Hydrologic Unit was estimated to be about 11,800 acre-feet per season, including the assumed increased import to the Eastern and Piru Subunits in the amount of 600 acre-feet per season.

The safe yield of water supplies available to meet requirements under ultimate conditions of development in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits would reflect the estimated 4,300 acre-foot per season reduction in safe yield of Oxnard Forebay Basin. Accordingly, it was estimated that this ultimate safe supply would be about 18,800 acre-feet per season during drought periods and about 19,900 acre-feet per season during mean periods.

Although it appears that underflow from Santa Paula Basin to Mound Basin would be reduced under estimated ultimate conditions, the magnitude of the probable reduction could not be evaluated with information at hand, and the safe yield of presently developed water supplies of the Mound Subunit was assumed to remain constant ultimately.

Calleguas-Conejo and Malibu Hydrologic Units. It was concluded that the ground water basins in the Calleguas-Conejo and Malibu Hydrologic Units are presently being utilized to the maximum practicable extent, and that any increased utilization thereof would either result in the establishment of overdraft or would increase existing overdrafts. Ultimate supplemental water requirements, therefore, were estimated by comparison of probable ultimate water requirements with safe yields of presently developed water supplies.

TABLE 48

ESTIMATED PROBABLE ULTIMATE MEAN AND DROUGHT PERIOD SEASONAL SUPPLEMENTAL WATER REQUIREMENTS IN HYDROLOGIC UNITS OF VENTURA COUNTY

(In acre-feet)

Hydrologic unit and subunit	Mean					Drought period					
	Water requirement	Safe yield	Net effect of imports and exports on safe water supply	Available safe water supply	Supplemental water requirement	Water requirement	Safe yield	Net effect of modified imports and exports on safe water supply	Net effect of changes in remaining items of water supply and disposal on safe water supply	Available safe water supply	Supplemental water requirement
Ventura											
Upper Ojai	3,700	400	0	400	3,300	4,000	400	0	0	400	3,600
Ojai	5,800	1,500	0	1,500	4,300	6,000	1,500	0	0	1,500	4,500
Upper Ventura River) Lower Ventura River)	24,000	6,000	-100	5,900	18,100	24,200	6,000	-100	0	5,900	18,300
Rincon	5,000	100	100	200	4,800	5,000	100	100	0	200	4,800
Subtotals	38,500	8,000	0	8,000	30,500	39,200	8,000	0	0	8,000	31,200
Santa Clara River											
Eastern	400	0	400	400	0	400	0	400	0	400	0
Piru	11,300	14,700	-3,400	11,300	0	11,400	14,700	-2,900	-400	11,400	0
Fillmore	20,700	16,400	4,300	20,700	0	22,000	16,400	3,300	2,300	22,000	0
Santa Paula	21,200	21,100	100	21,200	0	22,800	21,100	1,300	400	22,800	0
Mound	26,900	8,800	2,700	11,500	15,400	27,700	8,800	3,100	3,000	14,900	12,800
Oxnard Forebay) Oxnard Plain) Pleasant Valley)	146,900	22,400	-2,500	19,900	127,000	160,900	22,400	-3,600	0	18,800	142,100
Subtotals	227,400	83,400	1,600	85,000	142,400	245,200	83,400	1,600	5,300	90,300	154,900
Calleguas-Conejo											
Simi	20,100	6,100	0	6,100	14,000	20,800	6,100	0	500	6,600	14,200
East Las Posas) West Las Posas)	49,500	10,800	1,100	11,900	37,600	56,800	10,800	1,600	2,100	14,500	42,300
Conejo	26,600	2,600	0	2,600	24,000	28,300	2,600	0	600	3,200	25,100
Tierra Rejada) Santa Rosa)	8,100	3,100	0	3,100	5,000	10,200	3,100	0	1,100	4,200	6,000
Subtotals	104,300	22,600	1,100	23,700	80,600	116,100	22,600	1,600	4,300	28,500	87,600
Malibu	13,700	800	0	800	12,900	14,100	800	0	100	900	13,200
TOTALS	383,900	114,800	2,700	117,500	266,400	414,600	114,800	3,200	9,700	127,700	286,900

CHAPTER IV. PLANS FOR WATER SUPPLY DEVELOPMENT

It has been shown that current water resources problems in Ventura County include perennial and progressive lowering of water levels in certain ground water basins of the Calleguas-Conejo Hydrologic Unit, overdraft on ground water supplies in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Basins of the Santa Clara River Hydrologic Unit, resulting in the intrusion of sea water to pumped aquifers during periods of drought, and the utilization of both surface and ground water supplies in the Ventura Hydrologic Unit in excess of estimated safe yields. It has also been shown that there is an estimated mean seasonal requirement for supplemental water in the County of about 73,000 acre-feet at the present time. It has been further shown that elimination of present water resources problems, together with provision for anticipated future growth of the County, will ultimately require the development of supplemental water in the estimated mean seasonal amount of about 266,000 acre-feet.

Sources of supplemental water are available locally in the portion of runoff from watersheds of the Ventura and Santa Clara Rivers that presently wastes to the ocean, which portion would have averaged an estimated 230,000 acre-feet per season over the base period with the present pattern of land use and water supply development. Utilization of this presently wasted water will require the development of equalizing storage capacity either in ground water basins or in surface reservoirs, and construction of facilities to equitably distribute the water so conserved to areas of need. Studies described in this chapter indicate that, because of the erratic nature of the occurrence of runoff in Ventura and Santa Clara Rivers, in excess of 1,500,000 acre-feet of storage capacity would be required to effect complete salvage of this surface waste. Furthermore, because of the relatively high cost of developing surface storage capacity, together with a general paucity of feasible dam and reservoir sites, it is indicated that presently undeveloped ground water storage capacity should be exploited to the maximum practicable extent. It is concluded that under the limitations imposed by economic feasibility, insufficient local water could be conserved and equitably distributed to satisfy present supplemental requirements, and that final solution of the water resources problems of Ventura County must lie in importation of water from outside sources.

As was stated in Chapter I, the Division of Water Resources is presently conducting surveys and studies for the State-Wide Water Resources Investigation, under direction of the State Water Resources Board. This investigation has as its objective the formulation of The California Water Plan for full conservation, control, and utilization of the State's water resources, to meet present and future water needs for all beneficial purposes and uses in all parts of the State, insofar as practicable. Although the investigation is still in progress, it is sufficiently advanced to permit tentative description of certain major features of The California Water Plan which would provide supplemental water to meet the probable ultimate requirements of Ventura County. These projects, which are described in general terms in this chapter under the section entitled "Plans for Importation by Means of Feather River Project", would also provide supplemental water supplies for other water deficient areas of California. In addition, benefits from the projects would include hydroelectric power, flood and salinity control, mining debris storage, and incidental benefits in the interest of recreation and preservation of fish and wildlife.

In general, the major features of The California Water Plan which were mentioned in the preceding paragraph would be large multipurpose projects requiring relatively large capital expenditures. Additional study will be required to estimate final costs and to determine possible means of financing these major projects. Plans presented in this bulletin for the further development of local supplies are those under consideration for current financing, construction, and operation by appropriate local public agencies. The proposed local developments would be such that the works could be integrated into the foregoing major features of The California Water Plan.

Descriptions of various plans considered for the conservation and utilization of local water supplies in Ventura County, and of plans for importing water from available sources outside the County, are presented in this chapter, under section headings designated "Plans for Local Conservation Development", "Plans for Importation by Means of Feather River Project", "Plans for Importation by Means of Metropolitan Water District of Southern California", and "Discussion of Alternative Initial Plans for Water Supply Development". Included therein are estimates of costs of the various plans, estimates of the amounts of supplemental water that would be made available by their adoption and construction, and an evaluation of the plans from the standpoint of economic and financial feasibility.

Design of features of plans presented herein was necessarily of a preliminary nature and primarily for cost estimating purposes. More detailed investigation, which would be required in order to prepare construction plans and specifications, might result in designs differing in detail from those presented in this bulletin. However, it is believed that such changes would not result in significant modifications in estimated costs. The capital costs of dams, reservoirs, diversion works, conduits, pumping plants, and appurtenances included in the considered conservation, conveyance, and distribution systems were estimated from preliminary designs based largely on data from surveys made during the current investigation, both by the Division of Water Resources and other cooperating agencies. Approximate construction quantities were estimated from these preliminary designs. Unit prices of construction items were determined from recent bid data on projects similar to those in consideration, or from manufacturers' cost lists, and are considered representative of prices prevailing in the spring of 1953. Estimates of capital costs included costs of rights of way and construction, plus 10 per cent for engineering and 15 per cent of the construction costs for contingencies, and interest during one-half of the estimated construction period at 4 per cent per annum. Estimates of annual costs included interest on the capital investment at 4 per cent, amortization over a 40-year period on a 4 per cent sinking fund basis, replacement, operation and maintenance costs, and costs of electrical energy required for pumping.

Plans for Local Conservation Development

Consideration was given to enhancement of the presently developed yields of local water supplies, both through construction of equalizing storage capacity in surface reservoirs and in ground water storage. From the results of reconnaissance examination of many possible dam and reservoir sites throughout the County, it was concluded that detailed consideration should be given to ten of the more favorable sites, located in the Ventura and Santa Clara River watersheds. In connection with the studies of further conservation of local water supplies, consideration was given to transfer of surplus water between hydrologic units.

Planned operation of certain ground water basins of the County, either by their greater utilization or by changes in present pumping patterns, or both, would increase their utility by providing additional usable storage capacity for water supply regulation. The Ojai, Piru, Fillmore, Santa Paula, and Oxnard Forebay Basins were studied in this regard. In addition, the Simi and East and West Las Posas Basins were studied from the standpoint of providing regulation for potential imported supplies. Certain legal considerations regarding the vested rights of overlying users must be recognized in such planned operation of ground water storage.

As has been stated, water susceptible to capture by the construction of surface reservoirs in Ventura County, or by further development of ground water storage, is that which would waste to the ocean over a mean period of water supply and climate with the present pattern of land use and water supply development. Estimates were made, therefore, of the portion of this waste occurring during the base period which originated in the Ventura and Santa Clara Rivers and in each of the major tributaries of these rivers. The results of these estimates are presented in Table 49.

It should be pointed out that values presented in Table 49 for the Santa Clara River were derived under the assumption that Oxnard Forebay, Oxnard Plain, and Pleasant Valley Basins would be operated in accordance with their safe yield. Since records of surface outflow in the Santa Clara River during the base period are not available prior to the season of 1947-48, the values presented in Table 49 are based entirely on estimates, evaluated by methods and procedures described in Chapter II, and because of the nature of the studies are only indicative of magnitude. Seasonal amounts of waste to the ocean from the Santa Clara River system originating in each of the indicated major tributaries were determined from analysis of the monthly hydrologic studies presented in Chapter II.

Values presented for waste to the ocean from the Ventura River were determined by correcting measured amounts of runoff at the gaging station near Ventura for impairment by Matilija Reservoir prior to 1948, and for differences in actual historical diversion by the City of Ventura from the estimated present seasonal diversion and pumping requirement of that City. It was assumed that any other differences in the land use pattern and attendant use of water in the remainder of the Ventura River drainage area during the base period from that estimated for the present were negligible and would not affect measured runoff at the foregoing station. It was assumed that the measured runoff of Coyote Creek near Ventura represented the waste to the ocean from that stream during the base period. Estimates were made of the present impairment to the full natural runoff of both Matilija and North Fork of Matilija Creeks, to determine the portion of the previously estimated waste from the entire Ventura River system originating therein. Waste from the remainder of the Ventura River system, shown in Table 49, was then determined as a differential.

TABLE 49

ESTIMATED SEASONAL WASTE TO THE OCEAN FROM
VENTURA AND SANTA CLARA RIVERS DURING BASE PERIOD,
WITH PRESENT PATTERN OF LAND USE AND WATER SUPPLY DEVELOPMENT

(In acre-feet)

Season	Ventura River near Ventura					Santa Clara River at mouth						Total, Ventura and Santa Clara Rivers
	Coyote Creek	North Fork Matilija Creek	Matilija Creek	Miscellaneous runoff	Subtotal, Ventura River	Santa Clara River above County line	Piru Creek	Sespe Creek	Santa Paula Creek	Miscellaneous runoff	Subtotal, Santa Clara River	
1936-37	22,300	12,200	36,400	27,000	97,900	4,100	18,300	91,800	19,000	27,000	160,200	258,100
1937-38	26,600	21,700	73,000	65,300	186,600	36,800	88,000	217,000	38,000	84,700	464,700	651,300
1938-39	3,000	1,500	5,400	7,400	17,300	10,800	16,000	28,500	4,200	6,600	66,100	83,400
1939-40	2,400	700	1,500	4,300	8,900	2,800	5,600	14,100	1,800	2,700	27,000	35,900
1940-41	50,900	30,200	115,900	56,300	253,300	70,900	204,000	370,600	55,200	101,700	802,400	1,055,700
1941-42	3,600	2,600	5,900	7,000	19,100	27,500	20,500	28,700	4,900	5,800	87,400	106,500
1942-43	28,900	14,800	51,100	39,200	134,000	62,400	88,500	163,800	36,100	79,900	430,700	564,700
1943-44	15,200	8,600	29,500	19,200	72,500	67,500	99,500	125,100	18,800	42,500	353,400	425,900
1944-45	7,300	3,200	7,300	10,400	28,200	10,800	16,300	36,200	7,000	7,300	77,600	105,800
1945-46	3,600	3,700	10,300	4,000	21,600	6,400	12,400	37,100	5,900	8,800	70,600	92,200
1946-47	2,800	1,400	3,000	2,600	9,800	7,500	9,700	25,100	3,900	7,300	53,500	63,300
1947-48	0	0	0	0	0	0	0	0	0	0	0	0
1948-49	0	0	0	0	0	0	0	0	0	0	0	0
1949-50	1,500	0	0	900	2,400	0	0	0	0	0	0	2,400
1950-51	0	0	0	0	0	0	0	0	0	0	0	0
Average for base period, 1936-37 through 1950-51	11,200	6,700	22,600	16,300	56,800	20,500	38,600	75,900	13,000	24,900	172,900	229,700
Average for wet period, 1936-37 through 1943-44	19,100	11,500	39,900	28,200	98,700	35,400	67,500	130,000	22,300	43,800	299,000	397,700
Average for drought period, 1944-45 through 1950-51	2,200	1,200	2,900	2,500	8,800	3,500	5,500	14,100	2,400	3,300	28,800	37,600

Examination of Table 49 will show that the estimated mean seasonal waste to the ocean from the Ventura and Santa Clara Rivers under the present pattern of land use and water supply development is about 230,000 acre-feet. It is also indicated that during a drought period the average waste would be about 38,000 acre-feet per season, or about 16-1/2 per cent of the mean. During the wet period, the waste would average about 400,000 acre-feet per season, which amount approaches twice the mean, and is over ten times greater than the average amount for the drought period. Thus, it is evident that the effective conservation of local supplies requires development of carry-over storage capacity to reduce waste to the ocean during wet periods and make it available for beneficial use during periods of drought.

Potential Surface Storage Developments

Investigation of potential surface storage developments in Ventura County included hydrologic studies to ascertain the amounts of supplemental water that could be developed by construction of reservoirs, with various storage capacities at the several sites considered, geologic investigations to determine the suitability of dam sites as to type and height of dam, and estimates of capital and annual costs in order to establish economic relationships between various reservoir storage capacities at a given site and between the several sites. After preliminary reconnaissance, efforts were concentrated on more detailed investigation of the Casitas dam and reservoir site on Coyote Creek, a tributary of the Ventura River; the Ferndale site on Santa Paula Creek; the Cold Spring, Topatopa, Hammel, and Fillmore sites on Sespe Creek; and the Upper Blue Point, Blue Point, Devil Canyon, and Santa Felicia sites on Piru Creek. The locations of these dam and reservoir sites are shown on Plate 25, entitled "Potential Local Water Storage Developments and Conveyance Units for Importation of Water to Ventura County".

Reconnaissance investigation of potential dam and reservoir sites in the Calleguas-Conejo Hydrologic Unit, together with hydrologic studies, indicated that there are few feasible sites, and that present waste of water from Calleguas and Conejo Creeks is insignificant in comparison with present and probable future supplemental water requirements in the unit. Therefore, no further consideration was given to additional surface regulation and conservation of water supplies of these streams.

Estimates were made of monthly runoff during the base period at each of the ten dam sites given detailed consideration. Estimates were also made of mean seasonal waste to the ocean of runoff originating above each of the sites under the present pattern of land use and water supply development. For various selected reservoir storage capacities at each site, monthly operation studies were made, utilizing the aforementioned monthly estimates of runoff for the base period, in order to determine relationships between storage capacity and yield. Monthly values for reservoir evaporation were estimated from available records of evaporation in Los Angeles and Santa Barbara Counties. Net safe seasonal yields that would be developed with construction of the considered reservoir storage capacities were determined by deducting, from yields derived from the operation studies, the amounts of water that would have been put to beneficial use by downstream surface and ground water users without construction of the reservoir.

In operation studies for the proposed Casitas Reservoir, in the Ventura Hydrologic Unit, monthly percentages of seasonal reservoir draft were taken as equal to the estimated average monthly distribution of the seasonal demand for water of the City of Ventura, as shown in the following tabulation:

Month	Per cent of seasonal reservoir draft	Month	Per cent of seasonal reservoir draft
October	9	April	8
November	7	May	9
December	7	June	10
January	6	July	11
February	6	August	11
March	7	September	9

It was demonstrated in Chapters II and III that water problems in the Santa Clara River Hydrologic Unit are manifest in overdraft on ground water supplies on the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, and that in the future ground water overdraft probably will prevail in the Mound Subunit. It was also demonstrated that neither at the present time nor under assumed probable ultimate conditions of development would supplemental water be required in the Piru, Fillmore, or Santa Paula Subunits. Thus, salvage of water presently wasting to the ocean in the Santa Clara River would be for the primary purpose of alleviating ground water overdraft in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, and the measure of reservoir benefit would be the amount of new water that would be made available for beneficial use in these subunits.

The new water that would be developed by construction of reservoirs on tributaries of the Santa Clara River was determined from two operating criteria: (1) operation of the reservoirs on the basis of uniform seasonal releases to the Oxnard Plain, Oxnard Forebay, and Pleasant Valley Subunits, hereinafter termed the "uniform release" method, and (2) operation of the reservoirs on the basis of rapid releases to ground water storage in the Oxnard Forebay Basin, hereinafter termed the "rapid release" method.

Under uniform release operation, it was assumed that water stored in surface reservoirs would be released in equal seasonal amounts at monthly rates corresponding to the estimated average monthly percentages of seasonal demand for water in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits during a drought period. These monthly percentages are presented in the following tabulation:

Month	Per cent of seasonal reservoir draft	Month	Per cent of seasonal reservoir draft
October	10	April	7
November	8	May	10
December	6	June	11
January	5	July	11
February	4	August	12
March	4	September	12

With such uniform release operation, lands on the coastal plain requiring supplemental water would be supplied directly from the reservoirs. Analysis indicated that, because there are from 12 to 27 miles of pervious stream channel between the proposed reservoirs and the Oxnard Forebay Basin, transmission losses would be prohibitive unless the stored water were conveyed to the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits in a conduit.

In the uniform release operation studies, releases were also made from reservoir storage to satisfy prior rights of downstream surface and ground water users. Sufficient water was so released to maintain ground water levels in Piru, Fillmore, and Santa Paula Basins in the fall of 1951 equal to those which would have prevailed without the reservoirs and with the present pattern of land use and water supply development.

It was found that the maximum rate of extraction of water that could be maintained from each of the reservoirs was governed by the period of drought from 1944-45 through 1950-51. The net safe seasonal yield of a reservoir was taken as equal to this determined maximum seasonal extraction, less the average seasonal reduction in water supplies otherwise available for beneficial use in Oxnard Forebay Basin resulting from operation of the reservoir. It was found that a substantial increase in new water would be realized in Oxnard Forebay Basin were the aforementioned releases for prior rights in Piru, Fillmore, and Santa Paula Basins not made, thereby causing ground water levels in these three basins to experience greater lowering than would have occurred during the base period with the present pattern of land use and water supply development.

Under the "rapid release" method of reservoir operation, it was assumed that demands for supplemental water on the coastal plain would be met from Oxnard Forebay Basin, and that the proposed surface storage developments would be largely utilized for temporary detention of flood waters for their subsequent rapid release to this basin. In the operation studies, releases were made from the reservoirs after cessation of heavy winter flow in the Santa Clara River, and when sufficient ground storage capacity was available for percolation of the released water in Oxnard Forebay Basin. It was assumed that the released water would be conveyed to Oxnard Forebay Basin in natural channels of the Santa Clara River and its tributaries. A conduit for this purpose was considered infeasible, because of the prohibitive cost of providing sufficient conduit capacity to accomplish the required rapid reservoir depletion. The rates of release were large enough to minimize percolation and other losses in ground water basins upstream from Oxnard Forebay Basin, but the maximum rates were limited by the amount of flow which could be percolated in Oxnard Forebay Basin. Water was not released from the reservoirs when there was sufficient flow in the Santa Clara River to satisfy percolation demands in Oxnard Forebay Basin or when ground water storage in the basin was filled. It was attempted to deplete the surface reservoir storage each season, so that the maximum storage space would be available for capture of flood waters in the ensuing winter months.

Under this rapid release method of operation, the net safe seasonal yield of the proposed reservoirs was taken as the average seasonal increase in water made available for beneficial use in the Santa Clara River system during the drought period. This new water would be comprised of the net salvage of surface waste during the period, plus water held over from the wet period in surface storage, less reservoir evaporation loss. However, since present and probable future water problems in the Santa Clara River Hydrologic Unit are considered to prevail in the coastal plain only, any of the salvaged water retained in ground water storage upstream from Oxnard Forebay Basin should not be considered as a manifestation of reservoir benefit. As has been stated, the measure of benefit from proposed surface reservoirs is the amount of new water made available for beneficial use during a period of drought in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits. The effect of replenishing the upper ground water basins with salvaged water would be to reduce their utility as natural regulators of Santa Clara River water.

It was found that during the drought period reduction in waste to the ocean effected by the proposed surface reservoirs was about the same under either of the two methods of operation. However, the amount of new water made available at Oxnard Forebay Basin during the drought period was found to be substantially greater when a reservoir was operated by the uniform release method and the released water was conveyed to the coastal plan in a conduit.

Selected combinations of surface reservoirs of varying capacities at certain of the more favorable sites were operated coordinately under each of the two foregoing operational methods. Because of the effects of reservoir operation on downstream ground water supplies, the total yield developed by two reservoirs operated coordinately would be less, in some cases, than the summation of the yields of the two if operated alone.

It should be pointed out that the net safe yield of a reservoir operated under either of the two foregoing criteria could exceed the estimated mean seasonal waste to the ocean of water originating above the reservoir site.

By withholding runoff in surface storage, greater amounts of other waters in the system would have an opportunity to percolate to ground water storage than is presently the case. The demand on stored water to maintain ground water levels that would have prevailed without the reservoir construction would be accordingly reduced.

As has been stated, net safe yields of potential surface reservoirs in Ventura County were determined from water supply data for the base period from 1936-37 through 1950-51, and the magnitudes of yields so determined were governed by the critical drought period from 1944-45 through 1950-51. It is known that the period governing safe draft that can be maintained indefinitely from a reservoir is dependent on relationships between its storage capacity and the magnitude and regimen of flow of the particular stream. For any given stream, the critical period of water supply may change for different considered reservoir storage capacities. In general, for reservoir storage capacities considered in this bulletin, the drought period from 1944-45 through 1950-51 did govern the magnitude of safe yield. However, exceptions occurred in several of the larger reservoir capacities studied, particularly as storage capacities approached magnitudes required to completely control a given stream over the base period. For such larger capacities, it was estimated that the critical water supply periods that occurred either from 1922-23 through 1935-36 or from 1917-18 through 1935-36 would usually govern safe yields. In such instances, appropriate qualification of the values of safe yield presented herein has been made. It should be emphasized again, however, that reliable records of surface runoff in Ventura County are not available for seasons prior to 1927-28, and that runoff estimates are necessarily based either on rainfall-runoff relationships or correlations with runoff of streams in Santa Barbara or Los Angeles Counties. Furthermore, at only a few of the dam sites under consideration are there stream gaging stations of such proximity thereto, that reliable estimates of runoff during the base period could be made.

For each of the proposed surface reservoirs in Ventura County, consideration was given to future losses of effective storage capacity through sedimentation. The problem of reservoir sedimentation is of great significance in the County, and in comparable areas of southern California, because of the large bed loads carried by flood waters. Over a long period of years, the effective capacity of any reservoir will be destroyed through accumulation of sediment. The elapsed time prior to such complete loss of reservoir utility is dependent upon storage capacity of the development, and upon characteristics of the particular drainage area under consideration, such as soil type, vegetative cover, and nature and occurrence of runoff from the watershed. Brush and forest fires in a watershed reduce resistance to erosion and tend to increase sedimentation problems. Values for average seasonal rates of sedimentation utilized in this bulletin were obtained from reports by Harold Conkling, Consulting Engineer, entitled "Demand on Casitas Reservoir and Safe Yield", dated April, 1950, and "Development of a Supplemental Water Supply for Zone 2, Ventura County Flood Control District", dated September, 1949. The estimates in Mr. Conkling's reports were obtained from "Flood Frequencies and Sedimentation from Forest Watersheds", by Henry W. Anderson, California Forest and Range Experiment Station, United States Forest Service, Berkeley, California, dated February, 1949. For the proposed Casitas Reservoir on Coyote Creek, an average unit seasonal sediment production of 2.3 acre-feet per square mile of drainage area above the site was estimated. For watersheds of Sespe and Piru Creek, estimated seasonal values of 2.4 acre-feet per square mile and 1.6 acre-feet per square mile, respectively, were employed. The average unit seasonal sediment production of Santa Paula Creek was taken equal to that of Sespe Creek. Yields for all reservoirs considered in this bulletin were estimated on the basis of effective capacities that would remain after 20 years of operation. The constructed capacity of proposed reservoirs is hereinafter referred to as the "gross reservoir storage capacity", and the effective capacity remaining after 20 years of operation is referred to as the "net reservoir storage capacity".

Spillways for proposed dams and reservoirs in Ventura County were designed to pass the probable peak discharge from a flood having a frequency of once in one thousand years. Because of the preliminary nature of the designs, no consideration was given to the effect of surcharge storage in the reservoirs on reducing estimated peak flows over the spillways.

Because of the erratic nature of occurrence of runoff in streams of Ventura County, there might be a considerable lapse of time subsequent to construction of reservoirs before they would be filled and in effective operation. A large reservoir constructed at the beginning of the critical water supply period from 1922-23 through 1935-36 might have required as long as 20 years to fill. On the other hand, a reservoir constructed immediately prior to the wet period from 1936-37 through 1943-44 would have filled in a considerably shorter length of time. As has been stated, for over three years subsequent to its construction, in 1948, Matilija Reservoir was virtually dry. Runoff occurring during the one month of January, 1952, filled this reservoir. As an aid in selection of desirable reservoir capacities to be constructed at certain sites appearing favorable in other respects, operation studies were made for the period from 1894-95 through 1950-51, for which period only rough estimates of seasonal runoff in Ventura County streams are available, to determine the probable average number of years that would elapse prior to filling the reservoirs with various capacities.

