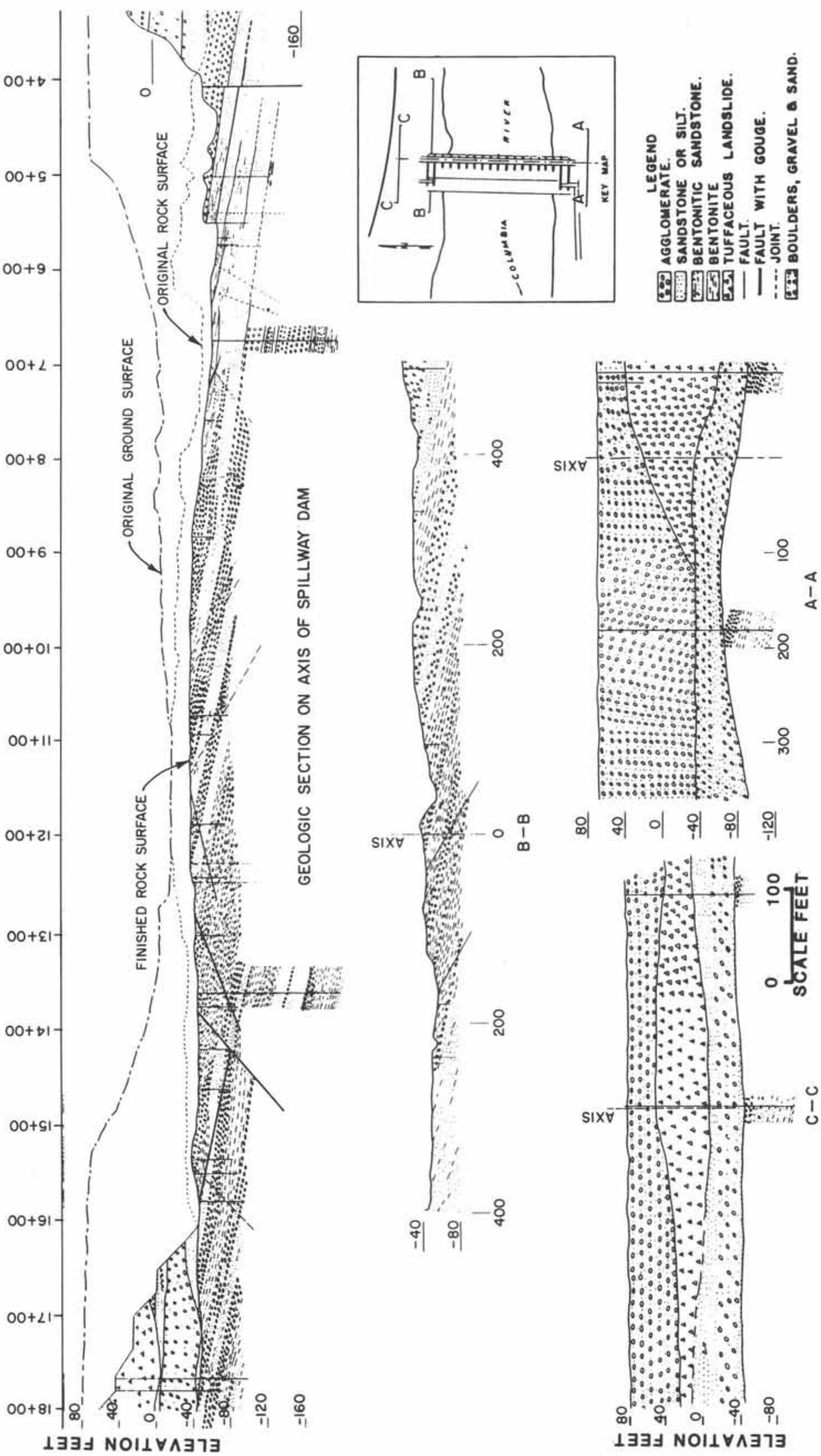


Figure 4. Geologic sections through the spillway dam, Bonneville Dam. Upper section is as viewed from the downstream side. The other sections are views to the north, to the Washington shore. From unpublished data, C. P. Holdredge, 1937.

diabase. Geology along the proposed alignment is shown on Figure 5. The top of Bonney Rock is exposed south of the existing navigation lock and may be the only in-place rock mass exposed at the site. Other than Bonney Rock, the top of sound in-place rock is below the proposed bottom elevation of the new navigation lock. The extent of the diabase at the proposed lock foundation elevation is roughly the same as the length of the lock, but the surface of the diabase dips steeply downward directly upstream and downstream at each end of the lock. The upstream and downstream approach channels will be excavated in overburden because the Weigle sequence lies below the bottom elevation of both channels.

The toe of the Tooth Rock landslide deposit is believed to extend into the upstream approach channel area. Although this landslide occurred an estimated 20 ka and is considered stable, past excavations in the toe area have resulted in local slide failures.

The Tanner Creek slide deposit, west of Bonney Rock (Figure 2) and upslope of the downstream approach channels, is a much larger ancient slide failure that was partially re-activated during the 1960 construction of Interstate Highway 80N (now I-84). The western edge of the failure mass is about 80 ft east of Tanner Creek, and the slide deposit extends about 1,000 ft along the highway and about 1,500 ft up- and downslope. Failure was apparently in decomposed Weigle blocks. A total of 6,400 ft of horizontal drains was installed by the Oregon Highway Department to stabilize the slide mass during construction of the highway in 1960 to 1963. In 1986, approximately 20,000 additional feet of horizontal drains were also installed within the slide mass to assure slope stability for excavation required to relocate the Union Pacific Railroad track as the first stage of constructing the new navigation lock.

CONSTRUCTION PROBLEMS

During construction of the original powerhouse and navigation lock, some difficulty was experienced with a relatively small landslide which has come to be called the Railroad slide (Figure 2). That slide was located just

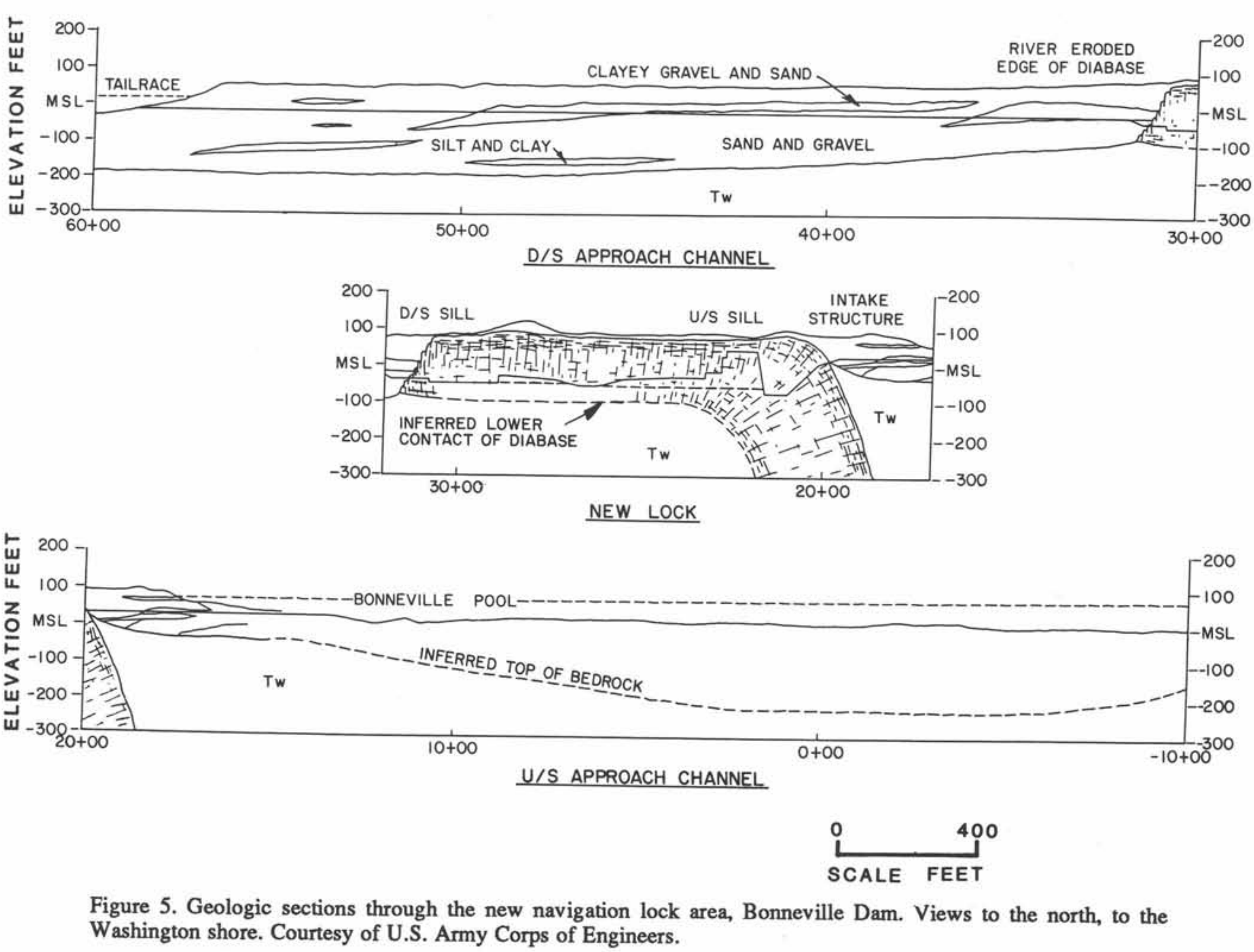


Figure 5. Geologic sections through the new navigation lock area, Bonneville Dam. Views to the north, to the Washington shore. Courtesy of U.S. Army Corps of Engineers.

east of Bonney Rock and about opposite the east end of the lock guide wall. The slide extended about 200 ft east and west and 200 ft or more south of the original railroad alinement. The sliding plane was in tuffaceous materials of the old landslide, and the slide was, therefore, of a secondary nature. At the back of this slide, the sliding plane on which it moved cut up through the sands and terrace gravels to the surface. For exploration purposes, one drill hole and one shaft were sunk on the line of the new railroad across this slide. When the cofferdam was built to permit construction, its south end abutted against the slide and for as long as it was present there was no movement. Before the railroad fill was constructed across the slide, all slide material was removed from beneath the planned fill area, but all slide material between the new fill and what used to be known as Bradford Slough was left in place. Loose rock was dumped into the excavation, and the railroad fill placed on the rockfill. During the construction of the upper lock guide wall piers, the south end of the cofferdam was removed. That portion of the slide that had been left immediately began to move again, carrying with it the old railroad (which was still in use at that time). This portion of the slide was in part supporting the toe of the new railroad fill, and as it moved away there was some slight movement in the new fill as well. Water had collected in the loose rock backfill beneath the "new" railroad and rewetted and lubricated the old sliding plane, thus causing the movement. After backfilling around the guide wall piers, the movement abated.

Relocation of the Burlington Northern Railroad as a result of the second powerhouse construction required building about 1,400 ft of railroad tunnel, which was mined entirely through Cascade landslide debris. Tunneling through the heterogeneous mixture of displaced rock and overburden materials presented several construction challenges. Ground control and stabilization in the weak materials were key to the successful completion of the tunnel.

Several minor rock slides occurred within the foundation rock cut slopes during excavation for the second powerhouse. A few of these failures required forming and concrete backfilling to reconstruct the failed slopes to their original cut slope configuration. A potential failure of the upstream "cofferdam" slope, from the upstream concrete cut-off wall to the upstream edge of the foundation excavation, was feared when movement occurred along a soft clay layer overlying bedrock. The layer extended beneath the slope and dipped approximately 15° downstream. With careful monitoring using slope inclinometers and horizontal drains to reduce porewater pressures along the clay layer, powerhouse construction was completed without a major slope failure.

OPERATIONAL PROBLEMS RELATED TO GEOLOGY

Potentially, the most significant operational problem related to geology is the impact of a maximum earthquake. At its nearest point, the St. Helens seismic zone is 21 mi from Bonneville Dam. Assuming that the seismic zone represents a fault capable of generating a major earthquake, a half-length rupture of 45 km with a strike-slip motion could generate an earthquake of magnitude 6.8.

Based on the assumption that the St. Helens seismic zone is a capable fault, earthquake and fault studies by the Portland District, U.S. Army Corps of Engineers have determined design values for ground motion parameters that could be generated by the maximum earthquake affecting Bonneville Dam. Although the seismic zone has not been directly correlated with a surface fault trace, the earthquake event pattern within the zone has been correlated with a postulated subsurface fault (U.S. Army Corps of Engineers, 1982). Recent earthquake activity in the zone and its proximity to Bonneville Dam dictate that consideration be given to the maximum potential ground motions that could be generated by the St. Helens zone. The following design earthquake values have been selected by the U.S. Army Corps of Engineers for re-analyzing the seismic stability of the Bonneville Project:

Maximum earthquake	Acceleration (%g)	Velocity (cm/sec)	Displacement (cm)	Duration (sec)
Without St. Helens Zone	0.17	14	10	13
With St. Helens Seismic Zone	0.24	20	15	19

The U.S. Army Corps of Engineers is conducting an analysis of the seismic stability to assure the project structures will remain stable if subjected to these values.

REFERENCES

Hammond, P. E., 1980, *Reconnaissance Geologic Map and Cross Sections of the Southern Washington Cascade Range*: Department of Earth Sciences, Portland State University, Portland, OR, 31 p., 2 sheets.

Hodge, E. T., 1932, *Report of Dam Sites on Lower Columbia River*: U.S. Army Corps of Engineers, Pacific Division, Portland, OR, 84 p., 37 plates.

Holdredge, C. P., 1937, *Final Geologic Report on the Bonneville Project*: U.S. Army Corps of Enguineers, Portland District, Portland, OR, 39 p.

U.S. Army Corps of Engineers, 1982, *Bonneville Lake, Earthquake and Fault Study, Design Memorandum No. 38*: U.S. Army Corps of Engineers, Portland District, Portland, OR, 69 p., 16 plates.

Waters, A. C., 1973, The Columbia River Gorge: Basalt stratigraphy, ancient lava dams and landslide dams. In *Geologic Field Trips in Northern Oregon and Southern Washington*: Oregon Department of Geology and Mineral Industries Bulletin 77, Portland, OR, pp. 133-162.

Williams, D. L.; Hull, D. A.; Ackerman, H. D.; and Beeson, M. H., 1982, The Mt. Hood region–Volcanic history, structure and geothermal energy potential: *Journal of Geophysical Research*, Vol. 87, No. B4, pp. 2767-2781.

Wise, W. S., 1970, *Cenozoic Volcanism in the Cascade Mountains of Southern Washington*: Washington Division of Mines and Geology Bulletin 60, Olympia, WA, 45 p., 1 plate.

Bonneville navigation lock during construction, about 1935. Photograph by U.S. Army Corps of Engineers.

The Dalles Dam

JOHN W. SAGER
U.S. Army Corps of Engineers

PROJECT DESCRIPTION

The Dalles Dam is on the Columbia River at the head of Bonneville Lake, 192.5 mi upstream from the mouth of the river and 3 mi east of The Dalles, Oregon. Portland, the largest population and industrial center on the Columbia River, is 97 mi downstream of The Dalles Dam. The Oregon-Washington state boundary lies along the main Columbia River channel, dividing the project between the two states. All of the project except the rockfill closure dam is in Washington. Construction began in 1952, and the project went into operation in 1957.

The Dalles Dam is a "run-of-the-river" dam (Figure 1). Celilo Lake provides a limited storage capacity of 332,500 acre-ft at normal maximum pool. The 23.5-mi-long lake serves as a power operating pool that has a head of 76 ft and navigation pool for river traffic. The dam is a 8,735-ft-long, Z-shaped structure made up of the following units: (a) a rockfill embankment with appurtenant fish passage facility on the Oregon (south) shore that serves as a south nonoverflow and closure dam; (b) a downstream channel-parallel powerhouse 2,089 ft long facing the south shore (containing 22 main and 2 auxiliary generating units rated at 1,807,000 kw capacity); (c) a south nonoverflow dam crossing a "button hook" island linking the powerhouse and spillway dams and serving as a switchyard; (d) a midstream concrete spillway 1,710 ft long rated at 2,290,000 cfs capacity; (e) a north shore concrete nonoverflow section with fish ladder; (f) a single lift 675-ft-long x 86-ft-wide navigation lock; and (g) a Washington shore earthfill embankment. Normal forebay elevation is 155 ft.

SITE GEOLOGY

General

The Dalles Dam is located near the southeast margin of a roughly circular topographic basin formed by the erosion of an approximately 1,000 ft thickness of Pliocene Dalles Formation from the northeast end of a synclinal trough in Columbia River basalt. The basin is approximately 10 mi in diameter and more than 1,000 ft deep. It is bounded on the north and west by the gently sloping east limb of Ortley anticline, which leaves the Cascade Range on a northeast trend, swings east around the north side of the basin, and trends into the Columbia Hills anticline. The south and east sides of the basin are formed by an erosional escarpment in tuffaceous sediments of the Dalles Formation. Youthful valleys of several small streams penetrate the escarpment. Between the escarpment and the base of Ortley fold is an area of low relief measuring approximately 6 mi east to west and 4 mi north to south. This area is underlain by flows of Columbia River basalt that dip gently south and are in part masked by Pleistocene and Recent alluvium. The Columbia River enters the basin from the northeast, follows a semicircular course around the east, south, and west margins of the central area, and leaves the basin through a gap in the Ortley fold, the east entrance to the gorge of the Columbia River through the Cascades.

