Waterpower Resources of the United States 1960-65

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1610-D

This volume was published as separate chapters A–D

UNITED STATES DEPARTMENT OF THE INTERIOR

STEWART L. UDALL, *Secretary*

GEOLOGICAL SURVEY

William T. Pecora, *Director*

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Waterpower Resources of the Bradley River Basin Kenai Peninsula Alaska

By F. A. JOHNSON

WATERPOWER RESOURCES OF THE UNITED STATES

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1610-A

Potential power of Bradley Lake and related geographic, climatic, and hydrologic factors

UNITED STATES GOVERNMENT PRINTING OFFICE, WASHINGTON : 1961

UNITED STATES DEPARTMENT OF THE INTERIOR

STEWART L. UDALL, *Secretary*

GEOLOGICAL SURVEY

Thomas B. Nolan, *Director*

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington 25, D.C.

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WATERPOWER RESOURCES OF THE BRADLEY RIVER BASIN, KENAI PENINSULA, ALASKA

By F. A. Johnson

ABSTRACT

A survey of Bradley Lake and the Bradley River, including underwater contouring at the lake, was made in 1955 for use in studying possible plans of waterpower development. The survey was supplemented by surface geologic examinations.

Illustrative plans of development in accord with the topography and geology are described in this report, and used with runoff records and estimates to compute the potential waterpower. Runoff records available in 1960 covered only two complete water years, October 1957 to September 1959. These were supplemented by estimates of monthly runoff from October 1949 to September 1957 based on a relationship with the runoff of the nearby Kasilof River and records for that stream. The runoff at the outlet of Bradley Lake was thus determined to have been about 334 cfs (cubic feet per second), or 84 inches per year on the drainage area of 54 square miles, from 1949 to 1959.

Runoff from an additional area of 14 square miles could readily be diverted into the Bradley Lake basin to increase the water supply about 26 percent, and the waterpower estimates were made accordingly. It was determined that a usable storage capacity of 300,000 acre-feet would have provided for substantially complete control in the 10-year period, 1949–59, with uniform release of 421 cfs. Development through a mean gross head of 1,131 feet to a powerhouse at tidewater some three and a half miles from the lake would provide for generation of 32,400 kw (kilowatts) continuously. The storage capacity could be obtained by a dam at the lake outlet to raise the surface 104 feet, or alternatively by a lower dam and a tunnel to tap the lake at depth.

The runoff from 1949 to 1959 is estimated to have been about 90 percent of that during the 40-year period prior to 1960. A lesser degree of development corresponding to control and utilization of about 80 percent of the 40-year mean runoff also was considered. This would have required about 171,000 acre-feet of capacity for uniform release of 375 cfs. Development through a mean gross head of 1,110 feet would provide for generation of 28,300 kw continuously. The storage capacity could be obtained by a dam at the lake outlet to raise the surface 68 feet, or by a combination of damming and lake drawdown.

Water could be conveyed from the lake to a powerhouse at tidewater by means of tunnels and pipelines along several alternative routes. It would be possible also to convey it along the Bradley River for development in two nearly equal stages at powerhouses located on the river.

Geographic, climatic, and hydrologic features of the Kenai Peninsula are discussed briefly. The runoff of the Bradley River is consistent with a mean annual precipitation of about 100 inches; whereas precipitation is less than 20 inches a short distance to the north in the rain shadow of the Kenai Mountains.

INTRODUCTION

PURPOSE AND SCOPE

The purpose of this report is to present an evaluation of the potential waterpower in the Bradley River basin as a basis for classifying the public lands with respect to their values for waterpower purposes. Possible plans of development, based on topography, geology, streamflow, climate and related information are outlined as a means of estimating the potential waterpower.

Records of the flow of the Bradley River that are available at the time of this report (1960) cover only two complete water years, 1957–1958 and 1958–1959. Figures of monthly runoff for the seasons 1949–1950 to 1956–1957 were estimated from the records of the nearby Kasilof River near Kasilof and used with the two-year record in reservoir operation schedules for given degrees of regulation.

Since the runoff may be considerably affected by changes of natural storage in snow or glacier ice, the climatic factors that determine such changes are discussed.

The utilization of a substantial part of the potential power of Bradley Lake probably will depend on the creation of new industries. Normal needs of farms and the small communities of the Kenai Peninsula would not absorb much of the Bradley Lake potential in the foreseeable future. A need may develop for some of the power at Anchorage through future growth of that city. A hydroelectric plant at Bradley Lake could be interconnected with a plant under construction at Cooper Lake (1960), and with other hydroelectric plants which may be constructed on the Kenai Peninsula, for joint transmission of power to the Anchorage area. The transmission distance from Bradley Lake to the vicinity of the Cooper Lake plant, near the lower end of Kenai Lake, is roughly 90 miles, and to Anchorage roughly 155 miles over a circuitous route, some of which is near existing highways.

OTHER INVESTIGATIONS

The Alaska District, U.S. Army Corps of Engineers, made a reconnaissance investigation of the Bradley Lake site in September 1954. A report based on this investigation and other available information was compiled by the Corps of Engineers (1955) at the request of the Governor of Alaska. It was estimated that an average discharge of 320 cfs would be available from about 54 square miles of drainage area directly tributary to the lake, plus about 4 square miles of the

adjacent Nuka River basin that could readily be diverted to Bradley Lake. It was further estimated that this supply could be completely regulated for generation of 23,000 kw of prime power, with 325,000 acre-feet of storage capacity at the lake.

Several possibilities were described for auxiliary development of run-of-the river power in the lower Bradley River basin and on nearby Battle Creek. It was pointed out that this seasonal power could be firmed up by providing additional storage and plant capacity in the Bradley Lake project.

The report of the Corps of Engineers includes descriptions of possible industrial sites at Homer, at Halibut Cove across the bay from Homer, and at Kasilof. Construction details and possible costs of power development, subjects outside the scope of the present report, also were discussed.

In October 1959 geologists and engineers from the office of the Alaska District, Corps of Engineers, made several field reconnaissances of the Bradley Lake power site; two core holes were drilled in the channel of the Bradley River near the lake outlet; and a test pit was dug in the saddle on the right abutment. It was concluded that the site is highly satisfactory for any type of dam, that rock available at the site is not suitable for coarse concrete aggregate, but that it has been found suitable for use in a rockfill dam.

ACKNOWLEDGMENTS

Information concerning recent investigations of the Corps of Engineers was supplied by the office of the District Engineer, U.S. Army Engineer District, Anchorage, Alaska.

GEOGRAPHY

GENERAL DESCRIPTION

Bradley Lake basin lies in the Kenai Mountains, which extend from southwest to northeast along the Kenai Peninsula. The lake is drained by the Bradley River, which flows in a northward direction about 3 miles through a very rugged canyon, and thence 3 miles westward to the head of Kachemak Bay, an arm of Cook Inlet. (See fig. 1.) The reach below the canyon is across the side of a wide delta formed by deposits from Bradley River and two glacial streams flowing southward to the bay; Fox River and Sheep Creek. The head of Kachemak Bay is featured by an extensive tidal mud flat, but the water deepens abruptly to 60 feet or more in and near Bear Cove, 4 miles southwest from the head of the bay.

The Bradley Lake basin is characterized by glaciers, extensive exposures of bare rock, and relatively little soil cover. (See fig. 2.) The

basin is uninhabited and entirely undeveloped. Some occupation by settlers has taken place along the north side of the Fox River valley and northwest of the head of Kachemak Bay within 10 miles of Bradley Lake. If such settlement increases sufficiently a road presumably may be extended to this area from Homer.

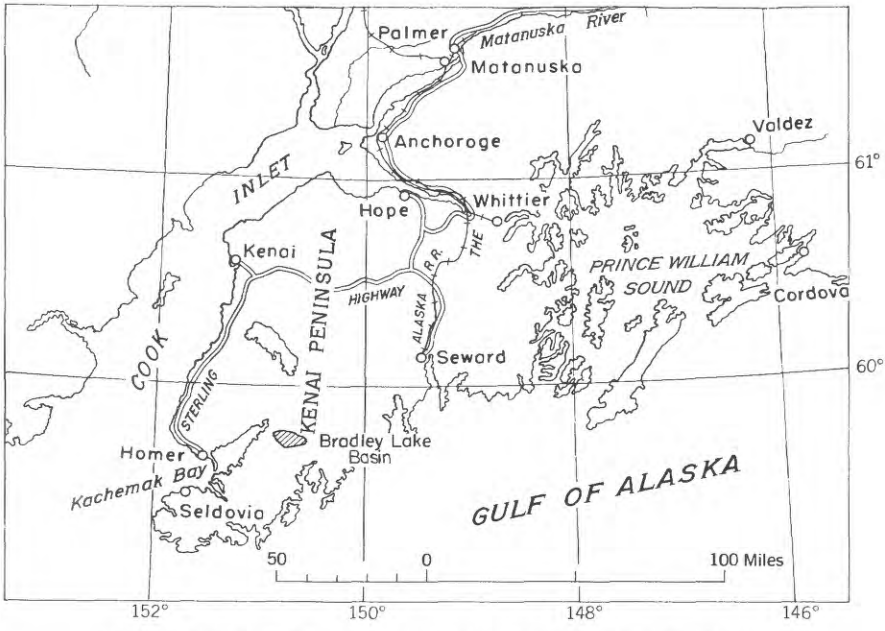

FIGURE 1—Index map of Kenai Peninsula, Alaska, showing location of Bradley Lake.

FIGURE 2—Bradley Lake looking southeast from right abutment at the damsite toward the headwaters of the basin. Kachemak Glacier in background, right of center, July 1955.

Areas above selected altitudes in the Bradley Lake basin and in nearby basins are listed in table 1.

TABLE 1—*Drainage areas in square miles above selected altitudes in Bradley Lake basin and adjacent basins*

Altitude (feet)	Bradley Lake basin		Nuka River diversion area		Diversion area north of lake		Lake basin plus diversion areas	
	Area	Percent of total	Area	Percent of total	Area	Percent of total	Area	Percent of total
1,000	54.0	100	4.1	100	10.2	100	68.3	100
2,000	36.3	67	3.6	88	10.1	99	50.0	73
2,500	31.3	58	3.3	81	9.6	94	44.2	65
3,000	25.5	47	2.5	61	8.3	81	36.3	53
4,000	8.4	16	0.8	20	4.1	40	13.3	19

Homer, the nearest town, is on the north side of Kachemak Bay about 25 miles to the southwest of Bradley Lake, at the end of the Sterling Highway. The population of Homer, according to the 1950 census, was 307. The Homer District which includes the villages of Anchor Point and Ninilchik (about 30 miles north) had a population 907. The town of Seldovia on the south side of Kachemak Bay, 15 miles southwest of Homer (but outside of the Homer recording district), had a population 437 in the 1950 census.

The principal industries at these places are fishing, and processing and canning fish. A number of homesteads have been established in the region near Homer, and to the north, and farming may increase in importance. Recently there have been extensive investigations of petroleum possibilities, and some discoveries have been made. Discovery of substantial reserves might have a considerable effect on development of the region. It appears, however, that demand for the substantial amount of power that could be generated at Bradley Lake will depend mainly on the creation of other industries. Manufacturing by electro-metallurgical or electro-chemical processes may offer the best possibilities. Lumbering and related activities will have little importance since forest growth in the region is scanty.

ACCESSIBILITY

At present the only practicable access to Bradley Lake is by float planes or small land planes. Development of the site for waterpower would necessitate construction of a road. This might be located from the region of deep water at Bear Cove along the hills bordering the south side of Kachemak Bay to the vicinity of Battle Creek on the Bradley River and thence southeastward to the lake, or from the north side of Kachemak Bay, connecting with a road from

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Homer, and extending around the head of Kachemak Bay to the Bradley River or Battle Creek. Possible routes up the mountainside to Bradley Lake are in rugged terrain, and considerable rock excavation would be required for road construction. (See figs. 3 and 4.)

FIGURE 3—Bradley Lake outlet and river downstream. The Bradley River at the bend in left foreground is 200 feet lower than the lake.

MAPS AND PHOTOGRAPHS

A map entitled "Plan and Profile, Bradley River and Bradley Lake, Alaska, Dam Sites" was published by the Geological Survey in 1956. The scale of the plan map is 1:24,000 and the contour interval is 20 feet. The topography is shown to about 200 feet above and to 250 feet below the lake surface. In addition, an area on a tributary of the Bradley River is shown at scale 1:24,000 and with a contour interval of 10 feet. (Works could be constructed there for diverting the stream into Bradley Lake.) A damsite at the outlet of Bradley Lake and one at mile 6.7 on the Bradley River are shown at a scale of 1:2,400 and with a contour interval of 10 feet.

FIGURE 4—Lower canyon of the Bradley River. Mile 6.7 damsite at point indicated by arrow.

The drainage basin of Bradley Lake and nearby regions are shown on the quadrangle maps:

Seldovia: Reconnaissance series, scale 1:250,000, contour intervals 200 and 1,000 feet.

Seldovia: C–2, C–3, C–4, C–5, D–2, D–3, D–4, D–5, scale 1:63,360, contour interval 100 feet.

The Bradley Lake basin is included in the quadrangles: Seldovia C–2, C–3, D–2, and D–3. The other quadrangles include regions in the vicinity of Kachemak Bay.

Soundings in Kachemak Bay are shown on Charts 8531 and 8554 of the U. S. Coast and Geodetic Survey. Chart 8531 is on a scale of 1:82,662 and Chart 8554 on a scale of 1:200,000. The soundings are shown in fathoms.

Aerial photographs used in compilation of the topographic maps are on file with the Geological Survey, Denver Federal Center, Denver, Colo.

GEOLOGY

The geology of the Bradley Lake powersite is described briefly in the report of the Corps of Engineers (1955), and in more detail by

Soward.[1] Some of Soward's findings are summarized briefly in the discussions of damsites and diversion routes given further on in this report.

CLIMATE

GENERAL

Most of the climatic records are for coastal points on the Kenai Peninsula, and none have been obtained in mountain regions near Bradley Lake.

Records of runoff and precipitation for several stations from Seward to Kenai show that precipitation in some parts of the mountains is greater than at Seward and very much greater than in lowlands or at coastal points on the northwest side of the Kenai Peninsula. Precipitation records indicate that the average annual precipitation at Kenai, Naptowne, and Kasilof probably is less than 20 inches, whereas it was about 66 inches at Seward during 39 years of record to 1959. The average annual runoff of the Snow River of the Kenai River basin (see fig. 1) is estimated to have been in excess of 90 inches, 1920 to 1959. This is from a basin of 166 square miles, ranging in altitude from 436 feet to more than 5,000 feet. The estimated runoff probably corresponds to a mean annual precipitation of roughly 100 inches. It is probable that precipitation in the headwaters area of the basin is considerably greater.

There is a marked decrease in winter temperatures northward from Seward to Naptowne, although summer temperatures are comparable at the two stations. Monthly mean temperatures at Naptowne from November to February may be 15°F to 20°F lower than at Seward and extremes may be even lower. Minimums of −40°F to −50°F apparently are not unusual at Naptowne and in the vicinity of Kenai Lake (at altitude about 500 feet), whereas those at Seward generally are about −10°F or higher. Records of runoff, precipitation, temperatures, and snowfall for this region are summarized by Johnson (1955). Climatic data for the Kenai Peninsula are published in "Summaries of Climatological Data" (U.S. Weather Bureau).

The general distribution of precipitation and temperature across the Kenai Peninsula in the vicinity of Bradley Lake probably is somewhat similar to that between Seward and Naptowne. The average annual precipitation at Kasilof is about 18 inches, as estimated on the basis of a relatively short record, and this is of about the same order of magnitude as that at Naptowne. Temperatures at Kasilof are somewhat higher than at Naptowne, and there probably is a gradual increase in precipitation and winter temperature around the coast to Homer, and beyond, where there is greater exposure to southerly winds from the Gulf of Alaska. The average annual precipitation at

[1] Report in preparation, 1961.

Homer is 25 inches, and the monthly temperatures generally are only a few degrees lower than at Seward. The deficiency in precipitation and the cold winters on the northwestern side of the Kenai Peninsula reportedly are due to the rain-shadow effect of the Kenai Mountains, and because the region is exposed to dry, north winds from the interior of Alaska.

The seasonal distribution of precipitation recorded at Kasilof differs greatly from that recorded at Homer, probably as a result of the factors just noted. The totals were computed for the two periods October to April and May to September, for the calendar years, 1940 to 1947, when nearly continuous records were kept. During the first period the catch at Kasilof was less than half that at Homer, being 72.0 inches as against 145.1 inches for the 8 seasons. However, during the second period, May to September, precipitation recorded at Kasilof was more than at Homer, being 91.5 inches as compared with 79.7 inches. Precipitation such as that recorded at Kasilof may be expected to result in runoff, especially snow runoff, much less than at Homer.

Temperatures at Kasilof and Homer also differ greatly in the winter period, November to March. The mean for that period is 19.5° F at Kasilof and 25.9° F at Homer, and average January temperatures are 13.3° F and 22.6° F respectively. Minimum temperatures as low as −30° F to −40° F are not uncommon at Kasilof; whereas temperatures below −15° F are unusual at Homer. During the period, April to October, monthly temperatures are about the same at both places; the mean for the period is about 46° F.

The seasonal distribution of precipitation and temperature at Bradley Lake possibly resembles the pattern at Homer, or at stations on the southeast side of the Kenai Peninsula since it is only about 20 miles northwest of the Gulf of Alaska, and is subject to the weather patterns of the Gulf of Alaska.

CLIMATE AT BRADLEY LAKE

The Bradley Lake basin is near the crest of the mountains and, as indicated by glaciers, and by the two years of runoff records, receives heavy precipitation. The runoff of the Bradley River at Bradley Lake for the water years ending September 30, 1958 and 1959, was recorded as 121 inches and 74 inches respectively. From comparisons of these figures with other runoff records and long-term precipitation records the mean annual runoff during the past 40 years is estimated to have been about 92 inches, which probably corresponds with a mean annual precipitation of about 100 inches.

Winter temperatures at the lake probably are lower than at Homer, since the altitude is a thousand feet higher. It is probable also that

winter temperatures at sea level near the head of Kachemak Bay are lower than at Homer, since the region is somewhat less shielded from north winds, and less exposed to southerly winds. Ice forms in a so-called "fresh-water" area at the head of the bay, but the bay reportedly is always open as far as Bear Cove. Local residents also report the occurrence of williwaws at the head of the bay when the air is fairly calm at Homer.

In 1955 ice on Bradley Lake prevented landing by float plane until after July 1. Temperatures in May and June 1955 were below normal, and the lake reportedly is usually open by about the middle of June. It was observed to have been free of ice June 4, 1958 and June 12, 1959, at times of visits to the streamgaging station.

Strong winds occur frequently at the lake. During the course of the survey in July and August, 1955, there were 12 days in a 31-day period when a small boat could not be operated. These winds generally were from east to west along the length of the lake. The shape of trees near the outlet of the lake and the absence of limbs on the eastward side indicate that this is the prevailing direction of strong winds. This circumstance possibly may be due to flows of cold air down-slope, and to southerly, storm winds modified in direction by the local configuration of the mountains.

PRECIPITATION

The magnitude of precipitation on the Bradley Lake basin cannot be estimated directly from precipitation records at other places, since none have been obtained at comparable mountain localities. It is reasonable to assume, however, that the year-to-year distribution of precipitation is roughly the same throughout a considerable area, both in the mountains and at coastal points. Precipitation indices thus may serve as measures of the wetness of given years or periods at Bradley Lake, in relation to the average for longer periods. The average of precipitation recorded at Seward and Homer may be closely representative of the year-to-year distribution at Bradley Lake. Continuous records have been published for both stations from 1940 through 1959, and rounded figures for the water years ending September 30 are listed in table 2.

The average of the indices for the period 1950 to 1959 is 89 percent of the 20-year mean, and the 20-year mean in turn is about the same as an average (partly estimated) for 39 years at the Seward station. The lowest water year of record occurred in 1952, both during the 20-year record for the two stations; and at Seward during 39 water years

of fairly complete records. The year 1951 also was relatively dry, but 1953 was one of the two wettest years of record.

Records of monthly and annual precipitation for Seward and Homer, 1940–59, are given in tables 3 and 4, and records of monthly and annual mean temperature for Homer are listed in table 5.

TABLE 2.—*Precipitation, in inches and percent of mean, at Homer and Seward*

Water Year	Homer		Seward		Mean, columns 3 and 5, percent
	Precipitation	Percent of mean	Precipitation	Percent of mean	
1940	31.4	130	87.1	131	130
1941	38.3	158	88.8	134	146
1942	24.0	99	73.8	111	105
1943	17.2	71	52.7	79	75
1944	33.6	139	101.1	152	146
1945	33.5	138	74.2	112	125
1946	27.0	112	58.0	87	100
1947	22.8	94	64.9	98	96
1948	22.8	94	67.6	102	98
1949	21.4	88	58.2	88	88
1950	19.5	81	66.2	100	90
1951	16.2	67	47.0	71	69
1952	11.9	49	42.8	64	56
1953	35.3	146	101.3	153	150
1954	20.7	86	51.6	78	82
1955	22.9	95	63.1	95	95
1956	17.6	73	48.0	72	72
1957	17.6	73	45.0	68	70
1958	28.8	119	87.4	132	126
1959	21.9	90	49.3	74	82
Mean	24.2		66.4		

TABLE 3.—*Monthly and annual precipitation, in inches, Seward, Alaska*

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Year
1940	6.9	5.6	9.8	12.0	2.8	5.6	8.6	3.0	1.3	2.1	10.7	18.7	87.1
1941	14.9	6.4	5.9	2.4	14.6	12.1	13.9	4.6	3.0	5.5	.8	4.7	88.8
1942	3.4	2.2	11.1	8.2	10.4	6.6	5.0	1.1	3.8	3.6	10.5	7.9	73.8
1943	7.1	.9	1.0	2.7	6.8	1.7	4.5	2.2	3.4	3.6	3.6	15.2	52.7
1944	11.3	18.3	18.2	11.3	6.1	2.0	1.1	7.0	1.6	4.8	11.4	8.0	101.1
1945	12.2	7.4	6.2	12.1	11.0	3.0	1.3	2.2	1.9	2.2	8.1	6.6	74.2
1946	15.8	2.1	4.8	5.6	6.2	3.2	3.4	4.9	1.2	.8	4.6	5.4	58.0
1947	11.7	4.3	2.9	3.0	6.4	7.1	1.8	6.7	2.9	2.3	5.0	10.8	64.9
1948	7.7	12.6	8.7	12.5	4.8	1.3	0	3.7	1.5	4.3	1.0	9.5	67.6
1949	13.5	4.4	1.6	4.7	.6	6.1	3.7	1.8	2.0	1.0	5.6	13.2	58.2
1950	8.1	10.9	4.7	1.2	.9	3.0	6.1	5.5	3.8	1.4	4.8	15.8	66.2
1951	4.0	.3	6.1	3.3	3.8	2.6	4.7	2.6	2.9	.9	3.0	12.8	47.0
1952	5.1	4.9	2.6	1.8	3.8	3.1	4.7	1.7	1.4	5.3	4.4	4.0	42.8
1953	21.7	19.2	15.1	2.6	16.0	2.1	7.6	.8	.6	1.1	5.6	8.9	101.3
1954	10.1	6.4	7.8	4.9	3.5	5.8	.4	2.4	.7	4.0	2.7	2.9	51.6
1955	10.6	8.8	2.8	10.8	2.6	3.5	3.2	4.4	2.4	3.6	4.5	5.9	63.1
1956	7.3	.8	7.1	2.0	4.2	2.3	4.1	7.7	.3	1.2	7.3	3.7	48.0
1957	3.0	7.7	2.0	2.3	2.1	3.3	.6	1.2	.1	1.5	5.1	16.1	45.0
1958	13.4	15.7	4.8	8.7	3.1	2.7	2.8	5.1	4.6	10.3	8.2	8.0	87.4
1959	6.0	7.0	2.4	1.3	3.6	.3	8.6	2.5	.3	6.2	1.8	9.3	49.3
Mean	9.7	7.3	6.3	5.7	5.6	3.8	4.3	3.5	2.0	3.3	5.4	9.4	66.4
Percent of mean annual	14.6	11.0	9.4	8.6	8.6	5.7	6.5	5.3	3.0	5.0	8.1	14.2	100.0

TABLE 4.—*Monthly and annual precipitation, in inches, Homer FAA, Alaska*

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Year
1940	3.4	1.7	8.0	3.8	0.2	2.2	1.2	0.7	0.3	2.1	3.2	4.6	31.4
1941	7.8	2.3	3.8	2.8	5.3	3.9	2.8	1.5	3.4	1.7	1.4	1.6	38.3
1942	1.4	1.1	4.4	1.3	2.4	2.1	2.4	.7	1.0	1.6	2.7	2.9	24.0
1943	2.3	.2	.9	1.1	1.7	.5	.9	1.3	.7	1.6	2.3	3.7	17.2
1944	2.5	7.3	5.0	3.8	1.1	1.5	.2	1.9	1.5	3.0	3.6	2.2	33.6
1945	3.3	4.0	1.8	4.9	4.0	3.8	.3	.4	.4	1.4	5.3	3.9	33.5
1946	6.3	2.9	1.3	3.3	2.3	1.3	1.2	2.0	.4	1.4	2.0	2.6	27.0
1947	6.8	1.4	1.3	.9	1.2	1.5	.7	.8	.3	2.1	2.2	3.6	22.8
1948	3.8	2.8	2.8	2.4	.6	.5	0	.7	.6	3.8	2.4	2.4	22.8
1949	3.5	1.0	.5	3.9	.4	2.3	.9	.5	2.2	1.4	1.3	3.5	21.4
1950	2.6	3.1	2.2	.7	.2	1.1	2.8	.5	1.4	1.0	1.3	2.6	19.5
1951	2.4	.1	1.4	1.5	.9	.4	1.4	.2	2.1	.7	2.1	3.0	16.2
1952	1.4	1.1	.3	.7	.8	.3	.6	.3	.5	1.7	2.0	2.2	11.9
1953	4.6	8.6	5.7	1.0	3.6	.2	1.5	2.0	.7	.2	4.8	2.4	35.3
1954	3.8	2.7	2.2	1.1	.8	1.9	0	.4	.3	1.9	4.1	1.5	20.7
1955	4.6	2.4	1.3	2.1	1.0	.4	1.1	.4	2.8	.9	4.0	1.9	22.9
1956	3.7	.1	2.6	1.0	.9	.7	1.5	1.2	.5	1.4	2.2	1.8	17.6
1957	1.4	3.0	.2	.9	.8	.4	.8	.4	.1	2.3	3.0	4.3	17.6
1958	3.6	6.0	2.4	3.7	.5	1.7	.9	1.1	1.1	2.5	2.9	2.4	28.8
1959	2.1	4.7	1.1	.8	1.8	.6	3.5	.6	.1	3.3	1.6	1.7	21.9
Mean	3.6	2.8	2.5	2.1	1.5	1.4	1.2	.9	1.0	1.8	2.7	2.7	24.2
Percent of mean annual	14.8	11.6	10.3	8.7	6.2	5.8	5.0	3.7	4.1	7.4	11.2	11.2	100.0

TABLE 5.—*Average monthly and annual temperatures, degrees Fahrenheit Homer FAA, Alaska*

Water year[1]	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Year
1940	34.8	27.8	27.6	31.1	33.0	29.5	41.6	44.4	49.3	53.8	53.8	48.4	39.6
1941	39.2	31.4	30.8	23.9	33.2	35.4	40.2	43.1	50.6	53.0	54.4	47.3	40.2
1942	38.9	25.9	27.6	33.3	36.4	29.7	38.6	47.7	50.9	53.9	54.7	52.4	40.8
1943	41.5	26.4	11.2	16.1	26.2	29.8	36.4	42.2	49.4	52.4	51.4	45.6	35.7
1944	40.1	33.4	29.9	25.3	34.4	28.4	34.8	44.4	50.9	54.2	55.5	47.4	39.9
1945	39.6	30.8	27.9	33.2	29.9	27.6	33.1	41.6	49.8	53.5	53.0	46.4	38.9
1946	37.6	20.8	22.9	24.7	24.2	21.1	33.6	42.0	47.4	51.7	50.8	46.0	35.2
1947	38.8	26.2	12.9	9.4	26.4	31.1	36.2	42.3	47.4	51.7	50.8	45.8	34.9
1948	37.4	32.8	28.2	25.1	21.0	25.2	31.9	42.2	48.1	50.4	49.4	44.0	36.3
1949	38.2	21.5	17.7	20.1	16.0	32.0	32.0	39.2	46.9	50.4	51.4	47.6	34.4
1950	37.5	32.5	17.9	20.1	16.0	32.8	34.9	40.2	47.2	50.6	52.8	47.2	35.8
1951	36.2	20.7	20.4	16.4	23.1	18.5	36.4	41.7	48.2	52.2	52.8	47.5	34.5
1952	35.3	30.1	21.3	14.4	26.5	27.0	33.2	38.6	45.3	52.0	52.5	46.8	35.2
1953	39.9	35.2	25.6	19.0	26.4	26.5	37.1	42.1	52.2	53.9	53.3	47.2	38.2
1954	35.8	29.2	27.5	18.3	14.1	26.5	32.7	43.6	49.2	52.8	52.4	48.5	35.9
1955	41.1	32.9	14.6	27.8	23.5	29.3	32.0	41.5	46.5	51.5	51.6	45.2	36.6
1956	35.0	22.3	15.0	14.1	17.4	22.8	34.1	41.4	46.7	51.5	51.5	44.9	33.1
1957	32.3	23.6	14.7	24.0	20.5	32.5	36.7	43.1	50.6	52.9	53 6	48.6	36.1
1958	41.4	37.5	18.7	26.[illegible]	29.1	33.2	38.4	44.4	50.5	53.6	42.7	45.4	39.3
1959	34.0	26.7	24.9	19.6	29.1	17.7	33.8	42.9	49.5	52.0	52.4	46.5	35.8
Mean	37.7	28.4	21.9	22.1	25.3	27.8	35.4	42.4	48.8	52.4	52.5	47.0	36.8

[1] Year ending Sept. 30.

GLACIERS AND SNOW ACCUMULATIONS

As interpreted from aerial photographs taken in 1950, glaciers in the Bradley Lake basin and in the nearby areas that can be made tributary to Bradley Lake cover about a third of the entire area. The distribution is as follows:

Drainage area	Area, square miles		Glacier area percent of total
	Total	Glacier	
Bradley Lake basin	54. 0	16. 9	31. 2
Upper Nuka R. basin	4. 1	3. 2	78. 0
Tributary to Bradley R.	10. 2	2. 8	27. 4
Total	68. 3	22. 9	33. 6

The largest is the Kachemak Glacier, the main body of which is below an altitude of about 3,600 feet, starting on a pass 10 miles southeast of Bradley Lake. This and a glacier which drains both into Bradley Lake and the Nuka River extend as valley glaciers down to altitudes of about 1,500 feet. The others are cirque glaciers generally above altitudes of 3,000 feet.

The substantial extent of the glaciers indicates that the snow packs must be relatively great, so that in the wetter and colder years it is to be expected that a substantial amount of water would be stored as transient snow to appear as runoff in subsequent drier or warmer years.

FACTORS THAT WOULD AFFECT THE DESIGN AND OPERATION OF POWER PLANTS

SEDIMENTATION

During the period of the surveys, June to August 1955, it was observed that Bradley River below the lake, and the main tributaries upstream, carried a considerable amount of dark sediment in suspension. This material evidently is largely the result of glacial erosion. The delta at the upper end of the lake and the valley bottom just upstream is composed of sand and mud. The material farther upstream at the glaciers includes gravel and boulders, grading into the finer materials downstream. It was also observed that some silt or rock flour was held in suspension in Bradley Lake. This, no doubt, decreases during periods of low inflow and it seems quite probable that the lake water will be fairly clear during the winter months.

If a reservoir were created at Bradley Lake, sediment undoubtedly would be deposited at the upper end during periods of high streamflow, and there probably would be some gradual impairment of the usable capacity from the outset. There is no basis for estimating the rate of sedimentation, since it would depend on the average sediment load of streams entering the reservoir, the amount that would accumulate in the active zone of the reservoir, and the amount that would be carried through or deposited in deeper parts of the reservoir below the

level of drawdown during filling of the reservoir in spring months. Allowance for sedimentation may depend on a somewhat arbitrary provision of extra storage capacity at the outset, as a safety factor, or on a provision for enlargement of the reservoir at some future time when the rate of sedimentation can be estimated from reservoir surveys.

If storage capacity were created at the lake by drawdown through a tunnel outlet or by combined drawdown and damming instead of damming alone this would introduce another kind of sedimentation (written communication, K. S. Soward, 1956). With reservoir levels below the natural lake surface, inflow at the head of the lake would erode the delta and start a new cycle of erosion in the channels upstream. A large amount of debris thus might be moved into the reservoir in a relatively short time. It seems doubtful, however, that such movement of material would seriously affect the useful life of the reservoir. To some extent a balancing effect is introduced, as the space formerly occupied by material carried into the lake at low stages would be available for storage at higher stages.

Fine materials carried through the reservoir and power plant might cause some wear or impairment of gates, waterways and turbines, but the trouble probably would be relatively minor.

ICE EFFECTS

Winter temperatures at Bradley Lake, and along waterway and transmission routes probably are lower than at Homer, but may not be as low as at Kasilof where minimums of −40°F to −50°F have been observed at times. However, structures such as pipelines and transmission lines would have to be designed for a fairly severe winter climate.

Ice would form on the surface of the reservoir during winter months, which would be the period of drawdown. Spill probably would not occur except in late summer of the wettest years after melting of the ice, if storage for a high degree of regulation is provided. Control of ice probably would have to be considered for protection of the dam and intake structure. At the time of the spring break-up large quantities of the ice accumulate at the downstream end of the lake because of the prevailing downstream winds. Such accumulation was observed early in July 1955.

WATER SUPPLY

TRIBUTARY TO BRADLEY LAKE

Continuous records of streamflow have been collected at the outlet of Bradley Lake since October 1, 1957. The figures of monthly runoff had about the same relationship to those of the Kasilof River during each of the two water years ending September 30, 1958 and

September 30, 1959. The relationship varies greatly in different seasons of the year because of widely different conditions for natural storage in the two basins but may be fairly stable from year to year. The Bradley River runoff in percentage of the Kasilof River runoff for the two years was as follows:

Water year	Oct..	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
1958	11.3	13.5	6.0	6.8	5.8	5.7	12.5	59.6	61.4	20.4	13.5	5.7
1959	7.9	7.2	7.1	8.0	8.2	7.8	6.9	43.1	52.3	18.3	12.3	5.4
Mean	9.6	10.4	6.6	7.4	7.0	6.8	9.7	51.4	56.8	19.4	12.9	5.6

Continuous streamflow records have been collected on the Kasilof River since July 1949. Estimates of the monthly runoff of the Bradley River were computed from the recorded monthly runoff of the Kasilof River by using the foregoing relationship, and are shown with the two years of record in table 6. As shown in table 4 the mean annual runoff for the period 1950–59 was 242 thousand acre-feet, which is equivalent to a mean flow of 334 cfs. The runoff record for the Kasilof River near Kasilof is listed in rounded figures in table 7.

TABLE 6.—*Monthly and annual runoff, in thousands of acre-feet, Bradley River near Homer, Alaska*

[Estimated October 1949 to September 1957; provisional records rounded to nearest thousand, October 1957 to September 1959]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Annual
1950	24	13	4	3	2	2	4	21	40	45	52	23	233
1951	22	10	4	2	1	1	3	22	46	54	57	28	250
1952	26	12	5	4	2	2	3	18	28	35	53	17	205
1953	25	17	8	5	3	3	4	24	61	58	62	23	293
1954	22	11	4	2	1	1	3	22	47	42	51	20	226
1955	22	13	4	3	2	2	3	18	28	32	43	19	189
1956	17	7	2	1	1	1	3	24	34	29	50	22	191
1957	20	9	3	2	2	2	2	20	60	58	52	36	266
1958	41	30	7	5	2	2	4	24	66	69	79	21	350
1959	14	6	4	2	1	1	2	19	49	48	48	18	212
Mean	23	13	4	3	2	2	3	21	46	47	55	23	242

The total estimated and recorded runoff of the Bradley River for the 10-year period was 2,420 thousand acre-feet, or 840 inches. This is approximately 14.0 percent of that of the Kasilof River for the corresponding period, although the Bradley River drainage area is only 7.3 percent of that of the Kasilof River. Half of the Kasilof River basin lies below an altitude of a thousand feet in a rain shadow northwest of the Kenai Mountains. The bulk of the runoff must come from the upper half of the basin which extends up to altitudes above 6,000 feet, and about half of which is covered by glaciers. It

TABLE 7.—*Monthly and annual runoff, in thousands of acre-feet, Kasilof River near Kasilof, Alaska*

[From published records, 1950–57; provisional records, 1958–59, rounded to thousands]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Annual
1950	255	125	62	46	31	33	39	41	70	233	400	405	1,740
1951	230	98	55	33	20	18	27	43	81	281	445	484	1,815
1952	267	120	76	51	33	28	34	36	49	178	407	296	1,575
1953	260	160	116	67	49	49	40	47	108	301	480	384	2,061
1954	224	104	55	34	21	17	26	43	83	216	394	337	1,554
1955	225	122	50	37	30	30	30	34	50	165	331	331	1,435
1956	174	69	22	18	16	17	27	46	60	151	389	386	1,375
1957	204	86	45	28	22	26	25	38	106	298	407	624	1,909
1958	362	220	113	70	40	35	35	40	108	339	582	369	2,313
1959	182	85	52	25	17	18	29	44	93	265	389	339	1,538
Mean	238	119	65	41	28	27	31	41	81	243	422	396	1,732

is to be expected from the proportionately larger area of glaciers in the productive portion of the basin that snow carryover would have a substantially greater equalizing effect in the Kasilof basin than in the Bradley basin. The annual figures of runoff listed in tables 4 and 5 verify this expectation. The standard deviation of the Kasilof River runoff is 16 percent of the mean compared with 20 percent for the Bradley River; and the ranges are 54 percent and 64 percent respectively.

The Bradley Lake basin is similar to the Snow River basin in altitude, glacier cover, and location with respect to the Gulf of Alaska. The Snow River basin is in the Kenai Mountains, about 75 miles northeast of Bradley Lake, and drains into the upper end of Kenai Lake. Estimates of its monthly runoff have been made by deducting from the recorded runoff of the Kenai River the recorded runoff of tributaries intermediate to the Snow River plus estimates for ungaged areas. The runoff, 1950–59, thus estimated was 816 inches, or not significantly different from the 840 inches estimated for the Bradley River. The standard deviation of the annual figures of runoff, 1950–59, for both basins also is approximately the same, 20 percent of the mean.

SUPPLEMENTAL WATER SUPPLY

The foregoing discussion has considered only the runoff from the area tributary to Bradley Lake. The flow from Bradley Lake could be appreciably increased by the diversion of the runoff from three adjacent basins into the lake.

DIVERSION FROM UPPER NUKA RIVER

The Nuka River originates in a large glacier about 5 miles southeast of Bradley Lake, and flows southward into an arm of the Gulf of

Alaska. Some drainage from the glacier also flows northward into the Bradley Lake basin. The glacier extends from its highest point at a ridge 4 miles to the southwest and terminates at the edge of a wide valley, at the place where there is a low divide between the Nuka River and Bradley Lake basins. The topography in this region is shown with a 10-foot contour interval on the published map of Bradley River and Bradley Lake. Diversion of all of the glacier drainage into Bradley Lake basin could be accomplished by construction of a short dike across the Nuka River channel, at a channel altitude of about 1,285 feet, and by excavation of a channel for a distance of about 2,000 feet to the north, largely or entirely through deposits of glacial debris. The highest ground along this route is just a little above an altitude of 1,300 feet. Construction equipment probably would have to be conveyed by barge from the lower to the upper end of Bradley Lake.

Maintenance of the diversion works probably would not be difficult under normal conditions. Intense, warm rainfall on the glacier conceivably might scour quantities of ice into the channel and cause temporary damming and damage, but this possibility seems unlikely. The surface of the glacier is fairly clean, so that abnormal quantities of debris are not to be expected. It appears that the only possibility for major trouble might be an advance of the glacier across the valley, such as might occur after a substantial and prolonged change in climatic conditions.

During the river survey, at about the end of August 1955, the flow of Nuka River at this place was estimated as several hundred cubic feet per second, and it appeared to be several times greater than the amount that drains toward Bradley Lake from the same glacier. It seems likely, therefore, that much of the runoff from the entire glacier region flows in channels under the glacier into the Nuka River, instead of dividing in accordance with the surface boundaries of the drainage basins. If that is the case, the increase in water supply by diversion of the upper Nuka River into Bradley Lake would be proportionately greater than the increase in the apparent drainage area would indicate, particularly since the glacier extends back to one of the highest and probably one of the wettest parts of the region.

DIVERSION FROM BRADLEY RIVER TRIBUTARY

Water from a tributary of the Bradley River could be diverted from that stream at a point about a mile north of Bradley Lake, into the nearby drainage area of another stream which flows directly into Bradley Lake. This diversion would tap a basin of 10.2 square miles, and increase the water supply perhaps 20 percent over that directly

tributary to the lake. The topography of the diversion area is shown in detail on the published map of Bradley River and Bradley Lake.

A diversion from the stream could be made at an altitude of about 2,180 or 2,190 feet, by means of a conduit or an excavated channel along the hillside to a point about 500 feet to the southwest. A small diversion dam at a channel altitude of about 2,180 feet would be required. A dike or training wall also would be required at a point 800 feet southwest of the creek, on a divide between the creek and lake basins.

It is estimated that the maximum discharge at the diversion point generally would be in the order of magnitude of 500 cfs or less, but that extreme discharges of several times that amount might occur. This possibility should be taken into account in the design of the diversion works. At the time of the survey, July 25, 1955, the flow was estimated to be at least 200 cfs, and it was noted that there was a fairly heavy load of dark sediment, similar to that of the Nuka River.

One of the major costs of construction might be for transport of equipment and materials to the site. The hillside north of Bradley Lake extends up to an altitude of 1,900 feet on a slope of about 1 in 2.

DIVERSION FROM BATTLE CREEK

Some water from the headwaters of Battle Creek in a region extending 2 miles south of the outlet of Bradley Lake could be diverted northward to the lake. Several diversion dams and conduits, and a short tunnel through the divide just south of Bradley Lake outlet, would be required for this purpose. The drainage area that could be tapped in this way is roughly 10 square miles, including 3 square miles of glacier area. The amount of required construction can be judged only roughly from the topography shown on the quadrangle maps, but the scheme appears to be relatively complicated, and perhaps of marginal feasibility. Potential water supplies from this source were not considered in the power estimates of this report.

The diversions from the upper Nuka River and the Bradley Lake tributary would increase the drainage area tributary to the lake from 54 to 68 square miles or an increase of 26 percent. Assuming that this additional area would have the same runoff characteristics as the area directly tributary to the lake, the mean annual discharge would be increased from 334 cfs to 421 cfs.

STREAM REGULATION

In the two years for which runoff records are available the runoff of the Bradley River was largely concentrated in the six-month period, May to October, about 90 percent of the total occurring then. This seasonal distribution is closely similar to that found in a number of

basins in Alaska at similar altitude for which records are available.

Since a demand for industrial or municipal power would be fairly uniform, the power possibilities of the Bradley Lake site were estimated only on the basis of regulated flow. It was judged from characteristics of the reservoir site that it would be impracticable to provide enough storage capacity for substantially complete control of the runoff, and that the maximum to be considered probably would be for control of 90 percent of the runoff during the past 40 years. This would correspond approximately to complete control during the relatively dry period, 1950–59.

The amount of storage capacity required during this period was computed by first estimating the monthly runoff of the Bradley River from the recorded runoff of the Kasilof River, 1950–57, and the average relationships that existed between the two streams during the period of overlapping records, 1958 and 1959. Reservoir operation schedules then were computed for the 10 years of estimates and records. A storage capacity equal to about 93 percent of the estimated mean annual runoff of the past 40 years would have been required for 90 percent utilization on a schedule of uniform monthly releases. This is about twice as much storage as for a corresponding degree of control in southeastern Alaska. The difference evidently is due to the greater variability of annual precipitation and runoff on the Kenai Peninsula. During the period 1950–59 annual indices of wetness for the Seward-Homer area had a range of −43 to +49 percent and a standard deviation of 27 percent, whereas the annual precipitation recorded at Juneau in southeastern Alaska had a range of −23 to +21 percent and a standard deviation of only 15 percent during the same period.

RESERVOIR SITES

Bradley Lake provides a favorable opportunity for the development of the required storage for adequate stream regulation. It is 3½ miles from tidewater at the head of Kachemak Bay, at an altitude of 1,090 feet, and is the only place in the region that is suitable for development of a substantial amount of storage capacity. The lake is about 3 miles long in an east-west direction, and has a surface area of 1,566 acres. Except at the upper end, it is bounded by steep, rocky slopes which generally extend underwater for about 200 feet. The valley for a mile and a half upstream from the lake has a comparatively flat gradient and is filled with glacial deposits, terminating in a delta about a mile wide. Tributary valleys from Kachemak Glacier to the east, and from the divide between the Bradley Lake and Nuka River to the south, join the wide valley about a mile above the lake. A considerable part of the potential storage capacity of the

site is in the wide valleys upstream from the lake, since they have a relatively flat transverse profile.