The following sections describe in some detail the results of investigation of each of the ten considered dam and reservoir sites in Ventura County. Certain of these results are depicted graphically on Plate 35, entitled "Relationship between Storage Capacity of Reservoirs and Capital Cost"; Plate 36, entitled "Relationship between Storage Capacity of Reservoirs and Net Safe

Seasonal Yield "; Plate 37, entitled "Relationship between Net Safe Seasonal Yield of Reservoirs and Annual Unit Cost; and Plate 38, entitled "Probable Time Required to Fill Reservoirs after Construction." Yields for reservoirs on tributaries of the Santa Clara River utilized in preparing Plates 36 and 37 were those determined from the uniform release method of operation, with releases for maintenance of ground water levels in Piru, Fillmore and Santa Paula Basins. Costs employed in preparing Plate 37, however, do not include the cost of a conduit that would be necessary to realize the indicated yields, and are, therefore, indicative of the annual cost per acre-foot of new water at the reservoirs.

Casitas Dam and Reservoir. The Casitas dam site is located on Coyote Creek, about 2.5 miles above its confluence with the Ventura River and about 0.7 mile downstream from State Highway 150. A county road, the Casitas Pass Road, passes along the right abutment of the site, and joins State Highway 150 about one mile upstream. Both the dam site and reservoir area are within a former land grant, designated Rancho Santa Ana. The stream bed elevation at the dam site is about 325 feet, U.S.G.S. datum. Construction of a dam and reservoir at this site would permit conservation of flood waters of Coyote Creek, and of the Ventura River diverted to the reservoir, and would be for the primary purpose of providing supplemental water to the Ventura Hydrologic Unit. Consideration was also given to conveyance of water from the reservoir to the Santa Clara River Hydrologic Unit.

The drainage area of Coyote Creek above the Casitas dam site comprises about 36 square miles, and produced an estimated average seasonal runoff of about 10,100 acre-feet during the base period. Under the plans considered, inflow to a reservoir at the Casitas site would be augmented by diversion of surplus waters from the Ventura River. Seasonal runoff at the considered diversion site would have averaged an estimated 33,500 acre-feet during the base period had Matilija Reservoir been in operation, from a drainage area of about 75 square miles.

Other Dam and Reservoir Sites Considered. Reconnaissance examinations were made of three other dam and reservoir sites in the Ventura River drainage area during the course of the investigation. The dam sites were located, respectively, on the Ventura River a short distance below the confluence of North Fork of Matilija Creek and Matilija Creek, designated the Nordhoff site; on the main thread of the Ventura River upstream from Foster Park and below the mouth of San Antonio Creek, designated the Arnaz site; and on San Antonio Creek immediately above its confluence with Ventura River, designated the San Antonio site. Although it was indicated that the two sites on the main thread of the river had certain advantages over the Casitas site, in that direct capture of the greatest portion of runoff of the Ventura River could be effected, probable costs of construction of dams at these sites were considered prohibitive. A dam at the Arnaz site would necessitate relocation of a branch of the Southern Pacific Railroad and a portion of U. S. Highway 399, and in addition would require the acquisition of several hundred acres of suburban residences in the reservoir area. Construction of a dam at the Nordhoff site would also necessitate relocation of U. S. Highway 399, which in this vicinity would be an expensive undertaking, and would inundate the existing Matilija Dam. The San Antonio Creek site was given no further consideration because of the relatively minor runoff in San Antonio Creek, and because it did not compare favorably with the Casitas site for offstream storage of Ventura River water due to the limited storage capacity available.

Areas and Capacities of Reservoir. The Casitas reservoir area was mapped up to an elevation of 550 feet in March, 1951, by the Ventura County Flood Control District, at a scale of 1 inch equals 400 feet, with a 25-foot contour interval. The District also mapped the dam site in 1949, at a scale of 1 inch equals 100 feet, with a 5-foot contour interval. Storage capacities of Casitas Reservoir at various stages of water surface elevation are given in Table 50.

TABLE 50

AREAS AND CAPACITIES OF CASITAS RESERVOIR

Depth of water at dam, in feet	Water surface elevation, U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	325	0	0
5	330	8	20
15	340	25	185
25	350	48	550
35	360	105	1,300
45	370	170	2,700
55	380	235	4,700
65	390	290	7,300
75	400	350	10,500
85	410	410	14,400
95	420	480	18,800
105	430	570	24,000
115	440	670	30,200
125	450	730	37,200
135	460	870	45,300
145	470	960	54,400
155	480	1,070	64,600
165	490	1,190	75,900
175	500	1,330	88,500
178	503	1,380	92,000
185	510	1,490	102,600
187	512	1,530	105,000
195	520	1,650	118,300
202	527	1,790	130,000
205	530	1,830	135,800
215	540	1,990	154,900
215.5	540.5	2,000	156,000
225	550	2,140	175,600

Geology of Dam Site. Geologic investigation indicated that the Casitas site is suitable for construction of an earthfill dam up to a maximum height of about 235 feet, which probably is about the upper limit from the topographic standpoint. The geology of the site was studied by George D. Louderback in 1948, by the D. R. Warren Company in 1946, and by J. B. Lippincott in 1934. During the course of the investigation, geologists from the Division of Water Resources examined the site and reviewed the prior geologic reports. Thirteen borings were made at the dam site in 1948, under the direction of Dr. Louderback, totaling about

1,310 feet of depth, of which about 924 feet comprised core borings. In addition, three tunnels totaling about 655 feet in length were driven into the right abutment. In 1946, the D. R. Warren Company drilled ten holes. During the Lippincott investigation, in 1934, six holes were drilled totaling about 366 feet in depth. In addition, exploratory trenching was done on both abutments.

The Casitas dam site is formed by a slight topographic constriction of the valley floor, where Coyote Creek has cut through resistant basal sandstone layers of Vaqueros age. These harder beds are inter-stratified with thicker, softer, shaly beds. The Vaqueros formation overlies a reddish, sandy, Sespe shale containing veinlets of gypsum, and in turn is overlain by grayish colored Rincon shale. All of these formations dip from 20 to 30 degrees upstream, which is a favorable attitude, and strike generally across the channel parallel to the proposed axis. The beds are slightly fractured, and although minor faults of slight displacement occur, they are generally sound and in reasonably good condition. The easterly extension of the more resistant Vaqueros beds, along the strike line, forms a relatively narrow ridge comprising the left abutment, with the downstream or southerly slope thereof being quite steep because of comparatively recent undercutting by Coyote Creek. Appreciably wide flat terraces are present on either side of the channel in the vicinity of the axis. The westerly or right abutment does not have so pronounced a ridge, although its upper portion shows the resistant Vaqueros beds, forming a small but sharply defined cliff above their contact in a ravine with the softer underlying Sespe formation.

Both abutments are covered with a moderately heavy soil blanket estimated to be from 5 to 15 feet in thickness. A small slide or slump exists between elevations of 325 and 425 feet on the right abutment upstream from the axis. Here the more brittle Vaqueros sandstone has slumped slightly out of position, possibly due to yielding of the less competent underlying beds. The slide comprises about 50,000 cubic yards of material. Most of this material exposed by the aforementioned tunnels appears to be reasonably firm and stable, although a final decision as to its suitability for foundation necessarily would have to await final stripping. From examination of the material exposed in cores, tunnels, and on the surface, it does not appear that the foundation area would accept much grout, unless large unknown seams or cavities are encountered during stripping operations. As the ridge forming the left abutment is rather thin, leakage from the reservoir could result unless the upstream slope was blanketed with impervious material.

Major faulting at the site was not observed or indicated by exploration work. However, it is apparent that numerous small faults and possibly shear zones exist in the foundation area. Others may come to light with additional exploratory work, particularly in the channel section. While a small amount of shaping may be necessary in the developed foundation, no serious defect is believed to exist. Since the Casitas dam site lies in a moderately seismically active area, proper consideration of this factor should be given in the design of any structure at this site.

Operation and Yield of Reservoir. As was stated, consideration was given to utilization of a reservoir at the Casitas site not only for impounding runoff in Coyote Creek but also for offstream storage of surplus waters diverted from the Ventura River. Diversion sites studied in this regard were located so as to enable capture both of flow in the North Fork of Matilija Creek and spill from Matilija Reservoir. Table 51 presents seasonal base period values of estimated runoff of Coyote Creek at the Casitas dam site, measured runoff of the North Fork of Matilija Creek, and estimated spill from Matilija Reservoir operated to give a gross seasonal yield of 3,700 acre-feet. Runoff of Coyote Creek at the Casitas dam site was estimated to be 90 per cent of measured runoff at the U.S.G.S. stream gaging station on Coyote Creek near Ventura.

TABLE 51

SEASONAL RUNOFF OF
COYOTE CREEK AT CASITAS DAM SITE
AND NORTH FORK OF MATILIJA CREEK,
AND SEASONAL SPILL FROM MATILIJA RESERVOIR,
DURING BASE PERIOD

(In acre-feet)

Season	Coyote Creek at Casitas dam site*	North Fork of Matilija Creek near Matilija	Spill from Matilija Reservoir*
1936-37	20,060	13,590	40,430
1937-38	23,900	22,920	77,230
1938-39	2,700	2,740	9,600
1939-40	2,190	2,250	5,300
1940-41	45,800	31,290	120,260
1941-42	3,270	4,300	9,630
1942-43	26,020	15,970	55,290
1943-44	13,670	9,870	33,650
1944-45	6,550	4,820	11,060
1945-46	3,240	5,150	14,270
1946-47	2,550	3,000	6,260
1947-48	50	760	0
1948-49	130	1,150	0
1949-50	1,320	1,630	0
1950-51	90	590	0
TOTALS	151,530	120,030	382,980
AVERAGES	10,100	8,000	25,530

* Estimated.

As a first step in the analysis of Casitas reservoir, estimates were made of the amounts of water susceptible to diversion from the Ventura River with works having capacities of from 50 to 200 second-feet, in increments of 50 second-feet. By analyzing daily records of runoff in the North Fork of Matilija Creek and estimates of daily rates of spill that would have occurred from Matilija Reservoir during the base period, the amounts of water that could have been diverted to Casitas Reservoir for each of the conduit capacities were determined. Neglecting for the moment prior rights to Ventura River water below the point of diversion to Casitas Reservoir, the water available for such diversion would have included all spills from Matilija Reservoir plus the entire runoff of the North Fork of Matilija Creek. The seasonal amounts of water that could have been so diverted to Casitas Reservoir during the base period, by the four capacities of diversion conduit and with Matilija Reservoir in operation, are presented in Table 52.

TABLE 52

ESTIMATED SEASONAL POTENTIAL FOR DIVERSION OF WATER FROM VENTURA RIVER TO CASITAS RESERVOIR DURING BASE PERIOD WITH MATILIJA RESERVOIR IN OPERATION AND WITHOUT PROVISION FOR DOWNSTREAM RIGHTS

(In acre-feet)

Season	Capacity of diversion conduit, in second-feet			
	50	100	150	200
1936-37	15,920	24,960	30,890	35,000
1937-38	18,500	28,490	35,410	40,690
1938-39	10,090	11,500	12,160	12,340
1939-40	6,000	6,910	7,150	7,240
1940-41	23,700	38,790	49,780	59,150
1941-42	11,710	12,600	12,960	13,260
1942-43	14,960	23,000	28,590	32,900
1943-44	15,920	21,630	25,520	28,410
1944-45	10,370	11,550	12,100	12,500
1945-46	9,020	10,670	11,530	12,340
1946-47	6,430	7,220	7,600	7,890
1947-48	760	760	760	760
1948-49	1,150	1,150	1,150	1,150
1949-50	1,610	1,630	1,630	1,630
1950-51	590	590	590	590
TOTALS	146,730	201,450	237,820	265,850
AVERAGES	9,780	13,430	15,850	17,720

By combining estimated values of diversions of Ventura River water for each of the four conduit capacities with estimated values of runoff in Coyote Creek at the dam site, total monthly inflows to Casitas Reservoir were determined. From these estimates, mass diagrams of cumulative monthly inflow were plotted. Graphic analysis of the mass diagrams indicated the variation in reservoir yield with storage capacity for each of the four diversion capacities considered, up to the maximum capacity of Casitas Reservoir required to regulate each of the diversions.

Yields indicated on the mass diagrams were corrected to take into account evaporation losses. This was done making operation studies of the selected reservoirs on a monthly basis throughout the base period. An estimated average depth of net seasonal evaporation of 2.00 feet, distributed monthly in accordance with the following tabulation, was employed in the operation studies:

Month	Net evaporation, in feet of depth	Month	Net evaporation, in feet of depth
October	0.20	April	0.15
November	0.09	May	0.20
December	0.05	June	0.25
January	0.04	July	0.28
February	0.05	August	0.30
March	0.11	September	0.28
		TOTAL	2.00

By the same method yields from the mass diagrams were further reduced by the amounts of rights to water of users downstream from the diversion point on the Ventura River. These rights were estimated as the reduction resulting from the diversion in the amounts of water that would otherwise have been available for beneficial use below the diversion point. The estimated average seasonal amounts of the rights are set forth in the following tabulation:

Capacity of diversion works, in second-feet	Estimated rights of downstream users to waters of Ventura River otherwise available for diversion, in acre-feet per season
50	2,450
100	2,800
150	3,000
200	3,050

In all reservoir operation studies of Casitas Reservoir, an allowance was made for loss of effective storage capacity by sedimentation in the amount of 2,000 acre-feet. This value represents the estimated loss after about 20 years of operation.

It was found that the series of wet years from 1936-37 through 1943-44 would have filled Casitas Reservoir by the spring of 1944 to all storage capacities considered, and that the reservoir would have been drained in the fall of 1951. Presented in Table 53 are the estimated storage capacities required to completely regulate runoff in Coyote Creek plus inflow from the Ventura River with the four capacities of diversion conduit considered. The table also shows the estimated net safe seasonal yields that would result from construction of the indicated developments.

TABLE 53

ESTIMATED STORAGE CAPACITIES AND NET SAFE SEASONAL YIELDS OF CASITAS RESERVOIR FOR VENTURA RIVER DIVERSION CONDUIT OF VARIOUS CAPACITIES

Capacity of diversion conduit, in second-feet	Gross reservoir storage capacity required for complete regulation, in acre-feet	Net safe yield, in acre-feet per season
0*	65,000	8,400
50	105,000	15,200
100	130,000	18,300
150	145,000	20,200
200	156,000	21,900

* With use of Coyote Creek water alone.

Rough analysis of earlier drought periods indicated that, with the exception of the 156,000 acre-foot capacity reservoir, the drought period from 1944-45 through 1950-51 was the most severe in regard to yield for all reservoir storage capacities studied. Had a Casitas Reservoir with capacity of 156,000 acre-feet, augmented by a 200 second-foot diversion from the Ventura River, been in operation during the critical water supply period from 1922-23 through 1935-36, it was estimated that the yield shown in Table 53 would have been reduced about 1,000 acre-feet per season.

The operation studies indicated that little increase in yield would be obtained for any given size of reservoir by increasing the capacity of the diversion conduit, unless the reservoir storage capacity exceeded that required for complete regulation of inflow. However, it was found that there was a relatively small difference in estimated costs of constructing conduits of varying capacities up to 200 second-feet, as hereinafter described. For this reason, it was concluded that a conduit with 200 second-foot diversion capacity should be provided, to assure filling of Casitas Reservoir during water supply periods with different regimens of flow than that of the base period and with possible longer and more deficient periods of drought. Table 54 presents estimates of the combined monthly inflow to Casitas Reservoir during the base period with a conduit capacity of 200 second-feet.

TABLE 54

ESTIMATED MONTHLY INFLOW TO CASITAS RESERVOIR DURING BASE PERIOD
WITH VENTURA RIVER DIVERSION CONDUIT CAPACITY 200 SECOND-FEET

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	120	80	2,420	2,000	19,740	15,040	9,200	3,700	1,610	700	300	150	55,060
1937-38	150	240	1,560	1,000	16,110	27,470	8,910	4,590	2,310	1,220	690	340	64,590
1938-39	440	500	2,550	2,980	2,190	4,060	1,280	670	150	90	70	60	15,040
1939-40	60	70	90	540	4,640	2,400	970	400	120	60	40	40	9,430
1940-41	50	60	4,610	11,940	19,630	29,040	20,380	9,790	4,290	2,710	1,530	920	104,950
1941-42	950	900	3,030	2,500	1,490	1,530	3,730	1,580	510	140	90	80	16,530
1942-43	90	90	110	17,050	11,420	19,190	5,580	3,070	1,330	560	270	160	58,920
1943-44	150	190	1,680	1,650	12,650	15,630	4,980	2,700	1,440	620	230	160	42,080
1944-45	180	1,240	750	760	7,360	4,470	2,360	1,150	440	160	100	80	19,050
1945-46	100	90	2,760	1,450	1,400	2,950	4,620	1,490	440	120	90	70	15,580
1946-47	80	1,560	3,680	2,390	1,130	880	420	120	70	40	40	30	10,440
1947-48	40	50	60	60	80	140	140	100	60	30	20	30	810
1948-49	40	50	110	120	90	350	150	150	100	50	40	30	1,280
1949-50	50	70	200	290	1,360	350	240	170	90	50	30	50	2,950
1950-51	50	70	70	100	80	100	70	60	40	20	10	10	680
Average seasonal inflow, 1936-37 through 1950-51													27,820

Estimates of the net safe seasonal yields that could be obtained with selected storage capacities of Casitas Reservoir, and with the 200 second-foot capacity diversion conduit, are shown in Table 55.

TABLE 55

ESTIMATED NET SAFE SEASONAL YIELDS
OF CASITAS RESERVOIR FOR SELECTED STORAGE CAPACITIES,
WITH 200 SECOND-FOOT VENTURA RIVER DIVERSION CONDUIT

(In acre-feet)

Reservoir storage capacity		
Gross	Net	Net safe yield
92,000	90,000	14,000
105,000	103,000	15,600
130,000	128,000	18,600
156,000	154,000	21,900

Design Features of Ventura River-Casitas Diversion. Investigation was made of three possible sites for weirs to divert Ventura River water to Casitas Reservoir. The uppermost of the three sites considered is on the North Fork of Matilija Creek about 0.6 mile above its confluence with Matilija Creek. Diversion at this site would involve conveying a portion of the North Fork flow through a tunnel into Matilija Reservoir, with release from that reservoir conveyed through a conduit to Santa Ana Creek, a tributary of Coyote Creek above Casitas Reservoir. The middle of the three sites considered is located immediately downstream from the confluence of the North Fork and Matilija Creek, and the diversion would include a conduit leading to Santa Ana Creek over a portion of the route of the preceding alternate. The lowermost of the three diversion sites studied is about 1.3 miles downstream from the confluence of the North Fork and Matilija Creek, and about one mile upstream from Meiners Oaks. The conduit to Santa Ana Creek from this site would also be aligned over a portion of the route utilized by the preceding alternatives. Estimates of cost for diversion works and conduits with capacities of 100 second-feet, 150 second-feet, and 200 second-feet, for each of the three sites, indicated that use of the middle site would be slightly more economical than the others. This fact, together with minor favoring engineering considerations, resulted in choice of the middle site for further study. As previously mentioned, a large diversion capacity may be needed in the future to assure filling of Casitas Reservoir under certain conditions of water supply. For this reason a diversion conduit from the Ventura River of 200 second-foot capacity was selected for cost analysis.

Preliminary designs for the diversion conduits and estimates of construction quantities were made from a profile prepared by the Ventura County Flood Control District in 1951, at a horizontal scale of one inch to 1,000 feet, and a vertical scale of one inch to 20 feet. Alignment and grade for those portions of the conduits above the limit of the County's location survey were determined by use of United States Geological Survey topographic maps at a scale of 1:24,000, and from information obtained during a field reconnaissance. Preliminary estimates of construction quantities for the diversion weirs were obtained from profiles at a scale of one inch equals 20 feet, both horizontally and vertically, prepared from field surveys by the Division of Water Resources.

The proposed diversion weir at the middle site would be of the concrete overpour type with ogee section, founded on bedrock at a stream bed elevation of 900 feet. The weir would be 10 feet in height above stream bed, and would be about 170 feet in length. Water would be diverted over a parapet wall into a side channel diversion box, and thence into a sand trap. From the sand trap, water would discharge either into a reinforced concrete pipe, 54 inches in diameter, or via sluiceways into the Ventura River. The pipe line would parallel the Ventura River on its right bank southerly for a distance of about 17,600 feet to a point west of Meiners Oaks, where it would discharge into a canal. The canal would extend about 14,730 feet southwesterly to discharge into Santa Ana Creek, about 3.5 miles upstream from the Casitas dam site. Included in the length of canal would be two flumes, comprising a total length of about 630 feet. The canal would be shotcrete lined, and would have a 5-foot bottom width, 1.5:1 side slopes, and a depth of water of 3.2 feet, with a freeboard allowance of 1.0 foot. The slope of the canal would be 0.002, and the velocity of water flowing therein at design capacity would be 6.4 feet per second. The two flumes would be of metal construction, 8.3 feet and 8.9 feet in diameter, and with slopes of 0.0022 and 0.0014, respectively.

The location of the proposed diversion weir at the middle site, and of the approximate alignment of the conduit are shown on Plate 42, entitled "Proposed Conveyance and Distribution Systems". General features of the Ventura River-Casitas Diversion are presented in Table 56.

TABLE 56

GENERAL FEATURES OF VENTURA RIVER-CASITAS DIVERSION
WITH CAPACITY OF 200 SECOND-FEET

Diversion Weir		
Type	Concrete gravity weir, with ogee overpour section; side channel diversion box, with overpour parapet wall, and 5 by 5 foot slide headgates in concrete headwall, and 5 by 5 foot slide sluicegate.	
Crest elevation, in feet, U.S.G.S. datum	910	
Height of weir above stream bed, in feet	10	
Length of weir, in feet	170	
Diversion Conduit		
Pipe Line		
Type	54-inch diameter, reinforced concrete	
Length, in feet	17,600	
Canal		
Type	Trapezoidal, shotcrete lined	
Length, in feet	14,100	
Side slopes	1.5:1	
Bottom width, in feet	5.0	
Depth of water, in feet	3.2	
Freeboard, in feet	1.0	
Slope	0.002	
Velocity, in feet per second	6.4	
Flumes		
Type	Lennon metal flume - semicircular section	
Length, in feet	600	30
Diameter, in feet	8.3	8.9
Freeboard, in feet	0.50	0.54
Slope	0.0022	0.0014
Velocity, in feet per second	9.1	7.6

Design Features of Casitas Dam and Reservoir. As a result of the previously described geologic investigation and yield studies, preliminary estimates of cost were made for dams at the Casitas site of 178 feet, 188 feet, 202 feet, and 215 feet in height from stream bed to spillway lip, creating reservoir storage capacities of 92,000 acre-feet, 105,000 acre-feet, 130,000 acre-feet, and 156,000 acre-feet, respectively. For all heights of dam, a rolled fill structure was considered, comprising an upstream impervious section of select earth material with a downstream section of random earth material. Both upstream and downstream slopes of the dams would be 3:1, with a slope of the downstream face of the impervious section of 1:1. Crest widths for the dams would be 25 feet. Random material was chosen for downstream sections rather than pervious fill because of the absence of suitable permeable material in the area. Utilizing random fill would require installation of gravel drains to remove any small amount of leakage that might occur through the impervious section. A gravel blanket, with a thickness of 6 feet normal to the downstream slope of the impervious fill, would be placed at the contact between the impervious and random fill, and would extend to a height of two-thirds of the distance between stream bed and spillway lip. Placing the gravel blanket to this height should amply cover that portion of the face of the impervious fill within the zone of saturation. Seepage intercepted by the blanket would be discharged into four longitudinal gravel drains extending to the toe of the random fill. These drains would be about 6 feet in thickness and 15 feet in width, and would be placed along each abutment and at one-third points across the stream bed. The upstream slope of the dam would be protected against wave action by placement of riprap to a depth 3 feet normal to the slope. The downstream slope of the random section would be stabilized and protected against the erosive action of rainfall by finishing off with topsoil, rolling in barley straw, and planting of bacharis shoots. Horizontal gutters, paved with cobbles, would be provided at 30-foot vertical intervals.

It was assumed that about 50 feet of alluvial sand and gravel would have to be stripped from under the impervious section. Under the random section in the stream bed, stripping depth was estimated to be 5 feet. Stripping requirements on the left abutment were estimated to be on the order of 10 feet under the impervious section, and 5 feet under the random section. On the right abutment, stripping would average about 20 feet under the impervious section and about 10 feet under the random section.

For estimating purposes for all heights of dam considered, it was assumed that the slide existing between elevations of 325 and 425 feet in the right abutment would be removed in its entirety, thereby adding about 50,000 cubic yards to stripping requirements. It was estimated that about 80 per cent of foundation stripping would be used for random fill, thus reducing required borrow. Field investigation indicated that sufficient borrow for the impervious fill could be obtained within a distance of about 3,000 feet from the site. Stripping excavation quantities were divided into common and rock classifications, in order to take advantage of the lower unit costs for excavation of large volumes of common material with tractor-drawn scrapers. It was assumed that rock excavation in the stream bed would consist of dressing-up the foundation surface with power shovels, bulldozers, or rooters. Foundation treatment would also include moderate grouting to insure against excessive seepage. Stripping of abutments would involve excavation of soil and solid rock in moderate quantities, and/or broken rock in relatively large quantities. It was assumed that both impervious and random material would be placed with the tractor-drawn scraping equipment and compacted with sheepsfoot tampers. Gravel for the drains and pervious blanket would probably have to be imported from the Santa Clara River near Saticoy, about 20 miles in distance. The nearest known source of rock for riprap is near Matilija Dam, which is about 11 miles from the site. Access to the site during and after construction could be maintained via the Casitas Pass Road.

It is indicated that excessive leakage might occur through the relatively thin rib that forms the left abutment of the Casitas dam site, and that it might be necessary to place an impervious blanket on the upstream slope of this rib. Provision for such a blanket was not included in the estimates for the 92,000 and 105,000 acre-foot reservoirs, it being assumed that if substantial leakage were observed after construction the reservoir could be drawn down and the blanket placed at that time. For the 130,000 and 156,000 acre-foot reservoirs, dam axes were moved a short distance upstream, and blanketing of both the left and right abutments was effected by impervious fill of the dam.

Spillways for all heights of dam considered would have a discharge capacity of 17,000 second-feet, which is the estimated peak discharge of a once in 1,000-year flood. The spillways were desiged as overpour chute types, with ogee weirs, concrete lined, and founded on bedrock in the left abutment. The designed maximum depth of water above the spillway lip varied from 9.4 to 11.0 feet for the several sizes of dam, and the residual freeboard comprised the remaining distance to the dam crest, which was 20 feet above the spillway lip. For the dam creating a reservoir of 92,000 acre-feet capacity, the spillway would be located in a saddle about 1,600 feet east of the center line of the stream channel. For the 105,000 and 130,000 acre-foot reservoirs, a saddle about 1,000 feet further east would be employed, whereas the spillway for the 156,000 acre-foot reservoir would be constructed about 400 feet still further to the east.

In the selection of spillway sites, consideration was given to utilization of a saddle in the reservoir rim, through which the Santa Ana Road enters the reservoir area, where spill could be discharged directly into the Ventura River. Preliminary estimates of cost indicated that this site did not compare favorably with the sites chosen.