Overburden

A generalized geologic map and typical geologic sections through the powerhouse, spillway, and navigation lock are shown on Figures 2 and 3. Overburden units in the foundation area consisted of recent silt, sand, and gravel in and adjacent to the river and older terrace silt, sand, and gravel higher on the abutments. This overburden has been removed from the foundation beneath most of the main structures, but small local alluvial remnants remain beneath a few of the appurtenant structures. A limited deposit of 3-in.-minus gravel was present upstream from the powerhouse axis, and fine silty sand covered portions of the spillway and spillway approach channel areas to depths of 30 ft. Moderately cemented Pleistocene gravel filled scabland channels in the lock area. In the spillway and south nonoverflow dam areas, gravel-filled channels cut into the bedrock surface were concealed by sandy overburden. A narrow band of sand and gravel was left in place beneath the midwestern portion of the closure dam's upstream toe, but overlying fine, wind-deposited sand with thin sand and gravel lenses was excavated from beneath the remainder of this upstream toe area. Extensive deposits of gravel cover much of the terrace north of the dam, and these terrace gravels were used both for construction fill and as concrete aggregate.

Figure 1. Aerial view northeast (upstream) of The Dalles Dam. The spillway dam dominates the view; the channel-parallel nonoverflow dam and powerhouse separate the forebay (left) from the original river channel and present tailrace. The fish ladders on the Oregon shore and embankment closure section are beyond the powerhouse. The light-colored exposures on the low hills in the upper right are Dalles Formation. A portion of the Washington shore fish ladder and the navigation lock are at the left. Photograph courtesy of the U.S. Army Corps of Engineers.

Bedrock

Foundations of the main dam structures have been excavated into the Priest Rapids and/or Roza members of the Wanapum Basalt, Columbia River Basalt Group (CRBG). During foundation explorations and construction, these flows were informally termed "Rufus Group", and individual basalt flows were given letter designations as shown on Figures 2 and 3 (U.S. Army Corps of Engineers, 1953, 1956). These basalt flows range from 60 to 100 ft thick; the thickness of individual flows varies by less than 10 ft. No persistent interbeds are found between flows, but some wind-deposited basaltic dust is found in fissures, and a few low areas on flow surfaces contain small lenses of fossil soil and preserved vegetation. Bedrock is a relatively uniform, dark gray basalt of high crushing strength, except in fractured zones and at some flow contacts where brown staining and some clay minerals occur. Rock strength, however, is not appreciably lowered by this weathering. Several flows include subflows or tongues of lava that solidified before being covered by later surges of the same flow. Flow contacts vary from very tight to open; voids contain ground water. Most flow surfaces in the excavations had a smooth, rolling ropey character (pahoehoe) and relief as much as 1 ft. Each flow includes an upper vesicular zone from 5 to 15 ft thick underlain by a "pinhole" vesicular zone of about equal thickness; a dense basalt zone makes up the remainder

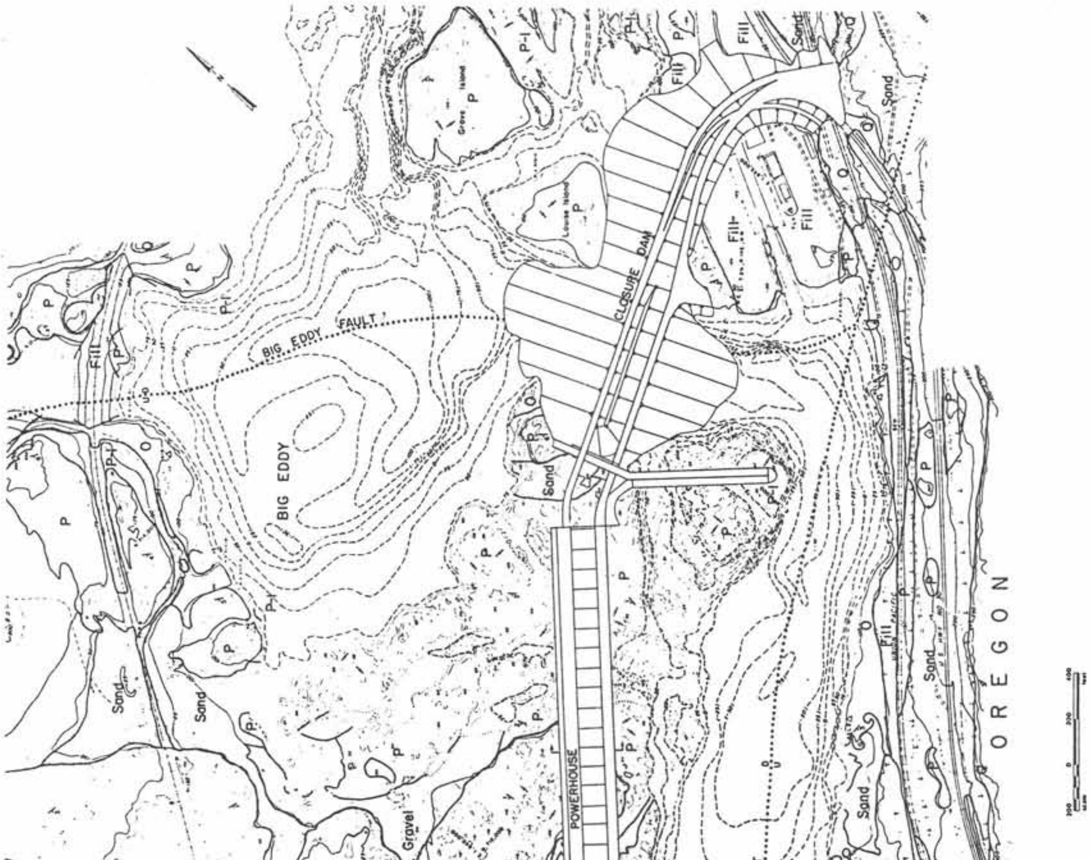


Figure 2. Generalized geologic map and plan of The Dalles Dam. S, Columbia River basalt (flow S); R, Columbia River basalt (flow R); Q, Columbia River basalt (flow Q); P, Columbia River basalt (flow P); P-1, Columbia River basalt (subflow P-1); O, Columbia River basalt (flow O). Dotted lines, location of concealed faults; Threemile fault is at the left margin of the map. Scale is approximately 1 in. = 900 ft. Courtesy of the U.S. Army Corps of Engineers.

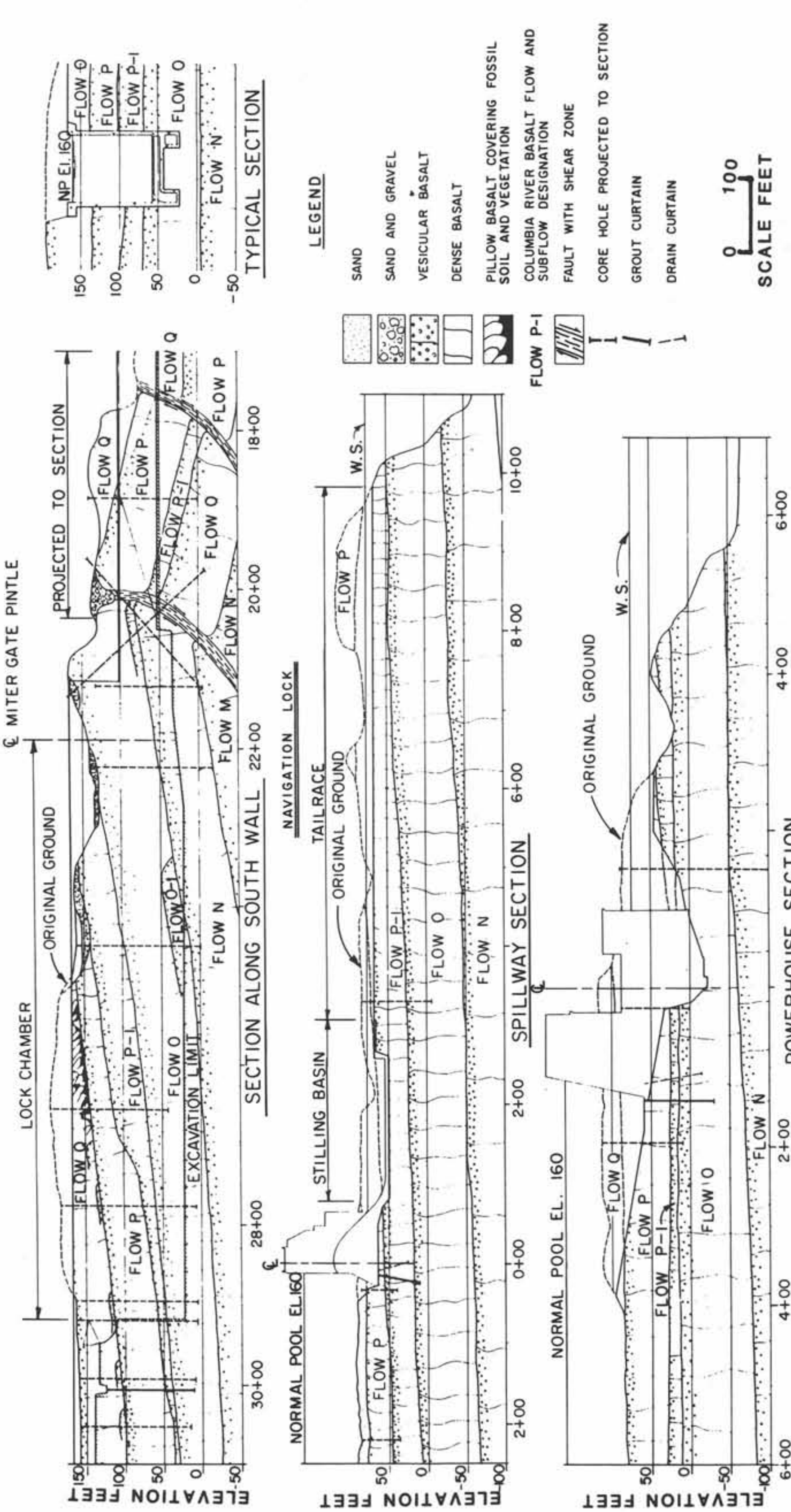


Figure 3. Geologic sections of The Dalles Dam along the south wall of the navigation lock (above), through the spillway dam area (middle), and through the powerhouse area (lower).

of the flow. The CRBG flows are estimated to be about 3,000 ft thick in this area and are overlain on the higher abutment slopes south of the dam by the Dalles Formation. The Dalles Formation, which is estimated to be about 1,000 ft thick, consists of thickly bedded andesitic ash-flow tuffs, tuff breccia, agglomerate, and flow rock with interbedded conglomerate. At the dam, both the Columbia River basalt and the Dalles Formation dip about 2° southwest and are at the south end of a small basin bounded by faults.

Joints and Faults

The joint system is different in each basalt flow, but the various joint set patterns have some common characteristics. Each flow includes several sets of cooling or tensional joints that are in part associated with the textural zones within the flow. Primary joints are vertical and form roughly rectangular prisms with joints spaced 4 to 8 ft apart.. One joint set parallels, and the other is normal to, the flow direction. Individual primary joints can locally be traced for several hundred feet on the surface of a flow. Most of the primary joints extend to the center of the flows, and many extend to the base. At depth these joints tend to bend away from the vertical, and the rectangular pattern also becomes less distinct. In the upper vesicular zone, horizontal slabbing joints parallel the flow surface and are from 0.5 to 2.0 ft apart. In the "pinhole" vesicular zone these joints are from 2 to 5 ft apart, and in the lower, dense zone they tend to be more curved and randomly spaced. Some of the basalt flows, however, have local basal zones with curved, subhorizontal joints from 1 to 6 in. apart. Numerous secondary joints dissect the blocks bounded by the primary joints. Secondary joints form many different patterns, and local foundation slopes tend to coincide with secondary joint surfaces. Almost all joint surfaces are coated with a thin black layer of glassy basaltic mineraloids that slake on exposure and tend to weather to a brown limonitic substance.

Three faults that exhibit gouge, brecciated rock, and shear zones ranging from 50 to 300 ft across cut the site area and determine the course of the river between Big Eddy and Threemile Rapids just downstream of the dam. Threemile fault crosses the navigation lock, and Big Eddy fault transects the closure dam (Figure 2). Beneath the downstream approach channel to the navigation lock, Threemile fault consists of two reverse faults that bound a highly brecciated fault splinter. Minor faults occur in the foundations of every structure. Most are low-angle faults that have displacements of only a few inches and no slickensides or brecciated zones. Thrust faults in the powerhouse foundation strike west and dip 5° to 20° N; they form an *en echelon* series of weakness planes with some local breccia zones in the denser basalts.

GEOLOGICAL ASPECTS OF SITING AND DESIGN

Prior to construction of the dam, the Columbia River flowed from Fivemile Rapids for about 3 mi through several narrow channels and closed rock basins to Threemile Rapids immediately below the present spillway. Through those channels the bedrock surface beneath the river was generally below sea level, and three gigantic "potholes" extended to 118, 195, and 213 ft below sea level. Of these, Big Eddy, the widest and shallowest, was a few hundred feet upstream from the dam axis. The two deeper potholes were in the main channel adjacent to and downstream from the spillway. A rock reef extending across the river formed a barrier at Threemile Rapids (Hodge, 1932). During construction the potholes were used as disposal areas, and they are now partly filled with waste rock.

The site of the powerhouse, spillway, and the connecting concrete nonoverflow dam was a rock peninsula around which the river flowed in a broad U-shaped course between Big Eddy and Threemile Rapids. Channels, basins, and potholes marked the surface of the peninsula, while on the riverward side vertical cliffs dropped 40 ft to low water.

The peculiar channel arrangement made this an ideal site for dam construction. It was possible to build the concrete structures without diverting the river and to finally close the river channel with a dumped embankment after the completion of all concrete structures. The peninsula provided space required for the long structures and made it possible to begin rough excavation for all concrete structures prior to cofferdam construction.

During preliminary planning, the downstream miter gate for the navigation lock was sited upstream of Threemile fault to avoid problems of anchoring the gate in brecciated rock.

CONSTRUCTION PROBLEMS

In general, the foundation rock conditions were very good, and construction problems as a result of geology were minor. During the excavation for the various structures it was noted that different rock types react differently to the same blasting procedure and that the amount of overbreak and the character of the final foundations were influenced by the rock structure. Several interesting minor rock structures were found in the excavations. Most of them required no special treatment, but a few required minor additional excavation to obtain sound rock. A few fissures were grouted before they were covered with concrete. A few large chamber or pipe-like openings, called spiracles, were found at the base of flows. Where these features occurred at or below final grade, they were cleaned out and backfilled with concrete.

Minor faults were found in the foundations of every structure. These were, in general, low-angle thrusts with displacements of only a few inches. Although they created no serious engineering problems, they were a nuisance, and much of the excavation below design grade was attributed to them.

OPERATIONAL PROBLEMS RELATED TO GEOLOGY

General

The Dalles Project has been remarkably free of significant operational problems that can be related to geology. Routine foundation drain and draft tube drain cleanout has been accomplished on a routine schedule of approximately once every 5 yr. Foundation instrumentation monitors underseepage, uplift pressures, and settlement. No foundation problems are known to exist.

Landslides

Although no problems have been experienced, the many landslides adjacent to the river create a potential for operational problems. The U.S. Army Corps of Engineers has documented the extent of slide activity adjacent to Lake Celilo, the reservoir for The Dalles Dam. General reconnaissance of the reservoir area indicates that no major slides have occurred since the 1968 survey year. Only small-scale activity continues. On the Oregon shore at Rufus, Quaternary fan and alluvial deposits are highly erodible and susceptible to mass wasting and can host small-scale slump and debris flows. Rockfalls and rockslides on the steep slopes surrounding Lake Celilo are potential failure sites. Between the Deschutes River confluence and Rufus, only minor rockfall sites exist. The thick sequences of surficial materials along the reservoir banks are possible sites for mass wasting, such as slumps. The steep cliffs between The Dalles and the Deschutes River confluence are sites of rockfalls and slides, especially near spring lines, faults and the interface between volcaniclastic deposits and basalt (U.S. Army Corps of Engineers, 1983).

REFERENCES

Hodge, E. T., 1932, *Report of Dam Sites on Lower Columbia River*: U.S. Army Corps of Engineers, Pacific Division, Portland, OR, 84 p., 37 plates.

U.S. Army Corps of Engineers, 1953, *Foundation Report, The Dalles Dam, The Dalles, Oregon, Part 1 Introduction and Spillway*: U.S. Army Corps of Engineers, Portland District, Portland, OR, 32 p., 14 plates.

U.S. Army Corps of Engineers, 1956, *Foundation Report, The Dalles Dam, The Dalles, Oregon, Part 2 Powerhouse*: U.S. Army Corps of Engineers, Portland District, Portland, OR, 42 p., plates 15-35.

U.S. Army Corps of Engineers, 1983, *The Dalles and John Day Lakes, Earthquake and Fault Study, Design Memorandum No. 26*: U.S. Army Corps of Engineers, Portland District, Portland, OR, 66 p., 19 plates.

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

John Day Dam

JOHN W. SAGER
U.S. Army Corps of Engineers

PROJECT DESCRIPTION

John Day Dam (Figure 1) is on the Columbia River in Oregon and Washington, 215.6 river miles from its mouth and approximately 112 mi east of Portland, Oregon, via Interstate Highway 84. The powerhouse is located in Oregon; the spillway and navigation lock are in Washington. The project is located in the rugged semiarid Umatilla Basin east of the Cascade Range. Lake Umatilla extends from John Day Lock and Dam upstream 76.4 mi to McNary Lock and Dam. The project became operational in April 1968, following approximately 10 yr of construction efforts.

John Day Dam was constructed for power, navigation, flood control, irrigation, and recreational use. Lake Umatilla has a 500,000 acre-ft storage capacity at normal maximum pool. The 76-mi-long lake also provides a 105-ft head for power generation. John Day Dam consists of a multiple-unit structure 5,900 ft in length. These units include: (a) a south shore fish ladder; (b) a 1,975-ft-long powerhouse with 16 generating units rated at 2,160,000 kw; (c) a 105-ft-high x 1,252-ft-long concrete spillway, rated for a discharge of 1,060,000 cfs; (d) a single-lift navigation lock, 86 ft wide by 675 ft long, adjacent to the spillway; (e) a north shore fish ladder and facility; and (f) a Washington shore rockfill embankment.

