THE SIGNAL HILL STORM DRAIN

Thesis for Degree of C. E.

Ropha V Pearsall

1915

THESIS

Dramaft

Civil engineering / Hydraulic engineering

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LONG BEACH, CALIFORNIA.
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By

R. V. PEARSALL.

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Long Beach California is a city of thirty-two thousand inhabitants which has enjoyed a very rapid growth. Property values have had a corresponding increase, resulting in scattering the population and making urgent the need of a vast amount of public improvement, among the most important of which is a system of storm drains.

The city has a south frontage on the Pacific Ocean with the San Gabriel river dividing north of the city, leaving low lands to the east and west. Between the city and the plain on the north is high ground known as Signal Hill.

The San Gabriel river is a mere creek during the summer months, but during the winter it is subject to serious floods becoming a raging torrent, miles wide in places. Immense quantities of silt are carried by these floods to the harbor of Long Beach and that of Los Angeles, adjoining on the west. So serious has this damage become that it is proposed to divert the flow of the San Gabriel river to the east of Signal Hill and Long Beach, thus disposing of a serious drainage problem and making industrial property of land now annually submerged.

A secondary benefit resulting from the diversion of the San Gabriel river is that it provides a splendid outfall for draining a basin lying just south of Signal Hill. This area has no natural drainage and contains approximately twenty-five hundred acres and lies partly out and partly within the corporate limits of Long Beach. Each year a lake forms in this basin

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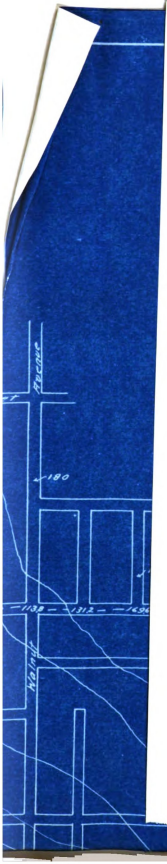
and its size has been increasing until now it covers approximately one hundred and twenty acres, appearing in January and after dwindling to about two acres in September, remains until the next rain in December. Doubtless the impervious area has been increased very much by the growing city and increased run-off accounts for the quantities of water accumulating in the basin. This pond is stagnant and a menace to health, yet surrounded on all sides by property selling at two thousand dollars or more per acre.

There appear three methods of draining this basin:- Draining to the ocean on the south; To tide lands on the east; or To the low lands on the west.

The characteristics of the basin are shown on the attached contoured map which discloses a steep water shed of five-hundred and forty-five acres on the north and a shed of nineteen hundred and twenty-five acres on the south draining to a pond between whose bed is eight and six tenths feet above city datum. This datum plain is seven and sixtenths feet above mean lower low water or about the highest high tide line. Tracks of the San Pedro Los Angeles and Salt Lake railway company are carried on a fill through California Avenue, thus dividing the southern water shed. Drainage of that portion lying west of California avenue three hundred and seventy acres in area presents a separate problem as its storm water is intercepted before reaching the lake and is easily carried to the low lands on the west. Outside of about four hundred acres lying south of Seventh Street the entire region is sparcely inhabited and the streets are oil dirt roads without walks and gutters. The south shed


PACIFIC
180
1130
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WALNUT

has a top soil of heavy loam one to two feet thick below which lie alternate layers of yellow clay and sand. The soil on the north slope is lighter and with equal slppes would have a much greater rate of percolation than that on the south.

Table one gives a record of rain fall for Long Beach during a twenty year period. This table was prepared to show the conditions which a storm water sewer would have to meet and shows a maximum rain fall of three and eighty-three hundredths inches. The severest storms of each year arelisted and show periods of daily rains and is intended for use in providing overflow area. The season maximum seems to lie arround twenty inches which amount has been approached during the years 1914-1915.

TABLE NO. 1.

Rainfall record for Long Beach.

