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Collapsing and Expansive Soils

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INTRODUCTION

Four types of soil in Washington are subject to large-scale settlement or expansion when wetted or partially dried. Of the soils subject to collapse or rapid consolidation when saturated, the most widespread is the loessial silt common throughout most of eastern Washington. It is called the Palouse silt or Palouse Formation and is of Pleistocene age, although extensive portions of these loessial soils are somewhat younger. The second relatively common settlement-prone soil is peat. Peat is present in almost all parts of the state; however, its lateral extent at any one location typically is relatively limited. Peat is subject to large-scale settlement under loading, drying, and, theoretically, oxidation. The third soil type subject to collapse when moistened is a unique group of silty and clayey soils that are subject to hydrocompaction. These materials are limited to the drier eastern portions of the state; they have been reported in the Ephrata and Wenatchee areas only. Finally, in isolated areas near Seattle, expansive clays, derived from the deep weathering of clay shales, are present. These soils have not been reported in other parts of the state; however, they may be present in areas underlain by clay shales.

LOESS

Wind-deposited or eolian silt, termed loess, blankets extensive parts of eastern Washington. This loessial silt is primarily rock flour derived from Pleistocene glaciers which invaded the northern parts of the state. The bulk of the loess deposits are located in the Palouse area around Colfax and Pullman, Washington, and Moscow, Idaho, although extensive exposures are reported from north of Spokane to south of Walla Walla and west to near Yakima. A major portion of these eolian deposits belongs to the Palouse Formation. In some areas, a younger loess overlies the Palouse sediments and later deposits. This younger loess apparently was derived from erosion of the Palouse silts and overlying Touchet beds, from younger rocks in part, or by deposition of rock flour created by late Pleistocene glaciation (Newcomb, 1961).

Typical Washington loess consists of tan to brown silt with some clay and fine sand. According to Browzin (1985), the material in Washington may be classified as a clayey loess containing about 32 percent fine fraction and 64 percent loessial fraction (0.01-0.05 mm). Browzin reports that the Palouse Formation is more than 100 ft thick in the Walla Walla area. The formation is described there as having a mealy cohesion and displaying low to low-medium plasticity when wet.

During test drilling, standard penetration test values ("N") on the order of 20 to 30 blows per foot of penetration are typical. Representative moisture content values of 15 to 30 percent have been measured at the Washington State University (WSU) campus at Pullman. Atterberg limits typically might include: Plastic Limit, 20 to 25; Liquid Limit, 35 to 45; and Plasticity Index, 10 to 20.

A wide variety of foundation treatments has been used for structural support in loess. Where the soil has been wetted to some degree or other, such as on much of the WSU campus, some preconsolidation of the loess has generally occurred. At the university, foundation systems used for building support include conventional footings, drilled piers, Augercast piling, driven "H" piling and displacement piles.

PEAT

According to Rigg (1958), peat deposits are present in 28 of the 39 counties in Washington, with the majority in the Puget Sound basin. Individual deposits range in size from less than 1 acre to more than 3,500 acres. Thicknesses of as much as 63 ft have been recorded, with minimum thickness being simply a highly organic, saturated topsoil horizon.

Peat may be fibrous or contain raw to partly disintegrated sedges and mosses. Also present may be Sphagnum peat, sedimentary peat, woody peat, and, to a lesser extent, salt-marsh peat near the coast. According to Rigg (1958), about half of the peat bogs contain one or more layers of volcanic ash. Radiocarbon dating (Rigg, 1958) of peat from several lakes and bogs reveals that, in western Washington, peat has apparently been accumulating at a rate of about 1 in. every 41 yr since the retreat of the last ice sheets about 10,000 to 14,000 yr ago.

Marachi et al. (1983) report peat moisture contents ranging from 100 percent to 500 percent. This range appears to be similar to values measured on samples obtained recently by the author in King, Pierce, and Snohomish counties. The dry density of the peat may range from 10 to 40 pcf, with those displaying the lowest dry densities having the highest moisture content.

Regardless of the measured physical properties of the material, any increase in loading on peat results in settlement of the surface. Generally, where the depth of peat exceeds the depth of fill which may be placed atop it, settlements of 6 to 10 in. per foot of fill depth should be anticipated. Typically, roughly half the settlement occurs over a 6-month to 2-yr period, with the balance taking place over about 20 yr. However, these settlements are not uniform, and any structures constructed atop such a fill will be subject to differential movements.

A bearing capacity failure may occur where more than a limited depth of fill or a heavy load is installed atop peat. In such instances, the peat is displaced laterally, creating "mud waves" to the sides of the embankment or structure being constructed. The fill or object then typically sinks vertically through the peat, possibly until it rests on the basal material beneath the bog. Filling over peat in this manner generally traps some of the organic material beneath or within the fill mass, permitting long-term differential settlements to occur. During the late 1930s in Germany, attempts were made to improve the effectiveness of this approach by detonating explosive charges beneath road embankments crossing peat bogs in an attempt to liquify the soft, compressible material and accelerate the settlement process. The intentional bearing capacity failure mode of construction has had only limited application or success in this country.

An alternate method of limiting post-construction settlement involves the construction of light-weight fill on top of the peat, this fill consisting of wooden building debris, hog fuel, or wood chips. Such a fill, being primarily wood or wood derived, would tend to float. When building debris is used, the fragments theoretically would tend to lock or knit together into a structure capable of spanning local soft areas. This type of fill normally is capped by crushed rock or pit-run gravel. Construction of streets and roads in this manner has had both successes and failures. Recently, some environmentalists have objected to the construction of fills of this type, contending that they could generate a leachate. However, leachate derived from wood debris or chips probably would not differ materially from the water in the underlying peat soils, which is in contact with similar organic matter.

HYDROCOMPACTION

Hydrocompaction occurs when dry, underconsolidated silty and clayey soils, in an arid or semiarid environment, lose strength on wetting and, as a result, settle or collapse. Ground subject to hydrocompaction commonly develops soil cracks and sinkholes, similar to those in karst topography. Surface structures may be warped or shifted out of shape, with the most severe distress typically affecting dams, canals, irrigation facilities, and wells. Soils subject to hydrocompaction apparently are not present in western Washington, owing to the moderate to high rainfall; they are relatively rare in the eastern part of the state, being reported only in Ephrata and Wenatchee.

It appears, in Washington, that soils susceptible to hydrocompaction were deposited in interfan areas, possibly by Pleistocene mudflows. Prokopovich (1984) inferred that, in California, these soils could have been frozen prior to drying, with the freezing conditions "puffing" the saturated silty and clayey deposits. He then proposes that, rather than thawing, the California deposits were "freeze-dried", a process induced in part by relatively dry westerly winds. Similarly, in Washington, cold, dry katabatic winds, possibly off Pleistocene glaciers, could have desiccated such mudflow deposits.

Although soils susceptible to hydrocompaction have been reported in Ephrata and Wenatchee in eastern Washington, they also may be present near Yakima and Ellensburg and in other localities wherever silty and clayey strata may have been subject to rapid erosion and deposition downstream under flash flood conditions. However, the area of deposition of these types of soils of necessity would be situated above the elevation of the ground-water table and be arid or semiarid, and the site, subsequently, could not have been subjected to flooding, irrigation or other prolonged wetting.

Washington soils subject to hydrocompaction have moisture contents ranging from 20 percent to 30 percent and dry densities of 75 to 85 pcf. Standard penetration test samples have "N" values on the order of 2 to 4. These values may not be totally representative, however, because one site explored settled owing to a ruptured water main and the other was subjected to intermittent irrigation activity.

EXPANSIVE CLAY SOILS

Expansive soils in Washington typically have developed atop deeply weathered rocks containing illite and montmorillonite clays. Such clays commonly are derived from the weathering of volcanic ash or feldsparrich rocks. Data in Washington are limited on the formations that display these characteristics. In the greater Seattle area, the Puget Group of Eocene age contains rocks that weather to expansive clays. However, because of glaciation in the area, most of the potentially expansive weathered soil has been removed from the northern parts of the state. It appears that since deglaciation in the Seattle area, some 13,500 yr ago, there has been insufficient time to develop a deeply weathered and expansive soil horizon on these rocks. Therefore, the expansive clays are present only in limited, isolated, and sheltered pockets in the Puget Sound area or other glaciated portions of the state. Expansive soils have not been reported in the Tri-Cities, Yakima, Ellensburg, Moses Lake, and Vancouver areas, which have not been glaciated. The majority of the soils in these areas appear to be relatively granular. It is not known at this time whether swelling soils are present in the Walla Walla, Centralia, or Chehalis areas. It appears likely, however, that once development begins to take place in outlying areas surrounding these communities, expansive soils may be found.

The primary clay minerals displaying expansive properties are montmorillonite and illite. Soils containing these clays have been subjected to a wide variety of tests locally, nationally, and internationally (for example, Kassif et al., 1969; McKeen and Hamberg, 1981). Such tests have included mechanical analysis, Atterberg Limits, moisture-density relationships, stiffness, suction, and various swell tests. Typical Puget Sound area expansive soils contain an average of 15 percent silt (0.074 mm-0.005 mm) and 84 percent clay (<0.005 mm), of which about 65 percent is colloidal sizes (<0.001 mm). Atterberg Limits included Liquid Limit, 85; Plastic Limit, 30; and a Plasticity Index of 55. This type of soil has a maximum dry density (AASHTO:T-99) of about 80 pcf and an optimum moisture content of about 35 percent. Under a confining pressure of 140 psf, about 15 percent swell may occur over a 48-hr period. One-dimensional expansion pressures on the order of 1,000 to 1,500 psf have also been recorded, although greater amounts of swell have been noted on samples from one site in the Tukwila area.

Stabilization of expansive soils normally can be accomplished during the sitework phases of a project. Methods utilized locally have included complete removal of the problem materials or isolation of the expansive soils by an adequate cap of nonexpansive, relatively impervious fill. Where expansive materials have been encountered on hillsides, a combination of partial removal and the construction of a buttress fill has been used to control sliding. Once buildings have been constructed, repair options appear to be relatively limited. Lime injection, which has been used elsewhere, is not performed locally because of lack of demand and experience. Underpinning buildings with deep foundations that are capable of resisting the anticipated uplift forces can be more costly than demolition and reconstruction. As a result, one local apartment building owner simply replaces ground floor-level slabs on a periodic basis and accepts the movement as part of the cost of a view location.

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Aerial view of the Hughes landslide, Sanpoil Arm, Franklin D. Roosevelt Lake (Grand Coulee Dam reservoir). The slide is in glacial lake sediments. Photograph by R. W. Galster, July 1979.

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The Influence of Man's Development on Geologic Processes

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INTRODUCTION

The series of preceding discussions has considered the various geologic factors within the state that influence the works of man. This discussion reverses the process to consider ways in which the works of man influence geologic factors and ultimately other works of man. Early inhabitants of the region lived on the land for thousands of years with little or no effect on the landscape nor on the geologic processes shaping it, owing to the lack of population density and to their limited needs. As man figuratively crawled out from the cave and invented tools for constructing things, the land and processes shaping the land also were altered.

Although western civilization began in Washington nearly 200 years ago, most construction of consequence has occurred during the 20th century. Because of this activity, some of which caused significant landscape alteration, some interesting and important changes in the effects of geologic processes, at times in the processes themselves, have occurred. Cultural development means the spread of urban areas and their infrastructure, transport systems, navigation improvements, and dams, the trappings of modern civilization without which engineering and thus engineering geology would not be necessary.

One should keep in mind that in the sense of long-term geologic processes and geologic time, man and his works are but slight indentations. Yet our influence on geologic processes as they affect our civilization can be tremendous; whether it is manifest in construction of a dam or a city or in hoeing a potato patch, the influence is clearly there. Until recently, however, the influence of construction on geologic process has rarely been considered. Even now, such changes are considered only on very large projects as a result of the environmental movement of the 1970s and 1980s. Because the environmental movement has tended to be concentrated more on the biological aspects of the environment, often to the exclusion of geological aspects, construction activities have occasionally resulted in a number of geologic problems best described by the immortal words of Pogo: "We have met the enemy and he is us."

EFFECTS OF URBAN CONSTRUCTION

Perhaps the most far-reaching influence of urban construction may be summed up in a single word—blanketing. In the process of covering the land surface, whether it be by pavement or buildings, the area available for infiltration of rain water and snowmelt to recharge ground water is drastically reduced; in many urban situations it is virtually eliminated. Thus, the natural movement of water through the ground to usual discharge points such as springs and streams is altered. Instead, the movement is replaced by storm sewers, by more concentrated entrance points of water into the ground, or by increased concentrations of surface drainage. In the first case, some storm sewers are combined with sanitary sewers to place undue loads on treatment facilities and provide additional contaminants at point discharge areas. In the last case, excessive amounts of erosion tend to occur in the streams draining such areas, with concomitant increased deposition of sediments and greater contaminant concentration at stream mouths. A multitude of examples of these problems is seen in the glacial drift terrain of the urban areas of western Washington where pre-urban infiltration was not particularly rapid in certain areas but is greatly reduced or nonexistent now. In areas of high, pre-urban infiltration rates (sand and gravel terrains), such as the Tacoma-Nisqually uplands and Spokane valley, urbanization has had similar effects, except that surface water and contaminants find their way directly into the ground water by rapid infiltration—a classic case of "fouling the nest".

The second influence of urban construction is simply the rearrangement of surficial geologic material for foundations, grading, and similar features; this includes that unique form of bioturbation known as construction of underground utilities and that is a characteristic of urban areas. The backfilling of such excavations is often such that new conduits for ground water are created. The resulting piping of materials results in settlement and collapse of streets and, in some instances, undermining of buildings. Breakage of utility lines usually aggravates the problem and may be both cause and effect. Perhaps the greatest example of earth rearrangement has been in the hydraulic regrading of some of Seattle's hills during the first half of the 20th century and creation of industrial lands on formal tidal areas. This included channeling of the tidal parts of the Duwamish River and creating Harbor Island. The post-World War II increase in urbanization in the Duwamish valley brought about the need for flood control on the Green River. Similar, though perhaps less spectacular, effects are seen elsewhere in the Puget Sound Basin. For the most part, other urban areas of the state have worked more with the landscape than re-arranging it in such a spectacular manner.

The grading of urban streets and building sites may frequently initiate landsliding in otherwise stable ground or re-initiate movement in marginally stable old landslide areas. Because landsliding is a normal geologic process, such grading aggravates the process either by removal of support or changes in surface drainage and ground-water conditions. To do so without seeking advance information and the means for mitigating has been found to be a costly folly, especially in the glacial drift that characterizes many of the urban areas of western Washington.

The influence of urban construction on flood plains and alluvial fans is usually evident in efforts to detour flood waters. Because flood-plain communities—such as Puyallup, Snohomish, Monroe, Centralia and Chehalis, Mount Vernon, Omak and Okanogan, Oroville, and Kelso and Longview—are frequently threatened by flood waters, they have resorted to systems of temporary and/or permanent manmade levees to minimize damage to urban areas. In so doing, a partial hydraulic dam is formed on the flood plain, which tends to increase the height of upstream flood peaks and concentrate erosion in adjacent areas. The thwarting of such a normal fluvial process can have some long-term effects on the flood plain in terms of future flooding, the necessity for higher and stronger artificial levees, increased sedimentation, changes in stream course, and increased erosion. For urban areas that are built on large alluvial fans (such as Wenatchee), the process of flooding and detrital deposition means occasional clean-up after infrequent flooding, while little is done to alter the basic process itself other than channeling the water and debris through the city streets as much as possible.

EFFECTS OF TRANSPORTATION NETWORKS

Transportation networks have developed at the pace of the growth of cities and towns of the state. Because valleys usually afford the easiest paths of construction in terms of both grade and alignment, rail and highway networks commonly use them whenever possible. However, in the process of constructing embankments along and across valley floors so that facilities will be above flood waters as much as possible, a series of confining dikes and barriers to normal discharge of flood waters is created. This not only influences the height of flood waters but also serves to alter the stream regime so that deposition of sediment and erosion are concentrated in areas that were formerly little affected. The lower Snohomish (Figure 1) and Green River valleys are typical examples. Channel confinement at stream crossings is often a disastrous culprit. Although it is repeated in many valleys, the Colville River valley in northeastern Washington is a good example of where confined stream channels at road crossings result in excessive upstream flooding and deposition.

An early and very evident effect of highway and rail construction was the clearing of rights of way which frequently resulted in changes in erosion and deposition. Excessive erosion of new cut and fill slopes and excessive detrital loading of nearby drainages can be a common change in the geologic process, usually felt for considerable distances downstream. Fortunately, engineers and construction contractors, as a result of the environmental movement, now attempt to control such effects both during and following construction. Both short- and long-term major changes in erosion or deposition are now commonly mitigated. The more mountainous the topography, the greater the cuts and fills required. Although early railroad construction frequently resorted to trestles and tunnels in mountainous terrain, the advent of modern earthmoving equipment and explosives has substantially altered not only the manner in which transportation networks are constructed but also their grade and alignment as well. As a consequence, increased slope stability and drainage problems now influence the normal geologic processes operating over the route. In some places, streams have suffered major diversions with gradient changes and resulting changes in erosion and deposition both upstream and downstream. Cuts and fills frequently intersect spring lines, altering the ground-water system as well as surface drainage.

Transport routes also have had some positive aspects in altering geologic processes relating to works of man. For example, the location of most of the Burlington Northern Railroad route along the shoreline between Seattle and Everett and the requirement to protect the route from storm wave erosion by more or less continuous stone walls and riprap have led to elimination of further sea-cliff erosion. This permits greater long-term stability for extensive residential use of adjacent lands, whereas similar areas around the periphery of Puget Sound have suffered continual loss of land. Many such examples may be found where rail lines or highways are adjacent to the state's waters.

EFFECT OF DAM CONSTRUCTION

Although low barriers across streams have only minimal effect on stream characteristics, larger dams, which characterize most of the Columbia, the lower Snake,

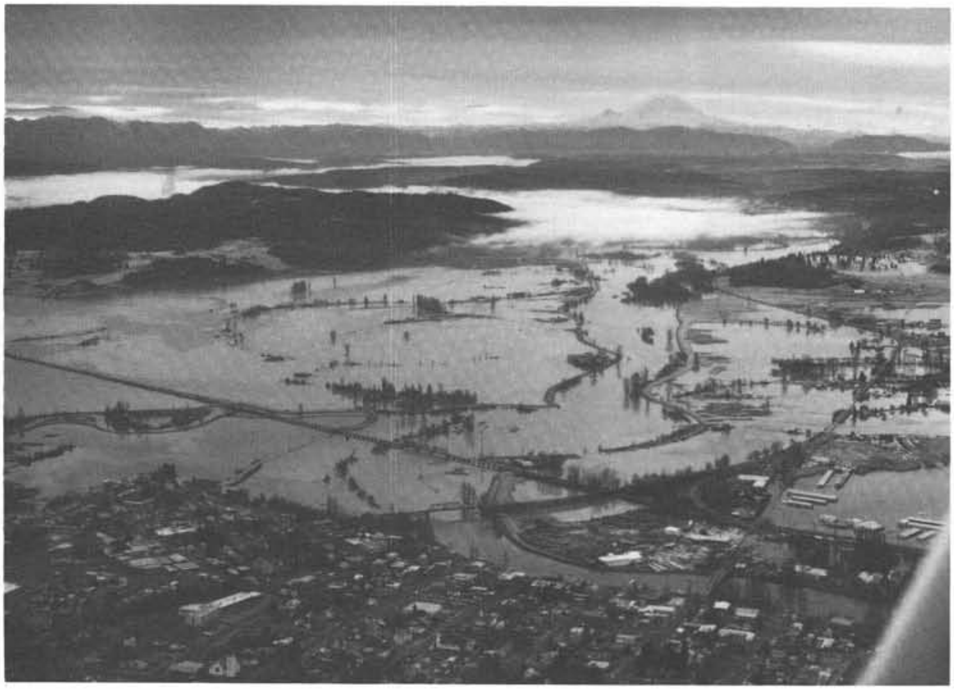

Figure 1. Aerial view southeast of the Snohomish River and valley during flood stage, December 1975. The town of Snohomish is in foreground. The river flows toward the viewer and off to the lower right; it is confined between dikes built on natural levees. The flow of the river during flood stage is constrained by highway and railroad embankments that cross the valley floor. Photo by the author.

Spokane, Pend Oreille, and many of the western Washington rivers, have had major impacts on river regime and some far-reaching effects both upstream and downstream. Some of the impacts on the Columbia and its major tributaries have been generated by dams outside the state.

Prior to advent of dams on the Columbia main stem, the free-flowing stream flooded annually, moving an estimated 10 million tons of detritus to its mouth, mainly during the spring floods (Phipps and Smith, 1978). Between 1935 and 1975 the floods were gradually reduced by dam construction, and much of the detrital load is now confined in more than 50 reservoirs throughout the drainage basin. Thus, the normal process of the flooding stream moving detritus furnished by its drainage basin into the sea no longer exists. While the overall positive aspects of water conservation are evident in terms of power, flood control, irrigation, and recreation, the negative effects on the geologic processes over the long-term also should be realized. For example, prior to dam construction a portion of the sand reaching the mouth of the Columbia River was distributed by littoral drift north and south along the Washington and Oregon coast between Point Grenville and Tillamook Head, creating the long barrier beaches of the Clatsop Plains, Long Beach Peninsula, Twin Harbors-Westport, and Ocean Shores-Copalis. The diminishing of this detrital supply appears to be having repercussions along the coast that are just beginning to be noticed (Galster, 1987). As a consequence, some areas previously characterized by deposition ultimately may become areas of erosion. The series of reservoir lakes also has significantly altered erosional patterns along the Columbia—changing them from fluvial current erosion/deposition to wind-generated wave erosion. Ground-water levels adjacent to reservoirs have been raised, and in some places the direction of ground-water flow has been altered.

Of special interest is the Hanford Reach of the Columbia River, which is the only remaining undammed section of the main stem. Mistakenly referred to as "free flowing" by some authors, the reach has been under control for more than a quarter century since completion of Priest Rapids and McNary dams. Although no longer subject to large floods, the reach has daily discharges that vary from a low of 36,000 cfs to about 100,000 cfs and a regulated 100-yr flood flow of 440,000 cfs (Rittenhouse Zeman and Associates, 1987). Velocities developed under such flows can move detritus as large as small cobbles. The reach is without tributary streams, but landsliding along the White Bluffs continues to provide fine-grained material to the river. The net result appears to be a reach largely characterized by coarse material and very stable channel islands; former river bars are no longer modified by major flood discharges.

The dammed rivers of western Washington exhibit some similarities to and some differences from the Columbia, and except for the Cowlitz, most of the dammed sections of western Washington rivers have higher gradients. None is so thoroughly dammed as the Columbia, and all are subject to heavy winter floods in addition to spring runoff. Control of flooding and detrital loads depends on the reservoir position with respect to tributaries and how the project is operated. In most rivers, detrital loads from upstream are deposited in the reservoirs, discharges downstream vary daily or seasonally, and the erosion/deposition regime is significantly changed. The single exception is Mud Mountain Dam, the only major single-purpose flood control dam in the state, which passes the detrital load of the White River through the dam, albeit with lower discharges than normally would occur. Although nearly all reservoirs are sediment basins, detrital movement on the lower reaches is controlled largely by downstream tributaries. In some instances, increased erosion is due to dam-related changes in discharge.

The manner of operation of a dam and reservoir may have a powerful impact on geologic processes. Rapid reservoir drawdown, following a long period of high reservoir levels, frequently increases landslide activities along the shore of the reservoir. Rapid and frequent changes in discharge may have both erosional and depositional effects downstream. For example, the Skagit River, between Gorge Dam and the Newhalem Powerhouse, is dry much of the year, owing to the entire flow being diverted through the tunnel penstock. The trapping of sediment by two dams on the Elwha River (Glines Canyon and Elwha) appears to have had major effects on the amount of littoral detritus available for natural maintenance of Ediz Hook (Galster, 1987). Several of the dams along the Cascade ice border (the western Cascade slope) have changed ground-water flow conditions appreciably.

The influence of dam construction on geologic processes often tends to be ignored by engineers and owners in the belief that such influences are long-term—related more to geologic time than to human time. However, such influence has become evident in many areas of the state within a short time and is a frequent source of frustration, costly repair, relocation, and legal action.

EFFECT OF COASTAL CONSTRUCTION

The balance between erosion and deposition along coastal areas is a function of material supply, prevailing wave action, and longshore currents. Any attempt to alter this action, either by reduction of littoral supply or by changing the action of the transport forces, has far-reaching effects. As with streams, coastlines exist in a crude equilibrium between supply and demand—a fact frequently lost on the designers and builders of coastal structures, whether these structures be riprap blankets to retard erosion or large breakwaters and jetties.

Coastal construction is usually required for one of three different reasons: navigation improvement, erosion mitigation, and port facilities development. Assessment of the effect of construction on prevailing geologic processes requires answering the following questions:

(1) What are the present geologic processes in effect and what are their seasonal variations?

(2) What is the size of the littoral cell involved, what is the source of littoral material in the cell, and what is its ultimate destination?

(3) What is the minimum construction that might be done to attain desired goals, and what will be its effect on the littoral cell?

Such questions are extremely important in high-energy coastal areas, such as along the ocean coast and the straits of Juan de Fuca and Georgia, but they are no less important along the periphery of the inland waters where energy levels may be less but are more variable.

Probably the most far-reaching effects of construction on coastal processes have been the results of construction of jetties at the mouths of the Columbia River and Grays Harbor. Although these structures have enabled easier navigation into their respective inlets, they have influenced the coastal process itself and substantially changed the outline of the southern Washington and northern Oregon coasts. They have established new controls for the accumulation of sand between Tillamook Head, Oregon, and Point Grenville, Washington, and much of the recent accretion at beaches between these headlands is a result of these structures.

Assessment of the less dynamic littoral drift of the inland waters is no less important and frequently more difficult. Breakwaters, boat basins, and their accompanying channels abound along the inland waters. A few of these have interrupted the littoral drift and caused some local erosion of adjacent lands, whereas others have been benign, in some instances through appropriate geological assessment or because of a fortuitous combination of geologic circumstances.

Construction of beach erosion protection facilities leads to a variety of changes to coastal process. The use of groins in one place may trap sand and gravel, but may starve the beach downdrift. Construction of sea walls, bulkheads, and riprap blankets, if not done smoothly and nearly simultaneously along a shoreline, is fraught with problems of end-around erosion in unprotected areas. When these structures are emplaced, continued wave action may lower the beach profile and undermine the structures unless the amount of detrital material available continues to be sufficient to maintain a dynamically stable beach. It is usually the geologically more ephemeral landforms—spits, bars, and hooks—which react most quickly and sometimes violently to process interruptions. This fact is most important in the granting of permits and in the construction of shoreline protective works. Fortunately, most major port facilities are situated in relatively calm areas that are not highly dynamic in terms of coastal processes. Exceptions to this are the oil ports along the dynamic shoreline of the Strait of Georgia in Whatcom County, where because of the use of piers on piles there has been no interruption basic coastal processes.

EFFECTS OF OTHER ACTIVITIES

Four other areas of human endeavor have had major impacts on geologic processes in Washington: agriculture, timber cutting, mining, and waste disposal. Perhaps the most critical of these is timber cutting, which has vastly altered the balance of water infiltration, runoff, and evapotranspiration over large areas of the Cascade, Olympic, and Coast Range mountains, the Puget Lowland, and, to some extent, parts of the Okanogan-Selkirk Highlands. In so altering this balance, an increased amount of the rainfall-snowmelt budget has become runoff. Before modification of logging practices, this increased runoff resulted in considerable erosion on steeper slopes. There is continuing evidence of the change timber cutting has made on the process of erosion in the form of occasional severe debris flows and landslides. In general, increased runoff has increased stream discharges and their erosive power and in some places added additional detritus to the streams for transport.

In the past, agriculture has significantly affected local geologic processes but usually less dramatically than other endeavors. Efforts to conserve top soil have been successful in most agricultural areas of the state; however, local instances of continuing erosion and detrital contribution to streams have occurred, especially on the Columbia Plateau. The irrigation in the Yakima, Wenatchee, and Okanogan basins and on the Columbia Plateau has influenced the position of the water table and movement of ground water in irrigated and adjacent areas. It has also changed the chemistry of the ground water and the surface water because of fertilizers and leaching. Elevated and, in places, perched ground-water tables that result from irrigation, especially overirrigation, tend to produce or aggravate slope instability along nearby valley sides and change the character of the erosional process.

Mining influences geologic processes in a variety of ways, depending on the magnitude of the surface and underground workings. Changes in stream regime by construction of tailings dams are obvious, as is the addition of detritus from old mine tailings prior to the control of mine-tailing disposal. Open pit workings with settling ponds have considerable influence on local surface runoff and ground water, as well as on the chemistries of these waters. Underground workings alter the ground-water movement and water-table level by draining large prisms of ground water and providing conduits for more rapid future movement.

The influence of waste disposal on geologic process, as in mining, depends on the scale of the effort. Although large sanitary landfills commonly change the drainage process, the major influence of waste disposal has been the local alteration of ground and ground-water chemistry and the contamination of surface waters. This has been especially true where waste has been disposed of indiscriminantly, with little or no attention paid to the underlying geology. These conditions are now changing, though past excesses continue to haunt us and provide much employment in mitigation efforts.

PLANNING

The interruption of geologic processes by the construction of urban areas, transportation networks, dams, and the other engineered trappings of modern civilization in Washington State can be largely mitigated by thorough geologic assessment, proper planning, and appropriate design. Of paramount importance in this effort is the establishment of a strong geological data base. The keeping of good records of subsurface exploration, excavations, position of underground utilities, and of the centralizing of such records are acutely necessary in urban areas. Without such records, proper geologic assessment, necessary for the planning process, is most difficult (Legget, 1973). Such data bases are necessary to establish the geologic constraints involved in proper land-use planning, which is nothing more than the conservative use of the land upon (and under) which our civilization is built. Often our geologic environment and the processes that created it are overlooked in the planning process (Mathewson and Font, 1974). To solve this problem requires education of the planners and the public by the engineering geology community. We have made great strides in incorporating local geologic factors in engineering design, but we have a great distance to go in incorporating the assessment of how we influence geologic processes, and how they, in turn, will affect us and our cultures.

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Howard Hanson Dam on the Green River; view upstream, spring 1960. The river has been diverted through a tunnel on the south (right) bank, upstream of which is a tall concrete structure. The downstream portal is in the lower center part of the photograph, just right of the downstream cofferdam. The embankment dam is to be built in the area between the two cofferdams, here being dewatered. Drilling and blasting of the spillway cut is in progress (right). Photograph by the Seattle District, U.S. Army Corps of Engineers.

The Practice of Engineering Geology in Washington

Richard W. Galster, Chapter Editor

Engineering Geology in Washington, Volume I
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The Role of Geologists in an Engineering Organization

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GENERAL STATEMENT

Although geologists have been integral members of mining and petroleum organizations for more than a century, the use of their services on the civil engineering and ground-water scene has been only during the past 60 years or so. This need for geologists resulted from two factors: (1) the large demand by the civil engineering profession for a better understanding of geologic factors involved in heavy construction; and (2) the increasing ability of many geologists to apply geological science in terms of engineering requirements (Burwell and Roberts, 1950).

Over the years, the role played by the geologist in an engineering organization has been, and still is, highly variable. It depends on the personality and ability of the geologist, the attitude and background of management, and the type of organization involved. There is little consistency as to what constitutes the duties and responsibilities of an engineering geologist, even within large federal organizations like the Department of the Interior's Bureau of Reclamation and the U.S. Army Corps of Engineers, and there is even less consistency among private organizations. The situation has become more complicated with development of an array of titles for those whose activities are slightly, partly, largely, or entirely in engineering geology. Such titles vary from civil engineer, geological engineer, materials engineer, engineering geologist, to hydrogeologist or geologist. There are also technician titles for engineering geology activities.

UTILIZATION OF GEOLOGISTS

A great diversity of opinion exists regarding the importance of engineering geologists in an engineering organization. In some private companies and governmental entities, an engineering geologist may be the chief executive officer or manager of a division. Sometimes with such organizations, engineers are relegated to duties of "number crunching" and signing plans. At the other extreme, there are civil and geotechnical engineering firms that either do not hire geologists or they relegate what few geologists are employed to a technician level. Above this latter type of an organization are those where engineering geologists are employed primarily as data gatherers; that is, they log test holes and draw maps, but otherwise they have very little input into the interpretation or decisions that affect geotechnical impacts on project planning. It appears that most geotechnical and some civil engineering firms operate in a mode somewhere between the extremes described. Additionally, with the development of a large need for hydrogeologic studies, especially hazardous waste assessment, the importance of engineering geologists has increased. In 15 states, including California, Oregon, Idaho, and Alaska, the professions of geology and engineering geology now are governed by regulations, and the practitioners are certified or licensed. In those states, geologists have become increasingly important in the operation, management, and direction of geotechnical firms. In Washington, however, registration or certification of geologists is still not required.

Normally, engineering geologists begin their professional careers as data gatherers and interpreters, logging test holes, obtaining samples, mapping outcrops, and performing routine field and laboratory testing. As their education continues to expand, duties may include the preparation of reports on the geologic observations in a format suitable for inclusion in a geotechnical report. The next step on the professional career ladder might involve compilation and interpretation of data, especially for more complex projects, in a form that would permit the understanding of subsurface conditions in three dimensions. Many geologists, for one reason or another, have difficulty in growing professionally beyond this level.

Options are available for professional growth and advancement. For example, it frequently is the practice to assign junior geologic personnel to construction inspection. Some geologists specialize in this area. They may manage a staff of junior-level inspectors and geologists and be able to advise clients on how to save money or minimize risks during site work. Some geologists specialize in laboratory testing, eventually becoming laboratory managers or directors. Other engineering geologists develop skills in budgeting and estimating and, as a result, enter the areas of marketing and proposal preparation. A fourth group, hydrogeologists, normally model ground-water flow with computers. Finally, some organizations train their entire staff, including engineering geologists, to perform relatively straightforward engineering work under the close supervision of a licensed engineer. In most geotechnical engineering firms, however, engineering geologists interpret to the engineering staff what the geologic conditions, as described by bore hole, geophysical, and surface data, really mean in terms of design and construction. In this way, they both support and complement the engineering design personnel.

REQUIREMENTS

In order for geologists to fit into an engineering organization, they must fulfill several important requirements:

First, they must be competent geologists (Burwell and Roberts, 1950). They must have the principles of geology so well in hand and feel so sure of their application that they are not the least disturbed at finding things of a geologic nature materially different from anything else they have ever seen (Berkey, 1929).

Second, they must be able to translate geologic discoveries and interpretations into terms of practical application that the engineer, designer, planner, construction contractor, or owner can understand. This requires a sound knowledge of engineering and design requirements (Burwell and Roberts, 1950) and a better knowledge of a planner's, contractor's, or owner's needs.

Third, they must be able to make timely, sound judgments and important decisions (Burwell and Roberts, 1950). The late Allen S. Cary, of the Seattle office of the Corps of Engineers, was fond of saying that good judgment comes from years of experience and that experience comes from years of exercising bad judgment. What he was saying is that one must not be afraid to make decisions based on sound judgment and incomplete data because it is not always economically practical or timely to eliminate all uncertainties. It is better for the geologist to make timely recommendations or decisions on geological matters with less than adequate information than to force someone else without geological expertise to make them. The integration of information at the proper time is essential to the proper functioning of any engineering organization (Burwell and Roberts, 1950).

In addition to the above attributes, a geologist must have the proper temperament to fit into an engineering organization. He or she must be neither an alarmist nor afraid to speak up at any time during a project's formulation, design, construction, or operation where matters influencing the design, cost, or safety are concerned. This is as true for residential development on a small lot as it is for a major project, for the small lot is a major project in the eyes of the owner. A small landslide on a major highway construction job which may require $100,000 to correct may not be of great significance, but such a cost can be devastating to the small residential developer. Thus, it is highly important for the geologist to make this adjustment in perspective when considering projects for various clients.

TEAM EFFORT

Above all, geologists must consider themselves as members of the team involved in development of a project—a team of numerous members—generally with a civil engineer or architect as the person responsible for the whole project. As one member of such a team, the geologist must be cognizant of the problems of all the other members and of the possible influences of geology on each of their specialties. The geologist must not work in a vacuum or consider his or her job completed when the report is submitted to the engineer. The geologist, with a commitment to the scientific process, examines the geologic environment, defines the need for and the type of subsurface investigation, and recommends remedial measures to assure that the earth materials will withstand the stresses placed on them by the structure. Geologists must constantly work with other members of the team throughout the siting, design, construction, and, sometimes, operational phases of a project to make sure their advice is timely and appropriate on geological matters. Only by so doing can the project be completed in a timely, cost-effective, and safe manner. Engineers or architects who fail to consider the geologist a full member of their team during the entire period of project development do themselves and their clients a great disservice and effectively cheat themselves by poor utilization of team members.

FUNCTIONS OF A GEOLOGIST

The functions and responsibilities of a geologist in an engineering organization may be divided into seven main categories (Burwell and Roberts, 1950) and may be summarized as follows:

(1) Providing geologic background for the site area.

(2) Selecting a site; the preliminary site investigation and assessment to determine general suitability, presence of "show stoppers", or features relating to geology that would require extraordinary design or construction costs.

(3) Developing geological details for design and contractual purposes, and providing timely advice.

(4) Developing information on natural construction material sources: sand, gravel, clay, and rock.

(5) Developing contract plans and specifications relating to foundation treatment, excavation, groundwater control, and the presentation of subsurface and surface conditions.

(6) Monitoring construction progress and problems, and providing advice to engineers and contractors on mitigation of construction problems.

(7) Monitoring the influences of the project on natural conditions—changes in slope stability or ground- and surface-water alterations that could influence the project and nearby areas over time.

It is most important that the geologist be called in during a project's formulative stage in order to keep funds from being unnecessarily spent on irrelevant site investigations. However, it is equally important for the geologist, who is initially trained in the scientific method, not to carry an investigation farther than it needs to go to accomplish the design and construction of the project. It has been said that the purpose of a boring in soil or rock is to check the geologist's interpretation and once the interpretation checks, further exploration is unnecessary. Although the number of borings may be in part a function of site geologic complexity, borings are also necessary to obtain samples for laboratory testing. The test of geologic predictability is appropriate when determining the amount of investigation required. There are instances of excessive exploration as well as examples of insufficient exploration, but it should be the geologist's function to determine such sufficiency levels and the adequacy of overall geological investigations.

ACKNOWLEDGMENTS

We wish to acknowledge the early dynamic thoughts of C. P. Berkey (1929), who is considered the father of engineering geology in the United States, and those of E. B. Burwell and G. Roberts (1950). An occasional review of their papers by geologists, engineers and architects alike is recommended. Review of this paper by G. E. Neff and his helpful comments were most appreciated.

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Kettle Falls, Columbia River. View to the northwest showing the condition of the former falls after 34 years of inundation by Franklin D. Roosevelt Lake. Photograph taken by R. W. Galster in May 1974 while the lake was drawn down for modification of Grand Coulee Dam.

Meeting at Priest Rapids core shed, March 5, 1955. From left to right: Clifford Wells, Chief Geologist, Harza Engineering Co.; Robert Reiss, Grant County Public Utility District; Fred O. Jones, Consulting Geologist; Keith Willie, Harza Engineering Co.; Robert Laughlin, Grant County Public Utility District; Allen S. Cary, District Geologist, Seattle District, U.S. Army Corps of Engineers; J. Hoover Mackin (kneeling), Consulting Geologist; W. (Brownie) Walcott, Project Geologist, U.S. Bureau of Reclamation.

Building Codes for Construction on Steep Slopes in Western Washington

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INTRODUCTION

Building and grading codes and regulations in Washington are controlled by city and county governmental administrations. Many of these city and county agencies rely on the provisions in the Uniform Building Code (UBC, published by the International Conference of Building Officials and periodically updated), particularly those in smaller municipalities or rural counties. However, larger entities in more developed areas have adopted their own regulations. The adoption of more detailed and sophisticated regulations usually, but not always, parallels the change in growth and population of the political unit. Thus, as a city or county becomes increasingly more populated, more pressure is put on the land (as less desirable property becomes developed), and a greater need to limit or control development is perceived. Justification to create restrictions on development comes from "hard-learned lessons" and from government's need to protect the public. Most regulating agencies list protection of the health, safety, and welfare of the general public as the purpose of development restrictions. Other agencies are more specific, such as the cities of Kent and Redmond, which state that the regulations are based on "known physical restraints".

One of the reasons that government agencies have adopted limiting regulations for geologically sensitive or hazardous areas is liability. Some governments have been sued because they issued building or grading permits for projects that subsequently have experienced stability, erosion, or drainage problems either on the permitted property or on adjacent properties. Therefore, the response of government has been to protect itself by making it more difficult to develop land that has physical constraints, such as steep slopes and compressible or subsiding ground. Some agencies also address seismic hazards, but none require special provisions for developing land in those areas identified as subject to seismic hazards.

Another reason for adopting regulations is to make the public aware of the risks involved in developing ground that is slide-prone or in other ways potentially hazardous. This public awareness can be accomplished by making available maps depicting such areas and by developing guidelines for studies on the designated properties.

The goals of restrictions on geologically sensitive areas are basically three: (1) to limit the density of development, (2) to restrict or prohibit development, and (3) to establish reasonable standards to guide professionals involved in the design for and construction on such land. The applicability of the regulations to a particular property can be determined by the angle of slope, the geologic conditions, and the ground-water conditions—sometimes by a combination of the three criteria.

ROLE OF THE ENGINEERING GEOLOGIST/ GEOTECHNICAL ENGINEER

The role of the engineering geologist and/or the geotechnical engineer is of great importance in the whole development process, whether the project is a single-family dwelling or for a commercial or industrial site. Single residences or subdivisions containing single- or multi-family residences comprise the overwhelming percentage of projects. The City of Seattle and King County have their own in-house geotechnical engineer and engineering geologist, respectively—a reflection of the size of the population and the increasing land development in those areas. These officials, however, have neither the time nor the staff to provide anything but review of submitted plans and engineering reports. The provision of services to the public is still the responsibility of consulting geologists and engineers.

Previous work by soil scientists and geologists provides the basic information that helps geologists, as well as the public, learn and research the general stratigraphy and geologic history of a site. Determination of the local or regional geologic conditions would be impossible or economically impractical for every small project. Therefore, good geologic maps, at quadrangle or larger scale (1:24,000 to 1:250,000) are essential to the performance of practical engineering geology. This kind of mapping for Washington is provided in U.S. Geological Survey (USGS) maps, USGS Water-Supply Papers, Soil Conservation Service maps for counties, Washington Division of Geology and Earth Resources maps, Washington State coastal resources folios, and masters and doctoral geologic theses from the state's universities.

Seattle, Bellevue, Kirkland, and King County also have completed their own geologic hazard mapping. Seattle's was the outgrowth of a doctorate thesis at the University of Washington by Tubbs (1974), Bellevue's was the product of a San Diego State University masters thesis by Gladden (1981), and King County's (1980) was produced by an in-house team of professionals and technicians. Kirkland's map (1980) was produced by a private consultant. While these works provide information for planning and zoning, they are still too general to use for evaluating the geologic suitability of a particular site. Such an evaluation is still the job of the consulting engineering geologist or geotechnical engineer. Without on-site evaluation of the topographic nature, the ground-water characteristics, and the detailed stratigraphy of the soil or rock, it is impossible to judge the suitability for development, the foundation types, or the drainage system for a particular piece of property.

Geologic evaluation of property that has possible hazards is carried out in two steps. The reconnaissance phase is usually accomplished during a prospective owner's consideration of purchase or during an owner's application for short plat of a large property. Information at this stage is often, but not always, preliminary and based on surficial observation. The engineering geologist's local experience and knowledge of geomorphic (landform) expression is put to the test at this phase of evaluation. Later, during detailed consideration of a property for final design of a structure, it is usually necessary to perform subsurface exploration.

GEO-INFORMATION REQUIREMENTS

Of the 18 cities and counties that responded to a request for copies of their procedures with regard to geologic hazards, 6 cities (Bellevue, Bothell, Kirkland, Mercer Island, Port Angeles, and Seattle) and 5 counties (Island, Jefferson, King, Snohomish, and Thurston) require certain geologic/geotechnical information in the form of a professional report for property deemed subject to control. Some cities, such as Kent and Redmond, have rules to determine the applicability of controls or restrictions on a property, but no specific criteria were listed for inclusion in a report. Other cities and counties simply require compliance with the UBC (International Conference of Building Officials, 1985) for building and grading. In both chapters 29 (Excavations, Foundations and Retaining Walls) and 70 (Excavation and Grading) of the UBC, the local building official is granted discretionary power to require a soils engineering and/or engineering geology report; however, on properties where proposed cut or fill slopes are steeper than 2 horizontal to 1 vertical (2H to 1V), a report is required.

The types of information generally required by many municipal or county agencies in the geologic portions of the report are:

(1) topographic data - contour map, if possible, but at least a description, including slope angles,

(2) subsurface data - boring logs and exploration methods - soil and/or rock stratigraphy - ground-water levels and seasonal changes of the levels,

(3) site history - prior grading or landsliding, and

(4) seismic hazards.

Geotechnical engineering information commonly deemed necessary by municipal and county officials for a report is as follows:

(1) slope stability studies and opinion of stability,

(2) angles of cut and fill slopes,

(3) foundation requirements,

(4) soil compaction criteria,

(5) estimated foundation settlements,

(6) site grading requirements,

(7) surface and subsurface drainage,

(8) lateral earth pressures,

(9) erosion susceptibility,

(10) suitability of on-site soil for fill, and

(11) laboratory data and soil index properties for soil samples.

Not all of the above criteria are required by any one government agency in Washington; however, publications by agencies in the larger, more populated areas, such as Seattle Department of Construction and Land Use (1984), Bellevue Public Works Department (1987), Kirkland Building Department (1984), Mercer Island Department of Community Development (1985a, b), and Island County Engineering Department (1982), have lists of required information in formal codes or guidelines to developers. Seattle's *Procedures for Permitting Construction in Potential Slide Areas* (1984) and Bellevue's *Natural Determinants Implementation Project* (1987) are the most comprehensive of these guides. They require nearly all of the information in the above list. In most cases the government agency's list of information is suggestive only, that is, the engineer or geologist need not address each issue if it is not applicable. However, this is not true in Bellevue and Seattle. Bellevue's reporting procedure has a three-tiered system of requirements, which are not negotiable. The level and detail of the report depends on on-site physical characteristics. Seattle has gone one step farther, requiring a checklist and identification of the page number or figure number where a certain piece of information occurs in the body of the report. Further, if a specific requirement is considered not applicable, the report must state why it is not applicable.

APPLICABILITY OF SLOPE REGULATIONS

Various methods have been devised by the cities and counties of Washington to ascertain whether slope hazard regulations are applicable for a certain piece of property. They are as follows:

(1) map only,

(2) slope angle only, and

(3) multiple determinants.

Where a map is sufficient, a government will have invested in the production of a geologic hazards map to identify those areas that are slide-prone. Maps of this type are available for Seattle (Tubbs, 1974; Seattle Engineering Department, 1976), Bellevue (Gladden, 1981), Renton (Morrison-Knudsen Co., Inc., 1985), King County (King County Department of Planning and Community Development, 1980), Kirkland (Kirkland Planning Department, 1980), Mercer Island (Mercer Island Engineering Department, 1982), Jefferson County (Washington Department of Ecology, 1978), and Thurston County (Thurston County Planning Department, 1986). Although general in nature, such a map provides some means to the public as well as to government officials to determine the need for detailed geologic and engineering studies. Because the criteria and thought processes behind the construction of the map are usually known only to the author of the map, lot by lot on-site evaluation is needed to determine the applicability of the types of studies and information. In the eyes of a government official, inclusion of a property within the boundaries of a hazard zone is often sufficient to trigger the requirement of additional studies without preliminary evaluation. Those governments using specific hazard maps are Seattle, Redmond, Kent, King County, Kirkland, and Bothell. Jefferson County and Thurston County use maps in the *Coastal Zone Atlas of Washington* (Washington Department of Ecology, 1977).

Many counties or municipalities use only slope angle to determine if soil studies must be performed or whether a property can be developed. This is the most capricious method because of the arbitrariness of the threshold angle. Those governments using slope angle as the prime determinant are Kirkland (Kirkland Building Department, 1984), Mercer Island (Mercer Island Department of Community Development, 1985a), Port Angeles (Johnson, 1987), Jefferson County (Nathanson, 1987), King County (King County Building and Land Development Division, 1987), Snohomish County (Snohomish County Planning and Community Development Department, 1986), and Thurston County (Thurston County Building Department, 1986). King County, at this writing, is preparing to undertake a revision of steep-slope applicability criteria. Port Angeles requires that all properties that are adjacent to the top of a particular steep bluff in the city have soils reports completed for the property.

The wide range of slope angles (15 to 50 percent) used to determine the need for the required soil studies for a property indicates the arbitrary nature of using these criteria as a sole or prime determinant. It has been the author's experience that very few slide-prone areas exist where slopes are less than about 25 percent, and slopes of 15 percent or less usually experience no geotechnical problems. On the other hand, slopes of 60 to 80 percent are commonly stable, owing to the very competent soils that underlie them.

Because slope angle is an arbitrary factor when used alone, more sophisticated methods have been developed to define geologically sensitive areas. Several government agencies have implemented a system of using two or more factors to ascertain whether the inclusion of a property in a hazard zone is warranted and if detailed geologic and geotechnical studies are needed. For example, Kent and Redmond both have similar multiple criteria, which include percent slope, seismic risk, and slide and erosion potential. These are combined on a city map which is maintained on file by the municipal agency. Bothell uses a double determinant: slopes that are both greater than 15 percent and considered unstable according to the Soil Conservation Service map (Snyder et al., 1973). All of the above agencies' requirements are minimal compared to the Bellevue state-of-the-art Natural Determinants Implementation Project (Bellevue Public Works Department, 1987). It employs a matrix (shown on Table 1) of slope angle, soil type (geologic unit), and ground-water table. The rating derived from this matrix then determines the level or details of geotechnical studies. Requirements for each of the levels of study are then detailed in the Bellevue text. The comprehensive nature of this document reflects the full participation of geologic and geotechnical professionals in its development.

SUMMARY

While it is not encouraging that municipal and county rules and regulations remove some of the professional judgment from the development and design process, it is significant that the rules are being developed by geoprofessionals, rather than by government bureaucrats. Prior to the implementation of more comprehensive guidelines for Seattle and Bellevue, government agencies were well aware of the wide range of scope of services and thoroughness of studies provided by professionals working on similar sites. The publication

Table 1. Site stability evaluation matrix; for a discussion of the categories, see Section 2.B.08.C, Bellevue Public Works Department (1987) from which this table is taken

	Slope								
	0-15%			15-40%			>40%		
	Adverse hydrologic factors								
Soils/Geologic Conditions	None	Surface	Ground water	None	Surface	Ground water	None	Surface	Ground water
Manmade fill	B	B	C	C	C	C	C	C	C
Organic soils and peats	C	C	C	C	C	C	--	--	--
Fine-grained soils	A	B	B	B	C	C	C	C	C
Granular soils	A	B	B	B	B	B	C	C	C
Fill	A	A	A	A	A	A	C	C	C
Superimposed units									
- Till/gran/fine	B	B	B	B	C	C	C	C	C
- Gran/fine	B	C	C	B	C	C	C	C	C
- Till/fine	C	C	C	C	C	C	C	C	C
- Colluvium	C*	C*	C*	C*	C*	C*	C*	C*	C*

* Indicates that this condition may present very high risks to development. Contact the Storm and Surface Water Utility Permit Center representative.

of regulations governing the applicability of and criteria for geologic studies of steep slope is one method of increasing the uniformity of investigative procedures, informing the public of the risks, decreasing government agency liability, and protecting the public safety.

