REPORT
ON THE
SEWAGE DISPOSAL
OF
THE CITY OF SAN DIEGO

BY

OLMSTED & GILLELEN

1919

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Allen H. Wright, City Clerk

By W.E. Bartlett, Deputy.

R E P O R T

on the

S E W A G E D I S P O S A L

of

T H E C I T Y O F S A N D I E G O

by

OLMSTED & GILLELEN

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OLMSTED AND GILLELEN
CONSULTING ENGINEERS
1112 HOLLINGSWORTH BUILDING
LOS ANGELES, CAL.

January 28, 1919.

Mr. F. M. Lockwood,
Manager of Operations,
San Diego, California.

Dear Sir:

We herewith submit to you our report upon the collection and disposal of the sewage of San Diego. This report is the result of a year's study and consideration. All the field work, such as the measurement of sewage flow, was done by the City Engineer's Department. These measurements were of necessity for only short periods and we recommend that more extensive measurements be taken before deciding the final capacity of the proposed improvements.

We wish to acknowledge the valued cooperation and assistance of yourself and the City Engineer's Department.

Respectfully submitted,

OLMSTED & GILLELEN,

By Frank Gillelen

R E C O M M E N D A T I O N S

Two general plans are possible for the improvement of the present sewage disposal for the City of San Diego. One is the construction of a separate disposal plant near the end of each of the existing outfalls; the other is the collection of all of the sewer flow, except that from the 32nd Street outfall, and the installation of a single treatment plant in the vicinity of Beardsley Street and Railroad Avenue. Although the first plan can be executed for less money than the second, the installation of a single treatment plant is more desirable.

We therefore recommend that the city build the intercepting sewers herein described, the small pumping stations for the territory below these interceptors, and install treatment plant in the vicinity of Beardsley Street and Railroad Avenue, and a small plant near the end of the 32nd Street outfall. Clarification of the sewage will remove a sufficient quantity of the total suspended matter to make the discharge of the effluent into tidewater unobjectionable, and we recommend that at both the Beardsley Street plant and the 32nd Street plant, fine screens be installed.

SUMMARY OF COST OF THE RECOMMENDED IMPROVEMENT

Intercepting sewers	$ 320,000
Screening Plant	225,000
Small pumping stations	32,500
Screen at 32nd Street outlet	25,000
Pumps at 32nd Street outlet	4,000
TOTAL	$ 606,500

PRESENT SANITARY CONDITIONS

The sewage from the main portion of the City of San Diego is at present discharged in a raw state into the bay at five different points known as the Olive Street, Market Street, 8th Street, Beardsley Street and 32nd Street outfall sewers. If a bulkhead were built along the waterfront so that at no point could the floor of the bay be exposed by low water, this discharge of crude sewage into the bay might be less objectionable, though highly undesirable under any circumstances. As it is, with several hundred feet of mud flat exposed on some tides, the accumulation of the slime and putrifying matter from the city's sewers is an offense and menace to the health of every person having occasion to use the ferries or visit the many plants and business houses along the water front. Your health department has condemned this crude and needless method of waste disposal and this condition can not continue long without condemnation by the State Board of Health.

At the foot of Market Street, the elevation of the last manhole is -8.0, City Datum, or +1.01 U. S. Coast and Geodetic Survey Datum. This outfall was originally built with some sort of a tide gate at its extreme ocean end, the idea presumably being to store the sewage tributary to this outfall for such a period as it could not discharge against the tide. The failure of such a device with a constantly increasing flow is made apparent by an examination of Plate No. 2, where it is seen that the tide is above elevation + 1.0 U. S. C. & G. Survey datum, 84% of the time. Having proved a failure, this device was wrecked, leaving the Market Street intercepting sewer subject to the action of the tides. The sewer cannot discharge freely when the tide is at or above +1.0 U. S. C. and G. Survey datum, which means that for about 80% of the time sewage is backed up in the Market Street sewer, sometimes as far as Third and K Streets, a distance of three quarters of a mile, the distance depending on the height of the tide. This produces a condition equivalent to a long horizontal cesspool beneath the city's streets, the gases from which are free to escape through the manholes and service connections.

The collecting system of sewers should be revised by the construction of a new interceptor or by the installation of pumping plants, so that the flow is not disturbed by the rise and fall of the tides. The surcharging of sewers located below the elevation of ordinary high tide is undesirable and always results in future troubles. In some instances, the construction of high line interceptors above the high tide mark will prove to be economical, while under other conditions a pumping system is the cheaper method. In any city located on tide water, there are some areas that are too low to be sewered by gravity and the sewage from these districts must be pumped. However, careful location of gravity flow interceptors will reduce the pumping areas to a minimum and thus reduce the annual operating expense for pumping and its necessary attendants. Two general schemes of improvement are applicable in the City of San Diego. One is the construction of a new interceptor that will pick up all the existing outlets except that at 32nd Street at such an elevation that about 85% of the total sewer flow can be conveyed by gravity to a common point; the other is the installation of pumping plants at the Market Street and 8th Street outlets so that the sewage can be discharged freely at all tides. Under either the interceptor system or the pumping system, the sewage will require treatment before it can be discharged into the bay, but the type of treatment plant will be the same in either case and the cost will be substantially the same. The choice between the interceptor system and the pumping system will affect the treatment work only in the matter of location; under the first collection method the treatment plant for the entire sewage flow, except the 32nd Street district, can be located near Beardsley Street and Railroad Avenue, while the second method will necessitate a disposal works to be built at each of the present outlets. In order to investigate the merits of each of these two general schemes, detailed estimates of the cost have been prepared. Investigations were made into the quantity of sewage to be handled, the present and future population of the city, the character of the various sewage districts, the density of the population etc.

PERIOD OF DESIGN In designing a sewage collecting system it is necessary to provide for a reasonable period of growth and at the same time not pass an economic limit which is set by the cost of construction. It would be poor practice to provide for a population so far in the future that the full efficiency of the sewer would not be realized during the life of the bond issue provided for the improvement. In accordance then with precedent and experience, a period of 40 years is taken as the economic basis for design and the interceptor is large enough to meet the requirements of the year 1960.

AREA The growth of the city in area, except to the east, will have no effect upon the design of the improvements proposed. To the north any addition of territory must find its drainage into the San Diego River. Point Loma and adjacent areas are cut off from the sewers herein considered by what was once the bed of the San Diego River, leaving only that territory known as East San Diego, Normal Heights and Encanto as possible additional drainage to the existing outfalls.

POPULATION Plate No. 3 shows the growth of the city from 1890 to 1918. It likewise shows the estimated growth for the coming 40 years. This curve is the result of a study of a number of cities similarly situated, together with a careful consideration of San Diego's industries, resources and location. To serve as a guide and indication of the growth to be expected, curves showing the growth of San Francisco and Milwaukee have been placed on the plate. The curves of the other cities with which San Diego has been compared are omitted to avoid confusion on the diagram. These curves of the two respective cities show the known rate of their growth from the time when their populations approximated 90,000 persons to a date 40 years later. The rapidity with which California has grown and the tendency of its cities to spasmodic augmentation of population precludes an accurate prediction of future rate of increase. From the data available, a future population of 400,000 seems to be a reasonable assumption in view of the many natural advantages the city enjoys, and to which it is rapidly turning its attention.

DISTRIBUTION OF POPULATION

On Map No. 2 is shown the distribution of population by precincts as computed from the registration for the year 1916, using a ratio of population to registration of 2.6. The year 1916 was chosen as being more representative of the true population owing to the abnormal fluctuation in 1917 and 1918 caused by the war. In order to show graphically and more clearly than might be shown by a written density for each precinct, the distribution of population has been represented by isoplephic lines which pass through the center of gravity of districts having identical population densities. The same map shows the distribution of buildings as determined from the Sanborn Insurance Maps, corrected to 1918, and a survey of the city. The purpose of such information and its bearing on the design of a sewage improvement is twofold.

It serves as a guide to probable future densities and as a check on the runoff per capita from sewer gagings. Obviously, discrepancies exist, and accurate forecasting in any sense is beyond human power, but from a collection and consideration of all available data and the determination of the proper weight of each fact in its relation to other known facts, a conclusion as to future conditions can be derived which approaches a mathematical probability.

No attempt has been made to allot a definite future population to each precinct as the carrying of prognostications to such a fine point is deemed unwarranted in view of the assumptions made. Instead, certain general conclusions have been reached and established as the basis for design with the belief that the result will be as reasonably accurate as that which might be obtained in any other manner. After a close study of the local tendencies toward building and consultation with those familiar with property values and the past history of the City, the area tributary to the sewers herein considered was divided into future districts, such as Commercial, Industrial, Apartment House and Residential. Of the present population, 89.6% is included in the area considered as tributary to the outfalls mentioned. It is estimated that when the City has attained a population of 400,000, the percentage residing in this district, which by that time will be extended to include East San Diego, will not exceed 60% of the total.

To each district has been assigned a probable future population density as determined by present conditions and a study of the distribution of population in similar districts in other cities. Map No. 3 shows the boundaries of these future districts and TableNo. 2 shows the future population density.

The residence districts are divided into two types which for convenience will be referred to as Type One and Type Two. The distinction between these types is chiefly due to the size of the lots, the density of the population and the runoff per capita. There can be no definite line drawn between the two types as one merges into the other and some times one type is intermingled with the other in a common drainage area.

MAIN DRAINAGE AREAS The main drainage areas of the city, tributary to the five outfall sewers above mentioned, are shown on Map No. 1. With the exception of the area tributary to the 32nd Street outfall, the future growth of the city will not affect the boundaries of these drainage areas which are fixed by the natural topography. It is not always true that topography can definitely determine a sewage drainage district, as in many cases they may be materially enlarged by pumping sewage from a district which could not flow into it by gravity. In the case of San Diego, however, it is thought reasonably safe to assume that in view of the opportunity for separate disposal for the districts to the north and west, these may be disregarded in considering the design of sewer improvements for the city proper.

QUANTITY OF SEWAGE Gagings were made under the direction of the City Engineer's office, at Quince and Californka Streets and at Nutmeg and California Streets to determine the present flow from this district. At Gaging Point No. 1 measurements were taken covering a period of one week, from January 2nd to 9th 1918.

In lieu of complete 24 hour gagings for flow, the following method was used in the application of the gagings to the computations for design. A conventional curve, Plate No. 10, computed from the hourly variation curves of many cities, was adopted showing the percentage of the total daily flow passing through the sewer at each hour of the day. The quantity of sewage passing the gaging point during the period in which measurements were taken was then equated to that portion of the total daily flow represented by the area under the conventional curve between the hours of gaging. Having thus determined the total daily flow, the average hourly flow was next computed and a curve plotted, using the same percentages that each hourly flow is of the average, as shown on the conventional curve adopted. A representative day was taken from the gagings made at each point, and a curve plotted as above described and as shown on Plates No. 7 to 10. Superimposed on these curves are the curves of the actual gagings. An inspection shows that the assumptions made are not unreasonably at variance with the actual gagings. Table No. 1 shows the result of computations made on the above basis from the measurements taken at each gaging point.

As an example to further illustrate the method used, a detailed case worked out thus:

At Gaging Point No. 1, the flow on Friday, January 4, 1918, from 8:45 a.m. to 4:00 p.m. was determined to be 92,290 gallons. From Plate No. 10, it is found that the flow between the hours of 8:45 a.m. and 4:00 p.m. is 37.6% of the total flow for the 24 hours. Taking 92,290 gallons as 37.6%, the total daily flow is computed to be 245,000 gallons, with an average flow of 10,225 gallons per hour. Taking this as the average we find from Plate No. 10 that at 12:00 m. the flow is 70% of the average; at 2:00 a.m. it is 65%; at 4:00 a.m., 74% etc. In this manner the flow curve for the entire day is plotted.

A close relationship generally exists between water consumption and sewage flow. But this is not always true as evidenced by the inconsistency between sewage gagings and metered water consumption, as determined not only in San Diego but in other cities. As the city water department has no measuring device on the main feeders between the balancing reservoirs and the smaller distributing mains, it is impossible to obtain information sufficient to show the hourly fluctuation in the water consumption. A curve showing the daily variation for a week in July and a week in November, 1918, is given on Plate No. 6. The water supply of La Jolla, Point Loma, Old Town, Ocean Beach and Pacific Beach, estimated by the city water department as 2,000,000 gallons per day was deducted, as this territory is outside the area considered in this report.

The average per capita consumption of water in the City of San Diego as given by the 1918 Handbook of the Neptune Meter Company is 137 gallons per 24 hours. In establishing the average sewer flow per capita the following data has been considered:

Mr. Kuichling in an investigation of 100 of the largest cities in the United States and Canada found the following average water consumption:

For cities of population of		Consumption per capita
1,000,000	and more	106 gallons
600,000	to 300,000	122 "
300,000	to 100,000	106 "
100,000	to 50,000	105 "
50,000	to 30,000	105 "

From statistics of the consumption for 1900 in 136 cities having a population exceeding 25,000, the relation of consumption to meters is roughly given by the following averages:

Percent of taps metered	Average consumption gallons per capita
Less than 10	153
10 to 25	110
25 to 50	104
more than 50	62

The Neptune Meter Company gives the following data concerning the relation between metered taps and water supply:

Percent of water metered	Number of cities	Average consumption
100%	26	85
90 - 99	23	109
80 - 89	6	128
70 - 79	13	103
60 - 69	14	113
50 - 59	13	117

Average flow of sewers from residential districts, Cincinnati, Ohio:

Average flow of sewers from residential districts, Cincinnati, Ohio.

Sewer District	Area in Acres	Population		Sewage flow from actual gagings				No. of gagings cover-ing 24 hour day	Dates of gagings
				Gallons per acre per day		Gals. per cap. p day			
		Total	Den-sity	Avg.	Max.	Avg.	Max.		
Ross Run	1,617	17,912	11.1	1,028	2,820	93	254	2	Dec. 3, 4
Mitchell Ave.	1,650	14,781	9.0	687	1,440	77	160	5	Nov. 19, 20, 21, 22, 23
Totals and aver-ages	3,267	32,693	- - -	857	2,130	85	207	- - - -	- - - - - -

Summary of Data obtained from gagings of Dry Weather sewage flow, made in 1910, Philadelphia, Pa.

Name of area	Character	Area in acres		Population census 1910		Average dis-charge per 24 hours, gallons	
		Total	Settled 1910	Total	Per settled acre	per settled acre	per capita
Thomas Run	Residential, mostly	320	240	15,012	62.5	14,200	227
	pairs of two &	426	337	21,677	64.0	9,860	153
	three-story houses	1,094	627	26,336	58.0	9,850	170
Pine Street	Residential, mostly solid four to six story houses	160	156	15,152	97.0	26,300	271
Shunk Street	Residential, mostly	208	208	25,754	123.0	10,500	85
	rows of two and three-story houses	331	331	37,916	114.0	10,600	93
Lombard St.	Residential, ten-ements & hotels	147	145	16,363	113.0	34,750	308
York St.	Residents and man-	358	354	33,340	94.0	36,000	383
	ufacturing.	58	36	The population contributing sewage is not shown by the census figures		99,250	.....
Market St.	Commercial	123	80			92,800	......

RESIDENCE DISTRICT RUNOFF

The sewer at California and Nutmeg Streets drains a residential district of Type One and gagings at this point show an average daily runoff of 111.5 gallons per capita. The future density of population in this district is set at seven persons per acre. From an analysis of sewer gagings in other cities from similar districts and from a consideration of all applicable data as determined at San Diego, the average daily per capita sewer runoff has been set at 100 gallons.

The average runoff per acre from this district, using the population density above mentioned, is 700 gallons daily.

