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Investigation and development of ground water of Chalone Creek, (Pinnacles National Monument, California)

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INVESTIGATION AND DEVELOPMENT OF GROUND WATER OF
CHALONE CREEK, (PINNACLES NATIONAL MONUMENT, CALIFORNIA)

BY

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THESIS

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ABSTRACT

The Pinnacles National Monument is located in San Benito and Monterey Counties about 130 highway miles south of San Francisco, California. Because of unusual patterns in volcanic rocks, the Monument was set aside as a recreation area on January 16, 1908, by President Theodore Roosevelt. It encompasses an area of approximately 12,818 acres.

Water quality and supply problems were experienced at the Monument in the few years prior to 1961. In 1961, a collection trench system was installed to alleviate the water supply condition. A horizontal trench system, 215 feet long and four feet below the dry season water table, was installed in the creek bed of Chalone Creek.

Driven well-points were used to determine fluctuations of the water level under pumping conditions. The use of well-points in a coarse alluvium, such as Chalone Creek alluvium, is not recommended by the author. Only 8 out of 18 well-points were successfully installed. Ripped screens in the well-points and bent pipe strings were the chief reasons for lack of success.

A channel sample was collected from the alluvium to determine the laboratory coefficient of permeability. The value was determined to be 5.5×10^2 gal/day/ft^2. A mechanical analysis was conducted on the channel sample giving a mean grain size of 16mm, sorting coefficient of 6.7, phi skewness value of 0.6, and phi kurtosis of 0.33.

A pumping test was conducted to determine the field permeability and transmissibility of the Chalone Creek alluvium. It was calculated to have a permeability of 6.3×10^2 gal/day/ft^2. The transmissibility was calculated as 3.2×10^4 gal/day/ft. The storage coefficient of the aquifer was determined to be .029.

Chemical analysis of water samples from Chalone Creek and Willow Spring, the largest of three springs in the area, indicated the quality of water is good for drinking purposes.

Utilization of the horizontal trench collection system should adequately supply the water demands of the Monument. Calculations indicate the trench could be pumped at a rate of 75 gal/min. for 1000 days without lowering the water table more than four feet.

PREFACE AND ACKNOWLEDGEMENTS

This dissertation was possible only with the cooperation of the National Park Service. Through a series of correspondence between Dr. James Maxwell and the National Park Service, several possible Thesis problems were suggested. It was decided that the project to be initiated at the Pinnacles National Monument would provide an opportunity to analyse the use of a horizontal trench water collection system.

The author would like to thank the National Park Service; Dr. James Maxwell of the Missouri School of Mines and Metallurgy; Ranger Robert Ramstad of the Pinnacles National Monument; Mr. Lynn Spaulding, Engineer in charge of the Pinnacles project; Mr. Frank Watz of the U.S. Bureau of Mines; and the author's wife, Carmen; for their cooperation in making this Thesis possible.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

I. INTRODUCTION

A. LOCATION AND PURPOSE OF STUDY

The Pinnacles National Monument is located in San Benito and Monterey Counties in California (Fig. 1) . It is about 130 highway miles south of San Francisco, 36 miles south of Hollister, and 35 miles north of King City via State Highway 25. The Monument is in the Gabilan Range which extends northwest-southeast between the Salinas and San Benito Valleys. It encompasses an area of approximately 12,818 acres.

Because of unusual erosion patterns in volcanic rocks, the Monument was set aside as a recreation area on January 16, 1908, by President Theodore Roosevelt. Land grants in 1923 and 1924, and purchasement by San Benito County in 1931 and 1933 brought the acreage to its present size.

Because of recent water shortages and a decline in the water quality at the Pinnacles, a water collection system was to be installed during the summer months of 1961. The purpose of this study was to determine if the collection trench system installed at Chalone Creek would adequately supplement the ground water reserves at the Pinnacles National Monument.

B. LOCAL GEOGRAPHY

The Pinnacles National Monument has a roughly rectangular shape. The longest dimension is north-south and extends over seven one-mile sections. At the northern end, the east-west dimension is four sections and at the southern boundary the east-west dimension is only two and

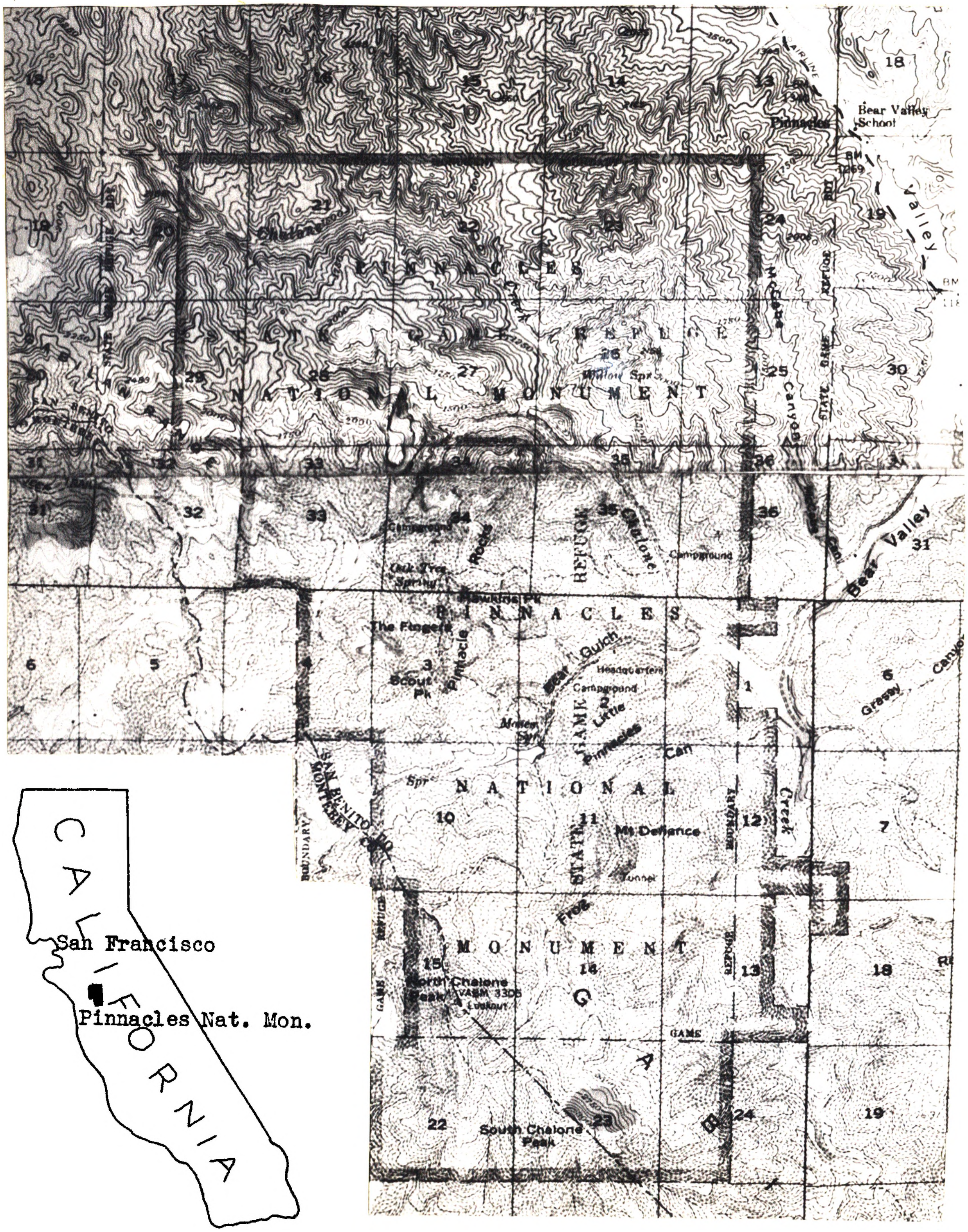


Fig. 1 Location map of Pinnacles National Monument

one-half sections.

The only paved access to the Monument is through Bear Valley (Fig. 1). The road enters the Monument at the east central border. The Monument headquarters and museum are located in the central part of the Monument, along Bear Gulch (Fig. 1). Near Moses Spring and the campground located in section 27 (Fig. 1) are large natural caves. Chalone Creek, the principle creek in the area, drains south from the northern border. Willow Spring, the largest of three springs in the Monument, rises in the northeast part of the Monument.

The Pinnacles National Monument is designed for the lover of nature. It has no hotels, swimming pools, golf courses, or night clubs, but boasts 23 square miles of beautiful craggs, pinnacles, and spires of buff, gray, and lavender volcanic rocks. Wildlife include deer, racoons, over 44 species of wild birds, and 200 species of wild flowers. Public facilities consist of three campgrounds, a ranger station, and a museum. Many of the spirelike rock formations are over 1000 feet high; notable is Chalone Peak (3,297 ft.) which commands a wide view of the surrounding country. The major part of the pinnacles are found on, or flanking, a north-south ridge which divides the Monument into east and west sides, unconnected by road. This central backbone has been cut through in two places by streams flowing east, leaving deep clefts. Into these notches, huge fragments of rock have fallen creating cool, dark, and unusual caverns. One of these caves is illuminated and easily approached by trails. Trails also lead to other major points of interest in the Monument.

C. CLIMATE

The Pinnacles National Monument has a Mediterranean type climate. The normal annual temperature is 60° F., with a normal winter temperature of 45° F, and a summer temperature of 65 to 75° F. The normal annual evaporation from pans is 60 inches, with May to October accounting for 55 inches. The normal annual evaporation from shallow lakes and reservoirs is 40 inches, and the normal annual excess of precipitation over evaporation is zero. The normal annual precipitation is 30 inches, of which 50% occurs in the winter, and 25% in the spring. About 18 inches or 60% of the total precipitation occurs from December to March inclusive. Normally the wettest month is January and the driest is July. The normal annual snowfall is one inch. The normal annual runoff is 10 inches which is 10 to 40% of the precipitation. The normal annual water loss by runoff and evaporation is 15 to 20 inches.

D. GENERAL GEOLOGY

Before 1932, geologic work in the Pinnacles National Monument was restricted to regional surveys. In 1794 the explorer, Captain George Vancouver, mentioned the area as an "unusual mountain." In 1894 H. W. Fairbanks made a reconnaissance of the area and briefly described the volcanic rocks. P. F. Kerr and H. G. Schenck (1925) investigated lavas six miles north of the Monument and considered them to be a continuation of the rhyolite that occurs in the Pinnacles.

In 1932, Phil. Andrews began his studies on the "Geology of the Pinnacles National Monument." The rock types and geologic sequence were worked out in detail (Plate 1, envelope). Andrews (1936, P. 5) states: "The oldest rocks in the region belong to the Sur Series, of which the Gavilan Limestone is a member." R. D. Reed (1933) dated the Gavilan Limestone as Carboniferous or Permo-carboniferous. The Sur Series contains both ancient sediments and igneous rocks. Andrews refers to the granitic rocks of the Sur Series as Santa Lucia. The sedimentary and igneous rocks are all pre-Franciscan.

Cretaceous and early Tertiary sediments are absent in the area which suggests that the area was a positive land mass during that time, or that subsequent erosion has removed any sediments that were deposited. Sometime during this interval the Santa Lucia granitic mass was almost completely unroofed. In what is believed to be middle Miocene time, orogenic activity began with the injection of a rhyolitic magma through fissures in the granite. As the activity continued, a central vent zone, trending north and south, was developed. Subsequent explosions and eruptions deposited a considerable thickness of pyroclastics over the earlier lavas. Weathering and erosion of the pyroclastics has resulted in the characteristic pinnacles of the area.

In later Miocene time, large fanglomerate deposits, consisting of both rhyolitic and granitic debris, were deposited along the margins of the Gablian Range. Andrews (1936, P. 6) states:

> "Farther from the source, the fanglomerates grade laterally into continental and shallow marine arkosic gravels which were in part interbedded with and overlain by diatomaceous shales of considerable thickness. The rather abrupt change from gravel to diatomaceous shale suggests a lowering of the land mass and consequent reduction in the amount of detritus entering the sea. The volcanic rocks or the Pinnacles were probably never submerged."

No Pliocene sediments have been recognized in the area, but river terrace gravels of probable Pleistocene age are distributed along the margins of the volcanics. The terrace deposits have been and are still in the process of being dissected because of readjustment of the land surface.

The area is five miles to the southwest of the San Andreas fault. Faults in the area roughly parallel the San Andreas and show considerable movement since Miocene Monterey time.

E. LOCAL GEOLOGY

Chalone Creek drains southward and occupies the approximate line of a major north-south trending fault (Plate 1, envelope). The bedrock in the Creek area, where exposed, is severely fractured. The creek bed consists of conglomeratic deposits of rhyolite, granite, granodiorite, and quartzite fragments which range in size from boulders 10 feet in diameter to silt particles.

The Temblor formation, to the east of Chalone Creek, is a fanglomerate of Miocene age. It is a thick continental deposit which grades from a fanglomerate eastward into arkosic gravels which conformably underlie white chalky diatomacious shales. More than 1000 feet of the fanglomerate is exposed just east of Chalone Creek and northwest of lower Bear Valley.

The Temblor formation is a crossbedded, poorly consolidated deposit consisting of angular granite blocks, pink to gray rhyolite, lapilli-tuff, and metamorphic rocks which occur in arkose beds. The average boulder size is 30 inches, but many range up to 6 to 8 feet in diameter.

The beds range from nearly horizontal attitudes to a dip of 10 degrees southwest. The color of the deposit is usually a dull tan, yellow, gray, or brown.

The Temblor lies in fault contact with rhyolites and granites south of Chalone Creek. The Temblor beds appear to be derived from the Gabilan Range on the basis of their composition. The formation was dated by its stratigraphic position and by fossils found in the diatomaceous shale which overlies it.

Terrace deposits of Pleistocene age occur along portions of Chalone Creek. They are poorly stratified and may vary in thickness from a trace to 100 feet or more. The average dip is 3 to 5 degrees eastward away from the central volcanic mass. The terrace deposits blanket granite and rhyolite bedrock and appear to be younger than much of the faulting in the area.

Rhyolitic masses form the western boundary of Chalone Creek. The rhyolite is predominately massive, but fracturing is common near the creek bed due to the fault zone. The rhyolite has been dated as Miocene.

F. HISTORY OF THE PROBLEM

A serious need for improvement of the water supply in the Pinnacles National Monument led to the investigation and subsequent development of the Chalone Creek development project. This project was sponsored by the National Park Service. Its purpose was to supplement immediately the water supply for Chalone Creek campground and ultimately supplement the rest of the Monument's water supply.

The project was initiated as a result of a letter to the National Park Service Region IV Headquarters, dated February 27, 1959, from the National Park Services Western Office of Design and Construction. It stated the unsatisfactory quality of the Pinnacles National Monument water supplies and that improvements should be made before the 1959 tourist season opened. As a result, emergency funds were appropriated by the Park Service to expedite the improvement program. Funds for a geological survey of water resources at the Pinnacles Monument had been previously approved for the 1960 budget. In view of the urgency of the problem, the U. S. Geological Survey district office in Sacramento agreed to conduct an investigation on March 31, 1959.

Mr. Curtis E. Richey, (1959, P. 2) the National Forest Service Consultant on the investigation party, stated a summary of the presently used sources of water.

> "Moses Spring now provides the domestic water supply for Bear Gulch area including the campground, residential areas for the National Park Service employees and the museum. Water flows from the spring at a 2.5 quarts per minute rate to a 20,000 gallon capacity masonry reservoir and serves the distribution system by gravity. The spring collection box and reservoir are of tight concrete construction with no apparent sanitary defects, but a series of samples collected over a period of several years indicates the supply is heavily contaminated. It is believed surface pollution is entering the aquifer without the usual filtering action of the soil. It was agreed Moses Spring with its very meager yield of less than 600 gallons per day should be given no further consideration as possible source of water supply for future development.

Bear Gulch Reservoir with a capacity of approximately 13 million gallons was constructed in the 1930's by C.C.C. labor to provide water for sanitary purposes and fire protection in the Bear Gulch Headquarters area. It was agreed the Reservoir might not provide an adequate supply of water in dry seasons andthat the cost of constructing and operating a complete treatment plant to treat the small amount of water required at Pinnacles would be prohibitive.

Willow Springs which supplies the Chalone Creek campground rises in a supply area at the head of a narrow canyon draining to nearby Chalone Creek. Flow from the spring is apparently adequate to supply present needs, but it was agreed that it would be difficult to capture the flow from the swampy area in a manner that would adequately protect the supply against surface pollution. This source of supply offers the advantage of gravity flow, but cost of development would be high due to the inaccessible location and the necessity of replacing the two mile long supply line from the Spring to the campground."

Ruling out the above sources, the only supply area, economically and geologically reasonable, would be the alluvial fill in the vicinity of Chalone Creek. Chalone Creek nearly conforms to a major north-south trending fault and the bedrock is extensively fractured. To the east of the fault line lies the Temblor formation, a fanglomerate that is believed to contain water. The channel of Chalone Creek is filled with unconsolidated conglomeratic alluvium and natural sand and gravel deposits.

