ANALYSIS OF FLOW IN THE WATER DISTRIBUTION SYSTEM OF EAST LANSING

Thesis for the Degree of B. S.

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Howard M. Kieft

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THESIS

ANALYSIS of FLOW

in the

WATER DISTRIBUTION SYSTEM

of

EAST LANSING

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THESIS

PREFACE

This thesis has been written primarily to bring up to date the analysis and recommendations for extension of the East Lansing Water Supply. An attempt has been made to briefly summarize previous reports on this subject, and to recommend with reasonable consideration, the present and future needs for expansion of the water supply in this fast growing residential city.

In the past few years new methods of analysis of flow in water mains and circuits have been developed, of which one of the more recent publications in 1936 by Hardy Cross, Professor of Structural Engineering at the University of Illinois, has been of great value to the author in this analysis.

Attention is called to the fact that this treatise on the subject is mainly analytical rather than factual. The necessary facts of analysis are included, but the main attention is devoted to the future needs of the water supply.

Sincere appreciation is extended to Professor F. R. Theroux, the City Engineering Department of East Lansing and all those who assisted in the presentation of this thesis.

HOWARD M. KI[illegible]

EAST LANSING, MICHIGAN

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ANALYSIS of FLOW
in the
WATER DISTRIBUTION SYSTEM
of
EAST LANSING

INTRODUCTION

During the past thirty years the East Lansing water supply and distribution system has been analyzed, or investigated in part by three other parties. The first occuring in 1910 when J. W. Knecht and P. G. McKenna wrote, "An Investigation of the Water Supply of East Lansing, Michigan." The second time in 1923 by L. M. VanNoppen entitled, "An Analysis of East Lansing's Water Supply System and Recommendations for Enlargement." The third time in 1925, "East Lansing's Fire Protection," written by C. H. Miller and P. H. Slack.

Another analysis of the distribution system at this time is seemingly appropriate to the existing conditions. Since the last analysis was made sixteen years ago many changes have taken place in the growth of the city, demand for water and the city's water supply has also increased much in size. One important factor that has been lacking in this period of time is the growth of the distribution system as will be pointed out later.

The purpose is to show the existing conditions of the distribution system with regards to maximum flow for domestic use and fire fighting facilities. The rapid growth of the city has heavily taxed the distribution system at times of maximum use in mid-summer when a great deal of water is used for sprinkling lawns. Many of the old two-inch mains are still in use, and many of the changes that were recommended in 1925 are not to be found in the system as yet.

This city is located in an area where no reasonably clean surface water supply, such as a large lake or non-polluted river is available. The only practical source of supply that would not require extensive treatment is from deep wells. The first city water supply consisted of a 6-inch bored well 362 feet deep that was constructed some time before 1910 when the city was still very small. * "The city obtains its supply of water from a bored well, 6-inches in diameter and 362 feet in depth. It is located near the western end of the city and at the edge of a marshy area." This location is now known as the old pump house located on the northwest corner of the Central Recreational Park, near the corner of Hillcrest and Oakhill Streets. The area which was previously marshy has been drained and developed into a recreational field in the approximate center of the city. The city has grown from about 850 population in 1910 to 5,839 in 1940. The first

*From thesis of J. J. Knecht and P. G. McKenna, 1910 water was drawn from that well on October 20, 1909 and after a few months service the pump was producing an average of 63.11 gallons per minute. Distribution mains served 90 houses, 60 of which were metered in orded to cut down the amount of waste and to provide a basis for charge of service.

The population prediction of 1910 proved fairly accurate for the city although that prediction as to the college population was greatly in error. He gave the population of the city as 100 in 1900 and 850 in 1910, and from his curve of the population he predicted the following: * "This would give an estimated population of seventeen hundred in 1918. We have every reason to believe that the college attendance will not increase much over 2,000 and at this figure it will probably remain almost constant." The population of the city was very near seventeen hundred in 1918, but as we know the college attendance had grown to 6,850 by 1940. Considering the short period of time for which records were kept previous to 1910, the predictions of Knecht and McKenna were very reliable.

Before 1923 the city's supply was increased by the construction of a 12-inch well 430 feet deep at the old pumping station near the corner of Hillcrest and Oakhill

*From thesis of J. W. Knecht and P. G. McKenna, 1910

Streets. The population of the city in 1923 was about 2,500 and at that time it was predicted to rise to 7,000 by 1940 and 10,000 by 1955.

Water drawn from this depth is very likely to be undesireable to the user because of hardness. In order to enable the water to reach the well it probably travels through a broad area of aquifier that has its outcrop many miles away. In this long underground travel of the water it will probably pick up a very heavy load of mineral matter which will make it hard, corrosive, or undesireable in other ways. That is one of the difficulties found in East Lansing, and as far back as 1923 the lime-soda method of softening was discussed as a possibility for the improvement and expansion of the city water supply.

About in the year 1924 another new well was drilled on the east side of the city on Orchard Street. That location is now known as the East Treatment Plant. This well was 12-inch diameter, about 480 feet deep, and capable of an outout of 250 gallons per minute.

The next and probably the most needed step in the development of the water supply was taken in the spring of 1933 when the people of the city voted in favor of the construction of a water softening and iron removal plant. At about the same time another 12-inch well was drilled on the intersection of Crohard and Beech Streets, which is about 400 feet north of the treatment plant. The plant is equiped with five cylindrical softening tanks, each seven feet in diameter and eleven feet high. Each tank contains 200 cubic feet of green-sand zeolite capable of removing 2,800 grains of hardness per cubic foot, or a total of 560,000 grains hardness removed per tank between regenerat-ion periods. The plant was built with two iron removal tanks of the same size but one of these tanks has been changed to a softening tank so now it has only one iron removal tank and six softening tanks. A 100,000 gallon elevated storage tank is a part of the plant.

In 1939 another new well of 12-inch diameter and approximately 400 feet in depth was drilled near the west end of the city on Saginaw Street. The West Treatment Plant was constructed on this property and started regular operation on January 3, 1940. This plant is equiped with three softening tanks, one iron removal tank and a 200,000 gallon elevated storage tank located about 1,400 feet north of the plant.

The first two wells located on the Central Recreation Park and used for some time, were discontinued from service over ten years ago but the exact time of discontinuation is not known.

The city water supply is independent of the college supply except for a two-way metered exchange box that is very seldom used. Any time that the city water supply is interrupted it can draw water from the college supply, or the city could help supply the college in case their supply is interrupted. Whenever water is exchanged in either direction it is metered and paid for by the reciever at a service rate. Owing to the fact that the city has three wells with two treatment plants and the college has many wells, about the only way that either supply could be interrupted for any appreciable length of time would be by complete disruption of electric power. The college owns and operates its own power plant and the city obtains its power from a different source so it would be practically impossible to have both supplies disrupted at the same time.

METHOD OF ANALYSIS

The data and information required in order to carry out a complete analysis of a water system of such a size as this probably requires more assumptions than the use of actual data. Therefore familiarity with the system and rechecking of data are of great importance to the obtaining of reliable results.

Maps of the system must be available with the sizes and length of all mains given. It would be advisable to make a tracing of the system and then assemble the required data as to size, length and coefficient of resistance directly on this tracing in order to compact the main data in a convenient form for computations. The coefficient of resistance is determined by the size, length and roughness of the pipe, and is solely dependent on these factors.

Records of the output of the plant over a period of two or three years should be available and thoroughly studied as an assistance in determining the probable maximum needs of the city. To this maximum need for domestic use an additional amount of flow must be added to provide for fire fighting use. An analysis of the distribution system is carried out to determine wether the system is capable of taking care of a maximum flow without unnecessary loss of head in distribution to the desired point. Therefore we analyze with maximum flow. A fair assumption for maximum domestic use might be taken at three times the average use, and as will be seen later this assumption was found to be very near correct in our work.

The method of analysis used was first developed by Hardy Cross, Professor of Structural Engineering at the University of Illinois, and was published by him in 1936. Since that time there have been many variations of this method, but investigation of these relates back to the basic principles set up by Cross.

To summarize, the method consists of assembling the data on the diagrams of the system in convenient form; assuming any flow distribution throughout the entire system; calculation of flow corrections due to erronous assumptions; application of these computed corrections and repeating the procedure until the head loss, by any path of travel between two points, is balanced.

The Hazen-Williams formula for loss of head in pipes is expressed as $\underline{h = rQ^n}$. In this equation $\underline{h}$ is the head loss, $\underline{r}$ is the coefficient of resistance, $\underline{Q}$ is the quantity of water flowing in the pipe and $\underline{n}$ is a constant depending on factors of roughness of the pipe. In determining the value of $\underline{r}$ account must be made of the roughness of the pipe, which is greatly dependent upon the length of time that the pipe has been in servise. Thus in obtaining $\underline{r}$ we must revert back to another coefficient $\underline{C}$ which gives the roughness in grades. Values of $\underline{C}$ run from 140 for extremely smooth pipe down to 60 for old pipe that is very rough. One of the most common values for cast iron pipe that has been in use for some years is 100. Then the coefficient r for any size and length of pipe can be computed from Table 1.

TABLE 1. Values of r for 1000 ft of pipe based on Hazen-Williams formula.

d-in.	c = 90	c = 100	c = 110	c = 120	c = 130	c = 140
4	340	246	206	176	151	135
6	47.1	34.1	28.6	24.3	21.0	18.7
8	11.1	8.4	7.0	6.0	5.2	4.6
10	3.7	2.8	2.3	2.0	1.7	1.5
12	1.6	1.2	1.0	0.85	0.74	0.65
14	0.72	0.55	0.46	0.39	0.34	0.30
16	0.38	0.29	0.24	0.21	0.18	0.15
18	0.21	0.16	0.13	0.11	0.10	0.09
20	0.13	0.10	0.08	0.07	0.06	0.05
24	0.052	0.04	0.03	0.03	0.02	0.02
30	0.017	0.013	0.011	0.009	0.008	0.007

"The flow correction is calculated by dividing the error in the head loss in each circuit by $1.85\Sigma rQ^{0.85}$ for that circuit. The error in the head loss is determined as follows: First calculate the loss of head due to the assumed flows which are clockwise in direction. Next, calculate the loss of head due to assumed flows which are counterclockwise.

The arithmetic difference of these totals gives the error in head loss. The value of $\Sigma rQ^{0.85}$ is computed for the entire circuit being studied and the summation is made without reference to sign."

A typical problem will greatly aid in explanation of the process used although there are some necessary changes in any actual problem. Figure 1 shows a simplified layout of a distribution system giving size, length and coefficients r as well as the amount of water drawn at each point.

Values of the o.85 powers of numbers from 0 to 99 are given in Table 2.

Figures 2, 3 and 4 show the successive corrections for the circuits of the example shown in Figure 1. It will be noted that there are three flow figures on pipes common to two circuits. This is caused by the use of corrected flows of one circuit for computation of the correction in adjacent circuits.

"The arrows placed at the right of the calculations and pointing from the smaller to the larger values of the head loss, h, indicate the direction of the flow correction in each circuit. The flow correction is added to or subtracted from the assumed flow depending upon wether its direction is the same or opposite to that of the assumed flow. The corrected flow is then placed on a new diagram and the procedure is repeated until the head loss, for clockwise and counter-clockwise flow, is balanced within any desired limit of error. It is rarely necessary in water works problems to apply more than three corrections."

TABLE 2. Values of the 0.85 power of numbers.

N	0	1	2	3	4	5	6	7	8	9
0	0	1.0	1.8	2.5	3.2	3.9	4.6	5.2	5.9	6.5
10	7.1	7.7	8.3	8.9	9.5	10.0	10.6	11.1	11.6	12.2
20	12.8	13.3	13.8	14.4	14.9	15.4	15.9	16.4	16.9	17.5
30	18.0	18.5	19.0	19.5	20.0	20.5	21.0	21.5	22.0	22.5
40	23.0	23.4	23.9	24.3	24.8	25.3	25.8	26.3	26.8	27.3
50	27.8	28.2	28.7	29.1	29.6	30.0	30.5	31.0	31.4	31.9
60	32.4	32.9	33.3	33.8	34.2	34.7	35.1	35.6	36.0	36.5
70	37.0	37.4	37.9	38.3	38.7	39.1	39.6	40.0	40.5	41.0
80	41.5	42.0	42.4	42.8	43.3	43.7	44.1	44.5	45.0	45.4
90	45.8	46.3	46.7	47.1	47.6	48.0	48.4	48.8	49.2	49.6

FIGURE 1. Sketch of pipe layout.

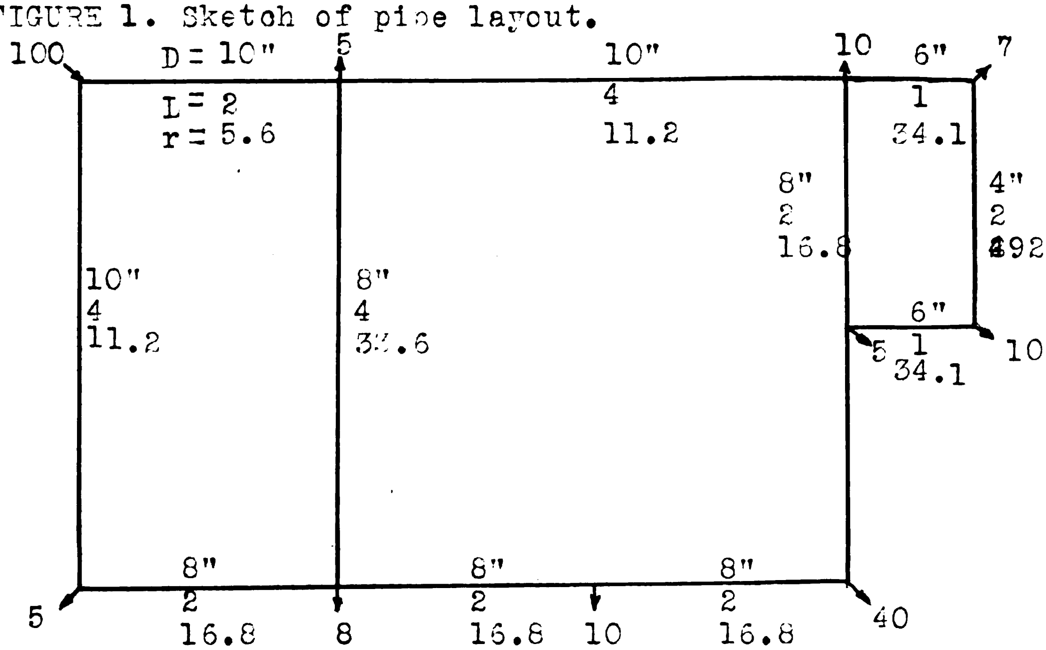
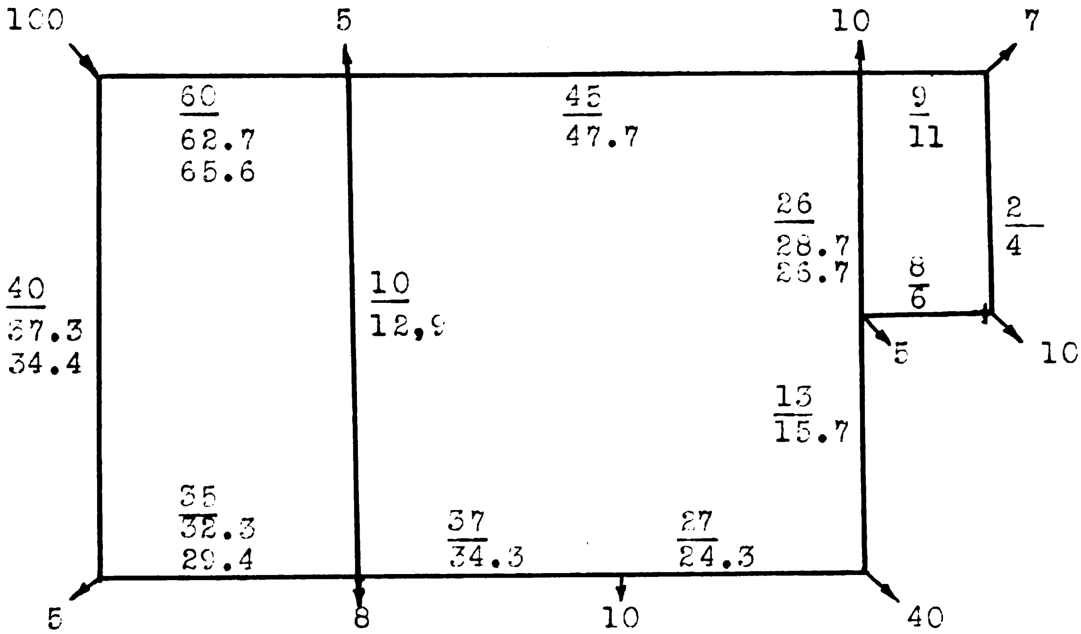


FIGURE 2. First correction.