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Special gratitude is given to Thomas Kirkland of Shannon & Wilson, Inc., not only for reviewing this paper, but also for his contributions to the engineering side of my engineering geologic education. In addition, his efforts toward development of Bellevue's and Seattle's steep slope regulations have been instrumental in their achieving state-of-the-art nature.

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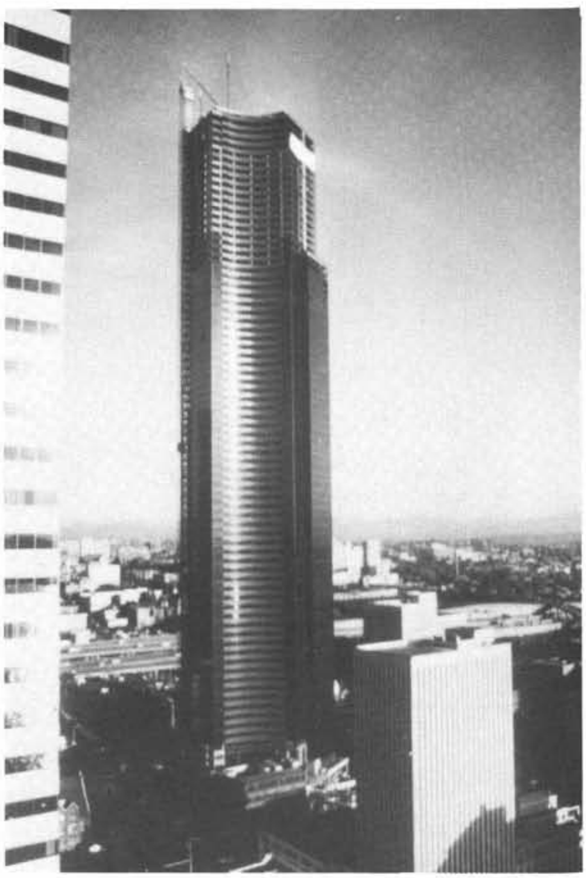

Columbia Center, Seattle, the tallest building in the western United States. Photograph courtesy of Shannon & Wilson, Inc.

Dakota Street/Delridge Way slide, Seattle, showing the damage to houses near the headscarp. Photograph by R. W. Galster, February 20, 1986.

Dakota Street/Delridge Way slide, Seattle, showing the slide following construction of the retaining wall at the toe along the east side of Delridge Way. The slide mass is covered by plastic sheeting. The house on the right is on the slide mass and was later removed. Photograph by R. W. Galster, March 1986.

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Legal Aspects of Engineering Geology in Washington

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Rittenhouse-Zeman & Associates, Inc.

INTRODUCTION

Unlike the members of many professions that directly influence human safety, geologists, specifically engineering geologists, are not licensed or certified by the State of Washington. As a result, a number of potentially negative impacts have developed. Foremost among these is the potential for unqualified persons to prepare geological reports, the results of which may adversely affect public health, safety, and personal and real property. Many engineering geologists work in geotechnical engineering firms where the responsible professional in charge is required by the State of Washington to be a licensed professional engineer who may or may not have an appropriate geological background. However, a geologist who is a private consultant or working within a government agency may be in responsible charge of work regardless of qualification and experience in engineering geology. An engineering geologist also may encounter difficulty establishing credibility in a courtroom or during arbitration proceedings if the opposition contends that, as an unlicensed individual, the geologist is not professionally qualified to act as an expert witness. Also because of their lack of legal standing, input from geologists may be discounted by individuals or public agencies when land-use decisions are made.

REGISTRATION HISTORY

During the early 1960s, numerous local governmental bodies in California, recognizing the need for geologic assessment in burgeoning urban and suburban development, established limits and controls on the activities of geologists. These controls and regulations were imposed because property damage claims, injuries, fatalities, and financial losses were occurring where inappropriate development took place in geologically sensitive areas. The intent of the various regulations was to establish a standard of competency and quality for engineering geology work. Because of conflicting requirements, costs, jurisdictional overlap, and a realization of need, the original California Geologist Act was passed in 1968. Subsequently, Idaho (1971), Oregon (1977), and Alaska (1980) established procedures for registration or certification of geologists and, in some instances, engineering geologists. By 1988, 15 states had established some type of licensing or certification relating to the practice of geology.

Recognizing the eventual need for professional licensing and following the lead of geologists in other states, geologists in Washington attempted to establish a licensing and/or registration program for the profession, beginning in 1964. Registration bills were introduced and studied by several legislative committees in 1971, 1972, and 1975. If enacted, these bills would have established a Washington State Board of Registration of Geology and Geophysics, similar to the boards that currently regulate the practice of geology in California and Oregon.

The proposed regulations for the registration of geologists in Washington have historically encountered opposition from a variety of groups. Certain civil engineers objected to the proposed act because of a perceived limitation on the scope of engineering services. Some geologists whose principle activity is in academics objected because of perceived restriction in their ability to provide geotechnical consulting services outside of normal teaching and/or research work. Some mining engineers and mining industry representatives objected because they were already subject to regulation and licensing under the State Professional Engineering Registration Act. Such objections have resulted in Washington being the only state on the west coast not providing assurance to public safety by the licensing of the geologic profession.

EFFECTS OF REGISTRATION

Where geologists and engineering geologists have been registered, licensed, or otherwise certified by state boards for a number of years, certain impacts on public safety and the profession have been the result of regulation. Codes and regulations requiring the expertise of the geologic professions have increased in number. Education and the general quality of geotechnical work have improved. Geologic hazards are more often identified and avoided or mitigated. Geologists have been elevated in stature in the eyes of courts, public officials, the general public, and members of other professions. In addition, the improved professional stature and importance of registered geologists have resulted in increased pay levels and professional responsibility, including professional liability (Slosson, 1984). Regulation has thus helped both the public and the geological profession.

Some local building and grading codes within Washington State require the services of an engineering geologist. For example, in Seattle there currently is a requirement for a geotechnical study prior to development of sites in areas identified as having potential landslide problems. However, there is no clearly documented mechanism for identifying or establishing qualifications for the individuals responsible for such work. A statewide geologist registration bill would establish criteria for qualification, which could be applied in Seattle, Mercer Island, or any other city where local codes may require services of engineering geologists.

ENGINEERING GEOLOGY AND LITIGATION

Analysis of ground conditions by engineering geologists requires interpretation of limited data and their extrapolation over extended areas. For example, samples from a core hole that is only 1 or 2 in. in diameter may be the basis for identifying subsurface conditions over a large area. These interpretations are subsequently used for planning earthwork-related development. If during construction, one party asserts that ground conditions differ from those anticipated, claims may arise and litigation may follow.

Similarly, if there are unmet expectations, claims and lawsuits are initiated. Unmet expectations have been related to: the desire of an owner for unattainable levels of perfection in the completed project; the discovery that construction and operational costs may be higher than anticipated; expected profits not being achieved; or the client simply trying to avoid paying the bill. Such projects with which the author is familiar include:

- A sewer district that demanded near perfection on pipe grades while not being willing to pay for adequate foundation material through soft-ground areas.
- A marina owner who claimed damages when faced with construction cost overruns.
- An earthmoving subcontractor who underbid a contract and tried to make a profit by claiming extras and changed conditions.
- A few developers who unfortunately give a bad name to an entire group by filing suit against the design team when expenses begin to rise.

Because such conditions develop, the possibility of claims for additional compensation that sometimes results in litigation can exist on any project, regardless of the accuracy of the geologist's report and especially if any party's expectations are not met.

Because geologists are not licensed, registered, or otherwise certified by the State of Washington, it may be more difficult to establish professional credibility in a courtroom in this state than in the neighboring states of Oregon and Idaho. The approach some attorneys take to erode the credibility of an engineering geologist is to contend that the problem could be solved by engineering analysis alone, and therefore, as an unlicensed person, a geologist should not be qualified as an expert witness. Because of these conditions, when an engineering geologist is called upon to take part in litigation, he or she should do the following:

First, the geologist must maintain clear communication with his/her client concerning the capabilities and limitations of an engineering geologist's field of expertise. If engaged as an expert witness to evaluate a claim or loss, he or she should explain to the client and the attorney the difference between a geologist's field of expertise and a geotechnical or civil engineer's area of specialization.

Second, the geologist must learn as many of the facts of the case as possible. This may involve report reviews, site inspections, subsurface exploration, and assessment of statements made by other parties relative to the problem. The entire process, including preparation, deposition, testimony, courtroom procedures, ethics, and suggestions for witness behavior, have been covered in detail by Waggoner (1981), Dovas (1963), Kiersch (1969), and Grainey (1981).

Third, and most importantly, if the geologist believes that he or she is qualified to serve as an expert witness, it will be necessary to demonstrate to the court that, on the basis of specialized training, education, and experience, the geologist is the expert in the particular field in question. Then, even if the opposition attorney attempts to question the geologist's credibility, he or she may be secure in the knowledge that he or she is the expert.

LEVELS OF LIABILITY

The amount of exposure an engineering geologist has of being named in a lawsuit is related to the nature of his or her practice and the type of employer he or she is affiliated with. For example, a single consultant who specializes in residential development work, especially hillside condominium projects, likely has a greater level of exposure to claims than a corporate engineering geologist who specializes in industrial facilities. As an independent consultant working alone, an engineering geologist can be held personally liable for any negligence claims where he or she is found to be responsible. Thus the personal level of exposure is quite high, and this liability can be expected to follow the engineering geologist even into retirement. By entering a partnership, it is possible to reduce individual risk levels because partners are usually held liable for each other's negligence. Participation in a partnership can, however, carry additional risks as well as benefits because one partner's error can affect all of the partners.

An engineering geologist, like any professional, is normally accountable for his or her own work and negligent acts. However, an engineering geologist, employed by a corporation or government agency typically is protected by the corporation or agency itself. Also, when there is a claim, a prudent attorney would logically be expected to seek redress from the party with the greatest ability to pay—the employer rather than the employee.

It is possible to try to shield oneself with insurance. However, insurance can be a mixed blessing. Although insurance can protect a person from catastrophic losses, insurance may not always be available. In addition, it is expensive; deductible amounts which the insured must pay can be large; and there may be exclusions and limitations on the coverage. Finally, an engineering geologist who has insurance may appear to a plaintiff to be a larger or potentially more tempting target than an uninsured individual.

MISCELLANEOUS HEARINGS

In order to protect life and property, governmental agencies at the state, county, and local level have the duty and responsibility to enact appropriate rules and regulations. These include zoning or grading codes, shoreline and wetland protection ordinances, rules governing operation of refuse disposal sites, regulations governing the drilling of wells, and a multitude of other matters. Frequently, engineering geologists are called upon to assist in the development of these regulations. A prime example is the set of rules developed by the City of Seattle Department of Construction and Land Use regulating development on steep slopes in the city. The original Directors Rule was written by the city staff assisted by a committee of geotechnical engineers and engineering geologists. After 3 years, the original requirements were updated, permitting what appears to be greater flexibility in the application of the regulations. The successful result of this type of approach is a credit to the cooperation between consultants and the city staff. The city staff gained a clearer understanding of the geotechnical community concerns, and the engineering geologists were able to more clearly appreciate the basis of the city's requirements.

In other areas, however, information developed by engineering geologists has not been fully considered in the formulation of regulations, such as some local wetland preservation ordinances. One local community planning department, ignoring information presented by biologists and engineering geologists, based its regulations on the premise that wetlands could not be constructed, improved, or relocated. When engineering geologists indicated that some of the identified wetlands slated for protection were manmade, their input appeared to be discounted. Later, when a botanist presented photos of a manmade wetland intentionally created as a wildlife habitat, his work was applauded but then ignored, in part perhaps because botanists, like engineering geologists, are not licensed professionals. Although it may not always be possible to influence regulations, professional status could help.

PROGNOSIS

Presently there are several obstacles to professional licensing of geologists and engineering geologists in Washington. First and foremost is the hesitation on the part of the Washington legislature to impose another board or agency on the public. Washington has therefore established a "Sunrise" statute (State of Washington, 1982), which, although binding on the medical profession, would likely apply to other occupations. Prior to establishing a new licensed profession or level of licensing, it would be necessary to show that current registration procedures and tort laws are insufficient to protect the general public.

There is, however, a sequence of events that may lead to professional registration of geologists. As population growth occurs around the state, more and more land subject to geologic hazards is being developed. The greatest hazard, traditionally, has been the risk of landsliding. However, slide-prone areas commonly also command extensive views and, if developed, typically are sold as premium residential property. Thus, eventually a situation may develop where excessive damage to public and private property occurs as a result of inadequate understanding of geologic hazards. This could result in the demand by the public for better regulatory codes and regulation of geological practitioners. If and when this occurs, geologists should be prepared to assist in the formulation of the appropriate regulations.

In the areas of liability and litigation, the future appears clouded. Geotechnical reports today generally appear to be superior in quality to work produced in the past. Most consultants are aware of liability problems and take steps to limit exposure. However, there appear to be increasing numbers of lawsuits based upon unmet expectations rather than actually negligent work or error. The best cure for problems of this type probably is related to a program of better communication and education. If owners, developers, and contractors learn the hidden costs of litigation—higher fees, stringent design criteria, costly bonding requirements, and additional regulations—some level of balance may ultimately be emplaced.

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Mats Mats Quarry near Port Ludlow. This quarry has furnished rock for many breakwaters on Puget Sound. Photograph by R. W. Galster, August 1980.

Part II: Engineering Geology Case Histories

Dams of Western Washington

Howard A. Coombs, Richard W. Galster, and William S. Bliton, Chapter Editors

Dams of Western Washington: Introduction — Early Projects

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INTRODUCTION

Within the 19 counties that comprise western Washington may be found approximately 500 dams of various shapes and sizes and constructed for a variety of purposes (Washington Department of Ecology, 1981). The dams range from minor 10-ft-high embankment structures to large concrete structures several hundred feet high. Most are on-stream structures constructed for hydropower generation, flood control, or water supply, and some are multi-purpose. Others are water-supply holding reservoirs, recreation facilities, or waste impoundments, many of which are off-stream facilities.

Major dam construction in western Washington occurred during two periods. The first was essentially during the economic boom of the second and third decades of the 20th century. During this period, 8 of the 24 major dams (those more than 100 ft high) were built (Table 1, Figure 1), all employing the concrete technology of the day. Most of these structures tapped the hydropower resources of the western Cascades and eastern and northern Olympics. Several water-supply projects were also constructed during this period.

A hiatus of dam construction occurred during the depression years of the 1930s. However, the construction of Mud Mountain Dam began late in this period, partly as a "make work" project, using early methods of embankment design and construction. This was followed by completion of Alder and La Grande dams on an expedited basis during World War II.

The post-World War II economic and population boom of the metropolitan areas of western Washington and northwestern Oregon resulted in construction of 13 major dams during the 25-yr period between the late 1940s and the early 1970s. Although most of these projects count hydropower as their major purpose, water supply, water conservation, and flood control requirements also have provided the impetus for major dam construction. In addition to improved post-war concrete technology, development of larger earth-moving equipment and advances in the science and art of soil mechanics and embankment design permitted economical construction of some of these projects as embankment dams.

The two basic periods of construction provide an interesting contrast in both dam design and construction methods.

EARLY PROJECTS

Advent of Hydropower

Prior to the onset of major dam construction, pioneering efforts in development of hydropower were made in western Washington during the final decade of the 19th century and the first decade of the 20th century. Electric lighting began in Seattle in 1886, followed by the onset of street car electrification 3 yr later, the year Washington became a state (Wing et al., 1987). A number of independent electric companies in the Seattle, Tacoma, and Bellingham areas were formed to serve limited territories, all obtaining power supplies using coal-fired steam generating plants. Most went into bankruptcy following the financial panic of 1893, temporarily stalling the young electric utility industry (Wing et al., 1987).

The birth of hydropower in western Washington must truly be placed in the hands of three men: Charles Hinkley Baker, a Seattle civil engineer with railway experience, and the combine of Charles A. Stone and Edwin S. Webster, electrical engineers who formed their partnership in Boston the year of Washington statehood. Baker generated the idea of the Snoqualmie Falls power development and proceeded to construct it. Stone and Webster consolidated many of the local bankrupt power companies about the turn of the century into a predecessor of the Puget Sound Power and Light Company (PSP&L) (Wing et al., 1987). During the first decade of the 20th century they searched out and developed several innovative hydropower projects, all of which continue to be in use.

Snoqualmie Falls Project

The late Pleistocene glacial diversion of the Snoqualmie River southward from its former channel onto an extensive ledge of Tertiary basalt set the scene for the world's first fully underground hydropower plant. The site was an obvious location for hydropower development, having a natural 268-ft vertical drop in the river

Table 1. Major onstream dams of western Washington

Dam type: CG, concrete gravity; E, embankment; GA, gravity arch; CM, cyclopean masonry

Owners: COE, U.S. Army Corps of Engineeers; PUD, Public Utility District of the named county; PSP&L, Puget Sound Power and Light Co.; PP&L, Pacific Power and Light Co.; CZ, Crown Zellerbach

Dam	River	Type	Year completed	Structure height (ft)	Crest length (ft)	Maximum reservoir capacity (x 1000 acre-ft)	Foundation	Engineer	Owner
Gorge	Skagit	GA	1961	285	670	10.8	gneiss	J.L. Savage	Seattle
Diablo	Skagit	Arch	1929	386	1,180	91.6	gneiss	Constant-Angle Arch	Seattle
Ross	Skagit	Arch	1949	540	1,300	1,633.4	gneiss	J.L. Savage	Seattle
Lower Baker	Baker	Arch	1925	290	570	136.9	limestone	Stone & Webster	PSP&L
Upper Baker	Baker	CG	1961	332	1,220	316.1	phyllite	Stone & Webster	PSP&L
Sultan (Culmback)	Sultan	E	1965	203	220	48	phyllite/ argillite	R.W. Beck	Snohomish PUD
Tolt River	S. Fork Tolt	E	1962	213	980	67.2	argillite/ andesite	Carey & Kramer	Seattle
Masonry	Cedar	CM	1914	225	980	175	andesite/ basalt	Seattle	Seattle
Hanson	Green	E	1962	235	500	106	andesite/ vol. breccia	COE	COE
Mud Mountain	White	E	1948	425	700	106	agglomerate/ mud flow	COE	COE
La Grande	Nisqually	CG	1945	217	710	3.1	andesite	Tacoma	Tacoma
Alder	Nisqually	GA	1945	330	1,600	244	andesite	Tacoma	Tacoma
Skookumchuck	Skookum-chuck	E	1970	195	1,340	35	basalt	Bechtel Inc.	PP&L

Table 1. Major onstream dams of western Washington, continued

Dam	River	Type	Year completed	Structure height (ft)	Crest length (ft)	Maximum reservoir capacity (x 1000 acre-ft)	Foundation	Engineer	Owner
Mayfield	Cowlitz	GA	1963	250	850	184.2	basalt/ andesite	Harza Engineering	Tacoma
Mossyrock	Cowlitz	Arch	1968	606	1,648	1,714.6	basalt	Harza Engineering	Tacoma
Merwin (Ariel)	Lewis	Arch	1931	313	1,250	423	basalt/ andesite	Ebasco	PP&L
Yale	Lewis	E	1953	323	1,550	402	lava, tuff flow breccia	Ebasco	PP&L
Swift	Lewis	E	1958	610	2,100	766	lava, tuff, mud flow	Bechtel Inc.	PP&L
Cushman #1	Skokomish (N. Fk.)	Arch/E	1926	275	1,111	478	basalt	Tacoma	Tacoma
Cushman #2	Skokomish (N. Fk.)	Arch	1930	235	460	8.7	basalt	Tacoma	Tacoma
Wynoochee	Wynoochee	CG/E	1973	175	1,755	76	basalt	COE	COE
Elwha	Elwha	CG	1912	160	490	8.1	gravel/con- glomerate	N.W. Power	CZ
Glines Canyon	Elwha	GA/E	1927	210	510	39.1	basalt, flow breccia, argillite	Thebo Starr, Anderson	CZ
Casad	Union	Arch	1956	202	400	4.4	basalt	J. Cunningham	Bremerton

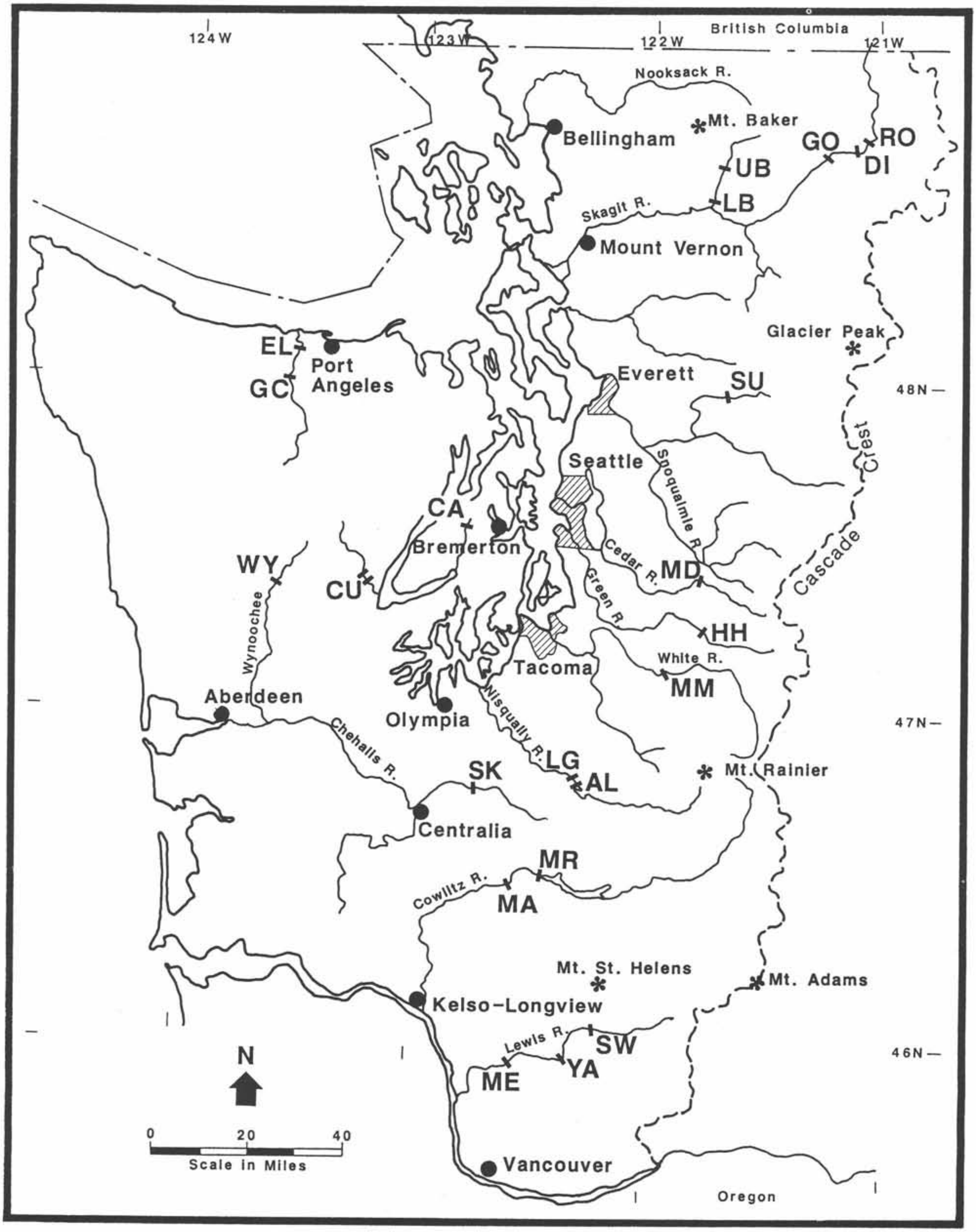

Figure 1. Major dams in western Washington. AL, Alder; CA, Casad; CU, Cushman 1 and 2; DI, Diablo; EL, Elwha; GC, Glines Canyon; GO, Gorge; HH, Howard Hanson; LB, Lower Baker; LG, La Grande; MA, Mayfield; MD, Masonry (Cedar); ME, Merwin; MM, Mud Mountain; MR, Mossyrock; RO, Ross; SK, Skookumchuck; SU, Sultan (George Culmback); SW, Swift; UB, Upper Baker; WY, Wynoochee; YA, Yale.

channel. The plunge pool at the base of the falls is 65 ft deep (PSP&L, unpublished data), and the falls have retreated about 1,000 ft since the late Pleistocene deglaciation (Galster and Olmsted, 1977).

The original project (Figure 2) consists of a concrete diversion dam 10 ft high, 16 ft wide, and 220 ft long just upstream from the lip of the falls. A 270-ft vertical penstock 300 ft behind the falls conducts diverted water into generating units in an underground powerhouse "cavity". Discharge is via a 650-ft-long tailrace tunnel. The powerhouse cavity and tailrace tunnel are unlined (Engineering News, 1900).

The Snoqualmie Falls Power Co. was organized early in 1898 with Charles H. Baker as president. With a small cadre of engineers and hired labor, construction was begun and pursued day and night until completion. The diversion dam was constructed first, followed by simultaneous excavation of the shaft from the top and the 12-ft-wide by 24-ft-high tailrace tunnel from a heading on the left bank adjacent to the foot of the falls. This was followed by enlargement of the underground chamber to its present dimensions: 200 ft long, 40 ft wide, and 30 ft high. The purpose of this plan was to protect the generating equipment from spray from the falls (Engineering News, 1900). The basalt is so massive that it has stood unsupported, unreinforced, and unlined for nearly a century. No ground water was encountered either during the original construction or subsequently. The bonding of the concrete diversion dam to the basalt

Figure 2. Aerial view of Snoqualmie Falls probably taken during the early 1930s. The tailrace tunnel for the underground powerhouse is at the edge of the spray to the right of the falls. The headworks for the underground powerhouse lies at the center of the photograph, at the cross ticks. The surface powerhouse, penstock, and gate house are at the lower center. Subsequent to this photograph, the surface facilities were expanded to include an additional penstock and powerhouse expansion. Photo courtesy of Puget Sound Power and Light Co.

bedrock foundation was augmented by additional blasting to roughen the bedrock surface and installation of steel rails 2 ft into the foundation prior to placement of concrete (Engineering News, 1900).

In 1910 a remote surface powerhouse was built, on the right bank of the river at a distance sufficiently downstream from the falls to eliminate spray problems. This facility is served by a 12-ft-diameter water tunnel to a surface gate house and thence by surface penstocks to the powerhouse. In 1957 the project was further modified by addition to the surface powerhouse.

Electron Project

In 1903 the Puget Sound Power Company was organized by Stone and Webster to construct the Electron Project on the Puyallup River in Pierce County. The project consists of a low wooden diversion wier and a 10-mi-long wooden flume conducting water to a four-unit powerhouse founded on bedrock. The project went into service in 1904, and the flume was rehabilitated in 1984-1985 (Wing et al., 1987).

Chester Morse Lake

Between 1902 and 1904 the City of Seattle, wishing to augment storage for its Cedar River water supply diversion at Landsburg, constructed a rock-filled timber crib dam at the outlet to Cedar Lake on the Cedar River in eastern King County. The project raised the level of Cedar Lake about 13 ft to store additional water. The dam was founded on fine-grained glacial outwash and lacustrine sediments near the up-valley end of a Pleistocene delta moraine deposited by the Puget lobe of the continental ice sheet in an up-valley direction (Mackin, 1941). The fine-grained sediments are intercalated with pervious sand and gravel and extend to an unknown depth. The dam was partly washed out in 1911 and rebuilt 5 ft higher to elevation 1,546 ft (U.S. Army Corps of Engineers, 1979). Following construction of Masonry Dam 1.5 mi downstream in 1914, the crib dam was frequently inundated by the Masonry pool. In 1986 the crib dam was replaced by a roller-compacted concrete dam.

Nooksack Project

Utilizing the 173-ft vertical drop of Nooksack Falls on the North Fork, Nooksack River in Whatcom County, the project was completed by the Whatcom County Railway and Light Company in 1906 (Wing et al., 1987). The project consists of a 9-ft-high wood crib diversion dam, 72 ft long, with concrete abutments; it is situated 1,130 ft upstream from the lip of the falls. A 1,025-ft-long unlined tunnel was constructed on the north (right) bank to a surface penstock that feeds a single unit powerhouse sited along the river bank 1,200 ft downstream from the toe of the falls. The tunnel is 7 to 8.5 ft wide and 8 to 10 ft high and was driven through andesite of the Wells Creek Volcanics on which the lip of the falls is developed. Shear zones intersecting the tunnel have permitted local stoping to as much as 30 ft into the crown. Otherwise, the walls of the unlined tunnel are seen to be in good condition 80 years after excavation. The powerhouse is apparently founded on glacial drift (till or outwash). A new powerhouse to be founded on glacial till is being planned (Kearnes, 1988).

White River Project

An innovative hydropower project involving diversion of the White River was planned by Stone and Webster and constructed by their Pacific Coast Power Company (later to become part of Puget Sound Power and Light Co.) between 1910 and 1914. The project, an off-stream facility, diverts a portion of the White River at Buckley via an 8-mi-long flume and open, partly lined channel into Lake Tapps, a many-fingered lake on the glacial outwash upland east of the Green River/Puyallup River valley. Fourteen low embankment dams were constructed in order to raise the lake level about 28 ft to a maximum elevation of 543 ft. Embankment dams range in height from 6 to 45 ft (CH2M Hill, Inc., 1981). The embankments were constructed using sand, gravel, and silt, dumped from rail cars on wooden trestles that were incorporated into the embankments. Apparently only water compaction was employed. The powerhouse is situated on the east side of the Green/Puyallup valley at Dieringer south of Auburn and is serviced via cut-and-cover and surface penstocks from Lake Tapps. About 480 ft of head is developed. All structures are founded on glacial drift except the wooden diversion weir across the White River, which is founded on coarse river alluvium.

Because the White River heads in the glaciers on the northeast side of Mount Rainer, the stream carries a heavy suspended load of rock flour, much of which is diverted into Lake Tapps. This gives the lake a turbid, blue-gray color, and when it is drawn down annually to its original level, rock flour is seen to coat all features below maximum lake level. Removal of rock flour build-up in the diversion channel is also periodically required.

First Nisqually Project

The short, deep canyon resulting from Pleistocene glacial diversion of the Nisqually River became the scene for the City of Tacoma's initial effort into hydropower development. Between 1910 and 1912, the first Nisqually Project was constructed utilizing the 300 ft of head available in the 2.5-mi-long Nisqually canyon between Alder and La Grande. A low diversion dam was constructed at the head of the canyon just upstream from the present Alder Dam, and a tunnel was driven through andesite bedrock to conduct water to a powerhouse near the canyon mouth at La Grande (Bretz, 1913; City of Tacoma, undated).

ADVENT OF MAJOR DAMS

The river basins of western Washington are generally subject to heavy runoff during winter storms and a relatively rapid runoff during the late spring/early summer snow melt. Thus, these early run-of-the-river hydropower plants frequently lacked a consistent volume of water to operate during dry periods. The need for a consistent source of water for hydroelectric generation and industrial/domestic supply required the construction of larger structures in the deep mountain valleys to provide such storage.

Interestingly, the first major dam constructed in western Washington was not near the major population centers on the eastern shore of Puget Sound, but rather on the northern Olympic Peninsula. Here, the Elwha Dam was constructed to provide both power and water for the pulp industry. During the same period (1910-1914) the City of Seattle constructed Masonry Dam on the Cedar River downstream from the crib dam which raised the level of Cedar Lake. The geologic problems relating to these two projects are chronicled in subsequent discussions.

In April 1924 Stone & Webster Engineering Corp. began construction of the Lower Baker Dam. W. D. Shannon (father of Shannon & Wilson's W. L. Shannon) was project engineer. During the next 7 yr, the Cushman project was built on the Skokomish River in the southeastern Olympic Mountains by the City of Tacoma; a second dam was built on the Elwha River; the City of Seattle completed Diablo Dam (the first of the major Skagit River dams); and Pacific Power and Light completed Merwin (the first of the Lewis River dams). These structures furnished the hydropower that carried western Washington through the depression years and, with the assistance of Bonneville intertie to Bonneville, Grand Coulee, and Rock Island dams, carried the industrial area of the Puget Sound Basin and Vancouver-Portland area through the years of World War II.

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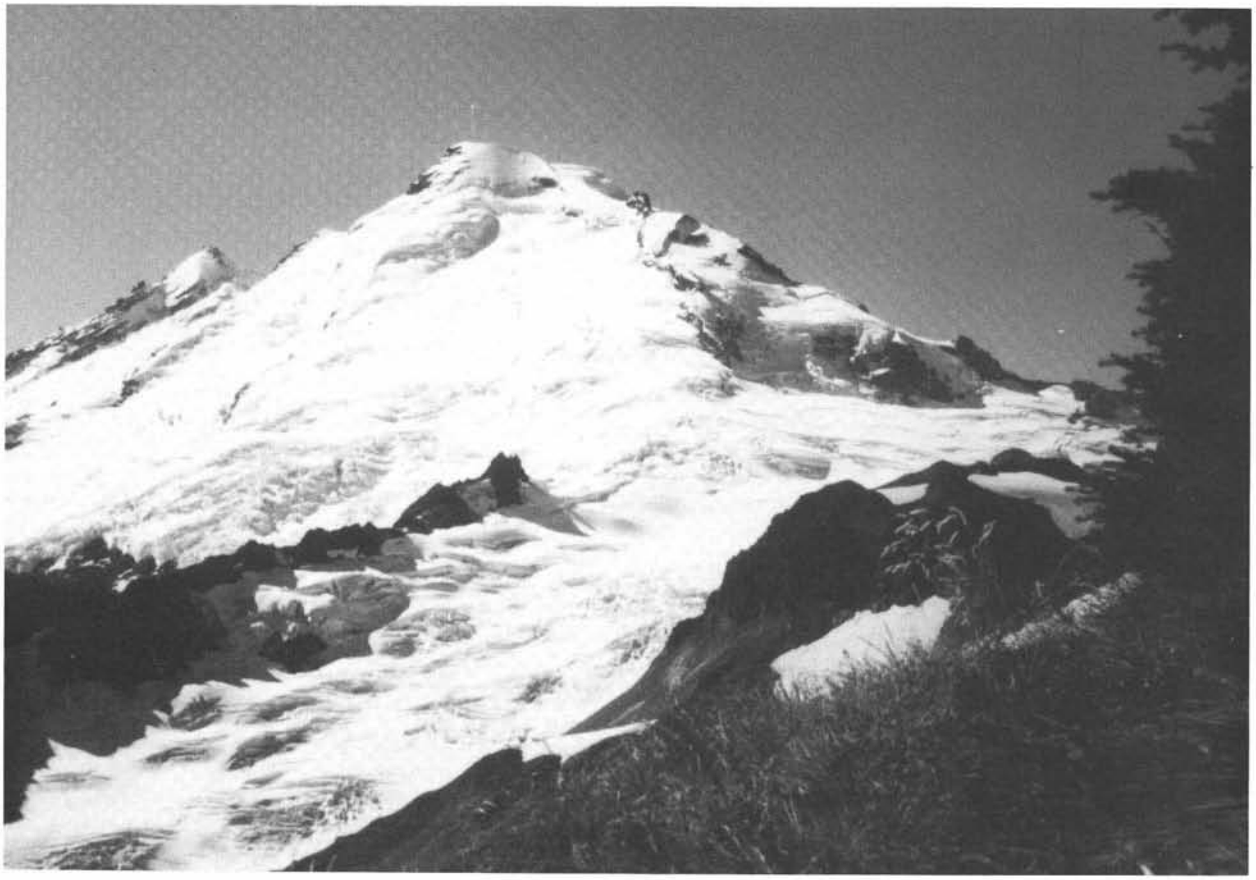
Aerial view to the southeast of the Nisqually River canyon showing the use of the canyon for damsites. La Grande Dam is at the lower left center, Alder Dam is in the upper center, with Alder Lake beyond. The original development consisted of a small diversion weir just upstream from the present Alder Dam and a tunnel that conducted water behind the south bank (right) to a powerhouse downstream of the present La Grande Dam. The present dams were constructed in the 1940s. Photograph by R. W. Galster, July 1980.

The Baker Project



East face of Mount Baker, headwaters of the Baker River. Sherman Peak is left of the main summit. Park Glacier dominates the upper slope, Rainbow Glacier the lower slope. Photograph by R. W. Galster, September 1985.

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The Baker Project

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INTRODUCTION

The Baker River originates in the North Cascades and flows southward until it meets the Skagit River at the town of Concrete. Puget Sound Power and Light Company constructed two dams on the Baker River to provide power for much of western Washington (excluding Snohomish County and the cities of Seattle and Tacoma). Lower Baker Dam is approximately 1 mi north of the confluence of these two rivers and forms a reservoir 7 mi long that extends to the toe of Upper Baker Dam.

AREAL GEOLOGY

The Baker River is just 25 mi west of the upper reaches of the Skagit River and Ross, Diablo, and Gorge Dams. In the space between the Baker and Skagit rivers the geology changes abruptly due to the presence of the Straight Creek fault (Figure 1). To add to the complexity, the Shuksan thrust of mid-Cretaceous age has juxtaposed one sequence of rocks on the west (the Shuksan Metamorphic Suite) over another sequence (the Chilliwack Group) to the east (Misch, 1966, 1979). Thus it is not surprising that the limestones, argillites, greenschists, and phyllites of the Chilliwack Group along the Baker River bear no resemblance to the-Shuksan Metamorphic Suite or to the gneisses and migmatites of the Skagit metamorphic suite in the upper Skagit valley.

With major fault zones so close to dams in the northern Cascades, the Federal Energy Regulatory Commission was concerned that renewed activity along such zones might generate earthquakes capable of damaging the dams. Hence it became necessary to evaluate the potential for renewed movement along these faults by searching for evidence of recent displacements (Miller and Vance, 1981). Studies of the Shuksan thrust provided no clues of renewed movement. However, the Straight Creek fault, as described by Vance (1957, 1977), was suspect since considerable displacement had occurred during the Cenozoic and activity might continue to the present. To the north the fault is cut by the Chilliwack composite batholith. Total possible dextral displacement along the fault zone is 120 mi.

In order to date movement of the fault, attention was directed to the Chilliwack batholith (Misch, 1979). The batholith is not accessible by road nor even by helicopter because this part of the northern Cascades is set aside as a wilderness area. Vance (1977) made a backpacking trip into the area to examine the geology and concluded that major movement along the Straight Creek fault ended 38 Ma on the basis of dates from samples from some of the intrusive rocks.

In order to determine if movement had occurred at the edge of the batholith, a diligent search was made for marginal faults by ERTEC Northwest, Inc. (1981). They found that a southern projection of the batholith was bounded by a fault on the west side. Stratigraphic studies of slope sediments covering parts of the fault showed no evidence of movement during the last 6,600 yr. This conclusion is based, in part, on the presence of undisturbed Mazama tephra in the sediments overlying the fault.

A series of scarps, suggesting a major fault zone, is present south of the Chilliwack batholith. At least 19 scarps have been mapped, and many others may be present but are covered by vegetation. Individual scarps may be as long as 7 mi, but most are less than a half-mile in length. Early studies in the area suggested recent tectonic movement. The name "uphill-facing scarps" has been given to these features. Characteristically, the scarps occur near the tops of ridges along steep-sided valleys. However, such scarps are not unique to the upper Baker River region. They have been described in many mountainous areas of the world, and in Europe they have been referred to as "sachung" according to Radbruch-Hall et al. (1976). In order to date movement on these scarps, tephrochronology was used again to assess the age of undisturbed sediments draped across the scarps. Ash falls from Mount Baker and Mount Mazama in Oregon have been dated, and the oldest of these undisturbed layers in the region contains the 6,600-yr-old Mazama ash. There is no evidence in the upper Baker region that the uphill-facing scarps are high-angle faults.

UPPER BAKER DAM

Project Description

Upper Baker Dam is a straight concrete gravity structure with a length of just over 1,200 ft and a height of approximately 300 ft (Figures 2 and 3). The reservoir behind Upper Baker Dam and the associated West Pass

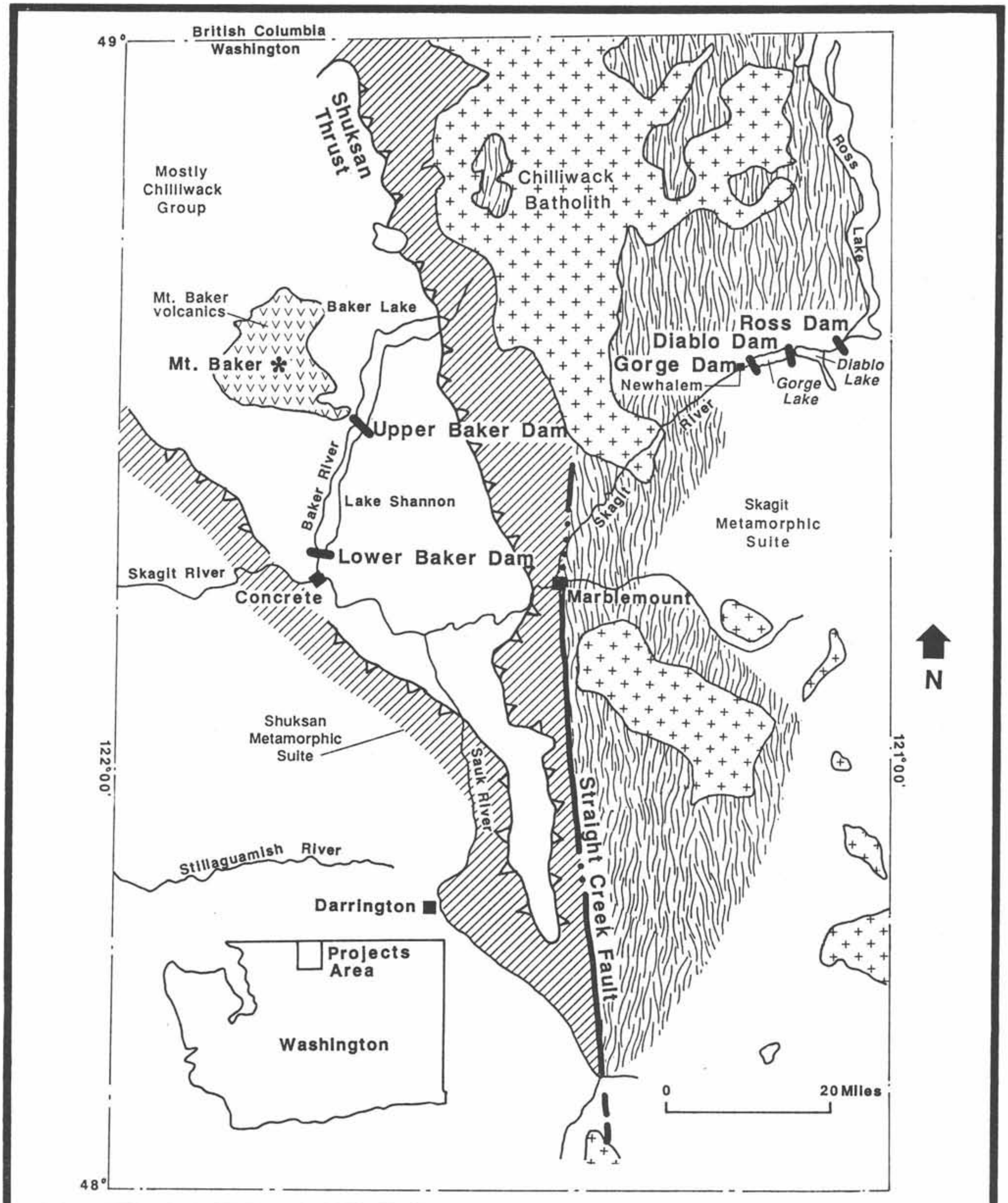

Figure 1. Location and geologic map of the Baker and Skagit projects. The Straight Creek fault juxtaposes contrasting lithologies. The Skagit metamorphic suite, in the area of the Skagit dams, is essentially quartz diorite gneiss. In contrast, the Shuksan metamorphic suite is composed of greenschists and phyllites. The Chilliwack Group below the Shuksan thrust contains slates, phyllites, and metavolcanic rocks of late Paleozoic age. The Eocene to Miocene composite Chilliwack batholith ranges from quartz diorite to granodiorite in composition.

Figure 2. Upper Baker Dam and powerhouse. Photo courtesy of Puget Sound Power and Light Co.

Dike is known as Baker Lake; it is 10 mi long and covers a maximum of 3,000 acres. The construction of Upper Baker Dam was completed in June 1959, and storage commenced in July of the same year. The dam was constructed in 5-ft lifts and with 25 monoliths of widths ranging from 25 to 59 ft. The contraction joints between monoliths were not grouted. A grout curtain is provided along a line of holes drilled on 10-ft centers at a location 3 ft downstream and parallel to the upstream face of the dam. The dam has an extensive series of drains under and within the structure.

Dam Site Geology

The geologic history of the Upper Baker Dam region is punctuated by both glacial and volcanic activity (Coombs, 1939; Frank et al., 1977; Hyde and Crandell, 1978). The ancestral Baker River, flowing on a pre-Cenozoic metamorphic terrane, was modified by alpine glaciers that left the valley sides and bottom covered with coarse glacial till. Still later, two flows from Mount Baker, totalling 350 ft in thickness, pushed the river out of its course and relocated it 2 mi to the east (Figure 4). Thus the Upper Baker Dam plus the postglacial lava flow form a barrier to retain water in the reservoir. The lavas appear to be solid; there are the usual shrinkage cracks plus some "ash". However, the fine ash is, in reality, comminuted clinkers along the tops and bottoms of the flows and some material rolled into the center of the flows as they advanced. The clinker and "ash" portions of the flows are quite porous.

The last major glacial event occurred when a lobe of continental ice in Canada moved southward, blocking the Skagit River and all its tributaries. As a result of this blockage, the Baker River and Sulphur Creek valleys were blanketed with varved lacustrine clays.

Sulphur Creek posed a potential drainage and landslide problem in that some of the flows in the valley sides were plastered with varved clays. Figure 5 illustrates the sequence of events that led to this situation (Stearns and Coombs, 1959). Figure 4 indicates the location of the exposed lavas in Sulphur Creek. As the

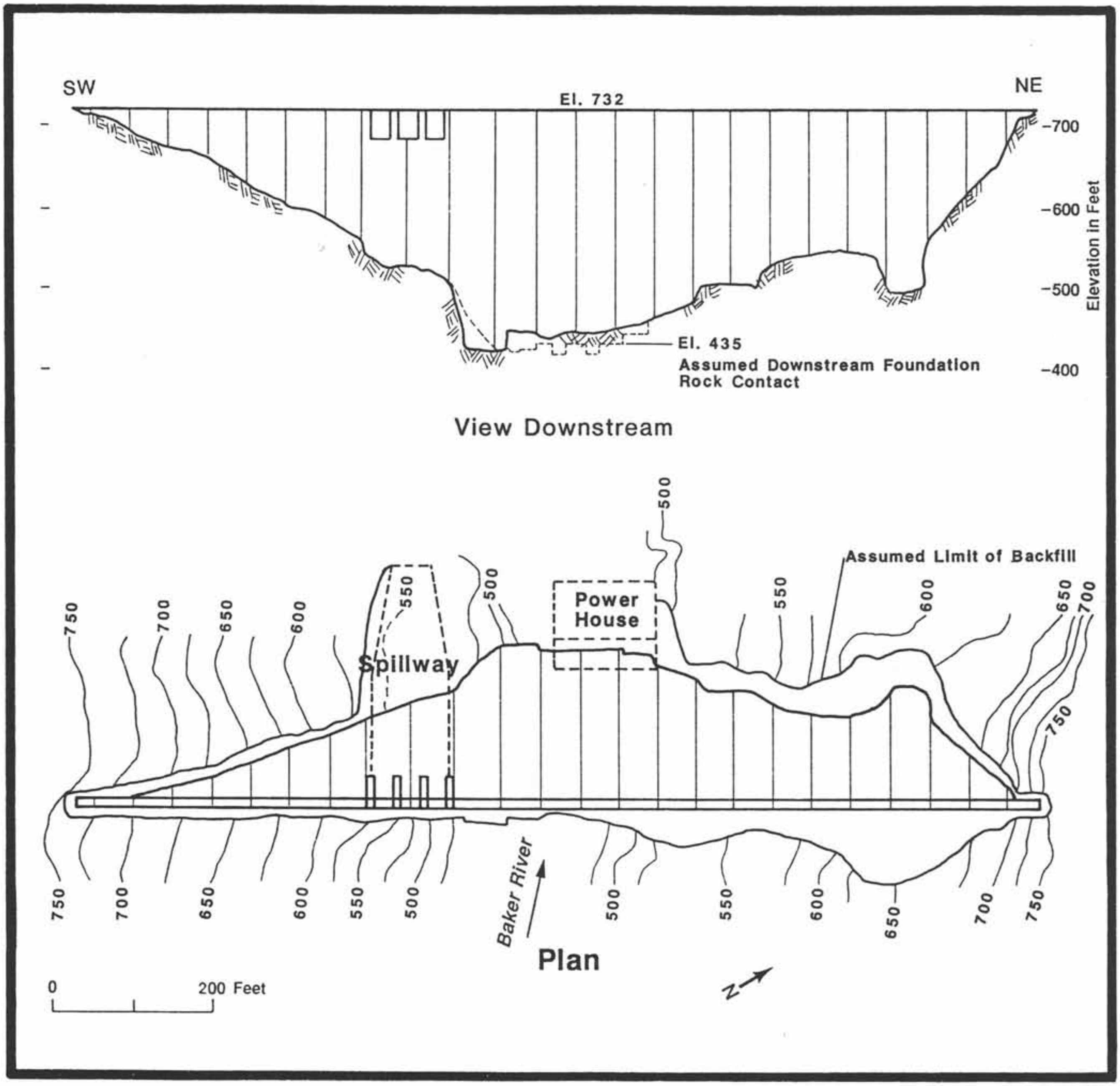

Figure 3. Plan and section, Upper Baker Dam.

water table is raised in the lava flows as a result of reservoir filling, pressure could build up behind the varved clays on the sides of Sulphur Creek. A sudden release of water from within the lavas could sweep down Sulphur Creek and back into the Baker River and flood the powerhouse at Upper Baker Dam. An adit (Figure 4) was driven through the clays and into the basalts along the creek to relieve water pressure.