Outlet works would be located in a circular reinforced concrete tower, located upstream from the dam on the right abutment, varying in diameter and height in accordance with the considered height of dam. Water would enter the tower through six gate valves, which would also vary in diameter in accordance with the considered reservoir capacity. Intake to the tower would be conveyed beneath the dam in a reinforced concrete cylinder pipe. The pipe would be encased in concrete and placed in a trench excavated in the foundation along the right abutment. Placing the outlet pipe on this abutment would be contingent upon finding satisfactory foundation conditions after removal of the aforementioned slide. For the 92,000 and 105,000 acre-foot reservoirs, 42-inch diameter outlet pipes were assumed, with 48-inch diameter pipes employed in the estimates for the two larger reservoirs considered. The outlet conduit would feed into a control house where a bifurcation structure controlled by gate valves would be placed, thereby allowing water discharged from the reservoir to enter either Coyote Creek or into the proposed distribution system.

It was estimated that two years would be required for construction of the 92,000 acre-foot and 105,000 acre-foot reservoirs, three years for the 130,000 acre-foot reservoir, and four years for the 156,000 acre-foot reservoir. It was assumed that the construction schedule would be arranged so that the embankment would be placed to stream bed level prior to the first winter season. Runoff during the first season would be passed over the embankment in a channel 50 feet in width, constructed along the right abutment. Outlet works would be constructed during the second working season. For the two dams requiring in excess of two years to complete, it was assumed that the embankment would be high enough during the second winter season so that sufficient storage would be available to handle floods of record, and that releases could be effected through the outlet works.

From study of aerial photographs, it was concluded that clearing of trees and brush would be required from about one-half of the Casitas reservoir area, or from about 800 to 1,000 acres. Approximately 3.5 miles of State Highway 150, and about 2.0 miles of county road would require relocation. Provision was made for a service road on the easterly side of the reservoir area. Relocation of certain other utilities also would be required, including a power line of the Southern California Edison Company. About 4,300 acres of privately owned lands and improvements would have to be acquired.

Estimates of costs for relocating State Highway 150, the county road, for acquisition of reservoir lands and improvements, and for relocation of utilities were made in 1951 by the Ventura County Flood Control District. A revised estimate of the cost of acquisition of lands and improvements was furnished by the Ventura County Flood Control District in 1953.

Pertinent data with respect to general features of the four sizes of dam and reservoir considered at the Casitas site, as designed for cost estimating purposes, are presented in Table 57. For illustrative purposes, a plan, profile, and section for the dam creating a reservoir with a capacity of 130,000 acre-feet are shown on Plate 26, entitled "Casitas Dam on Coyote Creek".

TABLE 57

GENERAL FEATURES OF FOUR SIZES OF DAM AND RESERVOIR
AT THE CASITAS SITE ON COYOTE CREEK

Earthfill Dam				
Crest elevation, in feet, U.S.G.S. datum	523	533	547	560
Crest length, in feet	1,665	1,695	2,540	3,970
Crest width, in feet	25	25	25	25
Height, spillway lip above stream bed, in feet	178	188	202	215
Side slopes, upstream and downstream	3:1	3:1	3:1	3:1
Freeboard above spillway lip, in feet	10.6	9	9	8.5
Elevation of stream bed, in feet, U.S.G.S. datum	325	325	325	325
Volume of fill, in cubic yards	4,715,400	5,461,800	6,934,100	12,441,800
Reservoir				
Surface area at spillway lip, in acres	1,375	1,530	1,790	2,000
Gross storage capacity at spillway lip, in acre-feet	92,000	105,000	130,000	156,000
Type of spillway	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute
Spillway discharge capacity, in second-feet	17,000	17,000	17,000	17,000
Type of outlet	Concrete tower with 42-inch diameter reinforced concrete cylinder pipe beneath dam, encased in concrete	Concrete tower with 42-inch diameter reinforced concrete cylinder pipe beneath dam, encased in concrete	Concrete tower with 48-inch diameter reinforced concrete cylinder pipe beneath dam, encased in concrete	Concrete tower with 48-inch diameter reinforced concrete cylinder pipe beneath dam, encased in concrete

Summary of Estimated Costs. Presented in Table 58 is a summary comparison of capital and annual costs of the four considered sizes of dam and reservoir at the Casitas site, and of the Ventura River-Casitas diversion with a capacity of 200 second-feet. Also presented in Table 58 are estimated unit costs of storage capacity and net safe yield of water that would result with construction of the indicated works. Certain of these latter relationships are depicted graphically on Plates 35, 36, and 37. Detailed estimates of cost for the four sizes of dam and reservoir, and for the Ventura River-Casitas diversion works and conduit, are presented in Appendix C.

TABLE 58

SUMMARY OF ESTIMATED COSTS OF DAMS, RESERVOIRS, AND YIELDS OF WATER AT THE CASITAS SITE ON COYOTE CREEK, WITH DIVERSION FROM VENTURA RIVER OF 200 SECOND-FOOT CAPACITY

Item	Reservoir storage capacity, in acre-feet			
	92,000	105,000	130,000	156,000
Capital Costs				
Dam and reservoir	$ 8,938,000	$ 9,678,000	$ 11,763,000	$19,636,000
Ventura River-Casitas diversion	1,112,000	1,112,000	1,112,000	1,112,000
Totals	10,050,000	10,790,000	12,875,000	20,748,000
Cost per acre-foot of storage capacity	109	103	99	133
Cost per acre-foot of net safe yield	718	692	692	947
Annual Costs				
Dam and reservoir	$467,000	$507,000	$615,000	$1,017,000
Ventura River-Casitas diversion	60,000	60,000	60,000	60,000
Totals	527,000	567,000	675,000	1,077,000
Costs per acre-foot of net safe yield	38	36	36	49
Cost per acre-foot of incremental net safe yield	---	25	36	121

Ferndale Dam and Reservoir. The Ferndale dam site is located on Santa Paula Creek about 0.4 mile southeast of its confluence with Sisar Creek, a principal tributary, and in Section 16, Township 4 North, Range 21 West, S.B.B. & M. State Highway 150, paralleling Santa Paula Creek, passes along the right abutment of the dam site, and traverses a portion of the reservoir area. Stream bed elevation at the dam site is about 910 feet, U.S.G.S. datum. Consideration was given to the construction of a dam and reservoir at the Ferndale site for storage of flood waters in Santa Paula Creek, and utilization of the waters so conserved in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit.

The drainage area of Santa Paula Creek above the Ferndale dam site comprises about 36 square miles, and produced an estimated average seasonal runoff during the base period of about 15,700 acre-feet. It was estimated that waste to the ocean of water originating above the dam site would have averaged about 12,000 acre-feet per season during the base period with the present pattern of land use and water supply development.

The Ferndale dam site was mapped up to an elevation of 1,225 feet in August, 1951, by the Ventura County Flood Control District, at a scale of one inch equals 200 feet, with a 5-foot contour interval. Reservoir areas and capacities for various heights of dam were obtained from available advance sheets of U.S.G.S. quadrangles, at a scale of 1:24,000 with a 20-foot contour interval. Storage capacities of Ferndale Reservoir at various stages of water surface elevation are given in Table 59.

TABLE 59

AREAS AND CAPACITIES OF FERNDALE RESERVOIR

Depth of water at dam, in feet	Water surface elevation, U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	910	0	0
10	920	5	15
20	930	7	75
30	940	9	155
40	950	12	260
50	960	15	390
60	970	26	600
70	980	37	910
80	990	47	1,330
90	1,000	58	1,850
100	1,010	78	2,530
110	1,020	99	3,400
120	1,030	120	4,500
130	1,040	140	5,800
140	1,050	160	7,310
150	1,060	185	9,050
160	1,070	210	11,000
165	1,075	220	12,100
170	1,080	230	13,200
180	1,090	250	15,600
190	1,100	270	18,200
200	1,110	290	21,000
210	1,120	310	24,000
220	1,130	330	27,200
230	1,140	350	30,600
240	1,150	380	34,200
250	1,160	400	38,100
260	1,170	430	42,300
270	1,180	450	46,700
280	1,190	480	51,300
290	1,200	510	56,300

A geologic investigation of the Ferndale dam site was made in 1951 by geologists of the Division of Water Resources. No prior geologic work at this site is known, nor has the site been drilled. Available information indicates that the site is suitable for construction of an earthfill or rockfill dam up to a maximum height of about 270 feet.

Formations at the dam site consist mainly of shale of the Modelo formations and extensive unconsolidated terrace deposits. Upstream from the site, Rincon shale, Pico sediments, Matilija sandstones, and Cozy Dell shale were noted, while Santa Margarita sandstone is in evidence immediately downstream from the site. Terrace deposits occur at various levels, varying from poorly stratified to unstratified in character, and apparently include old stream deposits, land slide, and colluvial material. Most of the terrace deposits contain many pebbles and cobbles, and in the case of the higher terraces include subangular blocks. The amount of fines in the terraces varies considerably, from limited quantities to instances where the amount of such binder material is appreciable. The shale exhibits considerable contortion and folding, with the strike varying from about North 60 degrees East to North 80 degrees East, and with a dip varying from about 55 degrees east to steep overturned dips to the southeast. A zone of thick colluvial cover, land slide material, and extensive travertine deposits occurs on the right abutment upstream from the dam axis.

Two major faults were identified in the vicinity of the site, together with a number of minor faults and shears. A fault trending about North 80 degrees East crosses Santa Paula Creek about 1,000 feet downstream from the dam site. The San Cayetano fault has been mapped, trending in a east-west direction about 1,500 north of the site. However, the Ferndale dam site appears to be free from major faults, so far as could be determined.

Based on estimates of runoff during the base period, yield studies were made for reservoir storage capacities at the Ferndale site of 12,000 acre-feet, 24,000 acre-feet, and 34,000 acre-feet, respectively. Runoff at the site was estimated to be 92 per cent of measured runoff at the U.S.G.S. stream gaging station on Santa Paula Creek near Santa Paula. Estimated monthly runoff of Santa Paula Creek at the Ferndale dam site during the base period is presented in Table 60.

TABLE 60

ESTIMATED MONTHLY RUNOFF OF SANTA PAULA CREEK AT FERNDALE DAM SITE DURING BASE PERIOD

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	460	220	1,370	1,910	9,600	8,630	3,860	1,560	740	480	280	250	29,360
1937-38	260	240	950	500	9,380	22,460	3,020	1,660	930	720	370	310	40,800
1938-39	310	300	1,640	1,150	760	1,440	750	380	240	200	150	470	7,790
1939-40	190	100	210	550	1,740	730	550	300	190	80	70	80	4,790
1940-41	100	110	910	2,750	11,140	19,190	11,710	3,360	1,590	1,060	650	510	53,080
1941-42	480	380	1,300	920	550	510	1,110	510	250	180	80	90	6,360
1942-43	110	130	170	11,950	6,880	11,780	2,370	1,220	710	530	380	320	36,550
1943-44	300	300	930	800	4,970	8,250	2,330	1,170	620	400	290	280	20,640
1944-45	290	990	410	400	3,250	2,460	1,490	760	520	280	200	160	11,210
1945-46	200	210	2,360	640	610	2,190	2,350	800	440	220	150	120	10,290
1946-47	150	1,690	1,980	990	520	450	350	260	130	80	50	90	6,740
1947-48	80	110	160	120	140	210	340	170	70	80	50	40	1,570
1948-49	60	50	130	200	190	560	240	170	100	60	40	20	1,820
1949-50	40	140	240	420	1,180	360	340	190	130	70	50	50	3,210
1950-51	50	90	120	130	120	160	100	80	20	20	0	10	900
Average seasonal runoff, 1936-37 through 1950-51													15,670

In all of the studies an allowance was made for reduction in effective reservoir storage capacity due to sedimentation, in the amount of 2,000 acre-feet. This amount represents the estimated loss after about 20 years of operation. An estimated average net seasonal depth of evaporation from the reservoir water surface of 1.70 feet, distributed monthly in accordance with the following tabulation, was employed in the operation studies.

Month	Net evaporation in feet of depth	Month	Net evaporation, in feet of depth
October	0.15	April	0.15
November	0.06	May	0.19
December	0.04	June	0.21
January	0.04	July	0.25
February	0.05	August	0.25
March	0.10	September	0.21
	Total		1.70

Monthly studies of operation of Ferndale Reservoir during the base period were made for the three sizes of reservoir considered under both the uniform release and rapid release methods of operation.

The estimated values of net safe seasonal yields that would be obtained under both the uniform release and rapid release operating criteria are presented in Table 61. The relationship between reservoir storage capacity and net safe seasonal yield, with Ferndale Reservoir operated by the uniform release method and with releases for maintenance of water levels in Santa Paula Basin, is depicted graphically on Plate 36.

TABLE 61

ESTIMATED NET SAFE SEASONAL YIELDS OF FERNDALE RESERVOIR

(In acre-feet)

Reservoir storage capacity	Uniform release operation		Rapid release operation	
	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, with releases for maintenance of ground water levels	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, without releases for maintenance of ground water levels	Available within Santa Clara River Hydrologic Unit	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits
12,000	2,500	4,000	2,500	2,000
24,000	4,900	6,500	4,900	3,000
34,000	6,600	8,500	6,700	4,200

As a result of the geologic investigation and the reservoir yield studies, estimates of cost were prepared for dams at the Ferndale site with heights of 165 feet, 210 feet, and 240 feet from stream bed to spillway lip, creating reservoir storage capacities of 12,000 acre-feet, 24,000 acre-feet, and 34,000 acre-feet,respectively. For all heights of dam, a rolled fill structure was contemplated, comprising an impervious core of select earth material, and upstream and downstream sections of pervious free draining material. Both upstream and downstream slopes of the dam would be 2.5:1 for the dams of 165-foot and 210-foot height, and 3:1 for the dam of 240-foot height. The impervious sections would have upstream and downstream slopes of 1:1. Crest widths would be 30 feet, comprised of a 10-foot width for the impervious core, and 10-foot widths each for the upstream and downstream pervious sections. The upstream face of the dam would be protected against wave action by rock riprap placed to a depth of 3 feet normal to the slope.

In the cost estimates, it was assumed that a depth of about 8 feet of sand and gravel would be stripped in the channel under the impervious core. On the left abutment, depths of from 5 to 50 feet of terrace material and from 4 to 6 feet of fractured shale would be removed. Under the impervious section on the right abutment, stripping requirements were estimated to comprise a depth of about 2 feet of surface soil, plus an average depth of about 12 feet of fractured shale. The cost estimates do not include provisions for removal of the aforementioned land slide and colluvial material from this abutment. Further exploratory work and examination during construction would be required to indicate the amount of additional stripping needed in this area. For the pervious sections, a nominal depth of stripping of 2 feet was assumed throughout the contact area. During actual construction, increased stripping might be required under the pervious sections of the dam to stabilize slopes, particularly in the land slide area on the right abutment. It was assumed that foundation treatment would include moderate grouting.

It is indicated that adequate material for the impervious section of Ferndale Dam is available within one mile upstream and downstream from the site. In this connection, it was assumed that terrace material stripped from the left abutment would be almost entirely usable in the impervious section. Two samples of material, taken from other possible borrow areas, were tested by the Division of Water Resources and were deemed adequate for use in the impervious section. Sufficient borrow material suitable for the pervious sections of Ferndale Dam is likewise available within about a mile of the site. It was estimated that a portion of the material stripped beneath the impervious section, and too coarse for use therein, would be used in the pervious sections. Matilija sandstone, outcropping about one mile upstream from the site, could be quarried for riprap. It was assumed that compaction of the impervious section of the dam would be effected by either sheeps-foot tampers or pneumatic rollers, and that pneumatic rollers would be used to compact the pervious section.

Spillways, for all heights of dams considered, would have a discharge capacity of 37,000 second-feet, which is the estimated peak discharge of a once in 1,000-year flood. The spillways were designed as concrete-lined overpour chutes, with ogee-weir control sections. For the two smaller dams, the spillway weir and chute channel would be excavated across the terrace easterly on the left abutment, and would discharge into a small ravine a short distance downstream from the dam. For the largest of the dams considered, topographic considerations required that the spillway be located across the right abutment. Depth of water above the spillway lip at design discharge capacity would be 20 feet for the dam of 165 foot height, and 25 feet for the dams of both 210 and 240 foot height. A depth of 5 feet of residual freeboard was provided in the spillways for each of the three heights of dam.

As it was estimated that the dam of 165 foot height could be constructed in one year, it was assumed that diversion of summer flow in Santa Paula Creek would be effected through the outlet conduit. For the dams with heights of 210 and 240 feet, requiring an estimated two years for construction, it was assumed that a 15-foot diameter concrete lined tunnel of horseshoe section would be constructed through the right abutment to provide for diversion of winter flows. The tunnel would be about 1,250 feet in length for the smaller dam and about 1,600 feet in length for the larger.

It was assumed that outlet works for both of the larger dams would utilize the diversion tunnel after construction. The approach channel for the outlet works would be 100 feet in length, with a varying bottom width and 1:1 side slopes. Maximum depth of cut would be about 40 feet. A submerged concrete intake structure would be located immediately upstream from the tunnel portal. This structure would consist of a concrete chamber, wherein would be located hydraulic and manual controls for a high pressure steel slide gate which would regulate discharge through the outlet pipe. The intake for the outlet pipe would be located about 20 feet above the floor of the tunnel. The outlet conduit would be placed in the tunnel, and would consist of 60-inch diameter steel pipe, supported by ring girders resting on the floor of the tunnel. The conduit would terminate at a control house located at the downstream portal of the tunnel, wherein releases would be further regulated by a 48-inch diameter needle valve. Access to the outlet pipe and intake structure would be maintained through the diversion tunnel.

For the dam with height of 165 feet, the outlet works would consist of an intake structure similar to those described for the two higher dams, from which water would discharge into a 42-inch diameter steel pipe. The pipe would be supported on ring girders and would be placed within a reinforced concrete conduit, 8 feet in diameter and horseshoe in section. The conduit would be placed in a trench excavated to sound rock across the right abutment, and would terminate at a control house at the downstream toe of the dam. Releases to the outlet pipe would be regulated at the intake structure by a high pressure steel slide gate, operated by controls similar to those for the two higher dams. Further regulation of reservoir releases would be obtained by installing a 36-inch diameter needle valve at the downstream end of the outlet pipe. Access to the pipe and intake structure would be maintained through the outlet conduit.

Construction of a dam at the Ferndale site would require the relocation of about 3.5 miles of State Highway 150. The cost of this relocation was estimated by the Ventura County Flood Control District in 1953 to be about $420,000. Included in the reservoir area is one large ranch, minor agricultural developments, and several small resort and suburban developments. In 1953 the Ventura County Flood Control District also estimated the cost of lands and improvements up to an elevation of 1,100 feet, which would accomodate a reservoir with storage capacity 12,000 acre-feet, and to an elevation of 1,200 feet, which would be required for storage capacities up to 34,000 acre-feet. These estimates do not include the cost of acquiring mineral rights in the reservoir area, which rights could substantially increase estimated acquisition costs. From the results of field examination by the Division of Water Resources, it was estimated that depending on the height of dam, from 270 to 450 acres of trees and brush in the reservoir area would require removal.

Presented in Table 62, are pertinent data with respect to the general features of the three sizes of dam and reservoir considered at the Ferndale site, as designed for cost estimating purposes. For illustrative purposes, a plan, profile, and section for the dam creating a reservoir with storage capacity of 12,000 acre-feet, are shown on Plate 27, entitled "Ferndale Dam on Santa Paula Creek."

TABLE 62

GENERAL FEATURES OF THREE SIZES OF DAM AND RESERVOIR AT THE FERNDALE SITE ON SANTA PAULA CREEK

Earthfill Dam			
Crest elevation, in feet, U.S.G.S. datum	1,100	1,150	1,180
Crest length, in feet	990	1,240	1,390
Crest width, in feet	30	30	30
Height, spillway lip above stream bed, in feet	165	210	240
Side slopes, upstream and downstream	2.5:1	2.5:1	3:1
Freeboard, above spillway lip, in feet	25	30	30
Elevation of stream bed, in feet, U.S.G.S. datum	910	910	910
Volume of fill, in cubic yards	2,311,400	4,101,300	6,334,800
Reservoir			
Surface area at spillway lip, in acres	220	310	380
Gross storage capacity at spillway lip, in acre-feet	12,000	24,000	34,000
Type of spillway	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute
Spillway discharge capacity, in second-feet	37,000	37,000	37,000
Type of outlet	42-inch diameter steel pipe, beneath dam in reinforced concrete conduit	60-inch diameter steel pipe, through diversion tunnel	60-inch diameter steel pipe, through diversion tunnel

Presented in Table 63 is a summary comparison of capital and annual costs of the three considered sizes of dam and reservoir at the Ferndale site. Also presented in Table 63 are estimated unit costs of storage capacity and net safe yields of water that would be developed by construction of the three sizes of reservoir. Yields referred to are those that would result under the uniform release method of reservoir operation. Certain of the relationships presented in Table 63 are depicted graphically on Plates 35, 36, and 37. Detailed estimates of cost for the three sizes of dam and reservoir at the Ferndale site are included in Appendix C.

TABLE 63

SUMMARY OF ESTIMATED COSTS OF DAMS, RESERVOIRS, AND YIELDS OF WATER AT THE FERNDALE SITE ON SANTA PAULA CREEK

Item	Reservoir storage capacity, in acre-feet		
	12,000	24,000	34,000
Capital Costs			
Dam and reservoir	$5,374,000	$7,249,000	$9,865,000
Cost per acre-foot of storage	448	302	290
Cost per acre-foot of net safe yield	2,150	1,480	1,500
Annual Costs			
Dam and reservoir	277,000	373,000	505,000
Cost per acre-foot of net safe yield	110	76	77
Cost per acre-foot of incremental net safe yield	---	40	78

Cold Spring Dam and Reservoir. The Cold Spring dam site is located on the upper reaches of Sespe Creek, in Section 6, Township 5 North, Range 22 West, S.B.B. & M. The site is about three miles downstream from the U. S. Highway 399 bridge across Tule Creek, a tributary of Sespe Creek. Stream bed elevation at the site is about 3,200 feet above an assumed datum of the Santa Clara Water Conservation District which approximates an elevation of 3,190 feet, U.S.G.S. datum. The dam site and most of the reservoir area are located on federally owned land within the Los Padres National Forest. Consideration was given to the construction of a dam and reservoir at the Cold Spring site for storage of flood waters in Sespe Creek, and utilization of the waters so conserved in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit.

The drainage area of Sespe Creek above the Cold Spring dam site comprises about 65 square miles, and produced an estimated average seasonal runoff during the base period of about 16,800 acre-feet. It was estimated that waste to the ocean of water originating above the dam site would have averaged about 14,500 acre-feet per season during the base period with the present pattern of land use and water supply development.

The Cold Spring dam site was mapped up to an elevation of 3,550 feet in 1932, by V. M. Freeman for the Santa Clara Water Conservation District, at a scale of one inch equals 100 feet, with contour interval of 10 feet. In 1925, J. B. Lippincott mapped the reservoir area up to an elevation of 3,410 feet, at a scale of one inch equals 600 feet, with contour interval of 10 feet. Reservoir areas and storage capacities at various stages of water surface elevation, computed from this map, are given in Table 64, but the elevations have been adjusted to the datum of the Santa Clara Water Conservation District dam site map by subtracting 10 feet. Above an elevation of 3,410 feet, the capacities were computed using areas measured from Army Map Service quadrangles, at a scale of 1:31,680, and with a contour interval of 50 feet. As previously stated, U.S.G.S. datum is approximately 10 feet lower than District datum.

TABLE 64

AREAS AND CAPACITIES OF COLD SPRING RESERVOIR

Depth of water at dam, in feet	Water surface elevation District datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	3,200	0	0
10	3,210	2	10
20	3,220	10	70
30	3,230	22	230
40	3,240	35	515
50	3,250	55	965
60	3,260	70	1,590
70	3,270	95	2,410
80	3,280	125	3,520
90	3,290	160	4,930
100	3,300	200	6,700
110	3,310	230	8,810
120	3,320	260	11,200
130	3,330	290	14,000
140	3,340	350	17,200
150	3,350	410	21,000
160	3,360	480	25,400
170	3,370	550	30,600
178	3,378	610	35,000
180	3,380	620	36,400
190	3,390	690	43,000
200	3,400	760	50,200
210	3,410	840	58,300
220	3,420	920	67,100
230	3,430	990	76,700
240	3,440	1,070	87,000
250	3,450	1,140	98,000
252	3,452	1,160	100,000
260	3,460	1,220	109,800
270	3,470	1,290	122,300
280	3,480	1,350	135,500
290	3,490	1,420	149,400
300	3,500	1,490	164,000
310	3,510	1,560	179,200

Based upon preliminary geological reconnaissance, the Cold Spring dam site is considered suitable for a properly constructed earthfill, rockfill, or masonry type of dam of low to moderate height. Geology was investigated by the Division of Water Resources in March, 1952. Two test pits and five core holes, totaling 586 feet in length, were drilled in 1948 by the Ventura County Flood Control District, and the cores were classified by Dr. T. L. Bailey, Consulting Geologist. Previous geologic studies of the site were made by Dr. Charles P. Berkey, Paul F. Kerr, and Hyde Forbes in the early thirties.

The rocks at the Cold Spring site are a gently dipping series of thick-bedded to massive fine-grain sandstones and more thinly bedded siltstones. A small amount of true shale is also present. The sandstones generally contain a considerable amount of silt, and perhaps some clay. The rocks probably belong to the Cozy Dell formation of Eocene age.

The beds on both abutments average nearly east-west in strike, and apparently without exception dip to the north on the flank of an anticline whose axis lies about three-quarters of a mile south of the site. The strike varies locally, largely because of a notable tendency of the beds to thicken or pinch out in short distances. No close folding or contortion of the bed was observed. The northerly dip varies from about 3 to about 20 degrees.

The rocks at the dam site are only moderately jointed, and no shearing or faulting was noted. The possible presence of a fault on the right abutment has been reported. More detailed exploration here is desirable if a dam is to be further considered at this site, but it is not believed that any fault on this abutment would be a major problem.

There is considerable uncertainty concerning the amount of runoff produced by the Sespe Creek watershed above the Cold Spring site. A U.S.G.S. stream gaging station on Sespe Creek near Wheeler Springs was established in 1948. This station is located about five miles upstream from the Cold Spring dam site, and measures runoff from about 50 square miles of watershed, or about

20 per cent of that at the U.S.G.S. stream gaging station on Sespe Creek near Fillmore, above which the drainage area comprises about 254 square miles. From 1948-49 through 1951-52, there occurred three relatively dry seasons and one wet season. Recorded runoff at the station near Wheeler Springs during the three dry seasons from 1948-49 through 1950-51 was about 5.5 per cent of that at the station near Fillmore. However, during the wet season of 1951-52 the runoff at the upper station was about 12 per cent of that at the lower station. It is indicated, therefore, that with an increase in relative wetness of a given season, the percentage of runoff at the upper station increases as compared with runoff at the lower station, and that runoff from various portions of the watershed is not proportional to the ratio of respective drainage areas.