SITE GEOLOGY

Overburden

A generalized geologic map and plan of the site and typical geologic sections through the major structures at John Day Dam are shown on Figures 2 and 3. Overburden units in the foundation area consist of thin, discontinuous layers of Holocene silt, sand, and gravel in and adjacent to the river and older terrace silt, sand, and gravel higher on the south abutment. Overburden has been removed from the foundation beneath all structures.

Bedrock

At John Day Dam, nine flows of the Columbia River basalt are exposed between the bottom of the Columbia River and the south rim of the gorge. The upper six flows were called the "Rufus" basalt group, now termed Wanapum Basalt. The Rufus basalt sequence is the same as that at The Dalles (U.S. Army Corps of Engineers, 1956), except that the uppermost flow is absent at the John Day Dam. The lower three flows exposed at the site were originally called the "Quinton" basalt group, now called the Grande Ronde Basalt.

The distinctive feature of the Quinton basalt group is that each is made up of two strikingly different types of rock. The basal portion of the flow is dense, dark, nearly featureless basalt, and the upper part is a highly fragmented and partially cemented aa breccia. The interphase between the basalt and the breccia is an extremely rough surface with relief of 20 to 50 ft. Numerous tapered wedges and tongues of basalt rise above the general level of the interphase and penetrate the breccia for distances of 20 ft. In places the interphase horizon is smooth and nearly horizontal over distances of 100 to 200 ft and then rises abruptly in a vertical or overhanging wall. On the Washington shore at John Day Dam and at Quinton, Oregon, 11 mi upstream, two of these flows form nearly horizontal stripped bedrock terraces with craggy topography, to which Hodge (1932) gave the name "Quinton" topography. Quinton topography is a distinct type of scabland topography and develops only in flow breccia or Quinton-type lava flows. It is the result of the removal of the soft aa breccia by stream erosion and the exposure of the rough basalt-breccia interphase surface.

Foundations of all structures were excavated on the upper Grande Ronde Basalt. During preconstruction exploration and design, individual basalt flow contacts were given the number designations shown on Figure 3. Three distinctive basalt flows form the foundations for all the dam structures. Individual flow thicknesses range from 94 to 160 ft and show little lateral variation in thickness. Intraflow structure from top to bottom includes a ropey to clinkery vesicular flow top comprising about one-third of the flow thickness. The remaining flow is made up of a colonnade and less distinct entablature that gives a blocky appearance. Flows are separated by a basal zone of glassy basalt altered to a brown, soft, weak, friable, highly fractured, slightly cemented rock. Engineering characteristics of individual flows include: a highly permeable flow top breccia that readily transmits ground water owing to pervasive interconnected

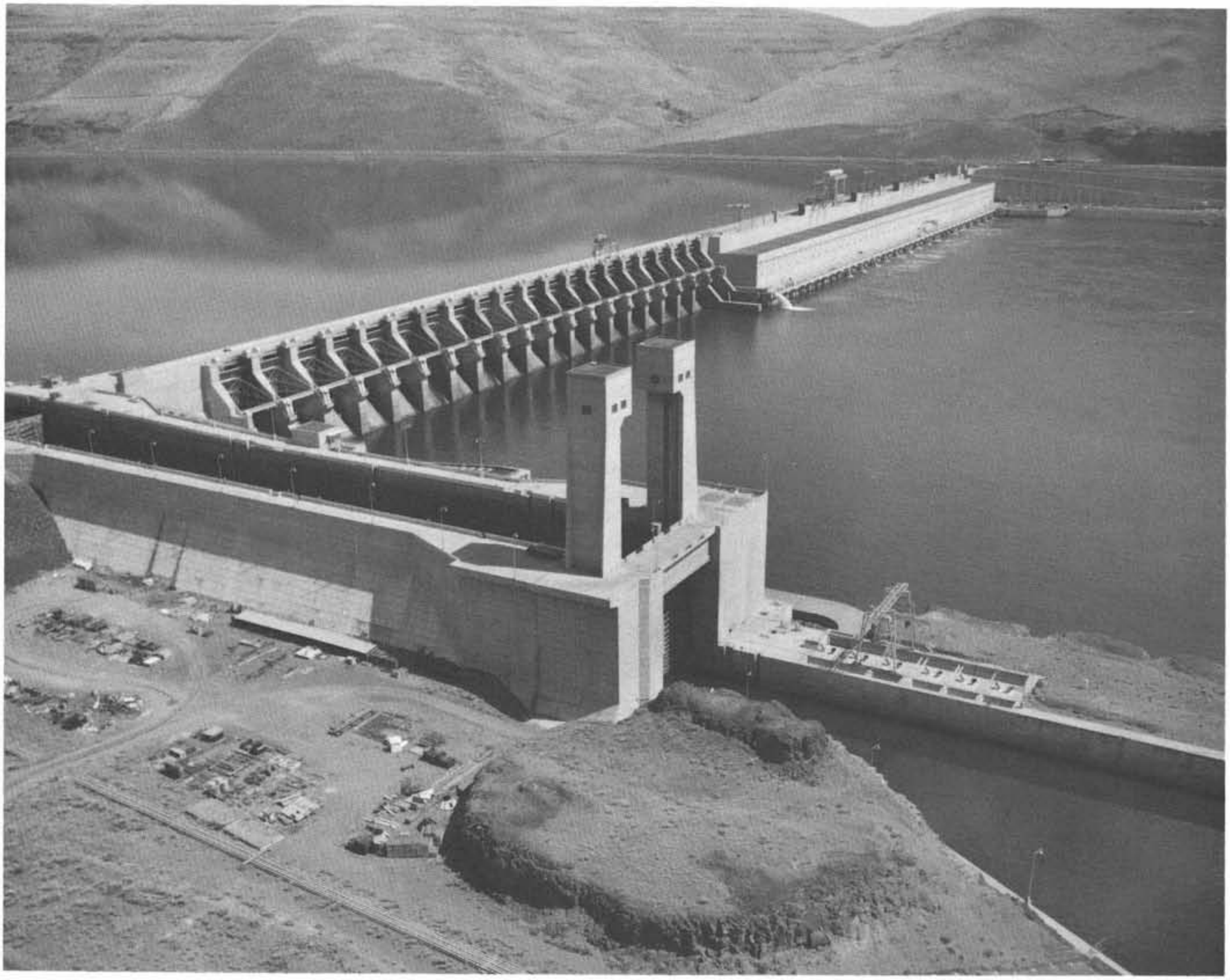


Figure 1. Aerial view of John Day Dam south (from the Washington shore) showing the navigation lock (foreground), spillway dam (center), and powerhouse/intake structure (upper right). Photograph courtesy of U.S. Army Corps of Engineers.

void spaces; a dense, very hard but open-jointed colonnade that also transmits water through joints and is separated from the flow top by irregular tiered zones that penetrate the flow top 20 or more feet; and a basal zone of weak weathered rock. Flow top breccia was particularly amenable to grout injection.

Jointing and Faulting

The joint system is different in each basalt flow, but the various joint set patterns have some common characteristics. Colonnade columns are larger than overlying entablature columns and typically are defined by pentagonal or hexagonal cooling (tensional) joints. Entablature columns tends to be four-sided. Some joints radiate locally, and some columns contain niche joints and ball-and-socket fractures. Regional conjugate joint sets overprint cooling and shrinkage joints. Regional sets are oriented north-south (conjugate east-west) and dip at high angles; regional dip is to the east. Regional structural responses to local tectonics have produced a number of shear and fault features in the foundation area. Among them are a high-angle thrust fault related to the south limb of the Columbia Hills anticline. This fault has open gouge and rubble zones and a maximum of 20 ft displacement. A small northwest-trending anticline has caused northeast-southwest arching and minor bedding and tensional folding shears. Shears are oriented along the axis of folding, are tightly closed, and contain little or no gouge. A series of N20°W-trending transcurrent faults has displaced the Columbia Hills structural axis at several locations. Minor high-angle fault zones subparallel and related to this trend appear in the foun-

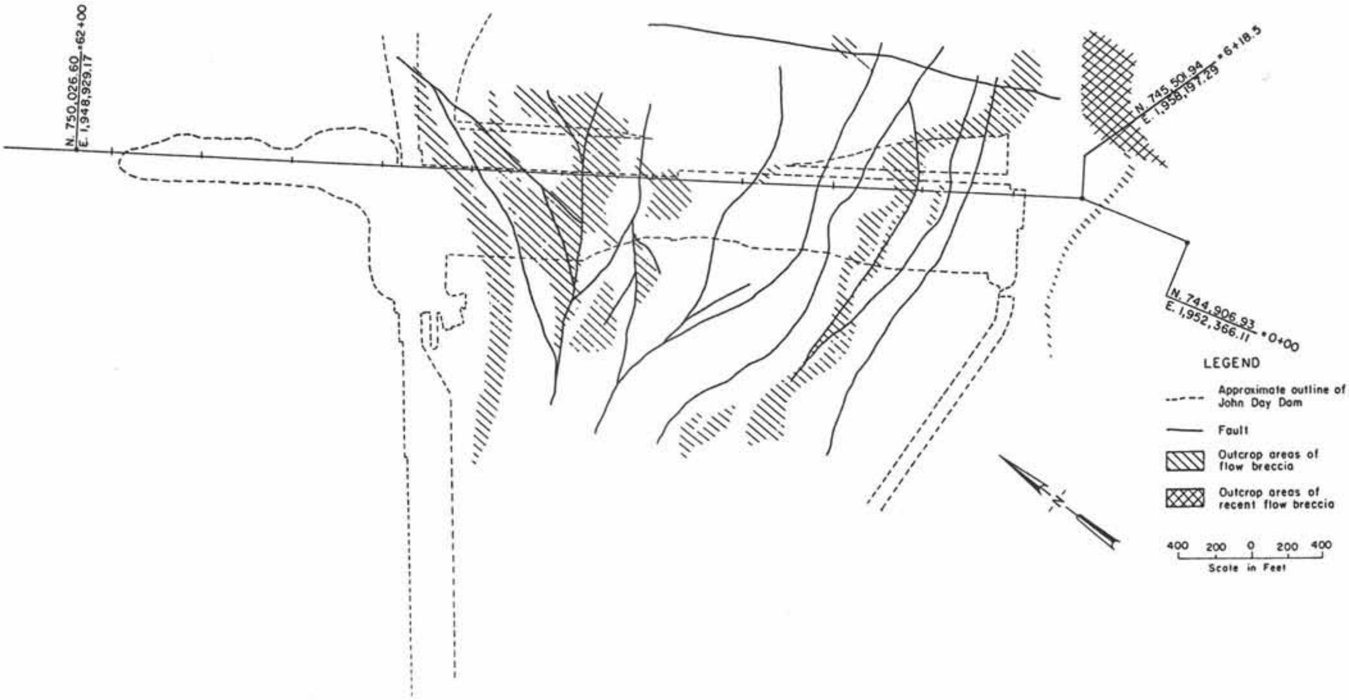


Figure 2. Geologic map of the John Day Dam area showing faults and areas of exposed flow breccia in the foundation and adjacent areas.

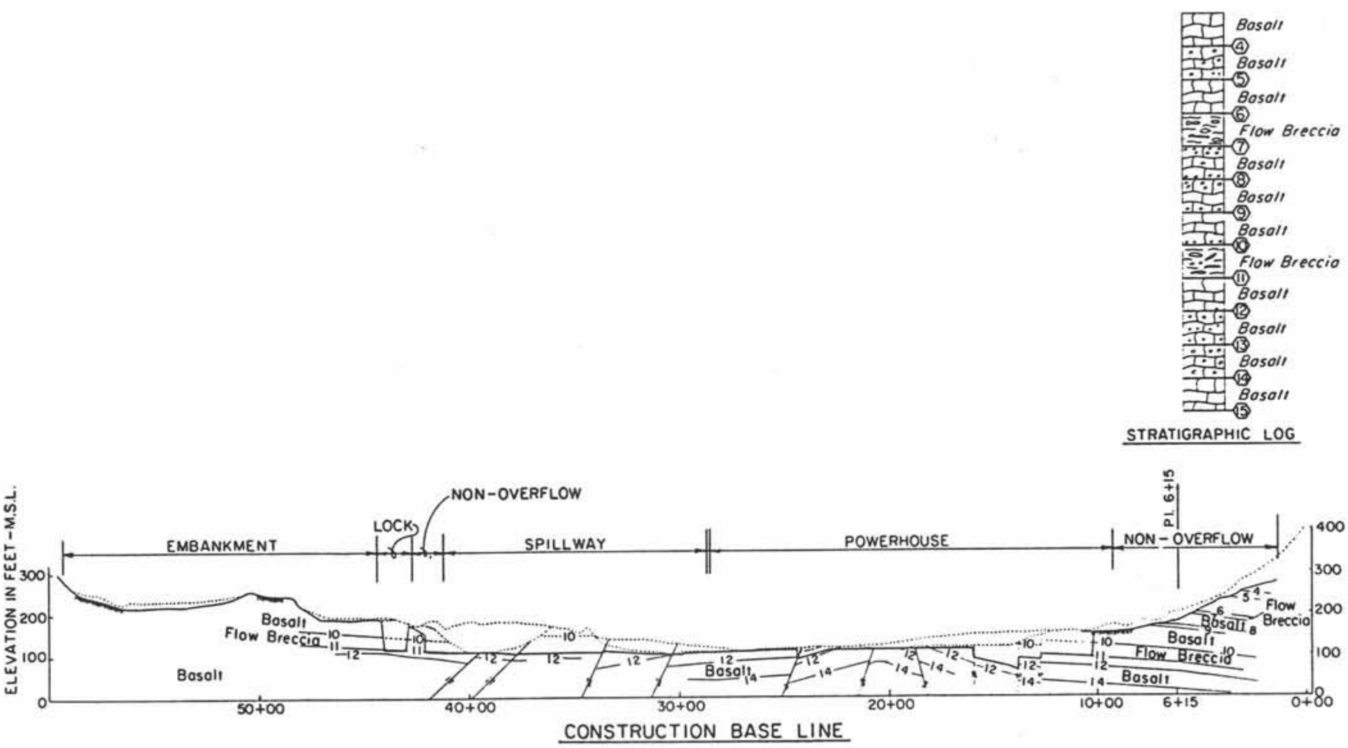


Figure 3. Geologic section of John Day Dam along the axis; view upstream. Contacts between the several flows and flow breccias of the Grande Ronde Basalt are numbered sequentially downward.

dation area. These faults are vertical strike-slip features and have minor gouge zones. Many are *en echelon* across several monoliths in the spillway and powerhouse, but the rock is not severely brecciated. However, one fault has a wide zone of breccia and gouge that cuts the spillway and stilling basin. Fault trends and displacement directions are shown on Figures 2 and 3. Faulting appears to predate deposition of the overburden units since none of the overburden units revealed during excavation were cut by faults.

GEOLOGICAL ASPECTS OF SITING AND DESIGN

Major geological features that influenced siting and design included the various structural features within the bedrock, the configuration of the top of bedrock, and the location and extent of the poor quality flow breccia. The existence of a deep, ancient, conglomerate-filled gorge downstream from the site was also a major siting factor. Bedrock drops off rapidly immediately downstream of the powerhouse. This low bedrock surface limited the downstream location of the structure. A similar low immediately upstream of the navigation lock established the upstream limits of that structure. The pattern of the bedrock surface, excepting the ancient gorge downstream from the dam, is related in part to faulting in the area but is more commonly due to erosion by glacial flood waters.

CONSTRUCTION PROBLEMS

The Columbia River valley in this area has produced several geologic phenomena that significantly affected the engineering design and construction of this project. These phenomena include not only the physical properties of the basalt and flow breccia bedrock as foundation materials, but also the quality of the bedrock and overburden for construction materials.

A major modification was necessary during construction as a result of an extensive flow breccia encountered at the design foundation grade for the station service bay, assembly bay, fishwater intake, and generator units one through nine. The extensive excavation required to remove this zone of poor rock was backfilled with mass concrete to provide a suitable foundation for the powerhouse structure.

Excavation for the navigation lock was accomplished after construction of a grout curtain around the perimeter of the lock area. The grout curtain controlled seepage into the excavation area and allowed the vertical rock cuts to be excavated with a minimum amount of slope instability.

A highly fragmented and partially cemented flow breccia, composed primarily of vesicular clinkers, was encountered near the downstream navigation lock gate monoliths, stilling basin, and spillway piers one through eight, and was removed to sound foundation rock. This additional excavation was backfilled with mass concrete, although in some areas where faults were crossed in bedrock, reinforcing steel was added to the mass concrete to provide a bridging action across such zones.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Navigation Lock Repair

Recent repairs to the John Day navigation lock (Figure 4) provide an example of a problem that occurred as a result of major differences in physical characteristics between the basalt and flow breccia foundations. Three layers of dense basalt were involved in the navigation lock foundation excavation, and between each was a layer of flow breccia. In general, the basalt units are composed of hard, fine-grained, dark gray, dense basalt. Flow breccia occurs below the upper basalt (horizons 10 to 11). It is composed in part of solidified lava and in part of ejected volcanic materials in the form of ash, lapilli, and bombs, commonly welded together. This flow breccia marks the original upper surface of a separate lava flow. Rapid cooling, weathering, alteration, and the heterogeneous nature of the rock has produced a brown layer that is soft, weak, crumbly, highly fractured (or brecciated), slightly cemented, and rough textured. Due to the nature of its deposition, both the top and bottom flow breccia contacts are very irregular. Unit thickness normally ranges from 25 to 40 ft. The flow breccia also contains a large number of void spaces, many of which are interconnected. It is this characteristic that makes the stratum very permeable and particularly amenable to grout injection. The original grout curtain was very effective in reducing seepage into the excavation and preventing ravelling and sloughing of the flow breccia during lock chamber excavation.

The basalt that underlies the flow breccia (below horizon 11) is similar to the upper basalt and is the lowest stratum involved in the foundation. The filling and emptying culvert sections for the lock are founded on the upper surface of the lower basalt (horizon 11).

