REPORT ON MEASURES FOR

ELIMINATION OF POLLUTION OF ROCK CREEK

AND ITS TRIBUTARIES IN WASHINGTON

WASHINGTON, D. C.

1935

Rock Creek Above Connecticut Avenue.

UNITED STATES DEPARTMENT OF THE INTERIOR
HAROLD L. ICKES, *Secretary*

NATIONAL PARK SERVICE
ARNO B. CAMMERER, *Director*

REPORT ON MEASURES FOR ELIMINATION OF POLLUTION OF ROCK CREEK AND ITS TRIBUTARIES IN WASHINGTON

(FEDERAL EMERGENCY ADMINISTRATION OF PUBLIC WORKS)
(*Federal Project No. 607*)

Prepared for the
EASTERN DIVISION, BRANCH OF ENGINEERING
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INTRODUCTORY REVIEW OF THE REPORT AND SUMMARY OF FINDINGS AND RECOMMENDATIONS

This report includes (*a*) an investigation of the source and amount of pollution in Rock Creek, (*b*) studies to determine the best remedial measures, and (*c*) cost of recommended remedial works.

1. *Location.*—Rock Creek, from its source near Mount Zion, Md., to its junction with the Potomac in the District of Columbia, drains an area of 77.8 square miles. The natural wooded valley along the banks of Rock Creek is occupied by the Rock Creek National Park and the Zoological Garden. The adjoining area is built up with many attractive homes. The lower section in the city of Washington is densely populated.

2. *Stream pollution.*—About 183,000 people live in the Rock Creek basin. Of this number, 160,000 in the District of Columbia are served by combined sewers; that is, sewers which carry both house sewage and storm water run-off. During dry weather the sewage is normally carried away by relatively small intercepting sewers to the station on lower Anacostia River. Whenever rainfall occurs the intercepting sewers become overcharged. The combined sewers then discharge sewage directly into Rock Creek. The effluent flushed into the creek during light rains, or the early part of heavy rains, is stronger and more foul than the diluted sewage following heavy storms. (See Analysis, par. 6, ch. II.) This large combined sewer area is the chief source of the creek pollution. There are 28 sewer outfalls now discharging into Rock Creek at times of even light rainfall. This happens about 40 times during an average year (ch. V–C, par. 31).

The territory above the Klingle and Luzon areas, including the Washington suburban sanitary district, is served in general by the separate system of sewerage; that is to say, the house sewage alone is conveyed to the existing Rock Creek interceptor and unpolluted storm water is discharged directly into Rock Creek (fig. 1 and chs. II and III). This separate system is not now 100 percent perfect. The pollution in different section of Rock Creek is shown by the analysis made by the United States Public Health Service. The relative pollution in Rock Creek is indicated by the following:

	Average biochemical oxygen demand	*B. coli, maximum*
Sherrill Drive	2. 85	1, 000
Above Zoological Garden	3. 03	10, 000
M Street	4. 35	100, 000

(ch. II, par. 4).

3. *Complaints.*—Many complaints have been made by visitors in Rock Creek National Park and by adjacent residents regarding insanitary conditions and foul odors. Maintenance crews have been employed periodically, following sewage outflows, in an effort to mitigate the nuisance by clearing the stream banks and by flushing.

4. *Basic remedies.*—Your consulting engineers advise as follows: The procedure for the upper basin of Rock Creek in Maryland should be to continue and to extend the existing separate system. We commend the program proposed by the Washington suburban sanitary district. The development of the work of this sanitary district has not kept pace with the needs for abatement of pollution in Rock Creek. Nine hundred and sixty-eight houses are now connected to sewers discharging into Rock Creek. The estimated cost of proposed extension is about $200,000 (ch. II, par. 3).

When a construction program is initiated to remove pollution contributed by the District of Columbia then the Maryland authorities should be called upon to complete that part of their sanitary program which will remove their contribution to Rock Creek pollution.

(*a*) The areas of Klingle and Luzon Valley and the Army Medical Center are now partly served by the separate system. Completed separate systems should be installed. We estimate the cost for such completion at $550,000. We recommend that this amount be allocated to the District of Columbia for said purpose with the understanding that future extensions and adequate policing of sanitary requirements be maintained in all the separate sewer areas of Rock Creek Basin in the District of Columbia (chs. III and VI).

(*b*) The best means for removal of pollution contributed by the lower area of Rock Creek are not as readily determined as for the two upper areas.

This lower section of the work is the largest and most expensive. Many alternative procedures may be used. This has been the subject of a number of reports and plans (ch. IV). Your engineers have considered and in part utilized much of the valuable data and suggestions contained in these reports.

5. *Need for essential data.*—None of these reports, heretofore made, contain basic figures of run-off or flow frequency which govern the size of relief sewers or remedial works. Several of the reports point out the need for such figures. Without them the merits of the various plans cannot be compared either on the basis of cost, accomplishment of results, or even feasibility.

6. *Derivation of run-off rates.*—Your consulting engineers have devoted a large part of their work to the essential task of deriving basic figures of run-off and frequency. Fortunately, through the foresight of the National Park Service in cooperation with the United States Geological Survey, self-recording gages had been established and operated for about 4 years. The sanitary department of the District of Columbia had taken frequent cup-gage measurements of the depth of stormflow in the combined sewers. The District had good automatic rain gage records at four stations in the basin of Rock Creek. The original rainfall records of the United States Weather Bureau at Washington were also furnished us. Our work, utilizing all the foregoing material, is presented in chapters V, V–A, V–B, V–C. Figure 9 gives the essential results of flow and frequency, by two entirely different procedures referred to as method A and method B. This part of the work, although highly technical, is not academic. It is founded on observed records. The two methods are in reasonable agreement. Figure 9 is a reliable basis for the practical design of any proposed relief sewers for the lower sections of Rock Creek Basin.

7. *Diversion by relief sewers recommended.*—Your consulting engineers have carefully considered the several possible procedures for eliminating pollution of Rock Creek from the combined sewers on both sides of Rock Creek from Piney Branch to the Potomac. This is an area of 7.35 square miles with a population of 150,000.

(*a*) *Change to separate system.*—This procedure would be most effective. It was originally recommended for this section by the sanitary engineer, Rudolph Hering, in 1890 (ch. IV, par. 1). His advice was not followed. Today the area is so built up and improved with pavement and underground utilities, combined sewer, water, gas, and electric conduits that the cost of installing separate sanitary sewers is at least $500,000 in excess of other adequate remedial measures.

(*b*) Local treatment of storm sewage, except possibly mechanical screening, is impracticable on account of the location. It is likewise inadvisable to utilize detention storage of storm water and sewage for the purpose of reducing the size of relief sewers. The possible nuisance created does not justify the minor reduction of cost.

(*c*) The application of clean flushing water to mitigate the nuisance of stranded sewage deposits is a useful palliative to partial or inadequate remedial measures. The relatively small channel scoured by even large quantities of artificially applied flushing water would not reach the outer banks covered by storm discharges from the sewers. Nevertheless, any measures which will increase the low water flow in Rock Creek are desirable. In any investigation of flood control in Rock Creek the possible use of storage for increasing the dry-weather flow should be considered. (See ch. II.)

(*d*) The only method to be seriously considered today is the collection of all objectionable flows by means of relief sewers as hereafter described.

8. *Choice of relief sewer routes.*—The various possible routes for relief sewers have been compared on the basis of construction cost, relative operating efficiency, and relative public nuisance during construction. Detailed studies and estimates are presented in ch. VI.

9. *Recommendation, valley line.*—Your consulting engineers recommend for adoption the plan designated as V–1, the valley line. The route of this line is shown in figure 17.

It starts at the Piney Branch outfall and extends down the parkway as far as Park Road where it turns into the first of the three cut-off tunnels. It emerges from this tunnel just below Lamont Street and continues down the east side of Rock Creek Park to Ontario Road where the second tunnel cuts across the creek bend occupied by the Zoo nurseries.

Between the second and third tunnels it again skirts the east side of Rock Creek Park under the Calvert and Connecticut Viaducts.

The third tunnel is in the line of Twenty-third Street, Massachusetts Avenue, and Twenty-second Street between Belmont Road and P Street. Below P Street the route cuts across the present bend in the creek, the old channel being filled and a new cut-off provided. The remainder of the route follows the east bank of Rock Creek to the outlet into the Potomac. The upper end of this relief sewer is a 9½-foot horseshoe sewer; at Pennsylvania Avenue the size is 14½ feet. Tunnel sections are indicated on the plan figure 19 and in profile on figure 18. There is also included in this project the proposed west-side relief sewer as shown in plan and profile on figure 20.

10. *Cost of valley line V–1.*—The total cost of this recommended valley line, together with the west-side relief sewer and complete with all auxiliaries, is $2,772,000.

11. *Tunnel line T–1.*—This line is discussed here because it is a close second choice to the valley line. It is so located from the Piney Branch sewer outlet to the Potomac as to be in tunnel to the greatest possible degree. See location figure 12 and profile figure 13. Like the valley line this project includes the west-side relief sewer.

12. *Cost of tunnel line T–1.*—The cost of the tunnel line T–1, including the west-side relief sewer complete, is $2,963,000.

13. *Comparison between the valley line V–1 and tunnel line T–1.*—The tunnel line is located almost entirely under the streets, east of Rock Creek Park. It therefore offers the least construction nuisance to the park. On the other hand, the blasting opera-

tions are carried on under a densely settled section of Washington. The operations at the several tunnel shafts will necessitate a certain amount of street and traffic obstruction. While the blasting would not cause property damage, the specifications would have to preclude blasting at night. The three tunnels of the valley line lie in comparatively isolated neighborhoods and can probably be driven without intermediate construction shafts, thus reducing to a minimum the obstruction of traffic and the annoyance to the public.

The tunnel line will require certain auxiliary sewers to collect the sewage of some of the upper areas west of the line.

Balancing all the elements and the merits for and against each of the two plans, your engineers, as heretofore stated, recommend adoption of the valley line.

14. *Outfalls.*—Consideration was given to the possible shortening of the two relief sewers by locating the outfalls into Rock Creek at the canal mouth instead of carrying them on to the Potomac River. This would save $130,000 and is quite practical if it is desired to lower the cost of the project, since the sewers will discharge only during rains, and the valley is largely industrial below this point. The effluent during light rains, however, will be polluted to the same extent as the present effluent from the 28 existing outfalls along the creek, and the shortening is not recommended if money is available to carry the sewers to the Potomac.

The relief sewers therefore are designed to discharge during time of storms into the Potomac. All the dry-weather sewage flow will be trapped into the existing interceptors and carried by them to the Anacostia station. Your engineers, after a study of stream flow and dilution water in the Potomac, do not, at the present time, deem it necessary to carry the outfalls out into the river. Should this procedure be considered worth while in the future, the extensions can be made at any time.

Your engineers forsee no occasion for attempting to carry the enormous quantity of storm water, discharged at times by the proposed relief-sewer outfalls, to some lower point on the Potomac River. It is probable that increased capacity of the interceptors leading to the Anacostia station may be provided in the future. In this event the proposed relief sewers are at grades which permit connection to supplementary interceptors leading to the Anacostia station.

15. *Design data.*—Many of the tables and other data developed for this report will be of service in expediting the construction designs in the event that the works herein recommended are authorized.

Your consulting engineers have presented quite complete details for the principal diversion chambers and to the greatest degree possible have kept them free from mechanical control.

Cost estimates are based upon labor conditions and efficiency such as has obtained on similar structures built by the Public Works Authority.

All elevations in this report refer to the District of Columbia datum.

16. *Future requirements.*—The design of the proposed relief sewers is based upon stormflow and not upon population. The sewers are adequate to serve any future increase in the population of the combined sewer area.

17. *Results to be obtained by the proposed remedial works.*—Sewage pollution of Rock Creek above the District of Columbia line will be entirely removed when the Maryland authorities extend their sanitary sewers to care for the present wastes of some 1,000 houses and continue their proposed works so as to keep pace with the growth of population.

Pollution of Rock Creek originating from the upper section of the District of Columbia, including the Luzon and Klingle areas, will be prevented if the recommended separate sewer system is completed for these two areas and if efficient policing is maintained for all of the separate sewer areas.

With the installation of the two proposed relief sewers running to the Potomac from Piney Branch on the east side and from Connecticut Avenue on the west side, all offensive effluent from the combined sewer area will be removed from Rock Creek. No overflow from the combined sewers will enter the creek until the run-off exceeds 1,200 cubic feet per second from the Piney Branch area and similar high rates from the other areas. Such overflows following the heaviest storms will occur about 3 times per year now, and may occur 4 times per year in the future when the area is completely built up. This means that overflows to the creek will be reduced from 30 or 40 times per year to 3 or 4 times per year. It is the light storms which produce the most frequent overflows that now wash out the filthiest effluent from the sewers (ch. II, par. 6–9). The proposed relief sewers will remove all of such effluent now entering the creek.

When the infrequent heavy storms occur, the first flow from the sewers, up to 1,200 cubic feet per second (or equivalent), washes out the filth and it is carried off by the proposed relief sewers. The overflow which may follow will be highly diluted and will come from relatively scoured or cleaned out sewers. It will not constitute a nuisance.

To be specific on this important point your consulting engineers have analyzed the situation that would have occurred during the period 1929–34 had the proposed relief sewers been in operation (ch. V–C, par. 33, and table 12).

During the period there were about 170 days when rains would have caused the combined sewers to empty into Rock Creek. If the relief sewers had been in operation, this overflow would have been prevented on 164 of these days. During 6 days of heavy storms, overflow would have occurred but the volume of such overflows would have been reduced by an average of 93 percent.

This showing based upon 4¼ years of record, is proof that the proposed relief sewers will be adequate to prevent pollution of Rock Creek. It is also proof that relief sewers of larger capacity than those proposed are unwarranted. Relief sewers sufficient to carry all the run-off from the maximum storm would have to be four times the capacity of those proposed.

When all the aforesaid remedial works in the basin are completed, the water in Rock Creek will be entirely free from foul odor or sewage deposits. The water will support fish and aquatic life. It will not be fit for drinking. It may be desirable to chlorinate the water for bathing purposes.

This is all that can be said for any stream, entirely free from sewage pollution, which serves a basin occupied by from 5 to 60 persons per acre.

18. *Partial remedies.*—The cost for these complete remedial measures to prevent stream pollution requires a large sum of money—almost $3,000,000. It may be pertinently asked, Will not some partial remedial works suffice?

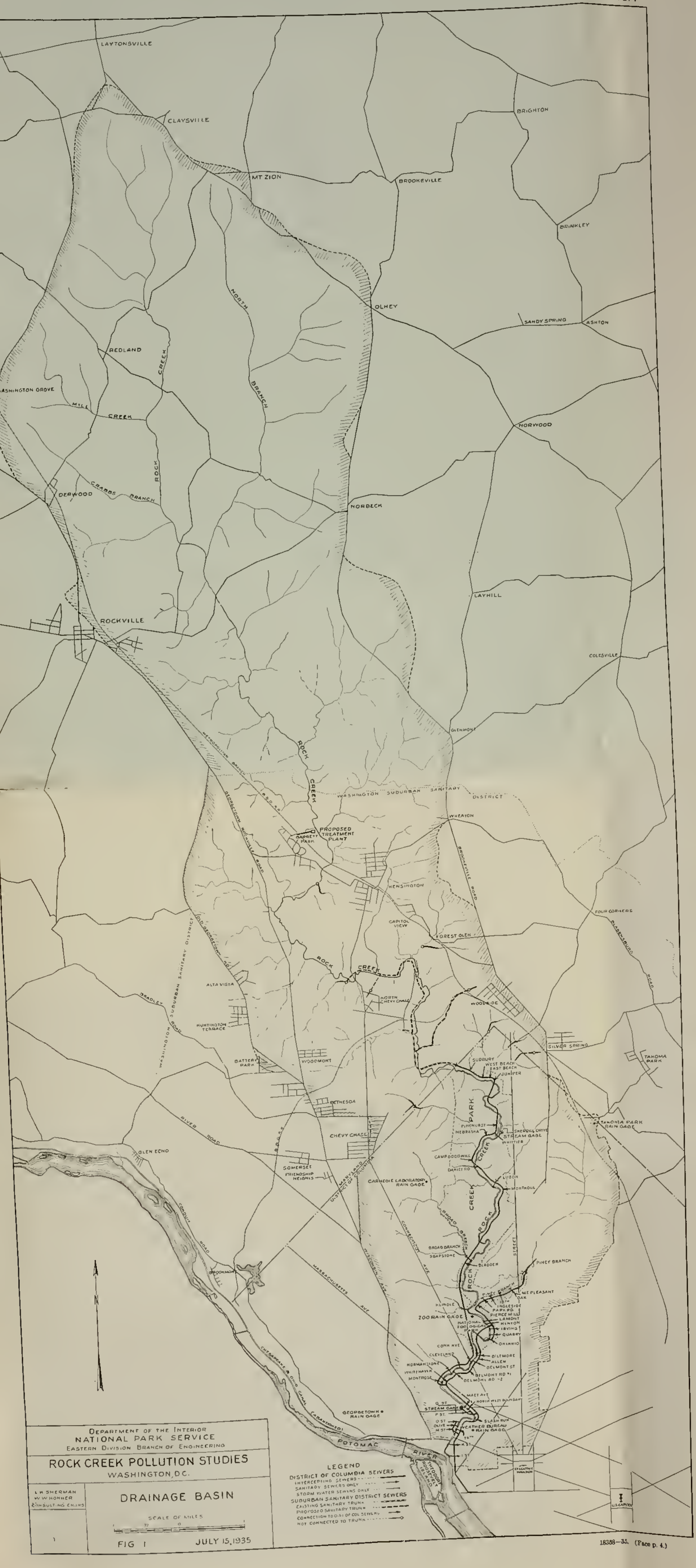
The completion of the separate sewer systems in the Klingle, Luzon, and Army Medical Centers at $550,000 is a worth-while project in itself. The National Park Service staff advise us that, for park purposes, unpolluted water in the creek above the Zoo is somewhat more essential than it is further down stream. Rock Creek affected by the lower combined sewers is now the foulest section of the stream. Your consulting engineers are of the opinion that any expenditure for partial or incomplete works in this lower section are not advisable or worth their cost. That complete remedial works should be built or none at all. Partial remedial works heretofore proposed cost from one-half to two-thirds of the amount required for the proposed complete relief sewers. With such partial works Rock Creek would still be a polluted stream.

19. *Acknowledgments.*—We wish to express our acknowledgments and appreciation to Mr. J. B. Gordon, director of sanitary engineering of the District of Columbia, and staff, for much of the data essential to this report; to Mr. Robert B. Morse, chief engineer of the Washington suburban sanitary district; to Mr. E. F. Kelly, chief of tests, and Mr. E. R. Shepard, research engineer, of the Bureau of Public Roads, for investigations to determine the depth of underlying rock; to Mr. John Nolen and staff of the National Capital Park and Planning Commission; to Mr. C. McDonough, chief engineer, and Mr. W. S. Merick, assistant engineer, Public Works Administration; to Mr. John L. Nagle, assistant chief, branch of engineering of the National Park Service, and staff, for guidance and cooperation in making this report.

Leroy K. Sherman,
Wesley W. Hormer,
Consulting Engineers.

Washington, D. C., *July 1935.*

FIG. I



18388—35. (Face p. 4.)

Chapter II

ROCK CREEK

THE SITUATION—STREAM POLLUTION

1. Rock Creek has a drainage area of 77.8 square miles. It follows a southerly meandering course from the upper end near Mount Zion, Md., to its mouth on the Potomac River in the northwest part of the city of Washington, D. C. (see fig. no. 1). The basin of Rock Creek is a beautiful wooded valley with steep to rolling hills. The creek in the District of Columbia forms the attractive feature of a national park and zoological garden.

The area of Rock Creek Park and the Zoological Gardens is 1,921 acres. The public investment in these parks in land value, bridges, roads, buildings, picnic facilities, etc., is $8,158,000.

The upper basin of Rock Creek in the District of Columbia and in Maryland is well developed with suburban homes. The District has installed excellent roads and pavements. Several monumental arch bridges span the creek. The lower part of the basin, including the Piney Branch or Takoma area, is densely built up with residences and apartments. The population of all the sewered districts in the Rock Creek Basin in 1930 was 182,800 (Report, Eddy, Gregory, and Greeley, p. 12). One hundred and fifty thousand of the above resided in the area probably served for all time by combined sewers. This is the area of 4,746 acres which includes Piney and the lower areas as shown in table on map no. 2. The Klingle and Luzon Valleys, with the Army Medical Center, which are now sewered on the combined plan but are to be converted to the separate plan have an area of 1,165 acres and a population of about 10,000.

2. Following all ordinary storms—that is 30 or 40 times per year—the sewage from the aforesaid population of 160,000 persons is now flushed out directly into Rock Creek. The National Park Service in cooperation with the United States Geological Survey has maintained self-recording gages recording the flow stages in Rock Creek since 1929. These stream records disclose an important phenomena. During the infrequent large storms of long duration there is recorded on the Q Street record an immediate sharp rise and sudden fall. This is followed, some hours later, by a second gradual rise and fall in the stage of Rock Creek. The first is the hydrograph of run-off due to water and sewage discharged by the adjacent combined sewers which bring down the run-off very quickly. The second rise is due to the flood run-off from the large open and unsewered area of over 60 square miles of the upper Rock Creek Basin. (See fig. 2a—Hydrograph No. 44B of Nov. 1, 1932.) Such large storms of wide extent, therefore, serve to wash out the antecedent sewage discharge from the creek. However, this subsequent flushing of the creek is infrequent. The ordinary rain produces a peak of sewage discharge 10 times as great as any after flow of relatively clean water. A typical example of this condition is shown by the hydrograph of July 25, 1933. (See fig. 6a.)

All this means that, following most of the rainfalls, sewage deposits are left stranded or pooled along the banks of Rock Creek between Piney and the Potomac. The offense is particularly apparent during warm weather. The sewer department of the District of Columbia sends out maintenance crews, following certain sewer overflows, to remove or flush stranded sewage and, so far as possible, mitigate the nuisance.

3. *Washington suburban sanitary district.*—Your consulting engineers have at various times conferred with the chief engineer of the Washington suburban sanitary district, and have examined the plans of the District for the additional trunk sewers immediately required to prevent the discharge of sewage into Rock Creek outside of the District of Columbia.

The most recent information on this matter is contained in a letter from Mr. Robert B. Morse, chief engineer, Washington suburban sanitary district, dated June 8, from which the following pertinent facts are abstracted:

> There are now approximately 2,838 sewered houses in the Rock Creek drainage area, 968 of which are connected to sewers discharging into Rock Creek, 1,834 to sewers connected directly or indirectly with the District of Columbia sewer system, and 36 to sewers leading to the Garrett Park sewage treatment works. Of the 1,834 houses, 1,487 discharge into the 36-inch Rock Creek intercepting sewer built by this commission.
>
> One hundred of the 968 houses, the sewage from which reaches Rock Creek can be removed from this category as soon as the District of Columbia extends its lines to the boundary near the north corner of the District. It will be evident therefore that, so far as our commission is concerned, more than two-thirds of the house sewage which might reach Rock Creek has been already eliminated.
>
> The Rock Creek intercepting sewer built by this commission is 5,600 feet long, of 36-inch diameter, and laid upon a grade of 0.19 percent. Its capacity flowing full is approximately 19 million gallons daily. Based upon the method of design used by this commission, this sewer is good for an average sewage

flow amounting to approximately 9 million gallons daily, or a population of approximately 75,000. The present population in the Rock Creek drainage area within the Washington suburban sanitary district is about 15,000.

The ultimate population of that part of the Rock Creek drainage area within the present limits of the sanitary district draining to the Rock Creek trunk sewer above the District of Columbia boundary we estimate at approximately 250,000. This is based upon an area of 12,242 acres and a density of from 15 to 25 per acre in different sections of the drainage area. The total average sewage flow from this population would be aproximately 35 million gallons daily.

It will be seen that at some time in the future the present Rock Creek trunk sewer will have to be relieved, but this will not be for many years. As to what will be done at that time I cannot predict, but it would appear that if available outlet sewer capacity in the District of Columbia were provided and if the charge by the District of Columbia for taking care of Maryland sewage were not too great, it would be advisable to discharge the flow from a relief trunk sewer in Maryland into the District of Columbia system. If either of the above factors did not eventuate it would appear to me advisable to divert either the excess flow in Maryland, or all of it, from the District of Columbia system and provide an outlet in the Potomac River above the District of Columbia boundary where not so intensive treatment of the sewage would have to be given as would be the case if it were discharged into Rock Creek.

I might say that the District of Columbia sewer along Rock Creek at the District of Columbia boundary is 36 inches in diameter and laid upon a grade of 0.15 percent. It will thus be evident that the trunk sewer which this commission has built is of slightly larger capacity than the District of Columbia sewer into which ours discharges.

To take care of the flow from Garrett Park, where now only 36 houses are connected with the sewerage system, the commission will start within the next month upon the construction of treatment works which will consist of primary sedimentation tanks and trickling filters.

The commission expects within the next few years to extend the Rock Creek trunk sewer upstream to a point a short distance west of Connecticut Avenue, where the last major sewer outlet is located. Above this point practically no sewage reaches Rock Creek below Garrett Park. Unless development becomes much more dense the trunk sewer will not be extended upstream for some time to come.

The area of the Washington suburban sanitary district is approximately 104 square miles and the present population about 75,000. There are 325 miles of water mains and 235 miles of sewer now in this district. There are 15,288 water-service connections and 11,625 sewer connections at the present time. In 1918 when the commission started operations there were 53 miles of water mains and 55 miles of sewer in existence, and total of 1,568 water-service connections and 1,630 sewer connections.

The present assessable basis of the sanitary district is $98,000,000. In 1918 it was $24,000,000.

Data for the basis of design is covered in a report to the sewerage commission of Montgomery and Prince Georges Counties, Md., February 3, 1914.

Robert B. Morse,
Chief Engineer.

The cost of the trunk sewer construction in Maryland required to produce a sanitary condition in Rock Creek Park is estimated by the Maryland authorities at $200,000.

4. *Stream analyses.*—The United States Public Health Service (S. Doc. 172, 72d Cong., 2d Sess.) made daily chemical and bacterial analyses of the water in Rock Creek from July 28 to September 9, 1932. The results were as follows:

	B Coli (coli-aerogenes) per cubic centimeter		5-day b. o. d. general average	Percent oxygen saturation
	General average	Maximum		
			p. p. m.	
Sherrill Drive	140	1,000	2.85	89
Above Zoological Garden	655	10,000	3.03	90
M Street	5,000	100,000	4.35	78
Potomac River (opposite Roosevelt Island)	115	1,000	1.40	85
Potomac River (below railroad bridge)	810	10,000	2.00	65

Analyses of the water from the creek at Sherrill Drive, indicate, from the somewhat decreased oxygen saturation values, from 51 to 89 percent, and the relatively high oxygen demand and the concentration of coli-aerogenes group organisms, the effect of sewage pollution entering the main stream and the Fenwick Branch from Maryland. Results from sampling stations just above the Zoological Garden at M Street, near the mouth of the creek, but above the Chesapeake & Ohio Canal and backwater from the Potomac River, indicate with the exception of the dissolved oxygen a progressive increase in pollution with the flow through the District. The sampling point above the Zoological Garden is located at the end of the narrow rocky section of the creek in which the water is efficiently aerated, which probably accounts for the increase in dissolved oxygen between Sherrill Drive and the Zoological Garden.

A sanitary survey of the Rock Creek watershed, within the District of Columbia, indicated that with the possible exception of accidental breaks in the sewerage system there was no direct sewage contamination of Rock Creek or its tributaries by domestic sewage when the storm water outlets of which there are 28 opening into the creek or its tributaries, are not in operation. As already noted, Rock Creek receives pollution in the form of domestic sewage from Kensington, Garrett Park, Forest Glen, and Woodside, Md. (U. S. P. H. Report, 1932; p. 46).

The bacteriological results on September 6 show in general a considerably higher concentration both in the main stream and most of the tributaries than that which existed a few days later after the effects of the storm were over. These special analyses indicate again the increase in pollution as the creek flows through the District of Columbia.

While there appears to be no evidence of sewage pollution within the District of Columbia, the bacteriological results indicate a high coli-aerogenes concentration in the water, due in part, it is believed, to the nature of the area which it drains. After long periods of decreased precipitation, the flow of the stream is made up primarily of ground water contribution from beneath a highly developed residential area on which past and present pollution has been excessive. It is believed that a considerable amount of the pollution of Rock Creek is derived from this source, and it seems very doubtful, even if the storm water overflows were removed from the watershed, that the water of the stream, especially in the lower section passing through the highly developed area along its channel, would be safe for bathing purposes. At times after the overflows have been in operation, a small percentage of the time in summer, greater pollution of the entire watercourse is indicated. In the winter and spring months of greatest rainfall, when the storm overflows are in more continuous operation, the creek is not being used for recreational purposes. (U. S. P. H. Report, 1932; p. 47).

5. *Bacteriological analysis of Rock Creek water by National Park Service*—

Analysis of water samples from Rock Creek on May 20, 1935, after 4 or 5 days of no precipitation and apparently no flow into the creek from the sewers within the District of Columbia.

Plates containing agar-agar media were set up for both the 21° C. and the 37° C. counts and the results expressed here are the number or organisms per cubic centimeter at the end of 48 hours.

Fermentation tubes containing 10, 1, and 0.1 of a cubic centimeter of the samples were set up and all of the same showed positive in less than 24 hours indicating the presence of gas-forming organisms. The presence of B. Coli was also shown and they existed in numbers greater than 10 organisms per cubic centimeter of the samples.

Samples taken approximately 150 feet downstream from the Highway Bridge at the District line at 2:15 p. m.

Temperature:	*Organisms per cubic centimeter*
21° C	11,500.
37° C	400.
Total	11,900.
B. Coli	More than 10.

Sample taken beneath the Connecticut Avenue Bridge at 1:30 p. m.

Temperature:	*Organisms per cubic centimeter*
21° C	15,000.
37° C	1,500.
Total	16,500.
B. Coli	More than 10.

Arbitrary standards for drinking water allow a total count of 100 organisms per cubic centimeter with no B. Coli, while swimming water standards allow a total count of 200 organisms per cubic centimeter with no B. Coli. In accordance with these standards the waters of Rock Creek at the time of sampling were unfit for any and all recreational purposes both at the District line and at the Connecticut Avenue Bridge. Inasmuch as the samples were taken at a time which appeared to be within a period of mean flow for the creek and when the water should have been at its best there is no doubt in my mind as to the very unfit condition of Rock Creek for all recreational purposes in all parts of the District of Columbia at all seasons of the year.

William R. Crane,
National Park Service.

6. *Analysis of sewage during storm overflow into Rock Creek.*—Under the direction of your consulting engineers observations and analyses were made of sewage flowing directly in Rock Creek from the Piney and Normanstone combined sewers. The purpose of these observations was to determine:

(*a*) The minimum rainfall which now begins to cause discharge of sewage into Rock Creek from the combined sewers.

(*b*) The strength of such sewage compared to the normal dry weather flow.

(*c*) The reduction in sewage strength, if any, during rains of long duration or of high intensity.

Samples were taken and analyzed at the Dalecarlia filter plant laboratory by William R. Crane of the National Park Service to determine the biochemical oxygen demand (b. o. d.), the turbidity and the total and organic solids. All figures are in parts per million (p. p. m.) except settleable solids which are recorded in parts per liter.

Following are the records reported by Messrs. W. R. Crane and L. R. Kemp: All the rains listed produced an overflow into Rock Creek and it is these overflows which are analyzed.

March 5, 1935.—Rain of March 5 reached a maximum intensity of 0.06 inch per hour for 10 minutes which occurred 15 minutes after rain began, the first 15 minutes being at a rate of 0.04 inch per hour. The average for the 3-hour duration being less than 0.02 inch.

No analysis is available for this rain, but the turbidities range from a maximum of 595, reached about 2 hours after rain started, to a minimum of 305; fluctuation between 305 and 400 for the last hour. There seems, however, to be no relation between turbidity and pollution for storm flow.

March 12, 1935.—Rain of March 12 shows several short peak rates, the first being at the rate of 0.6 inch per hour for 2 minutes, the second 0.9 inch for 2 minutes, the third 0.36 for 5 minutes and the fourth 0.6 for 2 minutes. These all occurred during the early hours of the morning and several hours prior to sampling. The average rate for the 8-hour duration was 0.06 inch per hour. The turbidities observed were low, the maximum being 155 and below 100 minutes. Biochemical oxygen demand was also low, the greatest being 240 parts per million and the lowest 53, due to the fact that the sampling was all done after the peak had passed.

March 25, 1935.—This rain had three peaks, the first 0.06 inch per hour occurred after rain had been falling for 30 minutes and was for a 10-minute period, the second was at the rate of 0.12 inch per hour for 10 minutes and came about an hour after rain started and the third at 0.06 inch per hour was for the last 30 minutes. The average for the 5-hour rain was 0.02 inch per hour.

The turbidities reached a maximum of 385 with biochemical oxygen demand 280 and settleable solids 4,000 parts per million 1 hour and 40 minutes after rain began while maximum pollution occurred 30 minutes later with biochemical oxygen demand of 410 and settleable solids of 4,800 parts per million with turbidity of 360; decreasing more or less uniform to a biochemical oxygen demand of 26 parts per million settleable solids 2,000 parts per million and turbidity 310 following one of less than 100 an hour earlier. There was a second period of increasing pollution during the afternoon which reached a maximum at 3:30 about 50 minutes after rain ceased when the biochemical oxygen demand was 112 with settleable solids of 1,800 parts per million and turbidity of 340. This was the last sample taken and states that flow was decreasing, but we are unable to say at what rate the pollution fell.

March 28, 1935.—Precipitation increased gradually to a peak of 0.18 inch per hour; reached 30 minutes after rain began to fall; it decreased to a rate of 0.06 inch per hour for the following 10 minutes with a second peak rate of 0.44 inch per hour which lasted for 15 minutes while for the last 10 minutes of rain the rate was 0.12 inch per hour, the average for the hour and 40 minutes being 0.16 inch per hour.

Maximum pollution occurred at the opening of the outfall gate, 1 hour and 15 minutes after rain began, when the turbidity was 535, biochemical oxygen demand 475, and settleable solids 12,000 parts per million. The turbidity increased but biochemical oxygen demand decreased during the next 10 minutes. Turbidity 675 with biochemical oxygen demand 260 and settleable solids 12,000 parts per million pollution decreased more or less uniformly thereafter to a minimum of; turbidity 525 and biochemical oxygen demand 77 with settleable solid 2,800 parts per million reached at 2:45 p. m.; about the time rain stopped falling. Sewage continued to flow into the creek after this.

The depth gage at Normanstone records depth of flow in the sewer; the maximum for this date being 0.6. The velocity at that depth was 10.2 feet and the flow 19 cubic feet per second from the 242 acres.

April 1, 1935.—Sampling started at 8:45 a. m. Monday after a rain which had continued at an average rate of about 0.06 inch per hour throughout the preceding day. The turbidities recorded ranged between 200 and 300 with fluctuating biochemical oxygen demand and settleable solid running from 9 to 150 parts per million and 200 to 600 parts per million respectively but having no relation to turbidities, the maximum and minimum both occurring with turbidities of 255.

Apr. 8, 1935

STATION A, PINEY BRANCH SEWER

Time	Turbidity	Percent stability	B.o.d.	Solids: Total	Solids: Organic	Solids: Settleable	Rainfall	Depth in sewer
a. m.								
9	238	80	84	343	75	0.6	Light	0.7
10	235	80	128	348	73	.4	do	.7
11	228	--------	173	915	624	.4	do	.6
Noon								
12	205	--------	140	390	100	.6	do	.6
p. m.								
1	215	--------	202	343	96	.6	do	.6
2	--------	--------	150	315	75	--------	do	.6
3	600	50	132	630	91	1.2	do	.6
NORMANSTONE SEWER								
a. m.								
9:30	134	--------	82	253	105	0.12	Light	0.4
10:30	109	--------	91	252	44	.12	do	.35
11:30	150	--------	167	383	151	.16	do	.35
p. m.								
12:30	164	--------	107	324	155	.20	do	.35
1:30	161	--------	75	513	155	.20	do	.30
2:30	140	--------	103	221	50	.08	do	.30
3:30	161	--------	56	885	218	.20	do	.30

No rain gage records are available for this storm, but Crane reports that sewer had been overflowing all of the preceding night when sampling started at 9 a. m. Monday. Biochemical oxygen demand ranged from 84 to 202 parts per million, the total and organic solids varying more or less uniformly (with two marked exceptions) as the biochemical oxygen demand. Total solids ranged between a minimum of 315 and a maximum of 915 parts per million, remaining generally between 300 and 400 parts per million with organic solids about 25 percent of the total for the 300 to 400 parts per million range.

The first marked exception occurred at 11 a. m. with biochemical oxygen demand of 173, total solids 915, organic solids 624, and settleable solids 400 parts per million, the second being at 3 p. m. when biochemical oxygen demand was 132, total solids 630, organic solids 91, and settleable solids 1,200 parts per million.

April 12, 1935.—Rain began at 10:40 a. m. and fell at the rate of 0.12 inch per hour for 10 minutes and at the rate of 0.06 inch per hour for the remaining 30 minutes of the rain. Maximum pollution occurred coincident with the opening of the outfall gate at 11:45 a. m., the turbidity, biochemical oxygen demand total, and organic solid at that time being respectively 340, 113, 485, 227 parts per million with settleable solids of 1,600. During the ensuing 5 minutes solids increased to 691 total, 288 organic, and 1,600 settleable, but biochemical oxygen demand fell to 70 parts per million.

Thereafter the pollution decreased with a minimum biochemical oxygen demand of 15 parts per million at 12:45 p. m. and fluctuated between 15 and 50 parts per million until 2:45 p. m., at which time the Piney Branch flush tank discharged.

7. *Dry-weather sewage.*—Following are analyses of domestic sewage during dry weather:

Normanstone sewer

Date	Time (a. m.)	Turbidity	Settleable solids (percent)	Percent stability	Biochemical oxygen demand
1935					
Mar. 31	9:30	100	0.2	55	41
Do	10:30	177	1.2	44	74
Do	11:30	152	.2	37	131
Apr. 2	9:05	307	10.0	35	221
Do	11:25	173	7.2	35	134
Apr. 3	9:00	270	5.2	60	135
Do	11:35	200	2.8	20	171
Apr. 4	9:05	236	3.6	21	--------
Apr. 5	8:47	263	4.0	35	146
Do	11:45	166	2.0	48	74
Apr. 6	8:50	246	4.0	37	75
Do	11:25	213	3.6	37	69
Apr. 7	11:15	230	1.6	37	133

Piney Branch sewer

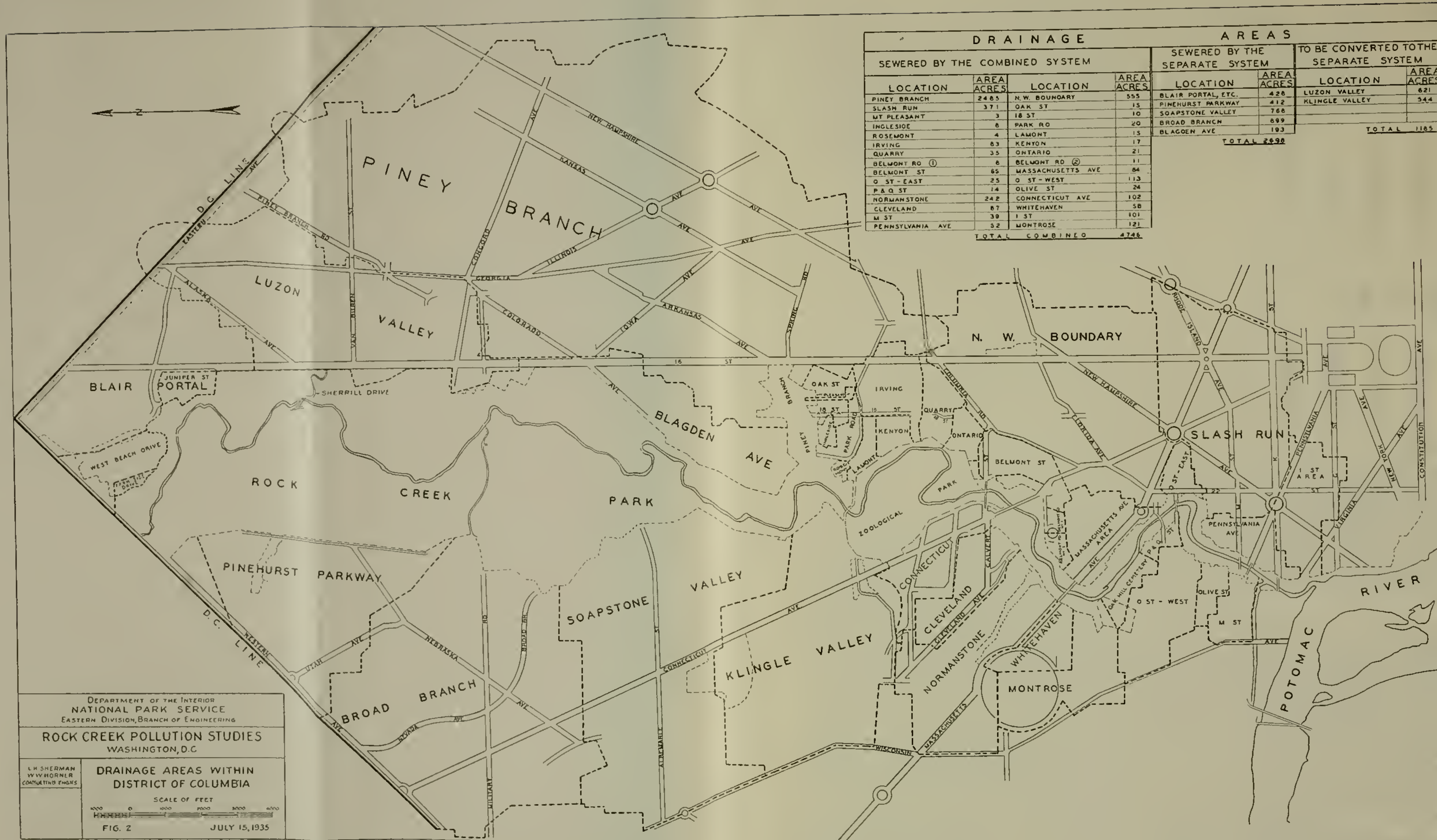
Date	Time (a. m.)	Turbidity	Settleable solids (percent)	Percent stability	Biochemical oxygen demand
1935					
Mar. 31	9:00	214	0.8	47	4
Do	10:00	228	.3	44	27
Do	11:00	235	1.0	37	122
Apr. 2	8:45	390	6.0	35	108
Do	11:10	288	7.2	35	318
Apr. 3	8:45	358	3.2	20	165
Do	11:15	273	4.0	35	194
Apr. 4	8:50	387	2.8	21	196
Apr. 5	8:30	390	8.4	11	134
Do	11:30	268	3.6	35	126
Apr. 6	8:33	400	4.8	37	155
Do	11:00	375	6.4	37	179
Apr. 7	10:55	368	6.0	37	134

The results of analysis of the domestic sewage taken at the same sampling points during dry weather are for the purpose of comparison with the storm overflow. The biochemical oxygen demand of the daily samples ranging between 60 and 200 parts per million is that of a weak domestic sewage and is in close agreement with the results of 24-hour biochemical oxygen demand tests of Washington sewage made during the same period by the United States Public Health Service.

8. *Variation of sewage strength with rainfall.*—Analysis of the samples shows generally an increasing turbidity, biochemical oxygen demand, and total solids during the first hour and a half of rainfall, reaching a maximum at about that time and being generally, for the rains observed, that of a strong domestic sewage.

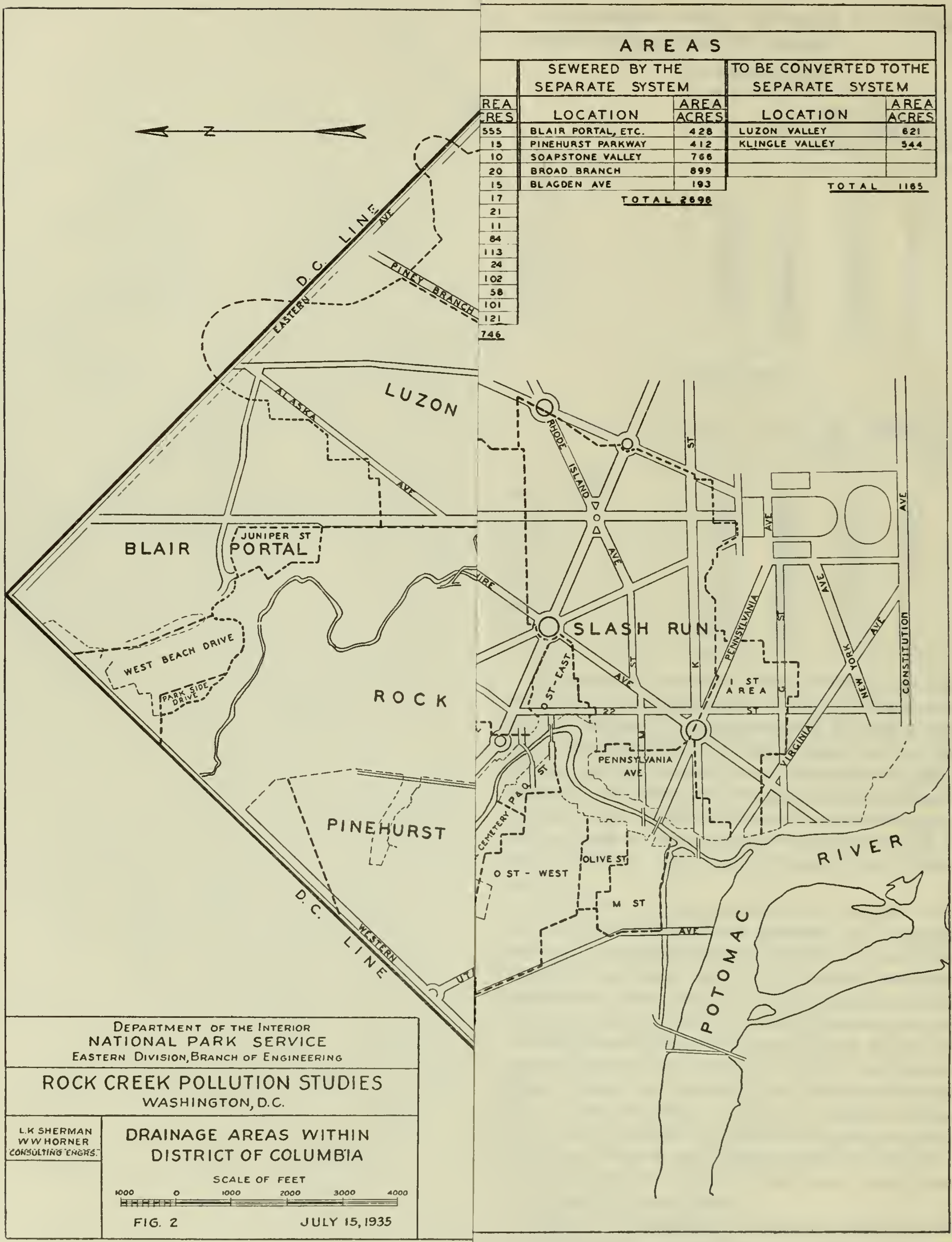
During the ensuing 30 minutes there is a slow, but gradual, decrease in strength to that of medium domestic sewage, and thereafter the pollution decreases more or less uniformly to that of a highly polluted stream, namely, with a biochemical oxygen demand of from 30 to 50 parts per million where the duration of the rain is such as to cause overflow to Rock Creek over a period of 4 to 5 hours.

Analysis of the samples for total and organic solids shows the same characteristics of increase during the first 1½ hours of rainfall, fluctuating but gradually decreasing thereafter.



DRAINAGE AREAS							
SEWERED BY THE COMBINED SYSTEM				SEWERED BY THE SEPARATE SYSTEM		TO BE CONVERTED TO THE SEPARATE SYSTEM	
LOCATION	AREA ACRES	LOCATION	AREA ACRES	LOCATION	AREA ACRES	LOCATION	AREA ACRES
PINEY BRANCH	2485	N.W. BOUNDARY	555	BLAIR PORTAL, ETC.	428	LUZON VALLEY	621
SLASH RUN	371	OAK ST	15	PINEHURST PARKWAY	412	KLINGLE VALLEY	544
MT PLEASANT	3	18 ST	10	SOAPSTONE VALLEY	768		
INGLESIDE	8	PARK RD	20	BROAD BRANCH	899		
ROSEMONT	4	LAMONT	15	BLAGDEN AVE	193		
IRVING	83	KENYON	17				
QUARRY	35	ONTARIO	21				
BELMONT RD (1)	8	BELMONT RD (2)	11				
BELMONT ST	65	MASSACHUSETTS AVE	84				
O ST - EAST	25	O ST - WEST	113				
P & Q ST	14	OLIVE ST	24				
NORMANSTONE	242	CONNECTICUT AVE	102				
CLEVELAND	87	WHITEHAVEN	58				
M ST	39	I ST	101				
PENNSYLVANIA AVE	52	MONTROSE	121				
		TOTAL COMBINED	4746	TOTAL	2698	TOTAL	1165

18368—35. (Face p. 9.)



18368—35. (Face p. 9.)

Organic solids vary from 25 percent of the total at the beginning of the rain to 55 percent at the time of maximum pollution, decreasing thereafter with a corresponding increase in the percentage of inorganic matter composing the street washings, etc.

9. *Conclusion.*—The greatest pollution is caused by the storm flows which, with their attendant higher velocities, transport the sewage solids that have become stranded in flat lateral sewers during dry weather, changing the character of the flow from that of a weak domestic sewage to that of a strong septic sewage carrying in extreme instances as much as 2 to 3 times the quantity of settleable solids present in the average flow.

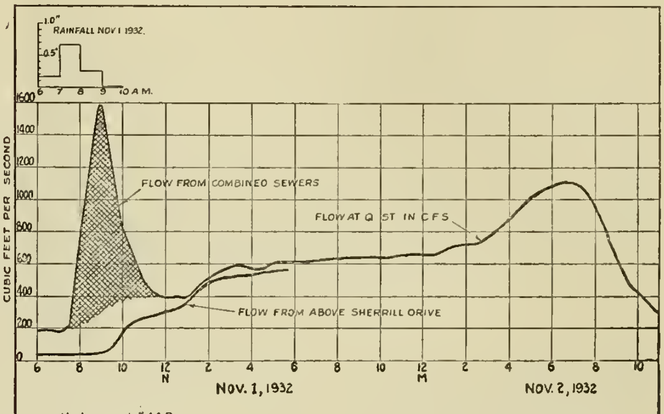
This period of increasing pollution extended generally over 1½ hours at which time peak pollution was reached in the rains observed. This time would be materially reduced by rains of high intensity and probably would not exceed 15 to 30 minutes for rains of the character for which the relief sewers are designed. The decrease in pollution, with heavy rainfall, after the peak is passed will be much more rapid than indicated by the light rains observed. The relief sewers as proposed will, by diverting this first overflow, prevent objectionable pollution of Rock Creek.

10. *Floods in Rock Creek.*—The highest flood in the records of the United States Geological Survey at Q Street (1929–34) occurred on August 23, 1933, with a peak flow of 4,300 cubic feet per second (table 12). The run-off studies from Piney Branch (table 8 and fig. 9, method B), which are based upon the 10-year rainfall records (1925–34), show that materially greater floods have occurred and will happen in the future.

The relief sewers herein proposed will reduce or eliminate the height of the numerous first peaks. The proposed relief sewers will not materially reduce the second flood peaks which come from heavy and long-continued rainfall upon the basin of Rock Creek above Sherrill Drive.

The height of floods in Rock Creek Park is augmented by numerous bad constrictions of the waterway by bridge abutments, minor structures, and drift.

The flood situation, stream-flow regulation, and bank protection are not in this assignment. However, the numerous hydrographs of flow of Rock Creek made in connection with this report can be utilized to determine flood flows from rainfall records where direct stream flows have not been taken.



TYPICAL DOUBLE PEAK HYDROGRAPH

FIG. 2A

Your consulting engineers have recommended plans for keeping filth from entering Rock Creek rather than the palliative of attempting to flush out the creek with clean water. In adopting this procedure we are not unmindful of the sanitary value of such minimum flow of clean water. Possible regulation of minimum stream flow, as a worth-while asset to Rock Creek Park, should be investigated in connection with flood control.

Chapter III

EXISTING SEWER SYSTEMS

1. *Use of combined and separate sewer systems.*—In chapter IV hereafter, it is shown that the early engineering reports on the development of the Washington, D. C., sewerage had definitely in mind the use of separate systems of sewers throughout the Rock Creek Basin. (See Hering report.) By the expression "separate systems of sewers", it is to be understood that one sewer system is designed to carry sewage and polluted water of all kinds; that this network and its outlets (generally referred to as the sanitary sewers) lead the flow of such liquids to points of satisfactory disposal; and that another scheme of sewerage is laid out for the sole purpose of collecting storm water from roofs, streets, and other ground surfaces and conveying such relatively unpolluted liquid to the nearest natural water course into which it can be discharged.

2. For some reason not clear from an examination of the records, these recommendations were not followed during a considerable part of the development period of the District, and instead, a number of the subdrainage basins in the Rock Creek Valley were actually sewered on the combined plan; that is, only one network of collecting sewers was constructed, carrying all liquid wastes and storm water to points of discharge in the Rock Creek Valley. This system of combined sewers was properly supplemented by intercepting sewers situated in and along Rock Creek Valley, designed to receive only the sanitary sewage from the combined sewer outlets of the various subbasins, and to carry this flow of sewage into the main sanitary trunk sewers of the District, these main sewers in general discharging into the Potomac near the Anacostia River.

After a number of subdistricts had been sewered on the combined plan, the policy was changed, and thereafter subdistricts were sewered in accordance with the original recommendation for separate sewers.

3. When the occupancy of the land in Maryland, adjacent to the District of Columbia, had been increased to such density as to require sanitary improvement, sanitary sewers for sewage only were built in a number of sections. At a later date, the Washington Suburban Sanitary District was organized, and this district proceeded with the construction of main trunk sanitary sewers, to furnish an outlet for the various isolated sewered areas.

On figure 1, the relation of the Rock Creek Valley to the District of Columbia and to the adjacent territory in Maryland is shown, and the area served by the Washington Suburban Sanitary District in the Rock Creek Basin is indicated. On figure 2, the subdrainage basins of the Rock Creek System within the District of Columbia are outlined, and on this figure the type of system installed in each subarea and the acreage drained by the system are listed. In general, it will be noted that the subbasins draining into the lower part of the Rock Creek Valley, that is, between the Potomac River and the mouth of Piney Branch, have all been sewered on the combined plan. The ordinary dry weather flow from these subbasins, consisting almost entirely of sewage, is diverted from the various outlets, into the Rock Creek intercepting sewer and its tributary known as the "Piney Branch intercepting sewer." As shown later, these sewers are capable of carrying a relatively small flow. Consequently, when the discharge from the combined sewer outlets is augmented, even to a small degree, by storm drainage, the bulk of the flow from the sewer, and in some instances all of the flow, is discharged into Rock Creek.

In the upper part of the valley, that is, between the mouth of Piney Branch and the District line the sewer systems for the subbasins are of the separate type, and the outlets for the sanitary systems in these basins are connected directly into the Rock Creek intercepting sewer. In consequence, the so-called "Rock Creek intercepting sewer" throughout that part of the valley above the mouth of Piney Branch is, in fact, a sanitary trunk sewer and not an intercepting sewer.

4. *Luzon and Klingle districts.*—Exceptions to the above-described conditions should be noted: (*a*) As to the sewers in the Luzon district, within which is the Walter Reed Hospital and Army Medical Center, and in the Klingle district on the west side of the valley; these sewer systems are actually to a great extent of the combined type, but a project to convert these sewers to a separate scheme has been adopted and partly accomplished. (*b*) The sanitary sewer systems for other subbasins in the upper valley are not entirely free from the influence of rainfall and consequent storm drainage, since the regulations of the District permit the connection of certain drains into the sanitary sewers. These are ground water drains installed for the purpose of keeping basements dry, and drains to serve bell traps and other inlets at the foot of areaway stairs and certain garage platforms. (*c*) It is known that, regardless of regulation to the contrary, downspouts from the roof guttering system occasionally become connected to the sanitary sewers. While no general investigation has been made as to the extent of such connections, a preliminary inspection of 2 separate systems showed that in 1 block no downspouts were connected to the sewer, but 1 yard drain was connected, and in another block 4 downspouts were

connected to the sewer, as were 14 back-yard drains. It is understood that sanitary authorities of the District of Columbia are proposing to undertake detailed inspection as to surface water connections into sanitary sewers, and to attempt to bring about a standard practice as to the permissible amount of such surface water allowed to enter the sanitary sewers, and that this matter will be subject to complete review and new enforcement regulations whenever the other steps necessary to develop sanitary standards in the Rock Creek Valley have been carried out. This question will be discussed in a subsequent chapter in connection with studies of the required capacity of relief and outlet sewers.

5. *Rock Creek intercepting sewers*.—The Rock Creek and Piney Branch intercepting sewers were built many years ago, and it is possible that they were expected to be used as sanitary outlet sewers, taking the flow entirely from sanitary sewer systems to which no surface water was admitted. Even under this criterion, it appears now that the capacities provided would be too low to comply with safe sanitary standards. As intercepting sewers serving the outlets of combined subsystems, their present capacity is inadequate in the extreme. As an illustration, the Piney Branch tributary of the Rock Creek interceptor has a usable capacity of about 35 second-feet for an area of nearly 3,000 acres. It is now running nearly half full in dry weather, and would become overcharged with the run-off of a small shower, as, for example, 0.1 of an inch of rain falling in 20 minutes. It is clear that with intercepting sewer capacity as small as this, crude sewage only slightly diluted will be discharged into Rock Creek many times a year.

With intercepting sewers of such inadequate capacity, it is not surprising that the United States Public Health Service, in its 1932 report, found unsanitary conditions in and along Rock Creek. (See ch. II.)

6. *Imperviousness*.—The present percentage of impervious area is one of the factors to be considered in the study of rainfall and run-off (ch. V). In order to derive a fair figure for this factor field observations and measurements of typical blocks were made. The results are given in paragraph 7. The average percentage of imperviousness for different sections was derived from the aforesaid field observations and study of airplane maps using the measured blocks as standards. These figures, recorded to the nearest 5 percent, are shown on the map in the accompanying appendix.

Impervious area includes roofs, pavements, and walks although all the latter may not be 100 percent impervious in delivering all rainfall to the combined sewers.

Observations on downspouts showed wide variation in direct connection to the sewers as shown in accompanying paragraph 8.

In method B (ch. 5–B) a figure of 50 percent impervious was used as an average for all of the combined sewer area above Q Street.

7.—RESULTS OF TYPICAL BLOCK COMPUTATIONS

	Percent impervious
Broad Branch—Block between McKinley, Morrison, 32d and 33d Streets	44.0
Soapstone Valley—Block between Veazey, Van Ness, 28th and 39th Streets	53.2
Klingle Valley—Block between McComb, Howell, 34th and 35th Streets	50.0
Normanstone Valley—Block between Normanstone Drive, Benton Street, Rock Creek Drive and 30th Street	36.0
Luzon Valley—Block between Alaska Avenue, 14th and Holly Streets	43.2
Piney Branch—Block no. 1, between Tuckerman. Sheridan, 5th and 7th Streets	57.5
Piney Branch—Block no. 2, between Gallatin, Farragut, 5th and 7th Streets	50.0
Piney Branch—Block no. 3, between Emerson, Farragut, 13th and 14th Streets	43.5
Northwest Boundary and Slash Run—Block between 17th 18th, Q and P Streets	93.7

All these impervious percentages represent present conditions with the vacant lots figured as pervious area.

If typical improvements are put on the vacant lots, the impervious percentages increase as follows:

Broad Branch Block		49.4
Soapstone Valley Block	(no change)	53.2
Klingle Valley Block	do	50.0
Normanstone block		52.7
Luzon Valley block		48.5
Piney Branch no. 1	(no change)	57.5
Piney Branch no. 2	do	50.0
Piney Branch no. 3		45.0
Northwest Boundary and Slash Run	(no change)	93.7

8.—RESULTS OF DOWNSPOUT OBSERVATIONS ON TYPICAL BLOCKS

Broad Branch block, sewered on the separate system:
- Downspouts all discharge on ground.
- 1 yard drain to sewer.