Underlined figures show assumed flow distribution with corrected figures below.

Main Circuit

r		$Q^{0.85}$		$rQ^{0.85}$		Q		h	
5.6	x	32.5	=	182	x	60	=	10900	
11.2	x	25.4		285	x	45		12800	
16.8	x	16.0		269	x	26		7000	
16.8	x	8.8		148	x	13		1920	32620
11.2	x	23.0		258	x	40		10300	
16.8	x	20.5		344	x	35		12050	
16.8	x	21.5		361	x	37		13350	
16.8	x	16.5		277	x	27		7480	43180
				2124					10560

$$\frac{10560}{1.85 \times 2124} = 2.68$$

Correct 2.7

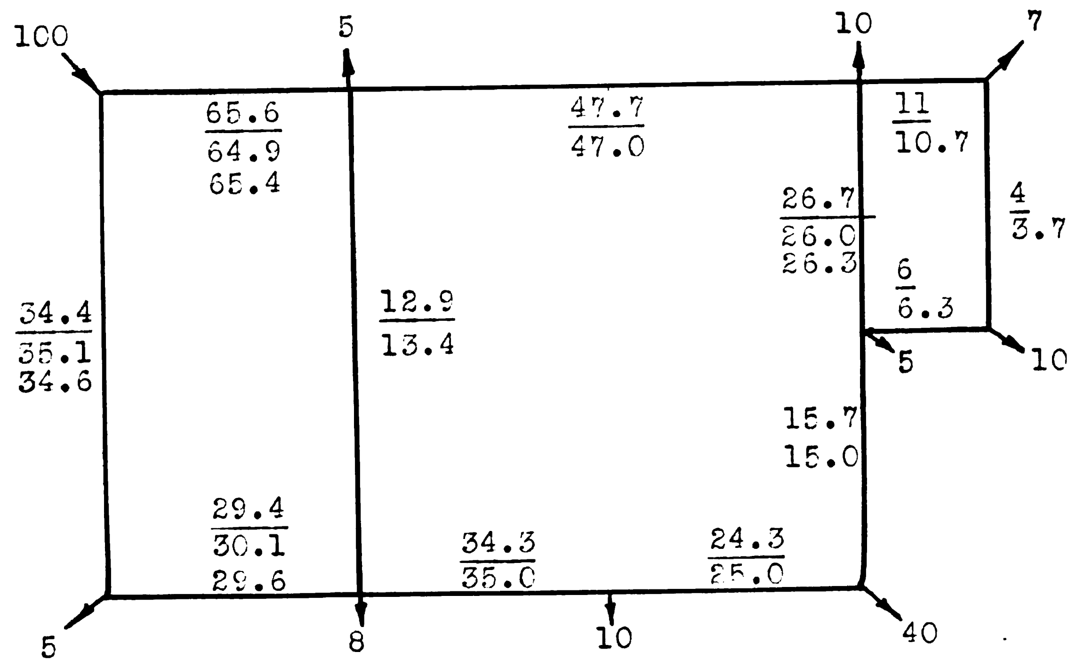
Side Circuit

34.1 x 6.5 =	222 x 9 =	1995		
492 x 1.8	885 x 2	1770	3765	
16.8 x 17.4	292 x 28.7	8390		
34.1 x 5.9	201 x 8	1610	10000	
	1600		6235	
Correct 2.1			1.85 x 1600	= 2.10

Cross pipe

5.6 x 33.6 =	168 x 62.7 =	11800		
33.6 x 7.1	238 x 10	2380	14180	
11.2 x 21.7	243 x 37.3	9070		
16.8 x 19.2	322 x 32.3	10400	19470	
	991		5290	
Correct 2.9			1.85 x 991	= 2.88

FIGURE 3. Second correction.



Main Circuit

r	$Q^{0.85}$	$rQ^{0.85}$	Q	h	
5.6	34.9	195	65.6	12800	
11.2	26.7	299	47.7	14300	
16.8	16.3	274	26.7	7310	
16.8	10.4	175	15.7	2740	37150
11.2	20.2	236	34.4	7790	
16.8	17.7	298	29.4	8750	
16.8	20.2	339	34.4	11640	
16.8	15.1	254	24.3	6160	34340
		2070			2810

$$\frac{2810}{1.85 \times 2070} = 0.73$$

<u>Correct 0.7</u>

Side Circuit

r	$Q^{0.85}$	$rQ^{0.85}$	Q	h	
34.1	7.7	263	11	2890	
492	3.2	1575	4	6300	9190
16.8	15.9	267	26	6950	
34.1	4.6	157	6	940	7890
		2262			1300

$$\frac{1300}{1.85 \times 2262} = 0.31$$

<u>Correct 0.3</u>

Cross Pipe

r	$Q^{0.85}$	$rQ^{0.85}$	Q	h	
5.6	34.7	194	64.9	12600	
33.6	8.9	299	12.9	3590	16190
11.2	20.6	231	35.1	8100	
16.8	18.1	304	30.1	9150	17250
		1028			1060

$$\frac{1060}{1.85 \times 1028} = 0.56$$

<u>Correct 0.5</u>

* "After the circuits are balanced to within an allowable error the loss of head in feet between any two points may be calculated by multiplying the sum of the h values between the points by (0.01 x total flow in gallons per minute)$^{1.85}$ divided by 100,000

$$\frac{h}{100,000} \times (\text{gallons per minute})^{1.85}$$

This equation makes it a relatively simple matter to construct a pressure contour map of the system."

The above statement could not be verified in computation.

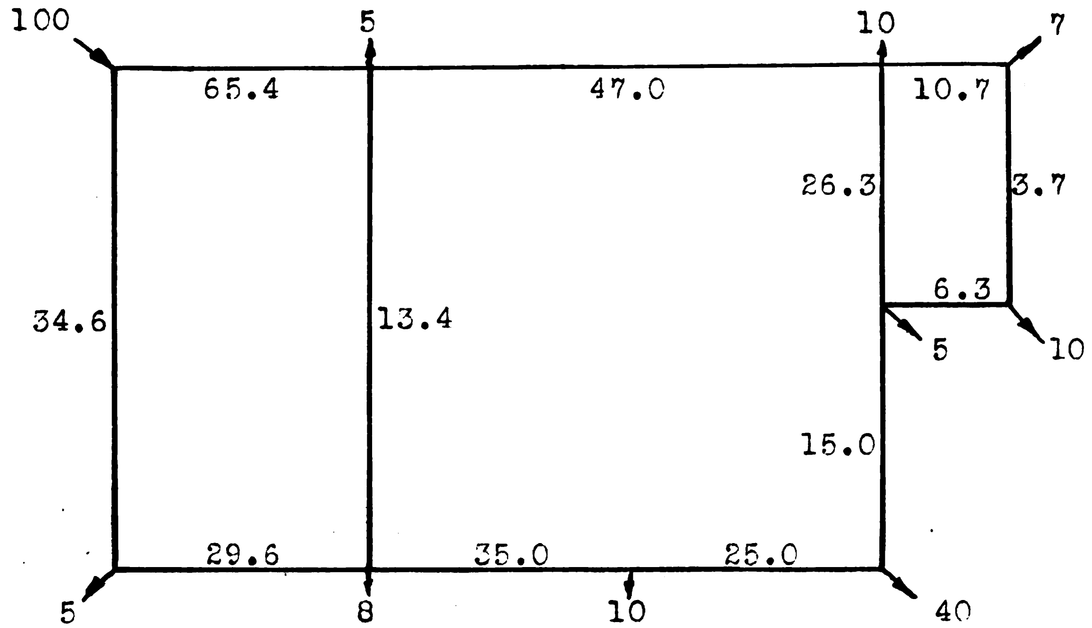
The final corrected values shown in Figure 4 are computed in terms of percentage of flow with 100% entering at one point and the percent of total flow in any pipe is shown by the final corrected value. From the known value of gallons per minute as total flow, each required percentage can be restated in terms of gallons per minute flowing and the loss in head computed between any two points in the circuit.

* "Multiple sources of supply and elevated storage may also be included in the study by this method.

The circuits may be calculated in any order desired and any direction of flow may be assumed. If the later is in error the counter balancing flow or flow correction will correct it. The flow correction will also eliminate accidental error in calculation provided they are not persistently made.

*J. J. Doland, Simplified Analysis of Flow in Water Distribution Systems., Eng. News Record Vol. 117

FIGURE 4. Solution.



Main Circuit

r		$Q^{0.85}$		$rQ^{0.85}$		Q		h	
5.6	x	34.9	=	196	x	65.4	=	12800	
11.2	x	26.3		294	x	47.0		13850	
16.8	x	16.1		270	x	26.3		7100	
16.8	x	10.0		168	x	15.0		2520	36270
11.2	x	20.3		228	x	34.6		7860	
16.8	x	17.8		299	x	29.6		8850	
16.8	x	20.5		344	x	35.0		12050	
16.8	x	15.4		258	x	25.0		6460	35220
				2057					1050

$$\frac{1050}{1.85 \times 2057} = 0.27$$

<u>No Correction</u>

Side Circuit

34.1 x 7.5 =	256 x 10.7 =	2740	
492 x 3.0	1475 x 3.7	5450	8190
16.8 x16.1	270 x 26.3	7100	
34.1 x 4.8	164 x 6.3	1030	8130
	2155		60

No Correction

$$\frac{60}{1.85 \times 2155} = 0.015$$

Cross Pipe

5.6 x 34.9 =	196 x 65.4 =	12800	
33.6 x 9.1	306 x 13.4	4100	16900
11.2 x 20.3	228 x 34.6	7860	
16.8 x 17,8	299 x 29.6	8850	16710
	1028		190

No Correction

$$\frac{190}{1.85 \times 1029} = 0.10$$

It is quite important that the total assumed inflow and outflow at any junction be balanced. Minor losses of head have been neglected in the solution presented."

The following computation indicates the change taken place when the flow is changed from percent to actual flow in gallons per minute or cubic feet per second.

Assume the 100 percent flow equals 1,000 gallons per minute. Then for simplicity the other indicated percents multiplied by 10 become flows in gallons per minute. The head at the entrance equals 50 pounds per square inch. What is the head loss and the pressure at the exit where

40 gallons per minute is taken off at the far corner of the circuit?

The following method of approach was used in finding the actual loss of head in feet for the circuit. The amount of flow in gallons per minute were changed over to cubic feet per second. Then knowing the quantity of flow and the size of the pipe, the loss per thousand feet of length could be determined from the, "Friction Loss of Head" diagram, Figure 132 on page 266, "Hydraulics" by Schoder and Dawson.

Main Circuit - Clockwise

Q(gpm)	Q(cfs)	Diam.	h/1000ft	L feet	h	
654	1.45	10"	3.0	2000	6.0	
470	1.05	10"	1.6	4000	6.4	
263	.58	8"	1.5	2000	3.0	
150	.33	8"	0.6	2000	1.2	16.6

Counter-clockwise

346	.77	10"	0.9	4000	3.6	
296	.66	8"	2.1	2000	4.2	
350	.78	8"	2.8	2000	5.6	
250	.56	8"	1.5	2000	3.0	16.2

Side Circuit - Clockwise

107	.24	6"	1.4	1000	1.4	
37	.08	4"	1.3	2000	2.6	4.0

Counter-clockwise

263	.58	8"	1.5	2000	3.0	
63	.14	6"	0.5	1000	0.5	3.5

Cross Pipe - Clockwise

654	1.45	10"	3.0	2000	6.0	
134	.30	8"	0.5	4000	2.0	8.0

Counter-clockwise

346	.77	10"	0.9	4000	3.6	
296	.66	8"	2.1	2000	4.2	7.8

Now the head on the outlet in the sample problem can be computed.

Head at entrance - - - - - 50 lbs./ square inch

Head loss in feet - - - - 16.6 ft. - clockwise

- - - - 16.2 ft. - counter-clockwise

- - - - 16.4 ft. - average

2.31 ft head = 1 pound

Head loss in pounds - - - $\frac{16.4}{2.31} = 7.1$ lbs./ square inch

Head at outlet - - - - - - - 42.9 lbs./ square inch

As will be noted a discrepancy of o.4 ft. could be possible in errors caused by droping off part of decimals in computation and in reading tables, so the average of the two is used.

In trying to find a simpler method of computing the actual head loss the author noted that the ratio that should exist between the comparative value of h and the actual values of $\underline{h}$, if investigated, could be converted to a simple proportion in the form of a constant figure. This would eliminate all computations of Q and conversions in computing the actual head loss.

The following table of computations proves this fact by establishing that proportion.

h (comparative)	h (actual)	$\frac{\text{h (comparative)}}{\text{h (actual)}}$
12800	6.0	2130
13850	6.4	2160
7100	3.0	2360
2520	1.2	2100
7860	3.6	2180
8850	4.2	2110
12050	5.6	2150
6460	3.0	2160
2740	1.4	1960
5450	2.6	2100
7100	3.0	2360
1030	0.5	2060
12800	6.0	2130
4100	2.0	2050
7860	3.6	2180
8850	4.2	2110

$$\frac{34300}{16} = 2140$$

These figures mostly lie a little above 2100 and the average was found to be 2140. Now this value of 2140 can be used with comparative accuracy to find the actual head loss in any single pipe or circuit, without any referance to the size or quantity of flow. This constant will change with each problem worked, but the constant for any problem can be determined with only a small amount of computation.

DATA

The most useful data of value in computation would be the data on the maximum use of water in each block in the city. Such data would give definite values as to the amount of water used in any section and the total use.