Bradley Lake is drained by the Bradley River, which flows northward three miles through a very rugged canyon down to an alittude near tidewater in the flat area above the head of Kachemak Bay. The channel at the lake outlet is about 150 feet wide, divided by a small rock island near the center.

The sides of the Bradley River canyon near the lake outlet are topographically suitable as abutments for a dam to an altitude of about 1,200 feet, or 110 feet above the lake surface. However, there is a saddle northeast of the right abutment, at an altitude a little above 1,150 feet, and another saddle west of the left abutment, at an altitude a little above 1,170 feet. Dams would be required in these saddles for reservoir flow lines above their controlling altitudes.

At a lake stage of 1,090 feet, the river drops only a foot in a distance of 300 feet below the island, and the canyon widens abruptly about 400 feet downstream from the island.

Soward [2] found that rock in the foundation and both abutments is mainly of massive graywacke, and is of good quality for a dam site. In the saddle area, northeast of the right abutment, this rock may be at considerable depth under a cover of gravel and talus.

The topography near the lake outlet also appears to be reasonably favorable for development of storage capacity by tapping the lake with a tunnel outlet and drawing it down to a depth as low as 965 feet or 125 feet below its natural surface. A study of the underwater contours shows that 965 feet is about the lowest altitude to which the lake can be drawn down. A considerable amount of capacity thus could be obtained by damming the lake outlet, by drawdown, or by a combination of the two methods. The potential capacities and corresponding surface areas are listed in table 8.

TABLE 8.—*Area and capacity of Bradley Lake reservoir site*

Altitude (feet)	Area (acres)	Capacity (acre-feet)		Altitude (feet)	Area (acres)	Capacity (acre-feet)	
		Below lake surface	Above lake surface			Below lake surface	Above lake surface
960	1, 050	169, 000	--------	1, 100	1, 990	--------	17, 800
980	1, 140	147, 000	--------	1, 120	2, 460	--------	62, 300
1, 000	1, 220	123, 000	--------	1, 140	2, 950	--------	116, 000
1, 020	1, 290	97, 900	--------	1, 160	3, 340	--------	179, 000
1, 040	1, 340	71, 600	--------	1, 180	3, 660	--------	249, 000
1, 060	1, 410	44, 100	--------	1, 200	4, 010	--------	326, 000
1, 080	1, 480	15, 200	--------	1, 220	4, 300	--------	409, 000
1, 090 [1]	1, 570	--------	0	1, 240	4, 600	--------	498, 000

[1] Lake surface

[2] Report in preparation, 1961.

The power possibilities of Bradley Lake were considered for two degrees of regulation that would provide for about 90 percent and 80 percent utilization, respectively, of the long-term mean discharge. They were estimated on the assumption that runoff from 64 square miles of drainage area or a mean discharge of 461 cfs could be available for regulation. For a controlled flow of 421 cfs, equal to the estimated mean for the period, 1950–59, or roughly 90 percent of the long-term mean, 300,000 acre-feet of capacity would be required. This could be provided by a dam at the lake outlet raising the lake level 104 feet to an altitude of 1,194 feet. A storage capacity of 171,000 acre-feet would have been required for 80 percent utilization and this could be provided by raising the lake surface 68 feet to an altitude of 1,194 feet.

The width across the river canyon at an altitude of 1,194 feet is 375 feet and at an altitude of 1,150 feet it is 275 feet. Storage to the higher level would necessitate construction of two saddle dams; one about 700 feet west of the left abutment, and the other 300 feet northeast of the right abutment. Soward [3] estimated that rock under the first saddle probably is at an altitude of 1,165 to 1,170 feet beneath 10 to 15 feet of fill; and that it may be at an altitude of 1,100 to 1,120 feet under 30 to 50 feet of fill at the saddle near the right abutment. The widths of the saddles at an altitude of 1,194 feet are 80 feet and 240 feet respectively. An auxiliary dam or core wall probably would be required at the right-bank saddle to seal this for storage to an altitude of 1,150 feet.

Development of storage by damming alone would have the considerable advantage of leaving the entire underwater capacity of the lake in dead storage to serve as a sediment catchment, and also the advantage that it would not result in a change in stream regimen upstream from the reservoir. It would also provide a greater head for power development.

The underwater topography is reasonably favorable for development of storage by a tunnel outlet for drawdown to an altitude of about 960 feet. This would necessitate tapping the lake at a point about 700 feet southeastward from the lake outlet. The nature of the lake bottom could not be judged closely from soundings at the time of the 1955 survey, and the water was too murky for visual examination. The possibility of development by drawdown alone, or by combined drawdown and damming may be seriously considered if further investigations show that conditions for dam construction are unfavorable.

The maximum discharge recorded at Bradley Lake during the two years of record was 3,470 cfs, August 13, 1958. This occurred at a time of general rainfall on the Kenai Peninsula and evidently was

[3] Report in preparation, 1961.

due both to rain runoff and snow melt. The discharge was equivalent to 64 cfs per square mile—not an unusually high rate. If a dam were constructed at Bradley Lake a spillway capacity at least five times that amount probably would be considered.

POWER DEVELOPMENT SCHEMES

The development of power from Bradley Lake can be accomplished by either of two general plans, both of which would utilize the lake as a reservoir to regulate the flow. In one plan the entire head from Bradley Lake to tidewater would be developed in one stage with the powerhouse at one of several possible locations near the shore of Kachemak Bay. In the other plan the head would be developed in two stages with one powerhouse at mile 8.2 on the river and the second powerhouse near mile 5.0 on the river. Water would be conveyed from the lake to the powerhouse at mile 8.2. The flow from this powerhouse would be diverted by a dam on the river at mile 6.7 and conveyed to a powerhouse near mile 5.0.

In the mile and a half reach downstream from the above mentioned powerhouse site, mile 8.2 to 6.7, the river drops only about 75 feet, or to an altitute of about 525 feet; and from mile 6.7 to 5.5 it drops 500 feet or to an altitude of 25 feet.

SINGLE-STAGE DEVELOPMENT

Water could be conveyed from the Bradley Lake reservoir almost due west about 3.7 miles, by way of tunnels, pipes and a penstock to a powerhouse on Battle Creek near Kachemak Bay. This is a route suggested in the reconnaissance report of the Corps of Engineers (called Route "A"). Alternative pipeline and tunnel routes to the same powerhouse site and to a possible site on lower Bradley River also were discussed. Conveyance Route "B" of that report include a 4,000-foot tunnel leading to the southwest from the reservoir into the upper Battle Creek drainage. Water would be conveyed thence by pipe and penstock roughly 4 miles along the hillside north and east of Battle Creek, to the powerhouse site on lower Battle Creek. (See fig. 5)

Conditions for tunnel construction along the general routes westward or southwestward from Bradley Lake were described by Soward.[4] Conditions along alternative routes to the northwest of Bradley Lake, leading to a powerhouse site on lower Bradley River also were discussed. Routes to the northwest could be followed by various combinations of tunnels and pipes. One possibility would involve about 2.2 miles of tunnel and 1.0 mile of pipe and penstock, with a total length of 3.2 miles. This is the shortest of several routes under consideration. It was found that a tunnel along this

[4] Report in preparation, 1961.

route probably would cross one large fault, and that a tunnel along Route "A" of the Corps of Engineers report westward to the Battle Creek site probably would cross two large faults, and encounter massive graywacke, chert beds, and interbedded graywacke and chert beds. Routes to the northwest would be in similar rock formations. A tunnel southwestward from the lake, as in Route "B" of the Corps of Engineers report, would be in massive graywacke for its entire length.

The lengths of the waterways to the west or northwest were measured from the damsite at the outlet of Bradley Lake. If storage capacity were developed there by drawdown of the lake surface, or by combined damming and drawdown, it would be necessary to locate the tunnel intake in the lake at a point as much as 700 feet southeastward from the lake outlet. However, drawdown of at least 100 feet could be obtained with underwater diversion only 200 feet from the left bank at the lake outlet.

TWO-STAGE DEVELOPMENT

Bradley River drops to an altitude of 600 feet at a point about 1.2 miles northwest of the lake outlet. Routes to this site have not been investigated closely in the field, but it may be found possible to convey water by way of a tunnel about 1.3 miles along the west side of the river, and thence by a short penstock to the powerhouse on the left bank of the river. The tunnel route would cross a sharp ravine, considered by Soward [5] to be in a probable fault zone. The tunnel could be terminated at this point, about a mile from the lake, and a penstock placed down the ravine to the powerhouse, but conditions for anchoring the penstock and protecting it from possible snow or rock slides may be unfavorable. Investigation may show that a somewhat longer tunnel route along the east side of the river would be preferable. In either event the powerhouse would be located at mile 8.2 on the Bradley River. In view of the extremely rugged terrain it might be advisable to place the generating equipment in a cave excavated in the canyon wall rather than in the more conventional powerhouse in the open.

The topography at mile 6.7 and immediate vicinity is suitable for building a diversion dam to a crest altitude of at least 580 feet, and at this level the width of the canyon is 150 feet. The altitude of the river surface at this site is about 525 feet. A possible by-pass area in a saddle 2,000 feet northeast of the damsite might be a controlling feature. The saddle is at an altitude of about 600 feet, and foundation conditions for an auxiliary dam are unfavorable. For power estimates of this report it is assumed that only a low dam on

[5] Report in preparation, 1961.

the Bradley River at mile 6.7, with a crest at an altitude of about 550 feet may be considered.

Water from the diversion dam could be conveyed to the northwest about 3,600 feet by tunnel, and thence to a powerhouse at the point where Bradley River emerges from its canyon; a distance of about 2,400 feet by penstock. This site is near mile 5.0 on the river, where the water surface is at an altitude of about 10 feet.

The conditions at the damsite, in the saddle area, and along the conduit route were discussed by Soward.[6]

Access to the upper powerhouse site would be difficult because of the rugged terrain, and it might be necessary to use a tramway or cableway from high ground for access down into the canyon. Access to the diversion damsite at mile 6.7 also would be difficult, although a road probably could be constructed to a point near the river about 2,000 feet downstream.

The two foregoing sites are described because they afford a means of developing the bulk of the potential power of the Bradley Lake site in two stages, of roughly equal size. Although the capital cost and operation costs for two plants probably would be greater than the costs for a single-stage development, it is conceivable that a market for all of the power may not be found in the near future. If a lesser development should prove to be economically feasible, and if a demand should develop for only about half of the total power at the outset, the possibility would be worth considering.

The waterway lengths for each of the two plants are less than half the length required for single-stage development, the overall length of tunnels and penstocks being about 2.5 miles. Storage requirements would be the same, but powerhouse and equipment costs per kilowatt of capacity would be increased because of the smaller installations and lower heads.

POTENTIAL POWER

The potential power, and related data, is shown in table 9 for the single-stage and two-stage plan of development. For the single stage plan two possibilities are shown, one with all of the storage developed above the normal lake level, and one with a combination of dam and drawdown. In the latter case half of the storage would be above and half below the normal lake level.

In the two-stage plan the data shown for stage one is for storage above the normal lake level. If a combination of dam and drawdown were used the head and resulting power would be decreased 8 percent and 5 percent, respectively for 90 percent and 80 percent degrees of regulation. The head and resulting power for stage two would be

[6] Report in preparation, 1961.

the same regardless of which plan is followed for the development of storage.

The power as shown in table 9 was computed from the formula $P=0.068QH$ in which P=power in kilowatts; Q, flow in cubic feet per second; and H, head in feet. This formula assumes an overall efficiency of 80 percent

TABLE 9.—*Estimated potential power and related data for Bradley Lake and Bradley River power sites.*

	Single-stage development—Storage				Two-stage development			
	Dam only		Combination of dam and drawdown [1]		Stage 1 [2]		Stage 2 [3]	
Height of dam (feet)	[4] 104	[4] 68	[4] 61	[4] 40	[4] 104	[4] 68	[5] 25	[5] 25
Storage capacity (acre-feet)	300,000	171,000	300,000	171,000	300,000	171,000	([6])	([6])
Altitude of reservoir (feet):								
Maximum	1,194	1,158	1,151	1,130	1,194	1,158	([6])	([6])
Minimum	1,090	1,090	976	1,029	1,090	1,090	--------	--------
Mean head (feet)	1,131	1,110	1,070	1,070	551	530	540	540
Controlled flow								
Percent of mean	90	80	90	80	90	80	90	80
Flow in cfs	421	375	421	375	421	375	421	375
Continuous power (kw)	32,400	28,300	30,600	27,300	15,800	13,500	15,500	13,800

[1] Storage capacity half above and half below normal lake level
[2] Power plant near mile 8.2 on Bradley River with diversion from lake
[3] Power plant near mile 5.0 on Bradley River with diversion by dam at mile 6.7
[4] Above normal lake surface
[5] Above river surface
[6] Stage 2 would follow stage 1 and would utilize the same storage and flow as stage 1.

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Waterpower Resources in Trask River Basin Oregon

By L. L. YOUNG

With sections on GEOLOGY OF SITES

By R. G. WAYLAND *and* D. L. GASKILL

WATERPOWER RESOURCES OF THE UNITED STATES

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1610-B

A discussion of potential power and of development possibilities including discussions of geology and out-of-basin diversion

UNITED STATES GOVERNMENT PRINTING OFFICE, WASHINGTON : 1963

UNITED STATES DEPARTMENT OF THE INTERIOR

STEWART L. UDALL, *Secretary*

GEOLOGICAL SURVEY

Thomas B. Nolan, *Director*

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington 25, D.C.

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WATERPOWER RESOURCES IN TRASK RIVER BASIN, OREGON

By L. L. Young

ABSTRACT

The Trask River originates in the Oregon Coast Range about 35 miles west of Portland and empties into Tillamook Bay at Tillamook, Oreg. The drainage basin is mountainous and sparsely settled. The runoff is derived principally from rainfall that occurs in a seasonal pattern, which results in 77 percent of the total annual discharge occurring in a 5-month period during the winter and early spring months. The available storage sites are not adequate to regulate completely the high winter flows or store the water for summer use.

The potential power is estimated by considering an illustrative plan including five sites. Powerplants at these sites could have generated 14,000 kilowatts of power continuously during the period 1938–52, or a minimum of 41,000 kilowatts from November 1 to March 31 each year during the period. At average discharge and total head the potential power of these sites would be 34,000 kilowatts.

A pumped-storage addition to one of the sites could produced a peaking potential of 68,000 kilowatts.

The Trask River could be diverted to the Tualatin River basin for municipal purposes and irrigation and still permit some power production. About one-third of the water now passing the gage near Tillamook could be diverted through or pumped over a ridge between the Middle Fork North Fork Trask River and the Tualatin River. The potential power of a diversion by gravity at an altitude of 800 feet is about 3,000 kilowatts continuous. By pumping the water 700 feet into a higher level canal and diverting it over the divide at an altitude of 1,500 feet, a considerable block of peaking power could be made available.

INTRODUCTION

PURPOSE AND SCOPE

The purpose of this report is to evaluate the waterpower potential of the Trask River basin. The basic data for the report are topographic quadrangles covering the basin, a special map of the river; large-scale damsite maps, geologic reports on the damsites, and streamflow records. The above data were all gathered by the Geological Survey. In addition to these, use was made of the precipitation and

evaporation records of the U.S. Weather Bureau and of the isohyetal map prepared by the Corps of Engineers, U.S. Army, for western Oregon.

Up to the present there has been little thought given to the possibilities of developing power on the Trask River. However, many of the more economical projects of the Columbia River basin have now received rather extensive studies and attention will shift to other sites. As the Columbia basin is developed the need for power to firm the system will also become greater. The Trask River could make a modest contribution in this respect. The Tillamook County Public Utilities District has filed an application with the Federal Power Commission (Federal Power Project No. 2274) for a power project on the Trask River and has also considered development of waterpower on the Nehalem River.

Any power development will require adequate provision for fish passage, road relocation, and reimbursement to land owners for damages resulting from inundation. Opposition to power development is lessened by improved methods of getting fish over dams and otherwise protecting them from harm by project works, added recreational benefits afforded by the new lakes, construction of new and better roads in the vicinity of power projects, and more adequate compensation to dislodged property owners.

The potential power of the river is estimated by considering a series of possible developments entirely within the basin, and an alternative plan is discussed in which its water could be diverted to the Tualatin River by gravity or by pumping.

PREVIOUS INVESTIGATIONS

Jones (1924) and Helland (1953), Engineers of the U.S. Geological Survey, have made investigations that included the Trask River.

The engineering firm of Cornell, Howland, Hayes, and Merryfield, of Corvallis, Oreg., recently prepared a report for the Tillamook County Public Utilities District entitled "Report of Reconnaissance Study of Hydroelectric Development of Trask River, June 1959."

MAPS RELATING TO THE AREA

A map of the Trask River was made and published by the U.S. Geological Survey (1955). This map was supplemented by the mapping of four damsites, resulting maps of which are in preparation for publication. Geologic examinations have been made of the following damsites and the results included in this report. The first four were examined by R. G. Wayland and the fifth by D. L. Gaskill.

Site	*Stream*	*Location*
Clear Creek	North Fork	Sec. 24, T. 1 S., R. 7 W.
Keyhole	do	Sec. 27, T. 1 S., R. 7 W.
Bark Shanty	do	Sec. 29, T. 1 S., R. 7 W.
Hollywood	South Fork	Sec. 6, T. 2 S., R. 7 W.
Ginger Peak	Trask River	Sec. 29, T. 1 S., R. 8 W.

The Blaine, Enright, Fairdale, Nehalem, Tillamook, and Timber topographic quadrangles, completed in 1955, scale 1:62,500, cover the entire basin.

GENERAL DESCRIPTION OF THE BASIN

PHYSICAL CHARACTERISTICS

The Trask River drains an area of about 170 square miles of mountainous timberland in northwestern Oregon. Most of the basin is within Tillamook County; minor portions are in Washington and Yamhill Counties. It heads in the Oregon Coast Range about 35 air miles west of Portland in mountains more than 2,000 feet high. Its course is almost west to its mouth at Tillamook on Tillamook Bay. It is one of five rivers entering the bay. The name is for an early settler and leader in Tillamook County, Elbridge Trask. The mouth of the bay is about 5 miles northwest of Tillamook and about 65 air miles west of Portland. The basin in outlined by a dashed line on figure 1.

The Trask River basin is bounded on the north by the Wilson River basin, on the east by the Tualatin and Yamhill River basins, on the south by the Nestucca River basin, and on the southwest by the Tillamook River basin.

The Trask and Wilson Rivers flow in the large Tillamook Valley for about 5 miles. The divide between them rises abruptly there to an altitude of 2,000 feet. On its eastern end this divide is above 3,000 feet. Hembre Ridge is 3,409 feet, an unnamed peak is 3,535 feet, and Saddle Mountain is 3,461 feet in altitude. The Tualatin and Yamhill basins are separated from the Trask by a north-south ridge, having peaks of 3,170, 2,941, 2,371, 3,423, and 2,878 feet in altitude. The lowest pass on this ridge is about 1,550 feet in altitude. The ridge dividing Trask and Nestucca River drainage is almost as high. Its peaks are 2,913, 2,700, 2,500, and 3,012 feet in altitude; the last is that of Grindstone Mountain. The highest peak on the divide between the Trask and Tillamook Rivers is Edwards Butte, 3,168 feet in altitude. Elevations on this ridge decrease rapidly from that point to the Tillamook Valley.

ECONOMICS

Tillamook is the only town within the basin. It had an estimated population of 4,261 (Oregon State Board of Census, 1960) July 1,

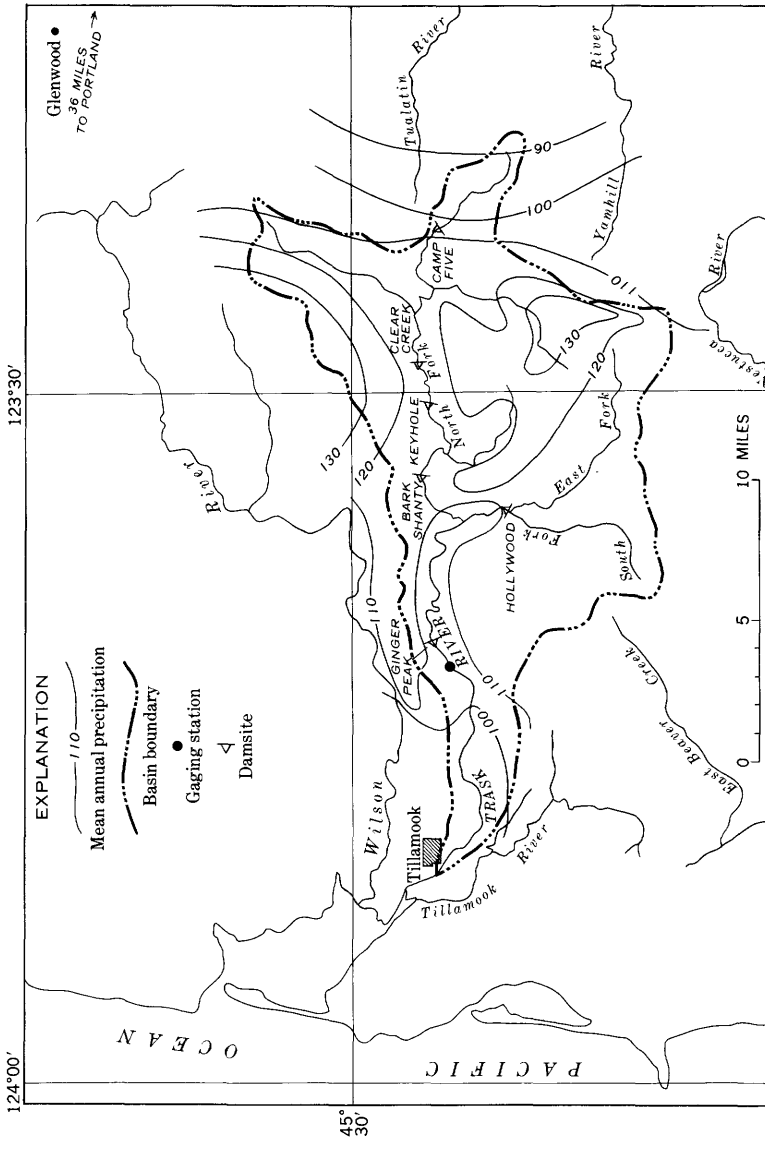

FIGURE 1.—Map of Trask River basin, Oregon, showing provisional isohyets and damsites.

1960, which was an increase of 15.6 percent over the 1950 population. There has been an increase of only 2 percent since 1950 in the population of Tillamook County. The 1960 population was 18,969, whereas in 1950 it was 18,606. Although most of the county is outside the basin, waterpower developed on the Trask River would benefit the whole county.

Captain Robert Gray sailed the *Washington* into Tillamook Bay in 1778 (Orcutt, 1951) 4 years before his discovery of the Columbia. Clark of Lewis and Clark visited the Tillamook coastal areas in 1806 (Orcutt, 1951). The first white settler, Joe Champion, arrived in 1851; and although he stayed only for the one summer season, others came and white settlement has been continuous.

For a great many years the only transportation link with civilization was the sea. The Tillamook Bay bar was exceptionally treacherous, however, and although some ships were built at Tillamook, transportation difficulties retarded economic growth.

The Trask River toll road between Yamhill in the Willamette Valley and Tillamook was established in 1871. This was the first wagon road to cross the Coast Range within Tillamook County. Mail and freight were carried over the road until 1911 when the railroad now owned by Southern Pacific was completed between Portland and Tillamook.

A surfaced highway now follows the river from Tillamook to the Trask guard station at the junction of the North and South Forks. Secondary roads cover the remaining reaches of stream and branch into the mountains to afford access to timber and for fire protection. There is no highway between Portland and the coast within the Trask basin. Oregon Route 6 crosses Tillamook valley and is within the basin for about 4 miles. U.S. Highway 101 along the Oregon coast passes through Tillamook.

The large valley around Tillamook Bay furnishes excellent farm and pasture land, and the early settlers were soon raising cattle and milking cows. The cattle were driven to market, and the dairy products were shipped as best they could be. Cheese became an important product because it is the least perishable of the conventional dairy products. The cheese industry makes a very substantial contribution to the present economy of the area.

The pioneers found the Trask basin covered by a dense forest of cedar, hemlock, fir, and spruce. Active exploitation of these abundant timber resources together with those of other basins brought a great number of people to the area. The period of growth began about 1890 and ended in 1933, the date of the great Tillamook burn. The burn devastated 140 of the 170 square-mile Trask River basin.

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Lumbering has been and probably will continue to be one of the principal industries. However, there is no longer sufficient timber to maintain former rates of exploitation. The burn caused immense tax losses to the county and a large part of the burnt area has been taken over by the State and Federal governments for reseeding. Subsequent burns in 1939 and 1945 which covered virtually the same area, have removed all possibilities of reforestation by natural reseeding. Reseeding by other methods is being done by land owners, State and Federal agencies, and citizen organizations.

Tourists, vacationers, and fishermen come to the area in ever-increasing numbers from year to year, and the basin has good possibilities for additional development of outdoor recreational activities. The river is fished for trout, steelhead, and salmon. If dams are constructed on the river, facilities for fish passage should be included. The new reservoirs would enhance the present recreational values.

WATER SUPPLY

PRECIPITATION

The U.S. Weather Bureau has records of precipitation at Tillamook for 42 years. Stations of the Bureau outside the basin at Seaside, Astoria, and Vernonia maintain precipitation records, which are indices of precipitation for the general area. The Corps of Engineers, U.S. Army, has prepared an isohyetal map of western Oregon that includes the Trask basin. An isohyetal map for the Trask basin was prepared by adapting data from the above sources to conform closely to the topography as shown on 1:62,500 scale quadrangles. The new map was used in estimating runoff by measuring areas between the several isohyets for the various drainage basin units. The approximate isohyets are shown in figure 1. The rainfall averages determined range from a minimum of about 90 inches on the coast and in the extreme eastern part of the basin to more than 130 inches on the higher mountain peaks. Throughout the basin precipitation has the usual coastal distribution, heavy in the fall and winter and lighter in the spring and summer.

RUNOFF

The average discharge of the Trask River from 1931 to 1955 was 982 cfs at a station 6 miles east of Tillamook, where the drainage area is 143 square miles. This is equivalent to 710,900 acre-feet annually.

A detailed analysis was made of the records for water years 1938 through 1952. For that period average discharge was 942 cfs, 40 cfs less than for the entire period of records. The lower yield 15-year period 1938 through 1952 is used in the computations for this report.

The difference between the determined precipitation and runoff depths for the drainage area above the gage was about 26 inches. Runoff amounts were determined for the subareas by subtracting 26 inches from annual precipitation depths determined in the previous section. The loss probably varies throughout the basin, but lack of discharge measurements makes refinement impractical.

Table 1 shows the rainfall and runoff estimates for the river at the damsites studied.

TABLE 1.—*Area and estimated precipitation and runoff at selected sites in Trask River basin*

Damsite, gage or basin	Drainage area (sq mi)	Estimated precipitation (inches)	Estimated average discharge (cfs per sq mi)	Estimated annual runoff [1] (acre-feet)	Percent of runoff at gage
Camp Five [2]	9	98	5. 36	35, 000	5. 1
Clear Creek	53	117	6. 69	257, 000	37. 7
Keyhole	57	117	6. 71	277, 000	40. 6
Bark Shanty	75	118	6. 77	368, 000	54. 0
Hollywood	49	115	6. 56	233, 000	34. 2
Ginger Peak	140	116	6. 63	672, 000	98. 5
Gaging station	143	116	[3] 6. 59	[3] 682, 000	100. 0
Mouth	170	114	6. 48	798, 000	117. 0

[1] Allows a difference of 26 in. between precipitation and runoff.
[2] Possible site for diversion to Tualatin River by gravity.
[3] Average for 1938–52.

A flow duration analysis (fig. 2) for the water years 1938 through 1952 shows Q95,[1] 79 cfs (0.55 cfs per sq mi); Q50, 490 cfs (3.43 cfs per sq mi); and Qmean, 942 cfs (6.59 cfs per sq mi). There is a pronounced seasonal variation in runoff corresponding to the seasonal variation in precipitation. The runoff is greatest during the period November through March and least during the period of July through September. The average monthly discharge, in cubic feet per second and the percentages of the average annual runoff for the Trask River at the gage are shown below.

Month	cubic feet per second	Percent of average annual	Month	cubic feet per second	Percent of average annual
October	489	4. 40	April	865	7. 54
November	1532	13. 36	May	521	4. 69
December	2148	19. 36	June	283	2. 47
January	1640	15. 13	July	160	1. 44
February	2069	16. 98	August	102	. 91
March	1399	12. 60	September	129	1. 12

[1] Discharge in cubic feet per second. Q95=discharge equaled or exceeded 95% of the time.

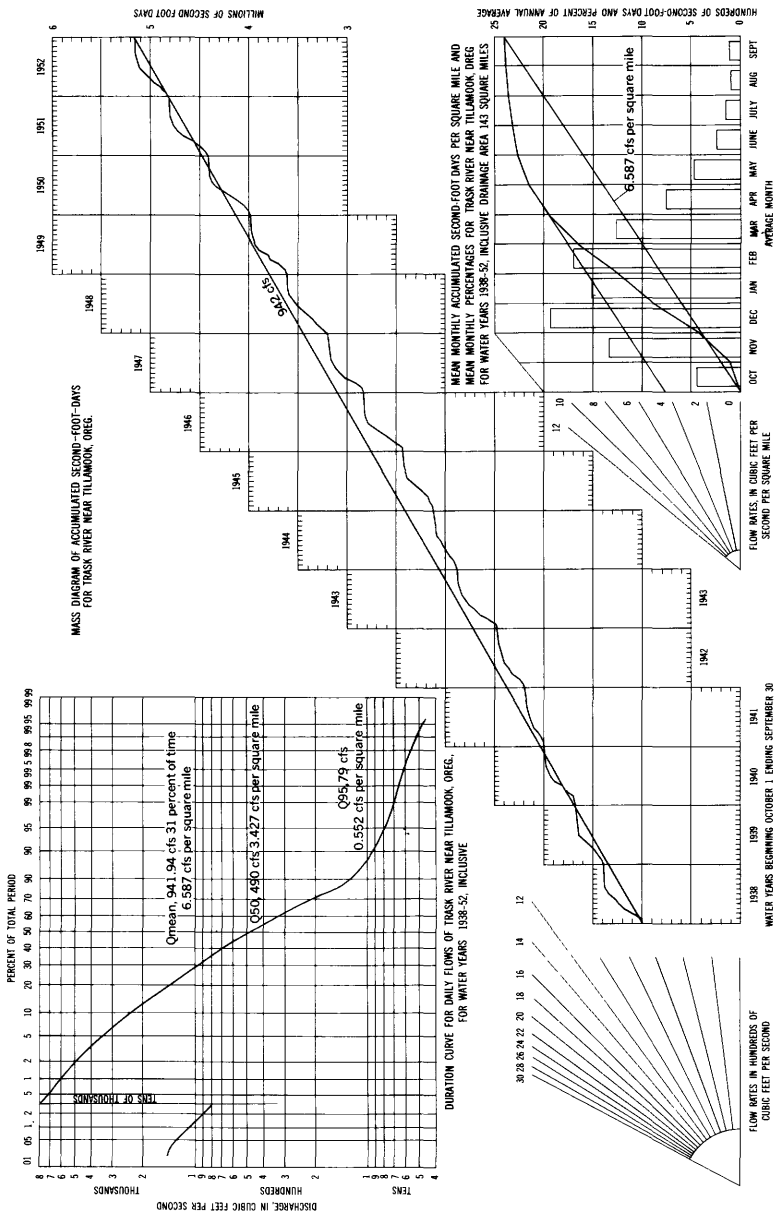

FIGURE 2.—Mass diagram and flow-duration curves, Trask River, Oreg.

The runoff pattern shown by this tabulation and by figure 2, which shows accumulated flows and flow duration, was used throughout the basin.

REGULATION AND STORAGE SITES

The distribution of annual runoff is such that reservoirs are necessary to regulate the flow by reducing the high winter runoff and increasing low summer flows. There are no reservoirs in the basin, but there are several sites topographically suitable for reservoirs. Costs of land acquisition and forest road relocations should be relatively moderate. The necessity for fish protection might tend to limit the development of reservoirs. It should be noted that rather high and massive dams will be required to create reservoirs with only relatively small storage capacities.

To find the storage requirements of the sites, the estimated total water yield at each damsite was distributed over the 15-year period studied by adapting the mass diagram and accumulated second-foot-day table for the gage to the particular site under consideration. In other words, the gaging-station records provide the pattern for distributing the runoff estimates made in the previous section throughout the year.

Estimates of evaporation losses were made from Weather Bureau records of evaporation rates in Oregon. An annual loss of 30 inches, of which 27 inches occurs between April 1 and October 31 and 3 inches between November 1 and March 31, was used.

The estimated monthly evaporation losses for the Trask River basin are as follows:

Month	Evaporation (inches)	Month	Evaporation (inches)
January	0. 3	July	5. 6
February	. 5	August	5. 1
March	. 8	September	3. 8
April	2. 5	October	1. 5
May	4. 0	November	1. 0
June	4. 5	December	. 4

The surface area of the reservoirs when they contained one-half their usable capacities was used to determine the evaporation loss for yearly, winter, and summer periods. An additional allowance equal to 5 cfs plus one-fifth the annual evaporation loss rate was made for leakage and other unmeasurable losses. The total loss was rounded to the nearest 5 cfs. Using this method, all losses are rounded to 10 cfs except those for Ginger Peak, which are 15 cfs for the annual and

summer periods and 10 cfs for the winter months. During reservoir-filling periods or during times when diversions from the basin were being made, a conservation release equal to the flow at the sites 95 percent of the time rounded to the nearest 5 cfs or 50 cfs, whichever is smaller, has been allowed.

Five reservoir sites are considered in this report; three on the North Fork Trask River, one on the South Fork Trask River, and one on the Trask River. Their locations on the river are shown in figures 1 and 3.

GEOLOGY

By R. G. Wayland

All damsites described in this report are in a series of volcanic rocks of Eocene age described by Warren, Norbisrath and Grivetti (1945, 1946) as the Tillamook volcanic series. Recent geologic mapping on a regional scale (Wells and Peck, 1961) has resulted in a tentative differentiation within the Tillamook series in the Trask River area. West of the forks at the Trask Guard Station, the dark-greenish-gray aphanitic to porphyritic basalt flows and flow breccias described here in the contribution by Gaskill on the Ginger Peak damsite would be an older series to be correlated with the Siletz River volcanic series of early Eocene age (Snavely and Baldwin, 1948; Baldwin, 1959). East of the forks, the upper part of the Tillamook series consists of some of the same type of basaltic rocks; but it also includes andesitic flows (or sills?) and interbedded dark-gray tuffaceous shale, siltstone, and thin-bedded sandstone. This upper part of the Tillamook series is now tentatively correlated with the Nestucca Formation (Snavely and Vokes, 1949) of late Eocene age, and may also include equivalents of the Yamhill Formation (Baldwin and others, 1955), of Eocene age.

The petrographic description by Snavely and Baldwin (1948) for the Siletz River volcanic series farther south seems to describe the basaltic flows and flow breccias of the upper Trask River so well that it is abstracted here:

> The predominant rock in the flows is a dark-greenish-gray aphanitic to porphyritic basalt. Rectangular phenocrysts of plagioclase and equant to rounded phenocrysts of augite are visible in hand samples. Vesicular and amygdaloidal basalts are common, with the amygdules composed of radiating zeolites and calcite. Unaltered basalt is rare, and locally chloritization is so advanced that a green-stone has resulted. Typical pillow structure is common through the series, individual pillows averaging 3 feet in diameter. Columnar joints which radiate from the center of the essentially ellipsoidal pillows can be seen in most exposures. Flow breccia is not rare, and is probably the result of autobrecciation by steam explosion which accompanies submarine extrusion. Sporadic pillows are found in many of the flow breccias, testifying to their subaqueous origin.

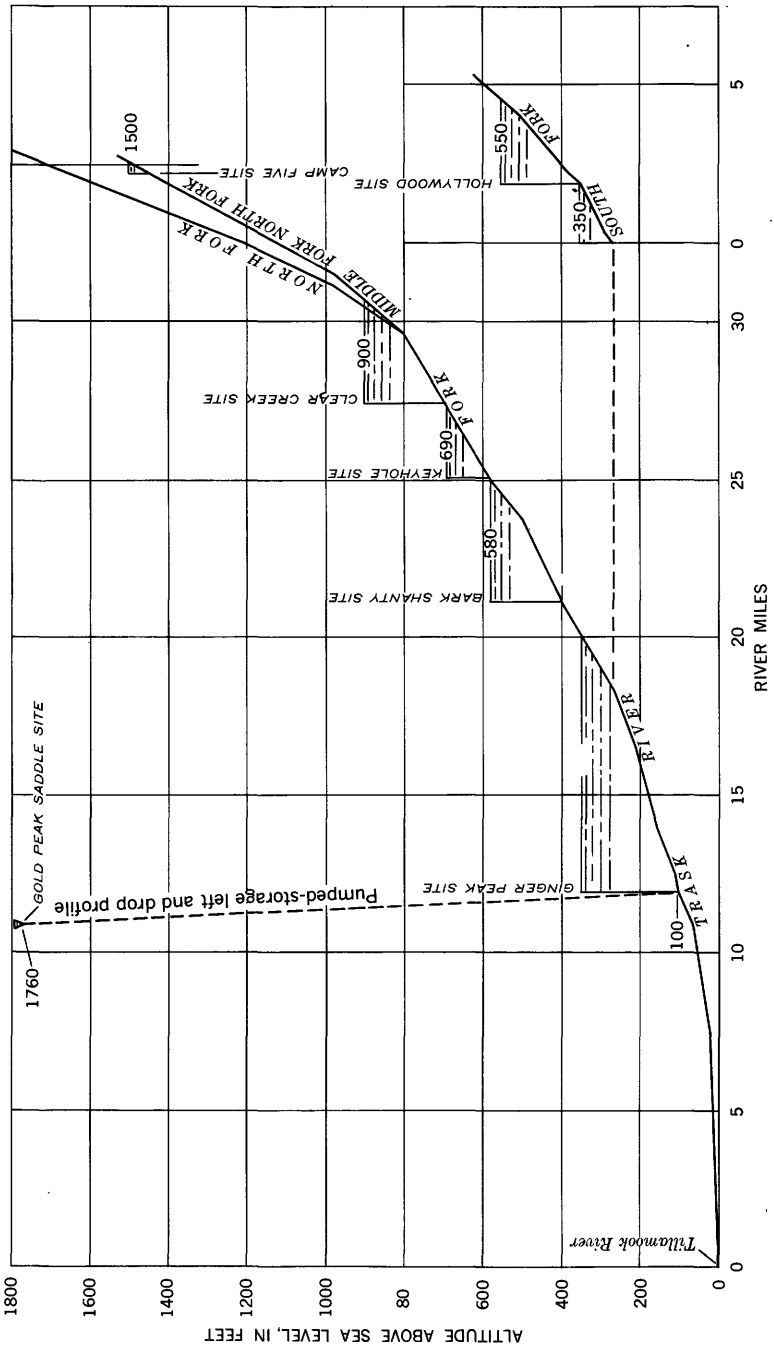

FIGURE 3.—Profile of Trask River, Oreg., showing power and storage sites.

Microscopic study shows that the basalt is holocrystalline to hemicrystalline with porphyritic to glomeroporphyritic and vesicular textures. The groundmass varies from trachytic to intersertal. Phenocrysts of plagioclase (* * * labradorite) and augite are set in a groundmass of plagioclase laths * * *, granules of augite and magnetite, and volcanic glass. The phenocrysts and laths of plagioclase constitute about 50 percent of the total rock. * * * Augite phenocrysts and granules constitute 2–25 percent of the basalt with only 1–5 percent occurring as phenocrysts. Magnetite is an important constituent of the groundmass, forming 5–15 percent of the sections. Glass in the groundmass of the basalt forms 15–65 percent of the sections, averaging 30 percent. In several of the basalts the glass was partly devitrified and altered to palagonite.

A hydrothermal type of alteration * * * has developed abundant secondary minerals. Zeolitic minerals (stilbite, mordenite, and natrolite) are common in most of the basalt and breccia, making up as much as 30 percent of some sections. Calcite is abundant, filling vesicules and irregular fractures, and chloritic minerals are present in most of the sections studied. Other secondary products identified are palagonite, epidote, opal, analcite, limonite, and serpentine.

The writer has also tentatively identified the mineral leonhardite, a variety of laumontite, among zeolites collected at the Clear Creek site. The identification was made in refractive oils, and the laumontite crystal structure was verified by Mary E. Mrose by X-ray (written communication, 1960). The augite phenocrysts appear from optical properties in oils to be the variety called pigeonite.

As pointed out by Baldwin (1947), the basaltic flow breccia when newly exposed is dark green to nearly black. Upon exposure to the atmosphere, the rock crumbles, becomes brownish buff in color, and the palagonite (altered basaltic glass) breaks down to yellow clay.

Snavely and Baldwin (1948) describe the tuffs and interbedded sediments as follows:

The pyroclastic rocks range from fine tuff to agglomerate. * * * In some exposures the fine tuff contains sporadic blocks of dense basalt. Microscopically the tuff contains altered basaltic fragments, calcic feldspar, and augite crystals set in a groundmass of opal, palagonite, calcite, and chloritic minerals. The interbedded sedimentary rocks are predominant water-laid tuff and tuffaceous siltstone and sandstone, with subordinate amounts of basaltic grits, sandstone, and conglomerate. Some of the finer tuff closely resembles a well indurated carbonaceous siltstone. These tuffaceous sediments weather to spheroidal masses, each concentric shell being coated with a film of manganese dioxide.

Observations by this writer were largely confined to the immediate localities of the Hollywood, Bark Shanty, Keyhole, and Clear Creek damsites. However, it was noted that the flows exposed along the North Fork near its mouth appear in general to dip gently southwestward, the dip direction ranging from westerly to southerly. Three miles northeast along the North Fork, in the vicinity of the Bark Shanty damsite, the rocks in the lower half of the valley section consist of interbedded flows, tuffs, dark tuffaceous shales, siltstones,

and sandstones, all dipping westward about 10°. The westward dip appears to persist for 2 more miles as one ascends the North Fork.

In the vicinity of the Keyhole damsite, the exposed rocks in the gorge are massive basaltic breccias similar to and perhaps belonging to the older series, equivalent with the Siletz River volcanic series. The younger, interbedded series makes up the higher valley walls above the "Keyhole," and where it is exposed along the road northeast of the Keyhole in NE¼ sec. 27, T. 1 S., R. 7 W., it is considerably folded and disturbed. This disturbed zone may include drag folding related to a fault of northeast trend shown on a new unpublished regional geologic map of Oregon west of the 121st meridian (Wells and Peck, written communication, 1960). This fault zone apparently follows the river valley through the west side of sec. 27 and then climbs the valley wall just north of the "Keyhole," passing out of the section at the northeast corner. The southeast side of this fault would be the upthrown side, bringing the older volcanic series up to where the river is now intrenching.

The principal anticlinal axis of the area appears to cross the valley in about the center of sec. 26. A mile farther east, at the Clear Creek site, the regional dip seems to be eastward at about 10°. The lower part of the valley here is entirely within massive flow breccias, the same flows as exposed at the Keyhole damsite.

The general westward dip observed along the North Fork between the Trask Guard Station and the Keyhole site suggests that the shales and sandstones in or near the valley bottom in this area are stratigraphically below the basaltic breccias and flows west of the guard station, or that there may be either a fault or monoclinal fold near the guard station, as yet unrecognized. However, as Baldwin (1947) and others have pointed out, in places in western Oregon there is a major unconformity which separates the beds of late Eocene age from those of middle Eocene age. The western margin of the upper Eocene overlap (Nestucca Formation) may be in the general vicinity of the guard station.

Lineaments observable on the topographic maps and aerial photographs suggest that the principal near-vertical jointing along the North Fork west of the Keyhole site strikes about N. 20° W. and dips steeply westward. Other less prominent jointing is suggested at N. 40° E. with a steep northwesterly dip and N. 55° W. with a steep southwesterly dip. East of the Keyhole site, the topography suggests that the most prominent jointing is vertical and strikes N. 10°–20° W.; other near-vertical jointing appears to strike approximately N. 60° W., N. 85° E., and N. 50° E. Along the South Fork about 2 miles south of the guard station near the Hollywood damsite, the most

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prominent topographic expression of near-vertical jointing is N. 20° E., probably with a steep westerly dip. Other lineaments in that area are N. 20° W. and N. 80° W.

In the following sections the geology of the Clear Creek, Keyhole, Bark Shanty, and Hollywood reservoir sites is by R. G. Wayland; that of the Ginger Peak site is by D. L. Gaskill.

CLEAR CREEK RESERVOIR SITE

DESCRIPTION OF SITE

The damsite is on the North Fork Trask River in sec. 24, T. 1 S., R. 7 W., about three-fourths of a mile downstream from the confluence of Clear Creek and the North Fork. The topography of this site is shown on plate 1. Drainage area at the site is 53 square miles, and the water surface altitude is 690 feet. A reservoir with a maximum water surface altitude of 900 feet would be 2½ miles long and would have areas and capacities as listed below.