During the base period, the maximum recorded seasonal flow of Sespe Creek near Fillmore was about 376,000 acre-feet in 1940-41, including corrections for upstream impairments. It was estimated that during such a wet season the runoff produced by the watershed above the station near Wheeler Springs would be equal to about 20 per cent of that above the station near Fillmore. Thus, it was assumed that for seasons producing runoff in excess of about 376,000 acre-feet at the Fillmore station, runoff at the upper station would be proportional to the ratio of the respective drainage areas. For seasons with lesser amounts of runoff at the lower station, runoff at the upper station was estimated from a curve drawn to show the relationship of runoff of Sespe Creek near Fillmore with that of Sespe Creek near Wheeler Springs during the four seasons of overlapping record. From this curve, runoff for each of the seasons of the base period without record at the Wheeler Springs stream gaging station was estimated.

To derive seasonal runoff at the Cold Spring dam site, estimated or measured seasonal runoff at the Wheeler Springs stream gaging station was increased by 30 per cent, or in proportion to the ratio of the respective drainage areas. Monthly distribution of seasonal runoff at the Cold Spring dam site for each season of the base period was estimated from the measured monthly percentage of seasonal runoff for Sespe Creek near Fillmore during seasons from 1936-37 through 1947-48, and from similar data for Sespe Creek near Wheeler Springs during seasons subsequent to 1947-48. Presented in Table 65 is the estimated monthly runoff of Sespe Creek at the Cold Spring dam site during the base period.

TABLE 65

ESTIMATED MONTHLY RUNOFF OF SESPE CREEK AT COLD SPRING DAM SITE DURING BASE PERIOD

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	500	110	2,480	1,890	10,720	8,770	2,610	950	450	200	130	100	28,910
1937-38	120	160	780	430	13,790	26,470	2,780	1,470	790	470	280	250	47,790
1938-39	100	110	750	660	400	880	310	170	100	50	40	250	3,820
1939-40	70	60	70	220	1,030	360	220	100	50	20	20	20	2,240
1940-41	110	100	4,660	8,840	26,390	32,840	17,730	4,440	1,880	1,020	650	490	99,150
1941-42	150	160	780	440	280	290	760	250	110	50	40	30	3,340
1942-43	70	100	130	9,840	6,520	9,070	1,400	690	370	210	140	100	28,640
1943-44	100	130	1,220	720	5,650	9,160	2,250	970	550	280	170	120	21,320
1944-45	80	600	190	150	1,920	970	510	210	120	60	50	30	4,890
1945-46	50	80	1,700	280	290	1,870	1,390	300	110	60	40	30	6,200
1946-47	30	930	1,530	460	230	190	140	80	50	30	20	20	3,710
1947-48	20	20	30	30	50	110	120	60	30	10	10	10	500
1948-49	10	20	50	60	60	180	120	100	40	10	10	10	670
1949-50	0	20	120	120	260	140	140	80	30	10	0	0	920
1950-51	0	10	20	50	50	80	50	40	10	0	0	0	310
Average seasonal runoff, 1936-37 through 1950-51													16,830

Based on the estimates of runoff, monthly studies of operation of Cold Spring Reservoir during the base period were made for four sizes of reservoir of 35,000 acre-foot, 43,000 acre-foot, 77,000 acre-foot, and 100,000 acre-foot storage capacity, under both the uniform release and rapid release methods of operation.

In all of the studies, an allowance was made for reduction in effective reservoir storage capacity due to sedimentation, in the amount of 3,000 acre-feet. This amount represents the estimated loss after about 20 years of operation. An estimated average net seasonal depth of evaporation from the reservoir water surface of 1.70 feet, distributed monthly in accordance with the following tabulation, was employed in the operation studies.

Month	Net evaporation, in feet of depth	Month	Net evaporation in feet of depth
October	0.15	April	0.15
November	0.06	May	0.19
December	0.04	June	0.21
January	0.04	July	0.25
February	0.05	August	0.25
March	0.10	September	0.21
		TOTAL	1.70

The estimated values of net safe seasonal yield that would be obtained under both the uniform release and rapid release operating criteria, are presented in Table 66. The relationship between reservoir storage capacity and net safe seasonal yield, with Cold Spring Reservoir operated by the uniform release method with releases for maintenance of water levels in Fillmore and Santa Paula Basins is depicted graphically on Plate 36.

TABLE 66

ESTIMATED NET SAFE SEASONAL YIELDS OF COLD SPRING RESERVOIR

(In acre-feet)

Reservoir storage capacity	Uniform release operation: Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, with releases for maintenance of ground water levels	Uniform release operation: Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, without releases for maintenance of ground water levels	Rapid release operation: Available within Santa Clara River Hydrologic Unit	Rapid release operation: Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits
35,000	5,000	5,500	5,100	3,500
43,000	6,500	7,000	6,600	4,200
77,000	10,500	11,800	11,600	6,600
100,000	12,000	13,800	12,200	8,800

As a result of the geologic investigation and the reservoir yield studies, estimates of cost were prepared for dams at the Cold Spring site with heights of 178 feet, 190 feet, 230 feet, and 252 feet from stream bed to spillway lip, creating reservoir storage capacities of 35,000 acre-feet, 43,000 acre-feet, 77,000 acre-feet, and 100,000 acre-feet, respectively. For all heights of dam, a rolled fill structure was contemplated, comprised of an impervious core of select earth material, and upstream and downstream sections of random material. Both upstream and downstream slopes of the dam would be 3:1 for the dams of 178-foot, 190-foot, and 230-foot height, and 3.25:1 for the dam of 252 foot height. The impervious sections would have upstream and downstream slopes of 1:1. Crest widths would be 30 feet, comprised of a 10-foot width for the impervious core, and 10-foot widths each for the upstream and downstream random sections.

The foregoing selection of random rather than pervious fill for the outer sections of the dam resulted from the absence of suitable permeable material in the area. Employment of the random fill would necessitate the installation of gravel drains in the downstream portion of the dam, to remove any leakage that might occur through the impervious section. A gravel blanket, with a thickness of 6 feet normal to the downstream slope of the impervious fill, would be placed at the contact between the impervious and random fill, and would extend to a height of two-thirds of the distance between stream bed and spillway lip. Placing the gravel blanket to this height should amply cover that portion of the face of the impervious fill within the zone of saturation. Seepage intercepted by the blankets would be distributed into four longitudinal gravel drains extending to the toe of the random fill. These drains would be about 6 feet in thickness and 15 feet in width, and would be placed along each abutment and at the one-third points across the stream bed. The upstream face of the dam would be protected against wave action by rock riprap placed to a depth of 3 feet normal to the slope. The downstream face of the dam would be stabilized and protected against the erosive action of rainfall by finishing off with top soil, rolling in barley straw, and planting bacharis shoots. Horizontal gutters, paved with cobbles, would be provided at 30-foot vertical intervals.

In the cost estimates, it was assumed that a depth of about 18 feet of sand and gravel would be stripped in the channel under the impervious core. On the left abutment, depths of 7 feet of rocky talus material, plus an additional 5 feet of bedrock, would be stripped for a vertical distance of about 100 feet above stream bed. Above this elevation the abutment consists of massive sandstone and thinner bedded siltstone outcrops, of which a depth of 5 feet would be stripped under the impervious core. Under the impervious section of the right abutment, depths of about 2 feet of soil and weathered rock, plus 5 feet of underlying jointed bedrock, would be stripped. For the random fill sections, a nominal depth of stripping of 2 feet was assumed throughout the contact area. It was assumed that foundation treatment would include moderate grouting.

Earthfill material considered suitable for the impervious section of the dam occurs in limited quantities in terraces both upstream and downstream from the site, but would probably require some sorting. By utilizing Rose Valley, about two miles from the dam site, as a borrow source for fill, it was estimated that sufficient material would be available for dams up to 272 feet in height.

Two samples of material, taken from possible borrow areas, were tested by the Division of Water Resources and were deemed adequate for use in the impervious section. Borrow material suitable for the random sections is available in somewhat limited quantities from stream gravels and from the coarse fraction in the aforementioned terrace deposits. It was estimated that a portion of the material stripped beneath the impervious section would be used in the random sections. The sandstones of the area would be quarried for riprap. It was assumed that compaction of fill material in both the impervious and random sections of the dam would be effected by either sheepsfoot tampers or pneumatic rollers. Gravel for the drains and pervious blanket would probably have to be imported from Cuyama Valley, where Tinta and Castle Creeks enter into the Cuyama River, about 24 miles distant.

Spillways, for all heights of dams considered, would have a discharge capacity of 50,000 second-feet, which is the estimated peak discharge of a once in 1000-year flood. The spillways were designed as concrete lined overpour chutes with ogee-weir control sections. For the dam of 178 foot height, the maximum depth of water above the spillway lip would be 17 feet, 5-foot residual freeboard. For the three larger dams, the maximum depth of water above the spillway lip would be 15 feet, with an additional 5 feet of residual freeboard. The spillway weirs and channels would be excavated across the nose of the left abutment, and would discharge into a small ravine downstream from the toe of the dam.

As it was estimated that the dams of 178 and 190 foot height, could be constructed in one year, it was assumed that diversion of waters in Sespe Creek would be effected through the outlet conduit. For the dams of 230 and 252 foot height, requiring an estimated two years for construction, it was assumed that a 16-foot diameter concrete lined tunnel of horseshoe section would be constructed through the left abutment to provide for diversion of winter flows. The tunnel would be about 1,520 feet in length for both dams.

It was assumed that outlet works for both of the larger dams would utilize the diversion tunnel after construction. The approach channel for the outlet works would be 90 feet in length, with a varying bottom width and 1:1 side slopes. Maximum depth of cut would be about 40 feet. A submerged concrete intake structure would be located immediately upstream from the tunnel portal. This structure would consist of a concrete chamber, wherein would be located hydraulic and manual controls for a high pressure slide gate which would regulate discharge through the outlet pipe. The intake for the outlet pipe would be located about 25 feet above the floor of the tunnel. The outlet conduit would be placed in the tunnel, and would consist of 60-inch diameter steel pipe, supported by ring girders resting on the floor of the tunnel. The conduit would terminate at a control house located at the downstream portal of the tunnel, wherein releases would be regulated by a 54-inch diameter Howell-Bunger valve. Access to the pipe and intake structure would be maintained through the diversion tunnel.

For the dams with heights of 178 and 190 feet, the outlet works would consist of an intake structure similar to those described for the two higher dams, from which water would discharge into a 54-inch diameter steel pipe. The pipe would be supported on ring girders and would be placed within a reinforced concrete conduit, 9.5 feet in diameter and horseshoe in section. The conduit would be placed in a trench excavated to sound rock across the left abutment, and would terminate at a control house at the downstream toe of the dam. Releases to the outlet pipe would be regulated at the intake structure by a high pressure steel slide gate, operated by controls similar to those for the two higher dams. Further regulation of reservoir release would be obtained by installing a 48-inch diameter Howell-Bunger valve at the downstream end of the outlet pipe. Access to the pipe and intake structure would be maintained through the outlet conduit.

The Cold Spring dam and reservoir area is owned by the United States

Government, except for one privately owned ranch containing about 44 acres. This ranch lies primarily along the bed of Sespe Creek, and is moderately rolling and undulating land, containing a small orchard, six modest frame buildings, and an outbuilding of the cabin type. The Ventura County Flood Control District, in January, 1952, estimated the cost of acquisition of the privately owned land and improvements to be $25,000. This amount does not include the cost of acquiring mineral rights in the reservoir area. The property has been leased for oil speculation, but the nearest drilling activity is a wildcat well several miles distant. Construction of the three larger dams at the Cold Spring site would require the relocation of about 27,000 lineal feet of U. S. Highway 399, and of two bridges, one crossing Tule Creek and the other Sespe Creek. No road relocation would be required for construction of the smallest of the four dams. An estimate of cost of relocating U.S. Highway 399 was made by the California Division of Highways in 1953. It was assumed that construction of an all purpose access road, approximately two miles in length, would be required for construction of all heights of dam. From the results of field examination of the reservoir area, it was estimated that, depending on the height of dam to be constructed, from 760 to 1,290 acres of minor clearing in the reservoir area would be required.

Presented in Table 67 are pertinent data with respect to the general features of the four sizes of dams and reservoirs considered at the Cold Spring site, as designed for cost estimating purposes. For illustrative purposes, a plan, profile, and section for the dam creating a reservoir with storage capacity of 35,000 acre-feet are shown on Plate 28 entitled "Cold Spring Dam on Sespe Creek".

TABLE 67

GENERAL FEATURES OF FOUR SIZES OF DAM AND RESERVOIR AT THE COLD SPRING SITE ON SESPE CREEK

Earthfill Dam				
Crest elevation, in feet, Santa Clara Water Conservation District datum . . .	3,400	3,410	3,450	3,472
Crest length, in feet	730	770	860	920
Crest width, in feet	30	30	30	30
Height, spillway lip above stream bed, in feet.	178	190	230	252
Side slopes, upstream and downstream . . .	3:1	3:1	3:1	3.25:1
Freeboard, above spillway lip, in feet	22	20	20	20
Elevation of stream bed, in feet, Santa Clara Water Conservation District datum.	3,200	3,200	3,200	3,200
Volume of fill, in cubic yards.	1,919,600	2,246,500	3,403,000	4,569,100
Reservoir				
Surface area at spillway lip, in acres.	606	690	995	1,156
Gross storage capacity at spillway lip, in acre-feet	35,000	43,000	77,000	100,000
Type of spillway. . .	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute
Spillway discharge capacity, in second-feet	50,000	50,000	50,000	50,000
Type of outlet. . . .	54-inch diameter steel pipe beneath dam	54-inch diameter steel pipe beneath dam	60-inch diameter steel pipe through diversion tunnel	60-inch diameter steel pipe through diversion tunnel

Presented in Table 68 is a summary comparison of capital and annual costs of the four considered sizes of dams and reservoirs at the Cold Spring site. Also presented in Table 68 are estimated unit costs of storage capacity and net safe yield of water that would be developed by construction of the four sizes of reservoir. Yields referred to are those that would result under the uniform release method of reservoir operation with releases for maintenance of historic ground water levels. Certain of the relationships presented in Table 68 are depicted graphically on Plates 35, 36 and 37. Detailed estimates of cost for the four sizes of dam and reservoir at the Cold Spring site are included in Appendix C.

TABLE 68

SUMMARY OF ESTIMATED COSTS OF DAMS, RESERVOIRS, AND YIELDS OF WATER AT THE COLD SPRING SITE ON SESPE CREEK

Item	Reservoir storage capacity in acre-feet			
	35,000	43,000	77,000	100,000
Capital Costs				
Dam and reservoir	$3,796,000	$5,613,000	$7,283,000	$8,571,000
Cost per acre-foot of storage	108	131	95	86
Cost per acre-foot of net safe yield	760	860	690	710
Annual Costs				
Dam and reservoir	199,000	292,000	378,000	446,000
Cost per acre-foot of net safe yield	40	45	36	37
Cost per acre-foot of incremental net safe yield	--	62	22	45

Topatopa Dam and Reservoir. The Topatopa dam site is located on Sespe Creek about 19 miles below the Cold Spring dam site, and is in Section 36, Township 6 North, Range 20 West, S.B.B. & M. Stream bed elevation at the site is about 2,100 feet, U.S.G.S. datum. Consideration was given to the construction of a dam and reservoir at the Topatopa site for storage of flood waters in Sespe Creek, and utilization of the waters so conserved in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit.

The drainage area of Sespe Creek above the Topatopa dam site comprises about 171 square miles, and produced an estimated average seasonal runoff during the base period of about 43,600 acre-feet. It was estimated that waste to the ocean of water originating above the dam site would have averaged about 37,600 acre-feet per season during the base period with the present pattern of land use and water supply development.

The Topatopa dam site and reservoir area were surveyed in 1950 by Fairchild Aerial Surveys, Inc., using photogrammetric methods, for the Ventura County Flood Control District, Zone 2. The dam site was mapped up to an elevation of 2,750 feet at a scale of one inch equals 100 feet, with a 5-foot contour interval. The reservoir area was mapped up to an elevation of 2,650 feet, at a scale of one inch equals 400 feet, with a contour interval of 20 feet. Storage capacities of Topatopa Reservoir at various stages of water surface elevation are given in Table 69.

TABLE 69

AREAS AND CAPACITIES OF TOPATOPA RESERVOIR

Depth of water at dam, in feet	Water surface elevation U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	2,100	0	0
20	2,120	5	50
40	2,140	17	270
60	2,160	35	780
80	2,180	61	1,740
100	2,200	90	3,250
120	2,220	120	5,310
140	2,240	150	7,950
160	2,260	180	11,200
180	2,280	220	15,200
200	2,300	270	20,000
220	2,320	320	25,900
240	2,340	370	32,900
260	2,360	430	40,900
280	2,380	510	50,300
300	2,400	580	61,200
320	2,420	650	73,500
322	2,422	660	75,000
340	2,440	730	87,300
355	2,455	790	100,000
360	2,460	810	102,600
380	2,480	900	119,700
400	2,500	1,020	138,900
420	2,520	1,110	160,300
440	2,540	1,230	183,800
460	2,560	1,350	209,600
480	2,580	1,480	238,000
500	2,600	1,590	268,700
520	2,620	1,720	301,800
540	2,640	1,850	337,500
550	2,650	1,920	356,300

Geologic investigation indicates that the Topatopa dam site is suitable for almost any type of structure up to heights above stream bed of the order of 400 feet. The geology of the site was studied by the Division of Water Resources during the current investigation. Previous geologic studies had been made by Dr. Charles P. Berkey, Paul F. Kerr, and Hyde Forbes, and by geologists of the Division of Water Resources in connection with the preparation of Division of Water Resources Bulletin No. 46. Some geologic work at the site has also been done by Thomas L. Bailey, Consulting Geologist. Three core holes were drilled at the Topatopa dam site in 1948 by the Ventura County Flood Control District totaling 302 feet in length. In 1952, 17 core holes were drilled by the United Water Conservation District, with a total length of 1,471 feet.

Rock exposed at the Topatopa dam site consists of hard, greenish-grey sandstone, interbedded with subordinate amounts of hard black or sandy shale. The sandstone beds vary from very massive to moderately thin bedded. The sandstone generally takes on a mottled appearance on weathering, and ripple-marked beds are present on both abutments just above the channel section. Strike of the bedding is across the channel and is quite consistent, averaging about north 30 degrees east. The dip is also uniform and averages about 18 degrees in a southeast direction, or downstream.

No positive evidence of a fault down the channel at the axis of the Topatopa dam has been found. However, a calcite deposit found in one of the drill holes of the United Water Conservation District suggest that a fault may exist in the channel at the point of drill hole No. 13. A fault was reported by Berkey and Kerr on the left abutment between about 0.25 and 0.5 mile upstream from the axis in a ravine containing a dry weather spring. The dip of the beds on either side of this fault varies from 30 degrees north on one side to 50 degrees south on the other, with gouge and calcite veins present between. This fault is now believed to extend along the left abutment downstream at an elevation of about 750 feet above the stream bed. It appears that the fault finally approaches the stream bed and crosses it immediately above the confluence of Sespe and Alder Creeks. Two minor faults were noted on the left abutment, one of which dips steeply upstream and shows a displacement of about 20 feet, and the other which appears to be a small thrust. Another minor fault was noted in the right abutment. Three sets of joints occur at the axis of the dam, and probably persist through the area of the site.

The right abutment of the Topatopa dam site has very steep rugged walls for the first 200 feet above the stream bed, and then slightly gentler slope with a blocky uneven surface. The rock is strongly jointed, with joints somewhat open near the surface. As a result of the drilling program of the United Water Conservation District, it was determined that sound rock in the channel section lies beneath about 40 feet of sand, silt, gravel, and boulders of sandstone and crystalline rock. The first 150 feet above stream bed on the left abutment consists of a nearly vertical cliff, with a talus deposit to an elevation about 50 feet above the base of the cliff at the dam axis. Above the cliff the slope of the abutment is slightly gentler. The entire abutment is strongly jointed, including some closely spaced sets. Borrow pit exploration for impervious material for a possible earth filled dam at the Topatopa site was conducted by the Division of Water Resources in 1951, using a bulldozer to expose an area located about one mile upstream from the site. Tests of nine samples from this area showed the material to be suitable for the impervious section of an earth filled dam. The United Water Conservation District explored the same area in 1952, and another area at a closer location to the dam site, by drilling auger holes. Drilling indicated that approximately 6,700,000 cubic yards of impervious material were available within one mile upstream from the site. Pervious material for a fill-type dam was determined to be quite limited.

Records of runoff at the Topatopa dam site are not available. However, estimates of runoff were made for the base period, utilizing the short record at the U.S.G.S. stream gaging station on Sespe Creek near Wheeler Springs, and the longer record at the U.S.G.S. station on Sespe Creek near Fillmore. Due to the generally easterly course of Sespe Creek above the dam site, it was assumed that the runoff characteristics would be similar to those at the similarly situated Cold Spring dam site. For this reason, the method of estimating runoff described for Cold Spring Reservoir was employed for the Topatopa site. To derive seasonal runoff at Topatopa Dam, estimated or measured seasonal runoff at the Wheeler Springs stream gaging station was increased by 242 per cent, or in proportion to the ratio of the respective drainage areas. Table 70 presents the estimated monthly runoff of Sespe Creek at the Topatopa dam site during the base period.

TABLE 70

ESTIMATED MONTHLY RUNOFF OF SESPE CREEK AT TOPATOPA DAM SITE DURING BASE PERIOD

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	1,300	290	6,430	4,900	27,800	22,740	6,770	2,460	1,170	520	340	260	74,980
1937-38	310	410	2,020	1,110	35,720	68,560	7,200	3,810	2,050	1,220	730	650	123,790
1938-39	260	290	1,940	1,710	1,040	2,280	800	440	260	130	100	650	9,900
1939-40	180	160	180	570	2,670	930	570	260	130	50	50	50	5,800
1940-41	290	260	12,080	22,920	68,430	85,150	45,970	11,510	4,870	2,640	1,690	1,270	257,080
1941-42	390	410	2,020	1,140	730	750	1,970	650	290	130	100	80	8,660
1942-43	180	260	340	25,520	16,910	23,520	3,630	1,790	960	540	360	260	74,270
1943-44	260	340	3,160	1,870	14,650	23,750	5,830	2,520	1,430	730	440	310	55,290
1944-45	210	1,560	490	390	4,980	2,520	1,320	540	310	160	130	80	12,690
1945-46	130	210	4,410	730	750	4,850	3,600	780	290	160	100	80	16,090
1946-47	80	2,410	3,960	1,190	590	490	360	210	130	80	50	50	9,600
1947-48	50	50	80	80	130	290	310	160	80	30	30	30	1,320
1948-49	30	60	140	170	150	480	320	250	90	30	20	10	1,750
1949-50	20	60	310	310	660	370	370	190	90	20	10	10	2,420
1950-51	10	20	50	120	130	200	140	120	30	10	0	0	830
Average seasonal runoff, 1936-37 through 1950-51													43,630

Based on the estimates of runoff, monthly studies of operation of Topatopa Reservoir during the base period were made for three sizes of reservoir, of 50,000 acre-foot, 75,000 acre-foot, and 100,000 acre-foot storage capacity, under both the uniform release and rapid release methods of operation. In all of the studies, an allowance was made for reduction in effective reservoir storage capacity due to sedimentation, in the amount of 8,000 acre-feet. This amount represents the estimated loss after about 20 years of operation. An estimated average net seasonal depth of evaporation from the reservoir water surface of 1.70 feet, distributed monthly in accordance with the following tabulation, was employed in the operation studies.

Month	Net evaporation, in feet of depth	Month	Net evaporation, in feet of depth
October	0.15	April	0.15
November	0.06	May	0.19
December	0.04	June	0.21
January	0.04	July	0.25
February	0.05	August	0.25
March	0.10	September	0.21
		TOTAL	1.70

The estimated values of net safe seasonal yield that would be obtained under both the uniform release and rapid release operating criteria are presented in Table 71. The relationship between reservoir storage capacity and net safe seasonal yield, with Topatopa Reservoir operated by the uniform release method with releases for maintenance of water levels in Fillmore and Santa Paula Basins, is depicted graphically on Plate 36.

TABLE 71

ESTIMATED NET SAFE SEASONAL YIELDS OF TOPATOPA RESERVOIR
(In acre-feet)

Reservoir storage capacity	Uniform release operation: Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, with releases for maintenance of ground water levels	Uniform release operation: Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, without releases for maintenance of ground water levels	Rapid release operation: Available within Santa Clara River Hydrologic Unit	Rapid release operation: Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits
50,000	8,000	8,400	8,100	6,000
75,000	12,400	12,900	12,500	9,000
100,000	16,500	17,000	16,700	12,000

As a result of the geologic investigation, yield studies, and reconnaissance type estimates of cost of dams of various heights and types, it was concluded that the most economical type of dam at the Topatopa site would be concrete arch, with a maximum physical limit in height of about 400 feet above stream bed. To determine the variation in cost with height of dam, and the accomplishments of reservoirs created by various heights of dam, estimates of cost were prepared for concrete arch dams 280 feet, 322 feet, and 355 feet in height from streambed to top of spillway gates, creating reservoirs with storage capacities of 50,000 acre-feet, 75,000 acre-feet, and 100,000 acre feet, respectively. The dams would be concrete arches, of the variable radius and variable angle type, and would be located so as to best fit the topography at the site.

In the cost estimates, it was assumed that a depth of about 40 feet of sand, gravel, and boulders would be stripped in the channel section. On the right abutment, it was assumed that a depth of about 25 feet of jointed rock would be stripped for the first 200 feet above the stream bed, and that above this elevation the depth of stripping would be about 35 feet. It was assumed that on the left abutment, a depth of about 18 feet of rock would be stripped for the lowermost 200 feet in elevation above stream bed, and that above this elevation the stripping depth would be about 35 feet. Water testing of several of the core holes drilled by United Water Conservation District indicated that moderate to heavy grouting of the foundation would be necessary. For cost estimating purposes, it was assumed that a concrete batch plant would be placed in the vicinity of the dam site during construction. Concrete aggregates could be made locally from a granite deposit located about three miles upstream.

Spillways, for all heights of dam considered, would have a discharge capacity of 82,000 second-feet, which is the estimated peak discharge of a once in 1,000-year flood. For each of the three sizes of dam, two spillways were incorporated in the design. A primary spillway would be provided along the extreme right end of the dam. together with a secondary spillway formed by a notch in the center of the dam. The primary spillway would be equipped with three tainter gates, each 30 feet in length and 20 feet in height. With the water level in the reservoir at the lip of the notched spillway and at the top of the gates, the gated spillway was designed to discharge 28,000 second-feet. With an additional depth of water of 10 feet, the gated spillway would discharge 52,000 second-feet, and the notched spillway 30,000 second-feet. A residual freeboard of 5 feet was provided above this maximum water surface elevation. No provision was made for cushioning of the stream bed, below the notched spillway, as such spill would be very infrequent. The primary spillway would consist of an ogee weir, with the aforementioned gates, and a concrete lined chute discharging into Sespe Creek about 400 feet downstream from the dam. The design of the primary spillway included a concrete gravity thrust block on its left side, separating the spillway weir from the arch. This thrust block would also act as the left training wall for spillway discharge.