In terms of the engineering characteristics of the foundation rock mass, it is the inherent differences in rock properties and physical behavior between the basalt and flow breccia that are most significant. The flow breccia exhibits considerably lower shear strength, unit weight, and elastic parameters than the basalts. Special emphasis during design and construction was placed on evaluating the flow breccia's load-carrying capacity and ability to serve adequately as a foundation material. Although the flow breccia rock was ultimately considered by the lock designers as a suitable foundation for the lock floor slab, it was removed where necessary to found the culvert sections on basalt. Instrumentation measurements (1978-1979) showed that very little vertical deformation occurred in the foundation during lock

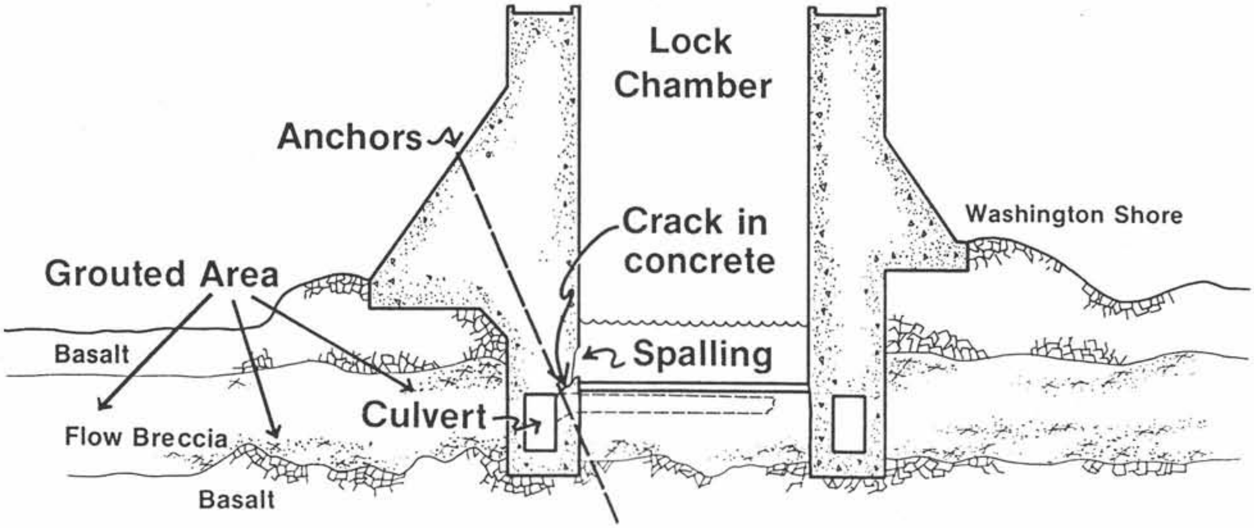


Figure 4. Typical section of lock repair required at John Day Dam showing locations of tendon anchors.

filling and emptying, so distribution of vertical stress within the foundation was not the problem. There is apparently a considerable degree of anisotropy in the rock mass deformation characteristics. This is a normal condition in layered rock, but has been increased by the effects of weathering and the presence of natural discontinuities in the rock left in place between the fishway and spillway.

Lateral restraint was minimized by the excavation configuration, and stress relief fractures undoubtedly occurred as a result of blasting and removal of the surrounding rock. Another factor that probably decreased the rock mass modulus of deformation in the lateral direction was the prolonged period of exposure to which the foundation slopes were subjected during construction before concrete was placed against them. This combination of factors, plus the presence of the much less rigid flow breccia, allowed considerable lateral, elastic deformation to occur within the rock mass between the riverside monoliths and the river. It was this elastic response that allowed the riverside monoliths to deflect during lock filling and eventually led to the wall cracking and spalling. Because of the elastic response of the rock and the reorientation of the monolith gravity load resultant, the monoliths returned to their original position upon lock emptying.

The flow breccia as a foundation material is weathered, and of low density and exhibits considerable variation in hardness, shear strength, and deformation parameters. Laboratory testing has shown a wide range in values for strength and elastic modulus, due mainly to the petrologic and mineralogic variations that occur both laterally and vertically within the flow breccia. Where portions of the breccia have been altered to palagonite (soft, brown or greenish-black alteration product of basaltic glass and composed primarily of montmorillonite), the strength and consolidation properties are more similar to those of soil than rock. At the other extreme, the flow breccia approaches a highly vesicular basalt and exhibits much higher strength, hardness, and related characteristics. The true strength and deformation properties of the entire flow breccia rock mass are also, of course, controlled to a large extent by the discontinuities that occur within the layer. The flow breccia also exhibits very high permeability and transmissivity due to interconnection of the large percentage of void space within the rock.

In 1980, consolidation grouting using cement grout was performed as the first phase of the foundation repair. The purpose of this grouting was to fill the open joints and void spaces in the rock to make the riverside wall foundation more rigid. The soft, weak, highly fractured flow breccia layer was the primary target of the consolidation grouting efforts. A specific goal was to increase the deformation modulus of the flow breccia from an assumed value of 500,000 psi to 1,000,000 psi, the modulus of solid flow breccia core samples tested in the laboratory. Finite element modeling during design showed that increasing the rigidity of the confining and supporting rock mass would significantly reduce the monolith deflections and decrease excessive tensile stresses around the filling culvert.

Consolidation grouting was completed by August 1980. Some 30,000 cu ft of grout was injected in an estimated 40,000 linear ft of 1.5-in.-diameter drill holes (approximately 600 holes). The grout was normally injected with a water-cement ratio from 3.0:1 to 5.0:1. The average grout take in the flow breccia zone was 1.55 sacks of cement per linear foot of drill hole. Cores from exploratory borings drilled after grouting demonstrated the effectiveness of the grout in consolidating the highly fractured flow breccia.

The second phase of the repair program was completed in September 1981. During this phase, 73 large, high-capacity rock anchors were installed through seven of the monoliths and into the foundation. Each rock anchor consists of a strand-type tendon installed in a 10-in.-diameter hole and tensioned to a working load of 1,300,000 lb. The tendons were spaced as close as 4 ft and averaged about 150 ft in length. Drilling was accomplished with three down-hole hammer drills modified and adapted to work from a specially constructed trestle attached to the face of the rock to provide access and allow the drills to move laterally between setups. Because of the position and orientation of the rock anchors, the allowable hole deviation tolerance was 1 ft in 100 ft to assure crossing the primary crack at the optimum location and avoid penetrating the lock chamber wall or filling culvert.

Landslides

There is a significant amount of landslide activity in the region near Lake Umatilla. General reconnaissance of the reservoir shoreline indicates no new major slides have developed in recent years. Large areas of deep bedrock instability exist on the south side of the Columbia Hills structure. On the Washington shore the normal sequence of exposed basalt members from top to bottom includes Elephant Mountain, Pomona, Umatilla, and Priest Rapids in the eastern lake area. Near John Day Dam the Roza Member overlies the Frenchman Springs basalt, the lowest exposed member. These flows are disturbed and juxtaposed by the Columbia Hills thrust zone. Generally the zone traverses the Washington shore area at mid-slope elevations. The Roza and Frenchman Springs flows have discontinuous interbeds of saprolite and minor tephra that reduce slope stability. The Priest Rapids basalt includes a 25-ft-thick tuffaceous siltstone and claystone unit that is the chief detachment plane for rotational and translational failures. The Umatilla Member has a sedimentary interbed, the Pomona Member is chiefly an intracanyon flow (but is present in a large area on the Oregon side), and the Elephant Mountain Member has a discontinuous tuffaceous interbed. Underlying the Elephant Mountain and Pomona members is an extensive volcaniclastic deposit of poorly sorted, weakly lithified andesitic to rhyolitic detritus chiefly erupted from Cascade volcanoes. This deposit is extensively involved in slumps and flows and serves as a detachment plane for overlying bedrock slides. Interbeds combined with the Columbia Hills thrust have caused extensive mass wasting and instability along sections of the Washington shore. Some minor instability, such as rock falls and slumps, also exists on the Oregon shore.

REFERENCES

Hodge, E. T., 1932, *Report of Dam Sites on Lower Columbia River*: U.S. Army Corps of Engineers, Pacific Division, Portland, OR, 84 p., 37 plates.

U.S. Army Corps of Engineers, 1970, *Construction History., John Day Dam, Columbia River, Oregon and Washington*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 37 p., 30 plates.

U.S. Army Corps of Engineers, 1983, *The Dalles and John Day Lakes, Earthquake and Fault Study, Design Memorandum No. 26*: U.S. Army Corps of Engineers, Portland District, Portland, OR, 66 p., 19 plates.

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

McNary Dam

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

PROJECT DESCRIPTION

McNary Dam (Figure 1) is a multiple-purpose project on the Columbia River on the border between Oregon and Washington and is approximately 292 mi above the mouth of the river. It is the fourth and uppermost in the series of dams on the lower Columbia Rivers.

The primary authorized purposes of McNary Lock and Dam are power generation and inland navigation, but other uses include fisheries, recreation, irrigation, and water quality maintenance. The project provides a dam 7,365 ft long, which raises the water surface about 85 ft and creates a lake (Lake Wallula) extending 64 mi upstream to a point approximately 27 mi above Pasco, Washington. The dam consists of a 2,495-ft-long earthfill embankment on the Oregon (south) side; a 1,422-ft-long powerhouse with 14 generating units at 70,000 kw each; a concrete, gravity-type spillway dam 1,310 ft long containing 22 lift gates; a navigation lock 86 ft wide by 675 ft long; a 1,620-ft-long earthfill embankment at the Washington (north) side; and two fish ladders for migratory fish. The maximum overall height of the concrete dam is 191 ft. The cities of Pasco, Kennewick, and Richland, Washington, near the upper end of the reservoir are protected by 16.8 mi of levees. Drainage and ground-water levels landward of the levees are controlled by 15 pumping plants (U.S. Army Corps of Engineers, 1982a).

Construction of the McNary Project began in May 1947. The reservoir was raised to full capacity (1,450,000 acre-ft), and the first power units were placed in operation in December 1953. All 14 power units were in operation by February 1957.

SITE GEOLOGY

The Columbia River valley at the dam site is approximately 250 ft deep and more than 1.5 mi wide. The dam is founded on the Umatilla Member of the Saddle Mountains Basalt; the upstream and downstream shells of both embankments and the left embankment tie-in are founded on *in situ* alluvial materials. The Mabton interbed and Priest Rapids Member of the Wanapum Basalt underlie the Umatilla Member. Extensive areas of glaciofluvial and reworked flood gravel terraces occur on both abutments of the dam. Eolian sand and silt mantle most of the gravel terraces. Recent alluvial gravels occur on both banks of the river downstream of the dam (Kienle, 1980; U.S. Army Corps of Engineers, 1982b). The geology near the dam is shown on Figure 2 and in the section along the centerline of the dam in Figure 3.

The main foundation rock is the Umatilla basalt. It is a hard, dark gray to black, dense, and vitric rock with small plagioclase laths in a fine-grained matrix. Jointing in the rock is prominent and ranges from randomly oriented ball and socket-type jointing to well developed columnar jointing. The joints and fractures are generally healed with dark green chloritic minerals or calcite. Locally, the flow dips about 1° southwest.

Two thrust faults are present in the foundation area. One fault is beneath the powerhouse, and one is beneath the spillway. Both faults trend at acute angles to the flow of the river and have offsets on the order of 10 to 15 ft (U.S. Army Corps of Engineers, 1950-53).

On the upper surface of the Umatilla basalt is a contact flow breccia of varied thickness. Much of the breccia was removed by river erosion, but it remains in some areas of the north and south embankment foundations. The breccia consists of hard to highly scoriaceous fragments in a soft, cindery matrix.

Immediately underlying the Umatilla basalt at depths ranging from 25 to 50 ft below foundation grade is the Mabton interbed. This sedimentary unit ranges in thickness from 40 to 60 ft and consists of thin bedded tuffaceous siltstones and claystones with some intercalated coarser materials.

The oldest rock unit penetrated by foundation drilling at the site is the Priest Rapids Member of the Wanapum Basalt. The basalt is dark gray to black and slightly vitric and has small plagioclase laths in a fine-grained matrix. Its contact with the Mabton interbed is brecciated and highly fractured and forms an artesian aquifer zone (U.S. Army Corps of Engineers, 1984).

GEOLOGICAL ASPECTS OF SITING AND DESIGN

The major geologic feature that influenced the siting of McNary Dam is the Service anticline (Figure 2). This structure is a north-trending fold with its axis perpen-

Figure 1. McNary Dam; view to the east (upstream). From left to right in the photo is a portion of the right (Washington) embankment, navigation lock, spillway dam, intake dam and powerhouse, and left (Oregon) embankment. Fish ladder facilities are located riverward of the navigation lock and along the downstream toe of the left embankment. Photograph courtesy of the U.S. Army Corps of Engineers.

dicular to and crossing the river approximately 1 mi downstream of the dam. As the Columbia River entrenched itself across this anticlinal fold, a series of rapids was formed. Over geologic time these rapids migrated upstream to the vicinity of the present dam, where they were referred to as the Umatilla Rapids (Monahan, 1957). Since two of the primary purposes of the dams on the Columbia and Snake rivers are power and navigation, each dam was sited to provide the maximum head for power and most favorable backwater profile for navigation. Hence, at McNary, full advantage was taken of the presence of the Umatilla Rapids. Additionally, downstream of the site, river erosion had cut through the Umatilla Member and exposed the Mabton interbed, which was considered unsuitable for founding concrete structures. At the present site, this latter unit is covered by a minimum of 25 ft of basalt, thus providing a suitable foundation for the dam.

CONSTRUCTION PROBLEMS

The basalt bedrock provided a structurally competent foundation on which to place the concrete dam—with the exception of the two thrust faults in the spillway and powerhouse areas (Figure 3) which required remedial treatment. The fault beneath the powerhouse had a narrow gouge zone; minor overexcavation was required to remove the gouge and highly weathered rock and to replace it with several feet of dental concrete. The fault beneath the spillway was more troublesome. The fault cut the foundation at a fairly low angle at this point and contained a red clay gouge several feet thick. Excavation was carried along the footwall of the fault until a sufficient mass of sound rock was present above the hanging wall to support the concrete structures (Monahan, 1957).

Figure 2. Photogeologic map of the McNary Dam area. Heavy solid lines indicate approximate position of contacts. Ql, loess; Qgf, glaciofluvial deposits; Tyem, Typo, and Tyum, Elephant Mountain, Pomona, and Umatilla members of the Saddle Mountains Basalt. Scale: 1 in. = approximately 2,000 ft. From U.S. Corps of Engineers (1982b).

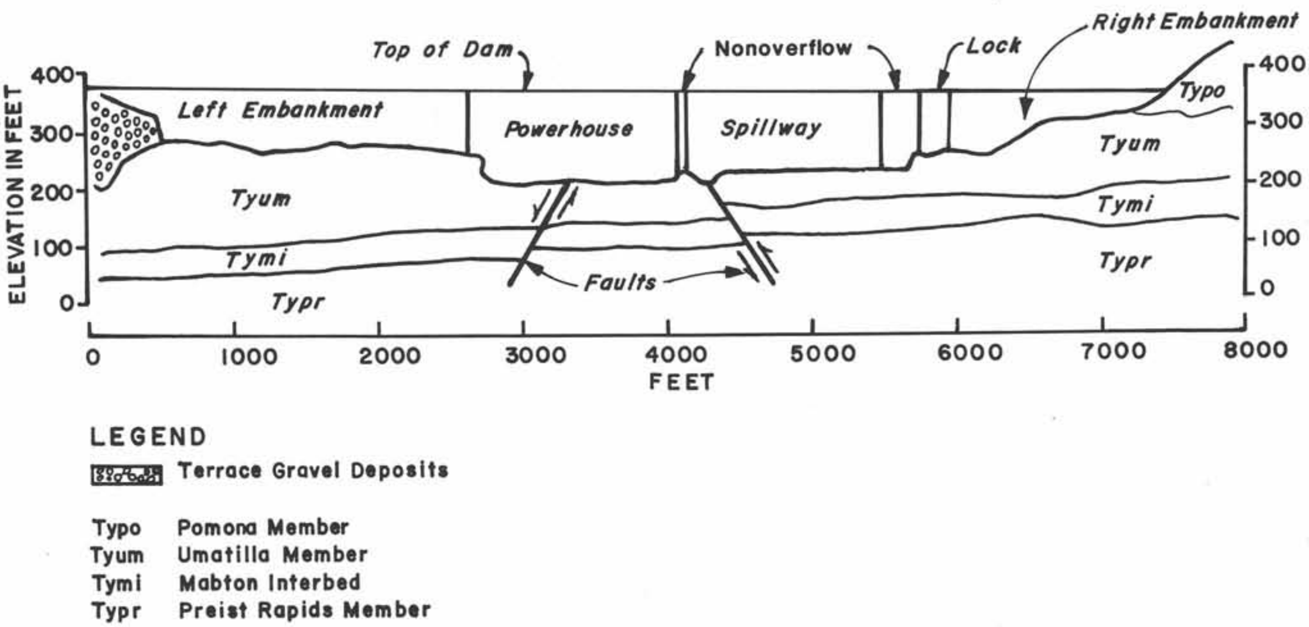


Figure 3. Geologic section at McNary Dam; view downstream.

The subcontact zone between the main basalt flow and the flow breccia is very irregular. River erosion had removed most of the breccia in the upstream portion of the foundation for the concrete structure, so very little excavation below design grade was required. In the downstream areas of the foundation, particularly in the powerhouse area, the contact was much deeper, but because draft tube design required a lower design grade, very little additional excavation was required. Flow breccia comprised most of the foundation rock for the south embankment dam and only a portion of the north embankment. Because the foundation strength requirements were less for the embankment dams, the flow breccia was left in place, except for that removed during normal foundation treatment to remove loose pieces or to provide a satisfactory surface for compaction of embankment materials.