Area and capacity of Clear Creek reservoir site

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
690	0	0	840	260	15, 100
720	20	300	880	420	28, 700
760	80	2, 300	920	520	47, 500
800	150	6, 900	960	740	72, 700

By comparison with the gaging station about 396,000 acre-feet of storage would be needed for controlling the average estimated flow of 355 cfs. It is obviously not feasible to effect a high degree of control at this site. By using 35,000 acre-feet for regulation, a discharge of not less that 130 cfs after losses could have been maintained during the period 1938–1952. The reservoir could have been operated to produce a minimum discharge of 460 cfs from November 1 to March 31 each year during the period if the summer releases were restricted to 30 cfs (Q95) when that was necessary for refilling in dry summers.

A possible use of the Clear Creek reservoir would be in connection with diversion to the Tualatin River basin. A tunnel 5.4 miles long could connect the 800-foot altitudes in both basins. The capacity of the reservoir above the 800-foot level would be 30,000 acre-feet.

In computing the divertible water in the above example a minimum discharge of 30 cfs (Q95) was left in the river for conservation purposes and losses were subtracted as explained.

GEOLOGY

At the Clear Creek damsite, plate 1, the North Fork of the Trask River flows in an inner gorge 400 to 500 feet deep cut in basaltic flows and flow breccias of probable early to middle Eocene age. Flow breccias predominate, and the individual flows are much the same in appearance and composition. Flow contacts are tight and irregular. One distinctive flow of porphyritic basalt (loc. 1) is readily observable along the left abutment about 150 feet above river level; it appears to dip to the east about 10°. Other flow contacts cannot be traced easily because of limited exposures. No interbedded tuffs or sediments were observed in the probable abutment areas. Some amygdaloidal basalt is present in most talus and in many outcrops, as at localities 2 and 3.

The exposed basaltic rocks are only slightly decomposed by weathering. Hydrothermal alteration includes some minor calcite and zeolite veinlets, amygdules, and breccia filling; the original basaltic glass is largely altered to palagonite, and in some areas there is considerable chloritization. Despite this alteration the rocks seem relatively impervious, moderately strong, and competent for well-spread foundation loads.

At locality 4 a slickensided shear zone with right-lateral movement strikes N. 70° E. and dips about 80° N. The zone was traced up the hillside into a chloritized area in the right abutment. No other evidence of faulting was noted. Jointing as observed in outcrops is widespaced, tight, irregular, and inconspicuous. At locality 2, the minor joints observed in the eastward dipping flows were random and inconsistent in strike and dip. Topographic expression within a 2-mile radius at the site suggests, however, some joint control of drainage, the joints striking approximately N. 10°–20° W., N. 60° W., N. 50° E., and N. 85° E.

Among the zeolites, pinkish natrolite is conspicuous at locality 2, and leonhardite was tentatively identified by the writer at locality 3.

Bedrock is exposed in the channel and the abutments, or is lightly covered with thin soil and with talus composed of relatively small gravel-sized fragments. Fresh rock can be exposed by shallow stripping. The slope on the right bank at section *A–A′* is about 35°, and at locality 5 the slope of this bank is about 40°. On the left bank at section *A–A′* the slope is as steep as 50°. The relatively smooth, uniform slopes and the small size of the talus fragments are a natural consequence of the autobrecciation within the flows. In contrast, the porphyritic flow (loc. 1) and some flows or sills observable near the top of ridges to the north and east of the site present oversteepened or clifflike outcrops and coarse talus. On exposure to weathering,

the flow breccias disintegrate along breccia fragment contacts more than they do along joints. It does not appear that this mode of physical weathering proceeds any faster than disintegration along joints in nonbrecciated rocks in the Trask River area, but it would be desirable to test the breccias for any possible rapid disintegration due to alternate wetting and drying in a reservoir or to wave action.

No deep-seated active landslides were noted or are discernible on the aerial photographs. Some shallow-seated sliding, creep or wash of the surface mantle into the reservoir may be expected where the adjacent valley sides have been denuded of forest cover by fire or lumbering operations.

Leakage would not seem to be a serious problem. Certain flows may contain vesicular tops that are slightly permeable, but the process of autobrecciation should have helped assure close configuration of each new flow with the irregular surface of the preceding flow. Subsequent cementation with zeolites and calcite would also have sealed most permeable openings. In the left abutment area, the contact between the porphyritic flow and the underlying flow breccia is a possible leakage zone; but due to its altitude, it would be under low hydraulic pressure. Water was observed in side gulches despite dry weather during and before field inspection.

There is no natural spillway site, but a spillway tunnel might be cut through the flow breccias on either abutment if an overflow spillway is not desired. Such a tunnel would require little structural support, but the rock may require lining and should be tested to determine its resistance to disintegration along natural breccia fragments under conditions of alternate wetting and drying, as mentioned above, and under static rock pressure. The known shear zone (loc. 4) should be avoided, as should other minor shear zones that may be found during a more extensive subsurface investigation of this site or during stripping.

As aggregate, the flow breccias should be viewed with suspicion, and tested for the presence of opal or chalcedonic silica. Better aggregate is probably available in river bars upstream, or possibly on some of the high river terraces. One such terrace appears to be above the left abutment at an altitude of about 1,200 feet. It is understood that gabbro or diorite sills or dikes occur in the SE¼ sec. 30, the S½ sec. 18, T. 1 S., R. 6 W., W.M. and in the east wall of the canyon of Elkhorn Creek. These localities were not visited. Dense fresh porphyritic basalt from nonbrecciated nonamygdaloidal flows would also be worthy of consideration. Such rocks, as well as the intrusive rocks, should be suitable for riprap.

Impervious materials might be found in the mantle in broader sections of the valley upstream 1¼ miles on the North Fork, or half a mile up Clear Creek. They may also be available on those highest flat-topped uplands that are remnants of the Miocene erosion surface (Baldwin, 1947, p. 50). This latter might include part of Gobblers Knob. Another possible source might be some of the shaly, tuffaceous beds of the Nestucca(?) Formation higher in the valley walls above the lava flows exposed at the damsite.

In conclusion, the site seems suitable for either an earth dam or a broad-based concrete dam. A concrete arch dam might be considered if testing and subsurface investigations recommended above give satisfactory results.

A diversion of water from a Clear Creek reservoir to the Tualatin basin has been considered. The regional maps suggest that any diversion tunnel would be largely in the Nestucca(?) Formation. Such a tunnel would require support in some sections, and lining might be required in the weaker tuffs and shales. Some Tertiary diorite or gabbro dikes and sills are known from regional mapping in this general area and might be encountered. A fault trending about N. 15° E. from Elkhorn Creek would apparently be intersected by a diversion tunnel in sec. 21, T. 1 S., R. 6 W.

KEYHOLE RESERVOIR SITE

DESCRIPTION OF SITE

The damsite is on the North Fork Trask River in sec. 27, T. 1 S., R. 7 W. The topography of the damsite is shown on plate 1. The drainage area there is 57 square miles. The altitude of the stream at the site is 580 feet. The maximum altitude of the reservoir surface would be limited to 690 feet, the stream altitude at the Clear Creek site. The dam for this reservoir altitude would have a crest length of about 260 feet. The site might be suitable for a gravity or an arch dam. Areas and capacities for this reservoir site are as follows:

Area and capacity of Keyhole reservoir site

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
580	0	0	680	80	4, 200
600	20	200	720	160	9, 000
640	50	1, 600	760	270	17, 600

The reservoir described above would provide capacity sufficient to reregulate discharges from Clear Creek reservoir and would create an average head of 95 feet for production of power.

GEOLOGY

The "Keyhole" is a conspicuous section of the inner gorge of the North Fork of the Trask River where the river flows between near-vertical cliffs cut in basaltic flows and flow breccias of probable early to middle Eocene age. Two particularly prominent outcrops are diagonally opposite each other (pl. 1, section *A–A′* and fig. 4) and suggest a natural site for a narrow base dam that could be higher than the 110 feet required by the scheme of river development presented. There are, however, certain problems of a geologic nature that must be thoroughly investigated, and drilling will be required before the site can be fully certified.

Both abutments are composed of the same basaltic rocks. The contacts between flows are irregular and tight. Without close examination it is difficult to distinguish the individual flows with certainty. There is little or no color contrast between the dark-greenish to black flows. Textural variations include pillow lavas and some amygdaloidal tops, but they are not conspicuous enough to allow individual flows to be followed far by eye or with binoculars from below the cliffs or from across the river. However, on the basis of several random observations, it appears that breccia predominates in the middle and upper part of the abutments and dense flows are at the bottom. It also appears that the flows dip gently north, northeast, or northwest. No interbedded tuffs or sediments are in the abutments.

FIGURE 4.—Keyhole damsite. View from downstream, looking east to the "Keyhole." Photograph by J. L. Colbert.

The basaltic rocks exposed in the abutments are virtually free of decomposition by weathering; but the talus fragments are relatively small, indicating physical disintegration along breccia fragment contacts. At locality 1, above the right abutment, the basalt is disintegrated and weathered to brownish buff.

Fresh vitreous augite phenocrysts are common in most flows and flow breccias, and feldspar phenocrysts are common in a few flows. The groundmass of the basalt is largely altered to palagonite, and the augite of the groundmass is partly chloritized. Zeolites may be found in some flows as amygdules and as local breccia filling, along with minor amounts of calcite. Some of the breccia readily absorbs water, indicating considerable local porosity but probably no extensive permeability. In general, the rocks of the abutments, section *A–A′*, are relativeley fresh, strong, and competent. No direct evidence of faulting at the actual site was observed. As stated under the general section on geology, there is some evidence that a northeast-trending fault may pass into the reservoir area just north of the Keyhole site, perhaps within a few hundred feet of locality 1. Northeast of the Keyhole site, the trend of the axial plane of some of the folding observable in the disturbed zone suggests the possibility of an intersecting fault from the south, just east of the Keyhole site. The rocks of the abutments at the actual damsite seem to be unaffected, but inspection of the canyon bottom may disclose a fracture zone or strong jointing that facilitated the rapid downcutting by the river.

Wide-spaced jointing at the damsite is no doubt responsible for the external shapes of the conspicuous outcrops. One joint plane observable in both abutments strikes N. 5°–20° E. and dips steeply west. Another strikes N. 10°–30° W., also with steep westerly to vertical dip. A third plane strikes N. 45°–55° W. and dips about 60° SW.; a fourth strikes east-west and dips moderately southward. At localities 2, 3, and perhaps 4, a near-vertical joint striking approximately N. 60° E. may be responsible for the parallelism of several faces of cliffs on the right abutment and adjacent outcrops. This joint was observed at locality 3, and at locality 5 just west of the left abutment, as well as in the left abutment itself. Topographic expression within a two-mile radius of the damsite confirms the major joint trends as N. 10°–30° W. and N. 5°–20° E., and also the joints striking N. 45°–55° W. The jointing, like the possible fault north of the site, may be of some small concern in the matter of reservoir leakage but would not seem to create a major problem in the founding of a dam at the actual site or be an important cause of reservoir leakage. A slab on the left abutment fronting an open joint will have to be removed or grouted.

At the site, the river is flowing on coarse alluvium consisting largely

of gravel, cobbles, and small boulders of porphyritic basalt. The depth to bedrock along section *A–A′* may be 10 feet, and possibly as much as 20 feet. Bedrock is at river level upstream on the left bank and downstream on the right bank but is covered by alluvium or talus at section *A–A′*. The nearest nickpoint in the stream profile appears to be slightly downstream (pl. 1); it is relatively inconspicuous, but supports the observation that the river is not at present cutting into bedrock where it crosses section *A–A′*.

Drilling will be necessary to determine not only the depth to bedrock under the channel but the nature of the bedrock. From observations in the reservoir area it is assumed to be flow breccia.

Shallow-seated landslides into the reservoir may be expected but would apparently be of little consequence. Leakage from the reservoir along the fault projected through the ridge north of the right abutment could be considerable. The ridge should be drilled for subsurface geologic information and for pressure testing. The left abutment may afford a good location for a tunnel spillway.

The flow breccias at the damsite and the intake and outlet of a spillway should be tested for resistance to disintegration from wave action, alternate wetting and drying, and current erosion. The presence of a natural bridge in the prominent outcrop at locality 6 demonstrates this need.

As aggregate, the possible presence of opal in the flow breccias as well as their tendency to disintegrate would suggest selection of other material, such as alluvial gravels just upstream from the site or the sources mentioned in connection with the Clear Creek site.

Impervious materials may include the tuffaceous shales overlying the basaltic flows, or possibly some materials available on the highest flat-topped upland remnants of the Miocene erosion surface.

In conclusion, the site seems suitable for a narrow-base concrete dam, if subsurface exploration of a possible fault zone north of the right abutment indicates that area is sufficiently strong and tight to withstand the relatively low hydraulic head from a dam 110 feet high, if the channel bottom is found tight and competent, and if the flow breccias pass tests for resistance to disintegration.

BARK SHANTY RESERVOIR SITE

DESCRIPTION OF SITE

The damsite is at river mile 2.8 on the North Fork Trask River in sec. 29, T. 1 S., R. 7 W. The drainage area at this location is 75 square miles, and the water surface is at an altitude of about 400 feet. Average discharge at the damsite is estimated to be 508 cfs. A dam that would raise water to the 600-foot level would create a reservoir

with a capacity of 27,000 acre-feet. Use of 25,000 acre-feet for regulating the stream could have provided a discharge of at least 145 cfs during the test period. Operated in conjunction with the Clear Creek reservoir, the minimum discharge could have been increased to 215 cfs. Both amounts are after subtracting an appropriate allowance for losses. The reservoir could have been operated to produce minimum discharge of 600 cfs between November 1 and March 31 of each year during the period if summer releases were restricted to 30 cfs (Q95) when that was necessary for refilling in dry summers.

The maximum reservoir altitude at this site would be limited to 580 feet if the Keyhole site were to be developed. Areas and capacities for this site are as follows:

Area and capacity of Bark Shanty reservoir site

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
400	0	0	520	150	7, 500
440	19	380	560	240	15, 300
480	93	2, 600	600	350	27, 000

Operated together, the reservoirs at the Clear Creek and Bark Shanty sites could have assured a minimum discharge of 700 cfs during the 5-month winter period.

GEOLOGY

At the Bark Shanty damsite the North Fork Trask River passes through a local constriction between a spur of the ridge on the right bank and an oversteepened left bank. A prominent terrace occurs about 130 feet above the river on the right bank, and various other inconspicuous terraces are discernible. The terraces slope gently to the west and appear to be due to differential erosion within a series of lavas, tuffs, siltstones and thin-bedded standstones which constitute the bedrock in this area. As discussed in the general section on geology, this series is now tentatively correlated with the Nestucca formation. Geology and topography at the site are shown on plate 1.

The valley constriction below the prominent terrace is due to the thick flows (or sills?) of gray porphyritic andesite. These flows are overlain by tuffs, tuffaceous sediments, and thin flows. Some interbedded tuff is probably also present in the andesite flows, as at locality 1 and as inferred in section *A–A′*. At the damsite the flows strike about N. 10°–30° E. and dip west at approximately 4° to 7°, considerably steeper than the 1° westerly stream gradient. The upper massive flow appears to be 60 to 80 feet thick. Its line of outcrop forms a vertical cliff on the left bank at stream level (loc. 2) and an ascend-

ing near-vertical cliff southwestward from there along the left abutment to and beyond locality 3. Although outcrops are discontinuous, it appears that this same flow forms the prominent terrace on the right abutment. A lower massive flow crops out in the road cut and was located tentatively in other areas on the basis of float (pl. 1). The thin flow at about 600 feet altitude, section *A–A′*, was located by float and topographic expression.

Upstream a quarter of a mile in the reservoir area, the left bank is seen to consist of distinctly interbedded flows, with underlying tuffs, dark shaly beds, and sandy beds. Here the andesite flows have an aggregate thickness of perhaps 150 feet.

On examination of the andesite under a petrographic microscope using refractive oils, the feldspar phenocrysts were determined to be andesine and the ferromagnesian phenocrysts to be augite. For this reason the rock is here classed as an augite andesite, although further petrographic examination with thin sections would be desirable. It was observed that the feldspars are somewhat altered to clay minerals and the augite to chlorite, even in specimens that appear on visual examination to be fresh. The fresh rock is very hard and tough. On exposure, it breaks along joints into large angular blocks.

Exposures are largely limited to the road cut and to the over-steepened parts of the left abutment. Joints in the exposed flows are closely spaced. Most appear to be crude columnar jointing, but some horizontal jointing is present, roughly parallel to the flow bedding. The largest joint blocks observed in the talus and the streambed were about 15 feet long. No persistent joint directions could be observed in the limited outcrops along the river, and the rock exposed in the roadcut has slumped out of position. The drainage pattern within a 2-mile radius of the damsite, however, suggests well-defined near-vertical jointing that strikes N. 5°–10° W., another joint of N. 55° W. strike (the course of the river at the damsite), another that strikes N. 20°–40° E., and still another than strikes N. 65°–75° E. The courses of the two minor streams mapped on the left abutment (pl. 1) appear to be governed by two of these joints.

Bedrock is not exposed in the valley bottom along section *A–A′*. The river here is flowing on alluvium ranging from coarse sand to boulders. In the channel most of the alluvium is well rounded, but off to the side of the channel, angular talus blocks predominate. Most of the alluvium and talus consists of augite andesite derived from the flows. The depth to bedrock in the channel section along the axis, and the nature of the bedrock, will have to be determined by stripping or drilling. About 1,000 feet downstream from section *A–A′* the top of the uppermost flow is exposed in the river channel; this outcropping suggest that the depth to bedrock at section *A–A′* is not more

than 10 feet. If the aggregate thickness of the massive andesite flows is 150 feet, the bedrock under the channel at section *A–A′* is the underlying tuffaceous sediments.

Because of the different physical characteristics of the flows and the interbedded and underlying shaly tuffs, the site is probably not suitable for a concrete structure but may be suitable for an earth dam as high as 200 feet above present river level. Leakage should be anticipated along the more permeable interbedded tuffs and along near-vertical joints in the flows, probably requiring considerable grouting. The ends of the dam can be well anchored in the competent flow rock of the abutments, but a cutoff wall in the less-competent beds under the channel at the axis will probably be required. Considerable stripping will be necessary to expose sound rock in the channel and the right abutment. There is a possibility that the right abutment may have been involved in a slide. Protection of tuffs and sediments from slaking and from wave action will probably be necessary. Pressure testing as part of subsurface investigation would seem necessary. Differential compaction of flows, tuffs, and sediments under load must be allowed for in design of rigid structures. Other possible flaws include a possible shear zone along one of the directions discussed above under the heading of joints, a deeper buried channel than suspected from the limited surface evidence, and a cavernous zone in the flows.

A spillway could possibly be laid on the dip slope of a competent bed or flow overlying the massive flows, with intake near the left end of the dam.

Rock fill, riprap, and aggregate can probably be obtained from the andesite flows at the site and from the talus and alluvium along the channel. Soils or clays for impervious curtains, screens, cores, grouting, and other uses, will be of limited availability in the damsite area and may be unsatisfactory in quality. Nonplastic borrow materials needed for an earth dam may probably be obtained from the tuffs and sediments of the reservoir area upstream from the damsite.

HOLLYWOOD RESERVOIR SITE

DESCRIPTION OF SITE

The damsite is on the South Fork Trask River below the mouth of East Fork in sec. 6, T. 2 S., R. 7 W., at river mile 1.9. Drainage area at this site is 49 square miles, and the water surface altitude is 350 feet. Average discharge at the damsite is estimated to be 320 cfs. The reservoir site has a storage capacity of 34,000 acre-feet at an altitude of 550 feet. By using 30,000 acre-feet for regulating the stream, a minimum discharge of 110 cfs after losses could have been

maintained during the test period. The reservoir could have been operated to insure a minimum discharge of 450 cfs from November 1 through March 31 of each year during the period if the summer releases were restricted to 25 cfs (Q95) when that was necessary to refilling in dry summers.

The topography of the damsite is shown on plate 1. Areas and capacities of the reservoir site are as follows:

Area and capacity of Hollywood reservoir site

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
350	0	0	460	161	6, 160
360	3	15	480	212	9, 880
380	12	165	500	279	14, 800
400	34	625	520	320	20, 780
420	68	1, 640	560	500	37, 180
440	111	3, 440			

GEOLOGY

At the Hollywood damsite the South Fork Trask River flows northward through a local constriction about half a mile long. The bedrock on both sides of the valley and in the channel consists of nearly horizontal basaltic flow breccia. At the narrowest part of this valley, near section *A–A′*, small igneous dikes across the valley. The greater resistance of these dikes to erosion probably accounts for the constricted cross section of the valley. Geology is shown on plate 1.

The basaltic flow breccias are typical of those described in the section of this report on general geology. They are of probable middle Eocene age, and may correlate with the Siletz River volcanic series. The individual flows are massive but thoroughly brecciated. Observable contacts between flows are tight, irregular, and inconspicuous. The larger breccia fragments consist of porphyritic basalt, some of which are amygdaloidal. The breccias are well compacted, and although they are porous enough to absorb water noticeably when dry, they do not appear to be sufficiently permeable to cause a leakage problem. The homogeneous nature of the flow breccias is illustrated in the damsite area by the smooth nearly continuous exposures in the right bank, as at locality 1; by the abundant potholes developed by the river in its present channel (fig. 5*A*); and by the broadly spaced inconspicuous jointing found in the outcrop areas (loc. 2).

White crystalline zeolites occurring as amygdules as well as breccia filling are common in the exposed flow breccias. Some calcite is also present, but analcite seems to predominate. Relatively little chloritization was observed in the outcrops, but under the microscope in-

A

B

FIGURE 5.—Hollywood damsite. *A*, View from camera station 1 showing pothole in flow breccia in the foreground and dike in the right background. *B*, View from camera station 2 showing dike ascending right abutment from stream level.

cipient chloritization of the ferromagnesian minerals is seen to be widespread. The groundmass of some breccia fragments appears to consist of altered basaltic glass; in others it is finely crystalline basalt or dense porphyry with a few small augite phenocrysts.

In the stream channel the flow breccia is very fresh in appearance. On the steep right abutment the exposed rock disintegrates into breccia fragments before it can be extensively decomposed by chemical weathering. Elsewhere, particularly on the left abutment where exposures are very poor, the depth of disintegration and decomposition may be considerable. The mantle derived from this weathering conceals nearly all bedrock. The outcrop of breccia shown at locality 3 is probably not firmly in place.

The dikes shown in figure 6 are of differing textures but of apparently similar composition. Judging from the color of their aphanitic groundmass and the presence of zoned feldspar phenocrysts, they are mostly porphyritic andesite. The larger dike at locality 4 and in figure 5*B* is a dense gray-green rock with some inconspicuous phenocrysts identified as andesine in refractive oils. The groundmass appears to be a dense mat of partly kaolinized feldspars and chloritized amphiboles or pyroxenes. The small dike 300 feet north is mainly an aphanite, but it contains some very small andesine and ferromagnesian phenocrysts. Upstream in the reservoir area about 800 feet south of section *A–A′* a highly porphyritic andesite dike crosses the road and river; it has numerous light-colored feldspar phenocrysts that are conspicuously zoned; it also has a few euhedral doubly terminated pyroxene phenocrysts that are pseudomorphously altered to chlorite.

In the mapped area the dikes are best exposed in the stream channel and along the right abutment. They all dip northward at about 65° to 70° and strike N. 85° W. Jointing in the dikes is feebly columnar in nature. On the left bank no natural outcrop of the dikes could be found because of the brush and the mantle; but joint blocks of porphyritic andesite were seen in a shallow excavation at locality 5, and rubble derived from such rock was observed up and down the hillside from this locality. It is assumed that the dike zone crosses the left abutment in the general area shown on plate 1 by the inferred contacts.

Jointing in the flow breccias, as seen in outcrops, has been described above and shown on plate 1. Sheeting was also observed, but sheets, once loose, tend to disintegrate into breccia fragments. The drainage pattern within a 2-mile radius of the damsite suggests relatively strong, near-vertical widely-spaced jointing that strikes N. 30°–40° E., N. 20° E., N. 20° W., and N. 75°–80° W. No direct evidence of faulting was observed within the mapped area.

Above the bedrock channel in which the river is now flowing is a terrace about 150 feet wide along section *A–A′* and wider to the north. It represents an earlier stream channel and flood plain. Terrace deposits consist of stream-sorted gravels and sands interbedded with poorly sorted mantle material washed down from the left bank. Bedrock near section *A–A′* is assumed to be higher under the terrace than in the present channel, but a buried channel under the present road is a possibility.

The right abutment appears to be free of danger from landslides. The left abutment may possibly be a remnant of an old deep-seated slide. In the reservoir area some minor sliding of the mantle should be expected.

Leakage would not seem to be a problem once solid flow breccia or andesite bedrock are exposed in the left abutment and under the terrace. The amount of material to be removed in these areas to expose sound bedrock cannot be estimated without a subsurface investigation. The dikes will probably tend to seal off percolation between flows or along joints in the flows. They will also add strength for the founding of masonry structures, probably without introducing a problem of differential compactibility.

A spillway might be cut through the left abutment. The resistance of the flow breccias to erosion and to slaking under alternate wetting and drying should be tested and taken into account in the design of structures.

Terrace gravels upstream or downstream from the site will probably provide suitable aggregate. The flow breccias should be avoided for this purpose or for riprap. If quarrying is necessary, andesite dikes are available in the reservoir area, and a coarse-grained intrusive igneous rock of intermediate composition is understood to be exposed a few miles southwest of the site, on Joyce Creek (H.G. Schlicker, Oregon Department of Geology and Mineral Industries, oral communication, 1960). Soils or clays for impervious cores may possibly be available in parts of the valley bottom a few miles downstream from the damsite, occurring as sediments in former temporary lakes. More likely, impervious materials will have to come from downstream below the mouth of the Trask River canyon. Nonplastic material needed for an earth dam will be available in river terrace deposits at several nearby localities.

GINGER PEAK RESERVOIR SITE

DESCRIPTION OF SITE

The damsite for this reservoir site could be at one of several possible locations between mile 5 and mile 8 on the Trask River in secs. 20, 21, 28, and 29, T. 1 S., R. 8 W. The water surface altitude

is 100 feet at the downstream end of this reach and about 180 feet at the upstream end. Raising the water surface from an altitude of 100 feet to an altitude of 350 feet, which backs water to the Hollywood site on the South Fork, would provide about 117,000 acre-feet of storage. Areas and capacities for this site are shown in the following table.

Area and capacity of Ginger Peak reservoir site

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
100	0	0	240	411	20, 750
120	21	210	260	535	30, 210
140	41	830	280	678	42, 340
160	80	2, 040	300	855	57, 670
180	150	4, 340	320	1, 003	76, 250
200	229	8, 130	340	1, 163	97, 910
220	311	13, 530	360	1, 312	122, 660

The geologic report, which follows, considers three possible sites within the reach. (See pl. 2.) The downstream site, *C–C′*, will be used for storage analyses. The average discharge is estimated at 928 cfs.

By using 100,000 acre-feet the reservoir could have regulated the stream to a minimum discharge of 395 cfs after losses during water years 1938 to 1952, inclusive. Operated in conjunction with the usable capacities of upstream reservoirs, a minimum discharge of 570 cfs could have been assured during the test period. If used to produce the greatest possible discharges during the 5 wet months, November through March of each year, the reservoir could have assured a minimum of 1,235 cfs by itself. With all usable storage in the upstream reservoir sites, this could have been increased to 1,510 cfs. These amounts are after losses. Average head at the Ginger Peak dam would have been about 200 feet for either type of operation. Simulated operating schedules for the most critical years are shown for winter months in table 3 in the summary of regulating possibilities.

GEOLOGY

By D. L. Gaskill

Rocks in the Ginger Peak damsite area seem to consist principally of a thick sequence of basaltic breccia flows, although some nonbrecciated flows are present. The breccia generally has a rough blocky appearance, but it may exhibit some vesicular, scoriaceous, or in places ropy structures. The larger components are composed of dark-gray, fine-grained or slightly porphyritic basalt well cemented by a basaltic matrix of finer particles in volcanic glass or palagonite(?).

Talus material locally includes specimens of basalt porphyry, amygdaloidal basalt, and important quantities of hydrothermally altered dioritic breccia. The flows are cut by many small basalt dikes and a number of larger diabasic and felsitic dikes. These rocks have been assigned to the Tillamook volcanic series of Eocene age by Warren, Norbisrath, and Grivetti (1945), who give an estimated thickness of 6,000 to 10,000 feet of lava exposed in the canyon of the Trask River from its mouth to its forks near mile 11.

Little is known of the local structure. Flow contacts generally appear to be very irregular and flow attitudes vary moderately from the horizontal. No faults were definitely recognized, but faulting has undoubtedly modified the local structure. A number of lineaments shown on plate 2 were interpreted from aerial photographs.

Remnants of alluvial terrace deposits are locally preserved along the Trask River. These deposits are probably related to three well-developed terraces at the mouth of the Trask River Canyon.

SECTION A–A′, MILE 7½

Bedrock is exposed in the channel of the Trask River and at weathered outcrops on the southwest abutment. These and other exposures downstream, particularly at localities 1 and 2 and in the vicinity of section *B–B′*, indicate that nearly all the abutment and foundation rocks are composed of thick sheets of basaltic flow breccia. More resistant dikes of dark-gray basalt cut these exposures locally. Upon exposure to weathering, the breccia is reduced to an unconsolidated rubble of its component parts. Deeply weathered breccia is exposed at scattered outcrops on the southwest abutment and on "The Peninsula" at locality 1.

No reliable attitudes were taken on these flow sheets, but the general attitude is near horizontal. At one weathered outcrop on the southwest abutment the flows strike roughly north and dip about 15° west. A few thin tight joint sets trend downstream and north-south in the river channel, but they are rather poorly developed and inconspicuous. Several lineaments appear to cut the southwest abutment on aerial photographs. These lineaments may represent zones of fractured or faulted bedrock. Both abutments are steep and covered with varying amounts of weathered residual mantle and slope wash, however, the northeast abutment is more massive and not so deeply weathered as the southwest abutment. Some 25 feet or more of unconsolidated alluvial terrace deposits overlies foundation rock on the northeast side of the river and lesser quantities of coarse alluvial material and slope detritus lie at the base of the southwest abutment.

Bedrock exposures are limited here, but they indicate a gentle attitude of the flow structures. Bedrock exposures in the river channel downstream from section *A–A′* indicate excellent foundation conditions. The southwest abutment appears to be much more deeply weathered and probably is structurally weaker than the northeast abutment.

Stripping requirements should not be excessive on the northeast abutment but may be extensive south of the river. This site seems to be very favorably located topographically.

SECTION B–B′, MILE 6

A view of this site is shown in figure 6. Foundation rocks are well exposed along the river in the area of section *B–B′*. These rocks exhibit layers of relatively fresh basaltic flow breccia, locally cut by thin, tight, widely spaced joints and large dikes of porphyritic basalt. Flow contacts are very uneven and undulating; but they appear to dip about 20° SE., although the topmost flow on the south bank has an apparent downstream or southwest dip at section *B–B′*. Joint sets trend north and northwest and dip steeply downstream. Less conspicuous fractures are healed with calcite and quartz veinlets. The most notable structures are the large resistant dikes, as much as 6 feet or more in width, which trend east-west and dip 40° to 50° north across the riverbed here. The dike rock has an aphanitic groundmass with small feldspar phenocrysts oriented parallel to the strike of the dike. The position of the largest dike with reference to the observed attitude of the bedrock suggests a possible offset of flow layers along this structure.

Abutment rocks are generally concealed by heavy forest vegetation. The steep bluff at locality 2 exposes massive outcrops of rough-textured vesicular scoriaceous blocky fragmental lava, cut by north-striking subvertical joints, a few irregular dikes and many small calcite veins (fig. 7). Secondary minerals, chlorite and zeolites, are abundantly developed in places along irregular fractures. Individual flows are not well defined, but they have an apparent near-horizontal position or dip moderately eastward. This outcrop, although considerably more altered than those in the river channel, is much less weathered than outcrops on "The Peninsula" at locality 1, and for the most part identical in structure and composition to all the exposures described in the area of sections *A–A′* and *B–B′*.

Flow layers in the river channel dip 20° to 30° southeast across the river, but they may be overlaid on the abutments by flows having a moderate downstream dip. Exposures upstream at locality 2, however, indicate near-horizontal flow attitudes. Joints and flow con-

FIGURE 6.—View downstream of section *B–B′*, Ginger Peak damsite, from locality 2.

FIGURE 7.—Typical exposure of basaltic breccia at locality 2, Ginger Peak damsite.

tacts may be effectively sealed in part by the large north-dipping dikes which apparently cut the southeast abutment.

Stripping requirements at this site are likely to be somewhat less than at sections *A–A′* or *C–C′*; however, considerable excavation of soil, slope debris, and weathered bedrock will be necessary on both abutments. Timber removal would be a more important phase of reservoir preparation than would be the case of a site selection at section *A–A′*.

The topography is very adaptable for a dam up to a maximum height of 500 feet above the river. Appurtenant works might be cut through the bedrock spur forming the southeast abutment.

SECTION C–C′, MILE 4.8

The Trask River flows here in a very narrow gorge cut in basaltic breccia. As many as eight individual flows are clearly exposed to a height of 15 feet or more above the river. Flow contacts are generally irregular and some of the thinner flows wedge out in cross section. On the northwest abutment, above the narrow bedrock channel, foundation rock is overlaid by alluvial terrace remnants and talus material. The terace deposits are probably more than 15 feet thick at section *C–C′* and are, in turn, overlain by a considerable thickness of roughly sorted slope debris including large boulders of hydrothermally altered breccia. The breccia is altered, in part, to greenstone. Basaltic

breccia is exposed to a height of 100 feet, or higher, on the steep southeast abutment. The flows dip about 20° north-northwest with a slight upstream component of dip. The breccia is cut by small basalt dikes and one very large andesitic dike, about 50 feet wide, that trends east-west and dips about 70° N.

The basaltic breccias are similar in composition and structure to those described upstream at sections *A–A′* and *B–B′*. The hydrothermally altered talus blocks apparently were derived from a source on the northwest canyon wall and are similar to altered dioritic breccia at the large quarry near the mouth of the Trask River Canyon. Similar metavolcanic rocks crop out in the canyon along roadcuts one-half to three-quarters of a mile below section *C–C′*.

This site is in the deepest and narrowest stretch of the Trask River Canyon. The canyon is 2,000 feet deep at this point with steep slopes that average at least 30°. Abutments above the 200-foot contour were not investigated.

This site might be considered for a high dam. Some 25 feet of terrace and slope detritus overlies bedrock on the northwest abutment, but bedrock is exposed in the narrow river channel and on the southeast abutment. Flow sheets dip gently upstream and into the northwest abutment. Where examined at or near river level, the bedrock appears structurally stable, perhaps more ideally so with regard to a dam structure than either of the upstream sites. A dam height of 260 feet (Helland, 1953), would make the crestline of a dam at section *C–C′* about 1,100 feet long or approximately equal in length to the crestline of sections *A–A′* or *B–B′*. Stream gradients from *A–A′* to tidewater average about 23.8 feet per mile and decrease to about 20 feet per mile at section *C–C′*. A disadvantage to this site might be the extreme height of the canyon walls with consequent exposure to landsliding.

It is highly probable that thin interbeds of pyroclastic or tuffaceous sedimentary material separate some of the flow layers in the area. The presence of such beds in the foundation or abutment rocks could have major engineering significance. An analysis of breccia matrix should be made together with tests of the bearing strength of this rock.

CONCLUSIONS AND RECOMMENDATIONS

Bedrock exposures in the Ginger Peak damsite area exhibit massive flow layers of homogeneous basaltic breccia. The fresh rock is tough and well bonded, but it crumbles into its component particles when exposed to weathering. Joints are not everywhere conspicuous, but where well developed, appear tight and impermeable. No evidence of recent fault movement was observed. Only moderate grouting should be required although more information would be necessary

for a critical evaluation. Crosscutting dikes are very resistant and must have important effect on the local topography and groundwater movement. Some of the larger dikes might serve as impermeable subvertical plates in foundation and abutment rocks where favorably alined. Contact zones between flow sheets are very irregular, often wedge-shaped, and in most places appear sufficiently watertight and insoluble to prevent excessive leakage around dam structures. A moderate dip of the flow layers in foundation and abutment rocks probably does not greatly detract from the structural stability of the bedrock, owing in part to the rough irregular contacts between flows. Lineaments discernible on aerial photographs appear to be developed along prominent fractures that may represent important structural anomalies. Lineaments should be carefully explored where they bisect a potential site.

Most of the reservoir site and drainage basin is within the area of the great Tillamook burn, with little reforestation of denuded slopes. This condition increases the possibility of flooding and abnormal siltation, particularly as these brushy-covered areas of the old burn are vulnerable to repeated fires at this stage of regrowth.

Stripping requirements of foundation rock at each of the sites investigated would be minimal. Unconsolidated deposits and much of the weathered bedrock on abutments could be removed by light excavation methods. Overflow spillways would be feasible, but light protection of bedrock from plucking may be necessary. Tunnels in the flow breccia would probably require lining where intersected by faults or badly fractured zones, but may require little or no support during construction in fresh rock.

The reservoir may be subject to some leakage along flow or fracture planes at the damsite, but the probable amount is believed to be negligible. The water table is assumed to be everywhere above the potential reservoir site.

Geologic conditions do not seem to be greatly different between sites within the area, but geologic structures are imperfectly known and should be explored in greater detail at the site selected.

All three of the sites described in this reconnaissance examination seem to be geologically suitable for a rockfill, earthfill, or possibly a concrete-type dam.

Moderate to strong seismic activity in this region (Coast and Geodetic Survey, 1950) should be considered in designing large construction projects in this area.

CONSTRUCTION MATERIALS

Rockfill and riprap material could probably be quarried at or near the damsites. Much of the unconsolidated material in this area, particularly talus, slope detritus, and the regolith mantle, would be unsuitable as construction material because of the susceptibility of basalt to weathering. Unconsolidated terrace deposits include roughly sorted gravel and cobbles, a few thin beds of clay, and a shallow soil cover. Large flood-plain deposits of sand and alluvial soil are available below the mouth of the Trask River canyon. All those basaltic materials are susceptible to rapid decomposition by ground water. According to Mielenz (1948, p. 8), "basaltic sands and gravels commonly become coated with opal * * * leached from the basalt pebbles. These opal coatings are deleteriously reactive with cement alkalies."

A large quarry has been developed in tough hydrothermally altered pyritized dioritic breccia at the mouth of the Trask River canyon in the SE cor. sec. 36, T. 1 S., R. 9 W. The breccia is composed of large angular blocks tightly bonded in a silicic matrix of smaller fragments. Irregular fractures indiscriminately cut the component blocks as well as the matrix. The breccia exhibits flow structures indicating a replacement or rheomorphic origin. Similar rocks are exposed along the highway in sec. 30, T. 1 S., R. 8 W. These breccia, as well as the porphyritic dikes, and possibly the thick basic nonbrecciated nonamygdaloidal flows of this area, should prove suitable for riprap or rockfill material.

SUMMARY OF REGULATING POSSIBILITIES

It would require about 1 million acre-feet of storage for complete regulation of the river at the Ginger Peak site. This is much beyond the capacity of the available reservoir sites. In fact, the reservoir sites are so small in relation to the volume of the dams required that they may not be economically feasible until and unless the need for peaking power is greater than at present. The 5 dams discussed in this report, assuming dams of earth-fill type, would have a combined total volume of about 6 million cubic yards and would form reservoirs with a combined total capacity of only 224,000 acre-feet.

The simulated operations shown in tables 2 and 3 assume that 195,000 acre-feet of the stored water would be available for regulation. The remainder would be used for conservation purposes and creation of head. The principal value of upstream storage would be for supplementing Ginger Peak storage. Use of the three largest of the upstream reservoir sites would make it possible to increase winter flows at Ginger Peak by 225 cfs over the amount that could be regulated by Ginger Peak alone.

The regulating capabilities of the reservoir sites are summarized in table 2. Table 3 shows simulated operations during the driest winters of the 1938 to 1952 test period. As previously mentioned, the locations of the reservoir sites are shown in figure 3.

POTENTIAL POWER

The sections on water supply and regulation show that power developments on the Trask River will not be practicable unless the river is regulated by storage reservoirs. Water yield is high, but the basin is small, and stream gradients are so steep that runoff is rapid. There are no spacious reservoir basins available, and except for the extra high-water yield there probably would be few, if any, powersites where development would be economic.

SUGGESTIONS FOR DEVELOPING POWER

Table 4 gives three optional-use suggestions for developing power, as follows:

1. For the maximum continuous regulation using all upstream usable storage plus the average amount of excess power and energy that would have been available between November 1 and March 31 of each water year between 1938 and 1953.
2. With the same storage, the maximum dependable power that would have been available during the five winter months with no power generation during the summer.
3. The maximum dependable power that would have been available during the 5 winter months with one reservoir on the North Fork (Clear Creek), one on the South Fork (Hollywood), and Ginger Peak on the main Trask.

TABLE 2.—*Regulating capabilities of reservoir sites in the Trask River basin*

Site	Reservoir capacity (acre-feet)		Regulated flow (cfs) [1]				Average head (feet)
			Annual		November 1–March 31		
	Gross	Usable	Individual site	Combined [2]	Individual site	Combined	
Clear Creek	39, 000	35, 000	130	130	460	460	165
Keyhole	7, 000	5, 000	40	140	([3])	485	95
Bark Shanty	27, 000	25, 000	145	215	600	700	150
Hollywood	34, 000	30, 000	110	110	450	450	160
Ginger Peak	117, 000	100, 000	395	570	1, 235	1, 510	200

[1] After losses to evaporation×1.2+5 cfs, rounded to nearest 5 cfs.
[2] Includes upstream usable capacity, if any.
[3] Not computed.

TABLE 3.—*Illustrative schedule of operations to produce greatest November–March discharge for the period Nov. 1, 1940, to Oct. 31, 1945, for a Ginger Peak reservoir with usable capacity of 51,000 second-foot-days (101,000 acre-feet)*

[Explanation of operations: I, without benefit of upstream regulation; II, with usable capacity of 17,000 second-foot-days at Clear Creek and 15,000 second-foot-days at Hollywood reservoir sites; and III, including II, with an additional 12,000 second-foot-days at Bark Shanty reservoir site. All figures in second-foot-days]

Date	Inflow	I Ginger Peak		II Ginger Peak, Clear Creek, Hollywood		III Ginger Peak, Clear Creek, Bark Shanty, Hollywood	
		Release [1]	Contents	Release [2]	Contents	Release [3]	Contents
1940							
Oct. 31	--------	--------	51,000	--------	83,000	--------	95,000
Nov	22,762	37,200	36,562	43,500	62,262	45,900	71,862
Dec	32,806	38,440	30,928	44,950	50,118	47,430	57,238
1941							
Jan	49,007	38,440	41,495	44,950	54,175	47,430	58,815
Feb	17,587	34,720	24,362	40,600	31,162	42,840	33,562
Mar	14,170	38,440	92	44,950	382	47,430	302
Apr.–Oct	77,400	26,492	51,000	14,700	63,082	15,700	62,002
Nov	31,339	37,200	45,139	43,500	50,921	45,900	47,441
Dec	78,739	72,878	51,000	50,660	79,000	47,430	78,750
1942							
Jan	25,387	38,440	37,947	44,950	79,437	47,430	56,707
Feb	42,490	34,720	45,717	40,600	61,327	42,840	56,357
Mar	22,300	38,440	29,577	44,950	38,677	47,430	31,227
Apr.–Oct	63,587	42,164	51,000	19,264	83,000	15,700	79,114
Nov	78,350	78,350	51,000	78,350	83,000	62,464	95,000
Dec	85,054	85,054	51,000	85,054	83,000	85,054	95,000
1943							
Jan	47,590	47,590	51,000	47,590	83,000	47,590	95,000
Feb	67,537	67,537	51,000	67,537	83,000	67,537	95,000
Mar	36,057	38,440	48,617	44,950	74,017	47,430	83,627
Apr.–Oct	92,866	90,483	51,000	83,973	83,000	81,493	95,000
Nov	19,606	37,200	33,406	43,500	59,106	45,900	68,706
Dec	41,707	38,440	36,673	44,950	55,863	47,430	62,983
1944							
Jan	34,669	38,440	32,902	44,950	45,582	47,430	50,222
Feb	33,101	34,960	31,043	42,050	36,633	44,370	38,953
Mar	27,119	38,440	19,722	44,950	18,802	47,430	18,642
Apr.Oct	71,549	40,271	51,000	14,700	75,651	15,700	74,491
Nov	23,202	37,200	37,200	43,500	55,353	45,900	51,793
Dec	21,805	38,440	20,367	44,950	32,208	47,430	26,168
1945							
Jan	57,693	38,440	39,620	44,950	44,951	47,430	36,431
Feb	56,197	44,817	51,000	40,600	60,548	42,840	49,788
Mar	63,146	63,000	51,000	44,950	78,744	47,430	65,504
Apr.–Oct	84,186	84,186	51,000	77,930	83,000	54,690	95,000

[1] Includes losses:
1. In winter minimum of 1,240 cfs less 5 cfs losses, net 1,235 cfs.
2. In summer minimum of 60 cfs less 10 cfs losses, net 50 cfs.

[2] Includes losses:
1. In winter minimum of 1,450 cfs less 15 cfs, net 1,435 cfs.
2. In summer minimum of 70 cfs less 20 cfs, net 50 cfs. The summer minimum was exceeded except in 1941 and 1944.