Outlet works would include a 60-inch diameter steel pipe, placed through the dam near the right abutment at an elevation of 2160 feet. Discharge from the reservoir would be controlled by a high pressure slide gate, 4.5 feet by 4.5 feet in dimensions, on the upstream face of the dam. Releases would also be controlled at the downstream end of the outlet pipe by a 54-inch diameter Howell-Bunger valve. A trash rack structure would be placed at the upstream end of the outlet pipe. It was estimated that construction of the dam of 280 foot height would require about two years, that of 322 foot height, two and one-half years, and the dam of 355 foot height, about three years. Diversion of the stream during construction would be accomplished by means of a flume or pipe, with the aid of a small coffer dam. Winter flood flows could be passed over a depressed section of the concrete dam.

It was estimated that between 510 and 790 acres of minor clearing would be required in the reservoir area, depending on the height of dam to be constructed. The Topatopa dam site and most of the reservoir lands are federally owned, and in the Los Padres National Forest. In 1952, the cost of acquisition of private lands in the reservoir area was estimated by the Ventura County Flood Control District to be about $25,000 for the two smaller dams, and about $62,500 for the larger dam. An all weather access road approximately 10.5 miles in length would be required before construction could start. The United Water Conservation District estimated in 1952 that this road would cost about $400,000.

Presented in Table 72 are pertinent data with respect to the general features of the three sizes of dams and reservoirs considered at the Topatopa site, as designed for cost estimating purposes. For illustrative purposes, a plan, profile, and section for the dam creating a reservoir with storage capacity of 100,000 acre-feet are shown on Plate 29, entitled "Topatopa Dam on Sespe Creek."

TABLE 72

GENERAL FEATURES OF THREE SIZES OF DAM AND RESERVOIR AT THE TOPATOPA SITE ON SESPE CREEK

Concrete Arch Dam			
Crest elevation, in feet, U.S.G.S. datum	2,395	2,437	2,470
Crest length, in feet . .	850	965	1,120
Crest width, in feet. . .	9	10	10
Height of dam, to top of spillway gates above stream bed, in feet	280	322	355
Freeboard, above top of spillway gates, in feet	15	15	15
Elevation of stream bed, in feet, U.S.G.S. datum.	2,100	2,100	2,100
Volume of concrete in dam, in cubic yards. . .	287,000	412,000	522,000
Reservoir			
Surface area at top of spillway gates, in acres.	510	656	788
Gross storage capacity, at top of spillway gates, in acre-feet. . .	50,000	75,000	100,000
Type of spillways	Notched overpour, and ogee weir with gates and concrete lined chute	Notched overpour, and ogee weir with gates and concrete lined chute	Notched overpour, and ogee weir with gates and concrete lined chute
Spillway discharge capacity, in second-feet	82,000	82,000	82,000
Type of outlet.	60-inch diameter steel pipe, through dam	60-inch diameter steel pipe, through dam	60-inch diameter steel pipe, through dam

Presented in Table 73 is a summary comparison of capital and annual costs of the three considered sizes of dam and reservoir at the Topatopa site. Also presented in Table 73 are estimated unit costs of storage capacity and net safe yields of water that would be developed by construction of the three sizes of reservoir. Yields referred to are those that would result under the uniform release method of reservoir operation. Certain of the relationships presented in Table 73 are depicted graphically on Plates 35, 36, and 37. Detailed estimates of cost for the three sizes of dam and reservoir at the Topatopa site are included in Appendix C.

TABLE 73

SUMMARY OF ESTIMATED COSTS OF DAMS, RESERVOIRS, AND YIELDS OF WATER AT THE TOPATOPA SITE ON SESPE CREEK

Item	Reservoir storage capacity, in acre-feet		
	50,000	75,000	100,000
Capital Costs			
Dam and reservoir	$ 9,155,000	$ 12,520,000	$ 15,540,000
Cost per acre-foot of storage	183	167	155
Cost per acre-foot of net safe yield	1,140	1,010	940
Annual Costs			
Dam and reservoir	482,000	652,000	805,000
Cost per acre-foot of net safe yield	60	53	49
Cost per acre-foot of incremental net safe yield	---	39	37

Hammel Dam and Reservoir. The Hammel dam site is located on the lower reaches of Sespe Creek, in Section 2, Township 4 North, Range 20 West, S.B.B. & M. The site is about four miles north and one mile west of the town of Fillmore, and about seven miles upstream from the confluence of Sespe Creek with the Santa Clara River. Stream bed elevation at the site is about 790 feet, U.S.G.S. datum. Consideration was given to the construction of a dam and reservoir at the Hammel site for storage of flood waters in Sespe Creek, and utilization of the waters so conserved in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunitsof the Santa Clara River Hydrologic Unit.

The drainage area of Sespe Creek above the Hammel dam site comprises about 246 square miles, and produced an estimated average seasonal runoff during the base period of about 92,000 acre-feet. It was estimated that waste to the ocean of water originating above the dam site would have averaged about 73,600 acre-feet per season during the base period with the present pattern of land use and water supply development.

The Hammel dam site and reservoir area were surveyed in 1950 by Fairchild Aerial Surveys, Inc. using aerial photogrammetric methods, for the Ventura County Flood Control District, Zone 2. The dam site was mapped up to an elevation of 1,325 feet, at a scale of one inch equals 100 feet, with a 5-foot contour interval. The reservoir area was mapped at a scale of one inch equals 400 feet, with a contour interval of 20 feet. Storage capacities of Hammel Reservoir at various stages of water surface elevation are given in Table 74.

TABLE 74

AREAS AND CAPACITIES OF HAMMEL RESERVOIR

Depth of water at dam, in feet	Water surface elevation U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	790	0	0
60	850	6	180
70	860	11	270
80	870	17	405
90	880	22	600
100	890	27	845
110	900	32	1,140
120	910	38	1,490
130	920	45	1,910
140	930	51	2,390
150	940	58	2,930
160	950	64	3,540
170	960	71	4,210
180	970	77	4,960
190	980	84	5,770
200	990	91	6,640
210	1,000	98	7,590
220	1,010	110	8,610
230	1,020	115	9,700
240	1,030	120	10,900
250	1,040	130	12,100
260	1,050	140	13,400
270	1,060	145	14,800
280	1,070	155	16,300
290	1,080	160	17,900
300	1,090	170	19,600
310	1,100	180	21,400
320	1,110	190	23,300
330	1,120	200	25,000
340	1,130	210	27,300
350	1,140	220	29,400
360	1,150	230	31,700
370	1,160	240	34,000
380	1,170	250	36,500
390	1,180	260	39,000
400	1,190	270	41,700
410	1,200	285	44,500
420	1,210	300	47,400
428	1,218	308	50,000
430	1,220	310	50,400
440	1,230	325	53,600
450	1,240	340	56,900
460	1,250	350	60,400
470	1,260	365	63,900
480	1,270	380	67,700
490	1,280	390	71,500
500	1,290	410	75,500
510	1,300	420	79,700

Based upon preliminary geological reconnaissance, the Hammel dam site appears suitable for a moderately high masonry structure. No prior geologic work at this site is known, nor has it been drilled. The dam site is located on the southerly limb of the Coldwater anticline, a distinct structural feature in both Coldwater and Sespe formations. The underlying Coldwater sandstone is exposed upstream from the dam site along the anticline, while the Sespe formation is the only rock exposed in the vicinity of the axis. The beds dip steeply downstream about 60 degrees south, and strike across the channel about north 65 degrees east.

The Sespe formation at the Hammel site is a medium to coarse grained, reddish brown, bedded sandstone, generally well indurated. Bedding planes and color banding in the various beds are noteworthy. There are relatively few joints and fractures, but one discontinuous open fracture parallels the left abutment in its lower third near the channel section at the axis. No serious structural defects were noted in this area. The harder beds of sandstone forming the abutments are several hundreds of feet in stratigraphic thickness, and have formed a narrow "V"-shaped canyon with slopes averaging steeper than 1:1 in the lower 300 feet of the cross section at the dam site.

The right abutment in the lower portion, to an elevation about 50 feet above stream bed, has talus blocks up to 50 feet in diameter. Average depth of talus in this area is 20 feet. Above the talus, the right abutment has a light cover of soil and talus over moderately jointed rock. In the channel section, about 120 feet in width, there is a filling of gravels, boulders, and blocks up to 30 feet in diameter. No signs of faulting or pronounced shears were noted in the channel section. A nearly vertical bare cliff rises about 250 feet above stream bed on the left abutment, with good quality rock exposed. Above the top of this cliff the exposed rock exhibits more pronounced jointing. The left abutment appears more favorable topographically for appurtenant features such as outlet tunnels. The canyon is narrow and appurtenant structures may be in a hazardous position due to the possibility of large blocks sliding into the canyon.

Records of runoff at the Hammel dam site are not available. However, runoff at the site was estimated equal to 97 per cent of the measured runoff at the U.S.G.S. stream gaging station on Sespe Creek near Fillmore, adjusted for diversions made upstream from the gaging station by the Fillmore Irrigation Company. The estimates were based on the ratio of watershed areas above the dam site and gaging station, weighted by estimated mean precipitation on the respective areas. The estimated monthly runoff of Sespe Creek at the Hammel dam site during the base period is presented in Table 75.

TABLE 75

ESTIMATED MONTHLY RUNOFF OF SESPE CREEK AT HAMMEL DAM SITE DURING BASE PERIOD

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	2,870	630	14,220	10,830	61,540	50,330	15,000	5,440	2,610	1,140	750	560	165,920
1937-38	580	760	3,780	2,100	66,880	128,360	13,480	7,140	3,820	2,270	1,360	1,220	231,750
1938-39	1,210	1,320	8,820	7,730	4,670	10,310	3,660	1,990	1,130	590	420	2,940	44,790
1939-40	970	870	1,030	3,090	14,460	5,020	3,070	1,470	650	340	280	260	31,510
1940-41	390	380	17,130	32,490	96,980	120,670	65,140	16,300	6,920	3,750	2,390	1,790	364,330
1941-42	1,810	1,940	9,550	5,410	3,450	3,550	9,390	3,020	1,370	630	440	410	40,970
1942-43	420	560	730	56,820	37,660	52,380	8,060	3,990	2,160	1,200	800	600	165,380
1943-44	650	820	7,940	4,710	36,810	59,650	14,650	6,340	3,560	1,840	1,080	790	138,840
1944-45	890	6,430	2,040	1,670	20,640	10,410	5,490	2,300	1,300	680	500	370	52,720
1945-46	480	770	17,190	2,800	2,950	18,850	14,050	3,030	1,130	560	380	320	62,510
1946-47	390	10,970	18,090	5,430	2,760	2,290	1,680	950	560	310	280	260	43,970
1947-48	290	290	460	510	800	1,680	1,820	960	480	210	180	160	7,840
1948-49	200	230	520	710	720	3,680	1,280	700	310	170	160	140	8,820
1949-50	170	420	1,770	3,230	5,780	1,620	1,750	770	330	200	160	190	16,390
1950-51	190	240	250	360	440	710	500	320	150	100	80	100	3,440
Average seasonal runoff, 1936-37 through 1950-51.													91,950

Based on the estimates of runoff, monthly studies of operation of Hammel Reservoir during the base period were made for two sizes of reservoir, of 25,000 acre-foot and 50,000 acre-foot storage capacity, under both the uniform release and rapid release methods of operation. In all of the studies, an allowance was made for reduction in effective reservoir storage capacity due to sedimentation, in the amount of 12,000 acre-feet. This amount represents the estimated loss after about 20 years of operation. An estimated average net seasonal depth of evaporation from the reservoir water surface of 1.70 feet, distributed monthly in accordance with the following tabulation, was employed in the operation studies.

Month	Net evaporation, in feet of depth	Month	Net evaporation, in feet of depth
October	0.15	April	0.15
November	0.06	May	0.19
December	0.04	June	0.21
January	0.04	July	0.25
February	0.05	August	0.25
March	0.10	September	0.21
		TOTAL	1.70

The estimated values of net safe seasonal yield that would be obtained under both the uniform release and rapid release operating criteria are presented in Table 76. The relationship between reservoir storage capacity and net safe seasonal yield, with Hammel Reservoir operated by the uniform release method, with releases for maintenance of ground water levels in Fillmore and Santa Paula Basins, is depicted graphically on Plate 36.

TABLE 76

ESTIMATED NET SAFE SEASONAL YIELDS OF HAMMEL RESERVOIR

(In acre-feet)

Reservoir storage capacity	Uniform release operation		Rapid release operation	
	Available to Oxnard Forebay, Oxnard Plain and Pleasant Valley Subunits, with releases for maintenance of ground water levels	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, without releases for maintenance of ground water levels	Available within Santa Clara River Hydrologic Unit	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits
25,000	4,000	5,800	4,100	3,000
50,000	9,500	11,300	9,600	8,000

As a result of the geological investigation and the reservoir yield studies, estimates of cost were prepared for two dams at the Hammel site with heights of 330 feet and 428 feet from stream bed to top of spillway gates, creating reservoir storage capacities of 25,000 and 50,000 acre-feet, respectively. For both dams, a concrete gravity structure was contemplated. The dams would have crest widths of 30 feet, 0.8:1 downstream slopes and 0.05:1 upstream slopes, except that the upstream slope for the higher dam would be 0.5:1 below an elevation of 823 feet. The two dams would have crest lengths of 470 feet and 810 feet, respectively.

In the cost estimates, it was assumed that the talus and a depth of about 15 feet of jointed rock would be stripped from the right abutment up to an elevation about 75 feet above stream bed. Above this elevation, about 3 feet of soil and talus and 30 feet of rock would be removed. In the channel section, a depth of about 25 feet of gravel, boulders, and blocks up to 30 feet in diameter, would have to be removed. It was assumed that a cut would be made in the cliff which forms the lower portion of the right abutment. The cut would be about 15 feet in depth in its lower half, and about 25 feet in depth in the upper half. Above the top of the cliff, approximately 250 feet above stream bed, the cut would be increased in depth to about 30 feet to include removal of weathered surficial materials.

Spillways, for both heights of dam considered, would have a discharge capacity of 90,000 second-feet, which is the estimated peak discharge of a one in 1000-year flood. The spillways would consist of a concrete overpour section in the center of the dam, and would be provided with four tainter gates, each 30 feet high and 40 feet wide. Maximum depth of water above the bottom of the gates would be 30 feet, and an additional 5 feet of freeboard would be provided. A spillway bucket would be provided at the downstream toe of the dam to deflect the high velocity flood flows into the air. A roadway, 10 feet in width, would be located on the crest of the dams and across the spillway near the upstream face, for access to the tainter gate controls.

Water would be released from the reservoir into a 54-inch diameter steel outlet pipe, located through the dam near the left abutment at an elevation of approximately 910 feet. The outlet pipe lengths would be 180 feet and 250 feet for the lower and higher dams, respectively. Releases would be controlled by a 48-inch diameter needle valve and a high pressure ring seal gate. A 54-inch diameter sluiceway pipe would be provided through the center of the dam at an elevation of 800 feet. The sluice pipe would be 310 feet in length for the lower dam and 400 feet in length for the higher dam, and would be controlled by two high pressure ring seal gates. Steel trashrack structures would be provided at the upstream ends of the outlet and sluice pipes, and access to the controls would be through chambers provided in the dam.

It was estimated that construction of a dam, either of 330 or 428 foot height, at the Hammel site would require about two years. Diversion of summer and small winter stream flows during construction would be through a 7-foot diameter concrete lined tunnel of horseshoe section located through the left abutment. Major floods would pass over a depressed section of the concrete dam. The diversion tunnel would be about 490 feet in length for the lower dam and about 560 feet in length for the higher dam. Following construction, the tunnel would be plugged at the upstream end.

Aggregate for a concrete dam could be imported to the Hammel site by truck or rail. Rail haul to within about five miles of the site is available. The aggregate could come from sources along the Santa Clara River area from 7 to 20 miles distant. After suitable testing, it might be determined that rock near the dam site is usable after crushing and screening.

It was estimated that from 210 to 320 acres of clearing would be required in the reservoir area, depending on the height of dam to be constructed. There are no improvements in the area. Approximately 170 acres are under private ownership, while the remainder of the property belongs to the Federal Government. In 1952, the cost of acquisition of private lands in the reservoir area was estimated by the Ventura County Flood Control District to be about $12,500 for both sizes of dam. Construction of an access road, approximately 2 miles in length, would be required before construction could start.

Presented in Table 77 are pertinent data with respect to the general features of the two sizes of dams and reservoirs considered at the Hammel site, as designed for cost estimating purposes. For illustrative purposes, a plan, profile, and section for the dam creating a reservoir with storage capacity of 50,000 acre-feet are shown on Plate 30, entitled "Hammel Dam on Sespe Creek".

TABLE 77

GENERAL FEATURES OF TWO SIZES OF DAM AND RESERVOIR
AT THE HAMMEL SITE ON SESPE CREEK

Concrete Gravity Dam		
Crest elevation, in feet, U.S.G.S. datum	1,125	1,223
Crest length, in feet . . .	470	810
Crest width, in feet. . . .	30	30
Height of dam, to top of spillway gates above stream bed, in feet. . . .	330	428
Freeboard, above top of spillway gates, in feet. .	5	5
Elevation of stream bed, in feet, U.S.G.S. datum. .	790	790
Volume of concrete in dam, in cubic yards	530,700	1,067,900
Reservoir		
Surface area, at top of spillway gates, in acres .	200	308
Gross storage capacity, at top of spillway gates, in acre-feet	25,000	50,000
Type of spillway.	Overpour, with gates and bucket	Overpour, with gates and bucket
Spillway discharge capacity, in second-feet . . .	90,000	90,000
Type of outlets	54-inch diameter steel pipe through dam, and 54-inch diameter steel pipe sluiceway	54-inch diameter steel pipe through dam, and 54-inch diameter steel pipe sluiceway

Presented in Table 78 is a summary comparison of capital and annual costs of the two considered sizes of dam and reservoir at the Hammel site. Also presented in Table 78 are estimated unit costs of storage capacity and net safe yields of water that would be developed by construction of the two sizes of reservoir. Yields referred to are those that would result under the uniform release method of reservoir operation with releases for maintenance of historic ground water levels. Certain of the relationships presented in Table 78 are depicted graphically on Plates 35, 36, and 37. Detailed estimates of cost for the two sizes of dam and reservoir at the Hammel site are included in Appendix C.

TABLE 78

SUMMARY OF ESTIMATED COSTS OF DAMS, RESERVOIRS, AND YIELDS OF WATER AT THE HAMMEL SITE ON SESPE CREEK

Item	Reservoir storage capacity, in acre-feet	
	25,000	50,000
Capital Costs		
Dam and reservoir	$12,890,000	$24,490,000
Cost per acre-foot of storage	516	490
Cost per acre-foot of net safe yield	3,220	2,580
Annual Costs		
Dam and reservoir	666,000	1,252,000
Cost per acre-foot of net safe yield	166	132
Cost per acre-foot of incremental net safe yield	---	107

Fillmore Dam and Reservoir. The Fillmore dam site, the lowermost of all sites considered on Sespe Creek, is located in Section 13, Township 4 North, Range 20 West, S.B.B. & M., about two miles north of the town of Fillmore and about 3.2 miles upstream from the confluence of Sespe Creek with the Santa Clara River. Stream bed elevation at the site is about 490 feet, U.S.G.S. datum. The location is such that practically complete regulation of the flow of Sespe Creek could be achieved through construction of a reservoir of sufficient size. Consideration was given to the construction of a dam and reservoir at the Fillmore site for storage of flood waters in Sespe Creek, and utilization of the waters so conserved in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit.

The drainage area of Sespe Creek above the Fillmore dam site comprises about 259 square miles, and produced an estimated average seasonal runoff during the base period of about 96,700 acre-feet. It was estimated that waste to the ocean of water originating above the dam site would have averaged about 77,400 acre-feet per season during the base period with the present pattern of land use and water supply development.

The Fillmore dam site was surveyed by the Ventura County Flood Control District in 1951, using instrumental methods. The map resulting from this survey is at a scale one inch equals 200 feet, with a contour interval of 2 feet on flat areas and gently sloping hill sides, and 25 feet on steep hill sides. The map extends up to an elevation of 850 feet on the right abutment and 800 feet on the left abutment. An area-capacity curve for Fillmore Reservoir, data for which were obtained from U.S.G.S. quadrangles, at a scale of 1:24,000, was provided by the Ventura County Flood Control District. Storage capacities of Fillmore Reservoir at various stages of water surface elevation taken from this curve, are given in Table 79.

TABLE 79

AREAS AND CAPACITIES OF FILLMORE RESERVOIR

Depth of water at dam, in feet	Water surface elevation U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	490	0	0
10	500	22	110
20	510	52	480
30	520	100	1,240
40	530	170	2,600
50	540	210	4,530
60	550	260	6,890
70	560	320	9,790
80	570	380	13,300
90	580	410	17,300
100	590	450	21,500
110	600	480	26,200
120	610	520	31,200
130	620	560	36,600
140	630	600	42,400
150	640	640	48,700
160	650	700	55,400
170	660	770	62,700
172	662	780	64,300
180	670	820	70,700
190	680	870	79,100
200	690	900	88,000
210	700	935	97,200
211	701	940	98,100
220	710	960	107,700
230	720	980	116,400
240	730	1,000	126,200
250	740	1,040	136,300
260	750	1,070	146,900
261	751	1,080	147,900
270	760	1,110	157,800
280	770	1,150	169,100
290	780	1,190	180,800
300	790	1,220	192,900
310	800	1,260	205,300

Based upon available geological information, including that resulting from reconnaissance examination and seismic surveys during the investigation, it was concluded that the only types of dam possible at the Fillmore site are earthfill or rockfill structures. Furthermore, the construction of such types of dam would only be feasible if further tests of the stability of the right abutment should give necessary results.

Geology in the area of the Fillmore dam site has been mapped by Kew, Hoots, Eldridge for the oil industry, and the geology of the more recent water-bearing deposits was reported on by Gentry in Division of Water Resources Bulletin No. 46. Much detailed work has been done on the older rock formations since publication of the aforementioned papers, but little has been done with the more recent water-bearing materials.

Rocks exposed on the left abutment and in the left channel section a few hurdred feet downstream from the axis of the dam are Miocene Modelo shales and siltstones, generally fine grained, thin bedded, and laced with slip or shear zones and gouge streaks. Material exposed over the wide gently sloping terrace between the shale and the right abutment appears to be old deposits of sand, gravel, and boulders, with a relatively thin soil cover. At the stream channel, a depth of from 15 to 20 feet of boulders and smaller fragments, and about 4 feet of overlying soil is visible at the edge of this terrace.

The right abutment, whose base is at an elevation about 100 feet above the stream bed, appears to be a portion or remnant of an old alluvial cone or terrace deposit, now considerably dissected. The materials comprising this abutment are generally unstratified, unsorted, and poorly consolidated. They consist of varying proportions of sandy and clayey material, containing rock fragments which vary in size to large subangular blocks. The upper surface of the abutment is relatively even and gently sloping, and supports a light brush and tree growth. The steep dissected side slopes have a heavy brush cover.

The San Cayetano thrust fault has been mapped by Kew and others, extending in a north-northwesterly direction near the center of the channel section at the Fillmore dam axis. The northeast limb of the fault is upthrown. If this mapping is correct, the Modelo shale of the left abutment does not extend to the west (right) of the fault, except at great depth.

Two 8-inch cable tool holes were drilled, and a seismic profile run by the Division of Water Resources to establish the presence or absence of the Modelo shale at shallow depths on the low right abutment terrace, and, if the shale was found to be absent, to determine whether other impervious materials suitable for a dam foundation were present. One hole was located on the sloping terrace near the base of the right abutment at an elevation about 35 feet above stream bed, and the other in the lower part of the sloping terrace at a site about 350 feet from the edge of the channel section, at an elevation about 20 feet above stream bed. The hole near the right abutment was drilled to a 60-foot depth, and the lower hole to a 67-foot depth. Neither of these holes encountered shale or comparable material, nor did they strike water table. They did, however, strike fairly tight silt and silty clay, commonly containing sand and pebbles, almost continuously from a few feet below the surface to the bottom of the holes. This material is apparently terrace material similar to that composing the right abutment.

Seismic profiles were run by the Division of Water Resources, from the shale exposed in the channel section upstream to the axis, where the shale is under the gravels, and thence along the axis of the dam toward the right abutment as far as Grand Avenue. Another profile was run a short distance along Grand Avenue both upstream and downstream from the axis. This survey indicated the seismic velocity in the shale at the ground surface to be about 6,000 feet per second. Materials with velocity up to 7,000 feet per second (probably saturated shale) were found underlying the gravels of the active channel section. Material of similar velocity was found to extend to Grand Avenue along the profile line, but at increasingly greater depths. Depths varied from zero in the channel section t about 100 feet at the hole in the lower terrace, and 200 feet at the intersection of the axis and Grand Avenue. This material may be saturated tight terrace material similar to that encountered in the drill holes. There are indications that this high velocity material may be pitching off in a downstream direction. A material having still higher velocity, on the order of 11,000 feet per second, was picked up toward the right abutment from the stream channel. This appears to have the seismic velocity of a consolidated sandstone, and may represent a small portion or fault sliver of Pico sandstone such as is exposed at the surface about a mile upstream.

About one-half mile downstream from the Fillmore dam site, an oil company has drilled through approximately 12,000 feet of more recent sediments without encountering the Modelo shale. Evidence from this well, from the shallow drill holes on the axis of the dam, and from the seismic profiles indicates that the mapped location of the San Cayetano thrust fault in the channel section near the left abutment is correct, and that there is little chance of finding the Modelo shale at any reasonable depth at the dam site to the right of the fault.

Records of runoff at the Fillmore dam site are not available. However, runoff at the site was estimated as equal to 102 per cent of the measured runoff at the U.S.G.S. stream gaging station on Sespe Creek near Fillmore, adjusted for diversions made upstream from the gaging station by the Fillmore Irrigation Company. The estimates were based on the ratio of respective watershed areas above the dam site and gaging station. The estimated monthly runoff of Sespe Creek at the Fillmore dam site during the base period is presented in Table 80.

TABLE 80

ESTIMATED MONTHLY RUNOFF OF SESPE CREEK AT FILLMORE DAM SITE DURING BASE PERIOD

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	3,020	660	14,950	11,390	64,710	52,930	15,750	5,720	2,740	1,200	790	590	174,450
1937-38	610	790	3,980	2,210	70,330	134,980	14,180	7,510	4,020	2,390	1,430	1,290	243,720
1938-39	1,280	1,390	9,270	8,130	4,920	10,840	3,850	2,090	1,190	620	440	3,090	47,110
1939-40	1,020	920	1,090	3,250	15,210	5,280	3,230	1,550	680	350	300	270	33,150
1940-41	410	400	18,010	34,170	101,980	126,890	68,490	17,140	7,270	3,950	2,520	1,890	383,120
1941-42	1,910	2,040	10,050	5,690	3,630	3,730	9,870	3,170	1,440	660	460	430	43,080
1942-43	440	590	770	59,750	39,600	55,080	8,480	4,190	2,280	1,260	840	630	173,910
1943-44	680	860	8,350	4,950	38,710	62,720	15,400	6,670	3,740	1,940	1,130	830	145,980
1944-45	940	6,760	2,140	1,750	21,710	10,940	5,770	2,420	1,370	720	520	390	55,430
1945-46	510	800	18,070	2,950	3,100	19,820	14,780	3,180	1,190	590	400	340	65,730
1946-47	410	11,540	19,020	5,710	2,910	2,410	1,760	990	590	330	290	280	46,240
1947-48	300	310	490	530	840	1,770	1,920	1,010	510	220	190	170	8,260
1948-49	210	240	540	740	760	3,870	1,340	740	330	180	160	140	9,250
1949-50	180	440	1,860	3,390	6,080	1,700	1,830	810	350	210	170	200	17,220
1950-51	200	250	260	370	460	740	520	340	150	100	90	100	3,580
Average seasonal runoff, 1936-37 through 1950-51													96,680

Based on the estimates of runoff, monthly studies of operation of Fillmore Reservoir during the base period were made for three sizes of reservoir, of 64,000 acre-feet, 98,000 acre-feet, and 148,000 acre-feet storage capacity, under both the uniform release and rapid release methods of operation. In all of the studies, an allowance was made for reduction in effective reservoir storage capacity due to sedimentation, in the amount of 12,000 acre-feet. This amount represents the estimated loss after about 20 years of operation. An estimated average net seasonal depth of evaporation from the reservoir water surface of 1.70 feet, distributed in accordance with the following tabulation, was employed in the operation studies.