Season	Precipitation in inches.	Precipitation for one day.	Precipitation during severe storms.	
			Days	Inches.
1894-1895	15.14	2.42	13	4.12
1895-1896	9.54	1.96	4	2.70
1896-1897	19.96	3.83	3	4.39
1897-1898	6.06	1.20	4	2.77
1898-1899	6.31	1.50	2	2.39
1899-1900	9.63	1.12	3	1.59
1900-1901	12.70	2.71	5	4.25
1901-1902	9.47	2.15	3	3.42
1902-1903	20.79	3.58	2	3.92
1903-1904	7.15	1.78	1	1.78
1904-1905	15.55	2.38	6	4.73
1905-1906	20.88	1.85	2	2.98
1906-1907	14.69	2.20	5	2.98
1907-1908-	10.04	1.49	6	4.29
1908-1909	18.13	2.18	2	2.70
1909-1910	10.42	1.72	3	3.38
1910-1911	13.19	1.06	2	2.55
1911-1912	7.56	1.02	9	4.08
1912-1913	8.14	2.00	3	4.20
1913-1914	17.27	2.80	2	3.85
1914-1915	2-.15	2.85	2	3.36

Additional rainfall data was obtained from the United States weather observer in Los Angeles and his records show maximum rainfall for periods of less than twenty-four hours. Table Number two is taken from his report and shows a maximum of one and nine tenths inches for one hundred minutes which figure I have used in my calculation.

TABLE NO. 2.

Excessive precipitation at Los Angeles, Cal., for the 15 years, 1899-1914.

Duration.	Amount in inches.
5 minutes	.36
7 minutes	.51
10 minutes	.66
15 minutes	.81
20 minutes	.85
30 minutes	1.12
60 minutes	1.51
100 minutes	1.90

Amount and velocity of run-off are rather indeterminate quantities but of prime importance in this subject. The problem is relatively simple once these factors are determined. The percentage of run-off was found by actual measurement in the pond with allowance made for evaporation and percolation. To measure this water I first found the area within each one foot contour lying within the flooded district. These areas and the method of computing the volume are shown in table number three.

TABLE NO.3.

Volume of Lake.

Contour.	Area-Sq.Ft.	Area Times depth	Difference in Area.	Difference in area x 1/3 distance bet. contours.
9	6125	6125		
10	21600	43200	15475	5160
12	79250	79250	57650	38430
13	144000	144000	64750	21580
14	281000	281000	137000	45670
15	1112500	1112500	831500	277170
16	2998800	5997600	1886300	628770
18	5072400		2073600	1382400
		7663600		2399180
				7663600
				10062780

208.0 Acre feet.

43560) 10062780

The volume of water shown 208 acre feet is that of the lake with the surface at contour 18 which has been its approximate level in February 1914-1915.

The maximum area of this lake as I before mentioned is approximately one hundred and twenty acres. Its life is from December to September or nine months. In February 1915 the surface of the lake was at elevation 17 when the accumulated rainfall was15.5 inches. This water had been accumulating since the last of December 1914 and would practically all be gone by the first of September. Evaporation statistics for Los Angeles in 1887-1888 are used in the following calculations. The average area of the lake while receeding was sixty acres and the time of receeding was six months. Evaporation for these six months was seventeen and eight-tenths inches per square foot or a total of three million eight hundred and sixty eight thousand cubic feet.

This evaporation had been taking place during the two months the lake was forming or 60 x 43560 (Area) x .36 (Evaporation) equals 941000 cubic feet. Now the contents of the lake 9062000 cubic feet minus 4809000 cubic feet or 4253000 cubic feet represents percolation into the ground in six months or the average rate per square foot of surface per month was 3½ inches. Assuming this rate to hold while the lake was forming we find 1524000 cubic feet to have percolated away previous to the time the lake contained 9062000 cubic feet and has had 941000 cubic feet evaporate. The average area of the lake has been 2613000 square feet so the direct rainfall has been 3376000 cubic feet. Adding evaporation and percolation and deducting direct rainfall gives 8151000 cubic feet as the run-off from 2470 acres and 15.5 inches of rainfall. This run-off is only 6% of the rainfall which small ratio is explained by poor drainage into the lake aided by heavy local percolation and evaporation. Soon after drainage is provided streets will be improved and the run-off will increase rapidly but will probably not exceed 20% within ten years.

As before stated there are three feasable methods of draining this area, directly south to the ocean, east to tide lands, or west to Cerritos Slough. Rapid run-off from the north water shed makes it necessary to have a trunk sewer at the foot of this slope. To put in a collecting system on the south slope designed to intercept the run-off would thus be robbed of the advantage of a shortened main sewer and since such a system not only calls for laterals but demands that street improvements be made I have decided to run all water to a reservoir located at the low point of the basin and bring the water by surface drainage to this point. As streets are improved new collecting sewers may be installed and the system now built will be in place to handle overflows

where the future sewers may prove inadiquate.

A drain to the south would be placed in Cherry Street because of an off-set in Walnut street which would make necessary tunneling under private property where such privileges would be hard to obtain. Cherry street is paved and any sewer placed in this street would need to be of a permanent nature so any trunk line installed should be of sufficient size to accommodate the entire basin. A plan and profile of this route and drain are shown on drawing No. 2.