[3] Includes losses:
1. In winter minimum of 1,530 cfs less 20 cfs, net 1,510 cfs.
2. In summer minimum of 75 cfs less 25 cfs, net 50 cfs.

TABLE 4.—*Potential power and energy of the Trask River, with regulation* [1]

Option	Discharge and potential power of Trask River for indicated powersite and average head (feet)											
	Clear Creek (165)		Keyhole (95)		Bark Shanty (150)		Hollywood (160)		Ginger Peak (200)		Total	
	cfs	kw	cfs	kw	cfs	kw	cfs	kw	cfs	kw	kw	kwhr
1. Continuous November–March	130	1, 460	140	900	215	2, 190	110	1, 200	570	7, 750	13, 500	118, 000, 000
Average excess	530	5, 950	570	3, 680	720	7, 340	490	[2] 5, 330	1, 160	15, 780	[3] 38, 080	138, 000, 000
Total												256, 000, 000
2. November–March only	460	5, 160	485	3, 130	700	7, 140	450	4, 900	1, 510	20, 540	40, 870	148, 000, 000
3. November–March only (three dams)	460	5, 160					450	4, 900	1, 435	19, 520	29, 580	107, 000, 000

[1] Regulation at the site and at upstream sites, if any.
[2] Varied at Ginger Peak between a low 5-month average excess of 340 cfs (winter of 1941) to a high average excess of 1,700 cfs (winter of 1946).
[3] The limits of variation would have been between 40,000,000 and 190,000,000 kwhr.

There would be a considerable amount of dump energy possible in any of the options because of the inadequacy of storage for saving water from wet winters for use in summer and for carryover from wet years for use in dry periods. The November–March excess in option (1) is a rough measure of the amount of such energy.

The average discharge at the gage was 942 cfs for the 1938–52 test period. The November–March average was 1,759, and the April–October average was 364 cfs during that period. Given adequate generating capacity, much of the excess winter flow could be utilized.

Figure 3 shows the dam and reservoir sites discussed in this report. The dams shown would develop 940 feet of the 1,500 feet of fall downstream from the Camp Five diversion site. Total head is used to measure the present potential of the various powersites. In computations of capability for producing power, average head is used. The average head for each site is considered to be the difference between the tailrace altitude and that of the water surface when the reservoir has had one-half its usable contents drawn out.

The illustrative development considers dams only as a means of developing head. The illustration is used for estimating the gross power potential and is not a suggested plan. It will probably not be practical to develop all the sites included. A maximum development might be a reservoir and powerplant on each fork and the Ginger Peak reservoir and plant on the main stem as shown in option II of table 3.

The Trask River could make a modest contribution to the Northwest's power network by its capacity to furnish firming power in winter. It is normal for the Columbia to flow at rates greater than average for 4 months, April, May, June, and July, and at rates less than average the remaining 8 months. The Trask's flow rates are higher than average for 5 months and less than average for the remaining 7 months. However, all these 5 high-rate months, November, through March, occur when the Columbia is low. They are also months when demand for power is highest. Figure 8 compares the monthly percentages of average annual discharge for the Trask and Columbia Rivers.

Potential power estimates based on natural flow and regulated flow are shown in table 5 for the 5 sites that have been discussed in this report. The estimates based on natural flow were made in the same manner as that used for measuring potential power in the United States for presentation in table 3 of Circular 367 (Young, 1954). The estimates based on regulated flow are intended to show the dependable power possible of development considerating the regulating capabilities of the site in question supplemented by upstream regulation, if any. All power is computed at 80 percent efficiency.

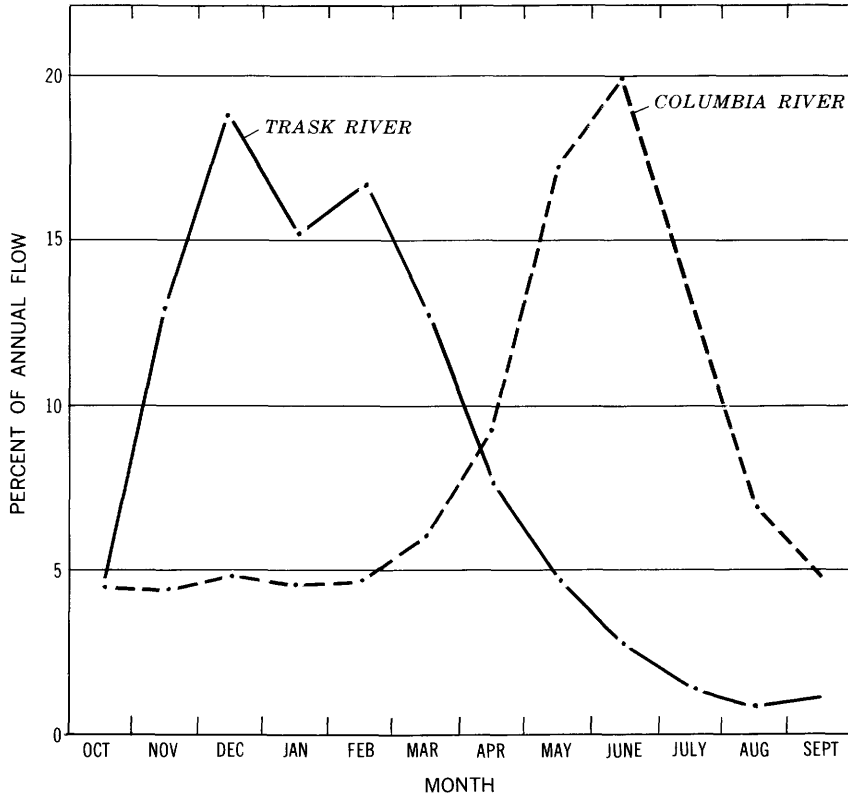

FIGURE 8.—Mean monthly percent of average annual flow for Trask River near Tillamook and Columbia River near The Dalles.

PUMPED STORAGE POSSIBILITIES

Since the coast streams may have their principal value for furnishing peaking power during the high-demand winter months, it is logical to look for pumped-storage sites near the reservoirs to which water could be pumped during the off-peak hours and used to meet morning and evening peaking demands. The Trask River contains such a site. A saddle on Gold Peak ridge at altitude 1,760 feet (in the NE¼ sec. 32, T. 1 S., R. 8 W.) could be turned into a reservoir with a capacity of 400 or 500 acre-feet by constructing 2 dikes, each about 800 feet long by 30 feet high. Water could be pumped from the Ginger Peak reservoir. The lift would vary between 1,410 and 1,690 feet, depending upon the level of the water in the lower and upper reservoirs. The penstock would be about 1 mile long.

Pumps delivering 300 cfs for 16 hours would lift about 400 acre-feet of water, which could be returned through the turbines at a rate of 600 cfs for 8 hours. Assuming an average upper reservoir altitude

TABLE 5.—*Summary of potential power with natural flow and regulated flow*

Site	Natural flow				Regulated flow			
	Percent of time or period	Estimated flow (cfs)	Head (feet)	Power (kw)	Period	Estimated flow (cfs)	Head (feet)	Power (kw)
Clear Creek	95	30	200	410	Continnous	130	165	1,460
	50	180	200	2,450	November-March	460	165	5,160
	Mean	355	200	4,830				
	November-March	530	200	7,210				
Keyhole	95	30	110	220	Continuous	140	95	900
	50	200	110	1,500	November-March	485	95	3,130
	Mean	380	110	2,840				
	November-March	570	110	4,260				
Bark Shanty	95	30	180	370	Continuous	215	150	2,190
	50	250	180	3,060	November-March	700	150	7,140
	Mean	508	180	6,220				
	November-March	720	180	8,810				
Hollywood	95	25	200	340	Continuous	110	160	1,200
	50	170	200	2,310	November-March	450	160	4,900
	Mean	321	200	4,370				
	November-March	600	200	8,160				
Ginger Peak	95	80	250	1,360	Continuous	570	200	7,750
	50	490	250	8,330	November-March	1,510	200	20,540
	Mean	928	250	15,780				
	November-March	1,730	250	29,410				

of 1,775 feet and a tailrace altitude of 100 feet, the average head would be 1,675 feet. The generating capacity at 80 percent efficiency would be 68,000 kilowatts, or 544,000 kilowatthours per day.

The pumped water would require about 1.4 kilowatthours of off-peak energy per kilowatthour of peaking energy (Hammond, 1958, p. 87). For a generating capacity of 68,000 kilowatts operating 8 hours a pumping capacity of 47,600 kilowatts would be required if operated the remaining 16 hours of each day.

An alternative arrangement that would use the same high-level reservoir, power drop and powerhouse could be effected. The water could be pumped from the Hollywood reservoir and carried along the north side of Gold Peak ridge. The lift pipeline would be about 1 mile long and the conduit length to the high-level reservoir, about 3 miles. There would be a saving of about 0.125 kilowatthour pumping energy per kilowatthour of generating energy and 42,000 kilowatts pumping 16 hours would lift the required water. The energy saved is not believed to be sufficient to justify construction of the added lift and conduit works.

DIVERSION TO TUALATIN RIVER BASIN

The Tualatin basin, especially in the areas being developed as Portland suburbs, is approaching a point where it must seek additional water. The Nehalem, Wilson, and Trask Rivers are possible sources for this water, and there are several sites in the headwaters of each where diversions to the Tualatin might be made. It seems possible to divert an average of about 800,000 acre-feet annually from the 3 streams combined. About 235,000 acre-feet could be diverted annually from the Trask River. A small diversion could easily be made by gravity. Diversion of a substantial amount of water will require reservoirs and tunnels or pumping.

CAMP FIVE SITE

Water could be diverted from the Middle Fork North Fork Trask River at a point in sec. 27, T. 1 S., R. 6 W. to the Tualatin River basin. The altitude at the point of diversion is 1,500 feet, and the drainage area is 9 square miles. Average annual runoff at the diversion site during the 15-year period 1938–52 was estimated to be about 35,000 acre-feet with an average discharge rate of 48 cfs. Because of the greater demand for water in the Tualatin basin, such a diversion should be regarded as a possibility. The river could be carried near the 1,500-foot altitude along the right bank of the Tualatin River for about 4½ miles and dropped to an altitude of about 550 feet at Haines Falls in the SE¼ sec. 20, T. 1 S., R. 5 W.

Some degree of regulation of the water could be effected to that a modest power installation might be justified. The winter diversion could average about 90 cfs. This diverted water if dropped through a head of 950 feet would generate nearly 6,000 kilowatts at 80 percent efficiency.

CLEAR CREEK SITE

Planners contemplating diversion of Trask River water to the Tualatin River should investigate the possibilities of diverting it from the Clear Creek reservoir by tunnel or pumping. The reservoir site has a capacity of 30,000 acre-feet above the 800-foot altitude. A tunnel at that altitude would begin at the junction of the North Fork and the Middle Fork North Fork Trask River in sec. 20, T. 1 S., R. 6 W., and would surface on the Tualatin River in sec. 19, T. 1 S., R. 5 W. Its length would be about 5.4 miles, according to the Fairdale 15-minute quadrangle. By carrying the water near the 800-foot altitude along the right bank of the Tualatin River for 5 miles, a head of 500 feet could be created at Cherry Grove in secs. 35 and 36, T. 1 S., R. 5 W. The Clear Creek reservoir could regulate the Trask sufficiently to permit a minimum diversion rate of 90 cfs after losses and

provision for passage of 30 cfs down the Trask River during periods of low water.

The potential power of 90 cfs acting through a head of 500 feet is 3,000 kilowatts at 80 percent efficiency.

Because of the higher value of winter peaking power, the water might be diverted in winter by pumping it to the 1,500-foot altitude from the Clear Creek reservoir, then carrying it along the right bank of the Trask River across the divide and down the right bank of the Tualatin River to the 950-foot drop at Haines Falls described for the Camp Five diversion.

The Clear Creek reservoir with 30,000 acre-feet usable could assure a minimum November through March discharge of 450 cfs. Acting through a head of 950 feet, the potential power would be 29,000 kilowatts at 80 percent efficiency.

If brought across the divide in winter, the water would have to be stored until needed for irrigation and municipal use. An adequate reservoir for that purpose could be constructed on Scoggin Creek.

A dam on Scoggin Creek to raise the water 190 feet in sec. 20, T. 1 S., R. 4 W., would have a crest length of 3,200 feet, according to the Gaston 7½-minute quadrangle. The reservoir would store more than 200,000 acre-feet of water. At a site 1½ miles downstream the same storage capacity could be created by raising the water 175 feet by a dam with a 1,500-foot crest length. The upper site might be a necessary choice, however, because of existing uses of the lands in the lower part of the valley. If the water were dropped at Haines Falls, it could be made to run into the Scoggin Creek reservoir by gravity through a 5-mile canal that would include a short tunnel in sec. 19, T. 1 S., R. 4 W. If dropped at Cherry Grove, the powerhouse tailrace might have to be raised somewhat higher than would otherwise be desirable to make gravity conduit to the reservoir possible. The conduit route would be about 3½ miles long including 1 mile of tunnel.

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Waterpower Resources in Nehalem River Basin Oregon

By L. L. YOUNG *and* J. L. COLBERT

With sections on GEOLOGY OF SITES

By D. L. GASKILL *and by* A. M. PIPER

WATERPOWER RESOURCES OF THE UNITED STATES

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1610-C

An estimate of the potential waterpower of the river and a discussion of possibilities for developing it

UNITED STATES GOVERNMENT PRINTING OFFICE, WASHINGTON : 1965

UNITED STATES DEPARTMENT OF THE INTERIOR

STEWART L. UDALL, *Secretary*

GEOLOGICAL SURVEY

Thomas B. Nolan, *Director*

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington, D.C. 20402

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WATERPOWER RESOURCES OF THE UNITED STATES

WATERPOWER RESOURCES IN NEHALEM RIVER BASIN, OREGON

By L. L. Young and J. L. Colbert

ABSTRACT

The potential waterpower of the Nehalem River is estimated on the basis of the streamflow for the period 1940–54 with complete development within the basin or with diversion of some water to the Columbia River.

The Nehalem River originates near Timber and empties into the Pacific Ocean near Wheeler in northwestern Oregon. The drainage basin is in a mountainous area and is sparsely settled. As is normal for Oregon coastal rivers, its waters are derived principally from rain rather than snow. Precipitation occurs in a seasonal pattern; about 80 percent of the annual runoff is concentrated in the 5-month period of November through March. In comparison with other coastal streams, the Nehalem River is well endowed with natural storage sites for regulating flow. Preliminary examinations indicate that these damsites and reservoir sites are geologically feasible.

The high winter runoff and the availability of storage sites for adequate regulation of the streamflow combine to make the Nehalem River attractive for power development, particularly during wintertime. An illustrative plan of development, including five sites, was considered in estimating the potential power of the stream. Storage sites in the basin could have regulated the streamflow to provide for the continuous generation of 70,000 kw during the period studied. If utilization of the water had been confined to the November 1–March 31 period, 170,000 kw could have been generated. The theoretical potential power of the stream, based on average discharge and gross head, is 97,000 kw. This potential, however, would require complete regulation of the streamflow, which is not possible.

Another way to use the Nehalem River for power production would be to divert it to the Columbia River. About half of the flow now passing the Foss gage could be diverted to a powersite on the Columbia River near Woodson. The potential power of a diversion by gravity at an altitude of 540 feet above sea level is about 51,000 kw, continuously. If the water were pumped into a higher level reservoir and diverted through a shorter tunnel at an altitude of 800 feet, a large block of peaking power would be available. Pumped-storage development at other sites is also possible.

Because of its proximity to the growing metropolitan area of Portland, any study of the water resources of the Nehalem River basin must include the possibility that water will be diverted to the Portland area. Indications are that an average of about 400 cubic feet per second could have been diverted for use in the Portland suburban area in the Tualatin River basin during the period studied.

INTRODUCTION

PURPOSE AND SCOPE

This report presents the results of studies of water-power resources made by the Geological Survey in the Nehalem River basin. The basic data used are topographic quadrangles of the basin, a special map of the river, large-scale damsite surveys, geologic reports, and stream-flow records, all gathered by the Geological Survey. In addition, precipitation and evaporation records of the U.S. Weather Bureau and an isohyetal map, prepared by the Corps of Engineers U.S. Army and showing rainfall intensities and distribution in western Oregon, were used in the study.

The contribution that the river might make to the Pacific Northwest power network was examined and found to be significant, particularly in supplying peaking power during the winter months when the Columbia River network will need it. Sites for developing reservoirs for streamflow regulation are discussed and preliminary geologic examinations of them are reported. Only those sites that might reasonably be expected to become economically feasible in the future are included; however, cost studies were not made.

The potential power of the Nehalem River is estimated by considering probable plans of development entirely within the basin, and an alternative plan is discussed in which the Nehalem River water would be diverted to the Columbia River by gravity or by pumped storage.

As power projects become otherwise feasible, problems relating to fish and wildlife and to population relocations will remain. These obstacles are not necessarily insurmountable, however. Opposition to power developments is lessened by improved methods of getting fish over dams and otherwise protecting them from harm by project works, by added recreational benefits afforded by the lakes, by the construction of new and better roads in the vicinity of power projects, and by adequate compensation to dislodged property owners.

PREVIOUS INVESTIGATIONS

E. C. LaRue and E. C. Murphy, engineers of the U.S. Geological Survey made investigations that included the Nehalem River. Mr. LaRue's study was made in 1917 to evaluate the water resources of lands forefeited by the Oregon and California Railroad Co. Mr. Murphy studied the coast streams of Oregon in 1923 to determine

which additional lands in addition to those forfeited should be reserved for waterpower purposes. The reports on these investigations were not made available to the public.

The following reports on the Nehalem River have been placed in the U.S. Geological Survey open file:

12–R–20 Report on potential waterpower of Nehalem and Wilson River basins, Oregon, by B. E. Jones and Warren Oakey, 1924, 16 pages.

12–R–33 Geologic features of damsites in Nehalem, Rogue, and Williamette River basins, Oregon, by A. M. Piper, 1937, 111 pages.

12–R–34 Water utilization within the Nehalem River basin, Oregon, by R. O. Helland, *with* Geology of damsites by A. M. Piper, 1937, 41 pages.

12–R–43 Waterpower of the coast streams of Oregon, by R. O. Helland, 1953, 46 pages.

Power companies and utility districts gave little thought to the possibilities of developing the power potential of the Nehalem River in the past. Recently, however, the Tillamook County Public Utilities District considered a development on the river and in 1959 had the firm of Cornell, Howland, Hayes, and Merryfield, Consulting Engineers, Corvallis, Oreg., make a study of waterpower possibilities. The firm's findings were submitted to the utility district in a report entitled "Report of Reconnaissance Study of Hydroelectric Development of Nehalem River." The utility district has suspended its plans for the Nehalem but has applied for and has been granted, a preliminary permit by the Federal Power Commission, project 2274, to investigate the Trask River.

MAPS RELATING TO THE AREA

A survey of the Nehalem River was completed by the U.S. Geological Survey in 1936 and published in 1938 under the title "Plan and Profile of Nehalem River from Mohler to Timber, Oregon, and Tributaries, Miscellaneous Damsites." That survey covered 102 miles of the river between sec. 36, T. 3 N., R. 10 W., near Mohler, to sec. 22, T. 3 N., R. 5 W., near Timber. It consists of seven sheets (four plan and three profile) at a scale of 2 inches=1 mile. The contour interval of the river-survey map is 20 feet for land and 5 feet for water. Five damsites were surveyed at a scale of 1 inch=400 feet with contour interval of 10 feet. The following damsite surveys were published on plan sheets of the river survey:

773-641 O—65——2

Damsite	At mile—	Location			Topography above river (ft)
		Section	T. N.	R. W.	
Tideport	36.9	23, 24	5	7	210
Elsie	30.4	4	4	7	250
Salmonberry	15.5	10	3	8	240
Nehalem Falls	8	23, 27	3	9	280
Stonehill	4.9	34	3	9	80

In 1957, additional surveys were made in the basin by the U.S. Geological Survey and the resulting map of a reservoir site on Fishhawk Creek and a tunnel route to the Columbia River entitled "Fishhawk Creek Reservoir and Damsite, Oregon," was published in 1957 at a scale of 1 inch=2,000 feet (1:24,000) with a contour interval of 20 feet. The 1957 surveys included the following damsites surveyed at a scale of 1 inch=400 feet with contour interval of 10 feet:

Damsite	At mile—	Location			Topography above river (ft)
		Section	T. N.	R. W.	
Fishhawk Creek		29	7	7	300
Squaw Creek	45.6	33	6	6	150
Upper Wakefield	14.4	15	3	8	430

The Squaw Creek and Upper Wakefield damsite maps were published in 1957 by the U.S. Geological Survey together with other damsite surveys under the title "Plan, Miscellaneous Damsites, Coast Streams, Oregon."

U.S. Geological Survey 15-minute quadrangle maps at a scale of 1:62,500 (approximately 1 inch=1 mile) cover the entire basin; they are listed below with the date of the last field check:

Quadrangle	*Date*	*Quadrangle*	*Date*
Birkenfeld	1955	Nehalem	1955
Cannon Beach	1955	Saddle Mountain	1955
Cathlamet	1953	Sensen	1955
Clatskanie	1952	Timber	1955
Enright	1955	Vernonia	1955

GENERAL DESCRIPTION OF THE BASIN

PHYSICAL CHARACTERISTICS

The Nehalem River drains an area of about 850 square miles of mountainous timberland in northwestern Oregon. The river heads near Cochran, 40 air miles northwest of Portland, in the Coast Range in mountains over 2,000 feet high. Its course is east to Timber, thence northeast to Vernonia and Pittsburg, northwest to Mist, west to Birkenfeld, southwest to Jewell, Tideport, Elsie, Batterson, and Foss. It

flows west and northwest past Foss to Mohler, meanders in large S-curves in a broad valley, passes Nehalem and Wheeler, and empties into Nehalem Bay. The mouth of the bay is about 5 miles southwest of Wheeler and about 65 air miles northwest of Portland. The basin is outlined by a broken line on figure 1.

The Nehalem River basin is bounded on the west by the basins of the Necanicum River and of other small streams which empty directly into the Pacific Ocean. The ridge line between the basins here averages about 1,900 feet in altitude. The lowest pass in the ridge is 720 feet and the highest feature (Onion Peak) is 3,064 feet in altitude. U.S. Highway 26 (the Sunset Highway) between Portland and the coast leaves the basin by a pass about 1,100 feet in altitude.

On the northwest, north, and east the adjacent drainage is all to the Columbia River. The adjacent basins are those of the Youngs and Klaskanine Rivers on the northwest, Clatskanie River on the north, and Scappoose Creek on the east. The average altitude of the divides is about 1,600 feet. Saddle Mountain (3,283 ft), Nicolai Mountain (3,020 ft), Clatskanie Mountain (2,680 ft), and Bunker Hill (2,040 ft) are among the higher peaks. The Tidewater Summit Pass on the Nehalem Highway (State Highway 202) is 1,221 feet high. The lowest altitude on these divides is 1,200 feet.

On the southeast, Tualatin River drainage adjoins for a short distance. The average altitude of the dividing ridge line is over 1,800 feet and there are several peaks over 2,000 feet in altitude. The lowest pass is 880 feet; it is on the Timber to Glenwood road (State Highway 8). U.S. Highway 26 enters the basin through a pass about 1,360 feet in altitude. The highest part of the Nehalem basin rim is on the south between it and the Wilson River basin. The average altitude there is 3,080 feet. The higher peaks in the reach are Larch Mountain (3,449 ft) and Lookout Mountain (3,510 ft). The lowest pass is 1,520 feet in altitude, but most are over 2,700 feet. The rest of the south and the southwest boundary adjoins drainage into Tillamook Bay by way of the Kilchis and Miami Rivers. The average altitude here is 2,100 feet and the extremes are Kilchis Peak (2,294 ft) and sea level. A pass near the coast has an altitude of 250 feet.

Altitudes in the interior of the basin are comparable with those on the ridge line, and, except for a rather large valley between Vernonia and Elsie, the canyons are narrow and deep. Foley Peak, with an altitude of 2,275 feet, is only 5 air miles from the ocean. The Nehalem River is still actively cutting through a plateau that lies diagonally between Saddle Mountain to the northwest (20 air miles from the coast) and Larch Mountain to the southeast (25 to 30 air miles from the coast). The river enters the plateau region near Elsie, about 20 miles

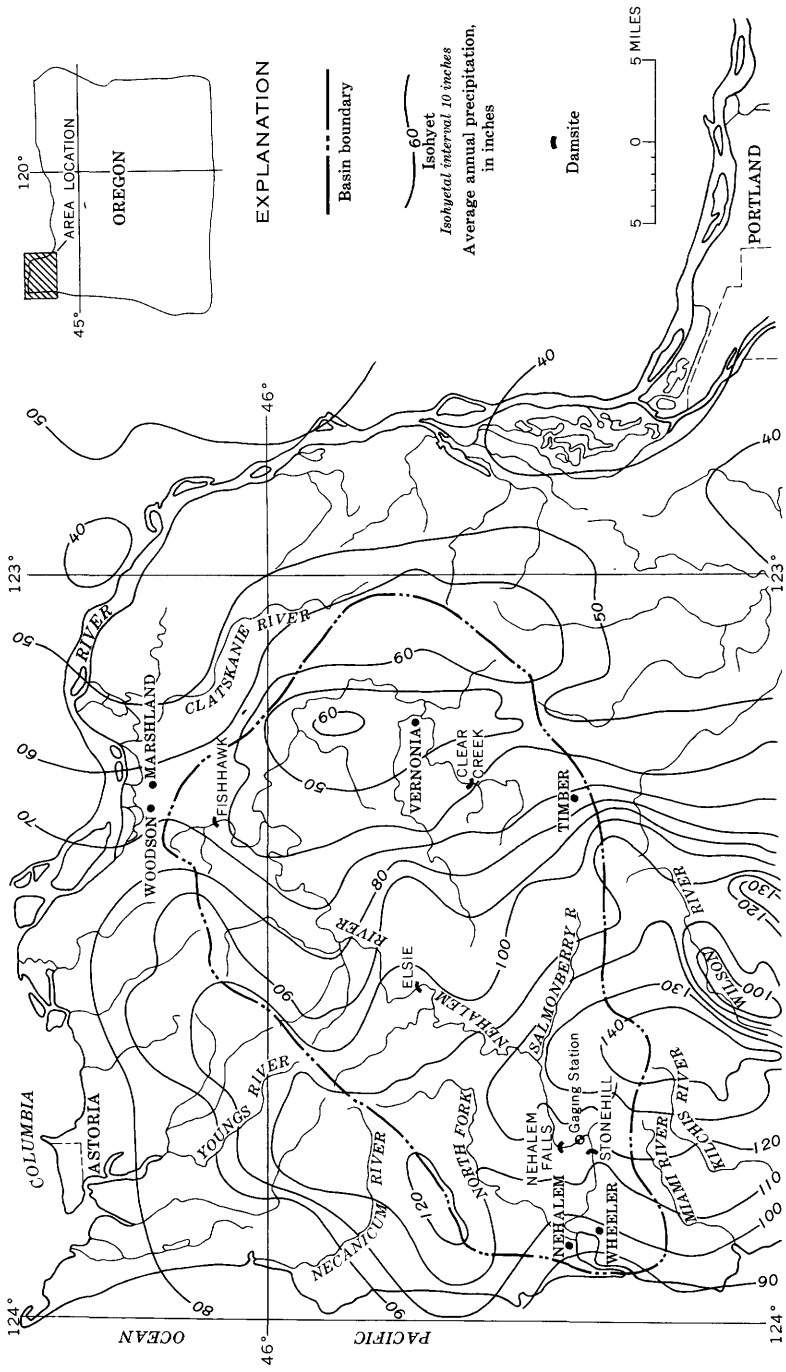

FIGURE 1.—Nehalem River basin, Oregon, with isohyets.

by air from the coast. The lower end of Nehalem Falls, at the west end of the active reach is only 8 miles from sea level.

ECONOMICS

The first paragraph under physical characteristics names the towns and most of the settlements in the basin. Vernonia, Columbia County, and Nehalem and Wheeler, Tillamook County, may be classed as towns. There are post offices at Timber, in Washington County, Mist and Birkenfeld, in Columbia County, Jewell, in Clatsop County, and Mohler, in Tillamook County.

The population of the basin has declined an estimated 18 percent since 1950. On July 1, 1960, Vernonia had a population of 1,095, a decline of 28 percent since 1950; Nehalem had a population of 244, a decrease of 16.2 percent; and Wheeler had a population of 230, a decrease of 14.8 percent (Oregon State Board of Census, 1960). The populations of the smaller communities and the number of farms appear to have remained much the same as they were in 1950.

Settlement of the Nehalem basin began about 1867 and some 150 people were there by 1877. There was little growth until 1920. Vernonia was named in 1876 for Vernonia Cherrington (Oregon Historical Society, 1941), the daugther of the first schoolteacher. The area is served with electric power by the R.E.A., West Oregon Electric, which is connected with Bonneville generating facilities. The Spokane, Portland and Seattle railway serves Vernonia and a 60-mile-long valley which includes the localities of Timber, Pittsburg, Mist, Birkenfeld, Jewell, and Tideport. The southern Pacific railroad line from Portland to Tillamook follows the Nehalem River upstream from Timber, then the Salmonberry and Nehalem Rivers to Wheeler.

A surfaced highway follows the river from Vernonia to Elsie and low-class roads provide access to the remaining reaches of the stream and branch into the mountains to afford access to timber and for fire protection. The main highway between Portland and the coast, U.S. 26, crosses the basin and would be affected by construction of the Rocky Point and Elsie dams. The Rocky Point reservoir would flood about a mile of the present highway, and a new bridge might be necessitated at Elsie.

The pioneers found the Nehalem basin covered by a dense forest that was difficult to penetrate. It was the exploitation of these abundant timber resources in the 1920's that brought the greatest number of people to the area; Vernonia is said to have been a town of over 2,500 population during that period (St. Helens Centennial, 1954).

Lumbering has been, and probably will continue to be, the principal industry. However, there is no longer sufficient timber to maintain

the former rate of production, and operations at the principal mills in the area are being curtailed. A reduction of operations at the International Paper Co.'s Long Bell mill at Vernonia is the principal cause of the decrease in the town's population. Other occupations in the basin are dairying, growing small fruits (berries), and diversified farming.

The basin has good possibilities for improvement of existing outdoor recreational activities. The river is fished for trout, steelhead, and salmon and there is a 9-hole golf course at Vernonia. If dams are constructed on the river, facilities for fish passage should be included. The new reservoirs would be beneficial to these recreational values.

WATER SUPPLY

PRECIPITATION

The U.S. Weather Bureau has records of precipitation at Vernonia for 24 years, and at Jewell Guard Station and Nehalem for short and intermittent periods. Stations of the Weather Bureau outside the basin at Seaside, Astoria, Clatskanie, and Tillamook, maintain precipitation records, which are indices of precipitation for the general area. In addition, the Corps of Engineers, U.S. Army, has prepared an isohyetal map of western Oregon that includes the Nehalem basin. From these data, a new isohyetal map was prepared that conforms closely to the topography of the basin as shown by quadrangle maps. The new map was used in estimating runoff by measuring the areas between the several isohyets for the drainage basin units under consideration. The isohyets of the map are approximated on the Nehalem River basin map (fig. 1), and the rainfall averages determined are shown in table 1.

The headwaters area of the basin is on the lee side of the Coast Range and therefore receives less precipitation than the western, or windward, slope. Annual precipitation varies from a minimum of about 50 inches in the eastern part to over 140 inches in the mountains in the southwestern part of the basin. The entire basin has the usual coastal-type distribution, heavy in the fall and winter and light in the spring and summer.

RUNOFF

Records of discharge of the Nehalem River at a gaging station near Foss where the drainage basin area is 667 square miles are complete from October 1939. A 3-year record of discharge (1926–28) was obtained for Rock Creek near Keasey, and records of miscellaneous measurements are available including those on the North Fork Nehalem River and on Salmonberry River. The difference between the

TABLE 1.—*Area, estimated average annual precipitation, and estimated average annual runoff at selected sites in Nehalem River basin*

Site	Drainage area (sq mi)	Estimated average annual precipitation (inches)	Estimated average annual runoff		Percent of runoff at Foss gage
			Inches	Acre-feet	
Rocky Point damsite	70	71	45.5	171,000	8.9
Rock Creek gage	38	81	61.0	123,000	6.3
Fishhawk damsite	11	67	41.5	23,650	1.2
Squaw Creek damsite	398	63	37.5	795,950	41.4
Tideport damsite	463	67	41.5	1,023,000	53.2
Elsie damsite	498	68.5	43.0	1,140,000	59.3
Spruce Run damsite	549	72	46.5	1,360,000	70.6
Salmonberry damsite	573	73.5	48.0	1,462,000	76.0
Upper Wakefield damsite	644	78	52.5	1,800,000	93.5
Wakefield damsite	647	78	52.5	1,810,000	94.0
Nehalem Falls damsite	660	79	53.5	1,877,000	97.4
Foss gage	667	79.5	54.0	1,923,000	100.0
Stonehill damsite	700	82	56.5	2,105,000	109.3
Gods Valley damsite	45	117	91.5	220,000	11.4
North Fork Nehalem Basin	96	114.5	89.0	457,000	23.7
Nehalem River Basin	850	87	61.5	2,788,000	145.0

estimated precipitation average and the measured runoff depth for the drainage area above the Foss gage was 25.5 inches. That amount was subtracted from the precipitation to obtain the runoff depth at each of the damsites considered in the section on regulation and storage sites.

The loss apparently varies throughout the basin, because a similar comparison of measured runoff at Rock Creek with the estimated precipitation indicates a loss of about 20 inches annually. However, the 3-year record on Rock Creek shows considerable variation from year to year and the indicated loss of 20 inches is probably not reliable because of the short period of record. The miscellaneous measurements of runoff on the North Fork Nehalem River also indicate that losses between rainfall and runoff may be less than 25.5 inches for this part of the basin. An annual loss of 25.5 inches is considered a conservative estimate for any subpart of the basin. Estimated annual runoff at the selected damsites and at other locations are shown in table 1. A comparison of the runoff at the Foss gage with precipitation at Vernonia in the upper part of the basin indicates a lag of 1 to 2 months between winter precipitation and runoff. On the average, the highest monthly percentage of annual precipitation occurs in December and the highest monthly percentage of annual runoff occurs in February. Lowest percentage of annual precipitation is in July and lowest percentage of annual runoff is in August.

A flow-duration analysis of the Nehalem River near Foss for the water years 1940–54 shows $Q95$ (discharge in cubic feet per second that is equaled or exceed 95 percent of the time) = 107 cfs (cubic feet per second) (0.16 cfs per sq mi), $Q50$ = 1,100 cfs (1.65 cfs per sq mi), and Q mean = 2,656 cfs (3.98 cfs per sq mi). The seasonal distribution of this runoff shows heaviest runoff during the period November

through March and very light runoff from July to the end of September. The average monthly discharge in cubic feet per second per square mile and in percentages of total runoff are shown below:

Month	Discharge (cfs per sq mi)	Percent of total runoff	Month	Discharge (cfs per sq mi)	Percent of total runoff
October	1.30	2.77	April	3.64	7.51
November	5.27	10.87	May	1.98	4.22
December	9.15	19.51	June	.843	1.74
January	8.82	18.80	July	.397	.85
February	10.50	20.38	August	.216	.46
March	5.77	12.29	September	.293	.60

This runoff pattern, modified to reflect wet and dry years as shown in figure 2, was used for making estimates throughout the basin.

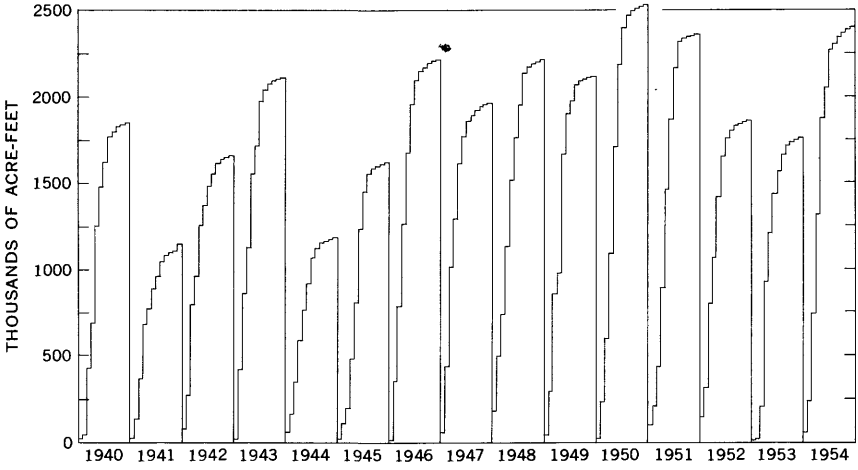

FIGURE 2.—Runoff in acre-feet accumulated monthly for water years 1940–54 for gage near Foss.

REGULATION AND STORAGE SITES

The distribution of annual runoff is such that reservoirs are necessary to regulate the flow by reducing the high winter runoff and increasing low summer flows. There are no reservoirs in the basin at present but there are a number of topographically suitable reservoir sites. Land acquisition, highway and forest road relocations, and the necessity for fish protection are factors which would tend to limit the construction of reservoirs.

To compute the storage requirements of the sites, the estimated total water yield at each damsite was distributed over the period studied by adapting the mass diagram and accumulated second-foot-

day table for the Foss gage to the particular site. In other words, the Foss gage provides the runoff pattern.

Estimates of evaporation losses were made from Weather Bureau records of evaporation rates in Oregon. An average loss in inches was determined for each month and these were totaled appropriately and multiplied by the surface of the reservoirs when they contained one-half their usable capacities to get the evaporation loss for yearly, winter, and summer periods. An additional allowance equal to 10 cfs plus one-fifth the annual evaporation loss rate was made for leakage and other unmeasurable losses. The total loss was rounded to the nearest 5 cfs.

During reservoir-filling periods or during times when diversions from the basin were being made, a conservation release equal to the flow at the sites 95 percent of the time for all reservoirs from Tideport upstream and 50 cfs for downstream reservoirs was assumed. The losses and conservation-release data for Rocky Point are used for the Gods Valley reservoir site on the North Fork Nehalem River. The dam and reservoir sites discussed below are shown on the river profile in figure 3.

ROCKY POINT RESERVOIR SITE

The Rocky Point damsite is at river mile 92.8, about 10 miles upstream from Vernonia, in sec. 23, T. 4 N., R. 5 W. The area of the drainage basin is 70 square miles and the water surface altitude at the site is 667 feet. An earthfill dam for raising the water surface to the 800-foot contour would have a volume of about 600,000 cubic yards, and would store 114,000 acre-feet. The reservoir would be 8.5 miles long and would have a surface area of 2,300 acres. Eight miles of State Highway 8 and a mile of U.S. Highway 26 would have to be relocated for a reservoir that has a maximum water surface altitude of 800 feet. The capacity of the reservoir at altitude 800 feet would be sufficient to store two-thirds of the estimated annual runoff at the damsite. Storage required to sustain the average annual discharge is about 250,000 acre-feet; this discharge would require a dam to the 850-foot altitude. The area and capacity data for the Rocky Point reservoir site are shown in table 2.

TABLE 2.—*Area and capacity of Rocky Point reservoir site, damsite at mile 92.8, secs. 22 and 23, T. 4 N., R. 5 W.*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
667	0	0	780	1,720	74,000
680	25	150	800	2,300	114,200
700	210	2,500	820	2,700	164,000
720	490	9,500	840	3,100	222,000
740	820	22,600	860	3,800	291,000
760	1,300	43,800	880	4,600	375,000

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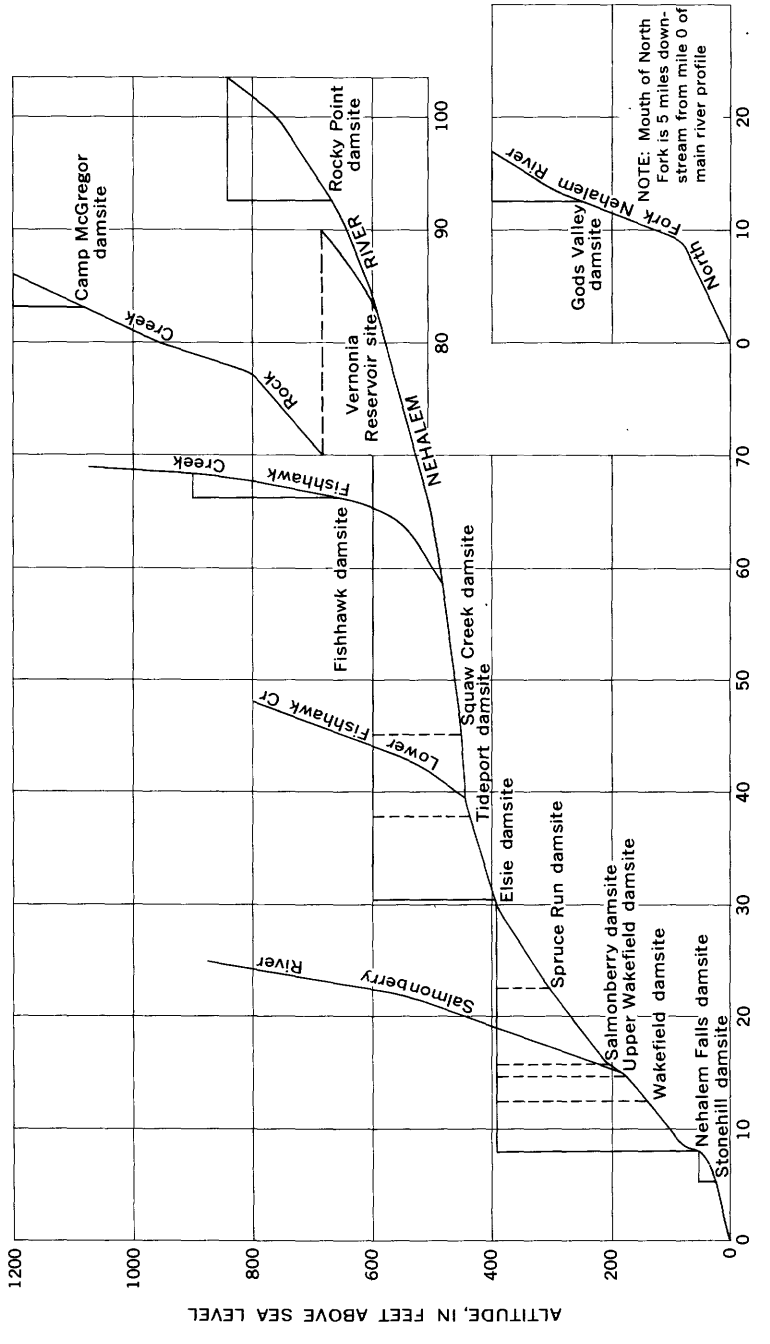

FIGURE 3.—Profile of Nehalem River showing prospective damsites and storage basins.

A reservoir with 114,000 acre-feet of active storage would have regulated the flow to 435 cfs after losses during the 5 winter months and 10 cfs after losses during the remaining 7 months of the year in the period studied, or a continuous discharge of 185 cfs after losses could have been maintained. A 250,000 acre-foot reservoir might be desirable if water is to be diverted to the Tualatin River basin. The larger reservoir would afford complete control of the stream and would shorten the tunnel length or the pumping head, depending upon the method chosen for diversion.

A possible use of water stored in a reservoir at the Rocky Point site might be for irrigation. The amount of irrigable land and possible water requirements have not been investigated. A reservoir of 100,000 acre-feet at the Rocky Point site would have provided a minimum of 355 cfs during a 4-month period June through September for all of the 15 years of record. Most of the time this irrigation-season flow would have been in excess of 400 cfs and there would have been substantial amounts of uncontrolled flow during years of above average flow.

A regulating dam at the Rocky Point site would be very beneficial to the downstream river basin. It could be so operated as to increase low flows and thus enhance the value of the river for fish, and farm and pasture lands between Vernonia and Elsie could be irrigated by releasing water during the dry season. If used for power or for diversion of water out of the basin, the other benefits would be reduced.

Rocky Point storage is not reflected in the data for any of the downstream sites because the purposes for which this reservoir is likely to be built may detract from, rather than add to, downstream power. The damsite has not been examined geologically.

VERNONIA RESERVOIR SITE

The Nehalem River has a rather flat gradient from Elsie, mile 30, to Vernonia, mile 83. This section is designated as the Vernonia reservoir site in this report. The maximum altitude for the development of this site would likely be 600 feet, to avoid inundation of Vernonia. Three alternative damsites are considered in this report for the development of the Vernonia reservoir site: (1) Squaw Creek near mile 46, (2) Tideport near mile 37, and (3) Elsie near mile 30. The Elsie damsite has probably been given the most consideration in development plans for this section of the river.

SQUAW CREEK DAMSITE

GENERAL FEATURES

The Squaw Creek damsite is at Nehalem River mile 45.6 in sec. 33, T. 6 N., R. 6 W., about three-quarters of a mile upstream from the

confluence of Squaw Creek. Drainage area at the site is 398 square miles and the water surface altitude is 450 feet. An earthfill dam 150 feet high that would raise the water surface to the 600-foot altitude would have a volume of about 1,500,000 cubic yards and create a reservoir of about 700,000 acre-feet capacity. By comparison with the runoff at the gage at Foss, the estimated average runoff at the site is 1,100 cfs. Control of this amount of runoff would require a reservoir of 1,160,000 acre-feet. Storage of this volume would require a dam about 30 feet higher. Such a dam is not considered to be feasible at this site. Area and capacity data for this reservoir site are shown in table 3.

