SAN GABRIEL VALLEY SEWERAGE SYSTEM.

REPORT OF

LOUIS KORN
and
EDWARD SEAVER
CALIFORNIA INSTITUTE OF TECHNOLOGY

JUNE 1921.

INDEX

	page
Introduction	
Demand for adiquate sewerage system.	1
Object	
Scope of investigation.	1
Topography	
General topography and location.	2
Population	
Present population.	3
Estimated future population.	4
Population curves	5
Reason for excluding Pomona.	11
Sewage disposal	
Class of sewage.	11
Type of system.	11
Method of disposal.	11
Irrigation	
Amount of sewage.	13
Value of effluent.	13
Design of line	
Alignment.	14
Size, velocity, and quantity.	15
Design and cost of conduits.	19
Excavation.	20
Right of way.	21
Man holes.	21
Trestle.	21
Pier and wood stave pipe.	21
Summary of costs.	21
Treatment plant	
Necessity of plant.	22
Degree of purification.	22
Type of plant.	22
Description of process.	23
Design of tank.	25
Design of sludge chambers.	25
Design of screens.	26
Economics of treatment plant.	26
Irrigation canal	
Design and cost.	27
Relative costs	
Saving on cost with treatment plant.	28
Increased cost with treatment plant.	28
Summary of costs.	29
Revenue and operating expense.	29
Interest.	29
Facts determined.	30
Reccomendations	31

SAN GABRIEL VALLEY SEWERAGE SYSTEM

Report by

LOUIS KORN and EDWARD SEAVER

Introduction.

Demand for adaquate sewerage system.

The disposal of sewage is the most important item in public sanitation. It is the most important present day problem in every city whether large or small. The direct cause of the majority of epidemics is the contamination of the water supply of the city by the excreta of man or animal. Public health varies directly as public sanitation,and if the public sanitation be good,the liability of sickness caused by contamination of the water supply is greatly lessened. When a city outgrows its sewerage system the public health becomes endangered. There are two causes for the increased amount of sewerage,increase in population and increase in industrial and manufacturing wastes. The main problem in this connection is the ultimate disposal of the matter which reaches the sewers.

Object.

Scope of this investigation

Before going further into the matter of sewage disposal it will be best to give in a brief way the problem which we have undertaken. It is the object of this study to make a preliminary investigation of the possibility for a sewerage system to serve the communities in Los Angeles County east of the Los Angeles River. If such a system is found feasible it would in conjunction with the proposed North Outfall Sewer of the city of Los Angeles provide for the sewering of the greater part of Los Angeles County west of the mountains.

Topography.

General topography and location.

The topography of this country to be served by this San Gabriel Valley System is a gradual slope to the San Gabriel River from the east and the west, the river flowing nearly from north to south. At a point about twenty miles from its mouth and about ten miles from the point where it leaves the mountains the river passes between the Montebello and Puenty Hills, this point being known as the Whittier Narrows. Above these narrows the river turns to the north east. To the east, reached by a pass of very low grade between the Puenty and the San Jose Hills, is the city of Pomona at the extreme eastern boundary of the county. To the north is the community of Claremont while to the west along the foothills are Laverne, San Dimas, Glendora, Azusa, Duarte, Monrovia, Arcadia, and Sierra Madra. To the extreme west on the bank of the Arroyo

Seco is the city of Pasadena, while to the south are South Pasadena, Alhambra, and San Gabriel. To the south of these cities and below the Whittier Narrows are Whittier, Downey, and Norwalk. From the mountains to the Whittier Narrows the grade of the river is medium steep but from this point to the ocean it is rather flat.

Population.

Present population

The present populations of these communities mentioned with their populations in 1900 and 1910 are given in the following tables. The sourse of these data was the census reports of 1900, 1910, and 1920.

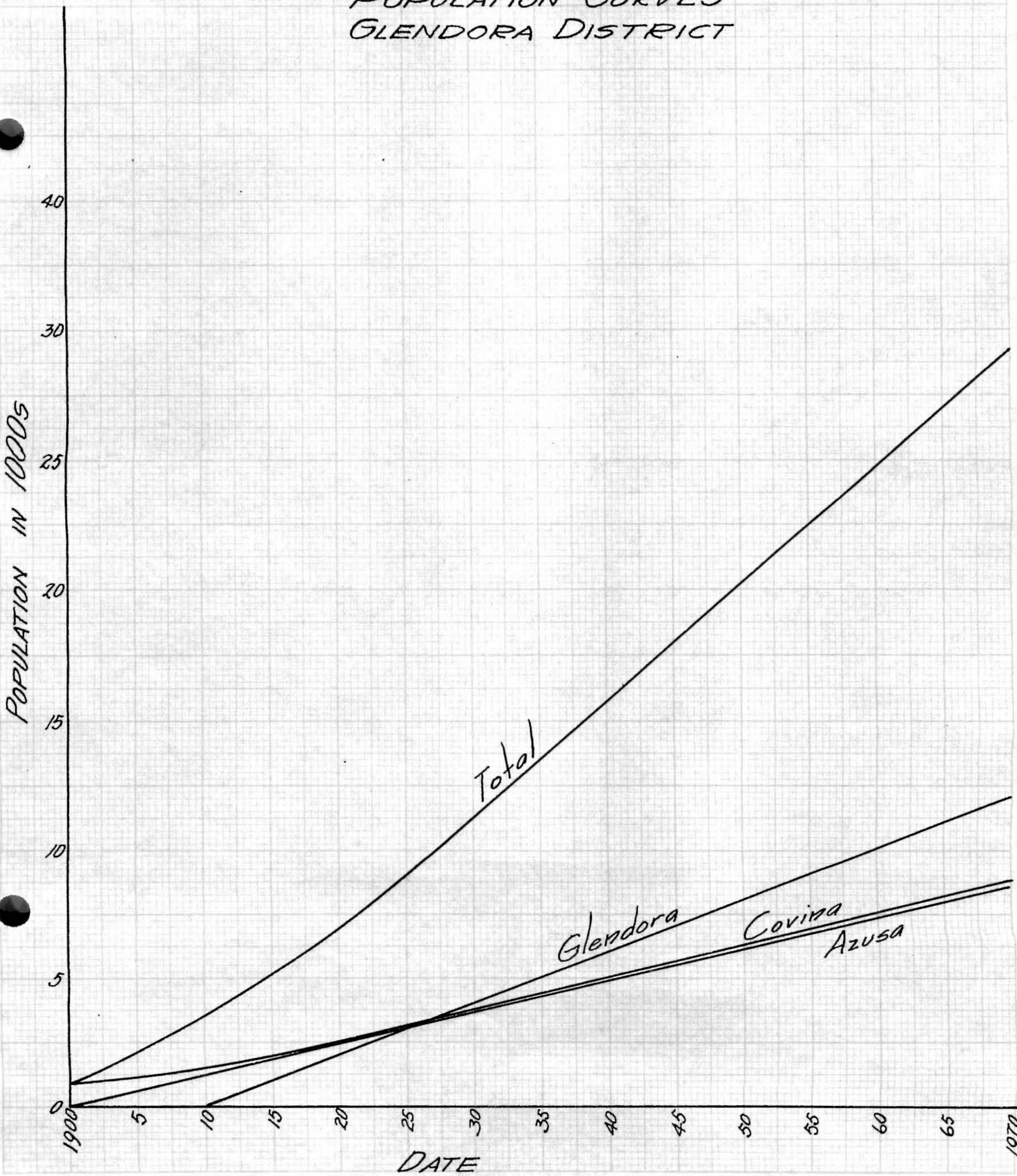
City	1900	1910	1920
Pomona	5526	10207	13505
Claremont	0	1114	1728
Laverne	0	954	1698
Azusa	863	1477	2460
Glendora	0	0	2028
Covina	0	1652	1999
Monrovia	1205	3576	5480
Sierra Madre	0	1303	2026
Arcadia	0	696	2239
Whittier	1596	4550	7997
Elmonte	0	0	1283
San Gabriel	------	------	2640
Alhambra	0	5021	9096
South Pasadena	1001	4649	7648
Pasadena	9117	30291	45354