Agreeing that a test well should be sunk in the Chalone Creek vicinity, the investigating committee made two location recommendations. The first proposed site was to be located about 200 feet downstream and on the opposite side of the road from Chalone Creek campground (Plate 2, envelope). A 100 foot deep test well was to be drilled at this location. Development of the well was to follow if favorable water bearing formations were encountered.

An alternate location was chosen approximately six tenths of a mile downstream from the first site. It lay in the wide valley below the confluence of Bear Gulch and Chalone Creek. The valley fill is much thicker at this site and the probability of finding water in the desired quantity is much greater than at the first site. The difficulty with this alternate site is that the cost of construction would be greater and the pumphouse could not be concealed as easily.

The Precision Drilling Company of Santa Rosa, California, was contracted to drill a test well at site one, on the west bank of Chalone Creek at an altitude of about 1,020 feet. Mr. R. E. Evenson of the U. S. Geological Survey (1959, P. 10) states

> "The well was drilled to a depth of 41 feet, cased with extra-heavy duty 8 inch casing to a depth of 35 feet below ground surface. The casing from 14 to 35 feet below ground surface was perforated by 8 inch long acetylene-torch cuts, spaced in a pattern of 4 cuts per linear foot of casing. When pumped, the well yielded 11 gallons per minute (gpm) at a pumping level of 14 feet below the top of casing and a drawdown of 4 feet below the standing level in the casing."

The well log (Table 1) shows the development of the well was unsatisfactory because of invasion of the walls of the well by drilling mud. Mr. Evenson expressed the belief that the factors that caused the invasion were the highly permeable character of the alluvium and the low hydraulic pressure of the groundwater. The yield of the well was much less than to be expected for sandy material, and even though the pumped water did clear of mud, the permeability of the ground around the well must have been reduced considerably.

A strong hydrogen sulfide odor was noticed when the water level was near the bottom of the well. After a sample of water was allowed to stand a few minutes, the odor was not present. Mr. Evenson stated that the sulfurous odor may have been due to water moving from the fault zone into the alluvium, but increase in water temperature was not apparent. A cement plug at the bottom of the well is believed to have sealed the sulfurous water from the well.

The well log as compiled by the driller and observers is shown in Table One.

A pump house was installed over the well and utilized, however, the water supply proved to be inadequate. This was probably a result of the drilling and impregnating the walls of the test well and reducing the permeability of the surrounding material. To supplement the water supply, a new water gathering system was designed by Mr. Lynn Spaulding, the Hydraulic Engineer for the San Francisco Office of the National Park Service. This system was to be installed at site one during the summer months of 1961. The author was employed to assist Mr. Spaulding in the investigation of the system and to collect additional data on the hydrological features of Chalone Creek.

The plans called for a cutoff trench 215 feet long, to extend across the channel of Chalone Creek at a depth of 4 feet below the water table. At the bottom of the trench, 12 inch perforated pipe was to be laid at a slope of one foot per hundred feet. It was to connect to the well casing beneath the existing pump house. A shallow runoff trench was to be constructed perpendicular to the cutoff trench to accommodate overflow.

TABLE I

Driller's Log of Chalone Creek Pump House Well

Date	Material	Thickness (feet)	Depth (feet)
July 9	Started drilling (11:30 a.m.). 9-3/8" rotary hole. Drilling fluid consists of bentonite and lime.		
	SAND, grayish brown, medium to coarse, soft, subangular; predominantly clear quartz, but also fragments of rhyolite and granite.	5	5
	SAND, as shown above, and gravel, loose. . .	13	18
	SAND, brown, coarse, soft, subangular; predominantly clear and white quartz . . .	4	22
	GRAVEL, medium gray, probably cobbles and small boulders, loose, almost no sand; fragments of granite pegmatite, rhyolite, andesite; white quartz is minor.	12	34
	BOULDERS, apparently larger than above because of very difficult drilling, fragments chiefly rhyolite	4	38
	RHYOLITE, either very large boulder or bedrock.	3	41
July 10	Stop drilling 5:30 p.m.		
July 13	Fluid level 9.5 feet. 4:00 p.m. cleaned hole to 41 feet, set 41'7" of heavy-duty casing, 8" I.D., with 1'6" above ground surface. Bottom 21 feet casing perforated with torch cuts. 6"-8" long, 4"-6" apart, 4 cuts around.		
July 14	Kelly lowered with packer at top of casing. Clear water pumped through casing into bore to wash out drilling mud. 5 1/2 tanks or about 2,500 gallons failed to clear well of drilling mud. Bailed about 130 bailers (14 to 25 gallons each) for a total of about 2,600 gallons and well still failed to clear.		

July 15 Depth to water 9.30 feet below land surface datum. Set test pump. Pumped at rates of 5 gallons per minute to 60 gpm, broke suction at 37 feet whenever pumping rate exceeded 7 1/2 gpm. Pumped about 10 hours or so. Depth to water level outside casing 17 feet when water inside casing was at 35 feet. Small amount of water moving through perforations. Failed to clear. Water has sulfur odor.

July 16 Depth to water 9.25 below land surface datum. Pulled casing 5.35 feet and 5.0 feet cut off. Top casing now 1.85 feet above land surface datum, and uppermost perforation at 13.75 feet below LSD*

Pumped from 9:20 a.m. to July 17 at 4:00 a.m. --about 19 hours--at rates from 12 to 30 gpm. Failed to clear.

July 17 7:30 a.m. Depth to water 9.06 feet below LSD. Alternating pumping and surging. Attempted pumping and simultaneous jetting between casing and bore with no observed improvement. Rigged up leather gasket and swabbed for about an hour. Two hours pumping 6:00 p.m. to 8:00 p.m. Water clear at 8:00 p.m.

July 20 Depth to water 9.03 feet below LSD, 10.86 feet below top of casing (measuring point). Started pump at 12:00 noon, pumping 7.5 to 9.5 gpm until 2:45 p.m., pumping level reached 13.40 feet below measuring pints. Increased rate to 13 gpm until 4:00 p.m.; pumping level reached 14.05 feet below MP** Averaged about 17 gpm until 7:50 p.m., pumping level reached 20.05 feet below MP.

July 21 Pumped at 10 to 15 gpm until 1:58 a.m.; pumping level 19 feet. Pump stopped for 1 minute. 2:00 a.m. to 3:00 a.m. pumped about 20 gpm to reach maximum pumping level of 28 feet below MP. Pumped 13.6 gp, for 3:15 a.m. to 5:10 a.m. At maximum pumping level of 28 feet below MP. Water level 17.10 below MP outside casing and 28.0 feet inside casing.

Pump stopped 5:10 a.m., water level recovered to 12.00 feet below MP at 6.00 a.m. Pump started 6:04 a.m., rate varied from 9 to 13 gpm until 10:27 a.m. when rate was reasonably stabliized at 11.5 gpm. Pumped until 7:00 p.m. with maximum pumping level at 14.90 feet below MP. Pump off 7:00 p.m. Water level 11.23 feet below Mp at 8:00 p.m.

July 22 Depth to water 11.00 feet at 5:45 a.m. Cement placed at 29 feet.

* LSD = Land Surface Datum

** MP = Measuring Point

II. FIELD WORK

The field work for the project was organized into four distinct phases. The first phase was orientation and planning for groundwater tests; the second consisted of preliminary investigation; the third was construction of the collection trench system; and the last was the well-point installation and pumping test.

A. ORIENTATION AND PLANNING FOR GROUNDWATER TESTS

The author arrived at the Pinnacles National Monument on June 27, 1961, and was introduced to the National Park Service Hydraulic Engineer in charge of the proposed collection trench system, Mr. Lynn Spaulding. The next few days were spent in investigating the area and becoming acquainted with the local geology. The author, with Mr. Spaulding, visited Willow Spring, where Mr. Spaulding suggested the author measure the flow of the stream and if possible, make an analysis of the water to compare it chemically with that of Chalone Creek.

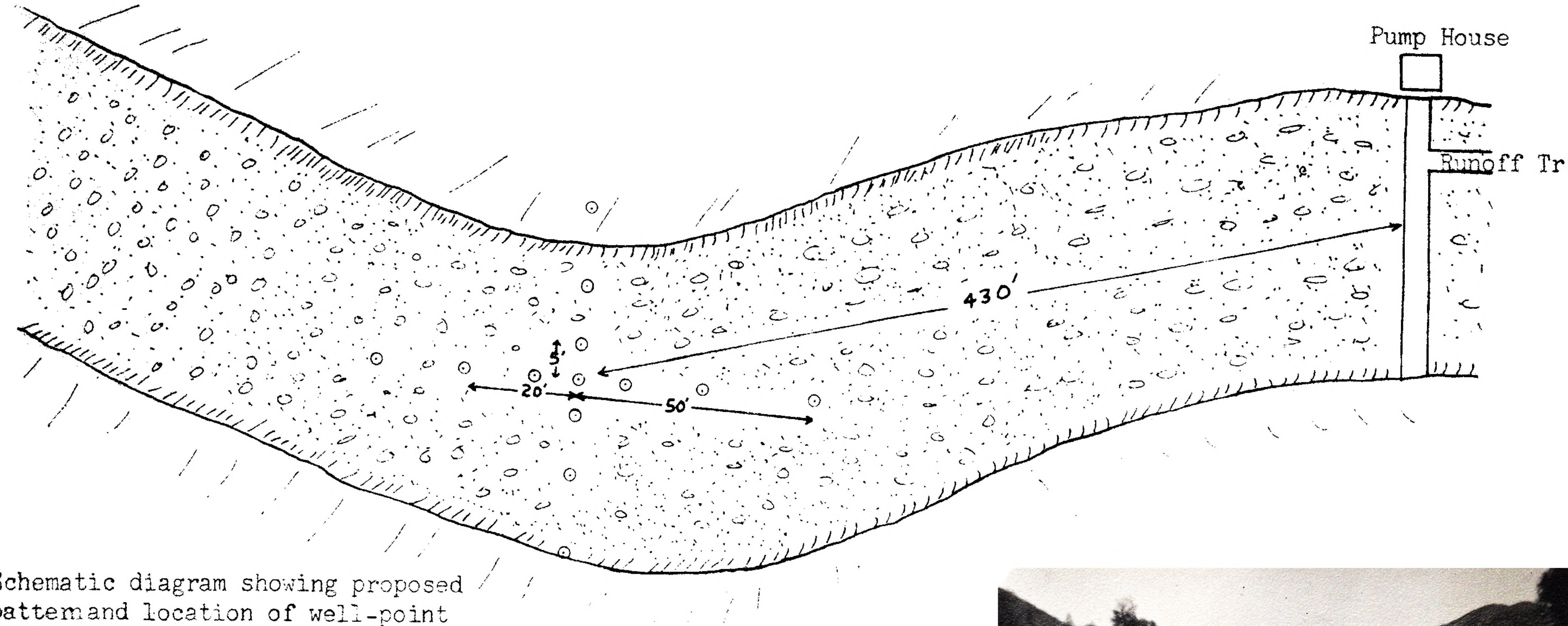
Mr. Spaulding described the proposed water gathering system, which he had designed, to the author. The choice of a collection trench system instead of a drilled well or pipe line was primarily one of economic necessity. The expense of either piping water from Willow Spring or drilling a new well and constructing a new pump house would be greater than that of a collection trench piping water to the casing of the existing pump house. The proposed trench would extend 215 feet across thechannel of Chalone Creek (Plate 2, envelope). It would be dug approximately 17 or 18 feet below the surface or 4 feet below the water table, because Chalone Creek bed has no surface flow except for the spring freshet season. Water was to be collected by placing 12 inch perforated pipe in the bottom of the trench, and connecting the perforated pipe to the casing of the existing pump house. A runoff trench was to be constructed perpendicular to the cutoff trench to channel overflow and help drain the cutoff trench during construction.

After examination of the creek bed in the area of the trench site, the author presented to Mr. Spaulding recommendations for the location of well-points for a pumping test. The proposed plan was for a system of well-points radiating from a central point at distances of 5, 20, and 50 feet from the central point (Fig. 2). It was agreed that the points should be placed upstream from the cutoff trench in order that the trench would not interfere with the groundwater flow to the test area. It was also agreed that at least two points should be placed on both sides of the cutoff trench.

A request was made by the author to the Park Service for the well-points to be used for the pumping test, and for authority to obtain a driving sleeve to install the points. Mr. Spaulding approved the requests.

B. PRELIMINARY INVESTIGATION

To facilitate the plotting of test well points and analysis of the pumping test, and to provide a base for water table mapping, a large scale (1:480) topographic map of the proposed test area was



Schematic diagram showing proposed pattern and location of well-point installations.

Figure 2. Proposed Pumping Site

View of proposed pumping area from downstream.

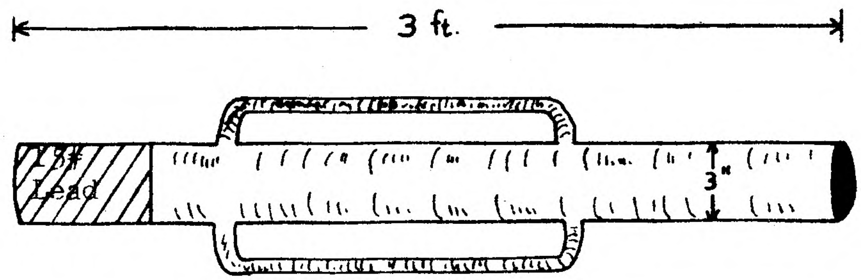
made. Because of temperatures in excess of 110 degrees in the afternoons, mapping was done in the early mornings and evenings.

After the maps were completed, sections of pipe to be used in driving the well-points were prepared. National Park Service Ranger, Robert Ramstad, supplied twenty foot long sections of 1.5 inch pipe that had been recovered from an abandoned water line. The author cut and threaded approximately 60 five-foot sections of the pipe (Plate 3).

The author designed a weighted driving sleeve, (Fig. 3), to facilitate the driving of the well-points. The idea was based on simply designed post drivers used on many farms in Iowa. A three-foot section of three-inch diameter pipe was taken to a blacksmith shop in Hollister, California. The blacksmith was instructed to put fifteen pounds of lead in one end of the pipe. To accomplish this, holes were drilled near the end of the pipe and a series of wires were placed through some of the holes in the pipe to reinforce the bond between the lead and the pipe. Other holes were left open to allow some of the lead to flow through the holes and also strengthen the bond between the lead and the pipe. Round iron rods, one inch in diameter, were welded to the outside for handles. The total weight of the sleeve with handles was fifty pounds.

The above steps completed the preparation for the installation of the well-points. Because delivery of the points had become delayed, time was available for the investigation of surface flow at Willow Spring.

Plate 3. The author threading pipe sections to be used in the installation of well-points.



Design of drive sleeve

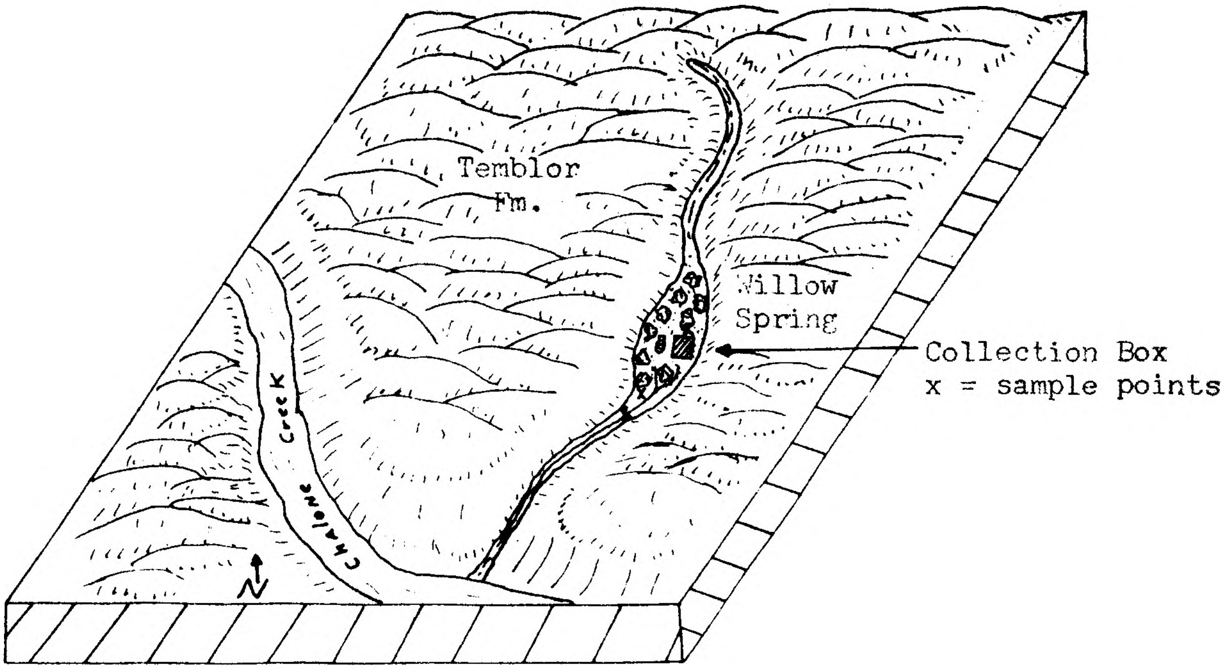
Utilization of drive sleeve
(pump house in background).