Soapstone block, sewered on the separate system:
- 4 downspouts to sewer (rear half of two houses on Thirty-eighth Street, represents 5 percent of total house area).
- 14 back-yard drains to sewer.

Klingle block, sewered on the combined system at present:
- 63 downspouts to sewer.
- 4 downspouts to ground.

Normanstone block, sewered on the combined plan:
- 30 downspouts to sewer.
- None to ground.

Luzon Valley block. sewered on the combined plan at present:
- 24 downspouts to sewer.
- 34 downspouts to ground.

Piney block no. 1, sewered on the combined plan:
- No downspouts to sewer.
- 112 downspouts to ground.

Piney Branch no. 2, sewered on the combined plan:
- 156 downspouts to sewer.
- No downspouts to ground.

Piney Branch no. 3, sewered on the combined plan:
- 97 downspouts to sewer.
- 4 downspouts to ground.

Northwest boundary and Slash Run:
- All downspouts drain to sewer.

Chapter IV

REPORTS HERETOFORE MADE ON POLLUTION OF ROCK CREEK

1. *The Hering Report, 1890.*—In June 1890 a report was submitted by Rudolph Hering, Samuel M. Gray, and Frederick P. Stearns. This Board of Sanitary Engineers was appointed by President Benjamin Harrison "to examine and report upon the system of sewerage existing in the District of Columbia, together with such recommendations as may by them seem necessary and desirable for the modification and extension of the same." This report, page 42, reads in part:

> Up the valley of Rock Creek, where new streets are being laid out, but where sewers are not yet built, the conditions are different from those which obtain in the populated parts of the City, and for this District we advise the adoption of a modified separate system. The surface water from the streets should not be allowed to enter the sewers which carry the sewage, but should be collected in separate underground channels discharging into Rock Creek, or its large branches, whenever underground ıemoval becomes necessary.

At the time the report was made all of the Piney Branch area, as well as other outlying territory, was unsewered. The District of Columbia did not adopt the recommendations of the Hering report for a separate sewer system in the Piney and other Rock Creek areas, and in 1906 proceeded with the construction of combined sewers. The present pollution of Rock Creek is primarily due to this failure to follow the Hering recommendations.

2. *The Gordon Report, 1928.*—In a letter to the Engineer Commissioner, dated July 27, 1928, Mr. J. B. Gordon, director of sanitary engineering of the District of Columbia, states: "The total cost of the inspectional and cleaning work incidental to the Piney Branch trunk sewer outlet amounted to $1,343.04 for the fiscal year ending June 30, 1928." He suggests four possible procedures for improving sanitary conditions of Rock Creek. In brief they are as follows:

> 1. Maintaining present sewers as storm water carriers only and constructing a separate system of sewers, to parellel the existing sewers, for the purpose of intercepting all sanitary flow.
>
> An approximate estimate of cost of a system of sanitary sewers as discussed above is $3,000,000.
>
> 2. Maintaining present system as a carrier of street water only and constructing a separate system of sewers to parallel the existing sewers for the purpose of intercepting all house laterals.
>
> Under this method, all existing house laterals would be intercepted by the smaller sewers, which sewers would thus be called upon to carry not only sanitary flow, but storm flow from roof leaders, yard and area drains. Consideration would be given to require only sanitary connections to be made to the new sewers in the case of future buildings, with the privilege of connecting roof leaders, etc., to the large sewers if so desired.
>
> An approximate estimate of cost for such a system as described under this method is $3,500,000.
>
> Without definite facts available, it is the opinion of the writer that this method no. 2 is impracticable in that the present interceptor, extending from the mouth of the Piney Branch trunk sewer down the valley of Piney Branch and thence along Rock Creek, is believed to have insufficient capacity to carry the flow to be expected in times of storm. The dry weather flow in this interceptor at present is now greater than half its capacity.
>
> 3. Constructing combined system interceptors along both banks of Rock Creek, north from Potomac River, to remove all D. C. sewage from Rock Creek.

Mr. Gordon's estimate of cost for this comprehensive plan of combined sewers is as follows:

Combined sewer on east side of Rock Creek	$4, 500, 000
Combined sewer from Luzon Valley	1, 000, 000
Or (alternate) separate system for Luzon and Walter Reed Hospital	350, 000
Combined sewer on west side of Rock Creek	750, 000
Klingle Road extension ($75,000+$450,000)	525, 000
Or (alternate) separate Klingle system	125, 000
Total minimum cost	5, 725, 000

> 4. Extending the Piney Branch trunk sewer to Rock Creek, thereby removing pollution from Piney Branch only.
>
> In considering this method, it must be borne in mind that although diluted sewage, during times of storm, would no longer be discharged into the upper end of Piney Branch, it would discharge directly into Rock Creek and consequently the sanitary condition of Rock Creek itself would not be improved.
>
> Estimated cost of this extension $600,000.

Mr. Gordon discusses the relative merits of the four projects as follows:

> Method no. 1 discussed above is not considered possible of accomplishment within a reasonable period of time, and the $3,000,000 expenditure necessary at this time is not believed warranted when it is considered that only a partial remedy would be in force for many years to come.
>
> Method no. 2 estimated to cost $3,500,000 is believed by the writer to be impracticable due to the insufficiency in capacity of the sewer system south from Piney Branch.
>
> Method no. 3, ranging from an estimated cost of from $5,725,000 to $6,775,000 is not considered justifiable in that, in the opinion of the writer, the benefits to be obtained are not proportional to the necessary expenditure, and furthermore, due to the fact that the District authorities have no control over the upper reaches of Rock Creek as to preventing pollution from Maryland.
>
> Method no. 4 is considered worthy of consideration, in that the estimated cost, i. e., $600,000 is possibly within the bound of good economics. Furthermore, this method would remove all pollution from the Piney Branch Parkway, and while not improving the conditions of Rock Creek would, with the single exception of the Luzon Valley sewer, remove all storm water overflows from the stream within the boundaries of Rock Creek Park.

3. *House Committee on Appropriations 1927–29.*—A discussion of this Piney Branch sewer "nuisance" appears at some length in the hearings before the Subcommittee of the House Committee on Appropriations. See 1927 hearings, pages 225–228; 1928 hearings, page 283; 1929 hearings, pages 275–278.

4. *Senate Document 172, Seventy-third Congress, Second session, 1933.*—"Disposal of Sewage in the Potomac River" made by the United States Public Health Service, contains the following references to Rock Creek conditions:

	Page
Population 1932	17
Sampling stations	23
Analysis of Creek water	34
Analysis of Creek water average	37
Comments pollution of Rock Creek	46
Special analysis September 1932	47
Special analysis September 1932 comments	47

(Noted in this report elsewhere.)

5. *Report of the National Capital Park and Planning Commission 1930.*—(See following report no. 6, p. 37.)

6. *Report of Eddy, Gregory, and Greeley 1934.*—On April 30, 1934, a board of sanitary engineers, consisting of Messrs. Harrison P. Eddy, John H. Gregory, and Samuel A. Greeley, presented a report to the Board of Commissioners of the District of Columbia on sewerage and sewage disposal. In part II of this report valuable data is presented on sewerage and drainage in Rock Creek basin together with recommendations and procedure to remove or limit the pollution of Rock Creek. This report is available in published form.

The following citations relative to Rock Creek are presented here:

Rock Creek and Piney Branch are objectionably polluted, in part as a result of the departure from the 1890 report, above mentioned, and by the limited allowance made for storm sewage in the intercepting sewers. An important source of pollution is the discharge of storm sewage, due to the limited size of the intercepting sewers. This condition may be aggravated by the complete closing of regulators of the float type during storm-flows. Another source of pollution is the sewage continuously discharged directly into Rock Creek within the limits of the Washington suburban sanitary district.

The effect of these pollutions is to cause odors at various points along these creeks when the streamflow is low and to leave objects of sewage origin stranded along the margins of the creeks. Much of the valley is a beautiful national park, and such pollution is highly objectionable.

This pollution may be controlled by the construction of a relief sewer from the upper end of Piney Branch at Sixteenth Street NW. to the Potomac River, and by flushing Piney Branch and Rock Creek below Piney Branch with settled Potomac River water taken from the Lydecker water tunnel of the District of Columbia aqueduct system. The relief sewer would greatly decrease the frequency and duration of overflows of storm sewage into these creeks. With the proposed flushings, the effect of the greatly reduced number of overflows would be negligible.

In addition to the intercepting sewers along Piney Branch and Rock Creek, some of the other intercepting sewers may need relief but, in general, only in the distant future (pp. ii and iii).

RECOMMENDATIONS

1. That a relief sewer be built in tunnel from the Piney Branch trunk sewer, near Sixteenth Street NW. to the Potomac River at the foot of New Hampshire Avenue, to assist in correcting the unsatisfactory conditions along Piney Branch and Rock Creek and that the storm overflow outlet in the river from this relief sewer be submerged.

2. That a pipe line be laid from the Lydecker water tunnel, where it crosses under Rock Creek near Massachusetts Avenue, to the upper end of Piney Branch, at Sixteenth Street NW. and that settled Potomac River water be drawn, by gravity, from the tunnel and used for flushing out Piney Branch and Rock Creek, immediately following each overflow of storm sewage into either of these streams.

4. That new areas in which sewers have not yet been built be sewered on the separate system.

5. That districts now sewered on the combined system so remain and that no attempt be made to change to the separate system except in certain relatively small areas where the District has already adopted the policy of making such a change; to wit: Potomac Heights, Luzon Valley, Klingle Valley, and possibly the Good Hope Road drainage area. In this conversion it is particularly important that the Army Medical Center undertake the revamping of its plumbing and drainage system to effect the separation of sewage from storm run-off.

6. That field investigations be made to determine the rates of sewage flow in each of the intercepting and main trunk sewers for the purpose of ascertaining, if and when relief may be needed, and also for determining how much flow should be taken in through each sewage-flow regulator, and that field investigations be made of the operation of each sewage-flow regulator for the purpose of improving its operation so as to thereby reduce the overflow of storm sewage to a minimum. These field investigations should be made at more or less regular intervals, say every 5 years, to ascertain the effects of changes in the distribution of population from forecasts made in this report, and of changes in other conditions.

7. That consideration be given to the drawing of settled Potomac River water from the Lydecker water tunnel through the pipe line heretofore recommended, and the discharge of the water into Piney Branch for the maintaining of a flow of water in Piney Branch and Rock Creek in daylight hours during periods of low stream flow, especially during the warmer months, so as to thereby greatly enhance the attractiveness of Rock Creek Park for recreation (pp. vi and vii).

There is no record of the number and duration of storm sewage discharges into Piney Branch and Rock Creek. They are sufficient, however, to make necessary routine cleaning of the channels at intervals of a few days to a few weeks depending on the frequency of rains and on the location along the creeks. Cleaning is especially necessary along Piney Branch and the channel of Rock Creek immediately below it (p. 35).

As stated in the 1930 report of the National Capital Park and Planning Commission, pages 120 and 121, four methods have been proposed and rejected for relieving Piney Branch and Rock Creek below it from pollution as follows:

Method no. 1 considered maintaining the present sewers in this valley as storm-water carriers only, and constructing a separate system of sewers to parallel the existing sewers for the purpose of intercepting all sanitary flow.

Method no. 2 considered maintaining the present system as a carrier of street water only, and constructing a separate system of sewers to parallel the existing sewers for the purpose of intercepting all house laterals, and carrying sewage and storm run-off from roofs and other areas.

Method no. 3 considered constructing combined system interceptors along both banks of Rock Creek, north from Potomac River, to remove all District of Columbia sewage from Rock Creek, including all storm run-off from all combined sewer districts.

Method no. 4 considered extending the Piney Branch trunk to Rock Creek, thereby removing pollution from Piney Branch only.

Still another method may be considered. This comprises the relief of the existing Rock Creek and Piney Branch interceptor and the Rock Creek main interceptor below it, by diverting a part or the whole of the sewage and storm run-off from sewer district B–8 at a point near Sixteenth Street NW. into a new relief sewer to extend from the lower end of the Piney Branch trunk sewer to the Potomac River (p. 37).

Considerable study will be required to determine the most appropriate slope and size for such a relief sewer. The minimum would seem to be a sewer 7 feet in diameter having a slope of 0.005. The outlet of this sewer at the Potomac River could be placed at such an elevation as to be submerged at all stages of the river. The sewer would discharge into the Potomac River in times of storm, but at no other time, and the quantity of storm sewage so discharged would be substantially the same as that discharged through the intercepting sewer overflows from the districts to be served by the relief sewer, and which now flows by way of Piney Branch and Rock Creek into the Potomac River. The flow of the Potomac River is so large that it would carry away quickly any such storm discharge. The cost of this relief sewer is estimated roughly to be $1,250,000 (p. 39).

A modification of the plan for a relief sewer would be to build such a sewer of sufficient size to carry the entire storm sewage discharge from sewer district B–8 (Piney) plus the sewage pumped during storms from the Rock Creek main interceptor. Such a relief sewer would have to be large enough to carry the run-off from the maximum rate of precipitation for which allowance should be made under conditions of ultimate development.

Such a sewer might be about 15 feet in size with a slope of 0.005 and a capacity of 3,400 cubic feet per second (2,200 million gallons daily). The cost of such a sewer is estimated at $2,800,000 (p. 40).

In our opinion, the large expense required for the elimination of storm sewage discharged from sewer district B–8 (Piney) is not justified (p. 41).

The cost of the pipe line from the Lydecker tunnel to Piney Branch at Sixteenth Street is estimated roughly at $250,000 (p 43).

7. *The Nagle report, 1934.*—Under date of November 7, 1934, Mr. John L. Nagle, assistant chief, branch of engineering of the National Parks Service, transmitted a report on improvement and cleaning up of Rock Creek. The report discusses streamflow regulation and intercepting sewers as possible means of improving the present objectionable conditions and pertinently states that various such schemes and combinations thereof have been given some thought, and tentative estimates set up. None of these studies or estimates is mature enough to afford a suitable basis for approval, or even for the allotment of funds for the accomplishment of the work.

Mr. Nagle refers to some of the proposed remedial plans, their costs and limitations. He discusses the Rock Creek problem as a Federal project.

Because Rock Creek Park, including practically the entire valleys of Rock Creek and its main tributaries in the District of Columbia, is under the National Park Service, it is recommended that the work be entrusted to the National Park Service, and that funds for the accomplishment of the work be allotted to the National Park Service.

8. *The valley line report, 1934.*—In a memorandum, dated November 30, 1934, Mr. C. McDonough, director of engineering, discusses the Eddy, Gregory, and Greeley recommendations and alternative plans insofar as the limited time would permit.

Our review of the problem leads us to the conclusion that the expenditure necessary to effect the ideal condition is not justified, at least, at present, and suggest the following solution which it is believed will adequately meet the requirements.

The salient facts of the plan are as follows:

It is proposed to construct a relief sewer of a size which will take all overflow from the present existing interceptors, except for from 5 to 10 days in the years of average rainfall. This relief sewer will begin at the Piney Branch sewer outlet and run continuously and increasing in size to its terminus in the Potomac River at F Street. At approximately five points on this relief sewer overflow outlets will be constructed. These outlets will be provided with a bar screen to remove all objectionable floating debris which might otherwise be discharged into Rock Creek. Provision would be made for chlorinating the overflow if necessary.

The cost of the plan herein recommended is roughly estimated at $2,000,000.

It is further proposed, as would be necessary in any plan, to convert the combined systems to separate systems in both the Klingle and Luzon areas. These conversions are estimated to cost $125,000 and $350,000, respectively.

Briefly, the reasons for the adoption of the plan herein recommended may be summarized as follows:

(*a*) It will divert from Rock Creek the first flushing of the present combined sewers, which is the chief sources of the nuisance, and will permit discharge of diluted sewage into the creek only at times when the creek would be in high stages.

(*b*) All overflow into the creek will be screened to eliminate objectionable solids and floating debris.

(*c*) The structures may be extended and paralleled by a sewer on the opposite side of the creek to effect 100 percent diversion if such treatment of the problem is found desirable in the future.

9. Your consulting engineers have considered and utilized many important and valuable features contained in the foregoing reports. The principal effort

remaining for your consulting engineers has been the derivation of actual run-off figures essential for estimating the relative merits of the several possible projects.

The development of a proper sanitary condition along Rock Creek will clearly require that no appreciable polluted flow from any sewer system be permitted to discharge into Rock Creek. The actual standard, or definition of appreciable pollution, as used here, is discussed elsewhere. An extreme corrective measure that might be contemplated would involve the prohibition of any flow into Rock Creek from the combined sewers and, consequently, the construction of a combined outlet sewer from Piney Branch to the Potomac River. Such a sewer would be of tremendous size and of a cost for which there could be no sanitary or economic justification. Between the extremes of the present inadequate intercepting system and the construction of a complete outlet sewer of great size, it is the aim of these particular studies to develop a project involving additional intercepting sewer capacity of such amount that all appreciably polluted flow will be carried independently to the Potomac, and all relatively unpolluted discharge from the sewers in excess of the capacity so provided to be permitted to discharge into Rock Creek.

In the development of this problem, it is obvious that some relation must be determined between the rate of flow from the sewers and the extent of pollution. Such a study is undertaken, and constitutes an important part of this report. With this study as a background, it is further important that the relative frequency of the discharge of particular quantities of flow from the subbasins be known. Such a study involves, first, a knowledge of the relative frequency of occurrence of specific rainfall rates; second, of the characteristics of the surface development of the various subbasins in order that coefficients may be applied to the rainfall rates, resulting in definite values of runoff rate; and, finally, the tributary area of each subbasin must be determined in order that, through the application of run-off rates to the area in the basin, discharges of determined frequency may be tabulated.

Basic data essential to such a study involve the determination of the relation between rainfall intensity and frequency for the various duration periods that are found to be critical for the particular draingage basins. It also involves the study of surface in various basins, resulting in the determination of values of percentage of impervious surface. The results of theses tudies are only applicable to a final recommendation through an exercise of judgment, after a thorough understanding of the conditions of the various sewer systems and their characteristics has been acquired.

Chapter V

BASIS OF DESIGN

RATES OF STORM RUN-OFF FROM COMBINED SEWERS

1. A preliminary study confirmed the conclusion drawn from some of the more recent reports and correspondence that by far the greater part of the pollution of Rock Creek and the insanitary conditions in Rock Creek Park results from the overflow of sewage at the outlets of the combined sewer systems serving subbasins. It is, accordingly, clear that any recommendations for removal of pollution must be based on a relatively intimate knowledge of the characteristics of these subbasins and of the systems of sewers serving them.

2. The development of the necessary information involved the collection of data from the existing records of the sanitary engineer's office of the District of Columbia and in the gathering of this information, consulting engineers and the officials of the National Park Service had complete cooperation throughout from the office of the sanitary engineer.

It was found that although systems of combined sewers in the various subbasins located in the lower part of Rock Creek Valley were constructed many years ago, the actual physical records of these sewers in the District office were, in general, quite complete. As basic data in the study to be undertaken, there were secured from the sanitary engineer's office copies of record plats showing the location of all such tributary sewers. These plats, in addition to showing the location of the sewers, generally showed with fair accuracy the location of manholes, the size of sewers, the elevation of the flow line or sole of the sewer, the elevations of street grades at critical points, and the location of storm-water inlets. In addition to this general record information, of which a large mass had to be assembled in order to accurately cover the area involved, there was also furnished some separate record of special sewer structures, such as overflows, regulator chambers, and sewer junctions.

There was also secured from the sanitary engineer's office, very complete information as to the location, size, and grade of the Rock Creek and Piney Branch intercepting sewers.

There were not available, in the records of the District, any computation sheets showing the original basis of design for any of these sewers, or any means of determining, except by extensive recalculation, the drainage area tributary to any of the sewers at a particular point.

3. *Existing sewer capacities.*—As it is obvious that even a preliminary study of the character here carried out must be based on a fairly accurate idea of the characteristics of the tributary sewers, it has been necessary to reproduce to some extent the form of calculation usually involved in the design of such systems, as from such a calculation only is it possible to determine (*a*) the actual capacities of the sewers themselves, and (*b*) the probable discharge of storm water with reference to critical frequency of occurrence. As an example of the type of calculation involved there is attached hereto table 1, covering the Piney Branch drainage basin. This particular table was prepared at an early stage of the studies undertaken to give a preliminary idea as to the adequacy of the main sewers of the Piney Branch system. In order to develop it, it was necessary to recalculate the subareas draining to each of the critical points in the sewer system that are listed on the horizontal lines in the table. In addition to the drainage area involved, the calculation also required the use of run-off coefficient in cubic feet per second per acre, which, in accordance with common practice, is decreased somewhat with the increase in the elapsed time of flow through the sewer system. This factor designated in the table by the symbol pI, represents a rainfall rate in inches per hour modified by percentage of run-off. For this preliminary calculation, a varying pI factor was taken which would normally be representative of a rainfall frequency of once in 10 years and a surface development involving approximately 45 percent impervious area. Actually, the later studies of the conditions in this drainage area and of rainfall frequency at Washington showed that the factors used were more nearly representative of a 4-year rainfall frequency for this particular basin. For comparison see diagram figure 8.

In this preliminary study, the capacities of the sewers were determined by the ordinary hydraulic formula, using the grades and sizes given in the records and a coefficient of roughness or n, 0.013. Later investigation in the field indicated that this is a conservative value for the whole sewer, although much of it is one or two points smoother.

4. *Sur-charge.*—This preliminary recalculation showed clearly that the sewers of the Piney Branch system are inadequate on the basis of accepted prac-

tice in combined sewer design; that while the capacity is being supplemented by the construction of a relief sewer in that part of the system between Fifth and Ingram Streets and Arkansas and Iowa Avenues (which relief sewer will add about 700 second-feet to the main trunk capacity between those points), nevertheless, the main trunk sewers will be overcharged throughout the greater part of their length, and the main trunk sewers in the lower part of the valley, particularly below Fourteenth Street, will be overcharged in the amount of 50 to 100 percent by the stormflows that may be expected about once in 4 years. This condition had a very direct bearing on some of the studies of more frequent stormflow discharge, described in a later chapter.

Table I also discloses other important characteristics of this particular drainage basin; as, for example, the relatively short time of flow through this system (33 minutes). This short time of flow for a drainage basin as large as 2,300 acres would indicate a quick or flashy reaction and sudden outflow as a result of any appreciable rainfall.

5. *Slopes.*—The slope column shows a badly varying condition of sewer grades and this in turn is reflected in the velocity column by a widely varying set of normal velocities, the velocity reaching 18 feet per second in the upper part of the basin, reducing to 6 feet in the middle, and varying from 12 to nearly 40 feet per second in the lower reaches. These velocities, of course, are theoretical and are those which would exist if each particular section of sewer were able to flow at the rate normal to the grade on which it is constructed. Actually, the interrelation of one sewer section to the corresponding sections above and below would appreciably modify the particular values shown. The general effect, however, is to produce a flow through the sewer system which will not be smooth or relatively uniform under any conditions, but will be interspersed with draw-down effects, hydraulic jumps, and large losses because of impact and turbulence.

6. *Coefficient of roughness.*—In order to secure a reasonable idea of the actual roughness coefficient in the sewers as constructed, an inspection of the interior surface condition of these sewers was made on several occasions at a number of points. An inspection of the Piney Branch main sewers and principal junctions developed the information shown in the following report by Mr. Chivvis. While this report calls attention to minor unsatisfactory conditions at a number of points and to the unsatisfactory junction angles of the incoming lateral sewers, it shows that, in general, the interior finish of the sewers, is above the average, that a coefficient of n, 0.012 would be applicable to considerable lengths of sewer, of n, 0.015 to a few short sections, and that an average of n, 0.013 could be accepted for the purposes of these studies.

PINEY BRANCH INSPECTION

At side manhole in Arkansas Avenue, just south of Varnum Street. $n=0.012$ both up and down stream. Flow very swift.

At side manhole in Arkansas Avenue at Allison Street. Cement spilled by manhole masons covers side of invert. An 18-inch lateral at the manhole and a 12-inch, some 50 feet above, are unclipped and project into the sewer 5 or 6 inches on the upstream side. $n=0.013$ upstream and down.

At side manhole in Arkansas Avenue at Buchanan Street. Cement spilled on side of invert under manhole. Laterals enter almost at right angles, one about 30 feet below manhole projects a few inches into main sewer barrel. $n=0.012$ above. $n=0.013$ below manhole.

Side manhole at south line of Iowa Street broken into sewer after construction. Twelve inch lateral just below manhole projects into sewer. Except for this, $n=0.012$.

Side manhole at north line of Iowa Street broken into sewer after construction. $n=0.012$.

New relief

Side manhole 50 feet north of north line of Iowa Street. Cement spilled on invert under manhole. Except for this, $n=0.012$. This manhole is 50 feet below junction of new relief sewer which is a very smooth piece of work spoiled by concrete spilled on the brick invert. Laterals come in at right angles, having been bricked in as intercepted instead of taking up part of lateral and relaying on a curve. As it is, $n=0.014$. Could be cleaned up to 0.11. (Allow head loss at laterals if 0.011 is used in computations.) The old sewer above the relief junctions is not as smooth as the relief. $n=0.012$ as far as could be seen upstream. All the circular sewer below this junction, as well as the old sewer just above, has a 120° brick bottom. The relief is brick to springline.

Deep side manhole in Ninth Street south of Gallatin Street. Manhole at center of reverse curve formed by 16-foot chords, which is typical of the curves above here. Cement spilled on invert beneath manhole. Sewer here and above is built of brick to spring line. Leakage at this spring-line joint has formed hard deposits down the brick invert. $n=0.015$.

Side manhole in sidewalk of Hamilton Street east of Eighth Street. Curve below manhole is built with 16-foot chords. Leakage from spring-line joint along south wall has formed deposits on brickwork. $n=0.013$.

Manhole in Longfellow Street south of Fourth Street. Cement spilled beneath manhole. Straight chord curve below manhole. Well-built junction chamber above. $n=0.015$ because of deposits and rough work.

Manhole on Upshur Street lateral in hospital grounds, near Fourteenth and Upshur Streets. $n=0.013$.

In general this is a high-class sewer, smooth and well built. It will average better than $n=0.013$ as is. Unfortunately, the footing is too slippery to permit walking between manholes.

7. *Rock Creek interceptor.*—As the capacity of the existing Rock Creek and Piney Branch interceptors is one of the most important basic values used, it is essential that it be known within fairly close limits of accuracy. The profile of this sewer, like that of some of the main combined sewers above referred to, is badly broken from point to point, steep grades and flat grades being generally interspersed. For this reason, it did not appear to be satisfactory to rely entirely on calculated capacities, and a field investiga-

TABLE NO. 1
PINEY BRANCH TRUNK SEWER
PRELIMINARY HYDRAULIC INVESTIGATION

LOCATION OF LINE	AREA NOS.	ACREAGE		% IMP.	RUNOFF		SLOPE %	SIZE	CAPAC. C.F.S.	LENGTH	VEL. F.P.S.	TIME	
		ADDED	TOTAL		C.F.S. PER AC.	TOTAL C.F.S.						ADD.	TOTAL.
IN CEDAR ST., R.R. TO 4TH	1	4.6	4.6	45	2.4	11	1.50	3' ϕ	83	200'	11.7	0.3	10.0'
IN 4TH ST., CEDAR TO ASPEN	2	15.1	19.7	"	"	47	1.67	3' ϕ	86	800'	12.0	1.1	10.3'
IN ASPEN, 4TH TO 3RD	3	27.1	46.8	"	"	112	1.10	$3\frac{1}{2}$' ϕ	107	580'	11.2	0.9	11.4
IN 3RD, ASPEN TO WHITTIER	4&5	41.9	88.7	"	"	213	3.30	4' ϕ	263	370'	21.0	0.3	12.3
" ", WHITTIER TO VAN BUREN	7	47.6	136.3	"	"	327	2.70	5' ϕ	439	480'	22.0	0.4	12.6
" ", VAN BUREN TO UNDERWOOD	6	16.4	152.7	"	"	366	1.08	$5\frac{1}{2}$' ϕ	356	480'	15.0	0.5	13.0
" ", UNDERWOOD TO RITTENHOUSE C. G. 54	8	73.2	225.9	"	"	542	0.82	$6\frac{1}{2}$' ϕ	483	1250'	14.6	1.4	13.5
" ", RITTENHOUSE TO QUACK. C. G. 52	9	31.8	257.7	"	"	619	0.82	$6\frac{1}{2}$' ϕ	483	480'	14.6	0.5	14.9
" ", QUACK. TO PEABODY	10	77.4	335.1	"	"	805	1.14	$7\frac{1}{4}$' ϕ	700	400'	17.0	0.4	15.4
3RD & PEABODY TO 3RD & OGLETHORP.	11	56.7	391.8	"	"	941	1.14	$7\frac{1}{2}$' ϕ	815	700'	18.0	0.7	15.8
IN 3RD PL., OGLETHORPE TO LONGFELLOW C. G. 49	12	45.1	436.9	"	"	1047	0.75	8'	800	1180'	16.0	1.2	16.5
DIAGONALLY ACROSS LONGFELLOW	13&14	135.1	572.0	"	"	1370	0.57	$8\frac{1}{2}$'	825	217'	14.5	0.3	17.7
IN 4TH ST., LONGFELLOW TO JEFFERSON	15	78.7	650.7	"	"	1560	0.40	9'	800	700'	12.6	0.9	18.0
4TH & JEFF. TO 5TH & INGRAHAM * C. G. 42	16	17.0	667.7	"	"	1600	0.40	9'	*800	930	12.6	1.2	18.9
IN 5TH ST., INGRAHAM TO HAMILTON	17	38.3	706.0	"	2.31	*1630	0.10	9'	400	430'	6.3	1.1	20.1
IN HAMILTON, 5TH TO 7TH	18	135.0 135.0	841.0 135.0	45% 10%	2.28 1.52	*2120	0.10	$9\frac{1}{4}$'	430	620'	6.4	1.6	21.2
7TH & HAMILTON TO ILLINOIS & GALLATIN	19	45.5 0.0	886.5 135.0	45% 10%	2.21 1.50	*2160	0.10	$9\frac{1}{4}$'	430	1000'	6.4	2.6	22.8
ILLINOIS & GALLATIN TO S. SIDE GEORGIA	20& 21A	191.0 22.0	1077.5 157.0	45% 10%	2.11 1.47	*2506	0.10	$9\frac{1}{4}$'	430	750'	6.4	2.0	25.4
GEORGIA TO FARRAGUT & ARKANSAS		0 0	1077.5 157.0	45% 10%	2.04 1.44	*2506 SHOULD BE COMBINED.	2.32	$9\frac{1}{4}$'	2100	209'	31.0?	0.1	27.4
FARRAGUT & ARK. TO DELAFIELD PL.	21 B	232.5 0	1310.0 157.0	45% 10%	2.04 1.44	*2900	0.61	$9\frac{1}{4}$'	1050	776'	15.5	0.8	27.5
IN ARKANSAS, DELAFIELD TO IOWA *	22	89.4 0	1399.4 157.0	45% 10%	2.00 1.42	*3023	0.38	$9\frac{1}{2}$'	*900	744	12.8	1.0	28.3
" ", IOWA TO BUCHANAN		0 0	1399.4 157.0		1.98 1.40	3023	1.10	$9\frac{1}{2}$'	1560	236'	20'±	0.2	29.3
" ", BUCHANAN TO WEBSTER	23	56.1 0	1455.5 157.0		1.98 1.40	3100	1.24	$9\frac{1}{2}$'	1480	900'	← MANNING'S FORMULA USED BELOW THIS PT. 20.8	0.7	29.5
" ", WEBSTER TO GRADE BREAK			1455.5 157.0		1.95 1.40	3100	1.84	$9\frac{1}{2}$'	1960	200'	27.6	0.1	30.2
" ", GRADE BREAK TO 14TH			1455.5 157.0		1.95 1.40	3100	3.71	$9\frac{1}{2}$'	2780	100'	39.4	0.04	30.3
ACROSS 14TH TO STA. 25+04	24	46.1 0	1501.6 157.0		1.95 1.40	3140	1.46	$9\frac{3}{4}$'	1900	293'	24.9	0.2	30.3
TO STA. 20+26	25 26 26A	25.4 87.2 158.5	1752.7 157.0	1909.7 SCALED. 1896.0 Act.	1.93 1.38	3600	0.96	$9\frac{3}{4}$'	1550	478'	20.2	0.4	30.5
TO STA. 17+76	27	26.7	1779.4 157.0		1.93 1.38	3650	2.10	$9\frac{3}{4}$'	2300	250'	30.+	0.1	30.9
TO STA. 8+12	28, 29, 30, 31	97.3	1876.7 157.0		1.90 1.38	3660	1.03	$9\frac{3}{4}$'	1600	964'	20.9	0.8	31.0
TO STA. 3+25		†360.8	2237.5 157.0	SCALED	1.90 1.38	4470	1.57	$9\frac{3}{4}$'	1980	487'	26.0	0.3	31.8
OUTFALL			2169.7 157.0	ACTUAL †	1.90 1.38	4340	1.70	10'×10'	2000	325	23.6	0.2	32.0

* Between 5th & Ingraham and Arkansas & Iowa, relief line will take about 700 c.f.s. which will reduce the flow in the main line between these points by this amount.

† This acreage taken from table supplied with cup gauge records.

⬭ Denotes overcharge.

n = .013 used for all of these computations.

tion was indicated. This field investigation involved a determination by inspection and judgment of the proper *n* value for this sewer, and a corresponding memorandum was prepared by Mr. Chivvis which indicated that the visible interior surfaces varied between 0.012 and 0.015 and that a value of 0.013 could be safely used throughout. (See Appendix.)

8. In order to determine the reaction of the various sewer sections on each other, an actual study of the water surface profile of the Rock Creek interceptor was carried out in the field. In this study, a stream of water from Rock Creek was turned into the intercepting sewer and the water level was observed at a number of manholes through its length. The flow was increased step by step until the sewer was running full at certain points found to be critical, and the depth of flow at other points was then recorded. An analysis of the information gained in the field indicated what particular sections of the sewer were appreciably below the average capacity of the sewer as a whole. A further study was undertaken of the possibilities of increasing the usable capacity by reconstructing the intercepting sewer at certain of these "choked" points, and the possible capacity of each intercepting sewer after such reconstruction is shown in the last column of table 2, 2*a*, 2*b* and 2*c*. The capacity so determined was necessarily an essential item of basic data in the later studies involving the economic additional capacity to be provided.

It was later determined that the capacity of the Rock Creek interceptor, even after being brought up to its best value, is quite small compared to the intercepting capacities that appear to be necessary to the recommended project. In the final disposition of this matter, it is proposed that the existing Rock Creek interceptor be relieved to the greatest possible degree by the new sewers proposed, and that thereafter it be treated primarily as a sanitary outlet sewer for the separate sewer districts above Piney Branch Parkway and for the interception of a certain proportion of the flow from a few small districts adjacent to Rock Creek, just below Piney Branch Parkway. The amounts taken from these districts are clearly shown on the maps, profiles, and design tables.

9. The sewage sampling studies (covered in chapter II) indicate high degrees of pollution for low storm flows out of the combined sewer outlets. There seemed to be no doubt that these combined sewers, possibly largely in the house connections, were acting to a considerable extent as settling basins during dry weather, and that the organic matter so settled out was later scoured and dispersed by the beginning of the storm flow. It was clear that complete sanitation could not be achieved unless the proposed relief sewers were designed for a very considerable flow of storm water; in fact, that these relief sewers would actually be designed as storm sewers of limited capacity. For this reason, it was immediately important to have an approximate idea of the relation between rainfall and run-off for storms occurring on the average of a number of times per annum, and that a sound economic recommendation could not be made until a preliminary determination of the relation between storm outflow and frequency has been made.

10. *Basic data.*—An examination of existing data indicated two possible approaches in the development of this relationship: (*a*) Through a study of actual flow in the larger combined sewers, made possible by the fact that the District of Columbia had established in the sewers certain cup gages capable of recording maximum water levels during particular storms. (*b*) Through a study of the flow in Rock Creek itself as indicated by gaging studies at Sherrill Drive and Q Street, carried on by the United States Geological Survey.

The studies actually made with the use of each of these sets of data are covered in detail in the subsequent chapters.

TABLE 2.—*Final condition of Rock Creek main interceptor after relief under tunnel plan*

Line	Increment (acres)	Run-off (cubic feet per second)	Total	Existing sewer, $n=0.013$			
				Size	Slope	Velocity	Capacity
Headwaters to Rockville							
Rockville to suburban sanitary district.							
Suburban sanitary district to District of Columbia line.	11,190 sanitary from suburban sanitary district.	11,190 sanitary at 0.0027	30				
District of Columbia line to Pineburst Parkway (9,854 feet of sewer to be rebuilt or relieved).	428 sanitary from Blair portal, etc.	428 sanitary at 0.008 11,190 sanitary at 0.0027	33	3' 0" diameter	0.0013	3.4	24
Pineburst Parkway to Luzon Valley (5,200 feet of sewer to be rebuilt or relieved).	412 sanitary from Pineburst Parkway.	840 sanitary at 0.008 11,190 sanitary at 0.0027	37	2' 0" by 2' 6" 2' 20" diameter	.0015 .00188	3.3 2.7	14 12
Luzon Valley to Broad Branch [1]	621 sanitary from Luzon Valley	1,461 sanitary at 0.008 11,190 sanitary at 0.0027	42	3' 6" by 5' 0" 4' 6" diameter	.0036 .001	7.2 4.0	103 63
Broad Branch to Klingle Road	1,858 sanitary from Soap stone Valley. Broad Branch and Blagden Avenue.	3,319 sanitary at 0.008 11,190 sanitary at 0.0027	57	4' 6" diameter 4' 6" by 5' 0"	.0018 .0018	5.3 5.3	84 101
Klingle Road to Connecticut Avenue.[1]	544 sanitary from Klingle Valley	3,319 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	63	5' 4" by 5' 0" 4' 3" diameter	.0018 H. g. .00135	5.9 4.6	126 63
Connecticut Avenue to Massachusetts Avenue (rebuild 185 feet of constricted section under Massachusetts Avenue).		3,319 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	63	4' 3" diameter 4' 3" diameter constricted to 8.17 square feet.	H. g. .00135 (0.002)	4.6 (4.3)	63 (35)
Massachusetts Avenue to proposed new creek crossing.	56 sanitary from Massachusetts Avenue.	3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	64	4' 3" diameter	H. g. .00135	4.6	64
New creek crossing (inverted siphon to replace present creek crossing).		3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	64	4' 6" diameter	[2]	4.0	64
New creek crossing to junction (270 feet of new 4' 6" sewer).		3,375 sanitary at 0.008 544 sanitary at 0.012 11.190 sanitary at 0.0027	64	4' 6" diameter	H. g. .001	4.0	64
Junction to northwest boundary	37.6 combined—16.8 sanitary from Piney Branch interceptor.	38 combined at 1.25 16 sanitary at 0.05 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	111	5' 0" by 5' 0"	H. g. .0014	5.3	111
Northwest boundary to O Street east.		38 combined at 1.25 16 sanitary at 0.05 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	111	6' 0" diameter	H. g. .00067	4.0	111
O Street east to N Street	25 combined from O Street	25 combined at 1.00 38 combined at 1.25 16 sanitary at 0.05 ,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	136	6' 0" diameter	H. g. .001	4.8	136
N Street to M Street	15 combined from N Street (Pennsylvania Avenue area).	40 combined at 1.00 38 combined at 1.25 16 sanitary at 0.05 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	151	6' 0" diameter	H. g. .00125	5.4	151
M Street to K Street	23 combined from M Street (Pennsylvania Avenue area).	63 combined at 1.00 38 combined at 1.25 16 sanitary at 0.05 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	174	6' 0" diameter	H. g. .0017	6.2	174
K Street to G Street	Pumping from K Street pumping station.[3] 14 combined from K Street (Pennsylvania Avenue area).	3 pumps at 13.0 77 combined at 1.00 38 combined at 1.25 16 sanitary at 0.05 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	227	6' 0" diameter	H. g. .0028	8.1	227

[1] It is assumed that there will be complete separation of all roof and yard drains in the Klingle and Luzon districts.
[2] Loss of head in siphon 0.5, h. g. 0.0025.
[3] 30 cubic feet per second from present drainage area plus 9 cubic feet per second from proposed west side relief sewer.

REMARKS

No addition for Connecticut Avenue, Cleveland Avenue, Normanstone Drive, or Montrose districts.
Float regulator assumed at Normanstone for the 3 districts.
Float regulators at L Street and I Street will divert all flow from old interceptors during storms.

TABLE 2a.—*Final condition of Rock Creek main interceptor after relief under valley plan*

Line	Increment (acres)	Run-off (cubic feet per second)	Total	Existing sewer, $n=0.013$			
				Size	Slope	Velocity	Capacity
Headwaters to Rockville							
Rockville to suburban sanitary district.							
Suburban sanitary district to District of Columbia line.	11,190 sanitary from suburban sanitary district.	11,190 sanitary at 0.0027	30				
District of Columbia line to Pinehurst Parkway (9,854 feet of sewer to be rebuilt or relieved).	428 sanitary from Blair Portal, etc.	428 sanitary at 0.008 11,190 sanitary at 0.0027	33	3′ 0″ diameter	0.0013	3.4	24
Pinehurst Parkway to Luzon Valley (5,200 feet of sewer to be rebuilt or relieved).	412 sanitary from Pinehurst Parkway.	840 sanitary at 0.008 11,190 sanitary at 0.0027	37	2′ 0″ by 2′ 6″ 2′ 20″ diameter	.0015 .00188	3.3 2.7	14 12
Luzon Valley to Broad Branch [1]	621 sanitary from Luzon Valley	1,461 sanitary at 0.008 11,190 sanitary at 0.0027	42	3′ 6″ by 5′ 0″ 4′ 6″ diameter	.0036 .001	7.2 4.0	103 63
Broad Branch to Klingle Road	1,858 sanitary from Soapstone Valley, Broad Branch and Blagden Avenue.	3,319 sanitary at 0.008 11,190 sanitary at 0.0027	57	4′ 6″ diameter 4′ 6″ by 5′ 0″	.0018 .0018	5.3 5.3	84 101
Klingle Road to Connecticut Avenue.[1]	544 sanitary from Klingle Valley	3,319 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	63	5′ 4″ by 5′ 0″ 4′ 3″ diameter	.0018 H. g. .00135	5.9 4.5	126 63
Connecticut Avenue to Massachusetts Avenue (rebuild 185 feet of constricted section under Massachusetts Avenue.		3,319 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	63	4′ 3″ diameter 4′ 3″ diameter Constricted to 8.17 feet.[2]	H. g. .00135 (.002)	4.5 (4.3)	63 (35)
Massachusetts Avenue to new creek crossing.	56 sanitary from Massachusetts Avenue.	3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	64	4′ 3″ diameter	H. g. .0014	4.6	64
New creek crossing (inverted siphon to replace present creek crossing).		3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	64	4′ 6″ diameter	H. g. .001	4.0	64
New creek crossing to junction (270 feet of new 4-foot 6-inch diameter sewer).		3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	64	4′ 6″ diameter	H. g. .001	4.0	64
Junction to northwest boundary	18 combined from Piney Branch interceptor.	18 combined at 1.25 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	86	5′ 0″ by 5′ 0″	H. g. .00085	4.1	86
Northwest boundary to O Street east		18 combined at 1.25 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	86	6′ 0″ diameter	H. g. .0004	3.1	86
O Street east to N Street	25 combined from O Street	18 combined at 1.25 25 combined at 1.00 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	111	6′ 0″ diameter	H. g. .0007	4.0	111
N Street to M Street	15 combined from N Street (Pennsylvania Avenue area).	18 combined at 1.25 40 combined at 1.00 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	126	6′ 0″ diameter	H. g. .00083	4.4	126
M Street to K Street	23 combined from M Street (Pennsylvania Avenue area).	18 combined at 1.25 63 combined at 1.00 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0027	149	6′ 0″ diameter	H. g. .0012	5.3	149
K Street to G Street	Pumpage from K Street pumping station.[2] 14 combined from K Street (Pennsylvania Avenue area).	3 pumps at 13.00 18 combined at 1.25 77 combined at 1.00 3,375 sanitary at 0.008 544 sanitary at 0.012 11,190 sanitary at 0.0021	202	6′ 0″ diameter	H. g. .0021	7.2	202

[1] It is assumed that there will be complete separation of all roof and yard drains in the Klingle and Luzon districts.
[2] 30 cubic feet per second from present drainage area plus 9 cubic feet per second from proposed west side relief sewer.

REMARKS

No addition for Connecticut Avenue, Cleveland Avenue, Normanstone Drive, or Montrose districts.
Float regulator assumed at Normanstone for the 3 districts.
Float regulators at L Street and Eye Street will divert all flow from old interceptor during storms.

TABLE 2b.—*Final condition of Piney Branch interceptor under tunnel plan*

Line		Increment (acres)	Run-off (cubic feet per second)		Total run-off	Existing sewer, n=0.013			
From—	To—		Area	Unit		Size	Slope	Length	Capacity
									Cubic feet per second
Oak Street	Eighteenth Street					2¾ by 4⅛	0.013	470	89
Eighteenth Street	Ingleside Terrace					2¾ by 4⅛	.017	450	102
Ingleside Terrace	Park Road					2¾ by 4⅛	.017	830	102
Park Road	Pierce Mill Road	6.2 sanitary from Park Road	6.2	0.05	0.3	2¾ by 4⅛	.01	450	78
Pierce Mill Road	Lamont Street	4.0 sanitary from Pierce Mill Road.	10.2	.05	.5	2¾ by 4⅛	.002	2,710	35
Lamont Street	Kenyon Street		10.2	.05	.5	2¾ by 4⅛	.002	630	35
Kenyon Street	Irving Street		10.2	.05	.5	2¾ by 4⅛	.002	260	35
Irving Street	Belmont Street	19.2 combined from Quarry and Ontario Roads.	19.2 10.2	1.25 .05	25	2¾ by 4⅛	.002	4,090	35
Belmont Street	Belmont Road (1)	6.6 sanitary from Allen Place	19.2 16.8	1.25 .05	25	2¾ by 4⅛	.002	855	35
Belmont Road (1)	Belmont Road (2)	7.6 combined from Belmont Road (1).	26.8 16.8	1.25 .05	34	2¾ by 4⅛	.002	1,110	35
Belmont Road (2)	Junction	10.8 combined from Belmont Road (2).	37.6 16.8	1.25 .05	48	2¾ by 4⅛	H. g.. 0037	1,780	48

TABLE 2c.—*Final condition of Piney Branch interceptor under the valley plan*

Line	Increment, acres	Run-off (cubic feet per second)		Total run-off	Existing sewer n=0.013			
		Area	Unit		Size	Slope	Length	Capacity
Oak Street to Eighteenth Street					2¾′ by 4⅛′	0.013	470	89
Eighteenth Street to Ingleside Terrace					2¾′ by 4⅛′	.017	450	102
Ingleside Terrace to Park Road					2¾′ by 4⅛′	.017	830	102
Park Road to Pierce Mill Road					2¾′ by 4⅛′	.01	450	78
Pierce Mill Road to Lamont Street	4.0 sanitary from Pierce Mill Road	04.0	0.05	0.2	2¾′ by 4⅛′	.002	2,710	35
Lamont Street to Kenyon Street		4.0	.05	.2	2¾′ by 4⅛′	.002	630	35
Kenyon Street to Irving Street crossing	Quarry and Ontario now enter here. Connection to be abandoned and all flow diverted to relief under valley plan.	4.0	.05	.2	2¾′ by 4⅛′	.002	360	35
Irving Street crossing to lower crossing (bulkhead below 2′ by 3′ cross connection at lower crossing and divert flow to main interceptor).		4.0	.05	.2	2¾′ by 4⅛′	.002	1,330	35
Lower crossing to Allen Place					2¾′ by 4⅛′	.002		
Allen Place to Belmont Road (1) remove interceptor below Calvert Street for construction of new relief sewer. Bulkhead below Calvert Street and above Allen Place.	6.6 from Allen Place	6.6	.05	.3	2¾′ by 4⅛′	.002	1,250	35
Belmont Road (1) to Belmont Road (2)	7.6 combined from Belmont Road (1)	7.6	1.25	10.0	2¾′ by 4⅛′	.002	1,110	35
Belmont Road (2) to junction	10.8 combined from Belmont Road (2)	18.4	1.25	23.0	2¾′ by 4⅛′	.002	1,780	35

Chapter V–A

STORM RUN-OFF BASED ON FLOW IN EXISTING SEWERS

1. The purpose of the studies outlined in this chapter is to determine the relation between rainfall on the combined-sewered area in Rock Creek and sewer discharge from these areas for storms occurring more frequently than once a year.

2. The information available to this study has been described in the first instance specifically with regard to the studies of the Piney Branch subbasin, in the preceding chapter, and may be summarized as follows:

(*a*) *Rainfall records.*—The information with regard to rainfall intensity in the Piney Branch Basin is of unusual adequacy due to the foresight of the District of Columbia in establishing recording rain gages at several points. The available gages are shown on the map, figure 1. The Piney Branch area is well covered between the gages at Takoma, at the Zoo and at Carnegie, the first two being the more valuable to the study. There was also available for substitution in certain instances, the records of the United States Weather Bureau gage at Twenty-second and M Streets, and this gage was particularly valuable in connection with the subsequent studies in the Slash Run Basin. In general, in the studies of the Piney Branch area, the 5-minute intensity values for the Takoma and Zoo gages were averaged as representative of the rain on the basin as a whole. In certain instances a weighted average was used.

(*b*) *Cup gages.*—Reference has been made to the available data with regard to sewer sizes and grades. The establishment of the cup gage installations made possible the determination of the maximum depth of flow in any particular storm within certain limits. These gages consisted of rods fixed in the sewer manholes, generally with their bottoms at about the midpoint of the vertical height of the sewer. Small conical cups were attached to these rods at 6-inch intervals. An examination of the cup gage after the rain determines the highest cup filled with water, and in that way fixes the maximum depth of flood flow during that rain as between the cup so filled and the next succeeding empty cup above.

3. In the use of this information, cup gages were chosen so located in the sewer as to be least affected by disturbed flow on account of the broken character of the sewer gradients. The sewer discharge was therefore computed from the height of water indicated by the upper wetted cup, this discharge in general being computed by the ordinary hydraulic formula, using a coefficient of n 0.013. In general, the discharge was computed using the flow line gradient of the sewer as constructed, although in certain instances this was modified where a water surface profile developed from adjacent cup gages indicated that the flow was not of uniform depth. In addition to the limitations due to disturbance of flow, the discharge values so computed are also possibly in error on account of the 6-inch intervals between cup-gage cups. The extent of this latter error has been tested by determining the effect on the ultimate storm factor, as between accepting the discharge as having been just level with the cup and, on the other hand, the possibility of its having been within half an inch of the succeeding cup. This test indicated that such a difference might introduce errors in the neighborhood of 10 percent. Such uncertainty unquestionably exists in the values developed, as hereafter described.

4. The studies of sewer flow, carried out in St. Louis and described in part in the "Proceedings" of the American Society of Civil Engineers, have indicated the entire propriety of attempting to compare such sewer discharges with average rates of rainfall for a basin of this kind. These St. Louis studies and the factors resulting from them, however, made possible the computation of a pluviagraph diagram for any particular rain, this diagram being essentially in the form of an hydrograph as it would exist for the specific rainfall intensities and the actual topography and time characteristics of the particular basin, except that the pluviagraph is an hydrograph only in pattern, its ordinates actually representing characteristics of outflow, without deduction of any losses from rainfall. It has been referred to as the 100-percent run-off hydrograph.

5. In the application of this method to Piney Branch, there were developed pluviographs for the particular rains for which cup-gage records were satisfactorily available. Such pluviographs were developed for the flow in the sewer at the location of the cup gage and were also developed for the probable flow in the sewer at its outfall at the Rock Creek end. The principle involved is that the discharge shown by the cup gage at a particular point, when compared with the peak of the pluviograph, or 100 percent run-off curve, may be used as a ratio or storm factor to develop an index relation between rainfall and run-off desired. The storm factor so developed at the cup gage may, with reasonable propriety, be considered as usable at the sewer outlet, and, when applied to the peak of the pluviograph at

that point, results in a calculated discharge value at the outlet of the sewer for the particular rain.

The application of this process in this way is, of course, open to a number of questions, and is subject to the same inaccuracies as are involved in all hydrologic computations where actual measurements of run-off over an extended period are not available. The propriety of the application has been tested by a brief study of the actual gagings along the main sewer in the Clarendon district of St. Louis, a drainage basin of characteristics similar to those of the Washington, D. C., sub-basins. This indicated that the storm factors for considerable variation of acreage in the upper end of the basin were in reasonable agreement.

6. The development of the pluviograph for the Piney Branch district followed the procedure that had been used in the analysis of the St. Louis gagings, although the procedure was simplified considerably as here used. The principal steps in the development may be briefly described as follows: By experience it was determined that a particular rain intensity lasting 1 minute would appear as flow in the sewer over a 20-minute period, reaching a maximum rate of flow at the end of 10 minutes. For example, the variation was taken along straight lines, as shown in figure 3. This triangular, 1-minute pluviograph was then developed into a pluviograph for 10-minute uniform intensity by means of a summation of ordinates for each minute taken separately. The 10-minute graph is also shown in figure 3. From this diagram there was taken off the ordinates of 5-minute intervals, and these ordinates were used as the basis of determining the flow at the end of each 5 minutes for a 10-minute rain of uniform intensity. The values of the ordinates are shown also in tabular form in table 3*a*.

7. *Time zones.*—In the application of this pluviagraph to the Piney Branch area, a study of the time of flow through the sewers was carried out, and the area was divided into 10-minute time zones on the basis of a time of flow above the outlet, or above the cup gage being studied, the actual position of the time zone lines being determined by adding to the time of flow through the sewer, an estimated flow time of 5 minutes on the surface from an extreme point in the particular subarea to the street inlet receiving water from that area. A time zone map for the Piney Branch area with zero at the outlet is shown herein as figure 4, and for further clarification there is included in tables 3*a* and *b*, a detail computation for one particular rain, carried through to the point of determining the estimated discharge at the sewer outlet.

8. For the Piney Branch sewer, six-cup gage readings and the corresponding rainfalls have been studied. From the records of depth of flow as shown by the cup gages, a calculation of the actual discharge or hydrograph peak at the particular point was made. Sketches showing the six rains and their corresponding pluviagraphs both at the gage and at the outfall, and the measured discharge as recorded by the cup gage, are shown on figure 5.

9. In table 4 is shown the result obtained for the six rains for which the cup gage records seem to be most acceptable. In this table has been shown the date of the rain, a rough picture of the rain pattern in the succeeding five columns, the pluviagraph peak at the cup gage in the succeeding column; then the calculated discharge from the cup gage record, then the storm coefficient derived from the two, and finally, the estimated discharge at the outlet, applying that coefficient to the pluviagraph for the full 2,327 acres. As information, two additional columns are added, giving the drainage area at the cup gage and the approximate condition of flow in the sewer at that point if full or partly full.

It will be noted that in most instances cup gage no. 29 has been used, as this gage seemed most nearly free from unstable flow conditions. For the rain of July 2, no record was obtained at that gage, but the indication was that the sewer should have been nearly half full. Consequently, in addition to using the next best gages—that is, 34 and 37—another pluviagraph has been inserted, based on the calculation that the sewer at gage 29 was approximately half full.

10. With the exception of the rain of September 12, 1934, the storm factor or run-off coefficient values are reasonably consistent. While an average would be a bit lower than 0.4, this figure is used in subsequent studies because of the inadequacy of the sewer at the present time and the possibility of its further relief in the future.

11. Pluviagraphs for 10 additional rains were developed and the storm factor of 0.4 was applied to arrive at an estimate of the discharge from these rains at the Piney Branch outfall. Table 5 gives these estimated run-offs for the 16 rains studied for the Piney Branch sewer. It should be noted that in this tabulation the storm factor 0.4 was applied also to the original six rains. The rainfall rates shown, where based on records from more than one station, were obtained by interpolation.

TABLE 3 A.—*Typical computations for 1 rain carried through to determination of the estimated discharge at Piney Branch outfall*

Ordinates to 10-minute unit graph. (See fig. 3.)	
Time in minutes	Ordinate in inches per hour
0	0.00
5	.15
10	.55
15	.75
20	.45
25	.10
29	.00

Application of 10-minute unit graph to rainfall of June 25, 1933. (See fig. 3.)

TABLE 3 A.—*Typical computations for 1 rain carried through to determination of the estimated discharge at Piney Branch outfall*—Continued.

Rainfall in inches per hour for each 10 minutes	0.78	1.15	2.75	0.60	0.44	100-percent run-off per acre in cubic feet per second
Time	4:40	4:50	5:00	5:10	5:20	5:30
4:40	0.00					0.00
4:45	.12					.12
4:50	.43	0.00				.43
4:55	.59	.15				.74
5:00	.35	.63	0.00			.98
5:05	.08	.86	.41			1.35
5:10		.52	1.51	0.00		2.03
5:15		.12	2.06	.09		2.27
5:20			1.24	.33	0.00	1.57
5:25			0.28	.45	.07	.80
5:30				.27	.24	.51
5:35				.06	.33	.39
5:40					.20	.20
5:45					.04	.04

TABLE 3 B.—*Typical computations for 1 rain carried through to determination of the estimated discharge at Piney Branch outfall*

[Computation of 100-percent run-off from area tributary to cup gage 29. (See fig. 3.)]

Time	100-percent run-off in cubic feet per second			
	For partial 20-minute zone, 507 acres	For 30-minute zone, 800 acres	For 40-minute zone, 117 acres	For total area tributary to cup gage 29, 1,424 acres
4:40	0			0
4:45	61			61
4:50	218	0		218
4:55	375	96		471
5:00	495	345	0	840
5:05	680	590	14	1,284
5:10	1,020	785	50	1,855
5:15	1,150	1,080	87	2,317
5:20	795	1,620	114	2,529
5:25	405	1,820	158	2,383
5:30	258	1,260	237	1,755
5:35	198	640	265	1,103
5:40	101	410	183	694
5:45	20	310	94	424
5:50		160	60	220
5:55		32	46	78
6:00			23	23
6:05			4	4

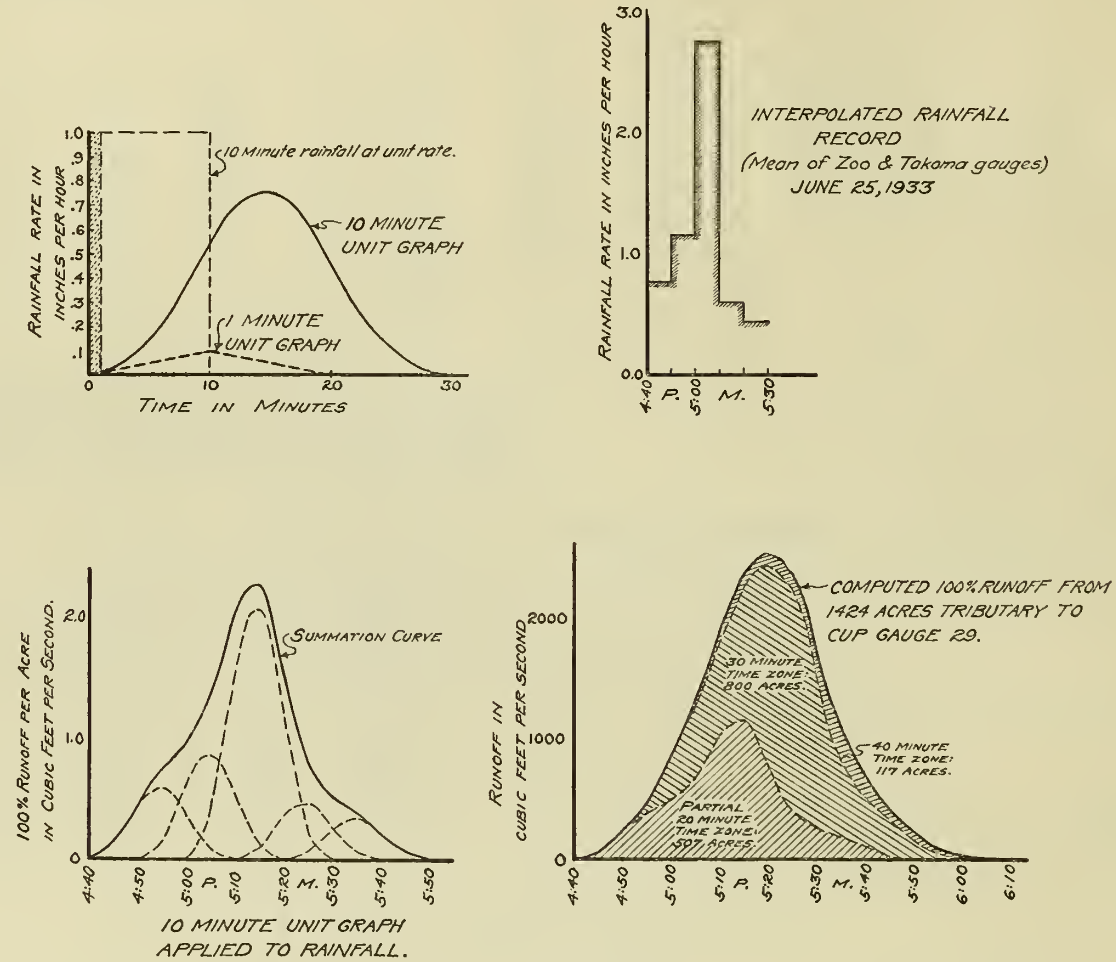
Record for cup gage 29 shows sewer flowing full with a capacity of 900 cubic feet per second. Therefore, for this particular rain and measured discharge, the storm coefficient is 900/2,529, or 0.36. Applying the figures for 100-percent run-off per acres to the full 10-, 20-, 30-, and 40-minute zones, it is found that the 100-percent run-off at outfall of Piney Branch is 3,395 cubic feet per second. Multiplying this 100-percent run-off at the outfall by the storm coefficient: 3,395×0.36=1,220 cubic feet per second estimated discharge at outfall.