Hawksleys formula gives an approximation as to the size of drain needed to carry the entireaffluent. This engineer designed some of the main sewers of London and Brooklyn which gives his formula a standing, it is-

Log(d)equals 3 log(A) plus log (N) plus 6.8 divided by 10.

D equals the diameter of the sewer in inches.

N equals distance in feet in which the fall is one foot.

A equals area in acres.

An outfall at the ocean would be located at an elevation approximating -3 and with an approximate length of 11500 feet with inlet at about 5 (N) would equal 1425. (A) equals 2470. The diameter thus becomes 8feet and 7 inches. The maximum cut would be 50 feet and the average 29 feet.

Buerkli gives as a result of his experience the following formula for the storm flow.

F equals $fr\left(\frac{S}{A}\right)^{\frac{1}{2}}$

f equals proportion of run-off.

F equals the flow per acre and cubic feet per second.

S equals the average slope.

A equals the area of the district in acres.

r equals the rate of rainfall in inches per hour during period of greatest intensity of rain. For suburban districts roughly paved he advises using a value of(f) of.44. (A) is 2470 and (r) is $1\frac{1}{2}$. (S) equals 2 for 1555 acres and 75 for 545 acres. This formula gives a value of (F) equal to 180 cubic feet persecond. Assuming the sewer to be concrete and using Cutters formula with (c) equal to 113 we get a value of (d) equal to 7 feet and 2 inches.

A formula easy to apply is vouched for by Adams of Brooklyn and is:-

$$D \text{ equals} \left(\frac{Q^2 L}{1542\ H}\right)^{1/6}$$

Q equals $\frac{\text{Acres}}{1.33}$ for rainfall of 2 inches per hours

D equals diameter of the sewer in feet.

L equals length of the sewer in feet.

H equals the total fall of the sewer.

This gives a value of(D)equal to 11 feet and 6 inches.

An application of McMaths formula shows a diameter of 9 feet and three inches to be required.

From these computations I feel safe in saying that a pipe 8 feet in diameter will be necessary. The installation of such a pipe will be expensive because of constructing an outfall in the open ocean, excessive cut, and replacing a large amount of pavement. A drain 9 feet in diameter was constructed in Wilminggon Delaware in 1903 with a cut averaging 24 feet at a cost of Twenty-four Dollars per foot. I estimate the proposed drain to cost more than the one cited and to entail an expense which is prohibitive under present conditions. Therefore a less expensive scheme must be sought.

A drain to the east may be constructed upon the right of way of the Pacific Electric Railway Company. An easement for this purpose has already been obtained and is a distinct advantage in that it obviates the danger of legal complications which may possibly occure if the streets of Long Beach are used. A plan and profile showing this route is shown on drawing No. 2. The outfall is on tide lands with an elevation of $1\frac{1}{2}$. The length of this route is approximately 11,700 feet and the maximum cut is 38 feet, the average being 23 feet.

The third and seemingly the most advantageous route is west through State Street. A plan and profile showing this scheme is given on drawing No. 3. This drain is 8,600 feet long and gains an outfall at elevation $2\frac{1}{2}$ into a rapidly flowing streem. The route through State Street is given preference over a more direct route in Twentieth Street because a better outfall is obtained and a hill lying near the Slough and crossed by Twentieth Street, is avoided. By taking the drain across private property from the intersection of Walnut Street andNineteenth Street to the intersection of Gundry Street and State Street a saving is made in the length of the drain and loss of head. Since the land thus crossed is neither improved nor under cultivation no trouble should be experienced in obtaining a permanent easement. The average cut needed for this drain is 12.4 feet with a maximum cut of 23 feet.

The exavation is so light in this case that a temporary drain is feasable as additional units may be built later at a reasonable expense and any drain now installed made a units of the larger system. The grade of this drain is sufficient to obtain a scouring velocity while the length and expense per foot----------

110
110

are less than those of the drain to the east. Since a considerable portion of the drainage basin lies within the corporate limits of Long Beach the use of its streets should be readily obtained. A relatively cheap construction is now desired by the community and this requirement eliminates consideration of a drain to the ocean. Having thus decided to drain west to Cerritos slough I will detail a design.