Figure 3. Drive Sleeve

Willow Spring (Fig. 4) rises in a marshy area of approximately one half acre. The Spring is located on the northeast side of the Monument and drains southwestward into Chalone Creek. The discharge from the Spring is sufficient to cause surface flow until it reaches the floodplain area of Chalone Creek, where it becomes entirely subsurface flow.

The spring rises about a quarter of a mile downstream from the head of a drywash canyon. The canyon is cut in a fanglomerate deposit, which is the Temblor formation as described by Andrews (1936). Outcrops of the Temblor could be seen on both sides of the canyon. The author did not note any bedrock exposed in the immediate area of the spring. The source of the spring is not known, but the hills behind the spring are probably the catchment area. Local residents in the area informed the author that to their knowledge the spring had not been dry in the past twenty years, even though other smaller springs in the area have dried up in some drought seasons. In the marshy area of the spring, water can be seen percolating down the sides of the canyon. Ferns are growing up the sides of the canyon approximately twenty feet and seem to outline the height above the valley floor from which water comes out of the Temblor formation to trickle down to the spring. The growth in the marsh consists predominately of willow trees, Edwardian ferns, and reeds.

Flow through the marsh consists of many small rivulets. On the south side of the marsh, a collection box was installed in the 1930's by Civilian Conservation Corps labor, to supply water to Chalone Creek



Schematic diagram showing location of Willow Spring and sample points.

Figure 4. View of Willow Spring from west bank. Collection box barely visable in lower center of picture.

campground. It consisted of a rectangular concrete box about four by five feet on a side, and four feet deep. A shallow trench had been dug around the box to permit drainage and collection of water. On the upstream side of the box a tile pipe six inches in diameter was put through the box to allow water to flow into the catchment box. On the downstream side of the box another tile was placed to allow overflow water to flow downstream.

The measurements at the spring were taken at three different locations; at the catchment box, on the outskirts of the spring, and 200 yards downstream. At the catchment box, where the spring water is trapped, the measured inflow of water was 15 gallons per minute. This was obtained by catching the inflow through the six inch tile pipe in a gallon can and measuring the filling time with a stopwatch. On the outskirts of the marsh area, several small streams flowed from the marsh and converged into the main trunk downstream. The largest of the channels was measured by digging a pit large enough to hold the gallon can and reducing the breadth of the channel above the can, by damming, until the entire flow was into the gallon can. The results were 15 gallons per minute. Further downstream where the full flow of the stream could be measured, the same method was used with a result of 30 gallons per minute. At this location, a water sample was collected to be analyzed and compared chemically with the water from Chalone Creek.

C. CONSTRUCTION OF THE COLLECTION TRENCH SYSTEM

On the morning of July 25, 1961, construction of the cutoff trench across Chalone Creek began. Mr. O. D. Bradford of Hollister, California, was contracted to clear and level the area of the proposed trench with his bulldozer. (Plate 4). The excavation was in typical mountain-stream conglomeratic alluvium, ranging in size from boulders four or five feetin diameter to silt particles. The ground was bulldozed in a V-shaped profile (Plate 4). The central portion was about six feet lower than the sides. As work progressed, definite tone patterns appeared in the soil. These tones represented varying amounts of soil moisture; darker tones presumably indicating more moisture than the lighter tones. Because the evaporation rate is greatest during July, it is probable that the dark tones represented capillary rise of water from the water table.

The author and Mr. Spaulding dug a hole with a hand auger at one of the dark-toned spots in the central low part of the V-shaped clearing. The hole was 140 feet from the pump house, along the line of the proposed trench. The purpose of the hole was to determine how deep the dragline that was to complete the job would have to excavate to strike water. Water was found at a depth of three feet. The water elevation in the hole was 1000.32 feet, while the water elevation in the pump house well was 1000.71 feet.

Mr. E. Henningson, from Salinas, California, had been contracted to dig the trench with his dragline. Excavation was started July 27, on the west side of the creek bed, 223 feet from the pump house (station

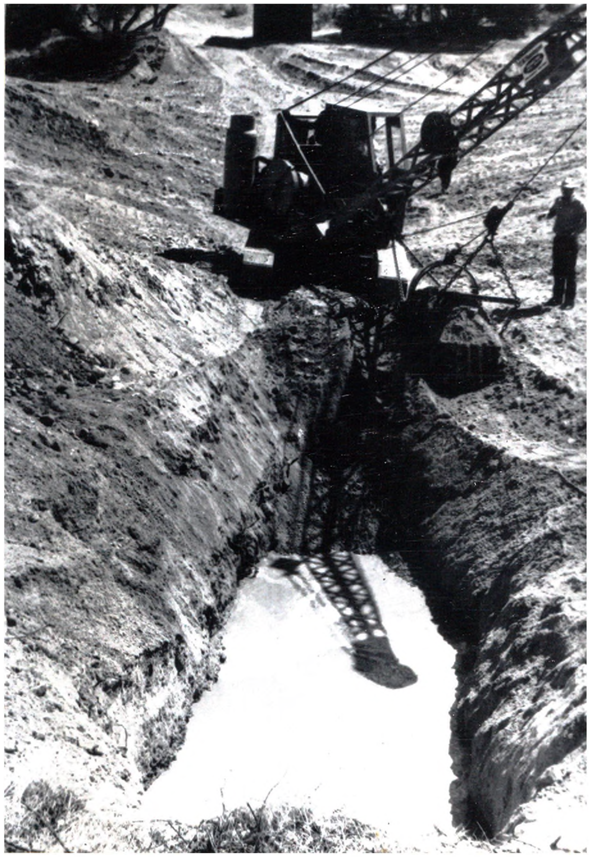
Partially cleared trench site
from pump house.

Pump house with trench site
in background.

Plate 4. Clearing of trench site.

2 + 23, Plate 2, envelope). The trench consisted of a cut about 4 to 5 feet wide at the bottom and 5 to 6 feet wide at the top. It was cut approximately four feet below the water table. Excavated material was dumped on both sides of the trench (Plate 5). As the excavation proceeded below the water table, water could be observed entering the trench from the upstream side. The dragline excavated the trench faster than the inflow of water could fill the trench. The result was an excellent view of the upper boundary of the water table. In the author's opinion, the water inflow was about 15 to 20 gallons per minute over an area of 3 feet in length and one foot above the standing water. The alluvium appeared to have a smaller range of sizes below the water table than above it. There was some trouble with caving of the walls at first, however as the trench was lengthened and deepened, the walls appeared to stabilize (Plate 6).

The dragline worked from west to east until it was within 60 feet of the pump house (station 0 + 60, Plate 2, envelope). A runoff trench was then constructed in a north-south direction downstream from the cutoff trench, to connect with the cutoff trench at station 0 + 60 (Plate 2, envelope). After the runoff trench had been excavated to within ten feet of the cutoff trench, the dragline returned to the cutoff trench and worked from the pump house westward to about station 0 + 50. This left a T-shaped plug of alluvium about ten feet long in the cutoff trench and ten feet long in the runoff trench.



Initial cut, west end of trench.

Dumping excavated material, pump house in background.

Plate 5. Dragline excavating cutoff trench.

Plate 6. Complete excavation from west end,
runoff trench in foreground.

Water elevations in the three separate trenches were measured at various times throughout the construction (Table 2). As can be seen from Table 2, the gradient of the water table in the downstream direction between the cutoff and runoff trenches was .75 of a foot per hundred feet. At station 'B', 3 + 00, downstream in the runoff trench (Plate 2, envelope), the water table intersected the surface and formed a marshy area. This was caused by a constriction of the valley by rhyolite flows, about 300 feet further downstream from station 'B', 3 + 00.

After the plug between the trenches was removed, the runoff trench functioned as a drain for the cutoff trench. The water levels in the cutoff trench were measured by use of a staff gauge driven in the bottom of the excavation 150 feet west of the pump house.

When the water level in the cutoff trench had dropped sufficiently to allow work in the trench, the bottom was leveled using hand shovels. The dragline operator had done an excellent job of leveling the bottom and the hand leveling was minor with the exception of the far east end of the trench, near the pump house. In this area the walls of the excavation were about 20 feet high, and proved to be extremely unstable. After a cave-in of about six cubic yards of the walls, it was decided the area must be supported. Two sections of timbering were used. Each was about 16 feet high and 10 feet long with three 1" by 12" planks nailed horizontally at one foot intervals, between 4 x 4 timbers, (Plate 7).

TABLE II

Water Elevations in the Three Trenches

West Trench

Date	Time	Elevation
July 29		1000.07
July 31		1000.06
Aug. 1	3:43 p.m.	999.97
Aug. 1	4:10 p.m.	999.94
Aug. 1	4:25 p.m.	999.92
Aug. 1	(at 4:54 p.m. first surface flow into runoff trench)	
Aug. 1	4:55 p.m.	999.87
Aug. 1	5:01 p.m.	999.59
Aug. 1	5:12 p.m.	999.25
Aug. 1	5:38 p.m.	999.13
	(at 5:45 p.m. estimated flow of 50 gal/min to runoff trench)	

East Trench

Date	Time	Elevation
July 29		1000.18
July 31		1000.18
Aug. 1	3:43 p.m.	999.96
Aug. 1	4:34 p.m.	999.77

Runoff Trench

(measurement taken 200 ft. downstream from cutoff trench)

Date	Time	Elevation
July 29		998.52
July 31		998.73
Aug. 1	3:43 p.m.	998.34
Aug. 1	4:36 p.m.	998.36

Dragline lowering timberset into cutoff trench.

Installed timber set.

Plate 7. Timbering used to support walls of cutoff trench.

The timbering held the walls very well and provided safe working conditions near the pump house. This was an important factor in that the dragline could not excavate the trench up to the well casing under the pump house, and work was done by hand in the trench. A small tunnel was shoveled about four feet under the pump house foundation exposing the well casing. After the casing was exposed, an electric drill was used to bore a semi-circle of one-half inch diameter holes into the casing where the perforated pipe would connect.

A pump was used to assist drainage of the cutoff trench and to facilitate installation of the pipe (Plate 8). The 12 inch perforated pipe sections were placed in the trench and bolted together (Plate 9). When all sections were installed, the trench was backfilled with the material originally taken from the trench.

D. PRODUCTION FROM CUTOFF TRENCH

Production of water from the cutoff trench began in early September, 1961. Demand on the water supply was small during the winter months. On March 14, 1962, Mr. James V. Lloyd, Acting Assistant Regional Director of the National Park Service (Region IV) informed the author by letter that the "pump has lately been operating on a reduced flow." He further stated he believed this was due to clogging of the holes drilled in the well-casing, where the perforated pipe joined the casing. The author concurs with this opinion as the drill holes were only one-half inch in diameter. Furthermore the number of holes (10) did not seem adequate at the time. The difficulty

Plate 8. Pumping from cutoff trench to drain it and facilitate installation of perforated pipe sections, (discharge to runoff trench).

Lamp-hole installed at west end of trench.

Perforated pipe in bottom of tunnel.

Plate 9. Perforated pipe sections installed in cutoff trench.

is to be corrected at a future date, possibly by slotting the well-casing.

E. WELL-POINT INSTALLATION

For observation of water table fluctuations during a pumping test, a series of driven well-points were installed in the Chalone Creek bed, near the collection trench system.

In general, small-diameter driven wells are constructed by driving into the ground sturdy metal drive-points fitted to the lower end of tightly connected sections of pipe (Fig. 5). The well-point consists of perforated pipe with a steel point at its lower end to break through pebbles or thin layers of hard material. The well-point must be driven deeply enough to penetrate a water bearing formation below the water table. They should not exceed 25 to 30 feet in depth if a pitcher pump is to be used for pumping as the lift of the pump would not be great enough to permit successful pumping. Sections of pipe 5 to 6 feet in length are generally used for the pipe string and serve as the casing on the completed wall. Driven wells are usually 1.5 to 2 inches in diameter.

An area about 430 feet upstream from the cutoff trench was chosen as the best location for a pumping test (Fig. 2). It was believed that this location was far enough upstream so that the trench construction would not interfere with the water table at the test area. The area was close enough to the trench, however, so that the trench could be used as a downstream observation point. An additional factor in choosing the test area was vegetation. At the proposed test site, there was for

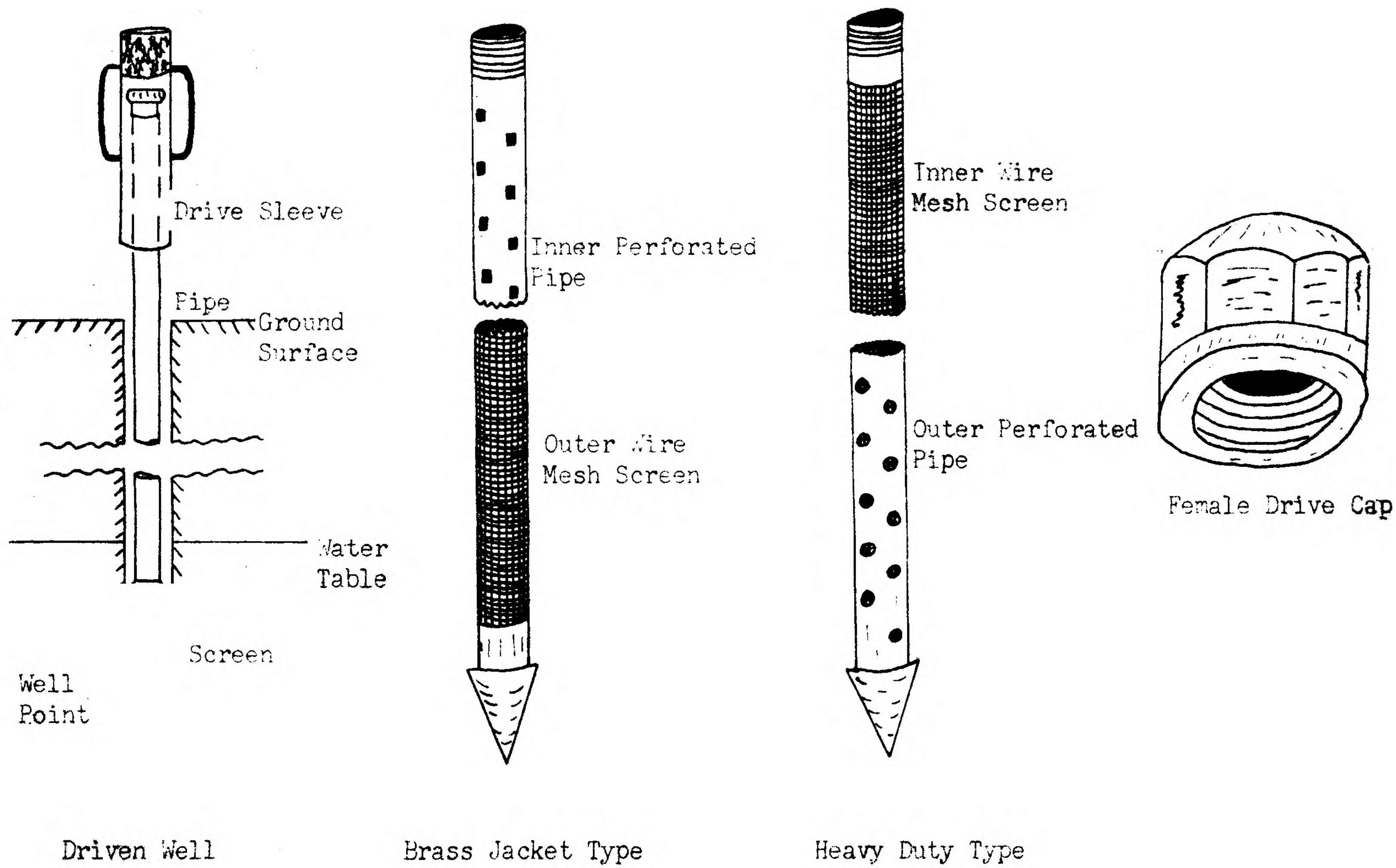


Figure 5. Driven Well Equipment

all practical purposes, bare ground. This would insure that fluctuation of the water table caused by phreatophytes would be a minimum. However, as will be described later, this site had to be abandoned due to large boulders hidden beneath the surface.

The proposed pattern of well-points consisted of a central point from which additional points radiated outward in four directions at distances of 5, 20, and 50 feet (Fig.2). This pattern would give maximum control in determining the shape of a drawdown cone while the central well-point was being pumped.

The well-point used in the tests were of two types. The first was the "brass jacket" type (Fig. 5) which consisted of a steel tip attached to $1\frac{1}{4}$ inch perforated pipe wrapped with wire mesh. The mesh was covered with a perforated brass sheet. The method of driving the point requires the pipe body to be strong, therefore, the number and size of holes in the pipe are limited. These holes, less the obstruction offered by the brass jacket, constitute the effective intake area in this type of well-point. The steel tip has a widened shoulder to push gravel or rocks aside and reduce the danger of ripping or puncturing the jacket. Nevertheless, it is the author's opinion that this type of well-point should be restricted to a fairly uniform sand because the mesh jackets ripped easily in the unsorted deposits of Chalone Creek.