Month	Net evaporation, in feet of depth	Month	Net evaporation, in feet of depth
October	0.15	April	0.15
November	0.06	May	0.19
December	0.04	June	0.21
January	0.04	July	0.25
February	0.05	August	0.25
March	0.10	September	0.21
		TOTAL	1.70

The estimated values of net safe seasonal yield that would be obtained under both the uniform release and rapid release operating criteria are presented in Table 81. The relationship between reservoir storage capacity and net safe seasonal yield, with Fillmore Reservoir operated by the uniform release method with releases for maintenance of water levels in Fillmore and Santa Paula Basins, is depicted graphically on Plate 36.

TABLE 81

ESTIMATED NET SAFE SEASONAL YIELDS OF FILLMORE RESERVOIR

(In acre-feet)

Reservoir storage capacity	Uniform release operation		Rapid release operation	
	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, with releases for maintenance of ground water levels	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, without releases for maintenance of ground water levels	Available within Santa Clara River Hydrologic Unit	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits
64,000	12,500	15,000	12,700	10,500
98,000	20,000	24,000	20,300	13,500
148,000	27,000	32,000	27,500	16,000

Although the Fillmore reservoir site affords an opportunity for the greatest degree of control of runoff from the Sespe Creek watershed, to achieve such control an earth or rock fill dam of considerable length would be required. Since suitable foundation material was not encountered at moderate depths, it was concluded that to extend the impervious section of a suitable dam to the underlying shale bedrock would not be feasible. Any structure contemplated at the Fillmore dam site would necessarily be floated on the terrace material overlying bedrock, using a shallow and narrow cutoff to reduce underflow. The high degree of development prevailing in the Fillmore Reservoir area would make acquisition of the necessary lands very expensive. For these reasons, it was concluded that construction of a dam and reservoir at the Fillmore site is not feasible at the present time. Therefore, design of the dam and appurtenant features, and estimates of costs, were limited to those of a reconnaissance nature necessarily made to arrive at the foregoing conclusion.

Reconnaissance type cost estimates were prepared for three earth-fill dams at the Fillmore site with heights of 172 feet, 211 feet, and 261 feet from stream bed to spillway lip, creating reservoir storage capacities of 64,000 acre-feet, 98,000 acre-feet, and 148,000 acre-feet, respectively. For all heights of dam a rolled fill structure was contemplated, with upstream and downstream slopes of 3:1, and a crest width of 30 feet. An open cut spillway, including an ogee weir section and a concrete lined chute, could be constructed across the left abutment. The cost estimates were based upon a freeboard of 10 feet from spillway lip to crest of dam.

A depth of about 5 feet of weathered material in the root zone should be stripped from the right abutment under the impervious section of an earthen dam. Depths of 5 to 10 feet of terraced material should be similarly stripped from the right side terrace. Gravel and boulders to a depth of 10 feet should be removed under the impervious section from the active channel, about 60(feet in width, and a depth of about 5 feet of weathered shale and siltstone should be removed from this vicinity where it is exposed. Practically all excavated materials could be salvaged. A depth of about 12 feet of boulders and gravel should be stripped under the impervious section from the low terrace on the left abutment, plus about 2 feet of fractured shale beneath these gravels and boulders. The bouldery fill should be similarly stripped from the upper terrace to a depth of about 20 feet. At least 70 per cent of this material would be recoverable for impervious section.

Materials taken from the terrace deposit upstream from the right abutment appear to be the main source of materials for an impervious section nea the Fillmore dam site. About one-third of this material would have to be screene to eliminate the boulders and large blocks, which could then be salvaged for blanket material. Compaction and permeability tests indicated that careful selection, and possible blending of materials, would be necessary to construct a suitable impervious fill from the terrace deposit. In addition to the materia of the right abutment, it is possible that the soil and underlying sediments of the low terrace between the right abutment and the channel section might be usable. Also, the material of the upstream terrace on the left abutment appears t be similar to that tested from the right abutment, and should be usable. Removal of trees, stumps, and roots might present a problem as to the suitability of this material. Pervious fill material is available in limited quantities in the channel of Sespe Creek both upstream and downstream from the axis of the dam, and large quantities of similar material could be obtained from the Santa Clara River channel about three miles downstream. The nearest heavy rock or riprap material available appear to be hard red Sespe sandstone located about three miles upstream near the Hammel Dam site.

The Fillmore reservoir area, to a distance of about 1.5 miles upstream from the dam site, contains several hundred acres of mature orange groves and suburban residences, and a number of oil rights and leases. Two county roads would be flooded and depending on the size of dam, several existing oil wells might possibly be inundated. A preliminary appraisal report prepared by the Ventura County Flood Control District in September 1951, estimated that the fair market value of property that would have to be acquired for construction of Fillmore Dam and Reservoir was $2,155,600.

Presented in Table 82 is a summary comparison of capital and annual costs of the three considered sizes of dam and reservoir at the Fillmore site. Also presented in Table 82 are estimated unit costs of storage capacity and net safe yields of water that would be developed by construction of the three sizes of reservoir. Yields referred to are those that would result under the uniform release method of reservoir operation with releases for maintenance of historic ground water levels. It is emphasized that the estimated costs are of a reconnaissance nature.

TABLE 82

SUMMARY OF ESTIMATED COSTS OF DAMS, RESERVOIRS, AND YIELDS OF WATER AT FILLMORE SITE ON SESPE CREEK

Item	Reservoir storage capacity, in acre-feet		
	64,000	98,000	148,000
Capital Costs			
Dam and reservoir	$18,966,000	$28,352,000	$44,680,000
Cost per acre-foot of storage	296	289	302
Cost per acre-foot of net safe yield	1,520	1,420	1,650
Annual Costs			
Dam and reservoir	968,000	1,445,000	2,273,000
Cost per acre-foot of net safe yield	77	72	84
Cost per acre-foot of incremental net safe yield	---	64	118

Upper Blue Point Dam and Reservoir. The Upper Blue Point dam site is ocated on Piru Creek in Section 10, Township 5 North, Range 18 West, S.B.B.&M., ome ten miles upstream from the confluence of Piru Creek and the Santa Clara iver. Stream bed elevation at the site is about 1,090 feet, U.S.G.S. datum. he drainage area of Piru Creek above the Upper Blue Point dam site comprises bout 370 square miles, and produced an estimated average seasonal runoff during he base period of about 48,700 acre-feet. It was estimated that waste to the cean of water originating above the dam site would have averaged about 32,800 cre-feet per season during the base period with the present pattern of land use nd water supply development.

Consideration was given to the construction of a dam and reservoir at he Upper Blue Point site as one of the several possible alternative locations 'or terminal storage of water imported from the Sacramento-San Joaquin Delta. 'his reservoir would regulate such water released from the southern California liversion conduit of the Feather River Project at a point near Quail Lake. The eleased water would flow through conduits and down natural stream channels, itilizing power drops for the generation of hydroelectric power, en route to Jpper Blue Point Reservoir. In the reservoir, the water would be available to ieet ultimate supplemental water requirements throughout Ventura County. Consi- leration was also given to use of Upper Blue Point Reservoir for storage of flood vaters of Piru Creek, and utilization of the waters so conserved in the Calleguas- Conejo Hydrologic Unit and in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit.

The Upper Blue Point Reservoir area was mapped in 1951 by Fairchild Aerial Surveys, Incorporated, using photogrammetric methods, for the Ventura County Flood Control District, Zone 2. The resulting map is at a scale of 1 inch equals 400 feet, with a 10-foot contour interval. An enlargement of the reservoir map in the vicinity of the dam site, to a scale of 1 inch equals 100 feet, was used for design of the dam and cost estimating purposes. Data on reservoir areas and capacities for various heights of dam were furnished by the Ventura County Flood Control District, and were based upon the aforementioned map of the reservoir area. Storage capacities of Upper Blue Point Reservoir at various stages of water surface elevation are given in Table 83.

TABLE 83

AREAS AND CAPACITIES OF UPPER BLUE POINT RESERVOIR

Depth of water at dam, in feet	Water surface elevation U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	1,090	0	0
10	1,100	15	78
20	1,110	23	270
40	1,130	68	1,170
60	1,150	140	3,220
80	1,170	180	6,400
100	1,190	230	10,500
110	1,200	250	12,900
130	1,220	300	18,400
150	1,240	350	24,900
160	1,250	380	28,500
170	1,260	410	32,400
190	1,280	490	41,400
205	1,295	540	50,000
210	1,300	560	51,900
230	1,320	590	63,500
260	1,350	750	83,700
280	1,370	820	99,500
310	1,400	930	125,800

As a result of preliminary geological reconnaissance, it was concluded that an earthfill dam of moderate height is the most feasible at the Upper Blue Point site, and that a high earth or rockfill dam or a masonry dam would be of doubtful feasibility. No geologic work at this site is known, other than the preliminary reconnaissance made in 1952 by geologists of the Division of Water Resources.

The Upper Blue Point site is located at a constriction in the canyon of Piru Creek. The rock includes light brown sandstone, varying from massive to thin bedded, and some shale. Massive sandstones are very prominent on the right abutment, whereas thinner bedded sandstones are prominent on the left abutment, although some massive rock is there also. A few beds of shale appear, particularly on the left abutment. Concentrations of ferruginous material approaching concretions appear in numerous places in the sandstones.

The left abutment is a fairly narrow nose falling back sharply downstream and somewhat less sharply upstream. The strata on the left abutment are overturned. They strike approximately across the channel and dip very steeply upstream. A similar attitude occurs in the upstream portion of the right abutment. However, south of a fault, which extends down the ravine opposite the approximate center of the left abutment face, the strike is cross-channel, and the strata dip downstream and toward the left abutment at a much gentler angle. The aforementioned fault trends southeasterly from the ravine on the right abutment, crosses the channel section, and probably lies just south of the left abutment face. Farther east, strong evidence of this fault appears in disturbed beds in the walls of the canyon extending eastward south of the left abutment face.

The sandstones on the left abutment are cut by a great number of fracture planes, trending in many directions. The fracture planes have been mostly re-cemented with limonitic material. The sandstones on the right abutment south of the fault appear to have been much less fractured, perhaps because of their massive nature. North of the fault on the right abutment, fracturing of the rocks is similar to that on the left abutment. Open joints are much more numerous on the left abutment, and on the right abutment north of the fault, than on the right abutment south of the fault.

Records of runoff at the Upper Blue Point dam site are not available. However, runoff at the site was estimated as equal to 85 per cent of the measured runoff at the U.S.G.S. stream gaging station on Piru Creek near Piru. The estimates were based on the ratio of respective watershed areas above the dam site and gaging station. The estimated monthly runoff of Piru Creek at the Upper Blue Point site during the base period is presented in Table 84. It may be noted that runoff at the Blue Point site, about 1,700 feet downstream was assumed to be the same as that at the Upper Blue Point site.

TABLE 84

ESTIMATED MONTHLY RUNOFF OF PIRU CREEK AT UPPER BLUE POINT AND BLUE POINT DAM SITES DURING BASE PERIOD

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	660	160	3,230	2,840	20,010	18,670	9,370	2,830	930	280	120	120	59,220
1937-38	150	210	1,420	990	17,770	70,850	8,610	4,190	2,130	1,160	1,090	880	109,450
1938-39	830	900	9,060	4,120	2,980	6,290	2,910	1,520	590	270	200	2,810	32,480
1939-40	780	700	810	1,780	6,800	2,540	1,790	780	290	90	50	90	16,500
1940-41	320	300	5,190	8,040	47,250	66,910	37,260	15,480	5,670	2,780	1,720	1,440	192,360
1941-42	1,660	1,750	5,700	4,100	2,580	2,620	4,820	2,160	940	450	290	300	27,370
1942-43	410	550	800	26,820	16,370	28,990	6,440	2,970	1,490	810	520	460	86,630
1943-44	640	830	4,820	3,250	23,050	39,170	17,480	9,440	3,820	1,990	1,060	880	106,430
1944-45	1,150	3,530	2,110	2,090	7,820	4,920	3,660	1,840	1,060	370	350	320	29,220
1945-46	780	900	5,920	1,840	2,070	5,100	7,330	1,790	750	440	300	260	27,480
1946-47	470	4,870	9,730	3,180	1,770	1,540	1,220	730	290	130	90	110	24,130
1947-48	160	400	660	570	780	1,190	970	490	220	70	60	70	5,640
1948-49	70	110	630	650	710	1,710	660	280	110	90	60	50	5,130
1949-50	50	120	780	1,160	1,940	900	730	310	50	50	50	50	6,190
1950-51	60	110	130	270	330	440	220	190	120	90	40	50	2,050
Average seasonal runoff, 1936-37 through 1950-51													48,690

It was estimated that a reservoir storage capacity of approximately 50,000 acre-feet would be necessary for terminal storage and regulation of water imported from facilities of the Feather River Project. To determine the safe yield of Upper Blue Point Reservoir with this storage capacity, if used for conservation of Piru Creek flood waters, monthly studies of operation during the base period were made under both the uniform release and rapid release methods of operation. The studies were based on the estimates of runoff of Piru Creek. An allowance was made for reduction in effective reservoir storage capacity due to sedimentation, in the amount of 12,000 acre-feet. This amount represents the estimated loss after about 20 years of operation. An estimated average net seasonal depth of evaporation from the reservoir water surface of 2.20 feet, distributed monthly in accordance with the following tabulation, was employed in the operation studies:

Month	Net evaporation, in feet of depth	Month	Net evaporation, in feet of depth
October	0.19	April	0.19
November	0.08	May	0.24
December	0.05	June	0.27
January	0.05	July	0.33
February	0.06	August	0.33
March	0.13	September	0.28
		TOTAL	2.20

The operation studies indicated that under the uniform release method of operation a net safe seasonal yield of 6,500 acre-feet would have been available to the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, with sufficient reservoir releases to have maintained historical ground water levels in affected basins. Without such releases for maintenance of ground water levels, the net safe yield would have increased to an estimated 9,300 acre-feet per season. Under the rapid release method of operation, a net safe yield of 6,700 acre-feet per season would have been available within the Santa Clara River

Hydrologic Unit. However, under this method of operation only 4,500 acre-feet per season of net safe yield would have been available to the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits.

Estimates of cost were prepared for a dam at the Upper Blue Point site with a height of 205 feet from stream bed to spillway lip, creating reservoir storage capacity of 50,000 acre-feet. The dam would be a rolled earth-fill structure, comprised of an impervious core of select earth material, and upstream and downstream sections of pervious free-draining material. Both upstream and downstream slopes of the dam would be 3:1, and the impervious section would have slopes of 1:1. The crest width of the dam would be 30 feet, comprised of a 10-foot width for the impervious core, and 10-foot widths each for the upstream and downstream pervious sections. The upstream face of the dam would be protected against wave action by rock riprap placed to a depth of 3 feet normal to the slope.

In the cost estimates, it was assumed that depths of about 4 feet of soil plus 4 feet of fractured rock would be stripped beneath the impervious section on the right abutment. Under the impervious section in the channel, a depth of about 60 feet of sand and gravel, including terrace material, would be stripped, and the exposed rock would be shaped. A depth of about 2 feet of soil plus 6 feet of fractured and weathered rock, including alluvial material, would be stripped from the left abutment. A prominent thin rock cliff at the southern end of the left abutment face might have to be removed, at least in part which removal was not included in the cost estimates. Further exploratory work and examination during construction would be required to determine the amount of stripping required on this cliff. Stripping under the pervious sections of the dam was assumed to be a nominal depth of 2 feet of loose surface material and vegetation.

Earth materials considered suitable for the impervious section of the dam occur in terraces both upstream and downstream from the site. Pervious material is available in the channel and in nearby sandy terraces. An estimated 60 per cent of the material stripped from the right abutment, nearly 100 per cent of that removed from the channel, and about 70 per cent of the material stripped from the left abutment could be used in the pervious section. The nearest source of riprap is a deposit of granite about three miles air line to the northeast of the dam site. It was assumed that compaction of the impervious section of the dam would be effected by either sheepsfoot tampers or pneumatic rollers, and that pneumatic rollers would be used to compact the pervious sections. It was also assumed that moderate grouting would be necessary to prevent minor leakage in the foundation and abutments.

The spillway considered would have a discharge capacity of 100,000 second feet, which is the estimated peak discharge of a once in 1,000-year flood. The spillway was designed as a concrete-lined overpour chute, with an ogee weir control section. The spillway weir and channel would be excavated through the thin left abutment ridge, and would discharge into Piru Creek downstream from the dam. Depth of water above the spillway lip at design discharge capacity would be 20 feet, and an additional 5 feet of residual freeboard would be provided.

It was estimated that the Upper Blue Point Dam would require about two years for construction. A 20-foot diameter concrete lined tunnel of horseshoe section, 1,250 feet in length, was included in the estimate to permit the diversion of Piru Creek waters during the construction period. The tunnel would be constructed through the left abutment of the dam.

After completion of the dam, the diversion tunnel would be used for the outlet from the reservoir. A concrete plug would be placed in the upstream end of the tunnel, and a 72-inch diameter steel pipe would be placed through this plug, extending to a circular reinforced concrete outlet tower located in the reservoir. Water would enter the tower through four 36-inch diameter inlet valves. The outlet pipe would be supported on ring girders through the tunnel and would terminate in a control house, where a bifurcation structure would be located to permit the discharge of water to either Piru Creek or a proposed conduit. The downstream releases would be controlled by a 48-inch diameter Howell-Bunger valve, and a 48-inch diameter needle valve would control releases to the conduit.

The dam site and a portion of the land in the Upper Blue Point reservoir area are privately owned, while the remainder of the reservoir area belongs to the Federal Government and is a part of the Los Padres National Forest. Cost of acquisition of the private lands was estimated by the Ventura County Flood Control District in 1952 to be about $33,300. There are no improvements which would have to be acquired or relocated. Field examination of the reservoir area indicated that approximately 640 acres of minor clearing would be required. Prior to construction of the dam, an estimated 1.5 miles of access road would have to be constructed, to replace an existing low standard road.

Presented in Table 85 are pertinent data with respect to general features of the dam and reservoir considered at the Upper Blue Point site, as designed for cost estimating purposes. For illustrative purposes, a plan, profile and section of the dam creating a reservoir with storage capacity of 50,000 acre-feet are shown on Plate 31, entitled "Upper Blue Point Dam on Piru Creek".

TABLE 85

GENERAL FEATURES OF DAM AND RESERVOIR AT THE UPPER BLUE POINT SITE ON PIRU CREEK, WITH 50,000 ACRE-FOOT STORAGE CAPACITY

Earthfill Dam	
Crest elevation, in feet, U.S.G.S. datum	1,320
Crest length, in feet	1,110
Crest width, in feet	30
Height, spillway lip above stream bed, in feet	205
Side slopes, upstream and downstream	3:1
Freeboard, above spillway lip, in feet	25
Elevation of stream bed, in feet, U.S.G.S. datum	1,090
Volume of fill, in cubic yards	4,986,000
Reservoir	
Surface area at spillway lip, in acres	542
Gross storage capacity at spillway lip, in acre-feet	50,000
Type of spillway	Ogee weir and concrete lined chute
Spillway discharge capacity, in second-feet	100,000
Type of outlet	Concrete tower, and 72-inch diameter steel pipe through diversion tunnel

Presented in Table 86 is a summary of capital and annual costs of a dam and reservoir at the Upper Blue Point site, to create 50,000 acre-feet of storage capacity. Also presented are estimated unit costs of storage capacity and net safe yield of water. The yield referred to is that which would result under the uniform release method of reservoir operation with releases for maintenance of historic ground water levels. Detailed estimates of cost of the dam and reservoir are included in Appendix C.

TABLE 86

SUMMARY OF ESTIMATED COSTS OF DAM, RESERVOIR, AND YIELD OF WATER AT THE UPPER BLUE POINT SITE ON PIRU CREEK WITH 50,000 ACRE-FOOT STORAGE CAPACITY

Capital Costs	
Dam and reservoir	$8,530,000
Cost per acre-foot of storage	170
Cost per acre-foot of net safe yield	1,310
Annual Costs	
Dam and reservoir	438,000
Cost per acre-foot of net safe yield	67

Blue Point Dam and Reservoir. The Blue Point dam site is located on Piru Creek in Section 10, Township 5 North, Range 18 West, S.B.B. & M., some ten miles upstream from the confluence of Piru Creek and the Santa Clara River, and approximately 1,700 feet downstream from the Upper Blue Point site. Stream bed elevation at the site is about 1,065 feet, U.S.G.S. datum. The drainage area of Piru Creek above the dam site comprises about 371 square miles, and produced an estimated average seasonal runoff during the base period of about 48,700 acre-feet. It was estimated that waste to the ocean of water originating above the dam site would have averaged about 32,800 acre-feet per season during the base period with the present pattern of land use and water supply development.

Consideration was given to the construction of a dam and reservoir at the Blue Point site as one of several possible alternative locations for terminal storage of water imported from the Sacramento-San Joaquin Delta with facilities of the Feather River Project, as described in connection with Upper Blue Point Reservoir in the preceding section. In Blue Point Reservoir, the imported water would be available to meet ultimate supplemental water requirements throughout Ventura County. Consideration was also given to use of Blue Point Reservoir for storage of flood waters of Piru Creek, and utilization of the waters so conserved in the Calleguas-Conejo Hydrologic Unit and in the Oxnard Forebay, Oxnard Plain and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit.

The Blue Point dam site and reservoir area were mapped in 1951 by Fairchild Aerial Survey, Inc., using photogrammetric methods for the Ventura County Flood Control District, Zone 2. The dam site was mapped up to an elevation of 1,700 feet, at a scale of one inch equals 100 feet, with a contour interval of 5 feet. The reservoir area was mapped up to an elevation of 1,250 feet, at a scale of one inch equals 400-feet, with a contour interval of 10 feet. Data on reservoir areas and capacities for various heights of dam were furnished by Ventura County Flood Control District, and were based on the aforementioned map of the reservoir area. Storage capacities of Blue Point Reservoir at various stages of water surface elevation are given in Table 87.

TABLE 87

AREAS AND CAPACITIES OF BLUE POINT RESERVOIR

Depth of water at dam, in feet	Water surface elevation U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	1,065	0	0
5	1,070	1	3
15	1,080	5	32
25	1,090	11	110
35	1,100	33	330
45	1,110	45	720
65	1,130	98	2,150
85	1,150	170	4,870
105	1,170	220	8,830
125	1,190	270	13,800
135	1,200	300	16,700
155	1,220	350	23,200
175	1,240	410	30,800
185	1,250	440	35,100
195	1,260	480	39,700
210	1,275	540	48,000
215	1,280	560	50,000
235	1,300	640	62,000
255	1,320	730	75,600
285	1,350	850	99,300
305	1,370	920	117,000

Geology of the region at the Blue Point dam site has been studied by Dr. Charles P. Berkey, Paul F. Kerr, Hyde Forbes, and Chester Marliave, and is described in Division of Water Resources Board Bulletin No. 46, published in 1933. The dam site has been explored by trenching on both abutments and by test hole drilling. Five holes were drilled in the stream bed, and four of these penetrated the stream gravels and were continued into bedrock. One hole was bored vertically into the right abutment at an elevation about 160 feet above the stream bed. The following is quoted from Bulletin No. 46 and was taken from a report by Chester Marliave:

"It is believed that on account of foundation conditions, only a flexible type of dam with a broad base should be constructed at this site. No good rock for such type of dam is available in the immediate vicinity but material for an earth fill is found just below the dam site. The earth fill type was therefore selected as the most suitable for this reservoir

- - -

"The region in the vicinity of the dam site is composed entirely of Tertiary sediments which are rather poorly cemented sandstones interbedded with clay shales.

"The regional structure is somewhat complex, the sedimentary beds being considerably folded and in the vicinity of the dam site they are overturned. The intense folding which some of the beds have undergone has resulted in numerous sharp anticlines and synclines which are conspicuous along the canyon in certain places. Accompanying these crustal movements there has been considerable local faulting and slipping, but no major faults were observed in this locality.

- - -

"The bedrock at dam site shows a formational contact. The red beds of the Sespe formation merge into the light colored buff beds of the Vaqueros formation. At the contact there are several hard thin strata of calcareous sandstone about a foot in thickness that are much more resistent than the accompanying strata and act as protective layers preventing disintegration of the softer underlying beds. On account of the inclination of the beds these hard sandstone layers form projecting ridges on each side of the canyon. The softer Vaqueros sediments underlying these harder strata weather easily so that there are high vertical bluffs on their downstream side. Resting upon these hard thin sandstone strata are the red beds of the Sespe formation which are composed of alternating hard and soft layers of sandstone and shale occupying an area 700 feet upstream from the dam site. On either side of the canyon the sedimentary beds dip uniformly upstream at an angle of 50 degrees from the horizontal, while the strike is at right angles to the direction of the stream channel.

"The channel section at the dam site is about 175 feet wide at the constriction of bluffs and somewhat wider along the axis of the dam site. The drill holes put down through the gravels show that bedrock under the stream bed lies close to 90 feet below the surface. - - - The material encountered in these holes where bedrock was reached is the same as that disclosed on the abutments of the dam site.

"There appears to be a minor fault running along the stream bed under the dam site. - - - The straight uniform channel of the stream for a distance of 6,000 feet below the dam site is indicative of a fault, but its continuation upstream is not in evidence although the fault may merge into one of the intense folds.- - -

. . . .

"The main portion of the left end of the dam should be confined to the small depression upstream from the prominent outcropping rib of harder rock. Two minor faults occur across this abutment within the limits of the dam site. The sediments of the left abutment dip uniformly upstream in a monoclinal structure across the site. There is a large amount of talus material scattered along the bottom of the draw over which the proposed dam would rest. All of this loose material would have to be removed before any type of dam could be built at this site.

"The right end of the dam should rest in the depression upstream from the prominent outcropped rib of the rock on that side of the canyon. Within the immediate limits of the dam site, the structure at this abutment is monoclinal but the upper portion merges into an inclined syncline which is badly distorted and faulted. One fault traverses the abutment in a vertical direction at an elevation of about 140 feet above the stream bed and has probably crushed the bedrock to a considerable extent. There is a large amount of talus material along the lower slope of this abutment resulting from the weathering of the Sespe formation higher up on the slope of the hill."