One authority places the velocity of run-off when over macadam andgravel roads, which condition I expect to see approached, as .4 foot per second for a slope of 1% with velocities varying as the square root of the slope. The average slope of the south shed is .2 percent for which the run-off velocity becomes .176 feet per second. The north shed has an average slope of 7½% for which the computed run-off is 1.096 feet per second. The average distance from the extremeties of the north slope to the drain is 5000 feet so the time required for water to reach the drain is $\frac{5000}{1.096}$ or 76 minutes. Likewise themaximum run from the south is placed at 8400 feet and the time required to reach the drain becomes $\frac{8400}{.176}$ equals 47700 seconds or 13 hours and 15 minutes.

Doubt has been expressed at the possibility of obtaining a satisfactory outfall into Cerritos Slough at State Street because of flood water at this point. The flood level of 8.4 feet noted on the annexed profile of State Street was for the North side of the street besides being a very unusual and transitory condition. I have recently been employed by the City of Long Beach to make a survey of the lands flooded by the Cerritos Slough. This included a contoured map of the region showing 1 foot contours and frequent sections of the river bottom. The result of this investigation was the decision to seek temporary relief from the floods by

in to the east. Since a comfortable
ability lies within the corporate
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construction is now desired by the
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ne decided to drain west to
a design.
velocity of run-off when over
condition I expect to see approach
slope of 1% with velocities varying
. The average slope of the south
the run-off velocity becomes .1%
ed has an average slope of 1%
in 1,000 feet per second. The
elevation of the north slope to the
sufficient for water to reach the
allowing the maximum run from the
the time required to reach the
.00 seconds or 18 hours and 18 minutes.
the possibility of obtaining
Carrollton Slough or State Street because
. The flood level of 8.6 feet noted
the street was for the north side of
unusual and exceptionally condition.
by the City of Long Beach to make
by the Carrollton Slough. This included
showing 1 foot contours and frequent
. The result of this investigation
rary relief from the floods by

straightening and cleaning the river channel. It is proposed to dig an eighty foot channel from a point 1500 feet south of State Street north to Willow Street. The grade of this channel will be .075% and the flow line at State Street will be at 1.4. The estimated velocity of flow will be 5 feet per second. This will increase the capacity of the stream many times and prevent recurrence of flood levels noted on the profile. Furthermore the diversion of water from Cerritos Slough previously mentioned though not necessary to the success of this plan may be anticipated in the near future and will remove all possibility of back pressure at the outfall.

With an outfall at 2½ fixed by the present channel bottom the grade of State street fixes the diameter of the drain at about three feet. Greater section may be obtained by duplicating the drain or using a box section.

Run-off from the north being so rapid carries considerable heavy debris and thus makes a settling basin desirable. The block now occupied by thepermanent pond contains 415000square feet and its purchase will furnish both a settling basin and reservoir thus reducing the size of the required storm drain. Since the water is to be collected by surface drainage the top of the reservoir is fixed by the surrounding ground level which is lowest to the west and by slight filling may be fixed at 15 feet. The bottom of this reservoir or the inlet to the drain is fixed at an elevation which will give a scouring velocity in the drain, which for a three foot drain is approximately .07%. A plan of the proposed reservoir is shown on drawing 5 giving proposed openings for admission of water without danger oferosion.

Maximum storm flow occurs at a time when water from the remote sections of the shed reach the drain and it is yet raining. Maximum flow from the north shed will thus occur from 76 minutes to 100 minutes after the beginning ofthe storm while on the south maximum storm flow comes after a period of 13 hours and 15 minutes. Plate 1. shows graphically the relation between run-off, outflow, and water level in the reservoir when a three foot drain is used. The reservoir volume used is 10% less than the actual to provide for silting which will surely occur. Flow in the pipe is computed by use of Cutters formula with (n) taken as .013 and advantage is also taken of initial pressure resulting from the water rising in the reservoir. After 76 minutes after the rain begins falling water will be flowing into the reservoir from the entire north slope and from a semicircle on the south whose radius is 76 x 60 x.176. The combined flow being 131.34 cubic feet per second. The drain will at this time be flowing full with water in the reservoir rising and outflow increasing. At 100 minutes the north flow will be the same but the south flow has increased so that the run-off is 134.93 cubic feet per second. At 176 minutes flow from the north due to the initial rate of rainfall has ceased but yet has a reduced flow due to the second rate of rainfall while the flow from the south is rapidly increasing. The drain flow is increasing slightly due to increased head. It takes 13 hours and 15 minutes for water to flow from the extremeties of the south slope so in 14 hoursand 55 minutes flow due to the first rate of rainfall will have reached the reservoir. The run-off during the period 176 minutes to 14 hours andn55 minutes is so great that the three foot drain proves inadequate as the water level stands at 17.48 feet or 2½ feet above the to p of

the reservoir while it is yet raining.