The second type was a heavy duty well-point (Fig. 5). It was designed for use in unsorted material, in that the wire mesh was placed inside the steel tube where it would be protected from ripping by rough stones. The 1.25 inch steel tube had offset circular openings about one inch in diameter. These holes constituted the effective intake area of the well-point. Although designed for installation in a coarse material, this type of well-point was only partially successful because of extremely difficult driving conditions.

The first well-point that was driven was the central well-point at the proposed pumping site. A five-foot section of pipe was connected to the well-point. All connections between pipe sections had to be air tight to get maximum suction when the pump was connected to the pipe. A driving cap (Fig. 5) was placed on the end of the five-foot pipe sections to protect the threads.

A sledge hammer was used to drive the first section of pipe as the driving sleeve had not yet been constructed. This method was unsatisfactory because of the awkwardness in driving the well-point and damage to the pipe sections. Unless the driving cap was hit squarely with the sledge hammer, the cap would deform the threads at the end of the pipe.

Utilization of the driving sleeve greatly decreased the effort needed to drive the pipe. Unfortunately, in spite of the satisfactory design of the drive sleeve, driven well-points proved to be unsuited for the field conditions encountered at Chalone Creek. The driving sleeve fitted over the pipe section and was lifted about two feet above the driving cap before being released. The sleeve weighed 50 pounds and in use was released with a downward velocity; yet the driving rate was extremely low. It took approximately 3,000 blows of the driving sleeve to obtain 14 feet of penetration. Approximately eight to ten hours were required to install one well-point 15 to 16 feet below the ground surface. Many times well-points had to be pulled out of the ground after striking boulders four or five feet below the surface. Five "brass jacket" type points had the outer wire mesh ripped by rocks (Plate 10). Extremely muddy water in the pipe casing indicated rips in the wire mesh. Five or six well-points of both types were bent due to driving them into boulders at depth. Of a total of 18 well-points driven, only 8 were successfully driven below the water table.

After two weeks' work at the proposed pumping site, only one well-point had been successfully installed by the last week in July. Two other points had been completely driven but one had its wire mesh torn, and the other point failed to penetrate the water table. It is believed that this pipe was deflected from the vertical by striking a large boulder. When a weighted line was dropped down the casing of the first point, muddy water and sand coated the weight, indicating a ripped screen. The second well-point was driven one pipe section deeper than the ones that were in water and was only five feet away from the other points but did not reach water. The pipe string must have veered off in a direction almost parallel to the water table.

As a result of the difficulty encountered in installing the well-points at the proposed pumping site, the author hired Mr. E. Henningson, the dragline operator, to attempt to drive the points with his machine.

Vertical pipes indicate abandoned well-points.

Top well-point, slightly bent, is heavy duty type. Bottom well-point is brass-jacket type with ripped screen.

Plate 10. Abandoned pumping site and damaged well-points.

Mr. Henningson had done this type of work before, and had a 1500 pound weight with a four-foot, three-inch diameter pipe extension that would act as a driving sleeve. The weight was hoisted up the boom and released to function as a pile-driver. To minimize the chance of the 1.5 inch pipe string buckling under the blows, a 2 inch diameter pipe, 20 feet long, was placed over the 1.5 inch pipe and into the 3 inch pipe extension from the weight. Mr. Henningson attempted to drive seven well-points and succeeded in reaching water with only two. Of the other five, two were recovered with torn screens and bent points. The remaining three well-points had struck boulders at depth and veered off at some angle other than vertical. They could not be driven successfully downward nor could they be pulled from the ground, so they were abandoned.

Because of the failure to establish the radiating system of well-points at the proposed pumping site, and the short time remaining to complete field tests, the pumping site had to be abandoned. Two well-points on each side of the cutoff trench had previously been established, therefore, a new pumping plan was devised. The well-points were at distances of 3 and 20 feet both upstream and downstream from the cutoff trench giving good control of a drawdown curve if the trench were pumped. At least one well-point had penetrated the water table 429 feet upstream from the trench; therefore, if well-points were installed 200 feet upstream and 400 feet downstream, the combined observation wells should give adequate control for a pumping test. The well-point

400 feet downstream was easily installed as the water table was only one foot below the land surface. The well-point 200 feet upstream was not established because of previously described difficulties in driving the points. Four attempts were made to drive the point, resulting in loss of two points, so the control spot was abandoned. The control for the pumping test therefore consisted of observation wells 430 feet, 20 feet, and 3 feet both upstream and downstream from the cutoff trench.

To conduct the pumping test, the author rented a gasoline pump from the Hollister Machine Works in Hollister, California. A fifteen-foot suction hose was attached to the pump. This was useful in that the pump could rest on the side of the trench and the screened end of the suction hose could be lowered into the trench. A fifty-foot fire hose was attached to the pump to route the discharged water to the east end of the upstream bank of the cutoff trench. The water was pumped at a rate of 60 gallons per minute.

In a pumping test, the measurements taken in the first five minutes of the test are valuable in determining the immediate reaction of the affected water table. Because the measurements for this test were to be taken in six moderately separated observation wells, help was required to take the readings during the first five minutes. Park Service Ranger, Robert Ramstad, and one of his staff, offered their assistance for the initial readings. Two well-points were assigned to each man. Measurements were to be taken approximately one minute apart in each well for the first five minutes. Measurements were made by dropping a weighted steel or cloth tape down the casing of the observation well. The fall of the water level in the trench was also recorded at one minute intervals during the initial five minutes. After the first ten minutes of pumping, measurements were taken at less frequent intervals (Tables 3-9).

The author and his wife recorded measurements of the water table throughout the night. At approximately 7:00 a.m. the next morning, water was noticed seeping from the upstream walls into the east end of the trench. The west end of the trench was checked and no water was observed entering the trench. The 50 feet of fire hose had not been long enough to discharge the water beyond the banks of the creek and eventually water from the fire hose had percolated down to the water table and flowed to the trench. To stop the flow of water into the cutoff trench, the author attempted to drag the fire hose across the cutoff trench to place it into the runoff trench. While doing this, the pump became dislodged and at 8:55 a.m. the pump fell into the cutoff trench (Plate 11). Measurements were continued in the observation wells and in the trench in order to obtain measurements of the recovery rate of the water table (Tables 3-9). Calculations of the permeability and recovery rate are presented in the next chapter.

TABLE III

Water Level Measurements in Well 1

Datum Elevation Top of Well Pipe 1010.58'

Date	Time	Hold	Cut	Depth	Elevation
Aug. 1	3:32 p.m.	16.00	2.50	13.50	997.08
Aug. 1	3:40 p.m.	13.60	2.70	10.90	999.68
Aug. 1	3:56 p.m.	12.00	1.84	10.16	1000.42
Aug. 1	4:18 p.m.	12.00	1.87	10.13	1000.45
Aug. 1	4:44 p.m.	12.00	1.99	10.01	1000.57
Aug. 1	4:53 p.m.	12.00	1.54	10.46	1000.12
Aug. 1	5:05 p.m.	12.00	0.66	11.34	999.24
Aug. 1	5:17 p.m.	12.00	0.67	11.33	999.25
Aug. 1	5:43 p.m.	12.00	0.65	11.35	999.23
Aug. 2	12:35 a.m.	12.00	0.68	11.32	999.26
Aug. 2	8:23 a.m.	12.00	0.68	11.32	999.26
Aug. 2	11:20 a.m.	12.00	0.68	11.32	999.26
Aug. 2	3:50 p.m.	12.00	0.68	11.32	999.26
Aug. 2	6:01 p.m.	12.00	0.64	11.36	999.22
Aug. 3	8:38 a.m.	12.00	0.62	11.38	999.20
Aug. 3	4:33 p.m.	12.00	0.55	11.45	999.13
Aug. 4	8:27 a.m.	12.00	0.50	11.50	999.08
Aug. 4	7:08 p.m.	12.00	0.49	11.51	999.07
Aug. 5	10:12 a.m.	12.00	0.47	11.53	999.05
Aug. 5	7:25 p.m.	12.00	0.45	11.55	999.03
Aug. 6	6:06 p.m.	12.00	0.38	11.62	998.96
Aug. 7	9:41 a.m.	12.00	0.37	11.63	998.95
Aug. 7	5:23 p.m.	12.00	0.34	11.66	998.92
Aug. 9	8:40 a.m.	12.00	0.30	11.70	998.88
Aug. 9	8:20 p.m.	12.00	0.25	11.75	998.83
Aug. 10	4:27 p.m.	12.00	0.23	11.77	998.81
Aug. 10	4:30 p.m.	13.00	1.10	11.90	998.68
Start of pump test 4:56 p.m.		Discharge = 60 gal/min		Datum = 1010.61'	
Aug. 10	4:57 p.m.	13.00	11.25"	12.06	998.55
Aug. 10	4:59 p.m.	13.00	11.25"	12.06	998.55
Aug. 10	5:00 p.m.	13.00	11.25"	12.06	998.55
Aug. 10	5:02 p.m.	13.00	11.00"	12.08	998.53
Aug. 10	5:03 p.m.	13.00	10.75"	12.10	998.51
Aug. 10	5:04 p.m.	13.00	10.75"	12.10	998.51
Aug. 10	5:49 p.m.	12.35	0.35	12.00	998.61

TABLE III (cont.)

Date	Time	Hold	Cut	Depth	Elevation
Aug. 10	6:01 p.m.	12.35	0.34	12.01	998.60
Aug. 10	6:22 p.m.	12.35	0.33	12.02	998.59
Aug. 10	7:53 p.m.	12.35	0.32	12.03	998.58
Aug. 10	8:17 p.m.	12.35	0.31	12.04	998.57
Aug. 10	9:37 p.m.	12.35	0.30	12.05	998.56
Aug. 10	10:14 p.m.	12.35	0.29	12.06	998.55
Aug. 10	11:52 p.m.	12.35	0.28	12.07	998.54
Aug. 11	12:25 a.m.	12.35	0.28	12.07	998.54
Aug. 11	2:40 a.m.	12.35	0.26	12.09	998.52
Aug. 11	3:14 a.m.	12.35	0.26	12.09	998.52
Aug. 11	6:44 a.m.	12.35	0.26	12.09	998.52
Aug. 11	8:10 a.m.	12.35	0.25	12.10	998.51
Aug. 11	9:28 a.m.	12.35	0.25	12.10	998.51
Aug. 11	9:47 a.m.	12.35	0.24	12.11	998.50
Aug. 11	10:38 a.m.	12.35	0.23	12.12	998.49
Aug. 11	10:57 a.m.	12.35	0.24	12.11	998.50
Aug. 11	3:47 p.m.	12.35	0.24	12.11	998.50
Aug. 12	10:27 a.m.	12.35	0.20	12.15	998.46
Aug. 12	4:06 p.m.	12.35	0.19	12.16	998.45
Aug. 13	10:12 a.m.	12.35	0.17	12.18	998.43
Aug. 13	4:12 p.m.	12.35	0.18	12.17	998.44
Aug. 14	9:27 a.m.	12.35	0.18	12.17	998.44
Aug. 14	4:58 p.m.	12.35	0.17	12.18	998.43
Aug. 15	8:12 a.m.	12.35	0.14	12.21	998.40
Aug. 15	9:47 a.m.	12.35	0.14	12.21	998.40
Aug. 15	9:55 a.m.	12.35	0.13	12.22	998.39
Aug. 15	10:00 a.m.	12.35	0.13	12.22	998.39
Aug. 15	10:07 a.m.	12.35	0.13	12.22	998.39
Aug. 15	10:19 a.m.	12.35	0.13	12.22	998.39
Aug. 15	11:11 a.m.	12.35	0.12	12.23	998.38
Aug. 15	1:12 p.m.	12.35	0.11	12.24	998.37
Aug. 15	1:30 a.m.	12.35	0.11	12.24	998.37
Aug. 15	5:14 p.m.	12.35	0.06	12.29	998.32
Aug. 16	8:30 a.m.	12.35	0.03	12.32	998.29

TABLE IV

Water Level Measurements in Well 2

Datum Elevation Top of Well Pipe 1005.66'

Date	Time	Hold	Cut	Depth	Elevation
Aug. 1	3:58 p.m.	7.00	1.50	5.50	1000.16
Aug. 1	4:15 p.m.	7.00	1.50	5.50	1000.16
Aug. 1	4:38 p.m.	7.00	1.50	5.50	1000.16
Aug. 1	4:56 p.m.	7.00	1.49	5.51	1000.15
Aug. 1	5:07 p.m.	7.00	1.46	5.54	1000.12
Aug. 1	5:15 p.m.	7.00	1.46	5.54	1000.12
Aug. 1	5:42 p.m.	7.00	1.44	5.56	1000.10
Aug. 2	12:40 a.m.	7.00	1.44	5.56	1000.10
Aug. 2	8:26 a.m.	7.00	1.44	5.56	1000.10
Aug. 2	11:25 a.m.	7.00	1.41	5.59	1000.07
Aug. 2	3:53 p.m.	7.00	1.41	5.59	1000.07
Aug. 2	6:03 p.m.	7.00	1.41	5.59	1000.07
Aug. 3	8:41 a.m.	7.00	1.40	5.60	1000.06
Aug. 3	4:35 p.m.	7.00	1.39	5.61	1000.05
Aug. 4	8:22 a.m.	7.00	1.37	5.63	1000.03
Aug. 4	7:10 p.m.	7.00	1.37	5.63	1000.03
Aug. 5	10:16 a.m.	7.00	1.36	5.64	1000.02
Aug. 5	7:29 p.m.	7.00	1.33	5.67	999.99
Aug. 6	6:08 p.m.	7.00	1.34	5.66	1000.00
Aug. 7	9:44 a.m.	7.00	1.33	5.67	999.99
Aug. 7	5:25 p.m.	7.00	1.32	5.68	999.98
Aug. 8	8:22 a.m.	7.00	1.30	5.70	999.96
Aug. 8	5:08 p.m.	7.00	1.27	5.73	999.93
Aug. 10	4:32 p.m.	8.00	2.70?	5.30?	1000.36?
Start of pump test 4:56 p.m.		Discharge = 60 gal/min		Datum = 1004.86'	
Aug. 10	4:56 p.m.	8.00	2'7"	5'5"	999.44
Aug. 10	4:58 p.m.	8.00	2'5"	5'7"	999.28
Aug. 10	5:02 p.m.	8.00	2'4"	5'8"	999.19
Aug. 10	5:14 p.m.	8.00	2'2"	5'10"	999.03
Aug. 10	6:03 p.m.	7.35	1.67	5.68	999.18
Aug. 10	6:20 p.m.	7.35	1.67	5.68	999.18
Aug. 10	7:57 p.m.	7.35	1.67	5.68	999.18
Aug. 10	8:15 p.m.	7.35	1.67	5.68	999.18
Aug. 10	9:40 p.m.	7.35	1.67	5.68	999.18
Aug. 10	10:08 p.m.	7.35	1.66	5.69	999.17

TABLE IV (cont.)