Estimated future population

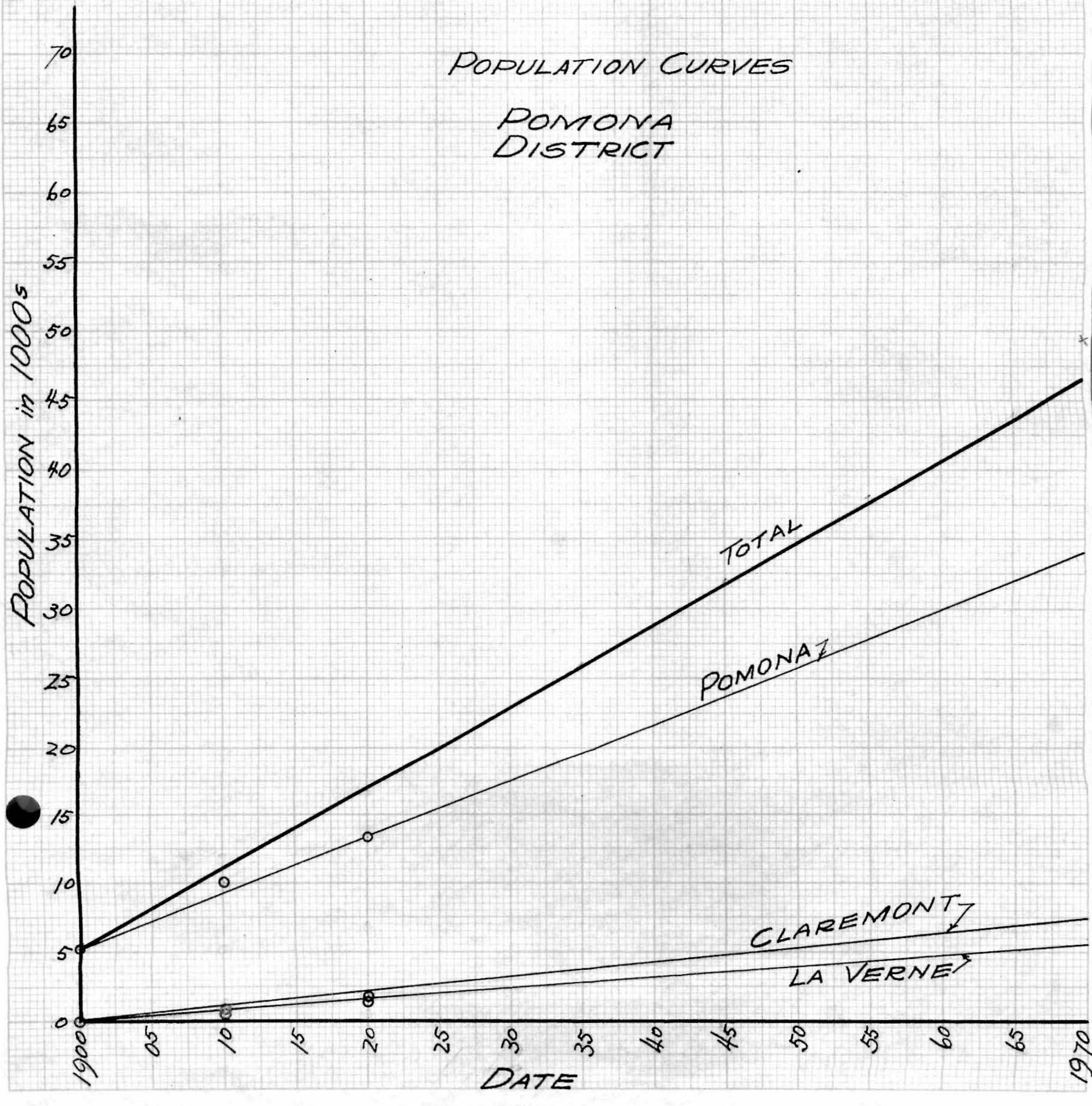
These data in the table on the previous sheet were the bases for the estimates of future population. As most of the communities considered are extremely small at the present time and due to the fact that they have been in existance for only ten or twenty years it was impossible to estimate their future growth by a comparison with other communities, but rather it was deemed the only practical method to assume their future growth to be along a tangent to the curve drawn through the known points at the last point. In cases where there was only one or two known points the growth was assumed to be along a straight line through the known points. The growth of Pasadena has been along a smooth curve with increasing radius so that at the present time it is nearly a straight line. For this reason it was thought fair to assume it to continue along this line. For a project so large it was considered unfair to the present generation to make them pay for a system much in excess to the probable required size. If any unforseen growth should occur the cost of the larger system should be carried by the people living here at that time and not by those who are here now. For convenience and clearness the communities considered have been grouped into districts representing the area served by the separate trunk lines.