Plate 2. shows a similar treatment for a 4 foot by 3 foot box drain. In this case the drain flows full in a trifle over 100 minutes after the start of the storm. Due to the outfall being under a head it is impossible to keep the pipe from flowing full but an increased rate is used during the time the pipe is filling. In this diagram we find the capacity of the pipe to be sufficient to carry the flow due to the later rate of rainfall and that the surface of the required reservoir is just above that desired, or 15 feet. I t is also shown that the reservoir will be emptied in less that two hours after water ceases to flow into it.

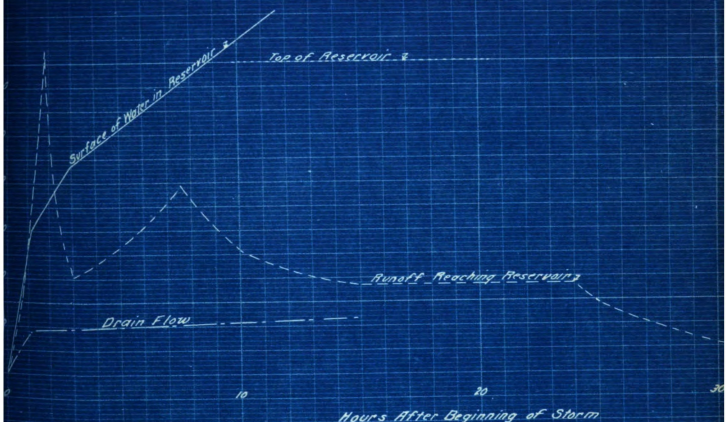
I have in plate 1. and plate 2. used the maximum rainfall for 24 hours with the maximum fall for 100 minutes at the beginning of the storm. Plate 3. shows the performance of the 4 x 3 drain assuming that the rain falls uniformly throughout the day. this shows the reservoir overflowing and makes either a larger drain or reservoir necessary. The reservoir volume could be increased by giving it a uniform section from elevation 9½ to elevation 15 and sloping the bottom from 8½ at the intake to 9½ at the sides.

Plat 4. shows the performance of a 4 x 3 drain under the conditions of an increased reservoir volume and uniform rainfall. The reservoir being again found inadequate I decided to try a 4 foot x by 3 foot 3 inches drain with the smaller reservoir. In plate 5 this scheme is shown a failure.

Plate 6. shows a 4 foot x by 3 foot 3 inch drain under the condition of uniform rainfall and with a maximum reservoir volume.

PLATE I

CIRCULAR DRAIN THREE FEET IN DIAMETER



RAINFALL
1.90 Inches in 100 Minutes
3.83 Inches in 24 Hours

RESERVOIR VOL
346000 Cubic Feet at
1557000 Cubic Feet at

Hours After Beginning of Storm

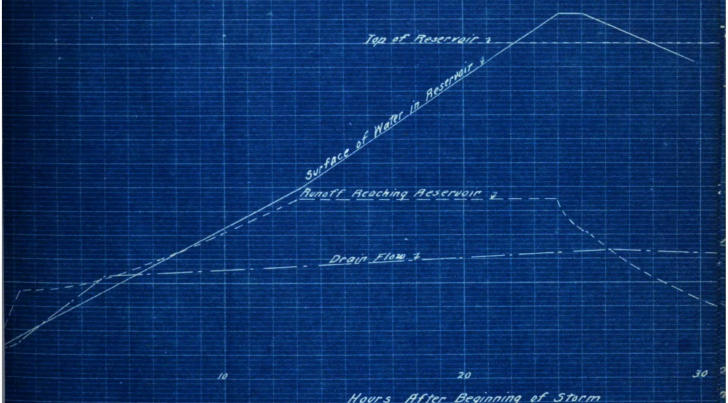
RAINFALL
3.83 Inches in 24 Hours

RESERVOIR V
346000 Cubic Feet a
1557000 Cubic Feet a

Elevation of Reservoir Surface
15
14
13
12
11
10
9
OLUME
Elevation 1150
Elevation 1150