Dam Foundation

Upper Baker Dam rests on phyllite and phyllitic siltstone on the left abutment and dolomitic hornfels to dolomitic marble and phyllite on the right abutment. There is no evidence of solution channels. The structure at the dam site is a homocline trending northwest and dipping 40° to the northeast. At the base of the right

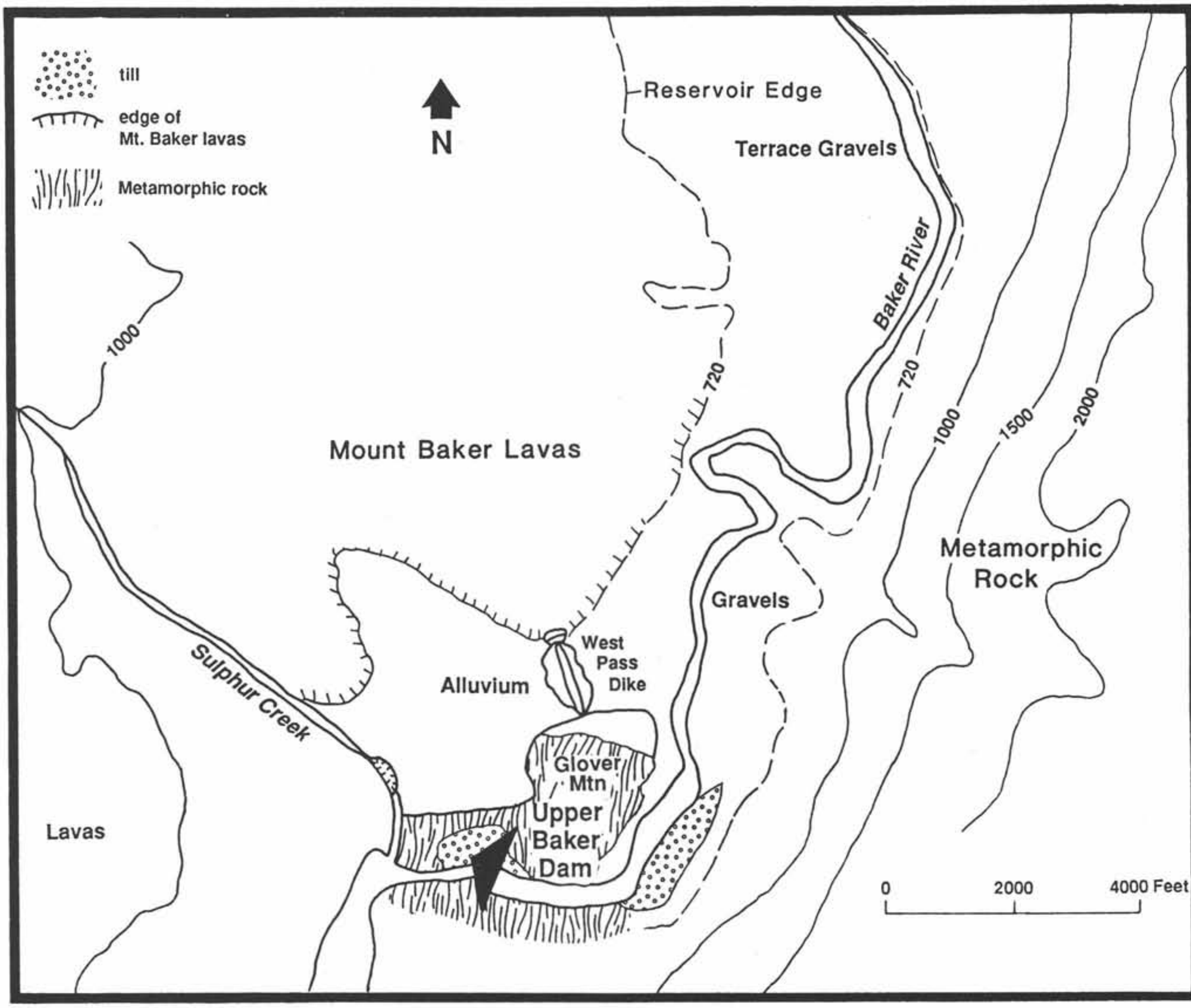

Figure 4. Generalized geologic map of the Upper Baker Dam area. To the west of the Baker River and the West Pass Dike are Mount Baker lavas and terrace gravels. A heavy, compacted till is found at the dam site and a mile upstream. The metamorphic rocks are phyllite, dolomitic hornfels, and dolomitic limestone.

abutment a heavy till containing boulders as much as 8 ft in diameter was so compact that blasting was necessary for removing it.

Joints in the bedrock are discontinuous and can be grouped into three distinct sets. The attitudes of the joints were plotted to determine if instabilities would result when the dam was constructed and reservoir filled. Joint orientations posed no problems. No discontinuities that would contribute to sliding are known to exist at depth in the foundation. The few small faults in the foundation area were easy to clean and backfill with concrete.

West Pass Dike

The West Pass Dike is located across a topographic saddle about 3,000 ft north of the Upper Baker Dam (Swiger, 1958, 1975) (Figure 4). It extends about 1,200 ft from the north slope of Glover Mountain to an unnamed bedrock (greenstone) knob, the top of which is above dike crest elevation of 734 ft. Before construction of the project, ground level in the saddle was about elevation 680 ft at the low point. This is 40 ft below full reservoir level (el. 724 ft) (Swiger, 1958, 1960).

Through the central part of the dike the foundation materials consisted of a thin cover of organic-rich materials underlain by 20 to 30 ft of sensitive, soft, organic silt. This in turn was underlain by about 30 ft of very sensitive, soft, blue inorganic clay. These glacial lake clays and silts rested directly on a very compact lodgement till and till on bedrock. At the south abutment the till extended up the north slope of a rock knob to well above the level of the dike crest. At the north abutment, the till was very thin or absent.

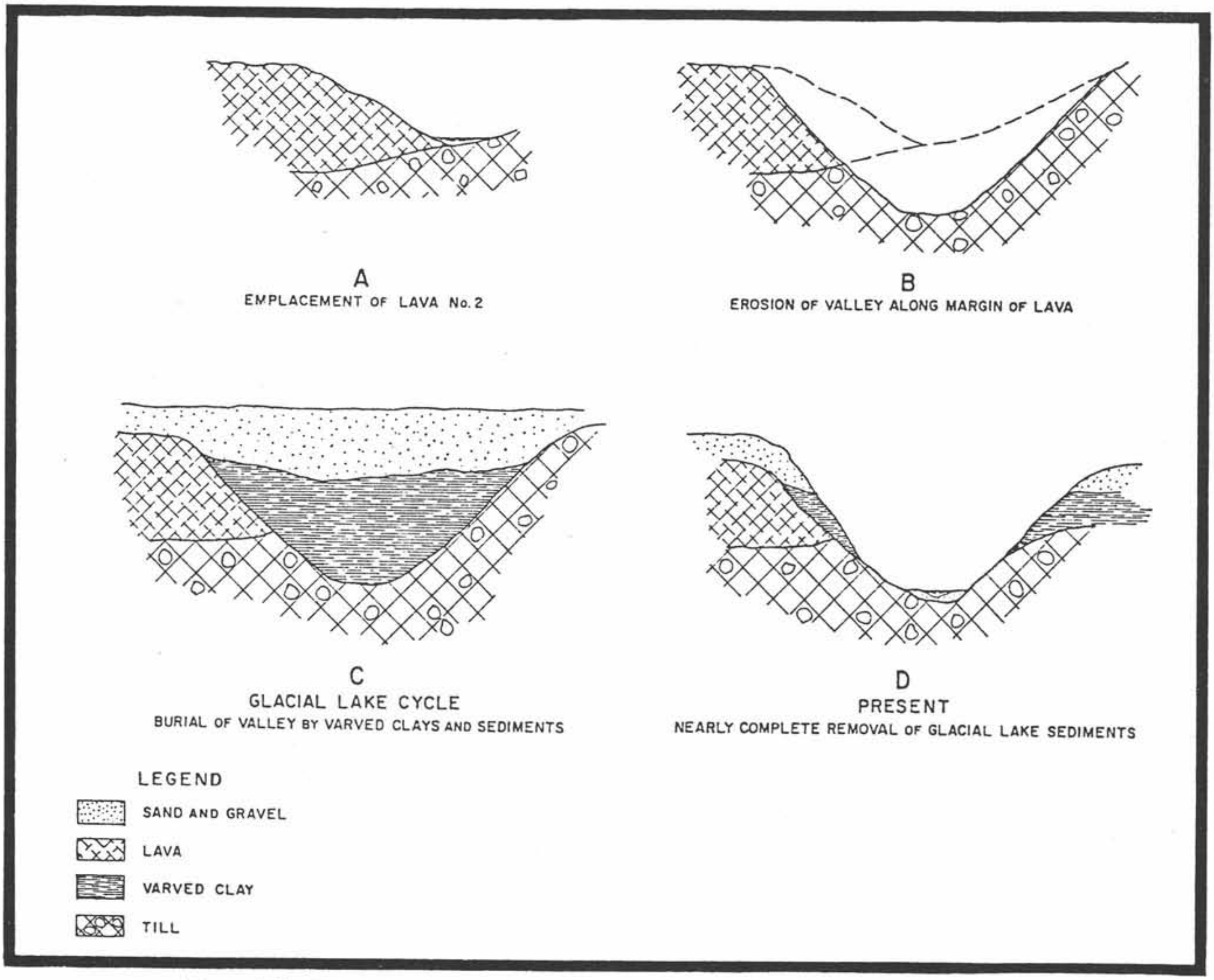

Figure 5. Sections across Sulphur Creek showing the development of the present geology.

The soft silts and clays were too weak and compressible to serve as foundation for the dike. Accordingly, they were completely excavated by a 6-in. cutter head hydraulic dredge. Shear strengths of the silts and clays were low, and stability analyses showed that sliding of these sediments would not permit dewatered excavation to the top of the till. As excavation proceeded, water level in the dredge pond was gradually lowered to a minimum level of approximately 30 ft above the top of the till, just within the operational limits of the dredge. No problems with slides were experienced.

The dike, which has a maximum height above its foundation of about 110 ft, was constructed as a sloping core, rockfill dam (Figure 6). Fill below the water level of the dredging pond was select, blocky lava. This was placed by dumping through the water. A filter zone of skip-placed sand and gravel was emplaced between the excavated clay slope on the upstream side and the dumped rock fill. Downstream excavation of the soft clays was carried beyond the limits of the dumped fill.

The downstream shell of the dike is built of compacted blocky lava. The core is of recompacted clayey till from a nearby pit. In places it was extremely dense and had a field moisture content below the optimum for placing. It was wetted in the pit and was again tempered by wetting, discing, and harrowing on the fill before compaction. There is a two-stage sand and gravel filter between the shell and the core. This filter extends upstream under the core. The core was carried upstream to contact with the *in situ* soft clay-silts. At this contact the two materials were knit together by careful rolling, and the juncture was then covered by a 4-ft-thick blanket of puddled clay. The upstream shell is of compacted blocky lava, select coarse material being placed on the face as dumped riprap. A sand and gravel filter to serve as a crack healer was placed between the core and the upstream shell.

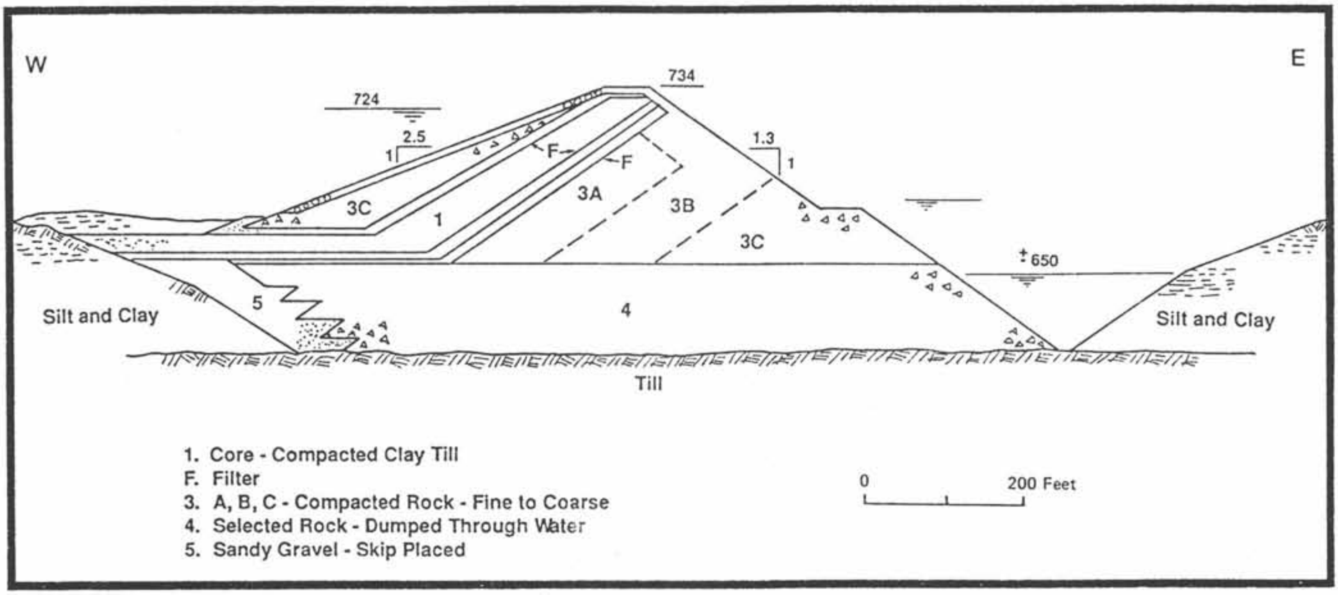

Figure 6. Section through the West Pass Dike. The section indicates the type of material in the dike and sediments on which the dike rests. Sketch by W. F. Swiger.

Volcanic Activity

On March 10, 1975, the crater of Mount Baker was shooting columns of steam thousands of feet in the air. New fumaroles were developed, along with the emission of ash and considerable quantities of sulfur and hydrogen sulfide (Hyde and Crandell, 1975). The U.S. Geological Survey, the University of Washington, and Puget Sound Power and Light Company cooperated in establishing a monitoring system including three seismometers and tiltmeters as well as areal surveillance. The U.S. Forest Service closed the southeast side of Mount Baker to the public for 1 yr because of volcanic hazards.

Because the Upper Baker Dam is located 10 mi southeast of the crater, there was concern that mudflows or floods entering the reservoir might set up waves or floods that could damage the dam. Puget Sound Power and Light Company agreed to lower the reservoir 21 ft so as to provide effective freeboard.

Spectacular steam activity continued until late 1976. As geophysical observations had not indicated abnormal activity and tiltmeters had not indicated inflation of the volcano, which generally suggests a potential eruption, the monitoring activity was called off. The volcano quieted down and has subsequently remained quiet.

Seismic Analysis

In 1982 a preliminary seismic analysis of the Baker dams was made by ERTEC Northwest Inc. This led to a comprehensive seismic analysis of the Upper Baker Dam by Stone & Webster Engineering Corp. (1984) to determine its behavior under earthquake loading. The assumed ground horizontal acceleration of 0.3 g was used for the dynamic analysis. The static effects considered in the three-dimensional analyses were gravity load, water load, and temperature effects.

Loads from the dam along the portion of the left abutment, where sliding could potentially occur were obtained directly from static and dynamic finite-element analyses. The sliding potential for low reservoir conditions was considered in detail because of the association with minimal concrete temperatures, which, in turn, reduce the normal stresses and frictional resistance between the dam and the abutment as well as between adjacent blocks.

The results of the sliding analyses along the left abutment, as well as at shallow depths below the contact with the dam, indicate the safety factor is adequate. Present data indicate there are no large-scale instabilities associated with the foundation. Should an earthquake greater than the one postulated occur, rockfalls or slides both upstream and downstream of the dam cannot be precluded.

LOWER BAKER DAM

Project Description

Lower Baker Dam (Figures 7 and 8) is a thick, concrete arch structure located 1 mi north of the town of Concrete. The dam is 285 ft high and impounds a 7-mi-long reservoir known as Lake Shannon (Figure 1).

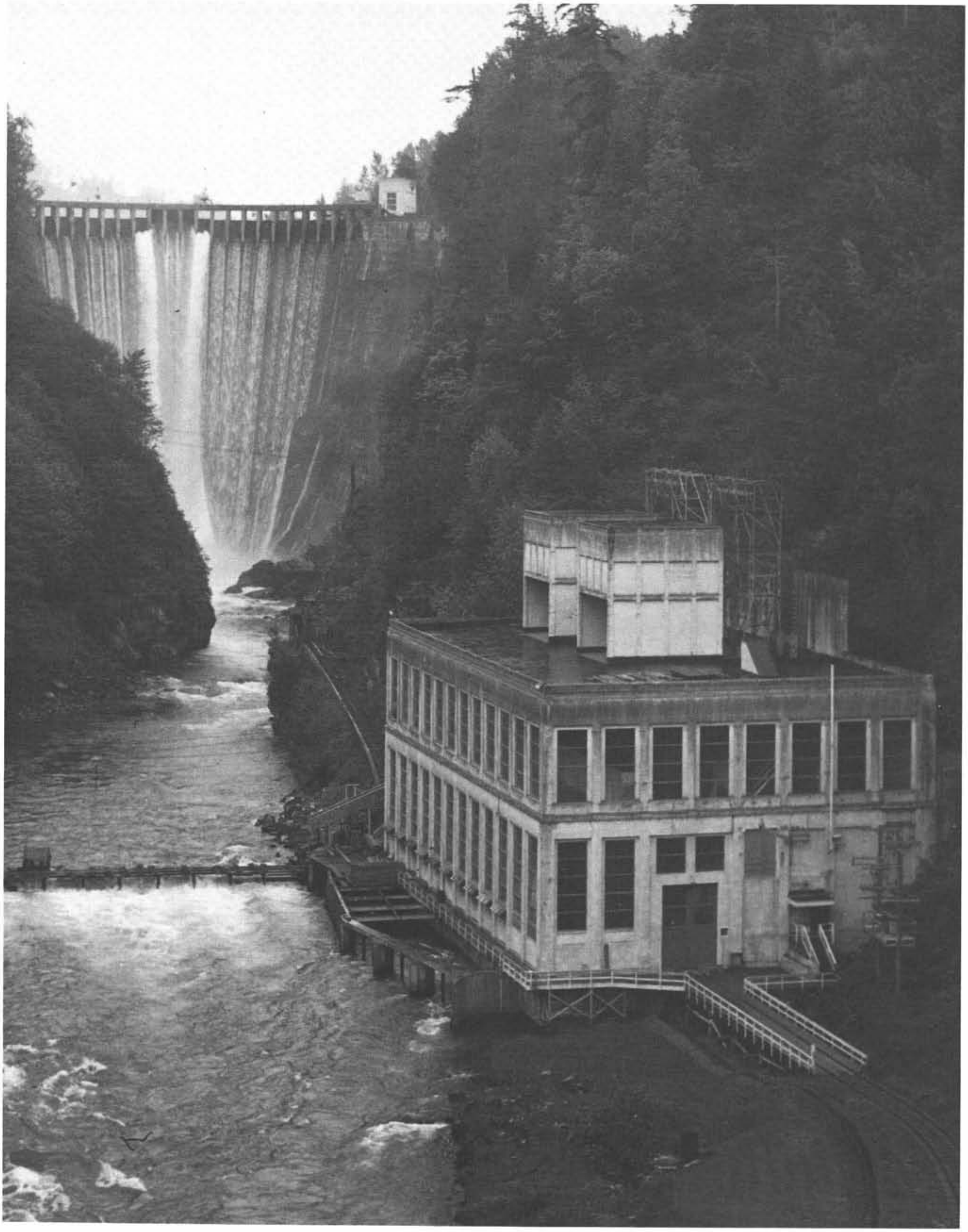

Figure 7. Lower Baker Dam and powerhouse. Photo courtesy of Puget Sound Power and Light Co.

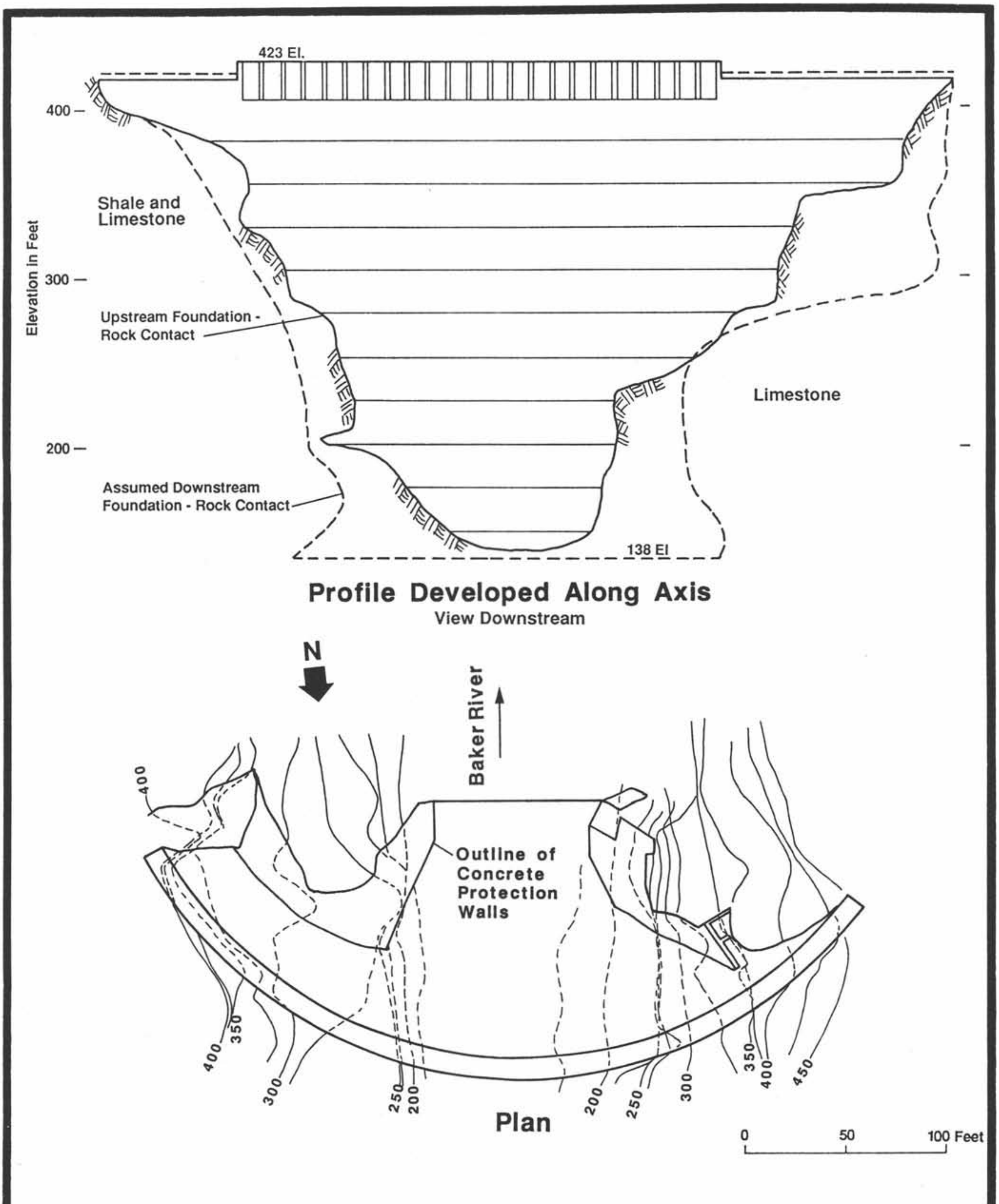

Figure 8. Plan and profile of Lower Baker Dam.

Limited information is available describing the construction of the dam. Most of the drawings and design calculations were stored in the original powerhouse, which was destroyed by a landslide in 1965 (Figure 9). The dam was completed in 1925 with a spillway crest elevation of 393 ft, and storage was commenced in November 1925. The dam was raised in 1927, increasing the spillway crest to the present elevation of 423 ft. As recorded in construction photographs maintained by Puget Sound Power and Light Company, the dam was constructed in blocks of various dimensions. No concrete temperature control methods were used. There were no formed contraction joints, and construction joints between blocks were not grouted. Grouting of the dam's foundation was not performed during the original construction. However, to reduce seepage, grouting was performed subsequently, in 1933, 1941, 1960, and most recently in 1983.

Dam Site Geology

The rock types at the dam site are limestone and intensely sheared shale, referred to as argillite. The right abutment consists mainly of limestone (Danner, 1966); the left abutment is shale with lenses of limestone. As might be expected with this type of foundation and abutments, there has been some excessive leakage. The sheared shale has a tendency to slake and the limestone to dissolve along small fractured zones. Thus solution channels form along the intersections of joints, faults, and bedding planes. The total volume of voids is small and has little influence on the integrity of the overall rock abutments with regard to strength.

Little or no folding can be seen in the limestone in the right abutment. Stress relief occurred by fracturing and shearing. In contrast, the rocks on the left abutment are intensely folded. The shale behaved plastically under pressure, even exhibiting overturned folds.

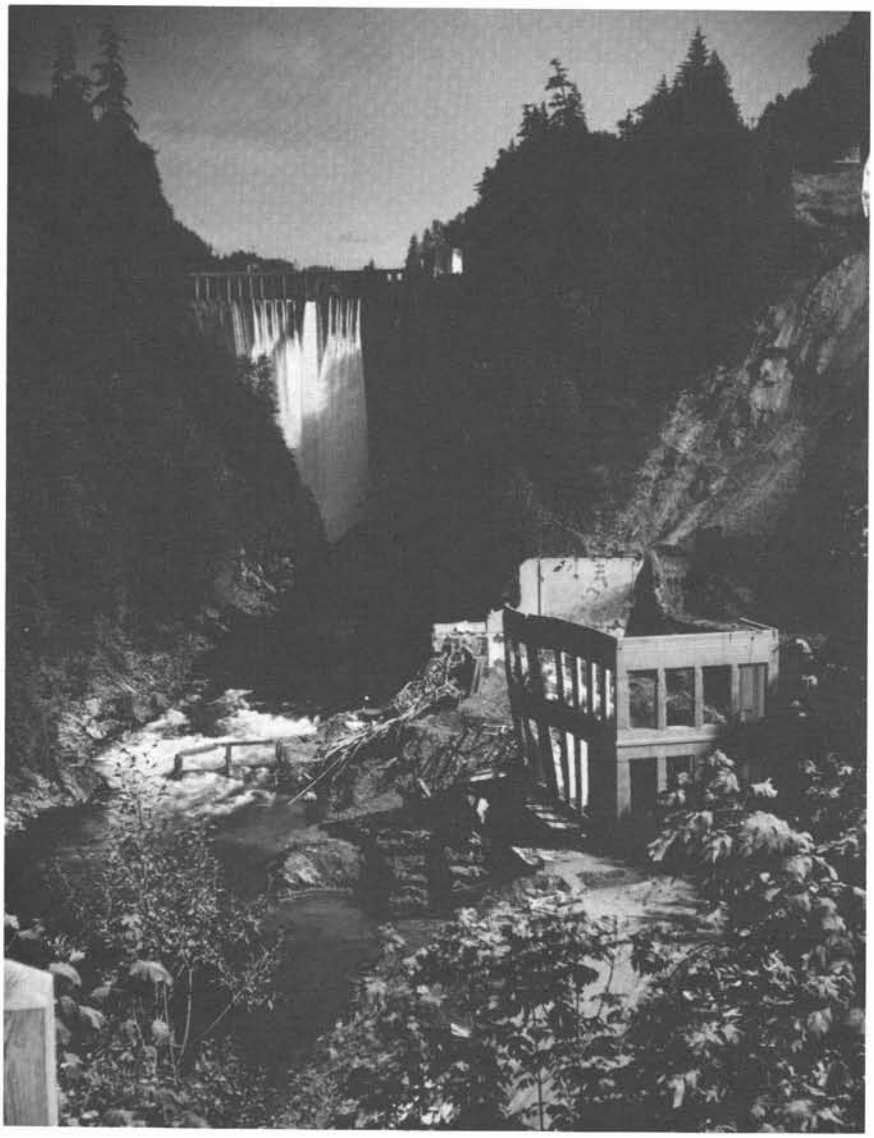

Figure 9. Lower Baker powerhouse after the slide in 1965. The far end of the powerhouse is in the river. Photo by R. W. Galster.

The displacement of rocks at the Lower Baker site, together with brecciation of the limestone and intense shearing of the shale, all suggest that the Baker River valley follows a large fault zone. Topographically, the valley is linear for at least 13 mi to the north. A faint topographic lineament can be traced for at least 20 mi to the south on the same trend. No offsets have been observed in Pleistocene lake beds in the Baker valley. The young flows from Mount Baker at the Upper Baker Dam area also show no evidence of offset. Hence the inferred fault is not considered active.

Seismic Analysis

In order to provide further evidence of dam stability during an earthquake, a rather exhaustive seismic analysis was made of both Baker dams (Stone & Webster Engineering Corp., 1984). At Lower Baker Dam the studies included limited field investigations identifying significant discontinuities potentially affecting foundation stability and the condition of reservoir walls. For the dam itself, studies were performed using three-dimensional linear, elastic, and finite element techniques—one set for static loading and a dynamic set for earthquake ground acceleration of 0.3 g. The results of the analyses and subsequent evaluation of the stability of the foundation under Lower Baker Dam indicate adequate factors of safety exist for both static and earthquake loading conditions.

Lower Baker Powerhouse Slide

On May 18, 1965, the powerhouse at the Lower Baker project was destroyed by a slide mass coming down over a steep cliff and literally pushing the powerhouse over. The events leading up to this disaster are worthy of note.

The earliest movement in the slide area above the powerhouse was detected in 1944 when the county road around the perimeter of the unstable area began to slump. In an effort to stabilize the entire area, drainage ditches were dug to divert water from the marshy ground. Several years passed without detection of movement in the slide area. Approximately a decade later it was noted that a transmission tower had moved out of line. Some remedial measures were taken, and the ground again seemed stable. In June 1964 a number of inclinometer installations provided a clue as to the location of the sliding plane. In August 1964 horizontal drains were placed to intercept a water-bearing stratum (Shannon & Wilson Inc., 1965). Despite all of these measures, small mudflows continued to spill over the bluff and down behind the powerhouse. Debris was removed from the toe of the slide and from back of the powerhouse.

In January 1965, 17 vertical relief wells were drilled in the slide area, and 12 encountered water-bearing strata. The wells were screened and pumps installed. In April of the same year, all movement stopped. Heavy rains soon reactivated the slide, and in May the mass came down, destroying the powerhouse.

The powerhouse lies at the base of an almost vertical cliff 220 ft in height. The rock in the cliff is sheared shale, argillite, and dirty limestone. Overlying this cliff is a mass of clay, silt, and sand covering an area measuring 700 ft x 800 ft. The surface of the slide rises 350 ft from the toe to the headscarp on a 3H to 1V slope.

The glacial lake sediments above the powerhouse were heavily over-consolidated by advancing ice. Severe cracking and fissuring of these lake beds resulted when erosion of laterally confining sediments released horizontal stresses imposed by the ice. The combination of steep slopes, highly fractured material, and abundant water created a marginally stable condition. The development of random but water-saturated, connected fissures and fractures within the silts and clays was the single most important characteristic that led to slope instability (Peck et al., 1965).

The cement mill at Concrete, now dismantled, obtained limestone from a quarry 1/3 mi from the slide area. Vibrations from blasting were monitored at the slide area to obtain some indication as to whether such vibrations could contribute to slide instability. On April 29, 1965, Seattle was in the epicentral area of a magnitude 6.5 earthquake, and vibrations were felt at Lower Baker Dam. It is possible that vibrations from these two sources could have contributed in some small way to slide instability, but excess water in the fractured slide material was the dominant cause of sliding.

Seepage Control

Shortly after the dam was completed in 1925 and water was raised behind the dam, leakage through both abutments was observed. By 1933 total seepage had increased to approximately 110 cfs. Both abutments were sealed with hot asphalt grout. The sealing was temporarily effective but seepage soon increased, and by 1960 it was leaking 55 cfs. Again, both abutments were grouted with hot asphalt (Stone & Webster Engineering Corp., 1960). By 1982 total seepage reached an estimated 140 cfs (Coombs et al., 1983).

Seepage channels followed shear zones and enlarged these zones by washing out fine gouge and dissolving some of the limestone. Since seepage was in the lower third of the abutments, it was difficult to reach the seepage channels from the steep canyon walls. Thus all drilling was done from the top of the dam.

Although hot asphalt had been temporarily effective in sealing the leaks, an effort was made to use some other form of grout that would be more permanent. Prior to the actual grouting, six holes were drilled in the abutments to penetrate possible leakage zones. These holes were carried to the base of the dam where the rock was relatively impervious. Several chemical grouts were tested in the laboratory prior to use in the field. One showed considerable promise and was used in the long grout holes. However, under actual hydrostatic conditions at the dam, the critical time period between the injection of the grout as a liquid and the point at which it became solid made use of that grout impractical.

Studies connected with the 1983 grouting program indicated that hot asphalt grout was most effective in stopping the leakage. Prior to injecting the hot asphalt, the drill holes were surged by air to remove gouge and shattered rock fragments in the faults and joints. It is believed this cleaning permitted a better penetration of the hot asphalt in these permeable zones. Once the flow is shut off, some of the chemical grout might be useful in lengthening the time interval between sealing by grouting.

ACKNOWLEDGMENTS

The writer acknowledges the many helpful discussions in the field and in the office on design and construction problems with W. F. Swiger, Vice President, Stone & Webster Engineering Corporation. Swiger prepared the section on the West Pass dike. J. K. Kearnes of Puget Sound Power and Light Company provided the photographs of Upper and Lower Baker dams, as well as background information on both dams.

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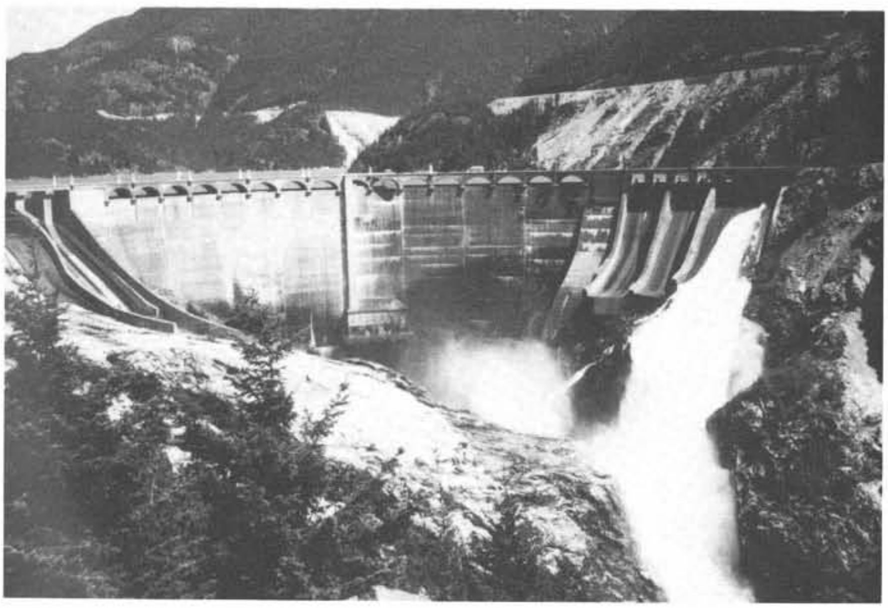
The Skagit Projects



Diablo Dam, the first major dam of the City of Seattle's Skagit Hydroelectric Project. Photograph by R. W. Galster, July 1965.

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The Skagit Projects: Ross, Diablo, and Gorge Dams

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PROJECT DESCRIPTION

The Skagit River flows southward from Canada through the rugged northern Cascade Range of Washington; then its course turns southwestward until it meets the waters of Puget Sound. The peaks of the northern Cascades average more than 7,000 ft in elevation, but the rivers, even in the heart of the range, average less than 1,200 ft above sea level.

More than half a century ago, J. D. Ross, then the superintendent of Seattle City Light, saw the possibilities for hydroelectric power in this primitive area less than 80 mi from Seattle. The plan was to construct three dams to make full use of the river from the settlement of Newhalem up to and a little beyond the Canadian border. The first application for a dam was made in 1917. Now completed, the three dams are: Ross Dam at the upstream end, Diablo Dam in the middle, and Gorge Dam at the downstream end.

Ross Dam is 540 ft in height and was planned to be 133 ft higher. However, the ultimate height would have backed water into Canada. Although Seattle City Light was originally granted permission for this encroachment of water into British Columbia in 1936, several decades later environmentalists in British Columbia raised objections. After considerable negotiations, involving the International Joint Commission, the matter was settled by an exchange of power, and it was agreed that no water was to be backed into Canada.

AREAL GEOLOGY

The northern Cascades have been sculptured by continental glaciers from Canada covering all but the highest peaks, as well as by extensive alpine glaciers (Crandell, 1965). As a result of the ice action, the highest parts of the range provide excellent exposures of the complex assemblage of metamorphic and igneous rocks in the upper reaches of the Skagit River.

The center of the range is composed of a crystalline core of gneisses, schists, migmatites, and granodiorites, all pre-Cenozoic in age. Two exceptions are the young volcanoes, Mount Baker and Glacier Peak.

The distribution of rock types is controlled by large strike-slip and thrust faults that divide structural blocks (Misch, 1966, 1979; Davis, 1977, 1984; Woodward-Clyde Consultants, 1978; Clayton and Miller, 1977; Vance, 1977) (Figure 1). The two largest faults are the Shuksan thrust and the Straight Creek fault. Misch (1966) has described in detail not only the structural complexity of the area but also the metamorphic history of the various rock types in this part of the North Cascades.

The dominant rock type in the upper Skagit Valley is the Skagit gneiss, a quartz diorite gneiss, largely migmatitic and commonly banded, with remnants of biotite schist and amphibolite. Large concordant bodies of orthogneiss are scattered throughout the area. This assemblage is known as the Skagit metamorphic suite. The upper Skagit canyon is one of the best exposed and most easily accessible migmatite zones in the world.

Interesting as these rocks are to the petrographer and structural geologist, the physical characteristics of the Skagit gneiss are also pertinent to engineering problems. Several pronounced sets of joints can be delineated at any given locality. Some are related to the gneissic structure, some are rebound joints resulting from ice unloading, but most are the result of several periods of deformation. Joints will be discussed more in detail in the sections treating dam sites.

ROSS DAM

General Description

Ross Dam (Figure 2) is the largest of the three dams on the Skagit River and provides most of the reservoir capacity for the system. It is founded on the Skagit migmatitic diorite-gneiss (Figure 3), as are Diablo and Gorge dams (Misch, 1966). This 540-ft-high dam was built in three stages in the periods 1937 to 1940, 1946 to 1949, and 1952 to 1957. Originally Ross Dam was designed with a waffle-iron face on the downstream side to facilitate planned modifications to bring it to its ultimate height of 661 ft. The waffle-iron face was constructed on all three lifts of the dam, but it was never

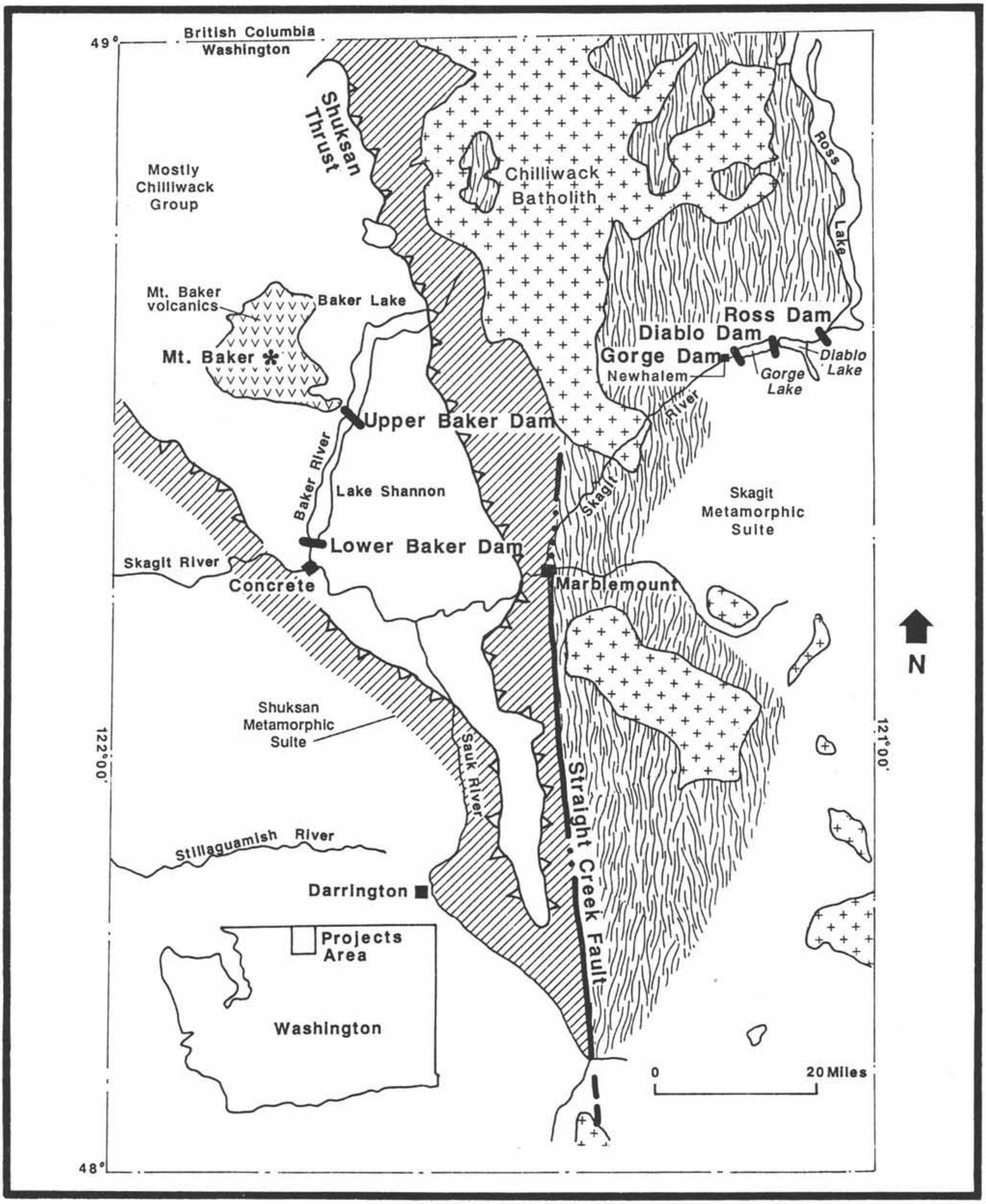

Figure 1. Generalized geologic map of the Skagit and Baker project area. The Straight Creek fault, which has great strike-slip displacement, brings together contrasting lithologies. The Skagit metamorphic suite, in the area of the Skagit dams, is essentially quartz diorite gneiss. In contrast, the Shuksan metamorphic suite is composed of greenschists and phyllites. The Chilliwack Group, below the Shuksan thrust, contains slates, phyllites, and metavolcanic rocks of late Paleozoic age. The Eocene to Miocene composite Chilliwack batholith ranges in composition from quartz diorite to granodiorite.

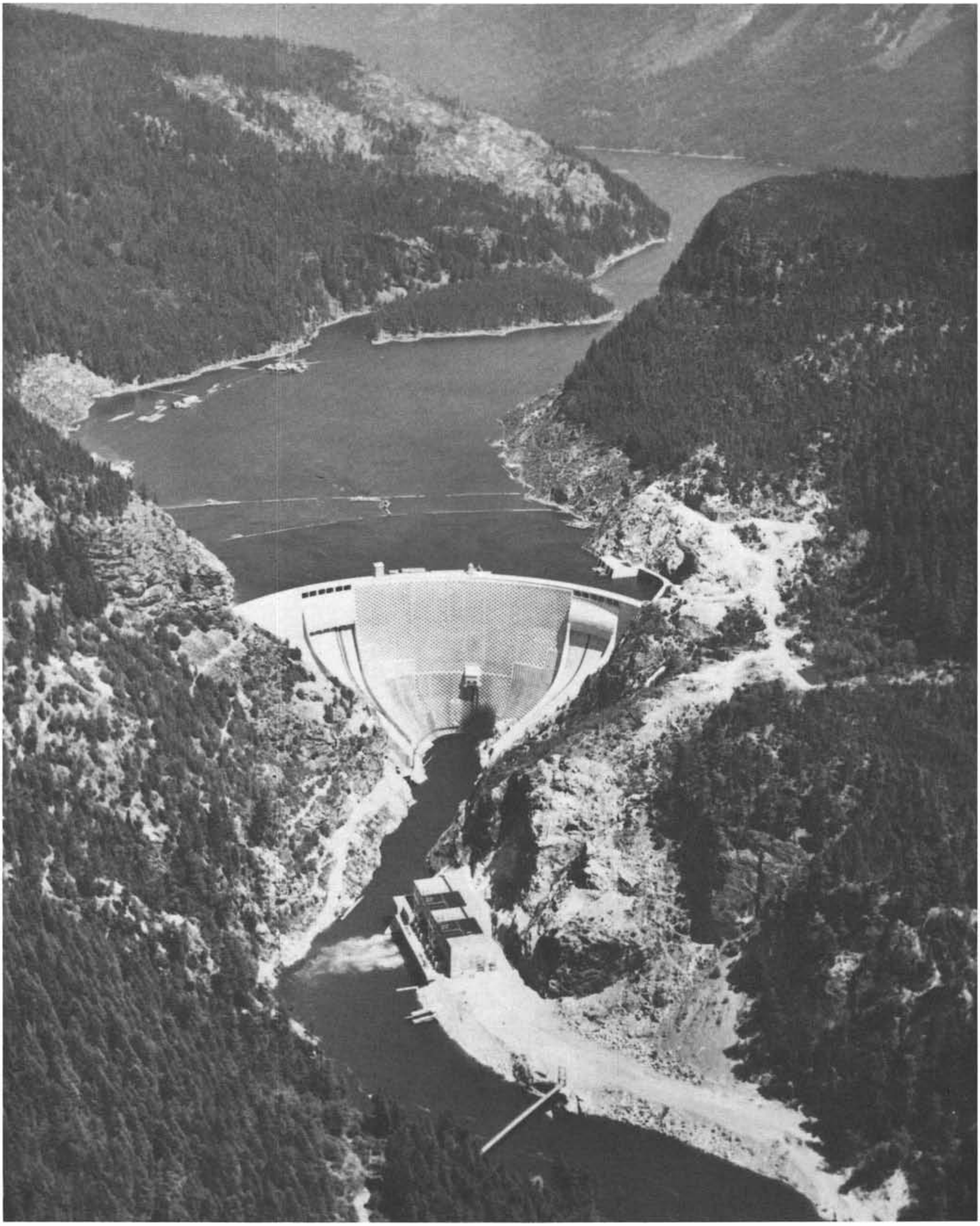

Figure 2. Ross Dam spillway and powerhouse. The power tunnel intake is on the right side of the dam. The "waffle iron" downstream face was designed to facilitate raising the dam. Photo by Seattle City Light.

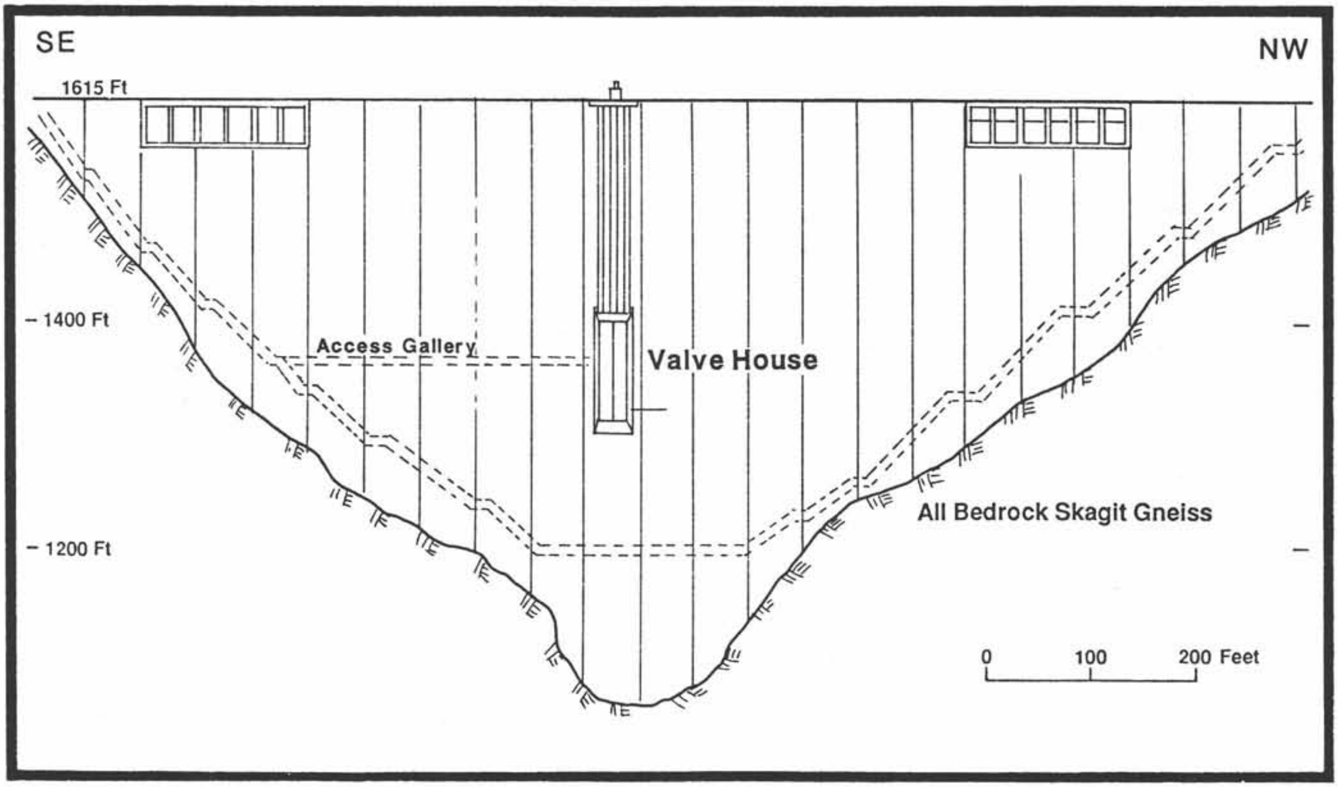

Figure 3. Section of Ross Dam; view downstream.

utilized because political complications prevented raising the dam to the planned height. Maximum reservoir elevation is 1,600 ft.

Ross Dam is a concrete gravity arch structure. Two chute spillways follow the downstream face of the dam and discharge into a deep natural pool at the toe of the dam. Two 24.5-ft-diameter power tunnels 1,900 ft in length are under the left abutment and serve the four-unit powerplant (Figure 4). Foundation treatment included removal of all weathered rock; a conventional grout curtain was constructed, and drain holes terminate in the gallery system within the dam.

Dam Site Geology

In 1936 C. P. Holdredge made the first geologic investigation of the dam site, paying particular attention to the fault and joint planes in the abutments. His example was followed by those who later carried on the geologic investigations (Berkey, 1943, 1944; Coombs, 1945). Decades later, as construction progressed, the location of major shear zones and joints became increasingly more important (Coombs and Sarkaria, 1971).