TABLE 3.—*Area and capacity of Vernonia reservoir site, with dam at Squaw Creek site, mile 45.6, sec. 33, T. 6 N., R. 6 W.*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
450	0	0	540	4,100	132,000
460	80	400	560	8,700	260,000
480	500	6,000	580	10,500	453,000
500	1,700	28,000	600	13,950	700,000
520	2,300	68,000			

A dam at Squaw Creek is considered only because a reservoir smaller than one formed by a dam at a downstream site might be better suited for the Fishhawk-Tunnel Ridge diversion to the Columbia River; a dam here is not considered justified for any other purpose. Usable capacity of the reservoir would be 570,000 acre-feet (the water stored above an altitude of 540 feet, the probable elevation for a diversion tunnel entrance). An advantage of the Squaw Creek site over downstream sites is that farms around Jewell and between Tideport and Elsie would not be affected.

Assuming a usable capacity of 570,000 acre-feet in the Vernonia reservoir behind Squaw Creek dam, a minimum flow of 870 cfs could have been diverted during the period studied. This minimum flow would have been met easily after the end of the critical period, April 1940 to January 1947. During spilling periods between January 1947 and April 1954, flow averages had a range of 1,450 to 2,650 cfs.

The 570,000 acre-feet of storage could be used to regulate flows released during the 5-month winter period November through March and thereby assure a minimum flow of 2,190 cfs. For this regulation the critical period was between November 1, 1940, and March 1, 1944. The reservoir would have been nearly empty on March 1, 1944, and would have refilled in 1948. There would have been excess water thereafter except for the drawing periods ending March 31, 1952 and 1953. The greatest excess would have been 190,000 acre-feet in the November 1949 to March 1950 period.

In computing the divertible water in the above examples, a minimum discharge of 40 cfs ($Q95$) was left in the river for conservation purposes and the appropriate losses were subtracted.

GEOLOGY

By D. L. Gaskill

At the Squaw Creek damsite (pl. 1), the Nehalem River cuts strata of the Pittsburg Bluff Formation (Warren and Norbisrath, 1946). These strata consist of alternating well-stratified micaceous thinly laminated to thick-bedded silty claystone, siltstone, and silty to fine-grained sandstone. The rocks are composed of subangular to subrounded grains of quartz, minor feldspar, and some tuffaceous material. Most of the finer grained beds are firmly compacted but contain little or no cementing material. A few thick beds of sandstone are well cemented with calcite. Crossbedding, ripple marks, mudcracks, carbonaceous material, calcareous clay and iron concretions, and other sedimentary features are present locally. The sedimentary beds probably grade laterally from one type to another within short distances. Fresh angular blocks of dark-gray dense very fine grained basalt form part of the slope wash on the east abutment of section *B–B′* (pl. 1). The basalt does not crop out in the damsite area and is probably derived from a dike or flow remnant near the crest of the ridge.

Bedrock exposures occur in the river channel at section *A–A′*, on the west bank at locality 3, and in highway and logging roadcuts. Thick flood-plain deposits of coarsely graded sand, gravel, and alluvium mantle most of the valley floor.

Structure of the bedrock is largely obscured by vegetation, deep weathering, and surficial movement. Large areas of valley and abutment slopes are slumping, and wet clay zones are exposed at most outcrops. Bedding attitudes vary, probably owing to bedrock slump, large-scale crossbedding, or possible faulting. The attitude of some resistant sandstone beds on valley slopes subject to surficial movement suggests a moderate downstream dip of the strata, whereas less competent strata in the river bed near section *A–A′* dip obliquely upstream into the east abutment. Beds are conspicuously fractured at locality 1 (pl. 1), and a few widely spaced joints cut bedrock near section *A–A′*. These fractures may be associated with a zone of minor folds and contorted bedding that trends about N. 70° W. across the river here. Aerial photographs reveal a few widely spaced lineaments that possibly represent faults or bedrock fractures. One lineament strikes about N. 30° E. through the damsite area.

The sedimentary beds exhibit a wide variation in texture, composition, and minor structures. They are generally weak, in part plastic when wet, and are subject to differential compaction, expansion, slumping, slaking, and disaggregation. Foundation and abutment rocks may fail along weak water-saturated bedding planes. The instability of abutment slopes is shown by the frequency of slumps and landslides, which are due in part to deep weathering and general absence of cementing material in bedrock. Bedrock dips slightly upstream at section *A–A′* in the river channel, but other bedrock exposures are probably influenced by some degree of surficial movement. The zone of folding in foundation bedrock at section *A–A′* should be investigated. Other areas of indicated structural weakness, as at locality 1, and the hypothetical lineament at locality 2 require exploration. Leakage should be anticipated through some friable sandstone or along bedding planes. Abutments will probably require deep excavation of slumped and loosely consolidated weathered rock. Some deep-seated sliding and creep of surface mantle into the reservoir area may be expected, particularly where valley slopes have been denuded of forest cover. A chute-spillway site at section *A–A′* might be developed on the west abutment. A spillway at section *B–B′* would probably have to be a side channel tied to the east end of a dam here. Spillways would have to be concrete lined. An impervious cutoff wall would probably be necessary along the top of the west abutment at section *A–A′*, dependent on reservoir level. Attitude and permeability of beds are particularly critical on the west abutment at section *A–A′*.

Impervious materials and aggregate are probably available from alluvial deposits in the Nehalem River valley. Nonfragmental and coarse fragmental volcanic rock is exposed in the Nehalem River valley near Jewell, about 8 miles downstream.

In conclusion, the Squaw Creek damsite has little to recommend it aside from topography, but it is probably adaptable to a flexible wide-base earth or rockfill structure.

TIDEPORT DAMSITE

GENERAL FEATURES

The Tideport damsite is at river mile 37, secs. 23 and 24, T. 5 N., R. 7 W., about half a mile upstream from the community of Tideport. The drainage area is 463 square miles. An earthfill dam that would raise the water surface from its present altitude of 437 feet to 600 feet would have a volume of about 1,500,000 cubic yards and store 1,060,000 acre-feet of water. Storage required for complete regulation to an estimated mean flow of 1,415 cfs is about 1,500,000 acre-feet, which would require a dam to near the 620-foot altitude. A dam to that height has

not been considered because of the altitude of the town of Vernonia. Area and capacity data for this reservoir site are shown in table 4.

TABLE 4.—*Area and capacity of Vernonia reservoir site, with dam at the Tideport site, mile 37, secs. 23 and 24, T. 5 N., R. 7 W.*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
437	0	0	520	5,000	141,000
440	40	200	540	7,000	261,000
460	420	5,000	560	12,000	445,000
480	1,380	23,000	580	15,000	716,000
500	2,700	64,000	600	19,600	1,063,000

The reservoir would provide about 70 percent of the storage required for complete regulation of the stream whereas the Squaw Creek site would provide only 60 percent of the storage required at that site. Lands that would be flooded by a dam at the Tideport site are perhaps more valuable than those farther upstream. A dam at Tideport of necessity would be higher but would have about the same volume of material as one at Squaw Creek. During the period studied, the reservoir could have assured a discharge of about 1,240 cfs if its entire contents were used as active storage. The reservoir would have an active capacity of 800,000 acre-feet above altitude 540 feet. This capacity could have provided a constant diversion of about 1,185 cfs to the Columbia River. If all the water were diverted during the 5-month period November through March, a uniform flow of 2,900 cfs could have been maintained.

The conservation release at Tideport is 50 cfs.

GEOLOGY

By A. M. PIPER

The bedrock at and near the Tideport damsite consists of thinly stratified shale and earthy sandstone overlain by dense nonfragmental volcanic rock (basalt) and fragmental volcanic rocks (basaltic tuff and agglomerate). In general, the rocks dip 5°–35° in a N. 20°W. to N. 20° E. direction. At the damsite, the exposures of bedrock are scarce and geologic features must be interpreted largely from regional features. A geologic map of this site is shown on plate 1.

At the site, dense volcanic rock is exposed at the base of the west abutment to a height of about 40 feet above a roadcut and for about 350 feet along the road. The exposed rock includes a layer of dense fine-grained basalt about 45 feet thick; another layer of basalt about 40 feet thick contains small globular masses of secondary nonmetallic minerals (amygdule fillings), and fine-grained fragmental material (tuff). These layers seem conformable and contacts are tight. These

rocks are inferred to form the greater part of the thin spur above the exposure that extends to the margin of the mapped area. Similar volcanic rock is inferred to underlie the opposite, or left, bank of the river.

The dense nonfragmental volcanic rock exposed at the Tideport damsite has high crushing strength; however, it probably will not form a rigid and thoroughly stable foundation at the right abutment, because the rock is highly fractured and partly altered. The exposure described in the preceding paragraph is crossed by two large fractures that strike N. 20° E. and dip about 70° westward. Between these two fractures all the rock is cut by closely spaced cross-fractures that strike in many directions; some of the rock is crushed thoroughly. In this fracture zone, many of the blocks are rotated rather steeply and their faces commonly are partly decomposed although some faces are silicified. All these features indicate that the exposed rocks are in a fault zone of appreciable displacement. The fault that strikes N. 20° E. locally controls the course of the river farther upstream. This fault suggests a line of serious weakness at the damsite. Other fracture zones may be concealed beneath the slope wash.

Coarse fragments of volcanic rock 2 feet across in a dense basaltic matrix form extensive outcrops on the left bank of the river downstream from the damsite. This rock is likely to occur at the site beneath the cover of slope wash and is moderately weaker than the nonfragmental volcanics.

The sedimentary rocks in the vicinity of the damsite are largely shale but include some earthy sandstone beds. These rocks are moderately low in bearing power and are probably plastic when wet. At or near the site these weak rocks are exposed on the right (west) bank at three places: (1) along the road at the downstream edge of the basalt outcrop described above, where they dip 70°–85° W. and strike N. 70° W. and are deeply weathered and air-slaked; (2) at an altitude of about 750 feet along the thin spur forming the west abutment; and (3) in shallow cuts along the road 1,300 to 1,600 feet upstream from the site where they dip 20°–35° N. 20° W. They probably form most of the right bank both upstream and downstream from the basaltic spur. The upper hillsides in these areas are scarred by several landslides. The scars are shallow and only a few yards across. Whether sedimentary rocks occur on the opposite, or left, bank is not known.

The line A–B on the map of the Tideport damsite (pl. 1) indicates the most feasible position for a dam having its foundation and abutments largely or entirely on the volcanic rocks. At this position the permissible maximum height of a dam probably is limited by the bearing power and perviousness of the narrow basaltic spur forming

the right abutment. Neither of these two limitations can be thoroughly evaluated from the data now available. However, this position seems wholly unsuited for a high rigid narrow-based dam because of the fractures that have been described and because of the extensive sheets of weak fragmental material (tuff) that may separate some of the layers of basalt. The position might be suitable for an earth or rock-fill dam.

At line C–D (Tideport damsite, pl. 1), a dam would be slightly narrower and both abutments would have considerable mass. There, the right abutment is probably the weak shale and sandstone beds; the character of the bedrock in the left abutment is unknown. Only a flexible dam that has a comparatively broad base would be stable. In summary, the Tideport site seems to be distinctly inferior geologically as a possible damsite to the alternative site near Elsie, which is described in the following section.

ELSIE DAMSITE

GENERAL FEATURES

The Elsie damsite is at river mile 30.4, sec. 4, T. 4 N., R. 7 W. The drainage area is 498 square miles and the water surface is at an altitude of about 395 feet. The axis of the dam would be near the present U.S. 26 highway bridge. An earthfill dam that would back water to the 600-foot level would have a volume of about 1,840,000 cubic yards and the reservoir a capacity of 1,500,000 acre-feet. A considerable acreage of farm, pasture, and timber lands, as well as numerous access roads and highways would be flooded. Complete regulation of the runoff at the site during the period 1940–54 would require a storage capacity of 1,600,000 acre-feet, on the basis of a comparison with runoff at Foss. This capacity would require a reservoir with a maximum altitude of 605 feet. The area and capacity data for this reservoir site are shown in table 5.

TABLE 5.—*Area and capacity of Vernonia reservoir site, with dam at the Elsie site, mile 30.4, sec. 4, T. 4 N., R. 7 W.*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
395	0	0	520	7,500	309,000
420	200	2,000	540	10,100	485,000
440	1,300	17,000	560	15,200	738,000
460	1,900	49,000	580	18,700	1,077,000
480	3,300	101,000	600	23,600	1,500,000
500	5,000	184,000			

The available storage in the Vernonia reservoir between altitudes 470 feet and 600 feet at the Elsie site would be 1,425,000 acre-feet and

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it is assumed that it would not be desirable to draw the reservoir lower in regulating for power production. To insure conservative estimates, an active capacity of 1,400,000 acre-feet has been used. This volume is sufficient to have maintained a uniform flow of 1,450 cfs (92 percent of the average) during the period from April 1, 1940, to September 30, 1954. The 1,400,000 acre-feet would have been drawn out by about October 31, 1945, and the reservoir refilled by January 15, 1951. The reservoir would have been drawn down only 600,000 acre-feet between January 15, 1951, and September 30, 1954. This favorable circumstance would have permitted operating from the top of the reservoir.

If the reservoir were to be used for firming winter flows, it could have maintained a firm flow of 2,195 cfs during the 5-month period from November through March and 1,000 cfs during the rest of the year. The greatest drawdown would have been about 1,370,000 acre-feet at the end of October 1945. The reservoir would have refilled by mid-January 1951, with only minor storage requirements thereafter through 1954. By reducing the summer flow to 50 cfs a minimum discharge of 3,535 cfs could have been assured for November through March of each year.

If water were to be diverted to the Columbia River via Fishhawk Creek, only 1,000,000 acre-feet (that part of the Vernonia reservoir above 540 ft in altitude) would be active storage. This storage could have provided a year-round diversion of 1,300 cfs of water. Other possibilities for diversion could have included a combination of a summer discharge of 470 cfs and a winter discharge of 2,645 cfs, or a November through March concentration of 3,280 cfs. After supplying these amounts, a minimum flow of 50 cfs would have been left in the river at all times.

GEOLOGY

By A. M. Piper

Three distinct bedrock units are recognized at the Elsie damsite (pl. 1) and they differ greatly in perviousness and bearing power. From oldest to youngest they are shale, nonfragmental volcanic rock (basalt), and fragmental volcanic rocks (also basalt).

The shale is thinly laminated, bluish gray when fresh, extremely fine grained, and moderately indurated. It is virtually impervious and low in bearing power. In part, the shale is somewhat clayey and probably is plastic when wet. Where exposed at the surface, it slakes and weathers deeply to a yellowish-brown clayey soil. Good outcrops of shale were mapped at four places along the left (east) bank of the stream, as follows: (1) in roadcuts 20 to 30 feet above the river near the downstream edge of the mapped area, where the shale dips

20° in a N. 40° W. direction; (2) at the inner edge of the Nehalem River flood plain about 450 feet downstream from line A–B (Elsie damsite, pl. 1), where the beds are nearly horizontal; (3) in a test pit for a pier of a highway bridge at an altitude of 440 feet 100 feet upstream from the line A–B; and (4) at the inner edge of the flood plain about 200 feet farther upstream, where the shale dips 30° to 90° eastward and may not be in place. Except in the central part of the site, along the line A–B (Elsie damsite, pl. 1), this shale is inferred to underlie the volcanic rocks.

The nonfragmental basalt is noncellular and microcrystalline throughout, and has a high crushing strength. This rock is exposed only in the central part of the site along the line A–B (Elsie damsite, pl. 1), where the river pierces it in a narrow chute. The outcrop is 200 to 400 feet wide along the river and rises to an altitude of about 480 feet on each side of the river, or about 90 feet above low river stage. In its lower half, this nonfragmental basalt displays conspicuous radial columns that average about 3 feet across; in the upper part, it fractures concentrically into shells from 3 to 8 feet thick (fig. 4). These partings firmly join one another and gouge is absent. The rock as a unit is impermeable.

The basalt outcrop just described is inferred to be the uppermost part of a steeply dipping dike that trends east across the river. The dike would form a rigid, near-vertical plate below river level.

The fragmental volcanic rocks, which are all basaltic, overlie both the shale and the dike of nonfragmental basalt and are probably as thick as 225 feet over most of the area (Elsie damsite, pl. 1). The fragmental rocks vary widely in physical character. On the right (west) bank of the river they are massive and are composed of a dense basaltic matrix in which fragments of basalt as much as 2 feet across are embedded. Here they form a nearly vertical bluff 100 to 150 feet high and therefore are believed to be fairly resistant and to have moderate bearing power. They probably are not highly pervious except where fractured. On the opposite, or east, side of the river the rocks are decidedly inferior in bearing power, in part are unstable, and probably are pervious inasmuch as they are composed of (1) coarse basaltic tuff and agglomerate in a weak fragmental matrix, all somewhat decomposed and cut by veinlets and scattered masses of silica; (2) discontinuous sheets of fine-grained basalt, most of which have been reduced to rubble by random fractures from 1 foot to 8 feet apart; and (3) a few thin ribs (dikes?) of fine-grained or glassy basalt, which ordinarily are brecciated throughout and at some places are silicified. In general, the fragmental rocks on the east bank are incoherent; thus, along the Wolf Creek Highway, which was built in 1937 across the

FIGURE 4.—Concentrically fractured columnar basalt at the Elsie damsite as viewed from the left bank. Photograph by A. M. Piper, 1937.

damsite area, a cut 700 feet long and 53 feet deep was excavated without blasting. (This road is now part of U.S. Highway 26.) Fully 90 percent of the hillward face of the cut exposes only a rubble of basalt blocks less than 1 foot across embedded in an earthy matrix (fig. 5). Not even at the base of the cut does massive basaltic rock crop out. Further evidence of the instability of these rocks is provided by the fact that several landslides have occurred along this road.

In addition to the slope wash that covers most of the Elsie damsite, the unconsolidated materials include landslide rubble and discontinuous stream deposits on the bed and banks of the stream and in a terrace about 20 feet above the stream.

FIGURE 5.—Basalt rubble in highway cut along the flank of the left abutment at the Elsie damsite. Photograph by A. M. Piper, 1937.

The stream deposits are composed of sand, pebbles, and cobbles and some boulders of local origin. In the central third of the mapped area they are not extensive and probably are thin; commonly they rest on rock shelves a few feet above river level.

Landslide rubble forms two flats along the left bank of the river; the southern edge of one is near the line A–B (Elsie damsite, pl. 1) and the other begins about 500 feet downstream from this line. The upstream flat lies at the foot of a landslide scar which is 450 feet wide and has a scarp with a vertical height of approximately 150 feet. The slide originated in the fragmental volcanic rocks. Above stream level the landslide rubble is composed of clayey earth mingled with basaltic blocks as much as 20 feet long. The earth matrix has been washed away where the rubble underlies the stream.

The weak shale described previously crops out at the sole of each of these two slides; thus, the cause of failure in the volcanic rocks most probably resulted from plastic deformation in the shale. The third slide formed in the same manner at a highway cut 0.6 mile east of the the damsite (fig. 6) within a year after the cut was made. The slide block at this locality is about 700 feet long and 50 feet in maximum thickness; its upper half is composed of fragmental volcanics, the lower

FIGURE 6.—Slide in fragmental basalt and shale in highway cut 0.6 mile east of the Elsie damsite. Photograph by A. M. Piper, 1937.

half of shale. All these features suggest that (1) the excessive fragmentation of the basaltic rocks on the east bank is due in part to subsidence, and (2) a considerable part of the area between the highway and the river on the east bank is unstable. Indeed, the entire east bank, which is characterized by large-scale hummocky topography, may be an old landslide whose form has been somewhat modified by erosion.

The lines A–B and C–D (Elsie damsite, pl. 1) indicate the two positions that seem to be best suited for dam construction at the Elsie site. The downstream position (A–B) crosses the mass of dense nonfrag-

mental basalt that is interpreted to be a rigid, steeply dipping plate extending to considerable depth through the shale. Accordingly, this position seems suitable for a thin rigid dam that is arched for stability to a height of about 90 feet above the river. This conclusion should not be considered firm, however, until the vertical extent of the nonfragmental basalt has been determined by test drilling in the river bed and on either bank along the axis of the proposed dam. Neither this position nor any other within the area (Elsie damsite, pl. 1) seems suitable for a thin rigid dam of greater height, because the upper part of either abutment would be composed of the fragmental volcanics.

If the site at Tideport were superseded by a dam at the Elise site, the pond level would reach an altitude of 600 feet or more—that is, at least 200 feet above the river. To be stable, a dam that would confine this depth of water should be broad and flexible. A fill of earth or rock is suggested as most feasible. The left, or east, abutment of a flexible dam at the downstream position (A–B) may even be unsound above an altitude of 480 feet. The reasons for the instability of this abutment are as follows: (1) the potential leakage may be excessive all along the abutment for about 2,000 feet from the river, and (2) a considerable part of the abutment may prove unstable if saturated to or above an altitude of 600 feet.

The upstream position (C–D) is suggested as a more feasible site than the downstream position (A–B) for a flexible dam up to an altitude of 600 feet. The advantages of that position, which tend to compensate for a moderately greater length, are: (1) because both abutments are broad, excessive leakage seems unlikely, and (2) the weak shale, which is a major cause of instability in the fragmental volcanics, is inferred to lie below stream level and therefore potential landsliding would be avoided.

For a rockfill dam, the nonfragmental basalt near the site is suitable. At any place above 600 feet in altitude, the rocks are fragmental volcanics; if quarried for erecting a dam, a moderately large proportion of these rocks probably would need to be rejected. The most promising quarry site seems to be the flank of the ridge that overlooks the right abutment.

Percolation under a low rigid dam could easily be restrained by grouting the fractures in the nonfragmental basalt. A high flexible dam would call for more extensive cutoff measures; how extensive cannot be estimated without thorough test drilling. The most critical requirement seems to be an impervious connection from the dam deep into the shale that underlies the volcanic rocks, unless the volcanic rocks are proven to extend far beneath river level.

Except for the nonfragmental basalt and the more massive parts of the fragmental volcanics, all materials that form the Elsie damsite would be eroded rapidly by swiftly flowing water of considerable depth. Thus, the design of an adequate spillway involves critical problems.

For the low rigid dam, an overflow spillway seems feasible because firm rock probably underlies the streambed at shallow depth for at least 500 feet downstream from the line A–B. Accordingly, an extensive downstream apron probably would not be necessary, although riprap or some other measure to protect the left bank might be desirable. As an alternative, a spillway tunnel could be driven through the massive rock in the right abutment without undue difficulty.

For a high dam, an overflow spillway seems undesirable. A spillway at the side of the dam would be topographically feasible across the east abutment only if the dam were placed in the less desirable downstream position (A–B). For assured stability, a spillway structure there would require a substantial and extensive cutoff and an extensive pavement downstream.

An alternative spillway site could be located in a saddle about 1.2 miles northwest of the damsite (Elsie spillway area, pl. 1). This site is equally suited for dam construction at either of the two positions shown on plate 1 (Elsie damsite). Within 1,000 feet of this saddle in every direction the bedrock seems to be entirely composed of shale. Like the shale that crops out at the damsite, this shale is thoroughly decomposed to a moderate depth beneath the land surface. Even if it were unweathered it would have low bearing power and would be abraded easily by swiftly flowing water. However, it is virtually impervious. Any spillway structure across this saddle should have an extensive pavement downstream. If the main dam is to rise above an altitude of 600 feet, the spillway structure must also serve as a dike to close the saddle. For this dual purpose, the structure should have a wide base and should be seated on unweathered shale. Its maximum feasible height cannot be estimated closely without loading and percolation tests of the foundation material.

SPRUCE RUN RESERVOIR SITE

The Spruce Run damsite is at Nehalem River mile 22.5 (sec. 24, T. 4 N., R. 8 W.) just downstream from the mouth of Spruce Run Creek and approximately 8 miles downstream from the Elsie damsite. The water surface at the Spruce Run damsite is about 305 feet in altitude and the drainage area is 549 square miles. Dams of various heights have been studied for the Spruce Run site. The damsite might be used either for a dam to back water to the Elsie or Tideport damsites

or for a dam that would back water to Vernonia. Table 6 shows the area and capacity data for the reservoir site.

TABLE 6.—*Area and capacity of Spruce Run reservoir site, damsite at mile 22.5, sec. 24, T. 4 N., R. 8 W.*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
305	0	0	440	3,000	125,000
320	80	600	480	5,400	254,000
340	130	2,700	520	9,520	552,000
360	320	7,100	560	19,190	1,126,000
380	545	15,700	600	28,520	2,000,000
400	885	30,000			

The approximate height and volume of earthfill dams and capacities of their respective reservoirs are as follows:

Purpose	Dam height (feet)	Volume (cubic yards)	Reservoir capacity (acre-feet)
Back water to Elsie damsite	90	260,000	30,000
Back water to Tideport damsite	132	615,000	115,000
Back water to Vernonia	295	5,200,000	2,000,000

The average discharge of the river at the damsite is estimated to be about 1,880 cfs. The 2,000,000 acre-foot reservoir would have completely regulated the stream during the test period and would make almost any desired water-use plan possible. The reservoir would hold 1,270,000 acre-feet above the 540-foot altitude. That amount of storage would have made possible a continuous diversion of about 1,600 cfs to the Columbia River during the test period.

The more likely use of the Spruce Run site, however, would be for a dam to back the water to the Elsie or Tideport sites, which are at altitudes 395 and 437 feet respectively. The total head created would be 90 or 132 feet and the reservoir would reregulate the releases from the upstream reservoir for power development.

The damsite area is shown on the previously described Saddle Mountain quadrangle, scale 1:62,500, and the 1936 river survey map, scale 1:31,680. A large-scale map has not been made, nor has a geologic examination been made of the area.

SALMONBERRY RESERVOIR SITE

GENERAL FEATURES

The Salmonberry damsite is in sec. 10, T. 3 N., R. 8 W., at Nehalem River mile 15.5, about half a mile upstream from the confluence of the Salmonberry River. Drainage area is 573 square miles and the water surface altitude is 205 feet. Table 7 shows the area and capacity data

for this reservoir site. The water surface at the Spruce Run damsite is 305 feet, and at the Elsie site is 395 feet. The approximate height and volume of the earthfill dams that would back water to these sites or to Vernonia, and the capacities of the respective reservoirs, are as follows:

Purpose	Dam height (feet)	Volume (cubic yards)	Reservoir capacity (acre-feet)
Back water to Spruce Run damsite	100		16,000
Back water to Elsie damsite	190	940,000	115,000
Back water to Vernonia	395	8,400,000	2,500,000

Dams high enough to back water to Spruce Run or to Elsie seem quite reasonable whereas the larger one probably is a remote possibility.

TABLE 7.—*Area and capacity of Salmonberry reservoir site, damsite at mile 15.5, sec. 10, T. 3 N., R. 8 W.*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
205	0	0	440	3,400	250,000
240	50	700	480	7,000	463,000
280	250	7,000	520	11,400	831,000
320	680	25,000	560	21,300	1,485,000
360	1,160	60,000	600	31,000	2,531,000
400	2,000	125,000			

A dam that would back water to the Elsie damsite seems the more likely. A 115,000 acre-foot reservoir between the Elsie site and the Salmonberry site could be used for reregulation of releases from a large reservoir at the Elsie site. The constant release from Elsie of 1,450 cfs on a continuous schedule could be regulated to a minimum flow at the Salmonberry site of about 1,845 cfs. Reregulating the summer (1,000 cfs) and winter (2,195 cfs) releases from Elsie could raise the summer minimum to about 1,310 cfs and the winter minimum to about 3,150 cfs at Salmonberry.

If the dam were constructed to the 600-foot altitude, complete control of the Nehalem at the Salmonberry site would be possible; in fact, only 2,100,000 acre-feet of storage would have been required for the period studied. That storage would have been depleted near the last of October 1945 and the reservoir would have refilled by the end of March 1951 with only minor drawdown thereafter.

The entire 2,500,000 acre-feet of water in a 600-foot altitude reservoir would be very valuable for peaking purposes, for it could be used at any time of the year in large or small amounts, or saved for use in dry years as needed. For example, a full reservoir in June 1940 could have been drawn upon at a rate greater than the average discharge through the dry years which ended during October 1945. This large

reservoir would have enabled a plant at Salmonberry to furnish some of the power lost at plants that have less storage and the excess water used could have been recovered by reducing releases in subsequent wet periods when water was adequate at other powerplants.

The storage above 540 feet at Salmonberry would be 1,420,000 acre-feet. If this site were used as an alternative reservoir for diversion to the Columbia River, this storage would have been sufficient for a constant flow of 1,730 cfs for the period May 1, 1940, through February 15, 1949. Utilizing this same volume of storage for concentrating the winter flows would permit a winter flow of 4,275 and a summer flow of 115 cfs.

GEOLOGY

By D. L. Gaskill

At the Salmonberry damsite (pl. 2), the Nehalem River flows in a deep canyon that is cut in basaltic flows of probable early Eocene age (Tillamook Volcanic Series of Warren and others, 1945). Canyon slopes rise to altitudes 1,200 feet above the damsite in a horizontal distance of ¼ to ½ mile from the river.

Foundation rocks consist of thick to massive flow layers of dark-gray dense equigranular fine-grained basalt and porphyritic basalt containing phenocrysts of feldspar 1 to 4 mm in length. Individual flows include some coarse-textured and amygdaloidal basalts. Some of the flows are separated by volcanic breccia, agglomerate, or tuffaceous beds of gray to blackish-red indurated clay. Local alteration of the basalt gives some exposures a green or olive-gray.

Bedrock on the east abutment is generally concealed by vegetation, talus, landslide, or rockfall detritus, in contrast to the drier, stable well-exposed bedrock on the west abutment. The river seems to have been formerly blocked at the damsite by rockfall material originating high on the east canyon wall. As a result of this rockfall, the river current has been directed against the west abutment. Although the present channel is still compressed against the west bank and is partially choked by block rubble, the river is cutting bedrock at and below section *D–D′* (pl. 2). However, 300 feet upstream from section *D–D′*, the river is still excavating thick coarse alluvial materials which have been deposited behind this rubble dam.

The basaltic flows dip upstream. The flow layers strike roughly east-west and dip about 28° N. in the damsite area. Irregular flow structures guide exfoliation of the basalts. Fractured, brecciated, and oxidized zones indicative of fault movement were observed at several localities in and near the damsite area. Study of aerial photographs suggests that the region is broken by many diverse faults or major zones of bedrock fracture. The lineament trending northwest

across the damsite area (pl. 2) follows a fractured, altered zone marked by deep gullies, but no offset of flows was observed. The lineament bisecting section *D–D′* is inferred from study of aerial photographs. A bed of indurated tuffaceous clay about 2 feet thick separates flow sheets on the west abutment at river level near section *D–D′*. About 200 feet downstream, a thin layer of basaltic agglomerate separates the massive flow layers at river level.

The attitude of the flow layers, at approximately right angles to the river and dipping upstream, is favorable for a dam here (fig. 7).

FIGURE 7.—Salmonberry damsite (section at *D–D′*, pl. 2), looking upstream. Photograph by D. L. Gaskill, 1957.

Fractures weaken the basalt, particularly on the east abutment where rock disintegration is facilitated by a moderate dip of the flow layers downslope. Most of the joint fractures are discontinuous and form tight random-sized interlocking blocks. The massive fine-grained basalt flows at river level and on the west abutment appear competent to sustain any proposed dam structure here. The thin beds of flow breccia, agglomerate, tuff, and clay are weak and permeable, but they represent only a small fraction of the exposed geologic section. These weak zones, in common with the massive flows, dip upstream and therefore oppose any tendency of a dam to overturn or slide downstream. The instability of the east abutment is perhaps the most serious handicap of this site. The dip of the basalt flows toward the river, in combination with an advanced state of solifluction, rock alteration,

exfoliation, and deep fracturing, has caused rapid bedrock deterioration and landsliding. The abutments probably have been weakened by high-angle shear zones and by one or more possible faults parallel to the river on the east abutment. Several springs emerge from beneath landslide rubble on the east abutment at river level, and seepage was observed along clay partings in the west abutment. Some leakage might occur along flow contact zones susceptible to hydraulic pressure or along high-angle joint planes trending downstream, but such leakage would probably prove unimportant and controllable by grouting. Stripping should prove minimal on the west abutment, but would require removal of considerable rock debris and deteriorated bedrock on the east abutment. Unconsolidated rockfall material, perhaps more than 50 feet thick in places, is present on the lower slope of the east abutment. Thick talus deposits are also present higher on the east abutment above the main landslide deposit. Loose, incoherent, disintegrated mantle rock, 10 to 15 feet thick, is locally exposed in roadcuts near the damsite area. Careful examination of the structural stability of a broad bedrock rib immediately northeast and above section *D–D′* on the east canyon wall would be advisable. A slide or rockfall originating on this spur might directly damage a dam or create damaging reservoir waves.

Impervious and aggregate materials are locally available in riverbar and flood-plain deposits above and below the damsite. The dense fresh basalt from nonbrecciated nonamygdaloidal flows might be a source of aggregate in the damsite area.

The Salmonberry site seems suitable for either an earth-rockfill or concrete dam.

UPPER WAKEFIELD RESERVOIR SITE

GENERAL FEATURES

The Upper Wakefield damsite is below the mouth of the Salmonberry River in sec. 16, T. 3 N., R. 8 W., near Nehalem River mile 14.4. Drainage area is 644 square miles and the water surface altitude is 175 feet. The reservoir site has an estimated storage capacity of 2,740,000 acre-feet with a dam to the 600-foot altitude. The volume of an earthfill dam to this height would be about 14,300,000 cubic yards. The average discharge of the river at the damsite is estimated to be 2,480 cfs. Storage required for complete regulation is estimated at 2,600,000 acre-feet. About 1,000 acres of the reservoir lies below the Salmonberry damsite, and most of this is in the Salmonberry River valley. The area and capacity data for this reservoir site is shown in table 8.

The most likely use of a dam at this site would be to raise water to the level of the Elsie site. Storage would amount to about 150,000

TABLE 8.—*Area and capacity of Upper Wakefield reservoir site, damsite at mile 14.4, sec. 16, T. 3 N., R. 8 W.*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
175	0	0	400	2,400	164,200
200	31	400	480	7,650	566,200
240	130	3,600	520	12,230	963,800
280	390	14,000	560	22,290	1,654,200
320	900	39,800	600	32,170	2,743,400
360	1,460	87,000			

acre-feet. This amount could be obtained with an earthfill dam 220 feet high having an approximate volume of 2,460,000 cubic yards.

A branch of the Southern Pacific railroad follows the Salmonberry River and the Nehalem River to the coast and about 4 miles of it would be flooded by a dam backing water to the Elsie site. The cost and difficulty of relocating the railroad would have to be considered in planning a dam at this site.

GEOLOGY

By D. L. GASKILL

At the Upper Wakefield damsite (pl. 2), the Nehalem River is confined in a deep canyon that is cut in basaltic lava flows of the Tillamook Volcanic Series (Warren and others, 1945). Valley walls rise to altitudes 1,800 feet above the river here. Bedrock is similar to that described at the Salmonberry damsite a mile upstream. Individual flows vary in texture but the predominant rock is a dense dark-gray fine-grained to porphyritic basalt. The flows exhibit slightly irregular contacts and many are separated by pyroclastic rocks and brick-red zones of baked tuffaceous clay. Many flows have well-defined flow partings.

The river is actively cutting bedrock in the damsite area. At several places along the river channel, fine-grained and porphyritic feeder dikes have been intruded along near-vertical fractures trending in a northerly or northwesterly direction. These dikes reach a maximum of 5 feet in width and contain a high percentage of glass and accessory magnetite. The basaltic flows strike roughly N. 80° W. and dip approximately 27° N. Several small faults or shear zones are indicated on plate 2 (Upper Wakefield damsite). The faults mapped in the area of section B–B′ are intruded by dikes. Displacement on faults does not indicate recent movement. These old faults are probably not critical planes of weakness in respect to a dam here. Joint systems represented by near-vertical, closely spaced parallel fracture sets are common in the foundation rock along the river, although large areas or individual flows may appear almost joint free. The

more conspicuous joint sets generally strike in a northerly direction and have secondary joints at right angles and diagonal to the major set. Most of the fractures are thin, tight, and discontinuous and probably do not seriously weaken the fresh unaltered bedrock. These joints are spaced about 3 to 6 inches apart in foundation rock at section B–B′ and at the river bend upstream from section A–A′.

Large areas of stable bedrock are exposed on the steep (40°–45° slope) north wall of the canyon. The north abutment is dryer and supports less vegetation than the heavily forested south abutment. Bedrock on the south side of the canyon is almost completely concealed by vegetation, talus, landslide rubble, and rock-soil creep. Limited areas of bedrock are exposed above an altitude of 300 feet on the south abutment along sections *A–A′* and *B–B′* (pl. 2). These rocks are separating from the cliff face in large blocks along vertical joint planes. West of the deep tributary gorge in the south canyon wall, most of the lower slopes below an altitude of 400 feet are covered by soil and rock debris that are in part saturated with water. This debris is more than 120 feet thick and is probably the result of landsliding. Slickenside surfaces observed on large boulder blocks comprising this material at river level indicate the presence of faults or landslip surfaces high on the south canyon wall. Many springs issue from the base of this debris along the south side of the river. The structurally weak and unstable character of the south abutment is due to the pronounced dip of the flow structures toward the river and the advanced state of bedrock disintegration and deterioration accompanying deep physical and chemical weathering.

Considerable excavation of deteriorated bedrock below the zone of open fractures would be necessary on the south abutment for a dam structure in this area. In contrast to the negligible stripping necessary on the north abutment at section *A–A′*, as much as 40 feet of talus debris would have to be removed on the south abutment below an altitude of 300 feet and 50 feet or more of bedrock above this altitude, depending upon the height of the dam.

Bedrock leakage probably would be limited to fractures and flow contacts, particularly the flow contacts dipping toward the river on the south abutment.

Core holes should be drilled along the south abutment profile to determine depth of unconsolidated material and bedrock deterioration.

An overfall type of spillway would subject the bedrock to some plucking, but would abrade the rock very slowly. A side-channel spillway or water-diversion tunnel on the north abutment would probably be feasible.

A damsite in the area of sections *A–A′* or *B–B′*, east of the tributary creek on the south abutment, would probably be suitable for a

rockfill or concrete-type structure of any desired height. However, reconnaissance of possible sites immediately upstream or east of the mapped area (Upper Wakefield damsite, pl. 2) is recommended.

A brief discussion of available construction materials in this area is included in the discussion of the Salmonberry damsite (p. 31).

WAKEFIELD RESERVOIR SITE

A dam at Nehalem River mile 12.8, about 1.6 miles downstream from the Wakefield site (fig. 3) would make a reservoir for a given dam height where capacity would be only slightly more than that of a reservoir at the Upper Wakefield site. For this reason the two positions offer equal advantages as reservoir sites.

NEHALEM FALLS RESERVOIR SITE

GENERAL FEATURES

The Nehalem Falls damsite is at Nehalem River mile 8 in sec. 27, T. 3 N., R. 9 W., where the water surface altitude is 65 feet. Raising the water surface to 395 feet, which would back water to the Elsie site, would provide about 362,000 acre-feet of storage. The volume of an earthfill dam of the required height would be about 8,300,000 cubic yards. Table 9 shows area and capacity data for this reservoir site.

The average discharge at Nehalem Falls is estimated at 2,585 cfs. Complete regulation of the river at this point would require about 2,740,000 acre-feet of storage. This amount of storage is much beyond the capacity of a reasonable dam at this site. Creation of head and reregulation of releases from upstream reservoirs would be more likely uses of any reservoir at Nehalem Falls.

TABLE 9.—*Area and capacity of Nehalem Falls reservoir site, damsite at mile 8, sec. 27, T. 3 N., R. 9 W.*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
65	0	0	240	850	51,000
80	6	40	280	1,200	92,000
120	90	1,400	320	2,000	153,000
160	250	8,200	360	2,750	248,000
200	540	23,000	400	3,900	380,000

If the river is regulated at Elsie to a continuous flow of 1,450 cfs, 347,000 acre-feet of storage at Nehalem Falls could reregulate to a constant flow of 2,200 cfs. This flow would leave 15,000 acre-feet of dead storage in a reservoir backing water to the Elsie site. If the river is regulated at Elsie to 1,000 cfs and 2,915 cfs for the April through October and the November through March periods, respectively, the reservoir at Nehalem Falls could regulate the river to 1,210

cfs and 3,900 cfs, respectively. By concentrating all the flow at Elsie except a conservation requirement of 50 cfs in the winter period, a flow of 3,535 cfs there could be increased to 5,385 cfs at Nehalem Falls.

Another possible plan of development that might affect the use of the Nehalem Falls reservoir would be to divert all the flow except a conservation release of 40 cfs at Squaw Creek to the Columbia River basin. In that event, the Nehalem Falls reservoir could be used to regulate discharges from the drainage basin downstream from Squaw Creek; the average flow at Nehalem Falls would be about 1,525 cfs (40 cfs conservation release at Squaw Creek and 1,485 cfs from the intervening drainage area). The 362,000 acre-feet of storage at Nehalem Falls would permit a minimum regulated flow of about 1,030 cfs after reservoir losses. If water is diverted to the Columbia River from a reservoir behind a dam at Tideport or Elsie, discharge at Nehalem Falls would be smaller but might still be large enough to warrant development there.

GEOLOGY

By A. M. Piper

The damsite at Nehalem Falls seems to involve only two extensive rock units—an incoherent terrace deposit, and a volcanic unit (basalt). A view of the damsite is shown on figure 8.

The terrace deposit consists largely of poorly sorted sand, pebbles, and cobbles as much as 3 inches in diameter that were derived from dense volcanic rocks, chiefly basalt. The rocks comprising the deposit are slightly weathered and all are bleached and iron stained. These stream-borne materials are exposed in the streamward faces of two terrace remnants about 30 feet above the river; one is on the left (east) bank at the upstream edge of the area shown on plate 2 (Nehalem Falls damsite), and the other is on the right bank near the downstream edge of the area. Near the landward edges of the two remnants, the stream-borne materials are mingled with slightly sorted slope wash. The terrace deposits rest on rock shelves from 2 to 15 feet above river level; the maximum thickness of the deposits is estimated to be about 30 feet. The unit as a whole is highly pervious and incompetent to sustain a rigid dam.

The greater part of the volcanic rock (basalt) is composed of several thick sheets that dip about 5° to 10° in a N. 30° E. direction or diagonally upstream. The basalt is fine grained, dense, and fresh; it has moderately high crushing strength, probably at least 20,000 pounds to the square inch. However, the following three features detract somewhat from its strength in mass:

1. Fractures which are undulatory, discontinuous, and commonly 10 feet or more apart are nearly vertical and form two sets. One

set trends northeast, the other northwest. A few fractures have crushed selvages as much as 4 inches thick. Cross fractures, random in strike and dip, are common. Taken as a whole, these fractures probably weaken large masses of the basalt only slightly, for the plates and blocks that intervene between fractures are random in size and shape and are rather closely interlocked. However, the thin spur that forms the east bridgehead near the downstream edge of the area is unstable and seems to be sliding toward the river; thus, the stability of the thin spurs on either side of the valley in the central part of the area also may be questioned.

2. Fragmental partings (basaltic scoria and flow breccia) separate the sheets of dense basalt at some places, as beneath the terrace deposit along the west bank of the river in the central part of the area. These fragmental partings are weak but, as exposed in fairly large and numerous outcrops, are neither thick nor extensive. Those that are exposed dip upstream and therefore would oppose the tendency of a dam to slide downstream.
3. Weak zones occur where the basalt is slightly or moderately decomposed or where it is cut by veinlets of nonmetallic minerals (drusy quartz and zeolites, in part cellular). Their number and extent are not fully disclosed by the outcrops, but they seem to be most extensive in the downstream half of the area.

A small part of the volcanic rock is composed of fine-grained dense basalt in thin dikes and of massive, coarsely fragmental rock (mostly basaltic agglomerate) similar to that at the Elsie site upstream. Both upstream and downstream near Nehalem Falls damsite, massive fragmental rock is extensive and commonly is deeply weathered. It may also be extensive in parts of the site that are covered by slope wash. This possibility is a critical feature of the site with respect to the erection of a dam, for the fragmental rock is materially weaker than the nonfragmental basalt and probably would not sustain an extremely thin dam. However, because landslide debris is absent, any fragmental rock at the Nehalem Falls site is probably stronger than that on the east bank of the river at Elsie site.

The best position for a dam at Nehalem Falls seems to be that indicated by the line *A–B* on plate 2 (Nehalem Falls damiste). At that position, both abutments are broad and it is inferred that fractures do not make either unstable. Nearly continuous exposures of dense nonfragmental basalt of high bearing power extend across the stream bed and also form the full height of the right abutment. The unconsolidated stream deposits are neither extensive nor very thick and can be removed with little trouble. However, the nature of the bedrock in

FIGURE 8.—Nehalem Falls damsite viewed upstream from the road bridge. Photograph by A. M. Piper, 1937.

the left abutment is not known and should be explored by core drilling. If all the bedrock is nonfragmental basalt such as forms the right abutment, the site probably is suitable for a rigid dam of gravity cross section, but the bedrock in the left abutment is largely fragmental, the site may be suitable only for a flexible dam. Such a dam might well be a fill of rock quarried from the dense basalt high on the right bank.