In 1939 the city water supply department made a survey of their records to determine the number of families, in each of three sections of the city, that used a specified amount of water over a three month period. Table 3 gives these values in consecutive order ranging from 0 to 10,000 gallons for the first interval and goes up to 100,000 gallons for the last interval, and lists the number of families using within the range of each interval. Two of the three sections are also divided into two parts.

Table 4 gives the most important pumping data for the year of 1940. The total monthly amount of water metered from the plant to the distribution mains is of importance in determining the months of the year when the plants are operating at a miximum rate.

Another important factor is the present and future expected population of the city. In an expanding city an allowance must be made in design for development of the water supply system. The city of East Lansing will probably develope to more than double its present population within the next twenty or thirty years for many reasons.

TABLE 3. Number of families using water and the number of gallons used per connection over a three month period.

Gallons of water used per connection (thousand)	District 1	2 - 1	2 - 2	3 - 1	3 - 2	City Total
	Number of Connections					
0 - 10	75	64	49	89	62	339
10 - 20	175	179	104	131	119	708
20 - 30	70	55	40	49	38	252
30 - 40	21	15	10	19	17	82
40 - 50	9	7	7	7	7	37
50 - 75	10	9	10	10	4	43
75 - 100	3	3	2	7	2	17
Over 100	7	3	4	7	4	25
						1503

TABLE 4. Data on East Treatment Plant - 1940

Taken from records of the East Lansing Water Supply Department.

Volumes in gallons

Month	Pumped into treatment plant	Metered from treatment plant	Average daily into plant	Average daily from plant
January	3,310,000	2,811,000	107,000	91,000
Ferbuary	7,400,000	6,290,000	255,000	217,000
March	4,300,000	3,660,000	139,000	118,000
April	4,540,000	3,850,000	151,000	128,000
May	7,180,000	6,100,000	231,000	197,000
June	8,180,000	6,950,000	272,000	232,000
July	15,670,000	13,320,000	505,000	430,000
August	12,330,000	10,510,000	399,000	339,000
September	7,950,000	6,760,000	265,000	215, 000
October	10,000,000	8,500,000	322,000	274,000
November	11,090,000	9,430,000	369,000	314,000
December	9,260,000	7,870,000	298,000	254,000

TABLE 4. (continued)

Data on West Treatment Plant - 1940

Volume in Gallons

Month	Pumped into treatment plant	Metered from treatment plant	Average daily into plant	Average daily from plant
January	15,420,000	14,024,000	498,000	453,000
February	11,260,000	10,245,000	388,000	353,000
March	13,000,000	11,821,000	420,000	382,000
April	13,470,000	12,128,000	449,000	403,000
May	11,290,000	10,161,000	364,000	328,000
June	9,990,000	9,078,000	333,000	302,000
July	12,820,000	11,666,000	414,000	376,000
August	13,800,000	12,556,000	445,000	405,000
September	11,760,000	10,693,000	392,000	356,000
October	12,700,000	11,576,000	410,000	373,000
November	11,790,000	9,263,000	393,000	307,000
December	11,160,000	10,154,000	360,000	327,000

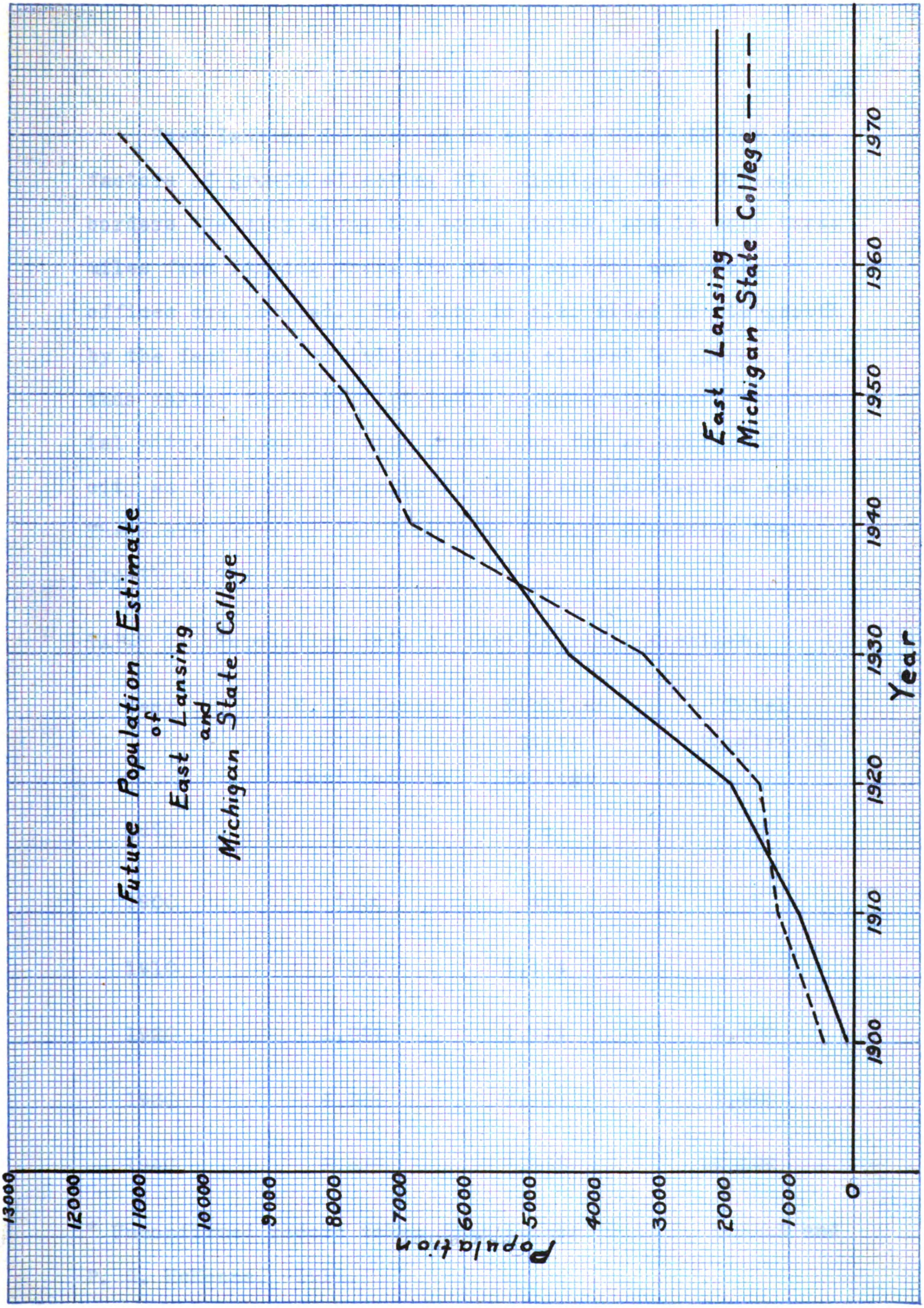
TABLE 4. (continued)

Total Treatment Data on Plants - 1940

Volume in Gallons

Month	Pumped into treatment plant	Metered from treatment plant	Average daily into plant	Average daily from plant
January	18,730,000	16,835,000	605,000	544,000
February	18,660,000	16,535,000	643,000	570,000
March	17,300,000	15,481,000	559,000	510,000
April	18,010,000	15,878,000	600,000	531,000
May	18,470,000	16,261,000	595,000	525,000
June	18,170,000	16,028,000	605,000	534,000
July	28,490,000	24,986,000	919,000	806,000
August	26,160,000	23,066,000	844,000	743,000
September	19,710,000	17,453,000	677,000	561,000
October	22,700,000	20,076,000	733,000	647,000
November	22,880,000	18,693,000	762,000	621,000
December	20,420,000	18,024,000	658,000	581,000

FIGURE 5.


Future Population Estimate
of
East Lansing
and
Michigan State College
East Lansing
Michigan State College
Year
1900
1910
1920
1930
1940
1950
1960
1970
Population
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000
12000
13000

East Lansing is located in an area where it has many factors of growth affecting it. Michigan State College borders the city along the south limits, and is only three miles from Lansing where the state capitol and state offices are located. The city is also aided in its growth by the fact that no industries are allowed to operate here, thus making the city a strictly uncommercialized, residential city. There is still great room for expansion of the city as it is now becoming a high class residential area. These are probably the four main factors affecting the growth of the city.

TABLE 5. Population Data of East Lansing, Michigan.

Year	U. S. Census of East Lansing	Michigan State College	Total Population
1890	?	?	?
1900	*100	463	?
1910	*850	1,174	?
1920	1,889	1,411	3,300
1930	4,389	3,211	7,600
1940	5,839	6,850	12,689

* Unconfirmed record taken from thesis of J. W. Knecht and T. G. McKenna, -1911.

The growth of East Lansing has been quite unusual to other cities in that it is only about forty years old, and in three of those four decades the population at the end of the decade was way over 100 percent of the population at the beginning of the decade. We have reason to believe that the city will continue to grow at this rate for some time to come. The expected population of the city in 1970 will be about 10,700. This figure may be found to be very small if the college continues to grow at its present rate although it is expected that the college population will drop next year in effect of the present selective service law, and the possibilities of this country entering into the European war may also affect the college growth. The college population increased only a few hundred in the decade from 1910 to 1920, and will probably repeat this procedure during the next ten years. Then both the city and college will grow at about the same rate as they have done in the past.

COMPUTATIONS

The preliminary computations with assumptions for the analysis of flow in the distribution system do not include very much work. The only necessary information to be determined is the maximum output of the treatment plant, and the approximate amount of water drawn off at each outlet.

The capacities of the two treatment plants in the city are not known exactly for any period of time and vary greatly with any change in efficiency. The new plant on the west side of the city has only one well and is capable of an output of 580,000 gallons per day. This is equal to 402 gallons per minute.

The treatment plant on the east side of the city has two wells, but does not have enough softening tanks to operate on full capacity of both pumps. Therefore when it is necessary to operate both pumps, one of both of them are partially shut down to prévent over-run of the softening tank capacities. This plant operates on an automatic time cycle so if the pumping is to great the softening capacity of the tanks may be over-run before the regeneration cycle is complete. When the plant is operating at its normal rate of 50 minute time cycle the plant is capable of an output of 866 gallons per minute. In case of a great emergency the two pumps could be put into maximum operation and by-pass the treatment plant. Under this condition the pumps could produce 1,000 gallons per minute, but it would be highly undesireable to by-pass the treatment process under any condition. The maximum combined capacity of the two plants is then established at 1,268 gallons per minute.

From the data of use of water in 1939 shown on page 23 it could easily be supposed by inspection of the figures that the values in the first five range intervals would represent the usage of water in private homes. The next two intervals would represent the use in large rooming houses, fraternity houses and sorority houses, and the last interval represents the largest users such as the apartments and restaurants.

It is desirous, from those figures, to determine the average amount of water used per connection in each of the three classes of size.

Average use in houses.

$$A = 1,000 \times \frac{\frac{339 \times 10}{2} + \frac{708 \times 30}{2} + \frac{252 \times 50}{2} + \frac{82 \times 70}{2} + \frac{37 \times 90}{2}}{339 + 708 + 252 + 82 + 37}$$

$$A = 1,000 \times \frac{1,695 + 10,620 + 6,300 + 2,870 + 1,665}{1,418}$$

$$A = 1,000 \times \frac{23,150}{1,418}$$

A = 16,320 gallons per connection per three months

In computation the average use per connection was raised up to 20,000 gallons per three months.

$$\frac{20,000}{90 \text{ days}} = 222 \text{ gallons per day per connection}$$

$$\frac{222}{1440} = 0.166 \text{ gallons per minute per connection}$$

Average use in rooming houses, fraternity houses and sorority houses.

$$A = 1,000 \times \frac{\dfrac{43 \times (50 \quad 75)}{2} + \dfrac{17 \times (75 \quad 100)}{2}}{43 + 17}$$

$$A = 1,000 \times \frac{2,687 + 1,488}{60}$$

$$A = 1,000 \times \frac{4,175}{60}$$

A = 69,580 gallons per connection per three months

In computation the average use per connection was used as 70,000 gallons per three months.

$$\frac{70,000}{90 \text{ days}} = 777 \text{ gallons per day per connection}$$

$$\frac{777}{1440} = 0.54 \text{ gallons per minute per connection}$$

There are twenty five establishments using over 100,000 gallons per three month period. The use in these places runs anywhere from 100,000 up to 1,000,000 gallons in the three months with the average probably nearer the lower bracket. Let us assume that the average use for these would be about 250,000 gallons per three months.

$$\frac{250{,}000}{90 \text{ days}} = 2{,}780 \text{ gallons per day per connection}$$

$$\frac{2{,}780}{1{,}440} = 1.93 \text{ gallons per minute per connection}$$

Hillcrest village uses over 1,000,000 gallons per three month period.

$$\frac{1{,}000{,}000}{90 \text{ days}} = 11{,}100 \text{ gallons per day}$$

$$\frac{11{,}100}{1{,}440} = 7.8 \text{ gallons per minute}$$

At this time the city maps were consulted to determine the number of houses drawing water from each of 124 outlets tentatively evenly distributed throughout the city. From these figures a much more accurate distribution could be assumed than by arbitrary distribution and flow assumption of the outlets. Combination of sets of these outlets are shown in Figure 7 as they were later combined into 31 outlets ranging in size from 0,5 percent to 5.0 percent of the total flow, and two outlets of 22.0 percent and 25.5 percent at the two hydrants down-town where it is assumed this amount is required for fighting fire.

In order to compute such a problem we must assume the domestic use is at a maximum, and that we have a fire raging that may need 4 to 6 hose lines to combat or to control such fire. Then the distribution system is operating under the worst possible condition of heavy flow.

At peak use of water in the later part of July and the first part of August in 1940 the treatment plants were operating at a peak capacity with the West Plant running as much as 24 hours a day, and the East Plant had one pump running 24 hours a day while the other pump was in operation from 4 to 8 hours on those days.

The West Plant can supply 402 gallons per minute and it has a storage capacity of 200,000 gallons. The East Plant can supply 866 gallons per minute and has an elevated storage tank of 100,000 gallons capacity. Then we can assume that the peak load in mid-summer of 1940 was 1,268 gallons per minute. In order to fight a disastrous fire at that time the water used in fighting the fire would have to, come from the storage tanks. If we were to depend on four fire streams at 250 gallons per minute, or six fire streams at 165 gallons per minute we would be drawing at a rate of 1,000 gallons per minute for fire flow. This seems to be a reasonable amount to expect for fire fighting. The toatal demand would then be 2,268 gallons per minute which is the basis for 100% flow in the mains.

TABLE 6. Various percents of total flow stated in gallons per minute

Percent	Gallons per minute
100	2,268
50	1,134
40	907

TABLE 6. (continued)

30	681
25	567
22	500
20	454
15	340
14	318
13	295
12	272
11	250
10	227
9	204
8	181
7	159
6	136
5	113
4	91
3	68
2	45
1	23
0.6	14
0.5	11
0.4	9
0.3	7
0.2	4.5
0.1	2.3

The procedure of the problem was to make a complete series of corrections over the entire system from the assumed flows. These corrections were made on the drawings, and then another compete series of corrections was made from the results of the first correction. After the second correction it was noted that the main circuits in the system were in error more than the side branches and smaller lines. The only way to correct this is by special computation and transfer from one treatment plant circuit to the other treatment plant circuit. The third correction was made on only portions of the distribution system. Five different paths of flow were taken from each plant to the location of the assumed fire, and corrections were made in only those circuits necessary. The fourth correction was made on only these ten paths of flow. The results of the fourth correction are used as final results of the problem. It is true that another complete correction series over the entire system would make some changes, but another correction would probably be so small that there would be only a little change in the results of the fourth series of corrections. These results are accurate enough to show the needs of the city.