Rock at the dam site is cut by joints which can be divided into primary, secondary, and tertiary sets. The primary, or regional, joint set strikes generally N 40° E and dips 60° northwest. The secondary set strikes N 60° W and dips approximately 50° northeast. The tertiary set strikes between N 30° E and N 75° E and dips 30° southeast.

Major faults are few and tend to be nearly vertical. The largest fault zone width was 6 ft. On the right abutment a vertical fault was encountered approximately half way up the second stage excavation; it trends parallel to the river. Another vertical fault, also parallel to the river, was encountered several hundred feet downstream on the right abutment. Prior to construction of the second stage of the dam, a vertical fault was traced in the left abutment roughly parallel to and under the axis of the dam. This bifurcated approximately half way up the abutment into two subparallel faults. All faults were cleaned and filled with concrete.

Several small shear zones were encountered in the abutments. They were narrow, varying from 6 to 18 in. in width but pinching and swelling from place to place. Some were so completely healed by later solutions that they were stronger than the gneiss on either side. In other places the shear zones were mechanically shattered and, locally, permitted weathering and decomposition to occur. All such zones were cleaned and grouted.

On the right abutment above the dam, a rock knob threatened to present some problems if the dam was to be raised to its ultimate planned height. A nearly horizontal shear zone at the base of the knob, coupled

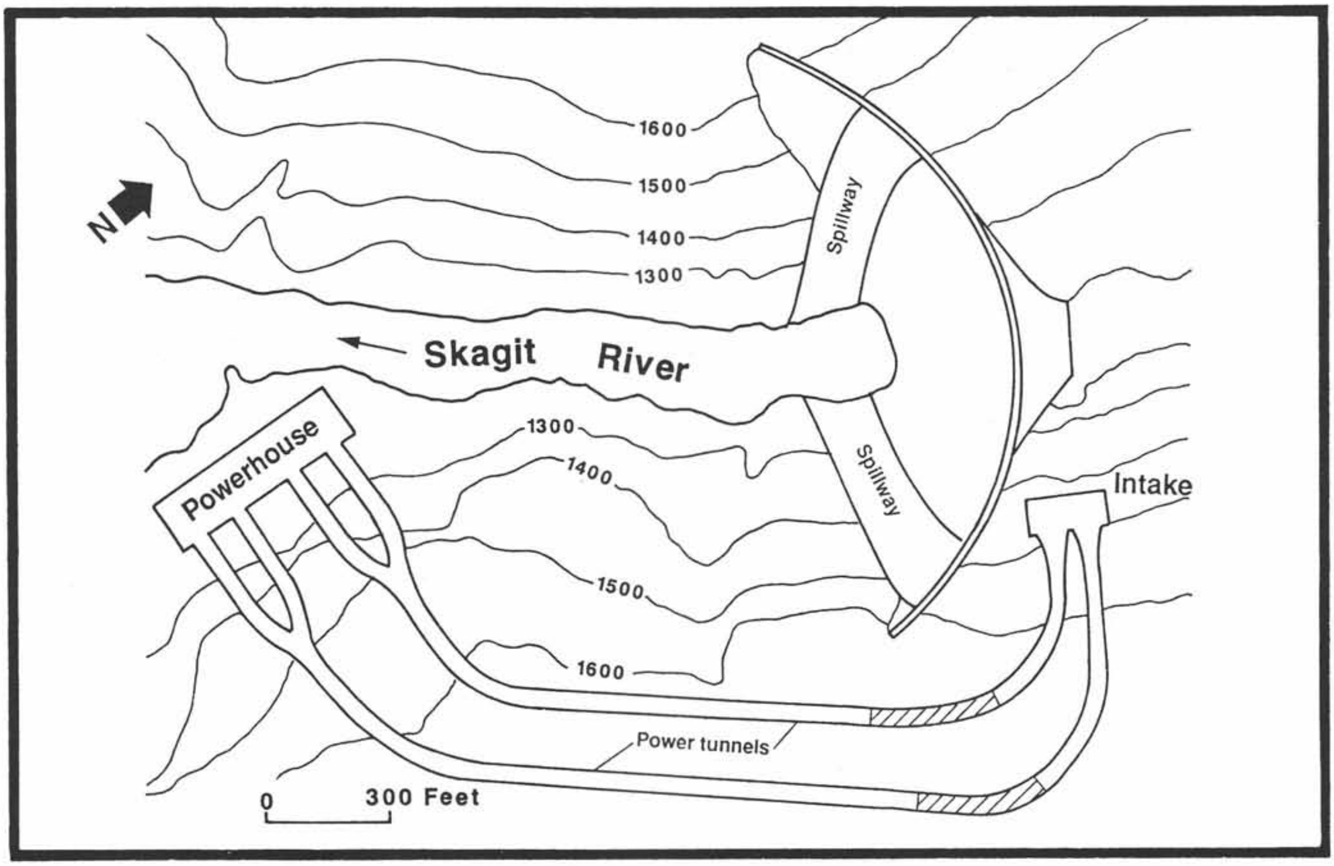

Figure 4. Plan of Ross Dam. Hachured areas of tunnels were excavated before pouring concrete in the second stage.

with joints dipping downstream within the knob, indicated the upper part of the abutment was not sufficiently strong to support the uppermost part of the structure. The design engineers avoided stressing this rock knob by an independent gravity wall that connects the arch to the right abutment upstream of the knob. An open transverse joint in the upper arch directs the thrust away from the rock knob (International Engineering Company, 1968). Since these studies were made, Seattle City Light has agreed not to raise the dam to its ultimate planned height.

Concrete

The concrete in Ross Dam is of exceptional quality. The average unconfined compressive strength is 7,000 psi, and the minimum flexural tensile strength is 580 psi. The aggregate used in the first stage construction of Ross Dam came from a site 3 mi upstream of the dam and thus was beneath the reservoir prior to the construction of the second stage a decade later.

To make concrete with commensurate physical properties to be acceptable in this high arch dam, various places downstream were tested for suitable aggregate. Test cylinders using aggregate from a site 13 mi downstream from Ross Dam gave excellent results (Coombs, 1954, 1969). However, both Diablo and Gorge Dams were between this source of aggregate and Ross Dam. All sand and gravel from this source had to be hauled by rail up past Gorge Dam to an incline at the toe of Diablo Dam, then up the incline and dumped into a barge on the Diablo reservoir and taken to the toe of Ross Dam (Pacific Builder and Engineer, 1946). The cement mill at the town of Concrete on the Skagit River provided the same quality, low alkali, cement for all stages of Ross Dam. The mill has now been dismantled.

Vibration Studies

Both the first and second stages of Ross Dam were built to store water. Ultimately the dam was to be used to generate electricity. The plan was to drive two 27.5-ft-diameter tunnels around the left abutment of the dam at an elevation approximately 60 ft higher than the first stage. The Board of Consultants, in 1944, thought it advisable to drive that portion of the tunnel near the dam before the second stage was constructed. The nearest power tunnel was less than 150 ft from the left abutment of the dam (Figure 4). There was some concern that setting off 800 lb of powder in the tunnel so close to new concrete in a second stage might damage the dam.

Consequently, an experiment was made at Ross Dam during the construction of the second stage. In order to obtain some quantitative appraisal of the potential effect of blasting vibrations on the abutment, Frank Neumann, Chief of Seismology of the U.S. Coast and Geodetic Survey, was called in. Several types of instruments were used for the study, but the most satisfactory was an inertia seismometer of the Wood-Anderson type. Only the horizontal components were measured. Some of the observation points were on the left abutment, some in the tunnel opposite from the blast, and others were on the dam itself (U.S. Coast and Geodetic Survey, 1944).

The greatest acceleration measured on the dam was 0.075 g when the blast was in the tunnel approximately 200 ft from the seismometer. Displacements in the concrete were from one-half to two-thirds those measured on the rock wall at equivalent distances. There was no significant change in the frequency of the dam vibration, nor was there any indication of amplification of the dam at its own natural frequency. While there was considerable consistency in the results obtained, the dispersion of vibrations throughout the rock wall was not generally uniform. There were changes in amplitude and frequency due perhaps in part to the presence of fault and joint surfaces and in part to the complexity of the vibrations themselves (Neumann, 1947; Coombs, 1949).

Evidence from the vibration studies and visible inspection of the first stage of Ross Dam indicated there was no damage from the blasting. In 1949 a 7.1-magnitude earthquake occurred near Olympia, Washington, approximately 150 mi from Ross Dam. This earthquake was felt at Ross Dam but apparently did no damage.

In summary, the vibration studies, interpreted on the basis of acceleration in terms of gravity, suggest that no damage would have resulted had the second stage been in place.

Powerhouse Foundation

Obtaining space for a powerhouse in the deep canyon at Ross Dam created a siting problem. The first stage of Ross Dam was completed in 1940, the second stage in 1949, but no electricity was to be generated until the completion of the third stage in 1957. The site of the powerhouse had been selected, but little information was available about foundation conditions until a drilling program was laid out in 1949. The small designated site was covered by aggregate used in construction of the second stage. The fit of the powerhouse on bedrock was so tight that a portion of the structure had to be cantilevered out over the river.

Care had to be taken in identifying bedrock because of the enormous blocks of rock that had fallen from the canyon walls and were now lodged as talus slopes near the river bottom. All cores were taken through 25 to 30 ft of unbroken rock to insure that a boulder was not being drilled. Many drilling core pulls of rock were in 10-ft, unbroken lengths and had to be broken to fit into the core boxes. The orientation of the gneissic banding in the cores was also checked and confirmed that the core exhibited the same banding orientation as the adjacent bedrock.

DIABLO DAM

General Description

This concrete arch (Figure 5) was completed in 1930 and is the oldest of the Skagit River dams. Bedrock in the foundation is the same quartz-diorite gneiss and migmatite found in the other two Seattle City Light dams on this river. At this site a steep-sided inner gorge broadens into a glacially scoured upper valley.

The arch dam spans both segments of its base and rises to a maximum height of 389 ft. Maximum reservoir elevation is 1,205 ft. The spillways are located on gravity sections on either side of the arch section (Figure 6). The power intake is located on the right abutment near the end of the spillway section. A single, concrete-lined 19.5-ft-diameter power tunnel extends 1,900 ft from the intake structure to the surge tank. In the next 140 ft the tunnel trifurcates into two 15-ft-diameter and one 5-ft-diameter steel penstocks. The penstocks are completely encased in concrete as a protection against falling rocks. For its entire length the tunnel is in sound, massive gneiss. The power plant is located west of the dam at the toe of a steep slope (Figure 5).

Foundation Conditions

Foundation treatment included a conventional, deep peripheral grout curtain and a row of drilled drains that lead into the gallery system in the dam. The body of the dam is drained by formed drains near the upstream face, which also connect with the gallery system.

Spill from the reservoir is carried primarily by three of the radial gates on the gravity section at the left end of the dam. Originally water falling from the lips of the spillways struck rock immediately below. However, in 1950-1951 the spillways were extended so as to throw water over the rocks and into the narrow gorge. There is no evident erosion as a result of this action.

On the right end of the dam the 12 radial gates spill directly on a shelf of bedrock. Spill took off a little of the slabby rock near the lip of the spillway. Some of the spillways were extended to remedy this situation.

Canyon Walls

The canyon walls of Diablo reservoir are quite similar to those at Ross reservoir in that they are composed of glacially scoured bedrock and have only a very thin, discontinuous soil cover. The slopes, though quite steep, are remarkably stable. Avalanches of small volumes of snow and loose rocks occur during the winter but have not generated destructive waves.

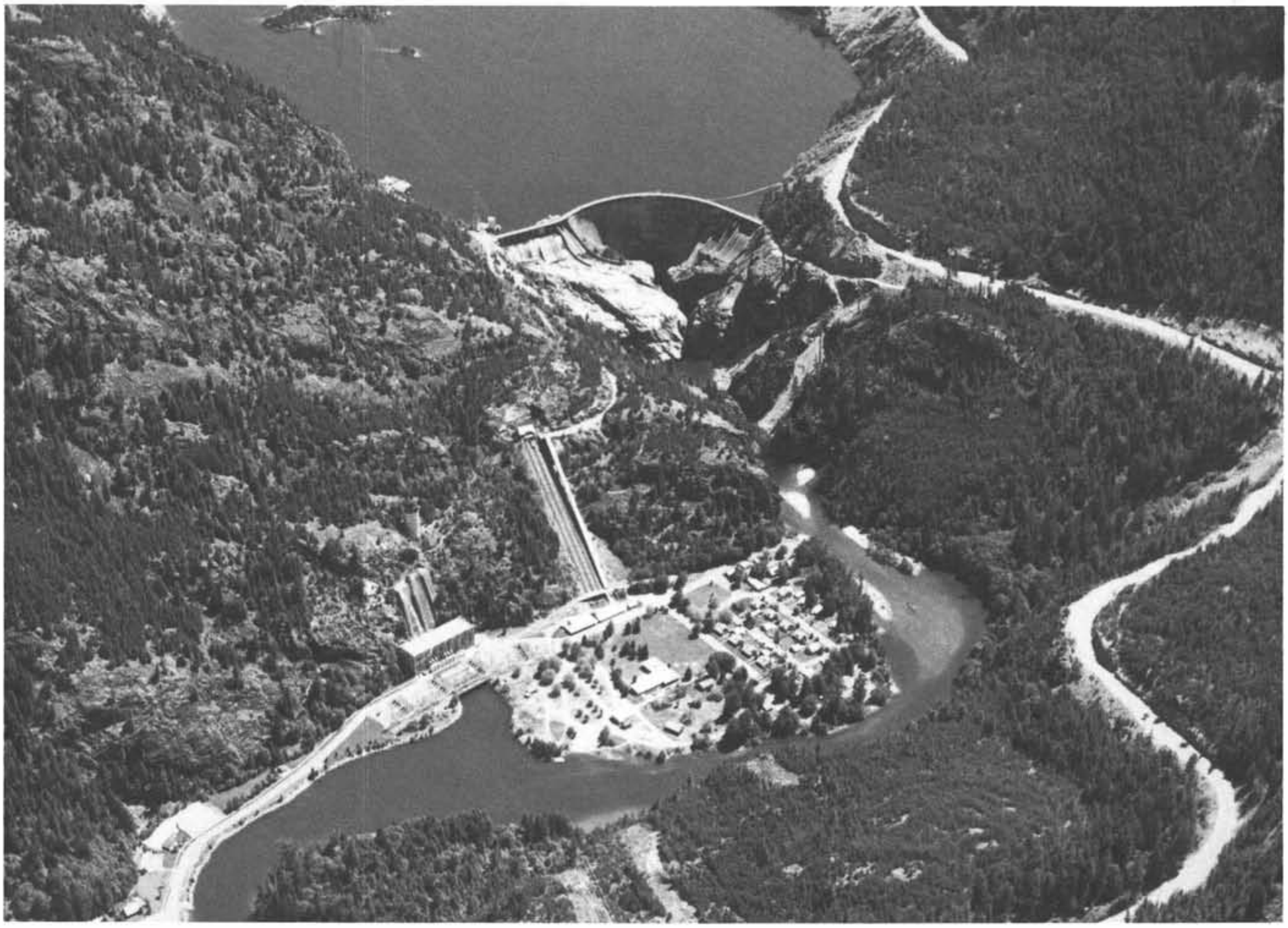

Figure 5. Diablo Dam with spillways on either side. The intake structure is to the left of the dam and spillway. The incline lift connects at the top with a road to the dam. The penstock and powerhouse are to the left of the incline. Photo by Seattle City Light.

GORGE DAM

General Description

In 1919 a simple crib structure founded on gravel was erected to divert the Skagit River into an 11,000-ft-long, 20-ft-diameter lined tunnel to a powerhouse located at the settlement of Newhalem.

In 1961 a 300-ft-high dam was designed and constructed immediately downstream of the old diversion structure. The dam is a concrete gravity arch (Figure 7) with the spillway on the left abutment; the spillway is separated from the arch by a buttress (Figure 8). The spillway chute is designed to direct the discharge away from the left abutment and into the natural plunge pool of the river. Maximum reservoir elevation is 875 ft.

Foundation Conditions

Geologically, the Gorge Dam site is of interest because of the W-shaped bedrock surface configuration beneath the dam (Figure 8) and the unusual groundwater problems imposed by having an existing reservoir only 400 ft upstream from the deep excavation required for the dam.

The 65 drill holes to test the site revealed an unusual bedrock topography beneath the valley floor consisting of two buried "slots" or channels. The Skagit River was flowing above a ridge of bedrock separating 160-ft-deep channels in the bedrock surface on either side of the river. The fill in the buried channels consisted of sandy alluvium, unsorted slope wash, and scattered large blocks of rock from the canyon walls. The bedrock surfaces in the slots were scoured and rounded by the sandy alluvium, presenting remarkably fresh surfaces for concrete after excavation and cleaning.

In excavating for the present Gorge Dam the limited space between the new site and existing crib dam required upstream excavation slopes of 1-1/2H to 1V or steeper to provide room for diversion facilities. Almost vertical cuts in the bedrock called for the use of rock bolts, as a safety measure, on the weathered rock faces above the deep excavation.

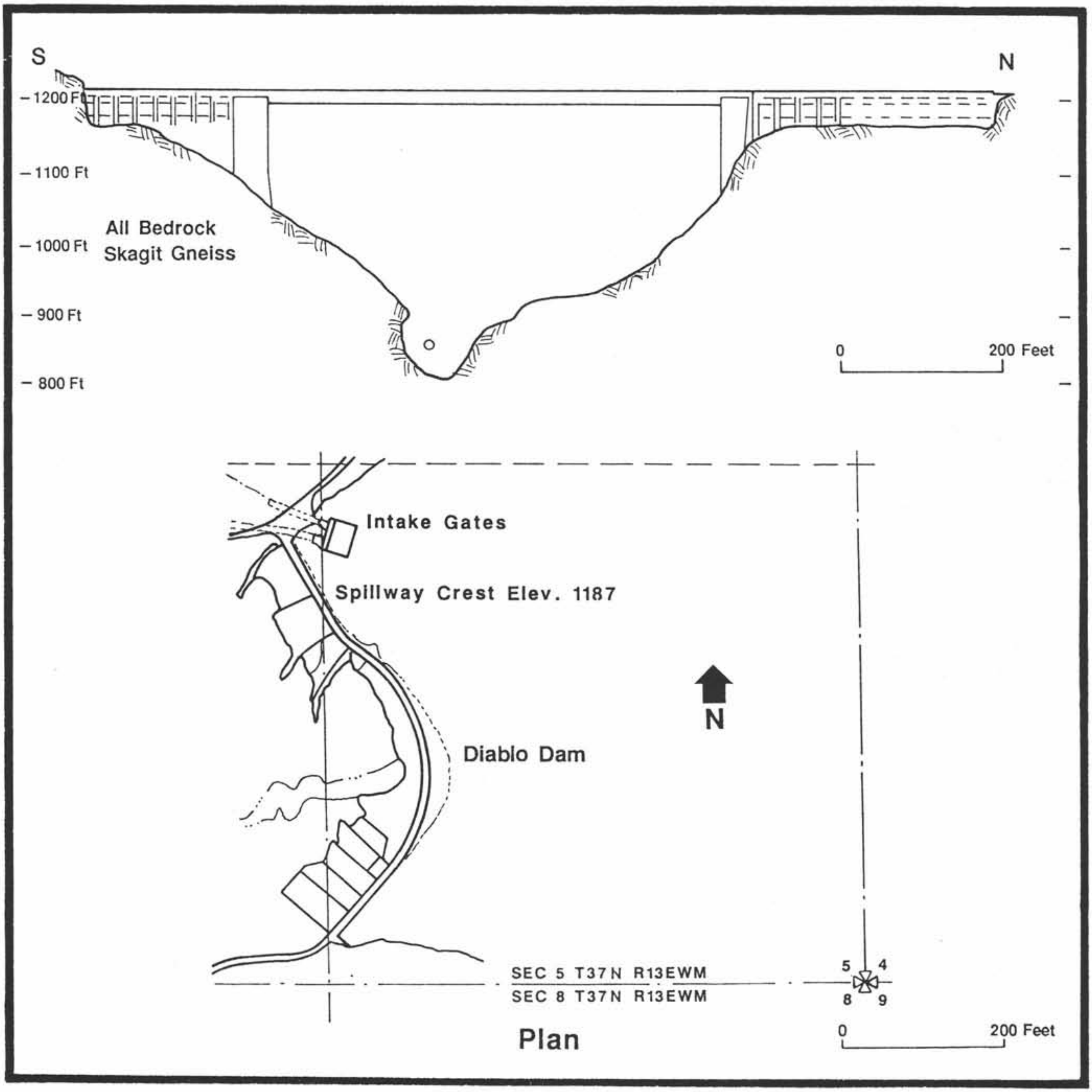

Figure 6. Plan and section of Diablo Dam; view downstream.

One of the most critical conditions for the construction of Gorge Dam was the excavation in the "slots" to a depth of 207 ft below water level downstream of the small diversion dam. Since the diversion dam was built entirely on alluvium, the slopes for the excavation had to be kept as stable as possible by preventing water seepage which might cause slipping or caving that would endanger the excavation for the new dam. The imposed hydrostatic head from the forebay of the old Gorge Diversion Dam and the steep hydraulic gradient from the diversion dam to the bottom of the excavation would mean exceedingly high velocities of ground water through the more permeable zones in the alluvium.

The method showing the greatest promise in preventing water from moving freely through this 400-ft path was to freeze an impervious curtain in the river alluvium just downstream of the old dam (Figure 9). The plan was to reduce the ground-water flow with grout and ice cur-

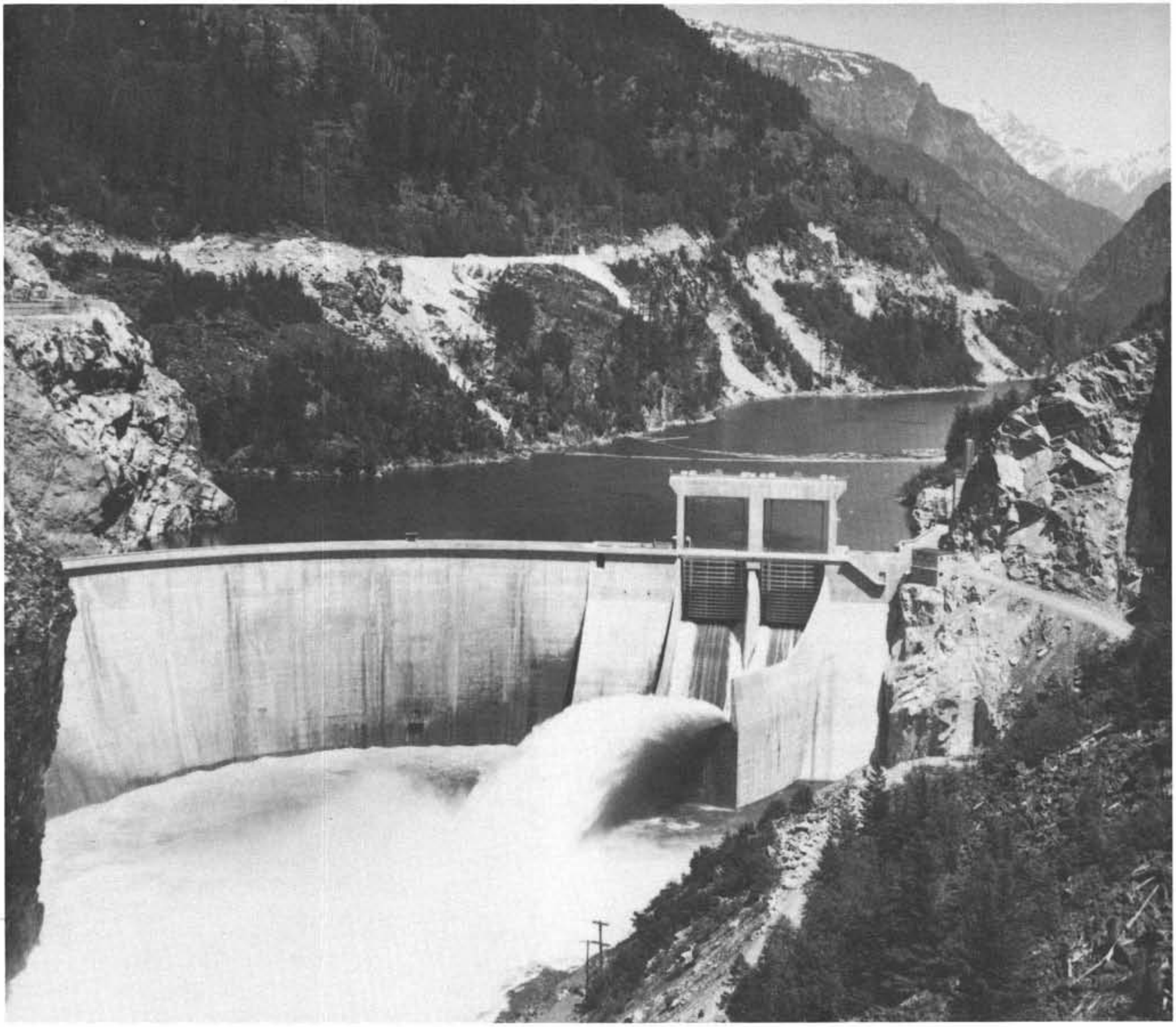

Figure 7. Gorge Dam during spill. Photo Seattle City Light.

tains and to intercept enough of the ground water by pumping wells so that the refrigeration plant could reduce the ambient temperature at the ice curtain to below freezing. Observation wells were drilled downstream of the proposed ice barrier location to give additional data to determine at what level water should be maintained in the pump wells for minimum flow past the freeze points.

First stage diversion called for dewatering the left channel to permit excavation for the gravity section and the pouring of concrete. The freeze curtain for the first stage was 400 ft long and consisted of 23,500 linear feet of holes ranging from 50 to 200 ft in depth and on 4-ft centers.

The installed capacity of the refrigeration plant was 392 tons with a pumping capacity of 350 gpm to circulate the calcium chloride brine. Thermocouples were installed about every 28 ft along the line of freeze points to monitor progress of freezing. Data obtained from thermocouples indicated local "hot spots" that were corrected by adding pumped wells upstream, grouting, or adding freeze points.

In spite of these methods, there was some leakage. The ice curtain was found to be the culprit. The holes drilled for the ice curtain were surveyed and found to be less than vertical. Some had crossed over to the next hole 4 ft distant. Once the gaps in the ice curtain were located, new holes were drilled and the corrected curtain proved to be an effective barrier.

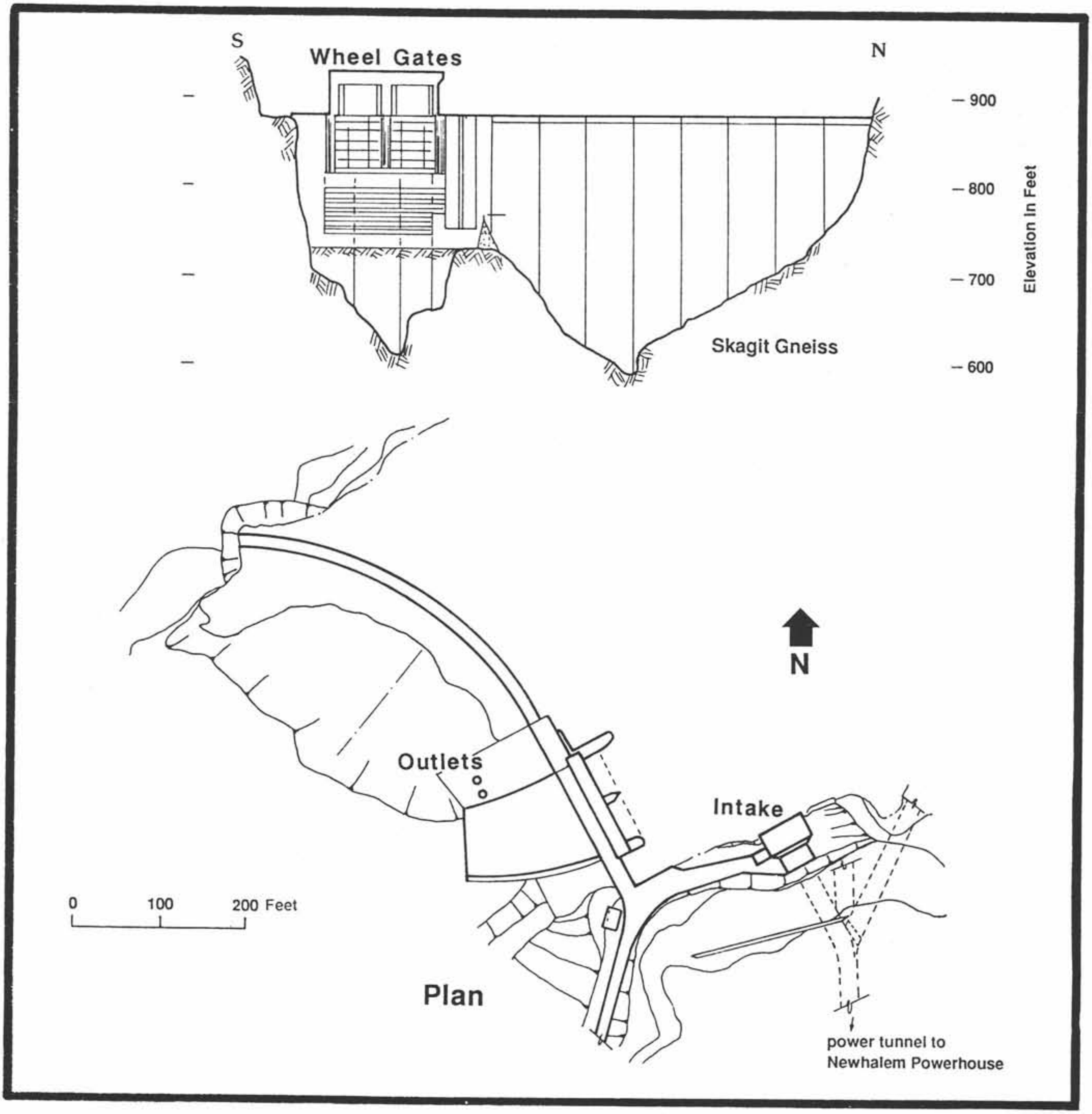

Figure 8. Gorge Dam plan and section; view downstream.

ACKNOWLEDGMENTS

The writer thanks L. R. Whitney, Manager, Civil Engineering, Seattle City Light, for information on the City of Seattle dams. He also expresses his appreciation for the many hours spent in the field discussing construction problems with Raymond Hoidal, former Chief Civil Engineer, Seattle City Light.

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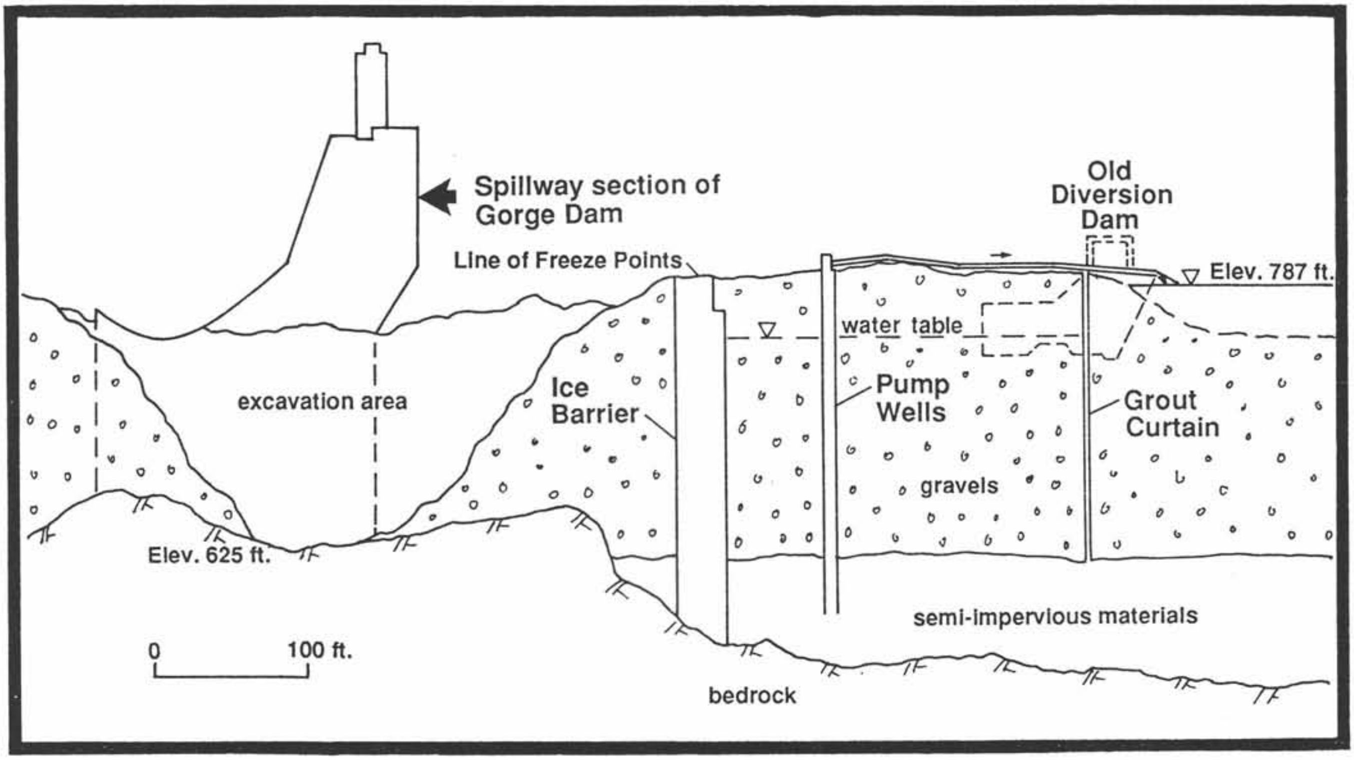

Figure 9. Sketch of Gorge Dam ice barrier to control ground water during construction.

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An aerial view of steam rising from Sherman Crater on Mount Baker in 1975. Photograph by R. W. Galster.

Cascade Ice Border Dams

Aerial view upstream of Mud Mountain Dam on the White River, in about 1948. The only access to the valve house (center) at this time was by a 100-ft-high exterior stairway. A roadway has subsequently been constructed down the face of the embankment dam and a concrete bridge constructed through the narrow rock gorge from the toe of the dam to the valve house. The ungated concrete spillway is to the left. Photograph by Seattle District, U.S. Army Corps of Engineers.

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Cascade Ice Border Dams: Geologic Setting

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The incursion of Pleistocene continental ice into the Puget Sound Basin is well documented, and the general stratigraphy and chronology of the glacial and interglacial sequence has been established (Willis, 1898; Bretz, 1913; Mackin, 1941b; Crandell et al., 1958; Crandell, 1965; Easterbrook et al., 1967; Mullineaux, 1970; Crandell and Miller, 1974; Blunt et al., 1987) (Figure 1). Four major glacial advances and several partial advances have been identified (Table 1). Each major advance occupied the entire basin and intruded into the lower extremities of the mountain valleys that head in the Cascade and Olympic ranges. During each incursion the ice border areas were the scene of a kind of geologic mayhem dominated by the impounding and diversion of ice meltwater-swollen drainages along the margin of a continually varying ice front. The result is a highly complex group of moraines, ice-marginal channels, deltas, and terraces containing an equally complex sequence of outwash, till, and lacustrine deposits. The sequence is further complicated by cutting and filling of canyons in earlier glacial drift and by the effects of alpine glaciations in the adjacent mountains, which, for the most part, were precursors of each continental ice advance.

The Pleistocene drainage diversion was especially pronounced along the eastern and southeastern border of the ice lobe (Figure 2). The volume of the meltwater increased southward in a complex of ice-marginal channels eventually discharging into the Chehalis drainage, thence westward into the sea. The discharge has been estimated to have been about 300,000 cfs (Cary, 1968). In each Cascade valley the drainage was first impounded, then diverted southward by the deposition of moraines and deltas. During interglacial and post-glacial times, the drainage was entrenched, usually into bedrock, along the south wall of each valley or was diverted entirely out of the preglacial valley, forcing cutting of a new canyon. Examples of the former are the Snoqualmie, Cedar, and White rivers; examples of the latter are the Sultan, Green, and Nisqually rivers. Most of the resulting bedrock gorges are narrow and steep sided, whereas the upstream portions of the valleys are wide, generally the result of repeated alpine glaciations.

Such favorable topography has attracted engineers to or near the points of Pleistocene drainage diversion. Here the fortuitous combination of canyons, many with partly exposed bedrock walls for dam abutments, together with broad upstream valleys for reservoirs is ideal for such projects from the standpoint of civil engineering (Coombs, 1969). Although the canyons and post-glacial channels offer tempting dam sites, each presents its own unique seepage and stability problems (Cary, 1968).

The most recent pervasive glaciation, the Vashon Stade of the Fraser Glaciation, deposited a series of extensive delta moraines across each of the pre-existing valleys along the Cascade mountain front, obliterating many of the landforms from earlier glaciations. These delta moraines are especially well exposed in the Skykomish, Snoqualmie-Cedar, and White river valleys where major cross-state highways pass through the mountain front. In several locations, modern lakes are impounded behind the delta morainal embankments (for example Cedar, Hancock, Calligan) (Figure 3). At other locations no natural lake exists because of drainage through the delta morainal embankment or downcutting by the stream sufficient to drain the glacial lake. Although the classic delta morainal embankment would *a priori* be composed of fine-grained sediments (lacustrine silt, clay, and fine sand) low in the deposit, grading to coarser material upward (sand, gravel, cobbles, and boulders), and the lake beds would rise time-transgressively up-valley (Mackin, 1941a), the embankments are found to be far more complex features. The amount and completeness of a blanket of till on the western face or top of any given embankment varies greatly depending on the history of the ice front and subsequent erosion. Thus, it is not possible to understand the capabilities of these delta morainal embankments to confine reservoirs without thorough investigation and analyses of the glacial geology. The success or failure of a dam/reservoir

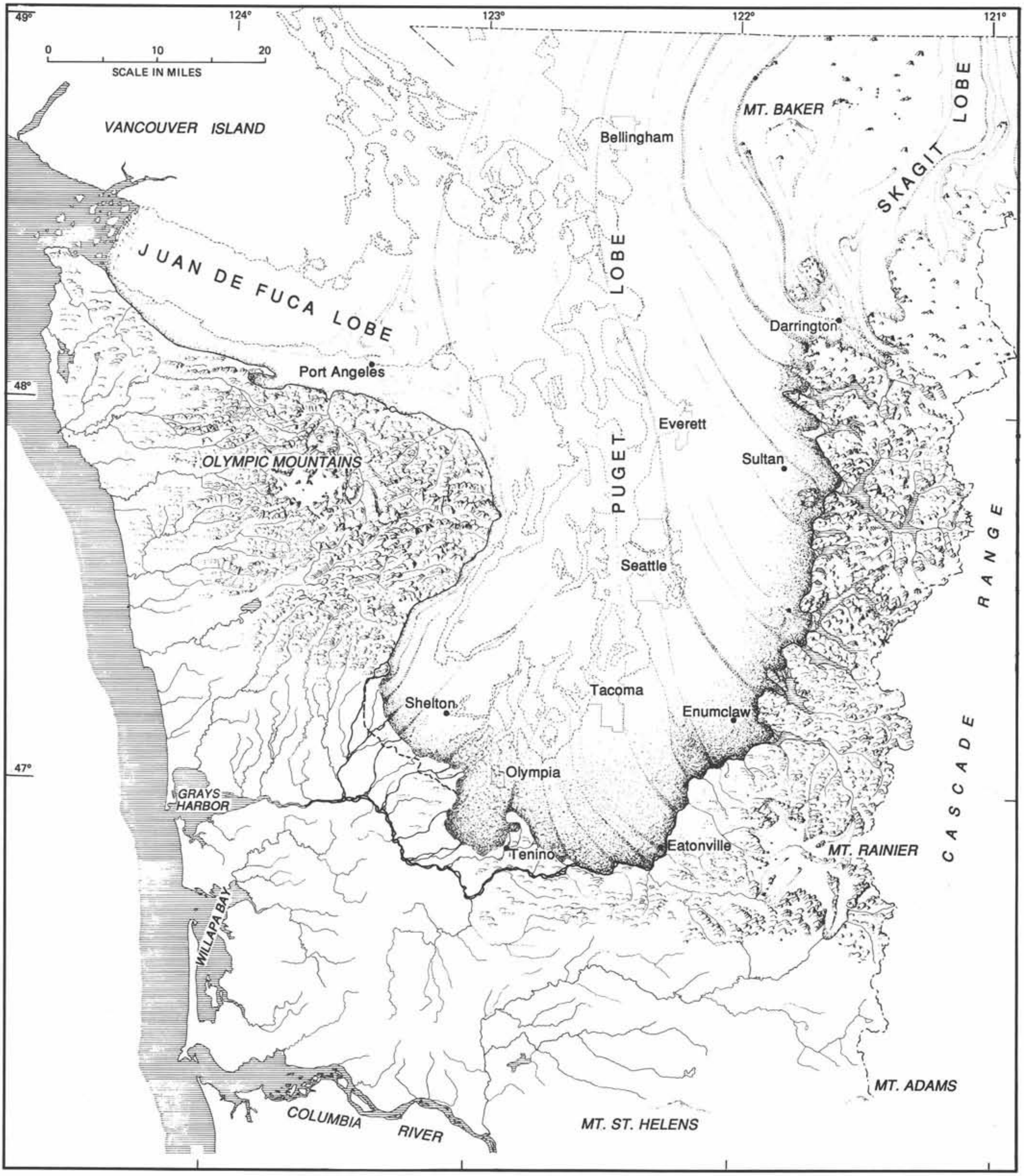

Figure 1. Maximum extent of the Puget lobe into the Puget Sound Basin during the Vashon Stade of the Fraser Glaciation about 14.5 ka. Modified from drawing by A. S. Cary, 1968.

Table 1. Pleistocene sequence, Puget Sound Basin and western Cascade Range (after Blunt at al., 1987; Crandell and Miller, 1974; Crandell, 1965; Lea, 1981; Westgate et al., 1987; J. B. Noble, 1989, oral communication)

Mag. Stage	Age ka	Geologic/climate units		Northern Puget Sound Basin	Southern Puget Sound Basin	Western Cascade Range	Volcanic Events
Brunhes (normal)	2	Winthrop Creek Glaciation	Garda Stade	alluvium & marine deposits	alluvium Electron Mudflow marine deposits	Garda Drift lahars	
			Burroughs Mtn. Stade			Burroughs Mtn. Drift, lahars	Rainier C
	3	"Hypsithermal interval"		alluvium, marine deposits	alluvium, marine deposits	alluvium, lahars	St. Helens Y
	4				Osceola Mudflow	Osceola Mudflow	Rainier F
	5						Rainier S Mazama O Rainier R
	10	Fraser Glaciation	Sumas Stade	Sumas Drift	alluvium	McNeeley Drift	
	12		Everson Interstade	Bellingham Drift Deming Sand Kulshan Drift	alluvium	alluvium	Glacier Peak
			Vashon Stade	recessional outwash Vashon Till Esperance Sand Lawton Clay	Vashon Drift: Steilacoom Gravel Vashon Till Colvos Sand	alluvium Vashon Drift	
			Evans Creek Stade		alluvium	Evans Creek Drift	
	20	Olympia Interglacial		"Discovery Fm" Quadra Fm	"Discovery Fm"	alluvium, lahars	
	60	Possession Glaciation		Possession Drift			
	80	Whidbey Interglacial		Whidbey Fm	Kitsap Fm		
	100	Double Bluff Glaciation		Double Bluff Drift	"penultimate drift"	Hayden Creek Drift	
Matuyama (reversed)	250 730	Salmon Springs	upper glacial		upper Salmon Springs Drift	"Mud Mountain complex"	"Lake Tapps" (Frigid Creek) tephra
	840		interglacial		Interglacial sediments, tephra	Wingate Hill Drift	
			lower glacial		lower Salmon Springs Drift		
		Puyallup Interglacial			Puyallup Fm	Lily Creek Fm	
		Stuck Glaciation			Stuck Drift	Logan Hill Fm	
		Aderton Interglacial			Alderton Fm		
		Orting Glacial			Orting Drift		
	2,000						

project in this geologic environment depends on geologic analyses to a greater extent than at many geologically simpler dam sites. Both the site selection and design elements required become extremely critical to controlling reservoir, foundation, and abutment leakage in glacial materials that range from impervious to highly pervious and have the potential for sudden and substantial failure when subjected to major changes in hydrostatic conditions.

The following case histories illustrate the varied geologic conditions encountered by existing reservoir projects associated with the Cascade ice border. Additional sites proposed for development are the focus of periodic study. Although each site is unique, some of the lessons learned from construction and operation of existing projects in this geologic environment are worthy of cognition by the designers of future projects.

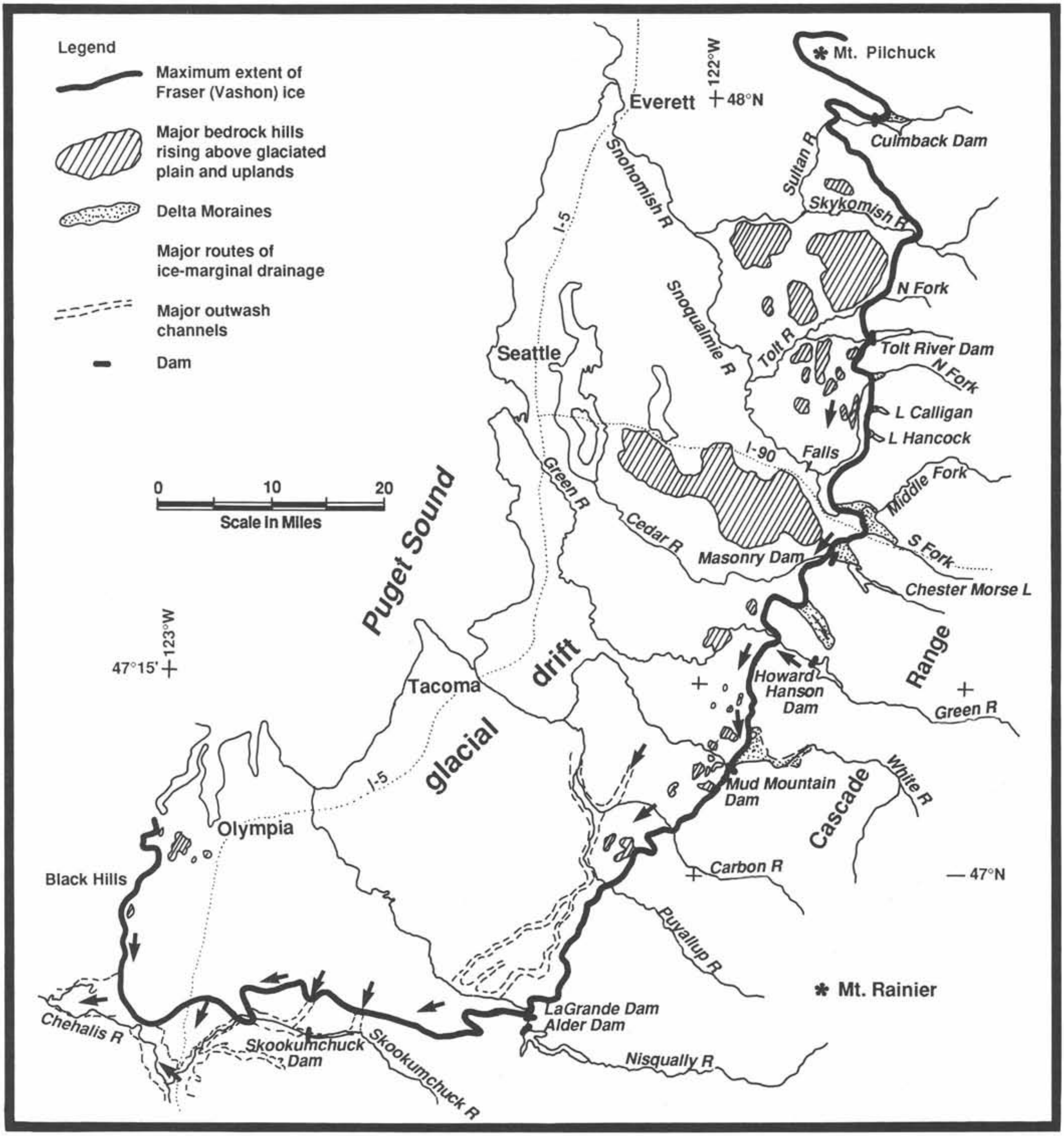

Figure 2. Cascade ice border showing the location of major delta morainal embankments, major ice-marginal channels, and dam/reservoir projects.

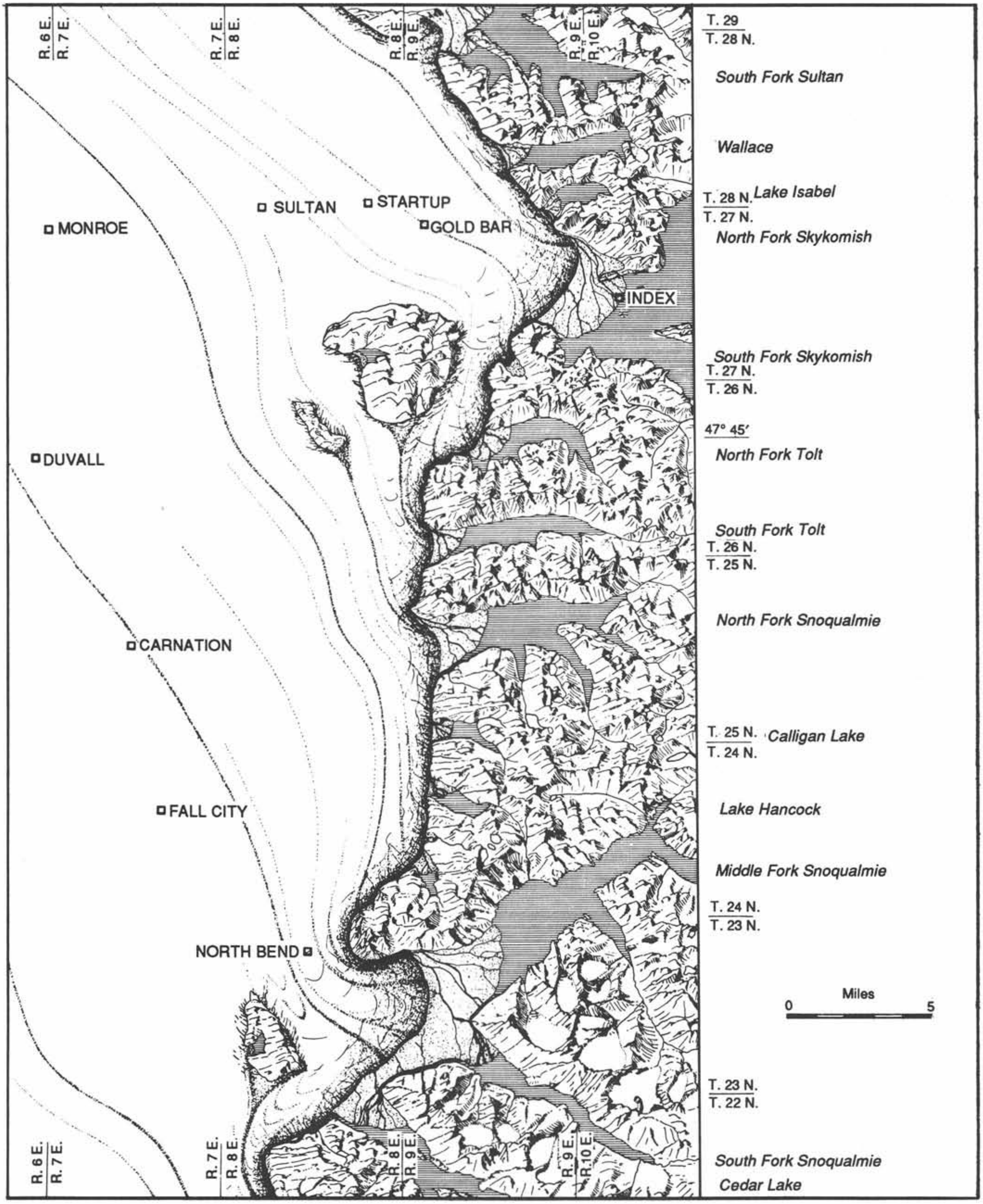

Figure 3. Central Cascade ice border at about 14.5 ka showing conceptual detail of delta morainal embankment formation and ice-marginal drainage. After drawing by A. S. Cary, 1968.