To restrain leakage, an impervious membrane seated in sound bedrock should extend somewhat below the floor of the chute which forms the "falls" about 250 feet downstream. In addition, a grout curtain to seal fractures in the abutments will probably prove desirable; the extent of such a curtain cannot be determined from data now available.

In the design of a permanent spillway, the geologic features of the damsite at Nehalem Falls offer no critical problems. The dense basalt that forms nearly all the bedrock along the stream is abraded very slowly by water flowing swiftly and in considerable depth; thus an overfall spillway seems feasible. Some paving may be necessary to prevent the plucking of small blocks from fracture zones but need not be extensive.

STONEHILL RESERVOIR SITE

GENERAL FEATURES

The Stonehill damsite is at mile 4.9, sec. 34, T. 3 N., R. 9 W., about 5 miles upstream from tidewater and 8 miles east of Nehalem. The water surface altitude at the damsite is about 23 feet. The average discharge of the river at the site is estimated to be about 2,900 cfs. An earthfill dam that would raise the water surface to the Nehalem Falls damsite, altitude 65 feet, would have a volume of 80,000 cubic yards. The reservoir would have a capacity of about 3,000 acre-feet. The purpose of the dam would be principally for creating head for power. Releases from the Nehalem Falls reservoir could be reregulated to some extent. One the basis of the Foss gage records the minimum flow increments to the Nehalem River between Nehalem Falls and Stonehill are estimated to be about 15 cfs during the driest months and about 170 cfs during the November through March period. The annual discharge, above reservoir losses, would be only 5 cfs and 160 cfs more than at Nehalem Falls for these periods.

GEOLOGY

By D. L. Gaskill

At the Stonehill damsite, the Nehalem River flows westward through a short valley constriction cut in rock of the Tillamook Volcanic Series (Warren and others, 1945). Bedrock consists of thick to massive flows of dark- to greenish-gray dense, finely crystalline and porphyri-

tic basalt, altered chloritized porphyritic basalt, and flow breccia. Flow breccia is exposed at a small quarry on the north abutment east of section *C–C′* (Stonehill damsite, pl. 2) and along the railroad grade near bench mark PTBM 90. At the quarry, the flow breccia is about 40 feet thick and consists of greenish-gray angular welded blocks of pyroclastic material chloritized to a greenstone. Vesicular and amygdaloidal zones are common, especially in the upper part of the flow. Joint faces are pyritized, veins of calcite cut the rock, and cavities are filled with quartz, calcite, and zeolite minerals. Massive flows of finely crystalline basalt and porphyritic basalt overlie and underlie the breccia. The flows at the quarry dip about 25° NE. Bedrock exposures on the steep north abutment include both unaltered and altered chloritized oxidized basalt. The river is abrading finely crystalline basalt along the base of the north abutment and seems to be flowing over bedrock a short distance upstream. Finely crystalline basalt and weathered porphyritic basalt outcrop locally along the abandoned rail grade on the south abutment. A thin bed of glauconitic(?) sandstone was observed between flows on the south bank near bench mark PTBM 66 (pl. 2). Most of the south abutment slope is covered by terrace and mantle deposits and obscured by vegetation. Only a thin veneer of alluvium is inferred to overlie bedrock in the river channel. Unconsolidated river terrace deposits are thin, probably less than 15 feet thick at section *C–C′*.

Bedrock on the north abutment is broken by numerous joint fractures. Lineaments plotted from aerial photographs and shown on plate 2 may represent weak fracture zones. Faulting is inferred on the north abutment, where highly oxidized zones are exposed along the highway and poorly exposed altered flow breccia crops out in railroad cuts. If the breccia is the same as that exposed at the quarry, a large displacement of flows is indicated, down thrown on the west. A series of northeast-trending faults cuts the volcanics and sedimentary rocks of the Cowlitz Formation several miles west of the damsite between Foss and Mohler, and a thick sequence of indurated faulted sediments (laminated hornfels) is exposed along the abandoned rail grade about half a mile east of the damsite on the south side of the river.

The north abutment seems to be structurally stable owing to the dip of the flow layers into the hill, in contrast to the inferred dip of the flows toward the river on the south abutment. The moderate upstream dip of the flow layers should afford good anchorage for a dam. Bedrock is weakened by joints and by local rock alteration, and probably by faults. The inferred fault zone mapped on the north abutment probably crosses the river and seems to be the most objectionable geologic feature of the damsite. Exploration would be required to deter-

mine the structure and degree of decomposition along this zone. Additional investigation of concealed bedrock structures, especially on the south abutment, is advisable. Joint planes and possible thin interbeds of tuff or sedimentary rock may be potential zones of reservoir leakage.

Impervious materials and aggregate appear to be available in large quantity from alluvial deposits in the Nehalem River valley near the damsite. Sand and gravel have been excavated from terrace deposits 2½ miles west of the damsite. Highway-metal quarries have been developed in and near the damsite area.

The Stonehill site is probably suitable for a wide base earth- or rock-fill structure.

GODS VALLEY RESERVOIR SITE

The Gods Valley damsite is in sec. 26, T. 4 N., R. 9 W., on the North Fork Nehalem River, about 1 mile upstream from Soapstone Creek. The water surface at the damsite is estimated to be about 240 feet in altitude. The only maps of the dam and reservoir site area are the U.S. Geological Survey 15-minute Cannon Beach and Saddle Mountain 1:62,500-scale quadrangles. A dam that would raise the water to the 400-foot altitude would have a crest length of about 500 feet; its volume is roughly estimated to be about 680,000 cubic yards. As measured from the quadrangle maps, the reservoir capacity would be about 75,000 acre-feet and would cover an area of 1,400 acres. There are no important roads or other improvements in the reservoir area. Average discharge at the site is estimated to be about 300 cfs. By using all the storage, a minimum discharge of 245 cfs could have been assured during the period studied. Other possibilities would be a minimum release of 535 cfs for power production during the November through March period or a diversion of as much as 300 cfs into an irrigation canal during a 4-month period June through September, after losses and a conservation release of 10 cfs.

A regulating reservoir at the Gods Valley site would be very beneficial to the North Fork basin regardless of whether a powerplant were included, and especially if it provided for passage or propagation of fish.

The damsite has not been examined geologically.

POTENTIAL POWER

Power development on the Nehalem River will not be practicable unless reservoirs are provided to regulate flow. Table 10 presents the regulated discharges and power that could have been available over the 15 water years 1940–1954, by regulation in four reservoirs on the main stem and one on the North Fork. Nehalem Falls benefits from

storage at Elsie, and Stonehill from storage at Elsie and Nehalem Falls. Rocky Point storage is not reflected at any of the downstream sites because the purposes for which this reservoir is likely to be built may detract from, rather than add to, downstream power. The various regulating possibilities of the reservoirs were discussed in the section on regulation and storage. Table 10 shows three alternative plans for utilizing the reservoirs: (1) by continuous regulation, (2) by assuring a small amount of water for power production during the summer months (when the Columbia River is flowing at high stages) in combination with a maximum amount of the remaining water for winter power production (when the Columbia River is at low stages and when demand for power is high), and (3) by assuring the maximum regulation for the winter months if no water is released for power during the rest of the year.

Figure 3 shows several damsites and reservoir sites on the main river, and one dam and one reservoir site each on Fishhawk Creek and the North Fork. A total head of 705 feet on the main river is available for development by dams or by combinations of dams and conduits. Three hundred and forty feet of head could be developed at the site on the North Fork by means of a dam and conduit. The illustrative plan of development presented here includes four dams on the Nehalem River and a dam and conduit on the North Fork. Fishhawk Creek is discussed separately because it would divert water out of the basin and thereby reduce the power potential of downstream sites. The basin's hydroelectric power potential is estimated as the combined amount that could be developed by dams at the five following sites: Rocky Point, Elsie, Nehalem Falls, and Stonehill on the Nehalem River, and Gods Valley on the North Fork. Some of the alternative dams discussed on pages C10–C40 and other possibilities for power development may have equal or greater merit, but to avoid confusion they have not been included in this section. The illustrative plan of development was chosen for estimating the gross power potential and is not necessarily the optimum one. In fact, the best use of the Nehalem River for power development might be a pumped-storage diversion to the Columbia River. That possibility is discussed separately at the end of this section.

The most valuable potential contribution of the Nehalem River to the Pacific Northwest power supply might be for furnishing peaking or firming power during the winter. The Columbia flows at rates greater than average for 4 months, April, May, June, and July, and at rates less than average during the remaining 8 months. The Nehalem River flows at rates greater than average for 5 months, November through March, and less than average for the remaining 7 months.

TABLE 10.—*Potential power of the Nehalem River*

Time of use	Flow (cfs) and potential power (kw) at indicated site (average head, in feet)										Total	
	Rocky Point (105)		Elsie (170)		Nehalem Falls (270)		Stonehill (40)		Gods Valley (305)		Kw	Kwhr
	Cfs	Kw	Cfs	Kw	Cfs	Kw	Cfs	Kw	Cfs	Kw		
Continuous	185	1,320	1,450	16,800	2,200	40,400	2,205	6,000	245	5,080	69,600	609,700,000
Combination:												
April–October	[1] 355	2,500	1,000	11,600	1,210	22,200	1,215	3,300	[1] 300	6,200	45,800	230,000,000
November–March	[2] 10	----------	2,195	25,400	3,900	71,600	4,060	11,000	10	----------	108,000	391,000,000
Total combination	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	----------	621,000,000
November–March only	435	3,100	3,535	40,900	5,455	100,000	5,615	15,000	535	11,000	170,000	616,000,000

[1] Covers the low-flow season only (June–September).
[2] Minimum with uncontrolled averages as much as 145 cfs for the October–May filling period.

The 5-rate months thus occur when the Columbia River is low; these are also the months when demand for power is highest. Figure 9 compares the monthly percentages of average annual discharge for the Nehalem and Columbia Rivers.

If the Vernonia site is utilized for a large reservoir, a high degree of control will be afforded and the schedule for producing power can be correspondingly flexible. Three different schedules are shown in table 10 for the five-dam development. The plan for continuous generation would have the lowest capability, but it would be 98 percent of the combination plan, which has the highest potential, and for that reason the energy output is almost the same regardless of the time of use. These high potentials are due to the regulating value of the Vernonia reservoir site. The greater value of winter peaking power makes the winter-use plans attractive, but the undesirability of completely closing plants during the summer is favorable to the combination or the continuous arrangement.

Diversion of water to the Columbia by pumping from the Vernonia reservoir with a dam at Elsie and by operation for year-round regulation could produce 666,000,000 kwhr of energy. A combination of winter and summer operations could produce 703,000,000 kwhr annually. Operated to produce winter peaking power only, the plan would produce 696,000,000 kwhr which would require expending about 350,000,000 kwhr of off-peak energy for pumping requirements.

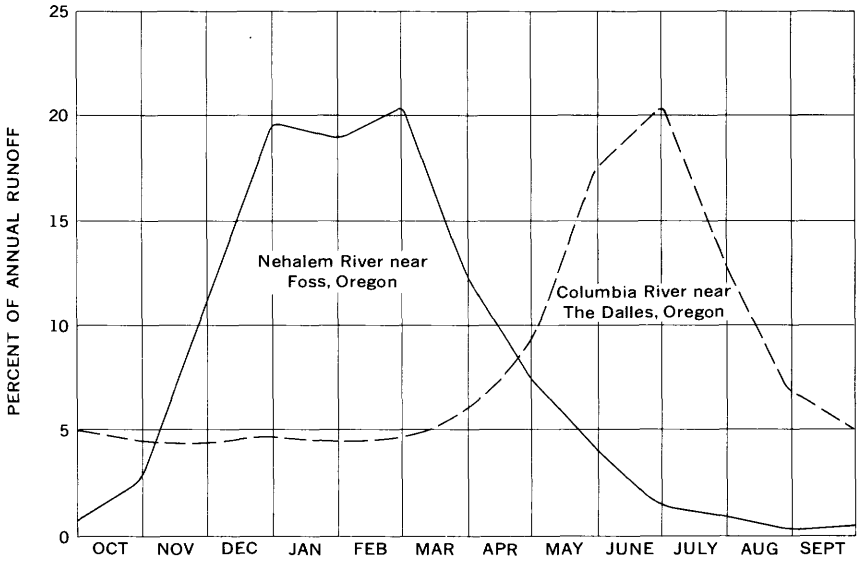

FIGURE 9.—Mean monthly percent of average annual flow for Nehalem River near Foss and Columbia River near The Dalles.

Table 11 summarizes potential power of the individual sites for the uncontrolled river, for continuous regulation and for maximum utilization of water between November 1 and March 31. These data show power potentials under existing conditions and by possible regulation. The data also indicate the importance of regulation in any practical waterpower development on the Nehalem River.

If the Rocky Point dam is constructed for irrigation or conservation, it might be found economic to include a powerplant. The reservoir operations might assure a minimum year-round flow, concentrate the flow in the winter months for power production, or hold the water for summer irrigation and fish propagation.

TABLE 11.—*Summary of potential power under natural-flow and regulated-flow conditions*

[Computations are for 80 percent efficiency]

Damsite	Head (ft)	Percent of time or period	Estimated flow (cfs)	Power (kw)
		Natural flow		
Rocky Point	130	Q95	10	90
		Q50	100	900
		Qmean	235	2,100
		November-March mean	465	4,100
Elsie	205	Q95	65	900
		Q50	650	9,000
		Qmean	1,573	21,900
		November-March mean	3,110	43,300
Nehalem Falls	330	Q95	105	2,350
		Q50	1,070	24,000
		Qmean	2,585	58,000
		November-March mean	5,110	115,000
Stonehill	42	Q95	117	334
		Q50	1,200	3,420
		Qmean	2,900	8,280
		November-March mean	5,738	16,360
Gods Valley	340	Q95	10	230
		Q50	125	2,890
		Qmean	300	6,930
		November-March mean	600	13,900
Total		Q95		3,900
		Q50		40,210
		Qmean		97,210
		November-March mean		192,660
		Regulated flow		
Rocky Point	105	Continuous	185	1,320
		November-March	435	3,100
Elsie	170	Continuous	1,450	16,800
		November-March	3,535	40,900
Nehalem Falls	270	Continuous	2,200	40,400
		November-March	5,455	100,000
Stonehill	40	Continuous	2,205	6,000
		November-March	5,615	15,000
Gods Valley	305	Continuous	245	5,080
		November-March	535	11,000
Total		Continuous		69,600
		November-March		170,000

A reservoir behind a dam at the Elsie site would store more than 1,400,000 acre-feet between altitudes 470 and 600 feet. It might be operated to produce a uniform year-round flow, a moderate flow from April through October with a firming winter flow about twice as great from November through March, or a concentrated winter flow with the summer release limited to water for operating fish ladders.

Regulated flows for the Nehalem Falls and Stonehill sites are based on regulated discharges from the Vernonia reservoir with a dam at Elsie.

If the Gods Valley dam is constructed for providing irrigation water, for fish propagation, or for other water-conservation purposes, addition of a powerplant might be feasible. A 3½-mile pressure conduit along the left bank of the North Fork Nehalem River at the 240-foot altitude would reach a favorable location for a penstock in sec. 32, T. 4 N., R. 9 W., where the river surface is estimated to be about 60 feet above sea level. The head would vary between 180 and 340 feet depending upon the water level in the reservoir. The purpose for which the reservoir is built would determine the type of water-power operation.

FISHHAWK CREEK DIVERSION TO COLUMBIA RIVER AND PUMPED STORAGE

The Nehalem and Columbia Rivers are separated by a distance of only about 8 air miles across Tunnel Ridge in the area of Birkenfeld on the Nehalem and Marshland on the Columbia. Water raised to an altitude of 600 feet in the Vernonia reservoir site would back up Fishhawk Creek toward the Columbia and thus shorten the distance between the two streams, and the water could be pumped into a storage site upstream on Fishhawk Creek. This arrangement would shorten the tunnel route and increase the head. This and other possible diversion sites are shown on plate 3.

FISHHAWK DAM AND RESERVOIR SITE

GENERAL FEATURES

The Fishhawk damsite is in sec. 29, T. 7 N., R. 5 W., on Fishhawk Creek, a tributary to Nehalem River. The water surface of the creek at the damsite is 670 feet above sea level and the drainage area is 11 square miles. An earthfill dam that would raise water to the 900-foot level would have a volume of about 3,300,000 cubic yards. The reservoir would store 45,000 acre-feet of water when filled. Table 12 shows the area and capacity data for this reservoir site. Runoff at the damsite is estimated to be about 23,000 acre-feet per year.

TABLE 12.—*Area and capacity of Fishhawk reservoir site (damsite in sec. 29, T. 7 N., R. 5 W.)*

Altitude (feet)	Area (acres)	Capacity (acre-feet)	Altitude (feet)	Area (acres)	Capacity (acre-feet)
670	0	0	860	400	25,000
700	5	75	900	600	45,000
740	40	975	940	840	74,000
780	130	4,000	980	1,220	113,000
820	240	12,000	1,020	1,670	172,000

GEOLOGY

By D. L. GASKILL

The geologic reconnaissance map of Fishhawk Creek and vicinity (pl. 4) covers a maturely dissected area of high relief athwart Tunnel Ridge which is the divide separating the Nehalem River-Fishhawk Creek drainage from the Columbia River drainage. The area contains no valuable mineral resources of record or material improvements other than a few unimproved access roads and an abandoned railroad grade across Tunnel Ridge. Much of the area is burned or logged over. The remainder is largely second-growth forest land.

BEDROCK

Bedrock is obscured over most of the area by vegetation and a deep residual mantle. Middle Tertiary sedimentary rocks of the area probably include the upper part of the Pittsburg Bluff Formation and units of the Scappoose Formation (Warren and Norbisrath, 1946). The upper part of the Pittsburg Bluff Formation consists of fine- and medium-grained tuffaceous shaly sandstone and tuffaceous shale. These beds are locally crossbedded, conglomeratic, and commonly contain carbonaceous and pumiceous material. The overlying Scappoose Formation is lithologically similar to the upper units of the Pittsburg Bluff, but grades upward into massive friable medium-grained micaceous sandstone. The sedimentary beds are locally overlain by the Columbia River Basalt. The Columbia River Basalt consists of dense and fragmental basaltic flows of varying thickness. Some of the flows are separated by pyroclastics or water-laid sediments.

STRUCTURE

The area was probably uplifted after deposition of the Scappoose Formation. Uplift was followed by a long period of erosion and possible subsidence, terminated by extrusion of the Columbia River Basalt in Miocene time. Subsequent to the close of volcanism, the region was warped into gentle northwest-southeast folds and subjected to periods of erosion separable into several epochs of stream adjustment (Warren and Norbisrath, 1946, p. 237).

The sedimentary strata are comparatively flat lying. The overlying basalt flows rest unconformably on an erosional surface of considerable relief. Remnants of basalt overlie sedimentary strata on Tunnel Ridge at 1,400 feet altitude and extend down to an altitude of 700 feet on Fishhawk Creek. Flow remnants also overlie sediments 1,200 feet below Tunnel Ridge on the Columbia River side. Some of these flows seem to have originated on Clatskanie Mountain, about 2 miles east of the area. Lineaments mapped on plate 2 are interpreted from aerial photographs. They represent hypothetical bedrock fractures and probably include some faults. Most of the bedrock is jointed to some degree and stream alignments usually parallel a joint set.

DAMSITE

The Fishhawk Creek damsite (pl. 4) is underlain by a thick near-horizontal bed of compact, very fine grained tuffaceous micaceous sandstone and siltstone (fig. 10). Downstream, near locality 1 (Fishhawk Creek and vicinity, pl. 4), these sediments contain calcareous concretions of fossil fragments and carbonaceous material as much as a foot in diameter. At locality 1 a thick bed of fine-textured tuff crops out and extends downstream. This weak bed apparently underlies the tuffaceous sand and siltstone at the damsite.

Basaltic flows unconformably overlie the tuffaceous beds on abutments and cross Fishhawk Creek at the upstream edge of the damsite

FIGURE 10.—Tuffaceous siltstone in bed of Fishhawk Creek, downstream from section *A–A'* (pl. 4). Photograph by D. L. Gaskill, 1957.

area (pl. 4). The basal flow is very agglomeratic and has tuffaceous clay matrix and lenses of clayey tuff and basaltic fragments. Finely crystalline basalt overlies the fragmental flow. A water-laid pebble to boulder conglomerate in a quartzose and tuffaceous sand matrix was observed locally at the base of the unconformity about 100 feet above Fishhawk Creek on the west abutment.

In the area of section A–A′ (pl. 4) on Fishhawk Creek, the sedimentary rocks are inconspicuously jointed. The few fractures are tight and widely spaced. Upstream, near the contact with the overlying volcanics, the sediments are conspicuously fractured by a well-defined system of tight, closely spaced joints. Near locality 1 (pl. 4) about one-fourth of a mile south of section A–A′, the stream beds are closely jointed and apparently faulted. The fault zone dips about 80° E. and seems to parallel the north-trending lineament plotted on the geologic map of the Fishhawk Creek damsite (pl. 4).

Many springs and points of seepage occur in the damsite area, mainly at the base of the Columbia River Basalt. Abutment slopes are largely obscured by vegetation, slope rubble, and some landsliding.

RESERVOIR SITE

The Fishhawk Creek reservoir site (pl. 3, and also Fishhawk Creek and vicinity, pl. 4) is largely covered by brush and cutover land. The floor of the reservoir site consists mostly of tuffaceous beds like those at the damsite, but includes some very finely laminated carbonaceous clayey siltstone or lacustrine sediments and less extensive areas of basalt and pyroclastic material. Very thick exposures of silty clay and fine-grained sandstone underlie agglomeratic basalt along the abandoned railroad grade on the west slope of the North Fork Fishhawk Creek. Volcanic boulders are exposed locally in the upper part of some thick tuffaceous clay tuff beds. Much of the basalt is highly fractured, locally faulted, and decomposed by weathering. Reservoir slopes are evidently subject to extensive slumping and landsliding.

TUNNEL RIDGE

Tunnel Ridge, between the Fishhawk Creek reservoir site and the Columbia River valley, is composed of sediments representative of the Scappoose Formation. Bedrock exposures along Tunnel Road and at localities 7–12 (pl. 4) along the abandoned railroad grade exhibit very thick massive beds of very fine- to medium- and coarse-grained friable, locally crossbedded micaceous sandstones composed of angular quartz grains, muscovite, biotite, some feldspar, and igneous rock fragments. Thin layers of claystone and conglomeratic lenses of igneous pebbles occur locally in these beds (fig. 11). The beds grade upward into fine silty sand and clay containing boulders of basalt.

At localities 15–19 (pl. 4) on the Columbia River slope, similar arenaceous sediments include lenses of coarse-grained carbonaceous pumiceous sandstone and finely laminated silty clay. At locality 13 and west along the abandoned railroad grade from locality 14, numerous exposures of massive siltstone and conglomeratic sandstone occur. The beds are either slumped or faulted near localities 15 and 16 along the lineament plotted here.

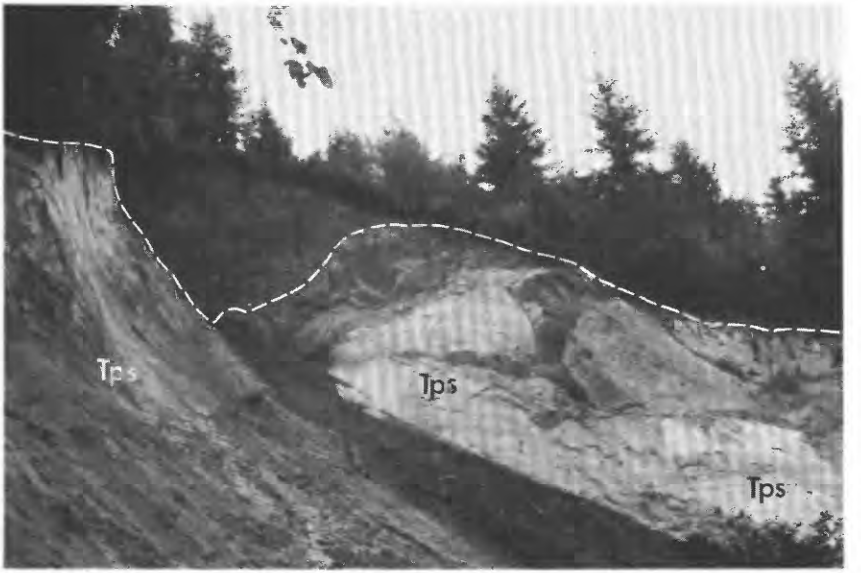

FIGURE 11.—Friable sandstone (Tps) overlain by basalt near crest of Tunnel Ridge (loc. 12, pl. 4). Photograph by D. L. Gaskill, 1957.

At intermediate altitudes along the north slope of Tunnel Ridge, between Eilertsen and Tandy Creeks, a topographic bench or terrace is capped with basalt. Sedimentary beds of Pittsburg Bluff lithology outcrop locally below the basalt and along the Columbia River Highway (U.S. 30) near Marshland.

CONCLUSIONS

The thick tuffaceous sedimentary bed flooring Fishhawk Creek at the damsite seems to have little or no intergranular cementation and probably owes most of its strength to compaction. After partial drying, such rocks tend to slake and disaggregate when acted upon by water and may be subject to slow plastic deformation under loads below their shearing strength (Meade, 1937). These rocks might be susceptible to differential settlement and rebound, but also may be relatively impervious and plastic enough to prevent formation of open joints. The basalt and interbedded pyroclastic flows are porous where

decomposed by weathering. In the damsite and reservoir area, these rocks are fractured and weakened by physical disintegration between breccia fragments or alteration zones in agglomerate. Interbedded lenses of tuff and flow contacts also constitute potential zones of weakness and permeability.

The erosional contact between the basaltic rocks and underlying sediments is probably the most critical zone of leakage and instability in the damsite area. On the west abutment, this erosional contact is infered to dip east at a moderately steep angle toward the damsite. On both abutments the basalt tends to fracture, slump, and landslide along this contact.

The bearing power and perviousness of both abutments seems to be vulnerable to reservoir pressure. The ridge between the reservoir area in sec. 30 and the creek in sec. 31, T. 7 N., R. 5 W. (Fishhawk Creek and vicinity, pl. 4) and the narrow spur forming the spillway site ridge (Fishhawk Creek damsite, pl. 4) should be explored for pervious zones, possibly along sandy beds, flow contacts, joints, or shear zones.

Topography limits the height of a dam at section *A–A′* (pl. 4) to about 200 feet. A cutoff wall might be required in foundation rocks and along the top of the east abutment.

A diversion tunnel under the Nehalem-Columbia divide at Tunnel Ridge would be largely in sedimentary rocks below the water table. The behavior of these rocks will vary greatly in respect to their porosity, permeability, water content, stratification, and structural deformation. The costs of tunnel construction depend mainly on the stand-up time of the material penetrated (Trask, 1950, p. 193). The physical properties of these sedimentary beds indicate conditions requiring continuous support and lining of a tunnel as excavation proceeds.

Impervious concrete aggregate and rockfill construction materials are probably available in the general area.

Foundation and abutment rocks at the Fishhawk Creek damsite are unsuitable for a concrete structure, but are adaptable to a flexible wide-base earth or rockfill dam.

POTENTIAL WATERPOWER OF THE DIVERSION PLAN

If diversion to the Columbia River is made, water probably will be raised in the Vernonia reservoir site to an altitude of 540 to 600 feet by constructing a dam at Squaw Creek, Tideport, or Elsie. Other damsites as far downstream as the Salmonberry River possibly could support dams to the 600-foot level; however, the three listed here are thought to be the best suited to the purpose. The volume of the dam, the area flooded, and the storage capacity desired would have considerable importance in the selection of the damsite.

Plate 3 shows tunnel routes at altitudes 540 and 800 feet and a section (*A–A′*) through Tunnel Ridge. The section illustrates a gravity surface-to-surface diversion at altitude 540 feet and also pumped-storage alternatives. The penstock could be on the surface or underground. If pumped storage is to be developed, pressure tunnels and an underground drop to the powerhouse site may be desirable in order to utilize head from the reservoirs. This pressurization could be done by connecting the tunnel at altitude 540 feet to the reservoir through a shaft or by pumping from the low reservoir into the high reservoir through a short tunnel or surface conduit and connecting the high reservoir with the Columbia River side of the divide by a tunnel at an altitude of 800 feet.

Alternative tunnel and conduit routes have been selected for illustrative purposes and are shown on the plan map in plate 3. One is at altitude 540 feet and begins in the NE¼ sec. 1, T. 6 N., R. 6 W., on Fishhawk Creek, passes under the deep part of Fishhawk reservoir site, and ends in the SW¼ sec. 8, T. 7 N., R. 5 W., on the south valley wall above the Columbia River near Woodson. This route is about 4.7 miles long. The other alternative is at altitude 800 feet and would conduct water from the Fishhawk reservoir through Tunnel Ridge to a point above Marshland. This route has been shown because the topography on the valley wall south of Marshland appears better suited for conduit and penstock routes. The penstock would be shorter and the powerhouse would be nearer the surge tanks. Tunnel routes at altitude 800 feet are shorter as the outlet end is moved toward section *A–A′* where it is only 1.8 miles from the intake as shown on the section in plate 3.

If pumped storage is added to the diversion scheme and the tunnel is at altitude 540 feet, water could be pumped into and discharged from the Fishhawk reservoir through a shaft connecting the tunnel and reservoir. The lift would range from 70 to 360 feet. An advantage of this plan is that it could operate directly from the Vernonia reservoir as a gravity unit when desired and thereby keep pumping to a minimum.

A purely pumped-storage development might also be effected by pumping water through a tunnel or surface conduit from the Vernonia reservoir to the Fishhawk reservoir. The lift would be between 200 and 360 feet because water below an altitude of 800 feet would be dead storage.

The amount of water available for diversion would depend upon the damsite chosen for the Vernonia reservoir. Some pertinent data on the various possibilities are shown in table 13.

Of the five sites, the Squaw Creek dam would give the most water per acre flooded by the reservoir. The Tideport site would give the most water per cubic yard of dam volume. If dam volume and area flooded are considered together, Tideport still gives the greatest unit volume of water, Elsie the second largest, and Squaw Creek the third largest. The Elsie site might be chosen because it would make more water available, and further examination may confirm its apparent geological superiority. The Elsie site is the most downstream point

TABLE 13.—*Water available for diversion from Nehalem River to Columbia River via Fishhawk Creek and Tunnel Ridge from Veronia reservoir with the dam at various sites*

Damsite	Dam		Reservoir		Divertible water (cfs) [2]	
	Height (feet)	Volume (cu yds)	Area (acres)	Capacity [1] (acre-feet)	Continuous	Nov.-Mar.
Squaw Creek	150	1,500,000	14,000	570,000	870	2,190
Tideport	163	1,500,000	19,600	800,000	1,185	2,900
Elsie	205	1,800,000	24,000	1,000,000	1,310	3,250
Spruce Run	295	5,200,000	29,000	1,270,000	1,550	4,065
Salmonberry	395	8,400,000	31,000	1,420,000	1,730	4,275

[1] Capacity of reservoir between altitudes 540 and 600 ft.
[2] Net water after conservation release and losses.

where a dam is considered to be feasible to raise the water to the 600-foot altitude. Squaw Creek would be chosen only if keeping the area flooded by the Vernonia reservoir to a minimum is desired. Because of the great volume of the embankments, high dams at Spruce Run and at damsites farther downstream probably are out of the question. Their value in storing a large quantity of water for use during disastrously dry periods seems to justify their inclusion in this report even though they are judged to be too costly to be feasible. Diversions from reservoirs behind dams at the Tideport or the Squaw Creek site would give proportionally less power than the reservoir and dam at Elsie; nevertheless, their peaking capacities would be very great.

Sample operations of a diversion plan using the amounts of water available for diversion from the Vernonia reservoir and a dam built at the Elsie site follow.

The active storage capacity of the Vernonia reservoir behind a dam at the Elsie site is 1,000,000 acre-feet (the capacity between altitudes of 540 and 600 ft.). Assuming a full reservoir on June 1, 1940, the amount of water which could have been diverted for various time periods and the corresponding power in kilowatts and energy in kilowatt-hours which could have been produced by an all-gravity diversion are shown in the following table. Computations are based on an efficiency of 80 percent. The average head is considered to be 575

Time of use	Net water (cfs)	Power	
		Kw	Kwhr
Continuous	1,300	50,800	445,000,000
Combination:			
April–October	470	18,400	95,000,000
November–March	2,645	103,000	373,000,000
Total combination			468,000,000
November–March only	3,280	128,000	465,000,000

feet, the approximate elevation of the Vernonia reservoir when 500,000 acre-feet have been drawn out.

For comparison with the gravity diversion, the following table presents the corresponding data for the all-pumped-storage scheme. The average head is considered to be 860 feet, which is the water-surface elevation the Fishhawk reservoir would have when half of the 38,000 acre-feet stored above the 800-feet level is drawn out. Computations are for an efficiency of 80 percent.

In total kilowatt-hours, the continuous generation plan produces 95 percent of the highest (the combination plan), but all three plans are very nearly equal in total output.

The discharge rates in the table are continuous during the specified period. If operated for peaking purposes, concentration of daily generation into one or two periods of several hours each would be desirable. For this type of operation the water available and the kilowatt output vary inversely with the length of the daily operating periods. The kilowatt-hour output would remain the same as long as all the water is used. For example, 6,560 cfs combined with appropriate installations in the powerhouse could produce 384,000 kw for 12 hours per day on the November–March only plan, or for 9½ hours per day from November–March on the combination plan. Use of 13,120 cfs together with an appropriate powerhouse could produce 768,000 kw for 6 hours per day on the November–March only plan, or for 4¾ hours per day from November–March on the combination plan. Capacities and operating schedules could thus be varied to fit many situations.

Time of use	Net water (cfs)	Power	
		Kw	Kwhr
Continuous	1,300	76,000	666,000,000
Combination:			
April–October	470	27,500	141,000,000
November–March	2,645	155,000	562,000,000
Total combination			703,000,000
November–March only	3,280	192,000	696,000,000

Modern pumped-storage plants expend about 1.4 kw of offpeak power for every kilowatt of peaking power when pumping head is equal to power head (Hammond, 1958). The Ffestiniog pumped-storage station recently constructed in North Wales is designed to operate at about this efficiency. It contains four pumps, each rated to deliver 745 cfs under a head of 1,000 feet at the expenditure of 93,600 hp (Water Power, 1961). If returned at the same rate, the water will produce 68,000 hp at 80 percent efficiency, or 1 hp for every 1.38 hp used in pumping.

The power head is sufficiently greater than the pumping head in the Fishhawk scheme to make at least 2 kw of peaking power available for each kilowatt used for offpeak pumping. This feature of the site is attractive.

Whether development of a pumped-storage diversion at this site would be economical is beyond the scope of this report, as is the determination of the economical size of tunnels and installation capacities. The economic feasibility of pumped-storage additions to power systems is well established, however, and this one should be investigated upon its individual merits by companies or agencies concerned with develpment of waterpower in the area.

OTHER PUMPED-STORAGE POSSIBILITIES

The Nehalem River from Humbug Creek to Nehalem Falls flows in a narrow canyon where bluffs as high as 2,000 feet above sea level are nearby. There are small lakes and reservoir sites into which water could be pumped for production of peaking power. Auxiliary pumped-storage plants could be added to any of the power developments discussed in this report. Table 14 presents the potentialities of several of these pumped-storage sites. The data of the table have been chosen to bring out characteristics of the sites such as reservoir capacities, head, quantities of water to be handled by pumps and conduits, and generating capacities. The reservoir capacities were estimated from measurements of area and depth shown on the 1 : 62,500-scale USGS topographic maps.

DIVERSION TO TUALATIN RIVER BASIN

The Tualatin River basin, especially in the growing Portland suburban area, is approaching the point at which it will need additional water. The Nehalem, Wilson, and Trask Rivers are possible sources for this water and there are several sites in the headwaters of each where diversions to the Tualatin might be made. Diversion of an average of about 700,000 acre-feet annually from the three streams combined seems possible. As much as 280,000 acre-feet

TABLE 14.—*Sites for pumped-storage developments in Nehalem River basin*

Site	Reservoir					Operating 9 hours per day [1]		
	Location			Capacity (acre-feet)	Head (feet)	Cfs	Kw [2]	Kwhr
	Sec.	T.N.	R.W.					
Lost Lake	17	4	7	900	1,170	1,210	96,000	864,000
Spruce Run Lake	17	4	7	1,000	640	1,345	59,000	531,000
Little Falls	30	4	7	800	1,220	1,075	89,000	801,000
Helloff Creek	11	3	9	1,500	950	2,017	130,000	1,170,000
Lost Creek	25	3	9	9,000	300	12,100	247,000	2,223,000
Fall Creek	16	3	9	1,500	560	2,017	77,000	693,000

[1] Filling and emptying the reservoir every 24 hours by pumping 15 off-peak hours and generating 9 hours.
[2] Computed at approximately 80 percent efficiency.

(the entire yield above the diversion points) could be diverted annually from the Nehalem River if reservoirs were built to regulate the river and to store the water from the November–March rainy period until mid-summer and early fall when its value is highest.

Some of the possible diversion routes from the Nehalem River, to the Tualatin River, all originating within the Rocky Point reservoir site, are shown on plate 3. The Rocky Point reservoir should be larger for a diversion plan than for providing water for use only within the basin. It is estimated that an active capacity of 200,000 acre-feet would be necessary to divert the entire yield and that some water would be diverted from Rock Creek, a downstream Nehalem River tributary.

The Rocky Point reservoir site has a capacity of about 240,000 acre-feet below altitude 845 feet and would contain about 210,000 acre-feet between altitudes 750 and 845 feet. The reservoir would be exceptionally valuable for recreation and conservation and therefore the amount of unused water below altitude 750 feet would be justified.

The diversion to Rocky Point reservoir from Rock Creek might best be accomplished by means of a dam on Rock Creek in sec. 14, T. 4 N., R. 6 W. (Camp McGregor), and a 1.2-mile tunnel at altitude 1,120 feet to Clear Creek. Clear Creek would carry the diverted water to the Rocky Point reservoir.

Drainage area above the diversion point on Rock Creek is about 32 square miles and 84 percent of that area is above the Keasey gage. Unit runoff would be higher at Camp McGregor and an additional 5 percent has been allowed for this. Calculations from table 1 indicate an average annual runoff of 109,000 acre-feet for water years 1940–54. Adding this to the annual average for Rocky Point (171,000 acre-ft) brings the total yield of the two areas to 280,000 acre-feet. An average of 387 cfs, less losses and necessary conservation releases,

would have been available for diversion from the Rocky Point reservoir had it been in operation for that purpose during the 1940–54 water-year period. The median year is 1947 with an average discharge of 393 cfs. Range is from 230 cfs (1941) to 510 cfs (1950).

Diversion could be made at altitude 750 feet to West Fork Dairy Creek (Manning tunnel route) or to Gales Creek by way of Beaver Creek (Glenwood tunnel route). The tunnels would be 3.7 and 3.6 miles long, respectively. Shorter tunnels would be needed if the water were pumped to a higher altitude. At altitude 1,040 feet the tunnel length could be as short as 0.5 mile to West Fork Dairy Creek. The tunnel would go under the divide near the present railroad tunnel but at a lower altitude than the railway. An even shorter tunnel at altitude 1,040 feet would put the water into Beaver Creek. Several alternative routes which would have similar tunneling requirements exist but are not shown. Conduits can be extended from the tunnels to power drops if it is found desirable to develop power for pumping of water or for other uses. At West Fork Dairy Creek, an extension of about 1 mile would connect the tunnel at altitude 750 feet with a point where the water could be dropped 400 feet into Manning reservoir site. An extension of about 2 miles would connect the tunnel at altitude 1,040 feet to a 700-foot drop site into the same reservoir site. The possible power drops into Beaver Creek would be 120 and 420 feet, respectively, from the 750- and 1,040-foot altitude tunnels to the Glenwood reservoir site. These heads are the difference between the estimated conduit terminal altitudes of 720 and 1,020 feet and the estimated full reservoir altitudes of 320 feet for the Manning site and 600 feet for the Glenwood site. If the Beaver Creek balancing reservoir is constructed there would be no drop from the low-level tunnel and only 300 feet of head from the high-level tunnel.

Provision of equalizing storage on the Tualatin River basin side of the divide would be desirable, and there are sites for doing this. The choice of whether to bring the water into West Fork Dairy Creek or divert it by way of Beaver Creek into Gales Creek might depend on the area of use. Both streams pass near Forest Grove, but if it were desired to deliver the water to the area around North Plains, the West Fork Dairy Creek Diversion might be preferred. Gales Creek appears to offer an advantage insofar as damsites and cultural developments in the reservoir site are concerned. The reach of Gales Creek between Glenwood and the open valley possesses damsites that appear to be relatively good. A site in sec. 23, T. 2 N., R. 5 W., about half a mile downstream from the settlement of Glenwood (Glenwood site) may be the best. On West Fork Dairy Creek the dam would probably have to be constructed in sec. 10, T. 2 N., R. 4 W., at

the settlement of Manning (Manning site). The Manning reservoir site appears larger than the Glenwood site, but existing improvements in it are considerably greater. Relocation requirements in the Manning reservoir site would include parts of highways U.S. 26 and State 47, several miles of the Spokane, Portland and Seattle Railway line, and the settlement of Buxton. The Beaver Creek reservoir site shown on plate 3 probably should be regarded as an alternative to the Glenwood site rather than a possible addition to it.

Existing maps are not suitable for determining the capacities of these reservoirs. The capacity is roughly estimated as 75,000 acre-feet for Manning and 30,000 acre-feet for Glenwood. No estimate was made of the Beaver Creek site.

Table 15 shows the potential power of the average dependable divertible water. The losses from the Camp McGregor and Rocky Point reservoirs would total about 30 cfs, continuously. By adding a 10-cfs conservation release to this 30 cfs and subtracting the total estimated loss from 387 cfs (the indicated gross annual average divertible water) a dependable average discharge of 347 cfs is obtained.

TABLE 15.—*Potential power and energy of 347 cfs for several Nehalem River to Tualatin River diversion routes*

[Computations at 80 percent efficiency]

Site	Head (feet)	Power (kw)	Energy (kwh per yr)
Tunnel at 750 ft:			
Manning	400	9,440	82,700,000
Glenwood	120	2,830	24,800,000
Tunnel at 1,040 ft:[1]			
Manning	700	16,500	[2] 145,000,000
Glenwood	420	9,910	[2] 87,000,000

[1] Pump-conduit diversion.
[2] Includes about 48,000,000 kwh per yr that would be required to pump the water from Rocky Point reservoir to altitude 1,040 ft.

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U. S. GOVERNMENT PRINTING OFFICE : 1965 O - 773-641

Waterpower Resources and Reconnaissance Geology of Sites in the Alsea River Basin Oregon

By L. L. YOUNG, D. W. NEAL, *and* D. L. GASKILL

WATERPOWER RESOURCES OF THE UNITED STATES

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1610-D

An estimate of the potential power of the river and a discussion of the possibilities for developing it

UNITED STATES GOVERNMENT PRINTING OFFICE, WASHINGTON : 1966

UNITED STATES DEPARTMENT OF THE INTERIOR

STEWART L. UDALL, *Secretary*

GEOLOGICAL SURVEY

William T. Pecora, *Director*

For sale by the Superintendent of Documents, U.S. Government Printing Office
Washington, D.C. 20402

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WATERPOWER RESOURCES OF THE UNITED STATES

WATERPOWER RESOURCES AND RECONNAISSANCE GEOLOGY OF SITES IN THE ALSEA RIVER BASIN, OREGON

By L. L. YOUNG, D. W. NEAL, and D. L. GASKILL

ABSTRACT

The Alsea River basin is a sparsely settled section in the central part of the Oregon Coast Range. The only major roadway through the basin is State Highway 34. The primary economic activity is, and probably will continue to be, the production of lumber and associated products although scenic and recreational activities are quite important.

The drainage area of the Alsea River basin is 473 square miles, and the annual runoff averages about 66 inches. Runoff follows precipitation closely in time and both are concentrated in the late fall and winter months. No storage reservoirs have been developed in the basin, but several good sites are present. The technically feasible power of the basin, estimated from an illustrative plan of development containing six sites, based on gross head, average discharge, and 100 percent efficiency, is 61,000 kilowatts. One site, the Scott Mountain, accounts for 34,000 kw and a second site, Tidewater, 11,000 kw.

A preliminary geologic examination of the Scott Mountain and Tidewater sites indicates that a number of problems must be investigated further before the location and height of the dams can be definitely selected.

There are many possibilities for the development of pumped storage in connection with the two sites selected for illustrating potential power. Sites with high head and small high reservoirs or sites with low head and large storage reservoirs might be developed.

There are two possible sites for diverting Alsea River water to the Willamette River basin, one in the headwaters area of the North Fork Alsea River, and the other in the headwaters area of the South Fork Alsea River.

INTRODUCTION

PURPOSE AND SCOPE

This report presents an evaluation of the waterpower potential of the Alsea River basin. The principal features for consideration include topography, geology, precipitation, streamflow, temperature, and evaporation. The power estimates are based on an as-

sumed plan of development considered adaptable to this basin. Actual development may not follow the plan outlined, but the plan serves as a means of estimating the potential power.

PREVIOUS INVESTIGATIONS

A report, "Tidewater Reservoir Site, Alsea River, Oregon," was prepared by R. O. Helland in 1945 and is in the Geological Survey open file. A supplement was added in 1952.