Gagings taken at Gaging Point No. 2 and Gaging Point No. 6, Map No. 1, show respectively 68.6 and 108.6 gallons average daily runoff per capita. Both of these gaging points are on sewers draining residence districts of Type Two.

For the same reasons given for residence district, Type One, the average daily per capita sewer flow has been set at 100 gallons.

The population density of Residence District Type Two, has been set at 12 persons per acre. The average daily runoff per acre is then 1200 gallons.

It is recognized that the water consumption per capita will be materially greater in districts of Type One, than in districts of Type two, but the excess is used for the irrigation of lawns, gardens and grounds in the Type One residence district.

APARTMENT HOUSE DISTRICT RUNOFF Gagings taken at Ash and Columbia Streets, Gaging Point No. 3, on a sewer draining a district of residences, hotels and apartment houses show an average runoff per capita of 120 gallons per 24 hours.

Gagings at B and Arctic, Gaging Point No. 4, on a sewer draining a similar district show an average runoff per capita of 148 gallons per 24 hours. The inclusion in the latter district of a portion of the commercial area accounts for the difference in runoff. The water consumption for the month of July, 1918 for two blocks bounded by Grape and Hawthorne, 2nd and 4th Streets, which were taken as typical of the apartment house district was obtained from the Water Department. It totaled 164,582 gallons for the month, which makes 1090 gallons per acre per 24 hours. This is so low that it is felt some error must exist and in view thereof the flow to be expected from this type of district has been set at 4800 gallons per acre average daily, in accordance with the measured sewer flow together with a consideration of the average water consumption of the city.

COMMERCIAL DISTRICT RUNOFF The commercial area consists of the down town business district which includes hotels, rooming houses, business and office buildings. The runoff of this type of district is composed of: that due to the resident population and, that due to the character of the district. The latter runoff comes from offices, stores, etc. and is entirely independent of the resident population. It would exist if there were no residents in the district. The normal per capita sewage flow from the resident population of a commercial district is equal to or a little less than that from a high class residence district. It is estimated that the flow from the commercial district due to the resident population, is 90 gallons per capita per day. The measured water consumption of a number of hotels, office buildings and stores in the commercial district, all tributary to a short sewer line on which gagings were taken, was found to be 7,412,600 gallons for the month of July, 1918. The ratio between the total area of six blocks in the heart of the business district, namely, those bounded by E, B, 4th and 6th Streets, and in which many of the buildings above mentioned are located, to the area of the buildings situated in these six blocks, was found to be 54%. If the buildings for which the water consumption wax taken were adjacent instead of being scattered through several different city blocks, they would occupy 10.1 acres, including street and open areas. The daily water consumption figured on the basis of the month of July, would then amount to 25,000 gallons per acre. From a study of the monthly variation of the water consumption in many towns, it is found that the consumption in July is 130 percent of the average monthly consumption for the year. On this basis the average daily consumption per acre for the commercial district amounts to 18,500 gallons.

Gagings taken at 3rd and J Streets, as summarized in Table No. 2, show an average daily flow of 33,230 gallons per acre. The discrepancy between the measured sewer flow and the metered water consumption is attributed to the fact that gagings were taken for three days only and for short periods during the heaviest flow. The metered water consumption for a number of blocks in the heart of the business district of Los Angeles for a period of one year showed that the average consumption per day per acre was 28,500 gallons. This fact would serve to bear out the theory that the metered water consumption was of more value for the purpose of design data than the sewer flow measured from the commercial district. The runoff due to the residents in this district is, at 90 gallons per capita per day, 9900 gallons per acre daily. In this character of territory substantially 100% of the water supply reaches the sewers. Therefore the runoff from this district due to sources other than the residents is 8800, making an average total of 18,700 gallons per acre daily.

INDUSTRIAL DISTRICT RUNOFF No gagings were made of a typical industrial district, nor could there be obtained from the Water Department data which might indicate the water consumption of such a district.

The per capita sewer flow from resident population in an industrial district is less than that from any other. The average runoff in similar districts in cities comparable to San Diego, is 40 gallons per capita daily, and it is safe to adopt this quantity here. In addition to the runoff from the residents, there is a flow due to the industrial plants, analogous to that in the commercial district. From the data collected from other cities, the anticipated average runoff from the San Diego industrial district is 3800 gallons per acre daily in addition to that from the residents. The future density of the population in the industrial districts is estimated to be 20 persons per acre and 40 gallons per capita will furnish a daily runoff per acre, of 800 gallons, or a total average daily flow of 4600 gallons per acre.

A summary of districts, together with their estimated future density average and maximum runoff is shown on Table No. 2.

<u>INTERCEPTOR SYSTEM</u> Based upon the foregoing data an intercepting sewer has been designed to convey the sewage to the vicinity of Beardsley Street and Railroad Avenue. Starting from this point, at an elevation sufficiently above high water to allow a discharge on all tides, the route of the intercepting sewer was laid out with a view to obtaining a minimum excavation and a maximum intercepted area. Unfortunately the location of this interceptor falls largely in a district in which the streets are all paved, an item which adds considerably to the cost of construction. The route of the interceptor is shown on Map 3 and is the result of much study and many trials for the most economical location. The areas of the several sewer districts draining to this interceptor with their subdivisions into residential, commercial and indus-

trial areas, the estimated population density, calculated future flow etc. is shown in Table 3.

The study of a sewer intercepting the entire area northwesterly from the Beardsley Street outfall indicated the possibility of omitting the present Olive Street outfall and allowing this district to discharge as at present. This scheme would shorten the interceptor by the distance from Grape to Chalmers Streets and reduce the size of the remainder. With this in mind a separate estimate was made of an interceptor which is termed No. 2 and which ends at Grape Street, as shown on Map 3. Table No. 3 gives the area of the different types of districts, the flow from each type of district in million gallons per day, the estimated total quantity in million gallons per day and cubic feet per second, also the accumulative total flow in million gallons per day and in cubic feet per second at each inlet; all based on the average maximum. Tables No. 4 and No. 5 give the summary of design data including sizes, grades, velocities etc. Plate 2 shows the profile of both interceptors. The route and depth of cut are the same for both, but No. 2 line ends at Grape Street and is of a smaller diameter than that of Line No. 1.

The difference in grades and pipe sizes is shown on Tables above mentioned. Some complication was encountered in the choice of a route from Grape Street north, and alternate profiles are shown, one being located along the lot line between Arctic and

California from Grape to Nutmeg Streets, the other continuing north on Arctic from Grape to Nutmeg, thence west to California, and north on California to Chalmers Street. Either of these locations will necessitate the construction of the sewer on a trestle across certain low ground. This construction, however, is not uncommon and the inevitable raising of the streets at some future time will eventually put these trestle sections under cover.

The sewer grades have been set with a view to obtaining the maximum efficiency of the commercial pipe sizes and at the same time provide scouring velocities.

The estimated cost of construction of these interceptors has been made on the basis of using reinforced concrete pipe. The cost of Interceptor No. 1 is $320,000 and that of No. 2 is $255,000. The details of these estimates are hereto appended. Since a circular section has been chosen for the sewer, no details of the hydraulic elements are included as the characteristics of such a section are standard.

As previously mentioned, the construction of an intercepting sewer does not eliminate the need for pumping the small districts lying below the grade of the new sewer. Plate No. 5 gives a general plan of the pumping station that will be required for these districts. The pumps are located in a dry well in such a manner that at all times they are accessible for cleaning and maintenance. They are installed in duplicate and are controlled by electrical devices which automatically start and stop the pumps and prevent injury to the apparatus in the event of the stoppage of the pump in action.

By present flow is meant the amount that immediate construction should provide for, and takes into account the expected increase during the next few years. The term future flow, is used to designate the quantity to be expected by 1960.

Districts Nos. 2,26 and 46, as shown on Map No. 3 will all be pumped into the proposed intercepting sewer by the pumping stations indicated as No. 1, 2, and 3, Map No. 3.

At pumping Station No. 1 the estimated total future flow from District No. 2 is 625 gallons per minute, which will require five inch pumps operated by 15 horsepower motors when discharged against a total static and friction head of 25 feet.

On the basis of the average future flow these pumps would operate a period aggregating approximately 21.4 hours per day. The yearly cost for power will be $1,065, on the basis of a current charge of 1½¢ per k.w.h. The cost of operation for present requirements would be materially less and is estimated at $120 per year. An allowance of $6,000 should be made for the construction of this pumping plant with its force main, exclusive of the cost of the land.

Pumping Station No. 2 would be located on the Market Street outfall at Atlantic and Market Streets. This Station would elevate the sewage from District No. 26 and deliver it to the main interceptor at 5th and Market. Table No. 3 gives the total area of District No. 26 and 335.29 acres, the estimated future maximum as 6.56 million gallons daily which is equivalent to 1820 gallons per minute average.

Eight inch pumps, operated by 25 h.p. motors will be required and the operating expense of pumping the estimated future flow is approximately $2,000 per year, on the basis of the current charge above mentioned. The present annual pumping cost will be $500. The estimated cost of the pumps, pump pit, motors, control equipment, building and force main is $18,000.

Pumping Station No. 3, located at the foot of 8th Street outfall, will elevate the sewage from District No. 46. The pumps will discharge into the interceptor at 9th and K Streets. Table No. 3 gives an area for this district of 72.29 acres and a maximum future runoff of 0.831 million gallons daily, which is equivalent to 232 gallons per minute. Four inch pumps operated by five horsepower motors are advised and the yearly power charge is estimated at $235. The amount necessary for the construction of the plant complete, including force main to the interceptor, is $7,000.

<u>PUMPING SYSTEM</u> The alternative to the construction of the intercepting sewer is the installation at each outfall northeasterly from Beardsley Street of a separate means of treatment before discharge into the bay, excepting the 8th Street outfall, the area tributary to which is so small that in the event of the

installation either of interceptors or separate plants, a pumping station is believed more practicable. At Olive Street and Beardsley Street the elevation of the existing sewer is sufficiently high to allow for a gravity discharge against high tide. The elevation of the present sewer at Market and Atlantic Streets is +1.0 U. S.C. and G. Survey data. In order to determine the percentage of the time that the tide is above any given elevation, computations were made from the government tables of predicted tides at San Diego for a period of one year. The time during which each tide staysabove any given elevation, ranging from zero to 6.5, was calculated. In the course of this investigation, Plate No. 1 was developed which materially served as a time saver in obtaining the final results. By means of this curve it is possible, knowing the range and duration of any tide to rapidly calculate percent of time the tide is above any given point. Plate No. 2 is the mean of the monthly curves computed. From this plate it is evident that with the Market Street sewer at an elevation of +1.0, U. S. C. & G. S. data, it can discharge by gravity only 15% of the time.

As the interceptor is designed for the ultimate future flow, the same basis is used for estimating the cost of installing and operating the larger pumping station that will be required at the Market Street outfall if the separate treatment plan is adopted.

The ultimate sewage flow to be expected at the Market Street plant is estimated as follows:

580 acres	Apartment House	@ 12,000	=	6,960,000
580 "	Commercial	46,700	=	27,200,000
368 "	Industrial	11,500	=	4,240,000
133 "	Residential #2	3,000	=	399,000
1661 Total				38,799,000

38,799,000 gals. per 24 hrs. maximum = 60 cubic ft. per sec.
15,520,000 " " " " average = 24 " " " "

Allowing for a total head against which the pumps would operate, of 25 feet, a battery of 12 inch centrifugal pumps in duplicate driven by 50 h.p. motors will be required. These pumps would be automatically controlled by means of float switches and electric controlling devices in such a manner than, as the tide rises above the elevation at which the sewage can flow into the ocean by gravity, the pumps would be thrown in one at a time as the fluctuating flow of the sewage demanded. Extra units would be retained as relays in case of the stoppage of one or more pumps, and would be automatically started by sewage reaching a higher level in the sump.

The estimated cost of the pumps, motors, control apparatus, storage sump, and building, is $42,500. The estimated yearly charge for power on the basis of the ultimate average flow pumped 85% of the time, amounts to $12,000 per year. The estimated cost of pumping the present average flow from this district is $3,600. The total operating charges for a period of 40 years would then be $312,000 which added to the total installation cost makes a total expenditure for the 40-year period, of $354,500.

At 8th Street the pumping plant necessary has been described above. The ultimate yearly power cost at the end of the 40-year period has been estimated at $225, the first cost of the plant and force main at $7,000 and the total power charges for 40 years at $7,500.

INTERCEPTORS VS. PUMPING

Interceptor System No. 1
Construction Cost.

Interceptor No. 1	$ 320,000
Pumping Station No. 1	6,000
" " No. 2	18,000
" " No. 3	7,000
Total	$ 351,000

Interest and operation costs to 1960:

Interest on $351,000, 40-year serial bonds @ 5% .	$351,000
Operation Cost, Pump. Sta. No. 1, to 1960 . . .	23,700
" " , Pump. Sta. No. 2 to 1960 . . .	48,000
" " Pump. Sta. No. 3 to 1960 . . .	7,500
Total	$430,200

Total construction, interest and operation cost to 1960:

Interceptor System No. 1 . . $781,200

Interceptor System No. 2
Construction Cost.

Interceptor No. 2	$ 255,000
Market St. Pump. Sta. 1960 capacity	18,000
Pump Station No. 3, " "	7,000
Total	$ 280,000

Interest and operation costs to 1960.

Interest on $280,000, 40-year serial bonds @ 5%	$280,000
Operation Cost of Market St. pump Sta. to 1960	48,000
" " " Pump Sta. No. 3, " "	7,500
Total	$335,500

Total construction, interest and operation costs to 1960:

Interceptor System No. 2 $615,500

Pumping System
Construction Cost:

Market St. Pumping Sta. 1960 capacity	$ 42,500
Pumping Station No. 3, " "	7,000
Total . .	$ 49,500

Interest and operation and construction cost to 1960:

Interest on $49,500 40-year serial bonds @ 5% . .	$49,500
Operation Cost Market St. Pump Sta. to 1960	312,000
" " Pump Sta. No. 3, " "	7,500
	$ 369,000

Total construction, interest and operation cost to 1960:

Pumping System . . . $ 418,500

The estimates of cost show that the installation of separate disposal plants at the end of each outfall, together with the nedessary pumping equipment, is less expensive than the concentration of the sewage flow for treatment at a single plant in the vicinity of Beardsley Street and Railroad Avenue. In spite of the fact that considerable expense is involved in the construction of a gravity interceptor, it is our opinion that this plan will prove to be more satisfactory.

SEWAGE DISPOSAL At present the sewage is discharged into San Diego Bay from the several outlets, in a raw state without receiving any preliminary treatment whatever. Under certain conditions this method is not objectionable. When the dilution is sufficient and the point of discharge is isolated, discharge of raw sewage into tidal water is generally the most economic and satisfactory means of disposal./ The present condition of the San Diego sewer outlets is the natural result of the growth of the city. Practically every city on tide water is going through the same stages in development of their sewage problem as is San Diego. At the time when existing sewer outlets were constructed one after another there was not sufficient sewage flow to seriously pollute the bay. Today, however, the quantity of sewage discharged through these outlets is sufficient to cause a nuisance and is rapidly becoming more and more objectionable. As the city grows, the objection to the existing method of discharge will become more serious and the expense of improvement will become greater. If the present condition is allowed to continue long enough there is no doubt but that the health authorities will demand improvement.

The proximity of San Diego to tide water renders any high degree of sewage purification unnecessary. Inland cities that discharge sewage into water courses from which other cities, further down the stream,must secure their water supply, of necessity are compelled to purify their sewage to a very high degree regardless of the cost. Simple clarification will be sufficient to render the sewage disposal into San Diego bay unobjectionable. Clarification is in no sense a purification process. The sewage is not altered chemically in any way. By clarification a sufficient portion of suspended solid matter is removed to make the discharge unobjectionable. The total quantity of suspended matter in the sewage is not great and only a part of the total suspended matter causes trouble when discharged into large bodies of tide water. Fortunately that part of the suspended matter which makes the trouble is the portion which is most easily removed.