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Sultan River Project

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PROJECT DESCRIPTION

The Sultan River Project is located on the Sultan River approximately 9.5 air mi northeast of Sultan. This is a multi-purpose project supplying water to the City of Everett and power generation for Snohomish County Public Utility District No. 1. Project features include Culmback Dam, Spada Lake Reservoir, and Henry M. Jackson Powerhouse (Figure 1).

Upstream of Culmback Dam the main streams that contribute to the Sultan River are Elk and Williamson creeks and the South and Main Forks of the Sultan River, all of which head in glacial cirques above 5,000 ft. At the present time small glaciers remain only in the Williamson Creek drainage. Below Culmback Dam, the Sultan River flows approximately 14 mi west and south, joining the Skykomish River near the town of Sultan. The Skykomish River flows approximately 10 mi where it joins the Snoqualmie River to form the Snohomish River, which empties into Puget Sound near Everett (Converse Consultants, 1983).

The project has been developed in two stages, with the first phase of development being for water supply. Stage 1 development, completed in 1965, included a 200-ft-high earth and rockfill dam with a crest elevation of 1,408 ft. This phase created an active reservoir capacity of 35,600 acre-ft with a normal operating water elevation of 1,360 ft. The project also includes a low-head diversion dam approximately 6 mi downstream of Culmback Dam and an approximately 1.5-mi-long tunnel west to Lake Chaplain, where water is passed through a filtration plant on its way to the City of Everett.

Culmback Dam is a zoned embankment structure with an inclined central impervious core and upstream and downstream filter zones. The shells consist of sand and gravel with an upstream slope of 2.5H to 1V and downstream slope of 1.8H to 1V. Oversized cobbles were placed for erosion protection on the shells. The spillway is a morning-glory type and has a 74-ft crest diameter and a 34-ft diameter tunnel (Converse Consultants, 1983).

The second stage of the project was completed in 1984 for the primary purpose of increasing the storage capacity of Spada Lake Reservoir and the addition of power generation facilities. The design of the first phase of Culmback Dam incorporated provisions for raising the structure. During the second phase, the dam was raised by 62 ft to an elevation of 1,470 ft, and the level of Spada Lake Reservoir was raised by 90 ft to an elevation of 1,450 ft (Figure 2). The rise in reservoir level increased the shoreline from 8.5 mi to 17.3 mi and the surface area from 770 to 1,900 acres. The spillway was raised by 90 ft to a crest elevation 1,450 ft and modified to a new 92-ft diameter. The second phase modifications increased the reservoir storage capacity from 35,800 acre-ft to 153,270 acre-ft (Snohomish County Public Utility District, 1982).

The addition of power generation facilities in the second stage included the construction of an intake structure and a downstream powerhouse connected with the reservoir by a combination power tunnel and pipeline. The intake structure is located a short distance upstream of the dam along the south side of the reservoir. The surface-type powerhouse is located approximately 12 river mi downstream of the dam. Water is conveyed to the powerhouse through a 14-ft-diameter, 4-mi-long power tunnel and a downstream 10-ft-diameter, 4-mi-long pipeline. The powerhouse contains two 47,500-kw turbine generators and two 8,400-kw units. The two smaller units are used to produce power from water flowing from Spada Lake to Lake Chaplain (Snohomish County Public Utility District, 1982).

PROJECT GEOLOGY

The landforms of the project area are primarily the result of Pleistocene glaciation in an ice-margin environment. Bedrock configurations and landforms indicate that the upper Sultan valley, above the Culmback Dam–Spada Lake Reservoir area, was once continuous with the Pilchuck River valley.

Geology and landforms in the Sultan and adjacent Pilchuck valleys are largely the result of glaciation. The original Sultan valley, particularly its higher elevations, were modified by erosion and deposition by alpine glaciers, whereas the lower elevations were modified by continental glaciation. During the Pleistocene a lobe of a continental glacier occupied the Puget lowland. As this lobe advanced up the original Sultan valley, drainage was blocked, creating an ice-marginal lake. Eventually,

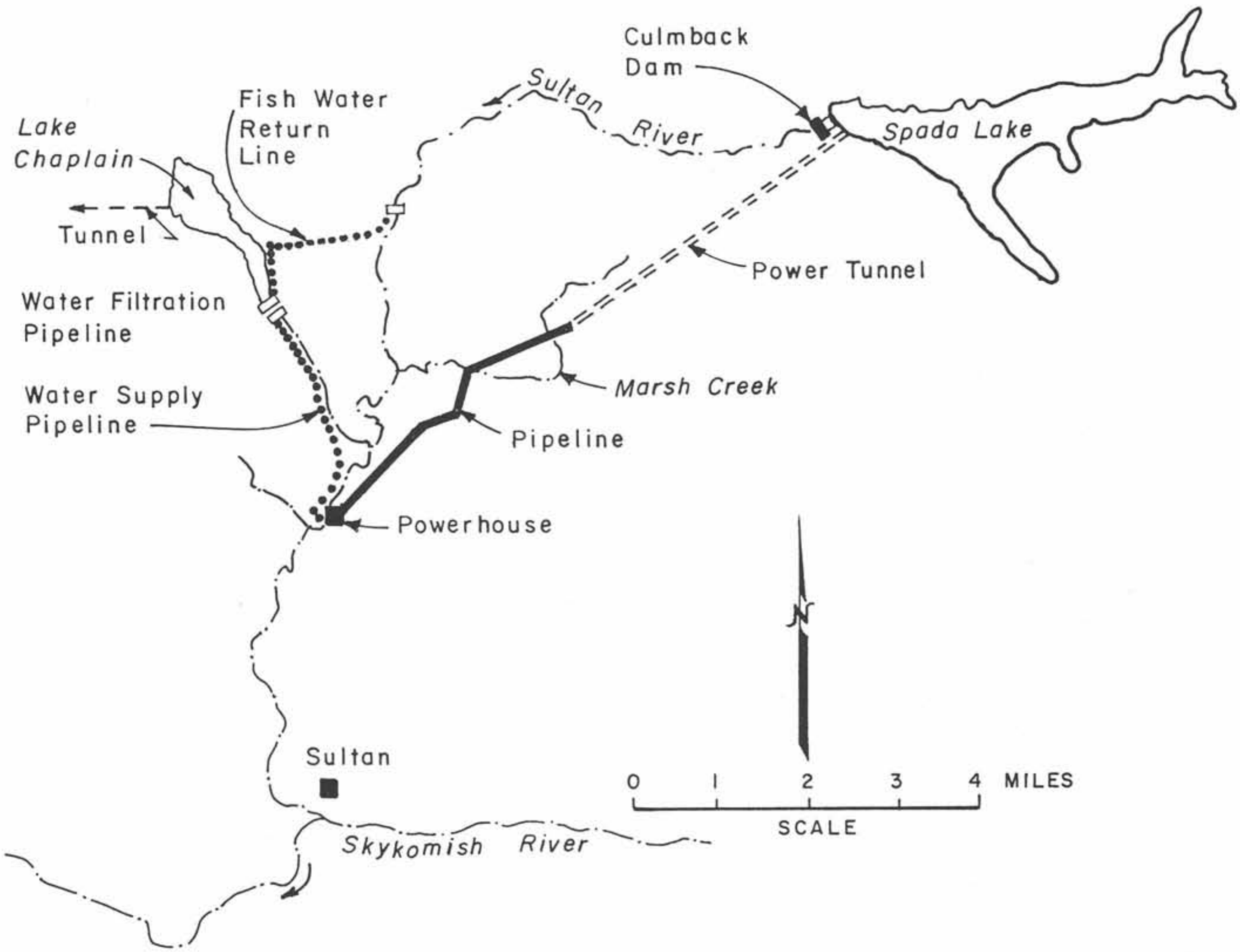

Figure 1. Sultan River project area.

a stream draining the lake along the southern edge of the valley became entrenched in bedrock and cut the steep, rock-bound Sultan River gorge. The ancestral drainage of the Sultan basin remains blocked by a thick sequence of glacial sediments referred to as the Pilchuck plug. The Pilchuck River presently drains that portion of the original Sultan valley downstream of the Pilchuck plug.

Bedrock

Bedrock in the vicinity of the Sultan River project consists of metamorphosed sedimentary rocks of Mesozoic age, including argillite, graywacke, slate, conglomerate, and minor chert. In addition, Tertiary meta-andesite, metabasalt, diorite, quartz diorite, and quartz monzonite are also present. Near Culmback Dam, the Mesozoic rocks are common and are dominated by graywacke that is massively bedded or interbedded with thin beds or laminations of argillite (Figure 3). Locally, the argillite also occurs in a thick massive sequence. Fracture spacing in both rock types ranges from extremely close to wide (Converse Consultants, 1983).

Prior to construction of Culmback Dam the Sultan River entered a steep-sided, rock-walled gorge incised between 100 and 150 ft into rock at a point a short distance upstream of the dam. Bedrock outcrops and exploration for the project indicate that the gorge was eroded on the south shoulder of the ancestral Sultan valley, at least 350 ft above the rock floor near mid-valley. Evidence indicates that the Sultan River gorge was already in existence prior to at least the last period of glaciation. Part of this evidence is in the form of glacially consolidated sediments that are typical of Vashon Stage deposits being present just above river level in the Sultan gorge a short distance downstream of Culmback Dam.

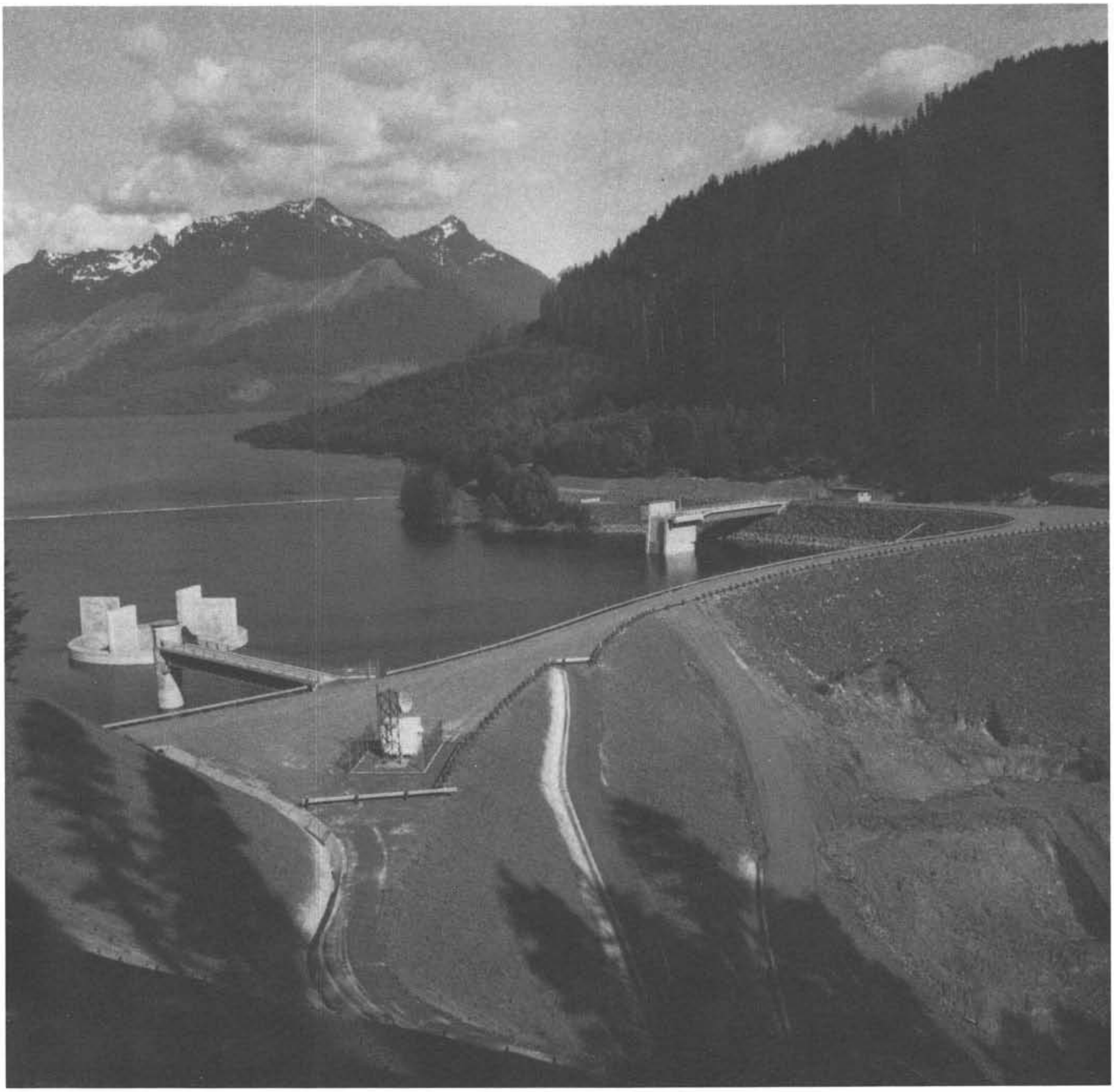

Figure 2. Culmback Dam and Spada Lake Reservoir after completion of stage 2 development. Morning-glory spillway is in the left-center of the photograph; the intake is in the right center. Photograph courtesy of R. W. Beck and Associates.

Overburden

Bedrock is exposed in the immediate vicinity of Culmback Dam up to an elevation of approximately 1,425 ft on the left abutment and approximately 1,450 ft on the right abutment (Figure 3). Above these elevations bedrock is mantled by till. On the left abutment, the thickness of till is interpreted to be relatively uniform and on the order of 25 ft. The upper approximately 10 ft of till is weathered and consists of stiff to very stiff, clayey to sandy silt and silty sand. The underlying unweathered till is very dense silty sand with rare gravel. In the right abutment area, the till increases in thickness as the bedrock surface slopes down to the north into the ancestral Sultan valley. This till is part of the Pilchuck plug.

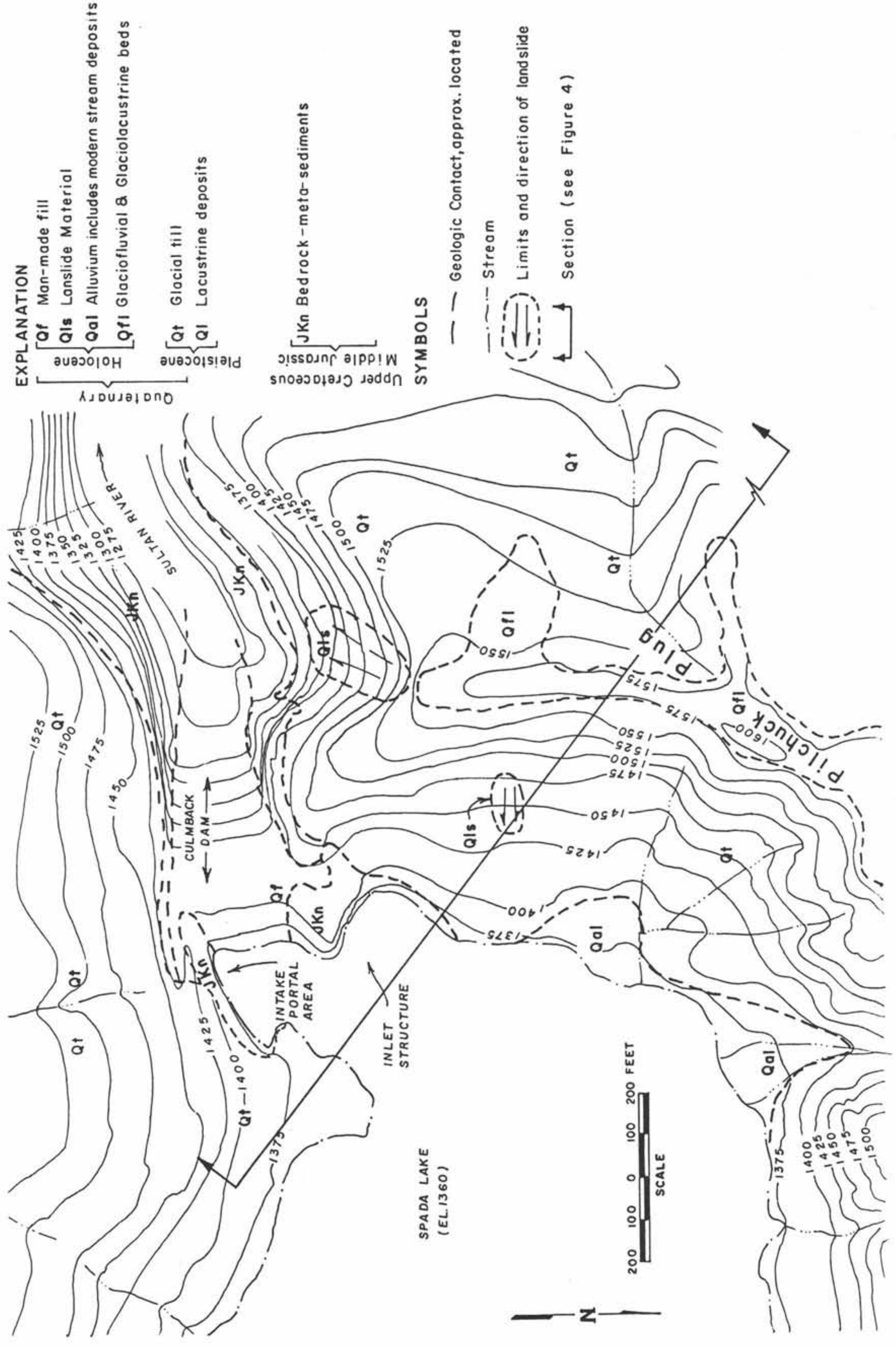

Figure 3. Generalized geologic map of the stage 2 development, Culmback Dam area. Adapted from Converse Ward Davis Dixon (1979).

The Pilchuck plug is a ridge-like feature that encompasses an area of approximately 1 sq mi and forms the extreme west shore of the reservoir and extends slightly more than a mile north and west from the dam's right abutment (Converse Consultants, 1983).

The glacial sediments near Culmback Dam and Spada Lake are of both alpine and continental glacier origin. However, the most common are the sediments related to continental glaciation. Most of the sediments that make up the Pilchuck plug are interpreted to have been deposited during the last period of continental glaciation. The western extent of the alpine glaciation can not be determined as these sediments are covered downvalley by continental glacial drift.

In view of reservoir seepage experienced by dam projects sited in similar geologic environments, an extensive geologic and geohydrologic investigation was completed prior to the construction and raising of Culmback Dam and Spada Lake Reservoir. This investigation was completed as an extension of the geotechnical investigation. These data were augmented by subsurface data acquired from investigations completed in 1962 for the first phase development.

For the area of the Pilchuck plug, available information suggests that at its deepest point near the central part of the ancestral Sultan valley, the elevation of the bedrock surface is on the order of 600 ft. A schematic geologic section through the plug area had been developed based on available data (Figure 4). The deepest deposits in the plug and ancestral Sultan valley are assumed to be alpine till and perhaps valley train deposits. Above these deposits are basal glaciofluvial gravels, which were probably subaerially deposited by high-energy streams transporting a large bed load of alpine outwash and reworked alpine till. The top of this deposit is between elevations 1,000 and 1,100 ft, and its surface slopes slightly downstream. The base level to which the uppermost part of this deposit was graded was a lake located several miles downstream whose surface elevation was about 800 ft. The glaciofluvial deposit is overlain by prodelta sands, which are lacustrine in origin and make up a major portion of the sediments of the Pilchuck plug. This unit is termed prodelta, indicating deposition ahead of the gravel delta-fan deposits, and it is distinctive for its relatively uniform sandy texture. Although predominantly sand, the unit does contain numerous thin beds and laminae of silt, clay, and gravel. The base of the unit is between approximately 1,025 and 1,100 ft. For the most part, the prodelta sands overlie the glaciofluvial gravels. The exception is the area near the right dam abutment, where the prodelta sand unit rests directly on bedrock. The lower part of the prodelta sand unit was probably deposited in response to a rapid rise in the glacially ponded lake to an elevation of approximately 1,100 ft.

The prodelta sand deposit is overlain by a thinner deposit of delta-fan gravels. The upper surface of the gravels is known to be at approximately 1,240 ft. The deposit has been interpreted as being the result of streams consistently occupying the northern portion of the valley as the lake level rose to an elevation of approximately 1,350 ft. Evidence, which includes the great thickness of the unit downstream of the mouth of the gorge, suggests that the gorge outlet was blocked by either ice or glacial deposits during the deposition of the delta-fan gravels.

The prodelta sands and delta-fan gravel deposits are overlain by a waterlaid till deposit and associated lacustrine sediments. The central part of the deposit is generally gray, hard, sandy silt or silty clay with small amounts of sand, gravel, and scattered cobbles. The lower and upper parts of the unit are typically dark gray, thinly bedded, lacustrine, clayey to sandy silts and silty clays with abundant dropstones and a few till lenses. The base of the unit is poorly defined; however, the top of the unit is well defined by an abrupt upward transition at about elevation 1,550 ft to brown lacustrine silt, sand, and gravel that lacks dropstones and till lenses. The dominant character of the unit is its till-like composition. However, where appreciable thicknesses of the unit were penetrated, bedding was generally observed. The waterlaid till unit was well exposed in required excavations in the right abutment area relative to the second-phase raising of the dam. The predominant texture in these exposures is overconsolidated, gray, clayey to sandy, slightly gravelly to bouldery silty till. Horizontal partings and faint but persistent bedding were formed by thin laminae of silty sand or silt.

The waterlaid till unit contains the first material deposited on the plug that had its primary sediment source from the continental glaciers. The deposit represents material that melted out of partially grounded and/or floating ice. Deposition likely occurred while the surface of the ice-marginal lake varied in elevation between 1,350 and 1,600 ft. Geologic mapping suggests that the north shore of Spada Lake and contiguous parts of the Pilchuck plug are covered by a substantial thickness of the waterlaid till and associated lacustrine sediments. These deposits appear to blanket the entire northern shore of both the stage 1 and stage 2 reservoirs; thicknesses range between several tens of feet to more than 100 ft. The waterlaid till unit gradually thins on the downstream side of the plug.

The youngest glacial unit in the plug area is the upper glaciolacustrine and glaciofluvial deposit, which overlies the waterlaid till. This unit was likely deposited as the lake level rose above an elevation of 1,600 ft. This unit consists of varved lacustrine clay, silt, and sand, as well as waterlaid till and outwash sand and gravel. Be-

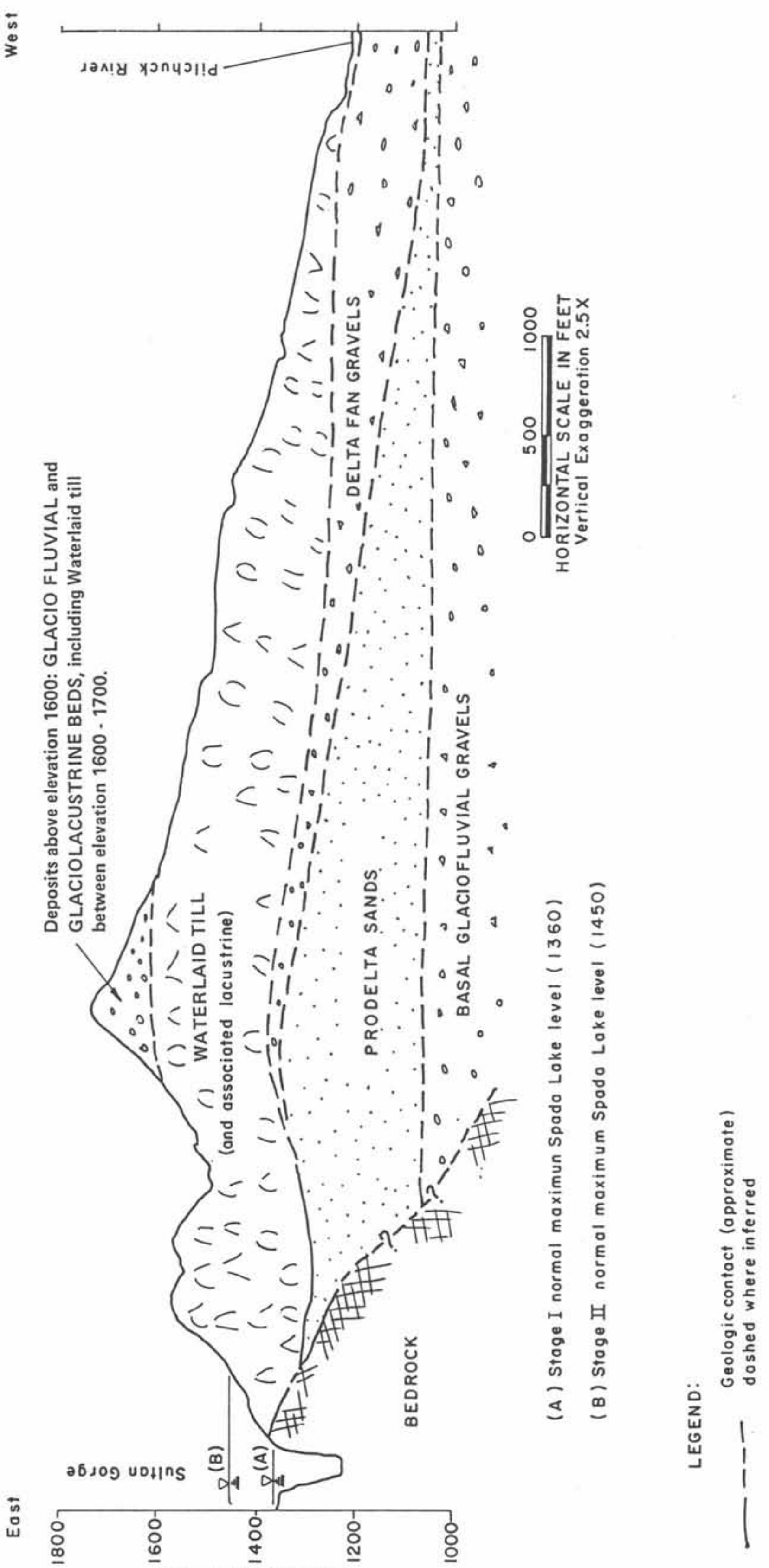

Figure 4. Schematic geologic section of the Pilchuck plug area. Adapted from Converse Consultants (1983).

cause this unit lies above the stage 2 reservoir level, it has not been studied in as much detail as the lower units.

A study of geology and landforms in the Sultan valley area indicates that the advance of the glacial lobe resulted in an increasing lake level. As the level of the lake rose, a series of topographic lows primarily along the south side of the valley provided spill areas into the adjacent drainage.

Glacial retreat or downwasting had relatively little effect on the Pilchuck plug. The main effect was erosion and downcutting of the upstream face of the plug and the deposition of various surficial deposits on both its upstream and downstream faces (Converse Consultants, 1983).

CONSTRUCTION AND OPERATION PROBLEMS RELATED TO GEOLOGY

No major construction or operation problems were reported relative to the stage 1 development. The dam at this level (crest elevation 1,408 ft) was founded entirely on bedrock. A part of the stage 2 development exploration was directed towards examining the stage 1 dam embankment to assess its performance. This involved the drilling of several borings, obtaining and testing representative samples, and installing piezometers to evaluate seepage conditions in the existing embankment. Evaluation of the phreatic conditions indicated the seepage within the embankment was occurring as planned in the original design. The evaluation of data obtained between 1964 and 1975 from a series of surface monuments installed to detect vertical settlement and horizontal deflection indicated a total vertical settlement of about 0.25 to 0.30 ft or 0.15 percent of the total dam height. Horizontal deflections of less than 2 in. were recorded. Inspection of the embankment crest and slopes prior to stage 2 construction indicated no cracking or deterioration of materials. All of the above data indicated that the stage 1 development was performing as designed.

The stage 2 development involved the raising of the existing embankment dam from elevation 1,408 to 1,470 ft and the reservoir from 1,360 to 1,450 ft. Additional facilities included an intake structure, power tunnel-pipeline, and surface powerhouse. In general, these structures were founded on bedrock. The power tunnel was excavated utilizing a tunnel boring machine and was completed with no major problems related to geology. The impervious core along the left abutment dam-raise areas was founded on bedrock after excavation of approximately the upper 5 ft of rock. The adjacent shells were founded on the natural till. The right abutment was founded on bedrock up to approximate elevation 1,450 ft. Above this elevation the embankment was founded on the till overburden, which was determined to be a suitable foundation. The landslides located on the right abutment were regraded as part of the stage 2 construction. However, there was some limited instability after construction which required remedial measures.

Although no evidence of abnormal seepage or instability was reported relative to the stage 1 development, there was concern that the rise in the reservoir surface elevation from 1,360 to 1,450 ft could result in future seepage problems. Extensive exploration in the form of borings and geologic mapping was completed in the area of the Pilchuck plug, the primary area of concern for seepage. Piezometers were installed in many of the borings in this area, and a baseline study of stream and spring flows was completed in the Pilchuck valley side of the Pilchuck plug and along the right side of the Sultan gorge a short distance downstream of the dam. Measurements were obtained from the piezometers and flow-measure points during the reservoir filling to its stage 2 level and for about 1 yr following filling. In general, these measurements indicated a slight rise in water level in the deeper piezometers prior to stabilization and little or no effect on the shallower piezometers. No increase in stream or spring flow has been attributed to the new reservoir level. In general, data obtained to date indicate that the sediments comprising the lower portion of the Pilchuck plug are relatively impervious and that the stage 2 development is performing as designed (Shaffer, 1988).

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Sultan dam site; view downstream. The diversion tunnel portal is to the right center of the photograph. The Sultan Riven was diverted by the continental ice sheet pushing upstream and forcing the river to create a new bedrock channel, which is the present canyon. The ancestral channel is to the right of the photograph. Note the smooth fault plane dipping toward the river. Glacial clays lie unconformably above bedrock and can be seen above the form work. Photograph by H. W. Coombs, 1963.

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Tolt River Project

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PROJECT DESCRIPTION

The Tolt River Project lies about 30 mi northeast of Seattle. The Tolt River drains part of the western slope of the Cascade mountains. The project provides approximately one-third of the water supply for Seattle and surrounding metropolitan areas served by the Seattle water system. The watershed contains the North and South Fork drainages; the rivers join approximately 8 mi below the South Fork Tolt River Dam. The town nearest the project is Carnation,which is 16 river mi downstream and 11 air mi southwest of the project at the junction of the Tolt and the Snoqualmie rivers (Figure 1).

The Tolt River basin above South Fork Tolt River Dam contains 18.8 sq mi of mountainous, heavily wooded terrain. Elevations in the basin range from 1,600 ft near the dam to 5,535 ft at McLain Peak. Normal annual precipitation in the basin is 120 in.

The basin was studied as a possible water source for the City of Seattle as early as 1935. Investigations of potential dam sites were started in earnest in 1952. In 1955 design studies, including site investigations, were initiated. The ultimate project development consists of a high dam impounding a storage reservoir on the South Fork and a diversion dam on the North Fork. The plan is to divert water from the North Fork into the water supply system whenever the flow exceeds the minimum flow required for fish spawning and propagation. Any deficiency of flow in the North Fork is to be supplemented by drawing water from the South Fork storage reservoir (U.S. Army Corps of Engineers, 1978).

To date, only the dam on the South Fork has been constructed in conjunction with the Tolt regulating reservoir and dams located approximately 5 mi downstream of the South Fork Tolt River Dam (Figure 1). The regulating reservoir provides a storage basin for water supply and reduces the head in the water-supply pipelines between the South Fork Project and Seattle.

The South Fork Tolt River Dam is a 200-ft-high embankment structure containing approximately 1,750,000 cy of material (Figure 2). The dam has an impervious core constructed of till and other impervious fill; upstream and downstream shells are composed of select sand and gravel fill. The upstream face is protected from erosion by 18 in. of rock spalls mantled by a 10-ft thick riprap blanket. The upstream slope and the downstream slope above elevation 1,675 ft were constructed at 2H to 1V (City of Seattle, 1981).

The dam has a crest width of 20 ft and a length of 980 ft at the design crest elevation of 1,775 ft. The dam impounds 56,000 acre-ft of water at the maximum regulated pool elevation of 1,765 ft. The spillway is a morning-glory type; it has a 40-ft-diameter mouth at crest elevation of 1,757 ft which transitions into a 18-ft-diameter vertical shaft. An 8-ft-high ring gate is used to increase the storage capacity by raising the reservoir to elevation 1,765 ft. The water-supply intake consists of a service intake tower near the toe of the upstream embankment slope and a 72-in.-diameter conduit. This conduit connects to the 54-in. South Fork Project pipeline, which conveys water to the regulating reservoir (U.S. Army Corps of Engineers, 1978).

The regulating reservoir is impounded by two dams: Tolt River West and Tolt River South (Figure 3). South Dam is a 35-ft-high earthfill structure with a crest width of 20 ft and a crest length of 320 ft. West Dam is a 30-ft-high earthfill structure with a crest width of 20 ft and a crest length of 250 ft. Both dams have a design crest elevation of 765 ft. Inflow into the basin from the South Fork Project pipeline is controlled by a 17-in.-diameter Howell-Bunger valve. The location of the regulating reservoir is at a point where water from the North Fork can also be combined when that portion of the project is developed.

GEOLOGIC SETTING

Bedrock underlying the Tolt River basin consists primarily of early Tertiary volcanic rocks that have been generally folded, faulted, and only locally metamorphosed (Tabor et al., 1982). Rocks of the Miocene Snoqualmie granitic batholith are present in the headwaters of the Tolt River (McKee, 1972). The rocks underlying the project area consist predominantly of layered volcanic flows with subordinate interbeds of volcanic breccia and tuffaceous sediments (Tabor et al., 1982). In general, the bedrock is mantled by a sequence of glacial deposits and is only sporadically exposed, primarily in the major drainages.

Although the Tolt River basin owes much of its present form to alpine glaciers that originated in the Cas-

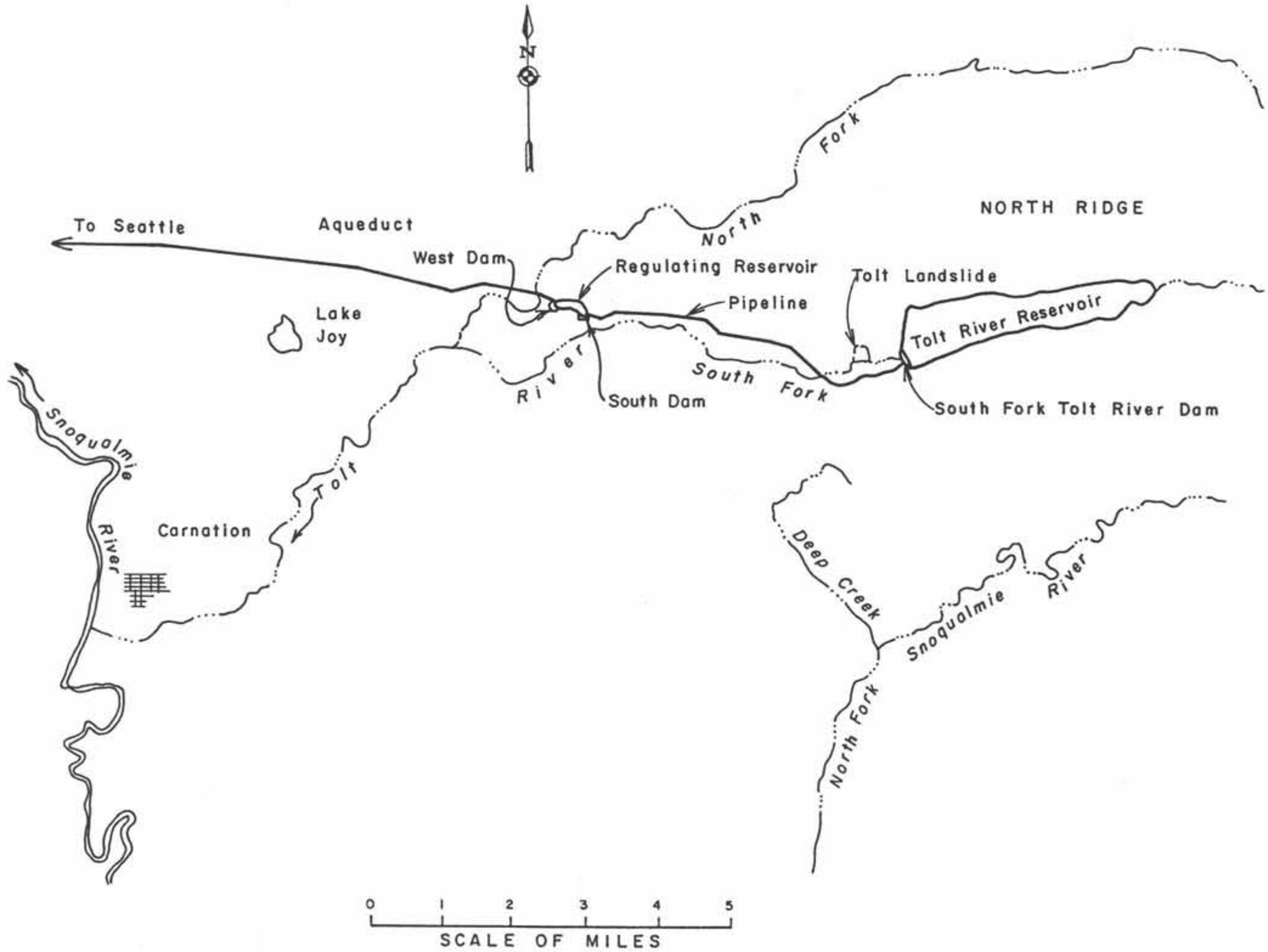

Figure 1. Tolt River project area.

cade mountains, the primary glacial influence was from the continental glacier, which advanced southward from Canada. Between 10,000 and 20,000 yr ago south-flowing continental ice blocked most of the valleys draining the northern part of the western Cascade slope. The glacier blocked the Tolt valley and deposited a thick sequence of glacial and related sediments. However, unlike the Sultan and Cedar drainages, the Tolt River did not subsequently become incised in a rock gorge.

PROJECT GEOLOGY

South Fork Tolt River Dam

At the South Fork Tolt River Dam, the river has eroded through the glacial sediments and exposed the bedrock. It is reported that bedrock was generally exposed in the reservoir area above elevation 1,800 ft except along the right side. Bedrock was exposed on the right side only at the location of the dam. In general, the bedrock surface near the dam slopes from the south valley wall north and down into the pre-dam valley bottom, then rises slightly on the north bank, and drops to a low point in the pre-glacial valley bottom somewhere beneath North Ridge (Figure 4).

Near the dam, a thick morainal complex forms the ridge referred to as North Ridge, which separates the North and South Forks of the Tolt in this area. The morainal complex is composed of four geologic units which, in ascending order, are: lacustrine, deltaic outwash, till, and outwash sediments (Figure 4) (Shannon & Wilson, Inc., 1981; U.S. Army Corps of Engineers, 1978).

The lacustrine sediments consist of a varved sequence of silts and fine sands; individual varves reach thicknesses of several inches. These deposits directly overlie bedrock or old stream alluvium and are interpreted as being deposited in the ponded drainage of the

Figure 2. South Fork Tolt River Dam; view to the south. Photograph courtesy Seattle Water Department.

Tolt valley during early blockage by the Puget lobe glacier. The top of this unit is at approximately elevation 1,600 ft at the dam axis. Permeability of the unit is very low and generally greatest in the coarser sand layers.

Overlying the lacustrine unit is a thick sequence of deltaic outwash sediments consisting of sand and gravel with various percentages of silt. Depending upon the silt content, this deposit has permeability that ranges from low to very high. The very high permeability was assigned to the parts of this unit that exhibited openwork texture.

The deltaic outwash sediments are overlain by a till unit composed of silty sandy gravel or gravelly silty sand with cobbles and boulders. The till shows crude sorting and deltaic foreset bedding. The till is reported to have an extremely low vertical permeability and low to moderate horizontal permeability, depending upon the percentage of silt present in any given section.

Overlying the till and extending to the ground surface is a section of recessional outwash deposits consisting primarily of sand and gravel (U.S. Army Corps of Engineers, 1978).

Approximately 0.7 mi downstream of the dam is an area referred to as the Tolt slide (Figures 1 and 5). Overburden sediments observed at the slide area are similar to those at the dam area. This suggested that the units have great lateral extent and raised concern relative to seepage from the proposed reservoir through the outwash sediments.

Figure 3. Regulating reservoir; view to the southwest. West Dam is in the foreground. Photograph courtesy of the Seattle Water Department.

Regulating Reservoir and Dams

The regulating reservoir lies in an abandoned glacial outwash channel that formerly joined the North and South Forks of the Tolt River. The geology of the area is characterized primarily by a complex assemblage of glacial deposits that vary widely both vertically and laterally. Bedrock consisting of andesite and basalt flows and some volcanic flow breccia is present at or near the ground surface near the South Dam and along the adjacent section of the South Fork Tolt River (Converse Consultants, Inc., 1982).

CONSTRUCTION AND OPERATIONAL PROBLEMS RELATED TO GEOLOGY

South Fork Tolt River Dam

The central portion of the dam is founded on bedrock, whereas the remaining portion is founded on glacial deposits (Figure 4). During the site investigation phase for the South Fork Tolt River Dam, concern was raised regarding the strength of and variations in the underlying lacustrine sediments. This concern resulted in the flattening of the design slopes of the dam and the right abutment and the addition of subsurface drainage. Owing to the design changes in dam slope, the morning-glory-type spillway was adopted rather than extending an originally proposed gravity spillway. A low-level outlet conduit, which was utilized as a diversion conduit during construction, exits into the heel of the spillway elbow. A 72-in.-diameter water-supply conduit connected to a service intake tower is located adjacent to the horizontal portion of the spillway conduit. All of these structures are founded on bedrock and were constructed by cut-and-cover methods (U.S. Army Corps of Engineers, 1978).

Because of the possibility that the highly permeable openwork gravels extended downstream to near the Tolt slide, attempts were made to grout this zone in at least the right abutment near the embankment interface. Downstream seepage control also included a 600-ft-long drain tunnel constructed in the right abutment and relief wells and piezometers in the tunnel invert. Horizontal drains were installed in bedrock in the downstream left abutment. An 8-in.-diameter open-jointed drain line extends from the central core of the embankment to the downstream open-channel spillway. Because of potential for seepage through the right abutment, a 5-ft-thick impervious blanket was placed on the upstream right reservoir bank during construction. The blanket extends approximately 1,000 ft upstream and from the edge of the pre-construction river up the face of the morainal ridge to an elevation of 1,675 ft.

The first filling of the reservoir began on March 26, 1962. This test filling was completed to evaluate leakage through the glacial deposits that constitute the right abutment. On June 27, 1962, during this test filling, a slide occurred approximately 3,300 ft downstream and downstream of the existing Tolt slide. At the time of the slide, the reservoir was at elevation 1,710 ft, 55 ft below the maximum normal pool elevation. On July 23, 1962, the reservoir was drawn down, and additional seepage control measures were considered.

As a result of analysis of the slide and seepage, the upstream impervious blanket was extended upstream an additional 2,000 ft and its upper level raised to elevation 1,768 ft. The extended blanket has a minimum thickness of 5 ft, and it is protected by a 3-ft thickness of riprap between elevations 1,692 and 1,768 ft. In addition, a drainage system consisting of random fill was placed behind the impervious blanket along with drains to relieve hydrostatic pressures behind the blanket during drawdown. Additional seepage control was installed downstream in the form of buried perforated drain pipes, piezometers, and weirs (U.S. Army Corps of Engineers, 1978).

After the construction of the additional seepage control, water was again impounded. In 1967, a relatively large slide occurred within the main Tolt slide, resulting in the temporary damming of the Tolt River and lateral shift of the active stream channel away from the slide area. In 1976, an additional slide occurred within the limits of the main slide (Galster and Olmsted, 1977). A plan of the slide area and limits and dates of activity are shown on Figure 5.

Investigations of the landslide generally concluded that the present period of instability is related to the Tolt Reservoir. Aerial photographs from 1958 and prior to dam construction reveal the presence of the slide. Initial instability is attributed to erosion of the underlying weak, erosion-prone lacustrine deposits that resulted in block failure of the overlying denser sediments. Progressive failure has continued at the head of the slide area and has probably resulted in restriction of flow from the openwork gravels, resulting in a buildup of hydrostatic pressures in the sediments. Measured spring flows in the slide area indicate a good relation to the water elevation in the Tolt Reservoir. Although the springs were present prior to the development of the reservoir, the post-reservoir spring flow is two to three times that of the pre-reservoir flow (U.S. Army Corps of Engineers, 1978; Galster and Olmsted, 1977).

The slides downstream of the South Fork Tolt Dam continue to be a major concern. The worst case would be a major landslide blocking the Tolt River, creation of a lake, and sudden drainage by the breaching of the weak slide mass, resulting in uncontrolled downstream flow.

Regulating Reservoir and Dams

The outwash channel contained a considerable thickness of Holocene organic and soft soils, which were excavated during construction. Approximately 600,000 cy of unsuitable material was reported to have been excavated and wasted. After excavation and shaping, pervious areas in the reservoir were blanketed and the slopes riprapped to control erosion. An underground collection system was constructed in the area of springs, which are reported to furnish an additional estimated 1 million gal/day to the reservoir (U.S. Army Corps of Engineers, 1978).

South Dam is constructed in a narrow rock valley. The embankment is founded on rock in the abutment areas and on till in the bottom of the valley after removal of unsuitable soils. The West Dam abutments are founded in part on glacial outwash materials and in part on compact till and pervious sand and gravel underlain by hard clay. No bedrock was encountered. A large landslide exists along the North Fork of the Tolt River directly west of the West Dam. The site of the West Dam was moved east approximately 200 ft so as to provide additional distance between the landslide and the dam (U.S. Army Corps of Engineers, 1978).

Seepage control at both dams was accomplished by construction of an impervious blanket on the upstream face of the dams and on the bottom and sides of the reservoir.

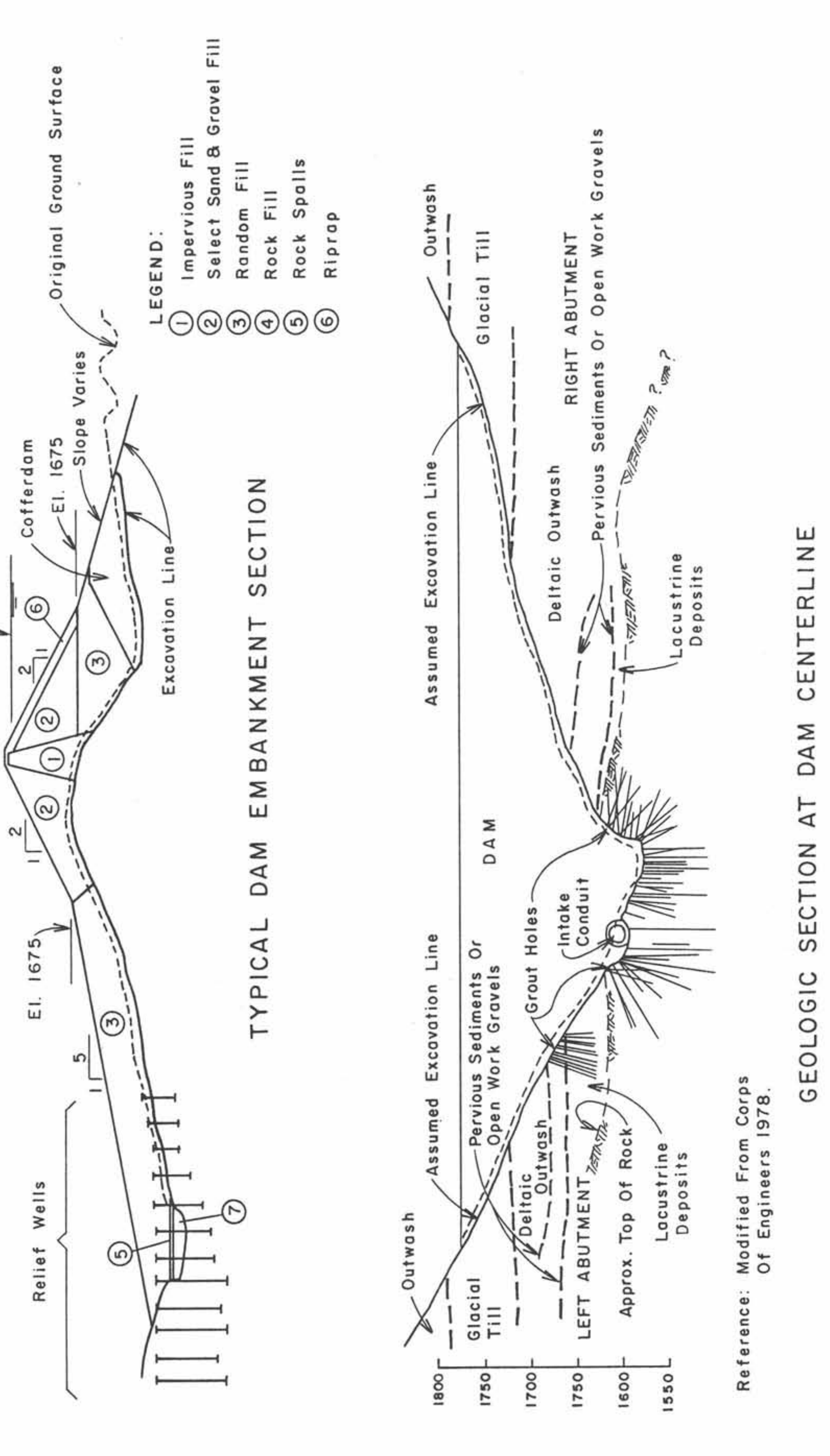

Figure 4. Embankment and geologic sections, South Fork Tolt River Dam.