TABLE 4.—*Washington, D. C., calculated discharges of Piney Branch trunk sewer based on cup gage records for various storms*

Date	Rainfall intensity for				Total rainfall for 40 minutes	Pluviagraph peak at cup gage, cubic feet per second	Discharge at cup gage, cubic feet per second	Storm coefficient	Computed outfall discharge, cubic feet per second	Gage number	Acres tributary to gage	Stage of flow at gage
	10 minutes	20 minutes	30 minutes	40 minutes								
June 25, 1933	2.75	1.95	1.56	1.32	0.88	2,529	900	0.36	1,220	29	1,500	Full.
July 2, 1933	1.58	1.17	.91	.76	.51	1,309	320	.25	530	34	1,160	⅔ full.
						1,181	320	.27	570	37	1,000	⅔ full.
						1,575	450	.29	590	29	1,500	½ full.
July 25, 1933	2.40	1.85	1.57	1.41	.94	2,560	900	.35	1,210	29	1,500	Full.
Aug. 23, 1933	.78	.72	.72	.68	.45	1,758	600	.34	530	29	1,500	⅔ full.
Oct. 1, 1933	1.32	1.32	1.32	1.11	.74	1,940	720	.37	1,050	29	1,500	0.8 full.
						1,841	1,050	.57	1,615	32	1,425	Full.
Sept. 12, 1934	3.72	3.30	3.10	2.70	1.80	4,753	900	.19	1,280	29	1,500	1.05 full.

TABLE 5.—*Estimated run-off—Outfall Piney Branch sewer*

Date of storm	Maximum average rainfall rate				Total precipitation 40 minutes	Pluviagraph peak outfall	Storm coefficient	Estimated discharge outfall	Rainfall records used
	10 minutes	20 minutes	30 minutes	40 minutes					
Apr. 21, 1927	3.12	1.71				2,660	0.40	1,064	Zoo only.
Nov. 17, 1927	2.46					1,940	.40	776	Zoo and Takoma.
Do	2.64	1.47	1.07	0.93	0.62	2,440	.40	976	Do.
Aug. 11, 1928	2.76	2.70	2.24	2.29	1.53	5,245	.40	2,098	Zoo only.
Aug. 16, 1928	1.26	.99	.90	.86	.57	1,980	.40	792	Do.
Aug. 21, 1928	1.92	1.56	1.63	1.41	.94	2,825	.40	1,130	Do.
Aug. 25, 1928	2.49	2.48	2.38	2.31	1.54	5,345	.40	2,138	Zoo and Takoma.
June 22, 1929	3.00	2.30	1.62	1.24	.83	3,710	.40	1,484	Do.
June 28, 1929	2.74	2.45	1.87			3,995	.40	1,598	Do.
June 16, 1932	2.82	2.58	2.07	1.81	1.20	4,400	.40	1,760	Zoo only.
June 25, 1933	2.75	1.95	1.56	1.32	.88	3,395	.40	1,358	Zoo and Takoma.
July 2, 1933	1.58	1.17	.91	.76	.51	2,055	.40	822	Zoo and Carnegie.
July 25, 1933	2.40	1.85	1.57	1.41	.94	3,465	.40	1,386	Zoo and Takoma.
Aug. 23, 1933	.78	.72	.72	.68	.45	1,045	.40	418	Do.
Oct. 1, 1933	1.32	1.32	1.32	1.11	.74	2,835	.40	1,134	Do.
Sept. 12, 1934	3.72	3.30	3.10	2.70	1.80	6,760	.40	2,704	Do.



UNIT GRAPH METHOD OF ESTIMATING 100% RUN-OFF

FIG. 3

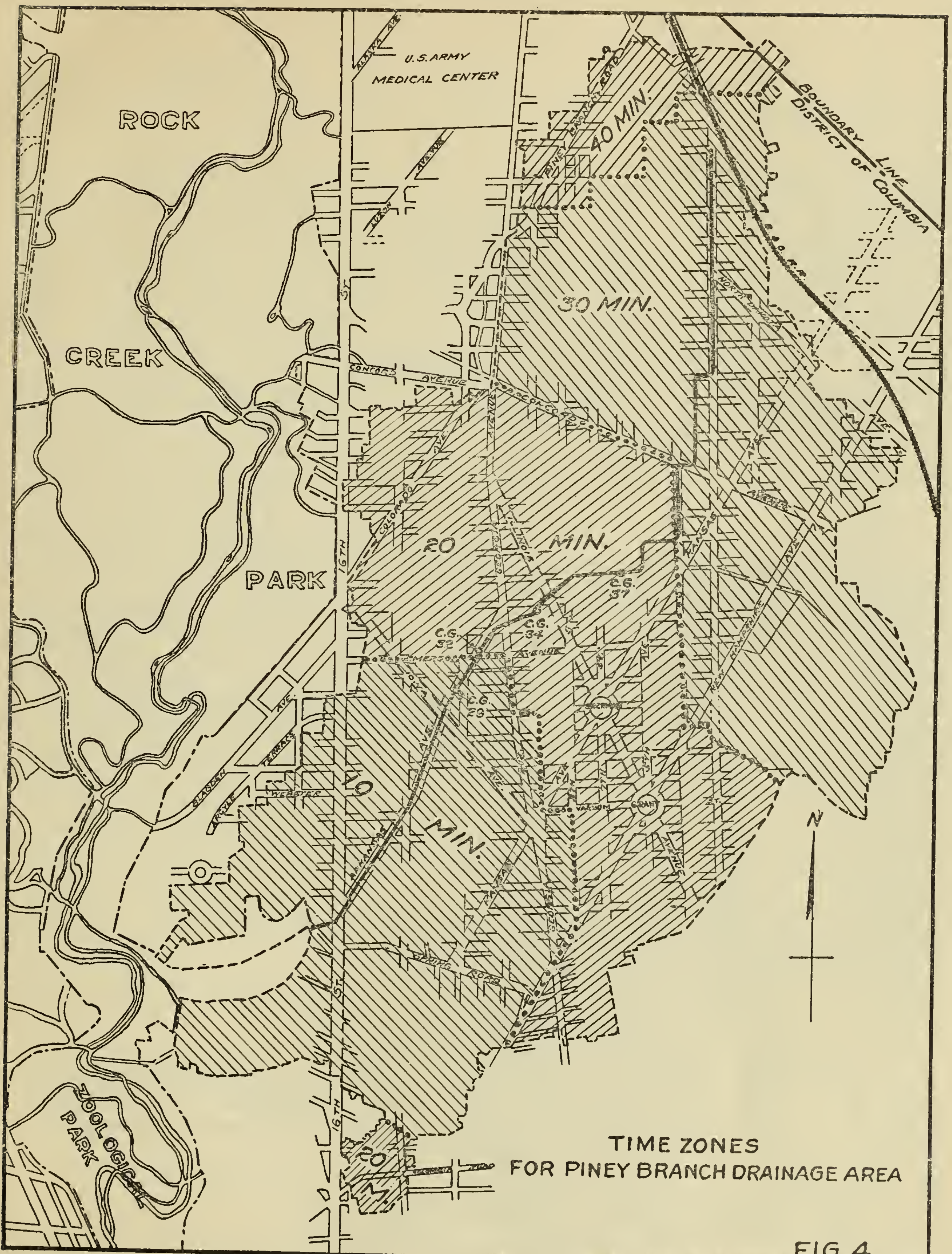
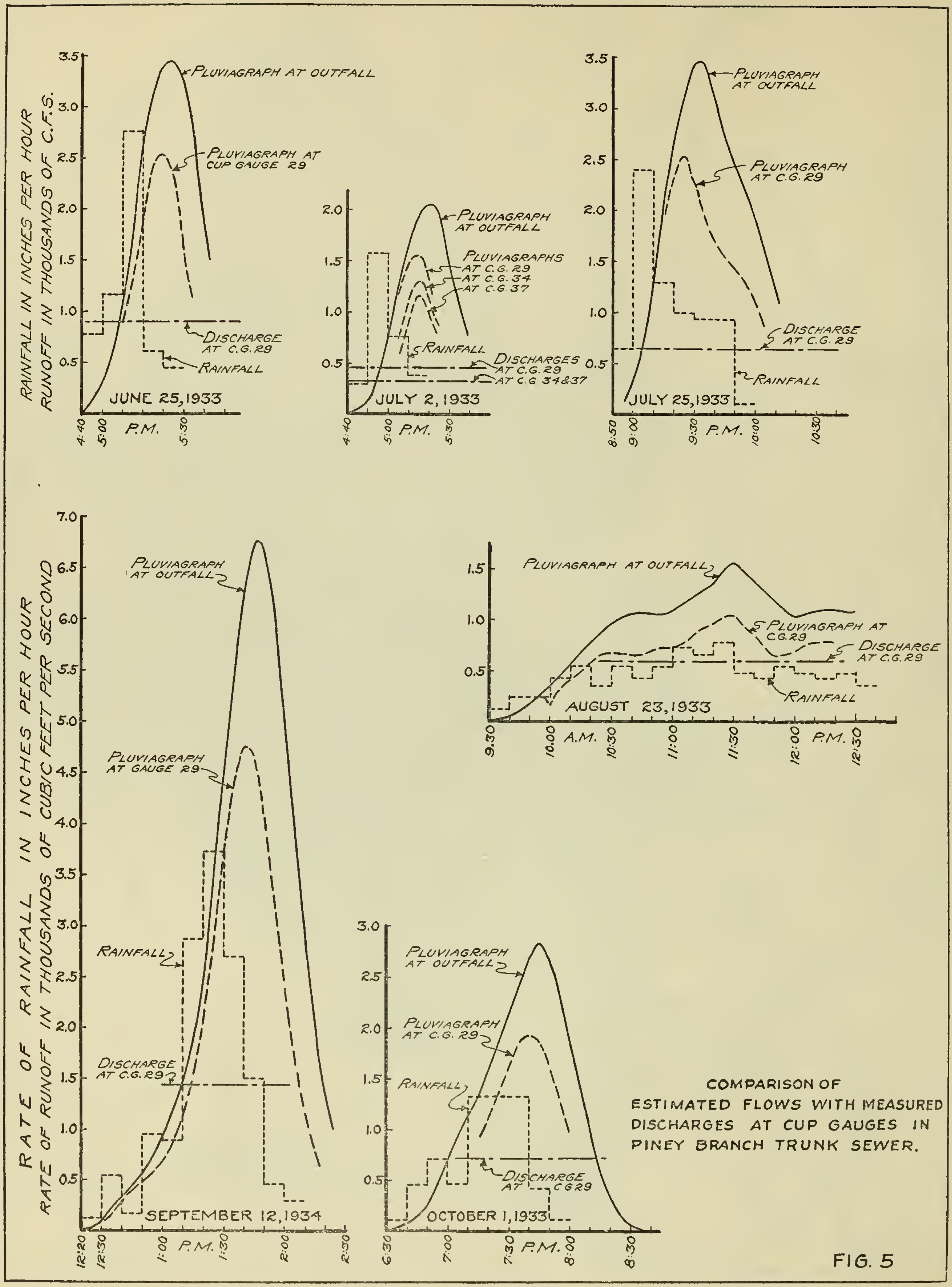

U.S. ARMY
MEDICAL CENTER
ROCK
CREEK
PARK
ZOOLOGICAL
PARK
BOUNDARY LINE
DISTRICT OF COLUMBIA
40 MIN.
30 MIN.
20 MIN.
10 MIN.
20 M.
16TH ST.
CONCORD AVENUE
C.G. 37
C.G. 34
C.G. 32
C.G. 29
N
TIME ZONES
FOR PINEY BRANCH DRAINAGE AREA

FIG. 4



COMPARISON OF ESTIMATED FLOWS WITH MEASURED DISCHARGES AT CUP GAUGES IN PINEY BRANCH TRUNK SEWER.

FIG. 5

Chapter V–B

METHOD B.—STORM RUN-OFF BASED ON FLOW IN ROCK CREEK AT SHERRILL DRIVE AND Q STREET GAGES

1. As an independent check, computations of discharges were made from the total combined and storm sewer areas of 4,000 acres [1] above Q Street. Through the foresight of the National Park Service, in cooperation with the United States Geological Survey, two self-recording stream gages were established on Rock Creek in 1929, one at Sherrill Drive and the other at Q Street. The available records covered a period of about 4 years. The recorded Q Street hydrographs, accompanying rainfall sufficient to produce any material run-off, show a sharp rise and peak immediately following rainfall. The hydrographs from the Sherrill gage (which is 35,400 feet above Q Street and above the combined sewer outfalls) showed no such peaks. As might be expected, the graphs of run-off from the open area above Sherrill were relatively long and flat. Under conditions where the surface offered high absorption or infiltration capacity, the maximum rate of run-off at Sherrill (drainage area=62.2 square miles) was frequently only one-tenth the peak rate of run-off from the sewered area of less than 7 square miles. Precipitation records from the self-recording rain gages were furnished by the Sewer Department of the District of Columbia as noted in chapter V–A.

2. *Peak run-off.*—Based on the foregoing stream flow records 44 hydrographs of run-off from the combined and storm sewered area of 4,000 acres were platted. The peak rates of run-off into Rock Creek, together with incidental data, are listed in table 7.

3. *Details of procedure.*—Figures 6*a* to *d*, hydrograph 16, for the storm of July 25, 1933, are presented as an example. The same procedure was used in all of the other hydrographs.

(*a*) *Rainfall.*—The rainfall records for each of the stations, Tacoma, Zoo, and Carnegie, are shown in table 6. The average rainfall for the three stations is also shown in figure 6.

(*b*) Run-off between Sherrill and Q Street. Following are the drainage areas here referred to:

	Square miles	*Acres*
Rock Creek above Q Street gage	75.8	-----
Rock Creek above Sherrill gage	62.2	-----
Rock Creek between Sherrill and Q Street	13½	8,600
Combined and storm sewer area above Q Street	-----	4,000
Open country area above Q Street [2]	-----	4,600
Outlet Piney Branch sewer below 16th Street	-----	2,327

[1] By planimeter from United States Geological Survey topographic map.

[2] Open country includes some 1,800 acres of separate sewered area as well as park and vacant lands.

On figure 6*a* the hydrograph of flow at the Q Street gage is platted from the self-recorded record of stages and the rating table furnished by the United States Geological Survey. There is also platted, in dotted line, the hydrograph for the same period of flow at Sherrill. The flow at Q Street includes the flow at Sherrill. Therefore, subtracting the flow at Sherrill from the flow at Q Street will give the run-off from the intervening tributary area of 8,600 acres.

(*c*) *Time of transit.*—Sherrill gage is 35,400 feet above Q Street and a given stage of water passing Sherrill gage does not reach Q Street until some time later. This time of transit is dependent upon the wave velocity. For a small change in stage the wave velocity "m" is derived from the Seddon formula:

$$m=dQ/dA$$

Where dQ is a small change in the rate of flow and dA is the corresponding change in cross section area of the water in the stream. The values of dQ and dA, for any given stage at Q Street or Sherrill, were given in the Rock Creek rating tables of the United States Geological Survey for each 0.1 foot of stage. The time of transit from Sherrill to Piney Branch was based on the Sherrill rating. The time from Piney to Q Street was based on Q Street rating. This approximation was found sufficiently accurate for practical purposes. The lower solid line, figure 6*a*, shows the Sherrill graph corrected for time of transit at Q Street.

(d) Hydrographs from drainage areas between Sherrill and Q Street. Figure 6*b* shows the hydrograph of flow from the 8,600 acres between Sherrill and Q Street. It is found from figure 6*a* by subtracting the time adjusted Sherrill flow *B—B* from the observed flow *A—A* at Q Street. The initial ground water flow at Q Street was 32 cubic feet per second. Ground water flow at Sherrill was 24 cubic feet per second. The peak rate of surface run-off from 8,600 acres above Q Street is ordinate *AC* figure 6*b*, or 2,550 cubic feet per second.

The peak rate from the 8,600 acres includes the run-off from 4,600 acres of open country. That is territory outside of the area served by combined and storm sewers. Therefore in order to determine the peak flow from this combined sewer area of 4,000 acres it is first necessary to estimate the run-off from the open country and deduct it from the observed total peak flow of 2,550 cubic feet per second. This was accomplished by the following procedure:

Given the average percentage of run-off, referred to rainfall, for any given storm. This is coefficient C as noted on the sheet for each net hydrograph.

$$C=\frac{\text{Volume of net hydrograph in } Af.}{\text{Volume of rain on 8,600 acres in } Af.}$$

Let A=percent of run-off from the impervious area in the combined and storm sewer area (2,000 acres).

Let B=percentage of run-off from the pervious areas (whether in the combined sewer area or the open area). (2,000+4,600=6,600 acres.)

We then have the following conditions:

1. $C \text{ percent}=\frac{2{,}000\ A \text{ percent}+6{,}600\ B \text{ percent.}}{8{,}600}$
2. B percent must be less than C percent.
3. Both B percent and C percent must decrease as rainfall volume decreases.
4. A percent (from the impervious area) must be high but cannot exceed 100 percent.

The impervious surface of the combined sewer area was taken as 50 percent. (See accompanying data from field survey.) A diagram was prepared in accord with the foregoing, showing values for A percent and B percent for any given value of C (fig. 7).

The total volume of run-off from the 8,600 acres, given by the hydrograph figure 6*c* is 350 acre-feet and the value of C, a ratio of volume of run-off to rainfall, is 0.356. The corresponding value of B percent, from the aforesaid diagram, is 20 percent. Hence the total volume of run-off from the 4,600 acres of open country is 107 acre-feet. This volume is the area of the hydrograph to be deducted from the total graph in figure 6*c* in order to get the net graph for the 4,000 acres of combined and storm sewers. This graph D, figure 6*c*, was drawn as follows: The run-off duration is the same as the hydrograph for the 8,600 acres, viz: from 9 p. m. to 2:40 a. m. The average rate of run-off was, therefore, 236 cubic feet per second. Draw this rectangle $CCMM$. The required graph must equal the area of the rectangle and the portion of this graph above the line MM must equal the area of the rectangle left outside the required graph. Approximations were made in favor of the largest net rate of peak run-off, AD, from the combined area.

Figure 6*c* is the hydrograph of run-off from the 4,000 acres of combined sewer area at Q Street. It was derived from figure 6*b* by subtracting the ordinates of graph D from graph A. The peak rate of run-off AD is 2,120 cubic feet per second. It was due to the mean rates of rainfall shown on figure 6*c* and table 6.

4. *Distribution graph.*—Figure 6*d* shows the percent of total volume of run-off that will occur during each half hour units of time. The ordinates of percent were derived by measuring each half hour section of graph, figure 6*d*, in acre-feet and dividing each of such measurements by 107 acre-feet. By the principle of the unit hydrograph this distribution graph is applicable to all rainfalls on this 4,000 acres of sewered area which have a duration of 130 minutes and which have a "pattern" of rain intensities similar to those in diagram figure 6*c*. The intensities of rain, during other applicable storms, may be fractions or multiples of the particular intensities shown in figure 6*c*. This procedure is useful in estimating run-off from rainfall data by analogy with observed rainfall and run-off.

5. *Relation of peak run-off from Piney Branch to total combined sewer run-off.*—Given the peak rates of run-off from the combined sewer area of 4,000 acres above Q Street, it was desired to find the corresponding peak rates of run-off from the Piney Branch area of 2,327 acres. This was accomplished by the procedure described by LeRoy K. Sherman in Transactions of the American Geophysical Union, 1932, page 332. The following peak flow relations were found:

Number of hydograph:	*Ratio peak flow of Piney Branch to peak flow at Q Street (percent)*
1	72. 5
2	71. 8
3	72. 5
13	72. 6
14	75. 0
16	66. 5
19	66. 8
Average	71. 1

In table 6 the factor of 71 percent was applied to the total-area peak rates to derive the column of peak rates of run-off for Piney Branch.

6. *Intensity-frequency curve by method B.*—The 15 maximum hydrograph peaks from the Piney Branch area, as listed in table 7, are platted in figure 9 together with the intensity-frequency curve derived by method A. The line of B peaks in about 80 percent of the A peak. This is largely due to the flattening effect upon sewer outfall peaks during time of transit in Rock Creek to the Q Street gage.

The two methods, made independently, are in fair agreement as a basis for design.

TABLE 6.—*Hydrograph no. 16, rainfall record July 25, 1933, beginning of 10-minute intervals*

Time	8:50 p. m.	9:00 p. m.	9:10 p. m.	9:20 p. m.	9:30 p. m.	9:40 p. m.	9:50 p. m.	10:00 p. m.	10:10 p. m.	10:20 p. m.	10:30 p. m.	10:40 p. m.	10:50 p. m.
Rate of rain inches per hour:													
Takoma Park	0	0	1.9	1.3	0.7	0.70	0.7	0.12	0.12	0.12	0.12	0.12	0.12
Zoo Park	1.3	2.9	1.3	1.3	1.2	1.20	.1	.10	.10	.10	.10	.10	.10
Carnegie Institute	0	1.0	3.1	2.4	1.2	.08	.08	.08	.08	.08	.08	0	0
Average inches per hour	.43	1.30	2.10	1.67	1.03	.66	.29	.10	.10	.10	.10	.07	.07
Accumulated depth of rain, inches	.072	.289	.639	.917	1.089	1.199	1.247	1.264	1.280	1.297	1.313	1.325	1.34

Maximum rain duration=130 minutes; average rate=0.617 inch per hour.
Maximum depth rain in 40 minutes=1.017 minutes; average rate=1.53 inches per hour.
Maximum depth rain in 10 minutes=0.350 minutes; average rate=2.10 inches per hour.

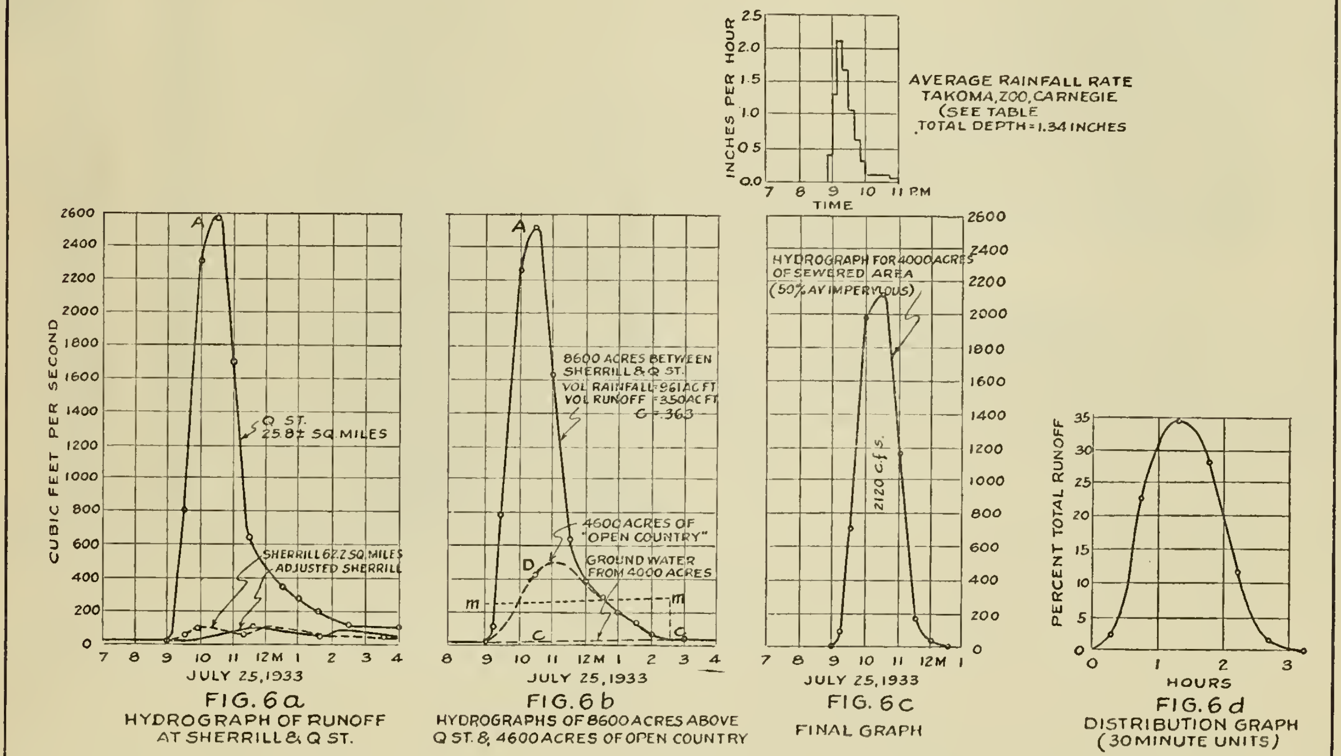


FIG. 6a
HYDROGRAPH OF RUNOFF AT SHERRILL & Q ST.

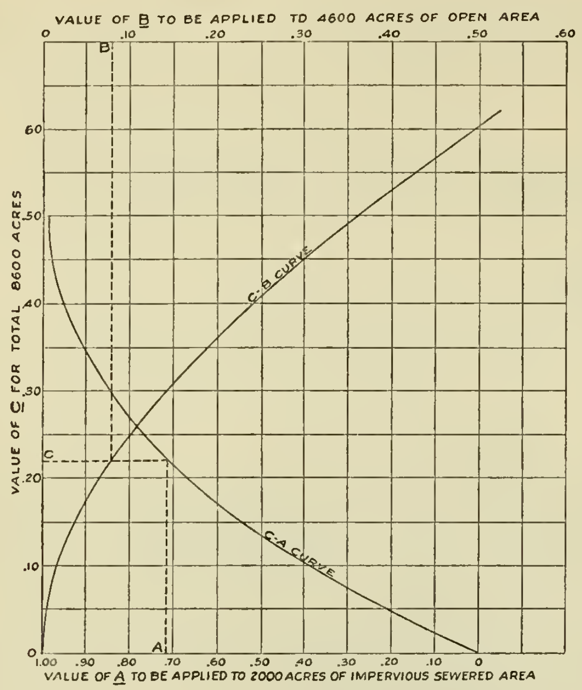
FIG. 6b
HYDROGRAPHS OF 8600 ACRES ABOVE Q ST. & 4600 ACRES OF OPEN COUNTRY

FIG. 6c
FINAL GRAPH

FIG. 6d
DISTRIBUTION GRAPH (30 MINUTE UNITS)

METHOD B FOR ESTIMATING STORM WATER FLOW FROM COMBINED SEWERS.
HYDROGRAPH NO. 16 OF JULY 25, 1933

FIG. 6



ROCK CREEK SEWERAGE
DIAGRAM SHOWING
C PERCENT OF RUN-OFF FROM TOTAL AREA = 8600 ACRES
B " " " " OPEN " = 4600 ACRES
A " " " " IMPERVIOUS
PORTIONS OF STORM SEWERED AREA = 2000 ACRES

FIG. 7

TABLE 7.—*Peak run-off rates from combined sewer areas and list of 44 observed hydrographs in Rock Creek*

Number of hydrograph	Date	Peak run-off 4,000 acres (cubic feet per second)	Peak run-off from 2,327 acres (cubic feet per second)	Order of magnitude
1	Sept. 5–6, 1932	2,360	1,677	First.
16	July 25, 1933	2,120	1,506	Second.
2	June 16, 1932	2,050	1,440	Third.
3	Mar. 27–28, 1932	1,870	1,329	Fourth.
42A	Oct. 5–6, 1932	1,800	1,279	Fifth.
19	Aug. 23, 1933	1,660	1,180	Sixth.
15	July 3, 1933	1,400	994	Seventh.
21	Aug. 10, 1931	1,300	923	Eighth.
44B	Nov. 1, 1932	1,270	902	Ninth.
32	June 17, 1930	1,160	824	Eleventh.
12	June 7–8, 1933	1,150	816	Twelfth.
4	May 27–28, 1932	1,070	760	Thirteenth.
5	Sept. 23–24, 1932	1,000	710	Fourteenth.
41	Sept. 23, 1931	915	650	Fifteenth.
13	June 25–26, 1933	1,200	852	Tenth.
17	July 26, 1933	820	581	
31	Apr. 6–7, 1930	730	518	
27	Nov. 19, 1931	720	510	
14	July 2, 1933	710	504	
6B	May 12–13, 1932	660	468	
7	Mar. 22, 1932	500	355	
11A	Mar. 6, 1932	460	326	
43A	Oct. 17–18, 1932	460	326	
18	Aug. 21, 1933	430	305	
9B	May 1, 1932	420	298	
39	Oct. 22, 1929	400	284	
33	Mar. 7, 1930	390	276	
9A	May 1, 1932	370	262	
8	Feb. 4, 1932	360	256	
44A	Nov. 1, 1932	360	255	
20A	July 15, 1931	350	248	
23	Aug. 15, 1931	340	241	
26	Aug. 27, 1931	330	234	
46	Nov. 9–10, 1932	320	227	
45	Nov. 6, 1932	310	220	
35	June 24, 1930	300	213	
6C	May 12–13, 1932	290	206	
20	Sept. 4, 1933	270	192	
40	Nov. 18, 1929	230	163	
25	Aug. 22, 1931	210	149	
28	Dec. 9, 1931	190	135	
24B	Aug. 21, 1931	170	121	
11B	Mar. 6, 1932	130	92	
34	Feb. 4, 1930	130	92	
24C	Aug. 21, 1931	60	43	

Chapter V–C

SUMMARY OF RUN-OFF STUDIES

1. The independent studies of the run-off as made separately through the use of the cup gages and through the use of the stream gages in Rock Creek developed values for about 16 occurrences with each method. The particular rains studied were generally not of the same dates under the two methods. Determination of the frequency of occurrence of discharges both above and below the range of those included in the specific studies was required.

It became evident that the frequency of discharge from the Piney Branch sewer for flows in excess of about 800 second-feet would be the critical factor on which final judgment would have to be based.

2. *Method A, run-off studies using cup gage and rainfall data.*—Because of the relatively short period covered by the cup-gage records and the fact that relatively few of these records were usable as being free from discharge complications, it was obviously impractical, by that approach, to develop a sufficient number of actual discharges for a frequency study. Accordingly, the frequency study was made through the use of rainfall records, these records being translated into discharge by uniform application of the pluviagraph method and the storm factor, already described.

The first step in this study involved a very extensive analysis of rainfall intensity, duration, and frequency, with regard to frequencies of a much lower value than have heretofore been studied in municipal drainage practice; that is, it was clear that the rainfall frequency study must be related to discharges occuring several times a year instead of once in 5 years or once in 15 years, as is generally involved in municipal sewer design.

In this study, the last 10 years' records of the United States Weather Bureau station located at Twenty-fourth and M Streets were used. The Weather Bureau very kindly sent the original charts to its St. Louis office for this work.

3. A preliminary analysis of the 16 rains studied indicated the lowest rates of rainfall for various durations that might have to be considered in such a study. Thereafter, for each year, each storm was analyzed, and the maximum intensity of rainfall for each of the duration periods commonly considered was plotted against a time scale. The resulting charts for the 10 specific years extending from 1925 to 1934, inclusive are given in the appendix. From these charts it is easy to determine the relative frequency of occurrence of any particular intensity for any one of the duration periods. Also, from a consideration of the vertical lines, it becomes an easy matter to determine the number of storms in each year in which intensities of interest occur.

4. This information was then condensed in the form shown in the appendix where the information for each duration period was brought into a single diagram of a 10-year record, and from these diagrams the rainfall intensity curves on figure 8 were worked out. These last curves were not used specifically in the further studies of discharge frequency but were available as a general check on the judgment of the engineers; from them two typical run-off curves were prepared as a background for the earlier studies of the Piney Branch sewer system.

5. In the further development of the run-off frequency study, each of the storms in the 10 years was examined in detail, and by inspection it was determined which of them developed discharges within the range of those given in table 8 for the first 16 rains that had actually been analyzed. In general, all rains were studied for which it appeared possible that the discharge from Piney Branch would exceed 800 cubic feet per second; and as the individual rains were analyzed, some of those originally chosen which were found to run below that figure were dropped from further analysis.

6. The total area in the Piney Branch subbasin is so located that the flow from the extreme section will generally reach the outlet within 40 minutes. Consequently, in the frequency studies for this basin, the rains of 40-minute duration were those of critical importance. In taking off rainfall intensities for duration periods between 10 and 40 minutes, in the rainfall study above referred to, the tabulation in most instances involved intensities occurring within a 40-minute period within the storm. However, this was not universally the case, and in order to reduce the further frequency studies to a standard basis, it was decided to so arrange the four 10-minute periods as to place the maximum 10-minute intensity first, and the remaining rainfall in the succeeding 10-minute periods calculated from the difference, e. g., from the maximum 20-minute intensity in the first 20 minutes and the maximum 10-minute intensity in the first 10 minutes, determining the rates in the second 10 minutes by necessary subtraction.

TABLE NO. 8

LIST OF WASHINGTON, D.C. RAINSTORMS FOR WHICH THE PROBABLE MAXIMUM RUNOFF RATE WOULD EXCEED 800 C.F.S. AT THE OUTFALL OF THE PINEY BRANCH TRUNK SEWER. COMPILED FROM RECORDS OF THE U.S WEATHER BUREAU FOR THE TEN YEARS 1925-1934.

DATE OF STORM.	MAXIMUM AVG. RAINFALL RATES FOR VARIOUS DURATION PERIODS.				RAINFALL RATES-SUCCESSIVE PERIODS-SYNTHETIC 40 MIN. STORM.				ESTIMATED MAX. RUNOFF FROM PINEY BRANCH. STORM COEF. 0.40	ORDER OF MAGNITUDE	RUNOFF FROM PINEY BRCH. PLUVIAGRAPHS
	10 MIN.	20 MIN.	30 MIN.	40 MIN.	FIRST 10 MIN.	SECOND 10 MIN.	THIRD 10 MIN.	FOURTH 10 MIN.			
1	2	3	4	5	6	7	8	9	10	11	12
7-10-25	2.76	1.44	0.96	0.72	2.76	0.12			805	63	
8-13-25	2.82	2.49	1.74	1.34	2.82	2.16	0.24	0.14	1249	34	
9-12-25	2.10	1.83	1.52	1.20	2.10	1.56	0.90	0.30	1290	31	
9-16-25	2.40	1.98	1.98	1.71	2.40	1.56	1.98	0.90	1652	15	
7-4-26	5.76	3.33	2.28	1.74	5.76	0.90	0.18	0.22	1923	8	
8-16-26	3.78	2.76	2.54	2.11	3.78	1.74	2.10	0.82	2109	6	
9-2-26	5.94	4.59	3.70	3.08	5.94	3.24	1.92	1.22	3132	2	
9-7-26	3.42	2.01	1.70	1.32	3.42	0.60	1.08	0.18	1398	25	
9-9-26	2.04	1.86	1.48	1.12	2.04	1.68	0.72	0.04	1081	41	
10-24-26	2.88	2.55	2.22	2.03	2.88	2.22	1.56	1.56	1852	11	
11-9-26	3.96	2.04	1.44	1.11	3.96	0.12	0.24	0.12	1194	36	
A 4-21-27	3.66	1.95	1.32	0.99	3.66	0.24	0.06		1090	40	1064
6-12-27	1.92	1.29	1.04	0.93	1.92	0.66	0.54	0.60	891	55	
6-22-27	1.92	1.29	0.98	0.84	1.92	0.66	0.36	0.52	839	60	
7-30-27	2.46	1.41	0.96	0.73	2.46	0.36	0.06	0.04	815	61	
8-13-27	1.98	1.41	1.14	1.10	1.98	0.84	0.60	0.98	987	45	
10-3-27	3.78	2.43	1.88	1.50	3.78	1.08	0.78	0.36	1571	18	
10-12-27	2.82	2.10	1.56	1.25	2.82	1.38	0.48	1.32	1370	28	
11-17-27	4.08	2.04	1.50	1.29	4.08	0	0.42	0.66	1261	33	976
7-28-28	2.58	1.41	1.02	0.82	2.58	0.24	0.24	0.22	880	57	
8-7-28	2.28	1.80	1.40	1.09	2.28	1.32	0.60	0.16	1184	37	
8-11-28	3.12	2.52	2.20	1.86	3.12	1.92	1.56	0.84	1885	9	2098
8-16-28	1.44	1.38	1.16	0.98	1.44	1.32	0.72	0.44	898	54	792
8-21-28	2.64	2.34	1.84	1.47	2.64	2.04	0.84	0.36	1381	26	1130
8-25-28	2.28	1.95	1.66	1.50	2.28	1.62	1.08	1.02	1417	23	2138
9-2-28	1.26	1.20	1.14	0.96	1.26	1.14	1.02	0.42	923	51	
C 4-21-29	2.10	1.44	1.04	0.93	2.10	0.78	0.24	0.60	904	53	
6-14-29	2.64	1.62	1.08	0.82	2.64	0.60			912	52	
6-21-29	4.56	4.14	3.00	2.52	4.56	3.72	0.72	1.08	2313	3	1484
6-24-29	2.64	1.71	1.16	0.90	2.64	0.78	0.06	0.12	988	44	
6-28-29	4.26	2.73	1.84	1.41	4.26	1.20	0.06	0.12	1569	19	1598
9-6-29	3.12	2.61	1.74	1.32	3.12	2.10			1511	21	
9-10-29	2.70	1.65			2.70	0.60			924	50	
9-29-29	3.18	2.04	1.42	1.23	3.18	0.90	0.18	0.66	1228	35	
6-24-30	2.82	1.65	1.12	0.84	2.82	0.48	0.06		946	49	
7-1-30	1.50	1.38	1.22	1.08	1.50	1.26	0.90	0.66	972	47	
9-13-30	2.16	1.38	0.94	0.76	2.16	0.60	0.06	0.22	809	62	
5-22-31	2.88	1.68	1.20	0.96	2.88	0.48	0.24	0.24	1011	42	
5-31-31	3.96	3.27	2.40	1.86	3.96	2.58	0.66	0.24	2029	7	
6-24-31	2.52	1.33	1.40	1.08	2.52	1.14	0.54	0.12	1178	38	
7-2-31	3.36	2.25	1.54	1.15	3.36	1.14	0.13		1301	30	
8-10-31	3.72	2.73	2.00	1.55	3.72	1.74	0.54	0.20	1737	14	
9-23-31	3.84	3.45	2.96	2.28	3.84	3.06	1.98	0.24	2126	5	
4-30-32	2.04	1.65	1.18	0.93	2.04	1.26	0.24	0.18	1008	43	
5-28-32	2.22	1.41	1.16	0.88	2.22	0.60	0.66	0.04	964	48	
6-16-32	3.60	2.49	2.10	1.61	3.60	1.38	1.48	0.14	1779	13	1760
9-5-32	3.36	2.85	2.58	2.37	3.36	2.34	2.04	1.74	2204	4	
9-23-32	2.94	2.16	1.98	1.59	2.94	1.38	1.62	0.42	1642	16	
10-31-32	1.98	1.32	1.04	0.84	1.98	0.66	0.48	0.24	881	56	
6-7-33	2.70	1.95	1.58	1.58	2.70	1.20	0.84	1.58	1379	27	
6-26-33	2.46	2.12	1.72	1.38	2.46	1.78	0.92	0.56	1464	22	1358
7-2-33	2.40	1.92	1.50	1.17	2.40	1.44	0.66	0.18	1269	32	822
7-25-33	3.48	2.70	2.22	1.86	3.48	1.92	1.26	0.78	1874	10	1386
9-4-33	2.04	1.38	1.00	0.76	2.04	0.72	0.24	0.04	847	59	
9-14-33	2.82	1.71	1.17	0.86	2.82	0.60	0.09		986	46	
10-1-33	3.48	2.25	1.66	1.42	3.48	1.02	0.48	0.70	1409	24	1134
7-25-34	2.82	1.50	1.02	0.77	2.82	0.18	0.06	0.02	855	58	
7-29-34	3.00	2.28	2.14	1.81	3.00	1.56	1.86	0.82	1783	12	
8-12-34	3.06	2.37	1.60	1.20	3.06	1.68	0.06		1360	29	
8-15-34	4.02	2.73	1.84	1.38	4.02	1.44	0.06		1562	20	
A 9-7-34	2.52	1.50	1.36	1.13	2.52	0.48	1.08	0.44	1124	39	
B 9-7-34	2.64	2.34	1.98	1.80	2.64	2.04	1.26	1.26	1574	17	
9-12-34	5.88	5.34	4.90	4.79	5.88	4.80	3.02	4.46	4007	1	2704

THESE RUNOFFS ARE BASED ON RAINFALL RECORDS OF WASHINGTON, D.C. WEATHER BUREAU

THESE RUNOFFS ARE BASED ON RAINFALL RECORDS OF TAKOMA, ZOO OR CARNEGIE RAIN GAGES OF WASHINGTON, D.C.

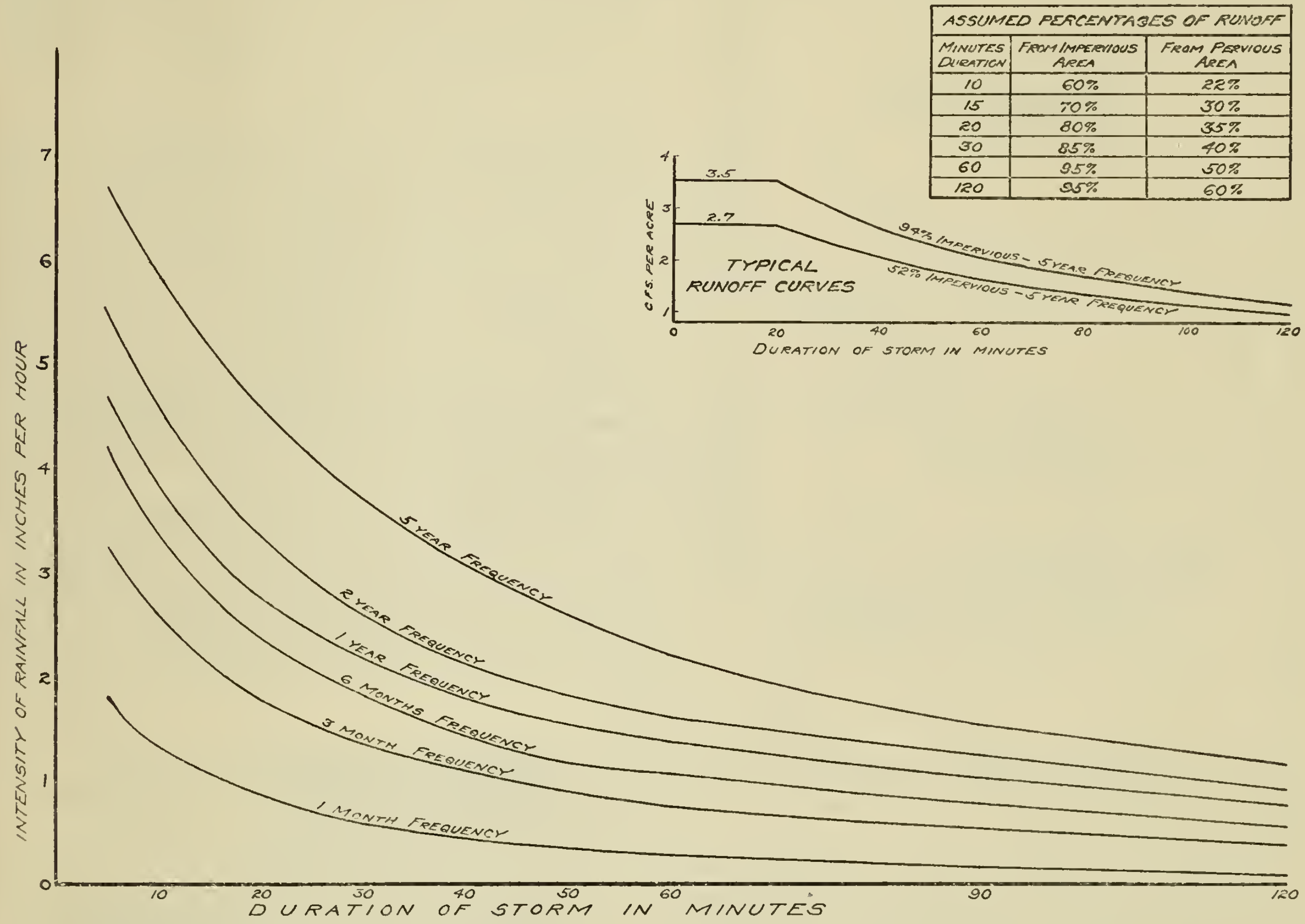
While this arrangement was arbitrary, it did not in many cases materially change the order in which the rainfall fell, the type of synthetic storm used here being probably the most common rainfall pattern encountered.

In order to determine whether the arbitrary placing of 10-minute values in this particular order had any peculiar effect on the peak rates of discharge, a test was made for several rains in which the 10-minute order was rearranged in other ways. It was found that the effect of such different arrangements on the calculated peak discharge was in most cases certainly within 2 or 3 percent, the possible exception being 1 or 2 rains of the double-hump type.

It should be noted that while 14 of the original 16 storms are again listed in table 8, their values are quite different, because in this 63-rain table the pluviographs are calculated from rainfall rates taken from the Weather Bureau records instead of from the adjusted rain rates actually occurring in the Piney Branch Basin; for short storms of this character the two sets of rainfall rates often differed widely.

7. Summarizing, it must be understood that the values given in column 10 of table 8 represent the discharges that would have occurred at the outlet of the Piney Branch sewer if the 63 rains with the specific rates recorded at the Weather Bureau station had been distributed over the Piney Branch watershed. These values were arrived at from the pluviographs calculated for these rains; as for the Piney Branch sewer by applying to the peak of the pluviagraphs the storm factor of 0.4, which had actually been determined for this location from the original 16 rains studied, from the original cup-gage studies. It is accordingly accepted that these 63 values within 10 years are a satisfactory basis for a frequency-discharge calculation of the Piney Branch outlet for all discharges having a value in excess of 800 cubic feet per second.

8. The values shown in table 8 have been plotted on figure 9 as a probability diagram. On this diagram there have also been plotted the discharge values resulting from the independent computation of the higher 15 rains analyzed by method B listed in table 6 and described in detail in chapter V–B. The lower



NOTE: Intensity curves based on rainfall records for the period 1925 to 1934 inclusive from the tipping-bucket gauge of the U.S. Weather Bureau in Washington, D.C.

RAINFALL INTENSITY CURVES FOR WASHINGTON, D.C.
APRIL 1935

FIG. 8

limit or sixty-third storm by method A described above was taken as 805 cubic feet per second.

9. It is possible from the diagram figure 10 to determine directly the rate of peak discharge that will be exceeded any number of times per annum. The minimum value on this diagram, 800 cubic feet per second, will be exceeded 6.3 times per annum. The curve

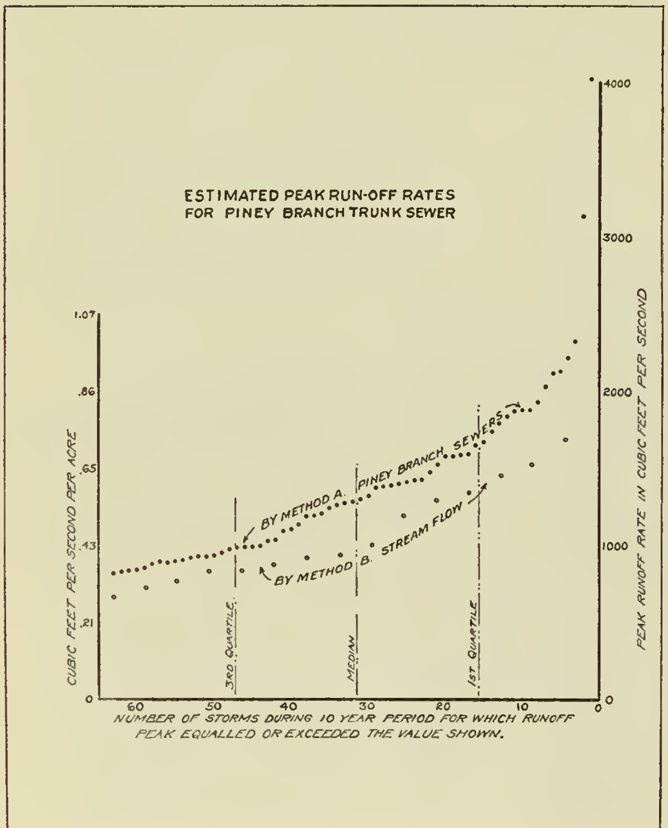


FIG. 9

rises slightly between the 6.1 frequency and the 4.3 frequency points, and is straight between the 4.3 and the low-frequency points.

10. *Selection of 1,200 cubic feet per second as relief-sewer capacity for Piney Branch area.*—In using figure 9 as a basis of decision as to the quantity of water to be diverted from Piney Branch, it had been decided that no diversion of less than 800 cubic feet per second would provide reasonable assurance against the discharge of polluted outflow into Rock Creek. If the diversion of 800 cubic feet per second were adopted, an overflow from the Piney Branch sewer would be expected more than 6 times per annum. Because of the flatness of the lower part of this curve, a raise of the diversion value to 1,000 cubic feet per second would have reduced the number of overflows to a little over 4 per annum. A further increase to 1,200 cubic feet per second reduces the number of overflows to about 3¼ per annum. After a full discussion of these possibilities with responsible officials of the National Park Service, it was agreed that the intensive use of Rock Creek in Rock Creek Park justified a very high sanitary protection, and the base value of 1,200 cubic feet per second for diversion at Piney was adopted.

11. *Slash Run run-off studies.*—Run-off studies similar to those for the Piney Branch area were made of the more highly developed Slash Run subdrainage basin, which has an area of approximately 370 acres. The time of flow through the sewers from an extreme point was found to be 15 minutes, with a 5-minute allowance for inlet time; this gives 20 minutes as the total time of flow from the extreme point to the outlet.

12. Of the cup gages, two of them were of value in this study. Gage no. 4, used for only one rain, has a tributary area of about 220 acres, but is located just above the junction of the main branches and is within 1 minute's flow time of the outlet. Gage no. 9, which was worked up for six rains, has a tributary area of about 100 acres and is located about 5 minutes' flow time above the outlet. The rains for this latter gage were worked up on the basis of a 10-minute pluviagraph before the time of flow above the point had been checked in detail. It appeared later that if a reasonable inlet time had been added, the 15-minute pluviagraph would have been required. However, the use of this material did not seem to justify a complete set of recomputations.

13. As shown in table 9 the storm factors average about 0.68; and if the two extremes are thrown out, the average is still about the same. If the pluviagraphs had been worked up on a 15-minute basis, the peaks would have been somewhat reduced; and the

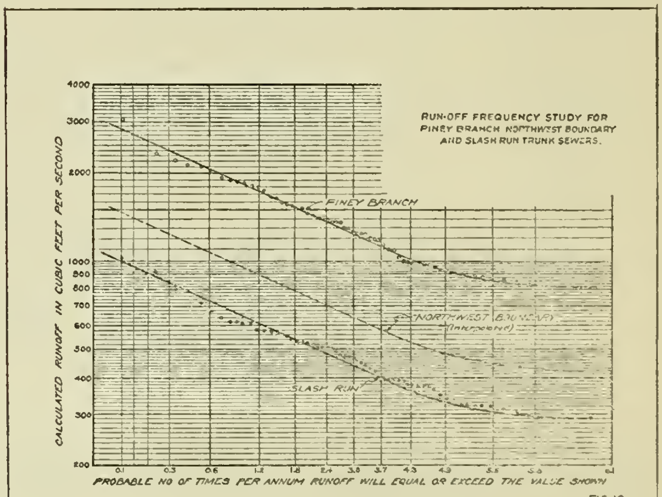


FIG 10

storm factor would have been accordingly increased. Even though this area is 94 percent impervious, the storm factor probably would not exceed 0.8 even on this basis. The estimated discharges based on the 0.68 factor, accordingly, may possibly be low by 10 to 15 percent.

TABLE 9.—*Washington, D. C., calculated discharges of Slash Run sewer based on cup-gage records for various storms*

Date	Cup gage no. 4, New Hampshire Avenue between L and M Streets; tributary area; 220 acres			Cup gage no. 9, Nineteenth and M Streets; tributary area, 101 acres		
	Cup-gage discharge	Pluviagraph discharge	Storm coefficients	Cup-gage discharge	Pluviagraph discharge	Storm coefficients
	Cubic feet per second	*Cubic feet per second*		*Cubic feet per second*	*Cubic feet per second*	
Aug. 21, 1928				271	278	0.97
June 21, 1929	640	1,020	0.73	309	469	.66
June 28, 1929				271	470	.58
Sept. 5, 1932				221	327	.68
Apr. 21, 1927 (A)				271	340	.80
Nov. 17, 1927 (B)				94.5	286	.33

Mean (of 6), 0.67. Median (of 7), 0.68.

14. Twenty-minute Slash Run pluviagraphs were calculated for the 63 rains (previously selected for the Piney Branch studies), and the storm factor of 0.68 was applied to their peak values to obtain the corresponding discharges at the Slash Run outlet. These have been plotted along with the similar data heretofore made up for Piney Branch, and are also shown on figure 10.

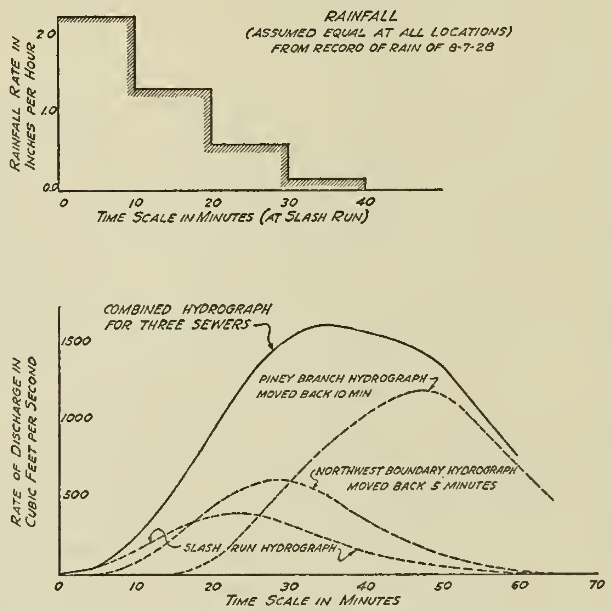
It is noteworthy that between the low frequency and the 4.3 frequency ordinates, both the Piney Branch and Slash Run curves are for all practical purposes straight lines; further, the Slash Run curve is a constant distance below the Piney Branch curve throughout. This constant difference represents the logarithm of $\frac{1,200}{430}$. The curve for the northwest boundary run-off was drawn in by interpolation, as explained subsequently in the discussion of proposed amounts to be intercepted.

15. *Estimating amounts to be intercepted.*—The earlier studies indicated that for Piney Branch, flows of over 1,200 second-feet, or 0.50 cubic feet per second per acre, may safely be discharged into Rock Creek. From the run-off frequency studies, it can be seen that such a flow will occur, or be exceeded, at the Piney Branch outfall on the average about 3.3 times a year. A similar frequency shows on figure 10 for Slash Run a discharge of 430 cubic feet per second, beyond which it may be presumed that overflows may be allowed. This 430 cubic feet per second value is equivalent to 1.16 second-feet per acre, or in round figures 1.2. On a pro rata basis, the northwest boundary area having an acreage of 555 would have a second-foot discharge value of 1.12, or a total outfall run-off of approximately 620 second-feet.

16. In figure 10, the run-off frequency graphs, the curve shown for northwest boundary sewer was interpolated from the Piney Branch curve, using the ratio 1,200:620.

17. Discharges in excess of 1,200 cubic feet per second, 620 cubic feet per second, and 430 cubic feet per second for Piney Branch, northwest boundary and Slash Run will occur then on an average of 3 to 3½ times a year, as a conservative estimate. Actually, for a number of years to come, this annual frequency of overflow would be somewhere between 2 and 3 times; however, if, and when, the Piney Branch and other overcharged sewers are relieved, the value may be nearer to the computed frequency. From the rainfall studies and the present capacity of the east interceptor, it appears that overflows in the past have average something over 30 per annum.

18. *Relief sewer diversion from several outfalls.*—In order that the capacity of such new interceptors as



COMBINED HYDROGRAPH FOR THREE MAIN SEWERS

FIG. 11

may be constructed might be properly estimated, it is further necessary that some knowledge be obtained as to the probable concurrence of the discharges of 1,200 cubic feet per second from Piney Branch, 620 cubic feet per second from northwest boundary, and 430 cubic feet per second from Slash Run. To that end, the hydrographs shown in figure 11 have been plotted. These were all based on one of the synthetic rains of the 63 rains previously studied, that of August 7, 1928, on the assumption that it was of simultaneous occurrence on all three drainage areas. The Piney Branch hydrograph has been moved over 10 minutes to allow for flow time through 11,000 feet of sewer down to about P Street; the calculated flow time through the sewer of 13 minutes has been reduced to 10 minutes to allow for the somewhat greater velocity of the flood wave, but this does not take into account the probable flattening out of the Piney Branch flood wave as it progresses. Similarly, the Slash Run hydrograph has been moved over 5 minutes to allow for the flow time in the new sewer between P Street and a point near M Street where the Slash water is introduced.

19. The combination of the three hydrographs on this basis indicates that the Slash Run and northwest boundary water practically all runs out ahead of the arrival of the Piney Branch peak. If the new sewer is to intercept both northwest boundary and Slash Run between Q and M Streets on Twenty-second Street, as might be the case of a tunnel outlet, then it would be probable to expect a first peak discharge from these small drainage areas in a combined amount of 1,100 or 1,200 cubic feet per second, and that this first discharge would pass out through the new sewer ahead of the Piney Branch water.

20. This would be the situation on the assumption previously stated that the rain was of simultaneous occurrence on all three of the drainage areas. As a matter of fact, inspection of several of the rains indicates that the more probable combination if for a somewhat later occurrence on Piney Branch, which would make the result of the hydrographs even more favorable. The peak of the combined hydrograph on the foregoing basis can be seen from figure 11 to be about 1,620 cubic feet per second. Assuming that the Slash Run and northwest boundary water was all introduced at about the same point (in the vicinity of P Street) instead of as proposed on figure 11, this peak might, from the three main tributaries, be increased to approximately 1,700 cubic feet per second. If to this is added the 200 cubic feet per second found to be contributed by the several smaller sewers to be intercepted, a total capacity of 1,900 cubic feet per second is indicated as ample for the lower part of the proposed relief sewer.

21. The discussion of the last three paragraphs is presented here to show the manner in which factors arrived at from the hydrologic studies are to be applied in the design of alternate relief sewer projects. The Piney Branch, northwest boundary, and Slash Run subbasins are the largest areas tributary to Rock Creek served by combined sewers, and the discharge from their sewers represents so great a proportion of the polluted outflow as to justify the detailed studies that have been outlined.