TABLE 7. Values of the 0.85 powers of numbers.

Number	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
0	0	0.14	0.25	0.36	0.46	0.56	0.65	0.74	0.83	0.92
1	1.00	1.09	1.17	1.25	1.33	1.41	1.49	1.57	1.65	1.72
2	1.80	1.88	1.96	2.03	2.10	2.18	2.25	2.33	2.40	2.47
3	2.54	2.61	2.69	2.76	2.83	2.90	2.97	3.04	3.10	3.18
4	3.25	3.32	3.39	3.45	3.52	3.60	3.66	3.72	3.79	3.86
5,	3.93	4.00	4.06	4.12	4.18	4.25	4.32	4.39	4.45	4.52
6	4.59	4.65	4.70	4.76	4.82	4.90	4.98	5.04	5.10	5.16
7	5.22	5.30	5.36	5.42	5.49	5.55	5.61	5.67	5.75	5.80
8	5.85	5.91	5.97	6.02	6.10	6.17	6.22	6.28	6.35	6.41
9	6.45	6.52	6.59	6.66	6.71	6.78	6.84	6.90	6.96	7.02
10	7.08	7.14	7.20	7.25	7.30	7.38	7.45	7.50	7.56	7.65
11	7.70	7.75	7.80	7.84	7.90	7.95	8.00	8.07	8.14	8.20
12	8.25	8.32	8.38	8.44	8.52	8.60	8.65	8.70	8.75	8.80

Values above 12.9 were computed as needed or taken from Table 2.

CITY OF EAST LANSING

Diagram of the distribution system of the city showing only mains of 4-inch size and over. 124 outlets to aid in assumption of flow in all sections of the city.

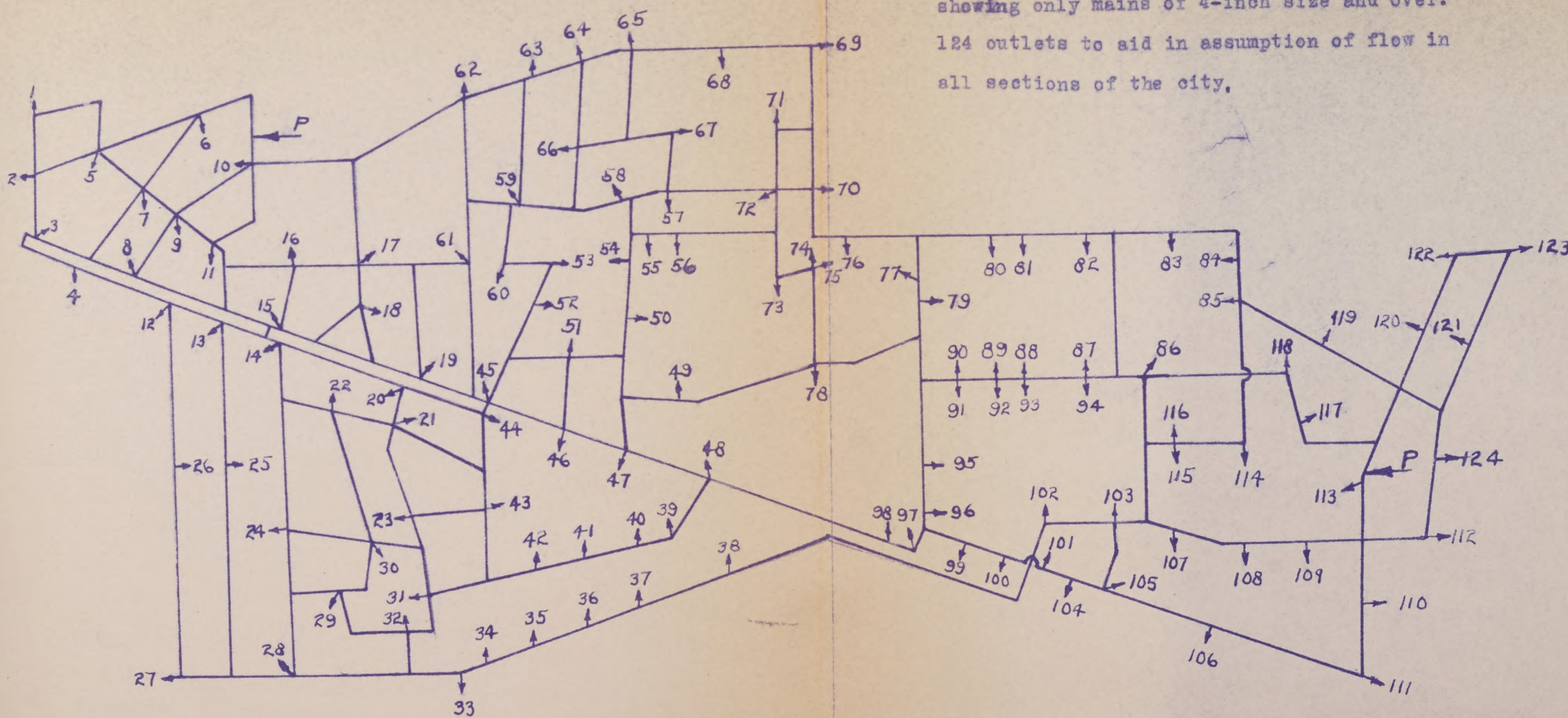


FIGURE 7.

CITY OF EAST LANSING

Diagram of distribution system after combination of first outlets into 33 major assumed outlets.

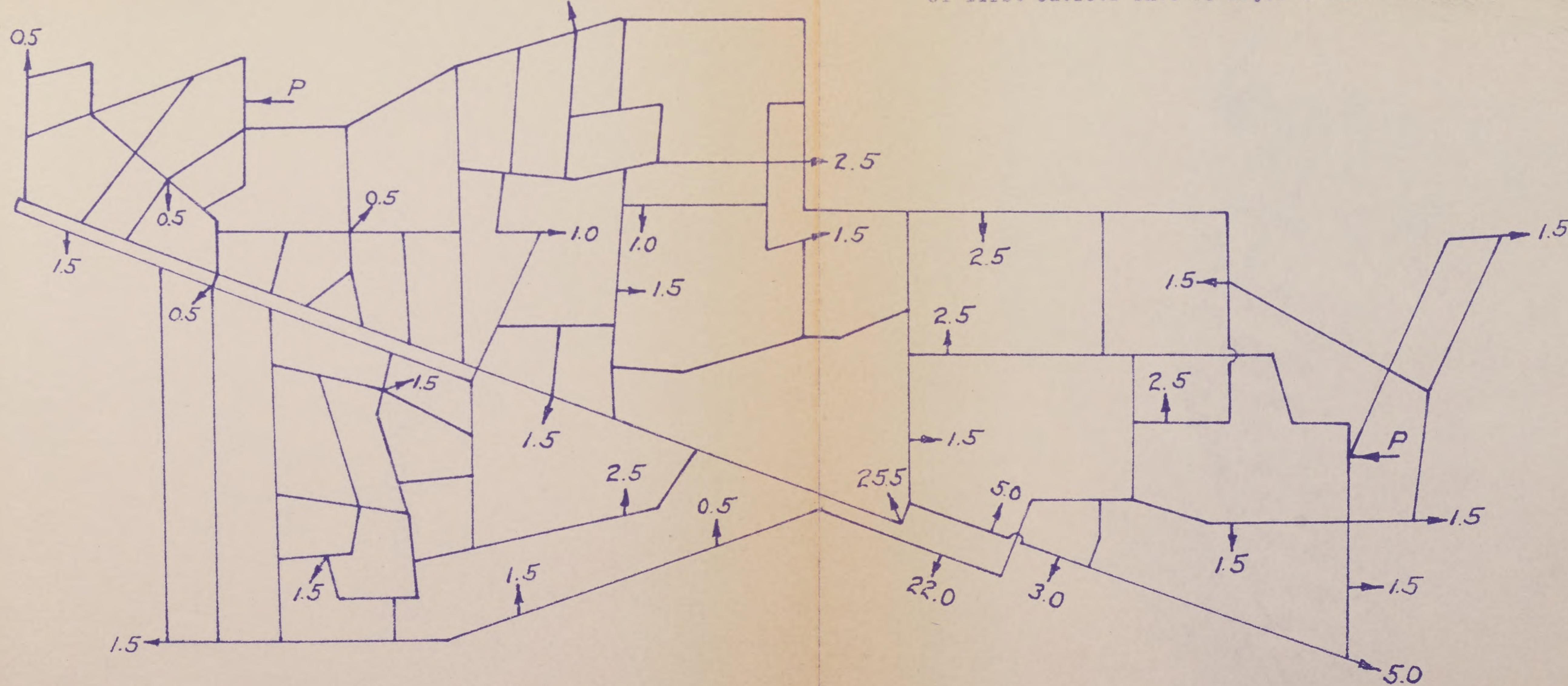
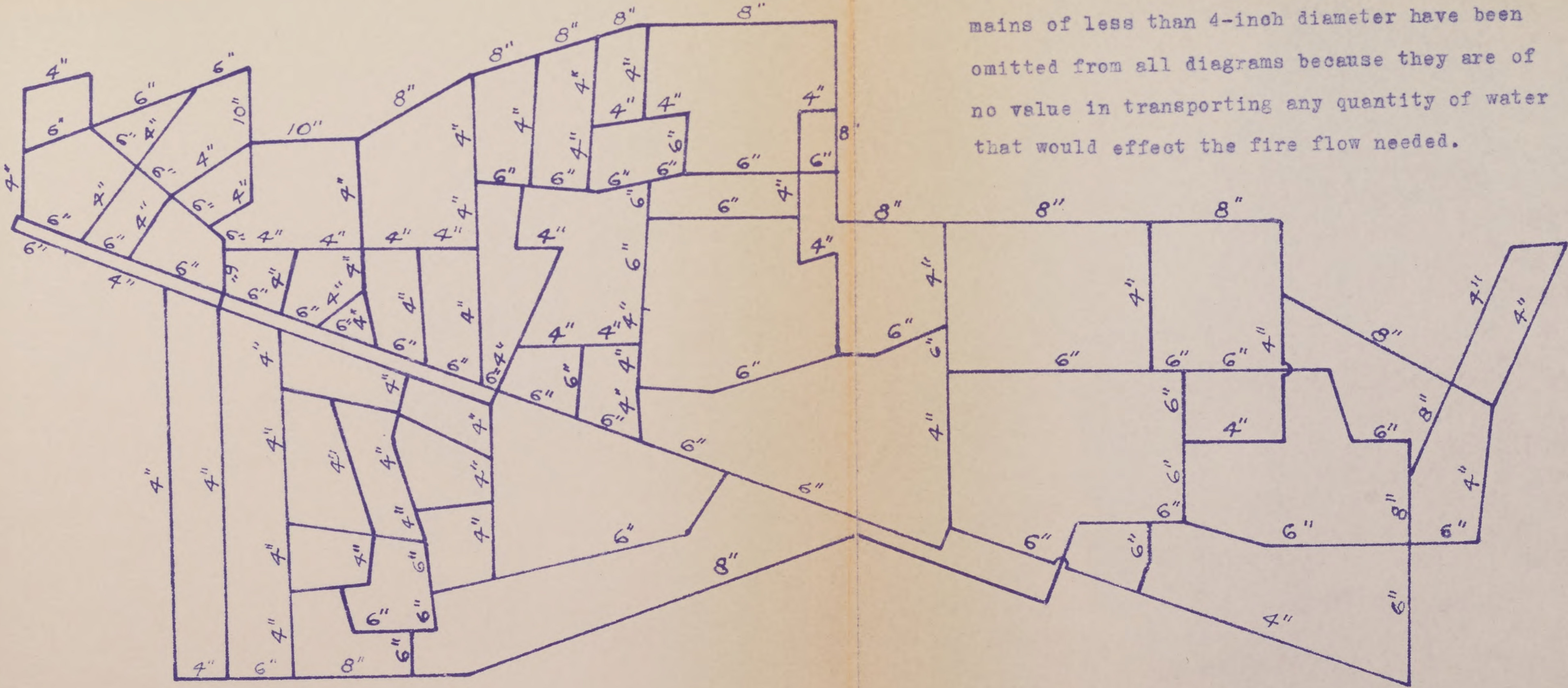


FIGURE 8.

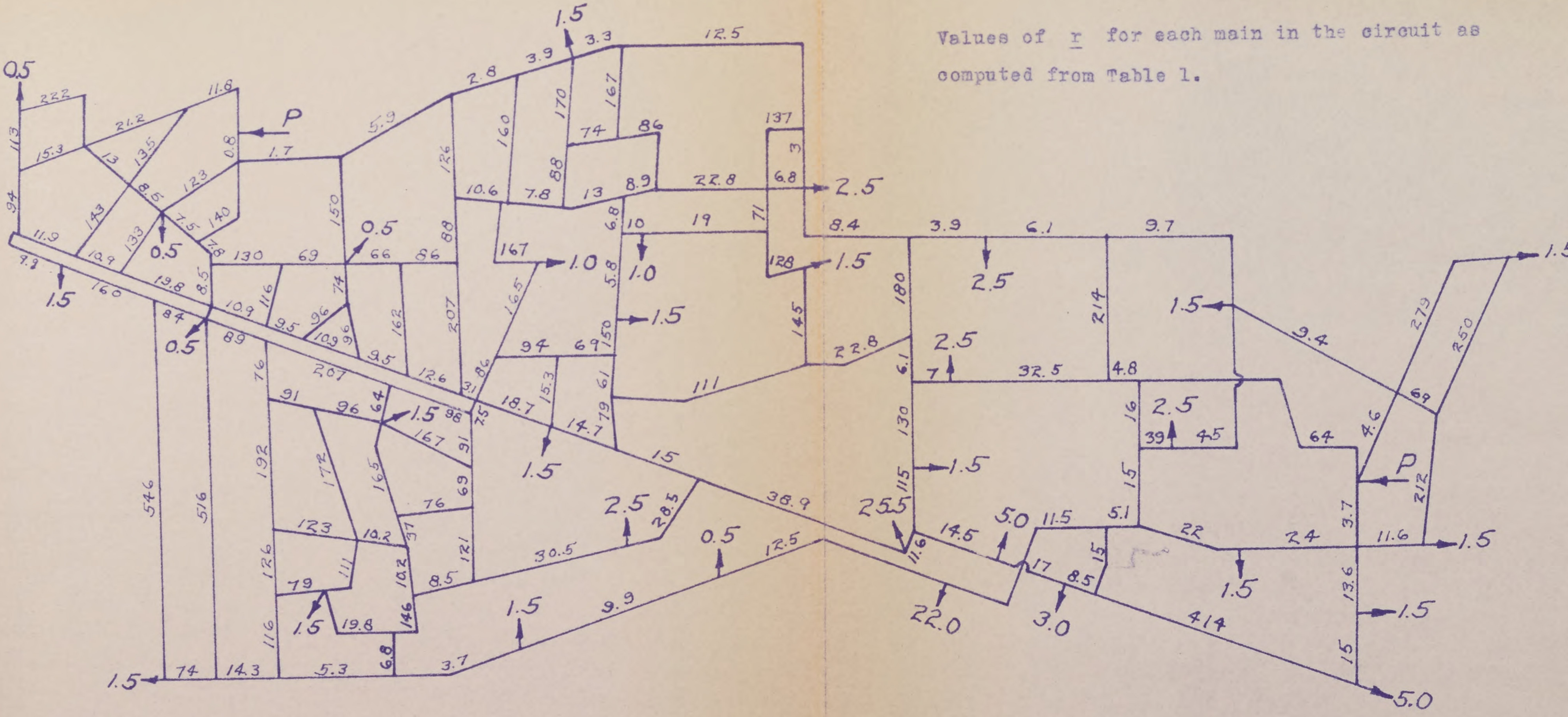
CITY OF EAST LANSING

Diagram of distribution system giving the sizes of all mains of 4-inch size or greater. All mains of less than 4-inch diameter have been omitted from all diagrams because they are of no value in transporting any quantity of water that would effect the fire flow needed.