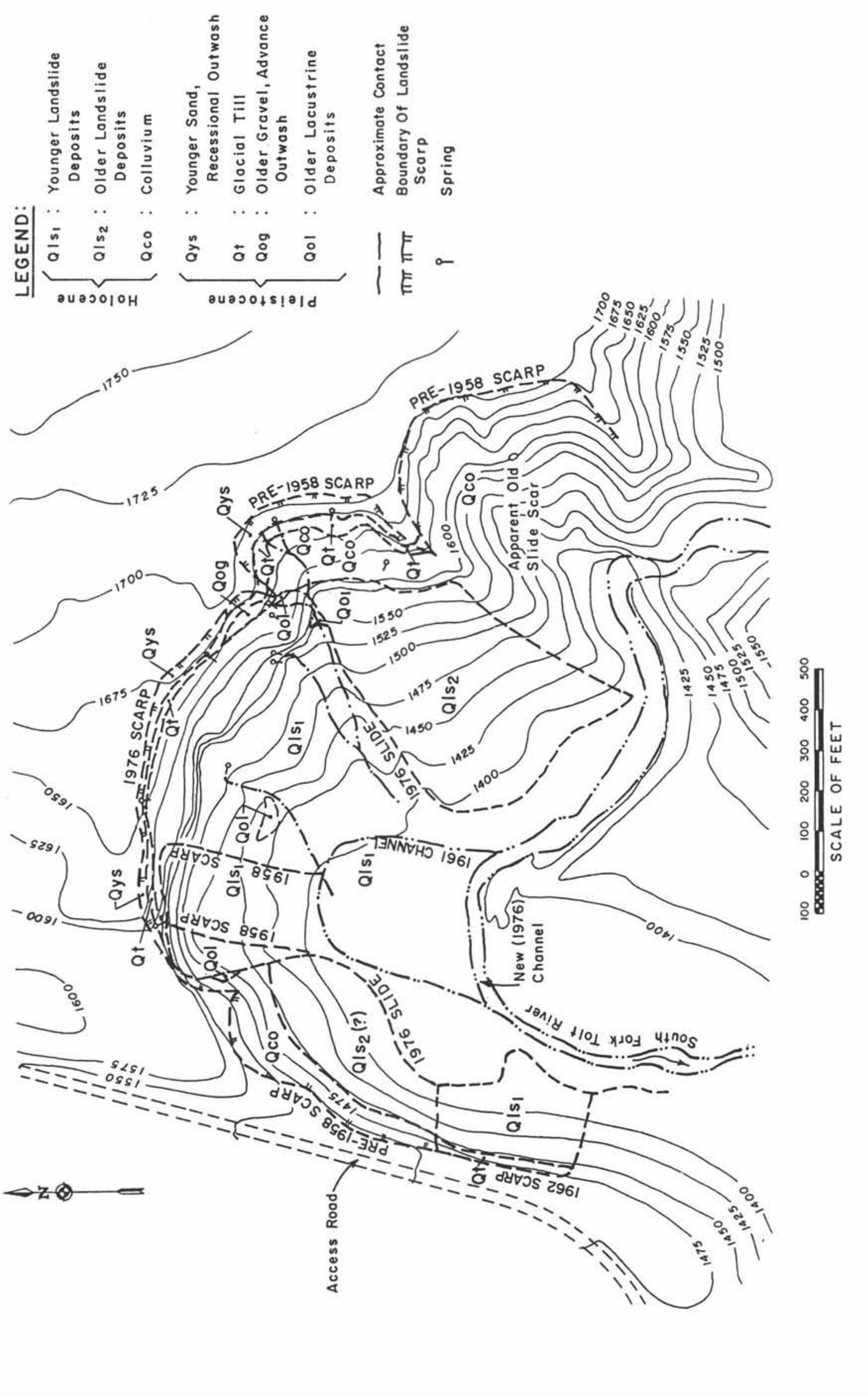

Figure 5. Generalized geologic map of the landslide area near the Tolt River project. Adapted from Galster and Olmsted, 1977.

There have been no reported major construction or operational problems with the regulating reservoir as a result of geologic factors. Some minor slumping of the impervious blanket was reported in the late 1960s.

FUTURE DEVELOPMENT

During the early 1980s, studies were undertaken to investigate the potential for addition of hydroelectric generation to the South Fork Tolt River Project. The proposed scheme consisted of a new 5-mi-long, 66-in.-inside-diameter pipeline paralleling the existing pipeline with a powerhouse located at the regulating reservoir. Water would be conveyed from the South Tolt Reservoir through the new pipeline to the powerhouse and then discharged into the regulating reservoir. The original pipeline would be maintained in a standby status. The powerhouse was designed to contain a single 15-MW Pelton turbine-generator. To this date, the power-generation facilities have not been added to the development (Converse Consultants, Inc., 1982).

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Cedar River Project

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PROJECT DESCRIPTION AND HISTORY

The Cedar River heads in the Cascade Range and flows generally westward. It empties into the southern end of Lake Washington at Renton (Figure 1).

Development of the Cedar River was started in 1900 with the construction of a dam and diversion works near Landsburg, approximately 12 mi southeast of Renton and approximately 11 mi downstream of Cedar Falls (Figure 1). The purpose of this project was to provide a municipal water supply for the City of Seattle. The first water from this system was delivered in 1911 through a gravity pipeline to Seattle (Hidaka and Garrett, 1967).

Later development of the Cedar River was concentrated in the upper reaches, specifically in the area upstream of Cedar Falls, which is located approximately 25 mi southeast of Seattle. A natural lake, Cedar Lake, drew early interest as a possible water supply. The large potential storage in the lake combined with approximately 600 ft of fall in the river below the lake suggested consideration for hydroelectric development. Investigations for the purpose of hydroelectric power were initiated as early as 1895.

Initial development at the dam site started in 1902 and consisted of a power plant at Cedar Falls. The headworks was a low timber crib dam with earthen dike extensions located approximately 1/2 mi below the natural outlet of Cedar Lake. The dam raised the lake level 13 ft to an elevation of 1,543 ft in 1904; the lake subsequently was renamed Chester Morse Lake. The crib dam was raised to elevation 1,549 ft in 1909. Part of the crest was washed out in 1911, and the dam was rebuilt to a spillway elevation of 1,546 ft and a spillway crest length of 108 ft (Figure 2). The raising of the lake provided approximately 25,000 acre-ft of storage in addition to the 33,500 acre-ft in the natural lake. The water was carried approximately 3 mi downstream to the powerhouse at Cedar Falls through 48- and 68-in.-diameter wood-stave penstocks (U.S. Army Corps of Engineers, 1979).

In 1910 Masonry Dam was proposed. The location was to be approximately 1.5 mi downstream of the crib dam, and the Cedar Falls powerhouse was to be enlarged. The dam was designed to have a spillway crest elevation of 1,605 ft and, at that elevation, would provide an additional 125,000 acre-ft of storage capacity. Subsequently, the dam was completed in 1914 with a crest height of 1,600 ft. The structure consists of a 230-ft-high concrete gravity dam with a crest length of 980 ft (Figure 3). The construction consisted of cyclopean concrete, defined as including large cobbles and boulders added to the concrete during placement. In the left abutment, the structure has an uncontrolled spillway channel with a crest elevation of approximately 1,588 ft and a width of approximately 100 ft. The dam was constructed to provide 60 ft of head above the level of Chester Morse Lake (elevation 1,530 ft) and 44 ft above the crib dam (elevation 1,546 ft). However, to raise the pool elevation to the design elevation of 1,590 ft, an earthen dike would have been required on the right abutment where the ground surface had an elevation of approximately 1,586 ft. This dike, although designed, was never constructed. For this apparent reason, a 37.5-ft-wide notch was left near the center of the dam; this functioned as a service spillway with an elevation of 1,554.6 ft. The pooling of reservoir water was started in December of 1914. When the water surface at Masonry Dam is below 1,546 ft, a separate pool is formed between Masonry Dam and the upstream crib dam; this is referred to as the Masonry Pool. When the water surface is higher than 1,546 ft, water rises over the crib dam, and the water level is continuous with Chester Morse Lake (U.S. Army Corps of Engineers, 1979).

An intake structure was built on the face of the Masonry Dam. Water is conveyed through an 11-ft-diameter concrete-lined tunnel. The initial tunnel alignment is parallel to and immediately upstream of the dam face; it turns downstream beneath left abutment and has a total length of about 1,300 ft. Downstream of the tunnel, water is conveyed by twin 6-1/2-ft-diameter penstocks to the powerhouse.

SITE GEOLOGY

General

The project lies along the southern margin of a deep re-entrant into the Cascade Range known as the Snoqualmie embayment. The most important element of the project geology relates to the deposition of a delta morainal embankment (or ice contact delta) across the mouth of the Cedar River valley by the Puget lobe of the Pleistocene continental ice sheet pushing up the em-

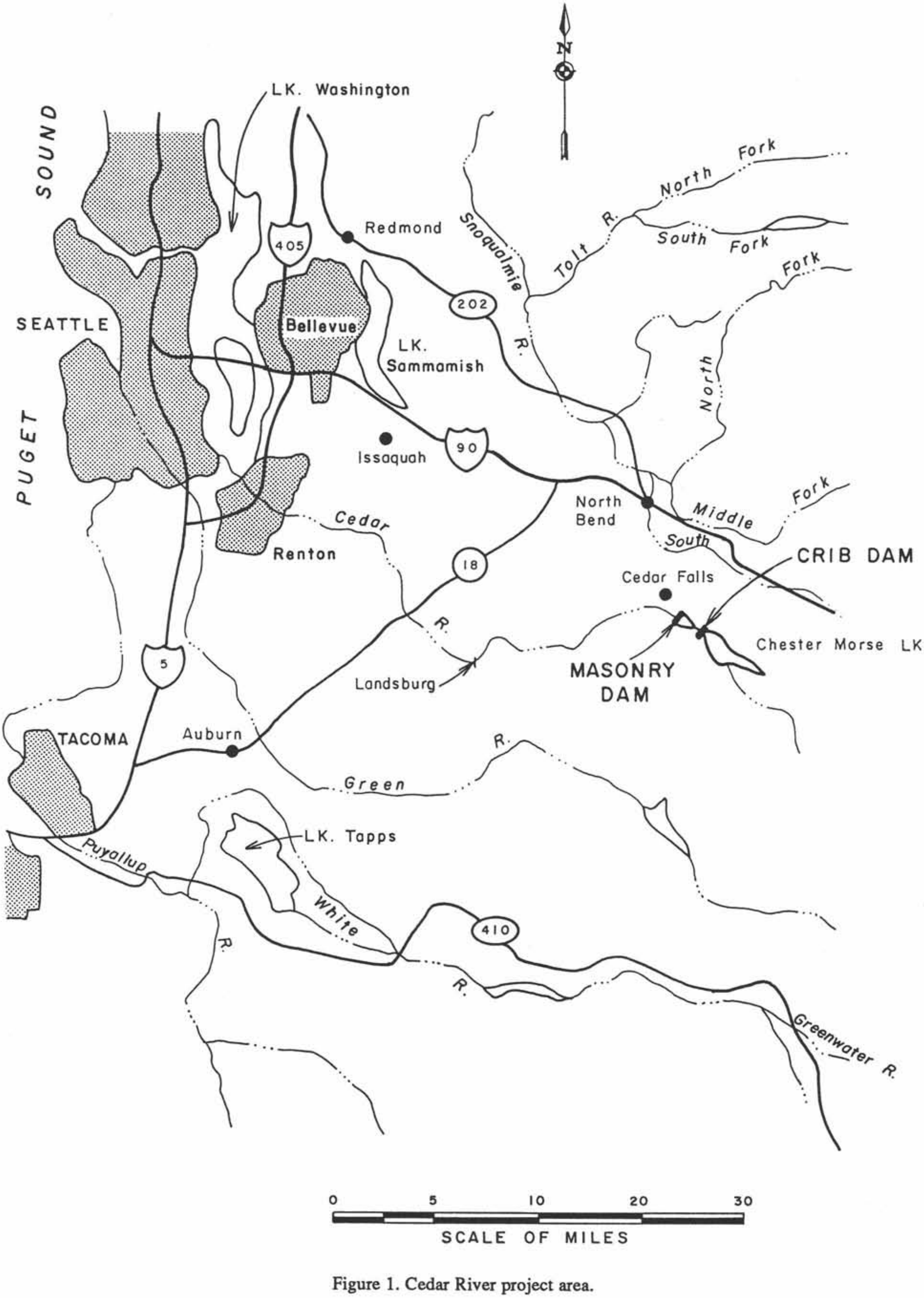


Figure 1. Cedar River project area.

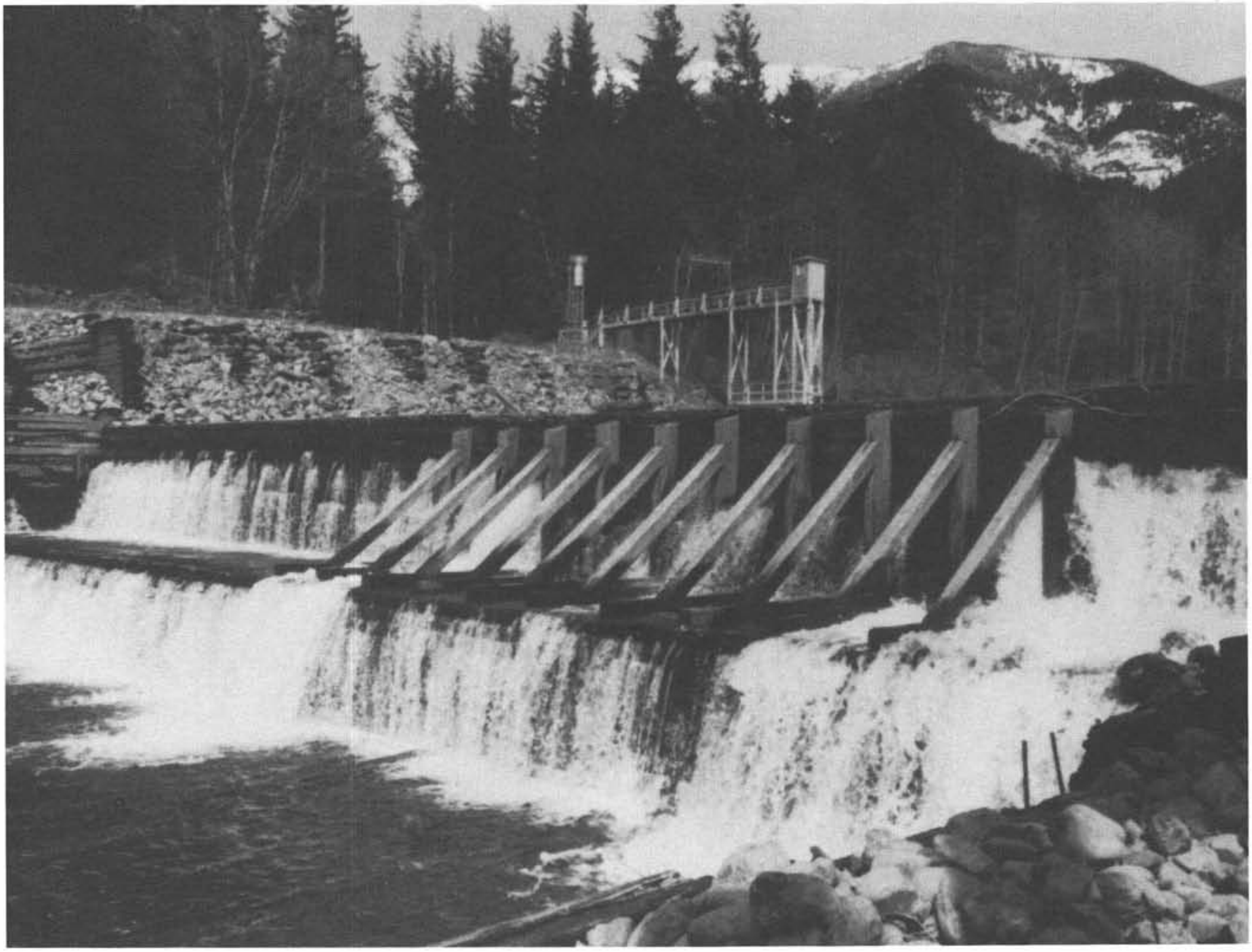


Figure 2. The Cedar River crib dam shortly before it was replaced by a new concrete overflow structure in 1986. Photograph courtesy of R. W. Beck and Associates.

bayment. As the embayment was invaded by ice, west-flowing streams were ponded against this advancing ice front, and fine-grained lacustrine sediments were deposited in the resulting lakes. As the glacier advanced up the valley, it deposited coarser outwash over the lacustrine sediments and forced the lake to attain higher elevations. Eventually, a new drainage path was established by an ice-marginal stream along the southern edge of the glacier and the adjacent mountain side. Subsequently, this stream became entrenched in bedrock, creating the Cedar River gorge (Mackin, 1941).

When the glacier retreated approximately 10 ka, the thick sequence of glacial deposits, referred to as the Cedar delta morainal embankment or Cedar embankment, remained as a barrier between the upper and lower portions of the original drainage. The upper surface of the embankment is a relatively flat surface that has a nominal elevation of 1,600 ft. The lacustrine deposits near the Cedar Lake outlet consist primarily of silt and clay that form an effective seal against seepage from the lake, even in its elevated position as Chester Morse Lake. Much of the lake is contained in a deep valley modified by alpine glaciation that pre-dated the deposition of the Cedar embankment. Unlike the morainal features in the adjacent Snoqualmie valley, which have been subsequently breached by erosion, the Cedar embankment remains essentially as it was deposited. This was primarily the result of the control of the gradient of the Cedar River by the resistant rock in which it had become entrenched (Figure 4).

Masonry Dam

The Masonry Dam is located where the Cedar River enters the narrow rock-bound Cedar River gorge. The bedrock underlying the dam site consists primarily of

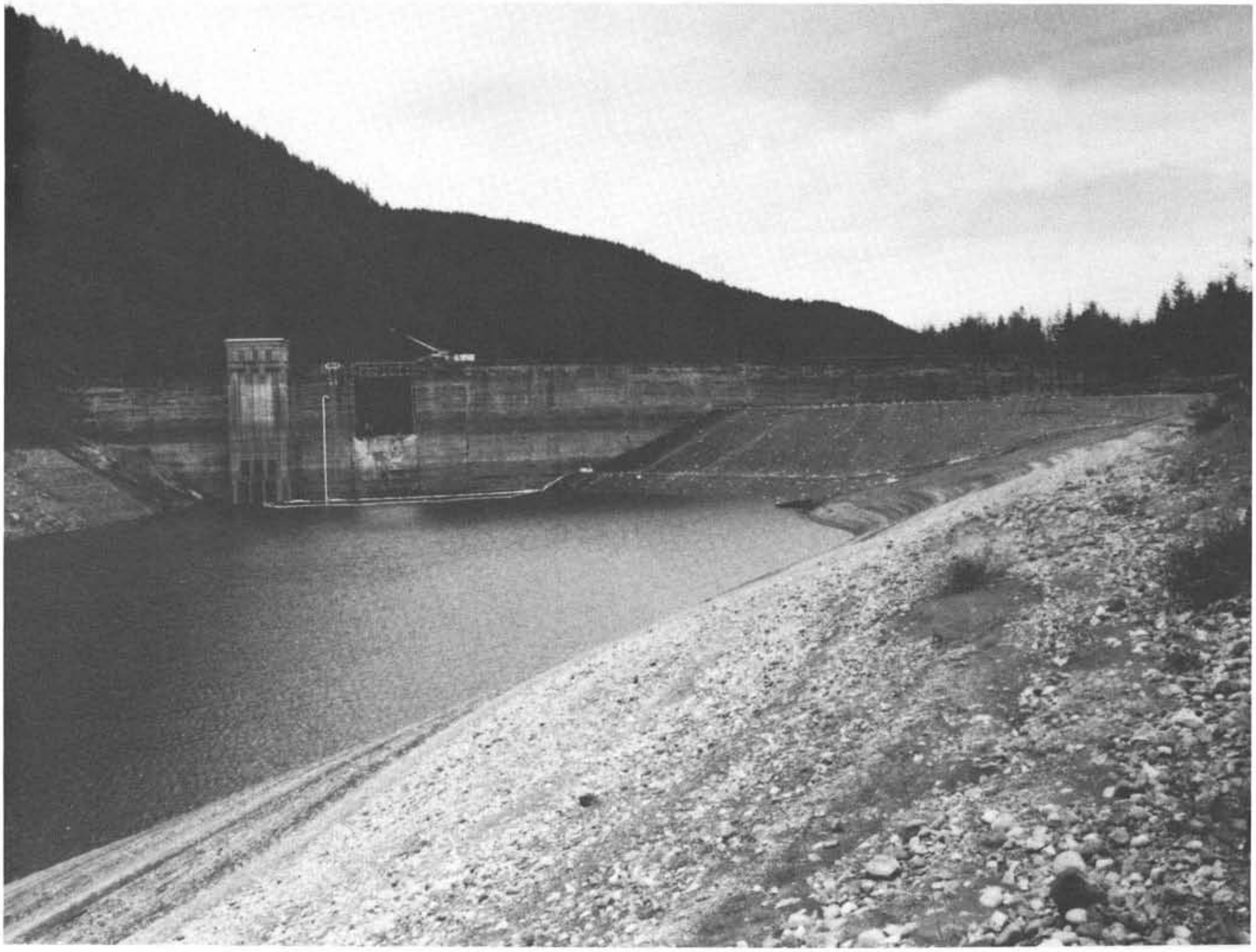


Figure 3. Masonry Dam; view downstream. The cofferdam in the right abutment area is related to construction of an emergency spillway in 1986. Photograph courtesy of R. W. Beck and Associates.

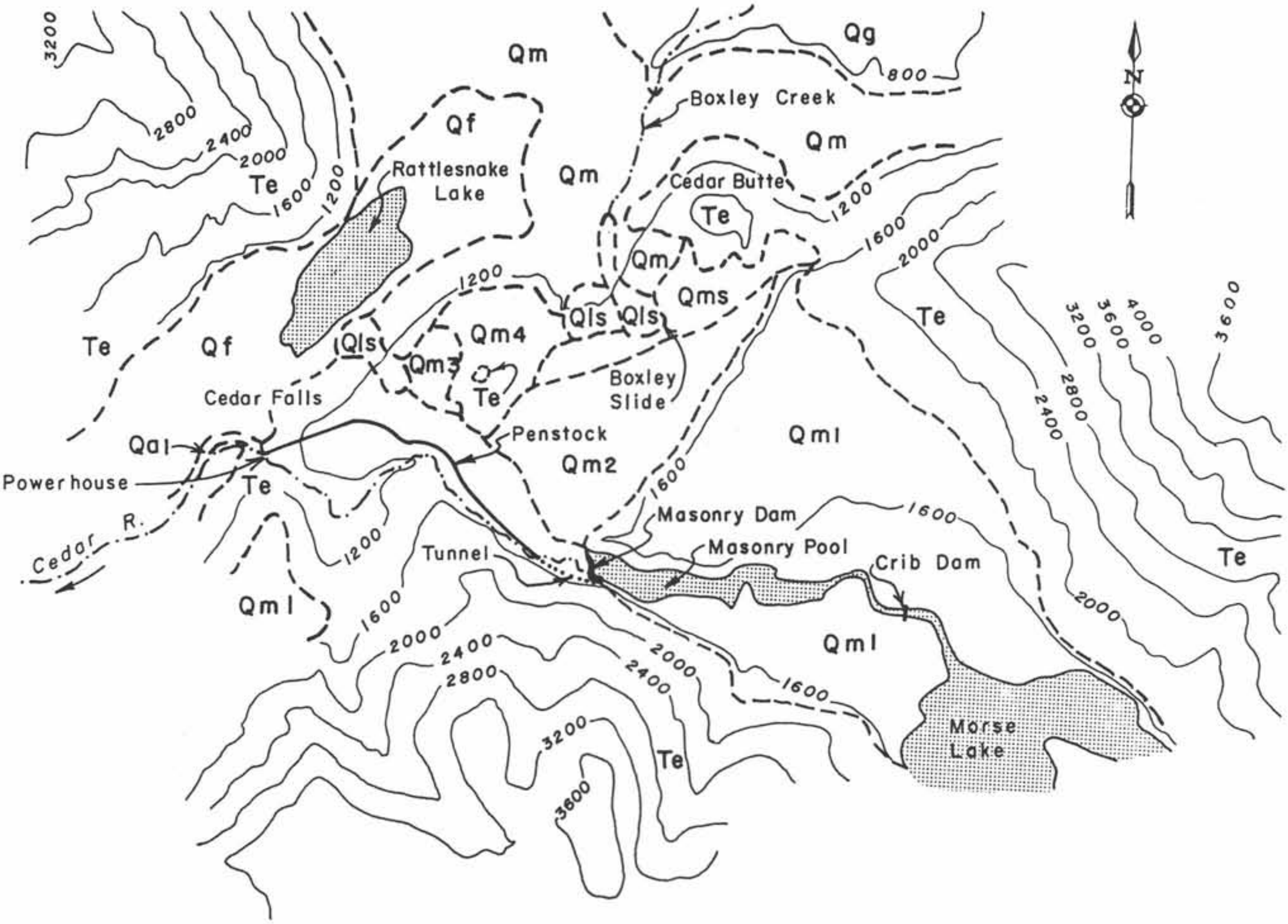
volcanic breccia and meta-andesite with some chert bands. These rocks belong to the Tertiary Enumclaw formation (see Figure 4). In general, the rock is medium hard to hard and slightly to moderately weathered and has very close to medium fracture spacing. Some of the fractures have clay and iron oxide infilling; others have been healed with calcite.

Bedrock is exposed along the southern side of the Masonry Pool and along most of the shoreline of Chester Morse Lake. Along the south side of Masonry Pool, the volcanic flows and interbedded sediments strike approximately N 40° to 50° W and dip 40° to 50° SW. Near the dam, two primary and one secondary joint systems are present. The primary joint sets strike approximately N 75° E, dipping 70° N to vertical, and N 20° W, dipping between 60° and 70° S. The secondary joint set strikes approximately N 50° W, dipping between 50° and 60° S.

The dam is located on a narrow bedrock high that slopes steeply toward the crib dam and Chester Morse Lake and northwest toward the central part of the preglacial valley (Figure 5). Exploration completed in 1982, which included seismic refraction, electrical resistivity sounding, and drilling, indicated that northeast of the Masonry Dam the unconsolidated sediments in the Cedar valley have a maximum thickness on the order of 800 ft. The upper 200 to 300 ft of the deposits consist of sandy gravel and gravelly sand. These sediments are interpreted to be underlain primarily by silty sand, silt, and clay (Converse Consultants, Inc., 1983).

Crib Dam

The crib dam is located in an area of thick proglacial sediments. Seismic refraction studies indicate that these sediments have a maximum thickness on the order of 500 to 600 ft. Borings indicate that at least the upper ap-



LEGEND:

Holocene	Qal	:	Modern River Alluvium.
	Qls	:	Landslide Deposits.
Pleistocene	Qf	:	Valley Fill Deposits Primarily Sand & Gravel.
	Qr	:	Recessional Outwash Deposits.
	Qm	:	Morainal Deposits With Terrace Levels 1 = Elev. 1600, 2 = Elev. 1560, 3 = Elev. 1480 & 4 = Elev. 1360.
	Qg	:	Glacial Sediments Undifferentiated.
Tertiary	Te	:	Bedrock: Keechelus Volcanic Group-Enumclaw Formation.

Figure 4. Generalized geologic map of the Cedar River project area. Adapted from Converse Consultants, Inc., 1983. Note: The Enumclaw formation was named by P. E. Hammond in 1963 in "Structure and stratigraphy of the Keechelus volcanic group and associated Teriary rocks in the west-central Cascade Range, Washington", a 264-p. University of Washington doctoral thesis. The term is not currently in use.

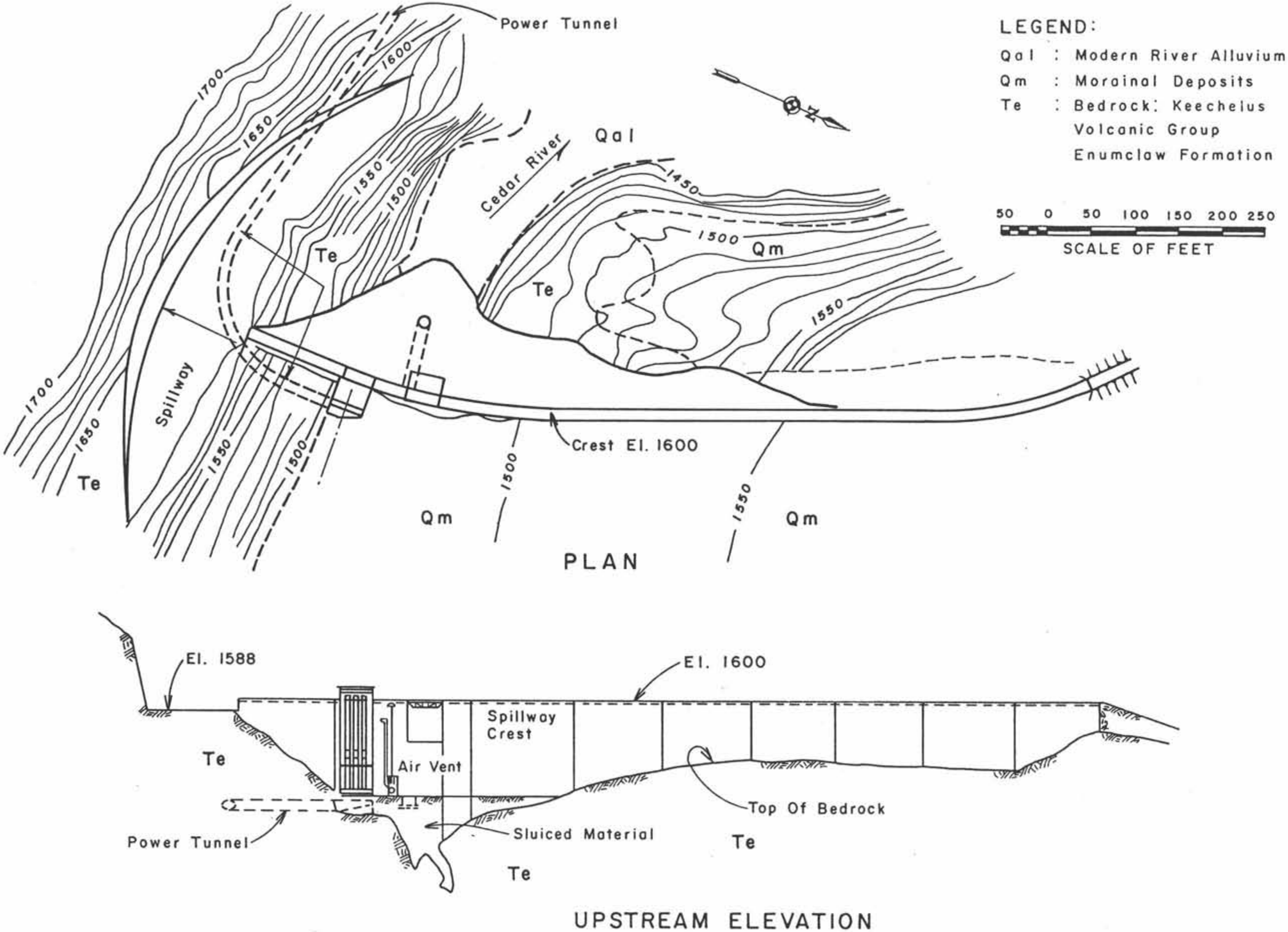


Figure 5. Generalized geologic map and section of Masonry Dam. Adapted from Converse Consultants, Inc., 1983.

proximately 75 ft of the sediments consist primarily of sand and gravel with increasing silt at depth (Converse Consultants, Inc., 1985).

CONSTRUCTION AND OPERATION PROBLEMS RELATED TO GEOLOGY

Prior to the construction of the Masonry Dam, Deans Landes and Roberts of the University of Washington were asked to report on the feasibility of the dam site. They had observed open-textured gravels in the area between the Crib Dam and the proposed Masonry Dam site and recognized them as being part of the morainal deposits occupying the valley. Pointing out the potential for high seepage from the reservoir, they recommended a preconstruction investigation of these deposits. Apparently, those recommendations were largely disregarded, and the construction of Masonry Dam proceeded (Mackin, 1941). The dam was completed in October 1914, and the water level in the Masonry Pool was permitted to rise to an elevation of 1,524 ft, equivalent to a water depth of 80 ft at the upstream dam face. The pooling of the reservoir water was accompanied by a great increase in spring discharge in the gorge downstream from the dam and along the base of the north face of the Cedar embankment northeast of Cedar Falls and approximately 6,000 ft from the Masonry Pool. After 30 days the water had been essentially drained from the pool by seepage; the calculated rate of loss was 30,000,000 gal/day. In the spring of 1915 the pool was permitted to fill to a level of 1,538 ft, approximately 8 ft below the level of the spillway crest of the crib dam. This again resulted in increased discharge from the downstream springs and caused the level of Rattlesnake Lake, a small lake near the base of the moraine, to rise from 868 ft to 881 ft between May 1 and May 8. This flooded the small town of Moncton (population 200), and the townsite was subsequently moved to Cedar Falls.

The seepage problem prompted a subsurface investigation. Test holes along the crest of the Cedar embankment and along the northwest side of the pool area encountered open-textured gravelly and bouldery deposits; holes near the dam penetrated rock at depths between 160 and 220 ft. Borings farther east failed to reach bedrock at depths on the order of 300 ft. Some of the gravel deposits had an estimated void ratio of as much as 35 percent. The conclusion from this investigation generally matched the early observations of Deans Landes and Roberts: extensive deposits of very permeable glacial deposits are present (Converse Consultants, Inc., 1983).

In 1916, an 11-ft-diameter tunnel was constructed through the dam, and the river was allowed to flow uninhibited. Between 1916 and 1918 various sealing operations were undertaken. These consisted primarily of sluicing of finer material onto the sides and lower reaches of the Masonry Pool. In the fall of 1918, water was allowed to rise in Masonry Pool to an elevation of 1,556 ft; this level was maintained between December 16 and 24, 1918. Seepage losses during the first part of the reservoir filling were estimated to have been on the order of 500 cfs and probably greater as the pool level was raised. Again, the pooling of water was marked by an increase in downstream spring discharge. Between midnight and 2:00 a.m. of December 23, 1918, a great outburst occurred in the eastern portion of the north face of the Cedar embankment approximately 6,000 ft from the dam. Saturated debris aggregating between 800,000 and 2,000,000 cy was washed northeastward into the drainage of the South Fork Snoqualmie River. This outburst, referred to as the "Boxley burst", occurred in an estimated 20 min to 2 hr and left a great amphitheater-shaped indentation in the embankment face. Discharge of water at the time of the initial outburst was estimated at between 3,000 and 20,000 cfs. The flood of debris moved northeast, destroying the tracks of the Milwaukee Railroad, the small town of Edgewick, and several sawmills and associated structures. After the outburst, spring discharge from the amphitheater quickly decreased to an estimated 250 to 300 cfs at 9:00 a.m. on December 23. The water level in Masonry Pool gradually lowered to elevation 1,548 ft on January 2, 1919, to 1,503 ft on January 17. The outburst resulted in lawsuits against the City of Seattle, which were eventually terminated in judgment against the city (Mackin, 1941).

In 1920, a proposal was recommending the construction of a rockfill dam with a crest height of 1,555 ft at the location of the crib dam and the abandonment of the Masonry Dam and Masonry Pool. This plan was not acted upon, and the project has remained as originally constructed, with the exception that the Masonry Pool is generally regulated at a lower elevation than originally intended.

SUBSEQUENT INVESTIGATIONS AND PROJECT MODIFICATIONS

Several geologic and hydrologic investigations of the project have been completed since the project was initially developed. The hydrologic studies generally examined the reasons for and quantities of seepage that occur at various pool levels in the Masonry Pool. Geologic studies generally emphasized the geologic history and relations (Mackin, 1941).

The original designers of Masonry Dam apparently assumed that the Cedar embankment was formed by a glacier advancing down the preglacial Cedar River drainage, that the embankment was composed primarily of impervious till, and that the open-textured gravels along the right bank of the Masonry Pool had limited extent. Evidence likely to have been interpreted as supporting this theory was the existence of the natural Cedar Lake. The purpose of Mackin's investigation was to obtain and review data to determine the characteristics and depositional history of the Cedar embankment.

The neighboring valleys of the Middle and South Fork Snoqualmie rivers contain morainal embankments whose position, size, and elevation are comparable to those of the Cedar embankment. Evidence that indicated a similar origin for these embankments included similar pebble rock types and degrees of weathering. Some of these pebbles are foreign to the upper reaches of the drainages, especially the Cedar drainage, which is underlain by volcanic rocks and some granitic intrusions in its upper reaches. The exotic rocks were correlated with outcrops to the north and in the lower reaches of the Snoqualmie River drainage. This evidence provided proof that all three embankments were formed by the same glacier moving south up the Middle and South Fork Snoqualmie River and into the Cedar River drainages (Mackin, 1941).

The embankments in the Middle and South Forks of the Snoqualmie River now differ from the Cedar embankment in that the Snoqualmie embankments have been eroded extensively, providing excellent exposures of the deposits making up the embankments. These sediments offer additional evidence that these embankments were formed by a glacier moving up the valleys, forming lakes that were subsequently drained. Mackin referred to the entire complex as the Snoqualmie embayment and to the glaciers which formed the embayment as tongues of the Puget glacier. The Cedar embankment remains essentially in its original form and uneroded because the Cedar River became entrenched in bedrock, which controlled its rate of downcutting (Mackin, 1941).

In addition, Mackin investigated the north face of the embankment, which separates embankment proper from the lower elevations of the Cedar River valley. This investigation concluded that the north face of the embankment had a mantle of till over the outwash deposits. In addition, the face is characterized by three distinct terraces occurring at elevations of approximately 1,560, 1,480, and 1,360 ft; in addition, a terrace at 1,600 ft forms the general surface of the embankment. These terraces were interpreted as erosional features produced by meltwaters when the glacier maintained various stands during its retreat. This action by the meltwaters likely resulted in the thinning of the till mantle. Mackin interpreted the Boxley burst as a failure of the embankment face caused by the build-up of hydrostatic pressure in the coarser outwash deposits behind the mantle of till. At the time of the Boxley burst, the thin till mantle was described as resembling a breached wall (Mackin, 1941).

An evaluation of the Cedar Falls Development in 1979 under the National Dam Safety Program concluded that some modifications to the crib and Masonry dams were necessary (U.S. Army Corps of Engineers, 1979). Items of concern were seepage from the Masonry Pool and the inability of the Masonry Dam to pass the probable maximum flood (PMF) without overtopping the topographic low a short distance east of the right abutment. Subsequent investigations in 1983 and 1985 again evaluated the above-mentioned items (Converse Consultants, Inc., 1983, 1985). A preliminary evaluation of seepage from the Masonry Pool included consideration of seepage control by a cut-off wall, drainage relief tunnel, and/or dewatering wells. In addition, evaluation of constructing a natural or artificial impervious blanket in the Masonry Pool was completed. All these methods of seepage control were determined to have construction or economic constraints, and the final recommendation was that the Masonry Pool should generally be operated at a maximum elevation of 1,560 ft. In 1986, the Masonry Dam was modified by the construction of a gated emergency spillway in the right-central part of the structure to facilitate the safe passage of the PMF. In addition, the crib dam was replaced by a new concrete overflow structure constructed by the roller-compacted method (Converse Consultants, Inc., 1985).

CONCLUSION

The Masonry Dam never operated as initially designed, primarily because of seepage from the Masonry Pool. The Masonry Dam illustrated the results of selecting a dam site with almost total disregard for local geologic conditions and for the advice of individuals who were skeptical of the suitability of the project siting and the lack of sufficient site exploration—which was completed only after project construction and the subsequent development of seepage.

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Howard A. Hanson Dam

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PROJECT DESCRIPTION

Howard A. Hanson Dam (Figure 1) is in Eagle Gorge on the Green River about 5 mi inside the western Cascade margin. The project, originally investigated as Eagle Gorge Dam, is used largely for flood control for the lower Green-Duwamish valley. The project also provides summer storage for the Tacoma water supply system. The dam lies within the City of Tacoma watershed in an area of controlled access.

The dam is a zoned embankment 235 ft high with a crest elevation at 1,228 ft. The embankment consists of two sand and gravel zones. The core is upstream from a central vertical gravel drain which is connected to a horizontal quarry rock drain at the base of the embankment that exits at the downstream toe. The downstream half of the dam is mostly rolled rock fill with a massive rock toe section. The upstream sand and gravel zone is connected to a 560-ft-long right bank blanket (Figure 2). A concrete-lined, gated chute spillway with a crest elevation of 1,176 feet is on the left bank immediately adjacent to the embankment. Water passage is normally by way of a 19-ft-diameter, 900-ft-long tunnel controlled by two tainter gates in a free-standing intake tower at the upstream end or through a 4-ft-diameter low-flow by-pass.

The embankment dam and ancillary features were constructed between 1959 and 1962. The project also required relocation of 13 mi of the Northern Pacific (now Burlington Northern) mainline railroad, mostly on the left valley wall, and construction of three major bridges.

SITE GEOLOGY

The project lies within a series of western Cascade volcanic rocks of Tertiary age (Hammond, 1963). These rocks are predominantly andesite flows, andesitic tuffs, and breccias with subordinate amounts of basalt and basaltic, pyroclastic, and felsitic rocks. Eocene sandstones of the Puget Group dip beneath the volcanic rocks at the mountain front, 5 mi west.

The Green River valley upstream from the dam was extensively modified by Pleistocene alpine glaciation, while the segment of the valley between the dam and the mountain front was not glaciated. Prior to early Pleistocene glaciation the Green River drained the Cascades via a northwest-trending valley that is now occupied by the North Fork in its southern part. The present valley between the dam and the mountain front was apparently a low area along the trace of the west-northwest-trending Green River fault and was occupied by lesser drainages including Bear Creek (Figure 3). Early Pleistocene blocking of the North Fork valley by Puget lobe ice and deposition of two extensive moraines permanently diverted the Green River, swollen by other glacial marginal drainage over the Bear Creek divide near the present dam (Ward, 1968). During one of the interglacial periods the Green cut its channel about 100 ft deeper than the present valley floor at the dam, rapidly excavating along the sheared rock of the Green River fault zone (Figure 4).

At the upper end of the new valley segment, the over-steepened slope collapsed, on at least two occasions, possibly a result of earthquakes generated by the Green River fault. The resultant slide debris diverted the river around the landslide toe, 1,300 ft southwest of its former channel, where it cut a short, steep-walled canyon into the bedrock to about elevation 1,000 ft (Figure 5). The result is a canyon about 200 ft wide with a steep rock face on the left wall rising to elevation 1,200 ft. On the north side, however, the bedrock surface rises to a narrow, largely buried septum between elevations 1,100 and 1,150 ft before dropping to about elevation 980 ft in a narrow, intermediate "channel" and ultimately to elevation 900 ft (Figure 5). The slide debris consists of a heterogeneous assemblage of rock blocks as much as 20 ft in diameter and finer debris with various amounts of interstitial fine-ground material. The base of the slide is nominally at elevation 1,090 ft. In the former channel area the slide debris is underlain by a sequence of fluvial and lacustrine deposits. The leakage path through these materials is about 2,000 ft long.

Bedrock that provides the foundation and abutments for the dam and ancillary structures is a highly varied assemblage of massive, moderately hard to soft andesitic and basaltic pyroclastic rocks, moderately hard to hard, generally jointed andesite and basalt with local hard, dense, felsitic dikes. Locally the assemblage is deeply weathered and/or hydrothermally altered. The site appears to lie on a southwest-plunging structural nose that is locally highly faulted and sheared. Post-canyon stress relief cracks along faults locally have secondary mineralization.

Figure 1. Aerial view upstream of Howard Hanson Dam. Intake tower and spillway are to the right. A gravel-filled crib constructed over spring area on right abutment is in the center of the photo just downstream from the embankment. Photo courtesy of Seattle District, U.S. Army Corps of Engineers.

GEOLOGIC ASPECTS OF SITING AND DESIGN

The location of the dam was dictated by two considerations: (1) the dam had to be downstream of the confluence of the Green and North Fork rivers for hydrologic reasons, and (2) the dam had to be as far as possible upstream of the City of Tacoma water-supply headworks to minimize the impact on that facility. The narrowest part of this valley segment was selected largely on the basis of topography. The site area was thickly timbered, maps were of poor quality and limited extent, and access was initially very difficult; consequently, the character of the slide and the materials beneath it were not well understood until the mid-1950s, even though the site had been investigated intermittently since 1947. The configuration of bedrock surface on the right bank forced siting of the tunnel, spillway, and other ancillary structures on the apparently sound rock of the left bank. This required crowding the relocated railway centerline farther into the mountainside. The highly pervious gravels in the old right bank channel, together with the generally high permeability of the slide debris, also required design of an extensive upstream blanket to control leakage around the right abutment.

A critical design aspect was the characteristic of the excavated rock and its suitability for rock fill in the dam. The paucity of accessible exposures and the limited number and depth of borings hampered a detailed analysis of the volcanic stratigraphy. The closely jointed

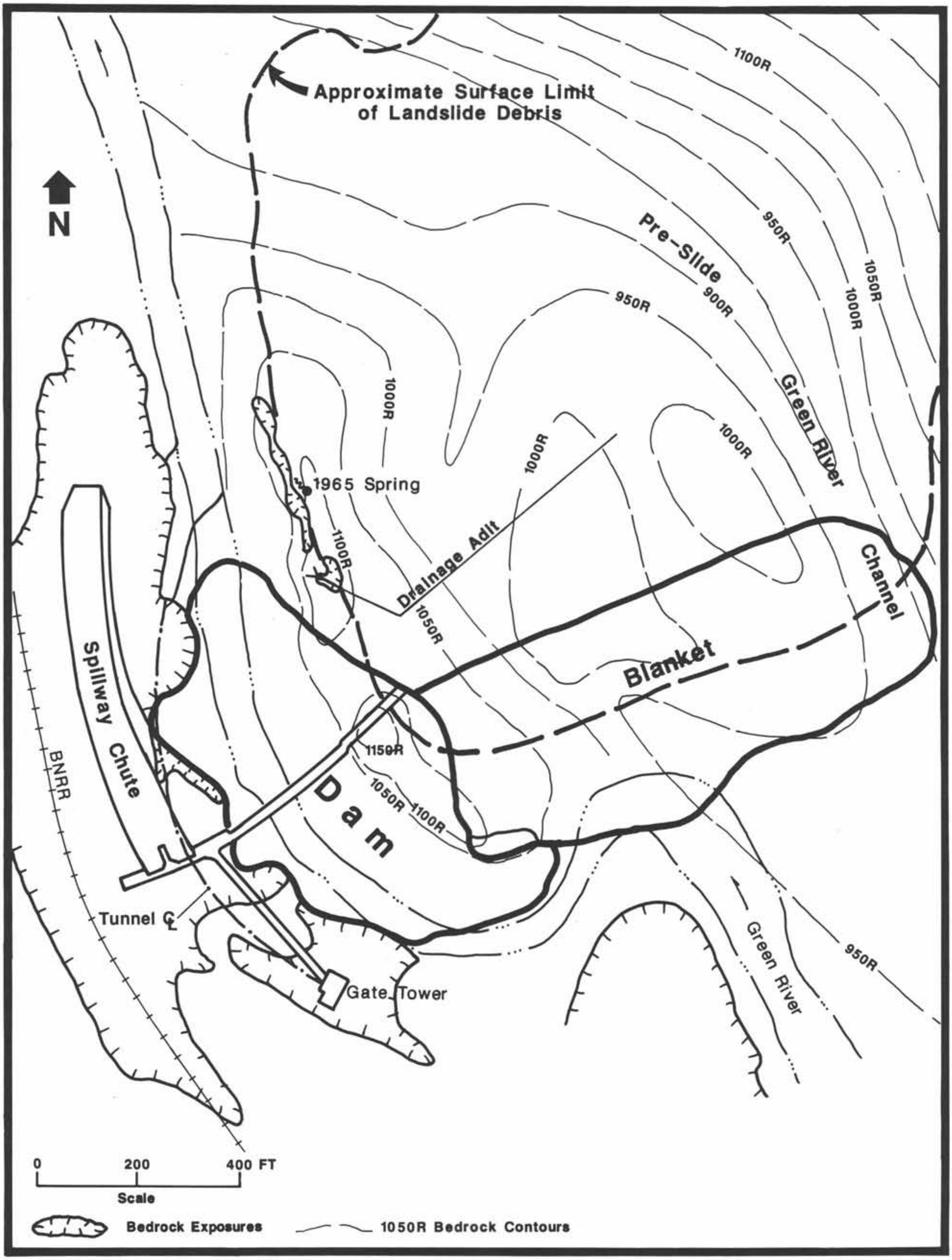

Figure 2. General site geology and plan of Howard Hanson Dam and ancillary features.

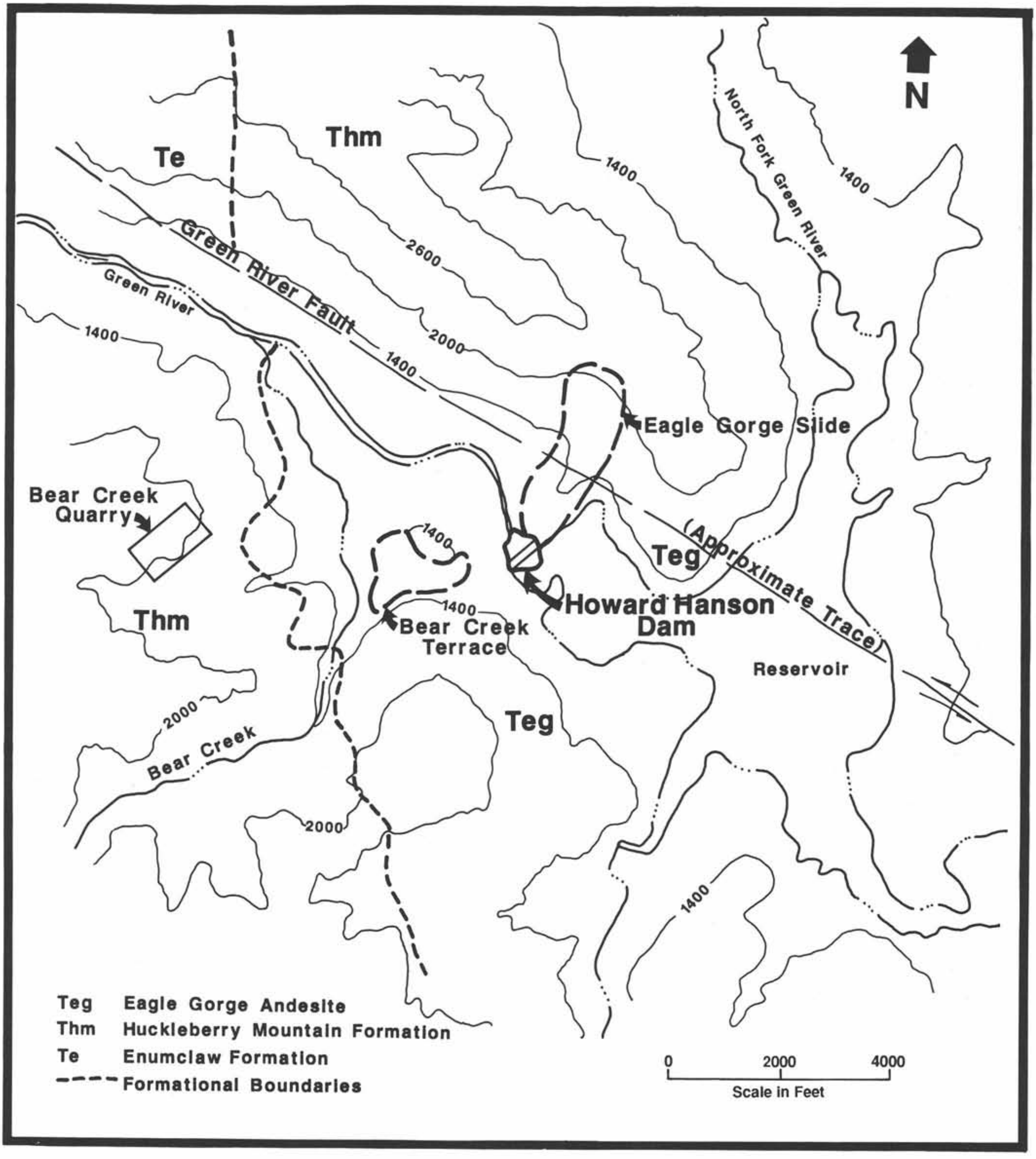

Figure 3. Generalized geologic map of the Howard Hanson Dam vicinity. Topography from U.S. Geological Survey Eagle Gorge quadrangle, 1951, scale 1:24,000. Geology after U.S. Army Corps of Engineers.