Unconsolidated sandy gravel terrace deposits form the left abutment tie-in of the dam. The bedrock surface at this location contains a deep scour channel, and a rock abutment tie-in would have required extensive excavation and replacement with embankment material. In lieu of that option, it was determined that a 1,000-ft-long seepage blanket placed upstream of the impervious core of the dam would minimize end-around seepage. The seepage blanket was tied to bedrock at the toe and extended upslope to a point above reservoir level. An internal drainage system was installed in the abutment to collect excess seepage and was extended downstream to empty into the river below the dam. The blanket and drainage system proved to be very effective; only approximately 3 cfs discharges from this area.

The primary foundation treatment consisted of a grout curtain placed along the centerline of the dam and inclined upstream at a 17° angle, measured from vertical. Water pressure testing during foundation explorations and experimental grouting programs in several areas of the foundation gave preliminary indications that the Umatilla basalt had low to moderate permeability and that the underlying Mabton interbed had extremely low permeability except in isolated zones. Therefore, the decision was made to extend the grout curtain through the Umatilla basalt to the top of the interbed.

The greatest construction effort was directed to the placement of the grout curtain beneath the concrete structures. The split-spacing, stage grouting method was used with the initial EX-size holes drilled on 20-ft centers and a final spacing of 5 ft. The grout holes averaged 75 ft in depth; the grout injection rate was approximately 0.29 sacks of cement per foot of drill hole. The grout curtain beneath the north (Washington) embankment was accomplished through EX-size holes drilled on 10-ft centers to an average depth of 80 ft. Grout injection rates were approximately 0.20 sacks of cement per foot of hole. Beneath the south (Oregon) embankment, EX-size holes were drilled on 10-ft centers and 30 ft into rock. Grouting was done in a single stage. Grout injection rates were less than 0.10 sack of cement per foot of hole (U.S. Army Corps of Engineer, 1950-53).

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Aside from the geologic problems encountered and corrected during and immediately following construction, McNary Lock and Dam has remained relatively trouble-free.

An uplift study initiated in 1953 to test the effects of the foundation grout curtain and drainage relief system revealed an uplift gradient above the design assumption in four of the powerhouse intake monoliths. The uplift pressures showed a seasonal fluctuation of 9 to 17 psi ; peak pressure occurred from January to March and lower pressures occurred from July through September. The pressures have been monitored since the initial study and have shown the same cyclical trend. One theory postulated for this cyclic trend is that during the winter, when the pressures are highest, the cool (39°F) water tends to contract the foundation rock, thus opening joints and fractures wider and allowing more seepage to penetrate a greater distance downstream. However, flow measurements from the drainage system do not reflect significantly larger flows.

Another concern (even prior to construction) was erosion of rock downstream of the spillway structure. The river has eroded completely through the Umatilla basalt, exposing the Mabton interbed in the crest of the Service anticline downstream of the dam. Upstream migration of the rapids formed in the Umatilla basalt had reached a point several hundred feet downstream of the spillway end sill at the time of construction. River diversion during construction concentrated flows into narrow channels where flow velocities approached 30 ft/sec; this undoubtedly accelerated channel erosion (Monahan, 1957). Soundings taken in 1938 and 1954, and again in the 1970s and 1980s, indicate a fluctuating erosion and sedimentation process that appears to represent a somewhat stable condition. However, the more recent investigations showed evidence of severe undercutting of the end sill. If it is determined that the erosion is progressing upstream, remedial treatment such as placement of large jetty stone or concrete mattresses downstream of the end sill may be necessary.

A geological and seismological review was conducted to assess the potential for seismic hazards near McNary Lock and Dam. It was concluded that the Wallula fault zone, located approximately 25 mi to the east and along the Horse Heaven anticline, was capable of generating a magnitude 6.5 earthquake and a sustained ground acceleration of 0.28 g (U.S. Army Corps of Engineers, 1982b). Seismic stability analyses recently completed indicate that the concrete structures could withstand those motions (U.S. Army Corps of Engineers, 1988). Analyses are continuing in order to determined the stability of the embankment dams under the predicted earthquake loadings and the stability of the Tri-Cities levees at the upper end of the reservoir.

REFERENCES

Kienle, C. F., Jr., 1980, *Geologic Reconnaissance of Parts of the Walla Walla and Pullman, Washington, and Pendleton, Oregon 1x2' AMS Quadrangles*: Prepared for the U.S. Army Corps of Engineers, Seattle District, by Foundation Sciences, Inc., Portland, OR, 66 p., 3 plates.

Monahan, C. J., 1957, Geologic Features at McNary Dam, Oregon-Washington. In Trask, P. D. (editor), *Engineering Geology Case Histories, Number 1*: Geological Society of America, Boulder, CO, pp. 33-38.

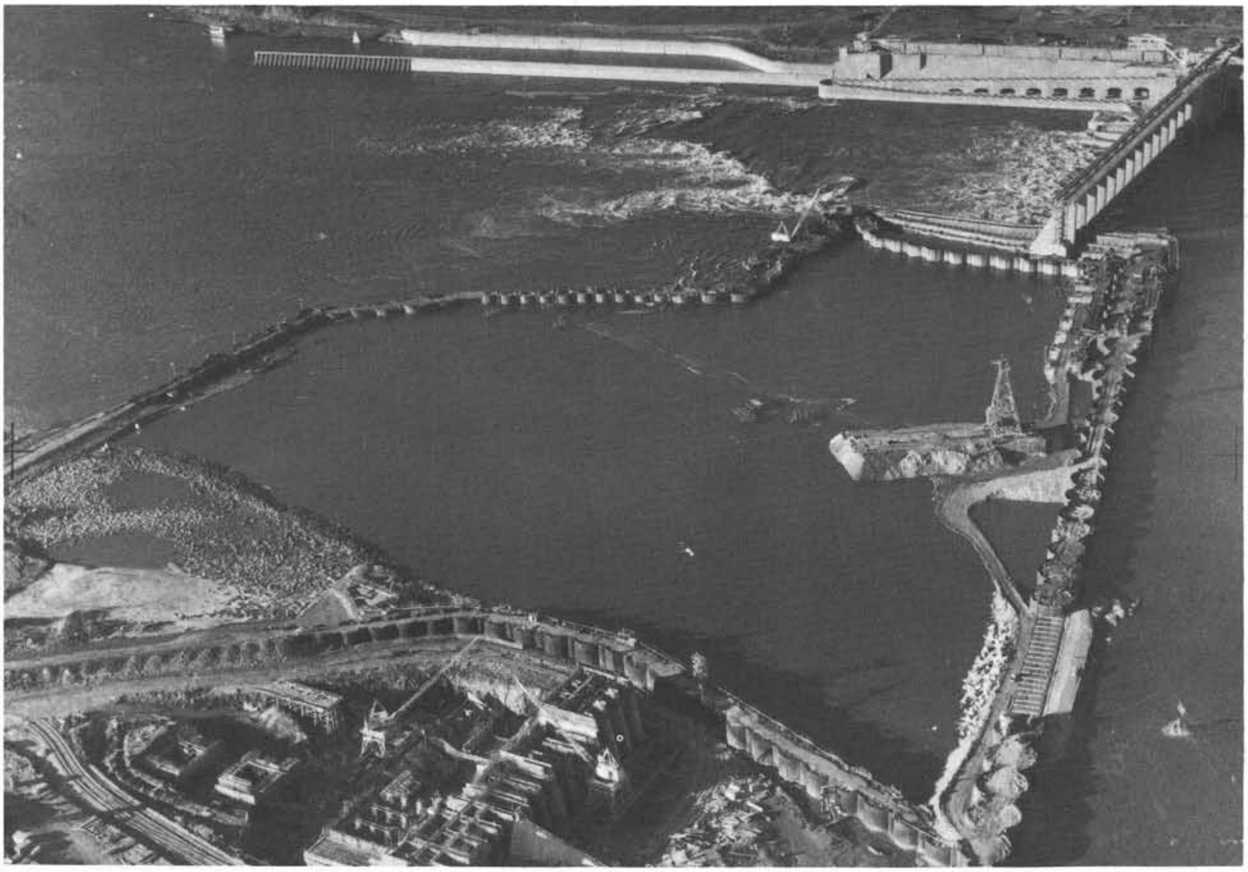
U.S. Army Corps of Engineers, 1950-53, *McNary Dam, Foundation Report, Chapters 1-11*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 99 p., 99 plates.

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, *McNary Dam; Geology, Foundations, Embankment, and Cofferdam–Design Memorandum No. 3*: U.S. Army Corps of Engineers, Walla Walla District, Walla Walla, WA, 39 p., 64 plates.

U.S. Army Corps of Engineers, 1988, *Special Report–Dynamic Analysis of Concrete Structures, McNary Lock and Dam*: U.S. Army Corps of Engineers, Walla Walla, WA, 10 p., 75 figures.



A general aerial view of construction, within the second-stage cofferdam, of the powerhouse on the Oregon shore at McNary Dam. The completed navigation lock and partially completed spillway are shown at the stop of this January 1951 photograph by the U.S. Army Corps of Engineers.

Dams of the Middle Columbia River

Introduction and Geologic Setting

Priest Rapids Dam

Wanapum Dam

Rock Island Dam

Rocky Reach Dam

Wells Dam

Aerial view of the middle Columbia River from the mouth of the Wenatchee River (lower right) upstream past Rocky Reach Dam (center). Photograph by R. W. Galster, March 1977.

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

Dams of the Middle Columbia River: Introduction and Geologic Setting

RICHARD W. GALSTER
Consulting Engineering Geologist
and
HOWARD A. COOMBS
University of Washington

Dams on the basically south-trending middle section of the Columbia River, between the mouth of the Snake River and the mouth of the Okanogan River, are unique in several respects. Unlike those on the remainder of the main stem, none of the existing dams were constructed by the federal government; all are products of local public utility districts (PUDs) or a private power company. All are "run-of-the-river" dams and, except for one, have no available storage above their respective power pool elevations. The first dam built on the Columbia River (Rock Island) lies within this middle segment, as does the dam most recently completed (Wells). The segment also contains the only available dam site remaining on the Columbia River in the United States (Ben Franklin). Unlike the lower Columbia dams, there are no navigation locks on the main stem dams above the mouth of the Snake. However, fish passage facilities are a feature of each dam in the reach. Also notable is the fact that all major tributaries enter the Columbia from the right (west) bank, draining the eastern slope of the Cascade Range. No major tributaries originate on the Columbia Plateau.

The mid-Columbia can be divided into four basic geologic segments (Figure 1). Between the mouth of the Okanogan and the mouth of the Wenatchee River, the river runs in a deep, steep-walled valley cut into the granitic/gneissic rocks of the northeastern Cascades. The valley has been modified by deposition of Pleistocene glaciofluvial fill, manifest in extensive spectacular terraces from Wenatchee upstream and known as the "Great Terrace of the Columbia". Two of the five dams (Rocky Reach and Wells) on the mid-Columbia lie within this segment.

The second segment lies in the several miles downstream from the Wenatchee River. Here the valley widens where it is underlain by softer Paleogene sedimentary rocks of the Chiwaukum graben. The Wenatchee urban area lies within this segment.

The third segment begins as the Columbia approaches the north side of the Wenatchee Mountains uplift; the river is deflected about 10 mi eastward onto the lava plateau underlain by the Columbia River Basalt Group. Southward, the river has cut a 600-ft-deep canyon into the generally flat-lying basalt. The canyon is considerably deeper in spectacular water gaps where the river passes through the anticlinal ridges: the Wenatchee Mountains at Rock Island, the Frenchman Hills at Vantage, and the Saddle Mountains at Sentinel Gap. Two dams (Rock Island and Wanapum) lie within this segment.

South of Sentinel Gap the river enters the final geologic segment, the Pasco Basin, a broad basin underlain by soft sediments of the Pliocene Ringold Formation and by Pleistocene glacial flood deposits. The sole exception to this is at Priest Rapids, where the river crosses basalt bedrock adjacent to Umtanum Ridge and is deflected 30 mi eastward by the Umtanum-Gable Mountain uplift to near the eastern margin of the Pasco Basin. Here the Columbia again turns south, cutting a 600-ft-high bluff in the Ringold sediments, the "White Bluffs of the Columbia", before crossing the southeastern part of the Pasco Basin where it is met by the Snake north of Wallula Gap. Only Priest Rapids Dam lies within this segment, although studies have been made at the Ben Franklin site in the Ringold Formation just upstream from Richland.

Much of the early stratigraphic work on the upper Columbia River basalts dates from the investigations for the downstream two dams: Priest Rapids and Wanapum (Mackin, 1961). This classic work has been enlarged upon and modified by numerous investigators (for example, Grolier and Bingham, 1978; Swanson et al., 1979; Reidel and Fecht, 1981). A stratigraphic diagram appropriate for the mid-Columbia is given in Figure 2. One of the continuing stratigraphic difficulties, especially toward the northern end of the basalt section on

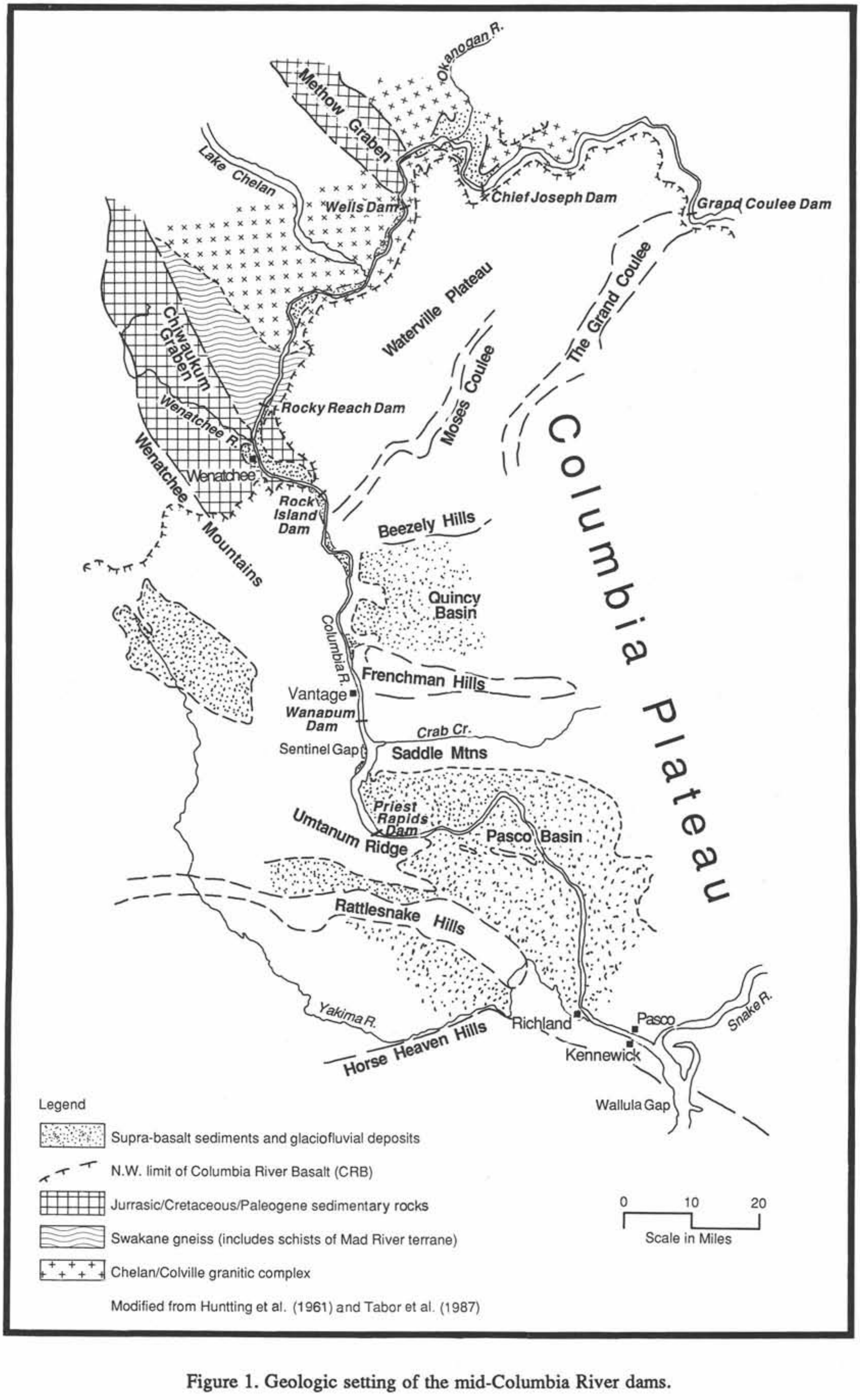


Figure 1. Geologic setting of the mid-Columbia River dams.