PLATE 4

RECTANGULAR DRAIN FOUR FEET BY THREE FEET



RAINFALL
3.83 Inches in 24 Hours

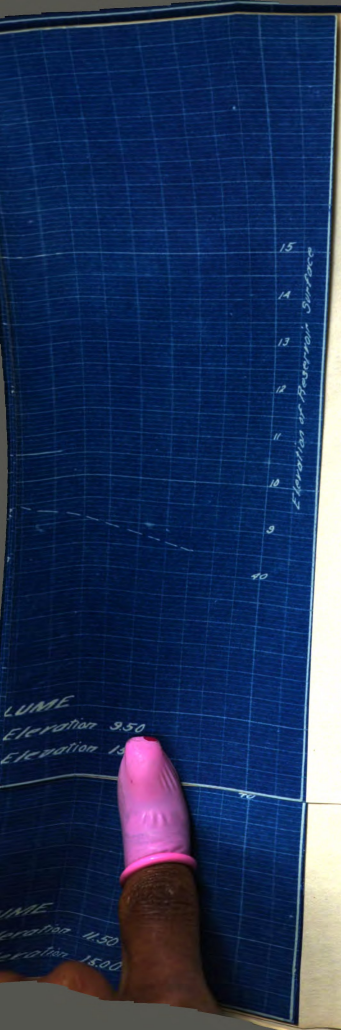
RESERVOIR VO.
115000 Cubic Feet at
2018000 Cubic Feet at

10 20 30

Hours After Beginning of Storm

RAINFALL
3.83 Inches in 24 Hours

RESERVOIR VO
346000 Cubic Feet at
1557000 Cubic Feet at


15
14
13
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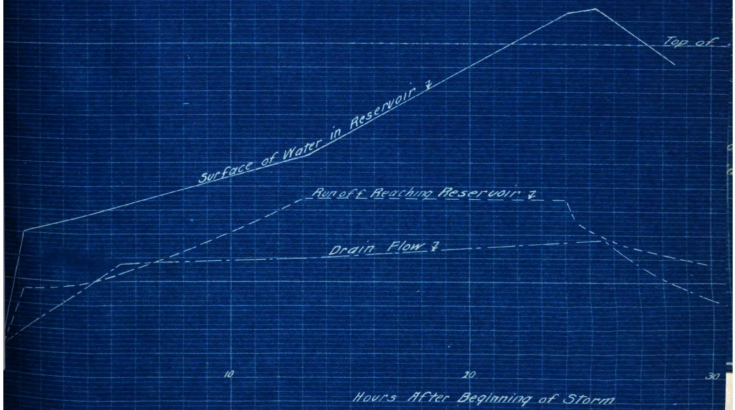
9
10
Elevation of Reservoir Surface
LUME
Elevation 9.50
Elevation 15
10
UME
evation 11.50
evation 15.00