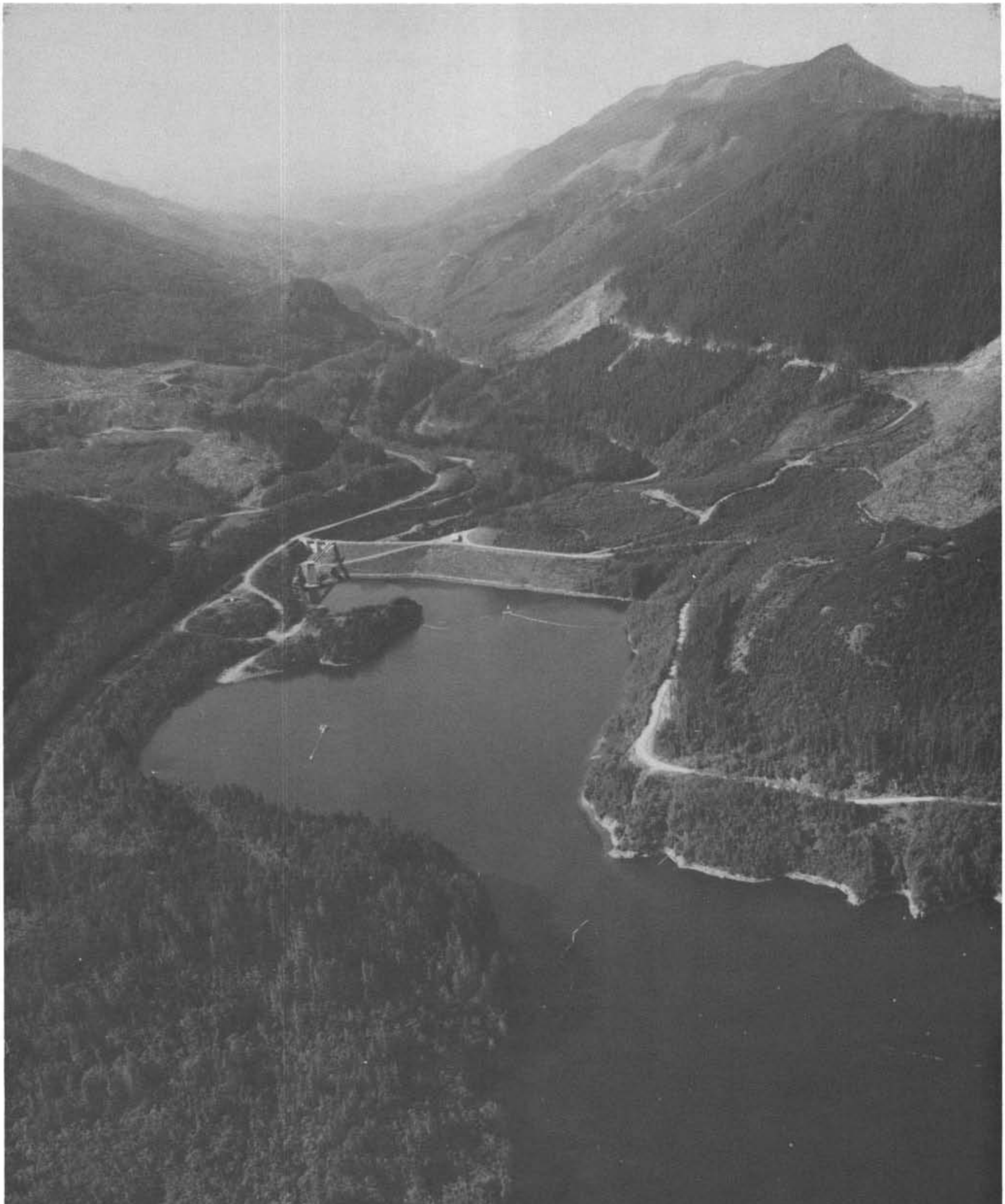

Figure 4. Aerial view downstream of Howard Hanson Dam along the post-early Pleistocene valley eroded along trace of the Green River fault. Intake tower is at left with gated spillway structure behind. The textural change near the center of the embankment reflects the transition between the embankment dam (left) and the blanket (right), the latter along the upstream side of the slide debris which forms the right abutment for the dam. Photo courtesy of the Seattle District, U.S. Army Corps of Engineers.

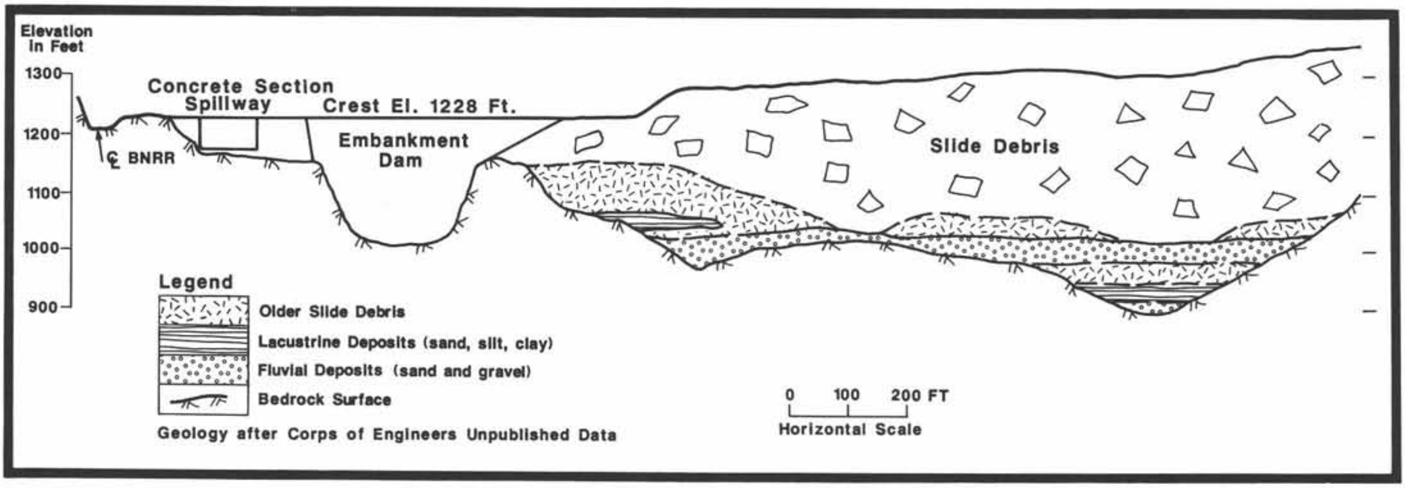

Figure 5. Geologic section at Howard Hanson Dam; view downstream.

character of much of the rock dictated inclusion of a shallow (20-30 ft) grout curtain in the bedrock foundation and abutments as well as beneath the spillway weir and around the tunnel collar. The apparent relative soundness in the exposed left wall of the canyon, together with limited drill hole data, suggested that, while some waste could be anticipated from rock excavation, the excavation would be expected to furnish an adequate quantity of rock fill for the embankment dam. Sand and gravel for the core sections of the dam were to be processed from an extensive high fluvial terrace (Bear Creek terrace) 1/2 mi west of the dam site. This material was determined to be unsuitable for concrete aggregate, however, owing to a high percentage of soft particles.

CONSTRUCTION PROBLEMS

Probably the most far-reaching construction problem was caused by the judgment call on durability of excavated rock. The disintegration after stockpiling of significant quantities of rock excavated from the spillway cut, tunnel, forebay, and intake channel required both major changes in embankment design and development of a source of durable rock. Pre-construction investigation indicated a possible source on the mountainside 1.5 mi west of the dam site, above the Bear Creek terrace (Figure 3). Drilling there had revealed a closely jointed to massive, hard, durable, basalt flow, overlain and underlain by a soft tuffaceous andesite. Stripping of remnants of the overlying rock unit along with the overburden was required. The resulting Bear Creek quarry furnished material for the large rock toe section as well as for the face riprap and rock drain sections of the embankment. Table 1 is a summary of rock characteristics.

Construction of the 100-ft-deep railway cut adjacent to the spillway forebay area (Figure 2), nearly 3 yr ahead of excavation for the dam and ancillary structures, allowed observation of the performance of the closely jointed rock slopes. Systematic rock bolting (slot and wedge type), netting, and a network of drain holes were employed on all later high rock cut slopes, most of which were on the order of 50 to 100 ft high. The irregularity of the volcanic stratigraphy resulted in several lesser design changes and, locally, additional excavation in zones of incompetent rock. A redesign of the gate tower to a free-standing structure was required due to discovery of a soft hydrothermally altered zone on one side of the excavation. The soft pyroclastic rock in parts of the spillway and gate tower foundations had to be covered by a thin layer of concrete immediately upon excavation in order to limit further deterioration of the foundation from air slaking (U.S. Army Corps of Engineers, 1963). The 19-ft-diameter horseshoe tunnel was driven full face by conventional drilling and blasting methods employing 10-ft delayed rounds. Overbreak was minimal when using a powder factor of 1 lb/cy. About one-third of the tunnel was found to be self supporting where it was driven through moderately hard, irregularly jointed andesite. A 32-ft-long, flat-roofed section at the upstream portal was supported by 24-in. I-beams on 18-in. centers resting on 8-in. posts. About 100 ft of tunnel length near the upstream portal and 50 ft near the downstream portal were supported by steel ribs on 4-ft centers; the majority of ribs were on 6-ft centers. Light loads were experienced, even though the supported sections were in interstratified soft andesitic pyroclastic rock and denser basalt and felsite.

Water inflow was experienced near both portals and adjacent to three fault zones in the central supported section. Upon completion of lining placement in the tunnel, the area outside the lining was contact grouted by a grout collar that was emplaced 20 ft into the rock surrounding the tunnel for a distance of 90 ft downstream

Table 1. Rock characteristics at Howard Hanson Dam (from U.S. Army Corps of Engineers, 1963

Rock type	Weathering resistance	Shot rock	Specific gravity	Absorption (%)	$MgSO_4$ (% loss) (5 cycles)	Freeze-thaw (% loss) (50 cycles)	Wet-dry (% loss) (25 cycles)
Basalt (general)	Very resistant	Blocky, durable	2.60-2.65	0.1-0.2	1.0	0.4	0.2
Basalt (Bear Creek quarry)	Very resistant	Blocky-fine durable	2.71	2.2	---	0.6	0.1 (20 cycles)
Andesite	Resistant	Massive with disintegration near shot	2.50-2.60	2.5	12.5	6.1	0.4
Basaltic tuff	Susceptible to breakdown	Breaks to fine particles which weather to gritty mass	2.30-2.35	6.4-7.6	9.8	92.3	1.9
Andesitic tuff	Very susceptible to weathering and breakdown	Breaks to silty clay mass with larger particles weathering readily	2.25-2.33	6.4-7.6	19.0	92.3	3.3

from the gate tower. All tunnel grouting was done at 50 psi (U.S. Army Corps of Engineers, 1963).

Foundation preparation for the embankment included removal of all channel gravel and other overburden in the canyon floor to expose shallow bedrock. The highly irregular bedrock surface was then backfilled with dental concrete in the sand and gravel core section so as to prohibit localization of seepage in narrow bedrock channels. The canyon walls were simply cleaned of debris as the embankment was constructed. About 50 ft of overburden was removed from the left abutment. Above elevation 1,115 ft on the right abutment only the colluvium and organic material was removed from the face of the landslide debris. The single line grout curtain was placed to depths of 25 ft into the bedrock beneath the upstream core section. The abutment grout curtains were extended 20 ft into bedrock, and the curtain beneath the spillway gate structure was extended to 30 ft. Grout takes were minimal (U.S. Army Corps of Engineers, 1963).

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Right Bank Leakage

Though initial impoundment of the reservoir began during the winter flood season in 1961 and summer conservation pools were held at elevation 1,141 ft, the first significant flood pool did not occur until February 1965 when an elevation of 1,161.8 ft was briefly attained. At this time a spring abruptly broke out at elevation 1,134 ft about 350 ft downstream from the downstream right abutment toe (Figure 2). The breakout was on a clay bed at the top of the fluvial-lacustrine sequence and at the base of landslide deposits. The spring initially flowed about 80 to 100 gpm and sapped the steep slope until after the reservoir was lowered. The spring was subsequently controlled by a gravel blanket supported by a crib wall. Horizontal drains drilled 50 to 260 ft into the slope between the right abutment of the dam and the spring were ineffective in lowering piezometric levels. After further analysis of piezometric data in the several aquifers behind the right abutment, a drainage adit was constructed in 1968 at elevation 1,100 ft and extending 650 ft into the mountainside. The portal and outer 80 ft of the adit are in a bedrock spur, and the remainder is in slide debris, which extends 20 to 25 ft below the adit floor. Twelve relief wells were drilled to intersect and extend 20 ft below the adit floor. This system appears to adequately control abutment leakage through the upper (slide debris) aquifer during flood pools (U.S. Army Corps of Engineers, 1982).

Embankment Dam

Monuments on the embankment indicated about 1 in. of settlement between late 1961 and 1974. A little more settlement was experienced at the crest than on the upstream or downstream slopes. About 10 percent of the central crest settlement appears to be a result of the 1965 Puget Sound earthquake (magnitude 6.5), and nearly 20 percent of crest settlement near the right abutment may be attributed to that event. It is estimated that a modified

Mercalli intensity of VII was experienced at the project (U.S. Army Corps of Engineers, 1965). In 1983 the project structures were analyzed using a permanent displacement (Newmark) analysis for a peak acceleration of 0.45 g and a duration of 14 sec; these values are greater than those expected from the proposed maximum credible earthquake. Analysis indicated a maximum displacement of less than 8 in. at 80 to 90 percent of embankment height, an acceptable order of magnitude (U.S. Army Corps of Engineers, 1983).

Reservoir Slides

Considering the reservoir slope declivity and geologic character of slope materials, the reservoir has been remarkably free from slides other than small failures of colluvium. The only large slide was a 1/4-mile-long slump/block failure of a terrace in glaciolacustrine sediments along the east side of the North Fork shortly after the reservoir was initially impounded. The slide was in a remote area and caused no damage to project or private facilities.

Erosion

Over the years of outlet works operation progressive erosion in soft andesitic pyroclastic rocks has occurred on the left bank just downstream from the stilling basin. The steep rock slope is locally undermined as much as 10 ft. As the stability of this rock slope is important to the integrity of the lower spillway chute, the progress of this erosion is closely monitored (U.S. Army Corps of Engineers, 1982).

ACKNOWLEDGMENTS

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Mud Mountain Dam

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PROJECT DESCRIPTION

Mud Mountain Dam (Figure 1) is located on the White River just inside the Cascade Range mountain front where the White River valley joins the Puget Sound basin. The project is used solely for flood control for the lower Puyallup basin and maintains little or no reservoir for much of the year. The dam is a zoned embankment 425 ft high consisting of a central core, flanking transition zones, and dumped rock shells. An uncontrolled, concrete-lined, chute spillway (crest elevation 1,215 ft) is situated on the right abutment immediately adjacent to the embankment (Figure 2). Water passage is normally by way of two 2,000-ft-long tunnels, both in the right bank: a 9-ft-diameter tunnel controlled by a single radial gate at the upstream end (elevation 895 ft), and a 23-ft-diameter tunnel (intake elevation 970 ft), which feeds into three steel penstocks controlled by Howell-Bunger valves at the downstream end (U.S. Army Corps of Engineers, 1982). The dam embankment was constructed from 1939 to 1941 and at that time was the highest embankment dam in the world.

SITE GEOLOGY

General

The complex topography and geology that influence the project area are a product of multiple Pleistocene glaciations in an ice border environment combined with periodic deposition of pyroclastic mudflows (lahars) and debris flows originating from the present and ancestral Mount Rainier volcanic center. The result has been a complex of ice-marginal fill terraces and outwash channels across and adjacent to the mouth of the valley and into flanking areas of the Puget Sound basin (Figure 3). The lahars and debris flows have further resulted in several diversions of the White River in this vicinity. The most recent diversion placed the river hard against the south side of the valley where the river has cut a deep canyon through the Pleistocene units and into the underlying volcanic bedrock. Except for this canyon, a narrow 2.5-mi-long, flat-topped ridge known as Mud Mountain blocks the entire width of the White River valley. The present form of the ridge is a product of the most recent (Vashon) glaciation. The ridge is composed of a sequence of ice-marginal outwash deposits (sand, gravel, and boulders) and lake beds (silt and fine silty sand), once part of an ice-marginal delta which extended about 3 mi up-valley. These deposits are underlain by earlier Pleistocene lahars. The northwestern or ice contact side of the ridge was modified by late ice meltwaters and subsequent landsliding. The southeastern side has been modified by erosion by the postglacial White River and related landsliding. The ridge crest, along with upstream remnants of the delta and a large area of the glacial outwash plain downstream, is mantled by the 5,700-yr-old Osceola mudflow originating from the Mount Rainier volcano (Crandell, 1971). The bedrock topography beneath Mud Mountain (the preglacial valley floor) has a nominal elevation of 950 ft, more than 50 ft below the present valley floor in the adjacent reservoir area. Two prominent channels, the older Boise Creek channel and the younger Big Springs channel, are cut at least 150 ft deeper and represent interglacial courses of the ancestral White River (U.S. Army Corps of Engineers, 1986).

Bedrock

The canyon walls at Mud Mountain Dam consist of volcanic bedrock of varied composition overlain by lahars, debris flows, and fluvial boulder gravels of the Mud Mountain complex, which is in turn overlain by the Hayden Creek Drift. The bedrock surface slopes gently northwest from a nominal elevation of 1,100 ft at the dam (1,125 ft on the left abutment).

Bedrock exposed in the canyon walls and mapped in the dam abutments during construction consists of andesite, andesite breccia, agglomerate, and thin beds of lithic tuff with local inclusions of sedimentary material, all part of the Miocene Enumclaw Formation (Hammond, 1963). The competence of the bedrock is highly varied. The andesites and andesite breccias are welded into a competent, though jointed, rock mass. The agglomerates form the majority of the cliff sections of the river canyon and are a weaker rock, generally more massive and more commonly hydrothermally altered. Locally, the agglomerate is less competent than the overlying lahar deposits. Most depositional contacts are highly irregular. The lithic tuffs tend to be thin beds which provide the major clue to structural attitudes. The bedrock is crossed by numerous local faults which may be related to the period of volcanic deposition (U.S. Army Corps of Engineers, 1939, 1942).

Figure 1. Mud Mountain Dam. Aerial view east on August 6, 1974, with reservoir at elevation 1,140 ft. At the lower center of the photo is the spraying discharge through three Howell-Bunger valves into the narrow, vertical-walled rock canyon downstream from the dam. The ungated spillway is right center. The lower part of the 1974 slide is at the upper left. Photo by the author.

Mud Mountain Complex

The Mud Mountain complex consists of a series of lahars, debris flows, water-laid tuffs, and related fluvial deposits approximately 200 ft thick and overlying the bedrock surface. The bulk of the lahar material is a hard, highly plastic, cobbly, gravelly silt and clay (CH, MH, GC, and GM), with minor amounts of sand, wood fragments, and pumice. The gravel and cobble clasts range from relatively fresh to totally weathered (or hydrothermally altered). Natural moisture contents in these materials vary from 30 to 50 percent, and dry unit weights range from 60 to 90 pcf. Laboratory testing indicates preconsolidation of the material by glacial overriding consistent with the geologic record. Commonly, the basal portion of the Mud Mountain complex consists of a fluvial boulder gravel and a discontinuous water-laid tuff bed sandwiched between the lowest lahar unit and an overlying thicker lahar unit. The uppermost part of the complex is locally characterized by a bouldery debris flow that covers fluvial channeling. Where the debris flow is missing, 10 to 15 feet of residuum characterizes the top of the sequence. Close examination indicates microvesiculation and charred wood fragments in some zones, indicating the material was still hot when deposited. Crandell and Miller (1974) suggest that materials are at least as old as middle Pleistocene and noted the lack of clasts from the modern Mount Rainier volcano, which suggests a source from an earlier volcanic center. In the south canyon wall the stratigraphy clearly suggests fluvial deposition, and boulders are common. The complex extends about 30 to 40 ft higher in elevation than on the north bank, suggesting erosion prior to the deposition of the overlying Hayden Creek Drift.

Hayden Creek Drift

Deposits of the Hayden Creek glaciation (Crandell and Miller, 1974) from the late Pleistocene Mount Rainier ice cap overlie the Mud Mountain Complex in this portion of the White River canyon. On the south bank the drift consists of about 25 ft of very dense gravelly clay till containing a slightly oxidized gravel.

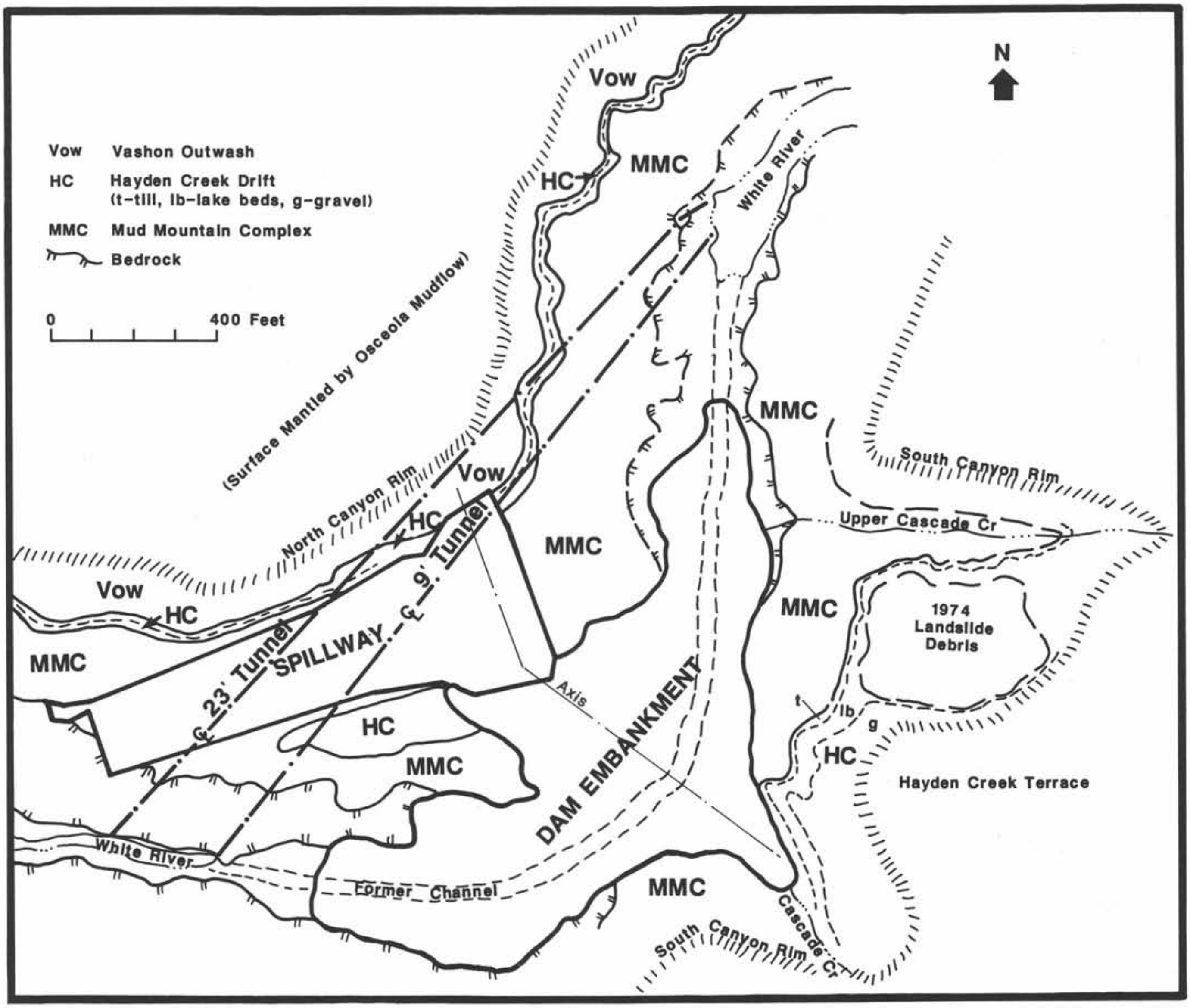

Figure 2. Project features and general geologic map of Mud Mountain Dam.

This is overlain by 30 ft of varved to thick-bedded clays, thin turbidites, and peat beds. These are overlain by more than 100 ft of oxidized gravels, which rise to the high terrace surface on the south bank. On the north bank the bulk of the Hayden Creek Drift has been removed by pre-Vashon erosion so that only the till and a portion of the lake beds remain. While there is some evidence for earlier glaciations in this region (Crandell and Miller, 1974) none has been recognized in this portion of the canyon.

Vashon Drift

In the right canyon wall the Vashon Drift is represented by a single unit of glacial outwash sand, gravel, and boulders above a nominal elevation of 1,240 ft. The materials tend to be loose and are highly pervious. A zone of perennial springs exits the valley walls at the top of the underlying till or lake beds of the Hayden Creek Drift. Upstream from the main canyon along the southeast side of Mud Mountain, the eroded surface of the pre-Vashon sediments drops nearly to current valley floor elevations. In these areas the Vashon drift consists of three units: a basal "advance deltaic" sand and gravel; a sequence of lake beds consisting of fine sand, silty sand, and silt with subordinate amounts of clay; and sandy gravelly outwash. The lake beds dominate the drift to a nominal elevation of 1,140 ft throughout much of the ridge (Mud Mountain), whereas the underlying "advance deltaic" sands and gravels appear to be erratically distributed. Overlying the lake beds and locally

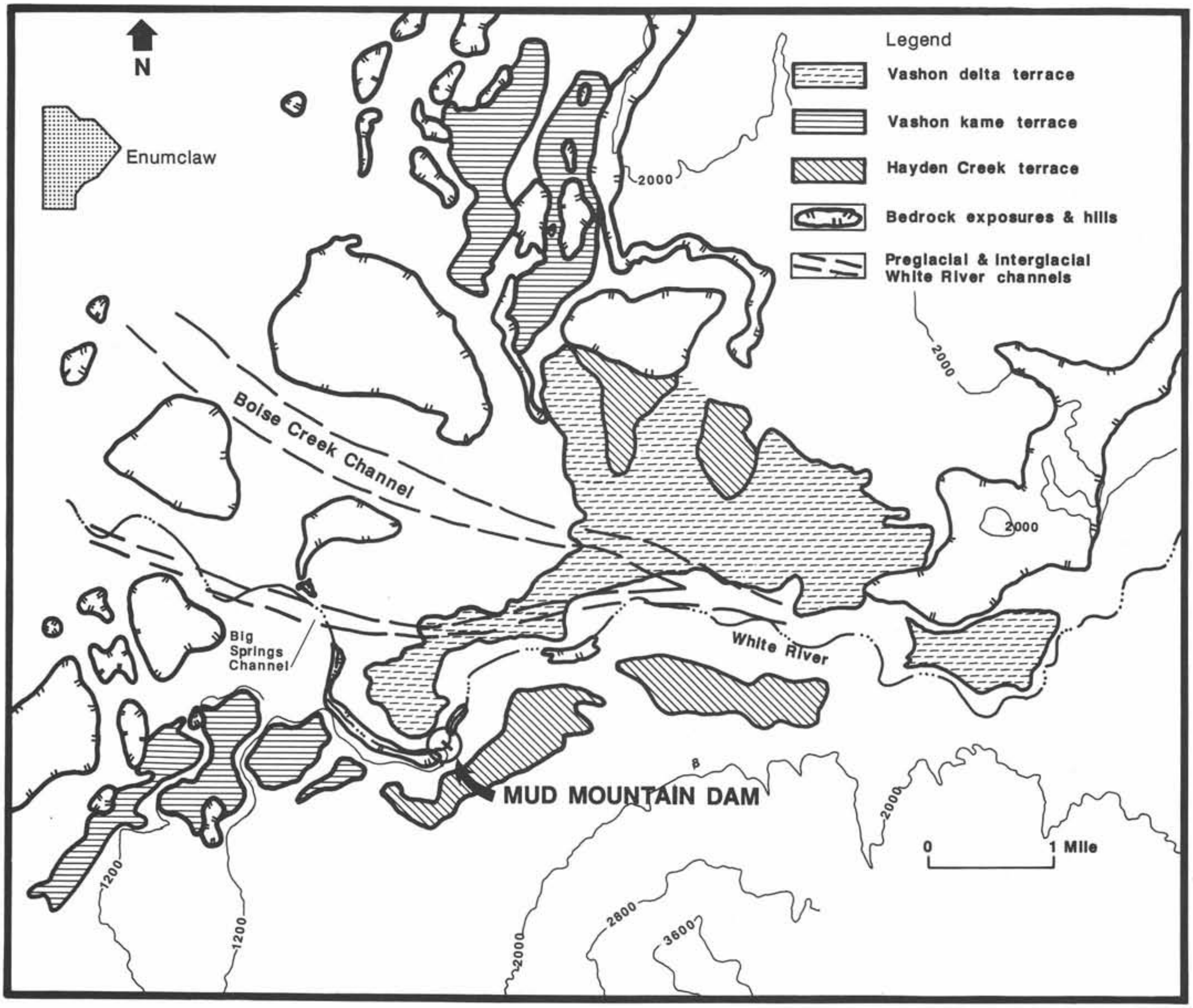

Figure 3. Geology of the Mud Mountain Dam area.

occupying channels in the lakebed surface is a thick section of sandy, gravelly, glacial outwash, crudely grading upward into the cobble and boulder gravels high on the ridge. It is these outwash materials, above elevation 1,140 ft, that appear to provide the principal conduit for "short path" reservoir leakage through the ridge. In the short canyon section of the reservoir midway along the southeast side of Mud Mountain, the semi-impervious pre-Vashon sediments rise nearly to the top of the ridge, well above maximum reservoir level. Here, the Vashon Drift is also represented by a thin unit of sandy, gravelly, glacial outwash.

Osceola Mudflow

Capping virtually all of Mud Mountain is a 5,700-yr-old mudflow which varies in thickness from 2 to 30 ft and consists of a heterogeneous mixture of boulders through clay (montmorillonite) and scattered logs and smaller wood fragments (Crandell, 1971; Crandell and Waldron, 1956). The mudflow provides a relatively impervious cap over the Vashon outwash; water tends to pond in low areas on the ridge top.

GEOLOGIC ASPECTS OF SITING AND DESIGN

The topography that resulted from glacial diversion of the White River onto the south side of the broad glaciated White River valley limited the choice of dam sites to this 3-mi-long canyon segment. The choice was further restricted to a mile-long segment in which the dam and appurtenant structures are now located. The dam axis lies essentially where the bedrock in the canyon walls is highest, minimizing the portion of the abutments in Pleistocene deposits (Figures 3 and 4). The spillway was positioned so as to be excavated in and founded on lahar deposits of the Mud Mountain complex (at that time thought to be glacial till); some of the excavation was made in Hayden Creek and Vashon drifts. Only the flip bucket is founded on bedrock, which is faulted and of poor quality. The left abutment was originally intended to be 200 to 300 ft upstream from its present position, which would have shortened the length of the section. Design engineers shifted the left abutment into the gully of Cascade Creek, using it as a "natural keyway" even though that required collection of the drainage and its discharge via pipeline well down on the upstream face of the dam (U.S. Army Corps of Engineers, 1946). While much of the canyon bedrock upstream is poor-quality agglomerate, the intake structures for both tunnels were founded on a small exposure of high-quality, jointed, andesite breccia on the right bank. The embankment dam below elevation 1,100 ft fills a narrow steep-sided bedrock canyon, 90 ft across at the bottom and 150 ft wide at the top, which exposed a heterogeneous assemblage of agglomerate, volcanic breccia, tuff, and subordinate andesite with numerous downstream-dipping and a few upstream-dipping faults. Individual faults exhibited gouge zones varying from a few inches to 5 ft in width (U.S. Army Corps of Engineers, 1942). Above elevation 1,100 ft most of the embankment is founded on and abuts into lahar deposits of the Mud Mountain Complex. Only a small part of the upper right abutment abuts into Hayden Creek till. The tunnels pass through a similar variety of volcanic rocks. The downstream valve house structure is sited on a vertical to overhanging section of rock face consisting of jointed andesite and andesite breccia.

CONSTRUCTION PROBLEMS

Preparation of the dam foundation included removal of river deposits from the entire area of the embankment; cleaning out potholes in bedrock on the canyon floor and low on the canyon walls, fault zones and weathered seams in the bedrock of the canyon walls; and backfilling all such irregularities with concrete. Foundation preparation in the lahar materials simply required removal of surficial weathered material and colluvium.

Perhaps the most interesting geologic problem during construction related to the borrow and placement of the core material. The borrow pit was located on the top of Mud Mountain, an area characterized by 5 to 15 ft of Osceola mudflow underlain by Vashon gravelly, bouldery outwash. The natural moisture content of the montmorillonite-rich mudflow was well above optimum, and the blend of 80 percent sand and gravel, 20 percent mudflow required rotary kiln drying prior to placement (U.S. Army Corps of Engineers, 1986). The sensitivity of this material to moisture further required placement and compaction under a large circus tent erected across the canyon during the winter.

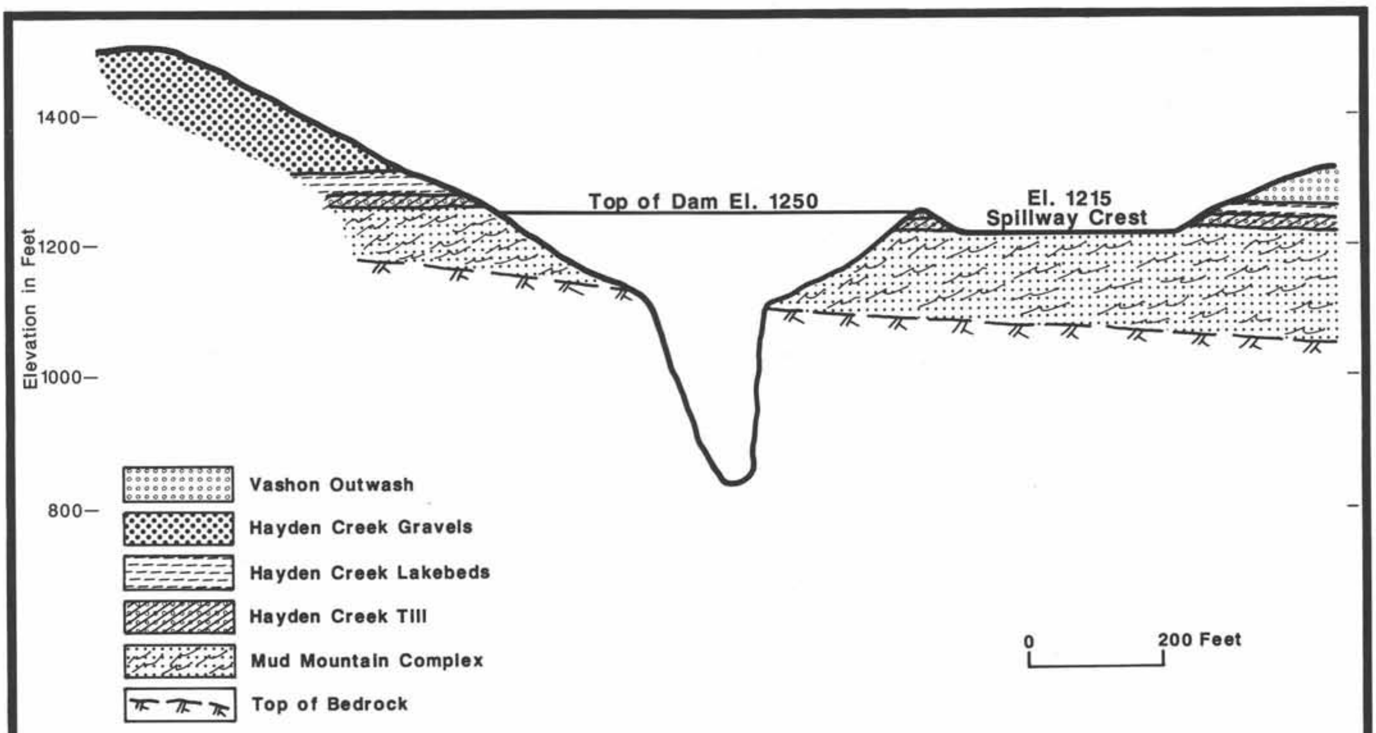

Figure 4. Geologic section of Mud Mountain Dam; view downstream.

The compacted transition zones consisting of 4-in.-minus crushed diorite from an off-site quarry turned out to be somewhat coarser than anticipated, resulting in less than satisfactory filtering characteristics. The coarse rock shells were placed by dumping and simultaneous sluicing. The quarry run rock (andesite, sp. gr. 2.72-2.75) had a specified gradation of 45 to 55 percent less than 0.5 ton, 25 to 35 percent at from 0.5 to 1.5 tons, and 15 to 25 percent at from 1.5 to 5.0 tons. The rock came from quarries in the ridge 1 to 1.5 mi northwest of Mud Mountain.

The tunnels were excavated by conventional methods using full face excavation and wood supports. Both tunnels are concrete lined.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

Reservoir Leakage

The general geologic character of the first 2.5 mi of the right side of the reservoir was recognized by the designers. However, the planned use of the project for the single purpose of flood control precluded impoundment of a reservoir for longer than a few days; this was not expected to compromise the integrity of the reservoir.

Four piezometers and a series of spring monitoring points on the northwest toe of Mud Mountain were emplaced in 1948 at the time the project became fully operational. Normal operation of the project over the succeeding 25 yr raised the reservoir to elevation 1,075 ft on numerous occasions. Levels were above 1,100 ft for 33 days in 1958 (maximum elevation 1,117 ft) and 3 days in 1965 (maximum elevation 1,131 ft). During the 1958 flood pool, discharge occurred where the Big Springs channel intersects the present White River below the dam and in an area (Cary Springs) along the northwest (downstream) toe of Mud Mountain.

In the early 1970s the U.S. Army Corps of Engineers conducted further geologic studies of the ridge and increased the number of piezometers and spring monitoring points. Two major leakage paths were identified: the Big Springs channel (long path), and leakage directly through the pervious Vashon outwash (short path) above a nominal elevation of 1,140 ft (the top of the Vashon lake beds). A test pool raise in 1974 that maintained reservoir levels above 1,100 ft for 3 months and held the level at elevation 1,150 ft for about one week confirmed the leakage paths.

Subsequent investigations have provided data to realistically estimate leakage and potential related sloughing activity on the downstream face of Mud Mountain, should higher reservoirs (to elevation 1,215 ft) become necessary. The U.S. Army Corps of Engineers is continuing these studies under their dam safety assurance program (U.S. Army Corps of Engineers, 1982).

Passage of Bedload

The source of the White River is principally from major glaciers on the north side of Mount Rainier. This results in fluvial characteristics typical of glacial stream: high gradient, braided channels, and extensive coarse bedload. Because the reservoir is usually empty, this bedload material, along with large quantities of sand and rock flour, is passed through the 9-ft-diameter tunnel with its invert (elevation 895 ft) at about the level of the original stream bed. During flood periods when a reservoir is impounded, the gravel/boulder bedload is trapped near the upper end of the reservoir and later sluiced on through as the reservoir is emptied. This chronically causes damage to the floor of the tunnel, which is reinforced by longitudinal steel rails with grout between them. Repair with some replacement of the reinforcing system has been required about every 2 yr.

Reservoir Slides

Small slides, generally related to failures of shallow, saturated colluvium, have been observed in the reservoir area. Some are the result of excessive rainfall, and some are the result of reservoir drawdown, especially after a reservoir of a month or more duration. During the drawdown from the long term 1974 pool elevation of 1,150 ft, small slides occurred where gravel outwash channels intersect the reservoir slopes. A major prehistoric slide about 0.5 mi upstream from the dam on the right bank, appears to have blocked the river about 1,800 yr ago, and an equally large prehistoric slide appears to have failed into the canyon from the left bank just below the dam. The only slide to influence operations, however, occurred on January 31, 1974, on the left bank just upstream from the dam. Rapid movement of a large mass of shallow, saturated colluvium on the headwall of an earlier slide high above the reservoir pushed older slide debris and trees across the flat surface of the Hayden Creek lake beds (elevation 1,300 ft). Part of the material plunged into the half-filled reservoir, and part dammed Upper Cascade Creek; the barrier failed about 30 min later. About 70,000 cy of earth, debris, and trees entered the reservoir, moving 800 ft upstream beyond the tunnel inlets and causing a 12-ft wave runup on the dam. When the reservoir was finally drafted, about 35 ft of muck remained on the reservoir bottom, with larger trees and snags standing upright.

The slide debris above the reservoir (Figure 2) was monitored by both aerial and terrestrial photogrammetry for several years following the failure. Only minor creep was detected. The slide resulted in abandonment of a cableway that was used for setting stop logs in the outlet structures and for removing reservoir debris.

Embankment Dam

Two items are of significant geologic interest with regard to the embankment dam. One concerns the design and construction and the potential for leakage through the structure. The other is related to the behavior of the structure during earthquakes.

Beginning in 1984, the U.S. Army Corps of Engineers embarked on field investigations and studies relating to the integrity of the core of the dam. Data indicate that a combination of (1) difficulty of compaction during construction and (2) settlement of the embankment, initially on the order of 2 to 3 ft, in the environment of steep canyon walls, together with (3) the inadequacy of the crushed rock transition zone for providing a proper filter for the core (clayey gravelly sand through silty sandy gravel), prevents the core from performing as a reliable seepage barrier and that progressive deterioration is occurring due to seepage from the occasional reservoirs. A concrete diaphragm wall is being constructed to mitigate the problem.

The embankment has experienced shaking from both the 1949 and 1965 Puget Sound earthquakes. These had estimated Modified Mercalli intensities of VII. Estimated site accelerations are on the order of 0.15 g (far field). There was no settlement of the embankment during these events although 1- to 1.5-in. longitudinal cracks opened up along the dam crest at the juncture of the core and rock fill (U.S. Army Corps of Engineers, 1949, 1965). In 1983 the structure was analyzed using a permanent displacement (Newmark) analysis for a peak acceleration of 0.45 g and a duration of 14 sec (greater than the values expected from the proposed maximum credible earthquake). The analysis indicated a maximum of 7 to 13 in. of permanent displacement at about 60 percent embankment height, an acceptable order of magnitude (U.S. Army Corps of Engineers, 1983).

Stress Relief Cracks

Stress relief cracks have been noted in and near the canyon walls. The cracks are apparently confined to the lahar sequence of the Mud Mountain complex. In 1948 a 200-ft-long crack was discovered in the narrow septum separating the south side of the spillway from the downstream canyon. The crack, subparallel to the canyon, was cleaned out and backfilled with mudflow and till materials. In 1974 a 75-ft-high mudflow cornice on the canyon wall, 200 ft south of the earlier crack, developed cracks high on the slope. The cornice was removed to lessen the hazard to personnel using the road on the downstream face of the dam. A 10-ft-deep crack opened up above the intake structure as a result of the 1949 earthquake (U.S. Army Corps of Engineers, 1949). Material outboard of the crack was removed. In 1984, while drilling high on the canyon wall above the intake structure, some 1,800 gal of drilling fluid was lost into the Mud Mountain complex in a zone of otherwise impervious lahar deposits. No fluid could be seen exiting on the steep slope below, and a stress relief feature parallel to the canyon wall appears a reasonable explanation. Thus the long-term stability of the steep slope may be compromised by a number of unknown stress relief features. However, no such failures have been noted during the period of project operation.

ACKNOWLEDGMENTS

The efforts of the late A. S. Cary, whose pioneer geological work at Mud Mountain Dam stimulated the writer's interest in the problems at the dam, are acknowledged. The review of the manuscript and helpful suggestions of K. D. Graybeal and R. E. Morrison of the Seattle District, U.S. Army Corps of Engineers, were most appreciated.

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Landslide debris and upright snags (as much as 30 ft high) from the landslide of January 31, 1974, upstream from the intake at Mud Mountain Dam in March 1974. The pool level was at 938 ft, and the river was cutting into the landslide debris. Photograph by R. W. Galster.

The snow line on the face of Mud Mountain Dam is evidence of the12 feet of runup created by the landslide of January 31, 1974. Reservoir elevation is 1,150 ft. Photograph by R. W. Galster, February 11, 1974.

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The Nisqually Projects: La Grande and Alder Dams

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INTRODUCTION

The Nisqually Project, consisting of Alder and La Grande dams, is located on the Nisqually River approximately 25 mi downstream from the western boundary of Mount Rainier National Park. The river originates in the Nisqually Glacier on the southwestern slope of Mount Rainier, turns northwestward, and empties into the south end of Puget Sound. This project area can be reached from Tacoma by driving almost due south on State Highway 7 (or 161), the road leading to the Nisqually entrance to Mount Rainier National Park.

During the early years of this century Tacoma bought its power from a private company. In 1907 the city investigated the present dam site in the Nisqually Canyon. In 1910 the decision was made to build the first La Grande Dam near the present site of Alder Dam, and the project was completed in 1912.

The winter of 1929-1930 brought a severe drought to the Pacific Northwest, and it was a critical time for power production for the City of Tacoma. The small reservoir behind La Grande Dam, together with electricity provided by Cushman Dam on the Olympic Peninsula, were inadequate to provide the city with power. A most unusual remedy was to "rent" the aircraft carrier *Lexington* from the Navy to supply additional power.

Both Alder and the new La Grande dams were constructed during World War II on an expedited basis in order to provide additional hydropower for the war effort.

LA GRANDE DAM

Project Description

The present La Grande Dam (Figure 1) is a concrete gravity structure 217 ft high and 710 ft long, situated 1.5 mi below Alder Dam. It consists of three main parts: a central, overflow spillway section, and two nonoverflow wings (Figure 2). A concrete cut-off wall, 5 ft thick and 180 ft long, abuts against the right end of the dam. The cut-off wall is buried in the overburden and is founded on bedrock. The dam diverts water through an intake structure on the left abutment into a power tunnel about 5,000 ft long to a surge tank, then to the remote powerhouse.

A grout curtain, galleries, and drains have been provided to reduce seepage and uplift. Since the spillway section, which is the major portion of the structure, is curved in plan, it is, in a sense, an arch dam. In addition, the canyon narrows quickly downward. Consequently, the spillway structure forms (in plan) a tapering plug firmly wedged between the canyon walls and is immune to any effects of the scour in the plunge pool.

Areal Geology

In a remarkable analysis of the Pleistocene glaciation of Puget Sound, Bretz (1913) recognized several advances of continental ice from Canada into the Puget Sound basin. In the La Grande Dam area, he mapped the maximum southern extension of the ice at La Grande, where a westward bulge in the Cascade Range front, Bald Hills, stopped further movement to the south.

During the Pleistocene, alpine glaciers moved westward from Mount Rainier and stopped approximately at the La Grande Dam. Thus the hills in the vicinity of the dam show marked evidence of glaciation (Crandell and Miller, 1974). Between the upland glaciated by alpine ice and the basin glaciated by Puget lobe ice, the Nisqually River has cut a spectacular canyon deep into the bedrock.

Bedrock at the site is similar to the lower to middle Tertiary volcanic and sedimentary rocks common to the southwestern Cascades (Figure 3) (Fiske et al., 1963; Walsh et al., 1987). These rocks have been subjected to the same deformation that formed the northwest-trending anticlines and synclines throughout this area. This sequence of rocks has been intruded by Miocene dioritic and granodioritic plutons. It is on the volcanic portion of this group of rocks that La Grande Dam is built.

Dam Site Geology

The topography of this part of the Nisqually Canyon is rugged and covered with timber. La Grande Dam was constructed in a narrow part of the canyon. In the

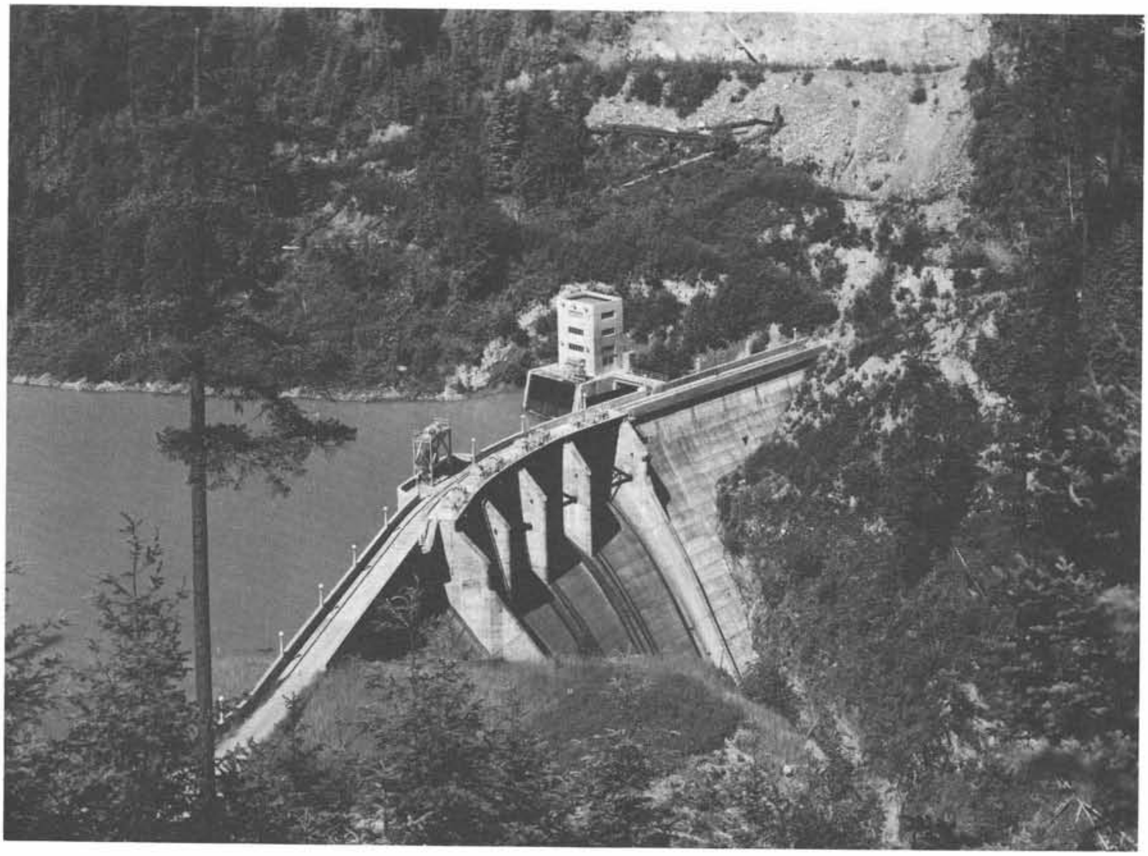

Figure 1. La Grande Dam. The intake structure for the tunnel shows above the right center of the dam. Photo by Tacoma City Light.