Date	Time	Hold	Cut	Depth	Elevation
Aug. 10	11:55 p.m.	7.35	1.66	5.69	999.17
Aug. 11	12:25 a.m.	7.35	1.66	5.69	999.17
Aug. 11	2:41 a.m.	7.35	1.66	5.69	999.17
Aug. 11	3:12 a.m.	7.35	1.65	5.70	999.16
Aug. 11	6:46 a.m.	7.35	1.65	5.70	999.16
Aug. 11	8:12 a.m.	7.35	1.65	5.70	999.16
Aug. 11	9:26 a.m.	7.35	1.65	5.70	999.16
Aug. 11	9:46 a.m.	7.35	1.63	5.72	999.14
Aug. 11	10:36 a.m.	7.35	1.63	5.72	999.14
Aug. 11	10:56 a.m.	7.35	1.63	5.72	999.14
Aug. 11	3:53 p.m.	7.35	1.61	5.74	999.12
Aug. 12	10:31 a.m.	7.35	1.62	5.73	999.13
Aug. 12	4:08 p.m.	7.35	1.60	5.75	999.11
Aug. 13	10:22 a.m.	7.35	1.57	5.78	999.08
Aug. 13	4:14 p.m.	7.35	1.58	5.77	999.09
Aug. 14	9:25 a.m.	7.35	1.58	5.77	999.09
Aug. 14	5:00 p.m.	7.35	1.58	5.77	999.09
Aug. 15	8:10 a.m.	7.35	1.58	5.77	999.09
Aug. 15	9:43 a.m.	7.35	1.55	5.80	999.06
Aug. 15	9:48 a.m.	7.35	1.55	5.80	999.06
Aug. 15	9:54 a.m.	7.35	1.55	5.80	999.06
Aug. 15	9:56 a.m.	7.35	1.55	5.80	999.06
Aug. 15	10:01 a.m.	7.35	1.54	5.81	999.05
Aug. 15	10:20 a.m.	7.35	1.54	5.81	999.05
Aug. 15	11:10 a.m.	7.35	1.53	5.82	999.04
Aug. 15	1:14 p.m.	7.35	1.53	5.82	999.04
Aug. 15	1:32 p.m.	7.35	1.52	5.83	999.03
Aug. 15	5:13 p.m.	7.35	1.52	5.83	999.03
Aug. 16	8:37 a.m.	7.35	1.51	5.84	999.02

TABLE V

Water Level Measurements in Well 3

Datum Elevation Top of Well Pipe 1006.81'

Date	Time	Hold	Cut	Depth	Elevation
Aug. 1	3:35 p.m.	9.00	1.74	7.26	999.55
Aug. 1	4:05 p.m.	9.00	1.78	7.22	999.59
Aug. 1	4:22 p.m.	8.00	0.78	7.22	999.59
Aug. 1	4:58 p.m.	8.00	0.76	7.24	999.57
Aug. 1	5:06 p.m.	8.00	0.77	7.23	999.58
Aug. 1	5:13 p.m.	8.00	0.76	7.24	999.57
Aug. 1	5:19 p.m.	8.00	0.76	7.24	999.57
Aug. 1	5:41 p.m.	8.00	0.76	7.24	999.57
Aug. 2	12:20 a.m.	8.00	0.72	7.28	999.53
Aug. 2	8:14 a.m.	8.00	0.70	7.30	999.51
Aug. 2	11:28 a.m.	8.00	0.70	7.30	999.51
Aug. 2	4:02 p.m.	8.00	0.70	7.30	999.51
Aug. 2	6:10 p.m.	8.00	0.65	7.35	999.46
Aug. 3	8:46 a.m.	8.00	0.70	7.30	999.51
Aug. 3	4:37 p.m.	8.00	0.67	7.33	999.48
Aug. 4	8:15 a.m.	8.00	0.70	7.30	999.51
Aug. 4	7:12 p.m.	8.00	0.70	7.30	999.51
Aug. 5	10:19 a.m.	8.00	0.69	7.31	999.50
Aug. 5	7:32 p.m.	8.00	0.68	7.32	999.49
Aug. 6	6:12 p.m.	8.00	0.68	7.32	999.49
Aug. 7	9:50 a.m.	8.00	0.69	7.31	999.50
Aug. 7	5:28 p.m.	8.00	0.67	7.33	999.48
Aug. 8	8:25 a.m.	8.00	0.67	7.33	999.48
Aug. 8	5:12 p.m.	8.00	0.67	7.33	999.48
Aug. 9	8:43 a.m.	8.00	0.67	7.33	999.48
Aug. 9	8:26 p.m.	8.00	0.63	7.37	999.44
Aug. 10	4:40 p.m.	9.00	1.47	7.55	999.26
Start of pump test 4:56 p.m.		Discharge = 60 gal/min		Datum = 1006.61'	
Aug. 10	4:57 p.m.	9.00	1.25	7.75	999.86
Aug. 10	4:59 p.m.	9.00	1.25	7.75	999.86
Aug. 10	5:04 p.m.	10.00	2.07	7.83	998.78
Aug. 10	5:10 p.m.	9.00	1.25	7.75	998.86
Aug. 10	5:41 p.m.	8.35	0.69	7.66	998.95
Aug. 10	6:06 p.m.	8.35	0.65	7.70	998.91
Aug. 10	6:17 p.m.	8.35	0.64	7.71	998.90

TABLE V (cont.)

Date	Time	Hold	Cut	Depth	Elevation
Aug. 10	8:00 p.m.	8.35	0.64	7.71	998.90
Aug. 10	8:13 p.m.	8.35	0.63	7.72	998.89
Aug. 10	9:45 p.m.	8.35	0.63	7.72	998.89
Aug. 10	10:05 p.m.	8.35	0.61	7.74	998.87
Aug. 10	11:59 p.m.	8.35	0.61	7.74	998.87
Aug. 11	12:20 a.m.	8.35	0.61	7.74	998.87
Aug. 11	2:48 a.m.	8.35	0.61	7.74	998.87
Aug. 11	3:08 a.m.	8.35	0.61	7.74	998.87
Aug. 11	6:51 a.m.	8.35	0.61	7.74	998.87
Aug. 11	8:15 a.m.	8.35	0.60	7.75	998.86
Aug. 11	9:20 a.m.	8.35	0.60	7.75	998.86
Aug. 11	9:43 a.m.	8.35	0.60	7.75	998.86
Aug. 11	10:27 a.m.	8.35	0.60	7.75	998.86
Aug. 11	10:50 a.m.	8.35	0.59	7.76	998.85
Aug. 11	3:55 p.m.	8.35	0.59	7.76	998.85
Aug. 12	10:33 a.m.	8.35	0.57	7.78	998.83
Aug. 12	4:11 p.m.	8.35	0.56	7.79	998.82
Aug. 13	10:17 a.m.	8.35	0.56	7.79	998.82
Aug. 13	4:15 p.m.	8.35	0.56	7.79	998.82
Aug. 14	9:30 a.m.	8.35	0.56	7.79	998.82
Aug. 14	5:01 p.m.	8.35	0.57	7.78	998.83
Aug. 15	8:15 a.m.	8.35	0.56	7.79	998.82
Aug. 15*	9:44 a.m.	8.35	0.55	7.80	998.81
Aug. 15	9:50 a.m.	8.35	0.58	7.80	998.81
Aug. 15	9:56 a.m.	8.35	0.54	7.81	998.80
Aug. 15	10:03 a.m.	8.35	0.54	7.81	998.80
Aug. 15	10:22 a.m.	8.35	0.54	7.81	998.80
Aug. 15	10:32 a.m.	8.35	0.54	7.81	998.80
Aug. 15	10:59 a.m.	8.35	0.54	7.81	998.80
Aug. 15	1:15 p.m.	8.35	0.53	7.82	998.79
Aug. 15	1:36 p.m.	8.35	0.51	7.84	998.77
Aug. 15**	5:11 p.m.	8.35	0.51	7.84	998.77
Aug. 16	8:40 a.m.	8:35	0.50	7.85	998.76

* Pump on
** Pump off

TABLE VI

Water Level Measurements in Well 4

Datum Elevation Top of Well Pipe 1010.96'

Date	Time	Hold	Cut	Depth	Elevation
Aug. 1	3:32 p.m.	16.00	2.50	13.50	997.46
Aug. 1	4:07 p.m.	14.00	0.64	13.36	997.60
Aug. 1	4:24 p.m.	14.00	0.67	13.33	997.63
Aug. 1	4:59 p.m.	14.00	0.73	13.27	997.69
Aug. 1	5:08 p.m.	14.00	0.74	13.26	997.70
Aug. 1	6:20 p.m.	14.00	0.76	13.24	997.72
Aug. 1	6:40 p.m.	14.00	0.78	13.22	997.74
Aug. 2	12:25 a.m.	14.00	1.21	12.79	998.17
Aug. 2	12:26 a.m.	13.00	0.21	12.79	998.17
Aug. 2	8:19 a.m.	14.00	1.50	12.50	998.46
Aug. 2	11:30 a.m.	14.00	1.55	12.45	998.51
Aug. 2	11:32 a.m.	14.00	1.56	12.44	998.52
Aug. 2	4:05 p.m.	14.00	1.65	12.35	998.61
Aug. 2	6:12 p.m.	14.00	1.66	12.34	998.62
Aug. 3	8:47 a.m.	14.00	1.74	12.26	998.70
Aug. 3	4:39 p.m.	14.00	1.77	12.23	998.73
Aug. 4	8:12 a.m.	14.00	1.75	12.25	998.71
Aug. 4	7:14 p.m.	14.00	1.77	12.23	998.73
Aug. 5	10:25 a.m.	14.00	1.75	12.25	998.71
Aug. 5	7:34 p.m.	14.00	1.75	12.25	998.71
Aug. 6	6:13 p.m.	14.00	1.78	12.22	998.74
Aug. 7	9:52 a.m.	14.00	1.78	12.22	998.74
Aug. 7	5:30 p.m.	14.00	1.77	12.23	998.73
Aug. 8	8:27 a.m.	14.00	1.76	12.24	998.72
Aug. 8	5:14 p.m.	14.00	1.76	12.24	998.72
Aug. 9	8:46 a.m.	14.00	1.76	12.24	998.72
Aug. 9	8:29 p.m.	14.00	1.74	12.26	998.70
Aug. 10	4:35 p.m.	14.00	1.72	12.28	998.68
Start of pump test 4:56 p.m.		Discharge = 60 gal/min		Datum = 1010.96'	
Aug. 10	4:57 p.m.	14.00	1.66	12.34	998.62
Aug. 10	4:59 p.m.	14.00	1.66	12.34	998.62
Aug. 10	5:01 p.m.	14.00	1.66	12.34	998.62
Aug. 10	5:45 p.m.	14.00	1.66	12.34	998.62
Aug. 10	6:09 p.m.	14.00	1.66	12.34	998.62
Aug. 10	6:16 p.m.	14.00	1.66	12.34	998.62

TABLE VI (cont.)

Date	Time	Hold	Cut	Depth	Elevation
Aug. 10	8:03 p.m.	14.00	1.66	12.34	998.62
Aug. 10	9:48 p.m.	14.00	1.66	12.34	998.62
Aug. 10	10:01 p.m.	14.00	1.66	12.34	998.62
Aug. 11	12:02 a.m.	14.00	1.66	12.34	998.62
Aug. 11	12:17 a.m.	14.00	1.66	12.34	998.62
Aug. 11	2:50 a.m.	14.00	1.65	12.35	998.61
Aug. 11	3:06 a.m.	14.00	1.65	12.35	998.61
Aug. 11	6:54 a.m.	14.00	1.65	12.35	998.61
Aug. 11	8:20 a.m.	14.00	1.65	12.35	998.61
Aug. 11	9:23 a.m.	14.00	1.65	12.35	998.61
Aug. 11	9:52 a.m.	14.00	1.64	12.36	998.60
Aug. 11	10:28 a.m.	14.00	1.63	12.37	998.59
Aug. 11	10:52 a.m.	14.00	1.63	12.37	998.59
Aug. 11	3:57 p.m.	14.00	1.63	12.37	998.59
Aug. 12	10:35 a.m.	14.00	1.64	12.36	998.60
Aug. 12	4:12 p.m.	14.00	1.62	12.38	998.58
Aug. 14	9:32 a.m.	14.00	1.59	12.41	998.55
Aug. 14	5:05 p.m.	14.00	1.59	12.41	998.55
Aug. 15	8:17 a.m.	14.00	1.54	12.46	998.50
Aug. 15*	9:45 a.m.	14.00	1.53	12.47	998.49
Aug. 15	9:51 a.m.	14.00	1.53	12.47	998.49
Aug. 15	10:04 a.m.	14.00	1.525	12.475	998.485
Aug. 15	10:23 a.m.	14.00	1.52	12.48	998.48
Aug. 15	10:31 a.m.	14.00	1.51	12.49	998.47
Aug. 15	10:50 a.m.	14.00	1.51	12.49	998.47
Aug. 15	1:17 p.m.	14.00	1.515	12.485	998.475
Aug. 15	1:37 p.m.	14.00	1.50	12.50	998.46
Aug. 15**	5:04 p.m.	14.00	1.50	12.50	998.46
Aug. 16	8:43 a.m.	14.00	1.59	12.41	998.55

* Pump on
** Pump off

TABLE VII

Water Level Measurements in Well 5

Datum Elevation Top of Well Pipe 1002.16'

Date	Time	Hold	Cut	Depth	Elevation
Aug. 10*	5:05 p.m.	4.80	0.23	4.57	997.59
Aug. 10	5:09 p.m.	4.80	0.23	4.57	997.59
Aug. 10	5:12 p.m.	4.80	0.23	4.57	997.59
Aug. 10	5:15 p.m.	4.80	0.23	4.57	997.59
Aug. 10	5:18 p.m.	4.80	0.225	4.575	997.585
Aug. 10	6:14 p.m.	4.80	0.225	4.575	997.585
Aug. 10	8:07 p.m.	4.80	0.225	4.575	997.585
Aug. 10	9:55 p.m.	4.80	0.23	4.57	997.59
Aug. 11	12:10 a.m.	4.80	0.23	4.57	997.59
Aug. 11	3:00 a.m.	4.80	0.23	4.57	997.59
Aug. 11	6:59 a.m.	4.80	0.23	4.57	997.59
Aug. 11	8:24 a.m.	4.80	0.23	4.57	997.59
Aug. 11	9:47 a.m.	4.80	0.23	4.57	997.59
Aug. 11	10:31 a.m.	4.80	0.23	4.57	997.59
Aug. 11**	3:59 p.m.	4.80	0.24	4.56	997.60
Aug. 14	9:38 a.m.	4.80	0.41	4.39	997.77
Aug. 14	5:07 p.m.	4.80	0.40	4.40	997.76
Aug. 15	8:21 a.m.	4.80	0.40	4.40	997.76
Aug. 15	10:26 a.m.	4.80	0.40	4.40	997.76
Aug. 15	11:07 a.m.	4.80	0.40	4.40	997.76
Aug. 15	1:18 p.m.	4.80	0.39	4.41	997.75
Aug. 16	8:47 a.m.	4.80	0.395	4.404	997.755

* Pump on
** Pump off

TABLE VIII

Water Level Measurements in Well A

Datum Elevation Top of Well Pipe 1013.63'

Date	Time	Hold	Cut	Depth	Elevation
Aug. 10*	5:14 p.m.	16.00	0.98	15.02	998.61
Aug. 10	5:54 p.m.	15.35	0.15	15.20	998.43
Aug. 10	8:24 p.m.	15.35	0.13	15.22	998.41
Aug. 10	10:22 p.m.	15.35	0.11	15.24	998.39
Aug. 11	12:35 a.m.	15.35	0.09	15.26	998.37
Aug. 11	3:26 a.m.	15.35	0.08	15.27	998.36
Aug. 11	7:09 a.m.	15.35	0.08	15.27	998.36
Aug. 11	8:29 a.m.	15.35	0.08	15.27	998.36
Aug. 11	9:59 a.m.	15.35	0.08	15.27	998.36
Aug. 11	10:43 a.m.	15.35	0.08	15.27	998.36
Aug. 11**	4:12 p.m.	15.35	0.08	15.27	998.36
Aug. 14	9:42 a.m.	15.35	0.11	15.24	998.39
Aug. 15	8:34 a.m.	15.35	0.12	15.23	998.40
Aug. 15	10:14 a.m.	15.35	0.11	15.24	998.39
Aug. 15	11:16 a.m.	15.35	0.11	15.24	998.39
Aug. 15	1:25 p.m.	15.35	0.12	15.23	998.40
Aug. 15	5:18 p.m.	15.35	0.11	15.24	998.39

* Pump on
** Pump off

TABLE IX

Water Level Measurements in Cutoff Trench

Date	Time	Elevation
Aug. 8	8:29 a.m.	998.49
Aug. 8	5:16 p.m.	998.49
Aug. 9	8:48 a.m.	998.49
Aug. 9	8:31 p.m.	998.49
Aug. 10*	4:57 p.m.	998.49
Aug. 10	5:20 p.m.	998.31
Aug. 10	6:07 p.m.	998.25
Aug. 10	6:23 p.m.	998.20
Aug. 10	7:59 p.m.	998.06
Aug. 10	8:14 p.m.	998.06
Aug. 10	9:44 p.m.	998.02
Aug. 10	10:07 p.m.	998.02
Aug. 10	11:40 p.m.	998.02
Aug. 11	2:45 a.m.	998.02
Aug. 11	6:47 a.m.	998.06
Aug. 11	8:05 a.m.	998.06
(Water noticed running into cutoff trench from upstream bank)		
Aug. 11	8:45 a.m.	998.07
Aug. 11**	9:10 a.m.	998.16
Aug. 11	9:25 a.m.	998.23
Aug. 11	9:40 a.m.	998.28
Aug. 11	9:55 a.m.	998.32
Aug. 11	10:26 a.m.	998.42
Aug. 11	10:34 a.m.	998.44
Aug. 11	10:48 a.m.	998.46
Aug. 11	10:54 a.m.	998.47
Aug. 11	3:45 p.m.	998.50
Aug. 12	10:37 a.m.	998.49
Aug. 12	4:14 p.m.	998.48
Aug. 13	10:20 a.m.	998.48
Aug. 13	4:19 p.m.	998.47
Aug. 14	9:25 a.m.	998.48
Aug. 14	5:10 p.m.	998.48
Aug. 15	9:17 a.m.	998.48
(Pumping to drain cutoff trench to facilitate pipe installation - pump on 9:35 a.m.)		
Aug. 15	9:42 a.m.	998.43
Aug. 15	9:49 a.m.	998.40
Aug. 15	9:55 a.m.	998.35
Aug. 15	10:02 a.m.	998.32
Aug. 15	10:08 a.m.	998.29

TABLE IX (cont.)