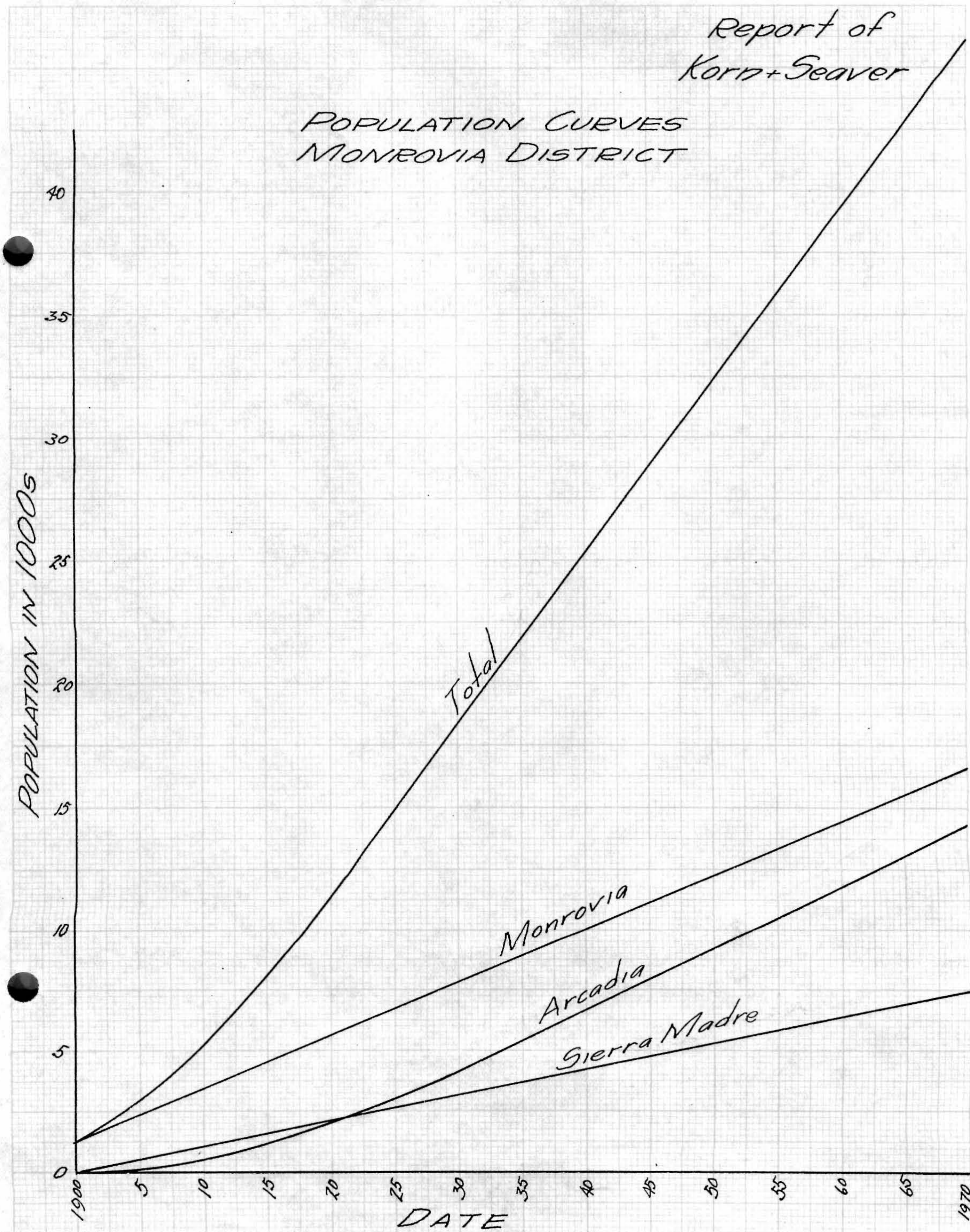
Report of
Korn + Seaver

POPULATION CURVES
GLENDORA DISTRICT

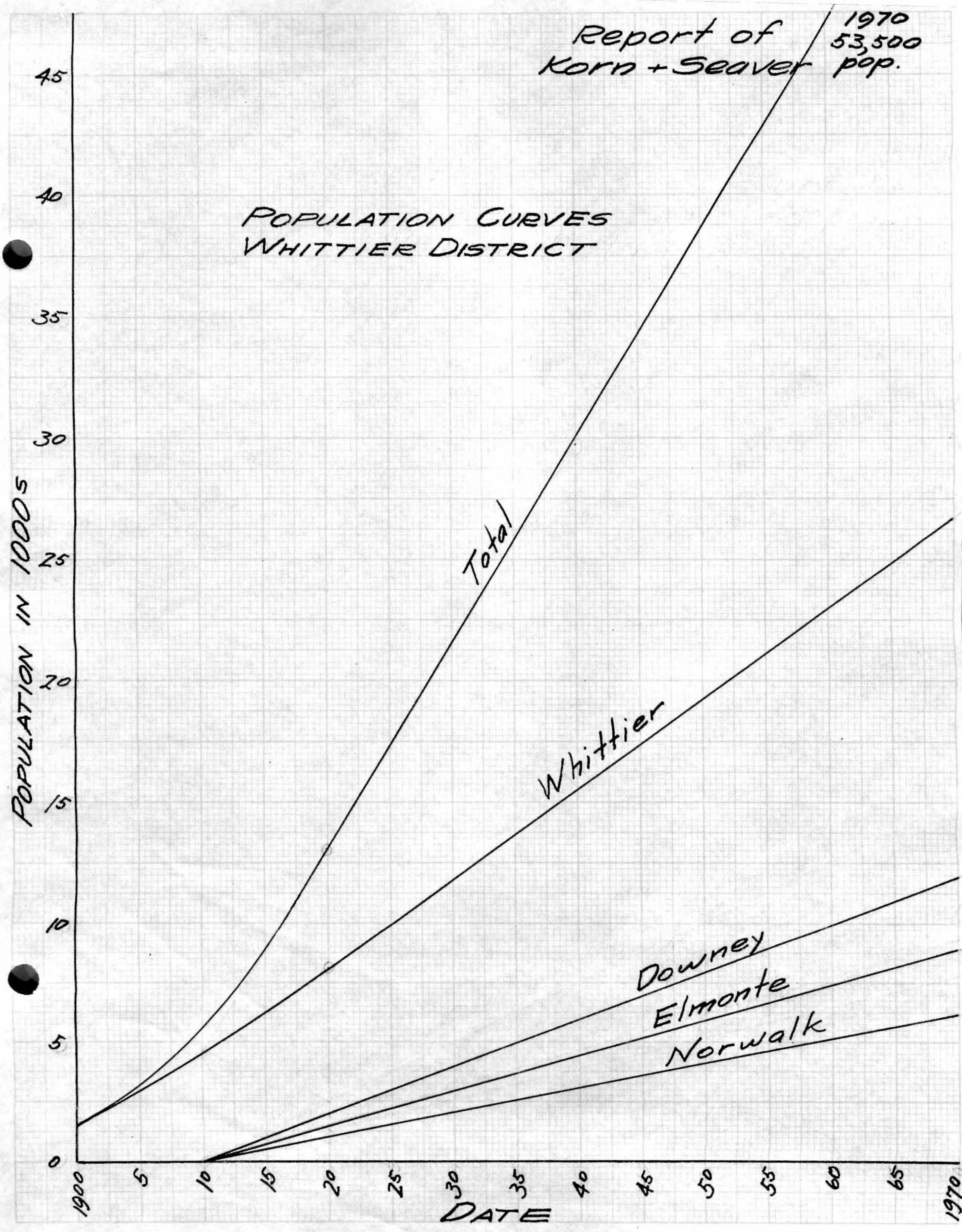


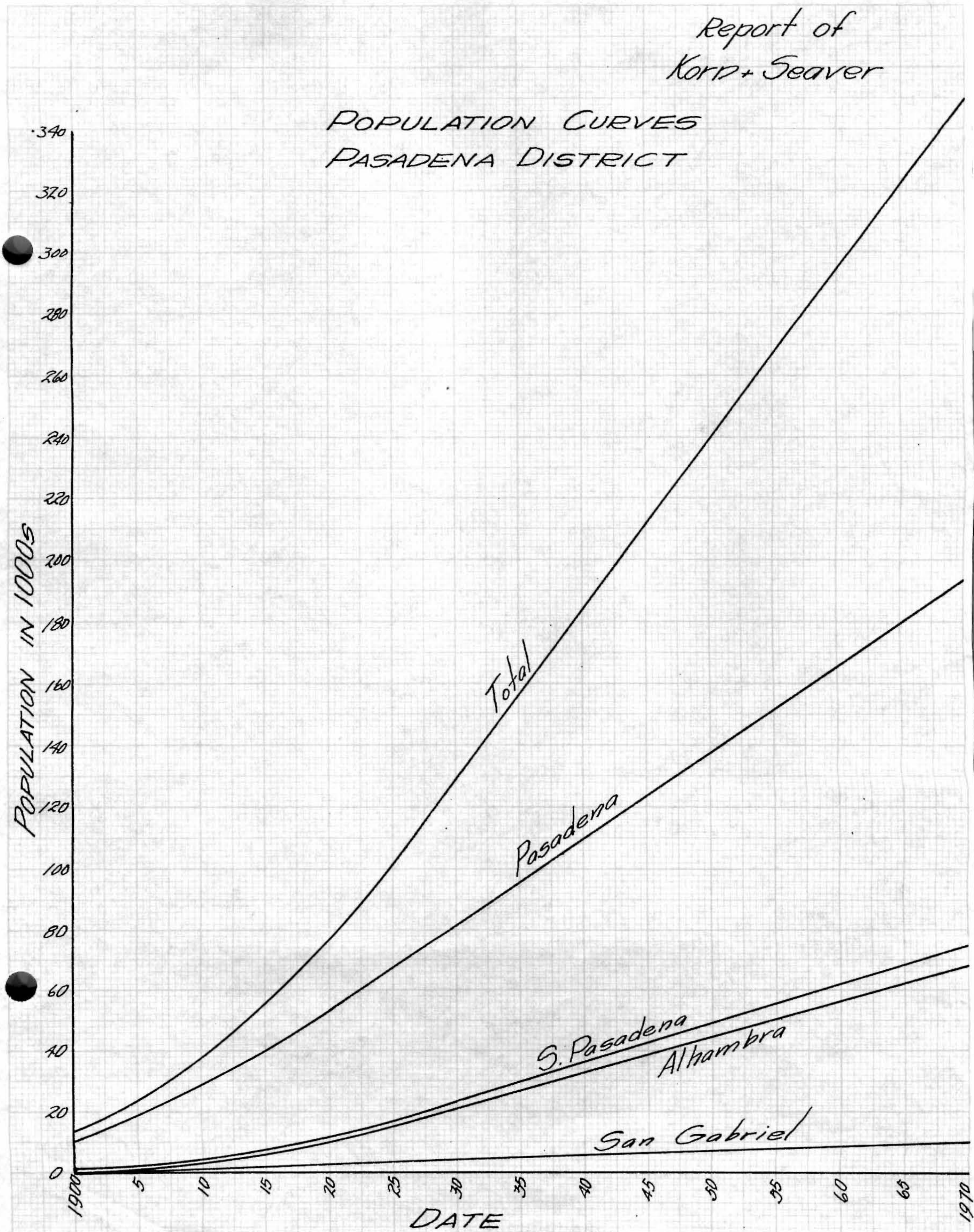
Report of
Korn + Seaver



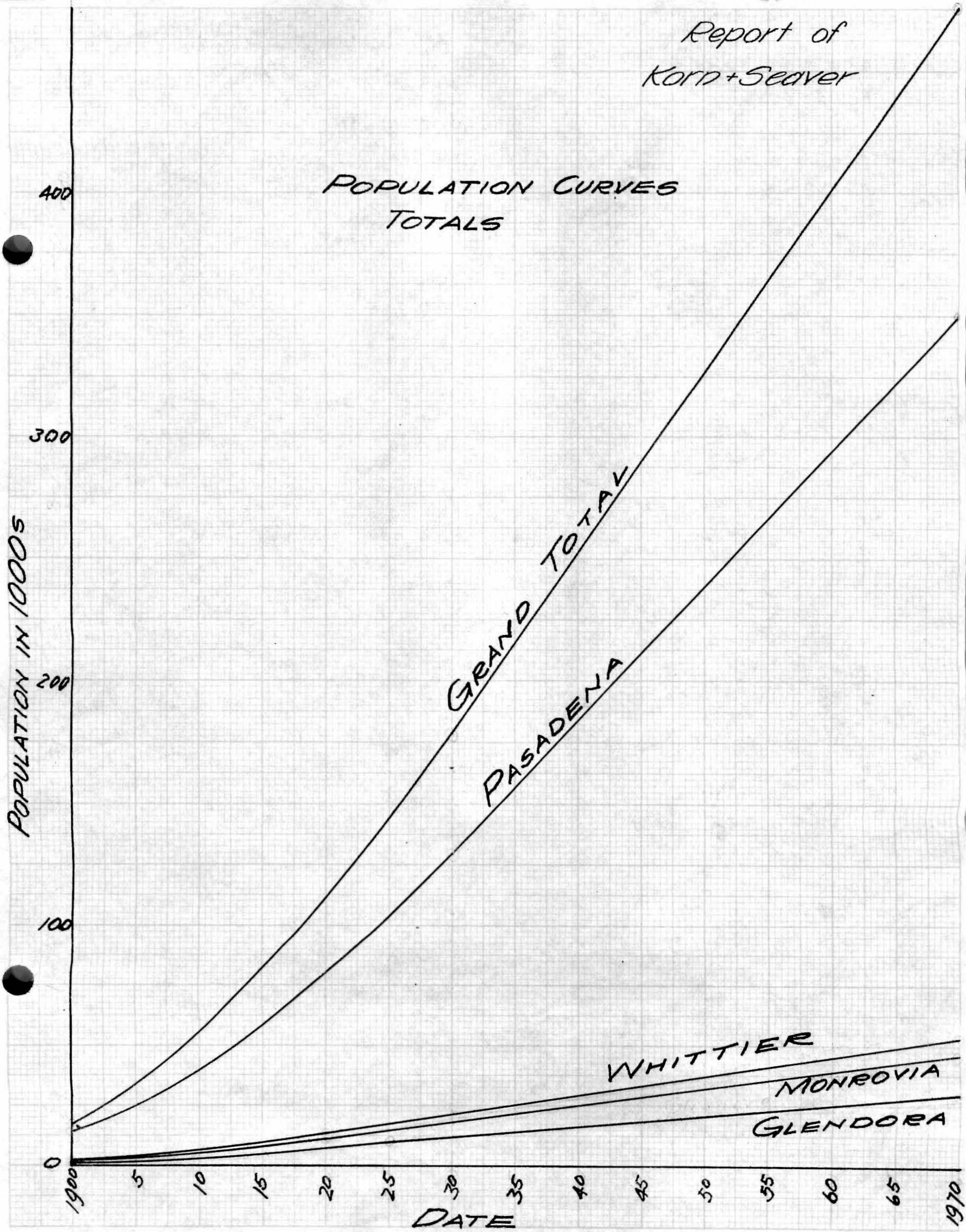


Report of
Korn + Seaver
POPULATION CURVES
MONROVIA DISTRICT
POPULATION IN 1000s
40
35
30
25
20
15
10
5
0
1900
5
10
15
20
25
30
35
40
45
50
55
60
65
1970
DATE
Total
Monrovia
Arcadia
Sierra Madre


Report of
Korn + Seaver
1970
53,500
pop.
POPULATION CURVES
WHITTIER DISTRICT
POPULATION IN 1000s
Total
Whittier
Downey
Elmonte
Norwalk
0
5
10
15
20
25
30
35
40
45
1900
5
10
15
20
25
30
35
40
45
50
55
60
65
1970
DATE



Report of
Korp + Seaver
POPULATION CURVES
PASADENA DISTRICT
POPULATION IN 1000s
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
Total
Pasadena
S. Pasadena
Alhambra
San Gabriel
1900
5
10
15
20
25
30
35
40
45
50
55
60
65
1970
DATE



Report of
Korn + Seaver
POPULATION CURVES
TOTALS
POPULATION IN 1000s
400
300
200
100
0
GRAND TOTAL
PASADENA
WHITTIER
MONROVIA
GLENDORA
1900
5
10
15
20
25
30
35
40
45
50
55
60
65
1970
DATE

Reason for excluding Pomona

Pomona being situated on the divide between the San Gabriel and the Santa Ana river valleys can be sewered in either of the two directions. As the line to connect it with the San Gabriel Valley System would be at a very flat grade thus necessitating a large pipe and since the entire cost of this line 90,000 feet in length would have to be borne by Pomona,Lavern,and Claremont it was deemed advisable not to consider it in connection with this system but rather allow it to connect with cities in the Santa Ana valley at such time as a system for their benefit is constructed.For this reason all future calculations are based on territory and population exclusive of Pomona district.

Sewage Disposal

Class of sewage

As the communities considered are now either residental or agricultural with no prospect of becomming to any great degree industrial it will be assumed that the sewage from these communities will be entirely domestic.

Type of system

Pasadena,the dominating factor in the system,now has a separate system of collection and for this reason all estimates have been based on a sanitary sewerage system rather than on the combined sanitary and storm sewer.

Method of disposal

The old idea of sewage disposal was to get rid of the sewage by the cheapest means that would protect the community from litigation.