Purification of sewage is much more difficult than clarification and also far more expensive. Nowhere is the purification of sewage accomplished in a manner which is entire satisfactory. Many processes are employed to accomplish purification, but none of them are universally applicable and the improvements that are being made in the methods of sewage purification are so rapid that the plants installed only a few years ago are now obsolete. Within the last few years the purification of sewage has been largely confined to the activated sludge process as developed and adopted by the City of Milwaukee; septic tanks followed by some form of filter bed; Miles acid process, which has recently been developed by the City of New Haven. There are numerous other methods which have been tried with varying success but they are mostly some variation or re-combination of one or more of the foregoing general methods.

Clarification has been accomplished by (1) plain sedimentation tanks; (2) septic tanks, of which the Imhoff tank is the best type and (3) fine screens. Since clarification is the only requirement at San Diego, all of the purification processes may be eliminated. Of the three mentioned methods of clarification, the first one, or plain sedimentation tanks, will not be considered, since this method has always been objectionable wherever installed, due to the difficulty in handling the sludge which is accumulated in the tanks. The two methods of clarification which should be considered for San Diego are Imhoff tanks and fine screens.

The Imhoff tank is a septic tank constructed in such a way that the low velocity in what is called sedimentation compartment allows portions of the suspended solids to settle by gravity and accumulate in a lower chamber of the tank, which is called the sludge digestion compartment. The plans for Imhoff tanks accompanying this report will illustrate the way in which the tank acts. Imhoff tanks have been used more universally than any other process so that the results which they accomplish are well established. Baltimore, Atlanta, Fitchburg, Plainfield and other cities have installed septic tanks and are now using them as a portion of their sewage treatment plants. The amount of suspended solids ordinarily removed by septic tanks is about 50% of the total sewage content. The suspended solids removed settle into the digestion compartment where it is converted by bacterial action into a dark colored, inert, humus-like substance which can be disposed of without any difficulty or objection. This process of digestion takes about six months so the capacity of the tanks is based upon this storage period. The effluent from the tanks has undergone no purification but the removal of the objectionable portions of the solid matter by the tanks, permits the effluent to be disposed of without causing objection. Finely divided solids, which are still contained in the effluent are rapidly disseminated by dilution and will be gradually oxidized by the ordinary dissolved oxygen in the sea water. The customary method of handling the sludge is to withdraw a portion of it periodically and after drying on sludge beds, to haul it away and waste it by filling low ground. As the sludge comes from the tanks, it contains from 90% to 95% water.

The construction of the sludge bed allows most of this water to drain off through underlying tile drains, the remaining moisture evaporates until the sludge becomes spadeable. Sludge drying beds are at the best a source of annoyance and an eyesore and should be tolerated only as a last resort.

Fortunately San Diego is so situated that it is believed the necessity of such beds may be avoided by a practice which is in use with entire success, in a number of California seaside towns. By the installation of a sludge pumping unit at each tank site, the sludge may be pumped from the tanks on the outgoing tide, through the outfall into the bay where the tidal currents would sweep it into the ocean. The quantity of sludge obtained from Imhoff tanks amounts to about 2.25 cubic feet per 1000 people. Contrary to popular conception, the fertilizer value of Imhoff tanks sludge is practically nothing. The biological process of digestion destroys its fertilizer value so that the dried sludge is a waste product.

The tanks herein considered for San Diego are designed on the following basis:

Quantity of sewage 130 gallons per capita per day.
Detention period, one hour.
Capacity of sludge compartment 5.25 cubic feet per 1000 persons.

Imhoff tanks are almost universally successful in small installations, and where the location of the tanks is in an isolated place not readily visible and a sufficient distance from the nearest habitation, so that odors which occasionally come from these tanks are not noticeable. The difficulties and objections to Imhoff tanks, where they are not isolated, are the large amount of ground space required, the appearance of the tanks, which can readily be recognized as sewage plants, acid foaming in the sludge compartment and the occasional odors which arise from the escaping gases which are formed in the process of digestion. The nearby town of Anaheim which at present is threatened with a damage suit from surrounding property owners who allege the location of the Anaheim Imhoff tanks depreciate their property values. At Plainfield, New Jersey, odors from Imhoff tanks were noticeable at a distance of about one quarter of a mile from the plant. Acid foaming sometimes occurs in these tanks which results inthe boiling or foaming of the sludge in the digestion chamber and results in a very considerable nuisance. While it is true that Imhoff tanks sometimes for months at a time will handle sewage without nuisance, they occasionally do cause nuisance in the vicinity and for this reason are not dependable.

The plan of locating a separate treatment works at each of the existing outlets would not be adaptable for Imhoff tanks. Imhoff tanks could be considered only by adopting the plan of conveying the sewage to one place, which involves as described above, the building of a new intercepting sewer. A consideration of tanks is confined then to the plan of collecting and delivering all of the sewage at the water front in the vicinity of Beardsley and Railroad Avenue. The area required for the installation of tanks at this site is 0.75 acres for a plant large enough to handle the present sewage flow and provision should be made for securing sufficient ground to provide for the future. The provision for future growth should allow an area of about 2.0 acres. From an investigation of real estate values in the vicinity of the proposed disposal plant, the estimated cost of the property is $10,000. The estimated cost of the tanks herein discussed includes housing over the tanks. They will be constructed in accordance with the accompanying general plan. The cost of constructing a battery of tanks to care for the present flow of 8,730,000 gallons per day, will amount to $250,000. The effluent from these tanks can be discharged through the existing outfalls at the foot of Beardsley Street without further treatment and without objection. No sludge beds have been provided in the plans for these tanks.

The method of removing the grosser portions of the suspended solid and silt by fine screens has been developing rapidly during the past ten years and in some ways it has certain advantages over any other means of clarification. Fine screens are now in use in Daytona, Fla., Long Beach, Santa Barbara and Stockton, California, and Brooklyn, New York and plans for the installation of fine screens are under way for Indianapolis, New York City and others.

One of the chief advantages of fine screening is the very small space required and the absence of nuisance of any kind. If a screening plant is kept clean, there will be no nuisance whatever at the plant. Consequently there have been no objections to the location of fine screening plants by surrounding property owners.

The amount of solid matter removed by the screens varies in different places from 15% to 40% of the total suspended solid content. The actual quantity of solid matter removed is not so important in clarification processes as the character of the material removed. Disposal of sewage by dilution into the San Diego bay will always be objectionable unless some preliminary treatment is adopted to partly clarify the sewage and to remove from it that portion of the suspended solids that causes the difficulty at the point of discharge.

In order to compare the cost anddesirability of screens and tanks we have designed and estimated the screening plant to handle the same quantity of sewage as that allowed for the tanks, namely 8,730,000 gallons per day. The plant designed is ample to take care of the immediate future and the ultimate future flow can readily be accomodated by the addition of similar units. The estimated cost of the screening plant is $225,000. The area required for screens at the present time is 0.25 acres, but provision for the future should be made so that double this area could be obtained. The cost of 0.5 acres for this purpose we estimate at $5,000.

The screenings, which is the term used to designate that portion of the solid matter which is removed by the screens, contains about 87% water and when dried produces a fertilizer base of considerable value. At the plant at Long Beach, California, the screenings are incinerated in a destructor. At the time this plant was built, the process of drying the screenings had not been developed so the method of incineration was adopted at this place as the most effective means of disposition of this material.

To determine the practicability and value of drying screenings an experiment was conducted at a reduction plant in San Diego. Five barrels of fresh screenings from the Long Beach screen were shipped to San Diego and taken to a reduction plant engaged in reducing fish scrap to poultry food. Briefly, the process employed in the reduction of the fish scrap consists in cooking with live steam, pressing in a screw press and drying in a rotary steam jacketed dryer. The dryer was thoroughly cleaned before making this test run. The screenings were run through the dryer only, the dried product resembling in appearance and odor ordinary commercial fertilizer. Samples of the dried screenings were analyzed by the chemists of the Reduction Company and by Smith-Emery of Los Angeles. The analysis of the latter

Company showed the lower values, and a copy is given below:

LABRATORY CERTIFICATE

SMITH, EMERY & COMPANY, LOS ANGEEES

Date July 13, 1918.

DETERMINATIONS

(On dry Basis)

Total Nitrogen (N) - - - - - - - -	5.06%
Availaboe Phosphoric Anhydride(P_2O_5)	1.52%
Water Soluble Potash (K_2O) - - -	0.76%

Respectfully submitted,

Smith, Emery & Company,

CHEMISTS & CHEMICAL ENGINEERS

Thequantity of screenings varies with the character of the sewage treated; the average removal amounting to about 2½ cubic feet per day per 1000 population.

Since septic tanks and screens will accomplish substantially the same results, each producing an effluent that can be disposed of by dilution without objection and since the cost of each kind of plant is about the same, the choice must be governed by other considerations. The location of the plant is

the most important factor in the choice of the kind of treatment works to install. Where an isolated location can readily be obtained, the occasional nuisance caused by septic tanks is not very important, but where the locaion of a treatment works is limited to built-up areas, the elimination of nuisance is the most important factor to be considered. As mentioned elsewhere in the report, a screening plant can be located at the end of each of the present outlets, whereas it would be impossible to maintain sewage tanks at the end of these outlets without incurring damage suits from nuisance. If an intercepting sewer should be built so that practically the entire runoff is conveyed to the suggested site at Beardsley Street and Railroad Avenue, it is possible that tanks might be built without incurring serious objection, but even at this location, the success of a tank installation would be doubtful. The batter of screens to care for the entire city's flow could be maintained at Beardsley Street and Railroad Avenue without causing any nuisance whatever.
<u>32ND STREET OUTFALL</u> This sewer is so located topographically that it is impossible to revise the flow by means of a higher level interceptor.

The only way in which the sewage from District 47 can be brought to the Beardsley Street site for disposal would be by the construction of a pumping plant at a point below the junction of the present 24 inch pipe and the proposed main for the Encanto

District. Some 3500 feet of force main would be necessary in addition to a gravity line approximately one and one half miles long.

In view of its remote location, a small separate treatment plant for this district is believed more practicable than to attempt to deliver this runoff to the main treatment plant. Tributary to this small plant would be all of District No. 47 as well as the East San Diego and Encanto Districts.

The future population for which this plant would serve is estimated as 80,000. The estimated future flow from this district is

27 x 11,500	=	310,000	
6610 x 3,000	=	19,850,000	
		20,160,000	gallons per 24 hours max.
		8,050,000	" " " " min. Average?
		31.1	cubic feet per second max.
		12.45	" " " " average.

The elevation of the present sewer at the last manhole on the 24 inch line is -8.43 city datum, corresponding to elevation +0.58 U. S. C.& G.S. datum. From Plate No. 2 it is seen it would be necessary to pump approximately 89% of the time in order that the sewer might discharge at all times. Owing to the difficulty experienced by disturbed flow in Imhoff tanks where the sewage is pumped into the flow chambers, intermittently, as is the case where pumps are located ahead of the tanks, and the high cost of placing the tanks deep enough for the pumps to be located behind the tanks, it is believed a screening plant at this point would prove more desirable and more economical.

On the basis of raising the sewage against a head of 20 feet, the ultimate power cost would be $6,100 yearly, but the present annual power cost will amount to only $1,200.

The estimated cost of a screen plant of sufficient capacity for the present and immediate future is $29,000.

OLMSTED & GILLELEN.

ESTIMATE

INTERCEPTOR NO. 1

Excavation	$ 41,500
Wasting and Backfill	16,200
Sheeting - labor	3,150
" - material	1,450
Flooding	400
Manholes	5,000
Removing paving.	15,800
Relaying	20,000
Pipe delivered on job	137,000
Pipe laying	13,570 218270
Insurance (11% on $90,000)	12,000
Interest (10% on $100,000 for 6 months) . .	6,000
Superintendent.	2,900
Auto	500
Fares	600
Timekeeper	1,100
Tools, Hdwr. & Sundries	1,450
Overhead	8,745
	$ 286,365
Bond, 1.5%	4,300
	$ 290,665
Engineering & Contingencies . . .	29,066
TOTAL	$ 319,731

Gaging Poing	MONDAY Gallons per 24 hrs.	MONDAY Gallons per hour	MONDAY Gals.per cap.per 24h.	TUESDAY Gallons per 24 hrs.	TUESDAY Gallons per hour	TUESDAY Gals. per cap. per 24 h.	WEDNESDAY Gallons per 24 hrs.	WEDNESDAY Gallons per hr.	WEDNESDAY Gals.per cap.per 24h.
#1	260,608.5	10,858.6	130.5	232,450.8	9,685.4	116.4	83,075.2	3,461.4	41.6
#2	494,577.6	20,607.4	64.8	507,592.8	21,149.7	66.3	508,358.4	21,181.6	66.4
#3	287,572.8	11,982.2	99.3	316,243.2	13,176.8	109.2	366,054.4	15,252.2	126.4
#4	981,289.3	40,887.0	130.3	1,250,146.0	52,089.4	166.0	1,250,146.4	52,089.4	137.9
#5	333,390.0	13,907.9	33.8	322,990.0	13,457.9	323.0	340,170.0	14,173.7	340.1
#5A	2,109,300.0	87,887.5	98.9	2,112,790.0	88,032.9	99.1	2,194,120.0	91,421.6	120.9
#6	2,544,570.0	106,023.7	10.50	2,520,336.0	105,014.0	104.0	2,689,974.0	112,082.2	111.0
#7	195,968.5	8,165.3	36.5	193,820.9	8,175.8	36.1	173,418.7	7,225.7	32.3

Gaging Poing	THURSDAY Gallons per 24 hrs.	THURSDAY Gallons per hour	THURSDAY Gals.per cap.per 24h.	FRIDAY Gallons per 24 hrs.	FRIDAY Gallons per hour	FRIDAY Gals. per cap. per 24 h.	AVERAGE Gallons per 24 hrs.	AVERAGE Gallons per hr.	AVERAGE Gals.per cap.per 24h.
#1	148,976.2	6,207.3	74.6	249,025.9	10,376.0	124.7	222,765.3	9,281.8	111.5
#2	508,358.4	21,181.6	52.1	607,886.4	25,328.6	79.4	525,354.7	21,889.7	68.6
#3	382,851.2	15,952.1	132.2	396,462.4	16,519.2	136.9	349,836.8	14,576.5	120.8
#4	1,072,414.4	44,683.9	142.4	1,012,166.4	42,173.6	134.4	1,113,232.4	46,384.6	147.8
#5							332,316.6	13,846.5	332.3
#5A							2,138,736.6	89,114.0	100.3
#6				2,617,272.0	109,053.0	108.0	2,631,812.4	109,658.8	108.6
#7				193,820.9	8,075.8	36.1	191,136.4	7,964.0	35.6

15 rooms per acre

TABLE NO. 2

FUTURE POPULATION DENSITIES AND RUNOFF

Classification	Area	Future density	Total future population	Flow per capita from resident pop.	Flow per acre due to resident pop.	Flow per acre due to character	Total flow per acre average	Total flow per acre maximum 250% of average	Total flow for District, maximum.
Commercial	533	110	58,630	90	9900	8800	18,700	37400 46,700	24,891,100
Industrial	1066	20	21,320	40	800	3800	4,600	9200 11,500	12,259,000
Apartment House	834	40	33,360	120	4800		4,800	9600 12,000	10,008,000
Residence No. 1	449	7	3,143	100	700		700	1400 1,750	785,750
Residence No. 2	10295.5	12	123,547	100	1200		1,200	2400 3,000	30,885,000
TOTAL	13177.5		240,000						78,828,850

	Gallons per capita	Date of design	Ult. Est.
Baltimore	300	1906	1925
Louisville	357	1906	1925
Milwaukee	350	1910	1950
Cincinnati	366	1913	1950
San Diego	328	1913	1960

ESTIMATED QUANTITY OF SEWAGE TO BE PROVIDED FOR AT MAXIMUM RATE IN 1960
SAN DIEGO INTERCEPTER NO. 1.