In the design of the relief sewer projects, the values of contaminated flow in cubic feet per second per acre to be intercepted, are as follows:

For the Piney Branch district, 0.50 cubic feet per second per acre; for the northwest boundary district, −1.1 cubic feet per second per acre; for the Slash Run district, −1.2 cubic feet per second per acre; for the smaller districts, both those on the east side of the valley and the considerable group of small areas on the west side of the valley, the quantities of outflow to be intercepted were varied between 0.75 and 1.25 cubic feet per second per acre in the designs hereafter described.

22. *West side relief sewer.*—The tributary districts on the west side are separated into two groups, one near the upper end and a second group below P Street. Between the two groups, a west side relief sewer would have a length of about 3,500 feet without lateral inflow, and the time of flow between centers of inflow of the two groups will be nearly 15 minutes. As the critical time for these small districts will not in any case exceed 15 minutes, the peak flow for the lower group will generally pass out of the sewer ahead of that from the upper group. The full combination of peak flows would only occur for longer and less intense rains. After study of these conditions the design factor for the lower west side group was reduced to values varying from 0.9 down to 0.75 second feet per acre. (See design tables 16 and 18.)

23. *Frequency and amount of combined sewage overflows into Rock Creek.*—The frequency and peak rates of overflow from the Piney Branch sewer outfall is shown for 63 computations by method A, in figure 9. This was checked by 15 independent computations by method B, also shown in figure 9. The 15 computations by method B are listed in table 12. The lower limit or sixty-third storm by method A was taken as 805 cubic feet per second. The corresponding storm, under method B, will have a peak rate of about 600 to 700 cubic feet per second. The exact minimum amount has little effect on the upper half of the frequency curve. The points by method B fall below the points derived by method A. This is to be expected. It is due to the effect of a falling peak in transit between Piney Creek and Q Street. For the purpose of a basis of design the check by the two methods is very satisfactory. Figures for flow as used for divisions of the combined sewer area can be used with confidence.

24. *Minimum rainfall to produce overflow into Rock Creek.*—Following is a list of the number of storms recorded by the United States Weather Bureau at Washington which had rainfall of 0.1 inch or more in 1 hour. The period is from October 1929 to October 1934, inclusive.

Year	Storms of 0.1 inch+in 1 hour				Storms of 0.2 inch+in 1 hour			
	Summer, 6 months		Winter, 6 months		Summer, 6 months		Winter, 6 months	
	May-October	April-September	November-April	October-March	May-October	April-September	November-April	October-March
1929	------	------	20	21	------	------	12	14
1930	15	19	12	8	11	13	4	2
1931	31	29	14	16	11	11	6	6
1932	30	32	27	25	15	14	7	8
1933	39	40	16	15	24	23	5	6
1934	33	------	------	------	26	------	------	------
Average, 6 months	27	30	17	17	17	15	7	7
Average, year	(1)	(1)	(1)	(1)	(2)	(2)	(2)	(2)

[1] 45 storms. [2] 23 storms.

TABLE 10.—*Light rains affecting Rock Creek gage at Q Street*

Hydrograph no.	Date	Rate of rainfall	Depth of rain
			Inches
39	Oct. 22, 1929	0.23 inch-hour for 10 minutes	0.04
39	do	0.065 inch-hour for 60 minutes	.07
32	June 17, 1930	0.20 inch-hour for 20 minutes	.07
35	June 24, 1930	0.25 inch-hour for 10 minutes	.04
35	June 25, 1930	0.17 inch-hour for 40 minutes	.11
24	Aug. 21, 1932	0.12 inch-hour for 50 minutes	.10
25	Aug 22, 1932	0.15 inch-hour for 15 minutes	.05
27	Nov. 19, 1931	0.3 inch-hour for 10 minutes	.05
43	Oct. 17, 1932	0.12 inch-hour for 30 minutes	.06
44	Nov. 1, 1932	0.09 inch-hour for 60 minutes	.09
45	Nov. 6, 1932	0.04 inch-hour for 60 minutes	.04
8	Feb. 4, 1932	0.10 inch-hour for 90 minutes	.15
11	Mar. 6, 1932	0.132 inch-hour for 60 minutes	.13
7	Mar. 22, 1932	0.25 inch-hour for 30 minutes	.13
6	May 12, 1932	0.08 inch hour for 90 minutes	.12
12	June 7, 1933	0.2 inch-hour for 30 minutes	.10
20	Sept. 3, 1933	0.2 inch-hour for 15 minutes	.05

TABLE 11.—*Percent of run-off from combined sewer area*

Hydrograph no.	Date	Average percent from 8,600 acres (C)	Average percent from 4,000 acres of combined sewer area, 50 percent impervious (C_1)	Duration of rain omitting intensities of 0.05 in./hr. or less	Antecedent rain for 5 days, total	Antecedent temperature, approximate average of maximum daily for 5 days	Maximum depth of rain falling in 40 minutes
				Minutes	*Inches*	*Degrees*	*Inches*
45	Nov. 6, 1932	0.551	0.750	120	1.79	58	0.10
19	Aug. 23, 1933	.514	.720	990	1.60	81	.41
6B	May 12–13, 1932.	.366	.666	270	1.10+	67	.21
2	June 16, 1932	.405	.662	80	1.00+	77	1.20
7	Mar. 22, 1932	.442	.656	60	.22+	52	.17
43A	Oct. 17–18, 1932.	.418	.634	420	.18+	62	.10
3	Mar. 27–28, 1932.	.435	.623	370	1.06	62	.56
42A	Oct. 5–6, 1932	.405	.622	300	.21+	78	.54
1	Sept. 5–6 1932	.369	.597	60	.66+	96	1.21
16	July 25, 1933	.363	.596	120	--------	92	1.00
44B	Nov. 1, 1932	.336	.545	180	.32	60	.44
15	July 3, 1933	.301	.500	180	1.55+	92	.40
5	Sept. 23–24, 1932	.310	.494	110	.16+	81	.24
46	Nov. 9–10, 1932.	.293	.493	360	1.58	62	.11
4	May 27–28, 1932.	.283	.481	90	--------	81	.50
21	Aug. 10, 1931	.278	.471	55	.06+	95	1.11
11A	Mar. 6, 1932	.237	.466	200	.80	55	.18
8	Feb. 4, 1932	.261	.453	610	.54+	48	.16
13	June 25–25, 1933.	.266	.453	50	.47	92	.75
17	July 26, 1933	.259	.444	300	1.68+	93	.22
14	July 2, 1933	.238	.422	60	.07	91	.56
12	June 7–8, 1933	.230	.406	55	.37+	86	(1) (2)
31	Apr. 6–7, 1930	.208	.381	570	.32	60	.36
41	Sept. 23, 1931	.201	.373	100	.05	86	.615
18	Aug. 21, 1933	.201	.365	420	.29	84	.150
9B	May 1, 1932	.198	.364	50	.07	70	.380
20A	July 15, 1931	.191	.361	25	1.90+	85	(3)
32	June 17, 1930	.194	.355	160	--------	85	.735
25	Aug. 22, 1931	.190	.354	170	1.68+	81	.130
27	Nov. 19, 1931	.179	.345	120	--------	70	.445
24B	Aug. 21, 1931	.183	.340	190	.60	85	.102
35	June 24, 1930	.175	.331	55	.17	87	.130
39	Oct. 22, 1929	.226	.331	390	--------	70	.295
11B	Mar. 6, 1932	.246	.323	180	.80	55	.175
33	Mar. 7, 1930	.172	.319	410	--------	50	.360
9A	May 1, 1932	.163	.307	100	.07	70	.540
24C	Aug. 21, 1931	.135	.296	40	.60	85	.080
23	Aug. 15, 1931	.138	.280	100	1.98	80	.485
28	Dec. 9, 1931	.150	.267	410	.09	50	.137
20	Sept. 4, 1933	.126	.249	30	.26+	81	.408
34	Feb. 4, 1930	.124	.241	440	.56	42	.110
44A	Nov. 1, 1932	.112	.226	330	.32	60	.242
40	Nov. 18, 1929	.102	.205	150	.50+	62	.207
26	Aug. 27, 1931	.098	.199	75	1.34+	77	(4) (5)

[1] 0.92 inch in 20 minutes.
[2] 0.11 inch in 35 minutes.
[3] 0.28 inch in 25 minutes.
[4] 0.565 inch in 15 minutes.
[5] 0.592 inch in 40 minutes.

25. The loss between rainfall and run-off on the 4,000 acres of sewered area (50 percent impervious), as estimated for the approximate period of surface run-off, ranges from 0.08 inch per hour to 0.22 inch per hour. This was determined from the following: Hydrographs nos. 35, 20A, 20, 14, 40, 4, 13, and 26. Average infiltration rate, inches per hours, 0.08, 0.086, 0.126, 0.139, 0.141, 0.144, 0.155, and 0.22.

26. Such low initial rates of rainfall as are listed in table 10 produce run-off because of the large impervious sewered area. Not all such light rainfall produces an effect on the Q Street gage. The effect of antecedent weather conditions has an effect.

27. This is indicated in the accompanying table 11 which shows the proportions of rainfall appearing as run-off into Rock Creek. C is the relation of total volume of run-off to the total rainfall for the given storm over the 8,600 acres between Sherrill and Q Street. C_1 is the same relation for the 4,000 acres of combined sewer area above Q Street.

28. Computations show the capacity of the existing Piney Branch interceptor to be not over 35 cubic-feet-seconds. A discharge of 35 cubic-feet-seconds might be produced from Piney Branch by a 10-minute rain of 0.2 inch per hour and 22.5 percent run-off rate from 700 acres. It might also be produced by a 20-minute rain of 0.12-inch average intensity and a 22 percent run-off rate from 1,300 acres. Such rainfall rates would apply to nearly every rain that occurs except such as we would distinguish as a mere drizzle.

29. Table 10 shows that many storms with total rainfall depth materially less than 0.10 inch in 60 minutes produced discharge from the sewered area into Rock Creek. The observed infiltration rates indicate that some run-off to Rock Creek will always occur from 50 percent impervious areas when the rainfall rate is 0.20 inch per hour. Comparison of United States Weather Bureau records of hourly depth of rainfall of 0.10 inch with District recording rain gages show that many of such hourly depths of 0.10 inch consisted of short rains with intensity rates of 0.20 inch or more per hour. The foregoing indicates that all of the 23 annual storms of 0.20 inch in 1 hour produced run-off to Rock Creek; that most of the 45, 23, or 22 annual storms with rainfall of 0.10 inch to 0.20 inch in 1 hour produced run-off; that some of the storms with rainfall less than 0.10 inch in 1 hour produced run-off.

30. Observations on direct discharge from the combined sewers during very light rainfalls are given in chapter II, Analysis of sewage during storm overflow into Rock Creek. These observations confirm the foregoing.

31. *Conclusions.*—Based upon all of the foregoing we find the following: Under present conditions there are on an average about 40 storms per year which produce some pollution in Rock Creek from the combined sewers. Twenty-five of such storms occur during the 6-month period including summer and 15 of such storms occur during the 6-month period including winter.

32. *Reduction of flow and pollution in Rock Creek.*—From the observations recorded on diagram figure 9 and from the hydrographs of run-off from the 4,000 acres now served by combined and storm sewers above Q Street, it was found that the peak flow of the individual sewers, as found by method A, was flattened by transit and storage in Rock Creek until, when measured at Q Street by method B, it amounted to only about 0.8 of the original peak. In other words, peak B equals 0.8 peak A, or peak A equals 1.2 peak B.

The simultaneous occurrence of peak flow in the proposed relief sewers near Q Street amounts to about 1,800 cubic feet per second. This by the 0.8 ratio gives a corresponding peak flow in Rock Creek at Q Street of about 1,500 cubic feet per second.

Therefore, combined sewer flows in Rock Creek from the 4,000 acres, when less than 1,500 cubic feet per second will be practically eliminated if and when the proposed relief sewers are installed. Also, any such flows from the combined sewers now in excess of 1,500 cubic feet per second will be reduced by as much as 1,500 cubic feet per second throughout their duration.

33. Table 12 shows the effect in reducing peak flow and sewage flows in Rock Creek at Q Street had the proposed relief sewers existed at the time of these recorded storms. The period of flow records in Rock Creek as used here dates from the fall of 1929 to 1934. The first figures in columns 3 and 7 are observed flows in Rock Creek as recorded by the United States Geological Survey gages at Q Street. The second figures give the flow had the proposed relief sewers been in operation. Column 5 is the flow from the existing combined sewers. This is from a computed hydrograph by method B as heretofore described for the case of hydrograph no. 16. Column 6 is the ratio of that part of the hydrograph above 1,500 cubic feet per second to the total area (volume) of the hydrograph noted for column 5. (See also hydrograph no. 44, fig. 2*a*.)

34. During this period there were about 170 days of rain when sewage was discharged into Rock Creek from the combined sewers. Had the relief sewers, as hereinafter proposed, been installed, then all of this sewer discharge for about 164 days would have been diverted from Rock Creek. Of the remaining 6 days of heavy rainfall 93 percent of all the water from the combined sewers would have been diverted from Rock Creek. The remaining 7 percent of flow for the 6 days would be so diluted as to be practically all rain water.

TABLE 12.—*Flow in Rock Creek at Q Street and change which would be effected by proposed relief sewer diversions*

Number of hydrograph	Order of magnitude for column 5	Date	Observed first peak at Q Street (cubic feet per second)	Composite peak at Q Street from 4,000 acres of combined sewers (cubic feet per second)	Percent reduction sewer flow in creek	Observed second peak at Q Street (cubic feet per second)
1	2	3	4	5	6	7
1	1	Sept. 5-6, 1932	3,300	2,360	79	130
1	With relief sewers		1,800	860		130
16	2	July 25, 1933	2,620	2,120	88	200
16	With relief sewers		1,120	620		200
2	3	June 16, 1932	2,675	2,050	90	330
2	With relief sewers		1,175	550		330
3	4	Mar. 27-28, 1932	2,660	1,870	92	940
3	With relief sewers		1,160	370		940
42A	5	Oct. 5-6, 1932	2,260	1,800	92	600
42A	With relief sewers		760	300		600
19	6	Aug. 23, 1933	4,300	1,660	99	4,300
19	With relief sewers		2,800	160		4,000
15	7	July 3, 1933	2,400	1,400	100	1,000
15	With relief sewers		900	0		1,000
21	8	Aug. 10, 1931	1,800	1,300	100	200
21	With relief sewers		300	0		200
44B	9	Nov. 1, 1932	1,590	1,270	100	1,100
44B	With relief sewers		240	0		1,100
13	10	June 25, 1933	1,400	1,200	100	200
13	With relief sewers		120	0		200
32	11	June 17, 1930	1,230	1,160	100	150
32	With relief sewers		130	0		150
12	12	June 8, 1933	1,390	1,150	100	500
12	With relief sewers		220	0		500
4	13	May 27, 1932	1,230	1,070	100	70
4	With relief sewers		60	0		70
5	14	Sept. 23-4, 1932	1,060	1,000	100	70
5	With relief sewers		70	0		70
41	15	Sept. 23, 1931	890	915	100	40
41	With relief sewers		40	0		40
6		May 12, 1932	1,110	290	100	920
6	With relief sewers		700	0		920
46		Nov. 9-10, 1932	500	320	100	690
46	With relief sewers		100	0		690

Chapter VI

COMPARISON OF PROJECTS FOR ABATEMENT OF POLLUTION OF ROCK CREEK—COST ESTIMATES

1. The sources of pollution of Rock Creek by domestic sewage may be classed into three geographical sections:

(*a*) Direct discharge of sewage from a relatively small population in the large area of the basin above the District of Columbia not now served by the separate system of sewers. This includes, in part, the Washington suburban sanitary district (fig. 1).

(*b*) Sewage discharged into Rock Creek in times of rainfall from the area now served in part by combined sewers in the upper part of the District of Columbia above the mouth of Piney Branch. This includes the areas of Klingle Valley, Luzon Valley, and the Army Medical Center.

(*c*) Sewage discharged, in times of rainfall, from the densely populated combined sewer areas of all that part of the Rock Creek Basin covered by the Piney Branch, northwest boundary and Slash Run areas on the east and all of the west part of the basin from Connecticut Avenue to the Potomac. (See fig. 2.)

2. *Upper section.* (*a*) The existing trunk sewers in the Washington suburban sanitary district and the additional construction immediately needed to remove sewage from Rock Creek at the District line, are shown on figure 1 reproduced from a map furnished by the chief engineer of the suburban district. It is understood that the estimated cost is under $200,000. So much of this work as is now needed to keep sewage from reaching the creek should be carried out simultaneously with remedial projects within the District. (See ch. II, par. 3.)

3. *Section* (*b*)—*Conversion of the subdistricts to the separate plan.*—Above the Piney Branch Parkway all sewer systems draining to Rock Creek Park have been constructed on the separate plan, excepting the large areas known as the Luzon and Klingle districts. For several years a conversion of these last two districts to the separate plan has been under consideration by the sanitary engineer of the District of Columbia. The matter has recently been the subject of conference by the consulting engineers, the sanitary engineer of the District, the engineers of the National Parks and Planning Commission and the assistant chief engineer of the National Park Service. After full examination of all of the circumstances we are of the opinion that such conversion to the separate system is essential to the development of sanitary conditions within Rock Creek Park, and is more economical than the extension of additional relief sewers to these upper areas.

Estimates of the cost of the work to be done in these districts have been prepared separately by the Engineers of the District of Columbia, and by your consulting engineers, and after conferences and modifications of estimates it is our judgment that the figures given below should be accepted as the probable cost of this work.

It should be understood by all agencies concerned that this work is essential to the sanitary improvement of Rock Creek Park, and must be included in a comprehensive plan. It is our understanding, however, that any allotment for this purpose would be made to the District of Columbia authorities.

4. *Estimate of cost, Klingle and Luzon separate systems.*—Following is our estimate of cost for completing the conversion of the Klingle and Luzon and Army Medical Center areas to a separate system of sewerage:

Luzon Valley—Estimate of cost

Storm water sewers:	
515 lineal feet 21-inch sewer, at $9	$4, 635
4, 585 lineal feet 18-inch sewer, at $8	36, 680
5, 115 lineal feet 15-inch sewer, at $7	36, 050
1, 490 lineal feet 12-inch sewer, at $6.30	9, 387
11, 705	
44 manholes, at $125	5, 500
Total for storm sewers	92, 252
Sanitary sewers:	
10, 295 feet 8-inch sewer, at $5.25	54, 049
450 feet 10-inch sewer, at $5.75	2, 588
10, 745	
40 manholes, at $125	5, 000
Total for sanitary sewers	61, 637
House connections:	
325 6-inch house connections, at $350	113, 750
Total for Luzon Valley	267, 639

Klingle and Luzon Valleys—Estimate of cost

Storm water sewers:	
2, 090 lineal feet 18-inch sewer, at $9	$18, 810
2, 980 lineal feet 15-inch sewer, at $8	23, 840
4, 710 lineal feet 12-inch sewer, at $7. 25	34, 148
9, 780	
42 manholes, at $150	6, 300
Total for storm sewers	83, 098

Klingle and Luzon Valleys—Estimate of cost—Continued

Sanitary sewers:	
1, 680 lineal feet, 18-inch sewer, at $9	$15, 120
3, 570 lineal feet, 12-inch sewer, at $7.25	25, 882
60 lineal feet, 10-inch sewer, at $6.50	390
5, 310	
22 manholes, at $150	3, 300
Total for sanitary sewers	44, 692
House connections:	
234 6-inch house connections, at $450	105, 300
Total for Klingle Valley	233, 090
Total for Luzon Valley	267, 639
	500, 729
Plus 10 percent for engineering	49, 271
Grand total for Klingle, Luzon, and Army Medical Center	550, 000

5. The procedure to be adopted with reference to abatement of pollution in Rock Creek under headings (*a*) and (*b*) is very definitely established.

6. *Section* (*c*), *Abatement of pollution from the combined sewer area of Piney and the lower basin.*—This section of the work is open to a wide variation in procedure, choice of projects, merits and objections to location, and costs of construction. Your engineers have considered the installation of the separate system. The present state of high development of this section would necessitate an expense for the separate system materially greater than other alternatives and would incur greater construction nuisance to the public. The use of local sewage treatment is out of the question. Detention storage of storm water has been considered as a factor to reduce cost. Sanitary objections in this section preclude such use.

7. *Relief sewers.*—By a process of elimination of other possibilities, your engineers find that the abatement of pollution of Rock Creek from the combined sewers can be best secured by the installation of relief sewers diverting the polluted storm water from Rock Creek and discharging it into the Potomac near the mouth of Rock Creek. The extent to which such relief diversion should be made is covered in chapter V–C. The dry-weather raw sewage will be carried to the Anacostia station by interceptors as at present.

8. *Outfall location.*—Your engineers find no occasion, at least for the present, in carrying the large volume of storm-water sewage beyond the Potomac outfalls. Should future conditions warrant further or other disposal then the outfalls as proposed herein may be extended or adjusted to such future plan.

Consideration has been given to the discharge of both the main and the west-side relief sewers directly into Rock Creek in the vicinity of L Street, as the creek below this point does not enter into any recreational use and is bordered by industrial property on one side. There is considerable justification for placing the outlets at this point in view of the fact that neither outlet will discharge dry-weather sewage at any time. It should be further noted that the extension of the west-side sewer to the Potomac involves some difficulty in the crossing of the Chesapeake & Ohio Canal, although this has been solved in a fairly satisfactory manner.

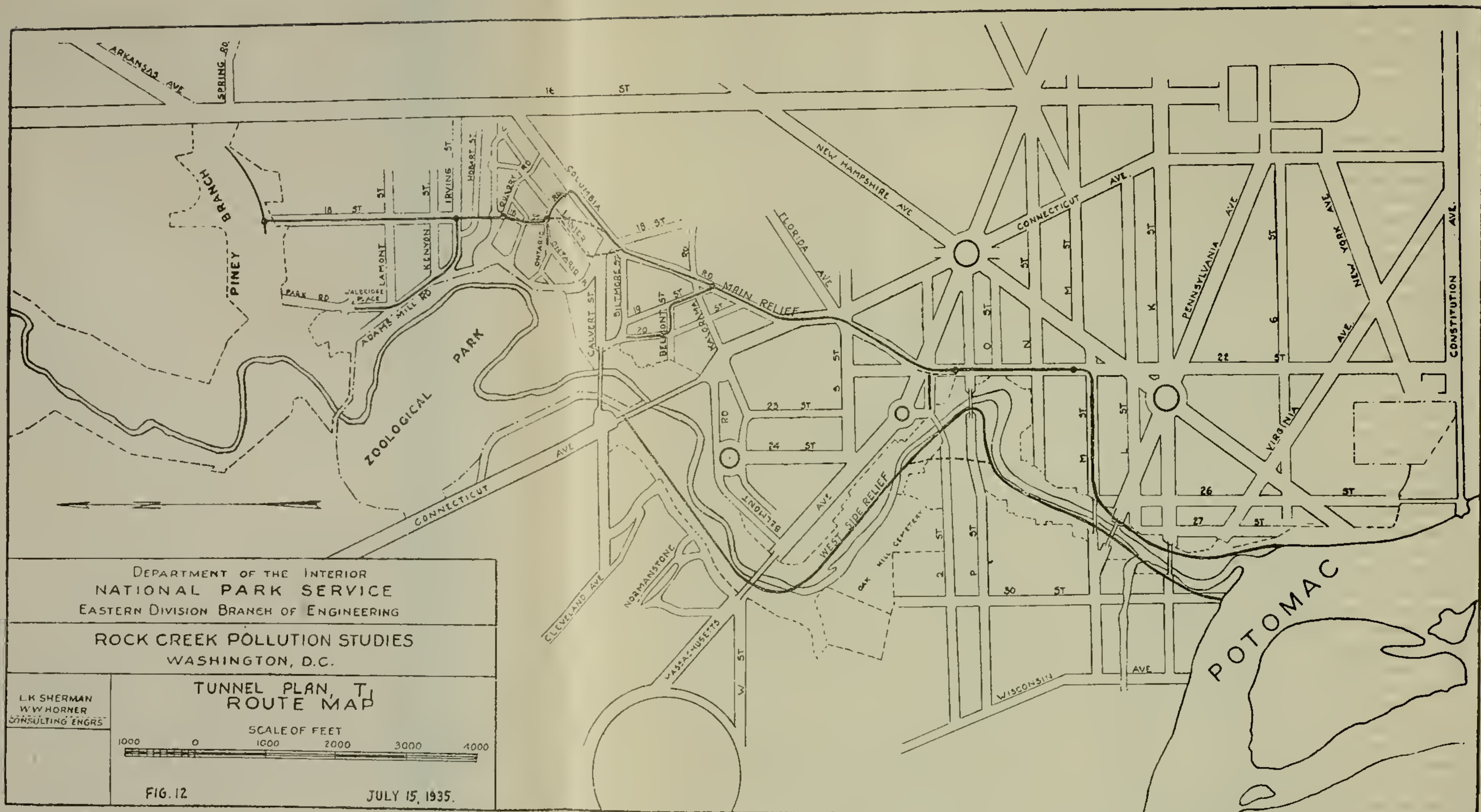
However, the slack water of lower Rock Creek would tend to cause sludge deposits; and in view of the possibility of eventual construction of submerged outlets to obtain the highest possible sanitary improvement, it is our recommendation that the outlets be at the Potomac River bank.

The location of the outlets at L Street would reduce the total cost of the project by about $130,000. If it eventuates that the total cost of the project must be in some manner reduced, then we would recommend as a first consideration a change in the outlet location.

9. *Selection of relief sewer routes.*—There has been discussed in some detail in earlier chapters the inadequacy of the existing Piney Branch and Rock Creek intercepting sewers and the frequent overflows of sewage on that account into Piney Branch and into Rock Creek below Piney Branch. Earlier reports already referred to have visualized that this nuisance could be most economically abated by the construction of relief sewers along Piney Branch and along the Rock Creek Valley below Piney Branch. Your consultants, after detailed studies outlined in preceding chapters, have arrived at the conclusion set out in these chapters as to the necessary capacity of any relief sewers to accomplish this purpose. The further discussion in this chapter relates to studies in the development of relief sewer projects which will carry out this program.

10. In the Eddy, Greely, and Gregory report there is a recommendation for the provision of the relief sewer in tunnel from the mouth of the Piney Branch sewer to the Potomac. While no specific mention is made in that report, it is clear that a complete sanitary development will require also a supplementary sewer for drainage basins on the west side of the valley. The report of the engineers of the Public Works Administration contains a suggestion for a relief sewer in the Rock Creek Valley itself. It appears that this sewer was presumed to be serviceable as an outlet for the subdistricts on each side of the valley. Our studies show that it is not feasible to provide a single sewer that will give such service to both sides, in that laterals from one side must pass under Rock Creek, and both the new sewer and the connecting laterals must be so designed as to grade and depth as not to interfere in a critical way with the old intercepting system or with the flow of the creek itself.

Your consulting engineers have examined each of the possibilities in detail, together with combinations of the two, and have determined that there is no other satisfactory plan for sanitary improvement that would accomplish the necessary end at a reasonable cost.



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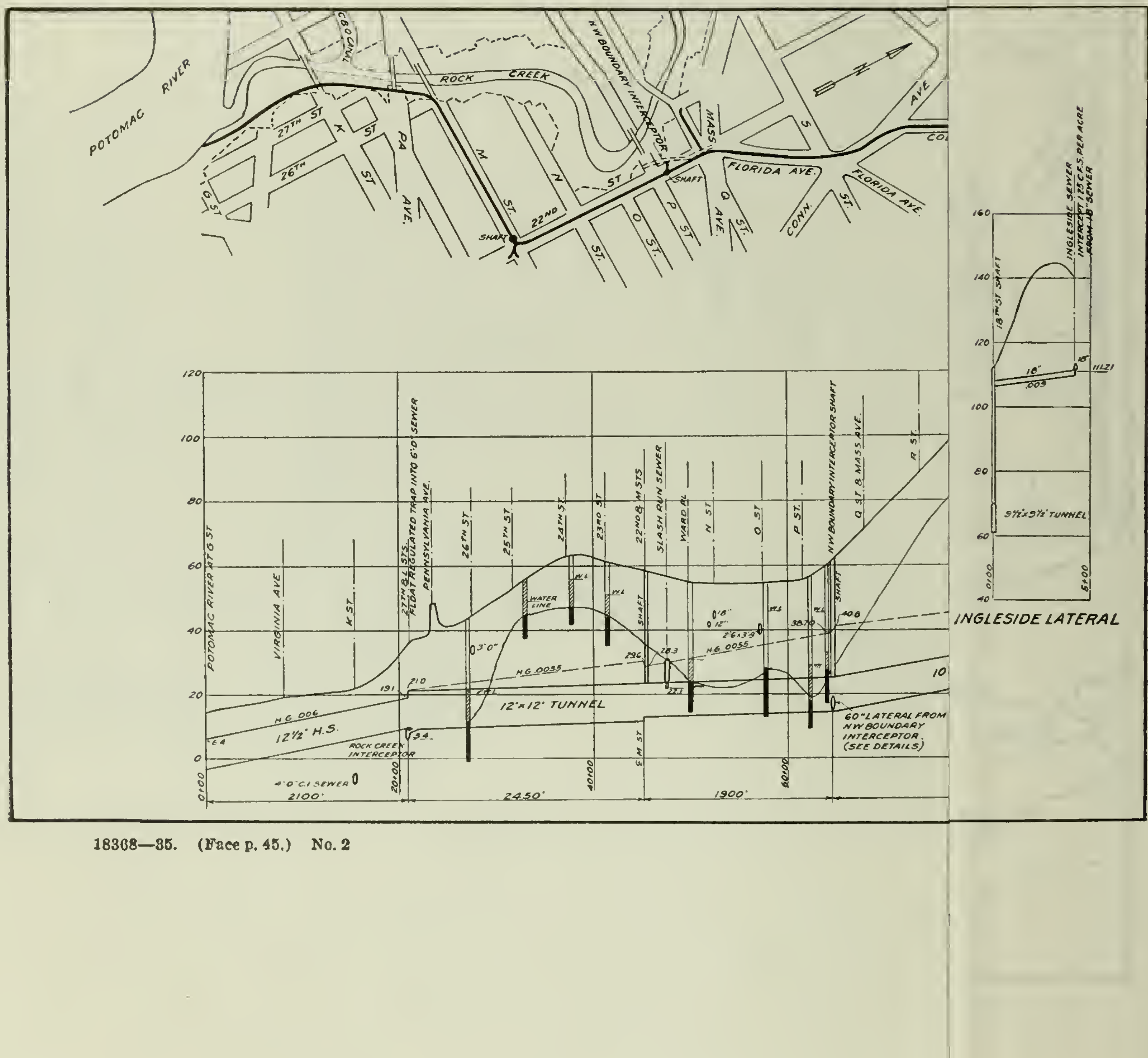

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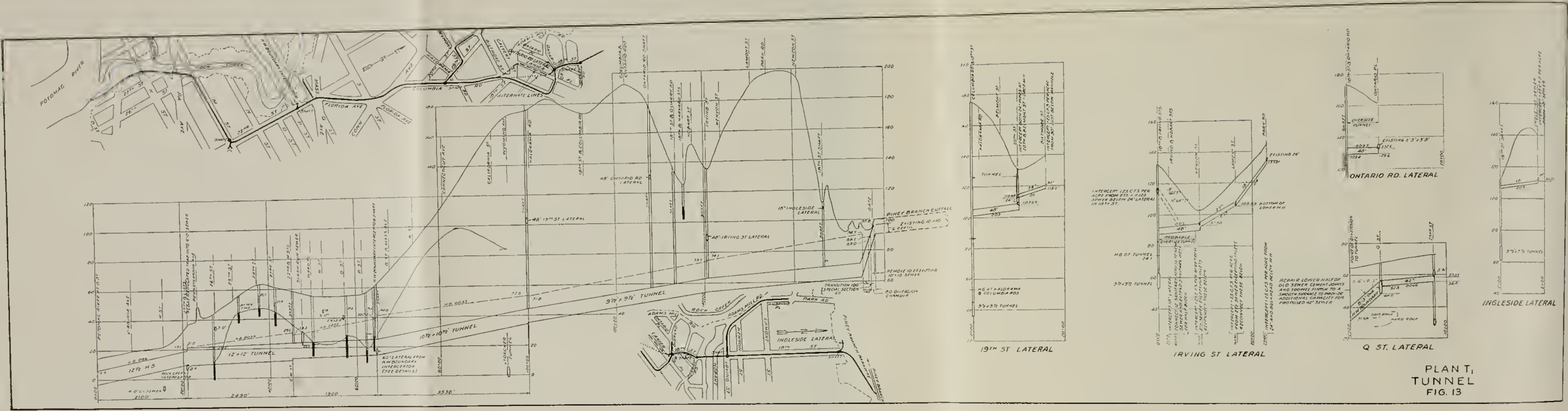
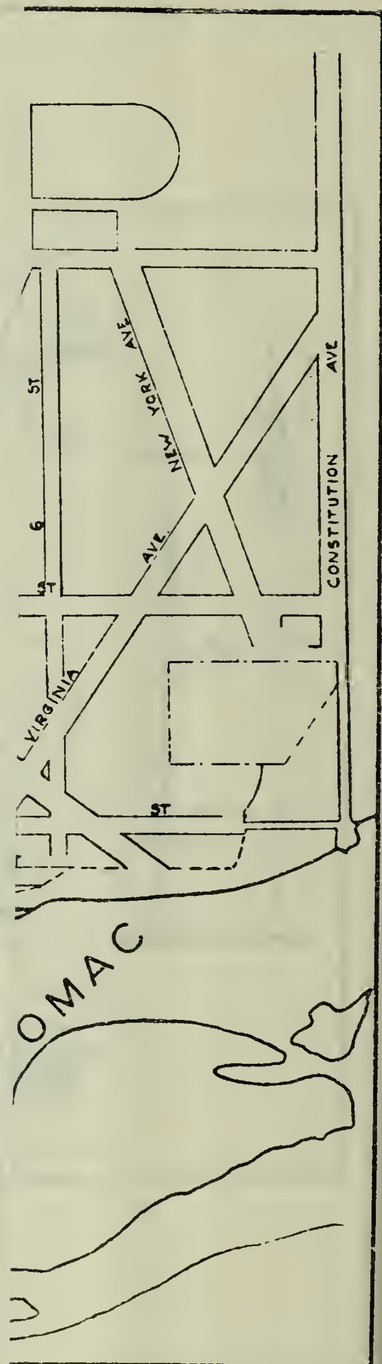


PLAN T1
TUNNEL
FIG. 13


NEW YORK AVE
CONSTITUTION AVE
VIRGINIA AVE
6 ST
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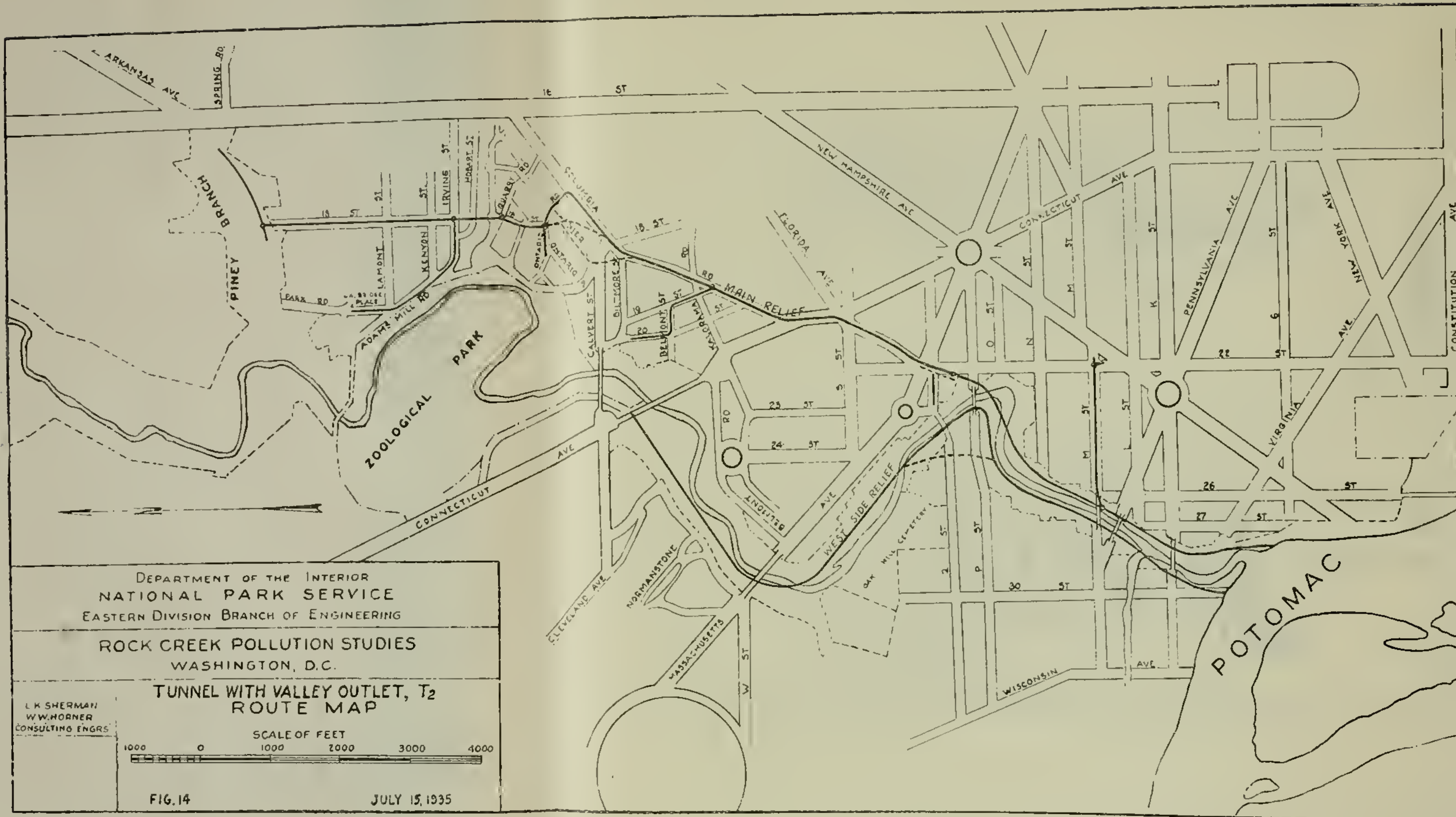

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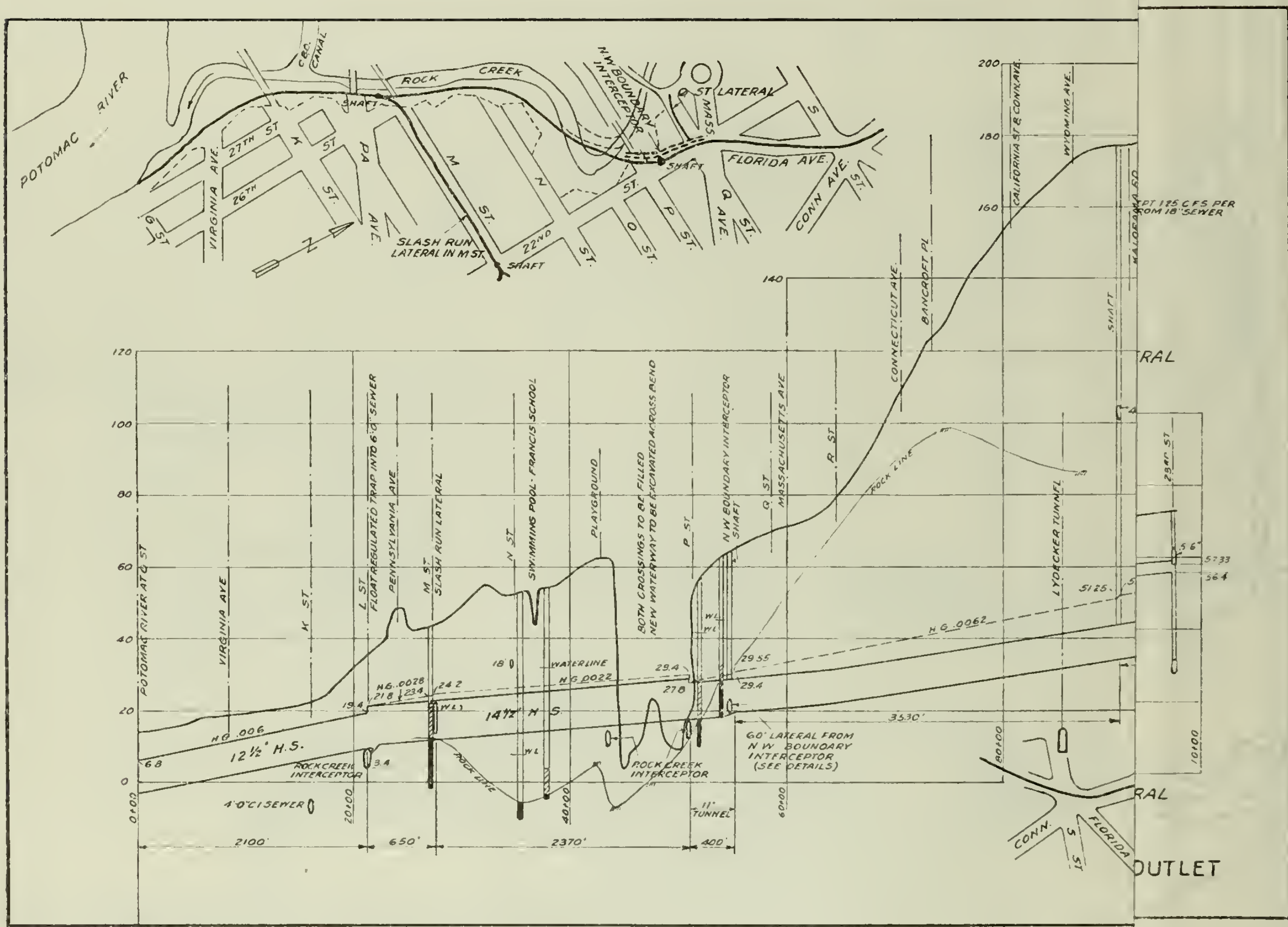

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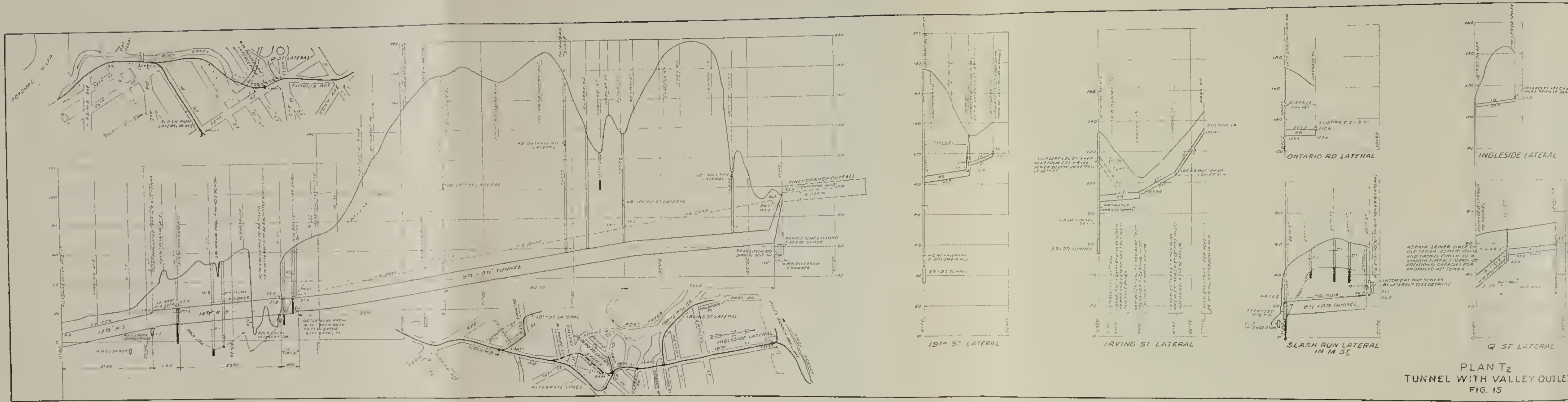
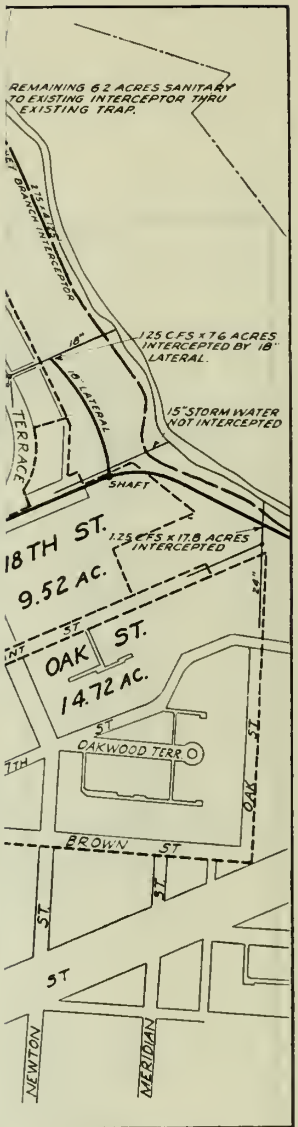


PLAN T_2
TUNNEL WITH VALLEY OUTLET
FIG. 15



...ARTMENT OF THE INTERIOR
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...IVISION, BRANCH OF ENGINEERING

...EK POLLUTION STUDIES
...WASHINGTON, D.C.

UPPER LATERALS OF THE TUNNEL PLANS - T_1 & T_2

SCALE OF FEET

200 0 200 400 600

FIG. 16 JULY 15, 1935

11. The detailed studies described in this chapter have involved the development of specific relief projects of either the tunnel or the valley-line type. We have discussed a number of practical routes under each of these plans, and in some instances have studied plans which involve the combination of the two basic ideas under which the suggested relief sewers would be constructed in tunnel throughout some part of their length and in open cut in others. After the possibilities had been examined in detail, it developed that any project of either type would involve in some part sections of the alternate type.

The plans which have received serious consideration are outlined in the following chapters.

12. *Scheme T–1, tunnel line.*—This proposal involves the construction of the main relief sewer for the east side of the valley in tunnel to the greatest possible degree.

As shown on map figure 12 and profile figure 13 the tunnel extends from the outlet of the Piney Branch sewer southward under Eighteenth Street; thence under Columbia Road, Twenty-second and M Streets to M and Twenty-sixth Streets. From this point it is built in open cut along the east bank of Rock Creek to the Potomac.

The tunnel route shown north of M Street has been selected, after consideration of a number of other possible lines, as being the most direct route possible within the zone which contains rock satisfactory for tunneling. The selection of this exact route was prompted somewhat by the necessity for arranging branch lines to service the small-sewer districts on the upper east side of the valley. Because of the necessity for making diversions from the outlets of certain of these districts, it was obviously desirable to keep the tunnel route as close to the edge of the valley as reasonable assurances of rock conditions would permit.

The branch diversion plans are shown on the plat above referred to and are indicated in more detail on the profile of scheme T–1, figure 13, and in the diversion map, figure 16, and the diversion table 17. Further information as to the main sewers in this group of districts and the detailed scheme for making diversions from them is shown on the general map of the area, figure 16; and the design capacities involved are shown in table 13.

Under this scheme T–1 all of the proposed sewers will be located either in park property or in public streets with the exception of the two-block section south of Irving Street where a right-of-way across private property for a deep rock tunnel should be secured at a nominal cost. The location of the line in Columbia Road or the possible alternate in Lanier Place shown in dash lines is proposed to avoid an excessively long diagonal right-of-way under expensive buildings. If right-of-way can be reasonably acquired, the short diagonal line, dotted on the maps, will reduce construction costs about $40,000.

The section to be constructed in the Piney Branch Parkway would be in tunnel, and this would avoid the disturbance of the new park road now being built along this route, except in the immediate vicinity of the diversion point. The proposed construction grade of the tunnel can be lowered in this vicinity if detailed examinations of the rock surface do not develop satisfactory rock cover at the grades shown.

The estimated cost of scheme T–1, including the west side relief, is $2,693,990. The detailed estimate is given in paragraph 17.

13. *Scheme T–2, tunnel with valley outlet below P Street.*—The general plan of scheme T–2 is identical with that of T–1 except as to the route below P Street. In scheme T–2 this section would be constructed in open cut along the high level bank of Rock Creek. The location of the main in this position involves some expensive construction but no serious difficulties, although because of rubbish filling in some places the sides of the trench would have to be supported with sheet piling. An unusual feature of this section is the proposal to construct the sewer on viaduct across the bend of Rock Creek below P Street, to later fill this bend and to provide a cut-off channel for the creek itself immediately west of the sewer. Under this plan the water diverted from the Slash Run sewer must be intercepted at Twenty-second and M Streets and a lateral sewer constructed in rock tunnel from this point westwardly in M Street to the main line. The diversions at Twenty-second and M Streets will have to be made under difficult working conditions but the streets can be kept open to traffic, except for very short periods of time.

The detailed location, profile, and grades of the main relief sewer under scheme T–2 are shown on figures 14 and 15 and in design table 14. The west side relief sewer for T–2 is the same as in T–1 and is shown in plan and profile on figure 20.

The tunnel lines, being of necessity far enough east of Rock Creek to lie in solid rock, must be supplemented by laterals extending west to the proper points of interception along the creek bank. The upper laterals are identical for plans T–1 and T–2 and are shown on figure 16.

At northwest boundary the laterals are made almost identical by diverting the tunnel line slightly from the center of Twenty-second Street, and allowing the intake shaft to be sunk in the landscaped triangular plat north of P Street under either plan.

Plan T–1 intercepts Slash Run at Twenty-second and M Streets without the aid of a lateral. (See fig. 25.) Plan T–2, having the valley outlet below P Street, requires the same Slash Run lateral as that used with the valley line. (See fig. 26.)

The estimated cost of this plan, including the west-side relief, is $2,600,090 for construction conditions such as have prevailed on P. W. A. projects during the past 2 years. The detailed estimate is given in paragraph 18.

14. *Valley plan (V)*.—The two preceding proposals, given as plans T–1 and T–2, involve the construction of relief sewers in tunnel to the greatest possible extent, and are particularly desirable, in that they accordingly avoid to the maximum degree interference with Rock Creek Park. They are objectionable in that they involve tunneling through residential districts of high value, with considerable disturbance to these areas on account of blasting and because of the handling of muck and concrete materials at shaft heads located in these districts. In general, also, it may be accepted that the cost of such tunnels per lineal foot is somewhat greater than the cost of sewers constructed in open cut under reasonably satisfactory working conditions.

The plan described here and shown on figures 17, 18, 19, and in design table 15, is, to a considerable extent, the scheme originally studied by the P. W. A. engineers (ch. IV, par. 8), the principal differences being in the location of the open-cut section below P Street and in the moving of a tunnel proposed by them in Twenty-fourth Street to Twenty-third Street and Massachusetts Avenue.

The original plan involved a tunnel in Twenty-fourth Street and a crossing to the west bank of Rock Creek immediately below it. This was abandoned because of the necessity of either recrossing to the east bank at an elevation high enough to allow the foul flow to be trapped into the existing interceptor, or of pumping this flow across Rock Creek, the one impossible and the other expensive.

For this original plan was substituted a Twenty-fourth Street tunnel with an open-cut section below it along the east bank of Rock Creek as far as P Street. This design escaped the difficulties pursuant to crossing to the west bank but had to be abandoned in turn, because the sewer grades, as economically worked out, would have required the raising of the low park road for a long distance above and below the Q Street Bridge and a corresponding raise in the expensive retaining wall along Rock Creek. This route was further objectionable because the sewer excavation would have had to be carried out at the foot of a sliding hillside, on the top of which are a number of expensive residences fronting on Massachusetts Avenue. It was felt that safe construction at the foot of this hill would involve extraordinary items of expense, that under any precautions some further movement might result, and that the sewer construction might, accordingly, be held responsible for serious damage to the residential properties.

The third and recommended plan is to move the tunnel into Twenty-third Street, as stated above, and continue it in Massachusetts Avenue and Twenty-second to P Street.

This tunnel is the lower one of three short tunnels included in this so-called "valley plan." The other two merely cut across sharp bends in creek in order to save length. This valley plan actually includes 7,450 feet of tunnel in a total length of 17,620.

The open-cut work in the parked portion of the valley is divided into three sections. In the lower of these, between Kalorama Circle and Calvert Street, affects a part of Rock Creek Park devoted largely to picnic and riding activities. Few valuable trees will have to be sacrificed, but a serious construction situation will exist for a distance of about 300 feet just south of Calvert Street, where the only available location is behind a retaining wall along Rock Creek, and at the foot of a high retaining wall against the bluff, the latter wall being in bad condition.

The second section of open-cut work centers on the Harvard Street entrance to the Zoo, and involves no difficulties except the street crossing, where traffic will undoubtedly have to be maintained. North of Harvard Street a few good trees must be sacrificed.

The third open-cut section will be located in the Piney Branch parkway, and because of the narrowness of this side valley, the sewer must actually be built, for the greater part of its length, in the park road now being paved. The estimates include an item for the removal and replacement of this pavement.

Scheme V–1 involves the construction of the west-side relief sewer, as shown on figures 20 and 21, identical with that included in schemes T–1 and T–2. The cost of scheme V–1 valley line is $2,519,820. (See details in par. 19.)

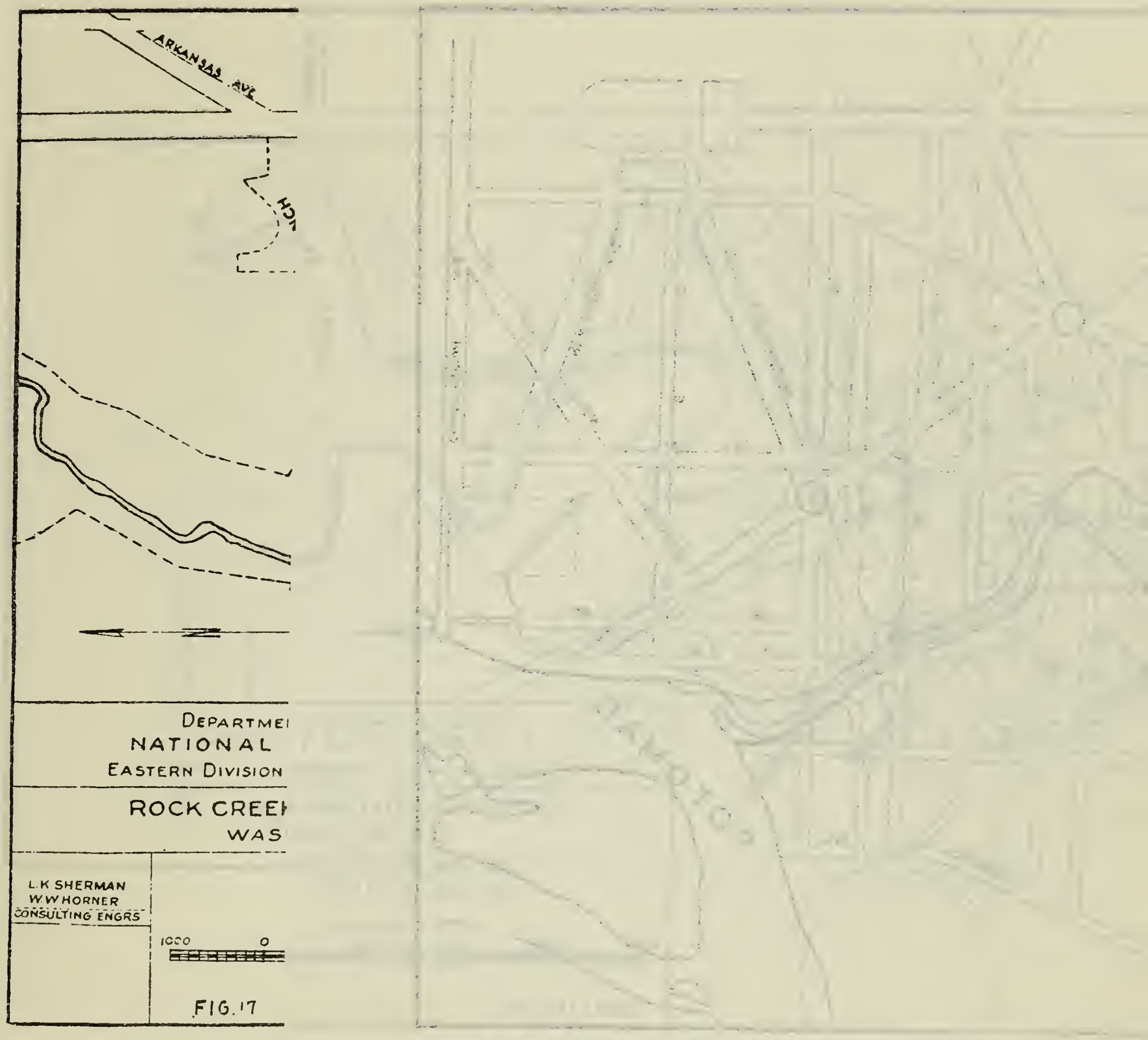
Your consulting engineers recommend the adoption of this plan, V–1, in preference to all the other plans herein considered.

15. *West-side relief sewer*.—A west-side relief sewer is needed with all plans in order to divert polluted run-off from the combined sewer areas on the west below Connecticut Avenue.

The recommended route is shown in plan on figure 21 and in profile on figure 20. The design quantities are shown in table 16.

As it is impossible to construct this line through the Oak Hill Cemetery property, the route involves two crossings of Rock Creek; but through careful adjustment of grades and a provision for using special sections at the creek, this is done in a manner entirely unobjectionable. In connection with the upper of the two crossings, advantage has been taken of the situation to construct a new crossing for the old Rock Creek interceptor, thereby removing the four-pipe crossing which was one of the bottle necks of the old interceptor. (See fig. 33.)