The more modern method is to treat the sewage to such an extent to render it harmless and then to waste it in the most convenient manner. But in the last few years a third and new method has been presented, that of utilizing the treated sewage, both sludge and effluent, for beneficial purposes and with compensation for the treatment. The sludge, being high in nitrates, is used as a dry fertilizer while the water, also high in dissolved nitrates, is excellent for irrigation. In a country such as that lying between the Puenty Hills and the ocean irrigation is practiced and as water is scarce the utilization of sewage effluent is made all the more practical and advisable. If this method of disposal be used there must also be some method of purification used in connection with it. From a careful study of the various methods of purification used at the present time some process of activated sludge was decided upon, this process rendering a high degree of purification. As the purified effluent would only be used eight months of the year it must be wasted for the remaining period. The treatment plant, being on the bank of the San Gabriel River, must discharge its waste into the river which will be carrying the winter flood thus preventing the developement of a nusiance either along the river or at its outlet at Alimitos Bay. A comparison of the most advisable method will be discussed later,

Irrigation

Amount of sewage . As a basis for calculating the amount of sewage from this district 100 gallons per capita per day has been used. With a population of 467,000 in 1970 , this would give 46,700,000 gallons per day or a 130.5 sec.ft. maximum flow for a 18 hour period. Assuming the average to be 80% of the maximum, this being the case in cities of like size, the average would be 104 sec.ft. This quantity represents 143 acre ft. per day.

Value of effluent. The present value of water for irrigation in the territory to be served is $6 per acre ft. For 143 acre ft. this value amounts to $930 per day or $223,000 for 240 day year. The duty of water in this location is 5ft. per acre per year. With a 3% evaporation and seepage loss per mile of canal the total loss in 6 miles amounts to 6,200 acre ft. per year leaving a net supply of 28,100 acre ft. With this duty of 5 acre ft. per year the available supply will serve 5,620 acres. The acreage which can be supplied by gravity is 9,420 ares showing that there is sufficient land to utilize all the water available.

Design of Line.

Alignment

In the design of the system the trunk lines for each district have been run from points slightly below the lowest point of the communities as they exist to-day.These trunk lines have been laid out so as to give the maximum service.This explains why the Glendora District trunk line does not go in the shortest path to its junction with the other trunk lines at Whittier Narrows,but rather goes first to the south to serve the communities of Covina and Baldwin Park.The same object is the reason for not joining the two present outfalls of Pasadena District instead of further diverging them to serve Ramona and that portion of San Gabriel which is not served by any line at the present time.These lines are shown on the accompanying U.S.G.S.Maps.

The two trunk lines serving districts west of the San Gabriel River join in the Whittier Narrows and after passing under the San Gabriel River join the trunk lines from the east.The outfall from this point follows the east bank of the river until it crosses the Santa Ana branch of the So,Pac.RR. below which point the lines from Downey and Norwalk join it at the location of the proposed treatment plant.If an outfall to the ocean be used instead of a treatment plant the outfall will continue from this point a-long the east bank of the river to Los Alam - itos where it will turn to the south east as the sewage could not be disposed of in the neighborhood of Alamitos Bay.A site to the east of the marsh at Bolsas Creek has been taken for the location of a pier to carry the outfall out to sea for a distance of 5000 ft. Other outfalls along the coast are not carried out to distances exceeding 2000 ft. but a nuisance has developed in some cases.For this reason it was deemed advisable to carry the outfall out for the distance stated above . This distance would insure that no trace of the sewage would be seen near the beach as the ocean currents at this location are fa-vorable to the carrying of the sewage out to sea and traces of sewage are not found more than one and a quarter miles from the outlet when sewage is discharged in ocean water. Wood stave pipe will be used for that porti on of the line beyond the pier.

Size, velocity, and quantity.

Table "A" on page 17 shows the various lines and sections together with their lenghts,popu-lation(both present and 70 years hence)served by each ,the normal 1920 flow and maximum 1970 flow in cu.ft.per mim.,the grade ,and the size of pipes to be used with their normal 1920 vel-ocities,maximum 1970 velocities and maximum capacities.The values of the latter items were calculated by Kutter's formula($V=C\sqrt{Rs}$).The grades in a large measure conform to the natural slope of the surface.The crossing of the Rio Hondo wash and the San Gabriel River however, require a deviation from this.In order not to require the construction of a tunnel 19,000ft. in length which would cost approximately $165,000 it was considered more economical to run the line at the surface for 12,000 ft. of this distance,bridge the Rio Hondo for 7,000 ft., and tunnel the remaining 2,000 ft.at a cost of approximately $70,000.These conditions are shown on the accompanying profile.

Velocites have been kept within the limits of two ft. per sec. minimum to 12 ft.per sec. maximum with two exceptions:a velocity of 14 ft.per sec. will result from the maximum discharge in 1970 in a portion of the Pasadena Outfall;while both maximum and minimum velocities in a section below the proposed treatment plant are less than two feet per sec., the minimum 1920 velocity being 1.3 ft.per sec. This low velocity in addition to the large pipe required are added reasons for not using the outfall to the ocean but rather to install a treatment plant at the point designated.

TABLE A

Line	Pop.1920.	Pop.1970.	Normal Discharge 1920	Max.Dis.1970, Cu.Ft.Per Min.	Max.Capacity
1-a	2028	12,000	33.6	403	534
1-b	2028	12,000	33.6	403	623
1-c	4488	20,250	75.6	680	733
1-d	4488	20,250	75.6	680	742
1-e	4488	20,250	75.6	680	860
1-f	4488	20,250	75.6	680	1109
1-g	4488	20,250	75.6	680	833
2-a	5480	16,750	92.4	564	751
2-b	5480	16,750	92.4	564	945
2-c	2026	7,500	34.2	252	334
2-d	2020	7,500	34.2	252	246
2-e	4265	21,750	72.	730	751
2-f	4265	21,750	72.	730	830
2-g	4265	21,750	72.	730	833
2-h	9745	38,500	164.	1295	1934
2-j	9745	38,500	164.	1295	1434
2-k	11028	46,000	185	1540	1576
3-a	6474	35,200	108.5	1182	2083
3-b	6474	35,200	108.5	1182	1288
3-c	58264	316,800	980.	10620	11630
3-d	58264	316,800	980.	10620	11442
3-e	58264	316,800	980.	10620	10658
3-f	64738	352,000	1085.	11800	14113
S	75766	398,000	1272	13350	14113
4-a	80254	418,250	1350	14050	15695
4-b	90251	466,950	1520	15700	15695
4-c	90251	466,950	1520	15700	16721
4-d	90251	466,950	1520	15700	16721
4-e	7997	28,200	134.5	948	1174
4-f	7997	28,200	134.5	948	1342

TABLE A Cont'd.