Records of runoff at the Blue Point dam site are not available. Runoff at the site was assumed to be the same as at the Upper Blue Point Site, about 1,700 feet upstream. The method of estimating runoff at the Blue Point dam sites is described in the preceding section, and the estimated monthly flow of Piru Creek at the sites during the base period is presented in Table 84.

It was estimated that a reservoir storage capacity of approximately 50,000 acre-feet would be necessary for terminal storage and regulation of water imported from facilities of the Feather River Project. To determine the safe yield of Blue Point Reservoir with this storage capacity, if used for conservation of Piru Creek flood waters, monthly studies of operation during the base period were made under both the uniform release and rapid release methods of operation. The studies were identical with those described in the previous section for Upper Blue Point Reservoir, and indicated that under the uniform release method of operation a net safe seasonal yield of 6,500 acre-feet would have been available to the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, with sufficient reservoir releases to have maintained historical ground water levels in affected basins. Without such releases for maintenance of ground water levels, the net safe yield would have increased an estimated 9,300 acre-feet per season. Under the rapid release method of operation, a net safe yield of 6,700 acre-feet per season would have been available within the Santa Clara River Hydrologic Unit. However, under this method of operation only 4,500 acre-feet per season of net safe yield would have been available to the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits.

Estimates of cost were prepared for a dam at the Blue Point Site with a height of 215 feet from streambed to spillway lip, creating reservoir storage capacity of 50,000 acre-feet. The dam would be a rolled earthfill structure, comprised of an earth core of select earth material, and upstream and downstream sections of pervious free-draining material. Both upstream and downstream slopes of the dam would be 3:1, and the impervious section would have slopes of 1:1. The crest width of the dam would be 30 feet, comprised of a 10-foot width for the impervious core, and 10-foot widths each for the upstream and downstream pervious sections. The upstream face of the dam would be protected against wave action by rock riprap placed to a depth of 3 feet normal to the slope.

In the cost estimates, it was assumed that depths of from 15 to 90 feet of gravel and boulders would be stripped from the stream channel under the impervious section of the dam, and that the exposed rock would be shaped. Under the impervious section on the abutments, depths of from 5 to 30 feet of soil and decomposed rock would be removed. For the pervious sections of the dam, it was assumed that no stripping would be necessary, except for a nominal depth of 2 feet of loose surface material and vegetation.

Earth materials considered suitable for the impervious section of the dam occur in terraces along Piru Creek near the site, and could be obtained from borrow pits located on both sides of the canyon about 1,500 feet downstream from the dam. The outer pervious zones of the dam would consist of stream bed sands and gravels, and materials salvaged from stripping operations. The nearest source of riprap is a deposit of granite about 3.5 miles to the northeast of the dam site. It was assumed that compaction of the impervious section of the dam would be effected by either sheepsfoot tampers or pneumatic rollers, and that penumatic rollers would be used to compact the pervious sections. It was also assumed that moderate grouting would be necessary to prevent minor leakage in the foundation and abutments.

The spillway considered would have a discharge capacity of 100,000 second-feet, which is the estimated peak discharge of a once in 1000-year flood. Because of the steep canyon walls on both abutments, any type of spillway placed across them would be extremely costly. For this reason, the spillway for Blue Point Reservoir was designed as a concrete lined tunnel, located through the left abutment. The control structure would consist of a concrete curved ogee weir, 310 feet in length. From the weir, the concrete training walls of the spillway would converge to a width of about 95 feet in a distance of 100 feet. At this point, a second ogee weir would control the flow entering the tunnel. The tunnel would be 1,075 feet in length, and would discharge into a concrete lined channel, 100 feet in length and thence into the channel of Piru Creek several hundred feet downstream from the dam. The spillway was designed to discharge 100,000 second-feet with the tunnel filled to 0.70 depth. With the flow at 0.93 depth, the spillway would discharge 130,000 second-feet, and flowing full it would discharge 120,000 second-feet.

It was estimated that Blue Point Dam would require about two years for construction. Assuming that the spillway tunnel, outlet conduit, impervious excavation, and embankment below the stream bed could be completed in one season, winter flood flows could be passed over the completed embankment without undue harm. The remaining embankment of less than 3,000,000 cubic yards could be placed in the next construction season, thus eliminating the necessity for a large diversion tunnel. It was further assumed that small summer flows could be diverted through the outlet conduit.

The outlet works would consist of a circular reinforced concrete tower located in the reservoir, and a 72-inch diameter steel pipe, 1,450 feet in length, placed in a trench excavated in rock beneath the dam near the right abutment and encased in concrete. Releases from the reservoir would be controlled by four 36-inch diameter gate valves in the outlet tower. The outlet pipe would terminate in a control house downstream from the dam, where a bifurcation structure would be located, permitting the discharge of water to either Piru Creek or a proposed conduit. The downstream releases would be controlled by a 48-inch diameter Howell-Bunger valve, and a 48-inch needle valve would control releases to the conduit.

A portion of the land in the Blue Point reservoir area is privately owned, while the dam site and the remainder of the reservoir area belongs to the Federal Government and is a part of the Los Padres National Forest. Cost of acquisition of the private lands was estimated by the Ventura County Flood Control District in 1952 to be about $33,300. There are no improvements which would have to be acquired or relocated. Field examination of the reservoir area indicated that approximately 640 acres of minor clearing would be required. Prior to construction of the dam, an estimated 1.2 miles of access road would have to be constructed, to replace an existing low standard road.

Presented in Table 88 are pertinent data with respect to general features of the dam and reservoir considered at the Blue Point Site, as designed for cost estimating purposes. For illustrative purposes, a plan, profile, and section of the dam creating a reservoir with storage capacity of 50,000 acre-feet are shown on Plate 32, entitled "Blue Point Dam on Piru Creek."

TABLE 88

GENERAL FEATURES OF DAM AND RESERVOIR AT THE BLUE POINT SITE ON PIRU CREEK, WITH 50,000 ACRE-FOOT STORAGE CAPACITY

Earthfill Dam	
Crest elevation, in feet, U.S.G.S. datum	1,305
Crest length, in feet	830
Crest width, in feet	30
Height, spillway lip above stream bed, in feet	215
Side slopes, upstream and downstream	3:1
Freeboard, above spillway lip, in feet	25
Elevation of stream bed, in feet, U.S.G.S. datum	1,065
Volume of fill, in cubic yards	3,497,700
Reservoir	
Surface area at spillway lip, in acres	536
Gross storage capacity at spillway lip, in acre-feet	50,000
Type of spillway	Tunnel
Spillway discharge capacity in second-feet	100,000
Type of outlet	Concrete tower, and 72-inch diameter steel pipe beneath dam.

Presented in Table 89 is a summary of capital and annual costs of a dam and reservoir at the Blue Point Site, to create 50,000 acre-feet of storage capacity. Also presented are estimated unit costs of storage capacity and net safe yield of water. The yield referred to is that which would result under the uniform release method of reservoir operation with releases for maintenance of historic ground water levels. Detailed estimates of cost of the dam and reservoi are included in Appendix C.

TABLE 89

SUMMARY OF ESTIMATED COSTS OF DAM, RESERVOIR, AND YIELD OF WATER AT THE BLUE POINT SITE ON PIRU CREEK, WITH 50,000 ACRE-FOOT STORAGE CAPACITY

Capital Costs	
Dam and reservoir	$8,171,000
Cost per acre-foot of storage	160
Cost per acre-foot of net safe yield	1,260
Annual Costs	
Dam and reservoir	420,000
Cost per acre-foot of net safe yield	65

Devil Canyon Dam and Reservoir. The Devil Canyon dam site is located on Piru Creek in Section 22, Township 5 North, Range 18 West, S.B.B. & M., some eight miles upstream from the confluence of Piru Creek and the Santa Clara River. Stream bed elevation at the site is about 980 feet, U.S.G.S. datum. The drainage area of Piru Creek above the Devil Canyon dam site comprises about 392 square miles, and produced an estimated average seasonal runoff during the base period of about 51,500 acre-feet. It was estimated that waste to the ocean of water originating above the dam site would have averaged about 34,700 acre-feet per season during the base period with the present pattern of land use and water supply development.

Consideration was given to the construction of a dam and reservoir at the Devil Canyon site as one of several possible alternative locations for terminal storage of water imported from the Sacramento-San Joaquin Delta with facilities of the Feather River Project, as was described in connection with Upper Blue Point Reservoir in a prior section. In Devil Canyon Reservoir, the imported water would be available to meet ultimate supplemental water requirements throughout Ventura County. Consideration was also given to use of Devil Canyon Reservoir for storage of flood waters of Piru Creek, and utilization of the waters so conserved in the Calleguas-Conejo Hydrologic Unit and in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit. Devil Canyon Reservoir, if constructed with sufficient storage capacity, could be used for joint regulation of imported water and conservation of local flood flows.

The Devil Canyon dam site and reservoir area were surveyed in 1951 by Fairchild Aerial Surveys, Inc., using aerial photogrammetric methods, for the Ventura County Flood Control District, Zone 2. The dam site was mapped up to an elevation of 1,450 feet, at a scale of one inch equals 100 feet, with a 5-foot contour interval. The reservoir area was mapped up to an elevation of 1,350 feet, at a scale of one inch equals 400 feet, with a contour interval at 10 feet. Data on storage capacities of Devil Canyon Reservoir at various stages of water surface elevation, based on the aforementioned map, were obtained from the Ventura County Flood Control District and are presented in Table 90.

TABLE 90

AREAS AND CAPACITIES OF DEVIL CANYON RESERVOIR

Depth of water at dam, in feet	Water surface elevation U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	980	0	0
10	990	7	40
20	1,000	22	18[illegible]
30	1,010	44	515
40	1,020	65	1,060
50	1,030	87	1,820
60	1,040	110	2,800
70	1,050	150	4,090
80	1,060	210	5,870
90	1,070	260	8,200
100	1,080	290	10,900
110	1,090	320	14,000
120	1,100	380	17,500
130	1,110	415	21,500
140	1,120	470	25,900
150	1,130	530	30,900
160	1,140	590	36,500
170	1,150	640	42,700
180	1,160	690	49,300
190	1,170	740	56,500
200	1,180	800	64,200
210	1,190	850	72,400
220	1,200	910	81,200
230	1,210	960	90,500
240	1,220	1,020	100,000
250	1,230	1,080	110,900
260	1,240	1,150	122,100
270	1,250	1,210	133,900
280	1,260	1,280	146,300
285	1,265	1,310	153,000
290	1,270	1,350	159,400
300	1,280	1,420	173,300
310	1,290	1,500	187,900
320	1,300	1,580	203,300
330	1,310	1,670	220,600
340	1,320	1,750	236,700
350	1,330	1,830	254,600
360	1,340	1,910	273,400
370	1,350	1,990	292,900

Based on preliminary geological reconnaissance, the Devil Canyon dam site is considered best adapted to a rolled fill type of dam of moderate height. A geologic investigation of the site was made in 1951 as a part of the current investigation. The geology of the region and dam site was previously investigated for and is described in Division of Water Resources Bulletin No. 46. The recent geologic examination considered greater heights of dam than were contemplated in Bulletin No. 46. Foundation exploration prior to 1933 included the drilling of five drill holes and the sinking of two test pits. Further exploration by the United Water Conservation District in 1952 included four drill holes.

The abutments and foundation at the Devil Canyon dam site lie in an area occupied by the Modelo formation, here exemplified by a series of thin interbedded sandstones and shales. They are not well cemented or indurated, although leaching of soluble salts to the surface has hardened some of the beds. Where naturally exposed, some of the beds are strongly weathered, but road cuts made 20 years ago in the thin shaly beds show the material to be in generally good condition. The steep slopes have a relatively thin soil cover, and support only a slight growth of grass and brush. The bedding is markedly evident, particularly on the left abutment. There is some evidence of openness along some of the joints and bedding planes of the Modelo formation near the surface, part of which may be due to solution.

Structurally, the dam site lies on the southerly limb of an east-west trending anticline, with the bed striking across the channel and dipping from 40 to 50 degrees downstream. The structure appears continuous, and no break is discernible in the channel section. The drill holes and test pits reported in Bulletin No. 46 showed the maximum depth of fill in the stream bed to be from 80 to 90 feet. The holes drilled in 1952 indicated that the depth of channel fill varied from 36 to 67 feet.

A small fault crosses the right abutment several hundred feet downstream from the axis of the dam, but it is not considered active and should present no insoluble problem. Although a large earthen dam would overlap this fault, only the downstream toe would reach it, and it is believed that only moderate additional excavation would be required. A few small seeps were noted near the base of the right abutment at the elevation of the road.

Records of runoff at the Devil Canyon dam site are not available. However, runoff at the site was estimated as equal to 90 per cent of the measured runoff at the U.S.G.S. stream gaging station on Piru Creek near Piru. The estimates were based on the ratio of respective watershed areas above the dam site and gaging station. The estimated monthly runoff of Piru Creek at the Devil Canyon dam site during the base period is presented in Table 91.

TABLE 91

ESTIMATED MONTHLY RUNOFF OF PIRU CREEK AT DEVIL CANYON DAM SITE DURING BASE PERIOD

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	700	170	3,420	3,010	21,190	19,770	9,920	3,000	980	290	130	130	62,710
1937-38	150	220	1,500	1,040	18,820	75,000	9,120	4,440	2,250	1,220	1,150	930	115,840
1938-39	880	950	9,590	4,370	3,150	6,660	3,080	1,610	620	290	210	2,970	34,380
1939-40	830	740	860	1,880	7,200	2,690	1,900	820	310	90	50	100	17,470
1940-41	340	320	5,490	8,510	50,030	70,850	39,460	16,390	6,000	2,940	1,820	1,520	203,670
1941-42	1,750	1,850	6,040	4,340	2,730	2,770	5,100	2,290	990	480	310	320	28,970
1942-43	430	580	850	28,400	17,330	30,690	6,820	3,140	1,570	860	550	490	91,710
1943-44	680	880	5,100	3,440	24,410	41,470	18,500	10,000	4,040	2,100	1,120	940	112,680
1944-45	1,210	3,740	2,230	2,210	8,280	5,210	3,880	1,950	1,130	390	370	340	30,940
1945-46	820	950	6,270	1,940	2,200	5,400	7,760	1,890	790	470	320	280	29,090
1946-47	490	5,160	10,300	3,370	1,870	1,630	1,300	770	310	140	90	110	25,540
1947-48	170	420	700	600	830	1,260	1,030	520	230	80	60	70	5,970
1948-49	70	120	670	690	750	1,810	700	300	120	100	60	50	5,440
1949-50	50	130	830	1,230	2,060	950	770	330	50	50	50	50	6,550
1950-51	60	120	140	280	350	470	230	200	130	100	40	50	2,170
Average seasonal runoff, 1936-37 through 1950-51													51,540

Based on the estimates of runoff, monthly studies of operation of Devil Canyon Reservoir during the base period were made for reservoir storage capacities of 100,000 acre-feet and 150,000 acre-feet, respectively, under both the uniform release and rapid release methods of operation, with utilization of the conserved water in the Santa Clara River Hydrologic Unit. Similar operation studies also were made for a Devil Canyon Reservoir of 150,000 acre-feet storage capacity, with utilization of the conserved water in both the Santa Clara River and Calleguas-Conejo Hydrologic Units. In these latter studies, the lower 100,000 acre-feet of reservoir storage was allocated to the Santa Clara River Hydrologic Unit, with operation under the uniform release criteria. The upper 50,000 acre-feet of reservoir storage was allocated jointly to both hydrologic units, with releases for the Santa Clara River Hydrologic Unit being under the uniform release criteria, and with releases for the Calleguas-Conejo Hydrologic Unit being as rapid as permitted by the capacity of the conduit to serve the Unit hereinafter referred to as the "Piru-Las Posas Conduit". Seven sizes of conduit were considered, varying in discharge capacity from 40 to 200 second-feet.

In all yield studies, an allowance was made for reduction in effective reservoir storage capacity due to sedimentation, in the amount of 13,000 acre-feet. This amount represents the estimated loss after 20 years of operation. An average net seasonal depth of evaporation from the reservoir water surface of 2.20 feet, distributed monthly in accordance with the following tabulation, was employed in the operation studies:

Month	Net evaporation, in feet of depth	Month	Net evaporation, in feet of depth
October	0.19	April	0.19
November	0.08	May	0.24
December	0.05	June	0.27
January	0.05	July	0.33
February	0.06	August	0.33
March	0.13	September	0.28
		TOTAL	2.20

The estimated values of net safe seasonal yield that would be obtained from Devil Canyon Reservoir under both the uniform release and rapid release operating criteria, with utilization of the conserved water in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit, are presented in Table 92. The relationship between reservoir storage capacity and net safe seasonal yield, with Devil Canyon Reservoir operated by the uniform release method with releases for maintenance of historic ground water levels is depicted graphically on Plate 36.

TABLE 92

ESTIMATED NET SAFE SEASONAL YIELDS OF DEVIL CANYON RESERVOIR IF OPERATED SOLELY FOR BENEFIT OF SANTA CLARA RIVER HYDROLOGIC UNIT

(In acre-feet)

Reservoir storage capacity	Uniform release operation		Rapid release operation	
	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, with releases for maintenance of ground water levels	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, without releases for maintenance of ground water levels	Available within Santa Clara River Hydrologic Unit	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits
100,000	15,000	19,000	15,500	10,500
150,000	22,000	27,000	22,700	15,000

The estimated seasonal amounts of water that could have been diverted to the Calleguas-Conejo Hydrologic Unit during the base period, with a Devil Canyon Reservoir of 150,000 acre-foot storage capacity operated for joint benefit of the Santa Clara River and Calleguas-Conejo Hydrologic Units, are presented in Table 93. The estimates include seasonal diversions with seven alternative sizes of Piru-Las Posas Conduit, and contemplated releases of water for the Santa Clara River Hydrologic Unit under the uniform release operating criteria, including the amounts necessary to maintain historic ground water levels in affected basins. Table 94 presents corresponding estimates of net safe seasonal yield that would be available to each of the hydrologic units with a Devil Canyon Reservoir of

150,000 acre-foot storage capacity operated as described. It should be noted that the yields shown in Table 94 could be increased substantially if reservoir releases were not required for maintenance of historic ground water levels.

TABLE 93

ESTIMATED SEASONAL POTENTIAL FOR DIVERSION OF WATER FROM DEVIL CANYON RESERVOIR TO CALLEGUAS-CONEJO HYDROLOGIC UNIT DURING BASE PERIOD, WITH OPERATION OF 150,000 ACRE-FOOT RESERVOIR FOR JOINT BENEFIT OF SANTA CLARA RIVER AND CALLEGUAS-CONEJO HYDROLOGIC UNITS

(In acre-feet)

Season	Discharge capacity of Piru-Las Posas Conduit, in second feet						
	40	60	80	100	125	150	200
1936-37	0	0	0	0	0	0	0
1937-38	16,980	23,470	33,960	42,450	47,460	49,710	51,780
1938-39	28,960	34,700	32,040	23,630	18,710	16,530	14,550
1939-40	15,730	3,320	0	0	0	0	0
1940-41	19,200	28,800	38,400	48,000	60,000	72,000	96,000
1941-42	28,960	39,870	42,980	40,280	34,270	28,260	16,250
1942-43	28,960	32,490	43,320	54,150	60,250	63,080	65,560
1943-44	28,960	43,440	57,920	55,780	61,730	72,000	83,800
1944-45	28,960	39,870	41,100	38,570	32,190	24,670	14,700
1945-46	23,940	12,850	6,930	5,380	5,380	5,380	5,370
1946-47	9,020	4,670	4,670	3,980	3,980	3,980	3,980
1947-48	0	0	0	0	0	0	0
1948-49	0	0	0	0	0	0	0
1949-50	0	0	0	0	0	0	0
1950-51	0	0	0	0	0	0	0
Averages	15,310	17,570	20,090	20,820	21,600	22,370	23,470

TABLE 94

ESTIMATED NET SAFE SEASONAL YIELDS OF 150,000 ACRE-FOOT DEVIL CANYON RESERVOIR, IF OPERATED FOR JOINT BENEFIT OF SANTA CLARA RIVER AND CALLEGUAS-CONEJO HYDROLOGIC UNITS

Discharge capacity of Piru-Las Posas Conduit, in second-feet	Available net safe yield, with reservoir releases for maintenance of historic ground water levels, in acre-feet per season		
	To Santa Clara River Hydrologic Unit, under uniform release criteria	To Calleguas-Conejo Hydrologic Unit	Totals
40	13,500	15,300	28,800
60	13,300	17,600	30,900
80	13,300	20,100	33,400
100	13,300	20,800	34,100
125	13,300	21,600	34,900
150	13,300	22,400	35,700
200	13,300	23,500	36,800

As a result of the geologic investigation and the reservoir yield studies, estimates of cost were prepared for dams at the Devil Canyon site with heights of 240 feet and 285 feet from stream bed to spillway lip, creating reservoir storage capacities of 100,000 acre-feet and 150,000 acre-feet, respectively. For both heights of dam, a rolled fill structure was contemplated, comprising an impervious core of select earth material, and upstream and downstream sections of pervious free-draining material. Both upstream and downstream slopes of the dam would be 3:1 for the dam of 240-foot height, and 3.25:1 for the dam of 285-foot height. The impervious section would have slopes of 1:1. Crest widths would be 30 feet, comprised of a 10-foot width for the impervious core, and 10-foot widths each for the upstream and downstream pervious sections. The upstream face of the dams would be protected against wave action by rock riprap placed to a depth of 3 feet normal to the slope.

In the cost estimates, it was assumed that a depth of about 75 feet of sand, gravel, and boulders would be stripped under the impervious core in the channel. On the right abutment, depths of about 5 feet of soil plus 5 feet of weathered fractured sandstone and shale would be stripped. On the left abutment, it was assumed that depths of about 2 feet of soil and 4 feet of fractured rock would be stripped. For the pervious sections of the dam, a nominal depth of stripping of 2 feet was assumed.

Sufficient materials for construction of Devil Canyon Dam are available within two miles or less of the site. Borrow pit drilling by the United Water Conservation District in 1952 revealed the presence of about 3,850,000 cubic yards of impervious fill material. Three samples of material taken from the possible borrow areas were tested by the Division of Water Resources and were deemed adequate for use in the impervious section. Suitable pervious material from the stream bed exists in unlimited quantities. Material salvaged from the stripping excavation would be suitable for this purpose. Granitic rock for riprap is available at a location about five miles upstream from the dam site. It was assumed that compaction of the impervious section of the dam would be effected by either sheepsfoot tampers or pneumatic rollers, and that pneumatic rollers would be used to compact the pervious section.

Spillways, for both heights of dam considered, would have a discharge capacity of 102,000 second-feet, which is the estimated peak discharge of a once in 1,000-year flood. Spillways for both heights of dam would be excavated throug the right abutment, would be of the chute type, and would be concrete lined throughout. For the lower dam, topographical considerations required that the spillway entrance be of the side channel type. For the higher dam, topographical considerations permitted use of the conventional ogee weir at the spillway entrance. Depth of water above the spillway lip would be 20 feet, and an additional 5 feet of residual freeboard would be provided.

It was estimated that the dam of 240-foot height could be constructed in two years, while the dam of 285-foot height would require three years for construction. Diversion of winter flood flows in Piru Creek would be effected through a 21-foot diameter concrete lined tunnel of horseshoe section constructed through the left abutment. The tunnel would be about 1,750 feet in length for the lower dam, and about 2,130 feet in length for the higher dam.

It was assumed that for either size of dam, the outlet conduit would pass through the diversion tunnel. For this purpose, the tunnel would be plugged at its upstream end, with the concrete plug encasing the outlet conduit. The approach channel for the outlet works would be 160 feet in length, with a varying bottom width and 1:1 side slopes. Maximum depth of the cut would be about 30 feet. For the lower dam, a submerged concrete intake structure would be located immediately upstream from the tunnel portal. This structure would consist of a concrete chamber, where in would be located hydraulic and manual controls for a high pressure slide gate which would regulate discharge through the outlet pipe. The intake for the outlet pipe would be located about 25 feet above the floor of the tunnel. The outlet conduit would consist of a 72-inch diameter steel pipe, and would be supported by ring girders resting on the floor of the tunnel. The conduit would terminate at a control house located at the downstream portal of the tunnel, wherein releases would be further regulated by a 60-inch diameter needle valve. Access to the pipe and intake structure would be maintained through the diversion tunnel.

For the higher dam, the intake structure would be a reinforced concrete tower in the reservoir, wherein would be located five 36-inch diameter inlet valves. The outlet conduit would be similar to the one for the lower dam, except that the pipe would terminate in a bifurcation structure in the control house. A 48-inch diameter needle valve would regulate releases of water into a pipe line, and a 48-inch diameter Howell-Bunger valve would regulate releases into the stream bed.

From field examination of the Devil Canyon Reservoir area, it was estimated that, depending upon height of dam to be constructed, from 1,050 to 1,500 acres of minor clearing would be required. The dam and reservoir area is owned by the Federal Government and is a part of the Los Padres National Forest, except for six privately owned parcels with minor improvements. The privately owned lands were appraised by the Ventura County Flood Control District in 1952, and their cost of acquisition was estimated to be $110,250. In the estimate, no valuation was placed upon mineral rights and oil leases.

Presented in Table 95 are pertinent data with respect to the general features of the two sizes of dams and reservoirs considered at the Devil Canyon site, as designed for cost estimating purposes. For illustrative purposes, a plan, profile, and section for the dam creating a reservoir with storage capacity of 150,000 acre-feet are shown on Plate 33, entitled "Devil Canyon Dam on Piru Creek".

TABLE 95

GENERAL FEATURES OF TWO SIZES OF DAM AND RESERVOIR
AT THE DEVIL CANYON SITE ON
PIRU CREEK

Earthfill Dam		
Crest elevation, in feet, U.S.G.S. datum	1,245	1,290
Crest length, in feet	1,050	1,180
Crest width, in feet	30	30
Height, spillway lip above stream bed, in feet	240	285
Side slopes, upstream and downstream	3:1	3.25:1
Freeboard, above spillway lip, in feet	25	25
Elevation of stream bed, in feet, U.S.G.S. datum	980	980
Volume of fill, in cubic yards	6,363,500	9,888,900
Reservoir		
Surface area at spillway lip, in acres	1,021	1,315
Gross storage capacity at spillway lip, in acre-feet	100,000	150,000
Type of spillway	Side channel and concrete lined chute	Ogee weir and concrete lined chute
Spillway discharge capacity, in second-feet	102,000	102,000
Type of outlet	72-inch diameter steel pipe through diversion tunnel	Concrete tower, and 72-inch diameter steel pipe through diversion tunnel

Presented in Table 96 is a summary comparison of capital and annual costs of the two considered sizes of dam and reservoir at the Devil Canyon site. Also presented in Table 96 are estimated unit costs of storage capacity and net safe yields of water that would be developed by construction of the two sizes of reservoir, with reservoir operation for the sole benefit of the Santa Clara River Hydrologic Unit under the uniform release operating criteria with release for maintenance of historic ground water levels. Certain of the relationships presented in Table 96 are depicted graphically on Plates 35, 36, and 37. Detailed estimates of cost for the two sizes of dam and reservoir at the Devil Canyon site are included in Appendix C.