Below the lower crossing of Rock Creek, just above Q Street, the line follows the west bank of Rock Creek


ARKANSAS AVE
DEPARTME
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WAS
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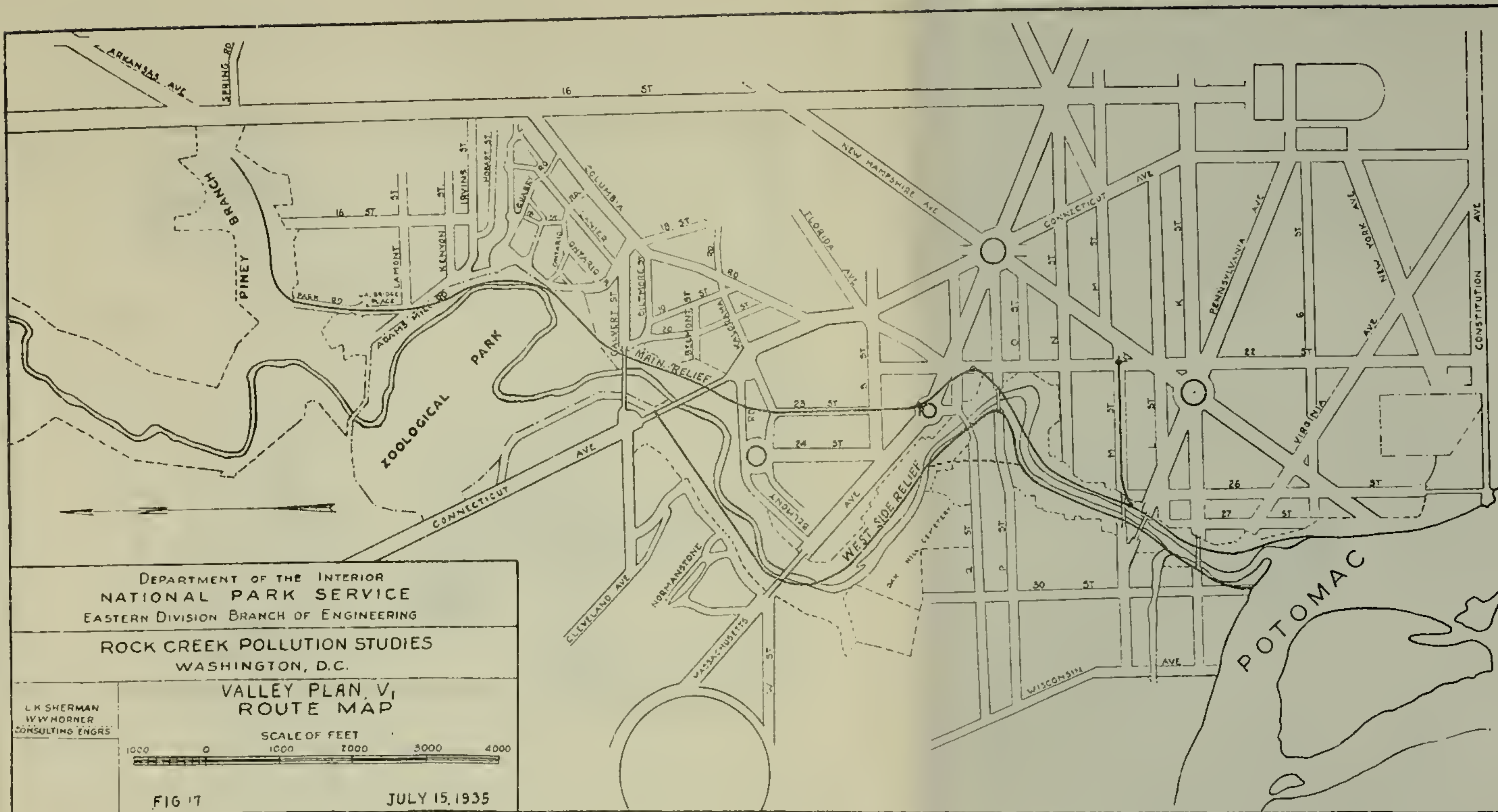
FIG. 17



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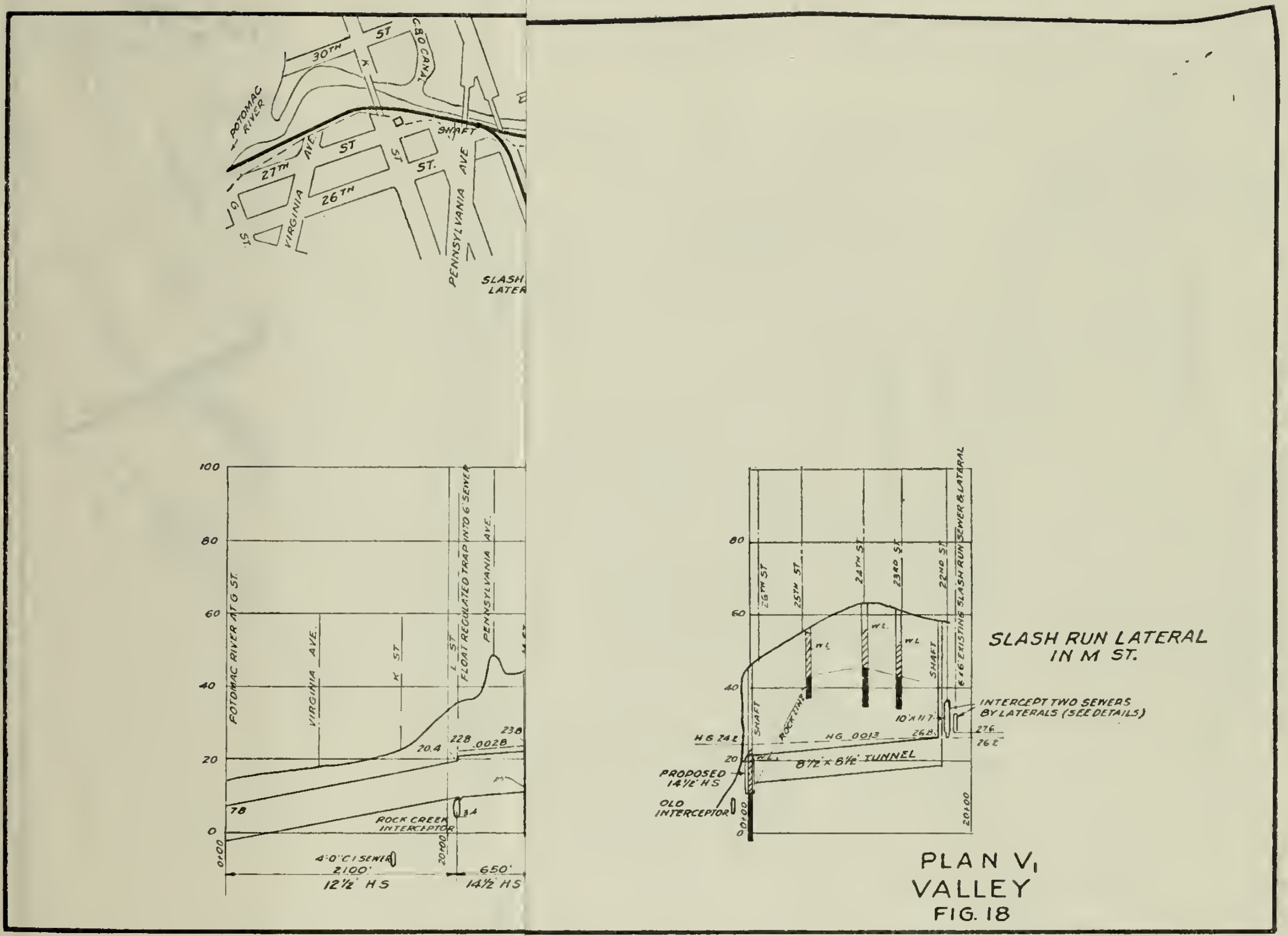

PLAN V_1
VALLEY
FIG. 18

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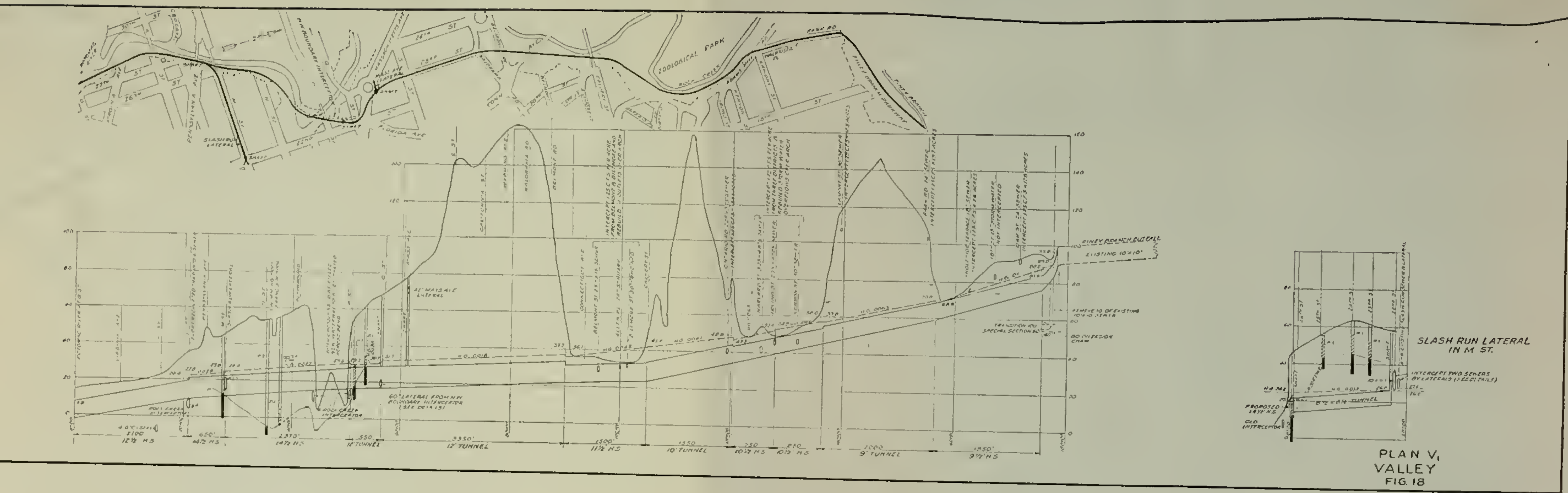
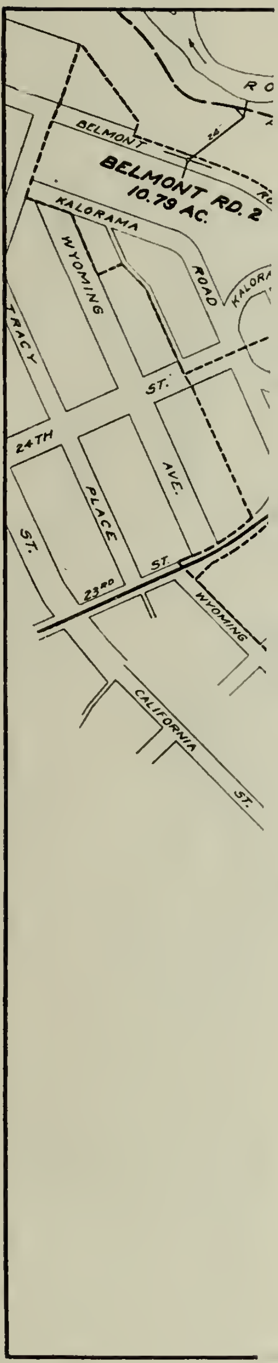
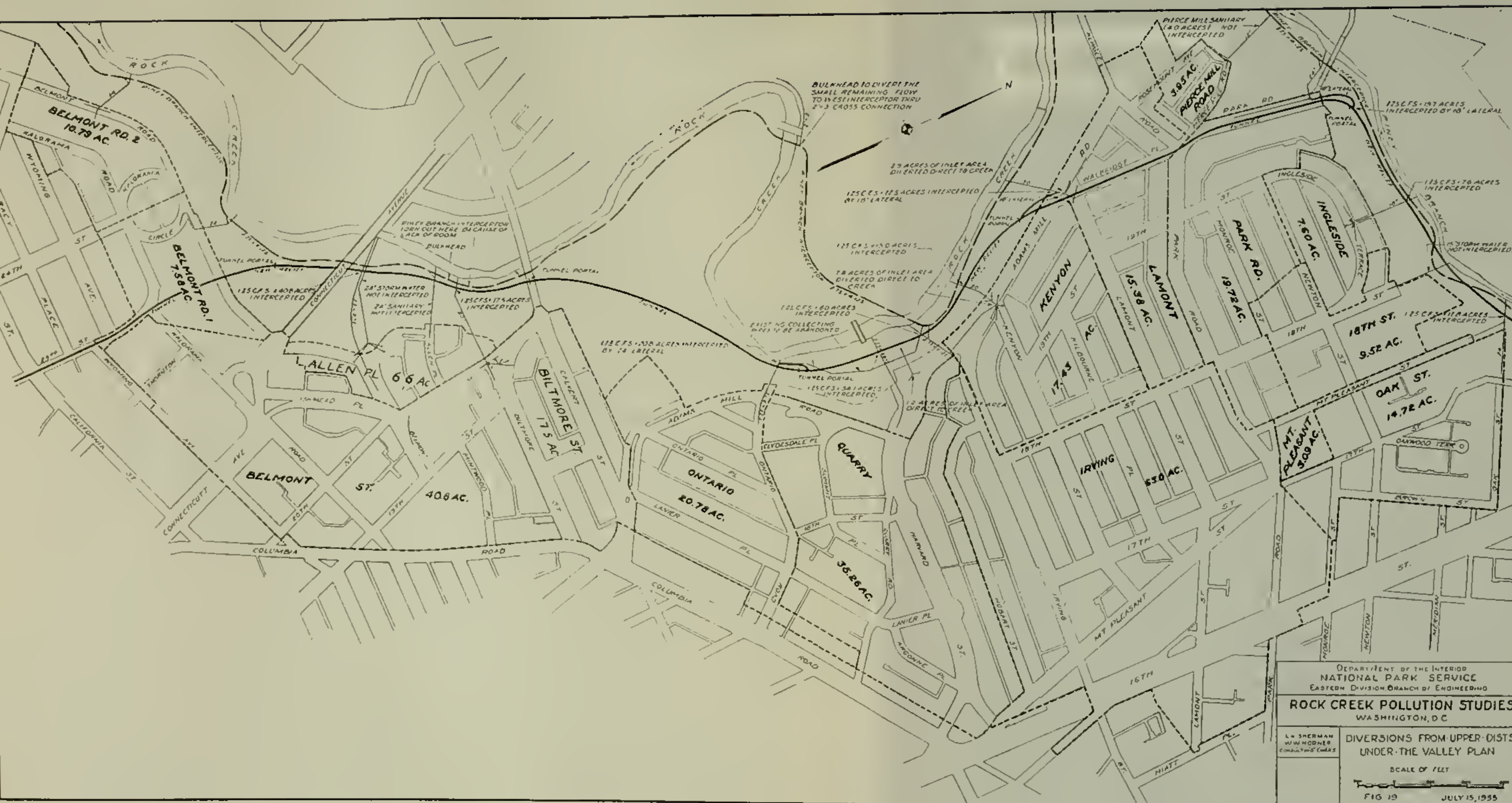
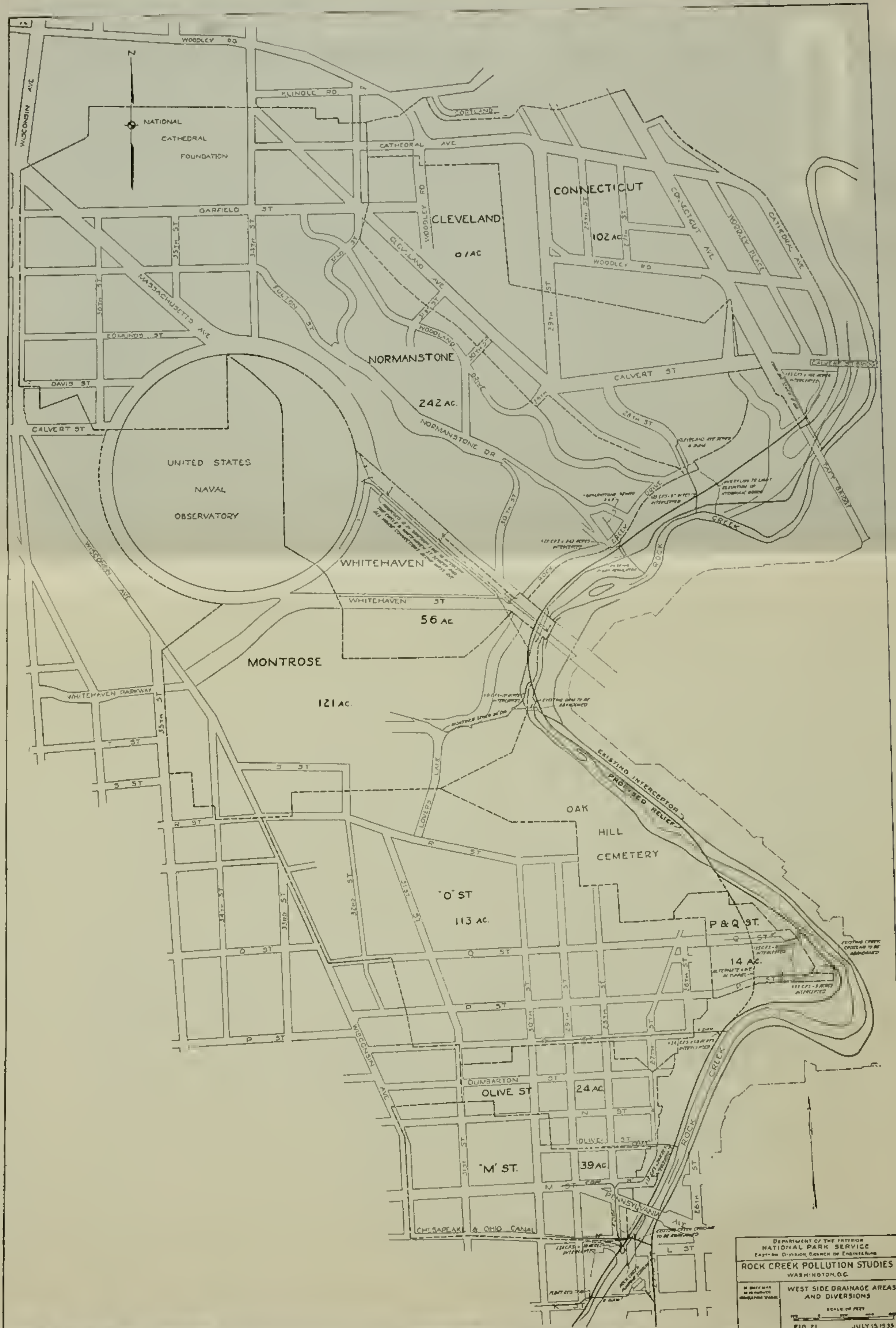


PLAN V1
VALLEY
FIG. 18


BELMONT
BELMONT RD. 2
10.79 AC.
KALORAMA
WYOMING
ROAD
TRACY
ST.
24TH
PLACE
AVE.
ST.
23RD
ST.
WYOMING
CALIFORNIA
ST.



18368—35. (Face p. 46.) No. 3



18308—35. (Face p. 47.)

and enters the Potomac at Thirtieth Street. In crossing under the old Chesapeake & Ohio Canal, the section is flattened and widened by gradual transitions.

The estimated cost of the west-side relief sewer is $418,770. (The detailed estimate is given in par. 20.)

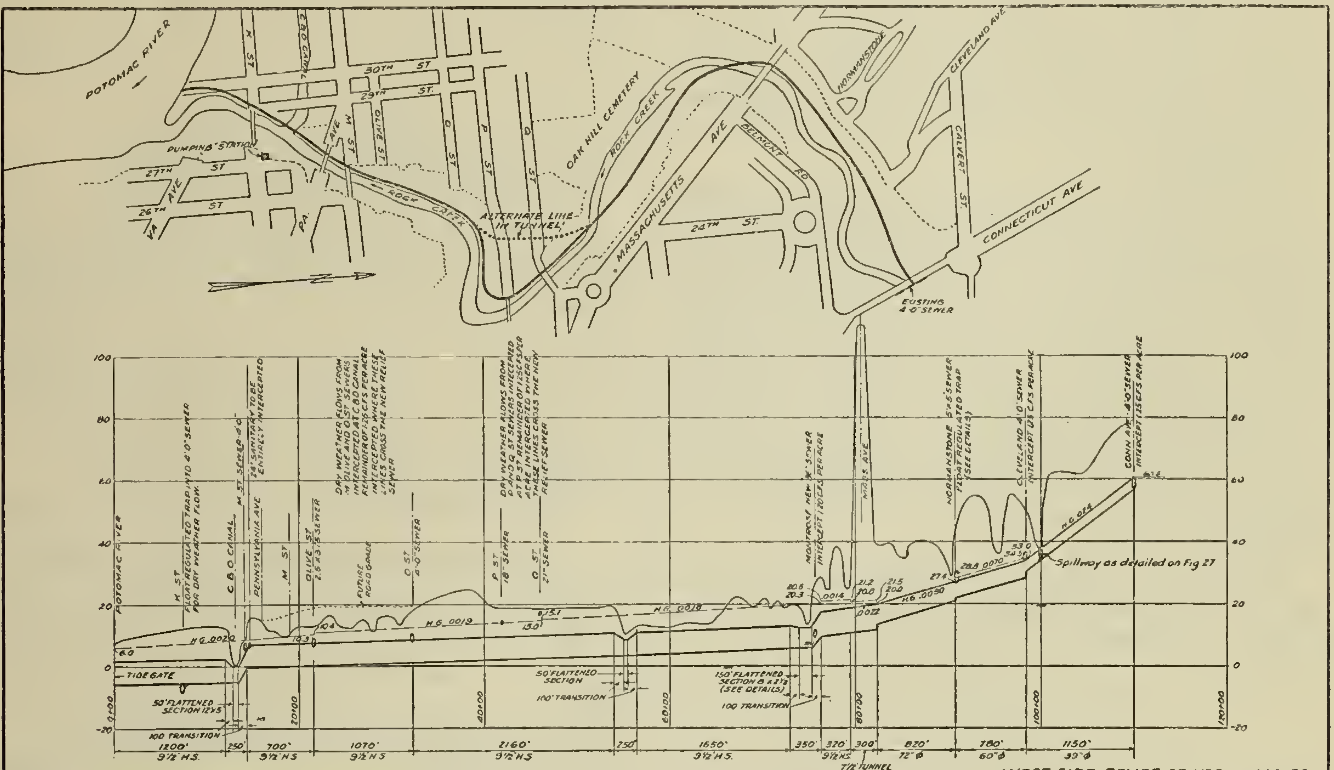
Alternate west-side relief with tunnel.—An alternate to the foregoing plan is to shorten the line by a tunnel cut-off across the bend at P and Q Streets as shown by the dotted line on the same map, figure 21. This alternate avoids some bad side-hill construction along the creek near Q Street, but involves the construction of a small interceptor to divert the flow from existing west-side sewers in P and Q Streets at a point near Rock Creek and carry it around the lower half of the bend to the tunnel portal. It is more expensive than the longer line by $40,000.

The Whitehaven district, which lies along Massachusetts Avenue between Rock Creek and the Naval Observatory, is definitely recommended for conversion to the separate system since this conversion can be accomplished very simply as shown in figure 32. The dry-weather flow of this area is at present trapped high up in the Massachusetts Avenue fill over Rock Creek and crosses this fill to the existing Piney Branch interceptor, while the remainder of the flow drops vertically into Rock Creek through a hole in the brick culvert arch. The increased interception necessary to keep light rains out of the creek cannot be carried by either of the existing interceptors below this point and it is cheaper to lay the 1,900 feet of 12-inch pipe required to convert this district to the separate system than to construct an overflow trap for the moderate rains and carry this water to the Potomac in the new sewer.

The estimate for this work is included with that of the west-side relief sewer.

16. *Plan V–2.*—The west-side relief sewer collects polluted run-off from Connecticut, Cleveland, and Normanstone districts at three points near its upper end. It then receives only one further addition, the Montrose sewer, until it reaches Q Street almost a mile below the Normanstone outlet.

Your consulting engineers have attempted two different plans for the elimination of this uneconomical upper length of sewer. The better of the two is combined with the main valley line and referred to as plan V–2. The two plans are as follows:



WEST SIDE RELIEF SEWER – FIG. 20

(*a*) The conversion of the four upper districts to the separate system, which would eliminate the west-side relief above Q Street.

The conversion of the Connecticut, Cleveland, Normanstone and Montrose districts to the separate system will cost about $700,000. This amount exceeds the saving to be made in the west relief sewer and the scheme was therefore abandoned.

(*b*) The siphoning of the polluted flow from the three upper districts across Rock Creek to the valley line, which would eliminate the west-side relief above the Montrose outlet. This expedient is not possible under the tunnel plans on account of the high gradient and the remoteness of the tunnels from the creek.

A plan and profile have been prepared for an outlet siphon crossing the valley from the three upper districts to the tunnel portal of the valley line north of Kalorama Circle. Because of the low lying level of parts of the Normanstone drainage area, and because the design of scheme V–1 involved a hydraulic gradient for the main valley line appreciably above the top of the proposed main sewer, very little fall is available for use in the hydraulic grade of this outlet sewer. Even though the Normanstone sewer is backed up some distance above its outlet, the resulting grades are very flat and the sewer is unusually large in comparison with the volume of flow to be carried.

The profile, figure 31, shows also that this sewer must be constructed as a siphon under Rock Creek, the siphon involving a very considerable depth and as the sewer would be used for the contaminated storm flow from the three districts, it would not be in continuous service and the siphon would require clean-out pumps and maintenance.

The cost of the collecting sewers, the siphon, and the necessary pumps and trap alterations, together with the extra size necessary on the lower valley line is estimated at $220,900. (The detailed estimate appears in par. 21.) The saving due to the elimination of the west side relief above Montrose is only $192,160 ($418,770—$226,610) and therefore, the scheme is not economical. It is cheaper to carry the west-side water down the west side.

This variation, however, has been called plan V–2 and is shown as a possible alternate, even though it is not recommended.

The total cost of plan V–2 including the short west-side relief below the Montrose Dam is $2,547,560 as shown in paragraph 21.

17. *Plan T–1—Estimate of cost of tunnel relief sewer and of west-side relief sewer*

Total estimate cost of tunnel relief sewer including right-of-way	$2, 275, 220
Total estimated cost of west-side relief sewer, including right-of-way	418, 770
Total estimated cost of plan T–1	2, 693, 990
Engineering, 6 percent	162, 010
Repairs to parks and contingencies, 4 percent	107, 000
Total	2, 963, 000

Summary of cost estimate of tunnel (plan T–1)

2,100 feet, 12½-foot horseshoe section in open cut, at $55	$115, 500
900 feet, 12-by-12-foot tunnel section in open cut, at $130	117, 000
1,550 feet, 12-by-12-foot rock tunnel, at $155	240, 250
5,430 feet, 10½-by-10½-foot rock tunnel, at $130	705, 900
7,540 feet, 9½-by-9½-foot rock tunnel, at $110	829, 400
Special diversion chambers and slash run lateral	136, 000
Q Street lateral	12, 260
Nineteenth Street lateral	37, 000

Summary of cost estimate of tunnel (plan T–1)—Continued

Ontario Road lateral	$18, 500
Irving Street lateral	43, 410
	2, 255, 220
Right-of-way	20, 000
Total estimated cost of tunnel	2, 275, 220

Cost estimate of tunnel (plan T–1)

POTOMAC RIVER TO L STREET

2,100 feet of 12½-foot horseshoe section in open cut: Average cut 19 feet, width 16½ feet, 12 cubic yards per foot; excavation, $29 per foot, including digging, disposal, bracing, and pumping; sewer barrel, $26 per foot, contract price; 2,100 feet × $55	$115, 500

TWENTY-SEVENTH AND L STREETS TO TWENTY-SECOND AND M STREETS

900 feet or 12-by-12-foot tunnel section in open cut: Average cut 34 feet, width 16 feet, 20 cubic yards per foot; 20 cubic yards, at $5; $100 full-sheeted throughout sewer barrel $30 per foot; 900 feet × $130	$117, 000
1,550 feet of rock tunnel, 12 by 12 feet, including 1 shaft, at $155 per foot	240, 250
	472, 750

TWENTY-SECOND AND M STREET TO COLUMBIA AND KALORAMA ROADS

5,430 feet of rock tunnel, 10½ by 10½ feet (including 2 shafts), at $130 per foot	$705, 900

COLUMBIA AND KALORAMA ROADS TO PINEY BRANCH

7,540 feet of rock tunnel, 9½ by 9½ feet (including 4 shafts), at $110 per foot	$829, 400

SPECIAL DIVERSION CHAMBERS

Transition at upper end of 9½-by-9½-foot tunnel diversion chamber, connection to Piney Branch sewer and overflow	76, 000
Special construction where northwestern boundary sewer is intercepted	35, 000
Special construction consisting of sump diversion and gate at L Street	10, 000
Diversion of slash run sewer and lateral, at Twenty-second and M Streets	15, 000
	2, 144, 050

Q STREET LATERAL

540 feet of 42-inch circular sewer; average cut 16 feet, width 6 feet, 3½ cubic yards per foot; excavation $13 per foot including digging disposal bracing, temporary and permanent pavement; 42-inch concrete pipe, $6 per foot; 540 feet × $19	12, 260

19TH STREET LATERAL

1,000 feet of earth tunnel, 48-inch inside diameter, at $30 per foot	30, 000
600 feet of 24-inch sewer at $10 per foot	6, 000
Intercept 4 existing sewers and build overflows	1, 000

Cost estimate of tunnel (plan T–1)—Continued

ONTARIO ROAD LATERAL

600 feet of tunnel 48-inch inside diameter, at $30 per foot	$18, 000
Intercept 2 existing sewers	500

IRVING LATERAL

890 feet of tunnel 48 inch, inside diameter, at $30 per foot	26, 700
780 feet of 27-inch sewer at $7 per foot	5, 460
650 feet of 18-inch sewer at $5 per foot	3, 250
Intercept 4 existing sewers and build overflows	1, 000
Reconnect inlets below overflows	1, 000
New interceptor between Ontario and Irving Streets	3, 000
Rebuild 3 traps and lay larger connecting sewers	3, 000
	2, 255, 220
Right-of-way	20, 000
Total	2, 275, 220

18. *Plan T–2, estimate of cost of tunnel relief sewer with valley outlet and of west side relief sewer*

Total estimated cost of tunnel relief sewer with valley outlet including right-of-way	$2, 181, 320
Total estimated cost of west side relief sewer including right-of-way	418, 770
Total estimated cost of plan T–2	2, 600, 090
Engineering, 6 percent	156, 910
Repairs to parks and contingencies, 4 percent	104, 000
Total	2, 861, 000

Summary of estimate of cost of tunnel with valley outlet (plan T–2)

2,100 feet of 12½-foot horseshoe section in open cut at $55	$115, 500
2,270 feet of 14½-foot horseshoe section in open cut at $110	249, 700
750 feet of 14½-foot horseshoe section on concrete piers at $75	56, 250
Replacement of swimming pool, etc	20, 000
400 feet of 11- by 11-foot tunnel and transition at $142.50	57, 000
11,070 feet of 9½- by 9½-foot tunnel including shafts at $110	1, 217, 700
1,780 feet of 8½- by 8½-foot tunnel (slash run lateral) at $100	178, 000
Special construction (diversions, etc.)	136, 000
New open channel Francis School Playground to P Street	20, 000
Q Street lateral	12, 260
19th Street lateral	37, 000
Ontario Road lateral	18, 500
Irving Street lateral	43, 410
	2, 161, 320
Right-of-way	20, 000
Total estimated cost of tunnel with valley outlet	2, 181, 320

Estimate of cost of tunnel with valley outlet (plan T–2)

POTOMAC RIVER TO L STREET

2,100 feet of 12½-foot horseshoe section at $55 per foot	$115, 500

Estimate of cost of tunnel with valley outlet (plan T–2)—Con.

L STREET TO CREEK CROSSING

2,270 feet of 14½-foot horseshoe section in open cut at $110 per foot	$249, 700
Replacement of swimming pool and other incidentals in connection with above line	20, 000
750 feet of 14½-foot horseshoe section across creek on concrete piers at $75 per foot	56, 250
New open channel between Francis School playground and P Street with outside of curve gunnited for vertical height of 12 feet	20, 000

CREEK CROSSING TO NORTHWEST BOUNDARY SEWER

400 feet of 11- by 11-foot tunnel including transition section at $142.50 per foot	$57, 000

NORTHWEST BOUNDARY SEWER TO PINEY BRANCH

11,070 feet of 9½- by 9½-foot tunnel including 4 permanent shafts and construction shaft at $110 per foot	$1,217,700

SLASH RUN LATERAL AND SPECIAL CONSTRUCTION

1,780 feet of 8½- by 8½-foot tunnel including shafts at $100 per foot	$178, 000
Junction at upper end with sewers at 22d and M Streets	15, 000
Special construction consisting of sump, diversion, and gate at L Street	10, 000
Special construction where northwest boundary sewer is intercepted	35, 000
Transition section at upper end of 9½- by 9½-foot tunnel, open cut, construction, diversion chamber, connection to Piney Branch sewer and overflow	76, 000
	2, 050, 150

Q STREET LATERAL

530 feet of 42-inch circular sewer, including temporary and permanent paving at $19 per foot	10, 260
Junction and diversion chambers	2, 000

19TH STREET LATERAL

1,000 feet of 48-inch inside diameter earth tunnel at $30 per foot	30, 000
600 feet of 24-inch circular sewer at $10 per foot	6, 000
Intercept 4 existing sewers and build overflows	1, 000

ONTARIO ROAD LATERAL

600 feet of 48-inch inside diameter tunnel at $30 per foot	18, 000
Intercept 2 existing sewers	500

IRVING STREET LATERAL

890 feet of 48-inch inside diameter tunnel at $30 per foot	26, 700
780 feet of 27-inch circular sewer at $7 per foot	5, 460
650 feet of 18-inch circular sewer at $5 per foot	3, 250
Intercept 4 sewers and build overflows	1, 000
Reconnect inlets below overflows	1, 000
New interceptor between Ontario and Irving Streets	3, 000
Rebuild 3 traps and lay larger connecting sewers	3, 000
	2, 161, 320
Right-of-way	20, 000
Total estimated cost of tunnel with valley outlet	2, 181, 320

19. *Plan V–1—Estimate of cost of valley relief sewer and of west side relief sewer*

Total estimated cost of valley relief sewer including right-of-way	$2, 101, 050
Total estimated cost of west side relief sewer, including right-of-way	418, 770
Total estimated cost of plan V–1	2, 519, 820
Engineering, 6 percent	151, 180
Repairs to parks and contingencies, 4 percent	101, 000
Total	2, 772, 000

Summary of estimate of cost of valley relief sewer (plan V–1)

2,100 feet of 12½-foot horseshoe section in open cut at $55	$115, 500
2,270 feet of 14½-foot horseshoe section in open cut at $110	249, 700
750 feet of 14½-foot horseshoe section on concrete piers at $75	56, 250
Replacement of swimming pool etc	20, 000
New open channel Francis School to P Street	20, 000
3,900 feet of 12- by 12-foot rock tunnel at $157.50	614, 250
1,500 feet of 11½-foot horseshoe section in open cut at $60	90, 000
1,550 feet of 10- by 10-foot rock tunnel at $125	193, 750
1,600 feet of 10½-foot horseshoe section in open cut at $45	72, 000
2,000 feet of 9- by 9-foot rock tunnel at $105	210, 000
1,900 feet of 9½-foot horseshoe section in open cut at $54	102, 600
1,780 feet of 8½- by 8½-foot tunnel (Slash Run lateral) at $100	178, 000
Special construction (diversions etc.)	136, 000
Small junction chambers and connecting sewers	20, 000
Rebuilding 3 traps and connecting sewers	3, 000
	2, 081, 050
Right-of-way	20, 000
Total estimated cost of valley relief sewer	2, 101, 050
Engineering, 6 percent	125, 950
Repairs to parks and contingencies, 4 percent	84, 000
Total	2, 311, 000

Estimate of cost of valley relief sewer (plan V–1)

POTOMAC RIVER TO L STREET	
2,100 feet of 12½-foot horseshoe section, open cut, at $55	$115, 500
L STREET TO CREEK CROSSING	
2,270 feet of 14½-foot horseshoe section, open cut at $110	249, 700
Replacement of swimming pool and other incidentals in connection with above line	20, 000
750 feet of 14½-foot horseshoe section across creek on concrete piers at $75	56, 250
New open channel between Francis School Playground and P Street with outside curve gunnited for vertical height of 12 feet	20, 000
CREEK CROSSING TO BELMONT ROAD NO. 1	
3,900 feet of 12- by 12-foot rock tunnel, including shafts, at $157.50	614, 250

Estimate of cost of valley relief sewer (plan V–1)—Continued

BELMONT ROAD NO. 1 TO CALVERT STREET	
1,500 feet of 11½-foot horseshoe section, open cut (to be designed as pressure line), at $60	$90, 000
CALVERT STREET TO ONTARIO ROAD	
1,550 feet of 10- by 10-foot rock tunnel, including shafts, at $125	193, 750
ONTARIO ROAD TO LAMONT PORTAL	
1,600 feet of 10½-foot horseshoe section, open cut to be designed as pressure line, at $45	72, 000
LAMONT PORTAL TO PARK ROAD PORTAL	
2,000 feet of 9- by 9-foot rock tunnel, including shafts, at $105	210, 000
PARK ROAD PORTAL TO PINEY BRANCH	
1,900 feet of 9½-foot horseshoe section in open cut including removal and replacement of paving, at $54	102, 600
SLASH RUN LATERAL	
1,780 feet of 8½ by 8½-foot tunnel, including shafts at $100	178, 000
Junction at upper end of Slash Run tunnel with existing sewers at 22d and M Streets	15, 000
Special construction consisting of sump, diversion, and gate at L Street	10, 000
Special construction where northwest boundary is to be intercepted	35, 000
Transition section at upper end of 9½-foot horseshoe open-cut construction diversion chamber, connection to Piney Branch sewer and overflow	76, 000
Other small junction chambers and connecting sewers	20, 000
Rebuild 3 traps and lay larger connecting sewers	3, 000
Total	2, 081, 050
Right-of-way for 23d Street tunnel	20, 000
Total estimated cost of valley relief sewer	2, 101, 050

20. *Estimate of cost of west side relief sewer*

POTOMAC RIVER TO O STREET	
2,970 feet of 9½-foot horseshoe section (not including box crossing under Chesapeake & Ohio Canal) at $33 per foot	$98, 010
250 feet of box crossing under Chesapeake & Ohio Canal at $60 per foot	15, 000
Removal and repaving of 400 feet of brick paving along 30th Street	4, 000
O STREET TO MASSACHUSETTS AVENUE	
4,130 feet of 9½-foot horseshoe section at $35 per foot	144, 550
250 feet of box section at lower creek crossing at $60 per foot	15, 000
350 feet of box section at upper creek crossing, including alteration to old interceptor and Montrose extension	40, 000
300 feet of earth tunnel under Massachusetts Avenue, 7½ by 7½-foot tunnel at $75 per foot	22, 500
MASSACHUSETTS AVENUE TO NORMANSTONE DRIVE	
820 feet of 72-inch circular sewer at $24 per foot	19, 680
NORMANSTONE DRIVE TO CLEVELAND AVENUE	
780 feet of 60-inch circular sewer at $21 per foot	16, 380

Estimate of cost of west side relief sewer—Continued

CLEVELAND TO CONNECTICUT AVENUE

1,150 feet of 39-inch circular sewer at $11 per foot	$12, 650
4 interceptions, $1,000 each	4, 000
3 interceptions (O Street, Olive Street, L Street), $1,000 each	3, 000
1 float regulated trap at K Street, $7,000 each	7, 000
1 tide gate	3, 000
Conversion of the Whitehaven district to the separate system	9, 000
Total	413, 770
Right-of-way	5, 000
Total estimated cost of west-side relief	418, 770
Engineering, 6 percent	25, 230
Repairs to parks and contingencies	17, 000
Total	461, 000

21. *Plan V–2—Estimate of cost of valley line with addition of three upper west side districts by a siphon crossing; including short west side relief sewer below Montrose*

Total estimate cost of main valley relief sewer (as estimated in plan V–1)	$2, 101, 050
Excess cost for increased size below siphon crossing	90, 000
Outlet sewer for Connecticut, Cleveland, and Normanstone district with siphon across Rock Creek	120, 900
Short west side relief sewer (including right-of-way)	226, 610
Conversion of the Whitehaven district to the separate system	9, 000
Total estimated cost of plan V–2	2, 547, 560

Plan V–2—Estimate of cost of short west side relief sewer

POTOMAC RIVER TO CHESAPEAKE & OHIO CANAL

2,150 feet of 7½-foot horseshoe section, open cut, at $28 per foot	$60, 200

CHESAPEAKE & OHIO CANAL TO O STREET

1,070 feet of 7-foot horseshoe section, open cut, at $26.50 per foot	28, 355

O STREET TO MONTROSE

4,780 feet of 6-foot horseshoe section, open cut, at $24.75 per foot	118, 305
4 interceptions at $1,000 each	4, 000
2 interceptions at $375 each	750
1 float regulated trap at K Street	7, 000
1 tide gate	3, 000
Total	221, 610
Right-of-way	5, 000
Total estimated cost of short west side relief sewer	226, 610

Plan V–2—Estimate of cost of outlet sewer for Connecticut, Cleveland, and Normanstone districts and siphon under Rock Creek to the valley line

1,030 feet of 42-inch circular sewer, to be designed as pressure line, at $20 per foot	$26, 600
640 feet of 7-foot horseshoe section, open cut to be designed as pressure line at $35 per foot	22, 400
735 feet of 8-foot horseshoe section, open cut, to be designed as pressure line at $40 per foot	29, 400
225 feet of 9-foot horseshoe section siphon, under Rock Creek at $100 per foot	22, 500
Trap revision	10, 000
Total estimated cost of outlet sewer with siphon	120, 900

TABLE 13.—*Tunnel line—T–1*

Line	Acreage intercepted: From—	Acreage intercepted: Increment	Total acreage	Intercepted run-off	Length, size, and grade of relief sewer, $n=0.012$
Piney Branch outfall to Oak Street sewer	Piney Branch	2, 465	2, 465	1, 200	400 feet of 9½- by 9½-foot tunnel, at 0.0048 h. g.
Oak Street sewer to 18th Street shaft	Oak Street Mount Pleasant	14. 7 3. 1	2, 483	1, 200	500 feet of 9½- by 9½-foot tunnel, at 0.0048 h. g.
18th Street shaft to Irving shaft	18th Street Ingleside	0 7. 6	2, 490	1, 200	2,550 feet of 9½- by 9½-foot tunnel, at 0.0048 h. g.
Irving shaft to Quarry shaft	Park Road Lamont Kenyon Irving	13. 5 12. 5 13 60	2, 589	1, 275	790 feet of 9½- by 9½-foot tunnel, at 0.0054 h. g.
Quarry shaft to Ontario shaft	Quarry	22. 1	2, 611	1, 275	500 feet of 9½- by 9½-foot tunnel, at 0.0054 h. g.
Ontario shaft to Kalorama shaft	Ontario	13. 6	2, 625	1, 275	2,800 feet of 9½- by 9½-foot tunnel, at 0.0054 h. g.
Kalorama shaft to northwest boundary	Biltmore Belmont	17. 5 40. 8	2, 683	1, 350	3,530 feet of 10½- by 10½-foot tunnel, at 0.0035 h. g.
Northwest boundary to Slash Run	Northwest boundary Massachusetts Ave., district	555 84	3, 322	1, 700	1,900 feet of 10½- by 10½-foot tunnel, at 0.0055 h. g.
Slash Run to old interceptor	Slash Run	371	3, 693	1, 900	2,450 feet of 12- by 12-foot tunnel, at 0.0035 h. g.
Old interceptor to Potomac River	----------	----------	3, 693	1, 900	2,100 feet of 12½-foot horseshoe, at 0.006. 17,520 feet, total length.

TABLE 14.—*Tunnel line, with valley outlet—T-2*

Line	Acreage intercepted		Total acreage	Intercepted run-off	Length, size, and grade of relief sewer, $n=0.012$
	From—	Increment			
Piney Branch outfall to Oak Street sewer	Piney Branch	2,465	2,465	1,200	400 feet of 9½- by 9½-foot tunnel, at 0.0048 h. g.
Oak Street sewer to 18th Street shaft	Oak Street Mount Pleasant	14.7 3.1	2,483	1,200	500 feet of 9½- by 9½-foot tunnel, at 0.0048 h. g.
18th Street shaft to Irving shaft	18th Street Ingleside	0 7.6	2,490	1,200	2,550 feet of 9½- by 9½-foot tunnel, at 0.0048 h. g.
Irving shaft to Quarry shaft	Park Road Lamont Kenyon Irving	13.5 12.5 13 60	2,589	1,275	790 feet of 9½- by 9½-foot tunnel, at 0.0054 h. g.
Quarry shaft to Ontario shaft	Quarry	22.1	2,611	1,275	500 feet of 9½- by 9½-foot tunnel, at 0.0054 h. g.
Ontario shaft to Kalorama shaft	Ontario	13.6	2,625	1,275	2,800 feet of 9½- by 9½-foot tunnel, at 0.0054 h. g.
Kalorama shaft to northwest boundary	Biltmore Belmont	17.5 40.8	2,683	1,350	3,530 feet of 9½- by 9½-foot tunnel, at 0.0062 h. g.
Northwest boundary to P Street portal	Northwest boundary Massachusetts Ave., district	555 84	3,322	1,700	400 feet of 11- by 11-foot tunnel, at 0.0044 h. g.
P Street portal to Slash Run			3,322	1,700	2,370 feet of 14½-foot horseshoe, at 0.0022 h. g.
Slash Run to old interceptor	Slash Run	371	3,693	1,900	650 feet of 14½-foot horseshoe, at 0.0028 h. g.
Old interceptor to Potomac River			3,693	1,900	2,100 feet of 12½-foot horseshoe, at 0.006.
					16,590 feet, total length.

TABLE 15.—*Valley line—V-1*

Line	Acreage intercepted		Total acreage	Intercepted run-off	Length, size, and grade of relief sewer, $n=0.012$
	From	Increment			
Piney Branch outfall to Oak Street	Piney Branch	2,465	2,465		
Oak Street to 18th Street	Oak Street Mt. Pleasant	14.7 3.1	2,483	1,200	1,900 feet of 9½-foot horseshoe, at 0.01 h. g.
18th Street to Park Road portal	Eighteenth Ingleside	0.0 7.6	2,490	1,200	
Park Road portal to Lamont portal	Park Road	19.7	2,510	1,200	2,000 feet of 9 by 9-foot tunnel at 0.0065 h. g.
Lamont portal to Irving Street	Lamont Kenyon	12.5 13.0	2,536	1,200	850 feet of 10½-foot horseshoe at 0.006 h. g.
Irving Street to Ontario	Irving Quarry	60.0 34.1	2,630	1,275	750 feet of 10½-foot horseshoe at 0.0068 h. g.
Ontario to Calvert	Ontario	20.8	2,650	1,275	1,550 feet of 10 by 10-foot tunnel at 0.0041 h. g.
Calvert to Belmont Road no. 1. (Cut old interceptor)	Biltmore Belmont	17.5 40.8	2,709	1,275	1,500 feet of 11½-foot horseshoe at 0.0042 h. g.
Belmont Road no. 1 to northwest boundary			2,709	1,350	3,350 feet of 12 by 12-foot tunnel at 0.0018 h. g.
Northwest boundary to P Street	Northwest boundary Massachusetts Ave., district	555 84	3,348	1,700	550 feet of 12 by 12-foot tunnel at 0.0028 h. g.
P Street to M Street			3,348	1,700	2,370 feet of 14½-foot horseshoe at 0.0022 h. g.
M Street to L Street	Slash Run	371	3,719	1,900	650 feet of 14½-foot horseshoe at 0.0028 h. g.
L Street to Potomac River			3,719	1,900	2,100 feet of 12½-foot horseshoe at 0.006.
					17,570 feet, total length.

TABLE 16.—*West side relief interceptor*

Line	Acreage intercepted			Unit run-off	Intercepted run-off	Length, size, and grade of relief sewer, $n=0.012$
	From—	Increment	Total acreage			
Connecticut Avenue to Cleveland Avenue	Connecticut Avenue	102	102	1.25	127	1,150 feet of 39-inch pipe, at 0.0024 h. g.
Cleveland Avenue to Normanstone Drive	Cleveland	87	189	1.25	236	780 feet of 60-inch pipe, at 0.0070 h. g.
Normanstone Drive to Massachusetts	Normanstone	242	431	1.00	431	820 feet of 72-inch pipe, at 0.009 h. g.
Under Massachusetts fill			431	1.00	431	300 feet of 7½- by 7½-foot tunnel, at 0.0022 h. g.
Massachusetts to Montrose crossing			431	1.00	431	320 feet of 9½-foot horseshoe, at 0.0014 h. g.
Creek crossing at dam			431	1.00	431	350 feet {150 feet of 6.75 by 10 feet 2 transitions, 100 each} 0.0018 h. g.
Montrose crossing to creek crossing above Q Street	Montrose	121	552	.90	497	1,650 feet of 9½-foot horseshoe, at 0.0018 h. g.
Creek crossing above Q Street			552	.90	497	250 feet {50 feet of 6.75 by 10 feet 2 transitions, 100 feet each} 0.0018 h. g.
Q Street crossing to Q Street			552	.90	497	750 feet of 9½-foot horseshoe, at 0.0018 h. g.
Q Street to P Street	Q Street	9	561	.90	505	450 feet of 9½-foot horseshoe, at 0.0019 h. g.
P Street to O Street	P Street	5	566	.90	509	960 feet of 9½-foot horseshoe, at 0.0019 h. g.
O Street to Olive Street	O Street	113	679	.75	510	1,070 feet of 9½-foot horseshoe, at 0.0019 h. g.
Olive Street to Chesapeake & Ohio Canal	Olive Street	24	703	.75	527	700 feet of 9½-foot horseshoe, at 0.0020 h. g.
Under Chesapeake & Ohio Canal	M Street	39	742	.70	527	250 feet {50 feet of 6 by 8 feet 2 transitions, 100 feet each} 0.0020 h. g.
Chesapeake & Ohio Canal to Potomac River			742	.70	527	1,200 feet of 9½-foot horseshoe, at 0.0020 h. g.
						11,000 feet, total length.

Chapter VII

SUBSURFACE EXPLORATION

As the character of the underlying rock in those sections where relief sewer tunnels are proposed is of the utmost importance in the development of a proper plan and reasonable estimates, and as the location of the rock surface at other points was also essential in estimating the cost of lines to be constructed in open cut, subsurface studies have been carried out to the extent necessary for a preliminary plan.

A shot drill and crew were rented from the United States Engineer Office and 17 core holes put down. A complete log of these holes appears in the appendix and copies have been given to the United States Engineers, the United States Geological Survey, and the Bureau of Public Roads. The actual cores are stored at the United States Engineers' Wharf, Eleventh and O Streets SE.

To supplement this positive method of exploration, which is expensive and slow, the seismograph recorder developed by Mr. E. R. Shepard, of the Bureau of Public Roads, and described by him in a recent paper delivered before the Academy of Science, was used with very satisfactory results. This recorder, together with material and operators, were donated by the Bureau and probably saved their entire cost on this one investigation.

A complete log of the seismographic soundings also appears in the appendix.

The conditions discovered by the subsurface exploration can be briefly described as follows:

1. *Unsatisfactory tunneling material below P Street.*—The core drilling, most of which was done below Q Street, proved that the original idea of a tunnel outlet in New Hampshire, or in New Hampshire and I, K or L Street, is impractical. Core log no. 1 at the intersection of Twenty-fourth, New Hampshire, and I Streets, shows that the entire barrel of the tunnel would lie in a water-bearing mixture of sand and gravel. Core log no. 2 at the intersection of Twenty-second, New Hampshire, and L Streets, is a little better in that the soft rock is overlain by clay, with the probable tunnel arch at the dividing line, but further drilling west on L was not encouraging; see holes nos. 3 and 12.

By moving one block further north into M Street, fairly good rock was found at tunnel depth excepting at the ends where it dips into Rock Creek on the west and into the old Slash Run gulch at Twenty-second Street on the east. M Street had the further advantage of being the best route for the Slash Run lateral to accompany a valley outlet, which had by this time entered the picture as a desirable solution, inasmuch as bad tunneling conditions had also been found in Twenty-second Street, between N and P where the only possible north and south tunnel line crossed the old Slash Run Valley.

Summarizing, holes 1, 2, 3, and 12 proved that a continuous tunnel outlet to the Potomac is not an economical solution. Holes 4, 11, 13, and 14 proved that M Street is a possible tunnel route from Twenty-second Street to Rock Creek for either the main relief sewer or the Slash Run lateral. Holes 5, 7, and 10 indicated that tunneling conditions in Twenty-second Street between M and P are not good, overlying, as this street does, the old junction of the Slash Run and Rock Creek Valleys. The tunnel arch will be in soft rock for most of this length and will undoubtedly encounter leakage from the water bearing gravel above.

2. *Satisfactory tunneling conditions above P Street.*—North of P Street, the seismograph indicates that the tunnel will be entirely in hard rock until the Piney Branch Valley is reached. No core drilling was done along the body of the tunnel route since the seismograph had already indicated good rock conditions, and no holes were put down in the Piney Branch Valley because of the cost of dragging in the machine and of piping in a water supply, and because rock is plainly visible in the creek bed. The one seismograph shot here indicates that it continues southward on a horizontal plane, but outcroppings encountered in grading the new Piney Branch road, show a higher level in spots. The tunnel construction grades under either plan T–1 or T–2 can be lowered at will in case additional drilling discloses unsatisfactory rock cover at the grades shown on the plans.

3. *Subsurface conditions on valley route.*—The three tunnels of the valley line were investigated by the seismograph recorder.

Locations nos. 20, 21, 24, and 25 for the upper tunnel seem contradictory, since only the first and last give definite indications of rock, but this is probably due to the fact that the rock face is sloping sharply toward the valley. This is borne out by the fact that location no. 22, 100 feet east of the line shows rock 35 feet higher than no. 20, almost opposite on the west. The data at 20 and 25 indicate 10 feet of rock cover over the arch.

Locations nos. 17 and 18 for the middle tunnel, while not on line, indicate 70 feet of rock cover in the middle of the tunnel, and with good solid outcrops visible at both the Calvert and the Ontario portals, there seems little cause for apprehension here, although it would be wise to investigate further in the deep valley just north of Calvert Street.

No exploration was done for the lower tunnel in Twenty-third Street, as this part of the line was moved from Twenty-fourth Street after the rock investigation was finished. Data from locations nos. 12, 13, 14, and 15 in Twenty-fourth Street, however, indicate high rock in the middle of the tunnel, but low at the north portal. No prospecting has been done in the vicinity of Massachusetts Avenue, but it is important that this end be thoroughly investigated on account of the value of the property and the necessity of obtaining a right-of-way through it.

As mentioned above, unsatisfactory tunneling conditions below P Street led to the selection of a valley outlet for the tunnel line, and this combined tunnel-valley plan is referred to as T–2. The outlet chosen has also been used in plans V–1 and V–2. It leaves Twenty-second Street between P and Q, crosses P Street midway between the riding school and the new bridge abutment and strikes out across the sharp bend of Rock Creek at Slash Run. Seismograph shot no. 5 indicates rock 16 feet below invert here, and it is intended to found the sewer on this rock by the use of concrete cross walls or side piers arched under the invert, and to extend the Slash Run outfall under the relief into the new creek channel which will be dug across the bend just west of the proposed sewer.

Leaving this section the sewer skirts the Francis School in deep open cut. Both seismograph and core drill agree that rock is from 9 to 18 feet below invert, rising to invert grade at M Street, and that the material over the rock is largely filled ground. See core logs nos. 8 and 9 and seismograph shots nos. 4 and 9.

With the exception of the peak of rock encountered between M Street and Pennsylvania Avenue, indicated by core log no. 4 and seismograph shots nos. 2 and 3, the remainder of the line to the Potomac River is in earth.

4. *Subsurface conditions on west side relief.*—Seismograph investigations nos. 26, 27, and 28 and core logs nos. 15 and 16 indicate that the west side relief sewer will encounter no rock at any point on its length.

Chapter VIII

DIVERSION CHAMBERS AND OTHER SPECIAL STRUCTURES

1. In the preceding chapters, there have been outlined the studies which determined the amount of flow to be diverted from the combined sewers in order to maintain satisfactory standards in Rock Creek. It is clear that the integrity of the relief sewer plans which are to be devised will depend to a large measure on the design of the diversion chambers.

The function of the diversion chambers to be constructed on each of the combined sewers is to lead from the combined sewer during time of storm, the full amount of flow intended to be diverted from Rock Creek. It is also clear that the diversion structures should not be capable of taking an appreciably greater amount than the predetermined value, in order that the relief sewers be not overcharged.

2. In the design of these chambers, it is also considered desirable that mechanical equipment be avoided, that the structures be as simple as possible, and be capable of functioning accurately in the predetermined manner without appreciable maintenance.

3. As the flow to be diverted from the Piney Branch sewer is the most important item entering into the capacity of the relief sewer, the most extensive study of the diversion chamber design was made on the Piney Branch system. In connection with this study, a location map of all of the sewers in the vicinity was prepared, and as the Piney Branch main sewer has been shown to be badly overcharged, the possibility of a future relief sewer for it was given definite consideration. An examination of the conditions on the ground indicated that the diversion structure could be satisfactorily constructed at a point immediately below the present outlet, and the plans which have been developed take the form of an extension downstream of the outlet of the present combined sewer. The location of the diversion chamber at this point not only simplifies the carrying out of the construction work by permitting the new structure to be built on the solid rock in the present creek bed, but by, also, doing away with an unsatisfactory outlet condition which has given a great deal of trouble to the sewer division of the District of Columbia and to the National Park Service. The moving of the outlet 150 feet downstream will permit the construction of a new apron and retaining walls in a way that will greatly reduce scour in the Piney Branch Channel.

4. The general location of the diversion structure and new outlet, and the manner in which future sewers can be connected, are shown on the plat, figure 22, and the details of the diversion chamber structure are shown in plan, profile and section, on figure 23.

The principle involved in the design of this structure is to permit a flow of 1,200 cubic feet per second to pass into the relief sewer without appreciable disturbance through change of section or abrupt change of direction, and to permit all of the maximum storm flows in excess of 1,200 cubic feet per second to pass over a long side spillway, also without abrupt change in direction of flow.

It was found that, theoretically, the Piney Branch sewer, when carrying 1,200 cubic feet per second at its outlet, would have a depth of flow of 4.3 feet without regard to possible draw-down effects. Under these conditions, the theoretical velocity is 28 feet per second. Because of various breaks in grade and junction chambers closely upstream, it did not appear to be safe to assume that this high velocity would actually exist, and, as a basis of design, it was arbitrarily assumed that the velocity would be 20 feet per second, which gives a depth of flow of 6 feet.

An important point in the design of this chamber is that the wetted area along the diversion channel is kept practically constant, and sufficient grade is given to the diversion channel to insure that the velocity will not be slowed up. This insures that the velocity of flow at the point where the diverted stream enters a closed section will not be less than 20 feet a second, nor the depth of flow for 1,200 second-feet more than 7.3 at any place. For this condition the velocity change will therefore occur inside the closed section, and there is proposed a long transition of about 100 feet to develop the change into the standard section. In this transition the velocity will be gradually reduced to the normal velocity for the relief sewer, and the difference in the velocity heads will be recovered and made available in increase in the hydraulic grade.

We have laid out the diversion channel on a 150-foot radius to insure against slopping out over the side weir, and have expanded the extended outlet of Piney Branch to a wide flat section, to give ample discharge capacity with the least increase in depth of flow.

The section is liberally designed to carry the excess flow above 1,200 second-feet; overdesigned, in fact, even without regard to the inevitable draw-down at the outlet.

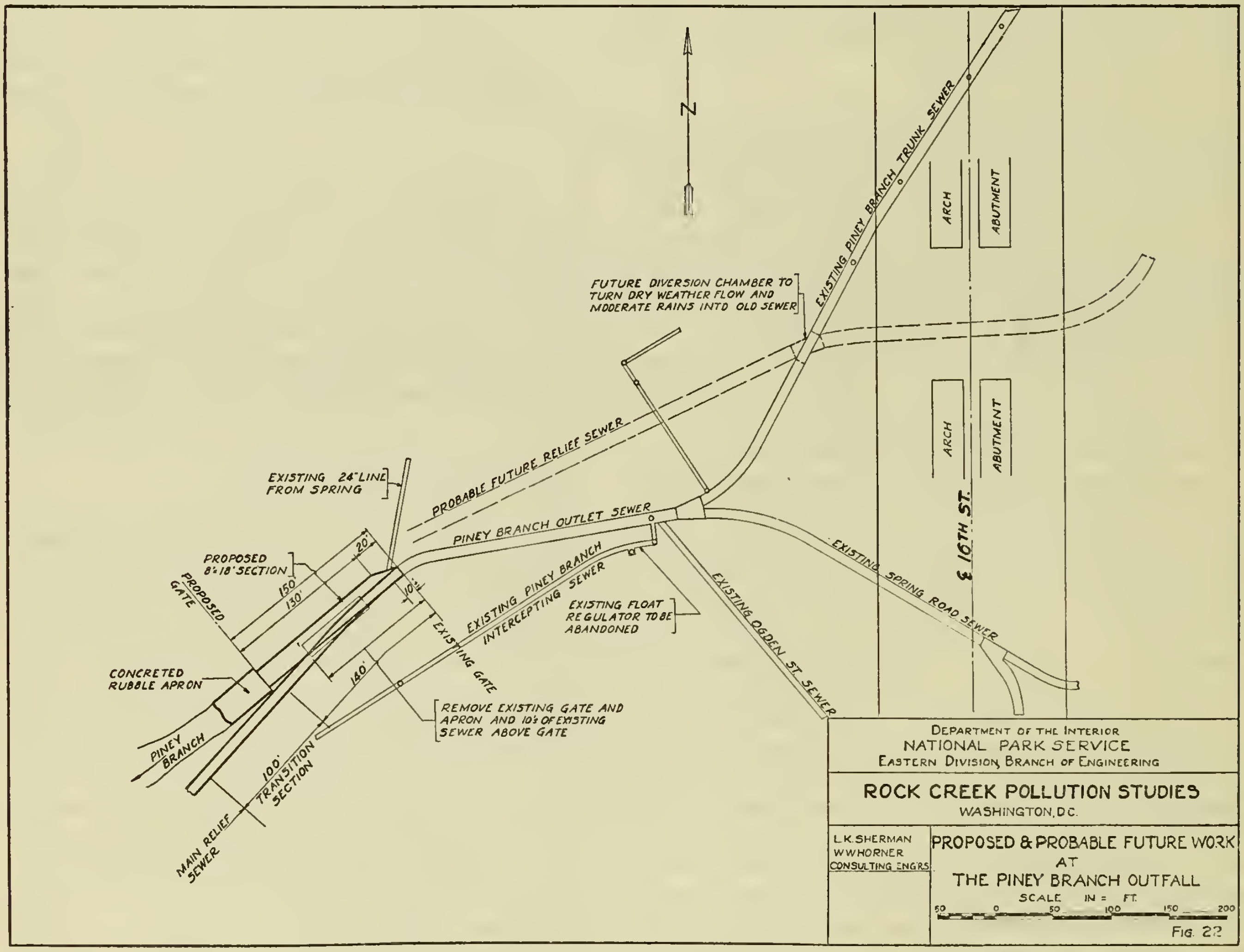
5. Your consulting engineers are of the opinion that a diversion of this type where extremely high velocities are involved, can best be thus worked out along the

lines of stream deflection. We have considered the possible effect on the relief sewer discharge of a full 4,000-second-foot flow in Piney Branch, and believe that the additional hydrostatic head available at the point of diversion would not be fully effective in increasing the relief-sewer discharge. If this head is not effective to the extent of more than 3 to 4 feet, a 1,200-second-foot discharge into the relief sewer will never be greatly exceeded.