	District number	Area in acres: Total area	Commercial	Industrial	Apartment	Residence No. 1	Residence No. 2	Million gallons per day: Commercial 46,700 gals per acre	Industrial 11,500 gals per acre	Apartment 12,000 gals per acre	Res. No. 1 1750 g.p.a.	Res. No. 2 3000 gals. per acre	Estimated Total Quantity M.G.D.	C.F.S.	Cumulative M.G.D.	C.F.S.
43	1	503.84				391.12	112.72				0.684	0.338	1.022	1.581	1.022	1.581
42 D	2	110.53		66.58			43.95		0.766			0.132	0.898	1.389	1.920	2.970
41	3	28.13		11.43		16.70			0.131		0.029		0.160	0.248	2.080	3.218
	4	1015.25			184.24	41.09	789.92			2.210	0.072	2.369	4.651	7.196	6.731	10.414
40	5	4.39		4.39					0.050				0.050	0.077	6.781	10.491
39	6	3.73		3.73					0.043				0.043	0.066	6.824	10.557
	7	40.42		4.61	35.81				0.053	0.430			0.483	0.747	7.307	11.304
	8															
38	9	6.37		6.37					0.073				0.073	0.113	7.380	11.417
37	10	9.00	2.41	5.49	1.10			0.113	0.063	0.013			0.189	0.292	7.569	11.709
	11	9.88	6.59		3.29			0.307		0.039			0.346	0.535	7.915	12.244
6	12	174.02	17.80		156.22			0.831		1.875			2.706	4.187	10.621	16.431
35	13	3.07	3.07					0.143					0.143	0.221	10.764	16.652
33	14	3.95	3.95					0.184					0.184	0.285	10.948	16.937
34	15	217.08	94.03		123.05			4.391		1.477			5.868	9.079	16.816	26.016
32	16	4.17	4.17					0.195					0.195	0.302	17.011	26.318
31	17	6.60	6.60					0.308					0.308	0.477	17.319	26.795
	18	2.42	2.42					0.113					0.113	0.175	17.432	26.970
30	19	3.95	3.95					0.184					0.184	0.285	17.616	27.255
29	20	1.32	1.32					0.062					0.062	0.096	17.678	27.351
	21	4.61	4.61					0.206					0.206	0.319	17.884	27.670
28	22	6.37	6.37					0.297					0.297	0.460	18.181	28.130
27	23	11.65	11.65					0.544					0.544	0.842	18.725	28.972
26	24	3.95	3.95					0.184					0.184	0.285	18.909	29.257
25	25	23.73	23.73					1.108					1.108	1.714	20.017	30.971
C	26	335.29	76.90	258.39				3.591	2.971				6.562	10.153	26.579	41.124
24	27	14.50	14.50					0.677					0.677	1.047	27.256	42.171
23	28	27.68	21.31		6.37			0.995		0.076			1.071	1.657	28.327	43.828

	District Number	Area in Acres: Total Area	Commercial	Industrial	Apartment	Residence No. 1	Residence No. 2	Million Gals. per day: Commercial 46,700	Industrial 11,500	Apartment 12,000	Residence No. 1 (1750)	Residence No. 2 (3000)	Estimated Total Quantity: M.G.D.	C.F.S.	Cumulative: M.G.D.	C.F.S.
	29	27.68	21.31		6.37			0.995		0.076			1.071	1.657	29.398	45.485
21	30	29.97	21.31	2.29	6.37			0.995	0.026	0.076			1.097	1.697	30.495	47.182
8	46	72.29		72.29					0.831				0.831	1.286	31.326	48.468
20	31	31.56	18.42	4.57	8.57			0.860	0.053	0.103			1.016	1.572	32.342	50.040
19	32	23.90	16.04	4.57	3.29			0.749	0.053	0.039			0.841	1.301	33.183	51.341
18	33	23.90	16.04	4.57	3.29			0.749	0.053	0.039			0.841	1.301	34.024	52.642
17	34	13.36	8.79	4.57				0.400	0.053				0.463	0.716	34.487	53.358
16	35	9.23	2.64	6.59				0.123	0.076				0.199	0.308	34.686	53.666
15	36	113.38	67.02	5.71	40.65			3.130	0.066	0.488			3.684	5.700	38.370	59.366
14	37	71.33	46.15	3.95	10.11		11.12	2.155	0.045	0.121		0.033	2.354	3.642	40.724	63.008
13	38	743.33	6.37		22.19		1714.97	0.297		0.266		5.145	5.708	8.831	46.432	71.839
9-10-11-12	39	210.28	5		101.08		109.20			1.213		0.328	1.541	2.384	47.973	74.223
6-7-8	40	704.01			121.95		582.06			1.463		1.746	3.209	4.965	51.182	79.188
5	41	21.75		21.75					0.250				0.250	0.387	51.432	79.575
4	42	31.86		31.86					0.366				0.366	0.566	51.798	80.141
	43	28.57		28.57					0.329				0.329	0.509	52.127	80.650
	44	21.98		21.98					0.253				0.253	0.391	52.380	81.041
3 2 1	45	329.37		240.60			88.77		2.767			0.266	3.033	4.693	55.413	85.734
	47	251.59		251.59					2.893				2.893	4.476	58.306	90.210
			533.39	1066.45	833.95	448.91	3452.71									

TABLE NO. 4

SAN DIEGO INTERCEPTING SEWER NO. 1.

Inlet	C.F.S.	Grade	$AC\sqrt{r}$	$\sqrt{S}$	Size	V
1	1.581	.00099	102.14	.0315	1-6"	1.8
2	2.97	.00099	102.14	.0315	1-6"	1.8
3	3.218	.00099	102.14	.0315	1-6"	1.8
4	10.414	.00117	309.23	.0342	2-3"	2.65
5	10.557	.00117	309.23	.0342	2-3"	2.65
6	10.557	.00117	309.23	.0342	2-3"	2.65
7	11.304	.00089	411.27	.0292	2-6"	2.5
9	11.417	.00089	411.27	.0292	2-6"	2.5
10	11.709	.00089	411.27	.0292	2-6	2.5
11	12.244	.00089	411.27	.0292	2-6"	2.5
12	16.431	.0010	532.76	.0316	2-9	2.84
13	16.652	.0010	532.76	.0316	2-9	2.84
14	16.937	.0010	532.76	.0316	2-9	2.84
15	26.016	.00098	836.69	.0314	3-3	3.1
16	26.318	.00098	836.69	.0314	3-3	3.1
17 Broadway	26.795	.00113	836.69	.0337	3-3	3.4
18	76.970	.00113	836.69	.0337	3-3	3.4
19	27.255	.00113	836.69	.0337	3-3	3.4
20	27.351	.00113	836.69	.0337	3-3	3.4
21	27.670	.00113	836.69	.0337	3-3	3.4
22	28.30	.00113	836.69	.0337	3-3	3.4
23 3+F	28.972	.00092	1021.1	.0303	3-6	3.2
24	29.257	.00092	1021.1	.0303	3-6	3.2
25	30.971	.00092	1021.1	.0303	3-6	3.2
26	41.124	.00118	1229.7	.0343	3-9	3.87
27	42.171	.00118	1229.7	.0343	3-9	3.87
28	43.828	.00096	1463.9	.031	4-9	3.6
29	45.485	.00096	1463.9	.031	4-0	3.6
30	47.182	.00118	1463.9	.0343	4-0	4.0
46	48.468	.00118	1463.9	.0343	4-0	4.0
31	50.040	.00118	1463.9	.0343	4-0	4.0
32	51.341	.00072	2007.	.0268	4-6	3.38
33	52.642	.00072	2007.	.0268	4-6	3.38
34	53.358	.00072	2007.	.0268	4-6	3.38
35	53.666	.00072	2007.	.0268	4-6	3.38
36	59.366	.00088	2007.	.0296	4-6	3.72
37	63.008	.00078	2659.0	.0279	5-0	3.76
38	71.839	.00078	2659.0	.0279	5-0	3.76
39	74.223	.00078	2659.0	.0279	5-0	3.76
40	79.188	.0092	2659.0	.0304	5-0	4.08
41	79.575	.0092	2659.0	.0304	5-0	4.08
42	80.141	.0092	2659.0	.0304	5-0	4.08
43	80.141	.0092	2659.0	.0304	5-0	4.08
44	85.734	.00105	2659.0	.0324	5-0	4.37
45	85.734	.00105	2659.0	.0324	5-0	4.37

N = .013

TABLE NO. 5

SAN DIEGO INTERCEPTING SEWER NO. 2

Inlet	C.F.S.	Grade	$AC\sqrt{r}$	$\sqrt{s}$	Size	V
6						
7						
8	.92				12"	
9	1.20				12"	
10	1.74				12"	
11	5.9				24"	
12	6.15					
13	6.44				24"	
14	15.52	.0011	532.76	.0323	33"	2.95
15	15.82	.0011	532.76	.0323	33"	2.95
16	16.29	.0011	532.76	.0323	33"	2.95
17	16.47	.0011	532.76	.0323	33"	2.95
18	16.75	.0011	532.76	.0323	33"	2.95
19	16.85	.0011	532.76	.0323	33"	2.95
21	17.17	.0011	532.76	.0323	33"	2.95
22	17.63	.0011	532.76	.0323	33"	2.95
23	18.47	.00093	674.	.0306	36"	2.9
24	18.76	.00093	674.	.0306	36"	2.9
25	20.47	.00093	674.	.0306	36"	2.9
26	30.62	.00106	1021.1	.0306	42"	3.45
27	31.61	.00106	1021.1	.0326	42"	3.45
28	33.33	.00106	1021.1	.0326	42"	3.45
29	35.00	.00103	1229.7	.0321	45"	3.6
30	36.68	.00103	1229.7	.0321	45"	3.6
46	38.00	.00103	1229.7	.0321	45"	3.6
31	39.54	.00103	1229.7	.0321	45"	3.6
32	40.84	.0011	1463.9	.0333	48"	3.8
33	42.14	.0011	1463.9	.0333	48"	3.8
34	42.14	.0011	1463.9	.0333	48"	3.8
35	43.16	.0011	1463.9	.0333	48"	3.8
36	48.86	.0011	1463.9	.0333	48"	3.8
37	52.50	.001	2007	.0316	54"	4.0
38	61.34	.001	2007	.0316	54"	4.0
39	63.72	.001	2007	.0316	54"	4.0
40	68.69	.00079	2659	.028	60"	3.8
41	69.07	.00079	2659	.028	60"	3.8
42	69.64	.00079	2659	.028	60"	3.8
43	70.15	.00079	2659	.028	60"	3.8
44	75.23	.00079	2659	.028	60"	3.8
45	75.23	.00079	2659	.028	60"	3.8

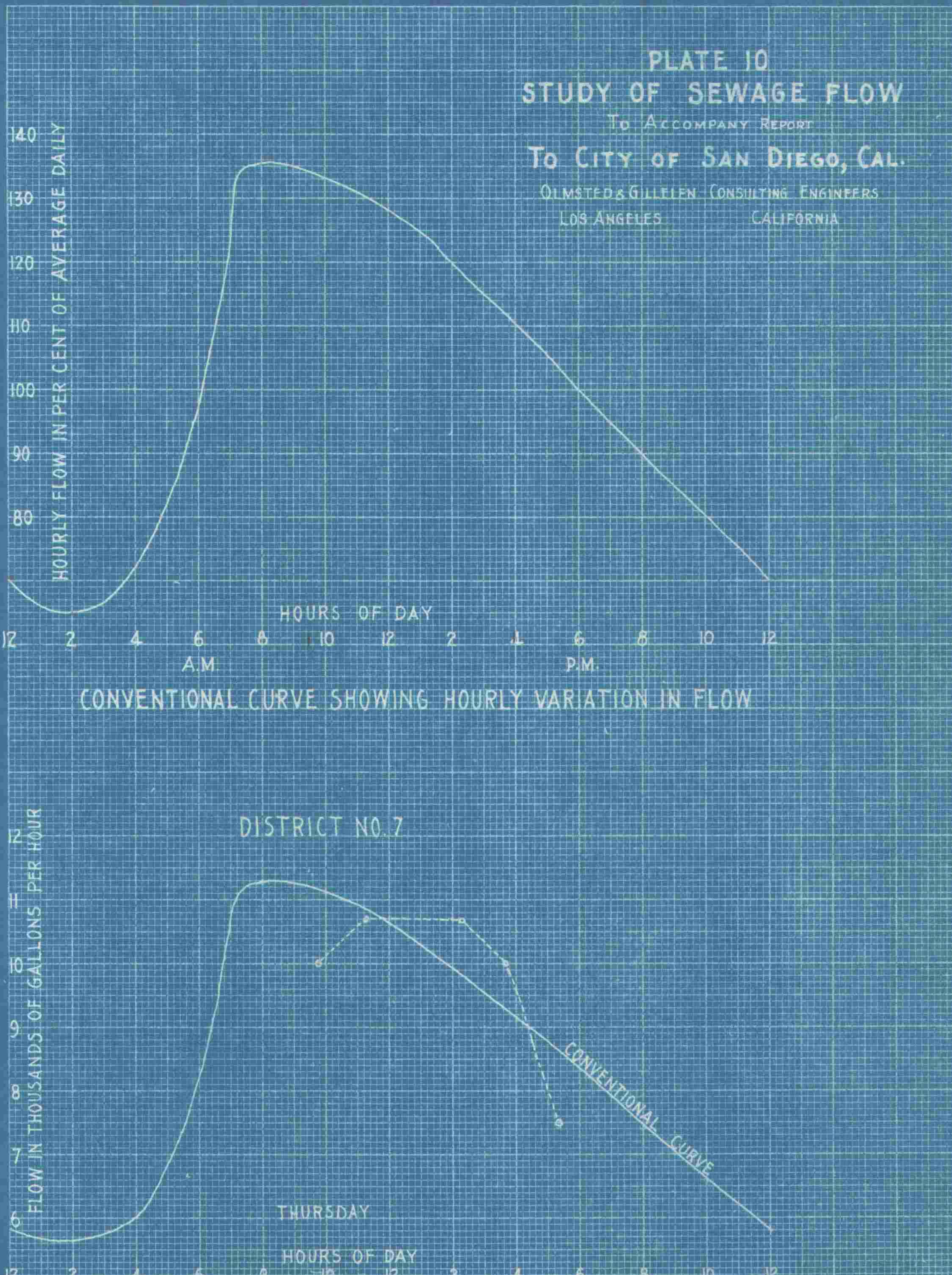
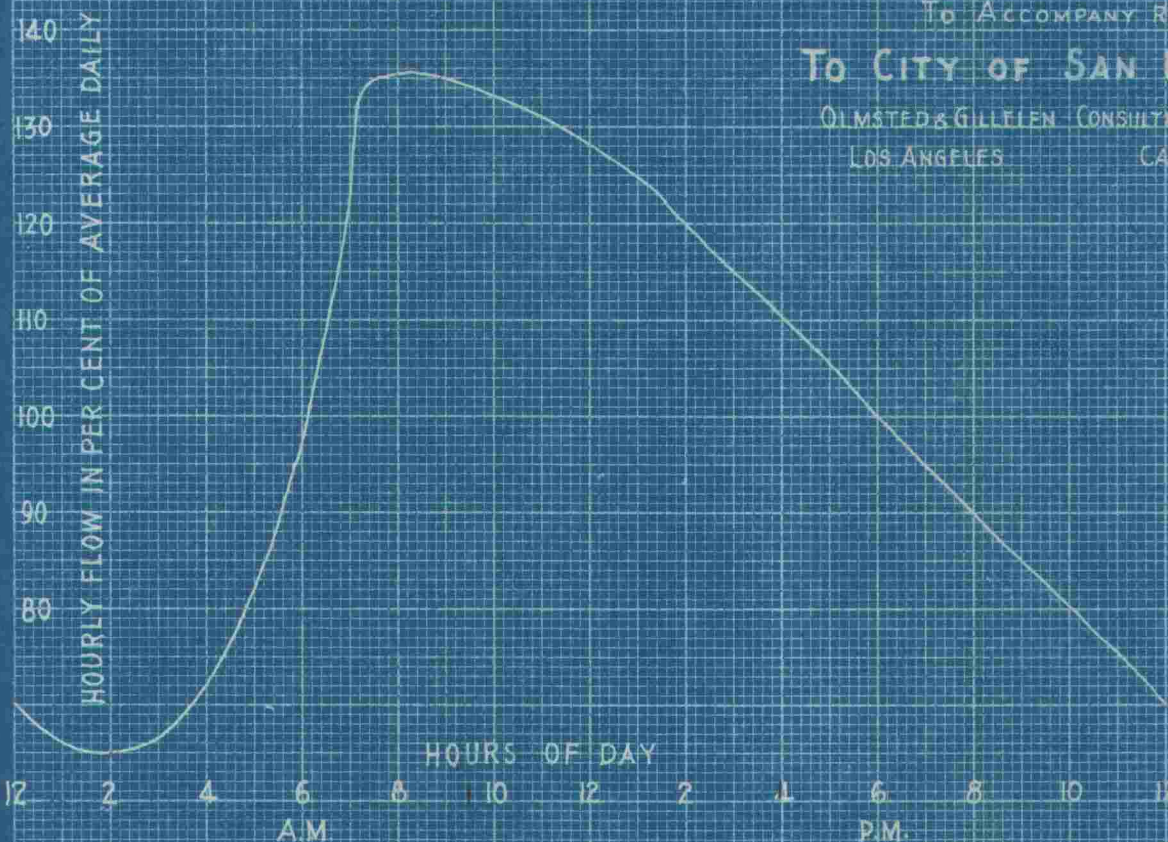