PLAT

RECTANGULAR DRAIN FOUR FEET B

PLATE 5

RECTANGULAR DRAIN FOUR FEET BY THREE FEET-THRE



RAINFALL

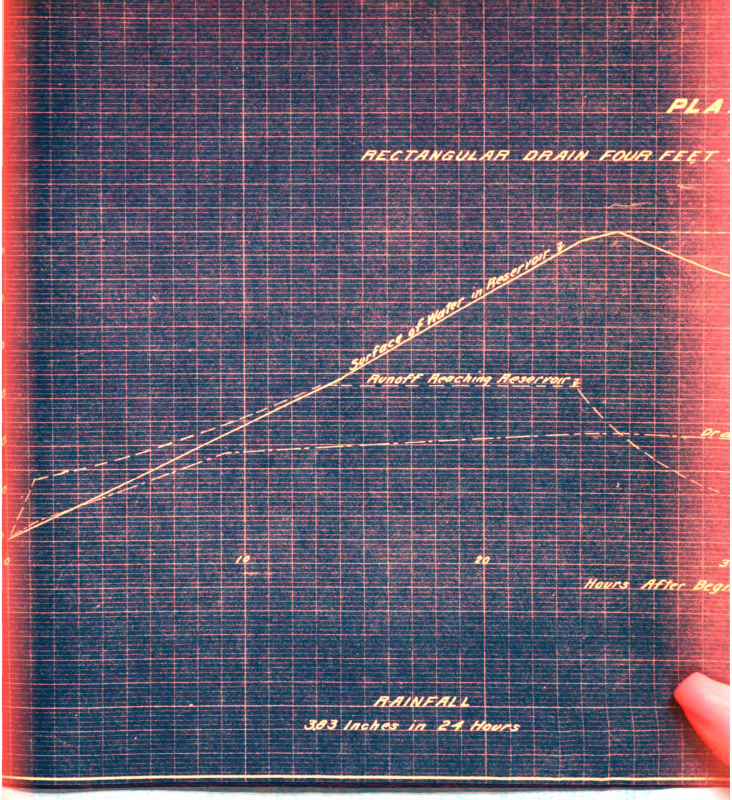
3.83 Inches in 24 Hours

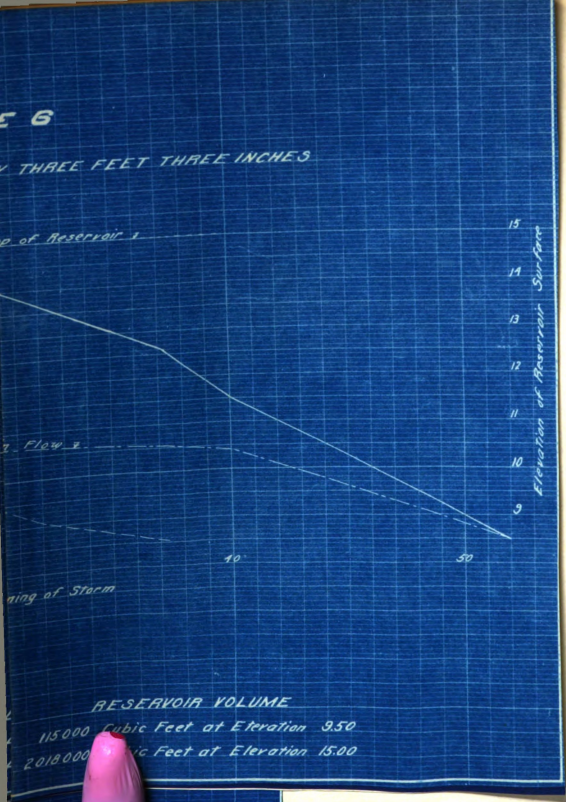
RESERVOIR VO

346000 Cubic Feet of

1557000 Cubic Feet of

E INCHES
Reservoir
15
14
13
12
11
10
9
Elevation of Reservoir Surface
LUME
Elevation 11.
Elevation 15.


PLA
RECTANGULAR DRAIN FOUR FEET
Surface of Water in Reservoir
Runoff Reaching Reservoir
Hours After Begi
RAINFALL
3.83 Inches in 24 Hours


E 6
THREE FEET THREE INCHES
of Reservoir
Flow
ning of Storm
10
50
15
14
13
12
11
10
9
Elevation of Reservoir Surface
RESERVOIR VOLUME
115000 Cubic Feet at Elevation 9.50
2018000 Cubic Feet at Elevation 15.00

The diagram shows the drain to be adequate and that the water level of the reservoir will begin to receed at about the time the rain ceases. Water would continue to reach the reservoir for 13 hours and 15 minutes but in 28 hours after the storm the reservoir would be empty. This assumes the most adverse conditions at the outfall thus providing an additional factor of safety.

The reservoir used in this design calls for an excavation of 58000 cubic yards this earth will raise all the low lands adjacent to the reservoir to elevation 15 and provide 21 acre feet additional fill. Since this locality is neither inhabited nor cultivated several blocks may be graded with this waste dirt thus raising them above the danger zone. This excavation may be handled very cheaply and theadditional cost of enlarging the reservoir is less than that of increasing the cross-section of the drain.

Lack of head room makes necessary the use of a box section over a considerable portion of the work and the section is continued uniform throughout since it obviates the danger of a reduced flow and consequent silting at the points of change in section. The walls of the drain are designed as simple slabs though a few longitudinal rods are introduced and the reinforcing carried continuous through the top and sides. Stresses are computed for a back fill weighing 120 pounds per cubic foot and the following table computed by Marston & Anderson was used in this work. In the streets a consentrated live load of 8670 pounds is used and represents the wheel load of a ten ton road roller. Beneath the right of way of the Pacific Electric Railway a surface load of 650 pounds per square foot is used and beneath

the Salt Lake Railway a load of 1500 pounds per square foot is used, these representing loads caused respectively by the heaviest interurban electric car and largest locomotive. In crossing private property a live load of 350 pounds per square foot is used and beneath sidewalks 200 pounds per square foot.

TABLE NO. 4.

Approzimate maximum loads in pounds per linear foot on pipe in trenches imposed by saturated sand filling weighing 120 pounds per cubic foot.

BREADTH OF TRENCH FOUR FEET.