6. *Northwest boundary diversion.*—It is important that the hydraulic grades for which the new relief sewer is designed be rather definitely fixed, and that there be absolute assurance that the pressure in the relief sewer tunnels will not greatly exceed the expected amount. Advantage has been taken of the necessary diversion construction at northwest boundary, to design for the same location a relief spillway, using the old northwest boundary outlet sewer to carry off any water that may flow over the spillway weir. Insertion of this provision requires a diversion chamber and overflow weir on the new relief sewer, a short connecting sewer, and a junction chamber on the northwest boundary outlet sewer.

The necessary hydraulic and construction grades for both the diversions and relief systems have been carefully worked out, and are shown on figure 24. These structures are a part of any of the relief-sewer schemes discussed, and a careful detailing of them is essential to the functioning of these sewers as planned.

7. *Slash Run diversion.*—The old Slash Run outfall was built in the bottom of a deep side valley and lies below the existing Rock Creek interceptor and below the hydraulic gradients of all relief plans at any point of diversion near Rock Creek. It was therefore necessary to place the diversion structure at Twenty-second and M Streets for all plans. This is unfortunate, as it will have to be built in deep cut, and the short sections of sewer which are required to complete the diversion will have to be built as tunnels under difficult conditions. As shown on figure 25, the diversion for the tunnel plan is worked out in some detail, using the same scheme as has been described for Piney Branch. It includes two diversion chambers, one on the main sewer and one on the M Street lateral, two short connecting sewers and a junction chamber on the new relief sewer.



8. Under plans T–2, V–1, and V–2 the Slash Run diversion must still be made at Twenty-second and M Streets as stated above. This requires an expensive lateral in M Street which fortunately can be built as a rock tunnel for the greater part of its length. The diversion at the upper end of this lateral tunnel is accomplished in practically the same manner as under plan T–1 and is shown in figure 26. This diversion of the Slash Run sewer and the necessary grades at which it is permitted to function not only fixed the grades of the Slash Run lateral, but also fixed the hydraulic grade of the main relief sewer at its junction with the Slash Run lateral.

9. *West side diversion chambers.*—Diversion chambers have been planned for the west side combined sewers that are tributary to the west side relief sewer. While these are much smaller in size, the accuracy of their design is as important to the functioning of the west side sewer as are the larger chambers to that of the main relief sewer. Typical diversion chambers for the west side sewers are shown on figures 27 and 28. Others have been drawn up for Connecticut and Montrose but are not included in this report.

10. *Further tests.*—In the preliminary design of these diversion chambers, the best theoretical information available has been used. Further refinements are possible, and should be carefully worked out, if these sketches are later reduced to construction plans. Because of the uncertainty of flow conditions at high velocities and the effect of side spillways such as have been suggested in these designs, it is desirable that a typical diversion chamber such as that at Piney Branch be theoretically designed in accordance with all the mathematical refinements possible; but we strongly recommend that thereafter the Park Service arrange with the Bureau of Standards to make accurate tests on a fairly large-scale model of the proposed chamber, and that the final design be further modified from the test results.

11. *Regulator chambers.*—At the present time, several of the combined sewers discharging into Rock Creek and from which sewage is diverted to the old Rock Creek intercepting system, are provided with automatic regulator gates which close whenever the Rock Creek intercepting sewers are filled to capacity. When the gates close under this condition, all sewage

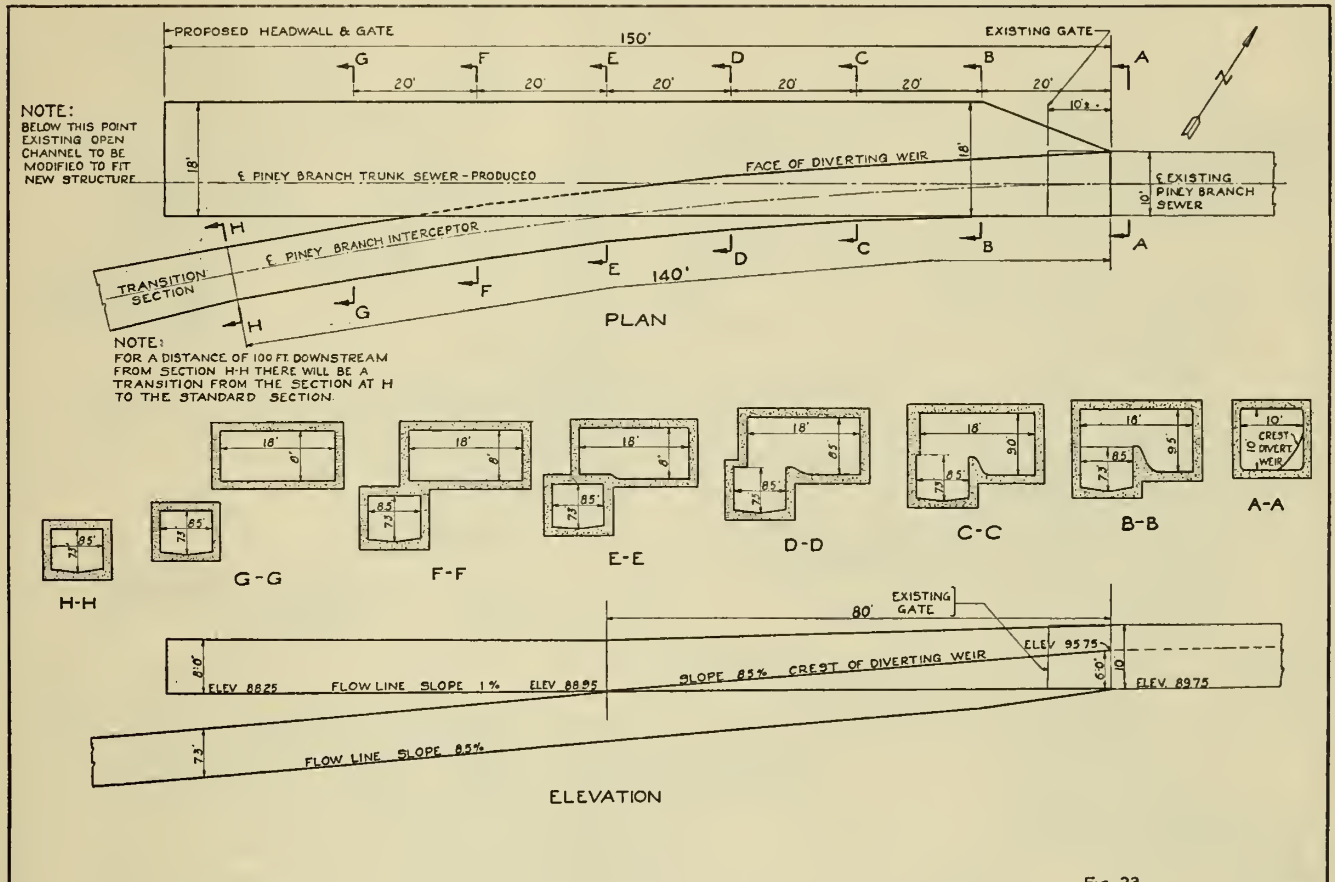


Fig. 23
PROPOSED CHANGES-PINEY BRANCH OUTFALL AND DETAILS OF NEW DIVERSION
ROCK CREEK POLLUTION STUDIES
SUBMITTED BY W.W. HORNER & L.K. SHERMAN CONSULTING ENGINEERS WASHINGTON D.C. 1935

flow, together with storm water, is discharged into Rock Creek. In all of the proposed relief plans such contaminated flow will not be discharged into Rock Creek, but up to a predetermined value will be diverted to the new relief sewer. Other combined sewer laterals are removed from the existing interceptors entirely and their entire flow is diverted to the relief up to the predetermined value.

12. The relief sewers will, therefore, not only carry contaminated storm flow to its outlet, but will also carry concentrated sewage in times of dry weather. This sewage flow must be returned to the old Rock Creek intercepting system at some point above the outlet of the relief sewers into the Potomac River. In the case of the main relief sewer, this return is provided for at a point near L Street where the relief sewer will cross over the top of the Rock Creek interceptor.

13. In order to avoid putting the old intercepting sewer under pressure at this point, regulating gates are required, and a special chamber for that purpose is outlined on figure 29. The gates in this chamber will operate very much as those now installed at the mouth of the Piney Branch sewer, shutting off the connection from the relief sewer entirely whenever the Rock Creek intercepting sewer is flowing full.

14. The dry-weather flow of the west side relief sewer is to be turned into the deep upper Potomac interceptor in K Street above the pumping station. This trap will also be float regulated in order to avoid overcharging the pumps. See figure 30.

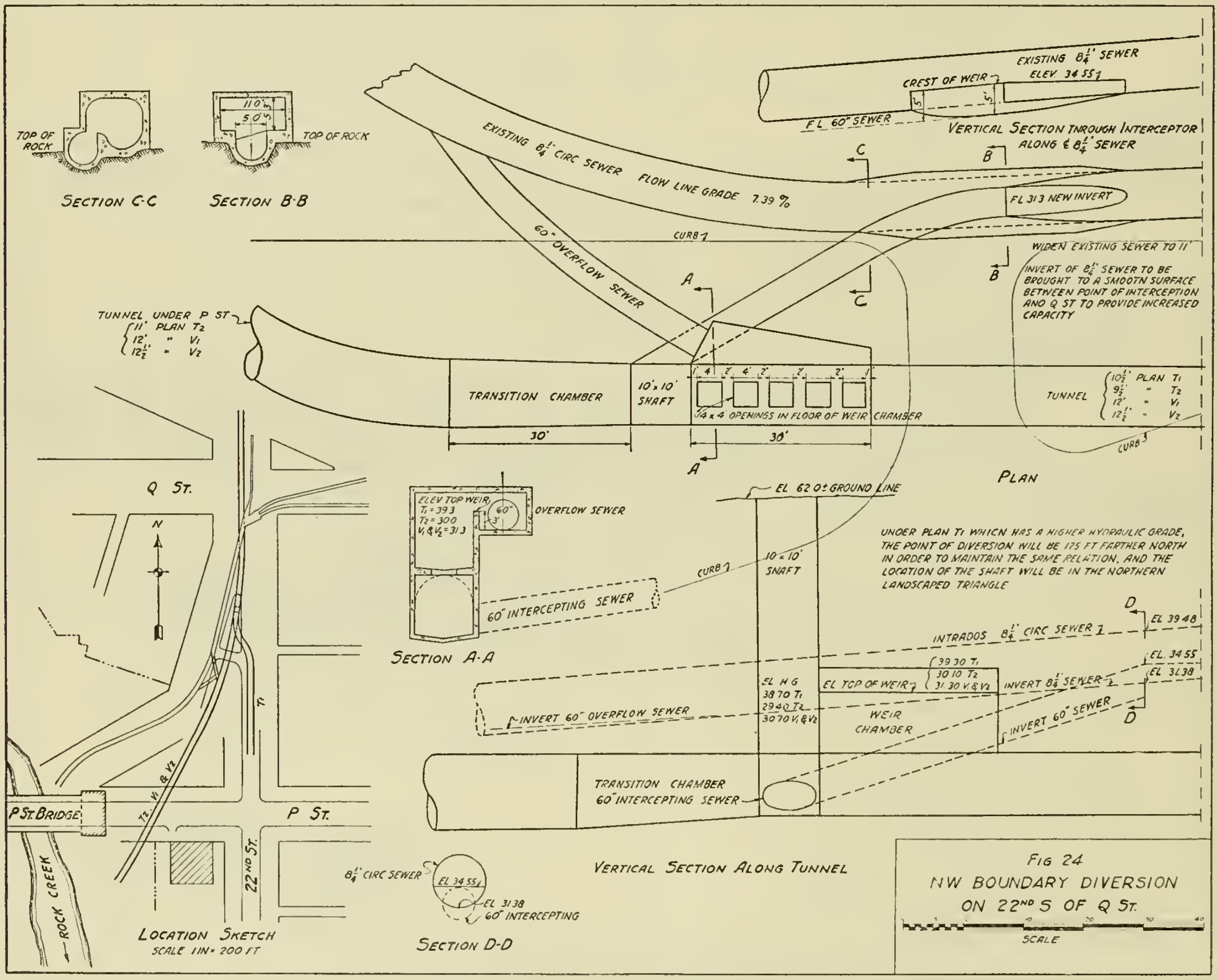
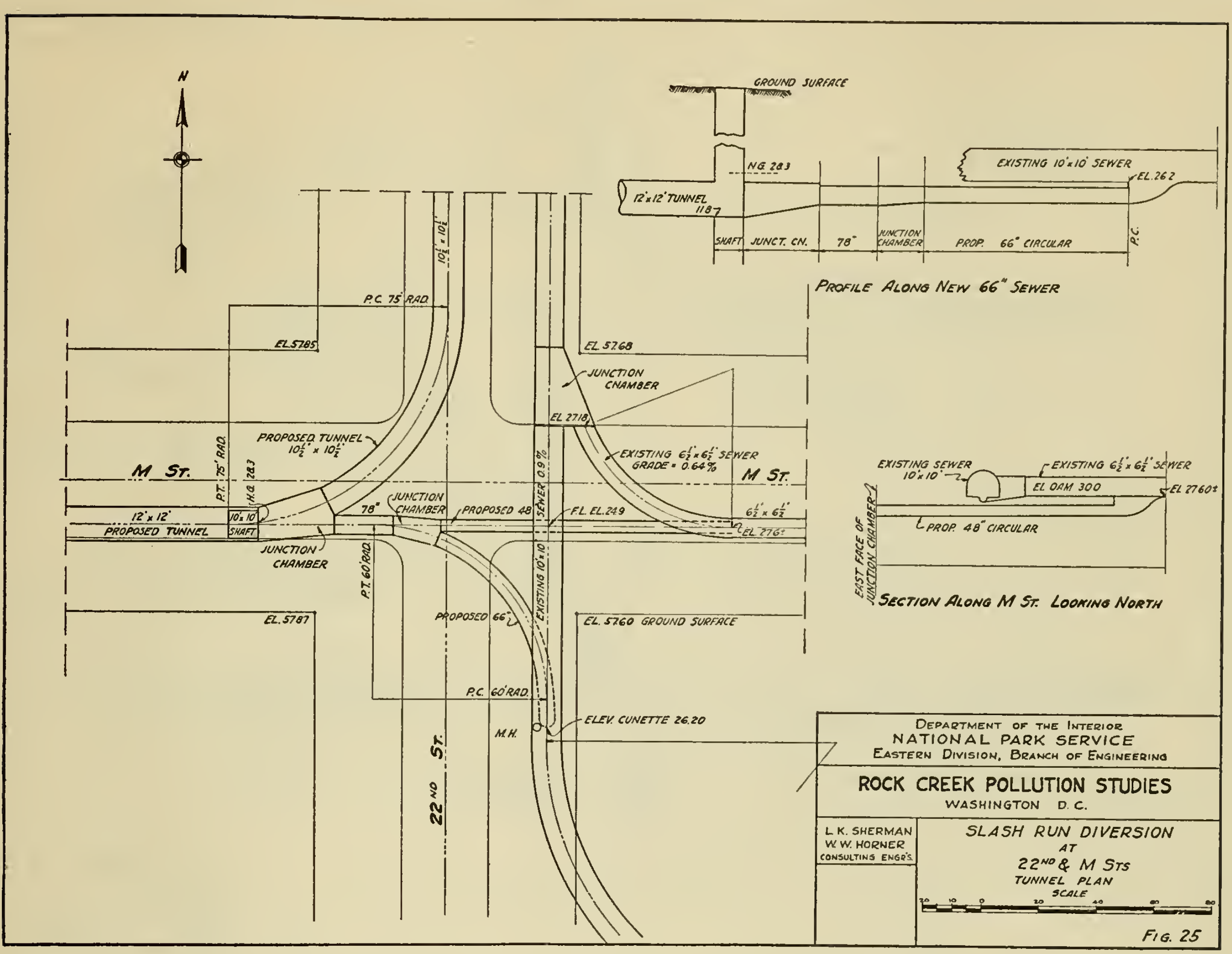


FIG 24
NW BOUNDARY DIVERSION ON 22ND S OF Q ST.

15. *Energy gradients.*—In the design of the new relief sewers, grades have been worked out that will avoid great or abrupt changes in velocity insofar as possible. In order to secure economically designed sections, however, some variation in average velocity between adjacent sections of sewer is inevitable. To the extent that these variations in velocity will involve appreciable differences in velocity head, this matter has been given serious consideration in the working out of hydraulic grades, and gain or loss of hydraulic grade on account of energy conversion has been taken into



account throughout. In order to conserve the available head to the greatest possible extent, it is suggested that all transitions between sections of different shape be worked out on long easy angles, and that no abrupt change in velocity be allowed to occur at any point. Such transition sections have been taken into account in the estimates of construction cost, and the additional cost of form work has been included in the estimates.

LeRoy K. Sherman,
Wesley W. Horner,
Consulting Engineers.

Washington, D. C., *July 21, 1935.*

Table 17.—*Diversions to main relief sewer*

Diversion	Acreage intercepted		Total acreage	Unit run-off	Intercepted run-off	Remarks
	From	Increment				
Piney Branch	Piney Branch	2,465		0.50	1,200	80 feet of diversion chamber, 60 feet of special section, 100 feet of transition section used with all plans.
Oak Street	Oak Street	17.8		1.25	22	New sump, with all plans.
Ingleside	Ingleside	7.6		1.25	9.6	420 feet of 18-inch used with both tunnel plans, new sump with valley plan.
Park Road	Park Road	19.7		1.25	24.7	300 feet of 18-inch, used with valley plan to Irving lateral-tunnel plans.
Lamont	Lamont	12.5		1.25	15.6	260 feet of 18-inch used with valley plan to Irving lateral-tunnel plans.
Kenyon	Kenyon	13.0		1.25	17.2	New sump with valley plan to Irving lateral-tunnel plans.
Irving	Irving	60.0		1.25	75	Do.
Quarry	Quarry	34.1		1.25	42.6	New sump with valley plan.
	Quarry	22.1		1.25	27.6	New sump and shaft used with both tunnel plans.
Ontario	Ontario	20.8		1.25	26	200 feet of 24-inch used with valley plan.
	Ontario	13.6		1.25	17	New sump, shaft, and 330 feet of 48-inch, used with both tunnel plans.

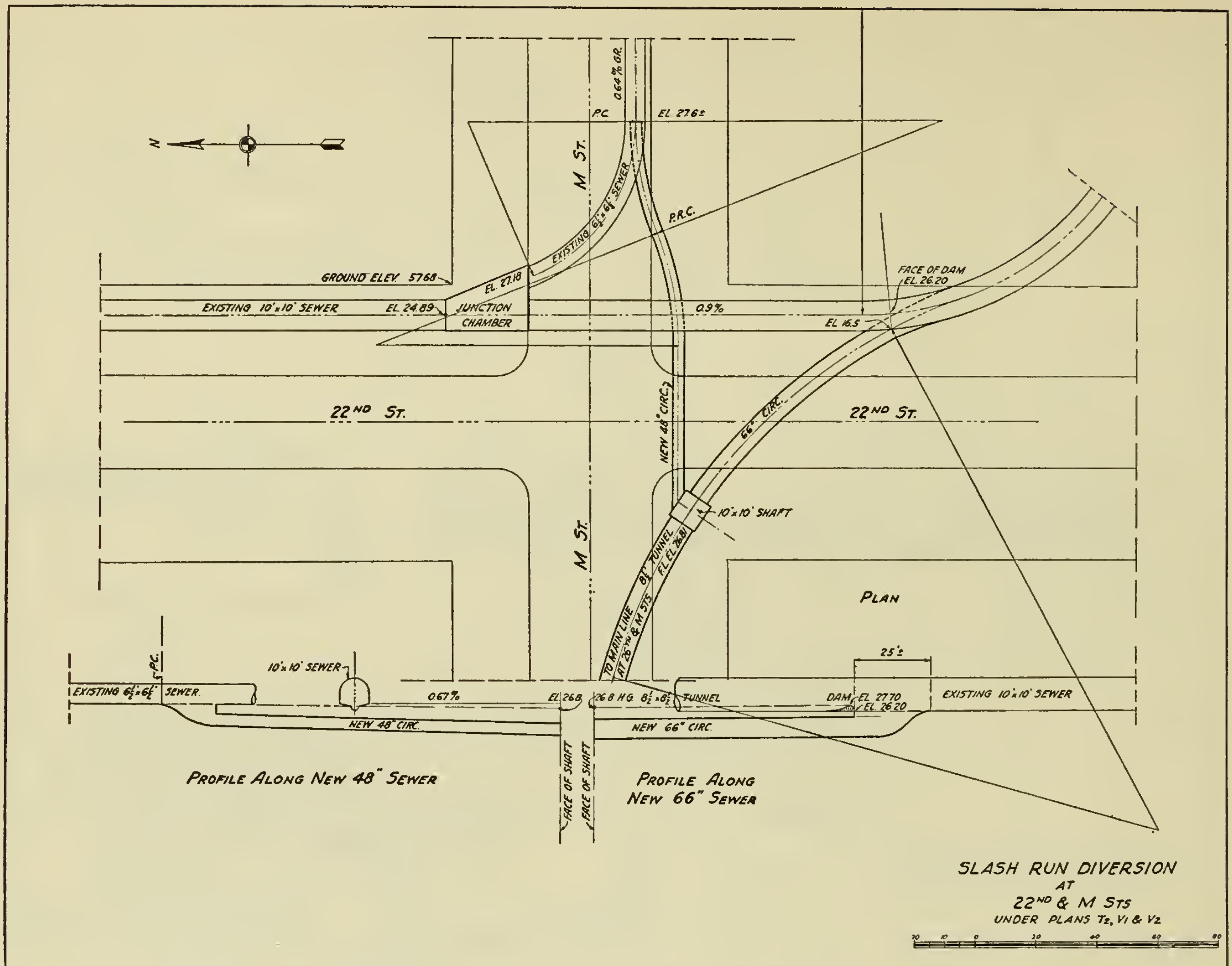
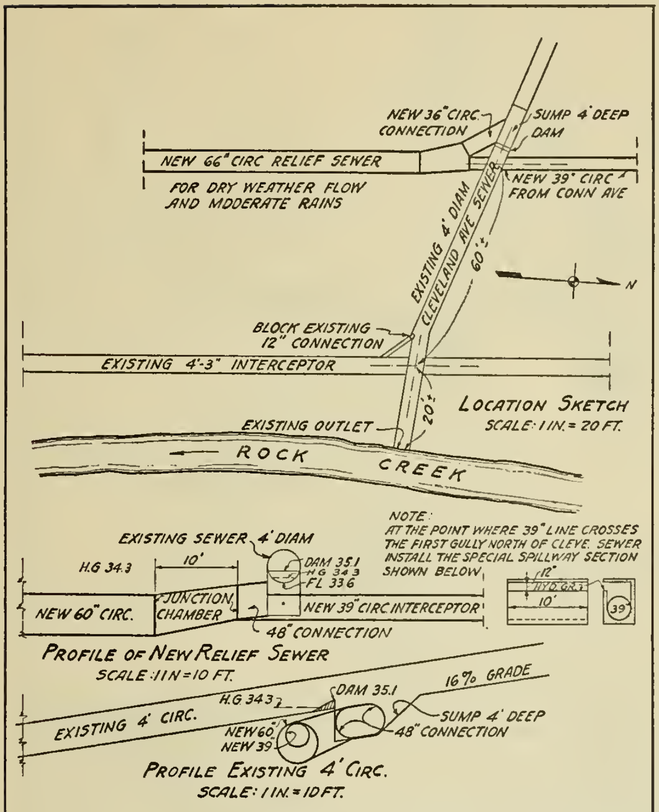


FIG 26

TABLE 17.—*Diversions to main relief sewer*—Continued

Diversion	Acreage intercepted: From	Acreage intercepted: Increment	Total acreage	Unit run-off	Intercepted run-off	Remarks
Irving Lateral	Park Road above Walbridge sanitary, below this point remains tributary to old interceptor.	13.5	13.5	1.25	17	650 feet of 18-inch.
	Lamont sewer at Adams Mill Road (2.9 acres tributary to inlets at this corner to be turned into Rock Creek).	12.5	26.0	1.25	33	750 feet of 27-inch.
	Kenyon sewer, at Adams Mill Road (4.4 acres tributary to inlets at this corner to be turned into Rock Creek).	13.0	39.0	1.25	49	900 feet of 48-inch.
	Hobart lateral	10.0	49.0	1.25	61	Used with both tunnel plans.
	Irving district (less Hobart lateral and 3 acres of inlet area at Adams Mill Road.	50	99	1.25	124	Intercept 62.5 cubic-feet-per-second from the Irving sewer just below the 24-inch lateral.
Biltmore	Biltmore	17.5		1.25	21.9	New sump used with valley line.
Belmont Street	Belmont Street	40.8		1.25	51	Do.
19th Street Lateral	Biltmore Belmont Street	17.5 40.8	58.3	1.25	73	600 feet of 24-inch, 1,000 feet of 48-inch used with tunnel plans.
Massachusetts Avenue at Sheridan Circle.	Massachusetts district	84	84	1.00	84	180 feet of 42-inch used with valley plan V-1.
23d and Q Streets	Massachusetts district	84	84	1.00		540 feet of 42-inch used with tunnel plan.
Northwest Boundary	Northwest Boundary Massachusetts district	55.5 84	63.9	1.12 1.00	7	64 feet of 60-inch used with all plans.
Slash Run	Slash Run	37.1	37.1	1.16	43	110 feet of 48-inch circular, 85 feet of 66-feet circular. 1,750 feet of 8½ by 8½-foot tunnel used with tunnel plant T-2 and valley plan.
	Slash Run	37.1	37.1	1.16	43	80 feet of 48-inch circular, 45 feet of 66-inch circular, 20 feet of 78-inch circular used with tunnel plan T-1.



DIVERSION STRUCTURE AT CLEVELAND
FIG. 27

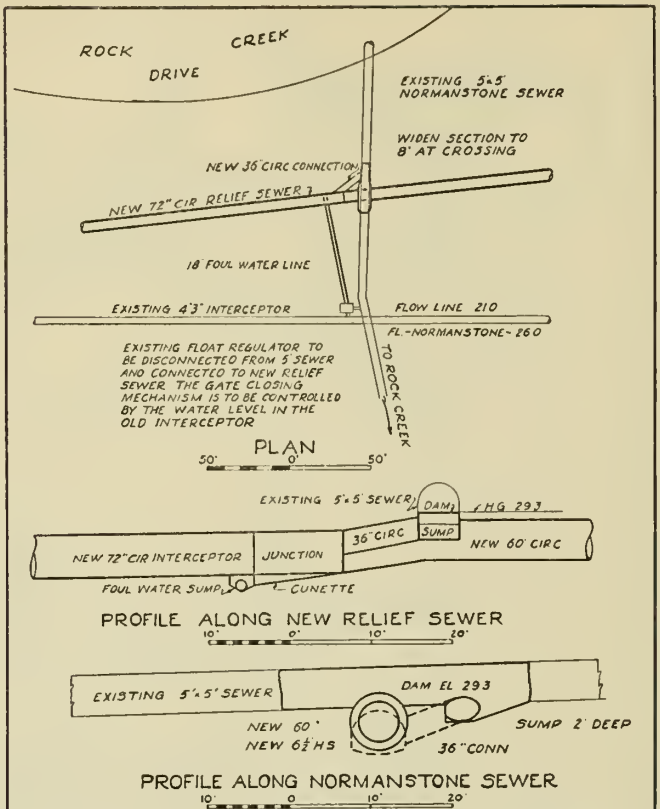


FIG. 28
DIVERSION STRUCTURE AT NORMANSTONE

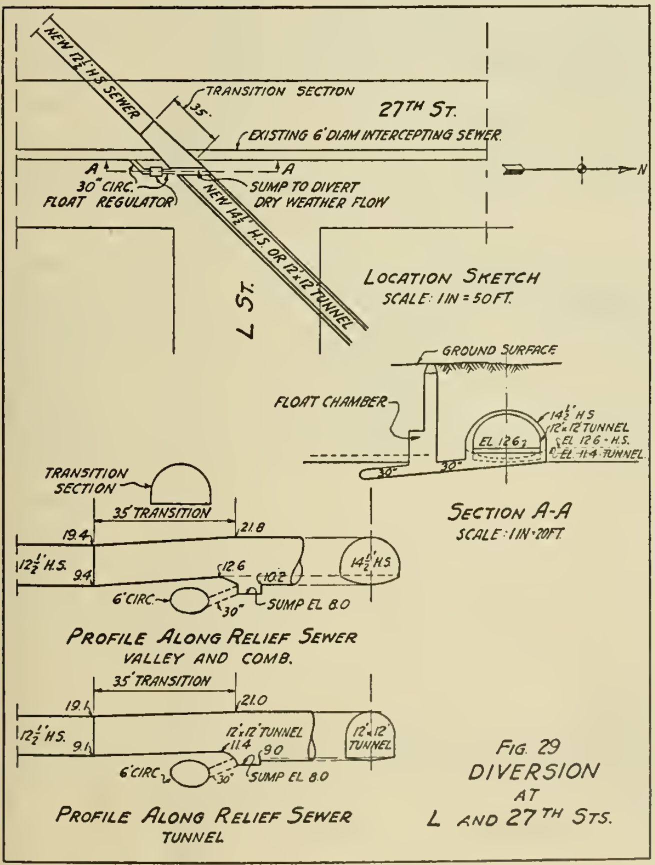


FIG. 29
DIVERSION
AT
L AND 27TH STS.

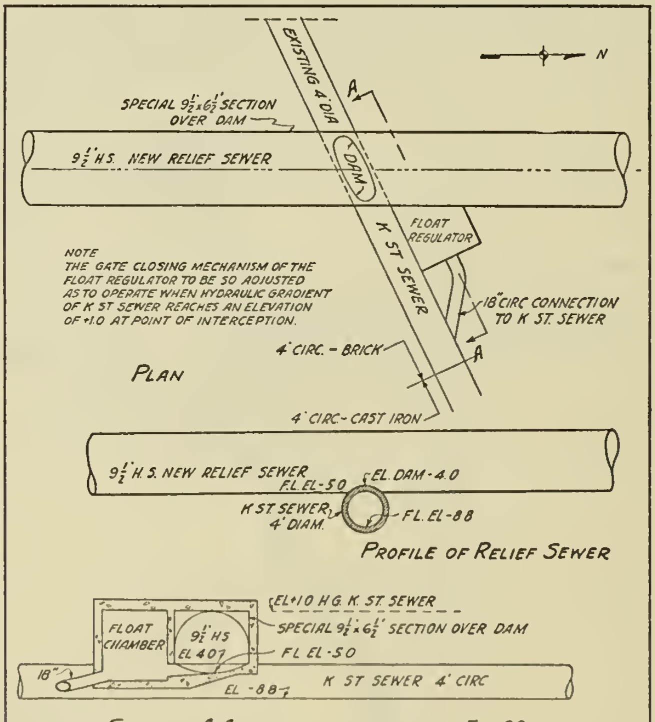
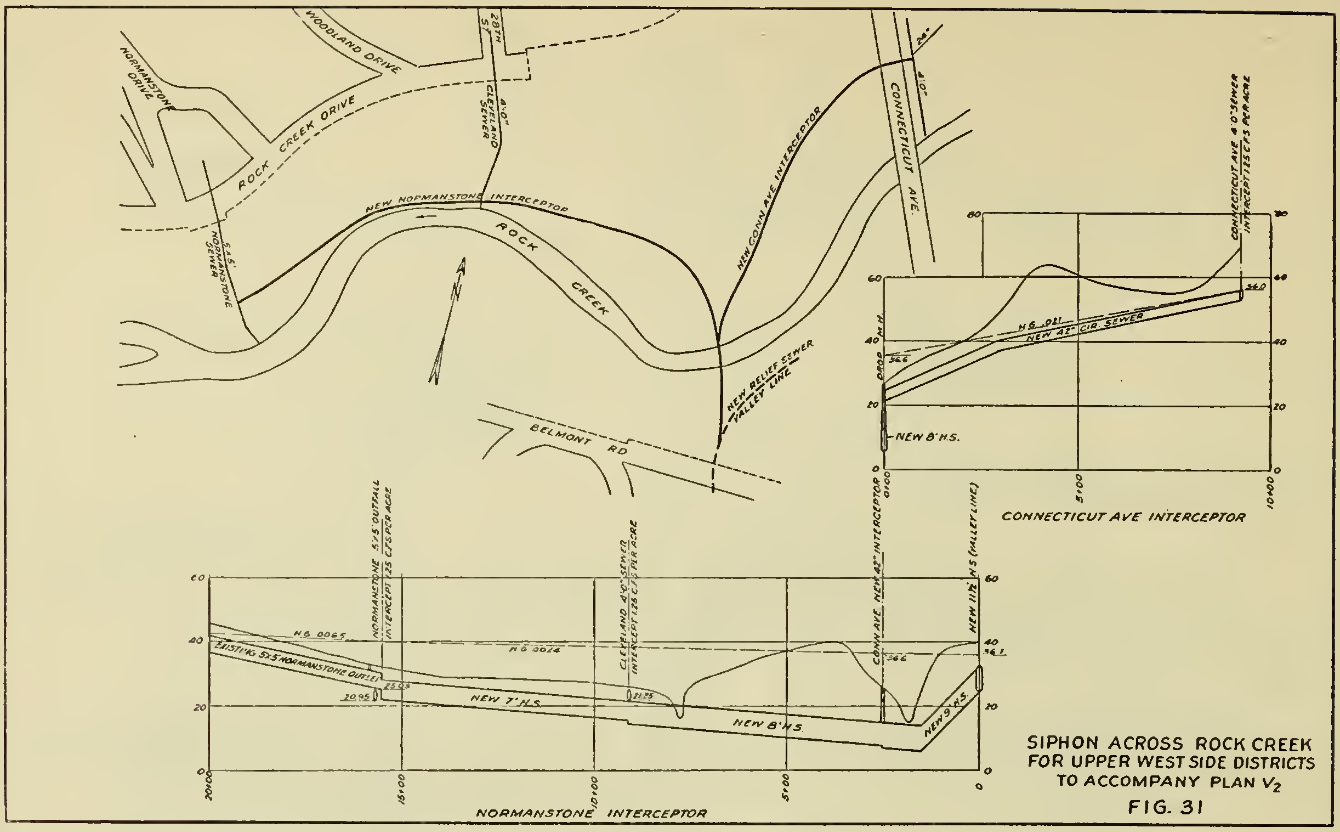


FIG. 30
FLOAT REGULATED TRAP
FOR DIVERSION OF DRY WEATHER FLOW
WEST SIDE RELIEF SEWER
AT K STREET
SCALE: 1 IN = 10 FT.



SIPHON ACROSS ROCK CREEK FOR UPPER WEST SIDE DISTRICTS TO ACCOMPANY PLAN V_2

FIG. 31

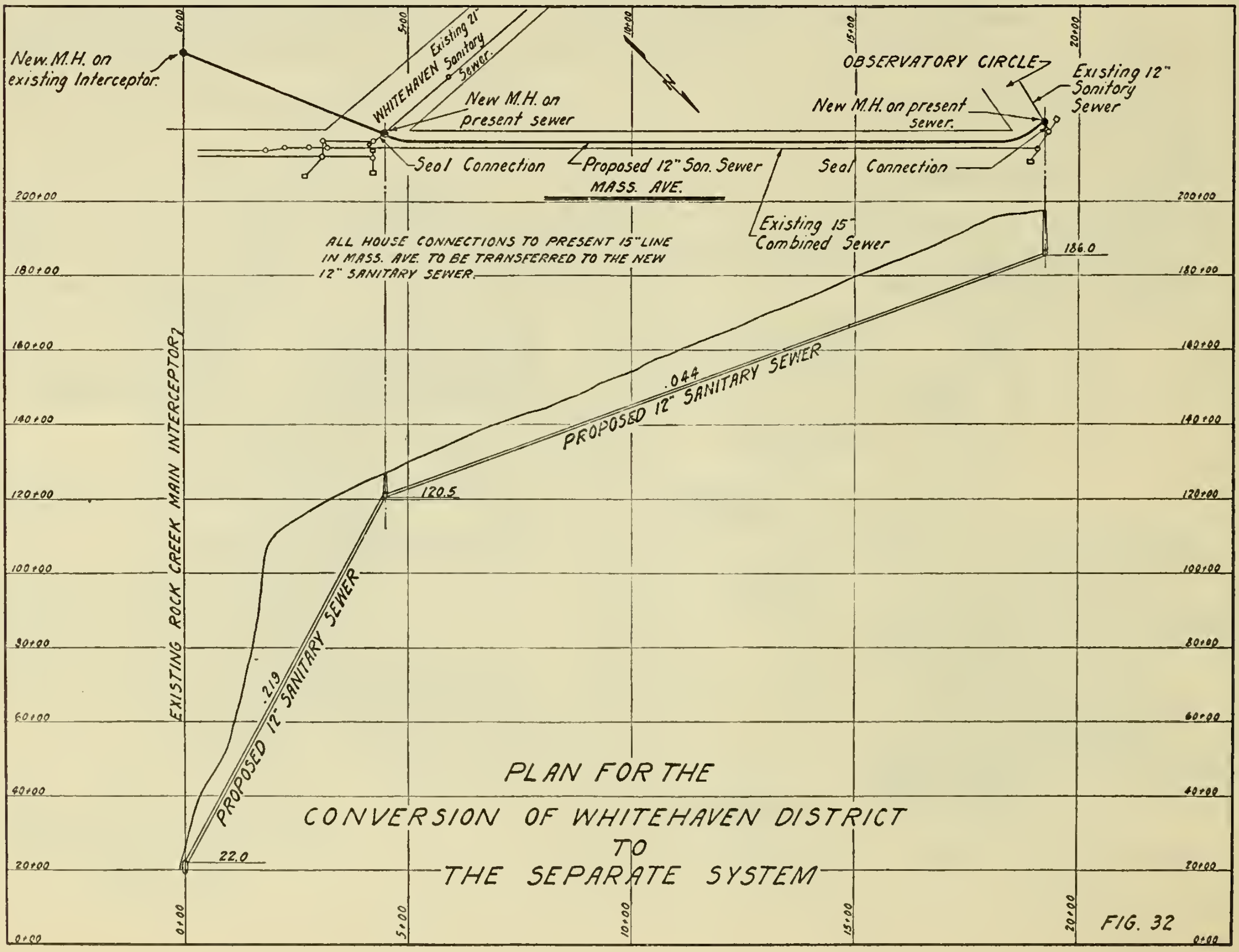


FIG. 32

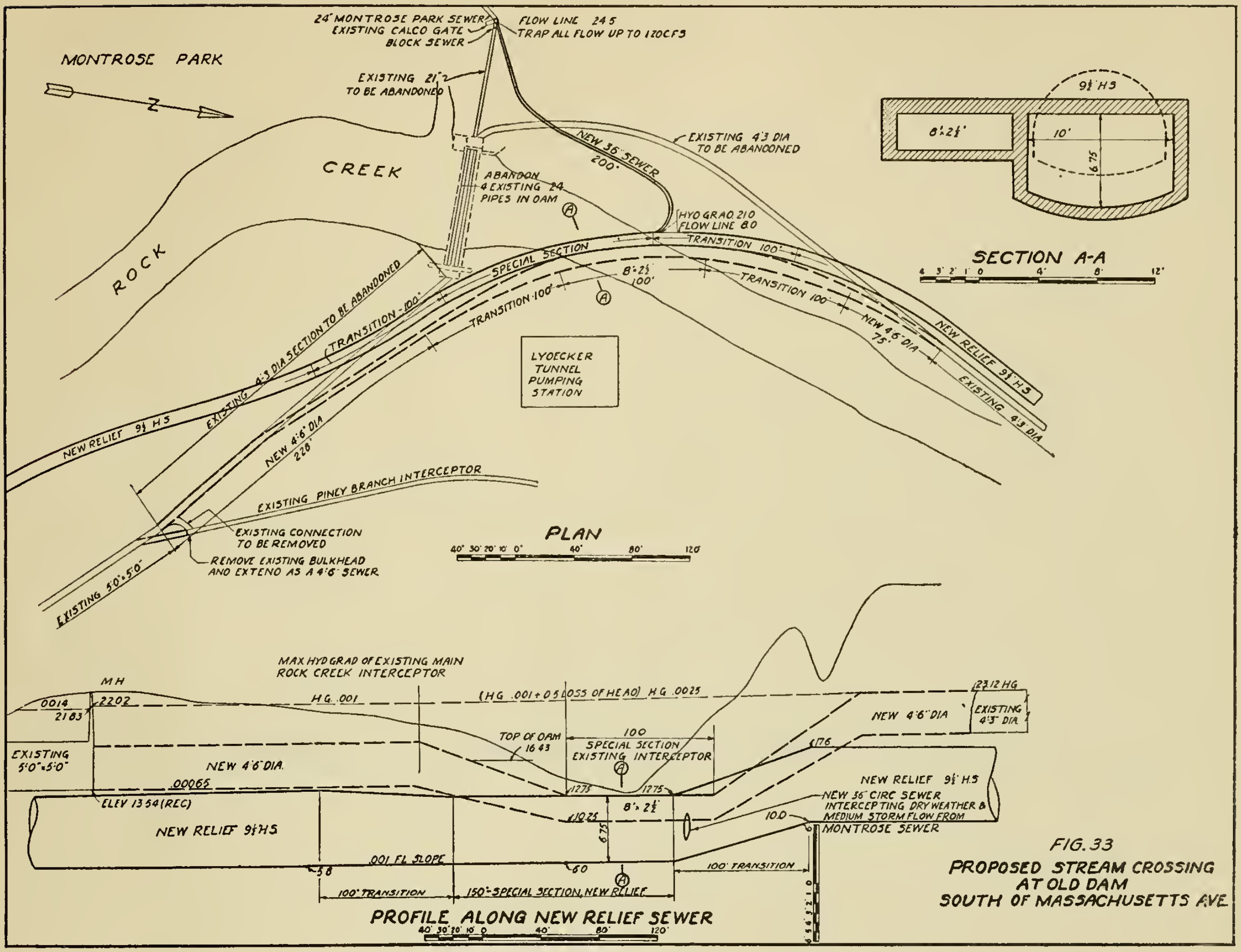


FIG. 33
PROPOSED STREAM CROSSING
AT OLD DAM
SOUTH OF MASSACHUSETTS AVE.

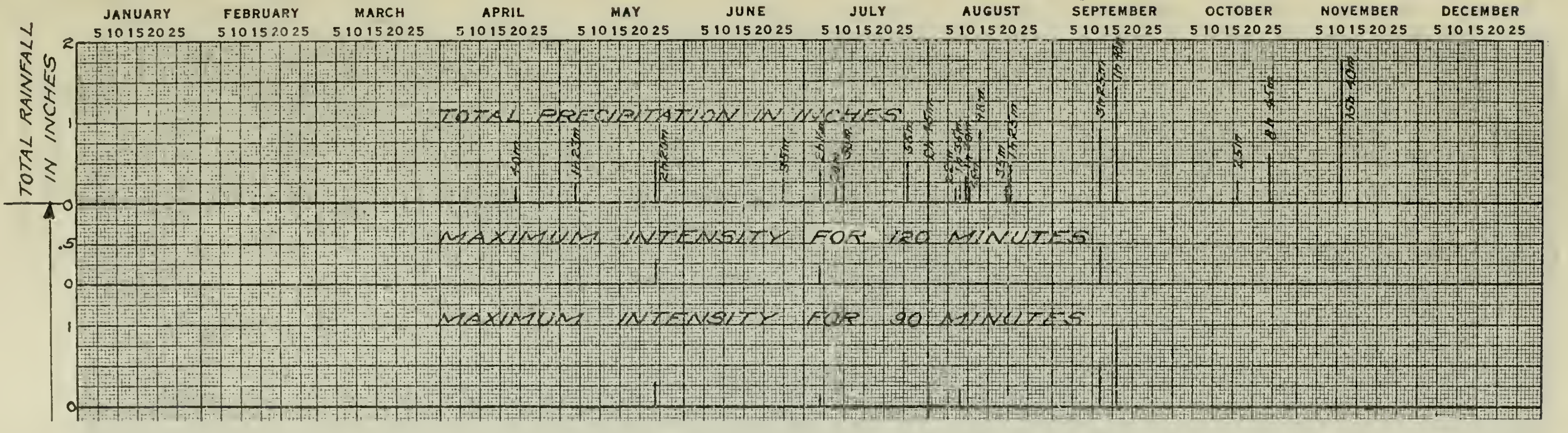
TABLE 18.—*Diversions to west side relief sewer used with all plans*

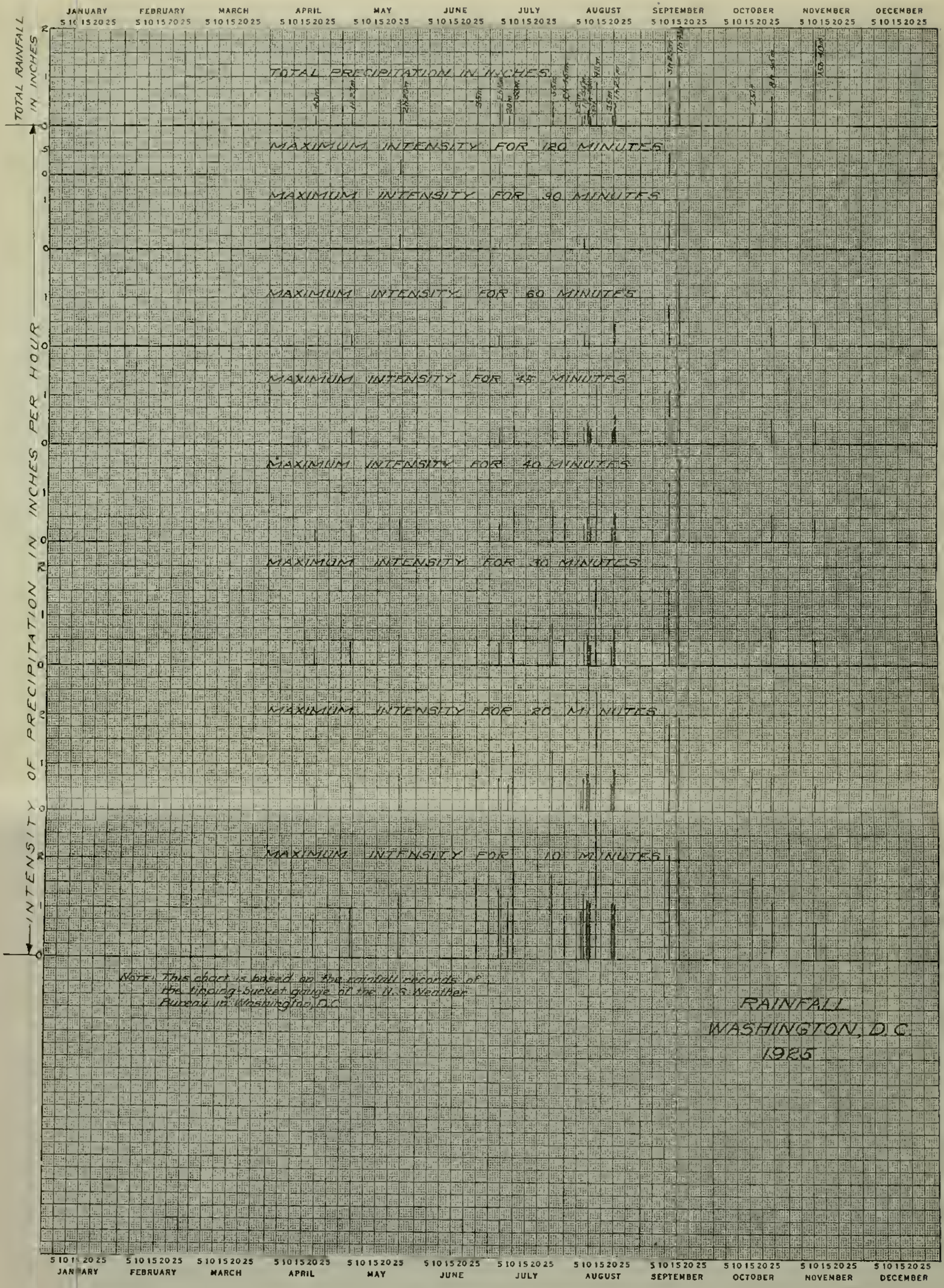
Diversion	Acreage intercepted: From	Acreage intercepted: Increment	Unit run-off	Intercepted run-off	Remarks
Connecticut Avenue	Connecticut Avenue	102	1.25	127	New sump.
Cleveland Avenue	Cleveland Avenue	87	1.25	109	New sump and ten feet of 48 inches.
Normanstone	Normanstone	242	1.25	302	New sump, 20 feet of 36 inches and 60 feet of 18 inches, for foul flow connection.
Montrose	Montrose	121	1.0	121	New sump and 200 feet of 36 inches.
Q Street	Q Street	9	1.25	12	New sump and connection.
P Street	P Street	5	1.25	6	Do.
O Street	O Street	113	1.25	141	Do.
Olive Street	Olive Street	24	1.25	30	Do.
M Street	M Street	39	1.25	49	Do.

APPENDIX TO REPORT ON ROCK CREEK POLLUTION

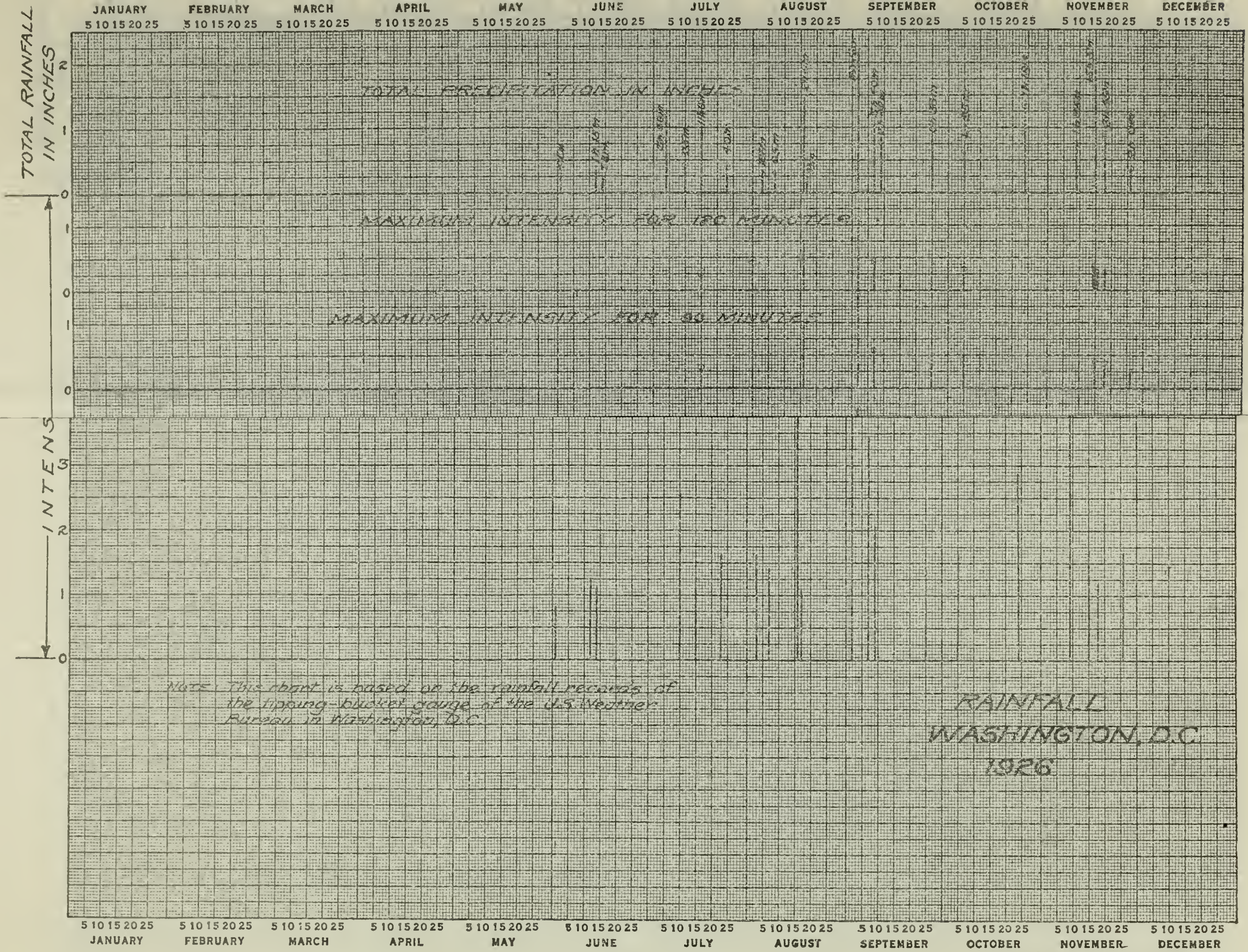
Appendix A

ELEVEN (11) CHARTS SHOWING RELATION OF FREQUENCY AND INTENSITY OF RAINFALL

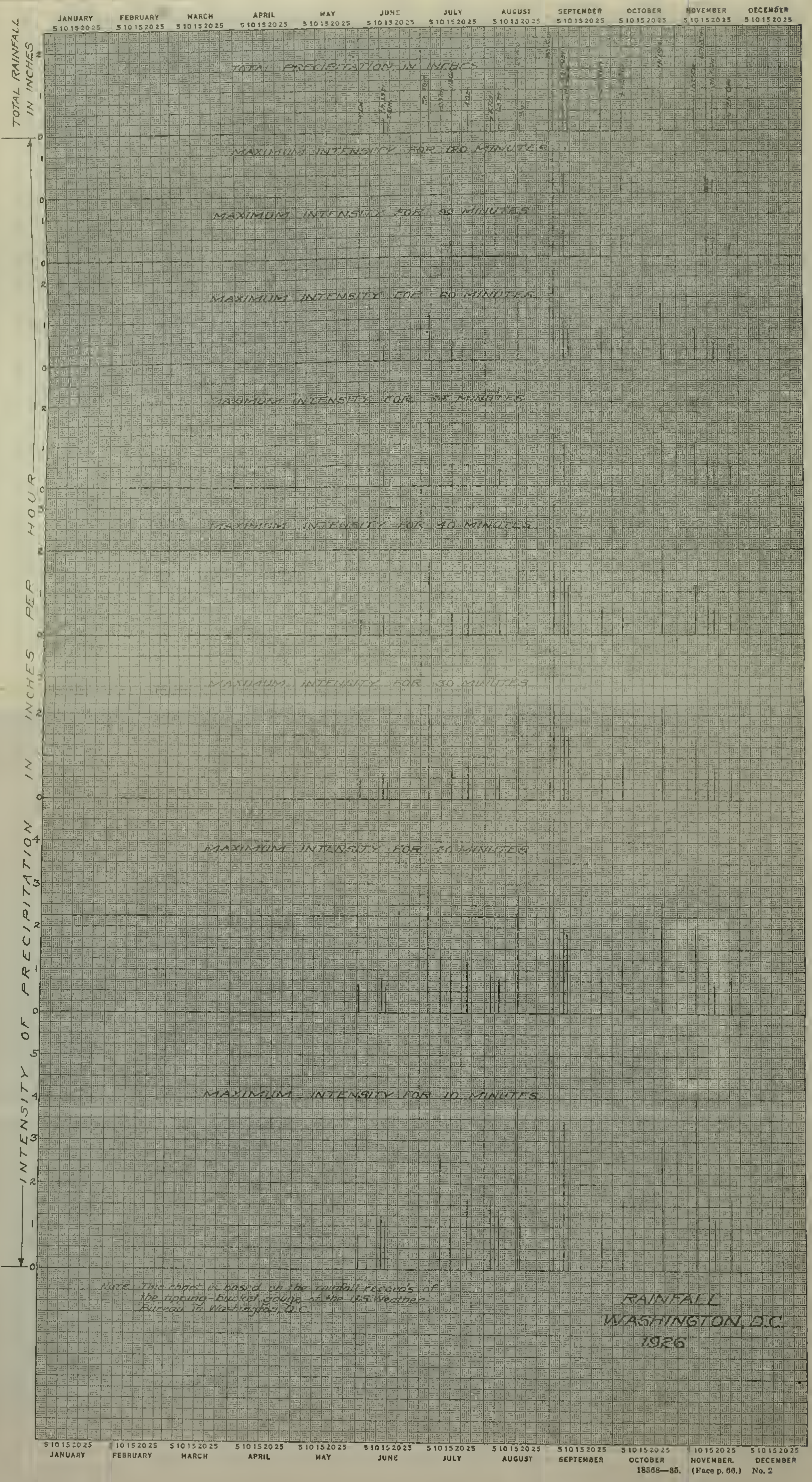

JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
TOTAL RAINFALL IN INCHES
TOTAL PRECIPITATION IN INCHES
MAXIMUM INTENSITY FOR 120 MINUTES
MAXIMUM INTENSITY FOR 90 MINUTES