PLATE 10
STUDY OF SEWAGE FLOW
TO ACCOMPANY REPORT
TO CITY OF SAN DIEGO, CAL.
OLMSTED & GILLELEN CONSULTING ENGINEERS
LOS ANGELES CALIFORNIA
HOURLY FLOW IN PER CENT OF AVERAGE DAILY
140
130
120
110
100
90
80
HOURS OF DAY
12 2 4 6 8 10 12 2 4 6 8 10 12
A.M.
P.M.
CONVENTIONAL CURVE SHOWING HOURLY VARIATION IN FLOW
DISTRICT NO. 7
FLOW IN THOUSANDS OF GALLONS PER HOUR
12
11
10
9
8
7
6
CONVENTIONAL CURVE
THURSDAY
HOURS OF DAY
12 2 4 6 8 10 12 2 4 6 8 10 12

PLATE 10

PLATE 10
STUDY OF SEWAGE FLOW
To Accompany Report
To City of San Diego, Cal.
Olmsted & Gillelen Consulting Engineers
Los Angeles California



CONVENTIONAL CURVE SHOWING HOURLY VARIATION IN FLOW

PLATE 10

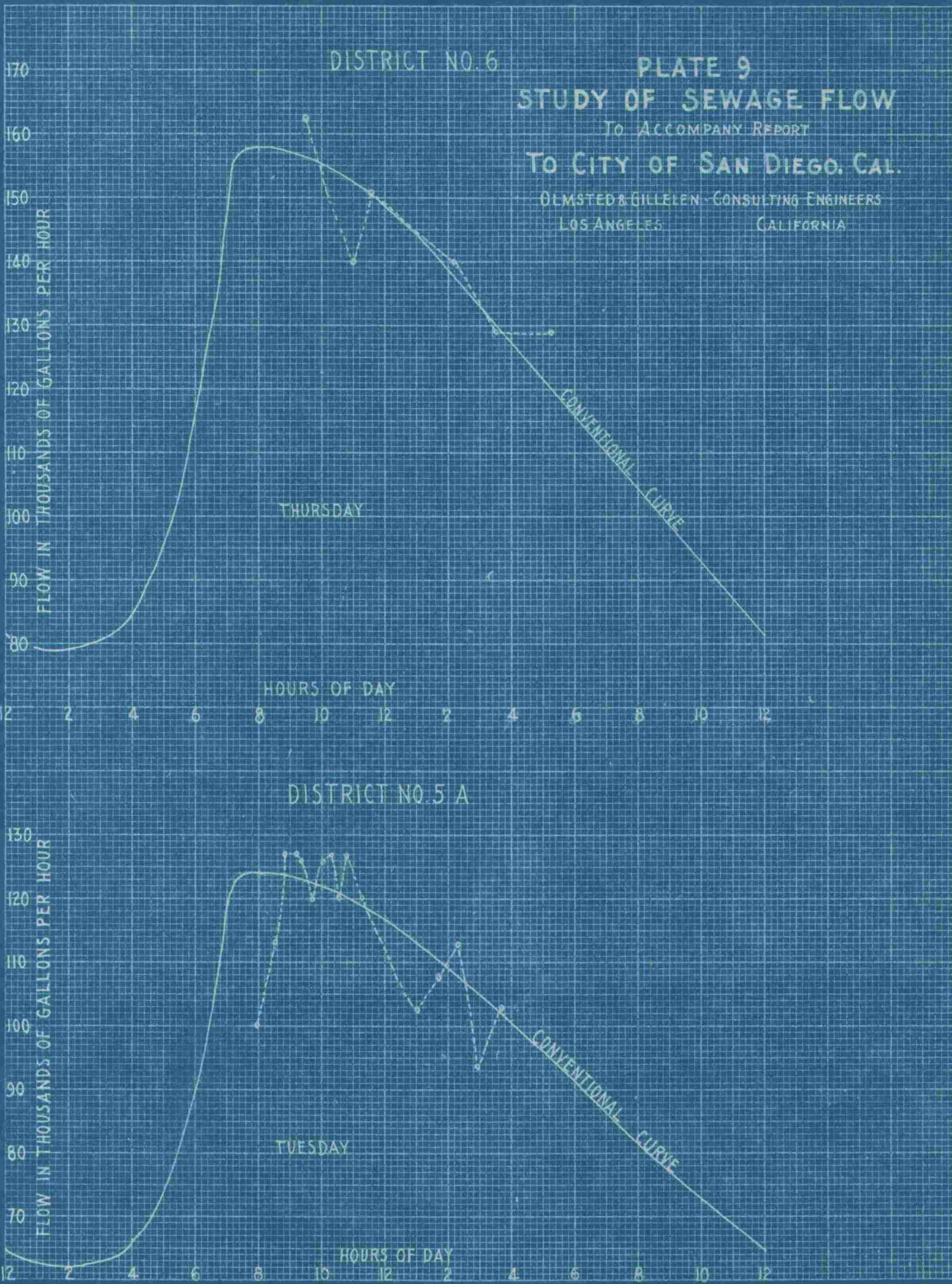


PLATE 9
STUDY OF SEWAGE FLOW
TO ACCOMPANY REPORT
TO CITY OF SAN DIEGO, CAL.
OLMSTED & GILLELEN - CONSULTING ENGINEERS
LOS ANGELES CALIFORNIA
DISTRICT NO. 6
FLOW IN THOUSANDS OF GALLONS PER HOUR
170
160
150
140
130
120
110
100
90
80
THURSDAY
CONVENTIONAL CURVE
HOURS OF DAY
12 2 4 6 8 10 12 2 4 6 8 10 12
DISTRICT NO. 5 A
FLOW IN THOUSANDS OF GALLONS PER HOUR
130
120
110
100
90
80
70
TUESDAY
CONVENTIONAL CURVE
HOURS OF DAY
12 2 4 6 8 10 12 2 4 6 8 10 12

PLATE 9

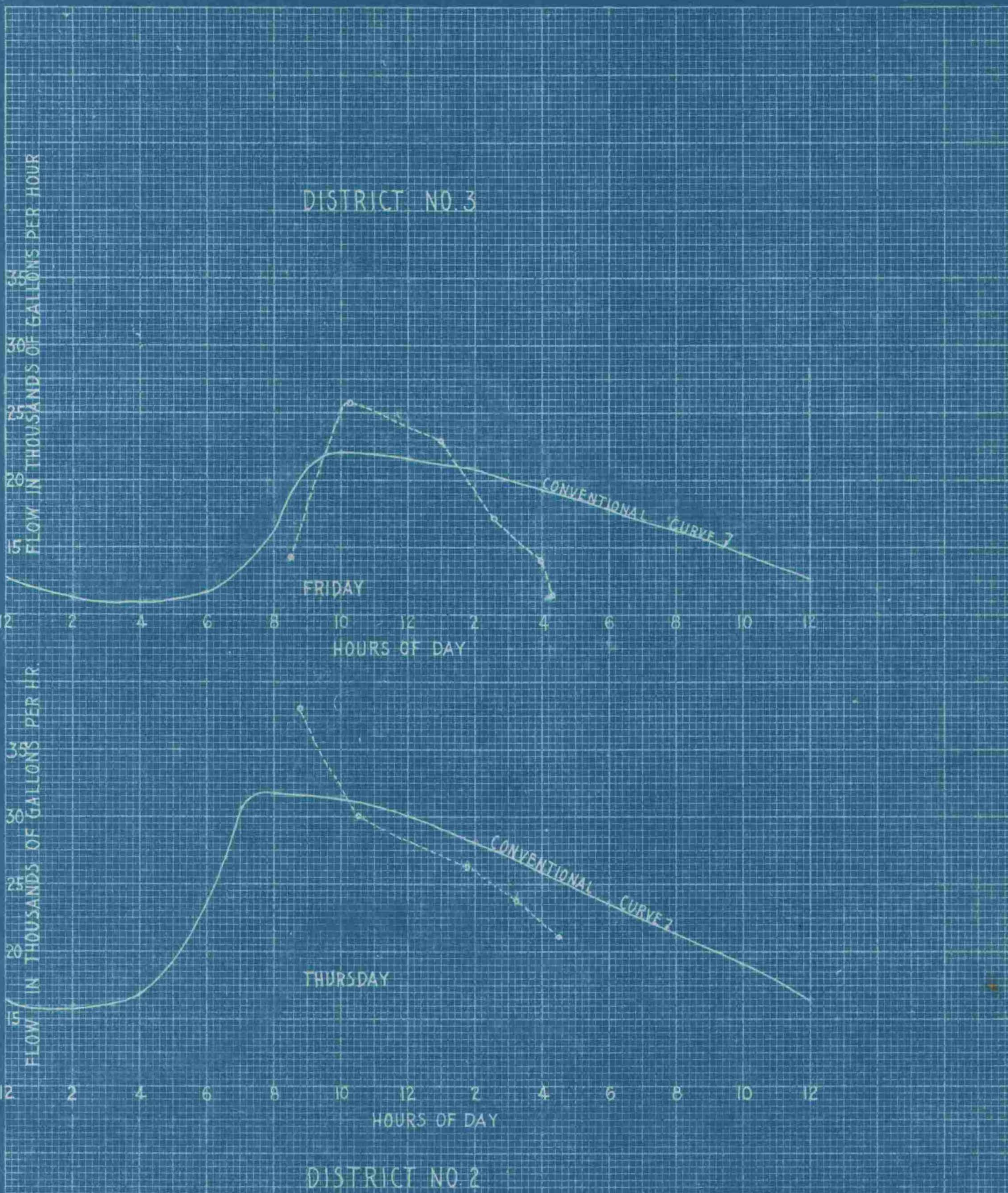


PLATE 7

STUDY OF SEWAGE FLOW

To Accompany Report

To City of San Diego, Cal.

Olmsted & Gillelen - Consulting Engineers

PLATE 7

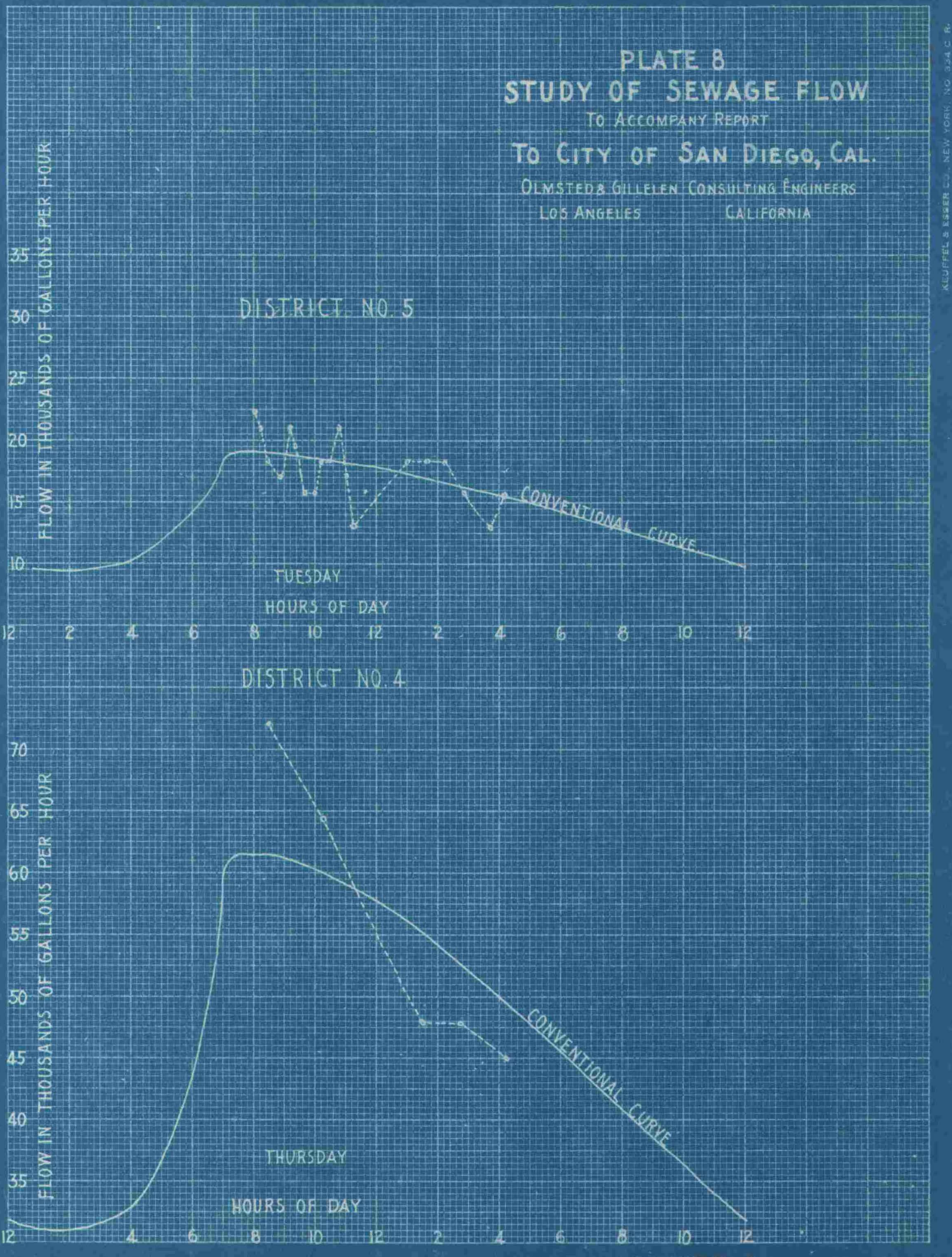


PLATE 8
STUDY OF SEWAGE FLOW
TO ACCOMPANY REPORT
TO CITY OF SAN DIEGO, CAL.
OLMSTED & GILLELEN CONSULTING ENGINEERS
LOS ANGELES CALIFORNIA
DISTRICT NO. 5
FLOW IN THOUSANDS OF GALLONS PER HOUR
35
30
25
20
15
10
CONVENTIONAL CURVE
TUESDAY
HOURS OF DAY
12 2 4 6 8 10 12 2 4 6 8 10 12
DISTRICT NO. 4
FLOW IN THOUSANDS OF GALLONS PER HOUR
70
65
60
55
50
45
40
35
CONVENTIONAL CURVE
THURSDAY
HOURS OF DAY
12 4 6 8 10 12 2 4 6 8 10 12
KEUFFEL & ESSER CO. NEW YORK. NO 354 C R.