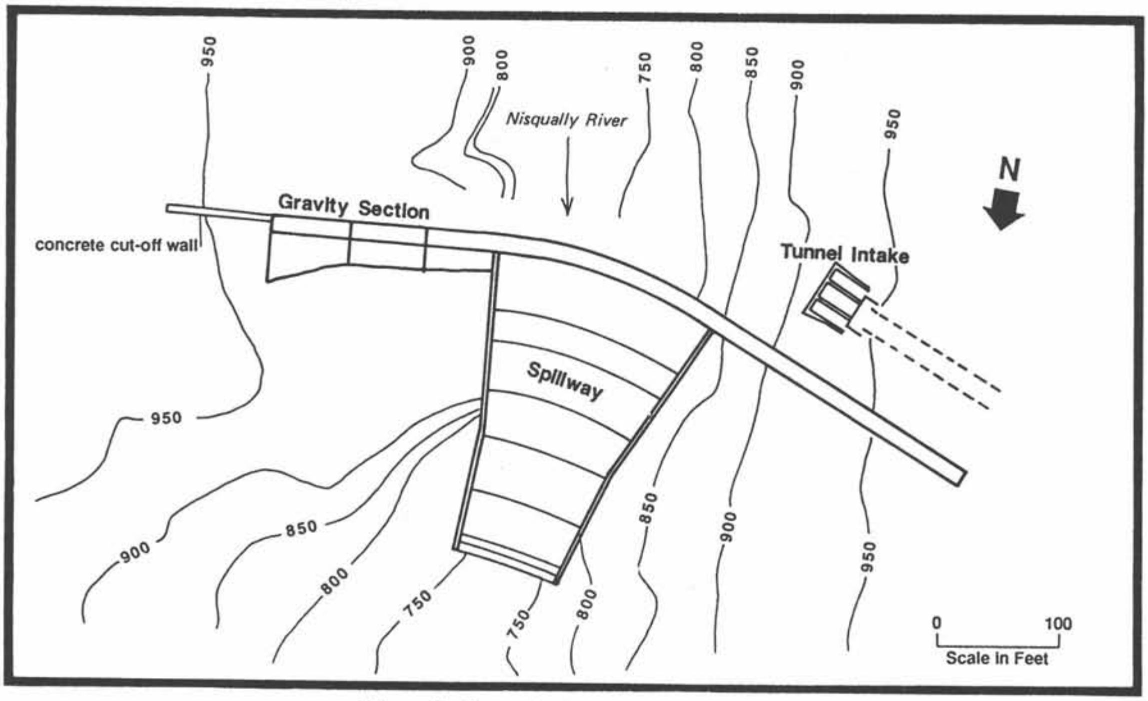

Figure 2. General plan of La Grande Dam.

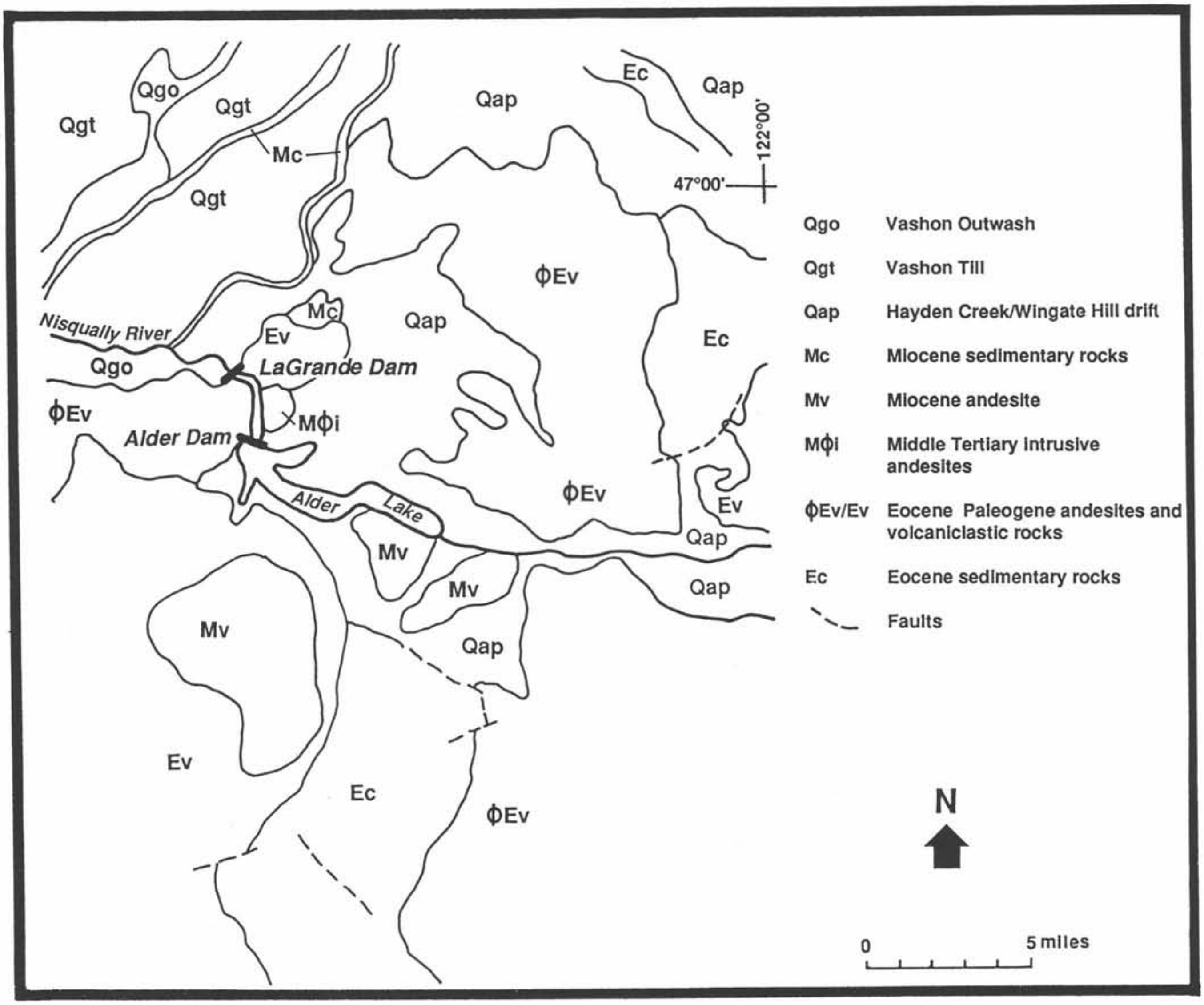

Figure 3. Geologic map of the La Grande Dam and Alder Dam area. Eocene to Miocene volcanic and sedimentary rocks cover most of this area. The exception is the continental ice marginal channels and outwash shown in the upper left corner of the map. After Walsh et al., 1987; Crandell and Miller, 1974.

vicinity of the dam the right canyon wall rises nearly vertically 160 ft to a poorly defined terrace that slopes gently upward to an elevation of 1,000 ft, or more than 60 ft above the crest of the dam (Figure 4). The canyon walls are notched by gullies eroded along nearly vertical dikes, faults, and shear zones that trend roughly normal to the axis of the valley. Immediately downstream of the dam, the right valley wall broadens out abruptly in the area of an old landslide.

The foundation and abutments of the dam were explored by a combination of vertical and inclined borings extending more than 200 ft into the abutments and about 100 ft below the bottom of the river channel. These borings show that up to an elevation of about 920 ft (or 22 ft below the top of the dam), the foundation and abutments are composed of uniformly hard and dense andesite. Although more than one flow may be present, there are no clear indications of soft interbeds or deep weathering between contiguous flows. Borings in the terrace on the right abutment encountered as much as 90 ft of sand, gravel, and talus blocks filling an old saucer-shaped, apparently glaciated, channel of the river. An impervious core wall, 5 ft thick, extending down into bedrock, was built across the old channel to cut off seepage under and around the right abutment. Sand and gravel deposits were also encountered in the left valley wall at an elevation about 900 ft. These deposits were removed and the dam founded on bedrock (Fucik, 1967).

The flow lines in the andesite could not be determined during the field inspection, but they may slope

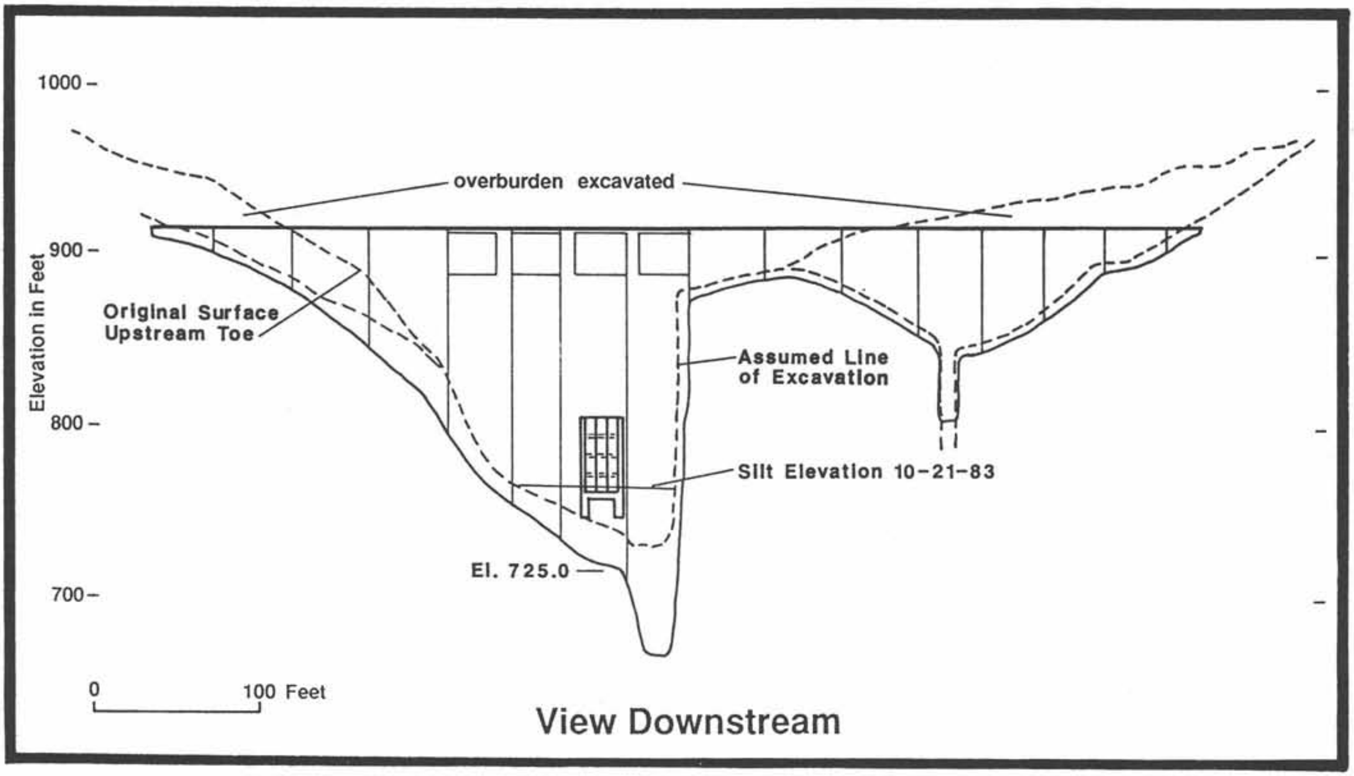

Figure 4. Section of La Grande Dam; view downstream.

into the right valley wall, more or less parallel to a set of widely spaced joints that dip at approximately 60°. Another much more conspicuous, nearly vertical set of joints trends parallel to the river channel. The joint spacing is generally more than 2 ft. Still another widely spaced, nearly vertical joint set trends about normal to the river channel. All joints are quite clean.

The topography of the valley walls and the borings indicated the presence of two nearly vertical fault or shear zones that strike obliquely to the river channel. One of these intersects the right bank upstream of the dam and may cut across the end of the straight gravity section. The other intersects the right valley wall near the upstream end of the spillway bucket and continues into the left abutment. Neither of these faults appears to act as a leakage channel. Another fault or shear zone in the former river channel and dipping steeply into the right abutment is visible in the construction photos. This fault was cleaned out and backfilled with concrete (Figure 4). The grout curtain helped to check leakage along the faults. Only a trickle of seepage can be seen along the face of the cliff that trends oblique to the right spillway retaining wall. No seepage is visible in the left abutment area. The core wall high in the right abutment appears to have been effective in checking leakage through the sand and gravel deposits filling the former channel of the river.

The spillway gate structure is curved in plan, with the flow terminating in a bucket which throws the water upward and away from the dam; the water flow lands in a pool in the river bed. The pool is about 80 ft downstream from the toe of the dam and is in good rock.

The wide spacing of the joints and faults and, more particularly, the apparent lack of any near-horizontal joints or other planes of weakness rule out the possibility of downstream sliding of the dam along shear planes in the foundation rock. The arch shape of the dam and the configuration of the gorge with respect to the foundation and abutments contribute further to the stability of the main part of the structure.

This is an area of moderate earthquake activity. During the 1965 magnitude 6.5 earthquake near Seattle, a large block of rock fell into the Nisqually Canyon between the powerhouse and the dam. No damage was reported. This was the second largest earthquake in the 150-yr recorded history of this area. To insure protection for the powerhouse, a deflecting wall has been constructed to divert falling rocks.

ALDER DAM

Project Description

Alder Dam (Figure 5), constructed in 1945, is 1.5 mi upstream from La Grande Dam and backs up a reservoir

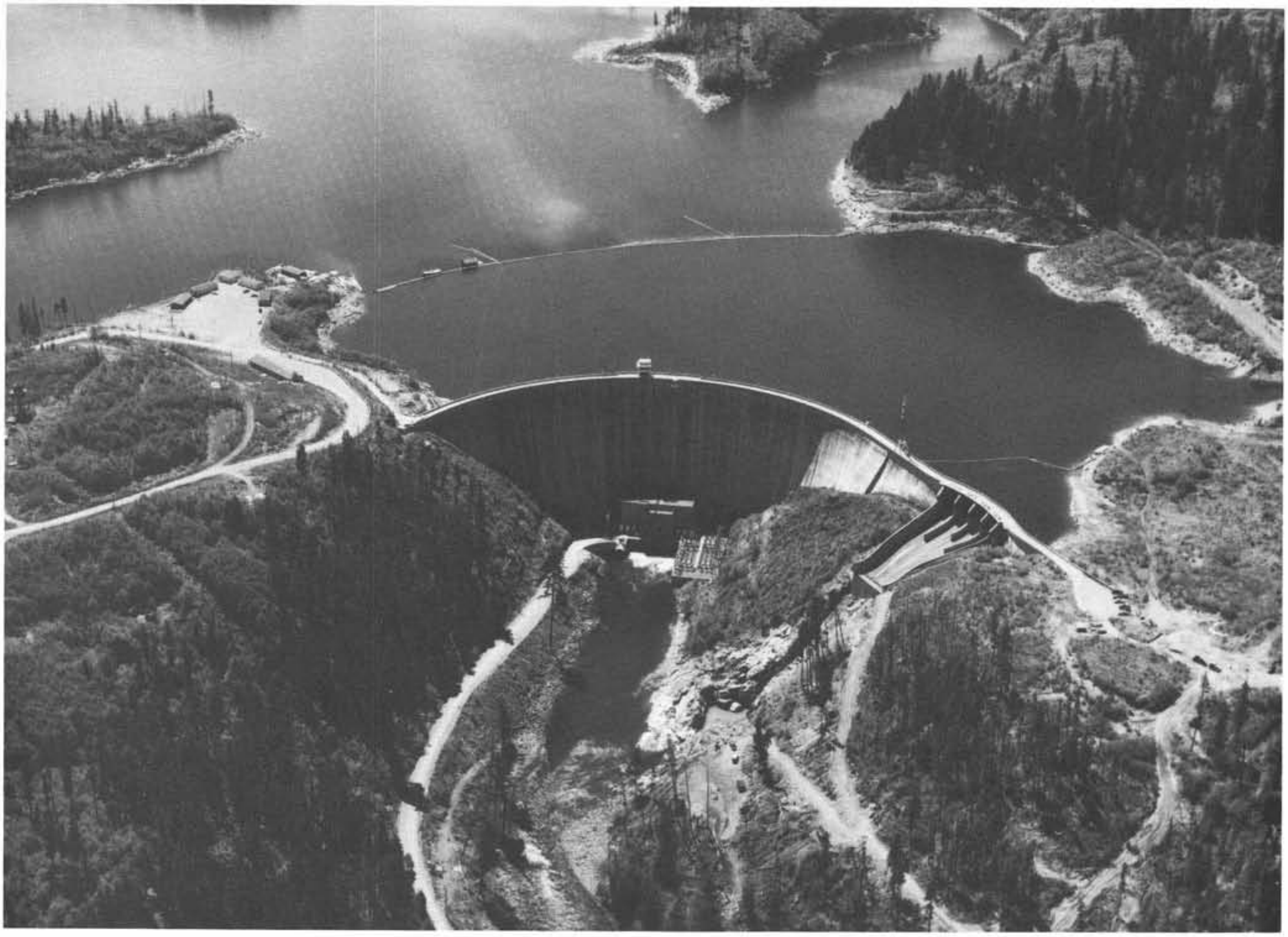

Figure 5. Alder Dam. Note the spillway on the right side of picture. The deep hole excavated by heavy spill is at the base of the cliff underneath the spillway. Photo by Tacoma City Light.

approximately 7 mi in length. Engineering design was done by the City of Tacoma, Light Division staff with C. P. Berkey as consulting geologist and J. L. Savage as consultant on design. In 1970 the Alder spillway gates were increased 7 ft in height to provide more storage.

The Alder project is composed of a 330-ft-high concrete arch dam 1,000 ft in length, a chute spillway on the left abutment, and a powerhouse (Figure 6). A single low sluiceway through the center of the dam, controlled by a Howell-Bunger valve, was provided to move accumulated glacial rock flour out of the reservoir. A low thrust block and a gravity, nonoverflow section 175 ft long link the left end of the arch to the spillway. The arch itself is fairly massive and of the variable radius type. The powerhouse is located in the riverbed next to the dam.

Dam Site Geology

Alder Dam is situated in a narrow gorge bordered by a thick series of massive andesite flows. The valley walls rise on a fairly uniform slope of 1H to 1V to an elevation 125 ft above the crest of the dam on the right side and a little less steeply on the left side.

At the time of construction, the foundation and abutments were explored by inclined borings spaced on roughly 100-ft centers along the axis of the proposed dam and extending to a depth of about 100 ft below the ground surface. The borings indicated that more than one andesite flow is present, but the number of flows involved and the dip and strike of the flow lines could not be determined in the field. The boring logs do not give clear indications of soft interbeds and present no conclusive evidence of significant weathered zones between individual andesite flows. Breccia and dikes were logged in the old power tunnel under the left abutment immediately upstream of the dam. It is inferred that the breccia is associated with the dikes rather than representing flow breccia near the top of separate flows. Surface exposures downstream of the dam tend to substantiate the inference that no weak interbeds, flow breccias, or deeply weathered zones exist in the founda-

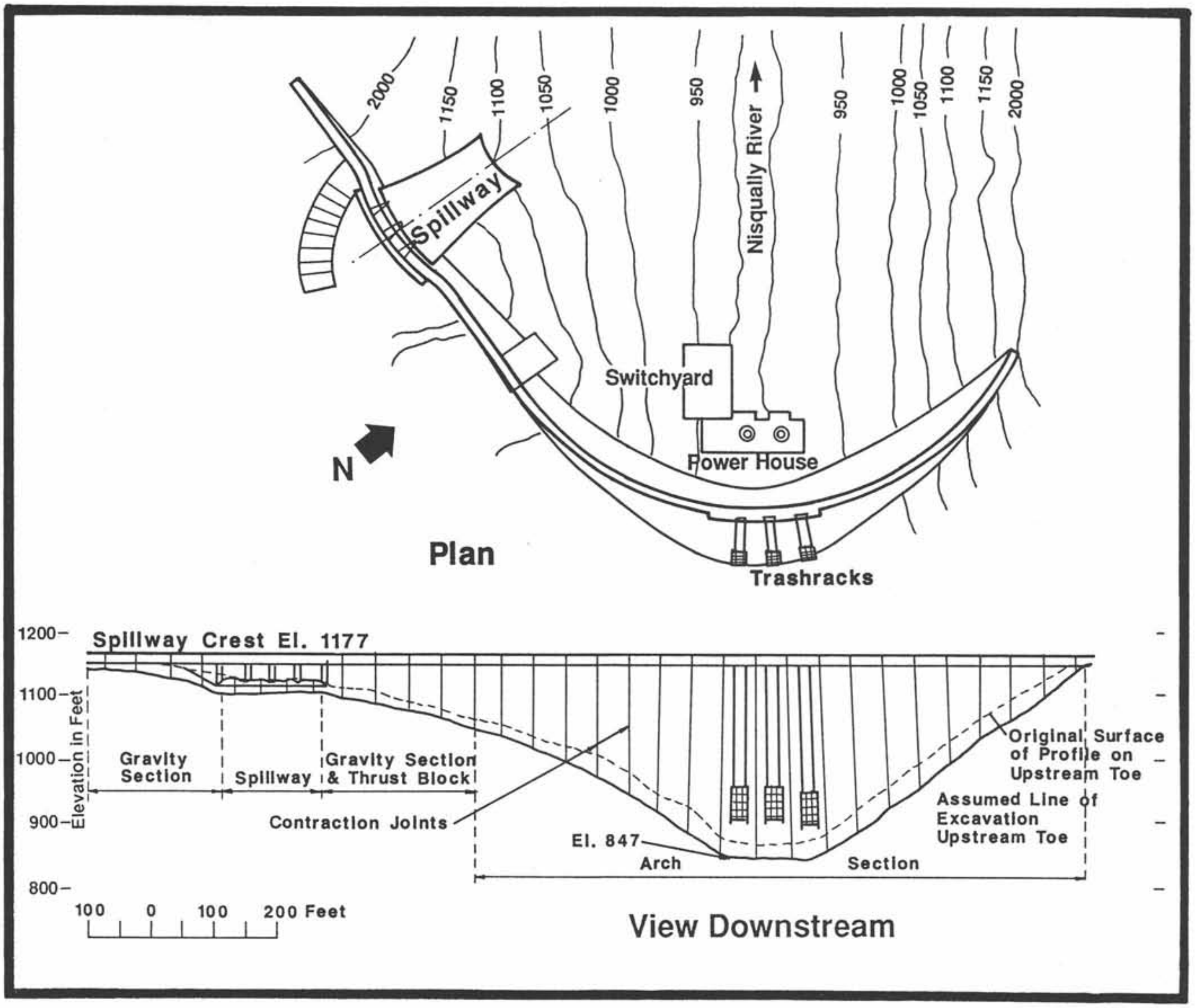

Figure 6. Plan and section of Alder Dam; view downstream.

tion and abutment rocks. Construction photos also tend to bear out this conclusion.

The valley walls are traversed by two prominent sets of widely spaced, steeply dipping to vertical joints; one set strikes nearly parallel to the axis of the valley, the other strikes nearly normal to the first. The spacing is generally more than 2 ft. Thin seams of hard minerals fill at least some of the joints.

In addition to the joints, the valley walls (the left wall particularly) are cut by nearly vertical fault and shear zones that trend roughly parallel to the valley. The zones' widths generally do not exceed a few feet. Seams of soft gouge occur in these fault and shear zones, which are generally too narrow to have much engineering significance.

A nearly vertical fault zone about 20 ft wide, trending parallel with the river channel, was encountered during excavation in the river bed. On the upstream side of the dam, the sheared and broken rock and associated fine-grained gouge was mined out to a depth of 92 ft and replaced with a massive concrete plug. A similar treatment was used on the downstream face, where the fault material was removed to a depth of about 45 ft. In the area between the two plugs, the concrete of the dam was designed to arch over the relatively weak faulted zone. The rock under both concrete plugs was grouted to considerable depth below the foundation. This treatment has been entirely satisfactory in controlling seepage.

The abutment grouting also appears to have been effective. Very little water is present at the downstream edge of the dam along both abutments. Some damp spots are visible, but no flowing water is observed. On the right abutment from the generator floor level to about 20 ft above, a small flow, less than 5 gpm, is visible about 50 ft downstream from the dam. This is an area which drained some water before and during construction, and the flow is probably surface drainage.

No drainage gallery was provided in the base of the dam. Foundation drain holes, drilled 10 ft on centers, collect seepage into a header and then to a transverse drain into a sump between the powerhouse and the dam. Very little flow has been observed.

The gated spillway on the left bank is a low structure, curved in plan, discharging into a restricted chute 225 ft long measured from the axis of the dam. The chute terminates in a bucket which throws the water upward, and the flow lands in a plunge pool to the left of the river bed (Figure 5). Construction photos show that the spill site is intersected by a nearly vertical zone of weakness a few feet wide that is filled with a dark-colored material; it could be either a weathered dike or gouge material. This line of weakness almost exactly parallels the centerline of the spillway chute. It is probably responsible for the slight indentation of the surface topography above the 1,080-ft contour line. The presence of the fault was well known at the time of construction. In view, however, of conditions imposed by the war, it was decided to delay any work on the hillside below the spillway chute until the eroding effects of its discharges could be more accurately determined.

Frequent discharges over the spillway in the first few years of operation scoured the narrow fault below the chute to a considerable depth. At the same time a plunge pool was eroded to roughly 100 x 150 ft in area by 80 ft deep (Figure 7). The plunge pool is at the base of the left bank slope, adjacent to the tailrace and about 800 ft downstream of the arch dam. The scoured material was deposited in the river, backing up the tailwater until removal was required.

In 1952 concrete was placed over the fault between the spillway bucket and the plunge pool to protect the spillway bucket from further erosion. To deepen the plunge pool and reduce the deposition of material in the river, a low massive concrete weir was placed along the outside rim of the plunge pool. Inside the plunge pool, reinforced concrete lining was placed where rock required protection.

Figure 7. The bottom of the hole excavated by heavy spill at Alder Dam. Photo by Tacoma City Light.

The plunge pool was pumped out and examined during the 1967 inspection. The rock exposed in the plunge pool is all hard and sound andesite, intersected by nearly vertical joints spaced many feet apart; one set of joints nearly parallels the axis of the spillway chute, the other is nearly normal to the chute. Widely spaced, nearly horizontal partings that may coincide with the boundaries between successive andesite flows are observable in the upper levels of the plunge pool. Another nearly vertical fault, trending about normal to the spillway chute axis, intersects the plunge pool. This fault has been scoured to form a marked "slot" on the downstream face of the plunge pool. Continued erosion may eventually round off the corner between this slot and the downstream side of the slot paralleling the axis of the spillway chute, but the geologic structure could not lead to undermining of the spillway bucket so long as the concrete filling the vertical fissure below it remains intact. There was no damage to concrete placed in 1952.

Judging from pictures taken at various times and from records and memory of operating personnel, the bulk of the material was eroded from the pool in the early days of operation. Some change in depth and horizontal area has occurred, but indications are that the pool is approaching a stable condition. Owing to the nature of the materials in the fault, some small changes in area may occur each time the spillway is operated. Since the rock in the area away from the faults is massive and sound, the change of area and depth due to future operations is expected to be small. Changes are not expected to affect in any significant way the safety of the dam (Coombs, 1972).

ACKNOWLEDGMENTS

E. E. Coates, Director of Tacoma Public Utilities, and L. H. Larson, Chief Civil Engineer, Light Division, have been most generous in supplying information on the La Grande and Alder hydroelectric projects.

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Skookumchuck Dam

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PROJECT DESCRIPTION

Skookumchuck Dam is located about 12 mi northeast of Centralia on the Skookumchuck River, a tributary to the Chehalis River. The project was constructed for Pacific Power & Light Company and Washington Water Power Company as a storage dam to provide a water supply for the coal-fired Centralia Power Plant. Water is released from the dam and is pumped 3 mi to the power plant northeast of Centralia from a diversion pumping plant located 10 mi downstream of the dam.

Skookumchuck Dam consists of a zoned embankment structure with an ungated side-channel, concrete chute spillway on the left (south) abutment (Figure 1). A gated intake pipe upstream of the left abutment feeds a steel outlet conduit, which passes beneath the dam. Facilities are also provided for the trapping of migratory fish that are then transported by truck around the dam. The crest of the dam is at elevation 497 ft, 175 ft above the stream bed. Maximum reservoir elevation is 477 ft, and minimum reservoir elevation is 400 ft. The reservoir extends 4 mi east of the dam.

The dam was designed and construction supervised by the Bechtel Corporation with Lockheed Shipbuilding and Construction Company as the prime contractor. Construction began in the spring of 1969 and was completed in October 1970.

SITE GEOLOGY

The Skookumchuck River drains a section of the Bald Hills/Huckleberry Hills region, a westward protuberance of the Cascade foothills lying south of the Puget Sound Basin. The segment of the valley in which the dam and reservoir are situated appears to be a small ice-marginal channel cut 300 to 500 ft below the general upland surface 2 to 4 mi south of and paralleling the Pleistocene ice border (Bretz, 1913). The valley floor is about 500 ft wide at the dam. Less than 1 mi downstream (west), the valley floor enlarges to a mile in width owing to confluence with a major ice-marginal channel. The valley floor narrows to less than 200 ft immediately upstream from the dam site. The glaciofluvial, glaciolacustrine, and alluvial deposits beneath the valley floor at the site range from 25 to 30 ft thick but locally attain a thickness of 60 ft. It appears that the site area was a bedrock rapids during periods of ice-marginal drainage, although the presence of silt units in the overburden sequence on the valley floor implies periods of ponding, presumably by downstream ice or detrital damming (Lea, 1984). Areas adjacent to the valley are mantled by deposits of decomposed gravel, sand, silt, and clay (mudflow?), probably in part equivalent to the early Pleistocene Logan Hill Formation (Bechtel Corporation, 1971).

The area is underlain by a series of gently west- to southwest-dipping basalt flows with intercalated flow breccia and tuff, all of which are assigned to the Eocene Northcraft Formation (Walsh et al., 1987; Bechtel Corporation, 1971). These volcanic rocks are overlain by coal-bearing sandstones and shales of the Skookumchuck Formation that are exposed a short distance downstream (Figure 2).

Several northwest-trending faults cut the volcanic rocks and the sediments (Walsh et al., 1987). The jointing in the Northcraft Formation consists of northeast- and northwest-striking sets, many of which have been healed by secondary mineralization (zeolites, calcite, and quartz). Near the edges of rock cliffs, relief joints have opened several inches (Bechtel Corporation, 1971).

The flow breccias and tuff zones are unevenly distributed both areally and stratigraphically in the basalt sequence. Though generally massive and sound, the breccias are locally chemically altered and soft to depths greater than 50 ft and are subject to rapid atmospheric slaking. The tuffs tend to react to exposure in a similar manner. The rock mass is deeply weathered (10 to 50 ft); the breccia and tuff are more susceptible than other rock types to this deep weathering.

Figure 1. Skookumchuck Dam. Aerial view east. Downstream berm is shown at the toe of the dam. Spillway cut is at the right, along the edge of the trees. Photo courtesy of Pacific Power & Light Co.

GEOLOGIC ASPECTS OF SITING AND DESIGN

The dam apparently was sited to take advantage of the farthest downstream segment of the basalt-flanked canyon that would provide suitable abutments and a reasonably shallow bedrock surface beneath the valley floor. The asymmetric character of the valley cross-section provided optimum siting of spillway and outlet facilities on the more gentle south side. This asymmetry also enabled placement of the outlet/diversion conduit in bedrock low on the south slope, thus eliminating the requirement for an expensive diversion tunnel.

The embankment slopes were designed 2.5H to 1V, consistent for founding the shells on granular alluvium. Where the foundation was underlain by fine-grained alluvium, the embankment required addition of toe berms.

CONSTRUCTION

Stages

Construction was accomplished using a four-stage diversion plan that included the use of the natural river channel, construction of two diversion channels, and, ultimately, diversion through the outlet conduit. Only minor dikes were required for cofferdams to permit construction of the grout curtain beneath the future core of the dam and low parts of the core and filter zones. Deep wells were drilled to assist in dewatering of core construction areas in the absence of cut-off walls to rock. The outlet works, conduit, and spillway were constructed in the dry above the water table.

Excavation and Foundation Preparation

Embankment Dam

The overburden on the valley floor was removed to the bedrock surface beneath the future core area (Figure 3). The rock face on the right abutment was resloped to 0.25H to 1V adjacent to the core area. This required 10,000 to 15,000 cy of rock excavation, which was accomplished in a single delayed blast. The shells of the dam were intended to rest on the sandy gravel alluvium of the valley floor and the locally alluviated bedrock surface on the right side of the valley.

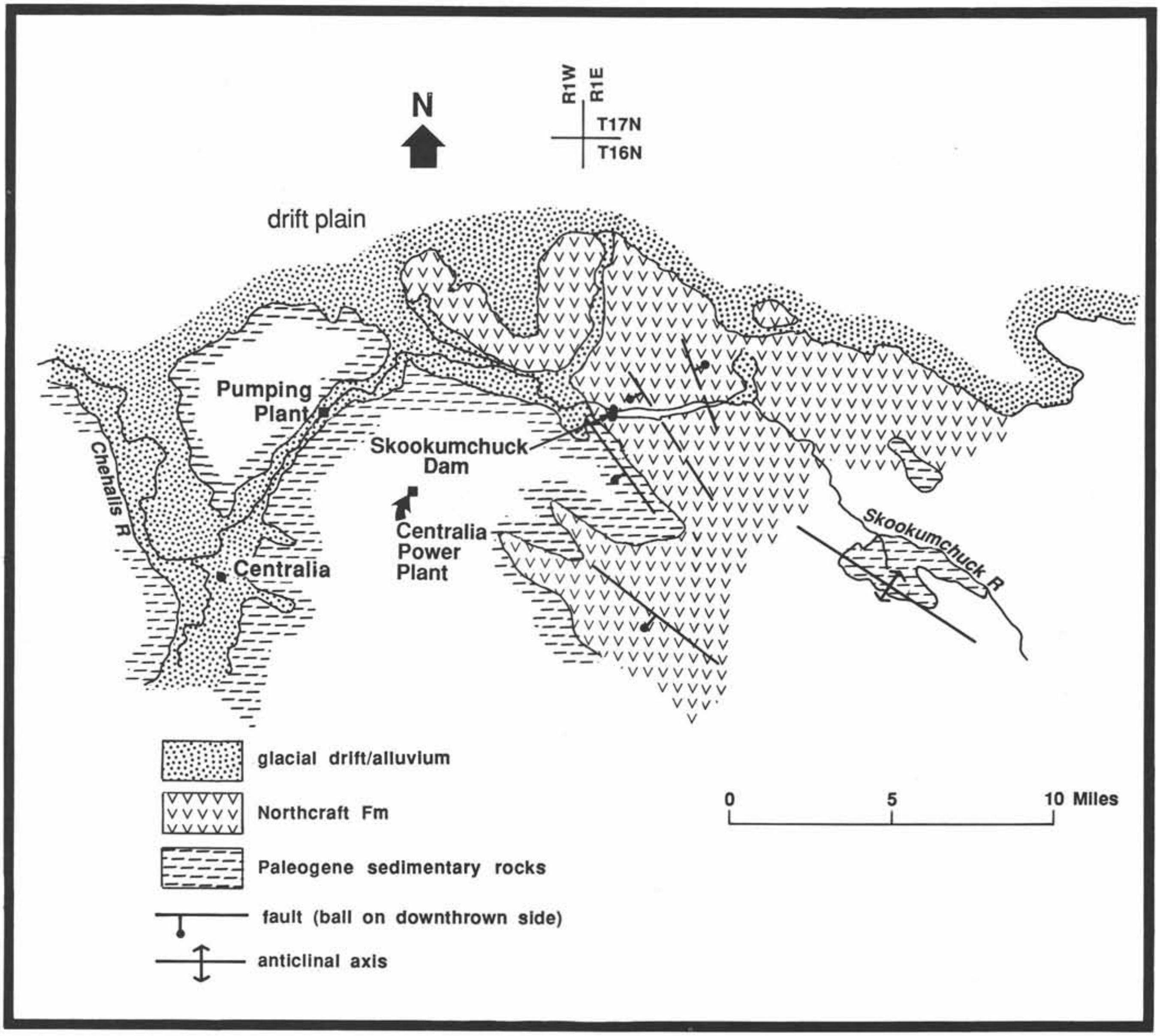

Figure 2. Generalized geologic map of the area near the Skookumchuck Dam and Centralia Power Plant. After Walsh et al. (1987).

A grout curtain was installed beneath the future core. Primary holes were 50 to 80 ft deep at 40-ft intervals (Figure 3). Secondary and tertiary grout holes were generally drilled to depths of 50 ft, and the final spacing of grout holes beneath the valley floor section of the core ranged from 5 to 15 ft. The rock abutments were grouted to depths of 60 ft (measured at right angles to the slope), using similar spacings as appropriate to grout takes experienced. The grout curtain was constructed beneath the future uncontrolled spillway weir and beyond; it extends 60 to 120 ft below foundation grade. The overall average grout take was 0.63 sacks/ft.

The bedrock foundation for the core and filter was prepared by mechanical and hand removal of loose material and cleaning with high pressure air-water jets. Where extensive jointing was present, slush grout was used to seal the foundation.

As excavation work progressed, an extensive deposit of silt, unrecognized during preconstruction investigation, was discovered beneath the valley floor. Exploration indicated that the deposit attained a thickness as great as 40 ft beneath the downstream embankment shell area, locally extending to the bedrock surface. As part of the shell had already been constructed, the silt was left in place and toe berms were designed and constructed both upstream and downstream to insure the stability of the embankment under dynamic (seismic) loading.

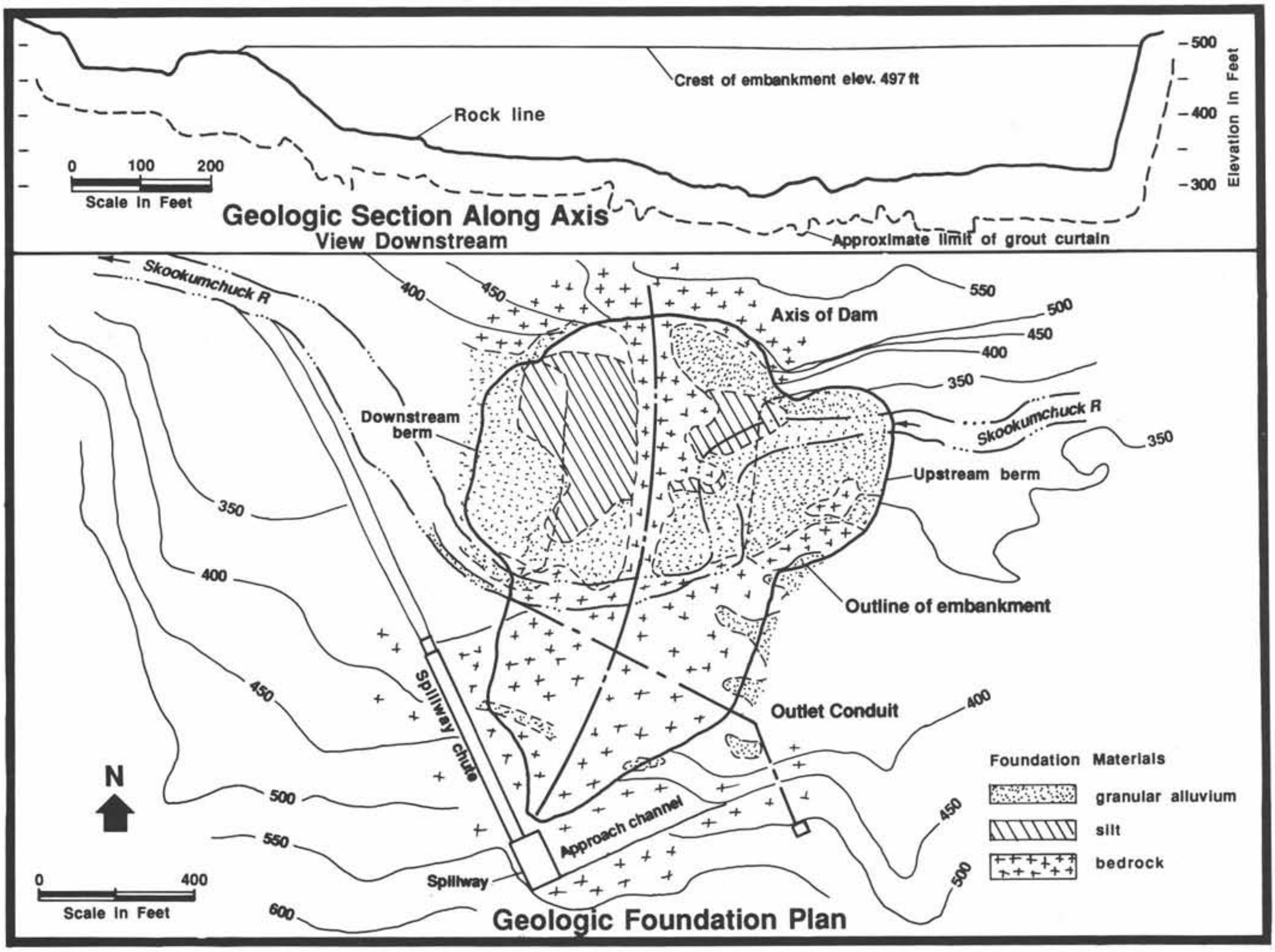

Figure 3. Geologic section along axis and centerline of grout curtain and foundation plan at Skookumchuck Dam. After Bechtel Corporation (1971).

Spillway

Construction of the spillway weir and chute required a combination overburden/rock cut, part of which is more than 100 ft deep. About 75 percent of the 220,000 cy of material removed required rock excavation. The overburden cut, originally designed for a 1.5H to 1V slope, was resloped to 2H to 1V after a 3,000-cy slide occurred. Rock excavation was made in 10- and 20-ft lifts using 7 x 7 ft and 5 x 5 ft blast hole patterns and a powder factor of 0.75 lb/cy. Grouted rock bolts were installed as needed to insure stability of the rock slope and the entire slope was mantled with wire mesh on the 0.5H-to-1V slopes above the concrete chute. The concrete chute was anchored by installation of reinforcing bars into the rock surface and by six keyways.

Outlet works

The small gate-operating platform at the head of the outlet works is founded on bedrock, as are the access road bridge piers. A 4-ft lowering of the foundation of one bridge pier was necessary to provide an adequate foundation on deeply weathered rock. The 48-in.-diameter steel intake pipe lies on a 27° slope; inlets are at elevations 450, 420, and 375 ft. The pipe rests on flange supports; some of these are on backfill and some are anchored into bedrock. The 3-ft- to 6-ft-diameter steel conduit beneath the dam lies in a notch as much as 10 ft deep in the bedrock surface, which was excavated in a single lift. The conduit was fully encapsulated in concrete prior to construction of the embankment.

Construction Materials

Except for cement, all major construction materials were obtained from project excavation or from borrow areas near the dam. Shell materials were obtained from a borrow area in glacial outwash gravels about 1 mi downstream. Impervious core material was obtained from flood-plain deposits both upstream and downstream of the dam and from mudflow deposits of the Logan Hill Formation a short distance upstream and above reservoir level. Shot rock for embankment slope protection and berm construction was obtained from excavation for the spillway, spillway intake channel, and a quarry in basalt on the north bank just upstream from the dam.

OPERATIONAL PROBLEMS RELATING TO GEOLOGY

No major problems relating to geology have become apparent during the succeeding years of project operation. A minor amount of leakage issues from a flow contact about 300 ft downstream from the north toe of the embankment. The spillway chute has been plagued by considerable raveling of the closely jointed rock exposed in the cut above the chute. Raveling has progressed to the point where some rock bolts are ineffective (U.S. Army Corps of Engineers, unpublished data, 1981).

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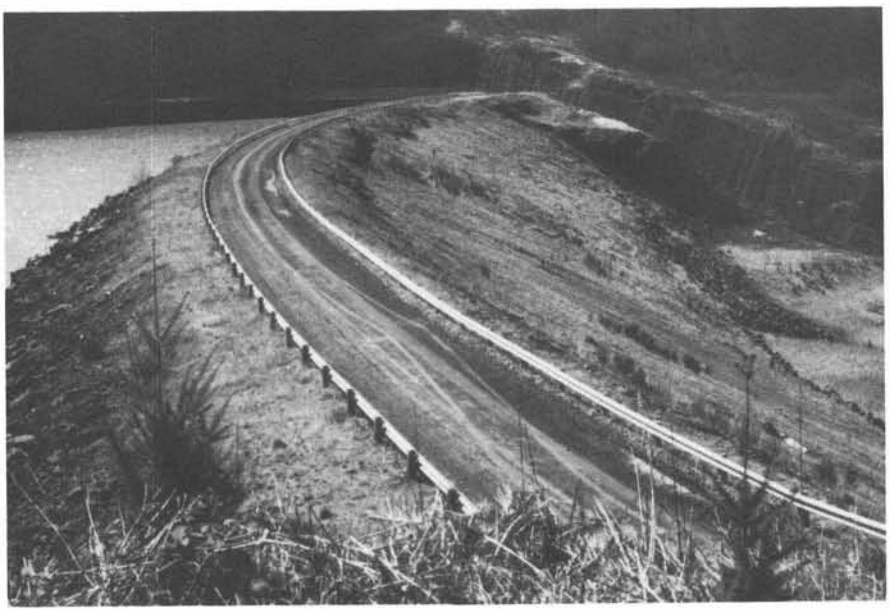
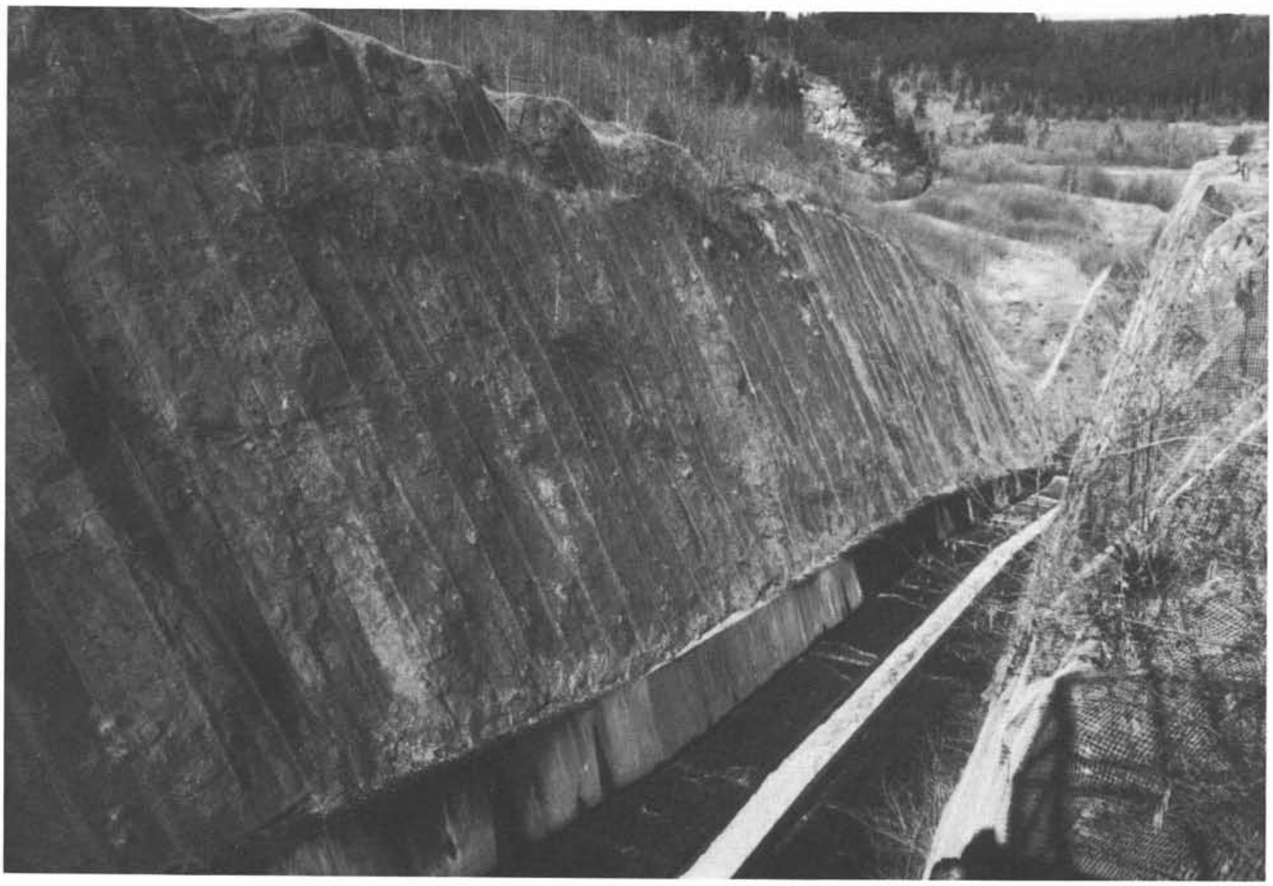


View to the south of the Skookumchuck Dam embankment from above the right abutment of the dam. The spillway structure is at the far end of the dam. Photograph by R. W. Galster, March 1981.



The Skookumchuck Dam spillway discharge chute on the left (south) bank. The cut is in basalt and has been draped with welded wire netting to control rock fall. The chute is designed so that low spillway flows are contained in a narrow central slot to permit ease of maintenance during low flows. Photograph by R. W. Galster, March 1981.