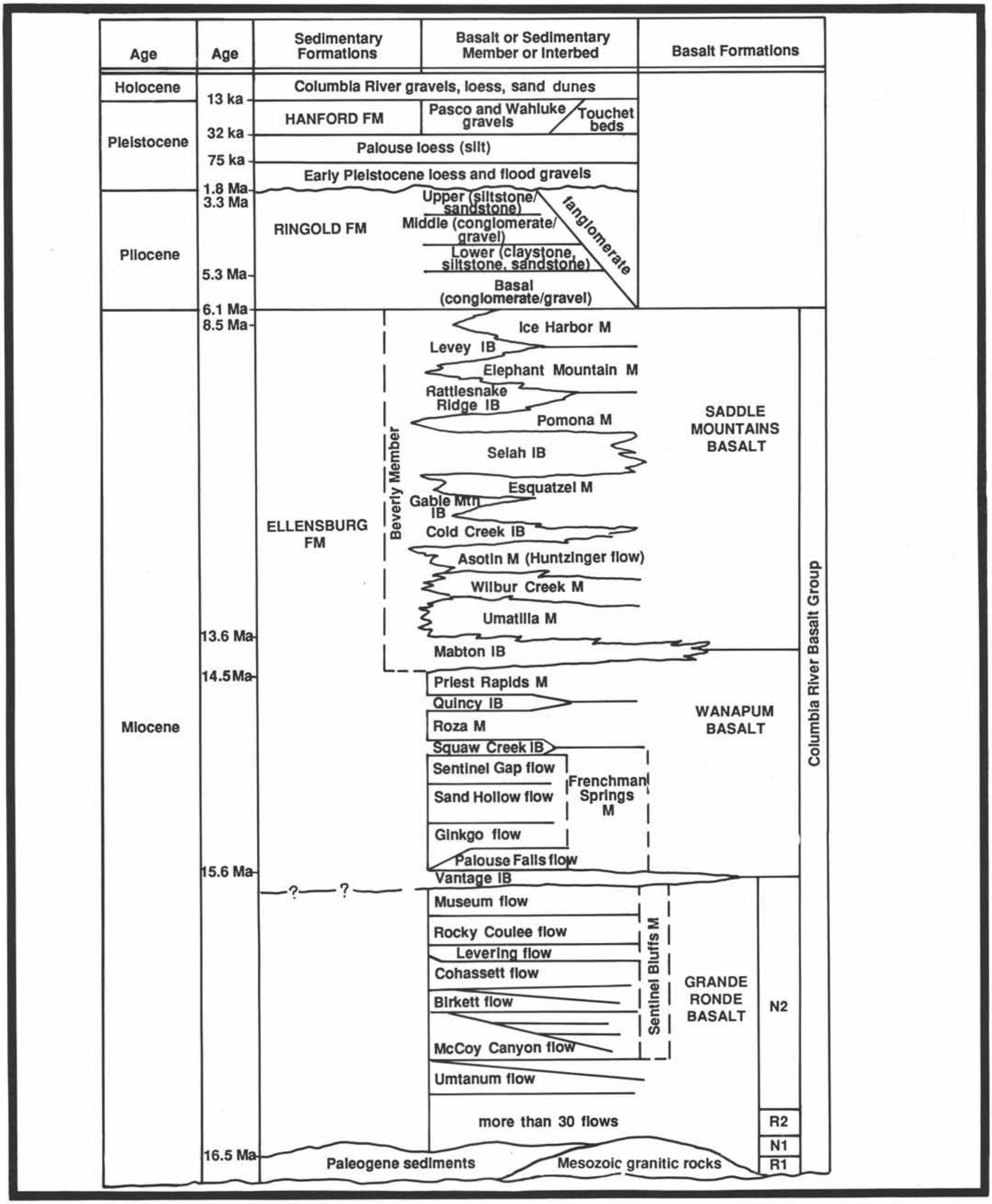


Figure 2. Stratigraphic column for the mid-Columbia River region. Modified from Mackin (1961), Reidel and Fecht (1981), and Swanson et al. (1979). M, member; FM, formation; IB, interbed; N, normal magnetic polarity; R, reversed magnetic polarity.

the mid-Columbia, is the relation with coeval sediments along the margins of the lava field. The problem is well illustrated at Rock Island where invasive relations between the lava flows and sediments are found.

Upstream, the basalt unconformably overlies both the Paleogene sedimentary rocks of the Chumstick and Wenatchee Formations (Gresens, 1983) and the granitic gneiss complex (Tabor et al., 1982).

A question under continuing study that has potential influence on the seismic stability of the region involves the style and timing of deformation of the basalts, especially the Yakima fold belt structures through which the Columbia and the Yakima rivers have maintained the spectacular water gaps on this middle segment. While there appears to be some evidence of deformation as early as late Grande Ronde time (Reidel, 1984), only local and sporadic deformation appears to have occurred until after Elephant Mountain time (ca. 10.5 Ma), culminating with the development of major faults and folds during latest Saddle Mountains to Ringold time (roughly 6-3 Ma) (Barrash et al., 1983). Post-basalt sediments of the Ellensburg and Ringold formations are clearly deformed along with the basalts.

REFERENCES

Barrash, W.; Bond, J.; and Venkatakrishnan, R., 1983, Structural evolution of the Columbia Plateau in Washington and Oregon: *American Journal of Science*, Vol. 283, pp. 897-935.

Gresens, R. L., 1983, *Geology of the Wenatchee and Monitor Quadrangles, Chelan and Douglas Counties, Washington*: Washington Division of Geology and Earth Resources Bulletin 75, Olympia, WA, 75 p., 3 plates.

Grolier, M. J. and Bingham, J. W., 1978, *Geology of Parts of Grant, Adams and Franklin Counties, East-Central Washington*: Washington Division of Geology and Earth Resources Bulletin 71, Olympia, WA, 91 p.

Mackin, J. H., 1961, *A Stratigraphic Section in the Yakima Basalt and the Ellensburg Formation in South-Central Washington*: Washington Division of Mines and Geology Report of Investigations 19, Olympia, WA, 45 p.

Reidel, S. P., 1984, The Saddle Mountains–The evolution of an anticline in the Yakima Fold Belt: *American Journal of Science*, Vol. 284, pp. 942-978.

Reidel, S. P. and Fecht, K. R., 1981, Wanapum and Saddle Mountains Basalts of the Cold Creek syncline area. In *Subsurface Geology of the Cold Creek Syncline*: Rockwell Hanford Operations RHO-BW1-ST-14, Richland, WA, pp. 3-1 to 3-45.

Swanson, D. A.; Wright, T. L.; Hooper, P. R.; and Bentley, R. D., 1979, *Revisions in Stratigraphic Nomenclature of the Columbia River Basalt Group*: U.S. Geological Survey Bulletin 1457-G, 59 p., 1 plate.

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.

Petroglyphs on basalt, Whale Island, Priest Rapids. These artifacts were removed for preservation prior to construction of Priest Rapids Dam. Photograph by R. W. Galster, January 25, 1955.

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

Priest Rapids Dam

RICHARD W. GALSTER
Consulting Engineering Geologist

PROJECT DESCRIPTION

Priest Rapids Dam (Figure 1) is situated near the lower end of Priest Rapids (river mile 397) where the Columbia River cuts close to Umtanum Ridge and was diverted eastward by the Umtanum uplift. Early development of hydropower at Priest Rapids began in 1907-1908 with construction of the Hanford Canal on the right bank from the head of the rapids to a small powerhouse at the foot of the rapids. This work was done by the Hanford Irrigation and Power Company (Rice, 1986). The site was investigated for a main stream dam in the early 1920s by Washington Irrigation and Development Co. and in the early 1930s and mid-1940s by the U.S. Army Corps of Engineers.

The dam consists of tandem straight-line concrete gravity spillway and intake-powerhouse and nonoverflow sections across the original broad, channeled valley floor. Flanking embankments cross higher river terraces on each side. The concrete sections rise to elevation 495.5 ft; embankment sections are 5 ft higher. Nominal reservoir level is at elevation 486.5 ft. The project includes fish ladder facilities on each bank at the junction between concrete and embankment sections.

The dam was engineered by Harza Engineering Company and constructed by Merritt-Chapman and Scott Corporation for the Public Utility District of Grant County between 1956 and 1960. The project is operated as a run-of-the-river hydroelectric project, with fishery enhancement and recreation as secondary purposes.

SITE GEOLOGY

The geology at Priest Rapids is dominated by the steep north face of the Umtanum Ridge anticline where the basalt flows are standing on end or slightly overturned. A steeply south-dipping thrust fault (Umtanum fault) is present at the toe of the ridge (Figure 2). This structural and topographic edifice lies immediately adjacent to the right abutment on the southwest and is in marked contrast to the geologic structure beneath the site proper. Columbia River terrace gravels overlie the bedrock surface on the right bank between the toe of Umtanum Ridge and the pre-dam river channel.

The rapids are developed on and underlain by relatively flat-lying basalt of the Priest Rapids Member of the Wanapum Basalt (Figure 3). This is the type locality for the member. Although the full section of the member is not exposed at Priest Rapids, it is seen to consist of four flows or flow units numbered in ascending order and having the following respective nominal thicknesses: 30 ft, 40 ft, 60 ft, and 90 ft (Mackin, 1961). The lower three flows (I-III) probably represent the Rosalia flow, and the upper flow (Priest Rapids IV) probably represents the Lolo flow of contemporary usage. The contact between flows III and IV is characterized by a thin weathered zone, consisting of as much as 6 in. of altered glass or several inches of ash. Development of the rapids removed much of the scoriaceous/vesicular top of flow IV (Lolo), and the surfaces of Panhandle and Whale islands, which dominated the valley floor, were developed on the eroded colonnade. The rock surface had been further modified by the cutting of a channel completely through flow IV near the west bank of the river, apparently during early Saddle Mountains time (ca. 13 Ma). The channel was filled with clay and compact gravel, probably belonging to the Beverly Member of the Ellensburg Formation. Some similar sediments were also exposed in the east channel. These sediments contained both partly petrified and unpetrified logs, as well as fresh-water mollusc shells (Mackin, 1961; Harza Engineering Co., 1966).

A second early Saddle Mountains channel lies adjacent to the left bank of the river. This channel was only slightly cut into the Priest Rapids basalt at the site proper, but downstream it lies within the Beverly sediments and is filled with the Huntzinger flow (Asotin Member, Saddle Mountains Basalt), well exposed on the left bank downstream from the dam. This rise in section is due to the gentle southeast plunge of gently undulating structures beneath much of the site.

The left bank is dominated by two terraces, a Columbia River flood gravel terrace rising as high as elevation 460 ft and the high Wahluke terrace, part of the extensive Wahluke Slope composed of catastrophic glacial flood gravels. The two terraces are separated by a northwest-trending linear depression known as Moran

Figure 1. Priest Rapids Dam. View to the north showing part of the right embankment, concrete spillway, intake dam/powerhouse section, and offset left embankment beyond. Fish ladder facilities flank the concrete dam. Photo courtesy of Grant County.

Slough, which contained perennial water in its floor (elevation 440 ft) before the dam was built. Moran Slough may be the surface expression of a fault. An early interpretation showed the feature as a graben trending with the topographic expression, which is about 1,000 ft wide and in excess of 4 mi long. Later drawings showed a single fault trending parallel to the Umtanum fault at an acute angle to the slough. Neither interpretation is well constrained by borehole data. Between the Umtanum and Moran fault zones, the structure (as determined by the position of the base of flow IV, Lolo) is a series of gentle south-plunging folds having structural relief on the order of 20 to 30 ft. The greatest of these is a syncline which passes directly under the powerhouse.

In addition to structural complications, the shape of the bedrock surface beneath the left bank is further complicated by its stratigraphic position across the Wanapum-Saddle Mountains contact. Structural drag on the west side of the Moran fault has preserved Beverly (Selah) sediments, which are also preserved downstream due to structural plunge. [These sediments were called "Selah Formation" during investigation and construction, and the overlying basalt flow(s) were called Wenas Basalt. Subsequent work by Reidel and Fecht (1981) has identified numerous flows and interbeds within the Sad-

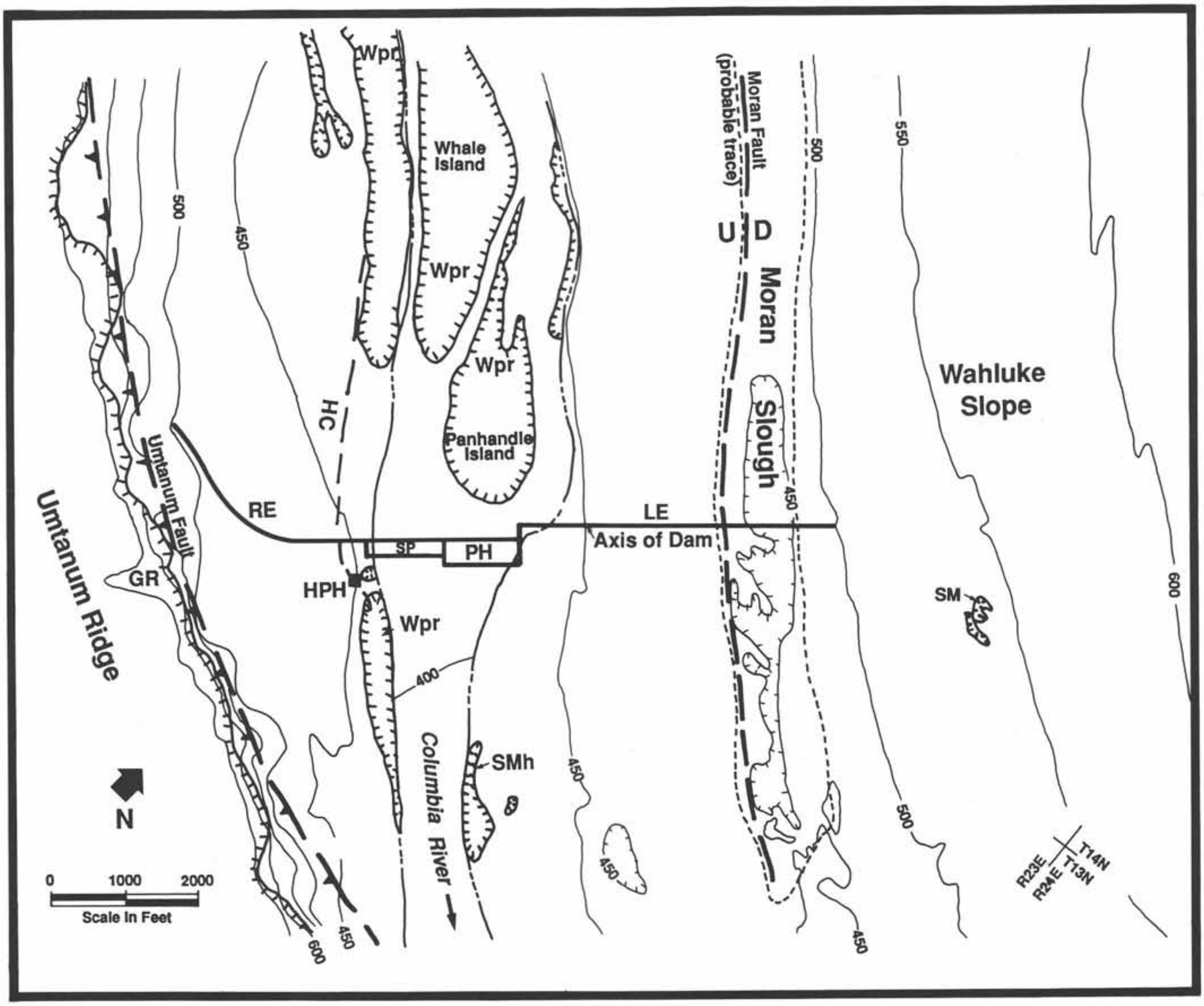


Figure 2. Generalized geologic map of the Priest Rapids Dam area. Heavy hachured lines indicate limits of rock outcrops. GR, Grande Ronde Basalt; SM, Saddle Mountains Basalt; SMh, Huntzinger flow of the Saddle Mountains Basalt; Wpr, Priest Rapids Member of the Wanapum Basalt. Other features: HC, Hanford Canal; the predam channel leading to HPH, the early Hanford Powerhouse; LE, left embankment; RE, right embankment; PH, powerhouse; SP, spillway.

dle Mountains Basalt/Ellensburg section, and the term Selah has been restricted to the sediments directly beneath the Pomona Member. However, where several Saddle Mountains flows are missing, it is better to use the more general term Beverly for sediments representing an otherwise unidentifiable interval.]

East of the Moran fault a younger basalt flow, probably Pomona, is preserved along with the underlying Beverly (Selah) sediments on the downthrown side of the fault, and locally a phaneritic to porphyritic basalt, possibly the Elephant Mountain Member, is preserved on the scabland surface underlying the Wahluke gravels.

SITING AND DESIGN

Basic siting of the dam was determined by the obvious availability of sound foundations for concrete structures. The original axis was to be about 1,200 ft upstream of the present site and across the lower end of Panhandle Island. However, by placing the structure downstream, less rock excavation was required for the powerhouse draft tubes and the volume of the left embankment reduced. The concrete spillway and intake/powerhouse sections essentially span the full width of the original river channel. In addition, it was important that the right end of the dam abut into basalt and

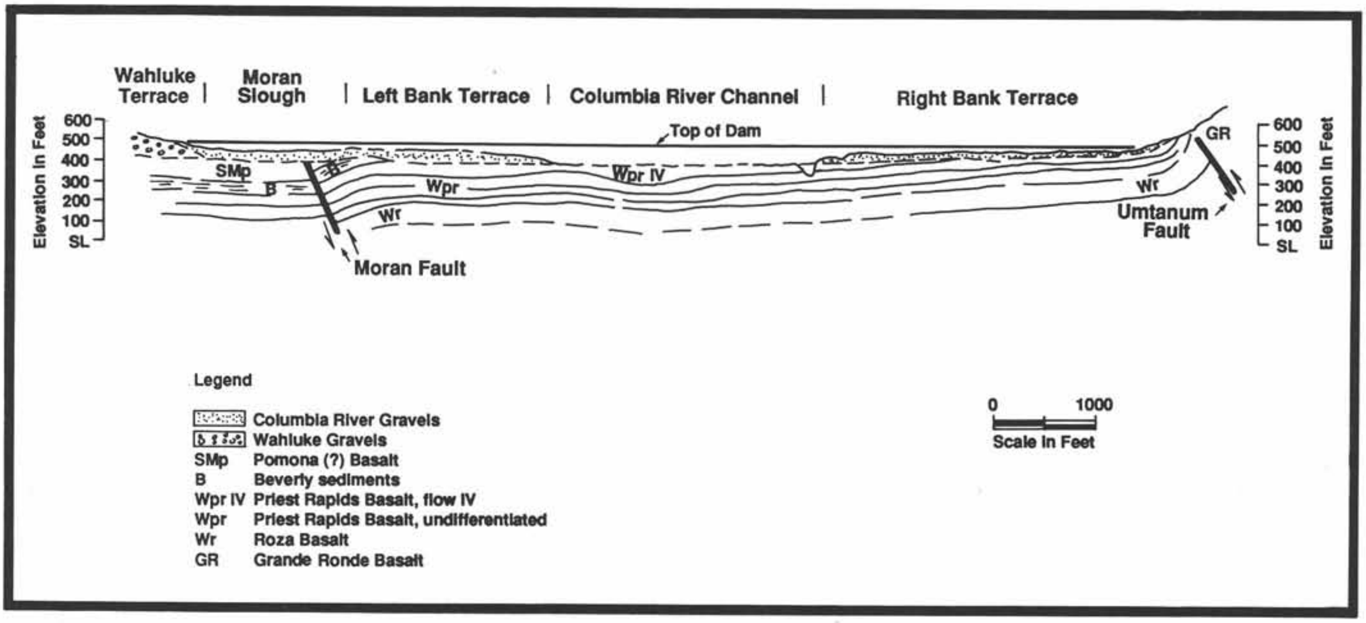


Figure 3. Geologic section at Priest Rapids Dam, view downstream.

not a sedimentary (Beverly/Selah) wedge on the footwall of the Umtanum fault; the latter was possible, due to the undulating nature of the lower plate. The right embankment was curved upstream to take advantage of the highest possible rock surface. Both embankments were designed to have cut-off trenches to bedrock, and the left abutment in the highly pervious Wahluke gravels was to have special treatment to control leakage. The offset between the left end of the powerhouse and the left embankment is to provide easier construction in event of a future powerhouse extension.