PLATE 8

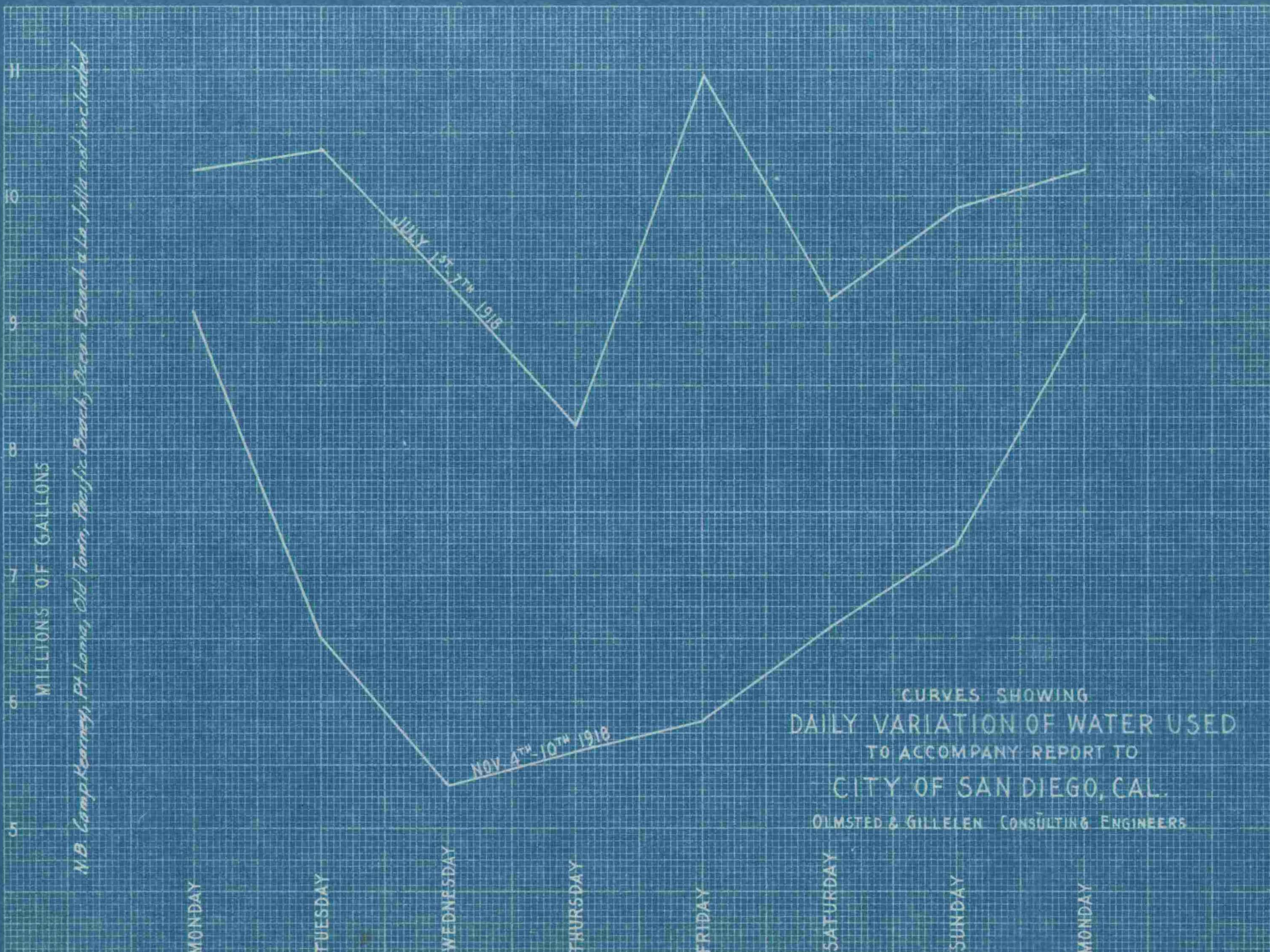
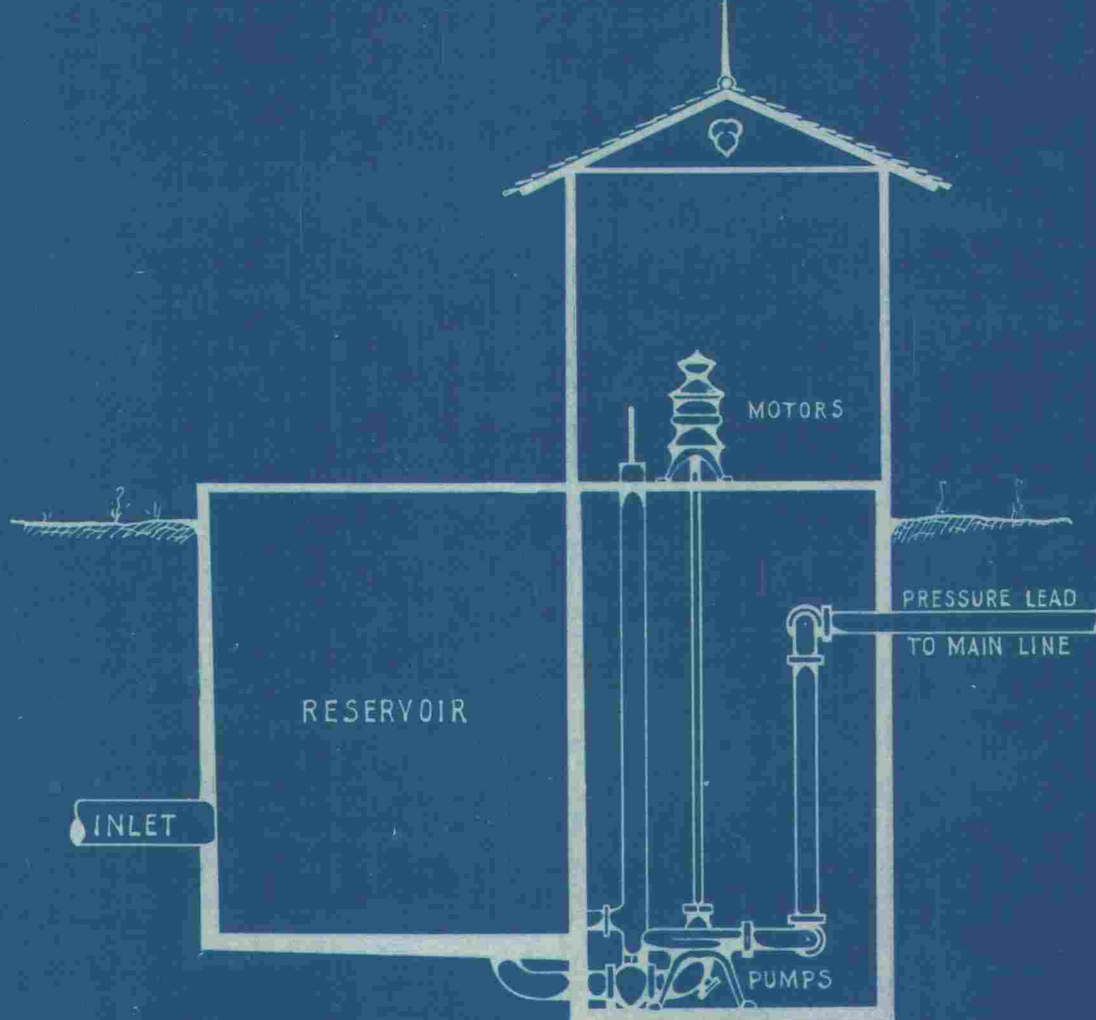
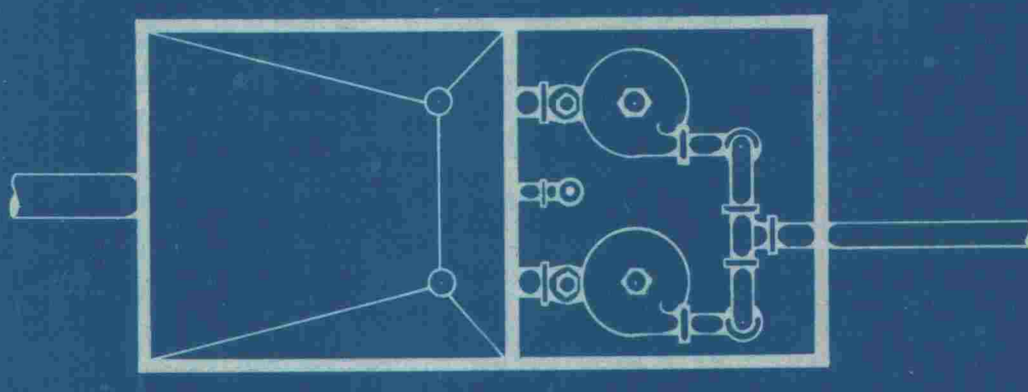

CURVES SHOWING
DAILY VARIATION OF WATER USED
TO ACCOMPANY REPORT TO
CITY OF SAN DIEGO, CAL.
OLMSTED & GILLELEN CONSULTING ENGINEERS
JULY 1ST - 7TH 1918
NOV. 4TH - 10TH 1918
MILLIONS OF GALLONS
N.B. Camp Kearney, Pt. Loma, Old Town, Pacific Beach, Ocean Beach & La Jolla not included
11
10
9
8
7
6
5
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
SUNDAY
MONDAY

PLATE 6



CROSS SECTION



PLAN

GENERAL PLAN

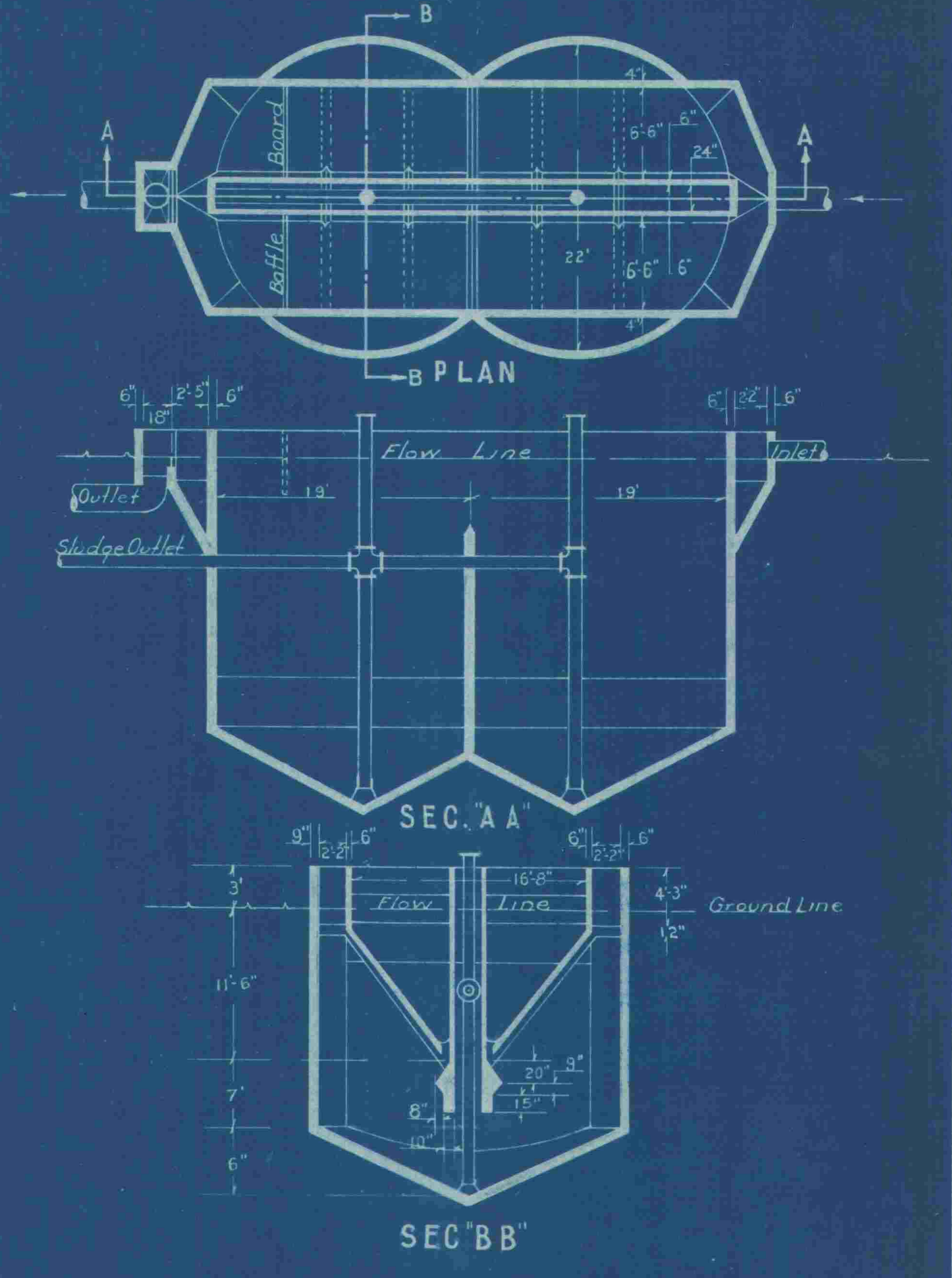
PUMPING STATION
SAN DIEGO SEWAGE TREATMENT

TO ACCOMPANY REPORT BY

OLMSTED & GILLELEN, ENG'RS., LOS ANGELES, CAL.