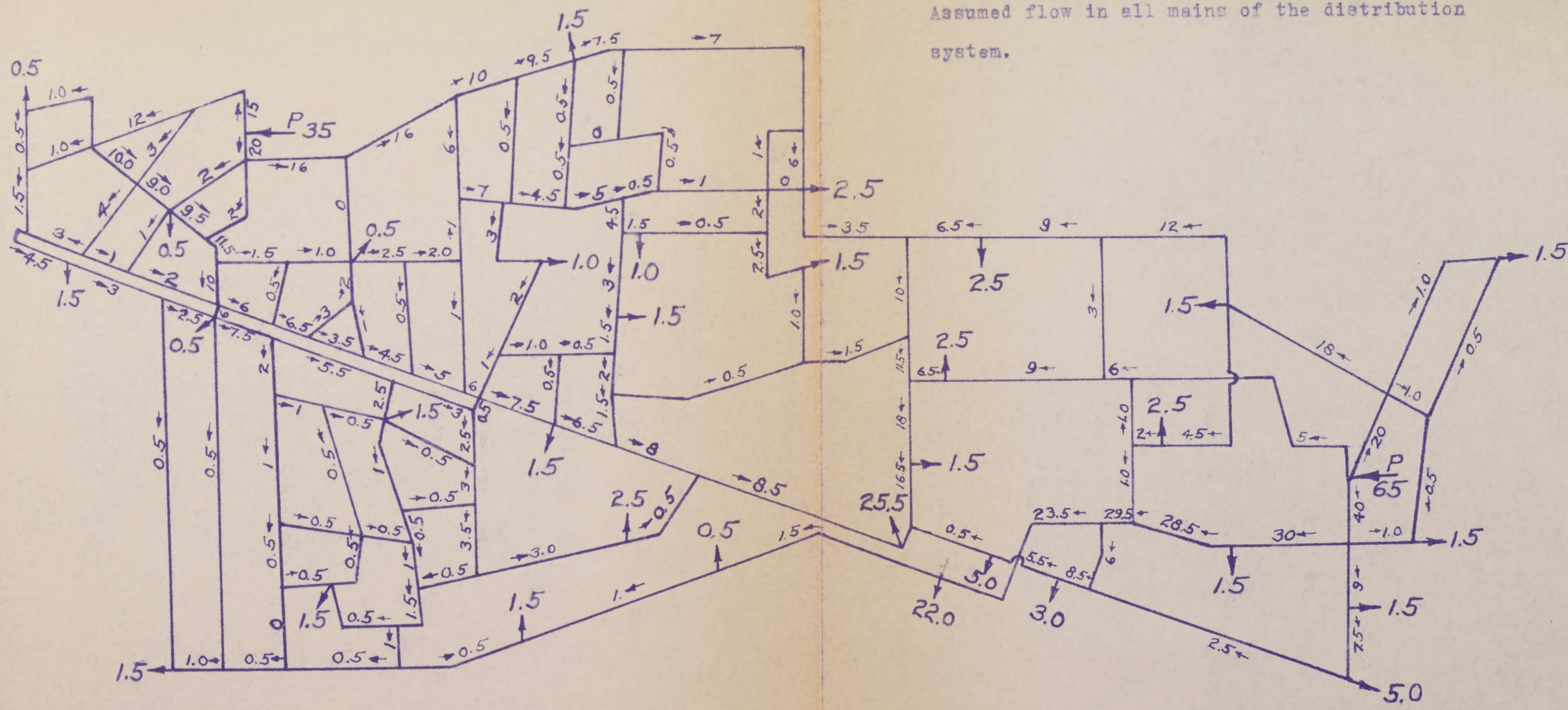
CITY OF EAST LANSING

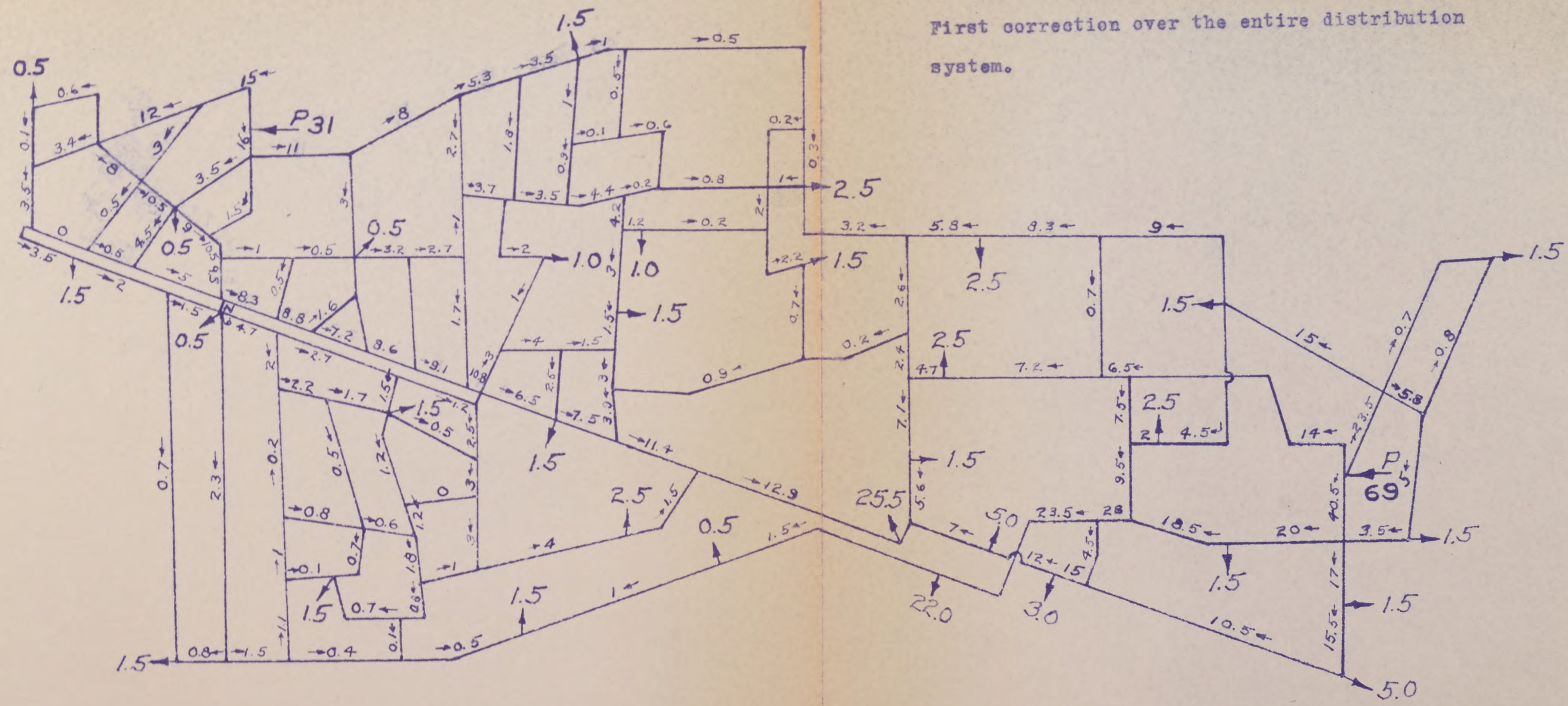
Values of r for each main in the circuit as computed from Table 1.



CITY OF EAST LANSING

Assumed flow in all mains of the distribution system.





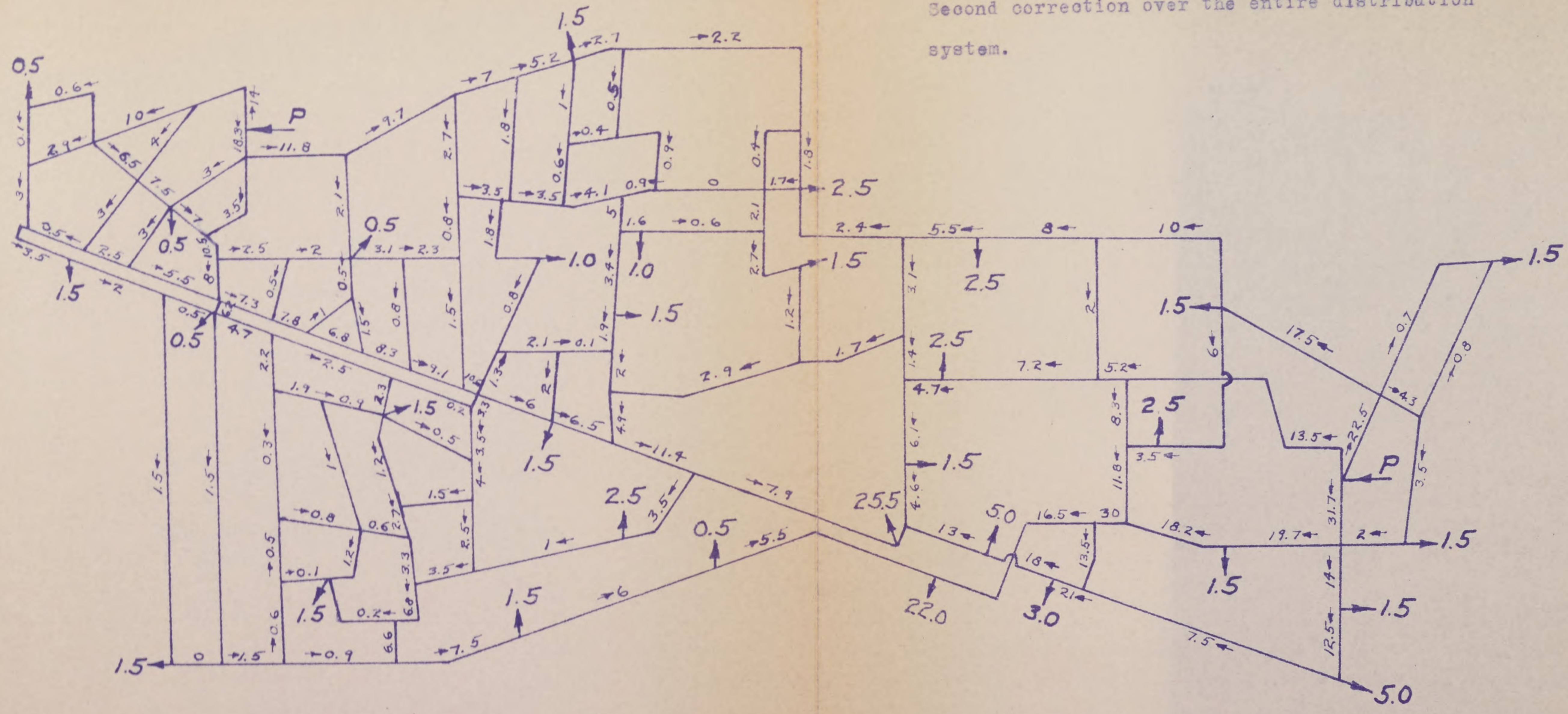
CITY OF EAST LANSING

First correction over the entire distribution system.

FIGURE 12.

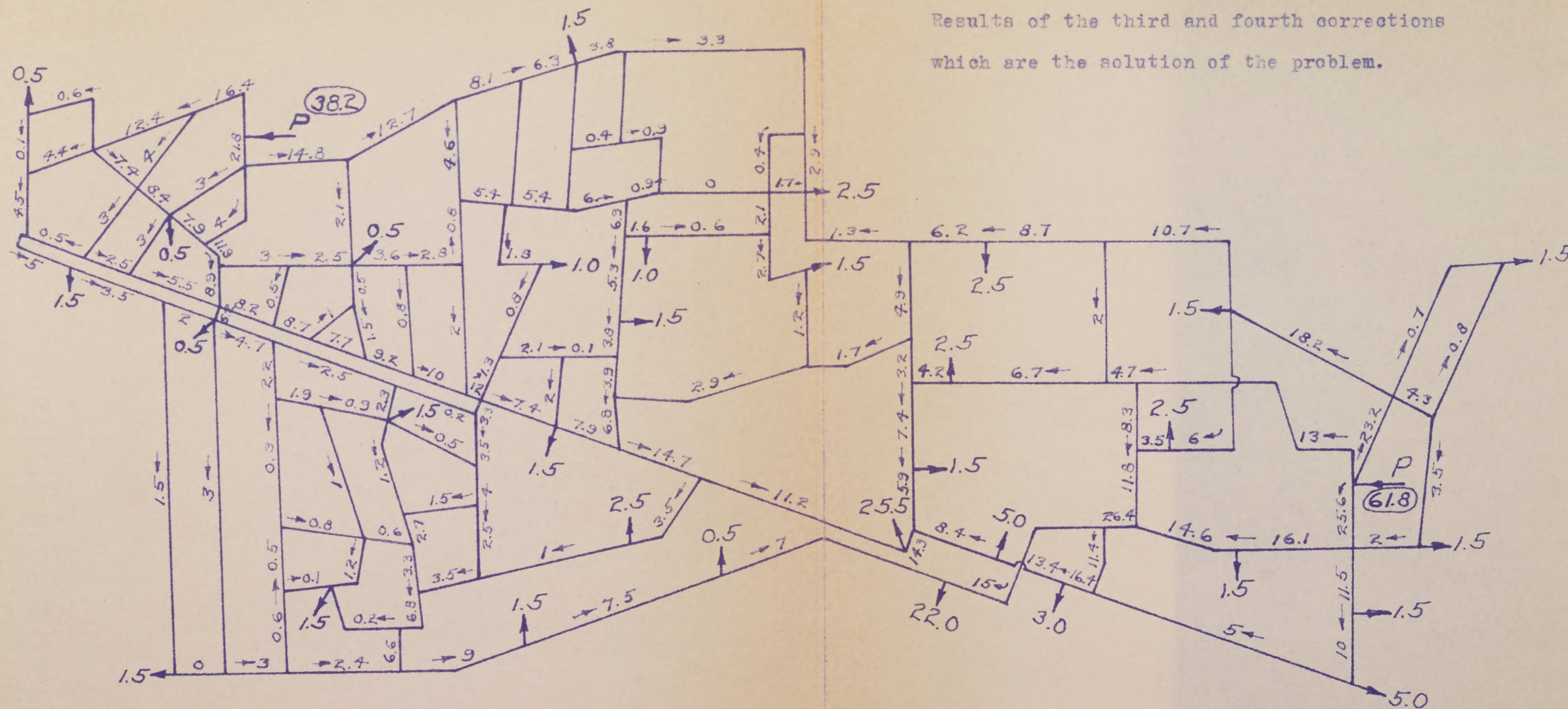
CITY OF EAST LANSING

Second correction over the entire distribution system.