CONSTRUCTION

Stages

The project was constructed in four stages. The first stage included construction of the powerhouse and left embankment, specifically the part behind left bank cofferdams. This stage began in September 1956 and continued throughout the entire construction period. Stage 2 was partial construction of the east (left) half of the spillway in 1957-1958. Stage 3 was the full construction of the west (right) half of the spillway behind right bank cofferdams and construction of the right embankment while the river was diverted through the low blocks of the east half of the spillway in 1958-1959. Stage 4 was the completion of the left half of the spillway in late 1959 and in 1960 (Harza Engineering Co., 1966).

Foundation Treatment

Grouting

An upstream grout curtain was constructed from the rock surface prior to any rock excavation for the concrete dam. The curtain extends for a distance of 1,500 ft beneath the core trench of the right embankment and for 1,800 ft beneath the core trench of the left embankment to about the point where the core trench rests on Beverly (Selah) sediments. The main target of the grout curtain was the contact between Priest Rapids flows III and IV, generally 50 to 60 ft below the rock surface beneath the concrete sections (Harza Engineering Co., 1966). Greatest grout takes were in the area of the left end of the powerhouse and left fishway north to the axis of the left embankment. Additional curtain grouting was done from a gallery in the spillway and intake sections after these sections were largely complete. Beneath the concrete structures some shallow area grouting of open joints was required.

Excavation

The concrete dam and powerhouse are founded on the colonnade of Priest Rapids flow IV (Lolo). The rock surface beneath the eastern half of the spillway was excavated to about elevation 380 ft, beginning with a gently undulating rock surface between elevations 390 and 400 ft. Locally, joints in the rock were mud-filled, requiring thorough cleaning. Beneath the western part of the spillway the rock surface was essentially channeled, culminating in a channel 50 ft deep and 90 ft wide filled by clay, sandstone, gravel, and conglomerate of Beverly (Selah) vintage. Although these materials were relatively tight, they were removed and the channel backfilled with concrete. The depth of excavation for the powerhouse was highly varied, extending to elevation 316 ft for the draft tubes. Considerable seepage into the excavation from the downstream side required control by pumping from french drains and riser pipes early during concrete placement. The drain and riser system was later grouted. Other than removal of loose material and cleaning, no special foundation preparation was required.

Embankments

Both embankments are founded on terrace gravel with core trenches extending to the bedrock surface. The right core trench extends to the irregular, eroded top of Priest Rapids flow IV (Lolo). The western half of the left embankment core trench also extends to the eroded surface of flow IV. East of this point, the core trench bottoms on sediments of the Beverly (Selah) Member for a distance of about 850 ft, then, crossing the Moran fault, rests on dense Pomona (?) Member basalt (termed Wenas during construction) until the left abutment contact with the Wahluke gravels is reached. The original design called for removal of the Beverly (Selah) sediments from beneath the future core wall, but the material was found to be a very dense clay, as impervious as the core, and it was left in place as a foundation for the core. Locally, pockets of gravel were removed. The Pomona (?) was also found to be tight and dry, and no leakage was experienced from it. However, because much of the left core trench floor was below the water table, heavy leakage into the excavation was experienced through the terrace gravels. Locally, leakage was also experienced from joints in the bedrock surface. This required considerable drainage and pumping. Gravel drains and risers from the rock surface were used; these were grouted as the embankment was raised. Because the rock surface beneath the right embankment core trench was above the water table, no drainage was required.

Left Abutment

Because the left (east) embankment abuts the highly pervious Wahluke gravels, a cut-off trench and blanket were designed and constructed extending both 1,000 ft upstream from the abutment along the face of the Wahluke terrace and down to the bedrock surface. The trench and blanket were designed to lengthen the leakage path around the abutment and thereby control leakage. The exit area for this water downstream from the abutment is controlled by a 5-ft-thick graded gravel blanket.

Construction Materials

Aggregate for concrete came mostly from the Columbia River gravels comprising the left bank terrace. A low-alkali type-II cement was required for use with these aggregates. Pozzolan was obtained by beneficiation of ash from the Beverly sediments 5 mi upstream on the west bank. Impervious material was obtained from re-worked Beverly sediments on the right bank just upstream from the dam. The low natural moisture of the material in place required addition of water to obtain the 14 to 15 percent optimum moisture. All other embankment materials, including rock riprap, came from project excavations or borrow immediately adjacent to the site.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Movement of water around the left (east) abutment was anticipated in the design and is appropriately controlled. The head loss through the impervious blanket is about 20 ft, and the general gradient of the phreatic surface around the abutment is 0.025 (CH2M Hill, 1974). Seepage has been clear and of small quantity and has had no change in character since initial filling of the reservoir (Harza Engineering Co., 1966). The leakage keeps Moran Slough downstream from the dam constantly wet.

ACKNOWLEDGMENTS

Although my relationship with the Priest Rapids project dates from early investigations in the mid-1950s, I am indebted to S. R. Brown and his staff of the Public Utility District of Grant County for their assistance in gathering construction and design data; some of these data were collected during mid-Columbia River shiplift studies with Rittenhouse Zeman and Associates under contract to the Seattle District U.S. Army Corps of Engineers. However, I am most indebted to the late J. H. Mackin, who was directly responsible for my being involved with the project during its formative stage and my early involvement with the geology of the mid-Columbia region.

REFERENCES

CH2M Hill, 1974, *Priest Rapids Development–Stability Study of the Embankments and Reservoir Shoreline*: Grant County Public Utility District, Ephrata, WA, 118 p.

Harza Engineering Company, 1966, *Priest Rapids Development–Final Report*: Harza Engineering Company, Chicago, IL, and Grant County Public Utility District No. 2, Ephrata, WA, 72 p., 23 plates.

Mackin, J. H., 1961, *A Stratigraphic Section in the Yakima Basalt and the Ellensburg Formation in South-Central Washington*: Washington Division of Mines and Geology Report of Investigations 19, Olympia, WA, 45 p., 9 plates.

Rice, D. G., 1986, Personal communication, U.S. Army Corps of Engineers, Seattle, WA.

J. Hoover Mackin, Professor of Geology, University of Washington; consultant to Harza Engineering Company, engineer for the Priest Rapids Dam. Photograph taken by R. W. Galster, December 16, 1954, during a lunch break on Whale Island at the Priest Rapids dam site.

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

Wanapum Dam

RICHARD W. GALSTER
Consulting Engineering Geologist

PROJECT DESCRIPTION

Wanapum Dam (Figure 1) is located between Columbia River miles 415 and 416, 5 mi downstream from the crossing of Interstate Highway 90 at Vantage. The dam has a "dog-leg" plan and consists of a concrete channel-parallel powerhouse/intake section (including space for six additional units) and a concrete spillway section. Flanking zoned embankments on each end cross older river terraces and the main river channel. The concrete sections rise to elevation 581 ft, and the embankment crests reach elevation 583 ft. Normal reservoir is elevation 570 ft. The project includes fish ladder facilities on each bank between the concrete and embankment sections of the dam.

The dam was engineered by Harza Engineering Company and constructed between 1959 and 1963 by Grant County Contractors (mainly Morrison Knudson Co.) for the Public Utility District of Grant County. The project is operated as a run-of-the-river hydroelectric project with fishery enhancement and recreation as secondary purposes.

SITE GEOLOGY

The 6,500-ft-wide valley floor at Wanapum Dam is underlain by thick sand and gravel alluvium, deposited partly by the Columbia River and partly by catastrophic glacial outburst floods during the late Pleistocene deglaciation to the north and northeast. The alluvium includes numerous boulders, ice-rafted blocks, and open-work gravels, as well as zones that are more silty and sandy. The adjacent valley walls expose the Roza and upper Frenchman Springs (Sentinel Gap flow) members of the Wanapum Basalt (Figures 2 and 3). The highly scoured bedrock surface, varying in depth from 50 to 150 ft beneath the valley floor, is formed on the sequence of basalts, palagonite tuff, and sediments which characterize the Wanapum-Grande Ronde Basalt boundary: the lower Frenchman Springs Member, Vantage interbed, and Museum and Rocky Coulee flows. This classic stratigraphy together with the presence of the southeast-plunging Ryegrass Mountain anticline and fault (Harza Engineering Co., 1955), which passes directly beneath the site, combined with the valley's history of glacial outburst flooding, have resulted in one of the most complex geological settings of any dam on the Columbia. The combination is also the reason for the complex layout of the various structures of the dam.

Much of the bedrock surface beneath the valley gravels on the left (east) bank of the river was developed on the Sand Hollow and Sentinel Gap basalt flows of the Frenchman Springs Member. Similarly, on the far right (west) bank the basalt facies of the Ginkgo flow forms much of the bedrock surface. Within the mid-valley third, however, the Ryegrass anticline and a high-angle reverse fault on its eastern flank bring the flows of the upper Grande Ronde (Museum and Rocky Coulee) to a structurally and topographically high position. The subcrop of the more easily erodible palagonite breccia of the lower part of the Ginkgo flow and the underlying Vantage interbed were channeled by the action of glacial flood waters (Figures 2 and 3). The topographically high, sound, bedrock surface developed on the colonnade of the Museum and the Rocky Coulee flows serves as the foundation for the concrete structures of Wanapum Dam: the spillway and powerhouse. The channel-parallel powerhouse location was required to conform with this sound bedrock configuration. The fault associated with the east flank of the Ryegrass anticline passes beneath the left embankment, essentially under the pre-dam river channel. Erosional channels along the fault zone and along the Vantage interbed also pass beneath the left embankment. A similar channel, developed in the Vantage-Ginkgo palagonite zone about 150 ft beneath the valley floor, passes beneath the right embankment. These channels in the bedrock surface converge downstream because of the plunge of the Ryegrass structure (Harza Engineering Co., 1955).

GEOLOGIC ASPECTS OF SITING AND DESIGN

An understanding of the erosional history of the mid-Columbia River valley was important in the siting of Wanapum Dam. With spectacular water gaps through the Frenchman Hills at Vantage and the Saddle Mountains at Sentinel Gap, selection of one of these two would have been expected to be considered, had the valley developed by normal river erosion. However, the series of Pleistocene catastrophic glacial meltwater floods concentrated their erosive power on the gaps so

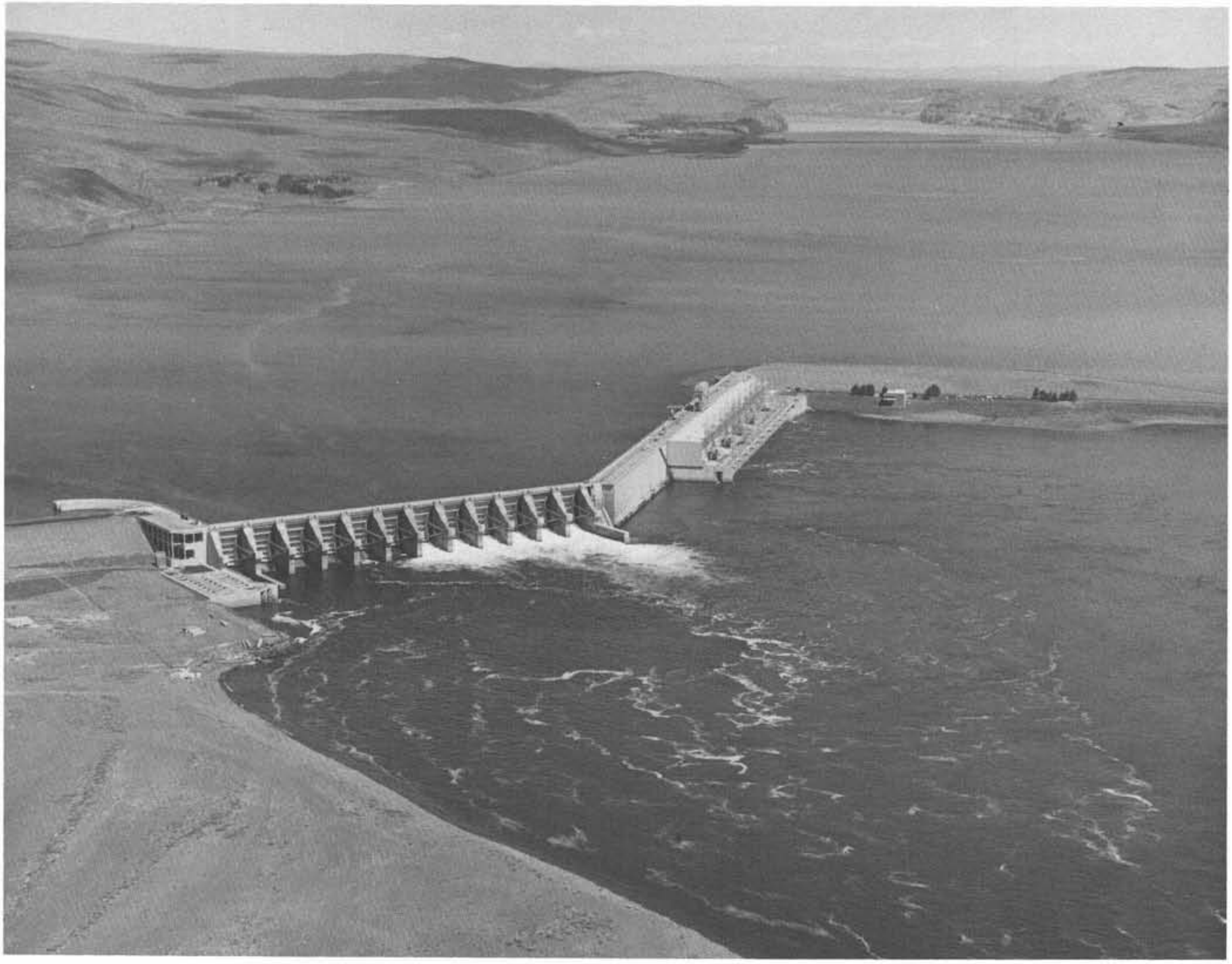


Figure 1. Wanapum Dam, View upstream (north) showing dog-leg arrangement of spillway and powerhouse and space for future units between. The left embankment is to the right. Fish passage facilities and a small part of the right embankment are to the left of the spillway. The Interstate Highway 90 bridge is in the background, at Vantage. Beyond the bridge is the canyon through the Frenchman Hills anticline. Photo courtesy of the Grant County Public Utility District

that the depth to bedrock beneath the valley floor at these locations is in excess of 250 ft (Harza Engineering Co., 1955). This made the founding of concrete structures uneconomical.

Mapping of the valley walls between the gaps revealed the presence of the Ryegrass structure and the likelihood of the resistant upper flows of the Grande Ronde Basalt being at an elevation sufficient to provide economical foundations. Location of the dam was ultimately based on an extensive seismic refraction survey which guided the early drilling program. The initial geologic interpretations proved to be true. The high, sound basalt units of the Museum and Rocky Coulee flows were available for founding the concrete structures, although the configuration of the rock surface dictated the orientation of the powerhouse parallel to the channel. The bedrock configuration also required the angled spillway and longer embankment sections flanking the concrete structures than would have been necessary had a straight-line axis been economical.