Date	Time	Elevation
Aug. 15	10:20 a.m.	998.21
Aug. 15	10:35 a.m.	998.12
Aug. 15	10:57 a.m.	998.06
Aug. 15	11:20 a.m.	997.90
Aug. 15	1:10 p.m.	997.50
Aug. 15	1:33 p.m.	997.40
Aug. 15**	2:02 p.m.	997.35
Aug. 15*	2:22 p.m.	997.47
Aug. 15**	5:03 p.m.	997.46
Aug. 15	11:20 p.m.	998.24
Aug. 16	6:56 a.m.	998.48
Aug. 16	8:30 a.m.	997.84

* Pump on
** Pump off

Pump in place.

Result of trying to relocate discharge hole (hose leads to pump).

Plate 11. Pump at beginning and end of pumping test.

III. ANALYSIS OF DATA

A. LABORATORY PERMEABILITY TESTS

A channel sample had been collected in the field for making laboratory tests of the permeability and grain-size distribution of the alluvium. Channel samples are collected by chipping a vertical trench or channel down the face of an outcrop. The diameter and depth of the channel must be at least as great as the dimensions of the largest particle in its path. A container or a cloth is placed at the base of the outcrop to catch all particles that are dislodged. The channel sample described here was collected from the north wall of the cutoff trench, about 150 feet west of the pump house. The channel was about seven inches wide and five inches deep to conform to the dimensions of the largest particle. Rocks larger than 2 inches were measured in the field and discarded because the weight would have been too great to transport back to Missouri.

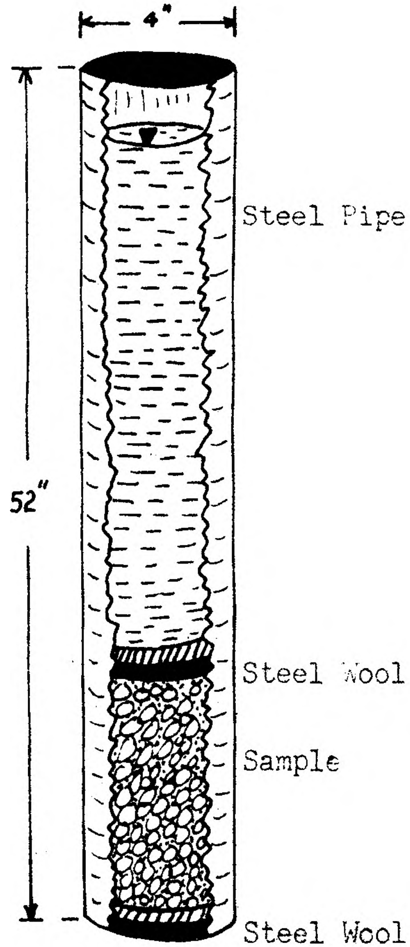
The channel was divided into three horizons on the basis of appearance of the wall of the trench. The upper horizon extended from the top of the trench to a depth of $1\frac{1}{2}$ feet. Below this point, larger pebbles and cobbles appeared to be more abundant. This layer of coarser material extended $3\frac{1}{2}$ feet and constituted the second horizon. The third horizon began where the upper surface of the water level in the trench was first observed and extended one foot below that point.

All three horizons were tested for laboratory coefficients of permeability using falling head type permeameters. The samples from horizons one and three were too small to use in any available permeameters at the Missouri School of Mines; therefore, the author constructed his own (Fig. 6). It consisted of a 4-inch diameter pipe, 52 inches long. Finely spun steel wool was placed below and above the sample to act as porous plates. A wire screen was fastened over the end of the pipe to retain the sample. The fall of the water head was measured by string attached to a cork that floated on the water surface. As the water level decreased, the string was lowered until it reached a predetermined depth. A stop watch was used to measure the time of head fall.

The sample from the second horizon was of sufficient size to be used in a permeameter furnished by the Missouri School of Mines Civil Engineering Department. The diagram (Fig. 6) illustrates the apparatus used. The procedure was to place the sample on a porous plate on the bottom of the permeameter and compact the material to approximate field consolidation. A second porous plate was placed on top of the sample and the top of the permeameter clamped in place. The system was filled with water from the bottom to assure that all air was removed from the system and to avoid disturbing the sample. The graduated reservoir tube was filled with water to the desired level. Water was then allowed to escape from the bottom of the sample cylinder while a stopwatch measured the time of head fall in the graduated tube.

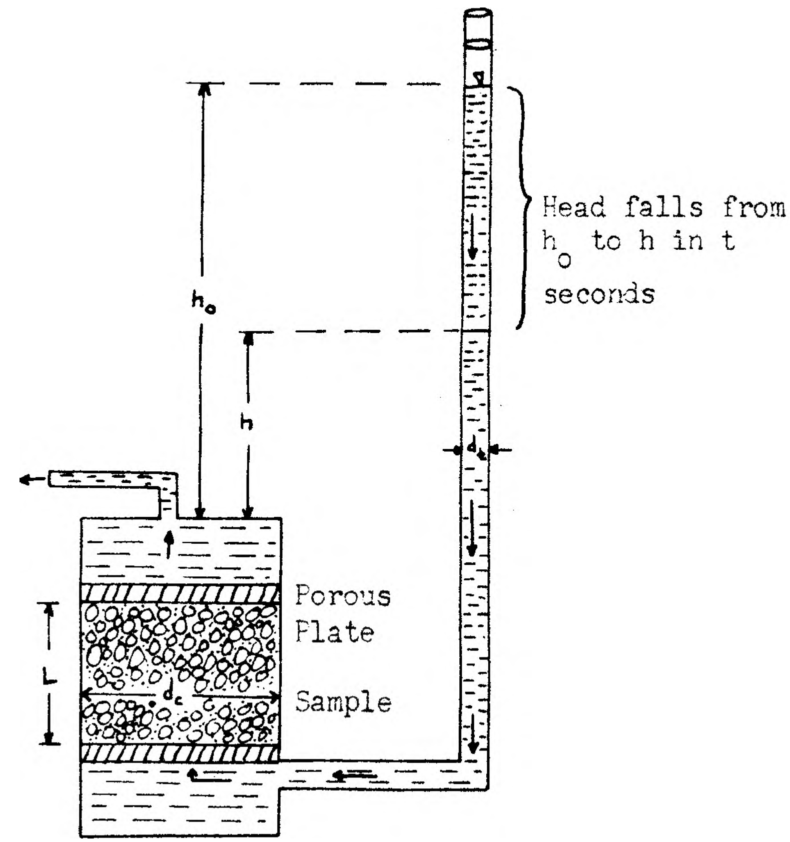
The formula used in calculations of results from both permeameters was Darcy's equation for a falling head test. It is

$$K = \frac{d_t^2 \; L}{d_c^2 \; t} \ln h_o/h \qquad \text{Eq. 3.1}$$



Author's Design

$$K = \frac{d_t^{\,2} L}{d_c^{\,2} t} \ln h_o/h$$



Standard Design

Figure 6. Falling Head Permeameters.

where K is the coefficient of permeability; d_t is the diameter of the graduated tube in feet; d_c is diameter of the sample cylinder in feet; t is the time in seconds; h_o is the height of water before test in feet; and h is the height of water in feet, after t seconds.

The results of the tests are shown in Table 10. Of the four tests run on horizon two, only the last three are considered accurate. The first test that was run did not give accurate results as the permeameter was not air tight and was out of round permitting only one porous retaining plate to fit into the sample cylinder. New gaskets were made to ensure an airtight seal between sections, and the cylinder was made round to accommodate the second porous plate. As a result of the improvements, the last three tests proved acceptable.

The mean coefficient of permeability of the three horizons were calculated to be 5.8×10^2, 1.4×10^2, and 9.3×10^2 for the first, second, and third horizons respectively. The average laboratory permeability for the entire channel sample is 5.5×10^2 gal./day/ft^2. Using the classification given by Todd (1959, P. 53), the alluvium of Chalone Creek would be considered a good aquifer. The permeability tends to increase below the water table as shown by the high permeability of the third horizon. The laboratory coefficient of permeabilities will be compared with the field calculations of permeability later in the chapter.

TABLE X

Permeameter Test Results

$$K = \frac{d_t^{\,2} L}{d_c^{\,2} t} \cdot \ln h_o/h$$

Standard design permeameter (2nd sample horizon)

$d_c = 20.32$ cm $d_t = 5.08$ cm $L = 14.60$ cm

Run 1 = $h_o = 175.5$ cm $h = 115.5$ cm $t = 82.99$ sec.

$K = 103.5$ gal/day/ft^2

Run 2 = $h_o = 175.5$ cm $h = 125.5$ cm $t = 50$ sec. $K = 138.2$ gal/day/ft^2

Run 3 = $h_o = 175.5$ cm $h = 115.5$ cm $t = 54.12$ sec.

$K = 155.5$ gal/day/ft^2

Run 4 = $h_o = 175.5$ cm $h = 115.5$ cm $t = 57.94$ sec.

$K = 138.2$ gal/day/ft^2

Author's design permeameter (1st horizon)

$d_c = 4$ in. $d_t = 4$ in. $L = 10$ in. $h_o = 51$ in. $h = 27$ in.

Run 1 = $t = 369.7$ sec. $K = 712.9$ gal/day/ft^2

Run 2 = $t = 486$ sec. $K = 517$ gal/day/ft^2

Run 3 = $t = 506.6$ sec. $K = 504.2$ gal/day/ft^2

(3rd horizon)

Run 1 = $t = 343.6$ sec. $K = 969.4$ gal/day/ft^2

Run 2 = $t = 364.2$ sec. $K = 970$ gal/day/ft^2

Run 3 = $t = 394.6$ sec. $K = 840.2$ gal/day/ft^2

B. MECHANICAL ANALYSIS OF CHANNEL SAMPLE

The principle purpose of a mechanical analysis of a sediment is to determine the distribution of grain sizes. This is usually accomplished by the use of a series of wire screens. Sorting of the particles in screens is accomplished by the use of a Ro-Tap testing sieve shaker. The sample is placed in the top of a series of 8-inch diameter sieves stacked in order of diminishing size of holes per unit area of mesh. The sieves are placed into a Ro-Tap machine which produces a circular and tapping action. As the sample is agitated, particles fall through the screens until they reach a screen with a hole spacing small enough to stop them. The result is a collection of grains of approximately the same size on each screen.

Three different horizons of the channel sample were tested. The first horizon was from the top of the trench to a depth of 1.5 feet. The second horizon extended 3.5 feet below the first horizon down to the original water level in the trench. The last horizon extended one foot below the original water level. Difficulty in collecting the sample was encountered at this horizon. Although extreme care was exercised in collecting the sample, it is probable that some of the fines were lost because of water action on the sample.

All the samples had to be oven dried before mechanical analysis could be begun, as they had previously undergone laboratory permeability tests. In the drying process, lumps tended to form in the samples. To get a truly representative size analysis, all individual grains had to be in a free state. A buckboard and wooden blocks were used to disaggregate the lumped grains. A wooden block, rather than metal, was used to avoid crushing the grains and reducing their size.

The maximum sample size that is desirable for Ro-Tap analysis is about 1,000 grams. All three horizon samples weighed more than 1,000 grams. Rather than cone and quarter the material, the author chose to hand sieve the coarser material from the sample to reduce the sample size. By doing this, the entire sample could be used for analysis. More effort is required to reduce the sample in this manner, but in the author's opinion the results are probably more accurate than if the sample was reduced by cone-and-quartering.

Four textural concepts are used in this investigation of the alluvium: 1) average or mean grain size, 2) sorting, 3) skewness, and 4) peakedness or kurtosis. These properties may be said to define a grain-size distribution. They are derived from the first four statistical moments of the phi grain-size distribution.

1) Mean Size and Standard Deviation

The mean size and the standard deviation (sorting) define a grain size distribution if the distribution of sizes falls into a symmetrical bell-shaped frequency curve. The mean locates the center of the curve over the size scale and the standard deviation measures the spread or width of the curve.

2) Sorting and Sorting Coefficient

Sorting is a measure of the spread of the distribution. It represents statistically the extent to which the grain sizes spread on either side of the average. The wider the spread, the poorer the sorting. The sorting coefficient is defined (Krumbein and Sloss, 1958, P. 73) as the "square root of the ratio of the larger quartile (the 25 per cent value, Q_1) to the smaller quartile (the 75 per cent value, Q_3)" of a cumulative curve. It is an index of the range of conditions present in the transporting fluid such as range of velocities and degree of turbulence. The equation is:

$$S_o = Q_1/Q_3 \qquad \text{Eq. 3.2}$$

where Q_1 and Q_3 are as previously defined.

3) Skewness

Skewness measures the lack of symmetry of the distribution and is generally marked by a spread of the size range on one side of the mean that is out of proportion to the spread on the other side of the mean. To express skewness in terms of sorting, one end is less well sorted than the rest of the distribution. The distribution is said to be skewed toward the end with the better sorting. Positive skewness in a grain-size distribution indicates a disproportional spread or poorer sorting in the finer sizes. Negative skewness indicates a disproportional spread or poorer sorting in the coarser sizes. The phi quartile skewness is found by the equation:

$$Skq = \frac{Q_3 + Q_1 - 2Md}{2} \qquad \text{Eq. 3.3}$$

where Q_1 and Q_3 are 25% and 75% intercepts of a cumulative curve of the distribution, and Md is the median grain size.

4) Kurtosis

Kurtosis is a measure of the peakedness of the distribution. In terms of sorting, kurtosis measures the relative degree of sorting that the center of a distribution bears to the two ends. A sand with a grain-size distribution which is better sorted in the center than on the two ends has a positive kurtosis. If it is poorer sorted in the center, it has a negative kurtosis. Kurtosis may be found by the formula:

$$Kq = \frac{QD}{(P_{90} - P_{10})} \qquad \text{Eq. 3.4}$$

where QD is the average of the 25 and 75% values on the cumulative curve, and P_{90} and P_{10} are the 90 and 10% values respectively, on the cumulative curve.

Figures 7, 8, and 9 are histograms of horizons 1, 2, and 3 respectively. Figure 10 is a histogram of the entire channel sample, and Figure 11 is a cumulative curve using the values of the entire channel sample. From Figure 11, the mean grain size, sorting coefficient, phi skewness, and phi kurtosis was calculated as 16 mm, 6.7, 0.6, and 0.33 respectively.

These values mean that this particular sample of the Chalone Creek alluvium had an average grain size (by weight) of 16 mm, was overall poorly sorted, with better sorting in the center of the distribution than in the finer sizes.