1919

PLATE 5



GENERAL PLAN

TREATMENT TANK UNIT

REPORT ON

SAN DIEGO SEWAGE TREATMENT

OLMSTED & GILLELEN ENGRS. - LOS ANGELES

1919

PLATE 4

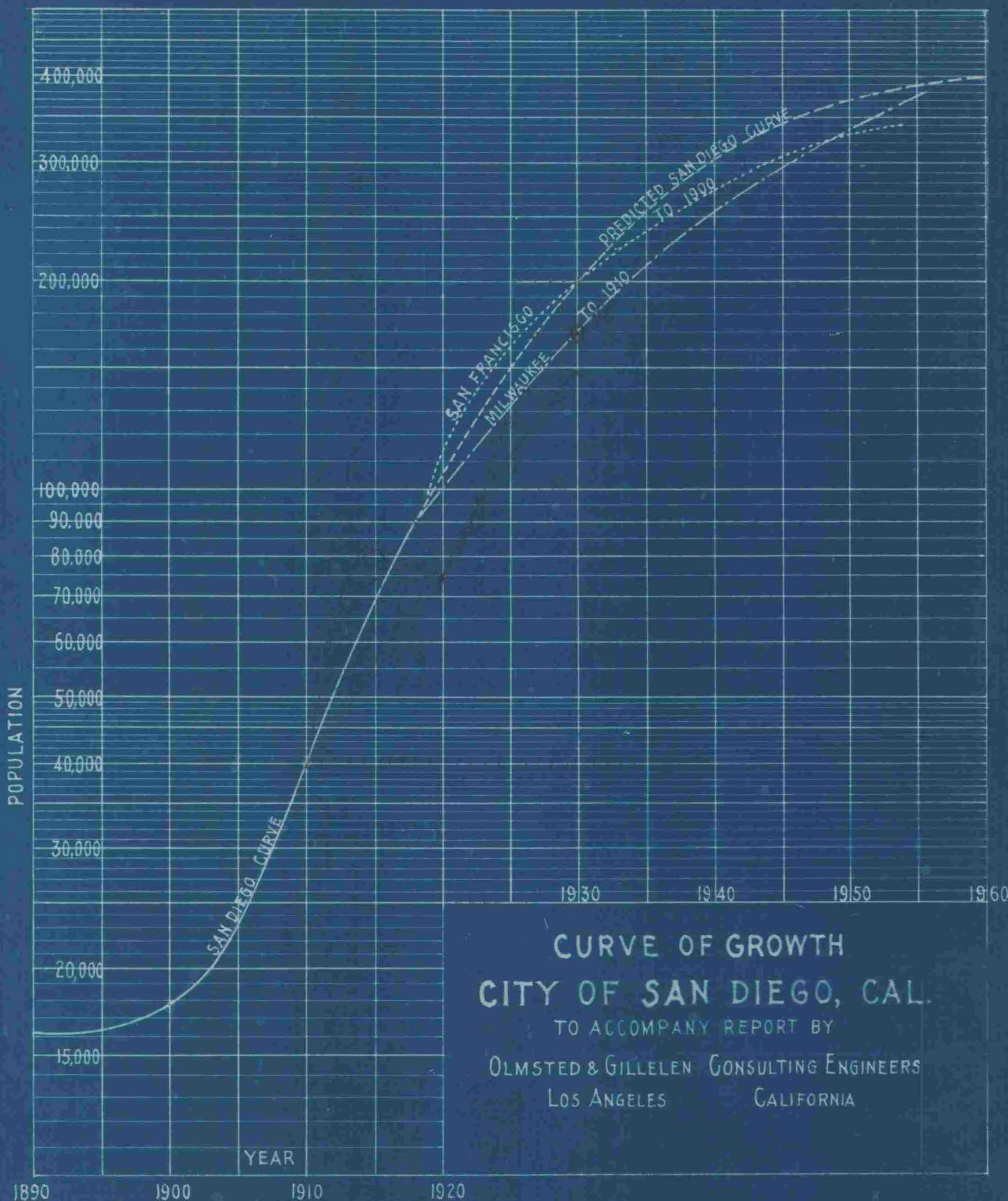

400,000
300,000
200,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
15,000
POPULATION
YEAR
1890
1900
1910
1920
1930
1940
1950
1960
SAN DIEGO CURVE
SAN FRANCISCO TO 1900
MILWAUKEE TO 1910
PREDICTED SAN DIEGO CURVE
CURVE OF GROWTH
CITY OF SAN DIEGO, CAL.
TO ACCOMPANY REPORT BY
OLMSTED & GILLELEN CONSULTING ENGINEERS
LOS ANGELES CALIFORNIA

PLATE 3

PER CENT (TIME) ELEV.	JAN.	FEB.	MAR.	APRIL	MAY	JUNE	JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
0.0	91.6	92.6	94.4	91.9	92.9	93.5	95.0	97.3	99.0	99.0	95.4	
0.5	88.4	90.4	89.0	85.5	88.2	89.5	91.5	93.2	95.0	94.6	91.0	
1.0	84.3	83.0	80.0	77.4	82.6	84.8	87.5	89.5	88.8	86.8	86.0	
2.0	70.7	65.7	64.2	61.6	69.6	73.7	76.6	73.0	69.7	71.6	70.7	
3.0	44.5	42.4	43.0	40.3	43.2	52.0	47.7	52.0	54.7	53.8	53.0	
4.0	24.4	24.7	22.3	19.0	18.6	21.5	28.9	34.2	32.8	30.4	25.3	
5.0	11.8	9.8	6.6	5.2	8.1	11.4	14.5	18.2	16.7	11.2	11.7	
6.0	2.8	1.5		2.3	0.9	3.0	4.7	4.2	1.6	2.6	3.9	
7.0	0.3							0.4				

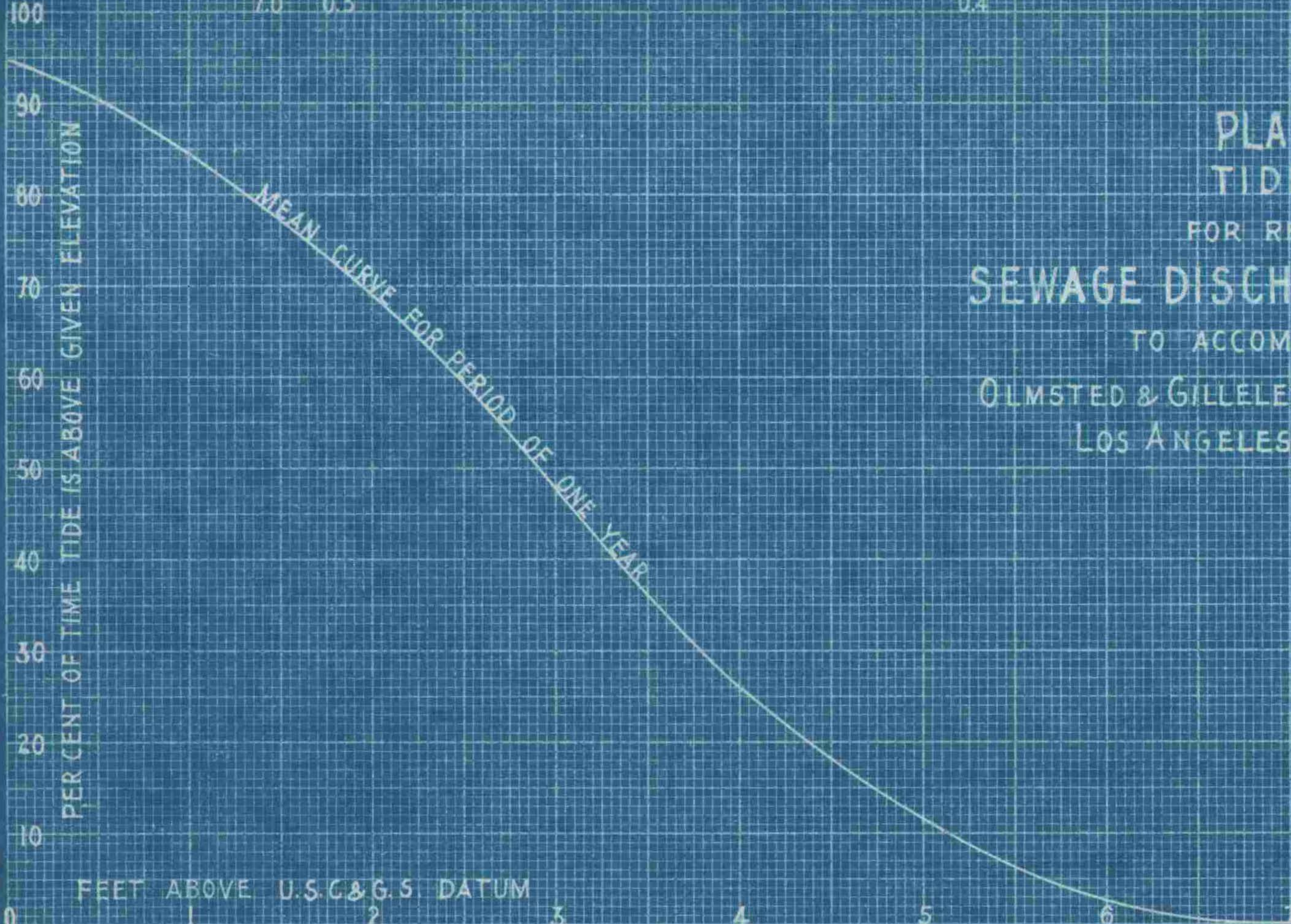
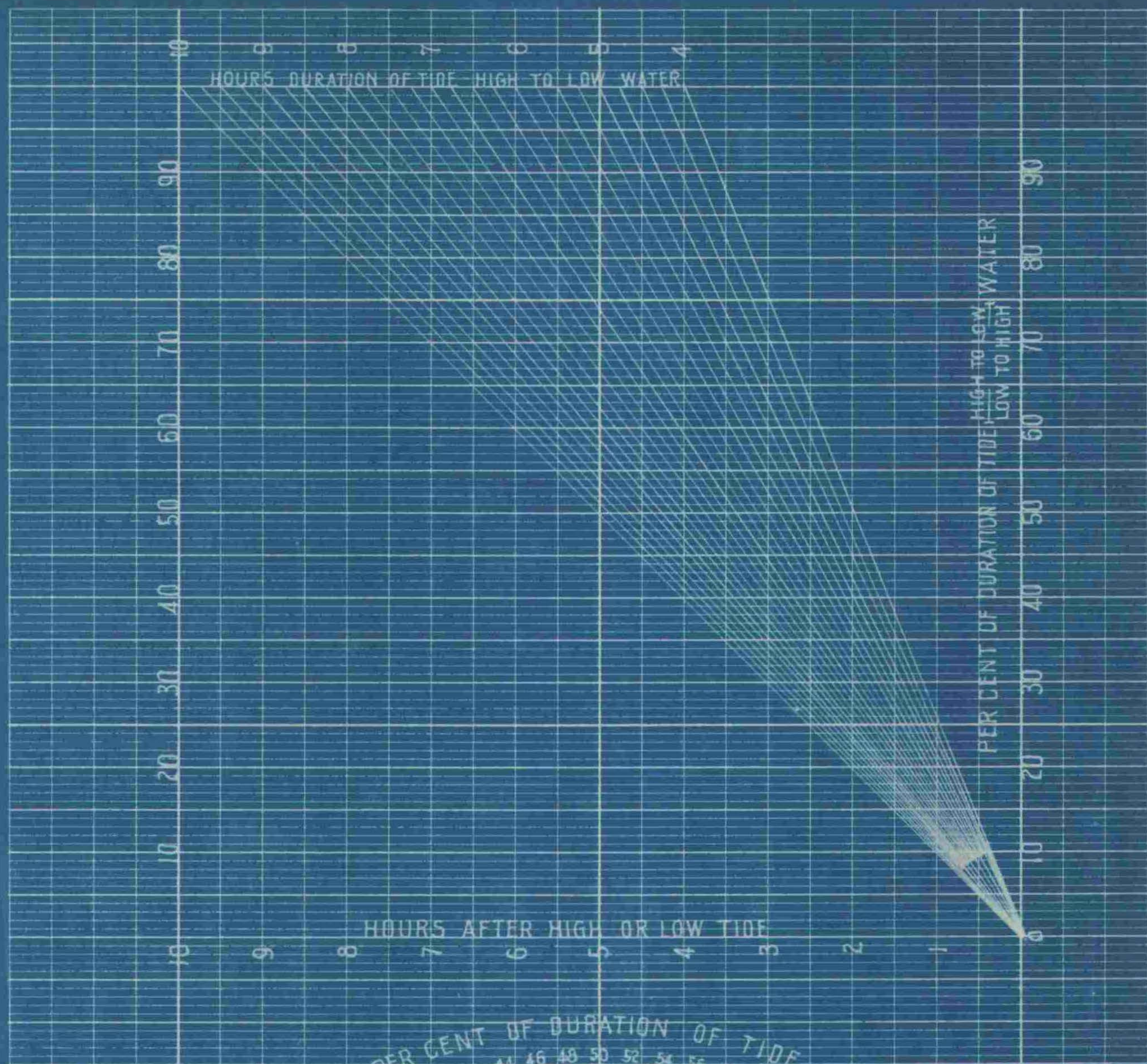