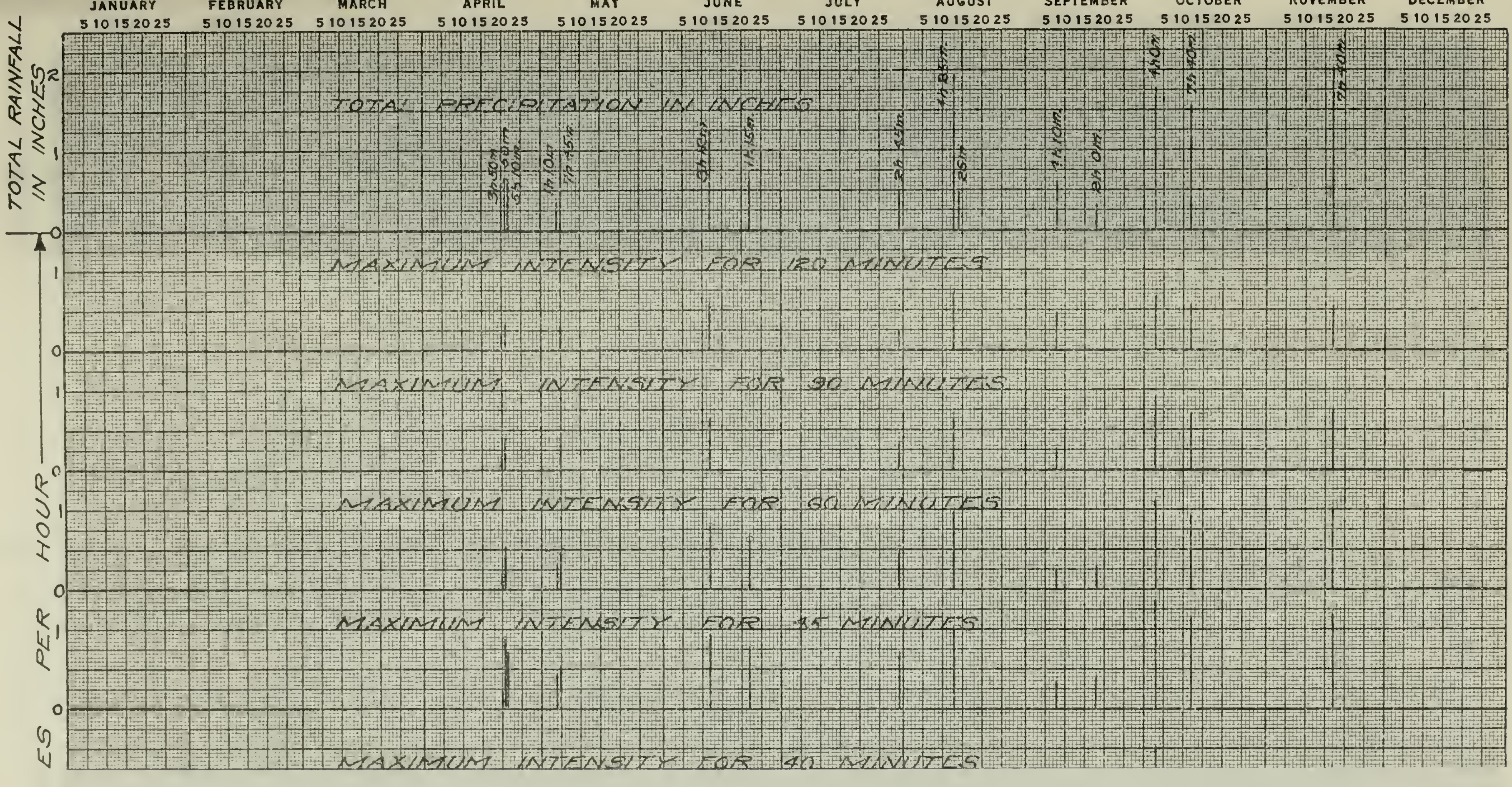
18308—35. (Face p. 66.) No. 1



18368—35. (Face p. 66.) No. 2



18368—85. (Face p. 66.) No. 2


JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
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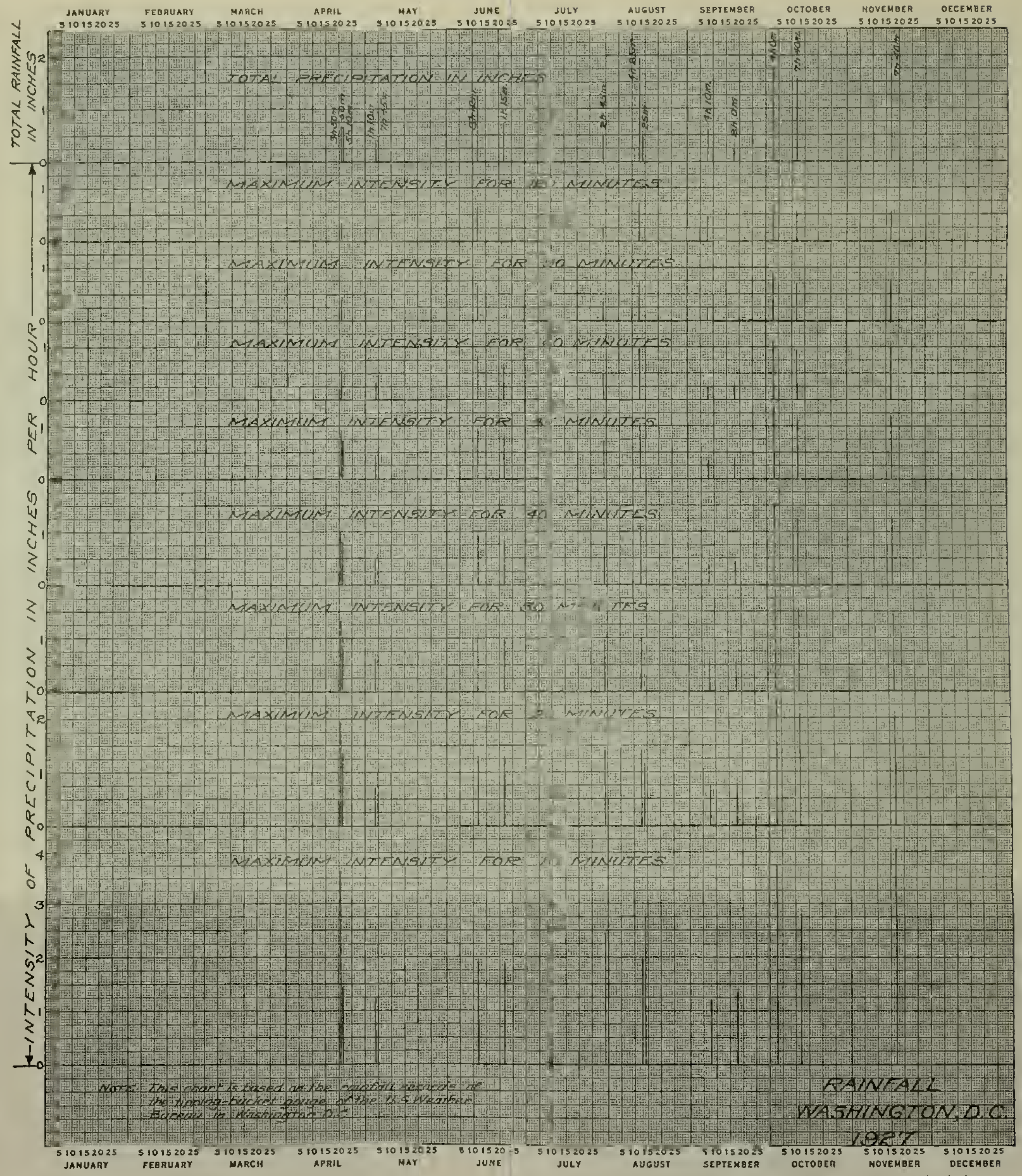
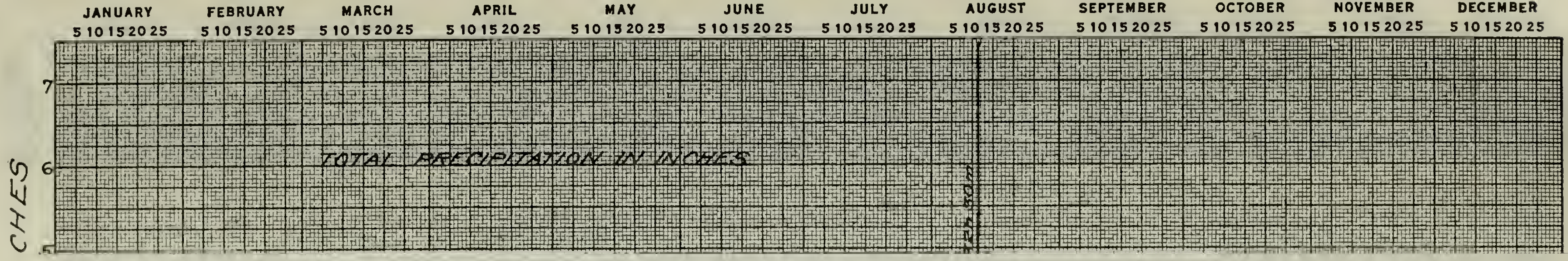
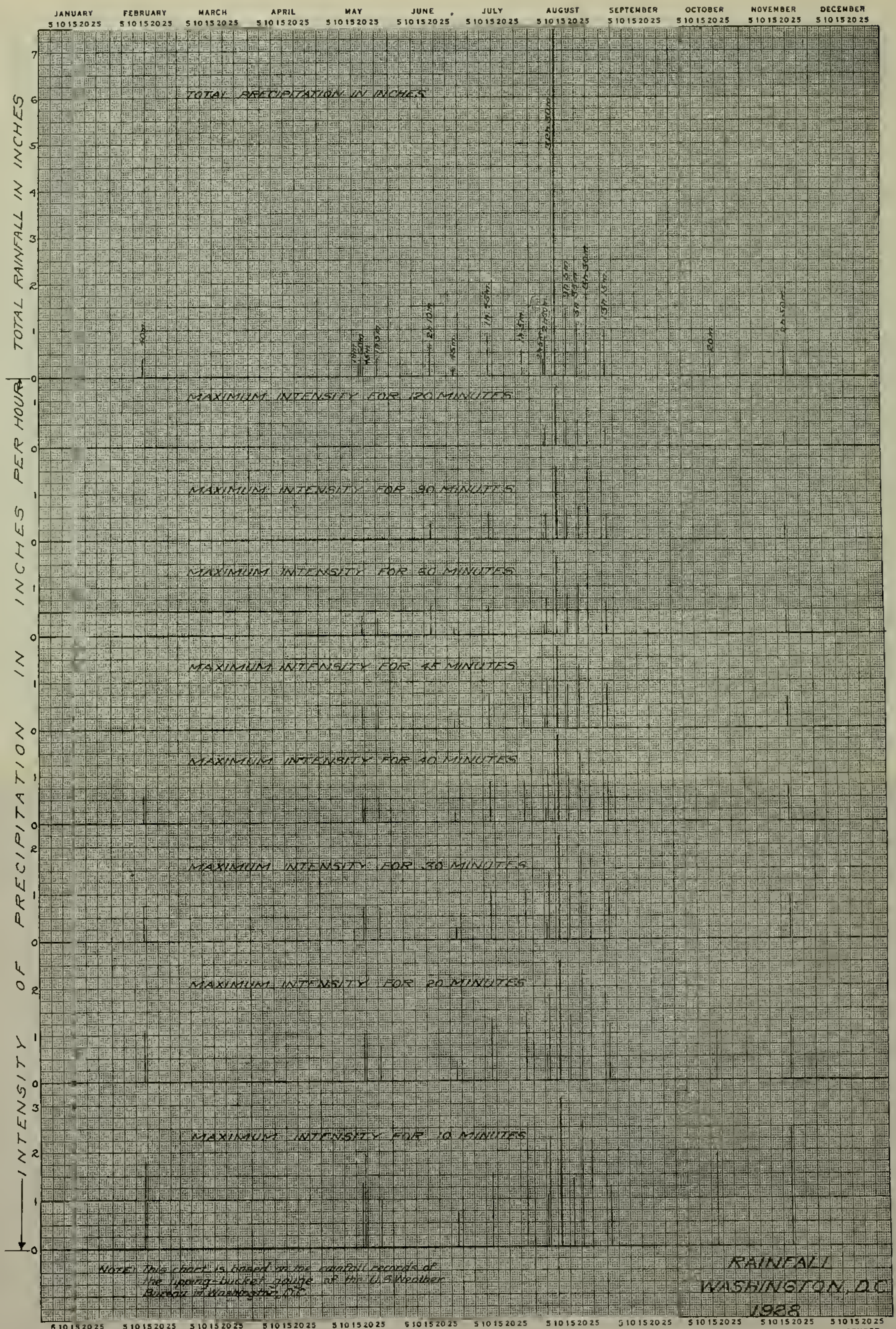
OCTOBER
NOVEMBER
DECEMBER
TOTAL RAINFALL IN INCHES
TOTAL PRECIPITATION IN INCHES
MAXIMUM INTENSITY FOR 120 MINUTES
MAXIMUM INTENSITY FOR 90 MINUTES
MAXIMUM INTENSITY FOR 60 MINUTES
MAXIMUM INTENSITY FOR 45 MINUTES
MAXIMUM INTENSITY FOR 40 MINUTES
HOUR
PER
ES

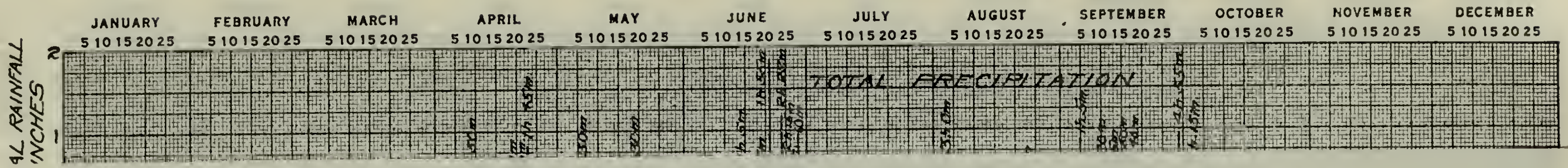


18368—35. (Face p. 66.) No. 3

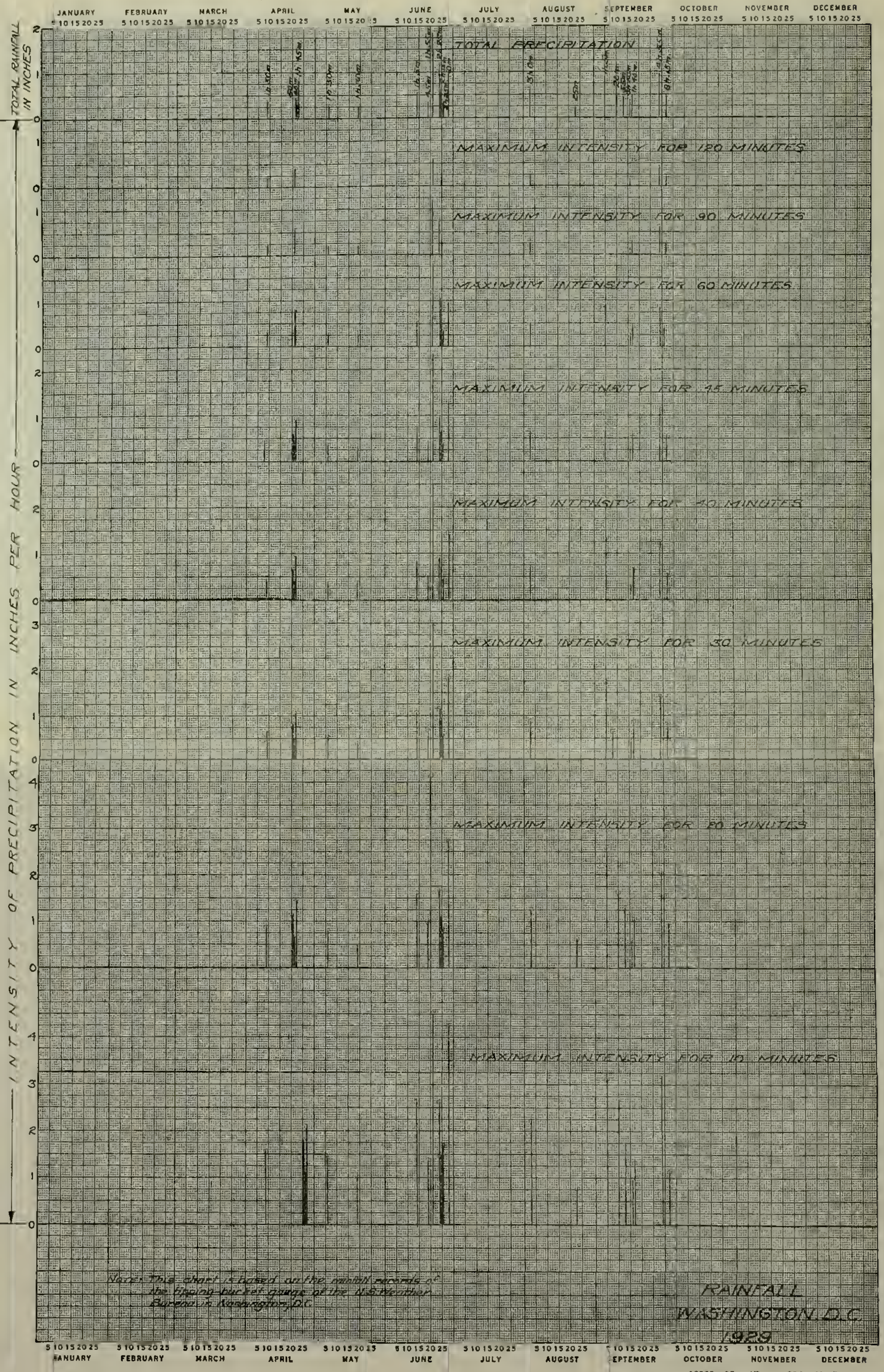


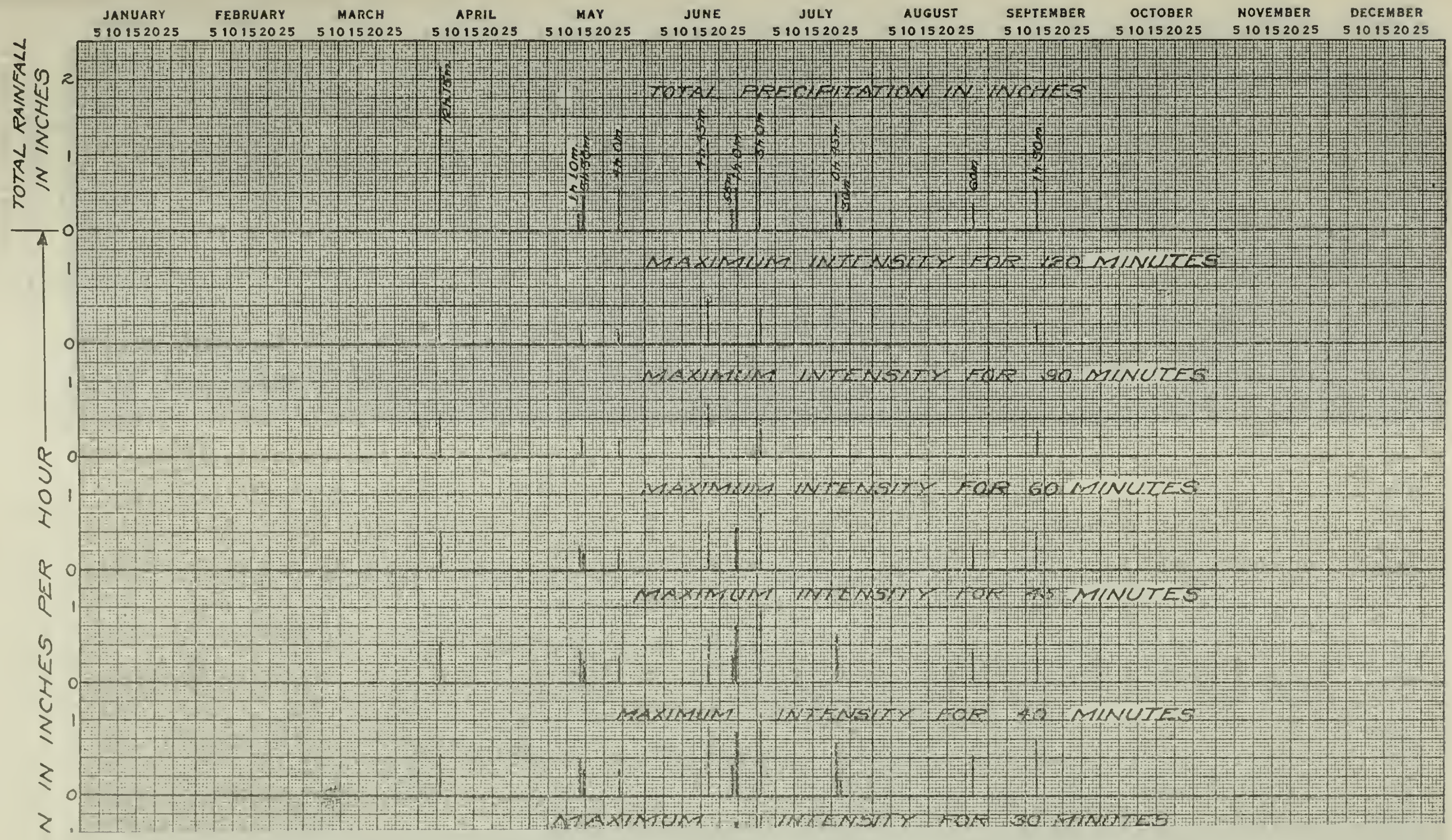
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
7
6
CHES
TOTAL PRECIPITATION IN INCHES


RAINFALL
WASHINGTON, D.C.
1928
TOTAL RAINFALL IN INCHES
INTENSITY OF PRECIPITATION IN INCHES PER HOUR
TOTAL PRECIPITATION IN INCHES
MAXIMUM INTENSITY FOR 120 MINUTES
MAXIMUM INTENSITY FOR 90 MINUTES
MAXIMUM INTENSITY FOR 60 MINUTES
MAXIMUM INTENSITY FOR 45 MINUTES
MAXIMUM INTENSITY FOR 40 MINUTES
MAXIMUM INTENSITY FOR 30 MINUTES
MAXIMUM INTENSITY FOR 20 MINUTES
MAXIMUM INTENSITY FOR 10 MINUTES
NOTE: This chart is based on the rainfall records of the tipping-bucket gauge of the U.S. Weather Bureau in Washington, D.C.
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER

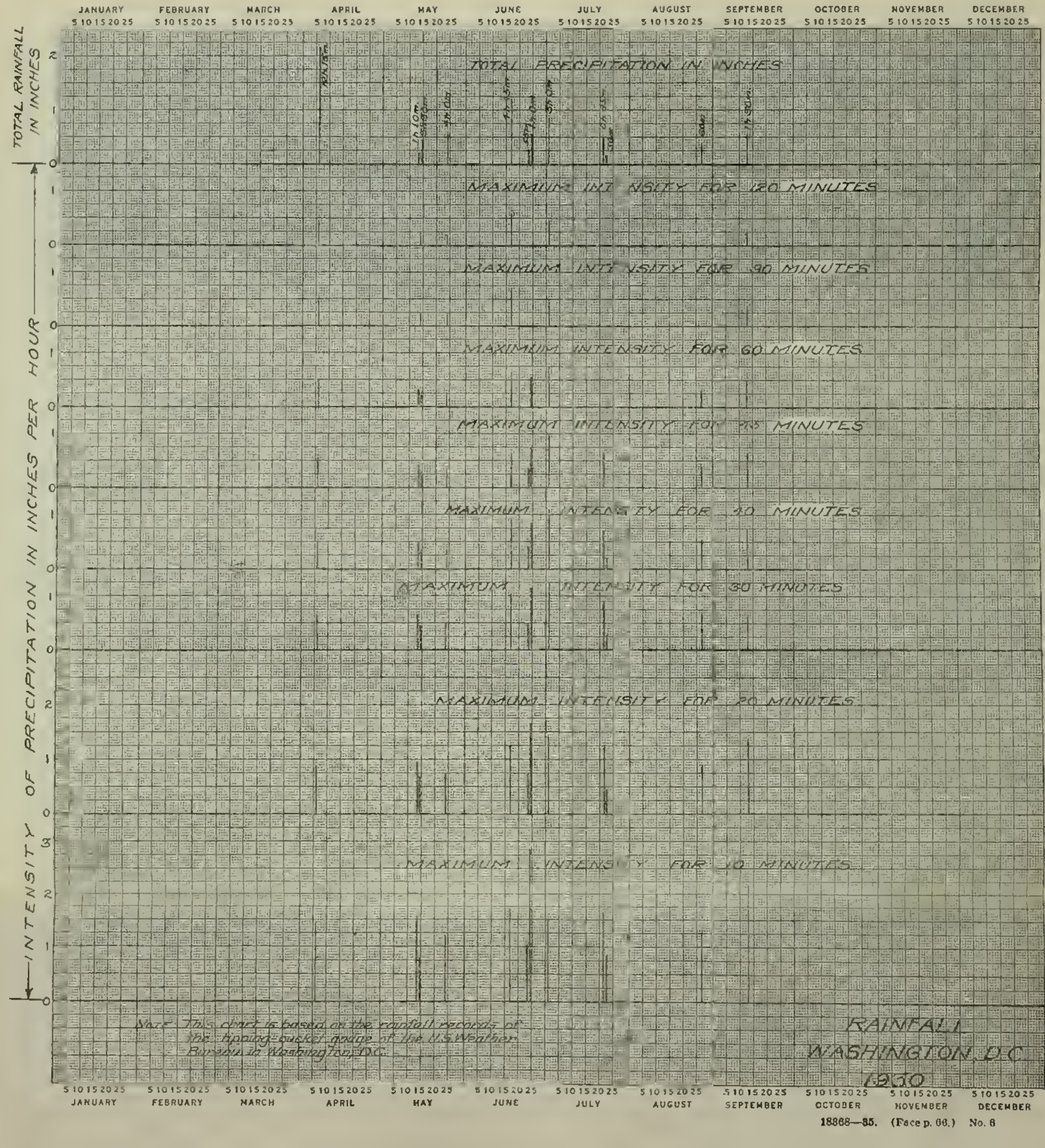


JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
TOTAL PRECIPITATION
AL RAINFALL
'NCHES

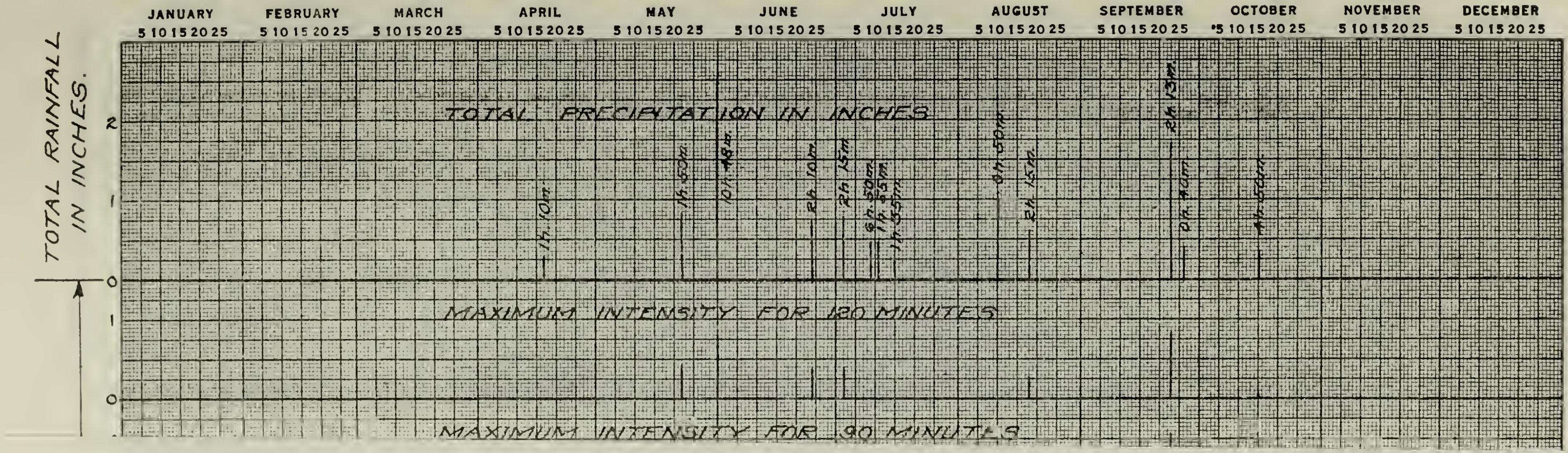

RAINFALL
WASHINGTON, D.C.
1929
TOTAL PRECIPITATION
MAXIMUM INTENSITY FOR 120 MINUTES
MAXIMUM INTENSITY FOR 90 MINUTES
MAXIMUM INTENSITY FOR 60 MINUTES
MAXIMUM INTENSITY FOR 45 MINUTES
MAXIMUM INTENSITY FOR 30 MINUTES
MAXIMUM INTENSITY FOR 20 MINUTES
MAXIMUM INTENSITY FOR 10 MINUTES
TOTAL RAINFALL IN INCHES
INTENSITY OF PRECIPITATION IN INCHES PER HOUR
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
Note: This chart is based on the rainfall records of the tipping-bucket gauge of the U.S. Weather Bureau in Washington, D.C.



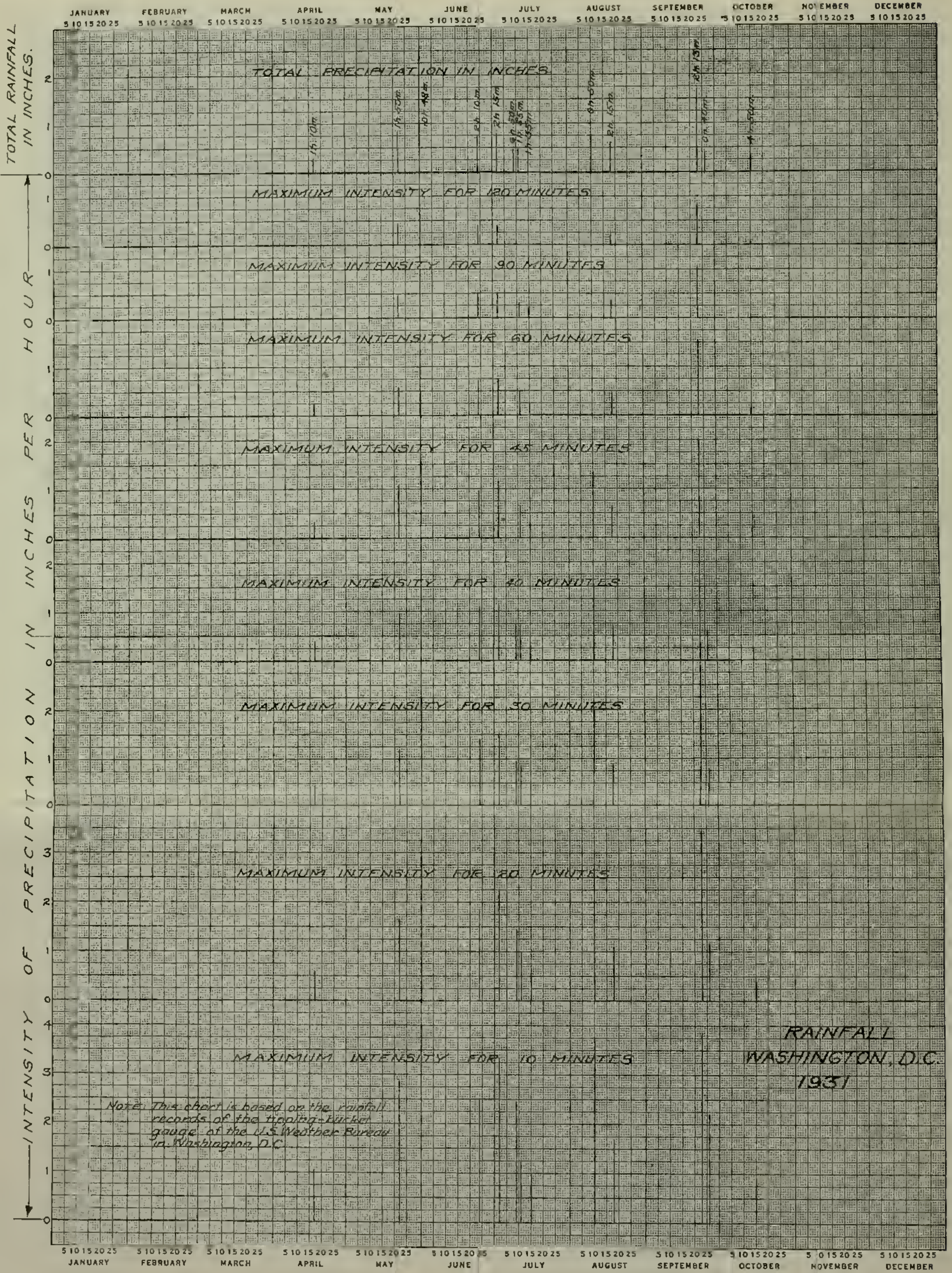
JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
TOTAL RAINFALL IN INCHES
IN INCHES PER HOUR
TOTAL PRECIPITATION IN INCHES
MAXIMUM INTENSITY FOR 120 MINUTES
MAXIMUM INTENSITY FOR 90 MINUTES
MAXIMUM INTENSITY FOR 60 MINUTES
MAXIMUM INTENSITY FOR 40 MINUTES
MAXIMUM INTENSITY FOR 30 MINUTES



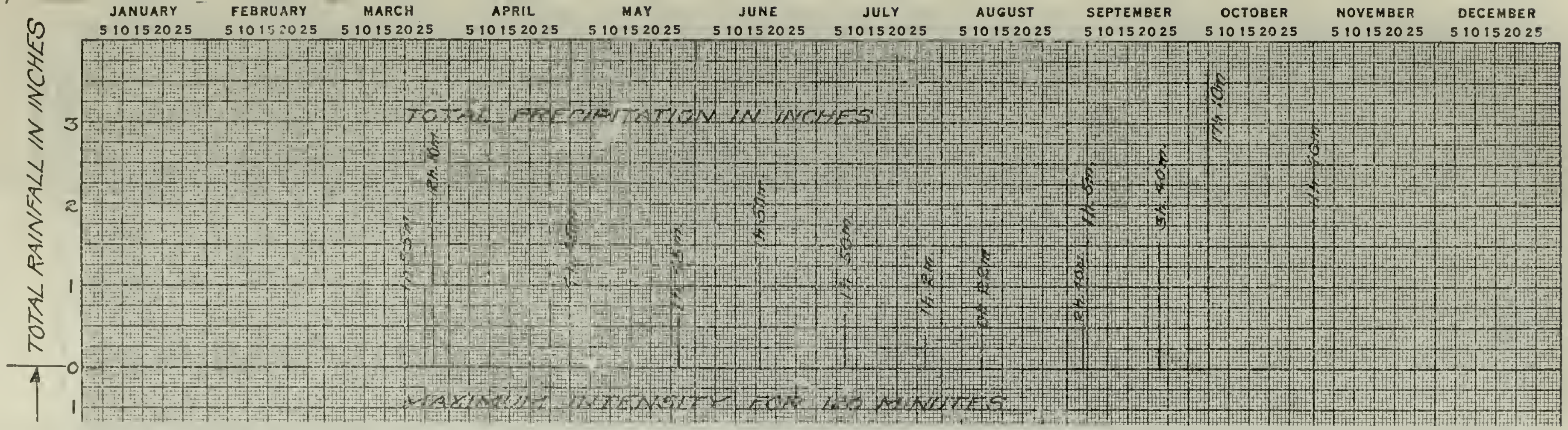
18368—35. (Face p. 66.) No. 6

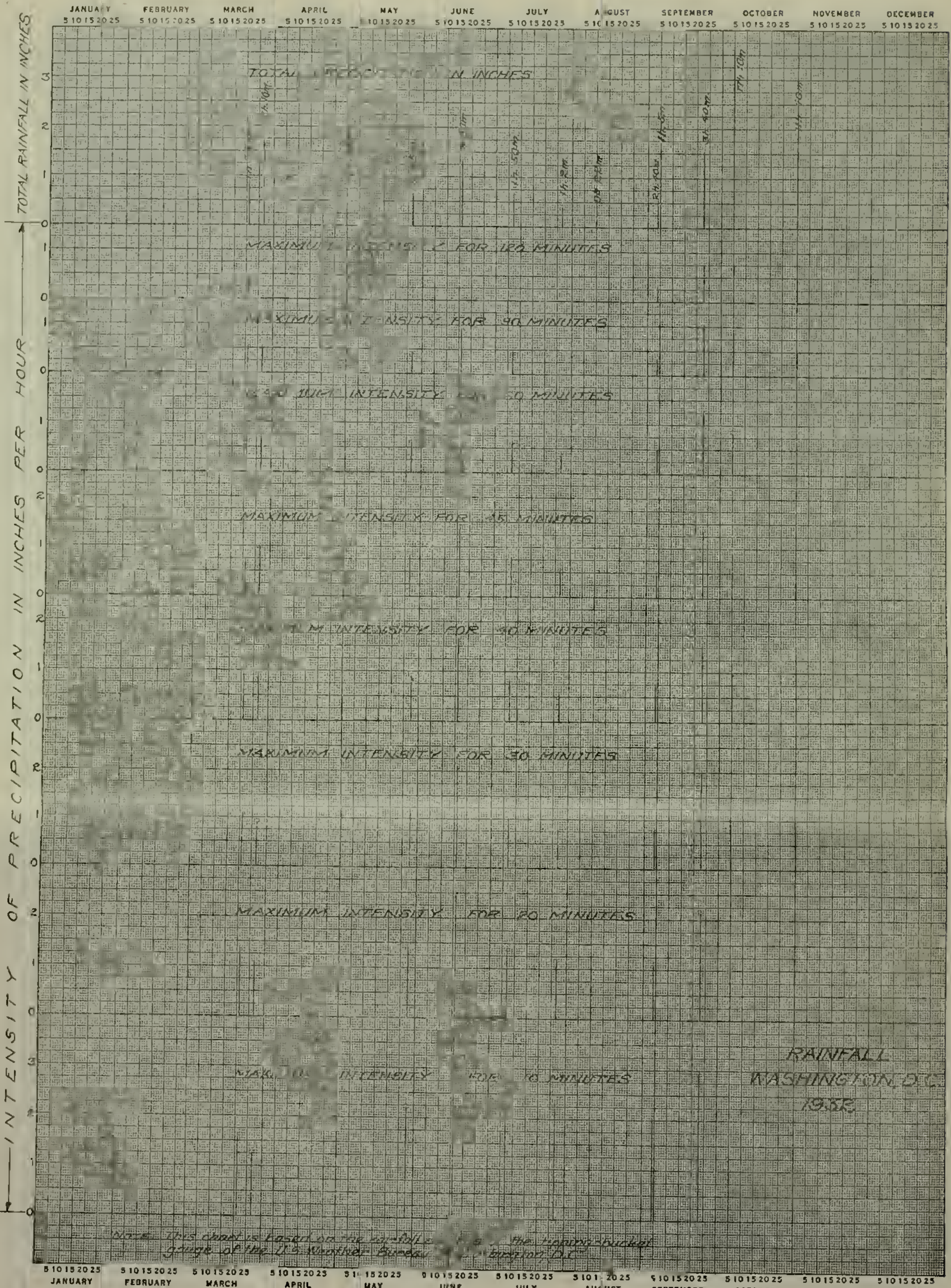


JANUARY 5 10 15 20 25
FEBRUARY 5 10 15 20 25
MARCH 5 10 15 20 25
APRIL 5 10 15 20 25
MAY 5 10 15 20 25
JUNE 5 10 15 20 25
JULY 5 10 15 20 25
AUGUST 5 10 15 20 25
SEPTEMBER 5 10 15 20 25
OCTOBER 5 10 15 20 25
NOVEMBER 5 10 15 20 25
DECEMBER 5 10 15 20 25
TOTAL RAINFALL IN INCHES.
TOTAL PRECIPITATION IN INCHES
MAXIMUM INTENSITY FOR 120 MINUTES
MAXIMUM INTENSITY FOR 90 MINUTES
1h 10m
1h 50m
10h 48m
2h 10m
2h 15m
6h 50m
1h 55m
1h 35m
0h 50m
2h 15m
2h 13m
0h 40m
4h 50m

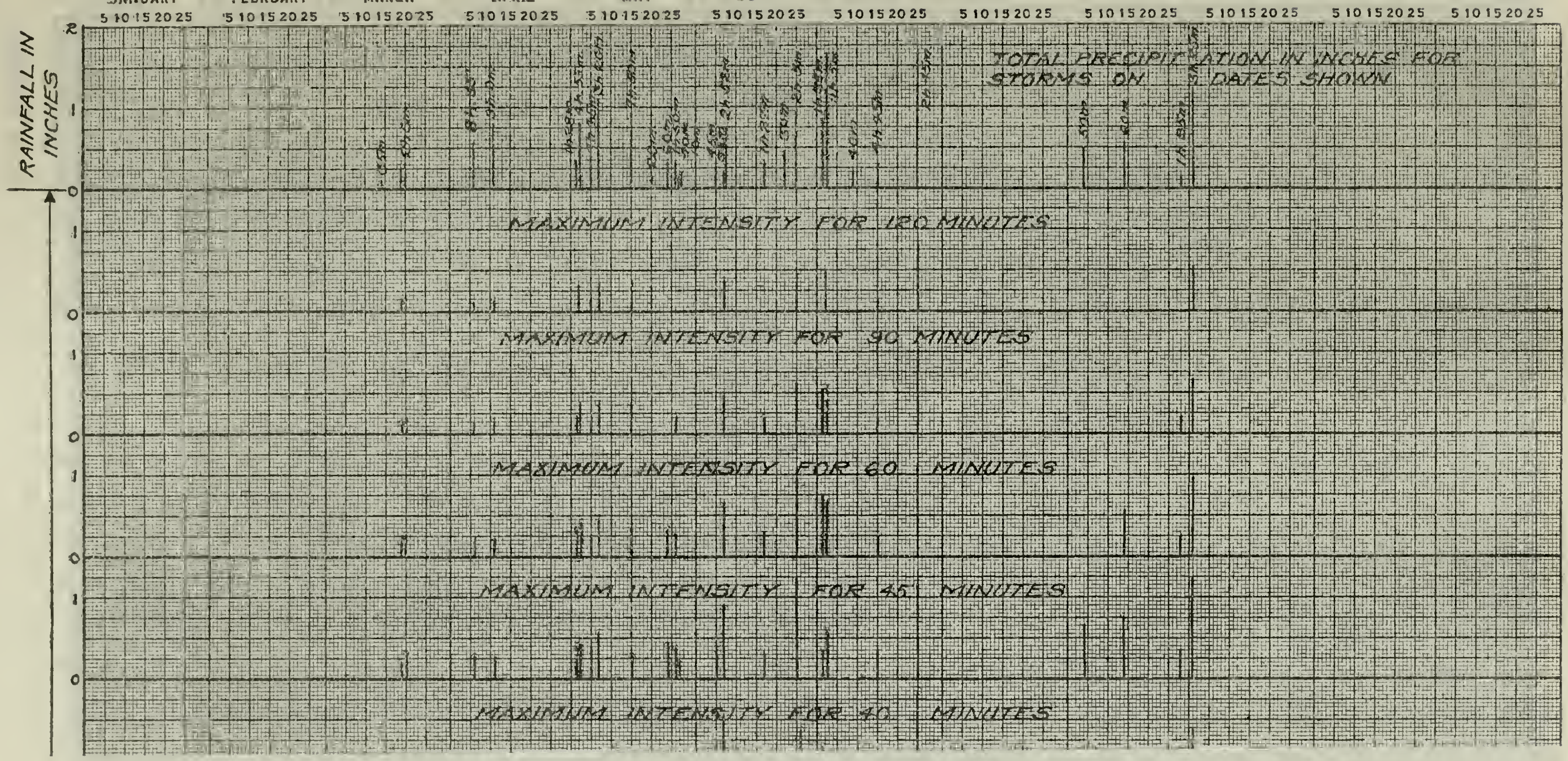


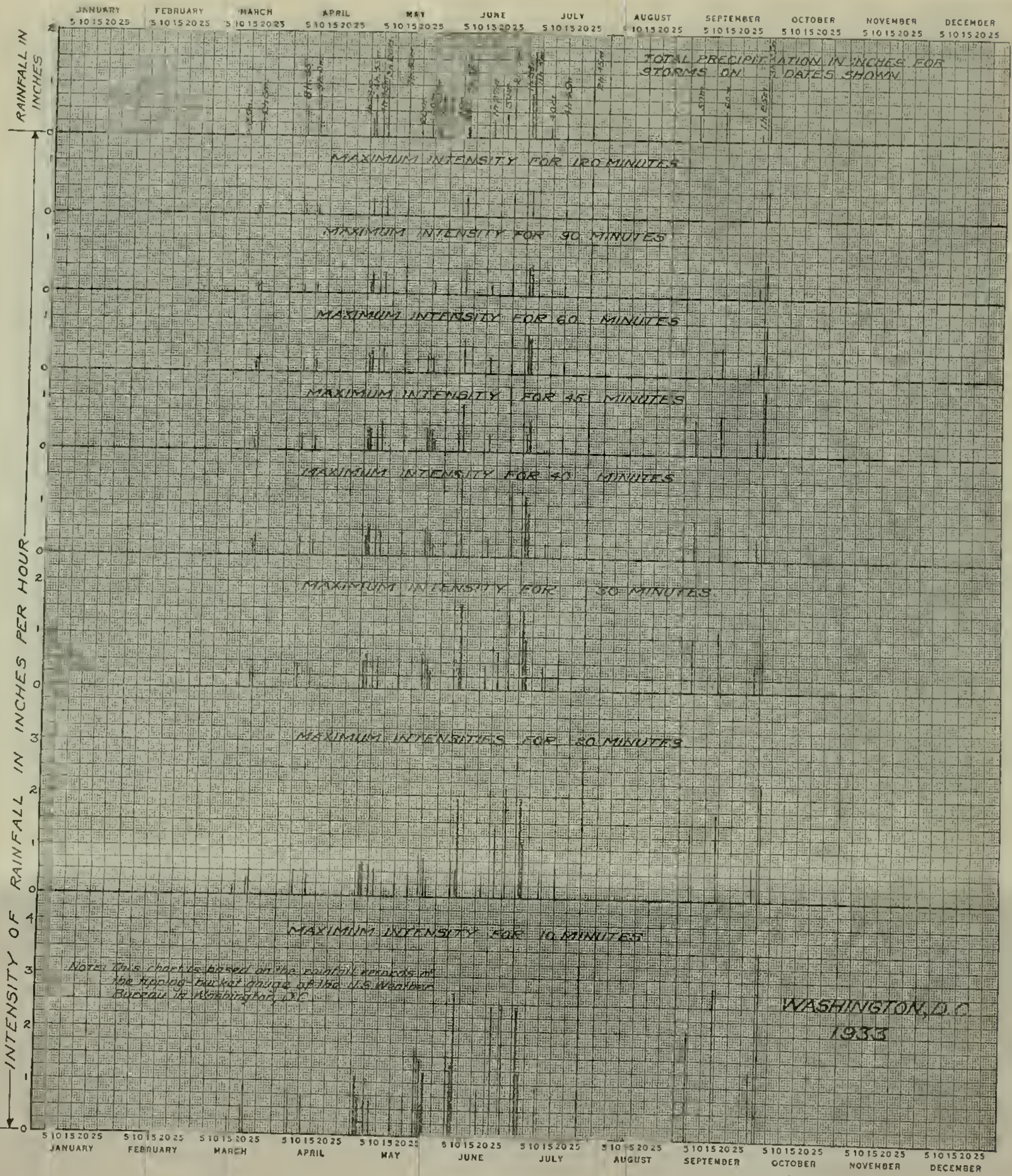
18308—35. (Face p. 66.) No. 7


JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
TOTAL RAINFALL IN INCHES
TOTAL PRECIPITATION IN INCHES

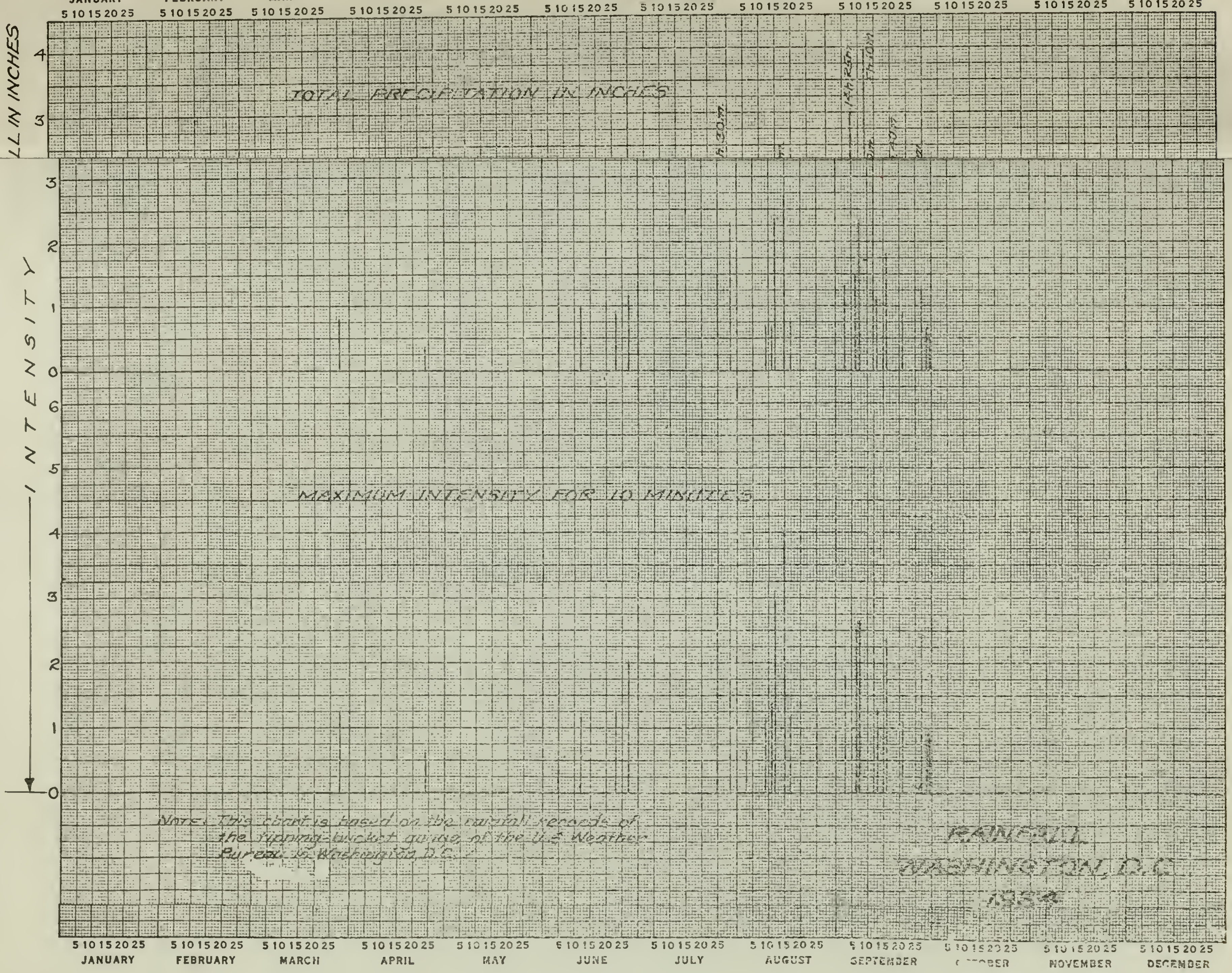


18308—35. (Face p. 66.) No. 8

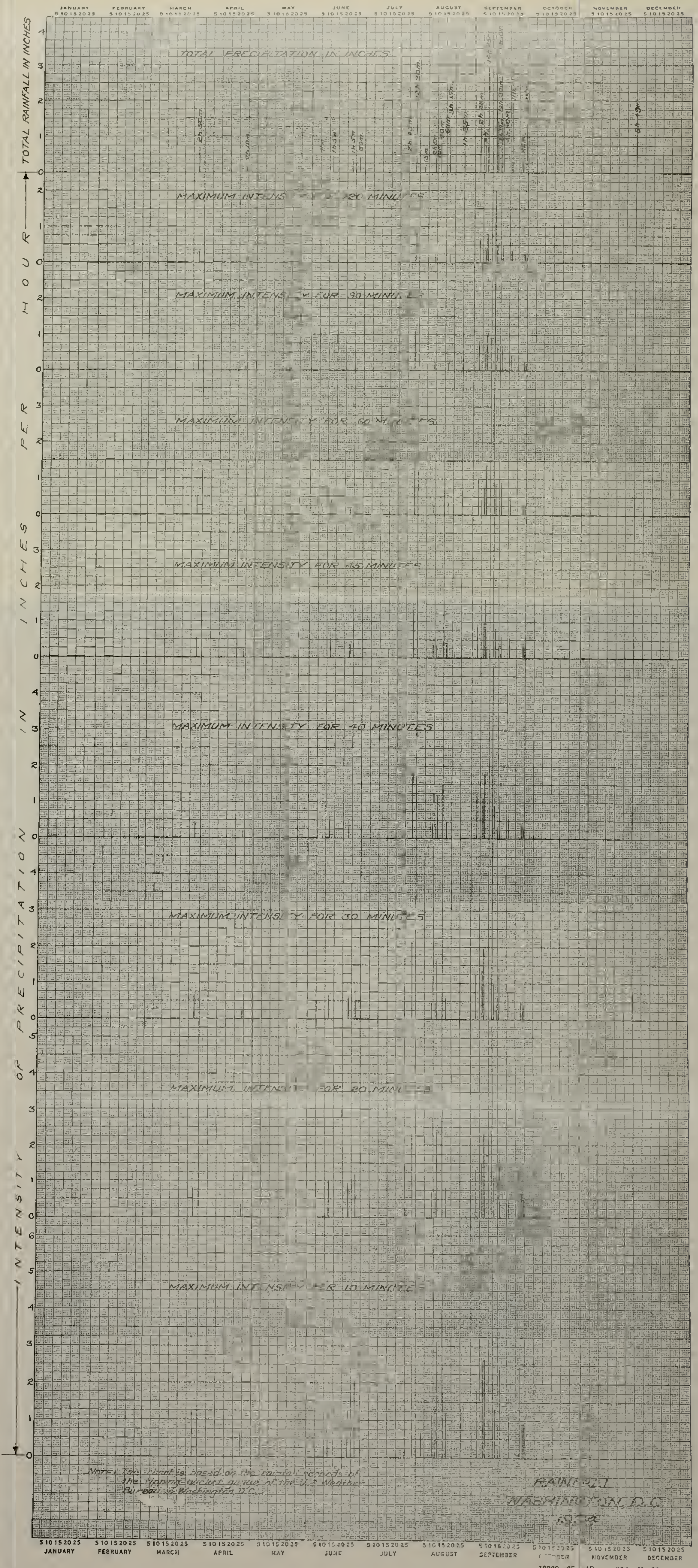

RAINFALL IN INCHES
TOTAL PRECIPITATION IN INCHES FOR STORMS ON DATES SHOWN
MAXIMUM INTENSITY FOR 120 MINUTES
MAXIMUM INTENSITY FOR 90 MINUTES
MAXIMUM INTENSITY FOR 60 MINUTES
MAXIMUM INTENSITY FOR 45 MINUTES
MAXIMUM INTENSITY FOR 40 MINUTES



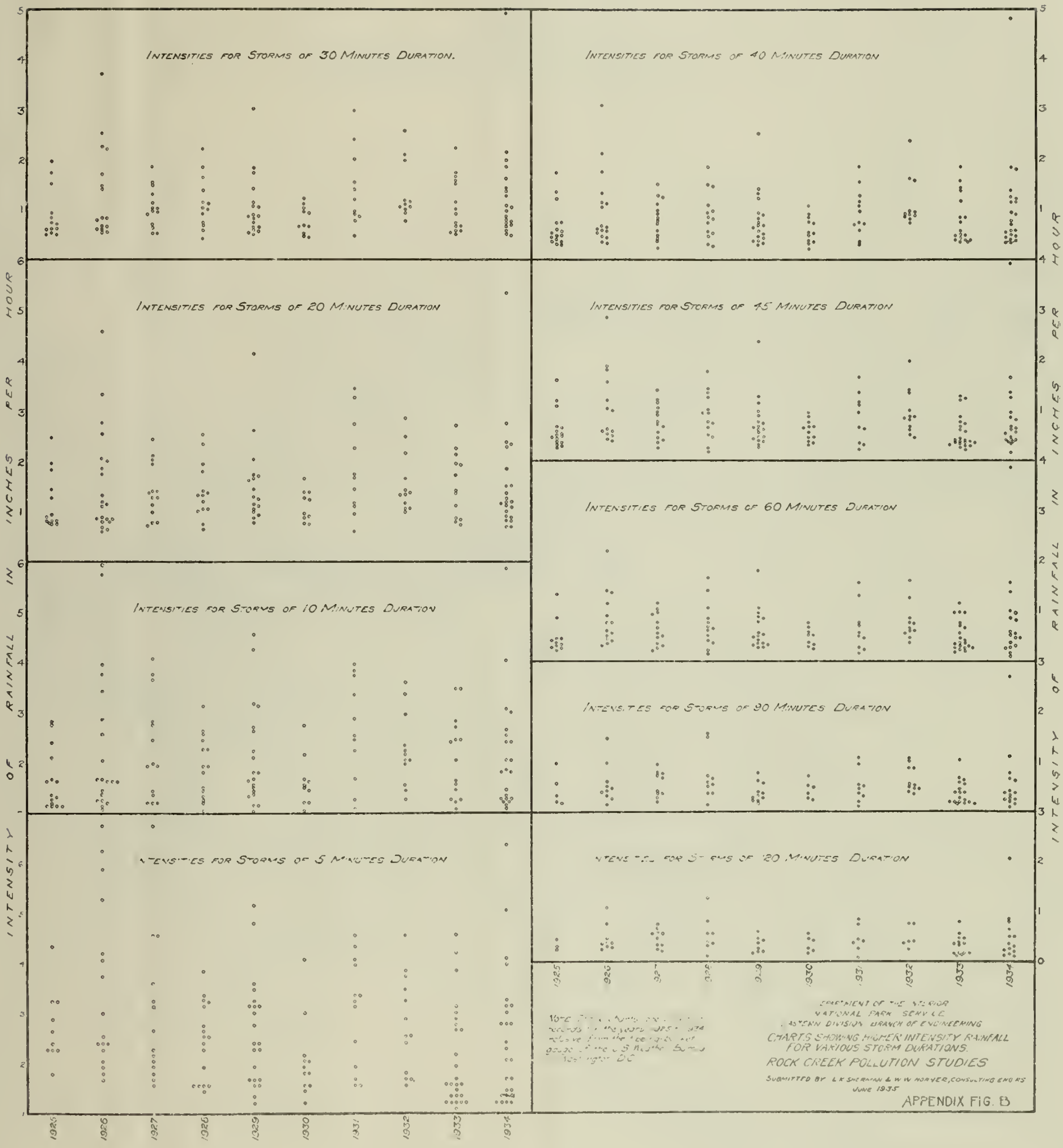
18368—35. (Face p. 60.) No. 9



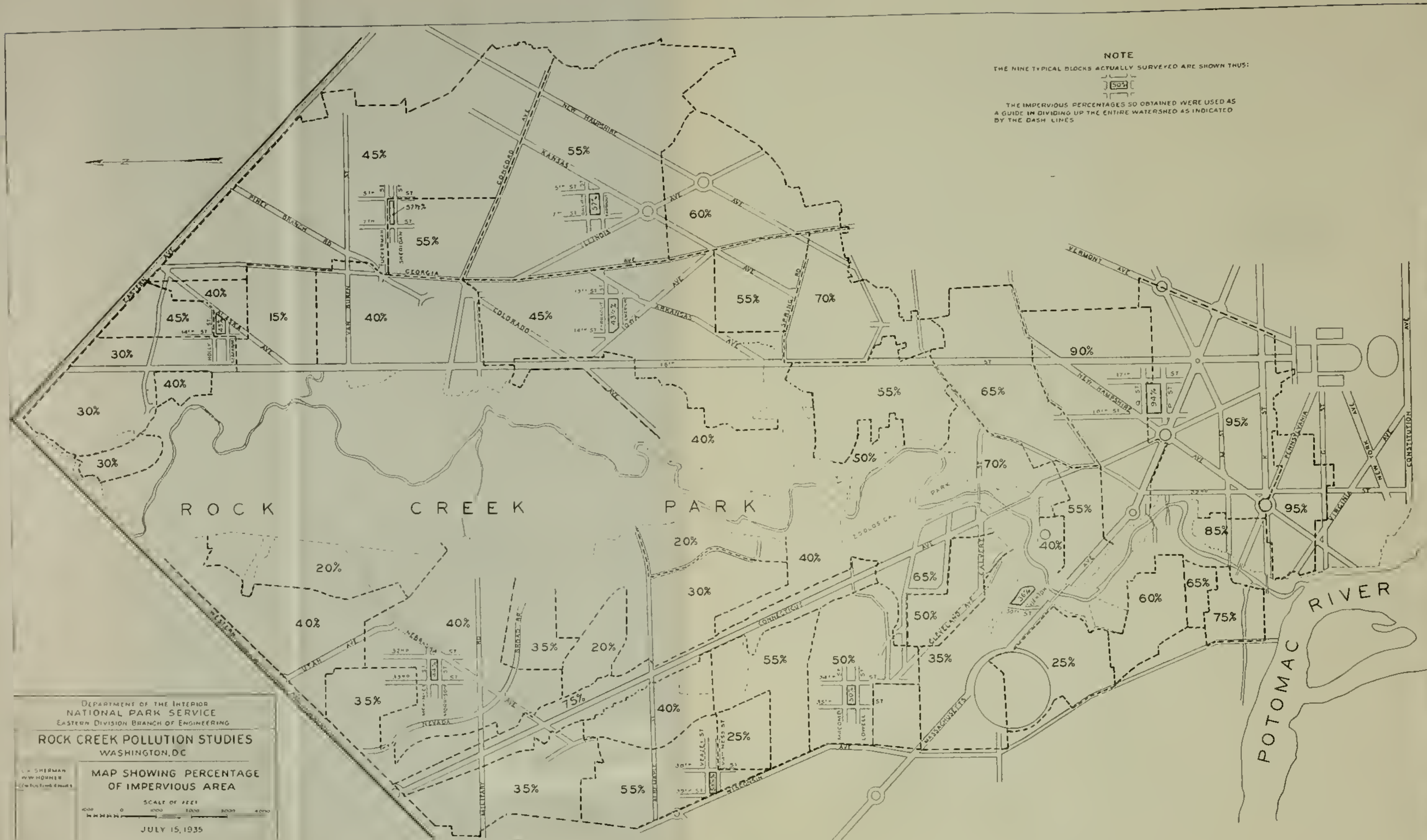
18368—35. (Face p. 66.) No. 10



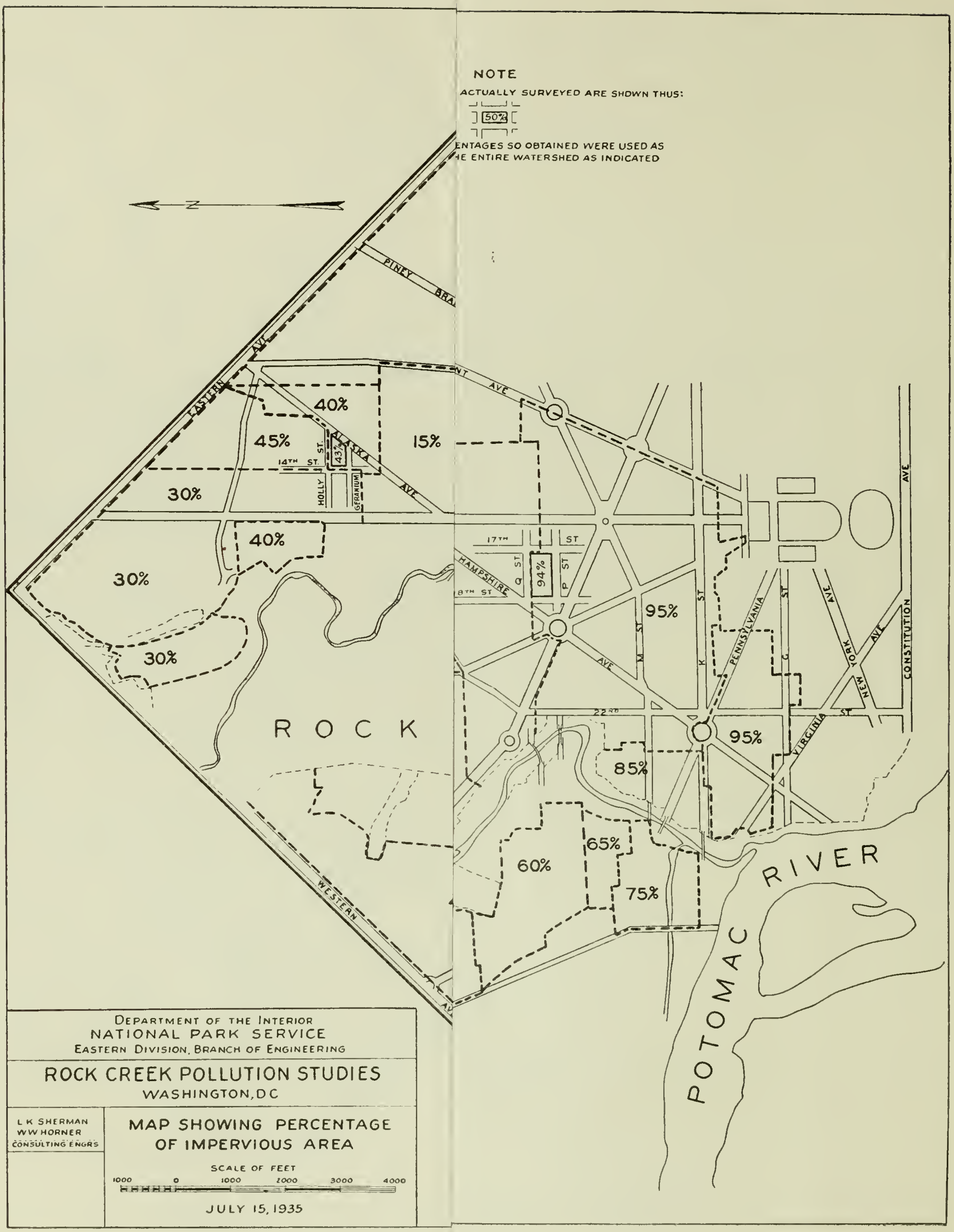
18308—33. (Face p. 00.) No. 10



APPENDIX FIG. B



NOTE
THE NINE TYPICAL BLOCKS ACTUALLY SURVEYED ARE SHOWN THUS:
50%
THE IMPERVIOUS PERCENTAGES SO OBTAINED WERE USED AS A GUIDE IN DIVIDING UP THE ENTIRE WATERSHED AS INDICATED BY THE DASH LINES
DEPARTMENT OF THE INTERIOR
NATIONAL PARK SERVICE
EASTERN DIVISION BRANCH OF ENGINEERING
ROCK CREEK POLLUTION STUDIES
WASHINGTON, D.C.
MAP SHOWING PERCENTAGE OF IMPERVIOUS AREA
SCALE OF FEET
JULY 15, 1935
ROCK CREEK PARK
POTOMAC RIVER
NEW HAMPSHIRE
KANSAS
COLORADO
GEORGIA
PINEY BRANCH RD
ILLINOIS
ARKANSAS
IOWA
SPRING RD
VERMONT AVE
PENNSYLVANIA
NEW YORK
CONSTITUTION
VIRGINIA
CONNECTICUT
CLEVELAND AVE
MASSACHUSETTS
NEBRASKA
NEVADA
UTAH
MILITARY
BROAD BR
ALBEMARLE
WISCONSIN
VAN BUREN
ALASKA
16TH ST
ZOOLOGICAL PARK
45%
55%
57½%
60%
40%
15%
30%
70%
90%
65%
95%
50%
85%
20%
35%
75%
25%



18368—35. (Face p. 67.)