A report, "Waterpower of the Coast Streams of Oregon," by R. O. Helland was released on open file in 1953. This report gives general information plus a short discussion of nine coast streams including the Alsea River.

POWER VALUE OF OREGON COAST STREAMS

The concentration of runoff during late fall and winter makes the coast streams of Oregon potentially valuable for furnishing firming power because most hydropower in the Northwest is produced on the Columbia River and its tributaries which have their low flows during this period. The Columbia reaches a peak in June when approximately 20 percent of the annual runoff occurs. From September through February, average monthly runoff of the Columbia River ranges from 4 to 5 percent of the annual average and is barely more than 25 percent for the 6-month period. The coast streams could be developed to concentrate power production in this critical period, and could thus increase the prime power of the Northwest power system.

GEOGRAPHY

LOCATION AND DESCRIPTION

The Alsea River basin is in Lincoln, Benton, and Lane counties on the central Oregon coast. The river is formed by its North and South Forks, which join at an altitude of 285 feet in sec. 1, T. 14 S., R. 8 W., near the settlement of Alsea. From this point the river flows west some 40 miles to its mouth at Alsea Bay near the town of Waldport. The major tributaries entering the Alsea River downstream from the junction of its forks are Fall Creek, Five Rivers, and Drift Creek. Major features of the Alsea River basin are shown in figure 1 on which isohyets have also been shown.

The Alsea River drains an area of 473 square miles. The drainage areas of its principal tributaries are as follows:

	Square miles
North Fork	63
South Fork	50
Fall Creek	30
Five Rivers	120
Drift Creek	70

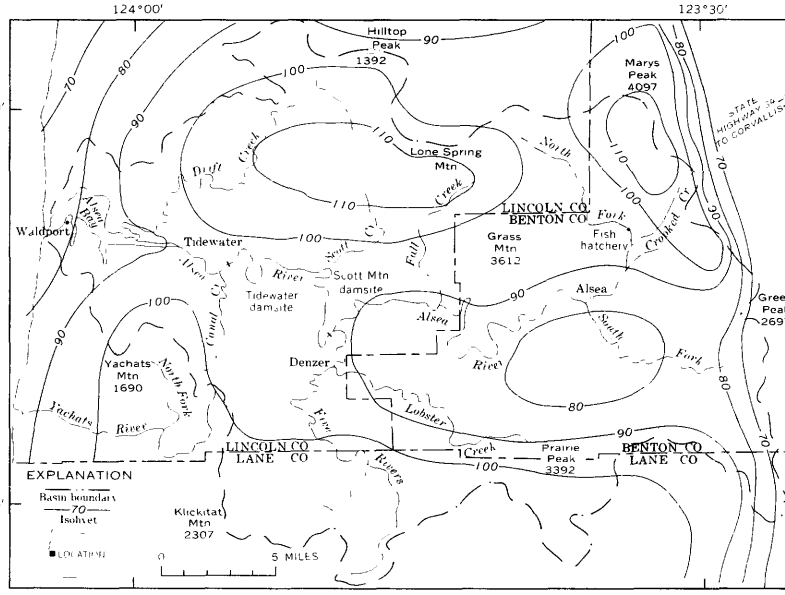

FIGURE 1.—Alsea River basin, Oregon.

The mouth of Fall Creek is in sec. 1, T. 14 S., R. 9 W.; Five Rivers joins the river in sec. 7, T. 14 S., R. 9 W., and Drift Creek in sec. 27, T. 13 S., R. 11 W. The basin is principally mountainous and rough, but valley bottoms are larger and smoother than those of many coast streams.

The North Fork heads on the slopes of Marys Peak, which at 4,097 feet in altitude is the highest point in the basin. Other altitudes in the North Fork basin range from 2,000 to 3,000 feet. Peak Creek, the principal tributary of the South Fork, drains the western slopes of Green Peak, which rises to nearly 2,700 feet in altitude. Other hilltops in the South Fork basin range from 1,800 to 2,000 feet in altitude. Yachats Mountain, within 3½ miles of the coast, on the south boundary of the basin is 1,700 feet in altitude. Highest altitude near the coast north of the river is about a thousand feet above sea level. In general, however, the drainage basin is higher north of the river than south of it. Tidewater extends about 14 miles up the Alsea River. The fall in the river between the junction of the North and South Forks and the upper limit of tidewater averages about 8 feet per mile. The adjacent drainage basins are Marys River on the northeast, Yaquina River on the north and northwest, Yachats River on the southwest, and Siuslaw River on the south and southeast.

MAPS AND AERIAL PHOTOGRAPHS

The Alsea River basin is covered by U.S. Geological Survey topographic quadrangles at a scale of 1:62,500 and a contour interval of 50 feet. They are the Toledo, Marys Peak, Monroe, Alsea, Tidewater, Waldport, Mapleton, Blachly, and Elmira quadrangles which were mapped in 1956 and 1957.

A map sheet by the Geological Survey dated 1957 and entitled "Miscellaneous Damsites, Coast Streams, Oregon," includes the Scott Mountain and the Tidewater damsites on the Alsea River. These were mapped on a scale of 1:4,800; a contour interval of 10 feet on land and 1 foot on the water surface was used.

The Salem sheet (1963) of the U.S. Army Map Service, 1:250,000 scale and a 200-foot contour interval, covers the basin. This map is printed and distributed by the Geological Survey. The Siuslaw National Forest map, which shows the Alsea River basin, is available from the U.S. Forest Service.

Aerial photographs of this region are available. Information concerning areas covered may be obtained from the Map Information Office, U.S. Geological Survey, Washington, D.C., or from the Forest Service in Portland, Oreg.

LAND USE

The primary land use in the Alsea River basin is timber production. The basin is covered by forest growth and heavy vegetation, and little clearing has been done because the steep terrain is not adaptable to agriculture. The valley is wider in the vicinity of Alsea, and much of the farming activity within the basin is in this area. The most common use of the open land is for grazing of cattle and sheep. The river valley is narrow below the mouth of Fall Creek, and there are only a few scattered farms. Some farms are located up tributary streams such as Five Rivers and Fall Creek.

Scenic and recreational activities are important to the region, and a considerable part of the economy depends on hunters and fishermen and on people vacationing on the beaches. Many permanent homes, vacation cabins, and tourist facilities are located along the river. Most of these are between the settlement of Tidewater and the mouth. Some salmon spawn in the basin, and the river has been partly cleared of obstructions to facilitate fish migration.

TOWNS AND ROADS

The ocean-front town of Waldport is the largest within the basin and is at the mouth of the Alsea River. The settlement of Tide-

water is about 1 mile by road and 3 miles by river downstream from the upper limit of tidewater. Alsea, the second largest town, is in a valley at and downstream from the confluence of the North and South Forks. Other small villages include Bayview on Alsea Bay and Fisher on Five Rivers. Scattered farms, churches, homes, and businesses are located throughout the basin. State fish hatcheries are in operation on North Fork Alsea River and Fall Creek. The population of Waldport was 689 in 1950, 667 in 1960, and 715 on July 1, 1961. The other towns were too small to be listed but the Waldport and Alsea census divisions had 3,413 and 857 persons respectively in 1960. These figures approximately represent the population of the Alsea River basin.

State Highway 34 is the major roadway through the basin; it connects the Willamette valley cities such as Corvallis and Albany with the ocean beaches. This surfaced highway follows the Alsea River, North Fork Alsea River, and Crooked Creek to the Coast Range summit and then northeast where it joins U.S. Route 20 near Philomath. Highway 34 is a narrow, rather crooked two-lane road, but it is being improved. The Oregon Coast Highway (U.S. Route 101) crosses the Alsea River at Waldport on the Alsea Bay Bridge, the largest and most impressive structure within the basin. These routes are important to the region because they bring an annual surge of tourists and provide access to other coastal areas.

Short parts of the Lobster Valley Road south of Alsea and the Bayview Road on Alsea Bay are surfaced. The remainder of the roads in the basin are unpaved access, farm-to-market, and logging roads.

CLIMATE

The climate of this region is summarized briefly as wet and mild. Extremes of temperature are uncommon, temperature seldom exceeding 90°F in summer, and there are few extended freezing periods in winter. The average temperature at Tidewater is 52.7°F, and the temperature range is 8°F–105°F. Weather Bureau records beginning in 1948 show the number of days that certain temperatures prevail. At the Tidewater station this interval averaged 225 days for the 32°F temperature, 284 days for the 28°F temperature, and 338 days for the 24°F temperature. Since 1955 a similar column is listed in Weather Bureau records for temperatures of 20°F and 16°F, but none have been recorded for both spring and fall of the same year.

WATER SUPPLY

Measurements of discharge in the Alsea River basin began in 1939 at a station near Tidewater. In 1957 and 1958, six new stations were added to the basin's network as follows: North Fork Alsea River at Alsea, South Fork Alsea River near Alsea, Fall Creek near Alsea, Five Rivers near Fisher, Drift Creek near Salado, and Needle Branch near Salado. The last named station has a drainage area of only 0.32 square mile and will not be used in this report.

In order to extend these short records to cover the 1940–60 period during which the river was gaged near Tidewater, precipitation and runoff data for stations in and near the Alsea River basin were analyzed as shown in table 1 so that short-term records could be compared with those for longer periods. The stations outside the basin chosen for this purpose are Newport, Corvallis, and Summit. The Newport station is near the coast. The Corvallis station is north of the city of Corvallis and northeast of the basin, and it is operated by Oregon State University. The Summit station is at a pass on the Coast Range north of the Alsea River basin.

According to the results obtained, the period of record of discharge of Alsea River near Tidewater (1940–60) approximately represents average conditions and therefore makes possible the assumption that average runoff over that 21-year period equals the long-term average. Runoff at the Tidewater gaging station averaged 62.9 inches during that period. Measured runoff in the basin subparts ranged from a low of 42.6 inches on the South Fork to a high of 76 inches (partially estimated) on Fall Creek. Long-term averages as indicated by the precipitation stations included in table 1 and the runoff records of the Alsea River near Tidewater range from a low of 41 inches on the South Fork to 74 inches for Fall Creek. Precipitation records at the Alsea fish hatchery date to 1955. Measured precipitation there averaged 97.3 inches for the 1955–60 period and 91 inches is the indicated projected average. At Tidewater an average of 96.8 inches of precipitation annually between 1944 and 1960 is reduced to 88 inches for the projected average.

The gages at the Alsea fish hatchery and at Tidewater measure precipitation at only two points and by themselves do not accurately represent the amount of precipitation that produces the runoff throughout the drainage areas. In order to estimate the

TABLE 1.—*Precipitation and runoff data for stations in or near the Alsea River basin, and estimated long-term runoff in basin subparts*

	Long-term average		Projected average		1940–60		1944–60		1955–60		1958–60	
	Actual		Indicated									
	Inches	Index	Inches	Index	Inches	Index	Inches	Index	Inches	Index	Inches	Index
Precipitation data												
Precipitation stations:												
Newport [1]	66.1	100	(2)	(2)	66.5	100.5	68.3	103.3	73.7	111.5	74.0	111.9
Corvallis [3]	37.3	100	(2)	(2)	38.5	103.2	39.1	104.8	41.0	109.8	39.4	105.6
Summit	66.1	100	(2)	(2)					67.4	101.9	67.9	102.6
Tidewater			88	100			96.8	110.0	93.4	106.1	91.8	[4] 103.8
Alsea fish hatchery			91	100					97.3	106.5	95	[4] 103.8
Runoff data												
Gaging stations:												
Alsea River near Tidewater	62.9	100	63	100	62.9	100	63.3	100.5	67.3	106.9	66.1	105.1
North Fork			56	100							58.0	[4] 103.8
South Fork			41	100							42.6	[4] 103.8
Fall Creek			74	100							[5] 76	[6] 102.6
Five Rivers			65	100							[5] 70	[7] 108.5
Drift Creek			71	100							[5] 73	[6] 102.6

[1] Records date to 1892.
[2] Same as figures under long-term average.
[3] Records date to 1890.
[4] Assumed to vary with average of Summit rainfall and Alsea River streamflow.
[5] Estimated.
[6] Assumed to vary with Summit rainfall.
[7] Assumed to vary with Newport rainfall and Alsea River Streamflow.

runoff from the basin subparts, precipitation and runoff measurements for stations in and near the basin were compared as shown in table 1. The runoff in inches shown in table 2 for the basin subparts are estimates made by using the relations established in table 1.

TABLE 2.—*Estimated average runoff for the Alsea River basin, by subparts*

Subbasin	Area (sq mi)	Runoff (in.)
North Fork	63	56
South Fork	50	41
Fall Creek	30	74
Five Rivers	120	65
Alsea River near Tidewater	334	63
Drift Creek	70	71
Remainder of basin	69	75
Alsea River basin	473	66

The short-term records for the gaging stations given in table 1 are summarized and the projected long-term averages, in cubic feet per second, are shown in table 3.

Rainfall and runoff are concentrated in the late fall and winter. The gages near Tidewater recorded 73 and 80 percent, respectively, of the annual precipitation and runoff during the November–March period. From April through October the complementary percentages of average annual precipitation and runoff are 27 and 20.

The concentration of runoff during fall and winter enhances the value of the coast streams for supplying power during a period of high demand. On the Alsea River about 80 percent of the annual runoff occurs between November 1 and March 31. Figure 2 illustrates the concentration of flow in the Alsea River during the 5-

TABLE 3.—*Summary of streamflow data, Alsea River basin, Oregon, and estimated long-term average discharges*

Stream	Gaging station	Period of record [1]	Drainage area (sq mi)	Discharge, in cubic feet per second			Projected long-term average (cfs)
				Average	Maximum	Minimum	
North Fork Alsea	At Alsea	10/57–9/61	63	286	8,820	13	260
South Fork Alsea	Near Alsea	10/57–9/61	49.5	167	4,340	7.2	150
Fall Creek	do	8/58–9/61	29.4	173	2,970	5.9	160
Five Rivers	Near Fisher	8/58–9/61	114	566	15,700	17	545
Alsea River	Near Tidewater	10/39–9/61	334	1,560	32,200	56	[2] 1,547
Drift Creek	Near Fisher	9/58–9/61	20.6	127	2,300	3.8	110

[1] 1961 included to improve usefulness of short records.
[2] 1940–60, actual.

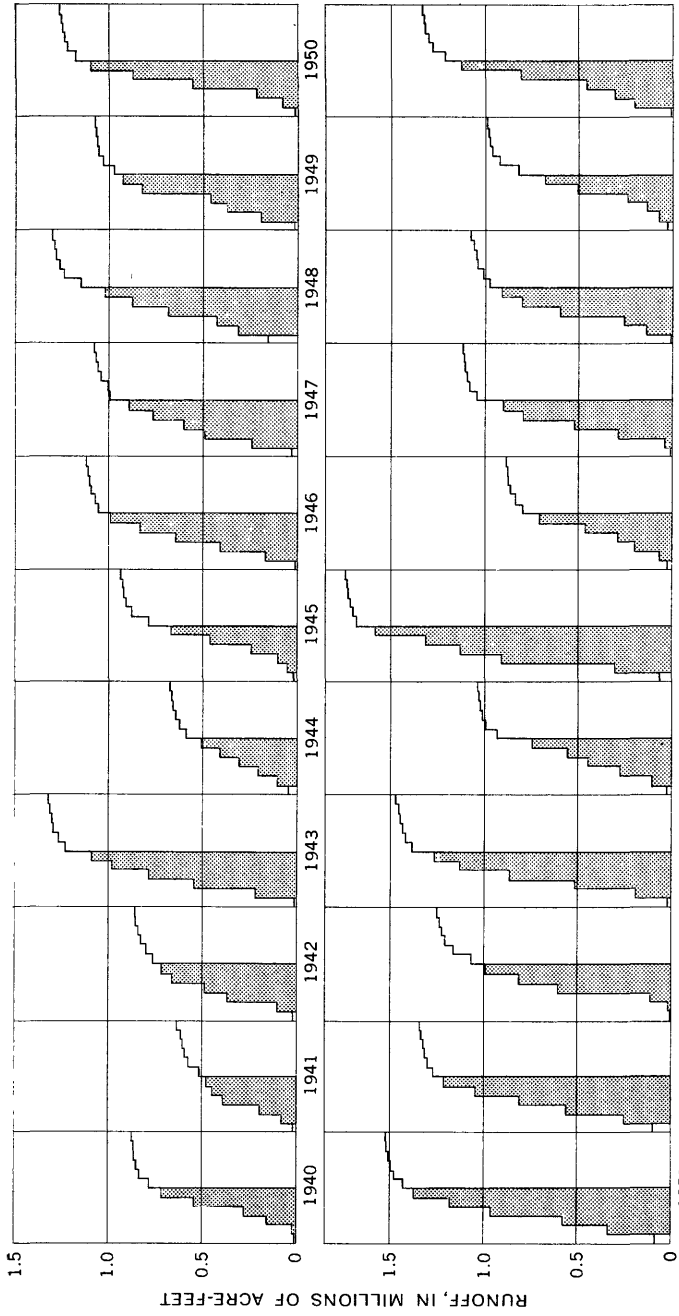

FIGURE 2.—Runoff in acre-feet accumulated monthly for water years 1940–61 for gage near Tidewater.

month period November through March (the shaded part of the yearly accumulation) for water years 1940–61. The water year begins October 1 and ends September 30. At first glance there might be a question as to whether the month of November or the month of April should be included in the 5-month high-yield period. However, the November yields are enough larger than those of April to be quickly verified by visual inspection. Actually, the November yield was greater than April during 17 of the 22 years shown. More detailed studies will probably show that the 5-month period should be started sometime during November and extended appropriately into April to obtain the maximum benefit. Wet and dry seasons as well as wet and dry years and the sequence of such wet and dry years are also illustrated in figure 2.

WATER RIGHTS

Records in the office of the state engineer in Salem, Oreg., show that 12.44 cfs (cubic feet per second) have been allotted from the Alsea River and its tributaries. Irrigation allotments were 83 percent of this total, and domestic use and fish culture were 9 percent and 8 percent respectively. About two-thirds of the allotment was from the main stem and one-third from tributary streams. It does not seem likely that water allotments will endanger the supply of water for power. The lowest flow recorded at the gaging station near Tidewater was 56 cfs.

STORAGE SITES

No storage reservoirs have been devloped in the basin. There are several potential storage sites, however, and many benefits would accrue from any measures taken to augment low summer flows and reduce excessive flows during the months of high precipitation. Available topographic maps are not suitable as to scale and contour interval for obtaining accurate reservoir capacities. However, approximate areas and capacities for several sites were obtained from 15-minute quadrangle maps, and this information follows the related discussion (p. D11–D15).

NORTH FORK

Small storage reservoirs could be built on the North Fork and its tributaries, and the capacity of one of these, the County Line site, is shown in the following tabulation. This site will be valuable if water is diverted to Marys River from North Fork. According

to the Alsea topographic quadrangle, scale 1:62,500, the dam would be about a thousand feet long at the 1,000-foot altitude. The drainage area is 25.3 square miles.

Area and capacity of County Line reservoir site, North Fork Alsea River

[Damsite in sec. 13, T. 13 S., R. 8 W.]

Altitude (feet)	Area (acres)	Capacity (acre-feet)
810	0	0
850	30	600
900	100	3,850
950	180	10,900
1,000	340	24,000

SOUTH FORK

The South Fork has some storage sites that appear to be reasonably good topographically. One of these, Peak Creek, is far enough upstream that it, also, might be used in a plan to divert water to the Willamette River basin side of the Coast Range. The area and capacity of this site are as follows:

Area and capacity of Peak Creek reservoir site, South Fork Alsea River

[Damsite in sec. 23, T. 14 S., R. 7 W.]

Altitude (feet)	Area (acres)	Capacity (acre-feet)
675	0	0
700	30	375
750	50	2,375
800	130	7,000
850	590	25,000
900	1,390	74,000

The 1:62,500-scale Alsea topographic quadrangle map indicates that the Peak Creek dam would be about 1,800 feet long at the 900-foot altitude. The drainage area is 30 square miles.

Reservoirs on the North and South Forks would probably be more valuable to farming, fish culture, and recreation than to power because of the small drainage areas affected. They would almost surely be built if water is diverted from these streams to the Willamette River basin tributaries on the east side of the Oregon Coast Range.

FALL CREEK

Fall Creek passes through valleys in which it appears possible to build small reservoirs that would have value for sustaining flows in aid of fish culture. None have been studied.

FIVE RIVERS

There are sites in the lower basin that probably could support rather high dams. In downstream order, they are on Five Rivers, the main stem, and Drift Creek. The Five Rivers damsite might be placed in sec. 18, T. 14 S., R. 9 W. The Tidewater topographic quadrangle, scale 1:62,500, indicates that the dam would be about 1,400 feet long at an altitude of 400 feet. The drainage area is 120 square miles. The area and capacity of this reservoir site are as follows:

Area and capacity of Five Rivers reservoir site, Five Rivers

[Damsite in sec. 18, T. 14 S., R. 9 W.]

Altitude (feet)	Area (acres)	Capacity (acre-feet)
80	0	0
100	40	400
150	470	13,150
200	1,560	63,900
250	3,030	178,700
300	5,320	387,000
350	7,280	702,000
400	9,520	1,122,000

Five Rivers and its tributaries flow in deeply cut valleys, and there are many alternative reservoir sites. The reservoir that would be created by the Scott Mountain dam on the main stem would also inundate the valley bottoms in Five Rivers and its tributaries.

MAIN-STEM SITES

Immediately downstream from Five Rivers, the Alsea makes a long mule-shoe bend around Stoney Mountain, a southward extension of Scott Mountain, and topography is favorable for a rather high dam. The Scott Mountain damsite in sec. 18, T. 14 S., R. 9 W., discussed in the section on potential power, was surveyed in 1957. The area and capacity of the reservoir site are shown in figure 3 and in the following tabulation.

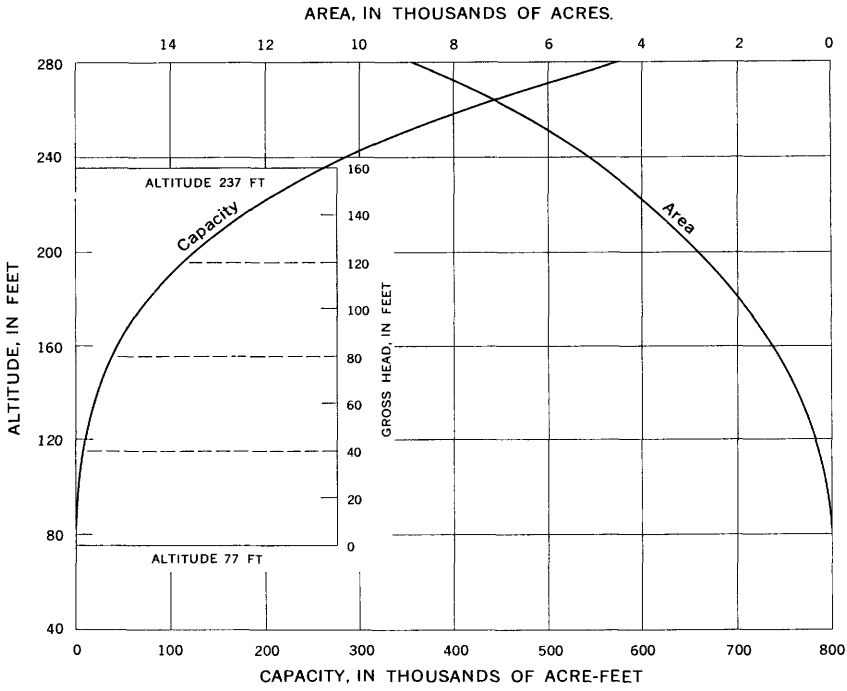

FIGURE 3.—Area and capacity of Scott Mountain reservoir site.

Area and capacity of Scott Mountain reservoir site, Alsea River

[Damsite in sec. 18, T. 14 S., R. 9 W.]

Altitude (feet)	Area (acres)	Capacity (acre-feet)
77	0	0
100	115	1,320
150	950	28,000
200	2,840	123,000
250	5,800	339,000
300	11,700	776,000

The damsite map, scale 1:4,800, indicates that a dam at the Scott Mountain site would be about a thousand feet long at an altitude of 237 feet (the height chosen in the illustrative plan discussed in the section on potential power). The drainage area is 321 square miles.

The capacity of the Scott Mountain reservoir site is 339,000 acre-feet at an altitude of 250 feet. The capacity of the Tidewater reservoir site at this same altitude is 724,000 acre-feet. The

Tidewater damsite is 13 miles downstream from the Scott Mountain damsite. The Tidewater damsite is a little below the tidewater limit, but there are other locations within a few miles upstream that appear to be about as good. The area and capacity of the Tidewater reservoir site follow and are also shown in figure 4.

Area and capacity of Tidewater reservoir site, Alsea River

[Damsite in sec. 32, T. 13 S., R. 10 W.]

Altitude (feet)	Area (acres)	Capacity (acre-feet)
7	0	0
50	695	15,000
100	1,590	72,000
150	3,000	187,000
200	5,000	387,000
250	8,500	724,000

The Tidewater damsite map shows that a dam would have a crest length of about 550 feet at an altitude of 77 feet. If continued to an altitude of 250 feet, the dam would be about 1,050 feet long. The drainage area is 357 square miles.

A dam at the Scott Mountain site built to back water to an

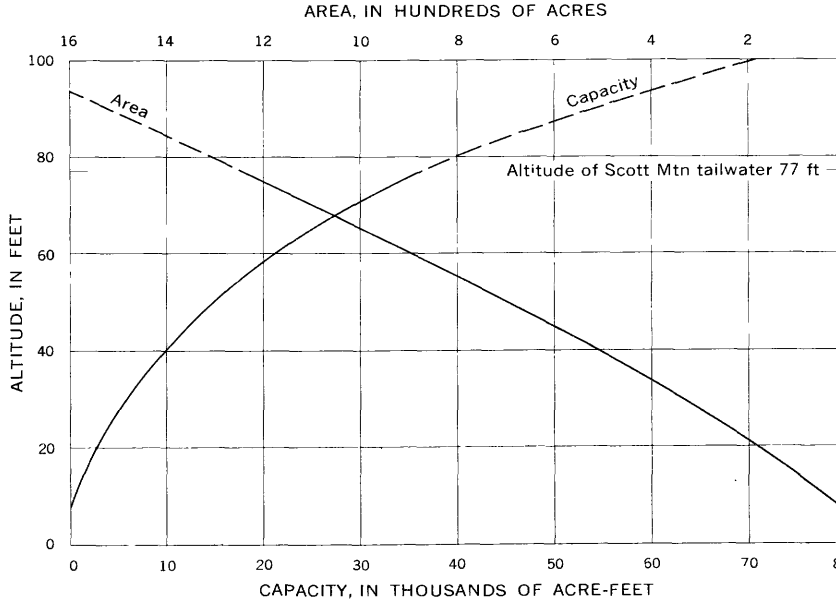

FIGURE 4.—Area and capacity of Tidewater reservoir site.

altitude of 300 feet or one at the Tidewater site built to an altitude of 250 feet would provide approximately equal storage capacity. At an altitude of 300 feet the town of Alsea would be almost completely covered by the Scott Mountain reservoir as would a large part of the tillable land in the valley. For this reason the Tidewater damsite might be preferable if a large reservoir is to be built.

The illustrative development plan presented in this paper on pages D24–D40 assumes a smaller reservoir with the dam at the Scott Mountain damsite. The water surface there is 77 feet above sea level and backwater would be at an altitude of 237 feet. This altitude limits the Tidewater reservoir site to a maximum capacity of 36,000 acre-feet.

DRIFT CREEK

There are two reservoir sites on Drift Creek: Slickrock, which has a damsite about 1 mile downstream from Slickrock Creek, and Trout Creek, which has a damsite about 1 mile downstream from Trout Creek. The drainage areas are 42 and 62 square miles, respectively, at the above damsites. The Tidewater topographic quadrangle map, scale 1:62,500, shows that a dam built to an altitude of 500 feet at the Slickrock site would be about 800 feet long, and that a dam built to an altitude of 250 feet at the Trout Creek site would be about 1,050 feet long. The areas and capacities of the two reservoir sites are shown in the following tabulations:

Slickrock reservoir site, damsite in sec. 3, T. 13 S., R. 10 W.

Altitude (feet)	Area (acres)	Capacity (acre-feet)
260	0	0
300	40	840
350	120	4,840
400	290	15,000
450	510	35,000
500	970	72,000

Trout Creek reservoir site, damsite in sec. 12, T. 13 S., R. 11 W.

25	0	0
50	40	500
100	190	6,250
150	420	21,500
200	690	49,300
250	1,000	92,000

GENERAL GEOLOGY OF THE ALSEA RIVER BASIN

By D. L. GASKILL

Rocks exposed in the Alsea drainage basin include marine and brackish-water sediments interbedded with basaltic lavas. These rocks are cut locally by basic and syenitic intrusives and include most of the sedimentary and igneous rocks of Cenozoic age known to occur in the central coast range of western Oregon.

The geology of the area has been mapped at a scale of 1:62,500. Most of the lower Alsea drainage basin is shown on a map by Baldwin (1955). The maps include brief geologic texts and references to earlier geologic work.

The predominant rocks are strata of the Tyee Formation of middle Eocene age (Baldwin, 1959; Wells and Peck, 1961; Snavely and Wagner, (1963). The name Burpee Formation, first given these rocks by Schenck (1927) and used by Vokes, Norbisrath, and Snavely (1949), has been abandoned by the Geological Survey in favor of the name Tyee Formation. The Siletz River Volcanic Series of Eocene age is exposed locally in the headwaters area (Baldwin, 1955).

Four of the storage sites discussed on pages D11–D15 are in the strata of the Tyee Formation, and two, the County Line dam and reservoir site on the North Fork and the Peak Creek dam site on the South Fork, are in the Siletz River Volcanic Series. Only the Scott Mountain and the Tidewater sites have been mapped with 10-foot contours and studied geologically for the purpose of this report. Accordingly, the following discussion concerns only rocks of the Tyee Formation.

The Tyee Formation in the lower Alsea drainage basin is 6,000–7,000 feet thick and has a regional dip of 15°–20° W., according to Vokes, Norbisrath, and Snavely (1949). They describe it as a * * * monotonous sequence of rhythmically bedded sandstones with mudstone partings * * *. Each bed is composed of medium- to coarse-grained, highly micaceous, arkosic sandstone that gradually grades upward into siltstones and mudstones in its upper part. Successive beds always begin with coarse sandy material which rests abruptly upon the upper mudstone zone of the underlying bed * * *. The sandstone * * * is firmly compacted, uniform, * * * somewhat calcareous * * * and is composed mainly of angular grains of quartz, feldspar, and tuffaceous fragments, with abundant wrinkled flakes of muscovite and biotite. The upper part of each bed is a dark, firm, sometimes sandy, mudstone. Fragmental plant material is almost universally present in these upper zones, and is commonly * * * present in the sands.

The Tyee Formation has been folded into broad gentle flexures. High-angle normal faults that have minor displacement may be observed in highway cuts along the Alsea River valley. Most of

these faults trend eastward. Abrupt deviations in attitude of the strata suggest that many other faults occur in this region (Vokes, Norbisrath, and Snavely, 1949).

RECONNAISSANCE GEOLOGY OF THE SCOTT MOUNTAIN DAMSITE

The Alsea River occupies a 100- to 150-foot wide bedrock channel at the base of the west abutment of the proposed damsite at Scott Mountain (pl. 1, section *A–A′*). A silty to sandy flood-plain deposit 400–500 feet wide separates the river from the east abutment. The river gradient here is about 9 feet per mile. Abutment slopes are forested and generally covered with 10 feet or more of soil and slope wash overlying weathered fractured bedrock.

A large quarry adjoining the damsite area on the northeast exposes more than 200 feet of massive firmly compacted greenish-gray micaceous silty sandstone containing thin beds, partings, and lenses of shaly claystone, mudstone, and siltstone (fig. 5). The sandstone beds range from less than 1 foot to 12 feet thick and average about 4 feet in thickness. Intervening shaly beds average about 4 inches in thickness but range from thin clay-mud-siltstone partings to beds several feet thick. (See pl. 1, columnar section.) Samples of noncalcareous silty sandstone examined under the microscope are composed of angular to subangular very

FIGURE 5.—Quarry in Tyee Formation adjoining Scott Mountain damsite at junction of Five Rivers and the Alsea River.

fine to medium grains in a matrix of loosely cemented silt-sized particles. The quarry strata is characterized by weak bedding contacts exhibiting mud cracks, ripple marks, and channeled surfaces. Most of the sandstone is rather loosely bonded, but some exposures are well cemented with calcium carbonate.

Sedimentary beds in the general damsite area dip gently southeast. The river channel at the damsite is cut into massive sandstone which is more than 5 feet thick and which exhibits a rectilinear pattern of more than three well-developed, commonly continuous subvertical and vertical joint sets trending parallel, diagonally, and at right angles to the river. The most persistent joints strike north parallel to the river and are spaced ½–4 feet apart; however, north of section *A–A′* (pl. 1) and opposite the quarry, the most conspicuous joints strike N. 73° W. across the river and are generally 9–30 feet apart. Several small east-trending faults were observed north and south of the damsite area. A basaltic dike, 3–4 feet wide, intrudes one such fault at the north end of the quarry (pl. 1).

A ridge extending southwest from Scott Mountain (pl. 1, index map) forms the west abutment at the damsite and the west side of the reservoir area for a distance of about 1 mile. The ridge forms a narrow peninsula between the reservoir area and the Alsea River valley downstream.

Bedrock on this ridge dips at a low angle south and southeast roughly parallel or obliquely downstream toward the damsite area and seems to be widely fractured and bisected locally by small faults. At an altitude of 80 feet above the river, this ridge is only 1,400–1,700 feet wide for a distance of about 1 mile along which reservoir water would have nearly direct access to bedding. At the 300-foot contour, 230 feet above the river, the ridge is barely 1,000 feet wide. The narrow width of the Scott Mountain ridge indicates a deep zone of weathering, which, combined with bedrock fracturing and the almost horizontal strike of bedding planes across the ridge, suggests the possibility of reservoir leakage through this barrier. Similar conditions might be anticipated on the east abutment ridge in sec. 18 between Five Rivers and the Alsea River where bedding attitudes suggest a gentle synclinal flexure or possible faulting between the two river valleys.

Additional exploration is necessary, particularly in regard to the permeability of the Scott Mountain ridge. Permeable zones may be present or develop as a result of reservoir pressure along fractures, weak bedding contacts, or poorly bonded permeable

sandstone beds. This ridge might prove too large an area for an economic grouting program.

Among the less favorable structural aspects of the damsite are the moderate dip of beds downstream and the persistent downstream-trending vertical joints in foundation and abutment rock. Leakage should be anticipated here through permeable zones that are subject to piping and scour unless properly grouted.

In view of the nearly horizontal position of the strata and the presence of massive sandstone beds, the damsite abutments are probably of sufficient strength and stability to support a wide-base earthfill or rockfill structure.

No faults were observed in foundation rock at or near section *A–A′* (pl. 1). The attitude of the sedimentary rocks at the damsite and in the immediate upstream reservoir area indicates little danger of slides during or after construction. Excavation requirements at this site would probably be moderate.

No examination was made of the general reservoir area. Unstable slopes are probably present in the reservoir area, as indicated by the steep mountain slopes and abrupt changes in attitudes of strata as shown in the published topographic and geologic maps.

The possibility of leakage through the Scott Mountain ridge seems to be the critical factor for a dam here. Consideration might be given to an alternate site where there would not be danger of reservoir leakage.

RECONNAISSANCE GEOLOGY OF THE TIDEWATER DAMSITE

The Tidewater site is about 5 miles downstream from the upper reach of tidewater and is subject to small daily tidal fluctuations. The Alsea River occupies most of the narrow valley at the proposed damsite (pl. 1, section *B–B′*), between steep abutments sloping 30°–40° and reaching altitudes of 800 feet above the river. Bedrock is exposed only in highway and logging roadcuts. Abutments are forested and covered with a thick soil mantle, local slope wash, slump, and some landslide debris. Valley fill consists of silty flood- and tidal-plain deposits.

Bedrock exposures in the area display sharply defined beds of sandstone, 1.5–9 feet thick, separated by shale layers 3 inches to 2 feet thick. The sedimentary beds dip 12°–19° downstream in abutment outcrops, but attitudes are probably affected by some slumping of exposures. Bedding attitudes near river level, both upstream and immediately downstream from the damsite area, indicate the dip of bedding is about 12° toward the southwest.

A system of closely spaced joints is exposed about a thousand feet southwest of the damsite area along the south bank of the river. The more prominent joints strike northwest and have 70°–75° dips. Other joints strike north and northeast and have dips of 55°–65° east and south. Bedding planes are offset as much as a foot along some of these fractures. Only a few north-south oriented joints were observed in highway cuts at the damsite.

A diabasic dike about 10 feet wide intrudes a fault on the north abutment. The dike strikes about N. 80° W. and dips about 40° N. If this fault orientation is correct, the dike should cut the south abutment west of section *B–B′* as shown on plate 1.

Normal faults trending about N. 80° W. are exposed along the highway between the damsite area and the community of Tidewater about a mile upstream. Other faults with similar orientation were observed near the mouth of Canal Creek. (See pl. 1, index map).

The depth of unconsolidated valley fill at section *B–B′* is not known, but it is probably less than 30 feet thick.

Stripping requirements are tied to thickness of the valley fill and obscure bedrock structures that might include weak bedding planes, shaly interbeds, poorly bonded sandstone, and other factors influencing permeability and bedrock stability. The abutment strata include thick massive beds of sandstone representative of the Tyee Formation. The sandstone beds apparently constitute the bulk of the sedimentary section at the damsite and should prove adequate support for a dam. The strike of the sedimentary strata, parallel to section *B–B′*, (pl. 1) is favorable to abutment stability, but the downstream dip of beds, though moderate, is not favorable to watertightness or hydrostatic reservoir pressure. At least one fault, intruded by a thick dike, cuts the north abutment and might bisect the line of section *B–B′*. This dike might prove a useful structural feature for selective-foundation or abutment treatment.

Construction would be in easily quarried rock. Tunnels will probably require strong support, and bedrock in spillway and stilling basins would require some protection from water erosion. Ground-water levels are assumed to be generally above possible reservoir levels. Stability of steep slopes above the damsite area should be investigated.

CONSTRUCTION MATERIALS

Intrusive rock has been crushed for road metal from a large gabbroic dike in the SE¼ sec. 36, T. 13 S., R. 10 W., 4 miles down-

stream from the Scott Mountain site. The quarried sandstone adjacent to the Scott Mountain site was apparently utilized for highway fill and embankment construction. Large quantities of earthfill material, gravel, and sand, are probably available from alluvial deposits in and near the damsite areas. Other intrusive bodies in the vicinity might be utilized as a source of aggregate and riprap.

POTENTIAL POWER

The highest altitude of perennial water in the Alsea River basin is about 2,300 feet above sea level. The median contour between this altitude and sea level is 1,150 feet. Average discharge is 4.63 cfs per sq mi at the gaging station near Tidewater. This rate of discharge applied to the 473 square-mile area of the basin equals 2,190 cfs and indicates a theoretical potential power of 214,000 kilowatts at 100 percent efficiency and average discharge. This theoretical power of the basin is not a measure of the power that could be developed. Inspection of the basin on the ground and by topographic maps indicates that it might be technically feasible to develop six sites that would have a total capacity of 61,000 kw at gross head, average discharge (mean flow) and 100 percent efficiency as shown in table 4.

This 214,000 kw is potential power that will never be economic. It might be of interest to note that the estimated technical power potential is 29 percent of the theoretical. This ratio appears to be reasonable. In Sweden, 43 percent of the gross theoretical is estimated to be technically feasible (Berglund and Larsson, 1959). In Italy, 20 percent and in Greece 12 percent of gross theoretical are considered exploitable at the present time (United Nations, 1961).

TABLE 4.—*Estimated potential power at technically feasible sites in Alsea River basin, Oregon*

[Gross head and 100 percent efficiency]

Site	Stream	Head (feet)	Kilowatts at percentages of time indicated		Kilowatts at mean flow
			95	50	
County Line	North Fork	300	130	1,280	2,660
Peak Creek	South Fork	300	130	1,020	2,300
Scott Mountain	Alsea	270	1,950	15,000	33,900
Tidewater	do	80	650	4,960	11,100
Slickrock	Drift Creek	225	290	2,010	4,200
Trout Creek	do	250	430	3,290	6,900
Total			3,580	27,560	61,060

The development scheme presented hereafter will, it is estimated, be economically feasible within the foreseeable future. It could produce 137,300,000 kilowatthours annually (equivalent to 16,000 kw continuous), about 10 percent of the above gross theoretical estimate of 214,000 kw reduced by an efficiency factor of 0.75.

There are no waterpower developments within the Alsea River basin.

UNDEVELOPED POWERSITES

COUNTY LINE

The North Fork drains 63 square miles in the northeast part of the Alsea River basin. Gaging-station records for 3 water years (1958–60) show a mean flow of 269 cfs and a corresponding mean unit flow of 4.27 cfs per sq mi. The long-term average discharge according to table 1 is 56 inches per year, equal to about 4.12 cfs per sq mi and about 260 cfs for the area above the gage. The County Line site would develop about 300 feet of head by diversion and conduit. Drainage area at this site is 25.3 square miles, which yields an estimated mean flow of about 105 cfs. Even with complete regulation, which would not be available, the power potential of this stream is small. It does not seem likely that streams of this size will be utilized for power in the foreseeable future. Although a certain potential does exist, the North Fork is not included in the plan of development in this report.

PEAK CREEK

The South Fork drains 49.5 square miles along the western edge of the Coast Range. Gaging-station records for water years 1958–60 show a mean flow of 155 cfs and a corresponding mean unit flow of 3.14 cfs per sq mi. Reduced to a long-term basis according to table 1 these amounts become 3.0 cfs per sq mi and 149 cfs for the area above the gage. The Peak Creek site would drain about 30 square miles and develop 300 feet of head by diversion and conduit. Estimated mean flow at the site is 90 cfs. The potential power of the South Fork is not considered in the plan of development.

FALL CREEK SITES

Fall Creek drains 30 square miles in the north-central part of the Alsea River basin. During water years 1959 and 1960 the gaging station recorded a mean flow of 168 cfs and a corresponding mean unit flow of 5.59 cfs per sq mi. Reduced to a long-term basis according to table 1 these amounts become 5.46 cfs per sq mi and 164 cfs for the area above the station. The Scott Mountain reser-

voir site on the main stem would back water about 3 miles up Fall Creek. A suitable powersite above this pool level would probably control less than 25 square miles. Small power developments could probably be undertaken on this stream but none are considered in the plan of development or in the power estimate of this report.

FIVE RIVERS

The next important tributary downstream is Five Rivers, and it has a drainage basin of about 120 square miles. The gaging station operated at Denzer Bridge during water years 1959 and 1960 recorded a mean flow of 509 cfs corresponding to a mean unit flow of 4.46 cfs per sq mi for a 114-square-mile drainage area. Extending this short record to the 1958–60 water years and making a further extension by the ratios developed in table 1, the long-term unit discharge becomes 4.79 cfs per sq mi. Average discharge for the entire 120 square miles in the basin would be 575 cfs.

Any significant development site on Five Rivers could also be utilized by constructing the dam downstream on the Alsea River. This explains the reason for the omission of a site on Five Rivers in the tabulation showing technically feasible waterpower sites. The Five Rivers subbasin contains a topographically good dam and reservoir site. A 200-foot dam near the mouth of the river would create a reservoir of 275,000 acre-feet. This reservoir would provide regulating storage plus a conservation pool of 100,000 acre-feet in excess of the 175,000 acre-feet required to regulate the stream to a minimum of 400 cfs. Average head during regulation would be 180 feet with that arrangement.

As previously mentioned, the Five Rivers storage capacity might be utilized by constructing the dam on the Alsea River down stream from Five Rivers. Should it be found undesirable to construct the dam on the Alsea River, a pumped-storage plan might be used for developing peaking energy with Alsea River water. The principal reason for such a development would be to avoid construction of a dam on the Alsea where it would interfere with passage of fish and would flood agricultural lands and a long length of very important highway. The Five Rivers reservoir would have a long and interesting shoreline and would be a valuable recreational asset. It would also provide easy access to timberlands and would be a deterrent to the spread of forest fires. This site is discussed further in the section on pumped storage (p. D40).

SCOTT MOUNTAIN AND TIDEWATER SITES

The Scott Mountain site near the mouth of Five Rivers and the Tidewater site near the upper end of tidewater influence have been considered in previous waterpower investigations. Helland (1945) discussed both of these in his report. Records at the gaging station between these two sites show a mean flow of 1,547 cfs and a corresponding mean unit flow of 4.63 cfs per sq mi. These sites are the only ones considered to approach economic feasibility. They are discussed in detail in the section, "Plan of development," on this page.

SLICKROCK AND TROUT CREEK SITES

Drift Creek drains 70 square miles in the northwest part of the Alsea River basin and enters the main river near its mouth. A gaging station was operated during water years 1959, 1960, and 1961 at an upstream location. The drainage area for this station is 20.6 square miles. This station recorded a mean flow of 118 cfs, and a corresponding mean unit flow of 5.73 cfs per sq mi. Extended to 1958–60 water years and from there to a long-term basis as in table 1, the unit discharge is 5.23 cfs. This is an average of 108 cfs for the gaging station and 366 cfs for the entire basin. Two sites on Drift Creek were studied. The Slickrock site a short distance below the mouth of Slickrock Creek drains 42 square miles. The Trout Creek site draining 62 square miles is in the SE¼ sec. 12, T. 13 S., R. 11 W. An annual average of about 220 cfs is indicated for the Slickrock site and about 325 cfs for the Trout Creek site. Reservoirs are possible at both sites.