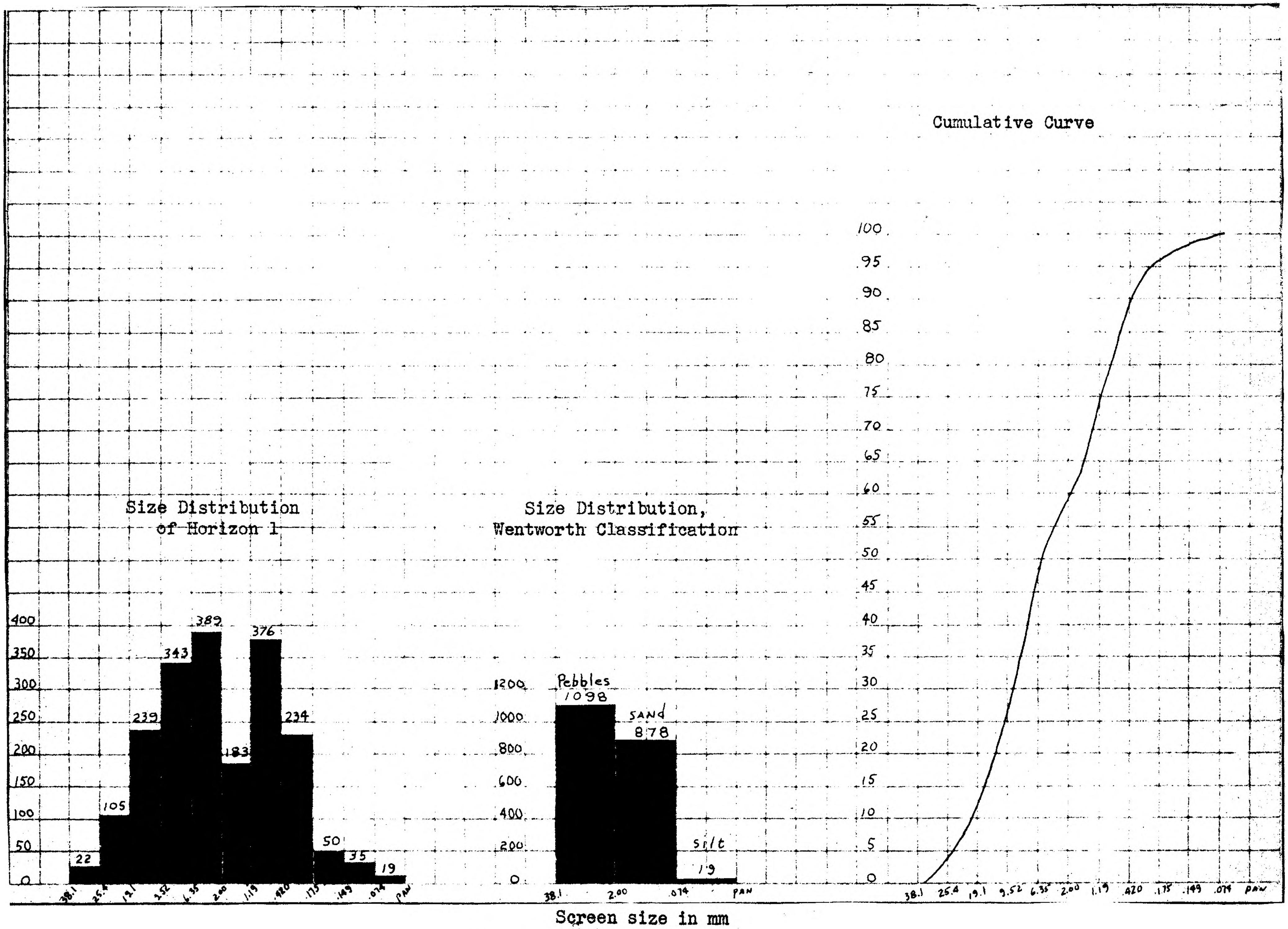


Fig. 7 Histogram of horizon 1

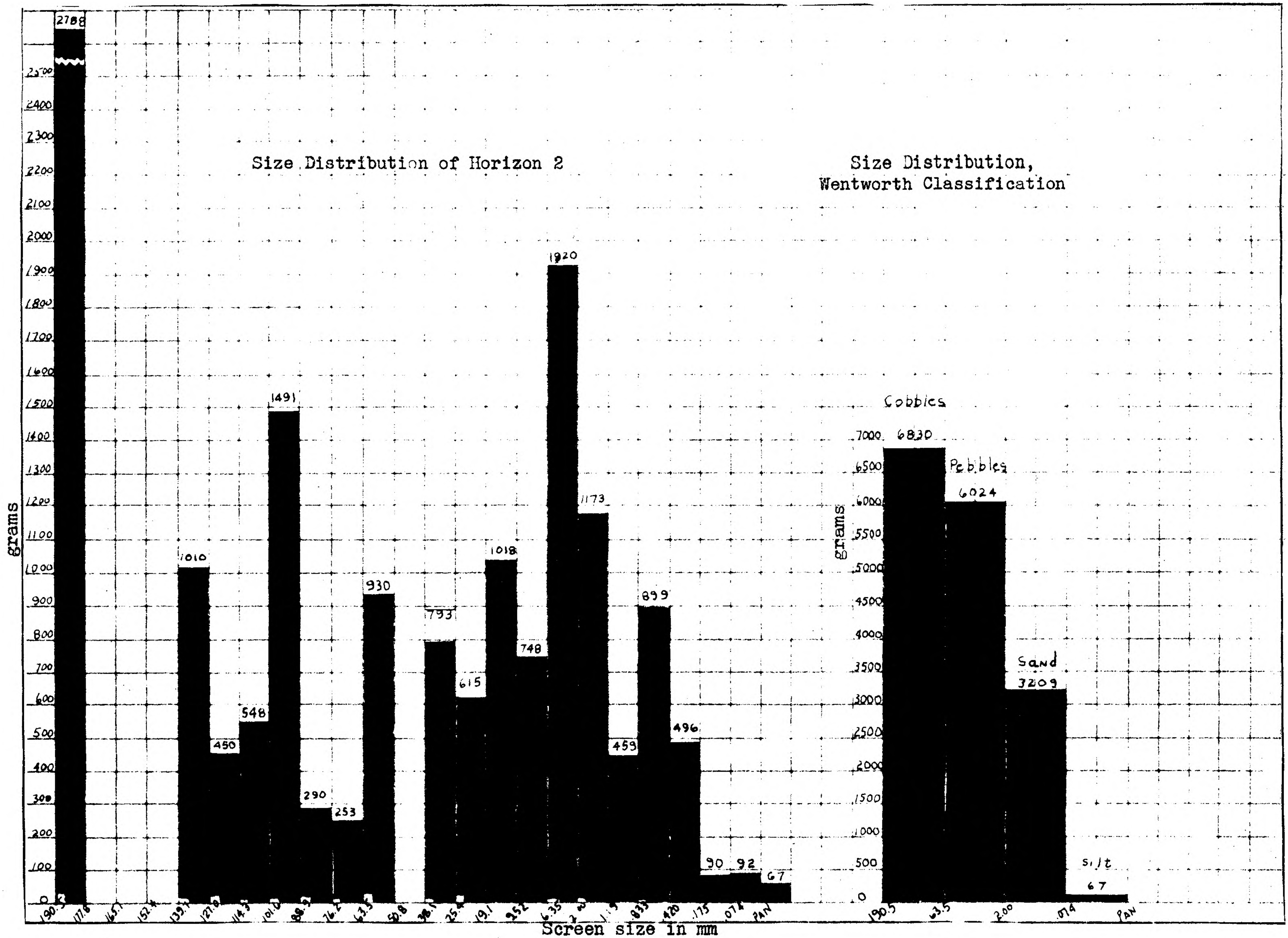


Fig. 8 Histogram of horizon 2

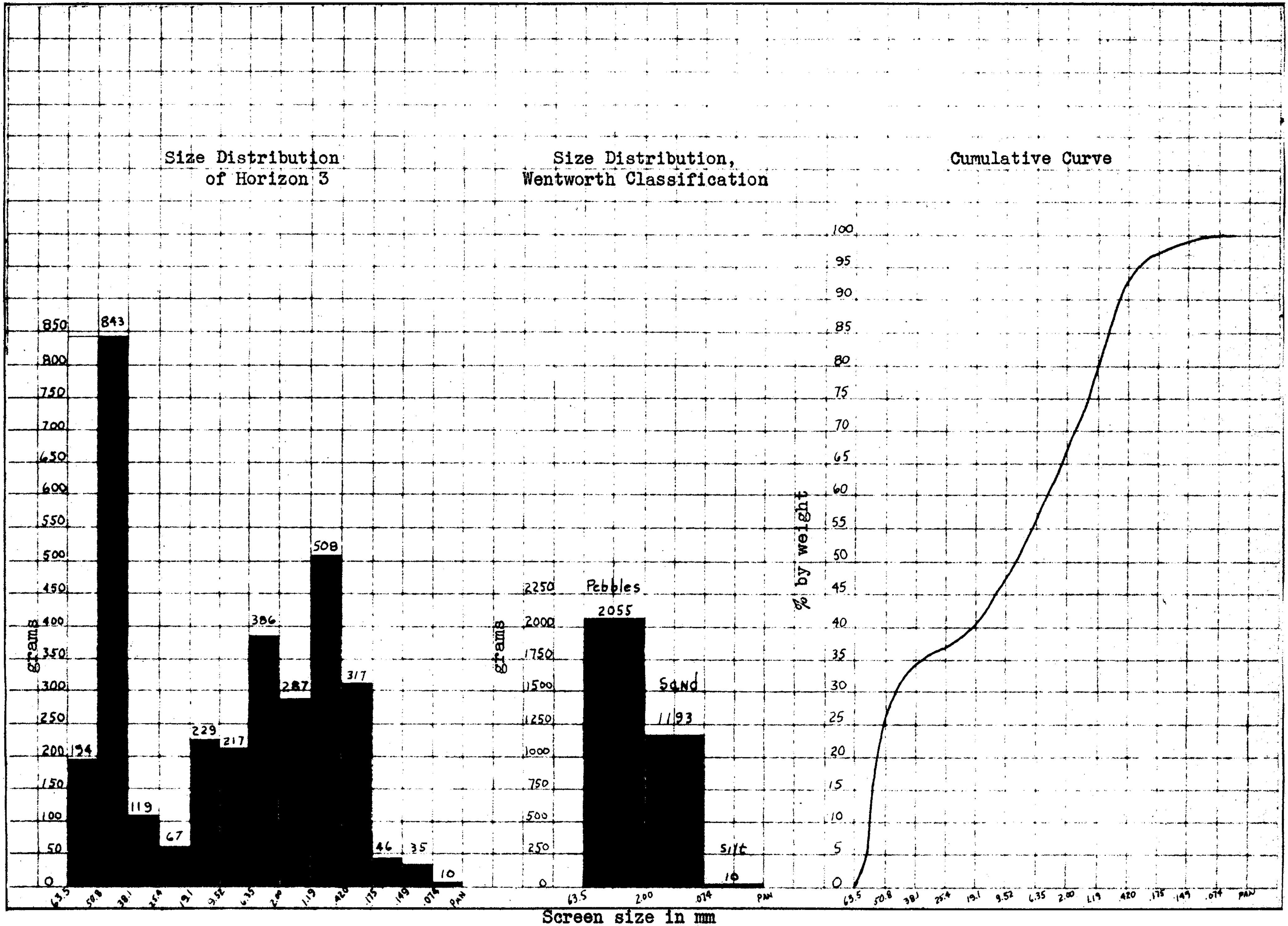


Fig. 9 Histogram of horizon 3

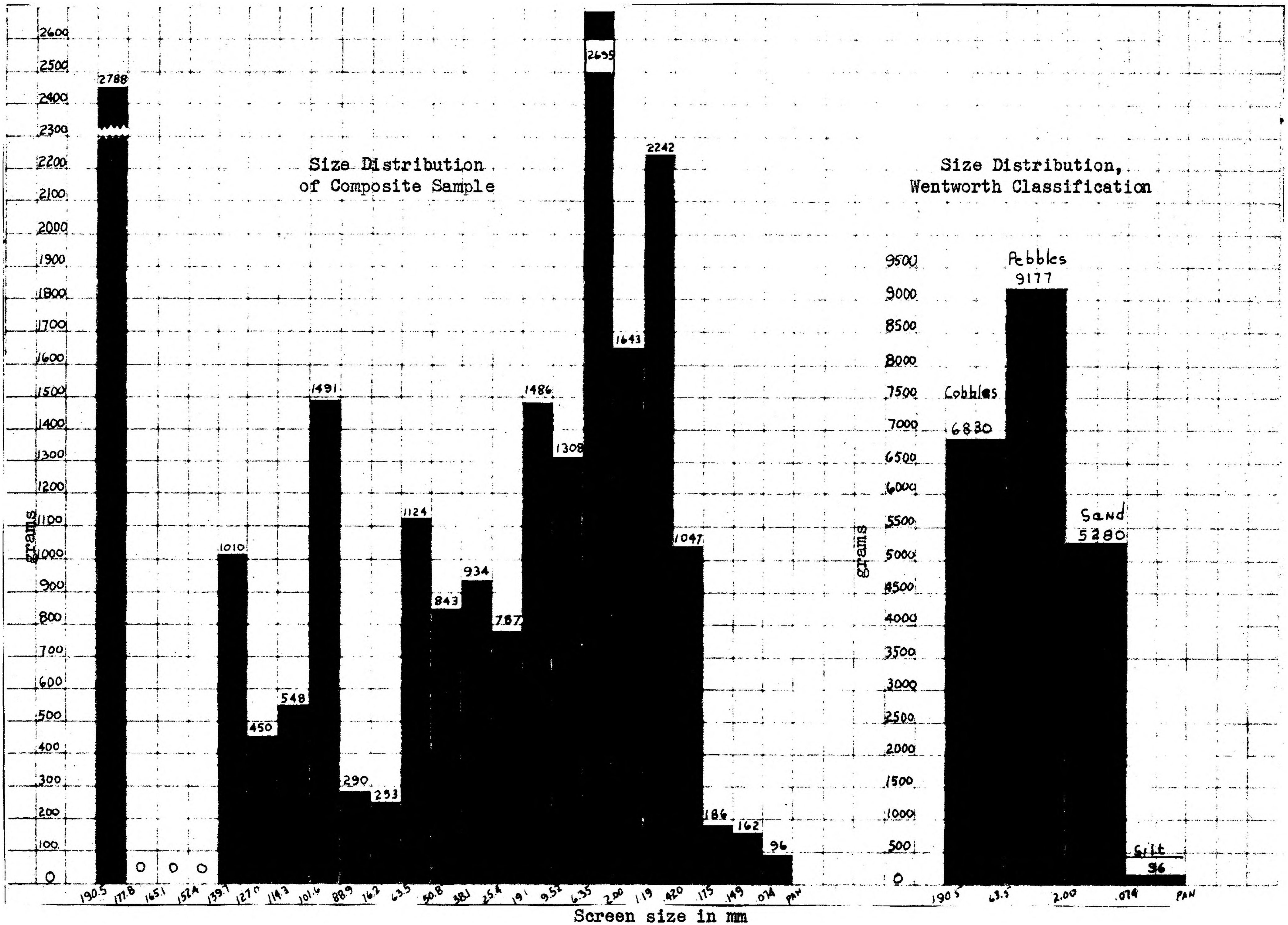


Fig. 10 Histogram of composite sample

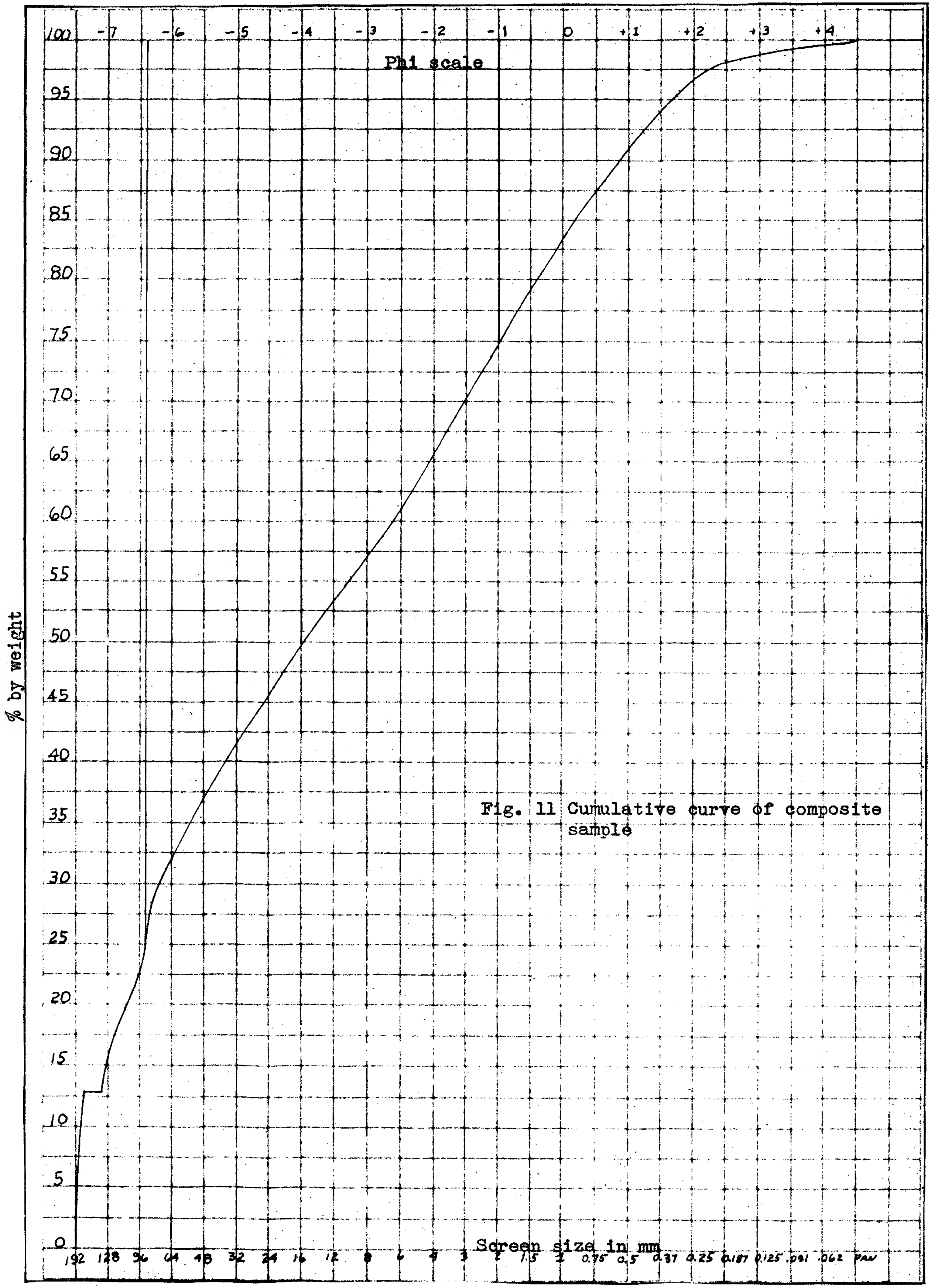


Fig. 11 Cumulative curve of composite sample

C. QUALITY OF WATER

The water sample collected from Willow Spring was analyzed for drinking quality by the United States Geological Survey. Table 11 lists the results of the analysis, along with a chemical analysis that had previously been conducted on a water sample taken from the pump house well at Chalone Creek.