Depth of fill above pipe.	Pounds per linear foot.
2	880
4	1280
6	2380
8	2980
10	3500
12	3980
14	4390
16	4750
18	5060
20	5330
22	5570
24	5780
26	5970
28	6140
30	6260

TABLE NO. 5.

Proportion of local superficial loads on backfilling which reaches the pipe in trenches with different ratios of depth to width.

SAND AND DAMP TOP SOIL.

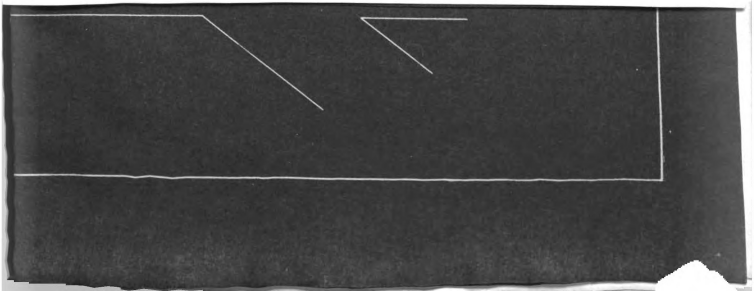
Ratio of depth of fill above pipe to width of trench.	Ratio.
1.0	.59
2.0	.35
3.0	.21

(TABLE NO. 5. CONTINUED).

1.5	.46
2.5	.27
4.	.12
5.	.07
6.	.04
8.	.02
10.	.01

The trench width is uniformly 5 feet. The material throughout is such that side forms will not be necessary and but light timbering will be needed. In placing the floor slab 2 inch planks may be mbeded at the sides thus forming seats or shoulders for the side walls and assisting infloating and bringing the floor to perfect grade. I also suggest placing greased wooden pins on these planks before being embeded thus forming sockets to receive the rods,reinforcing the sides and top. The form work for this design is the simplest possible. There are no intricate stresses to be considered in the design and no difficult steel spacing nor placing. The elements, quantities and number of feet of each section are shown in a table on drawing number 4. The location of these sections are shown on drawing No. 3.

The inlet is placed near the center of the reservoir and details of its construction are shown on drawing No. 4. Its location is chosen with a view of having the heaviest debris deposited before reaching the drain. The mouth of the drain is protected by a number of small of galvanized pipes fixed in the concrete of the upper slab and slightly embeded in the lower slab. Since the reservoir will seldom contain water it may be used as a small park in which case the mouth of the drain would need to be covered to keep children out. These rods may be easily lifted



up permitting entrance for cleaning thus obviating the need of an extra man hole.

Since there are neither curbs nor sidewalks along the south side of State Street from Locust Avenue West the drain is here carried 5 feet from the property line. This reduces the slab sections very materially as the drain here lies very near the surface thus necessitating heavy construction if placed in the roadway. The top of the drain comes just below the gutter grade of the intersecting streets and will form a splendid sub-grade for the sidewalks when built. Manholes in this section are placed in street or alley intersections as property owners will seriously object to a manhole cover in the sidewalk in front of their property.

The outfall is placed on a line with the east face of the opening beneath the present State Street bridge. This insures an outfall into rapidly flowing water without danger of being damaged by the stream or of having a sand bar form in front of it either of which injuries might occur were the outfall located elsewhere. A reinforced wall is run to a depth of 10 feet below the grade line at the outfall as a protection against the pool which may form. Details of the construction at this point are shown in drawing 4. A small footing is provided for the wall since it can be added at little or no extra expense, a 3 foot trench being needed in placing the wall. This footing is placed beneath the drain because it leaves firm earth in front of the wall and will withstand more undercutting before being destroyed. The wall receeds at each side of the outfall as it will look betterthus and have greater stability than a straight wall.

Manholes are provided at intervals slightly exceeding 600 feet. Details of the design of both the manhole and cover are shown on drawing No. 4. The walls are carried up square and a round cast iron ring and cover used. By this design much expensive form work eliminated and nothing lost in efficiency. There are 14 manholes and their location is given in drawing No. 3.

The yearly floods and stagnant pool at the base of Signal Hill are both unsightly and unsanitary. They have also been a great deterrent to the growth of this entire section of the city. I have endeavored to show in this paper that ther is a feasable method now open for draining this land, that it is possible to care for a vast flow of water with a comparatively small drain, and that money thus expended will not be sacrificed for temporary relief since the drain herein designed will become a unite of the larger system to be installed when future growth and improvement make it a necessity and a financial possibility.

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