Line	Max. Vel. 1970	Normal Vel. 1920	Grade	Pipe Diameter	Length feet
1-a	7.25	3.96	0.02	15"	5000
1-b	5.87	3.21	0.0104	18"	12000
1-c	3.89	2.28	0.0031	24"	18500
1-d	5.67	3.32	0.008	20"	7500
1-e	4.56	2.68	0.0044	24"	24500
1-f	3.76	2.2	0.0022	30"	19000
1-g	5.26	3.08	0.0061	22"	9500
2-a	7.08	4.98	0.0148	18"	7000
2-b	3.21	2.26	0.0016	30"	10000
2-c	10.18	6.75	0.0765	10"	1500
2-d	5.23	2.40	0.0141	12"	11000
2-e	7.08	3.77	0.0141	18"	2500
2-f	6.34	3.37	0.012	20"	8000
2-g	5.26	2.80	0.0065	22"	3000
2-h	6.57	4.1	0.0065	30"	3000
2-j	4.02	2.5	0.0024	33"	10500
2-k	5.35	3.35	0.0041	30"	23000
3-a	7.08	3.87	0.0069	30"	9000
3-b	6.85	3.74	0.0091	24"	9000
3-c	7.10	3;77	0.0035	72"	7500
3-d	9.71	5.16	0.0067	60"	7500
3-e	14.13	10.60	0.0205	48"	4000
3-f	8.32	6.24	0.0039	72"	19000
S	8.32	6.24	0.0039	72"	2000
4-a	6.48	4.86	0.0023	84"	20500
4-b	6.48	4.86	0.0023	84"	37500
4-c	6.06	4.54	0.0012	96"	24000
4-d	1.76	1.32	0.0002	120"	57500
4-e	8.97	5.84	0.022	20"	4000
4-f	4.56	3.20	0.003	30"	10000

Design and cost of conduits.

All pipe 24" in diameter and less will be of vitrified clay while larger sizes will be of brick with concrete foundations as shown in the accompanying sketch. Table B gives the cost of the vitrified pipe; table C gives the cost of the brick and concrete pipe.

TABLE B

Cost of vitrified pipe.

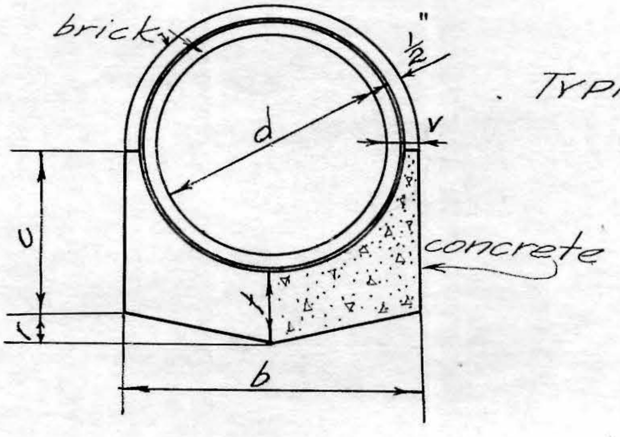
1500'	10" pipe	@	$0.45	=	$ 675.00
11000'	12" "	@	$0.60	=	6600.00
5000'	15" "	@	$0.90	=	4500.00
21500'	18" "	@	$1.17	=	25200.00
19500'	20" "	@	$1.45	=	28250.00
12500'	22" "	@	$1.65	=	20600.00
52000'	24" "	@	$1.93	=	100400.00
			Total		$ 186325.00
	+engr.(15%)				$ 214500.00
	+contr(15%)				$ 246400.00

The above prices are for pipe in place.

TABLE C

Cost of cement and brick pipe.

d	b	c	r	v	t	Brick	Cost	Cu.Yds. Con.	Cost
30"	47.5"	20"	4"	8.75"	4.5"	2620000	$ 91600	1100	$ 15400
33"	50.5"	21.25"	4.25"	8.75"	4.75"	407000	14250	169.5	2380
48"	65.5"	28.5"	5.5"	8.75"	5.62"	227000	7950	97.5	1370
60"	86.25"	33.5"	5.25"	13.12"	6.37"	530000	18580	288.5	4040
72"	98.25"	39.5"	8.25"	13.12"	7.25"	2420000	84700	1375	19250
84"	110.25"	45. "	9.25"	13.12"	8. "	5750000	201500	3380	47300
96"	122.25"	51. "	10.25"	13.12"	8.75"	2715000	95000	1670	23400
120"	146.25"	62.5"	12.25"	13.12"	10.25"	8140000	285000	5380	75600
Unit cost per M brick = $35.00					Total		$798580		$188740
" " " cuyd.Con.= $14.00					Cost of pipe				$987320
					+engr.(15%)				$1135000
					+contr(15%)				$1305500



TYPICAL CROSS-SECTION OF CONDUIT

Excavation

The unit cost for excavation was taken as $ 1.00 per cubic yard for depths not exceeding ten feet and varying directly from this value to $ 2.00 per cubic yard at twenty feet. The cost of tunneling was taken as $ 3.00 per cubic yard of earth removed. Table D shows the cost of said excavation and tunneling.

TABLE D

Line	Width	Depth Ave.	Cu.Yds. per ft.	Length	Totalvol. cu.yds.	Unit Cost	Cost
1-a	2'	8'	.59	5000'	2960	$1.00	$ 2960
1-b	2	16.5	1.22	12000	14650	1.65	24200
1-c	3	15	1.67	18500	30900	1.50	46400
1-d	3	9.5	1.07	7500	8030	1.00	8030
1-e	3	7.25	.81	24500	19750	1.00	19750
1-f	3	10.	1.10	19000	20900	1.00	20900
1-g	3	12.	1.32	9500	12650	1.20	15200
2-a	2	6.	.45	7000	3110	1.00	3110
2-b	3	15.	1.67	10000	16650	1.50	25000
2-c	2	6.	.45	1500	660	1.00	660
2-d	2	10.	.74	11000	8150	1.00	8150
2-e	2	14.	1.04	2500	2590	1.40	3630
2-f	2.5	16.	1.48	8000	11850	1.60	18950
2-g	2.5	12.	1.11	3000	3340	1.20	4010
2-h	3	9.	1.00	3000	3000	1.00	3000
2-j	3.5	6.	.78	10500	8180	1.00	8180
2-k	3	10.	1.10	23000	25300	1.00	25300
3-a	3	6.	.67	9000	6000	1.00	6000
3-b	2.5	8.	.74	9000	6660	1.00	6660
3-c	7	6.	1.56	7500	11650	1.00	11650
3-d	6	10.	2.20	7500	16500	1.00	16500
3-e	5	10.	1.85	4000	7420	1.00	7420
3-f	7	bridge	------	(7000)12000	------	------	------
S	7	tunnel 9.	2.34	2000	4680	3.00	14050
4-a	8	8.	2.37	16500	39200	1.00	39200
4-a	8	tunnel10.	2.96	4000	11850	3.00	35600
4-b	8	10.	2.96	37500	111000	1.00	111000
4-c	9	5.	1.67	24000	40000	1.00	40000
4-d	11	----	------	57500	-------	------	-------
4-e	2.5	8.	.74	4000	2970	1.00	2970
4-f	3	10.	1.10	10000	11000	1.00	11000
						Total	$ 539480.00
					+engr.(15%)		$ 620000.00
					+contr(15%)		$ 712000.00

Right of way. The land which must be acquired for right of way for this project has an average value of $500.00 per acre. Although a right of way 25 ft. wide will be required on the lower end of the outfall the average width required may be taken as 20 ft. With a line 383,550 ft. in length the cost of right of way will amount to $\frac{383,500 \times 20 \times 500}{43560}$ = $89,500.00

Man holes. The number of man holes required cannot be estimated with a very great degree of accuracy but in consideration of the number required on similar projects, 200 man holes was determined upon as a fair figure for the project under consideration. The average depth of man hole will be 10 ft. which with a unit cost of $7.50 per foot plus engineering and contracting costs gives a total cost of $ 19,800.00 for man holes.

Trestle. As previously stated a trestle will be required to cross the Rio Hondo Wash, the average heigth of which will be 5 ft., the maximum to be 15 ft. With bents spaced 16 ft. apart and constructed of one vertical and two battered piles the cost per foot was computed to be $8.00. With this unit cost 7000 ft. will cost approximately $56,000.

Pier and wood stave pipe. From cost data for piers and wood stave pipes on similar projects a unit cost of $1.00 per foot of pier including the pipe was found to be representative. With a 5000 ft. pier the cost of construction will be $500,000.00.