Appendix B

PHOTOSTATIC REDUCTION OF MAP SHOWING PERCENTAGE OF IMPERVIOUS AREA

Appendix C

INTERIOR CONDITION OF MAIN ROCK CREEK INTERCEPTOR

1. *At Klingle Road manhole.*—Concrete walls and arch very smooth between form joints. At every joint, however, there is a visible offset where forms failed to line up. Furthermore, leakage at some of these joints has produced stalactites which hang from the arch. Walls and arch as far as visible will average 0.012. The flat-brick bottom was invisible but it felt like 0.015. Average $n=0.013$.

2. *At lower portal in Zoo.*—This manhole is built on a steeply inclined circular brick sewer just below the transition from the tunnel section. Velocity very high at manhole and below, over 15 feet per second. Waves from hydraulic jump at foot of incline reach top of sewer. Stalactites hang down 15 inches below arch at a joint some 50 feet below manhole. Average n for transition and incline$=0.014$.

3. *Quarry manhole.*—Could not enter this manhole because of 4-inch-gage pipe held in center of hole by turnbuckle braces. This pipe has caught a mass of rags which back up a 15-inch wall of water.

Depth to water above obstruction, 3.60.

Depth to water below obstruction, 4.92.

Depth to water in pipe, 4.80.

4. *Manhole above Calvert Bridge barricade.*—Water within 20 inches of arch. Concrete smooth between form joints, but consecutive sections do not line up. Curve above manhole built with straight sections of form. Average $n=0.013$.

5. *Manhole below Cleveland Avenue.*—Average $n=0.013$.

6. *Manhole at Normanstone Valley.*—$n=0.013$ upstream because of bad form alinement. Trash caught on some of the worst joints. $n=0.013$ downstream.

7. *Manhole above Massachusetts Avenue.*—$n=0.013$ upstream and 0.013 down.

A straight-form angle to the left about 30 feet below manhole raises a wave halfway across stream. No evidence of the choke section shown in details as far as visible downstream.

8. *Manhole just below junction of east interceptor.*—Some of the worst joints have been plastered smooth. Trash caught on others. General alinement of sections good. Average $n=0.012$.

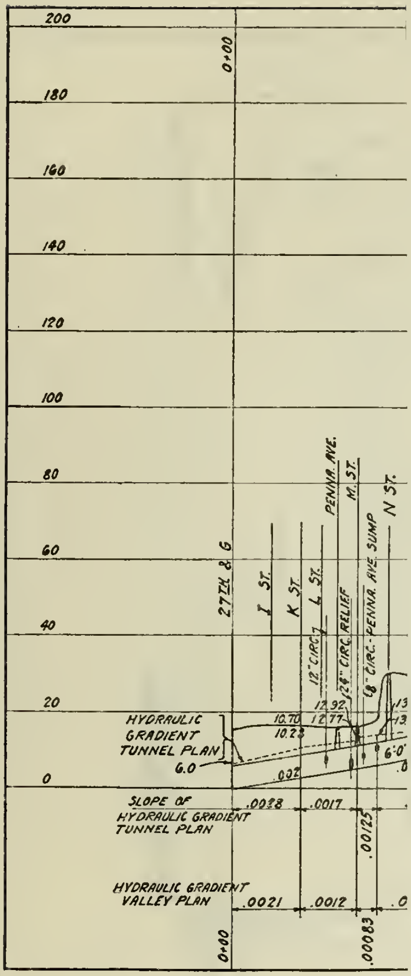
INTERIOR CONDITION OF PINEY BRANCH INTERCEPTOR

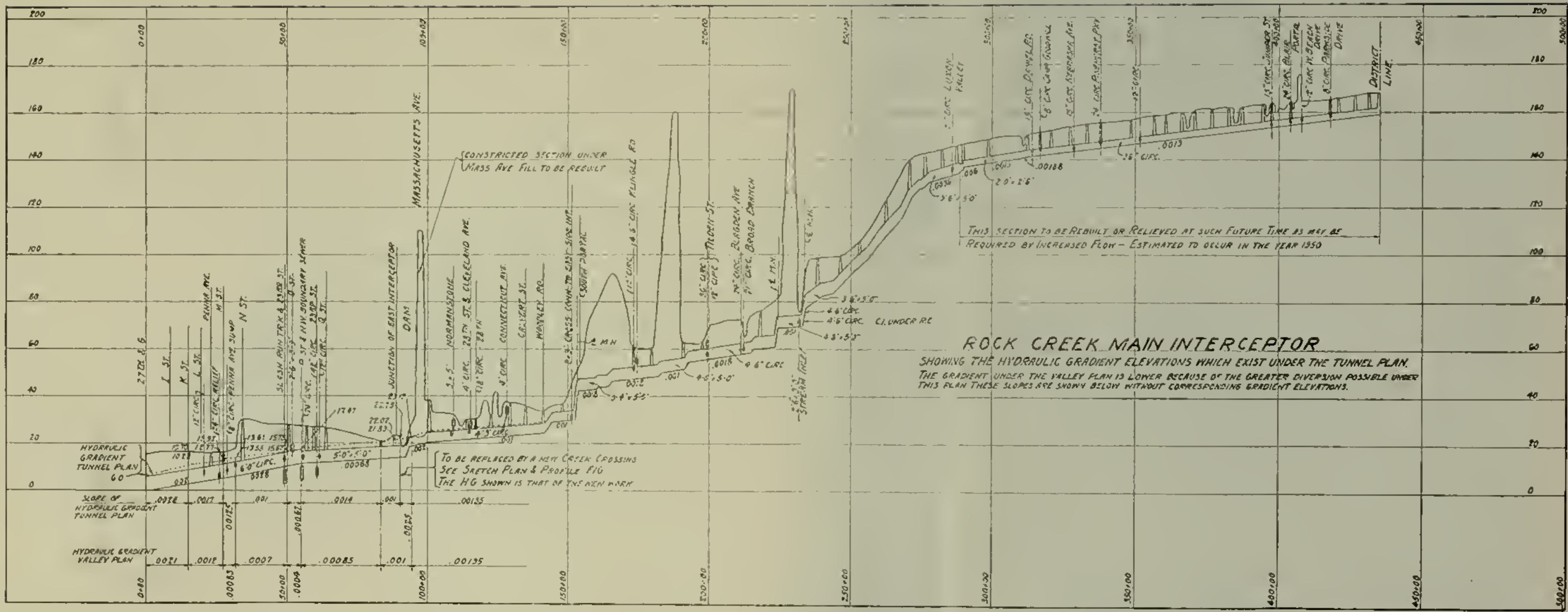
At third manhole below double 30-inch pipe creek crossing.—(This manhole is at the ford above Calvert Street.)

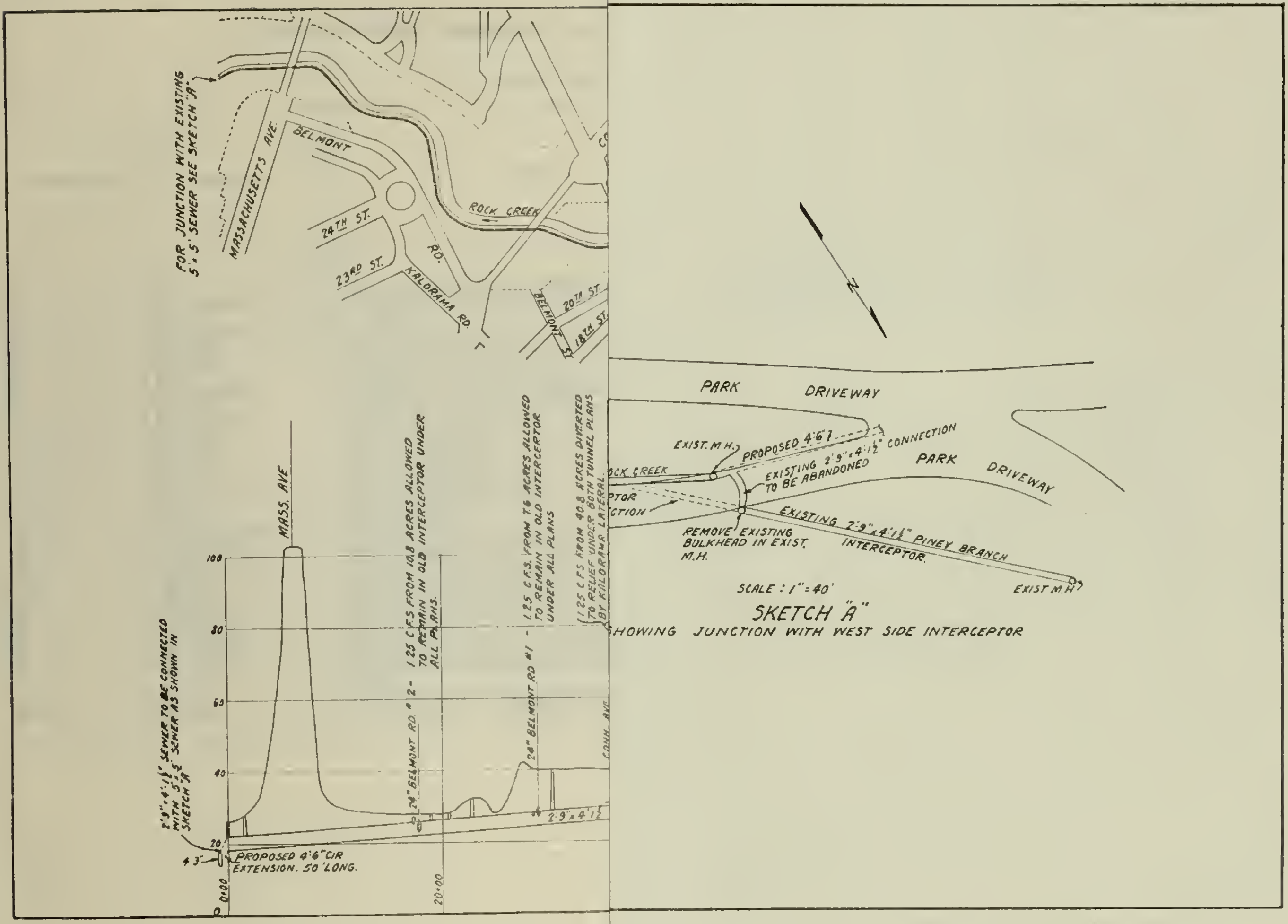
Very nice egg-shaped sewer; smooth brick walls and concrete arch; growths sticking to brickwork below spring line prevent calling it "n 0.011." Curves are rounded angles, too abrupt to be called good. Average $n=0.012$.

At dam below Massachusetts Avenue.—Entire egg-shaped sewer built of brick. Not smooth masonry but apparently stronger than section above.

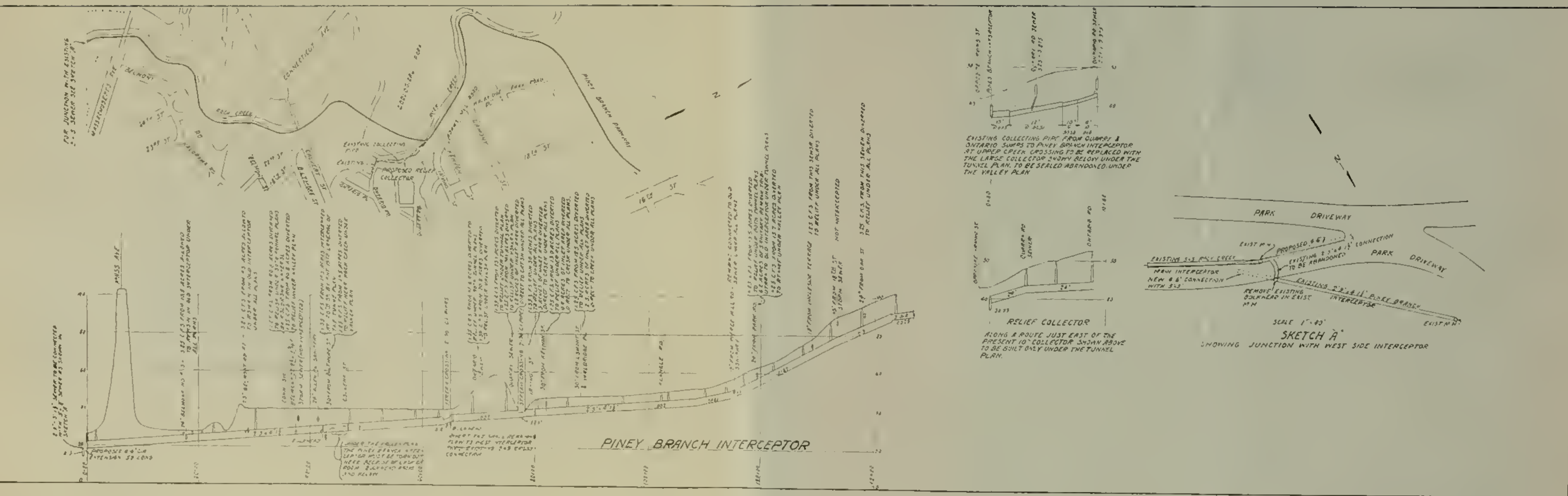
Abrupt angle instead of curve about 20 feet above manhole. $n=0.015$ upstream and 0.014 down.


200
180
160
140
120
100
80
60
40
20
0
0+00
27TH & G
I ST.
K ST.
L ST.
PENNA. AVE.
M. ST.
N ST.
12" CIRC.
24" CIRC. RELIEF
18" CIRC.-PENNA. AVE. SUMP
HYDRAULIC GRADIENT TUNNEL PLAN
6.0
10.70
10.23
12.92
12.77
.002
6'0"
SLOPE OF HYDRAULIC GRADIENT TUNNEL PLAN
.0038
.0017
.00125
HYDRAULIC GRADIENT VALLEY PLAN
.0021
.0012
.00083
0+00


ROCK CREEK MAIN INTERCEPTOR
SHOWING THE HYDRAULIC GRADIENT ELEVATIONS WHICH EXIST UNDER THE TUNNEL PLAN.
THE GRADIENT UNDER THE VALLEY PLAN IS LOWER BECAUSE OF THE GREATER DIVERSION POSSIBLE UNDER THIS PLAN THESE SLOPES ARE SHOWN BELOW WITHOUT CORRESPONDING GRADIENT ELEVATIONS.
CONSTRICTED SECTION UNDER MASS AVE FILL TO BE REBUILT
TO BE REPLACED BY A NEW CREEK CROSSING SEE SKETCH PLAN & PROFILE FIG
THE H G SHOWN IS THAT OF THE NEW WORK
THIS SECTION TO BE REBUILT OR RELIEVED AT SUCH FUTURE TIME AS MAY BE REQUIRED BY INCREASED FLOW – ESTIMATED TO OCCUR IN THE YEAR 1950
HYDRAULIC GRADIENT TUNNEL PLAN
SLOPE OF HYDRAULIC GRADIENT TUNNEL PLAN
HYDRAULIC GRADIENT VALLEY PLAN
MASSACHUSETTS AVE.
DISTRICT LINE



18368—35. (Face p. 68.) No. 2



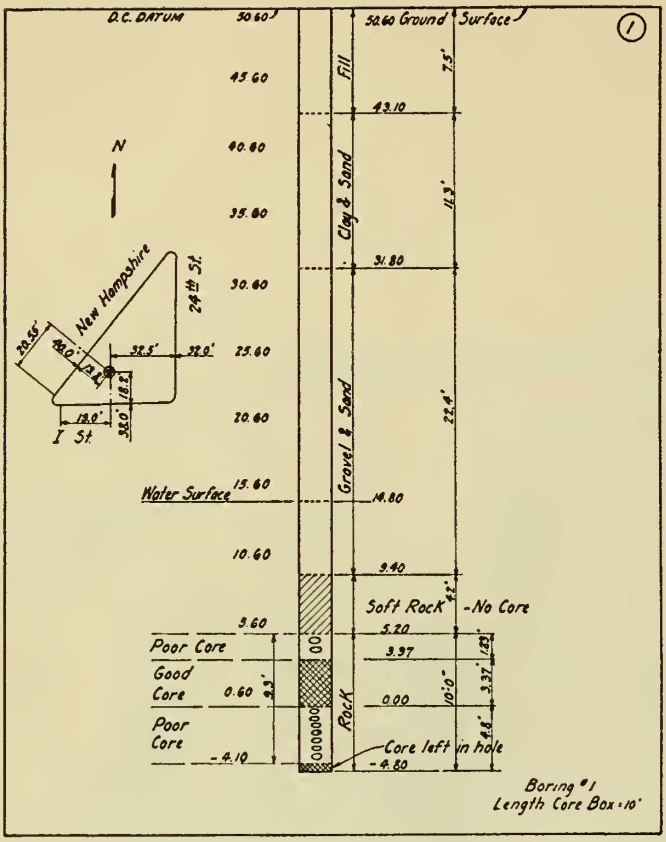
PINEY BRANCH INTERCEPTOR
PINEY BRANCH PARKWAY
EXISTING COLLECTING PIPE FROM QUARRY & ONTARIO SUMPS TO PINEY BRANCH INTERCEPTOR AT UPPER CREEK CROSSING TO BE REPLACED WITH THE LARGE COLLECTOR SHOWN BELOW UNDER THE TUNNEL PLAN. TO BE SEALED ABANDONED UNDER THE VALLEY PLAN
RELIEF COLLECTOR
ALONG A ROUTE JUST EAST OF THE PRESENT 10" COLLECTOR SHOWN ABOVE TO BE BUILT ONLY UNDER THE TUNNEL PLAN.
PARK DRIVEWAY
PROPOSED
NEW INTERCEPTOR
REMOVE EXISTING BULKHEAD IN EXIST M H
SKETCH "A"
SHOWING JUNCTION WITH WEST SIDE INTERCEPTOR

Appendix D

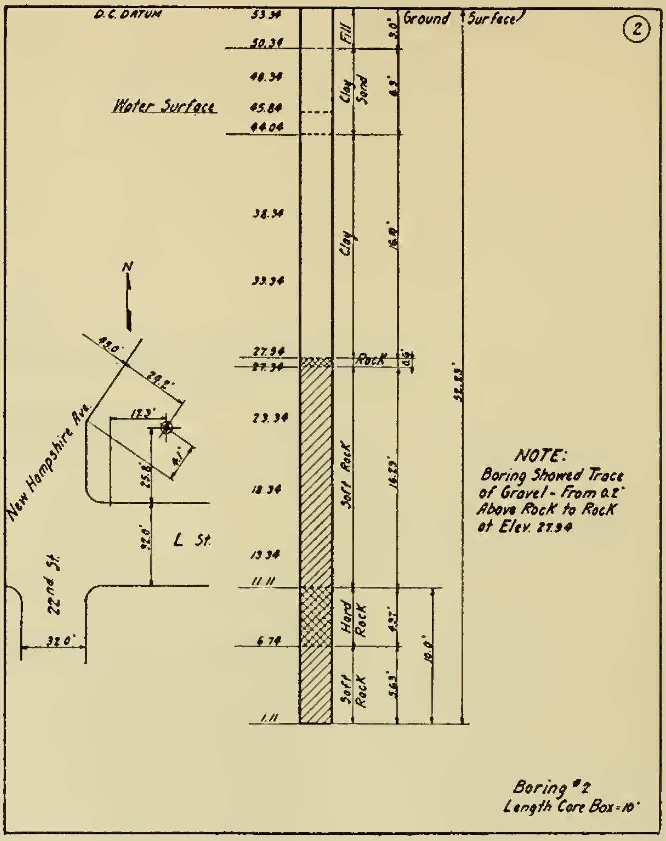
LOG OF CORE DRILLING AND SEISMOGRAPHIC INVESTIGATION

Log of seismographic soundings

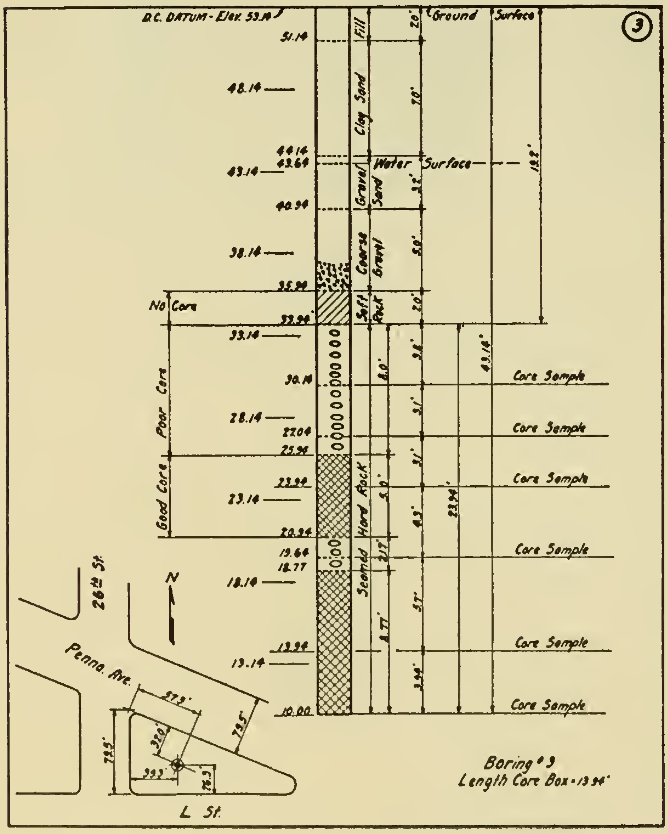
Location no.	Record no.	Location	Elevation of ground, District of Columbia datum	Depth of	Approximate elevation of rock
			Feet		
1	219	Triangle at 26th and Pennsylvania	52	20 feet	32 feet.
2	220, 221, 222	West side 26th, between M and Pennsylvania	48	32 feet	16 feet.
3	223	do	35	30 feet	15 feet.
4	224, 225, 226	Francis School playground	68	62 feet	6 feet.
5	227, 228	West bank Rock Creek in loop below P Street	90	20 feet	
6	229, 230	East end P Street Bridge	60	31 feet	29 feet.
7	231	Triangle at 22d and Massachusetts	73	36 feet	37 feet.
8	232, 233, 234, 237	Florida and Connecticut Avenues	125	26 feet	99 feet.
9	235, 236	Bridle path north of Francis School	29	36 feet	7 feet.
10	238, 239, 240	18th south of Piney Branch	171	No rock at 75 feet	None at 96 feet.
11	241, 242	Piney Branch valley at 18th	86	10 to 14 feet	72 to 76 feet.
12	268, 274	24th and Tracy Place	135	No rock to 40 feet	None at 95 feet.
13	269, 278	24th and Kalorama	145	28 feet	117 feet.
14	270, 271, 277	East side Kalorama Circle	124	No rock to 80 feet	None at 44 feet.
15	275, 276	North side Kalorama Circle	99	31 feet	68 feet.
16	279	20th and Allen	134	No rock to 60 feet	None at 74 feet.
17	283, 84, 85, 313	20th and Calvert	143	29 feet	114 feet.
18	286, 287, 314	Clydesdale and Adams Mill	154	32 feet	122 feet.
19	288, 289, 323	Zoo at Harvard	49	13 to 20 feet	29 to 36 feet.
20	309, 310	Adams Mill, 250 feet north of Kenyon	107	39 feet	68 feet.
21	299, 300	Adams Mill and Lamont	116	No rock to 50 feet	None at 66 feet.
22	312, 315	East side Adams Mill, 420 feet north of Kenyon	122	19 feet	103 feet.
23	290, 291	Clydesdale and Ontario	154	23 feet	131 feet.
24	301, 302, 322	Park Road, 400 feet north of Pierce Mill Road	123	No rock to 45 feet	None at 78 feet.
25	303, 304, 311	Piney Branch Park, east of Park Road	102	24 feet	78 feet.
26	316, 317	West bank Rock Creek, 200 feet above Cleveland sewer.	28	6 to 10 feet	18 to 22 feet.
27	318, 319	Rock Creek below Massachusetts dam	17	15 to 19 feet	−2 to 2 feet.
28	320, 321	West side tunnel, 150 feet west of Q	69	No rock at 70 feet	None at −1 foot.



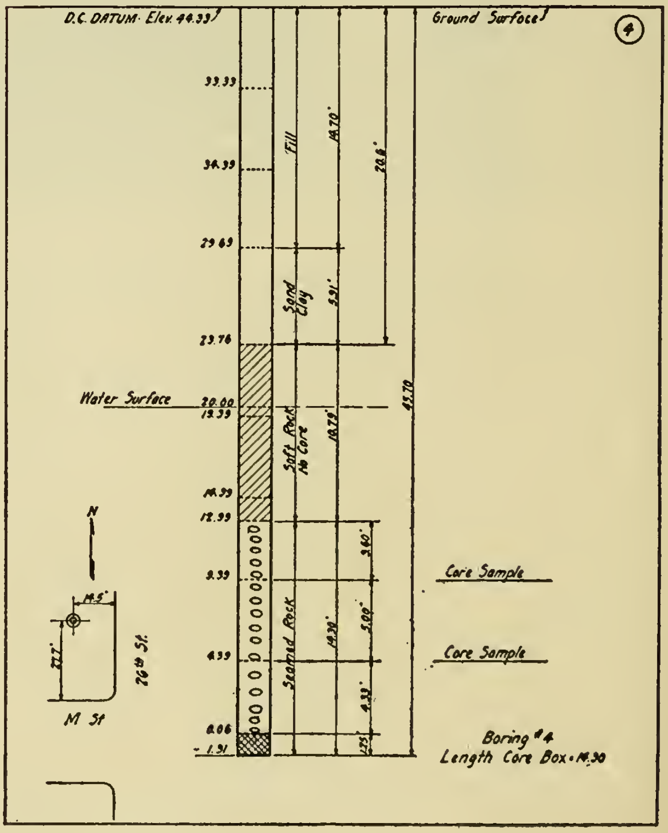
D.C. DATUM
50.60 Ground Surface
Fill
7.5'
43.10
Clay & Sand
11.3'
31.80
New Hampshire
24th St.
I St.
Gravel & Sand
22.4'
Water Surface 15.60
14.80
9.40
Soft Rock -No Core
Poor Core
Good Core
Poor Core
Rock
Core left in hole
-4.80
Boring #1
Length Core Box = 10'



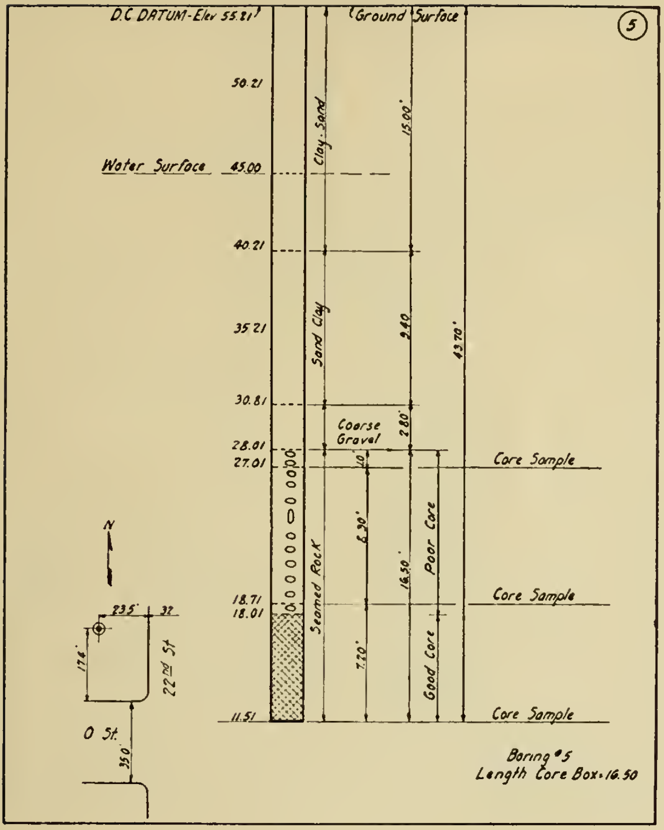
D.C. DATUM
Ground Surface
Fill
Water Surface
Clay Sand
Clay
New Hampshire Ave.
L St.
22nd St.
Rock
Soft Rock
Hard Rock
Soft Rock
NOTE:
Boring Showed Trace of Gravel - From 0.2' Above Rock to Rock at Elev. 27.94
Boring #2
Length Core Box = 10'

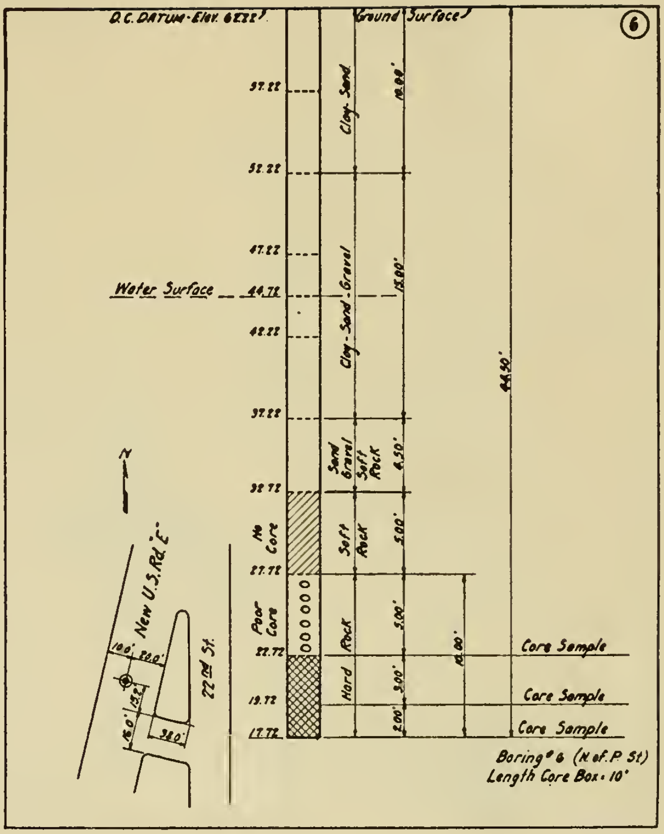


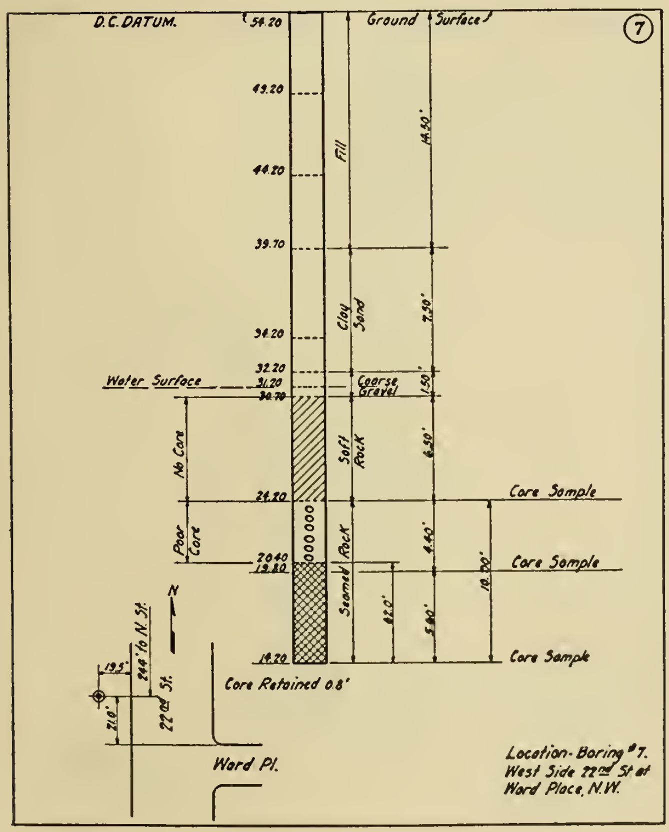
D.C. DATUM - Elev. 53.14
Ground Surface
Fill
Clay Sand
Water Surface
Gravel Sand
Coarse Gravel
Soft Rock
No Core
Poor Core
Good Core
Hard Rock
Seamed Rock
Core Sample
Core Sample
Core Sample
Core Sample
Core Sample
Core Sample
26th St.
Penna. Ave.
L St.
Boring #3
Length Core Box = 13.94'

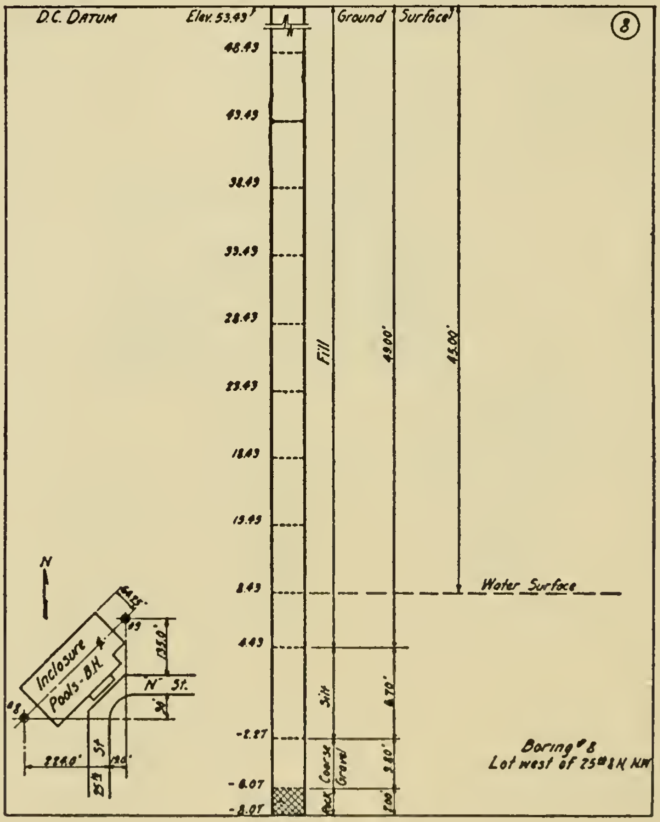


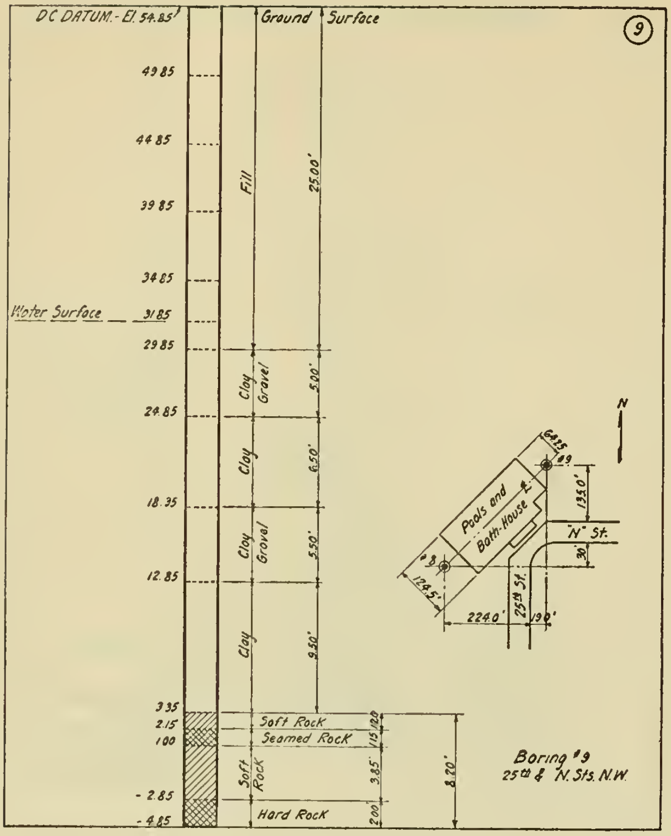
D.C. DATUM Elev. 44.33
Ground Surface
Fill
Sand Clay
Water Surface
Soft Rock No Core
Seamed Rock
Core Sample
Core Sample
26th St.
M St.
Boring #4
Length Core Box = 14.30

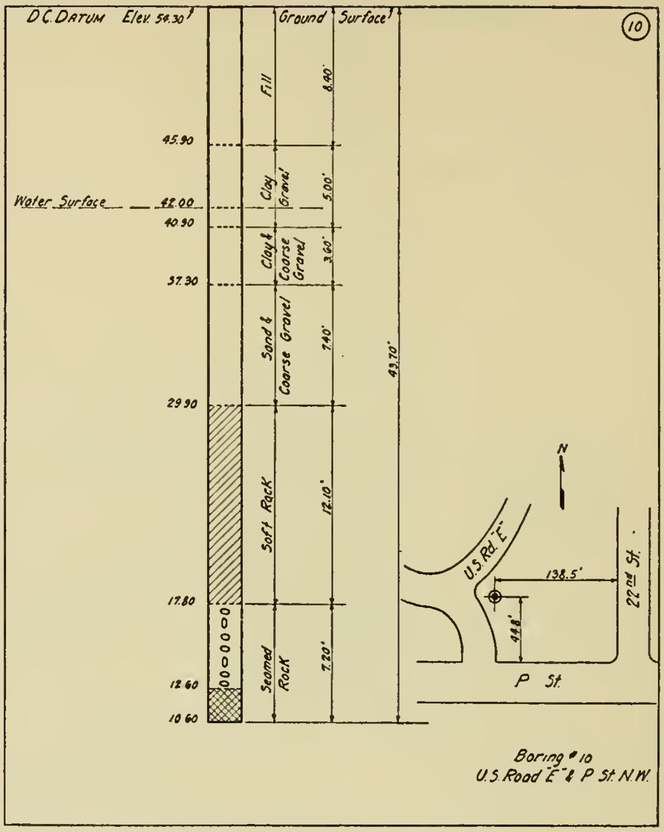

D.C. DATUM-Elev 55.21
Ground Surface
Water Surface 45.00
Clay-Sand
Sand Clay
Coarse Gravel
Seamed Rock
Poor Core
Good Core
Core Sample
Core Sample
Core Sample
O St.
22nd St
Boring #5
Length Core Box = 16.50

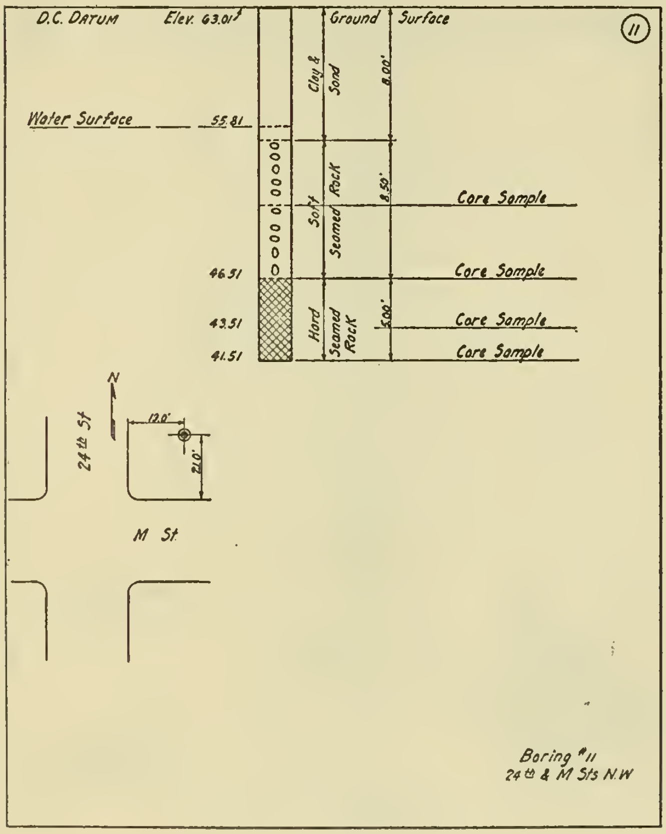

D.C. DATUM-Elev. 62.22
Ground Surface
Clay-Sand
Water Surface
Clay-Sand-Gravel
Sand Gravel Soft Rock
No Core
Soft Rock
Poor Core
Rock
Hard
Core Sample
Core Sample
Core Sample
New U.S. Rd "E"
22nd St.
Boring #6 (N of P St)
Length Core Box = 10'

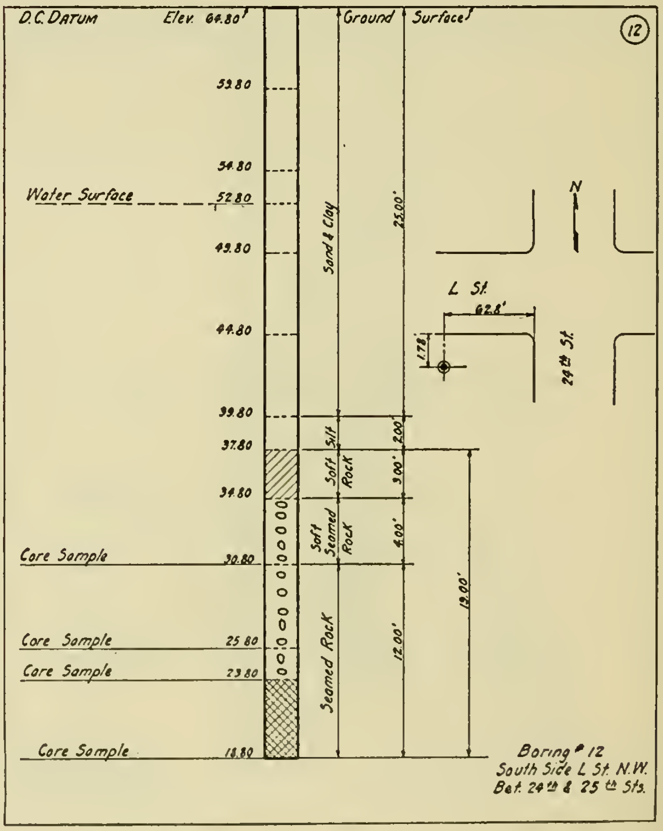

D.C. DATUM.
Ground Surface
Fill
Clay Sand
Water Surface
Coarse Gravel
No Core
Soft Rock
Poor Core
Seamed Rock
Core Sample
Core Sample
Core Sample
Core Retained 0.8'
22nd St.
Ward Pl.
Location-Boring #7.
West Side 22nd St at
Ward Place, N.W.

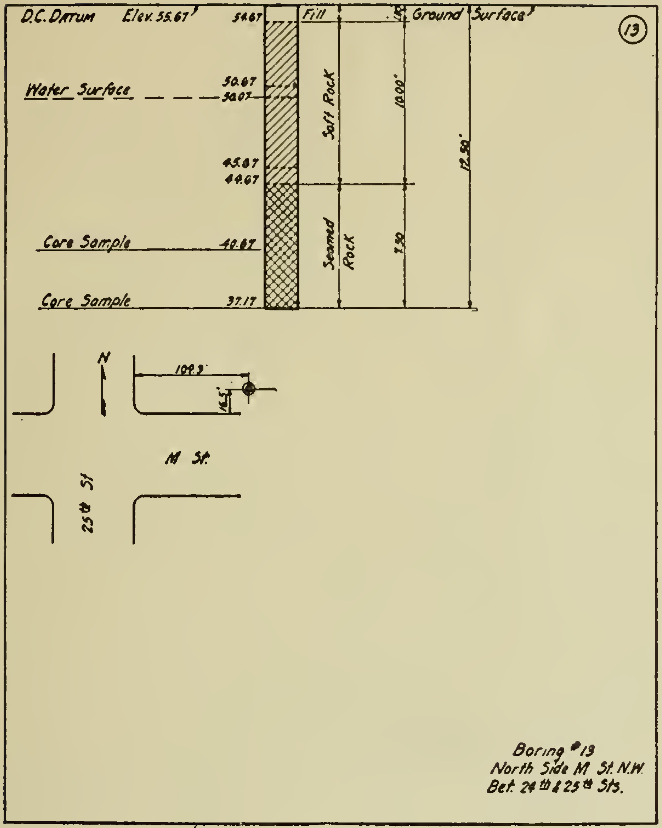

D.C. DATUM
Elev. 53.43
Ground Surface
Fill
Water Surface
Silt
Coarse Gravel
Rock
Inclosure
Pools-B.H.
N St.
Boring #8
Lot west of 25th & N N.W.

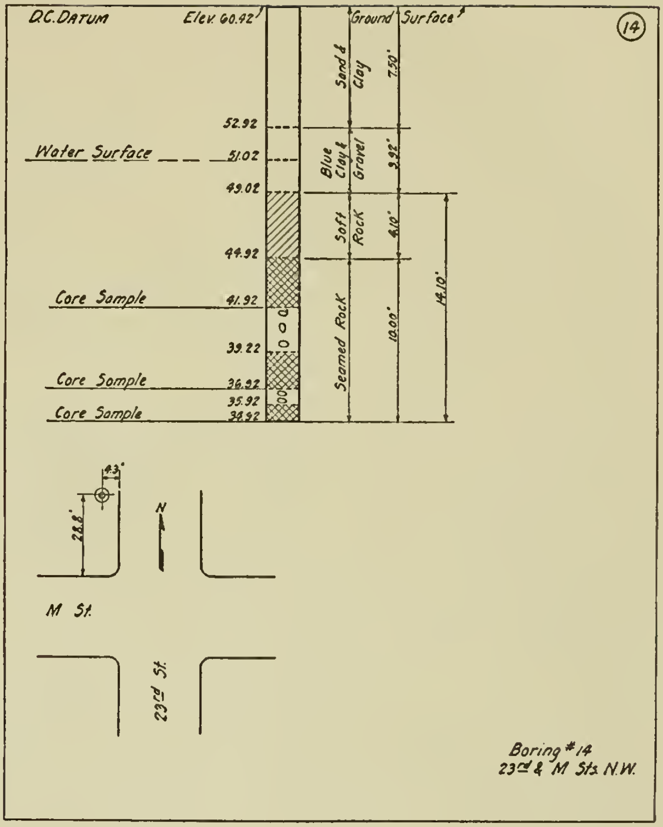

D.C. DATUM.-El. 54.85'
Ground Surface
Water Surface 31.85
Fill
Clay Gravel
Clay
Soft Rock
Seamed Rock
Hard Rock
Pools and Bath-House
"N" St.
25th St.
Boring #9
25th & N. Sts. N.W.

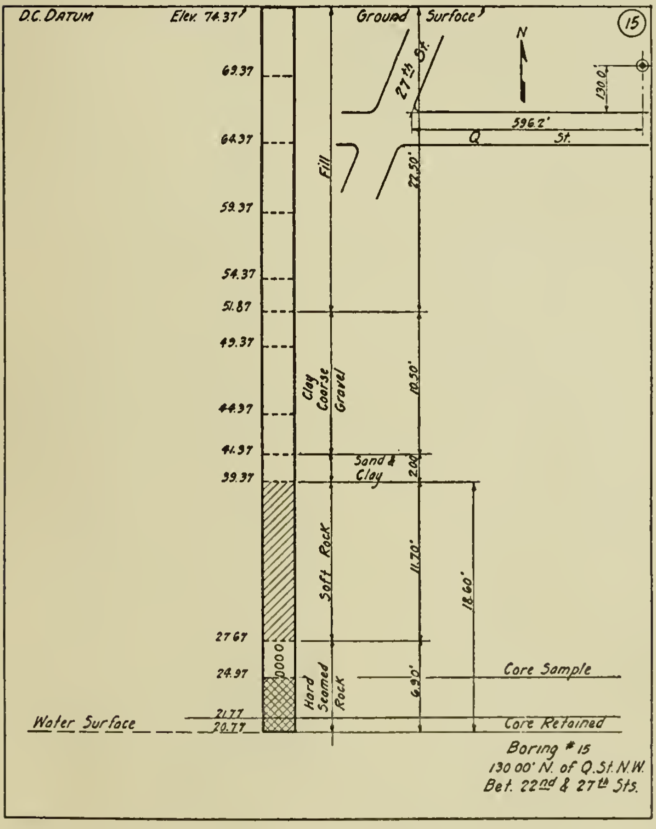

D.C. DATUM Elev. 54.30'
Ground Surface
Water Surface 42.00
Fill
Clay Gravel
Clay & Coarse Gravel
Sand & Coarse Gravel
Soft Rock
Seamed Rock
U.S. Rd. "E"
22nd St.
P St.
Boring #10
U.S. Road "E" & P St. N.W.

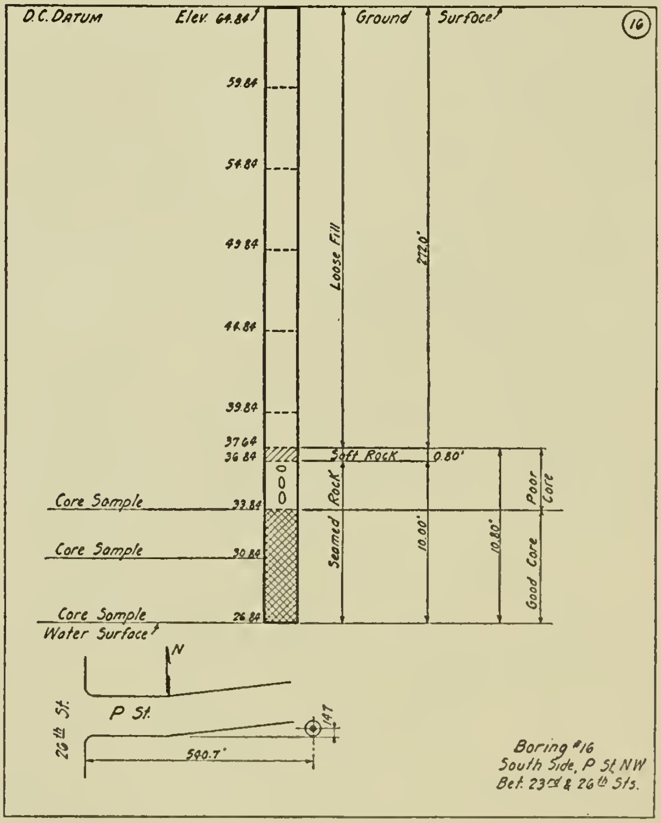

D.C. DATUM Elev. 63.01'
Ground Surface
Water Surface 55.81
Clay & Sand
Soft Rock
Seamed
Hard Seamed Rock
Core Sample
24th St.
M St.
Boring #11
24th & M Sts. N.W.

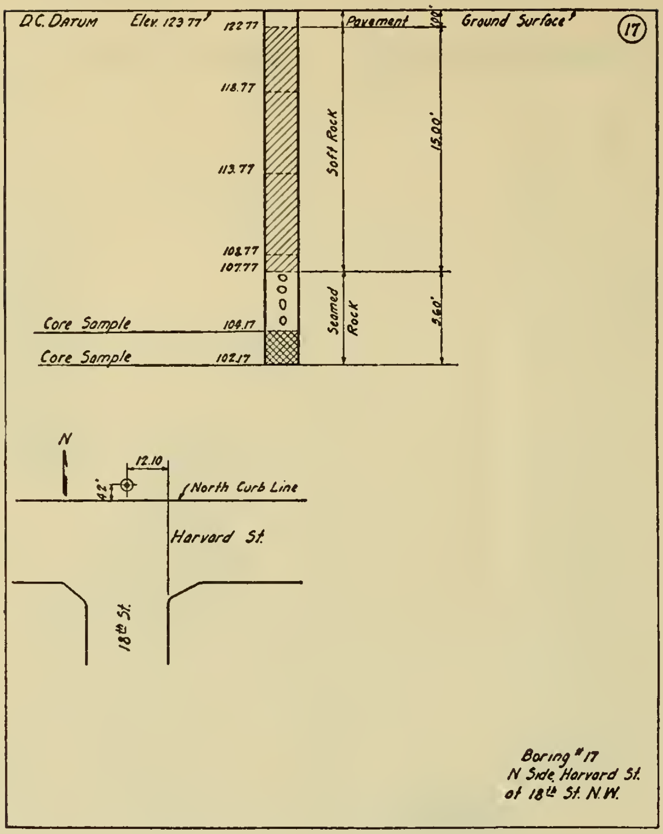

D.C. DATUM Elev. 64.80'
Ground Surface
Water Surface 52.80
Sand & Clay
Silt
Soft Rock
Soft Seamed Rock
Seamed Rock
Core Sample
L St.
24th St.
Boring #12
South Side L St. N.W.
Bet. 24th & 25th Sts.


D.C. DATUM Elev. 55.67'
54.67
Fill
Ground Surface
13
Water Surface
50.67
50.07
Soft Rock
10.00'
12.50'
45.87
44.67
Core Sample
40.67
Seamed Rock
7.50
Core Sample
37.17
N
109.9'
16.5'
M St.
25th St.
Boring #13
North Side M St. N.W.
Bet. 24th & 25th Sts.


D.C. DATUM Elev. 60.92'
Ground Surface
14
Sand & Clay
7.50'
52.92
Water Surface
51.02
Blue Clay & Gravel
3.92'
49.02
Soft Rock
4.10'
44.92
Core Sample
41.92
Seamed Rock
10.00'
14.10'
39.22
Core Sample
36.92
35.92
Core Sample
34.92
4.3'
28.8'
N
M St.
23rd St.
Boring #14
23rd & M Sts. N.W.


D.C. DATUM Elev. 74.37'
Ground Surface
15
69.37
27th St.
N
130.0'
596.2'
Q St.
64.37
Fill
22.50'
59.37
54.37
51.87
49.37
Clay Coarse Gravel
10.50'
44.37
41.37
Sand & Clay
2.00'
39.37
Soft Rock
11.70'
18.60'
27.67
24.97
Hard Seamed Rock
6.90'
Core Sample
Water Surface
21.77
20.77
Core Retained
Boring #15
130.00' N. of Q St. N.W.
Bet. 22nd & 27th Sts.


D.C. DATUM Elev. 64.84'
Ground Surface
16
59.84
54.84
Loose Fill
27.20'
49.84
44.84
39.84
37.64
36.84
Soft Rock
0.80'
Core Sample
33.84
Seamed Rock
Poor Core
10.00'
10.80'
Core Sample
30.84
Good Core
Core Sample
26.84
Water Surface
N
P St.
26th St.
14.7'
540.7'
Boring #16
South Side, P St. NW
Bet. 23rd & 26th Sts.


D.C. DATUM
Elev. 123.77'
122.77
118.77
113.77
108.77
107.77
104.17
102.17
Core Sample
Core Sample
Pavement
Ground Surface
17
Soft Rock
15.00'
Seamed Rock
5.60'
N
12.10
North Curb Line
Harvard St.
18th St.
Boring #17
N Side Harvard St.
at 18th St. N.W.