CITY OF EAST LANSING

Results of the third and fourth corrections which are the solution of the problem.



RESULTS

In all reference to such a problem as this one there were no problems found in which the amount of water drawn from the treatment plant had to be assumed and corrected, In all others where more than one source of supply was used a definite amount of water was taken from each plant. Many difficulties arise in having to assume the amount of water flowing from each plant, and make corrections by the method used in this problem. A change of only one percent of the flow from one plant to the other may easily effect the distribution in the entire system. These difficulties have been overcome satisfactorily, and therefore the final results can be given with justifyable accuracy.

The head losses in the two main paths of flow from the plants to the fire are corrected in the following computations. The relative head losses are first found as in all computations, and then the method of transfer from relative head loss to actual head loss is calculated as in the example problem. Then the actual head loss is determined.

Line from East Plant

r		$Q^{0.85}$		$rQ^{0.85}$		Q		h
3.7	x	15.8	=	57	x	25.6	=	1495
24	x	10.7		257	x	16.1		4140
22	x	9.75		214	x	14.6		3130
5.1	x	16.1		82	x	26.4		2170
15	x	7.90		118	x	11.4		1350

8.5 x 10.8	92 x 16.4	1505	
17 x 9.05	154 x 13.4	2060	
14.5 x 6.10	88 x 8.4	743	
11.6 x 9.58	111 x 14.3	1590	18183

Line from West Plant

11.8 x 10.8 =	127 x 16.4 =	2090	
21.2 x 8.52	181 x 12.4	2240	
13. x 5.49	71 x 7.4	528	
8.5 x 6.10	52 x 8.4	435	
7.5 x 5.80	44 x 7.9	344	
7.8 x 8.20	64 x 11.9	760	
8.5 x 6.41	54 x 8.9	485	
10.9 x 5.97	65 x 8.2	533	
9.5 x 6.28	60 x 8.7	520	
10.9 x 5.67	62 x 7.7	475	
9.5 x 6.59	63 x 9.2	575	
12.6 x 7.08	89 x 10	890	
3.1 x 8.25	26 x 12	307	
18.7 x 5.49	103 x 7.4	758	
14.7 x 5.80	85 x 7.9	672	
15 x 9.80	147 x 14.7	2160	
38.9 x 7.80	304 x 11.2	3400	17172
	2771		1011

$$\frac{1011}{1.85 \times 2771} = 0.197$$

No Correction

This correction is less than two tenths of one percent so no correction will be made. The relative head loss will be taken as an average of the two totals at 17,678. The following computations serve to determine the actual head loss, and to check the ratio between the actual and relative head loss.

[illegible] C = 110 upon which [illegible]

Q(gpm)	Q(cfs)	Diam.	h/1000ft	L feet	Actual h	Relative h	Relative h / Actual h
372	.827	10"	[illegible]	270			
372	.827	6"	13.6	360	4.90	2090	428
281	.625	6"	8.0	620	4.96	2240	452
168	.373	6"	3.0	380	1.14	520	463
190	.422	6"	4.0	250	1.00	435	435
179	.398	6"	3.5	220	0.77	344	447
270	.600	6"	7.5	230	1.73	760	439
201	.447	6"	4.2	250	1.05	485	462
185	.411	6"	3.9	320	1.25	533	426
197	.437	6"	4.1	280	1.15	520	452
175	.389	6"	3.3	320	1.06	475	448
209	.465	6"	4.4	280	1.23	575	468
227	.505	6"	5.3	370	1.96	890	454
272	.605	6"	7.5	90	0.67	307	458
168	.373	6"	3.0	550	1.65	758	460
179	.398	6"	3.5	430	1.51	672	445
334	.742	6"	11.0	440	4.84	2160	447
255	.567	6"	6.5	1140	7.40	3400	460
					38.27		

581	1.29	8"	7.0	440	3.06	1495	486
365	.812	6"	13.5	700	9.45	4140	437
332	.738	6"	12.0	650	7.80	3130	402
599	1.33	6"	32.0	150	4.60	2170	452
259	.575	6"	6.5	440	2.86	1350	472
372	.827	6"	13.6	250	3.40	1505	443
254	.564	6"	6.4	500	3.20	2060	644
190	.422	6"	4.0	480	1.92	743	387
325	.722	6"	11.9	340	4.04	1590	394
					40.55		11761

$$\frac{11761}{26} = 452$$

We find the actual head loss in the circuits to be 39.41 feet or 17.1 pounds pressure loss. The pressure at the pumping plants is usually about 50 pounds per square inch. There will be some change in head due to difference in elevation of the parts of the city, but the land is so near level that we will assume the elevation of all points to be the same. Then the pressure at the hydrants nearest the fire is 32.9 pounds per square inch. This seems to be an extremely great loss of pressure, but it must be kept in mind that this problem has been computed on the basis of maximum flow that could be expected at any time. Under ordinary conditions the flow would not be greater than one-fourth the maximum flow required for domestic use and fire fighting purposes. With the normal flow of only one-fourth the maximum possible flow, the loss of head between the plant and the downtown district, or probably the most distant outlet in the city, would not be expected to exceed two pounds per square inch.

At this point we can now determine the loss of head in any main in the city from the information we have available. The constant ratio between the relative head loss and the actual head loss in this problem is 438 to 1 as will be noted from the previous computation.

For example if we wish to find the loss of head in the length of 8-inch pipe on Beach Street running from Orchard Street to Bailey Street, From Figure 13, of final corrections we find that 18.2 percent of the total flow is passing through that main. The loss of head in any main can be expressed in an equation for either loss in feet of head, or loss in pounds as follows;

Head loss in feet

$$h = \frac{r\ Q^{1.85}}{\text{Problem Constant}}$$

Head loss in pounds

$$h = \frac{r\ Q^{1.85}}{2.31 \times \text{Problem Constant}}$$

In these equations, r is the value of resistance coefficient for the pipe as shown in Figure 9, Q is the quantity of water flowing in the main expressed in percent of the total flow, and the problem constant of this problem was determined at 452.

Loss in length of pipe on Beech Street running from Orchard Street to Bailey Street.

$$h = \frac{r\ Q^{1.85}}{452}$$

$$h = \frac{9.4 \times 18.2^{1.85}}{452}$$

$$h = \frac{9.4 \times 214}{452}$$

$h = 4.48$ feet loss

$$h = \frac{4.48}{2.31}$$

$h = 1.94$ pounds per square inch loss

This again checks with computation of the head loss from the friction loss diagram shown on page 266 in "Hydraulics" by Schoder and Dawson.

It has here been shown that the East Lansing water distribution system is inadequate to needs of maximum flow, but no mention has been made as to the availability of the water. If the two plants are pumping at maximum capacity as they were in the later part of July and the first part of August in 1940 in order to supply the domestic need, all flow for fire fighting at that time would have to come from the elevated storage tanks. Assuming the fire flow of 1,000 gallons per minute and the maximum capacity of the storage tanks at 300,000 gallons, we could obtain flow from them for a total of three hundred minutes of flow or equal to five hours. This would be considered to be a short period of time in the case of a conflagration. During this time a very serious loss of pressure would occur at the pumping plants, thus causing a more serious shortage of water. At the dry season in 1940 some drop of pressure was noted.

The question may arise as to wether the assumed fire flow of 1,000 gallons per minute is to large to expect for a city of this size. This and other questions concerning fire flow are best answered in quotations by the National Board of Fire Underwriters.

What constitutes a good fire stream? * " Some years ago this was defined as one of 250 gallons a minute discharge. The basis of this was the experience with hose lines in the mills of New England, where normal pressure on the systems ranged from 60 to 100 pounds, and streams were taken direct from hydrants. For inside lines to be handled by one or two men this size of stream is about correct, and for the usual type of residential or small store occupancy it is reasonable to base requirements on the number of 250-gallon streams which may be needed. Actually, present and future practice will be to use more and smaller streams in fire fighting in this class of occupancy. Effective use both on

* National Board of Fire Underwriters, Bulletin No. 116 January 15, 1941.

the fire and to protect exposures, is being made with $1\frac{1}{2}$-inch hose lines wyed from $2\frac{1}{2}$-inch feeders.

The introduction of the automobile pumper in fire service has brought about a material increase in fire demand upon water works adequacy. A good $1\frac{1}{4}$-inch stream, such as would be used as an outside line where a large building was involved, runs from 310 gallons at a 45-pound nozzle pressure, which gives a reach of about 70 feet as a solid stream, to 350 gallons when the nozzle pressure is 60 pounds and the reach is about 80 feet. The automobile pumping engine, thanks to the aid of the National Board of Fire Underwriters to the manufacturers, is capable of pumping its full capacity continuously and at a good pressure, therefore it is not unreasonable for the fire department to expect the water system to have an adequacy such that the full use can be made of these larger streams.

There is today a growing appreciation of the value and use of streams too strong to be handled by hand. Fixed and portable turret or monitor nozzles are becoming standard equipment in even small departments. The English are finding in their almost daily contending with conflagrations caused by bombing, that ladder pipes are one of the most effective means of preventing the spread of fire from one building to another. These various devices for powerful streams add further to the demands of adequacy. They provide more ready means for the use of the full capacity of the pumping engines in the fire department. Through the use of 3-inch hose and of siamesed lines of $2\frac{1}{2}$-inch hose, $1\frac{1}{2}$-inch and larger nozzles can be used effectively. These give discharges ranging from 500 gallons to over 1,000 gallons a minute.

Even where great congestion of buildings is not present it appears reasonable to expect at least two fixed powerful streams to be used on a fire which is threatening to spread to other buildings. These would be in addition to eight or ten hand lines. Using as an average, 600 gallons from the fixed nozzles and 300 from the hand lines we get 3,000 to 3,600 gallons a minute as a minimum requirement.

The National Board of Fire Underwriters, in its Grading Schedule, has set up requirements for adequacy of the water system based upon average conditions found in communities of various sizes. These are as follows:

Population	Required Fire Flow, Gallons per Minute for Average City
1,000	1,000
2,000	1,500
4,000	2,000
6,000	2,500
10,000	3,000
13,000	3,500
17,000	4,000
22,000	4,500

28,000	5,000
40,000	6,000
60,000	7,000
80,000	8,000
100,000	9,000
125,000	10,000
150,000	11,000
200,000	12,000

Over 200,000 population 12,000 gallons a minute with 2,000 to 8,000 gallons additional for a second fire.

Some of the quantities given above may appear rather large, especially where the fire department has not been added to as the community grew. In this connection it is worthwhile to consider the other feature of the modern motor pumper, which is, its ability to cover long distances at a good rate of speed.

Water supply systems must be designed on the basis of possible outside aid. In all of the recent large fires, including those in cities as big as Chicago, aid has been used, either at the fire or to care for other fires,

It is of importance for every fire department to know the ultimate capacity of its water system. This is particularly true of the smaller places. Water is only effective when it is thrown on the actual burning material and to do this the stream must be of good volume and discharged with sufficient velocity to give it a good reach. Fires have been known to get away where a limited supply of water was wasted through the use of too many streams, none of which were effective, rather than a smaller number properly located and used. Where the pumps in the water system are known to have a total capacity of 2,000 gallons a minute, not more than eight $2\frac{1}{2}$-inch lines should be used, and probably more effective use would be obtained with six lines using $1\frac{3}{4}$-inch nozzles than the larger number with smaller nozzle tips, but certainly these six would be better than 10 or 15 streams with $1\frac{1}{4}$-inch tips where the discharge from each was 200 gallons or less, with an effective reach of less than 35 to 40 feet.

Adequacy is not just a question of the total supply of water which will be delivered to the district under consideration, but also whether there are enough hydrants to permit the water to be put on the fire without excessively long hose lines. A study made by the National Board of Fire Underwriters has indicated that the hydrants should be spaced upon an area-served basis rather than linear spacing of hydrants.

The adequacy is also controlled somewhat by the size of the distribution main. The average fire department pumper has a capacity of 750 gallons. This flow through a 4-inch pipe produces a friction loss of about 20 pounds in a hundred feet. Even if the 4-inch pipe is fed from both ends, the friction loss is 5 pounds per 100 feet and as many blocks are 400 to 600 feet long a hydrant in the middle of the block will not recieve an adequate supply if pressures are reduced generally, as they will be in a big fire, below 20 to 30 pounds. This is because there is also friction loss in the hydrant branch, the hydrant and the suction hose. Hydrants installed twenty years or more ago were in many cases of poor design and had a high friction loss which seriously restricted delivery. A replacement of small pipe and hydrants in high value areas should be one of the objects of the fire department.

The figures decided upon for the area-served basis are as follows:

Fire Flow Required Gallons per Minute	Average Area Per Hydrant, Sq. Ft. Engine Streams	Direct Hydrant Streams
1,000	120,000	100,000
2,000	110,000	85,000
3,000	100,000	70,000
4,000	90,000	55,000
5,000	85,000	40,000
6,000	80,000	40,000
7,000	70,000	40,000
8,000	60,000	40,000
9,000	55,000	40,000
10,000	48,000	40,000
12,000	40,000	40,000

Reliability of water supply is of vital importance in connection with fire fighting. Great improvement has been made in water works practices in the past generation, but equipment, such as pumps, still has to be overhauled and repaired, and water mains break under certain conditions. Instances are on record of serious interruption of water s supply at times when the fire department was fighting a fire. In addition it must be remembered that any interruption of supply introduces a probability that even a small fire may not be controlled and may spread to conflagration proportions, as was the case when the earthquake put the water supply of San Francisco out of commission.

The installation of seperate fire main systems is seldom justified in cities not having them. The considerable amount of money required for installing and operating such systems can be used to better purpose in providing better adequacy and reliability of existing water supply systems."

ADVISED CHANGES OR ADDITIONS TO THE EAST LANSING WATER SUPPLY IN ORDER OF IMPORTANCE

Drilling of at least one new well, probably at the West Treatment Plant. An accute shortage of pumping facilities may occur this summer, or is almost certain within the next two or three years.

Replace 2-inch main by a 6-inch main on Gunsón Street from Ann Street to the alley running parallel to Grand River Avenue just north of that street, and thence laying a 6-inch line from that point across Grand River Avenue to connect onto the present 4-inch line on River Street. This was advised as far back as 1925 and still has not been done. By making such change three serious dead-end points would be connected to make continuous flow circuits, and would aid service greatly.