Summary of Costs. The summary of the various costs relative to the construction of the entire outfall to the ocean with no treatment is shown in the following table.

TABLE E

Costs, outfall to ocean-no treatment.		
Item		Cost
1. vitrified pipe	$	246,400.00
2. brick and conc.conduit		1,305,500.00
3. excavation		712,000,00
4. right of way		89,000.00
5. man holes		19,800.00
6. trestle		55,000.00
7. pier		500,000.00
	$	2,928,200.00
approximately	$	3,000,000.00

Design of treatment plant.

Treatment Plant

Necessity of plant.

Treatment of sewage is necessary in order that no nuisance may be created from odors or pollution of streams. During the rainy seasons the effluent will not be used for irrigation purposes and it will be necessary to turn it into the San Gabriel River. Therefore, when sewage must be turned into this natural drainage channel and a high degree of stability is essential towards the maintainance of a high sanitary condition, a purification plant is justified and must be kept in continual operation.

Degree of purification.

The effluent after treatment must always satisfy every requirement of state and local boards of health. It must not contaminate ground waters nor the drinking water supply, nor must it prove a nuisance along the banks of the San Gabriel River or along the ocean front. It must not be favorable to the breeding of mosquitoes or flies and without objection of nuisance generally wherever water may be used. The effluent must therefore be clear, pure, and injurious bacteria reduced to a minimum as is humanly possible.

Type of plant.

We recommend a modified activated sludge treatment plant, because we are of the opinion that a plant of this type will give under proper management and design a stable, non-putrescible effluent. This type of plant has proven efficient, has low air consumption requirements, and fairly high nitrogen recovery capacities from the sludge.

Description of process.

The sewage is essentially of the domestic type and will accordingly be treated as follows:-

First:

It will be screened before entering the plant by a self-cleaning revolving drum screen which will catch and remove large matter, tough and non-digestible solids, and permit the passage of the soft and easily digested solids. These screenings may be burned or dried and compressed and sold for its fertilizing value in accordance with market demands.

Second:

The sewage will then pass thru a Venturi meter which will automatically register and record the flow of sewage at all periods. This meter will be large enough to take care of 130.5second feet of flow in 1970.

Third:

From here the sewage will flow directly into the aeration-sedimentation tanks. These tanks are divided into two compartments by a horizontal tray placed 0.6 of the height from the bottom of the tank. Sedimentation will take place in the upper compartment andaeration in the lower compartment. On the bottom of each tank are filtros acting as the diffusing medium. The exposed surfaces of these filtro tiles are swept and kept clean by two radial arms with squeegees attached and suspended from a central vertical shaft. As the sludge settles in the sedimentation compartment it is shoved out by these arms towards the periphery of the tank and caused to drop into the aerating compartment thru downcast wells. In this lower compartment the sludge settles on to the filtros but is immediately pushed out again towards the periphery. In this manner a circulation of sludge is effected and a concentration of fresh sludge kept in the aerating chamber. A clear overflow is removed continuously around the top of the tank by means of a peripheral launder. Provision is made for withdrawing accumulated sludge which may build up and escape with the overflow in the final sedimentation chamber.

Approximately 75% of the air is introduced into the first unot and 25% into the second unit. An important feature of the design is that it utilizes the natural lifting effect of the air and increases circulation of the activated sludge and the incoming sewage feed. This gives an intimate and prolonged contact between the air, biologic flocs, and sewage, with resulting air economy.

The main walls of the tanks will be constructed of reenforced concrete.

Fourth:

From these tanks the sludge is withdrawn and spread on drying beds. The beds will have 3 in. sub-laterals connecting into 6 in. main laterals. These laterals will be of tile material. On these sub-drains will be place sand and gravel one foot thick. When the sludge becomes sufficiently dried it will re - moved by cars run directly to the beds on two-foot gauge track, with 12 lb. rails and steel ties. The supply of sludge will be so regulated that each bed will be cleaned in rotation and maintainance costs reduced to a minimum.

Fifth:

After the sludge is removed it will be slod directly to users. The demand for this sludge must be developed by dissemination of propaganda, but we are of the opinion that a market already exists as farmers in Los Angeles County have knowledge of fertilizing value of sludge and will make use of same where land is difficient in nitrogeneous matter.

Design of tank.

Normal flow in 18 hours will be

$$\frac{46,695,000}{18 \times 7.5} = 345,000 \text{ cu.ft.per hr.}$$

Each tank has capacity for 15,000 people. Population in 1970 will number 466,950. Therefore the number of tanks neccessary in 1970 will be

$$\frac{466,950}{15,000} = 31.$$

Year	1930	1940	1950	1960	1970
Population	180000	252500	325000	400000	466950
No. Tanks	12	17	22	27	31

Maximum capacity of each tank

$$\frac{345,000}{31} = 11,200 \text{ c.f.}$$

Detention period will be 9 hours. Therefore required capacity of each tank will be

$$9 \times 11,200 = 100,800 \text{ c.f.}$$

Size of tanks will be 65 ft. in dia. and 30 ft. in depth

Design of sludge chambers.

Each chamber will have a capacity of 11,820 c.f. which will take care of 15,000 people for five months based upon 0.00525 c.f. of sludge per capita per Day. For every 1000 people contributing sewage there will be required 350 sq.ft. of drying area. Therefore, for

466,950 people an area of 164,500 sq.ft. will be required. There will be 55 separate drying beds, each 100 ft. in length and 30 ft. in width, separated by 2 in. redwood partitions. Thru the center ofthe bed there will be 12 in. concrete partition wall. Following is a table giving the required number of beds at each ten year interval.

Year	1930	1940	1950	1960	1970
Chamber	21	30	38	47	55

Design of screens.

The screens will be 1/2" by 2" iron bars spaced 2" and inclined at an angle of 45 degrees to the horizontal.

Economics of treatment plant.

CONSTRUCTION

Bar screen and chamber					$ 450.00
Venturi meter					3000.00
Year	1930	1940	1950	1960	1970
Tanks	$386000	547000	707000	868000	997000.00
Beds	18350	26300	33200	41200	48100.00
Miscellaneous					5000.00
Cost of land (plant and beds only) 10 acres @ $500.00					5000.00
Total					$1,058,550.00
+engr.(15%)					$1,215,000.00
+contr(15%)					$1,395,000.00

MAINTAINANCE

Chemist and bacteriologist	2500.00
5 men @ $0.50/hr.	
1 foreman @ $0.75/hr.	
1 nightman @ $0.55/hr.	10950.00
Labor per year $	13450.00
Power	97000.00
Depreciation (2%)	5000.00
Contingencies	3000.00
Total	$ 118450.00

REVENUE

Undissolved NH_3 3400#

Available " 3400#×65%(settles out)×75%(is lost) will give 1660#

Value of sludge @ $0.35/# for one year $ 212000.00

Gross earnings in 1970 $ 93550.00

Calculations of construction:

Sludge beds

concrete

(excavation) bottom 100×30×1÷27=111.5 @ $0.75 $ 83.70

sides 100×2×1.5÷27=11.15cu.yds.

ends 30×2×1.5÷27 = 3.33 "

partition 30×1×1.5÷27=1.67 " @ $14=226.00

Underdrainage system		$ 50.00
Sludge pipes and valves		75.00
Sand 27.9 cu.yds. @ 1.50		41.80
Gravel 139.3 cu.yds. @ 2.25		300.00
Wooden walls		25.00
Total per bed		$ 801.50
for 55 beds	$ 44,200.00	
Track, switches and four cars	3,893.00	
Total(1970)	$ 48,093.00	

Tanks	
Mechanical equipment	$ 21,411.00
Material and pump house	$ 10,737.00
Total	$ 32,148.00
for 31 tanks(1970)	$ 997,000.00

Design and cost.