The most unusual part of the design was the use of a backfilled slurry wall as a permanent cut-off beneath the dam embankments, which rest on valley gravels. Preliminary design provided only a partial cut-off wall under the embankments; the cut-off consists of a core extending 40 ft below the valley floor. The cut-off was to extend an additional 20 ft with steel sheet piles (Harza Engineering Co., 1967a). Permeability tests showed the impracticality of dewatering below the water table and of driving steel sheet piles to any great

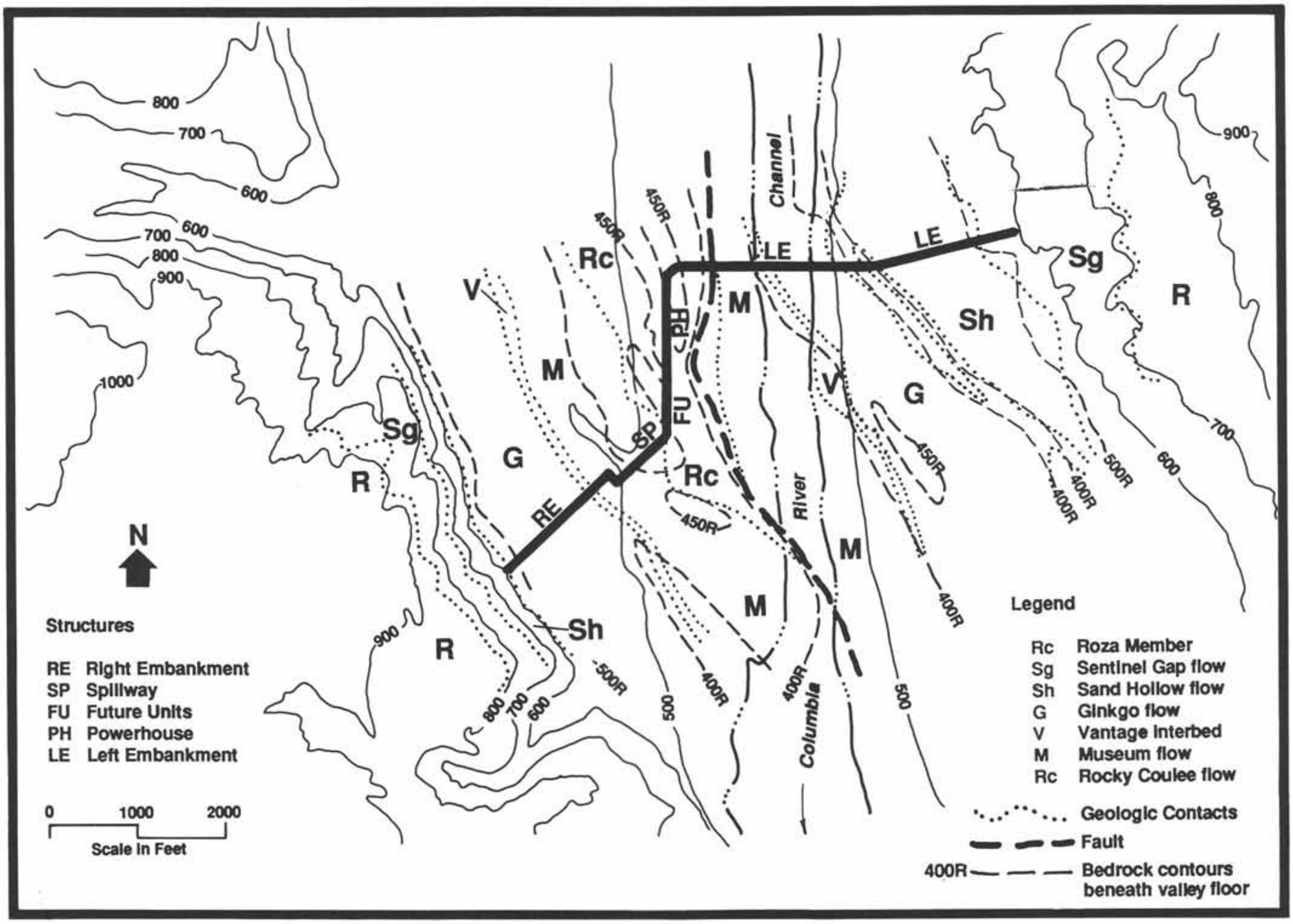


Figure 2. Generalized geologic plan at Wanapum Dam. Geology modified from Harza Engineering Co., unpublished drawing FPC No. 2114-106, February 1959.

depth because of the high permeabilities and large clast size of the flood gravels. Permeabilities were found to range from about 820 ft/day (0.3 cm/sec) to 8,200 ft/day (3 cm/sec). The design was therefore changed to install a permanent backfilled slurry cut-off wall to bedrock beneath the core. As the practicality of installing such a slurry trench to depths greater than 100 ft was open to question, two limited areas beneath the left embankment, where the bedrock surface was in excess of 80 feet below the water table, were to be grouted between the bottom of the slurry wall and the rock surface.

CONSTRUCTION

Stages

Construction of the dam began in July 1959 with emplacement of cut-off trenches under the right and left embankments, except for the river section. Upstream and downstream cofferdams, designed with backfilled slurry trenches similar to those for the dam embankments, were concurrently constructed and tied to embankment cut-off sections, permitting excavation for mid-valley concrete structures to begin in October 1959. Construction of dam embankments "in the dry" across the river terraces then proceeded along with construction of concrete structures behind the cofferdams until September 1962. At that time the river was diverted from its natural channel through future unit intake openings by concurrent removal of cofferdams and construction of cofferdams across the river channel. This permitted completion of the left embankment prior to high water in 1963. The reservoir was raised in 5 days in May 1963.

Concrete Structures

The gently undulating and scoured bedrock surface in the mid-valley spillway and intake dam-powerhouse area was at about elevation 450 ft, dropping off to about 430 ft at the right gravity dam-fish facility and again dropping rapidly into a scoured fault zone east of the powerhouse. The contact between the Rocky Coulee flow and the overlying Museum flow passes through the

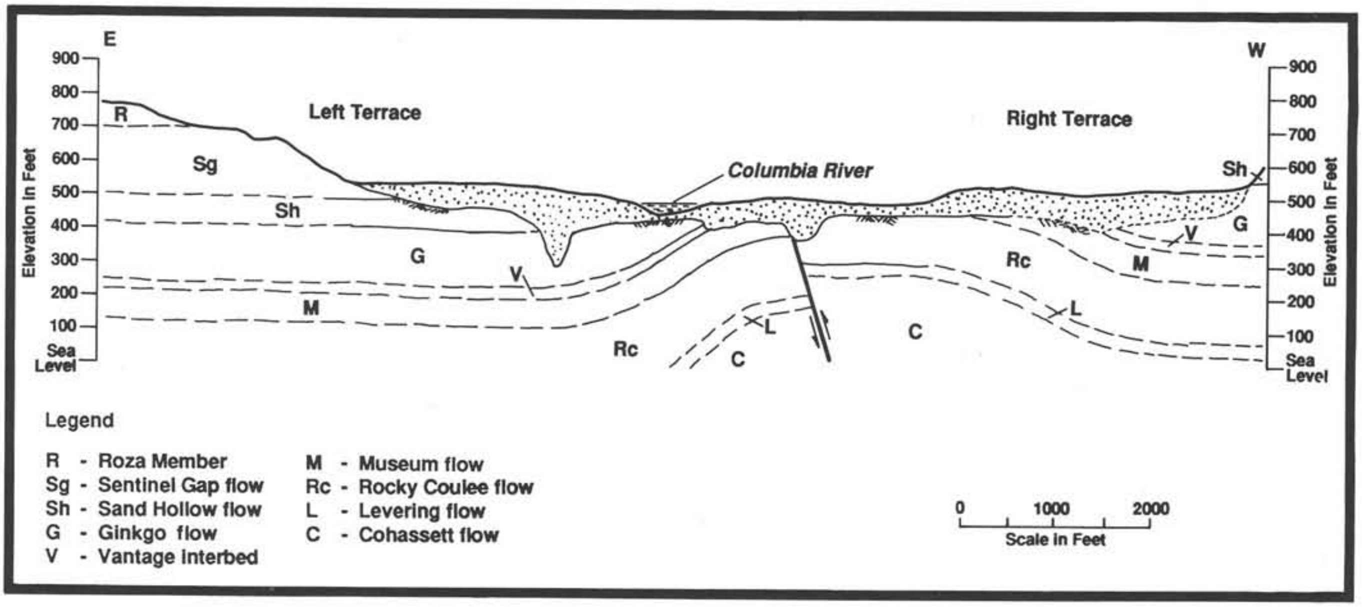


Figure 3. Generalized geologic section across the Columbia River valley at Wanapum Dam showing bedrock stratigraphy and the extent of the Ryegrass Mountain fault and anticline. After Harza Engineering Co. (1955).

southern (downstream) three future powerhouse units (Figure 2). The contact, dipping gently westward, was characterized by volcanic cinders and fractured basalt, requiring about 10 additional feet of rock excavation before a suitable foundation could be found. The intake dam is founded at about elevation 435 to 440 ft. The deepest part of the powerhouse excavation (draft tubes) required a little more than 50 ft of rock excavation (to elevation 394 ft). The erection bay sump required about 75 ft of rock excavation.

Most of the spillway is founded on the flood-scoured bedrock surface. Only minor foundation excavation and cleanup were needed. However, the right gravity dam and fish facilities required about 30 ft of rock excavation to accommodate the structures. The spillway apron required installation of shallow steel rock anchors, an activity somewhat hampered by the presence of artesian water in many of the anchor holes, probably related to the Museum-Rocky Coulee flow contact. In the future units intake dam, post-tensioned cable anchors were installed in 16-in.-diameter holes 70 to 75 ft deep, well into the Rocky Coulee flow, to provide structural stability in the absence of a completed powerhouse.

A grout curtain was installed near the upstream edge of concrete structures. The curtain is inclined 1H to 5V upstream and varies from 28 to 50 ft deep beneath the spillway and 61 to 72 ft deep beneath the completed section of the intake dam/powerhouse. The curtain was extended to elevation 360 ft (90 ft depth) beneath the future units intake dam, the full depth of cable tendons, to reduce or eliminate water in tendon anchor borings. Additional foundation grouting was also done for this purpose. Most of the grouting in the powerhouse area was accomplished prior to rock excavation, whereas in the spillway area grouting was accomplished from the drainage and grouting gallery in the dam. Grout pressures varied from 70 to 150 psi, depending on depth of section being grouted.

A vertical drainage curtain extending a little more than two-thirds the depth of the grout curtain was installed just downstream from the grout curtain and exits into the drainage and grouting gallery. Considerable leakage into the northern end of the powerhouse excavation, possibly due to the proximity of the excavation to the fault zone, required installation of french drains, which were later grouted upon completion of concrete work.

As the embankment cut-off trenches at each end of the concrete section were approached, stepped, steel sheet piling walls were required to provide stability to the fluid backfill of cut-off trench.

Embankments

Cut-off Trenches

Cut-off trenches for both embankments were constructed from a working berm varying in elevation from 490 to 495 ft. The berm was higher toward the left abutment. This elevation required as much as 30 ft of excavation in the valley gravels but kept the berm above the nominal water table controlled by the Columbia River. Construction slopes in gravel were 1.35 H to 1 V. The slurry trench was constructed 12 ft wide because of the size of the bucket employed. The viscosity and density of the slurry were closely controlled, especially near backfilling time, to prevent segregation of backfill. A concrete cap was placed at the bottom of the cut-off trench (in most places, top of bedrock). Cationic exchange between the lime in the concrete and the sodium in the bentonite-sand slurry caused some flocculation of the slurry, although much of the contaminated slurry was later incorporated in the silty, gravelly sand backfill (Harza Engineering Co., 1967b).

The depth of the 1,740 lineal feet of cut-off trench beneath the right embankment ranged from 35 to 74 ft (Figure 4). The west end of the trench abutted against a steep rock slope very near the right abutment.

The cut-off beneath the left (east) embankment required construction in three stages: sections west of the river, east of the river, and, later, the river section. A short distance west of the river channel (now the powerhouse tailrace) a depression in the bedrock surface to about elevation 385 ft, created in the less resistant Ginkgo palagonite breccia and the Vantage interbed, was far beyond the 80-ft contract depth for a slurry trench cut-off (Figure 4). The trench was constructed to about elevation 415 ft and the gravels beneath were grouted using three rows of holes on 6-ft centers.

East of the river channel, a nearly 500-ft-wide channel had been scoured by glacial flood waters to elevation 320 ft. A zone of cemented gravel across this channel, ranging in elevation from 465 ft on the east to 410 ft on the west, prevented the drag bucket from penetrating farther. Thus, the underlying gravels were grouted the entire distance to the bedrock surface in a manner similar to the grouting on the west bank (Harza Engineering Co., 1967b). The cut-off trench beneath the left embankment terminates about 600 ft west of the present state highway where the rock surface rises above elevation 500 ft.

Foundation and Abutments

The embankments rest on the valley gravels. Removal of 10 to 15 ft of overbank and side channel silty material from the western portion of the right embankment foundation area was required. Otherwise, only grubbing and removal of some eolian sand was necessary. The right abutment is against a steep rock

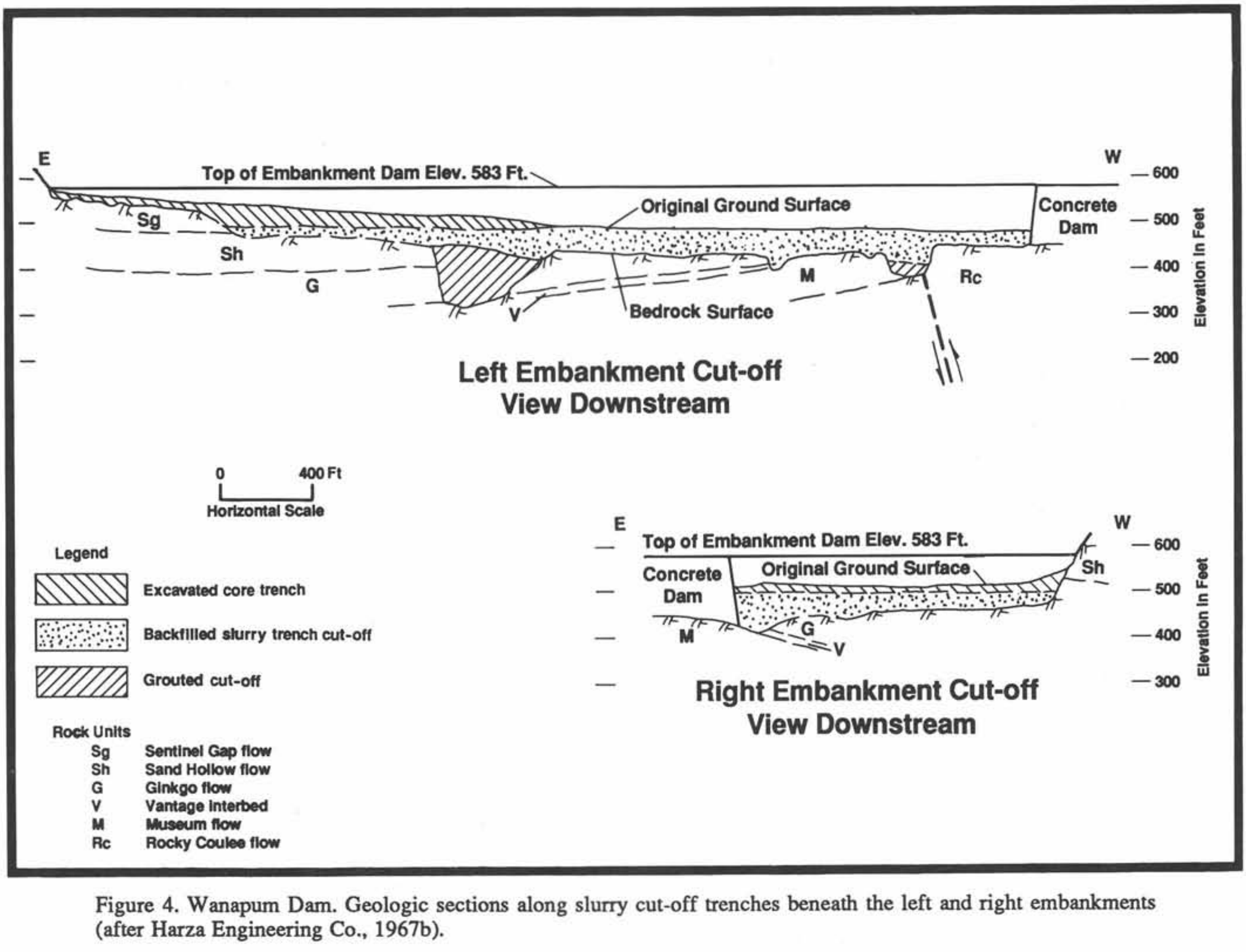


Figure 4. Wanapum Dam. Geologic sections along slurry cut-off trenches beneath the left and right embankments (after Harza Engineering Co., 1967b).

slope in the Ginkgo and Sand Hollow flows, and the contact between the two units is at about valley floor elevation. The left abutment is on a gentle slope in the Sentinel Gap and Sand Hollow flows, and the contact between the units is at about elevation 490 ft. As the abutment was approached, pockets of sand, silt, and ash were encountered and removed.

Construction Materials

Except for bentonite and cement, all construction material came from the project area, much of it from required excavation. Pervious and "impervious" materials were available on both sides of the valley, although "impervious" material from the left embankment came from a borrow area in eolian sandy silt about 1 mi east. Pervious fill required no processing, although processing was required for transition zones and concrete aggregate. Rock quarries were opened on each side of the valley to provide rock shell and riprap material. The left bank quarry is in the colonnade of the Sentinel Gap flow, and the right bank quarry is in the colonnade of the Roza flow. Derrick stone was difficult to obtain, and ultimately much of that was obtained from a waste area near Sentinel Gap, 5 mi downstream.

OPERATIONAL PROBLEMS RELATED TO GEOLOGY

Wanapum Dam has been free of geologically related operational problems during more than a quarter century of its operation. There have been minor erosion of the spillway apron and training walls and minor changes in river bottom elevations 200 to 300 ft downstream from the spillway apron (CH2M Hill, 1975). The grouting and drainage system appears to have been effective without significant change in drain flows. The cut-off trench beneath the embankments has performed well, and its tie to the well designed and well constructed embankment has stood the test of time. The reservoir periphery is not subject to major landslide or erosion problems, although local slumps and some minor wave modification of the shoreline can be seen.

ACKNOWLEDGMENTS

Although my relationship with the Wanapum Project dates from early investigations in the mid-1950s, I am indebted to S. R. Brown and his staff of the Public Utility District of Grant County for their assistance in gathering construction and design data. Some of the data were collected during mid-Columbia River shiplift studies with Rittenhouse Zeman and Associates under contract to the Seattle District, U.S. Army Corps of Engineers. However, I am mostly indebted to the late J. H. Mackin, who was directly responsible for my being involved with both the project during its formative stage and the geology of the mid-Columbia region.

REFERENCES

CH2M Hill, 1975, *Wanapum Development, Fourth FPC Safety Report*: Grant County Public Utility District Ephrata, WA, 71 p., 2 appendices.

Harza Engineering Co., 1955, *Priest Rapids Hydroelectric Project, Columbia River, WA, Volume III-A*: Harza Engineering Co., Chicago, IL, and Grant County Public Utility District, Ephrata, WA, 43 p., 17 exhibits.

Harza Engineering Co., [1967a], *Priest Rapids Project, Wanapum Hydroelectric Development, Final Report for Public Utility District of Grant County, Volume I*: Harza Engineering Co., Chicago, IL, and Grant County Public Utility District, Ephrata, WA, 143 p., 4 plates.

Harza Engineering Co., [1967b], *Priest Rapids Project, Wanapum Hydroelectric Development, Final Report for Public Utility District of Grant County, Volume II*: Harza Engineering Co., Chicago, IL, and Grant County Public Utility District, Ephrata, WA, 194 p., 172 figs. and plates.