The water sample from Chalone Creek has a greater concentration of all constituents except potassium, sulfate, and fluoride. This is to be expected as the water in Chalone Creek flows through an extensive aquifer containing a high percentage of acidic igneous rocks.

In general, the analysis showed that the water of Willow Spring has a good drinking quality (compared to standards of the U. S. Public Health Service shown in Table 12), low hardness (43 ppm), a pH of 6.9, and 264 parts per million (ppm) of dissolved solids. The analysis also indicated, by trace element comparison, that Willow Spring could be contributing to the groundwater reserves of Chalone Creek. This supports the author's field observations of the discharged flow from Willow Spring.

D. PUMPING TEST CALCULATIONS

Permeability is a measure of the ease with which a fluid will flow through a porous medium. The field coefficient of permeability is defined by Todd (1959), p. 50) as "the flow of water in gallons per day through a cross-section of aquifer 1 ft. thick and 1 mile wide under a hydraulic gradient of 1 ft/1 mile at field temperature." A pumping test

TABLE XI

Chemical Analysis of Chalone Creek and Willow Spring Water Samples

Chalone Creek Pump House Well. Sampled July, 1959.

Chemical	Parts Per Million
Silica (SiO_2)	47.0
Calcium (Ca)	23.0
Magnesium (Mg)	60.0
Sodium (Na)	34.0
Potassium (K)	2.1
Bicarbonate (HCO_3)	126.0
Sulfate (SO_4)	24.0
Chloride (Cl)	51.0
Fluoride (F)	0.4
Nitrate (NO_3)	1.2
Boron (B)	0.1
Dissolved Solids	280.0
Hardness as $CaCO_3$	
Total	83.0
Non-Carbonate	0.0
Specific Conductance (Micromhos at 25°C)	407.0
pH	6.7

Willow Spring. Sampled July, 1961

Silica (SiO_2)	43.0

TABLE XI (cont.)

Chemical	Parts Per Million
Calcium (Ca)	13.0
Magnesium (Mg)	2.6
Sodium (Na)	42.0
Potassium (K)	3.7
Bicarbonate (HCO_3)	82.0
Sulfate (SO_4)	45.0
Chloride (Cl)	18.0
Fluoride (F)	0.4
Nitrate (NO_3)	0.7
Dissolved Solids	264.0
Hardness as $CaCO_3$	
Total	43.0
Non-Carbonate	0.0
Specific Conductance (Micromhos at 25°C)	285.0
pH	6.9

TABLE XII

Comparison of Drinking Water Standards and Samples Analyses

Chemical Characteristics:

Constituent	Upper Limit, ppm	Chalone Creek	Willow Spring
Lead (Pb)	0.100	0.0	0.0
Fluoride (F)	1.500	0.4	0.4
Arsenic (As)	0.050	0.0	0.0
Selenium (Se)	0.050	0.0	0.0
Hexavalent Chromium	0.050	0.0	0.0
Copper (Cu)	3.000	0.0	0.0
Iron (Fe) and Manganese (Mn)	0.300	0.0	0.0
Magnesium (Mg)	125.000	60.0	2.6
Zinc (Zn)	15.000	0.0	0.0
Chloride (Cl)	250.000	51.0	18.0
Sulfate (SO_4)	250.000	24.0	45.0
Phenol	0.001	0.0	0.0
Total solids, desirable	500.000	280.0	264.0
Total solids, permitted	1000.000		

The limits for the first five constituents are mandatory; the others are recommended.

was conducted during August 10 and 11, 1961, to determine the field coefficient of permeability. Tables 3-9 list the measurements of the water levels at that time and Plate 12 (envelope) illustrates the drawdown curves of the six observation wells during pumping and the effect of the pumping on the water level in the trench. The extreme fluctuations of the drawdown curves of wells 1, 2, 3, and 4 during the first 30 minutes, are due to measurement techniques. A cloth tape was used for the initial measurements in these wells. The surface of the tape had a water-proof covering which prevented accurate reading of the waterline on the tape.

Using the above information, calculations of the field coefficient of permeability were arrived at using the Darcy equation for a horizontal trench or drainpipe in the case of a deep unconfined aquifer (Butler, 1957, P. 141). The equation is:

$$q = \frac{P\ (y - y')}{2.3 \log (x/x')} \qquad \text{Eq. 3.5}$$

where q is the discharge in gallons per day per foot length of trench or pipe, P is the permeability in Meinzers, y and y' are points on the drawdown curves in feet, and x and x' are distances in feet from the center of the trench to the observation wells.

The maximum error of measurement of the drawdown values (differences in water depths), distances of observation wells, and discharge are $\pm$ 20%, $\pm$ 7%, and $\pm$ 2% respectively. The value for permeability calculated using equation 3.5 is therefore accurate to $\pm$ 29%. This accuracy may be considered tolerable as it is the author's purpose to calculate the magnitude of permeability rather than a specific value.

The permeability was calculated between three sets of observation wells. The y values were calculated from the difference between the water level in a well and the stabalized pumping water level in the cutoff trench at the same instant. This difference between y values at the same instant supplied the value for y - y'. The rate of pumping was 60 gallons per minute or 86,400 gallons per day. The length of trench was 215 feet, giving a value for q of 402 gallons per day per foot of trench. The distances to the observation wells were known, therefore, the only unknown factor in the equation was the permeability.

The calculated values for the permeability of the Chalone Creek alluvium were 5.1×10^2 gal/day/ft^2 between observation wells 1 and 2; 7.6×10^2 gal/day/ft^2 between wells 3 and 4; and 3.5×10^3 gal/day/ft^2 between wells A and 1. The permeabilities between wells 1 and 2 compared favorably with that between wells 3 and 4, and the laboratory determinations of permeability. The permeability coefficient of 3.5×10^3 did not compare with either the laboratory coefficient or the field permeability coefficients between the other observation wells. Because the difference between 3.5×10^3 and the other permeabilities is much greater that the probable error of equation 3.5, and the fact that the observation well 5, at a distance downstream similar to the upstream distance of well A, did not show a proportional influence from the pumping, it is the author's opinion that this value may be disregarded.

The average value for the field coefficient between observation wells 1 and 2, and wells 3 and 4, is 6.3×10^2 gal/day/ft^2. This compares favorably with the average laboratory coefficient of permeability of 5.5×10^2 gal/day/ft^2. Both values indicate a good aquifer using Todd's (1959, P. 53) classification.

Pumping water from the cutoff trench at a rate of 60 gallons per minute for 10 hours lowered the water level in the trench only 49 hundredths of a foot. In the interval from 8:55 a.m. (when the pump stopped) to 3:45 p.m. on August 11, a total of 6 hours and 50 minutes, the water level in the trench had recovered to its initial elevation. This quick recovery rate also indicates the alluvium has a high effective permeability.

The hydraulic properties of an aquifer are sometimes expressed in terms of transmissibility rather than permeability. The coefficient of transmissibility is defined as the "rate of flow of water in gallons per day through a vertical strip of the aquifer one foot wide and extending the full saturated height under a hydraulic gradient of 100 per cent at a temperature of 60°F" (Wisler and Brater, 1949, P. 206). It may be found by multiplying the coefficient of permeability by the saturated thickness of the aquifer.

The total depth of the aquifer is not known in the central part of Chalone Creek; however, the drill records of the pump house well (Table 1) show the alluvium to be 41 feet thick on the east bank of the creek. Assuming the alluvium to be at least 1.5 times as thick in the central part of the channel, and the saturated zone beginning 10 feet below the surface, the saturated thickness of the aquifer would be 51.5 feet. Multiplying 51.5 by the mean coefficient of permeability value 6.3×10^2, the coefficient of transmissibility in the central part of the channel would be 3.3×10^4 gal/day/ft.

To check the above value of transmissibility, the author used a method derived by C. E. Jacob (Todd, 1959, P. 94) to arrive at a value for transmissibility by plotting the drawdown curve on semi-log paper. Figure 12 illustrates the drawdown curve of observation well 2. After the drawdown has been plotted, a straight line is fitted to the part of the curve having the greatest concentration of data. The difference in the drawdown over one complete log cycle of the fitted line is Δs. To calculate the transmissibility the formula is:

$$T = \frac{264\ Q}{\Delta s} \qquad \text{Eq. 3.6}$$

where T is the coefficient of transmissibility in gal/day/ft, Q is the pumping rate in gal/min, and Δs is the drawdown difference in feet.

The value for the coefficient of transmissibility using this method was 3.2×10^4 gal/day/ft. Dividing this value by the permeability of 6.3×10^2, the saturated thickness of the aquifer is 50.02 ft.

The storage coefficient of the aquifer may also be determined using Figure 12. The storage coefficient is defined by Todd (1959, P. 31) as "the volume of water that an aquifer releases from or takes into storage per unit surface area of aquifer per unit change in the component of head normal to that surface." It may be calculated using the formula:

$$S = \frac{0.3\ T\ t_o}{r^2} \qquad \text{Eq. 3.7}$$

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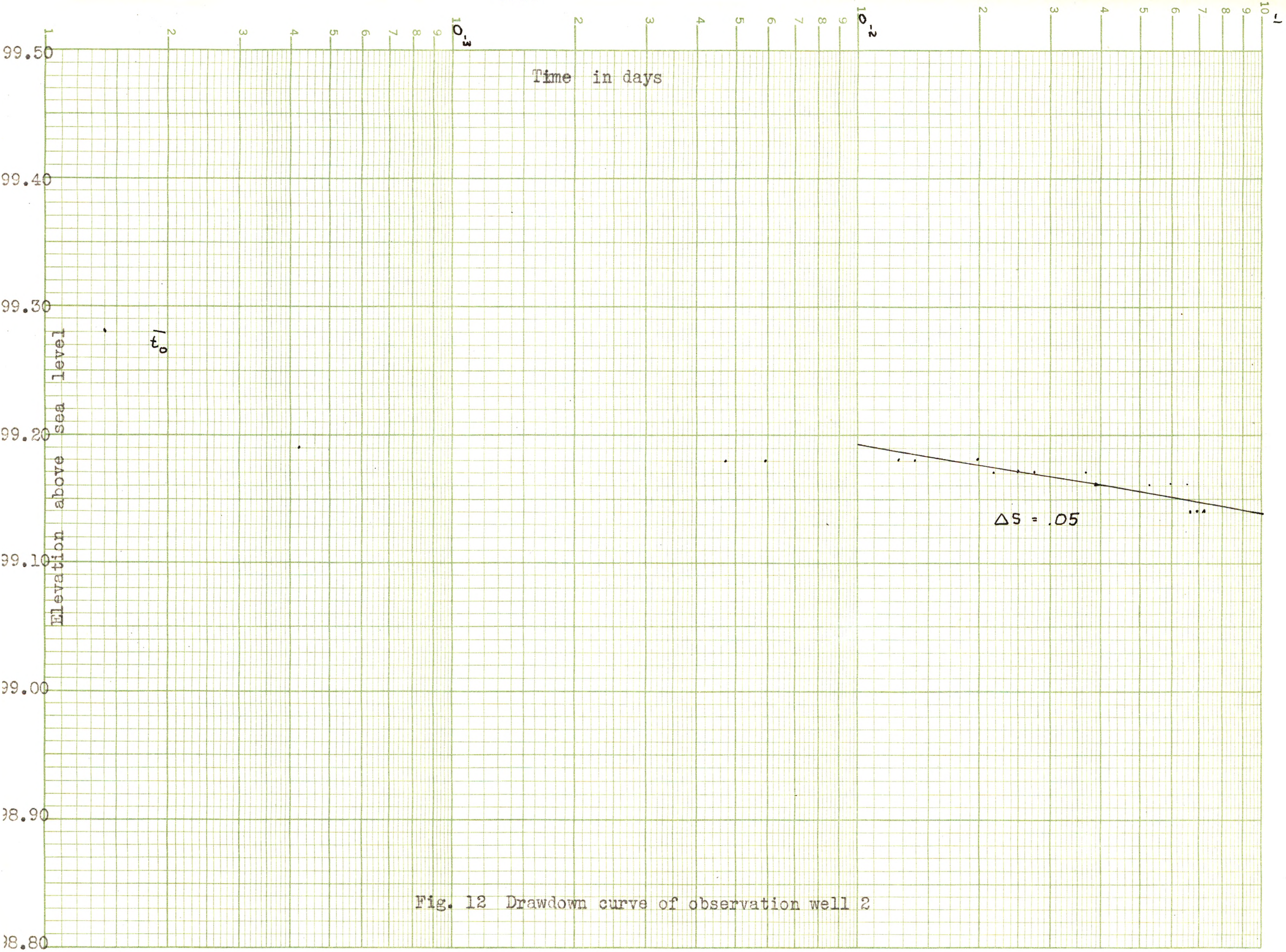

Fig. 12 Drawdown curve of observation well 2

where S is the storage coefficient, T is the transmissibility in gal/day/ft, t_o is the time intercept on the zero-drawdown axis of Figure 12, and r is the distance of the observation well from the pumped well. For observation well 2:

$$S = \frac{(0.3)\ (3.2 \times 10^4)\ (.0019)}{(25)^2} \qquad \text{Eq. 3.8}$$

$$S = .029 \qquad \text{Eq. 3.9}$$

Once the transmissibility and storage coefficients are known, the rate at which the aquifer may be pumped for a period of time without losing the pump suction may be determined by equations:

$$U = \frac{1.87\ R^2 S}{T\ t} \qquad \text{Eq. 4.0}$$

$$Q = \frac{\Delta h\ T}{114.6\ W(U)} \qquad \text{Eq. 4.1}$$

where U is function based on the exponential integral W(U) termed a well function, R is the radius of the pumped well, T is the transmissibility in gal/day/ft, S is the dimensionless storage coefficient, t is the time in days of pumping, Q is the well discharge in gal/min, and Δh is the drawdown difference in feet. Plates 13 and 14 (envelope) are nomograms (Worley, 1961 p. 29) used to simplify the calculations.

A desired time of pumping in days is chosen on Plate 13 (envelope). A line from that column drawn to the column indicating the transmissibility of the aquifer, crosses reference line A at some point (x). Another line, constructed from the column indicating the storage coefficient to the radius of pumping influence, crosses reference line B at some point (y). A line is then constructed from (x) to (y). Where it crosses the column of values for U, a specific value is attained. Plate 14 (envelope) is then used to determine the pumping rate Q. A line is constructed from the transmissibility column to the desired drawdown (four feet was considered the maximum desirable drawdown as the trench bottom was only four feet below the dry season water table) crossing reference line C at some point (z). The value of U is located on the W(U) column and a line drawn from that value to point (z). The value for Q is read where this last line crosses the Q column.

Using the nomograms, it was calculated the aquifer could be pumped at rates of 1.4×10^2, 1.1×10^2, 1.0×10^2, 0.9×10^2, 80, 75, and 70 gal/min for 1, 5, 10, 50, 100, 500, and 1000 days respectively and have a maximum drawdown of four feet. This indicates the water collection trench should more than adequately supplement the water reserves of the Pinnacles National Monument.

IV. CONCLUSIONS

A. SUPPLY AND QUALITY OF WATER

In the Pinnacles National Monument, groundwater is the principal source of water for camping and sanitary purposes. The groundwater is supplemented by surface flow during only a minor portion of the year. Recharge from springs in the area contribute to the groundwater reserves, but the largest of the three known springs in the area, Willow Spring, has a flow of only 30 gallons per minute.

During the summer of 1961, the greatest number of people using the Monument was about 600 on July 4, 1961. The total number of people using the Monument per year probably does not exceed 3000. The National Park Service assumes 11 gallons of water per day per camper. This would create a maximum demand of 3.3×10^4 gallons per year. Calculations show the pumping capacity of water from the collection trench to be about 5.2×10^5 gallons per year. Now that the water collection system has been installed, the quantity of water available for public use appears to be adequate for the facilities of the Monument.

The field coefficient of permeability of the Chalone Creek alluvium was calculated to be 6.3×10^2 gal/day/ft^2. The coefficient of transmissibility was found to be 3.2×10^4 gal/day/ft. Using nomograms, it was calculated that with that permeability and transmissibility, the cutoff trench could be pumped at rates of 1.4×10^2, 1.1×10^2, 1.0×10^2, 0.9×10^2, 80, 75, and 70 gal/min for 1, 5, 10, 50, 100, and 1000 days respectively without lowering the water level enough to break pumping suction. This would more than adequately supply the water demands during the tourist season.

The Chalone Creek water is of good drinking quality, has a pH of 6.7, of medium hardness, and contains 280 parts per million of dissolved solids.

B. TRENCH METHOD

The National Park Service undertook a somewhat unusual method when they decided to install a horizontal trench water collection system rather than drill additional wells to supplement the water supply of the Pinnacles National Monument. This method was chosen because movement of water down Chalone Creek is predominately underflow. The trench system is, in effect, a horizontal well 215 feet long and four feet below the water table to intercept the groundwater movement.

In the author's opinion this method, if properly installed, will supply more water and be more dependable than a drilled well, although it may be more costly. Present reduction in flow is believed to be caused by inadequate connection between the perforated pipe in the trench and the pump house well-casing. This is to be corrected at a future date.

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