Repalcement of all 2-inch mains by 4-inch mains throughout the city, with consideration of the length of the main and the number of years it has been in service. That is to say, replacement of the longer lines in the original distribution system should be made first. Most of these lines were advised to be changed as early as 1925 and have since that time been in use for sixteen years. Proof of the need of this change can easily be obtained. Pick for example the 2-inch main running through the alley between M. A. C. Avenue and Grove Street from Elizabeth to Ann Street. Bordering on this main are 24 lots which are almost all built upon. If each house used as little as 4 gallons per minute this would be a total of 96 gallons per minute for 24 houses or 0.214 cubic feet per second. With this amount of flow the head loss is 230,feet per 1,000 feet of length. The length of that main is 850 feet so the actual loss of head would be 196 feet or 89 pounds if all the flow were coming in one end and out the other. If we assume the main as two parts with half of the flow coming in at each end we have only 425 feet of length and a quantity of 0.107 cubic feet per second. This gives a head loss of 12 pounds in passing from entrance to the center of the distance. That loss of pressure is even to great to be called a servicable main.

Another new well will probably be needed in about five years so it would be advisable to plan on that in enlarging the treatment plant to handle such capacity.

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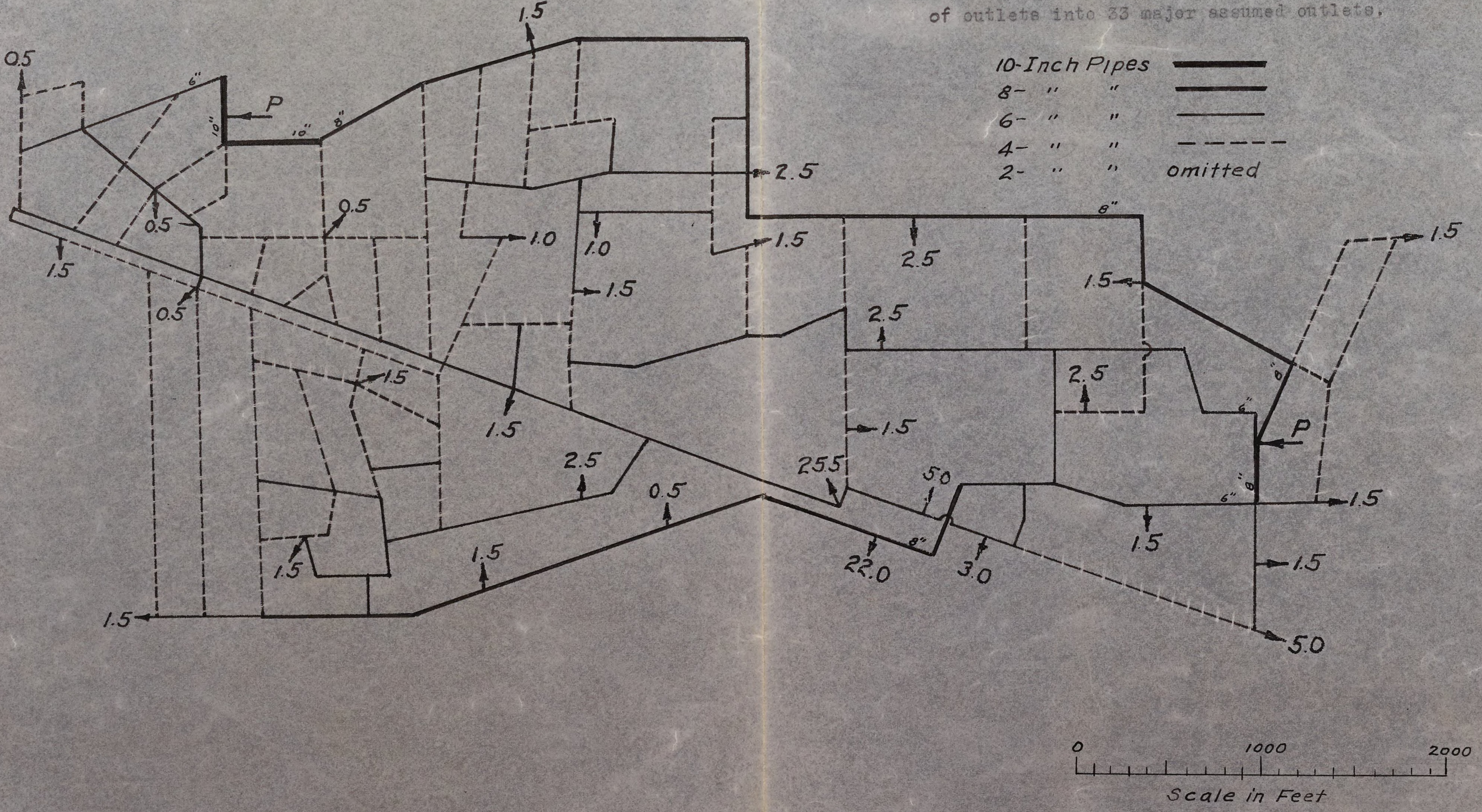
CITY OF EAST LANSING

Diagram of distribution system after combination of outlets into 33 major assumed outlets.



CITY OF EAST LANSING

Diagram of the distribution system of the city showing only mains of 4-inch size and over. 124 outlets to aid in assumption of flow in all sections of the city.

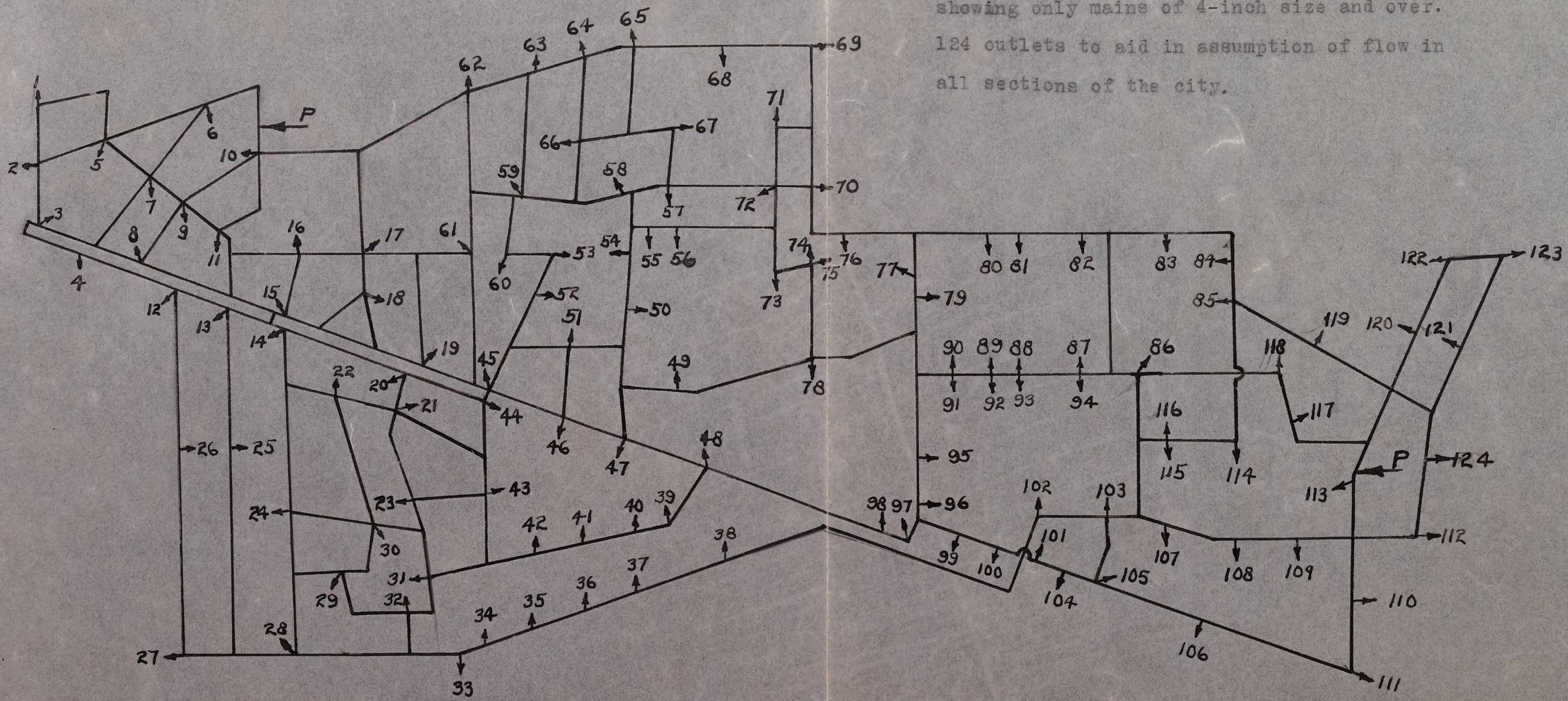
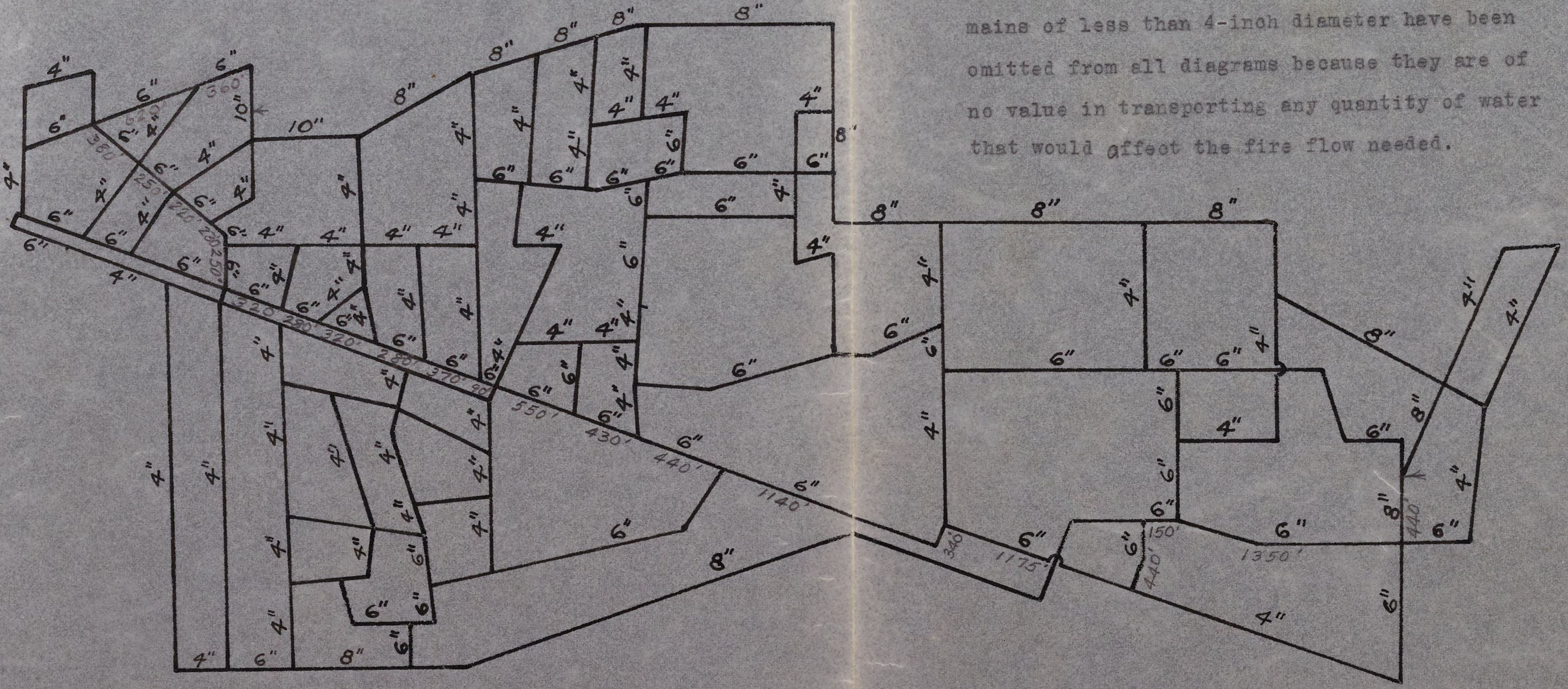


FIGURE 8.

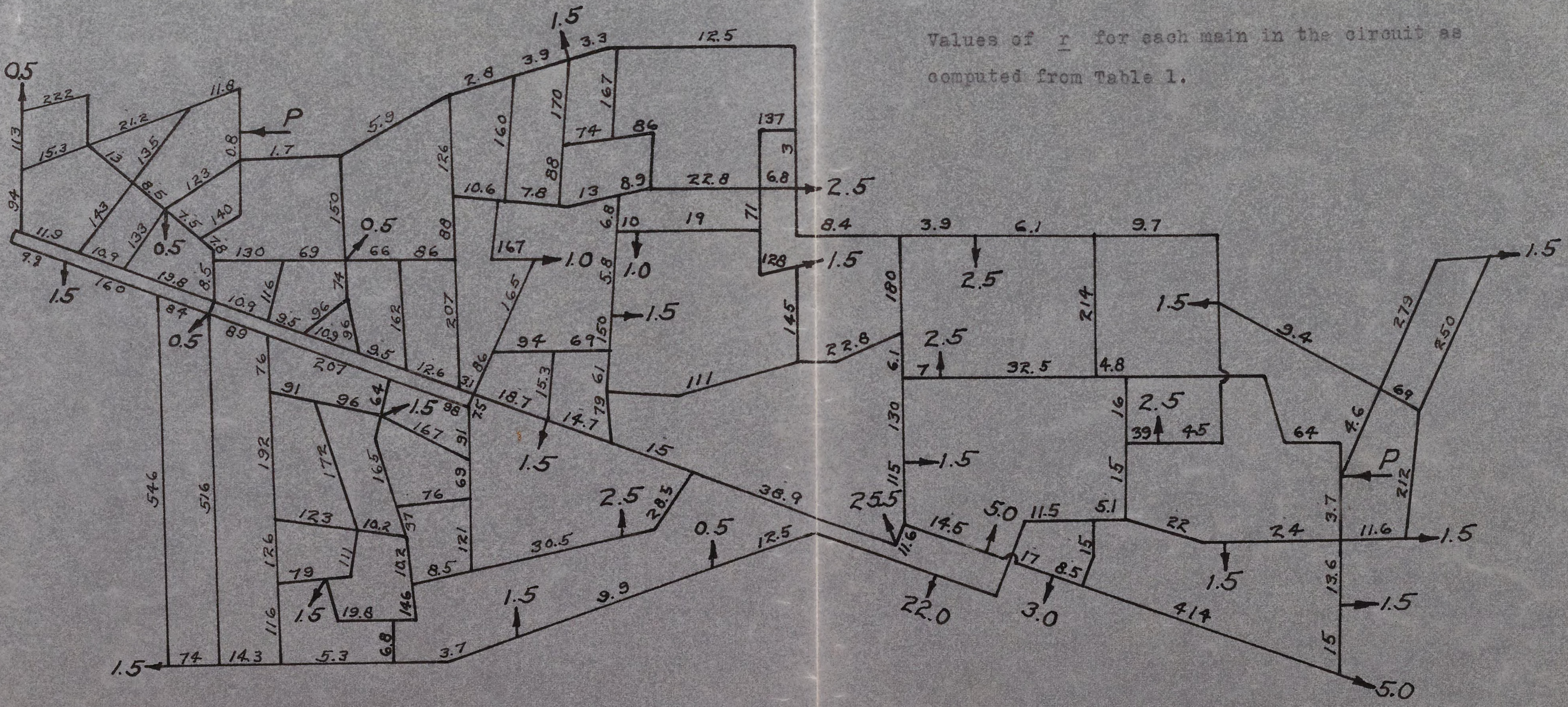
CITY OF EAST LANSING

Diagram of distribution system giving the sizes of all mains of 4-inch size or greater. All mains of less than 4-inch diameter have been omitted from all diagrams because they are of no value in transporting any quantity of water that would affect the fire flow needed.