Irrigation Canal.

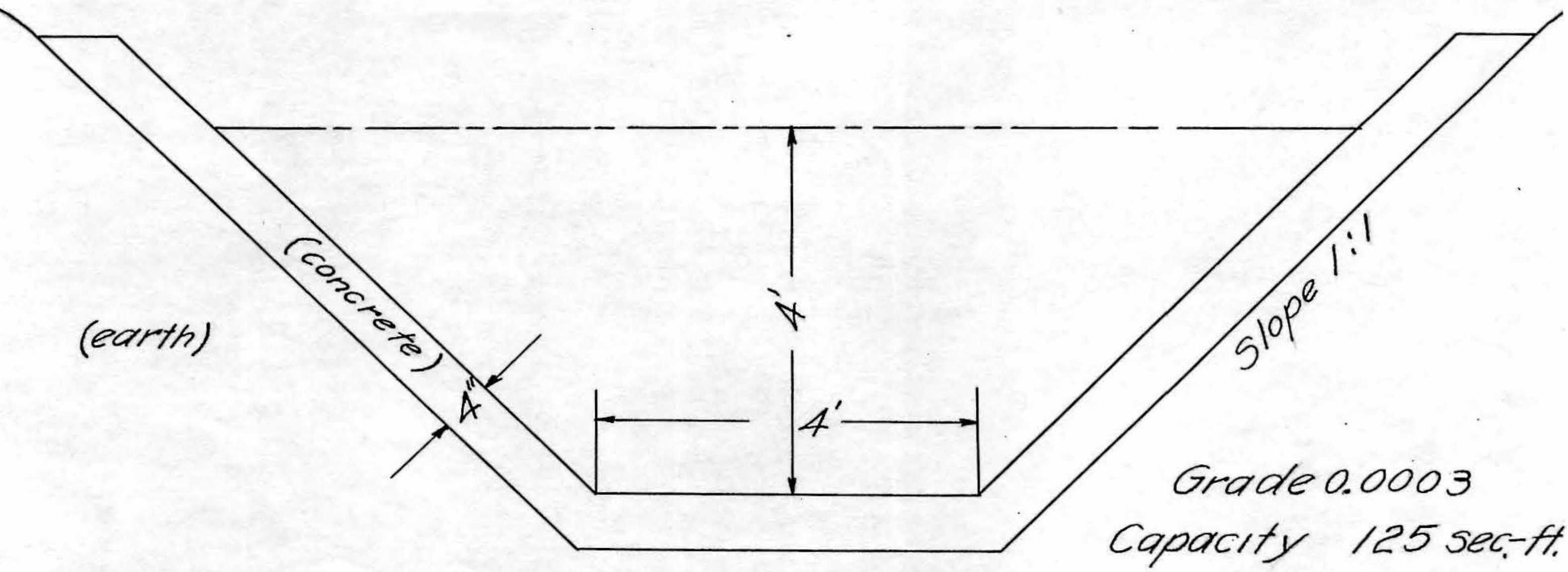
In conjunction with the treatment plant below Whittier there will be required canals to transport the effluent to the land to be irrigated, this land being designated on the project map. Only the main canal is here investigated , said canal to run to the east from the treatment plant keeping as high as the grade will permit. To insure against large seepage losses the canal is to be lined with 4" concrete. This canal will be 31,700 feet in length and will have a capacity of a 125 sec.ft. A cross section of the canal is here shown. The cost of this canal will be ;

$1.05 per foot for excavation
$1.17 per foot for concrete lining
$2.22 per foot--total cost

31,700 ft. @ $ 2.22 per ft. = $ 70,500.00

The right of way required for this canal will 20 ft. wide and 31,700 ft. long which at $500.00 per acre, will cost $7,300.00.

Cross Section of Canal



Saving on cost of line with treatment plant.

Relative costs.

In order to determine the advisability of such a system as proposed and to compare the relative costs of an outfall to the ocean with no treatment to a shorter outfall with treatment plant and system for irrigation the following savings of the latter method have been computed. As the water discharged from the treatment plant will be utilized for irrigation in the summer and will be wasted into the San Gabriel River in the winter ther will be no need of an outfall below this point. This embodies a saving of $500,00.00 for pier, $540,500.00 for conduits, $40,000.00 for excavation, $18,500.00 for right of way, and $2,000.00 for man holes. These items represent a saving of $1,325,500.00.

Increased cost of with treatment plant.

As has been previously been stated, the treatment plant will cost $1,400,000.00, while the canal system represents an additional sum of $70,500.00 making a total increased cost with the use of the treatment plant of $1,470,500.00.

Summary of costs.

The summary of costs relative to the construction of outfall, treatmentplant, and irrigation system is given in the following table:

TABLE F

Costs. Outfall with Treatment Plant.

	Item	Cost
1.	vitrified pipe	$ 246,400.00
2.	brick and conc. conduit	540,500.00
3.	excavation	672,000.00
4.	right of way	71,000.00
5.	man holes	17,800.00
6.	trestle	55,000.00
7.	treatment plant	1,400,000.00
8.	canals	70,500.00
9.	canal right of way	7,300.00
	Total	$ 3,080,500.00
	approx.	$ 3,000,000.00

Revenue and operatign expense

With the use of a treatment plant there will be a revenue which at present prices would be $ 212,000. for sludge and $ 223,000. for effluent in 1970 providing there be a market for same. These two figures represent a revenue of $ 435,000. in 1970, which with operating expense of treatment plant of $ 118,450. gives a net revenue of $ 316,550. As prices are varyable and demand uncertain no definite conclusion can be reached as to the actual value of the byproducts of the treatment, but under present conditions this revenue for the fifty year period would amount to $ 7,900,000.

Interest

With an investment of $ 3,000,000. the interest charges on fourty year bonds to pay 6 % would be $ 7,200,000. It will be seen that this figure is approximately the same as that for accumulative income on the fifty year period from the treatment plant, showing that it may be assumed that the income from the plant will pay the fixed charges of the project.

Conclusion

Facts determined

In order to judge clearly the comparative merits of the two methods investigated, namely, an outfall to the ocean with no treatment, and a short outfall with treatment plant and irrigation system, the facts determined will be enumerated.

First. The required investment is approximately the same in both cases.

Second. The revenue from the treatment plant may be assumed to pay the fixed charges.

Third. The effluent from the treatment plant will supply a great need of farmers in the district.

Fourth. An ocean outfall would cause strife between the communities served and the beach cities even though no nusience be created.

Fifth. The outfall below the proposed treatment plant would be at such a flat grade that velocity would be far below the allowable minimum.

Sixth. The treatment plant and irrigation system are along the line of present endeavor, that of utilizing all available resources for production.

Reccomendations.

In consideration of the facts mentioned, all of which would tend to point toward the treatment and utilization of the sewage and effluent, we reccoment that a system as previously described be constructed to serve the communities in the San Gabriel Valley with the exception of Pomona, Lavern, and Claremont, said system to consist of trunk lines from the various districts joining on the east side of the San Gabriel River at the point in the Whittier narrows as designated on the map of the project.

We further reccomend that a treatment plant of the activated sludge type be constructed on the outfall at a point below the crossing of the Santa Ana branch of the S.P.R.R. and the San Gabriel River.

We thirdly reccomend that an irrigation system be constructed from said treatment plant to serve the land to the south east.

We finally reccomend that the waste water from said plant be wasted into the San Gabriel River during the winter when it is not needed for irrigation purposes.

Respectfully submitted,

Louis Korn

Edward Seaver