TABLE 96

SUMMARY OF ESTIMATED COSTS OF DAMS, RESERVOIRS, AND YIELDS OF WATER AT THE DEVIL CANYON SITE ON PIRU CREEK, WITH RESERVOIR OPERATION SOLELY FOR BENEFIT OF SANTA CLARA RIVER HYDROLOGIC UNIT

Item	Reservoir storage capacity, in acre-feet: 100,000	Reservoir storage capacity, in acre-feet: 150,000
Capital Costs		
Dam and reservoir	$12,120,000	$15,490,000
Cost per acre-foot of storage	121	103
Cost per acre-foot of net safe yield	810	700
Annual Costs		
Dam and reservoir	625,000	798,000
Cost per acre-foot of net safe yield	42	36
Cost per acre-foot of incremental net safe yield	---	25

Estimates of annual unit costs of net safe yields of water from a Devil Canyon Reservoir of 150,000 acre-foot storage capacity, with seven alternative sizes of Piru-Las Posas conduit, operated for the joint benefit of both the Santa Clara River and Calleguas-Conejo Hydrologic Units, are presented in Table 97. The estimates were based on the previously described criteria of reservoir operation, including releases of water for the Santa Clara River Hydrologic Unit by the uniform release method, and releases for maintenance of historic ground water levels in affected basins.

TABLE 97

ESTIMATED UNIT COSTS OF YIELDS OF WATER
FROM 150,000 ACRE-FOOT DEVIL CANYON RESERVOIR,
WITH RESERVOIR OPERATION FOR JOINT BENEFIT OF SANTA CLARA RIVER
AND CALLEGUAS-CONEJO HYDROLOGIC UNITS

Discharge capacity of Piru-Las Posas Conduit, in second-feet	Annual costs per acre-foot of net safe yield at reservoir
40	$28
60	26
80	24
100	23
125	23
150	22
200	22

Santa Felicia Dam and Reservoir. The Santa Felicia dam site is located on Piru Creek in the Rancho Temescal land grant, some five miles upstream from the confluence of Piru Creek and the Santa Clara River. Stream bed elevation at the site is about 870 feet, U.S.G.S. datum. The drainage area of Piru Creek above the Santa Felicia dam site comprises about 422 square miles, and produced an estimated average seasonal runoff during the base period of about 55,800 acre-feet. It was estimated that waste to the ocean of water originating above the dam site would have averaged about 37,600 acre-feet per season during the base period with the present pattern of land use and water supply development. Consideration was given to the construction of a dam and reservoir at the Santa Felicia site for storage of flood waters in Piru Creek, and utilization of the waters so conserved in the Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits of the Santa Clara River Hydrologic Unit.

The Santa Felicia dam site and reservoir area were surveyed in 1951 by Fairchild Aerial Surveys, Inc., using photogrammetric methods, for the Ventura County Flood Control District, Zone 2. The dam site was mapped up to an elevation of 1,300 feet, at a scale of one inch equals 100 feet, with a 5-foot contour interval. The reservoir area was mapped up to an elevation of 1,250 feet, at a scale of one-inch equals 400 feet, with a contour interval of 10 feet. Storage capacities of Santa Felicia Reservoir at various stages of water surface elevation, derived from the foregoing reservoir area map, are given in Table 98.

TABLE 98

AREAS AND CAPACITIES OF SANTA FELICIA RESERVOIR

Depth of water at dam, in feet	Water surface elevation U.S.G.S. datum, in feet	Water surface area, in acres	Storage capacity, in acre-feet
0	870	0	0
10	880	11	60
20	890	47	350
30	900	65	860
40	910	110	1,730
50	920	150	3,050
60	930	210	4,870
70	940	270	7,270
80	950	390	10,600
90	960	500	15,100
100	970	580	20,400
110	980	690	26,800
120	990	750	34,000
130	1,000	810	41,800
140	1,010	880	50,300
150	1,020	960	59,500
160	1,030	1,030	69,400
165	1,035	1,070	74,600
170	1,040	1,100	80,100
180	1,050	1,190	91,500
187	1,057	1,280	100,000
190	1,060	1,320	104,000
200	1,070	1,420	117,800
210	1,080	1,510	132,400
220	1,090	1,600	148,000
230	1,100	1,710	164,500
240	1,110	1,810	182,100
250	1,120	1,940	200,900
260	1,130	2,070	220,900
270	1,140	2,210	242,300
280	1,150	2,330	265,000
290	1,160	2,460	289,000
300	1,170	2,590	314,300
310	1,180	2,730	340,900
320	1,190	2,870	368,900
330	1,200	3,010	398,300
340	1,210	3,160	429,200
350	1,220	3,300	461,500
360	1,230	3,460	495,300
370	1,240	3,620	530,700
380	1,250	3,730	567,400

Based upon preliminary geological reconnaissance, the Santa Felicia dam site appears to be suitable for a moderately high earthen dam. Dr. Charles P. Berkey reported on the geology of the Santa Felicia dam site in 1947. A program of foundation and borrow area exploration at this site, including soil testing, was conducted by the United Water Conservation District in 1952, under the direction of M.F. Thiel. Except as noted, the geology hereinafter described is based upon preliminary geological reconnaissance conducted by the Division of Water Resources in 1951.

The axis of the proposed Santa Felicia Dam is located on the southwesterly or downstream limb of an anticline in Modelo sandstones, siltstone, and shales of Miocene age. The strike is across the canyon, more or less east to west, and dip at the axis of the dam is about 40 to 50 degrees downstream. Several of the sandstone members, being more resistant to erosion, stand out prominently and help to create a slight constriction in an otherwise uniformly wide valley. The sandstone and shales are well bedded, and generally the shale beds are much thinner and more broken. Acid tests reveal very little calcareous cement in the sandstones at the dam axis. The anticlinal structure is very pronounced. The northward dipping limb is about 0.25 mile upstream from the dam site, with a crushed zone near the axis of the fold where it apparently snapped. A large number of producing oil wells have been drilled on this structure in the vicinity of the dam site. Beds are moderately well jointed but should not be expected to leak excessively.

The nose of the ridge forming the right abutment is not very thick. Both abutments are relatively steep, with slopes on the order of 1.5:1, though not entirely uniform. However, above about 200 feet from stream bed the right abutment flattens out thus providing a good location for the spillway. Terrace deposits of silty, clayey, and gravelly sands, on the order of 40 feet deep, are found on the ridge in the proposed spillway area, both up and down stream from the axis of the dam. However, one hole drilled in this area by United Water Conservation District indicated a depth of terrace material of about 66 feet.

Under the program of foundation and borrow area exploration conducted y United Water Conservation District in 1952, 36 holes were drilled, amounting o about 2,000 lineal feet of overburden drilling and about 1,300 lineal feet of ock core drilling. Overburden in the stream bed was found to be composed of mixture of sand, gravel, cobbles, and boulders, while on the abutments and pillway site it was composed of gravel, sand, silt, and clay. Overburden in he stream bed was a maximum of 85 feet in depth, and in the spillway area the epth of overburden varied from 0 to 66 feet. Rock cores taken showed the dam ite to be underlain by thick beds of moderately soft massive sandstone and hick beds of soft to hard shale. Water pressure testing of most of the holes rilled in the stream bed indicated that little or no grouting in the bedrock ould be required, and led to the assumption that the bedrock will be practically ater tight. It was further assumed that in all probability the reservoir will lso be water tight, due to the tightness of the bedrock and the general mpervious nature of the soil overlying the bedrock except in the stream bed.

Records of runoff at the Santa Felicia dam site are not available. owever, runoff at the site was estimated as equal to 97.5 per cent of the easured runoff at the U.S.G.S. stream gaging station on Piru Creek near Piru. he estimates were based on the ratio of respective watershed areas above the am site and gaging station. The estimated monthly runoff of Piru Creek at the anta Felicia dam site during the base period is presented in Table 99.

TABLE 99

ESTIMATED MONTHLY RUNOFF OF PIRU CREEK AT SANTA FELICIA DAM SITE DURING BASE PERIOD

(In acre-feet)

Season	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Total
1936-37	760	180	3,710	3,260	22,950	21,420	10,740	3,250	1,060	320	140	140	67,930
1937-38	170	240	1,630	1,130	20,390	81,260	9,880	4,800	2,440	1,320	1,250	1,000	125,510
1938-39	950	1,030	10,390	4,730	3,410	7,220	3,330	1,750	670	310	230	3,220	37,240
1939-40	900	800	930	2,040	7,800	2,910	2,060	890	340	100	60	100	18,930
1940-41	370	350	5,950	9,220	54,200	76,750	42,740	17,750	6,500	3,190	1,970	1,650	220,640
1941-42	1,900	2,010	6,540	4,700	2,950	3,000	5,530	2,480	1,070	520	340	340	31,380
1942-43	470	630	920	30,760	18,780	33,250	7,390	3,400	1,700	930	600	530	99,360
1943-44	730	950	5,530	3,720	26,440	44,930	20,050	10,830	4,380	2,280	1,220	1,010	122,070
1944-45	1,320	4,050	2,420	2,400	8,970	5,640	4,200	2,120	1,220	420	400	370	33,530
1945-46	890	1,030	6,800	2,110	2,380	5,850	8,400	2,050	860	510	340	300	31,520
1946-47	540	5,590	11,160	3,650	2,030	1,760	1,400	830	340	150	100	120	27,670
1947-48	180	460	750	650	900	1,360	1,110	570	250	80	70	80	6,460
1948-49	80	130	720	740	810	1,960	750	320	130	100	70	50	5,860
1949-50	60	130	890	1,330	2,220	1,030	830	320	60	60	60	60	7,050
1950-51	70	130	150	310	380	500	250	230	130	100	40	50	2,340
Average seasonal runoff, 1936-37 through 1950-51													55,830

Based on the estimates of runoff, monthly studies of operation during the base period were made for storage capacities of 50,000 acre-feet, 75,000 acre-feet, and 100,000 acre-feet at the Santa Felicia site, under both the uniform release and rapid release methods of operation. In all of the studies an allowance was made for reduction in effective reservoir storage capacity due to sedimentation, in the amount of 14,000 acre-feet. This amount represents the estimated loss after about 20 years of operation. An estimated average net seasonal depth of evaporation from the reservoir water surface of 2.20 feet, distributed monthly in accordance with the following tabulation, was employed in the operation studies.

Month	Net evaporation, in feet of depth	Month	Net evaporation, in feet of depth
October	0.19	April	0.19
November	0.08	May	0.24
December	0.05	June	0.27
January	0.05	July	0.33
February	0.06	August	0.33
March	0.13	September	0.28
		TOTAL	2.20

The estimated values of net safe seasonal yield that would be obtained under both the uniform release and rapid release operating criteria are presented in Table 100. The relationship between reservoir storage capacity and net safe seasonal yield, with Santa Felicia Reservoir operated by the uniform release method, and with releases for maintenance of ground water levels in affected basins, is depicted graphically on Plate 36.

TABLE 100

ESTIMATED NET SAFE SEASONAL YIELDS OF SANTA FELICIA RESERVOIR

(In acre-feet)

Reservoir storage capacity	Uniform release operation		Rapid release operation	
	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, with releases for maintenance of ground water levels	Available to Oxnard Forebay, Oxnard Plain, and Pleasant Valley Subunits, without releases for maintenance of ground water levels	Available within Santa Clara River Hydrologic Unit	Available to Oxnar Forebay, Oxnard Plain, and Pleasan Valley Subunits
50,000	6,600	9,500	6,800	4,600
75,000	11,000	14,300	11,300	7,500
100,000	15,000	19,000	15,500	10,500

As a result of the geologic investigation and the reservoir yield studies, estimates of cost were prepared for dams at the Santa Felicia site with heights of 140 feet, 165 feet, and 187 feet from stream bed to spillway lip, creating reservoir storage capacities of 50,000 acre-feet, 75,000 acre-feet and 100,000 acre-feet, respectively. A dam with height of 187 feet is the highest that could be constructed without flooding the Blue Point site. Furthermore, with higher dams it would not be possible to utilize the ridge which forms the right abutment for a spillway location, and spillway costs would be proportionately greater.

For all heights of dam considered, a rolled fill structure was contemplated, comprising an impervious core of select earth material, and upstream and downstream sections of pervious free draining sands and gravels. Both upstream and downstream slopes of the dam would be 2.5:1 for the dam of 140-foot height, and 3:1 for the two higher dams. The impervious section would have slopes of 1:1. Crest widths would be 30 feet, comprised of a 10-foot width for the impervious core, and 10-foot widths each for the upstream and downstream pervious sections. The upstream face of the dam would be protected against wave action by rock riprap placed to a depth of 3 feet normal to the slope.

In the cost estimates, it was assumed that depths of about 4 feet of soil and 6 feet of fractured and weathered sandstone and shale would be stripped from the right abutment under the impervious section. In the stream channel, about 600 feet in width, an estimated depth of 75 feet of sands, gravels, and cobbles would have to be removed under the impervious section. Most of this stripping would be below the water table, and would require dewatering during excavation and backfill. Under the impervious section on the left abutment, estimated depths of about 3 feet of soil and 6 feet of fractured and weathered rock would be stripped. For the pervious sections of the dam, a nominal depth of stripping of from 2 to 4 feet was assumed.

The borrow pit drilling by the United Water Conservation District in 1952 revealed the presence of about 3,850,000 cubic yards of impervious fill material within about 1.3 miles of the dam site. Soil tests showed that these borrow soils, when compacted, are impervious, well-graded mixtures of gravel, sand, and silt or clay. In addition, a portion of the material stripped from both abutments could be salvaged for impervious fill. Pervious material from the stream bed exists in unlimited quantities, and all of the material stripped from the channel section could be salvaged for the pervious fill. The nearest source of granite for riprap is some five miles distant, or stream bed gravels, cobbles, and boulders could be used for riprap purposes. It was assumed that compaction of the impervious fill of the dam would be effected by either sheepsfoot tampers or pneumatic rollers, and that pneumatic rollers would be used to compact the pervious sections. The estimates included provision for light grouting of the foundation, increasing with the height of dam.

Spillways, for all heights of dam considered, would have a discharge capacity of 103,000 second-feet, which is the estimated peak discharge of a once in 1,000-year flood. The spillways were designed as concrete-lined chutes, with ogee weir control sections. They would be constructed across the ridge forming the right abutment of the dam, and would discharge into Piru Creek below the dam. Depth of water above the spillway lip at design discharge capacity would be 15 feet, and an additional 5 feet of residual freeboard would be provided.

As it was estimated that the dam of 140-foot height could be constructe in one year, it was assumed that diversion of summer flow in Piru Creek would be effected through the outlet conduit. For the two higher dams, requiring an estimated two years of construction, a 22-foot diameter concrete lined tunnel of horseshoe section was included in the estimates, to provide for diversion of winter flows. The tunnel would be about 1,080 feet in length for the dam of 165-foot height, and about 1,270 feet in length for the dam of 187-foot height

It was assumed that outlet works for both of the larger dams considered would utilize the diversion tunnel after constructed. The approach channel for the outlet works would be 100 feet in length, with a varying bottom width and 1:1 side slopes. Maximum depth of cut would be about 50 feet. The first 60 feet of tunnel would be plugged with concrete, encasing the outlet pipe. A submerged concrete intake structure would be located immediately upstream from the tunnel portal. This structure would consist of a chamber, wherein would be located hydraulic and manual controls for a high pressure slide gate, which would regulate discharge through the outlet pipe. The intake for the outlet conduit would be located about 25 feet above the floor of the tunnel. The outlet conduit would consist of a 72-inch diameter steel pipe, supported by ring girders resting on the floor of the tunnel. The conduit would terminate at a control house located at the downstream portal of the tunnel, wherein releases would be further regulated by a 60-inch diameter needle valve. Access to the pipe and intake structure would be maintained through the diversion tunnel.

For the dam with height of 140 feet, an intake structure similar to those for the two larger dams was planned. However, the outlet conduit would follow an alignment along the contour of the bedrock on the left abutment. The conduit would be constructed of reinforced concrete, horseshoe in section, and

9.5 feet in diameter, and would be placed in a trench excavated to sound rock. A 60-inch diameter steel outlet pipe, supported on ring girders, would be placed within this concrete conduit. The outlet pipe would terminate at the downstream toe of the dam in a control house. Further regulation of reservoir releases would be obtained by installing a 54-inch diameter needle valve in the pipe line. Access to the pipe and intake structure would be maintained through the outlet conduit.

Based upon field examination, it was estimated that, depending upon the height of dam to be constructed, from 1,030 to 1,490 acres of light brush and some trees would have to be removed from the reservoir area. The cost of acquisition of private lands, and improvements on private and public lands was estimated by the Ventura County Flood Control District in 1952 to be about $446,650. In 1952, the United Water Conservation District estimated the cost of necessary road relocation to be about $150,000, and oil well damages to be about $200,000.

Presented in Table 101 are pertinent data with respect to the general features of the three sizes of dams and reservoirs considered at the Santa Felicia site, as designed for cost estimating purposes. For illustrative purposes, a plan, profile, and section for the dam creating a reservoir with storage capacity of 100,000 acre-feet are shown on Plate 34, entitled "Santa Felicia Dam on Piru Creek".

TABLE 101

GENERAL FEATURES OF THREE SIZES OF DAM AND RESERVOIR
AT THE SANTA FELICIA SITE ON PIRU CREEK

Earthfill Dam			
Crest elevation, in feet, U.S.G.S. datum	1,030	1,055	1,077
Crest length, in feet	1,040	1,160	1,240
Crest width, in feet	30	30	30
Height, spillway lip above stream bed, in feet	140	165	187
Side slopes, upstream and downstream	2.5:1	3:1	3:1
Freeboard, above spillway lip, in feet	20	20	20
Elevation of stream bed, in feet, U.S.G.S. datum	870	870	870
Volume of fill, in cubic yards	3,037,900	4,527,000	5,428,000
Reservoir			
Surface area at spillway lip, in acres	884	1,066	1,280
Gross storage capacity at spillway lip, in acre-feet	50,000	75,000	100,000
Type of spillway	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute	Ogee weir and concrete lined chute
Spillway discharge capacity, in second-feet	103,000	103,000	103,000
Type of outlet	60-inch diameter steel pipe, in reinforced concrete conduit beneath dam	72-inch diameter steel pipe, through diversion tunnel	72-inch diameter steel pipe, through diversion tunnel

Presented in Table 102 is a summary comparison of capital and annual costs of the three considered sizes of dam and reservoir at the Santa Felicia site. Also presented in Table 102 are estimated unit costs of storage capacity and net safe yield of water that would be developed by construction of the three sizes of reservoir. Yields referred to are those that would result under the uniform release method of reservoir operation with releases for maintenance of historic ground water levels. Certain of the relationships presented in Table 102 are depicted graphically on Plates 35, 36, and 37. Detailed estimates of cost for the three sizes of dam and reservoir at the Santa Felicia site are included in Appendix C.

TABLE 102

SUMMARY OF ESTIMATED COSTS OF DAMS, RESERVOIRS, AND YIELDS OF WATER AT SANTA FELICIA SITE ON PIRU CREEK

Item	Reservoir storage capacity, in acre-feet		
	50,000	75,000	100,000
Capital Costs			
Dam and reservoir	$ 7,128,000	$ 8,417,000	$ 9,029,000
Cost per acre-foot of storage	143	112	90
Cost per acre-foot of net safe yield	1,080	765	600
Annual Costs			
Dam and reservoir	369,000	435,000	469,000
Cost per acre-foot of net safe yield	56	40	31
Cost per acre-foot of incremental net safe yield	---	15	8

Conveyance and Distribution of Supplemental Water

This section describes the various conveyance and distribution systems that were considered for delivery of locally developed supplemental water to areas of need in Ventura County, and presents preliminary cost estimates thereof. The location and alignment of the systems studied are shown on Plate 42, entitled "Proposed Conveyance and Distribution Systems". In general, preliminary design of the conveyance and distribution systems was made by the use of available U.S.G.S. topographic maps, at a scale of 1:24,000 and with a contour interval of 20 feet, and from information obtained during field reconnaissance of the proposed routes. In most cases, design of the systems was limited to the main laterals extending to strategic points in each of the hydrologic units, and no attempt was made to estimate the cost of connection with individual water users. Except as noted, preliminary estimates of cost acquisition of right of way and relocation of existing facilities, when necessary, were made on the basis of field examination and appraisal during the course of the investigation.

Distribution System for Casitas Reservoir. In the preliminary design for a distribution system to serve water developed by Casitas Reservoir, it was assumed that sufficient reservoir storage capacity would be constructed to provide new water in an amount equal to the estimated present supplemental water requirement in the Ventura Hydrologic Unit, of about 4,000 acre-feet per season, plus an allowance to provide for a portion of the probable ultimate supplemental water requirement of about 31,000 acre-feet per season. It was assumed that the initial distribution system from the reservoir would deliver about 13,360 acre-feet of water per season, distributed in accordance with the following tabulation:

Hydrologic Subunit	Seasonal delivery of water, in acre-feet
Upper Ojai	760
Ojai	1,200
Upper Ventura River	3,320
Lower Ventura River	6,920
Rincon	1,160
TOTAL	13,360

The amounts shown in the tabulation may be compared with values for present and probable ultimate supplemental water requirements in the Ventura Hydrologic Unit presented in Tables 47 and 48.

In November, 1951, the Ventura County Flood Control District prepared a report entitled "Distribution of Water Stored in Casitas Reservoir and Matilija Reservoir to Lands and Users in the Year 1975", describing a distribution system for water from Casitas Reservoirs. In accordance with the request of the Board of Supervisors of the Ventura County Flood Control District, this report was reviewed by the Division of Water Resources and the results of the review were submitted to the District on June 30, 1952. The distribution system described herein conforms in general alignment to the plan prepared by the Ventura County Flood Control District, and is based on surveys, appraisals, and designs made by that District. However, certain revisions were made in line capacities and estimates of cost. The locations of Casitas Reservoir and the distribution system therefrom are shown on Plate 42.

From the outlet control house at Casitas Dam, the main feeder line of the distribution system would follow Casitas Pass Road generally downstream along the right bank of Coyote Creek, and would connect with the Foster Park intake of the City of Ventura's water system. This feeder line would convey the entire supply from the reservoir for the Upper Ojai, Ojai, and Upper and Lower Ventura River Subunits. It would have a discharge capacity of about 32 second-feet, and would be capable of delivering about 2,000 acre-feet per month. This amount represer about 15 per cent of the assumed seasonal supply available from the reservoir, which is somewhat greater than the estimated maximum monthly percentage of seasonal demand for water in the Ventura Hydrologic Unit. By designing the ma feeder and laterals under this criterion, some additional peaking capacity was obtained. The main-feeder line to Foster Park would be about 14,000 feet in length, and would be constructed of 36-inch diameter centrifugal spun reinforc concrete pipe.

From Foster Park, a smaller line would extend northerly a distance o about 9,000 feet to the vicinity of Lacrosse, where a wye would be installed. This 27-inch diameter reinforced concrete cylinder pipe would have a capacity about 14 second-feet, and would deliver about 5,280 acre-feet per season.

From the wye near Lacrosse, one branch line would continue northerly about 18,000 lineal feet generally parallel to the Ventura River to State Highw 150, where another wye would be located. This line would consist of a 24-inch diameter reinforced concrete pipe with a capacity of 10 second-feet, and would deliver a seasonal supply of about 3,750 acre feet. A regulatory reservoir of about 50 acre-foot storage capacity would be located north of Oak View on the line. It was assumed that a pumping plant, required on this line to lift the water about 350 feet, would consist of three pumps installed in series, each wi a 200 horsepower motor.

From the wye near Lacrosse, a second line would extend northeasterly along San Antonio Creek, a distance of about 42,800 feet to a regulating reservoir, about 1.8 miles easterly from the town of Ojai at an elevation of 880 feet. This line would consist of a 16-inch diameter reinforced concrete cylinder pipe with capacity of 4.0 second-feet, and would deliver a seasonal supply to the regulating reservoir of about 1,525 acre-feet. It was assumed that two pumping plants would be utilized on this line, each equipped with a 100 horsepower motor and each lifting the water about 310 feet.

From the wye at State Highway 150, one line would extend westerly across ne Ventura River about 13,400 lineal feet to a regulating reservoir of 50 acre-oot storage capacity in the Santa Ana Creek watershed. This 14-inch diameter einforced concrete cylinder pipe, with capacity of about 3.5 second-feet, would eliver a seasonal supply of about 1,315 acre-feet to the regulating reservoir at n elevation of 665 feet.

From the wye at State Highway 150, another conduit would extend about 1,400 lineal feet to a point immediately north of Meiners Oaks, where an inter-onnection would be made with the existing pipe line from Matilija Reservoir. This 5-inch diameter reinforced concrete cylinder pipe, with capacity of about 4.5 econd-feet, would deliver a seasonal supply of about 1,700 acre-feet. It was ssumed that a pumping plant, required on the line to lift the water about 360 eet, would consist of three pumps, each equipped with a 120 horsepower motor.

From the aforementioned regulating reservoir easterly of Ojai, a line ould extend northerly to provide an additional interconnection with the existing atilija pipe line. This lateral would consist of about 4,500 lineal feet of 12-nch diameter reinforced concrete cylinder pipe, with a capacity of about 1.2 econd-feet, and would deliver a seasonal supply of about 450 acre-feet. From he same regulating reservoir, another conduit would extend easterly about 10,200 ineal feet to serve the Upper Ojai Subunit. This 12-inch diameter reinforced oncrete cylinder pipe, with capacity of about 2.0 second-feet, would deliver a easonal supply of about 760 acre-feet, and would terminate at an elevation of ,312 feet in a small terminal reservoir. It was assumed that a pumping plant, equired in this line to lift the water about 450 feet, would consist of two umps connected in series, equipped with 100 horsepower and 50 horsepower motors, espectively.

From the terminus of the existing Matilija pipe line, a new line would extend northeasterly about 9,000 lineal feet to a regulating reservoir of about 40 acre-foot storage capacity at an elevation of 1,300 feet. This 10-inch diameter welded steel pipe, with capacity of about 1.2 second-feet, would deliver a seasonal supply of about 450 acre-feet. A pumping plant, required on the line to lift the water about 420 feet, would consist of two pumps installed in series, equipped with 50 horsepower and 25 horsepower motors, respectively.

Immediately west of Ojai, an extension from the existing Matilija pipe line would be constructed northerly about 5,000 lineal feet to a regulating reservoir of 40 acre-foot storage capacity at an elevation of 980 feet. This 14-inch diameter reinforced concrete cylinder pipe, with capacity of about 4.4 second-feet, would deliver a seasonal supply of about 1,640 acre-feet.

From the Foster Park intake of the City of Ventura, a pipe line would b constructed southerly to Canada Larga and thence northeasterly along Canada Larga This would consist of about 29,200 lineal feet of 6-inch diameter welded steel pipe with capacity of about 0.5 second-feet, and would deliver a seasonal supply of about 175 acre-feet. A pumping plant, required to lift the water about 400 feet to an elevation of 760 feet, would consist of two pumps, installed in series equipped with 10 horsepower and 20 horsepower motors, respectively.

Commencing at the outlet control house at Casitas Dam, a pipe line woul extend westerly along the relocated Casitas Pass Road to Casitas Summit, a distance of about 1,300 feet, and thence southwesterly to the ocean at Sea Cliff, a distance of about 21,000 feet. From Sea Cliff, one lateral would extend along the ocean a distance of about 45,000 feet to the Ventura River, and another would extend westerly a distance of about 20,000 feet to the vicinity of the County line near Rincon Point. This system would be constructed of welded steel