CITY OF EAST LANSING

Values of r for each main in the circuit as computed from Table 1.



CITY OF EAST LANSING

Assumed flow in all mains of the distribution system.

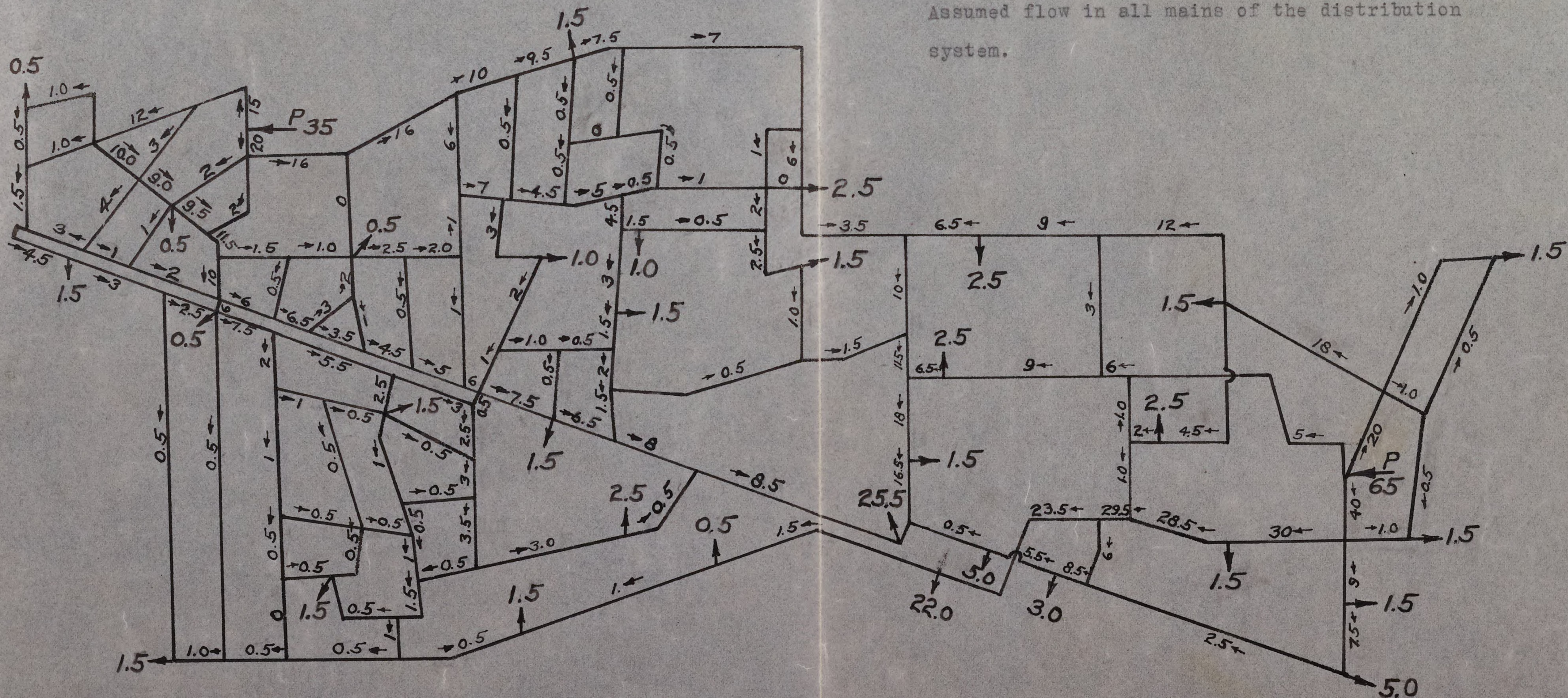
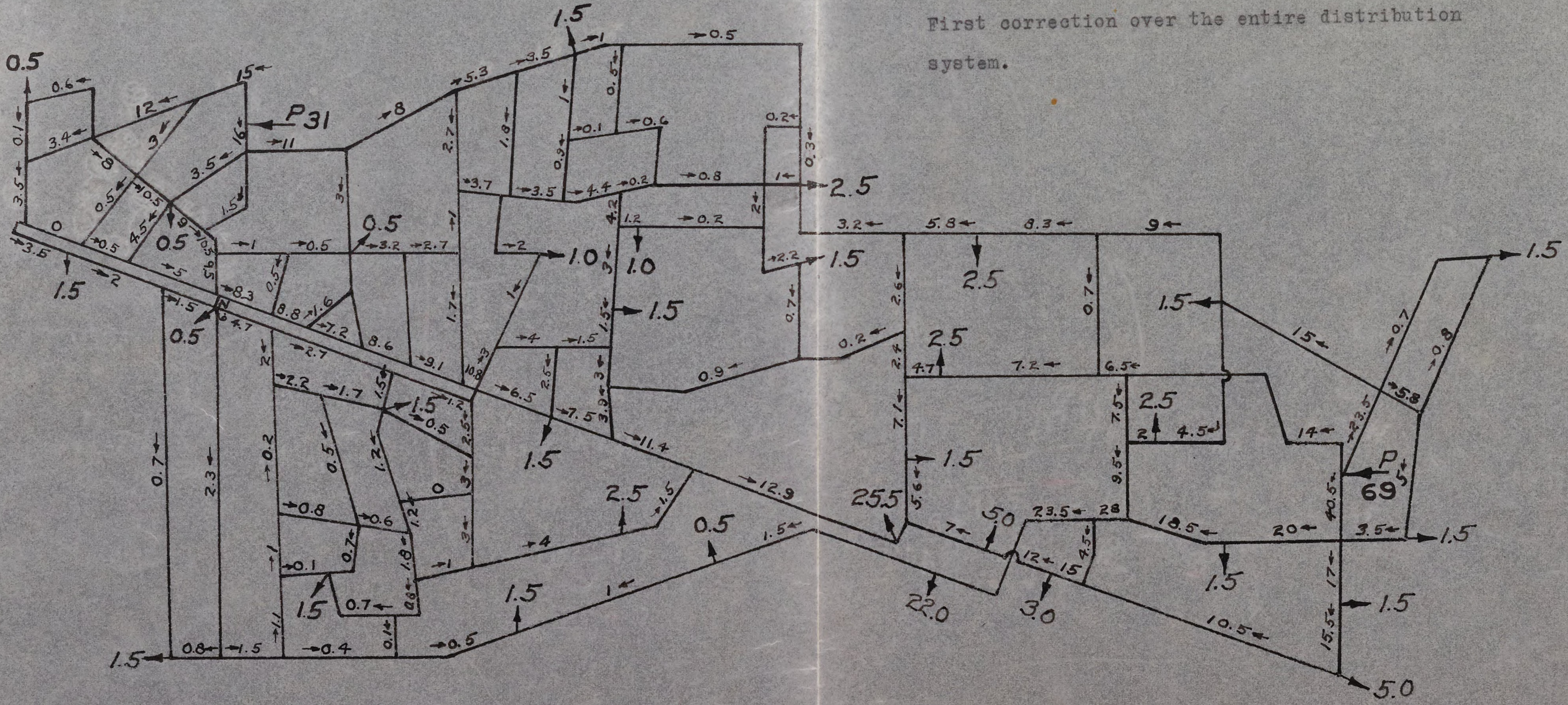


FIGURE 11.

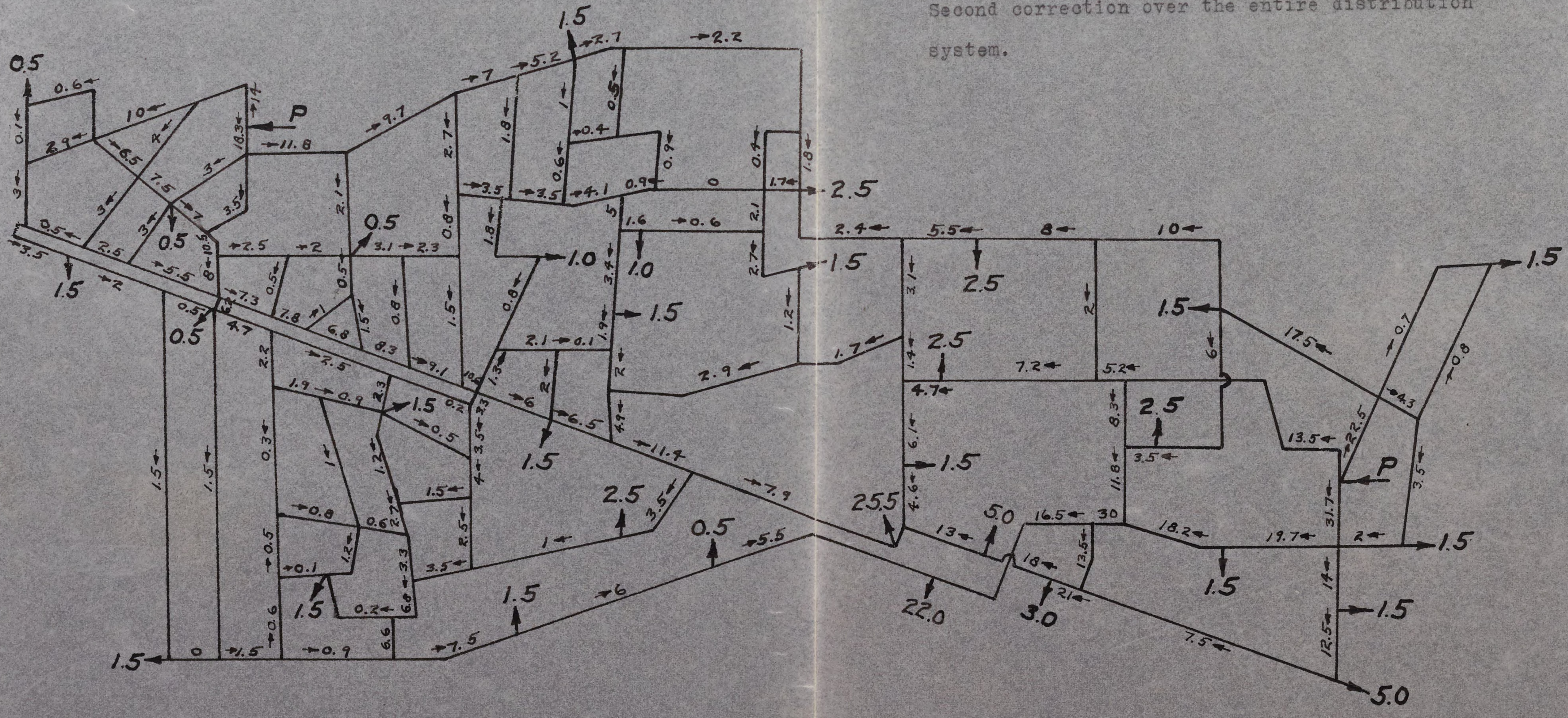
CITY OF EAST LANSING

First correction over the entire distribution system.



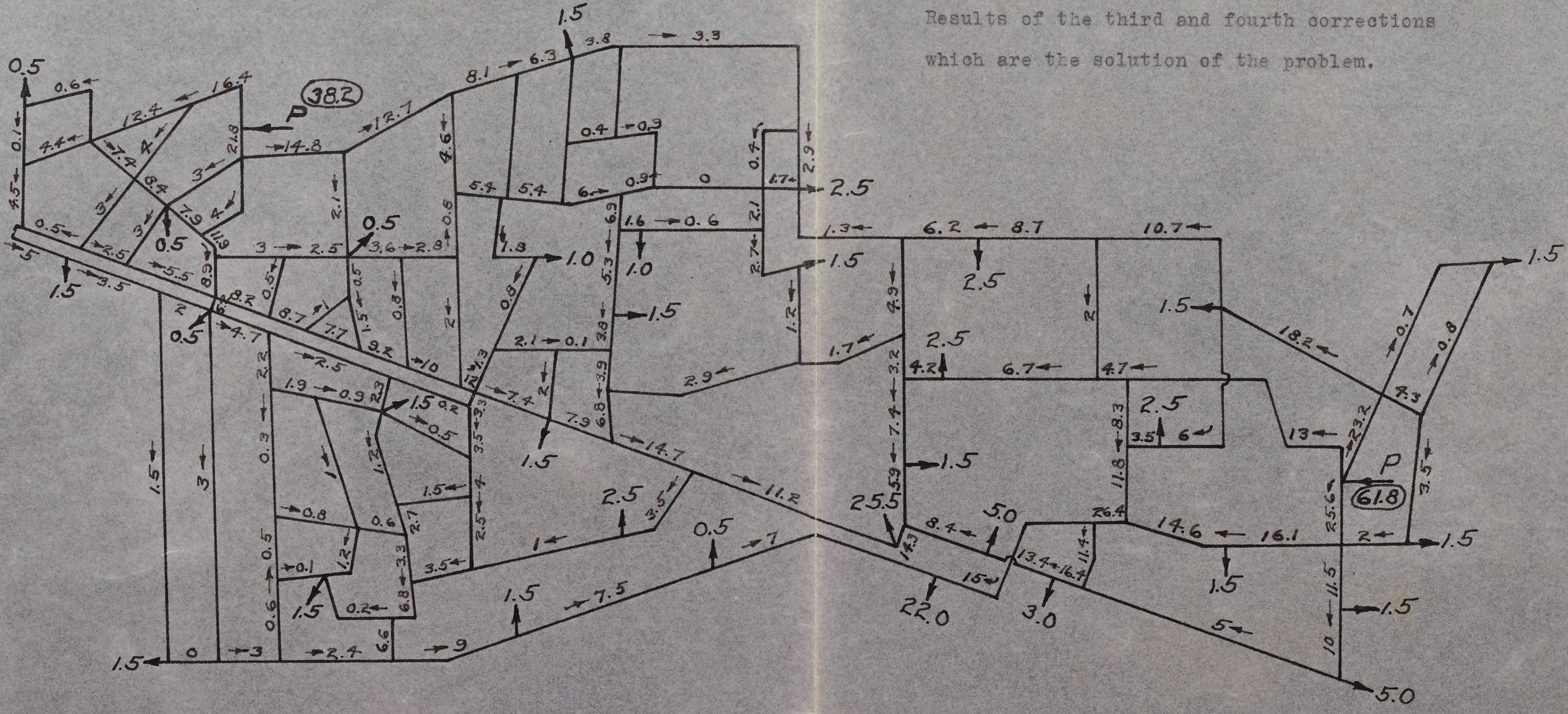
CITY OF EAST LANSING

Second correction over the entire distribution system.



CITY OF EAST LANSING

Results of the third and fourth corrections which are the solution of the problem.



CITY OF

Diagram of distrib

of outlets i