PLAN OF DEVELOPMENT

Storage is essential for development of power on the Alsea River owing to considerable fluctuations in the flow. The Scott Mountain site would provide this storage, and would be the key power development in the basin. A development at the Scott Mountain site would limit the maximum water surface of the Tidewater reservoir site to an altitude of 77 feet and its capacity to 36,000 acre-feet. A reservoir of this size would be primarily valuable for reregulating Scott Mountain releases and for pondage. The Scott Mountain and Tidewater reservoir and powersites probably approach economic feasibility. None of the other sites discussed seem worthy of consideration for construction at this time.

SCOTT MOUNTAIN

Drainage area at the gaging station is 334 square miles, and 96 percent of this, or 321 square miles, is drained at the Scott Mountain site. The natural runoff at the site was estimated from gaging-station records on a ratio of drainage areas. Table 5 gives this estimated natural runoff at the Scott Mountain site in monthly acre-foot values. It shows that runoff during the period of November through March averaged more than 100,000 acre-feet each month and that January had the highest runoff at 217,100 acre-feet. The volume of runoff falls off sharply beginning in April to a low of 7,100 acre-feet in August.

Complete regulation of this stream would require a much greater storage volume than for operations confined to the 5 high-yield months. These 5 high-yield months correspond to the low-flow months at the Columbia River power stations. Power produced from November through March is valuable for peaking in the Northwest power pool, and the Alsea River requires less storage for regulation during this period. Therefore, the plan of development presented here will confine most power production to this period. A conservation release of 100 cfs or 6,000 acre-feet per month would be allowed to remain in the stream to maintain fish life, boating, and esthetic qualities. This release is slightly lower than the August average of 7,100 acre feet. However, extreme low flows would be eliminated because 18 months during the 21-year period 1940–60 had less than 6,000 acre-feet of runoff. During the 7-month period from April through October, power could be produced with the 100 cfs conservation release. A considerable quantity of secondary power could also be produced during full-reservoir periods.

The method used here to determine storage required does not assume full regulation but seeks to find optimum use of the water supply. The recurrence interval of deficient flow estimates the frequency of water-supply deficiencies for varying amounts of storage less than that required for full regulation. This gives a quantitative index by which quick comparisons can be made of the effect reduced storage would have on water release schedules.

By examination of table 5 the following minimum water-release schedules were prepared:

Schedule A: 110,000 acre-feet per month November–March and 6,000 acre-feet per month April–October, or 592,000 acre-feet per year.

TABLE 5.—*Estimated natural runoff, in thousands of acre-feet, at the Scott Mountain damsite*

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Year
1940	9.4	6.6	131.7	118.4	251.2	165.7	65.9	53.8	14.5	7.8	4.8	5.2	834.9
1941	11.9	58.7	116.5	181.0	55.2	35.6	39.7	54.3	17.7	9.1	6.8	25.8	612.2
1942	16.2	80.2	252.0	118.2	163.2	60.1	36.3	41.9	26.8	14.7	8.2	5.5	823.3
1943	6.6	204.3	310.5	229.3	193.8	104.8	132.1	36.3	24.6	11.5	8.5	5.6	1,267.8
1944	43.5	55.3	100.0	97.2	94.1	91.9	86.1	35.1	21.7	10.1	6.4	5.4	646.8
1945	6.0	40.1	47.4	141.3	203.0	201.1	114.3	90.6	28.2	11.6	7.0	8.9	899.5
1946	6.0	158.3	225.0	228.9	179.9	151.9	56.8	24.3	16.4	12.1	6.5	7.5	1,073.6
1947	29.2	198.6	244.1	104.9	155.2	120.3	85.3	23.8	38.0	14.5	9.8	8.1	1,031.8
1948	148.8	149.9	107.1	249.1	182.6	138.8	127.8	86.4	27.7	13.7	8.7	7.9	1,248.4
1949	13.9	75.0	269.5	81.5	351.1	98.5	42.3	64.7	16.7	9.6	6.4	5.8	1,035.0
1950	9.0	67.0	132.4	326.5	296.8	220.9	79.6	42.0	17.2	9.4	6.2	5.5	1,212.5
1951	83.7	236.1	234.6	361.0	210.0	185.3	56.8	48.9	18.2	10.0	5.4	4.8	1,454.9
1952	94.4	142.4	291.3	241.5	228.7	159.8	59.3	27.9	16.3	10.8	6.5	4.9	1,283.7
1953	4.5	8.6	95.2	464.7	206.7	170.9	73.5	91.4	41.3	15.9	11.1	7.7	1,191.5
1954	22.9	158.8	312.2	333.7	250.0	121.6	121.6	27.2	18.4	11.3	8.1	9.1	1,404.7
1955	21.3	82.9	158.8	159.4	106.6	184.8	181.2	53.1	18.2	13.0	6.4	8.0	993.6
1956	59.1	224.4	399.2	397.3	178.5	258.2	84.9	26.0	11.2	8.2	5.3	5.4	1,660.7
1957	23.1	37.2	130.1	87.8	163.4	234.6	88.6	41.2	20.9	11.3	8.0	5.4	851.5
1958	11.3	30.5	235.9	219.8	260.8	104.7	129.8	40.8	18.4	8.9	4.9	5.4	1,071.2
1959	8.1	132.4	110.0	322.3	189.7	104.8	70.6	42.0	21.6	11.1	6.1	20.0	1,038.8
1960	30.7	39.0	67.1	95.4	253.1	170.2	136.6	98.9	35.0	13.4	8.1	5.9	953.4
Average	31.4	104.1	189.1	217.1	199.2	146.9	89.0	50.0	22.5	11.3	7.1	8.0	1,075.7

Schedule B: 150,000 acre-feet per month November–March and 6,000 acre-feet per month April–October, or 792,000 acre-feet per year.

Schedule C: 160,000 acre-feet per month November–March and 6,000 acre-feet per month April–October, or 842,000 acre-feet per year.

Table 6 was prepared for the above schedules listing active storage required for periods of deficient natural streamflow.

The monthly entry in table 6 represents the difference between scheduled release and actual runoff as shown in table 5 since the deficient period began. This value represents the storage that would have been required at the beginning of the deficient period to maintain the specified release schedule. Entries were placed in table 6 as long as total runoff for the period was less than specified release schedules even though runoff for certain months during the period exceeds the release schedule. Months for which no entries appear indicate lack of a deficient period and monthly runoff exceeding that specified by the release schedule. The maximum active storage required for each water year and its ranking number are given in the last two columns for use in the statistical comparison of storage required and recurrence interval of deficient flow. A dead storage of 30,000 acre-feet was assumed at all times to maintain 70 feet of head. The sum of dead and maximum active storage required for each year was placed on a data sheet in ranked decreasing order. The recurrence interval of deficiencies for each ranked entry was computed by the formula: $R.I. = n + 1/m$, where n=years of record and m=rank.

Figure 6 shows a graph of recurrence interval and storage required for the three release schedules considered. The storage required and corresponding dam height for any recurrence interval desired may be determined from this graph. With a 3-year recurrence interval, schedules A, B, and C would require dams that would raise the water surface 118 feet, 152 feet, and 159 feet, respectively. The 4-year recurrence interval would require corresponding dams to raise the water 126 feet, 160 feet, and 167 feet. A 10-year recurrence interval would require dams of sufficient height to raise the water 136 feet, 177 feet, and 187 feet. Schedule B at a 4-year recurrence interval has been selected to illustrate potential power for this report. This schedule will require 235,000 acre-feet of usable storage from the 265,000 acre-foot reservoir provided by a dam to raise the water surface 160 feet. A dam of this height would have a crest length of about a thousand feet. The full reservoir would rise to an altitude of

TABLE 6.—*Active storage, in thousands of acre-feet, required at the Scott Mountain site for periods of deficient natural streamflow*

[Rank is from the greatest to least requirement]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Maximum in year	Rank
Schedule A														
1940		103.5	81.8	73.4							1.2	2.0	103.5	6
1941		51.3	44.8		54.8	129.2	95.5	47.2	35.6	32.5	31.7	11.9	129.2	2
1942	1.7	31.5				49.9	19.6					.5	49.9	9
1943						5.2						.4	5.2	13
1944		54.7	64.7	77.5	93.4	111.5	31.4	2.4				.6	111.5	5
1945	.6	70.5	133.2	101.9	8.9								133.2	1
1946													0.0	20
1947				5.1									5.1	15
1948			2.9										2.9	16
1949		35.0		2[illegible].5		11.5						.2	35.0	11
1950		43.0	20.6									.5	43.0	10
1951											.6	1.9	1.9	17
1952												1.1	1.1	19
1953	2.6	104.0	118.8										118.8	4
1954													0.0	21
1955		27.1			3.4								27.1	12
1956											.7	1.3	1.3	18
1957		72.8	52.7	74.9	21.5							.6	74.9	8
1958		79.5				5.3					1.1	1.7	79.5	7
1959						5.2							5.2	14
1960		71.0	113.9	128.5								.1	128.5	3
Release schedule	6.0	110.0	110.0	110.0	110.0	110.0	6.0	6.0	6.0	6.0	6.0	6.0		

TABLE 6.—*Active storage, in thousands of acre-feet, required at the Scott Mountain site for periods of deficient natural streamflow*—Continued

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Maximum in year	Rank
Schedule B														
1940		143.5	161.8	193.4	92.2	76.5	16.5				1.2	2.0	193.4	8
1941		91.3	124.8	93.8	188.7	303.1	269.4	221.1	209.4	206.3	205.5	185.7	303.1	3
1942	175.6	245.4	143.4	175.2	162.0	251.8	221.5	185.7	164.9	156.1	154.0	154.4	251.8	4
1943	153.8	99.5				45.2						.4	153.8	9
1944		94.7	144.7	197.5	253.4	311.5	231.4	202.4	186.7	182.6	182.2	182.7	311.5	2
1945	182.7	292.7	395.3	404.0	351.0	299.9	191.6	107.0	84.8	79.2	78.2	75.3	404.0	1
1946	75.3	67.0											75.3	13
1947				45.1	39.9	69.6							69.6	15
1948		.1	43.0			11.2							43.0	17
1949		75.0		68.5		51.5	15.2					.2	75.0	14
1950		83.0	100.6									.5	100.6	11
1951											.6	1.9	1.9	20
1952		7.6										1.1	7.6	19
1953	2.6	144.0	198.8										198.8	6
1954						28.4							28.4	18
1955		67.1	58.3	48.9	92.3	57.5							92.3	12
1956											.7	1.3	1.3	21
1957		112.8	132.7	194.9	181.5	96.9	14.3					.6	194.9	7
1958		119.5	33.6			45.3					1.1	1.7	119.5	10
1959		17.6	57.6			45.2							57.6	16
1960		111.0	193.9	248.5	145.4	125.2						.1	248.5	5
Release schedule	6.0	150.0	150.0	150.0	150.0	150.0	6.0	6.0	6.0	6.0	6.0	6.0		

TABLE 6.—*Active storage, in thousands of acre-feet, required at the Scott Mountain site for periods of deficient natural streamflow*—Continued

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar	Apr.	May	June	July	Aug.	Sept.	Maximum in year	Rank
Schedule C														
1940		153.5	181.8	223.4	132.2	126.5	66.5	18.8	10.2	8.5	9.6	10.5	223.4	8
1941	4.6	106.0	149.5	128.5	233.3	357.7	324.0	275.7	264.0	260.9	260.1	240.3	357.7	3
1942	230.2	310.0	218.0	259.8	256.6	356.5	326.2	290.3	269.5	260.7	258.6	259.0	356.5	4
1943	258.4	214.1	63.6			55.2						.4	258.4	6
1944		104.7	164.7	227.5	293.4	361.5	281.4	252.4	236.7	232.6	232.2	232.7	361.5	2
1945	232.7	352.7	465.3	484.0	441.0	399.9	291.6	207.0	184.8	179.2	178.2	175.3	484.0	1
1946	175.3	176.9	112.0	43.1	23.2	31.3							177.0	10
1947				55.1	59.9	99.6	20.3	2.5					99.6	14
1948		10.1	63.0			21.2							63.0	17
1949		85.0		78.5		61.5	25.2					.2	85.0	15
1950		93.0	120.6									.5	120.6	13
1951											.6	1.9	1.9	20
1952		17.6				.2						1.1	17.6	19
1953	2.6	154.0	218.8										218.8	9
1954		1.2				38.4							38.4	18
1955		77.1	78.3	78.9	132.3	107.5							132.3	11
1956											.7	1.3	1.3	21
1957		122.8	152.7	224.9	221.5	146.9	64.3	29.1	14.2	8.9	6.9	7.6	224.9	7
1958	2.3	131.8	55.9			55.3					1.1	1.7	131.8	12
1959		27.6	77.6			55.2							77.6	16
1960		121.0	213.9	278.5	185.4	175.2	44.6					.1	278.5	5
Release schedule	6.0	160.0	160.0	160.0	160.0	160.0	6.0	6.0	6.0	6.0	6.0	6.0		

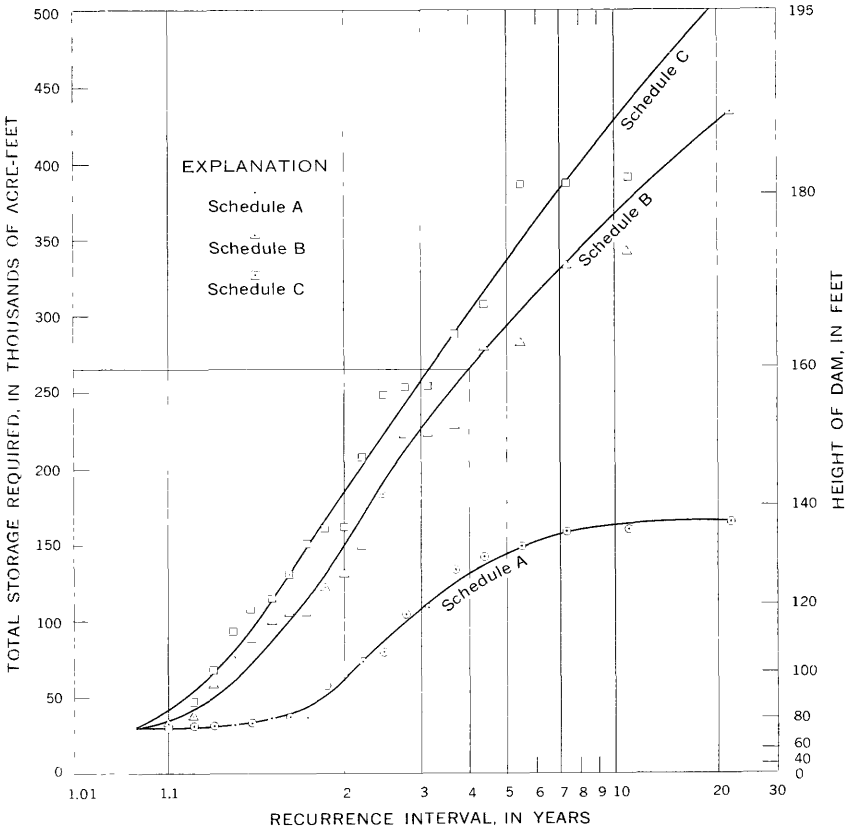

FIGURE 6.—Recurrence interval of flow deficiency for selected release schedules, Scott Mountain site.

237 feet and inundate approximately 10 miles of State Highway 34. The present water surface at the damsite has an altitude of about 77 feet. Table 7 shows the reservoir contents in acre-feet that would have resulted if schedule B had been used with a 160-foot dam (altitudes 77–237 ft) during water years 1940–60. The reservoir is full at 265,000 acre-feet and is not allowed to drop below 30,000 acre-feet.

Evaporation losses shown in table 7 were computed from evaporation records at Corvallis. A 3,000-acre reservoir was assumed. These losses total 31.55 inches per year. For the November–March period, average reservoir volume from table 7 was 194,000 acre-feet. The area-capacity curve in figure 3 shows a corresponding reservoir height of 144 feet for this volume of water. This head has been used in computing the potential power at the Scott Mountain site.

TABLE 7.—*Reservoir volumes, in thousands of acre-feet, for development of schedule B at the Scott Mountain site*

[Active storage is 235,000 acre-ft (265,000 — 30,000)]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
1940	[1] 2.9	[1] 3.4	[1] 129.1	33.5	134.7	150.4	209.6	256.3	263.5	263.6	260.9	259.0
1941	264.4	173.1	139.6	170.6	75.7	[2] 30.0	62.9	110.1	120.5	121.9	121.2	140.0
1942	149.7	79.8	181.8	150.0	163.2	73.4	102.9	137.7	157.2	164.2	164.9	163.4
1943	163.5	217.8	Full	Full	Full	219.8	Full	Full	Full	Full	Full	263.6
1944	Full	170.3	120.3	67.5	[2] 30.0	[2] 30.0	109.3	137.3	151.7	154.1	153.0	151.4
1945	150.9	41.0	[2] 30.0	[2] 30.0	83.0	134.1	241.6	Full	Full	Full	264.5	Full
1946	264.5	Full	Full	Full	Full	Full	Full	do	do	do	263.9	264.4
1947	Full	do	do	219.9	225.1	195.4	do	do	do	do	Full	Full
1948	do	264.9	222.0	Full	Full	253.8	do	do	do	do	do	Do.
1949	do	190.0	Full	196.5	do	213.5	249.0	do	do	do	263.8	262.7
1950	do	182.0	164.4	Full	do	Full	Full	do	do	do	263.7	262.2
1951	do	Full	Full	do	do	do	do	do	do	do	262.9	260.6
1952	do	257.4	do	do	do	do	do	do	do	do	263.9	261.8
1953	259.9	118.5	63.7	do	do	do	do	do	do	do	Full	Full
1954	Full	Full	Full	do	do	236.6	do	do	do	do	do	Do.
1955	do	197.9	206.7	216.1	172.7	207.5	do	do	do	do	263.9	264.9
1956	do	Full	Full	Full	Full	Full	do	do	do	do	262.8	261.2
1957	do	152.2	132.3	70.1	83.5	168.1	250.0	do	do	do	Full	263.4
1958	do	145.5	231.4	Full	Full	219.7	Full	do	do	do	262.4	260.8
1959	262.4	244.8	204.8	do	do	219.8	do	do	do	do	263.6	Full
1960	Full	154.0	71.1	230.0	133.1	153.3	do	do	do	do	Full	264.0
Evaporation loss	.5						.8	1.1	1.3	1.7	1.5	1.0

[1] Reservoir buildup period in October, November, and December of 1940 water year.
[2] Occurrence of flow deficiency. Reservoir never drawn below 30,000 acre-feet.

Table 8 gives releases that would have occurred from the Scott Mountain reservoir site if schedule B had been followed during periods when the reservoir was not full. This table is used for estimating runoff available at the Tidewater site downstream. It shows that deficiencies occurred during 6 months of 4 separate years. The deficiency during 3 of the 6 months amounted to less than 20,000 acre-feet. The first 3 months of the period of record are reserved for reservoir buildup and deficiencies to meet the release schedule during this period are not considered above.

TIDEWATER

Drainage area at the Tidewater damsite is 357 square miles. The runoff at the site, after regulation at the Scott Mountain site, was estimated to be the sum of releases at the Scott Mountain site (table 8) and the flow increment entering the stream between the two sites. This runoff is shown in table 9. Drainage areas at the Scott Mountain and Tidewater sites are 96 percent and 106 percent respectively of the area drained at the gaging station. The flow increment entering between the two sites, therefore, is estimated to be 10 percent of the runoff at the gaging station.

The altitude of the water surface at the Tidewater damsite is 7 feet, and at the Scott Mountain damsite it is 77 feet. These altitudes fix the lower and upper limits of the Tidewater reservoir. Because the height of the dam is fixed, recurrence-interval and storage-required curves would serve no purpose. Power development will be confined primarily to the November–March period, because runoff is highly concentrated during this time by the Scott Mountain reservoir releases. Table 9 shows that average monthly runoff during this period ranges from 164,700 acre-feet in November to 232,800 acre-feet in January. A conservation release of 100 cfs or 6,000 acre-feet per month would be assured during the April through October period. Figure 4 shows the full reservoir volume to be 36,000 acre-feet. A dead storage of 5,000 acre-feet would be kept to maintain a minimum head of 21 feet.

Table 10 shows the reservoir volumes that would have resulted if each of the three schedules had been used during the period of record. Evaporation losses shown in table 10 were computed from evaporation records at Corvallis on the basis of a 1,000-acre reservoir. These losses total 31.55 inches per year.

By examination of table 10 the following three water-release schedules were prepared:

TABLE 8.—*Release from Scott Mountain site, in thousands of acre-feet, if schedule B had been used*

[Aim: 150,000 acre-ft November-March; 6,000 acre-feet April-October]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
1940	[1] 6.0	[1] 6.0	[1] 70.0	150.0	150.0	150.0	6.0	6.0	6.0	6.0	6.0	6.0
1941	6.0	150.0	150.0	150.0	150.0	[2] 45.7	6.0	6.0	6.0	6.0	6.0	6.0
1942	6.0	150.0	150.0	150.0	150.0	150.0	6.0	6.0	6.0	6.0	6.0	6.0
1943	6.0	150.0	263.3	229.3	193.8	150.0	230.1	35.2	23.3	9.8	6.9	6.0
1944	35.7	150.0	150.0	150.0	[2] 131.6	[2] 91.9	6.0	6.0	6.0	6.0	6.0	6.0
1945	6.0	150.0	[2] 58.3	[2] 141.3	150.0	150.0	6.0	60.2	26.9	9.9	6.0	7.4
1946	6.0	157.8	225.0	228.9	179.9	151.9	56.0	23.2	15.2	10.4	6.0	6.0
1947	28.1	198.6	224.1	150.0	150.0	150.0	15.0	22.7	36.8	12.7	8.2	7.1
1948	148.3	150.0	150.0	206.1	182.6	150.0	115.8	85.3	26.5	11.9	7.1	6.9
1949	13.4	150.0	194.5	150.0	282.6	150.0	6.0	47.7	15.5	7.8	6.0	6.0
1950	6.2	150.0	150.0	225.9	296.8	220.9	78.8	40.9	16.0	7.7	6.0	6.0
1951	80.4	236.1	234.6	361.0	210.0	185.3	56.0	47.9	18.0	8.3	6.0	6.0
1952	89.5	150.0	283.7	241.5	228.7	159.8	58.5	26.8	15.0	9.1	6.0	6.0
1953	6.0	150.0	150.0	263.4	206.7	170.9	72.7	90.3	40.1	14.1	9.6	6.7
1954	22.4	158.8	312.2	333.7	259.9	150.0	92.4	26.1	17.2	9.6	6.6	8.1
1955	20.8	150.0	150.0	150.0	150.0	150.0	122.9	52.0	17.0	11.2	6.0	6.0
1956	58.5	224.4	399.2	297.3	178.5	258.2	84.2	24.9	12.9	6.4	6.0	6.0
1957	18.8	150.0	150.0	150.0	150.0	150.0	6.0	25.0	19.7	9.6	6.4	6.0
1958	9.2	150.0	150.0	186.2	260.8	150.0	83.7	39.8	17.2	7.1	6.0	6.0
1959	6.0	150.0	150.0	262.1	189.7	150.0	24.6	40.9	20.4	9.4	6.0	17.6
1960	30.2	150.0	150.0	[2] 136.5	150.0	150.0	24.1	97.9	33.7	11.6	6.6	6.0

[1] Reservoir buildup period, October, November, and December of 1940 water year.
[2] Occurrence of flow deficiency.

TABLE 9.—*Estimated runoff, in thousands of acre-feet, at the Tidewater site after regulation at the Scott Mountain site*

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Year
1940	7.0	6.7	83.7	162.3	176.2	167.3	12.9	11.6	7.5	6.8	6.5	6.5	655.0
1941	7.2	156.1	162.1	168.9	155.7	49.4	10.1	11.7	7.8	7.0	6.7	8.7	751.5
1942	7.7	158.4	176.3	162.3	167.0	156.3	9.8	10.4	8.8	7.5	6.9	6.6	877.9
1943	6.7	171.3	295.7	253.2	214.0	160.9	243.9	39.0	25.8	11.0	7.8	6.6	1,435.9
1944	40.2	155.8	160.4	160.1	141.4	101.4	15.0	9.7	8.3	7.1	6.7	6.6	812.5
1945	6.6	154.2	63.2	156.0	171.1	170.9	17.9	69.6	29.9	11.1	6.7	8.3	865.6
1946	6.6	174.3	248.4	252.7	198.7	167.7	62.0	25.8	16.9	11.6	6.7	6.8	1,178.1
1947	31.2	219.3	269.5	160.9	166.2	162.5	23.9	25.2	40.7	14.2	9.3	8.0	1,130.8
1948	163.8	165.6	161.2	232.0	201.6	164.5	129.2	94.3	29.4	13.3	8.1	7.7	1,370.7
1949	14.9	157.8	222.5	158.5	319.2	160.3	10.4	54.4	17.2	8.8	6.7	6.6	1,137.3
1950	7.1	157.0	163.8	259.9	327.7	243.9	87.1	45.2	17.8	8.6	6.6	6.6	1,331.4
1951	89.2	260.7	259.0	398.6	231.9	204.6	61.9	53.0	19.9	9.4	6.6	6.5	1,601.1
1952	99.4	164.8	314.1	266.7	252.5	176.4	64.7	29.7	16.7	10.2	6.7	6.5	1,408.3
1953	6.5	150.9	159.9	311.8	228.2	188.7	80.4	99.8	44.4	15.8	10.8	7.5	1,304.6
1954	24.7	175.4	344.7	368.4	287.0	162.7	105.1	28.9	19.1	10.8	7.4	9.0	1,543.2
1955	23.0	158.6	166.5	166.6	161.1	169.3	141.8	57.5	18.9	12.6	6.7	6.8	1,089.3
1956	64.7	247.8	440.8	438.7	197.1	285.1	93.0	27.6	14.4	7.3	6.6	6.6	1,829.5
1957	21.2	153.9	163.6	159.1	167.0	174.4	15.2	29.3	21.9	10.8	7.3	6.6	930.1
1958	10.3	153.2	174.6	209.1	288.0	160.9	97.3	44.0	19.1	8.1	6.5	6.6	1,177.6
1959	6.9	163.8	161.5	295.7	209.5	160.9	32.0	45.3	42.9	10.5	6.6	19.7	1,155.3
1960	33.4	154.1	157.0	146.4	176.4	167.7	38.3	103.1	37.4	13.0	7.4	6.6	1,040.8
Average	32.3	164.7	207.1	232.8	211.3	169.3	64.4	43.6	22.1	10.3	7.2	7.7	1,172.7

TABLE 10.—*Thousands of acre-feet of water in Tidewater reservoir, using the releases from Scott Mountain reservoir shown in table 8*

[Asterisk indicates that the reservoir was full, 36,000 acre-ft, 31,000 acre-ft usable]

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Schedule D												
1940	[1] 0.8	[1] 1.5	[1] 5.0	12.3	33.5	*	*	*	*	*	*	*
1941	*	*	*	*	*	(2)	8.9	14.2	15.6	16.0	16.2	18.5
1942	20.1	23.5	*	*	*	*	*	*	*	*	*	*
1943	*	*	*	*	*	*	*	*	*	*	*	*
1944	*	*	*	*	22.4	(3)	13.7	17.0	18.9	19.3	19.5	19.7
1945	20.2	19.4	(4)	6.0	22.1	*	*	*	*	*	*	*
1946	*	*	*	*	*	*	*	*	*	*	*	*
1947	*	*	*	*	*	*	*	*	*	*	*	*
1948	*	*	*	*	*	*	*	*	*	*	*	*
1949	*	*	*	*	*	*	*	*	*	*	*	*
1950	*	*	*	*	*	*	*	*	*	*	*	*
1951	*	*	*	*	*	*	*	*	*	*	*	*
1952	*	*	*	*	*	*	*	*	*	*	*	*
1953	*	31.9	*	*	*	*	*	*	*	*	*	*
1954	*	*	*	*	*	*	*	*	*	*	*	*
1955	*	*	*	*	*	*	*	*	*	*	*	*
1956	*	*	*	*	*	*	*	*	*	*	*	*
1957	*	34.9	*	*	*	*	*	*	*	*	*	*
1958	*	34.2	*	*	*	*	*	*	*	*	*	*
1959	*	*	*	*	*	*	*	*	*	*	*	*
1960	*	35.1	*	27.4	*	*	*	*	*	*	*	*
Evaporation loss	.17						.25	.36	.42	.58	.51	.33

See footnotes at end of table.

TABLE 10.—*Thousands of acre-feet of water in Tidewater reservoir, using the releases from Scott Mountain reservoir shown in table 8*—Continued

Water year	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.
Schedule E												
1940	[1] 0.8	[1] 1.5	[1] 5.0	7.3	23.5	30.8	*	*	*	*	19.0	*
1941	*	32.1	34.2	*	31.7	(5)	8.9	14.2	15.6	16.0	16.2	18.5
1942	20.1	18.5	34.8	*	*	32.3	35.8	*	*	*	*	*
1943	*	*	*	*	*	*	*	*	*	*	*	*
1944	*	31.8	32.2	32.3	13.7	(6)	13.7	17.0	18.9	19.3	19.5	19.7
1945	20.2	14.4	(7)	(8)	16.1	27.0	*	*	*	*	*	*
1946	*	*	*	*	*	*	*	*	*	*	*	*
1947	*	*	*	*	*	*	*	*	*	*	*	*
1948	*	*	*	*	*	*	*	*	*	*	*	*
1949	*	33.8	*	34.5	*	*	*	*	*	*	*	*
1950	*	33.0	*	*	*	*	*	*	*	*	*	*
1951	*	*	*	*	*	*	*	*	*	*	*	*
1952	*	*	*	*	*	*	*	*	*	*	*	*
1953	*	26.9	26.8	*	*	*	*	*	*	*	*	*
1954	*	*	*	*	*	*	*	*	*	*	*	*
1955	*	34.6	*	*	*	*	*	*	*	*	*	*
1956	*	*	*	*	*	*	*	*	*	*	*	*
1957	*	29.9	33.5	32.6	*	*	*	*	*	*	*	*
1958	*	29.2	*	*	*	*	*	*	*	*	*	*
1959	*	*	*	*	*	*	*	*	*	*	*	*
1960	*	30.1	27.1	13.5	29.9	*	*	*	*	*	*	*
Evaporation loss	.17						.25	.36	.42	.58	.51	.33

See footnotes at end of table.

TABLE 10.—*Thousands of acre-feet of water in Tidewater reservoir, using the releases from Scott Mountain reservoir shown in table 8*—Continued

Water year	Oct.	Nov.	Dec.	Jan.	Feb	Mar.	Apr.	May	June	July	Aug.	Sept.
Schedule F												
1940	[1]0.8	[1]1.5	[1]5.0	(9)	16.2	18.5	25.1	30.4	31.5	31.7	31.7	31.9
1941	33.0	24.1	21.2	25.1	15.8	(10)	8.9	14.2	15.6	16.0	16.2	18.5
1942	20.1	13.5	24.8	22.1	24.1	15.4	18.9	22.9	25.3	26.2	26.5	26.8
1943	27.2	*	*	*	*	31.9	*	*	*	*	*	*
1944	*	26.8	22.2	17.3	(11)	(12)	13.7	17.0	18.9	19.3	19.5	19.7
1945	20.2	9.4	(13)	(14)	11.1	17.0	28.7	*	*	*	*	*
1946	*	*	*	*	*	*	*	*	*	*	*	*
1947	*	*	*	31.9	33.1	30.6	*	*	*	*	*	*
1948	*	*	32.2	*	*	35.5	*	*	*	*	*	*
1949	*	28.8	*	29.5	*	31.3	35.5	*	*	*	*	*
1950	*	28.0	26.8	*	*	*	*	*	*	*	*	*
1951	*	*	*	*	*	*	*	*	*	*	*	*
1952	*	35.8	*	*	*	*	*	*	*	*	*	*
1953	*	21.9	16.8	*	*	*	*	*	*	*	*	*
1954	*	*	*	*	*	33.7	*	*	*	*	*	*
1955	*	29.6	31.1	32.7	28.8	33.1	*	*	*	*	*	*
1956	*	*	*	*	*	*	*	*	*	*	*	*
1957	*	24.9	23.5	17.6	19.6	29.0	*	*	*	*	*	*
1958	*	24.2	33.8	*	*	31.9	*	*	*	*	*	*
1959	*	34.8	31.3	*	*	31.9	*	*	*	*	*	*
1960	*	25.1	17.1	(15)	16.4	19.1	*	*	*	*	*	*
Evaporation loss	.17	----------	----------	----------	----------	----------	.25	.36	.42	.58	.51	.33

[1] Reservoir buildup period, October–December of 1940 water year.
[2-15] Active storage exhausted. Deficiency in acre-feet:

2. 74.6.	7. 87.4.	12. 63.6.
3. 36.2.	8. 4.0.	13. 97.4.
4. 77.4.	9. 2.7.	14. 9.0.
5. 83.9.	10. 104.8.	15. 6.5.
6. 49.9.	11. 11.3.	

Schedule D: 155,000 acre-feet per month November–March and 6,000 acre-feet per month April–October, or 817,000 acre-feet per year.

Schedule E: 160,000 acre-feet per month November–March and 6,000 acre-feet per month April–October, or 842,000 acre-feet per year.

Schedule F: 165,000 acre-feet per month November–March and 6,000 acre-feet per month April–October, or 867,000 acre-feet per year.

If schedule D had been used, the reservoir would have been full 87 percent of the time, and three deficiencies would have occurred. The reservoir would have been full 79 percent of the time, and four deficiencies would have occurred if schedule E had been used. By using, schedule F the reservoir would have been full 63 percent of the time. Seven deficiencies would have occurred, but only three of them would have exceeded 7 percent of the scheduled release. Schedule F has been selected, because it is about the maximum release for which deficiencies in meeting the schedule can be tolerated. For the November–March period, average reservoir volume from table 10 (schedule F) was 27,930 acre-feet. Figure 4 shows the corresponding reservoir height to be 61 feet for this volume of water. This head has been used in computing the potential power at the Tidewater site.

The Tidewater reservoir would inundate the town of Tidewater and some homes along the river. The hillside above the present roadway between the Tidewater site and Hellion Canyon is more gradual than most places along the river. Road relocation would be over relatively favorable terrain and would not involve an extra hill because the present road climbs to an altitude of 250 feet to pass through a saddle east of Tidewater.

ESTIMATE OF POWER AND ENERGY

Power is computed by the formula $P=0.068QH$ when $P=$ power in kilowatts at 80 percent efficiency, $Q=$flow in cfs, and H $=$ head in feet. A summary of power and energy is given in table 11. The major part of the energy could be used in the Northwest power pool during critical periods in winter months. During the November–March period, 24,500 kw could be generated continuously at the Scott Mountain site and 11,400 kw at the Tidewater site for a total of 35,900 kw. The annual energy figure listed is the product of power and total hours in a year; this is a theoretical value based on complete use of water listed in the water-release schedule. In actual operation all the water listed in the schedule might not be used owing to demand fluctuations. However, a great

TABLE 11.—*Estimated potential power and energy, Alsea River, Oregon*

Powersite	*Q* Flow (cfs)	*H* Head (ft)	*P* Power (kw)	*E* Annual energy (kwh)
Scott Mountain:				
November to March	2,503	144	24,510	88,820,000
April to October	99	144	969	4,977,000
Total				93,797,000
Tidewater:				
November to March	2,754	61	11,420	41,386,000
April to October	99	61	411	2,111,000
Total				43,497,000
Total, power and energy			35,930	137,294,000

deal of secondary energy could be produced during certain periods, and actual energy output could exceed that shown in table 11. Energy potential from table 11 for the November–March period is 88,820,000 kwhr at the Scott Mountain site and 41,386,000 kwhr at the Tidewater site for a total of 130,206,000 kwhr.

PUMPED-STORAGE POSSIBILITIES

Pumped-storage powersites are plentiful in the Alsea River basin. One of these, the Five Rivers dam and reservoir site, has already been mentioned. To utilize this site for pumped-storage would require construction of a low dam downstream on the Alsea, probably at or below Tidewater, to raise the water to the tailrace of the Five Rivers powerplant site, which would be 80–90 feet above sea level. Water would be pumped from this reservoir into the upper reservoir on Five Rivers during off-peak hours, and energy would be supplied from the network or some of it could be generated at a powerplant installed at the Tidewater dam. The amount of power made available would depend upon the size of the installations and the length of the periods of use.

A pumped-storage development might also be achieved in connection with the Tidewater reservoir. Pumping capacity and head could be combined as desired to achieve a large or small development. A large development is described.

A hilltop in the SE¼ sec. 20, T. 13 S., R. 10 W., rises to an altitude of 1,750 feet above sea level within about 1½ miles of the Tidewater damsite. At an altitude of 1,600 feet the hilltop has an area of approximately 37 acres. By removing the top above this altitude and using it for embankment material, a reservoir of about 1,750 acre-foot capacity could be constructed. The embankment would be about 70 feet high, and have a 10-foot top and slopes of

2.5:1 in the reservoir and 2:1 outside of it. Average head would be about 1,560 feet, the difference between the reservoir at one-half capacity (estimated at altitude 1,637 ft) and the maximum pool of the lower reservoir (altitude 77 ft). The pumped water could also be passed through the turbines of the plant (see p. D33) below the Tidewater dam.

The potential capacity of the site would depend upon the schedule of operations. If, for example, the upper reservoir were filled and emptied twice a day, the filling being accomplished in 6- to 12-hour periods and the emptying in 3-hour periods, 7,060 cfs would be available for producing power. Acting through a head of 1,560 feet, this water would generate 749,000 kw at an operating efficiency of 80 percent. This would make 4,494,000 kwhr of energy available daily for use during hours of high demand. Pumping requirements would be in the neighborhood of 6 million kwhr daily of off-peak energy. This energy requirement is several times greater than the potential production capability of the Alsea River plants, and interconnecting high-voltage transmission lines would be a prerequisite.

A potential storage site on Canal Creek, which enters the Alsea in sec. 32, T. 13 S., 10 W., about half a mile downstream from the Tidewater damsite might be used for pumped storage. Drainage area of the creek is only 13 square miles, and for that reason the creek has little value for power production by itself. However, it could be used as a storage basin for a pumped-storage development in which as much as 400 feet of head could be achieved if a topographically suitable damsite in sec. 5, T. 14 S., R. 10 W., is geologically sound. According to the Tidewater quadrangle (scale 1:62,500, contour interval 50 ft), the reservoir would have the following areas and capacities:

Area and capacity of Canal Creek reservoir site

[Damsite in sec. 5, T. 14 S., R. 10 W.]

Altitude (feet)	Area (acres)	Capacity (acre-feet)
40	0	0
50	30	150
100	80	2,900
150	170	9,000
200	300	21,000
250	510	41,000
300	750	73,000
350	1,020	117,000
400	1,390	177,000

Water could be pumped into the Canal Creek reservoir from the Tidewater reservoir. A very large installation having a tailrace altitude of 10 feet could be made near tidewater for producing peaking kilowatts, or the relatively large volume of water could be held as standby storage for carrying a smaller plant for a considerable time. The reservoir would hold enough water for a discharge of 3,000 cfs for 30 days. When the reservoir was drawn to one-half capacity, head would be about 310 feet. It could produce nearly 63,000 kw at 80 percent efficiency.

DIVERSION TO THE WILLAMETTE RIVER BASIN

Alsea River water could be diverted to the Willamette River basin by way of tributaries to Marys River. Most of the water available for diversion falls as precipitation during the fall and winter months when there is ample water in the Willamette tributary streams in the area, and suitable storage sites to hold the water for use during July, August, and September would be necessary. The diverted water would probably be used principally for municipal supplies as the Corvallis-Philomath area population increases, but it would also be capable of providing supplemental irrigation water to lands on the benches on either side of Muddy Creek and the Marys River south of Philomath. Diversion routes from North and South Forks Alsea River are shown on plate 2.

An extensive study of the need for diversion of water from the Alsea River to the Willamette River basin is not within the scope of this paper. However, the amount of water available for diversion and the general physical barriers to such diversion are properly briefly described. Before a diversion is undertaken, a study should be made of the possibility for pumping the necessary water from the Willamette River and its tributaries and for providing the water in the Willamette when needed by constructing reservoirs inside the basin upstream from Corvallis.

NORTH FORK

The drainage basin of the North Fork Alsea River upstream from the County Line site could be connected with Marys River by way of Crooked and Wells Creeks in a canal which would be about 7 miles long and which would include some tunneling. In lieu of the canal and tunnel combination, a tunnel 3.7 miles long beginning at an altitude of about 850 feet would connect the County Line reservoir site and a point near a balancing reservoir site on Crooked Creek in the Alsea River basin. A second tunnel about

0.8 mile long at an altitude of 800 feet would connect the balancing reservoir with Wells Creek, a tributary of Greasy Creek in the Marys River basin. A conduit about 1½ miles long could carry the water from the tunnel outlet to a point on Greasy Creek where it can be dropped 290 feet (altitude of 790 ft to an altitude of 500 ft) if development of power in connection with the diversion is deemed advantageous.

It is estimated that 62,000 acre-feet could be diverted after allowing an average discharge of 10 cfs to pass downstream and after deducting 10 cfs for losses by evaporation and seepage in the County Line reservoir. The County Line reservoir site has a capacity of only 24,000 acre-feet for a 200-foot dam, and an additional storage site will be needed in order to assure delivery of water during dry periods. The most favorable site for this purpose is on the east side of the Oregon Coast Range on Greasy Creek, a Marys River tributary. A dam in sec. 16, T. 12 S., R. 6 W., that would raise the water surface from an altitude of about 310 feet to the 500-foot contour as shown on the Corvallis and Monroe 15-minute quadragles would create a reservoir of 144,000 acre-feet. The reservoir would be entirely downstream from the Wells Creek powersite previously mentioned, and the water could be retained in the reservoir until needed for municipal purposes or for irrigation. The area and capacity of this site are shown in the following tabulation:

Area and capacity of Greasy Creek reservoir site

[Damsite in sec. 16, T. 12 S., R. 6 W.]

Altitude (feet)	Area (acres)	Capacity (acre-feet)
310	0	0
350	180	3,600
400	590	22,850
450	1,160	67,000
500	1,950	144,000

SOUTH FORK

The drainage basin of the South Fork upstream from the Peak Creek site in secs. 23 and 26, T. 14 S., R. 7 W., could be connected with Marys River by way of Oliver and Muddy Creeks through a tunnel 2.2 miles long at an altitude of 800 feet. A tunnel 2.5 miles long at the same altitude would surface on Rainbow Creek, which is tributary to Oliver Creek in the Muddy Creek basin. This

conduit could be extended about three-fourths of a mile in a canal to a 385-foot drop site should it be found desirable to develop waterpower in connection with the diversion. The average yield of the South Fork and Peak Creek at the Peak Creek site is estimated to be about 65,000 acre-feet per year. About 50,000 acre-feet would be available for diversion after providing 10 cfs average discharge downstream at the Peak Creek damsite and allowing an additional 10 cfs for evaporation and leakage in that reservoir. The Peak Creek reservoir site having a dam that would raise the water 225 feet to an altitude of 900 feet would have a capacity of about 74,000 acre-feet. The area-capacity tabulation shows that 67,000 acre-feet of this water would be stored above an atlitude of 800 feet. This storage would make the diversion schedule very flexible without any storage required on the east side of the Coast Range.

The valley bottoms on Muddy Creek and its tributaries are wide, and the streams have gentle gradients downstream from the powersite suggested on Rainbow Creek. Damsites are lacking, however, and it would be necessary to construct long dikes to retain the water until the dry season. A reservoir having a maximum altitude of 400 feet might be created on Muddy Creek in the valley around the settlement of Alpine. A dam 110 feet high would be 0.9 mile long at a site in the SW¼ sec. 19, T. 14 S., R. 5 W., and the SE¼ sec. 24, T. 14 S., R. 6 W. Auxiliary dams about 50 feet high would be required in saddles on each side of the main embankment. The area and capacity of this site as measured from the Monroe 1:62,500-scale quadrangle map, which has a 50-foot contour interval, are shown in the following tabulation.

Area and capacity of Alpine reservoir site, Muddy Creek

[Damsite in sec. 19, T. 14 S., R. 5 W., and sec. 24, T. 14 S., R. 6 W.]

Altitude (feet)	Area (acres)	Capacity (acre-feet)
290	0	0
300	730	3,650
325	1,460	31,000
350	2,450	79,900
375	3,070	148,900
400	4,320	241,000

If this site is to be used, it would probably be limited to a dam height that would create a reservoir having a maximum pool of not more than 350 feet in altitude unless it is to serve also as a

storage site for excess runoff from Oliver and Reese Creeks and the Long Tom River. Water from these streams would have to be pumped into the reservoir, but that might become economically feasible and desirable. In any event the site is so large in comparison with the South Fork Alsea water available that complete regulation would be easily achieved by its use. The reservoir could be useful for flood control on Muddy Creek and Marys River.

A Corps of Engineers channel rectification project has been authorized on Muddy Creek and Marys River to benefit 16,275 acres of land.

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