PLATE NO. 2
TIDE CURVE
FOR REFERENCE TO
SEWAGE DISCHARGE, SAN DIEGO, CAL.
TO ACCOMPANY REPORT BY

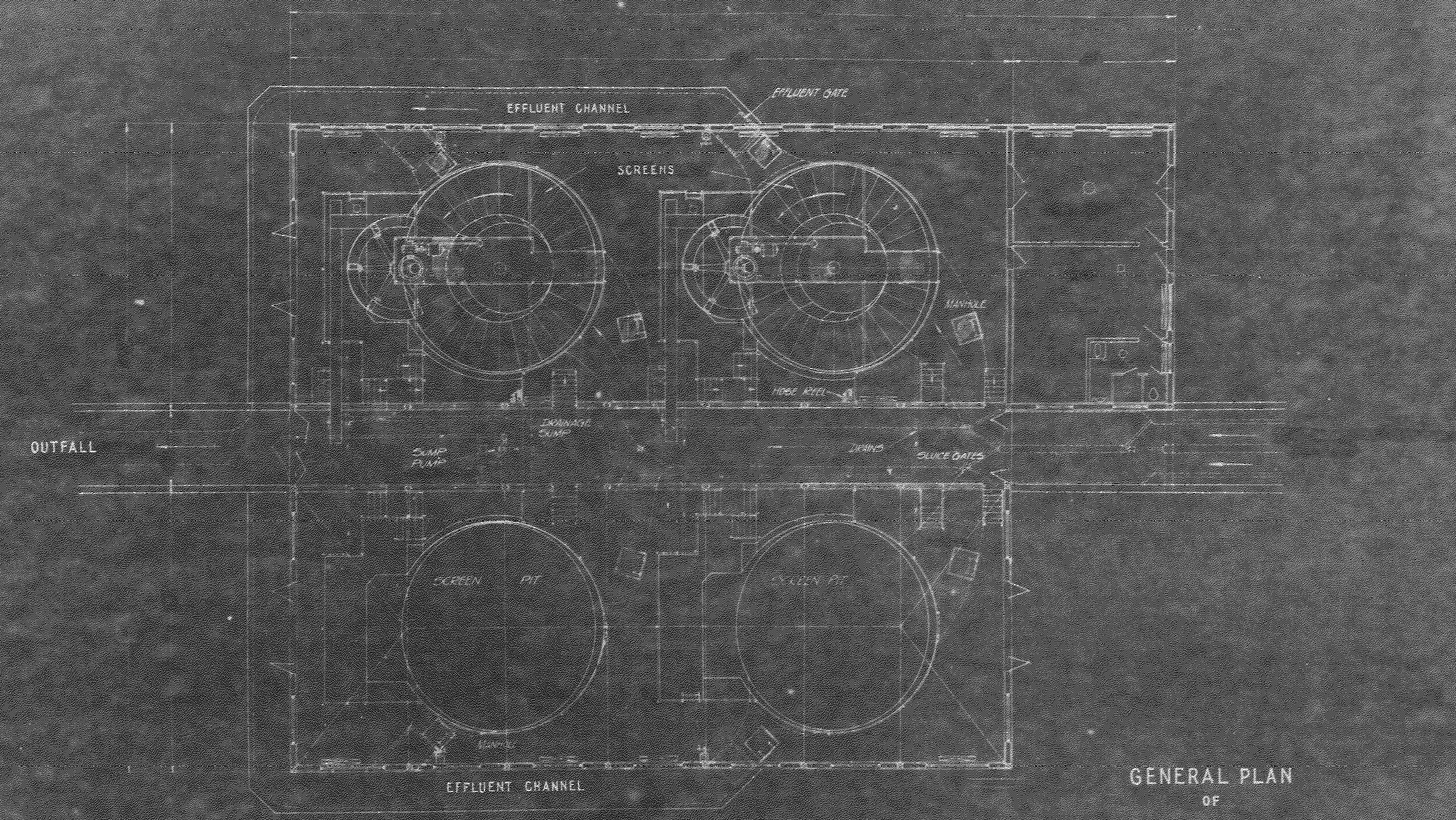
OLMSTED & GILLELEN — CONSULTING ENGINEERS.
LOS ANGELES — CALIFORNIA

PLATE 2



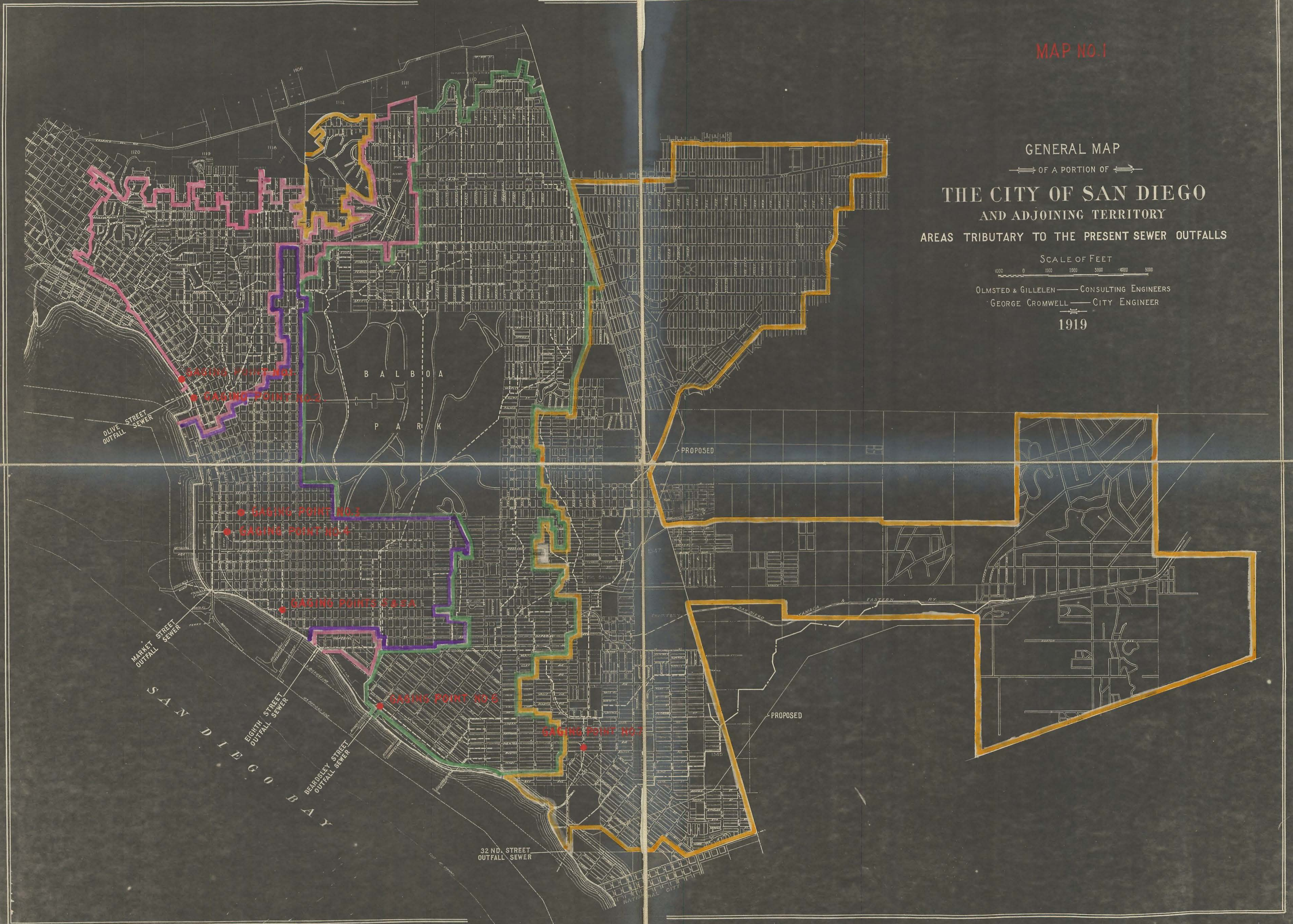
REPORT ON SAN DIEGO SEWAGE TREATMENT & DISPOSAL

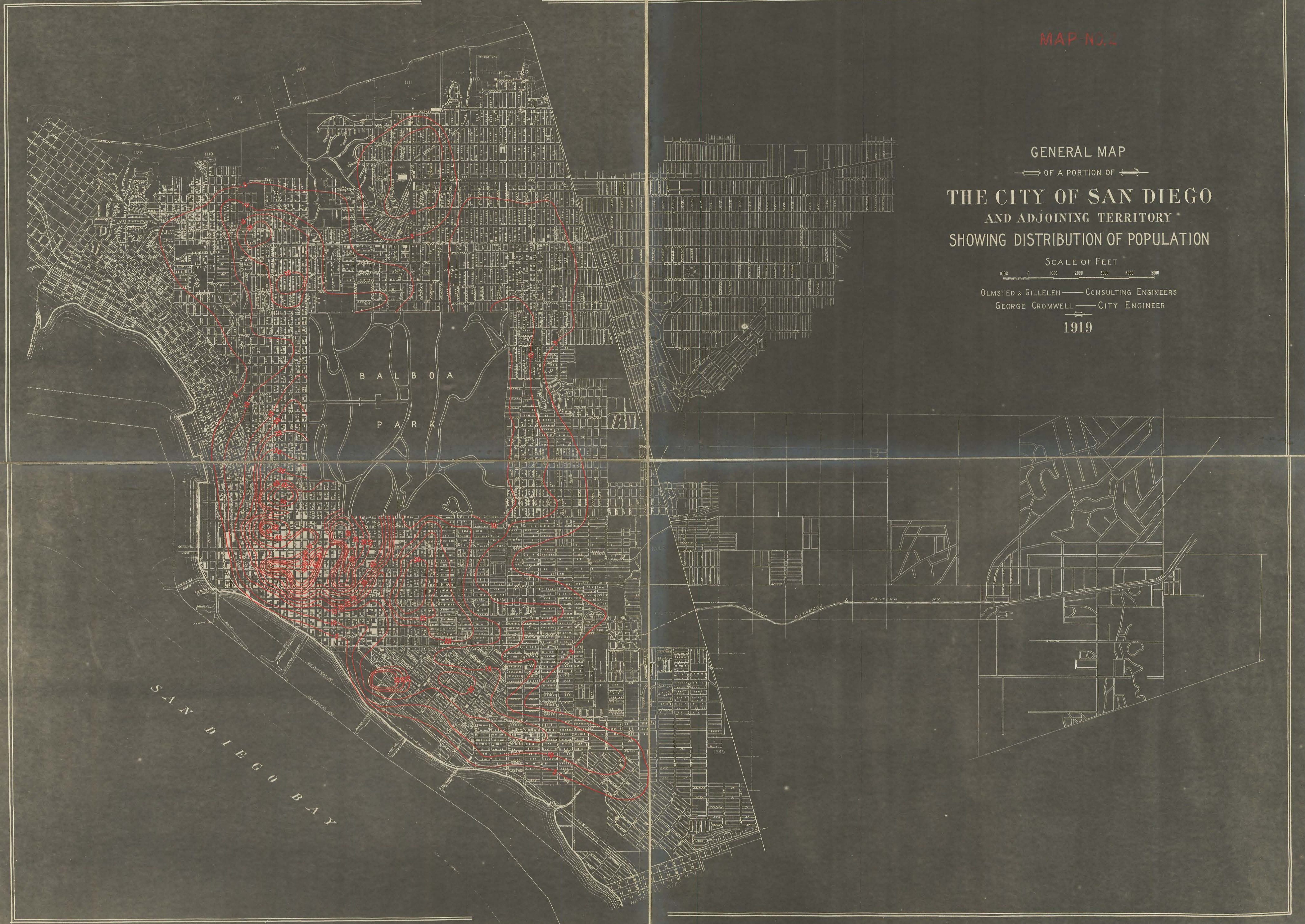
PLATE I



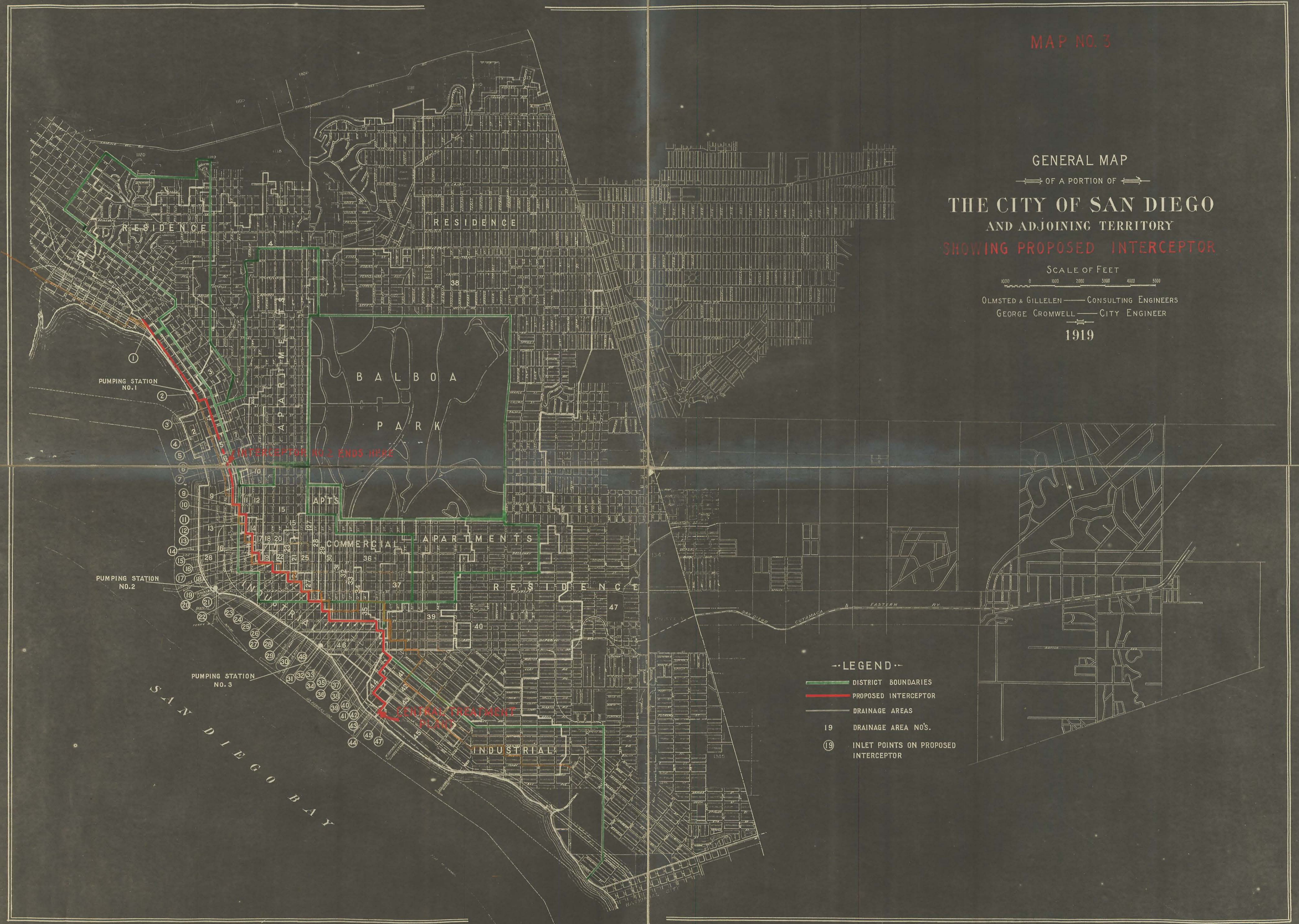
GENERAL PLAN
OF
CENTRAL SCREENING PLANT
FOR
SAN DIEGO SEWAGE TREATMENT
TO ACCOMPANY REPORT BY
OLMSTED & GILLELEN CONSULTING ENGINEERS
LOS ANGELES, CALIFORNIA
1919

GENERAL ELEVATION
OF
CENTRAL SCREENING PLANT
FOR
SAN DIEGO SEWAGE TREATMENT
TO ACCOMPANY REPORT BY
OLMSTED & GILLELEN CONSULTING ENGINEERS
LOS ANGELES, CALIFORNIA
1919


MAP NO. 1
GENERAL MAP
OF A PORTION OF
THE CITY OF SAN DIEGO
AND ADJOINING TERRITORY
AREAS TRIBUTARY TO THE PRESENT SEWER OUTFALLS
SCALE OF FEET
OLMSTED & GILLELEN — CONSULTING ENGINEERS
GEORGE CROMWELL — CITY ENGINEER
1919
BALBOA PARK
SAN DIEGO BAY
GAGING POINT NO. 1
GAGING POINT NO. 2
GAGING POINT NO. 3
GAGING POINT NO. 4
GAGING POINT NO. 6
GAGING POINT NO. 7
OLIVE STREET OUTFALL SEWER
MARKET STREET OUTFALL SEWER
EIGHTH STREET OUTFALL SEWER
BEARDSLEY STREET OUTFALL SEWER
32 ND. STREET OUTFALL SEWER
PROPOSED
PROPOSED



MAP NO. 2
GENERAL MAP
OF A PORTION OF
THE CITY OF SAN DIEGO
AND ADJOINING TERRITORY
SHOWING DISTRIBUTION OF POPULATION
SCALE OF FEET
1000 0 1000 2000 3000 4000 5000
OLMSTED & GILLELEN —— CONSULTING ENGINEERS
GEORGE CROMWELL —— CITY ENGINEER
1919
BALBOA PARK
SAN DIEGO BAY
SAN DIEGO CUYAMACA & EASTERN RY.



MAP NO. 3
GENERAL MAP
OF A PORTION OF
THE CITY OF SAN DIEGO
AND ADJOINING TERRITORY
SHOWING PROPOSED INTERCEPTOR
SCALE OF FEET
1000 0 1000 2000 3000 4000 5000
OLMSTED & GILLELEN —— CONSULTING ENGINEERS
GEORGE CROMWELL —— CITY ENGINEER
1919
LEGEND
DISTRICT BOUNDARIES
PROPOSED INTERCEPTOR
DRAINAGE AREAS
19 DRAINAGE AREA NO'S.
19 INLET POINTS ON PROPOSED INTERCEPTOR
RESIDENCE
RESIDENCE
APARTMENTS
BALBOA PARK
APTS
COMMERCIAL
APARTMENTS
RESIDENCE
INDUSTRIAL
INDUSTRIAL
INTERCEPTOR NO. 2 ENDS HERE
CENTRAL TREATMENT PLANT
PUMPING STATION NO. 1
PUMPING STATION NO. 2
PUMPING STATION NO. 3
SAN DIEGO BAY
SAN DIEGO CUYAMACA & EASTERN RY.

PROFILE OF PROPOSED INTERCEPTING SEWER FOR CITY OF SAN DIEGO, CALIF.

OLMSTED & GILLELEN CONSULTING ENGINEERS
GEORGE CROMWELL CITY ENGINEER

PLATE NO. II