COMPREHENSIVE SANITARY SURVEY OF WOONSOCKET, RHODE ISLAND

By John S. Bethel, Jr.*, Member

As Mr. Hammann has stated in the first portion of this combined presentation, most of the industries in Woonsocket are related to textiles in some form. At present Woonsocket is undergoing a period of industrial transformation similar to that being experienced in many other northern textile centers, and it is not possible to predict the type of industries that Woonsocket will support in the future. However, the trend throughout the New England area appears to be away from soft-goods industries which require large volumes of water and produce large quantities of wastes, to hard-goods industries requiring little water and producing small volumes of wastes. This transformation was important in our study of water and sewerage problems.

Population Studies

The prediction of future populations of a city is generally based upon statistical methods. These methods are usually successful if there are within the City's limits large areas for industrial expansion and housing development; if the City has shown an increase in population over the years; and if the growth of the city can be compared to the growth of larger cities having similar industries and occupations. However, the problem of predicting the future population of Woonsocket could not logically be solved by usual methods for the following reasons: the areas available for future industrial and housing expansion are limited; the population of Woonsocket has been static for nearly 30 years at 50,000 persons, with only minor fluctuations during the depression of the 1930s and later during the years of industrial activities stimulated by World War II; and finally, no historical data are available for estimating future growth based upon comparison with other New England cities which have lost a large number of their industries and are in the process of attracting new ones. Because of the inapplicability of these conventional methods,

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it was necessary to conduct a detailed study of the present density of population of the various areas of Woonsocket and to estimate the ultimate population of each of these areas. A visual survey of the City was made in which built-up and vacant areas were noted. This information was checked against aerial photographs. Park, industrial, and similar areas unavailable for construction of housing were omitted from the density calculations.

Also considered in this study were types of housing. In the early part of the present century, the multiple dwelling unit was the more common form of housing in Woonsocket. However, after the first World War, and even more markedly after the second World War, the trend has been to single dwelling units. In those unpopulated portions of the City where the available land exceeded 100 vacant acres per precinct, it was assumed that the future density would be approximately 8.4 persons per acre. This density was based on the assumption that a new small house might occupy approximately 10,000 sq. ft. of land and provide housing for 3.6 persons. Where there were less than 100 vacant acres available in a precinct, it was assumed that the future development would approximate the present density. On these bases, it was estimated that the ultimate population of Woonsocket might be approximately 70,200 persons.

The area density method of determining future population does not give one important factor required for engineering design, namely, the year in which the future population will be realized With the apparent static population of Woonsocket, there is little reason to believe that under present conditions there will be any rapid growth in the next 10 years. Under no conditions, would it appear reasonable to assume an increase of 1,000 persons per year, which rate is equal to the present natural increase in population. It was, therefore, estimated that Woonsocket would not attain a population of 70,200 persons prior to the year 2000, or 45 years hence, and barring additional industrial expansion might never reach this population.

Taking into consideration the various factors previously discussed, it was also estimated that the population of Woonsocket might be approximately 55,000 by the year 1965, an increase of about 5,000 over the present population. In general, therefore, this population was used for the design of those works which could be expanded readily, whereas the ultimate population was used in the design of those works which would be difficult or costly to expand.

Water Consumption Supply and Treatment

By far the most pressing problem facing the City is that of water supply. This supply, barely adequate for many years, is now critically inadequate. Previous studies made in 1925 and 1943 stressed the inadequacy of the supply and in the earlier report, a recommendation of additional sources of surface water which might be available to the City was presented. Unfortunately, no action was taken on these reports and the problem today remains unchanged, except that at present the need for an augmented supply is infinitely more acute than it was in 1925. The seriousness of the problem was demonstrated in 1930 and again in 1953 when the reservoir supplying water to the City was nearly depleted.

In addition to the problem of development of an additional water supply, Woonsocket is faced with a second problem relative to the quality of the supply. The color, odor, taste, and iron content of the water do not meet the U. S. Public Health Service standards recommended for drinking water. Moreover, the pollution of the watershed of the present supply is such that adequate treatment of the water should be undertaken to protect the health of the people of the City.

Quantity of Water. Before undertaking any study of additional water supplies available for development, it was necessary to estimate the future quantities of water needed. A study of the past consumption of water, both domestic and industrial, was therefore made.

Past records indicate that during the years 1925 to 1953 water consumption increased from approximately 3 mgd. to 5.3 mgd., a rise of nearly 80 per cent. However, as the population during that period remained substantially constant, the increase in consumption was the result of an increase in per-capita consumption from 52 gcd. to 80 gcd. In our opinion, this is the result of a generally improved standard living as well as the trend from apartment-type housing to single family dwellings. It was estimated, therefore, that the domestic consumption will continue to increase to 90 gcd. by 1965 and to 100 gcd. by the year 2000.

Concurrent with this study of domestic water consumption an investigation of the water needs of all industries in Woonsocket was also conducted. The industrial consumption was divided into two categories, namely, the amount required for "normal production," and that required for "capacity production." We estimated that in 1965 the industrial consumption would be equal to that required for the present "normal production" and that by the year 2000 it would be equal to the present "capacity production."

On the basis of these assumptions, we estimated that the total water requirements of the City will be 6.5 mgd. in 1965, provided industrial demands remain at their present level; and will be 9.5 mgd. when the population of Woonsocket has reached 70,200, providing industrial demands at that time equal the demand of present industries operating at capacity levels. Further details of water consumption are given in Table 1. The average daily consumption was used

TABLE 1
WATER CONSUMPTION

Item	Year			
	1954	*1960*	*1965*	*2000*
Population of Woonsocket	50,200	52,600	55,000	70,200
Gallons per capita per day	80	85	90	100
Industrial Production	Normal	Normal	Normal	Capacity
Water demand in mgd.				
Woonsocket—domestic	4.03	4.47	4.95	7.02
Woonsocket—industrial	1.04	1.15	1.29	2.05
Villages outside City limits	0.26	0.28	0.30	0.40
Total average consumption	5.33	5.90	6.54	9.47
Max. 4-month demand	6.2	6.9	7.6	11.0
Max. daily demand	8.6	9.5	10.5	15.2

to determine the quantity of additional water required; whereas the maximum daily consumption was used to determine the capacity of the proposed water treatment plant and the necessary additions to the major pipes of the distribution network.

Crookfall Brook Supply. The present water supply for the city of Woonsocket is obtained from Crookfall Brook, a tributary of the Blackstone River. Crookfall Brook has a watershed area of 7.9 sq. mi. and is located just south of the City in the towns of North Smithfield, Smithfield, and Lincoln. The watershed area is wooded and sparsely settled, except along the Louisquissett Pike which has been lightly developed in recent years. There is some swamp area on the upper reaches of the watershed, but on the lower portion there are large areas of swamp along the course of the brook.

When the water supply system was taken over by the City from the Woonsocket Water Company in 1885, there were two small reservoirs on the lower part of the watershed, namely, Reservoir No. 1, a small reservoir of 30 mil. gal. which is still used as a storage reservoir for the present pumping station; and Reservoir No. 2, a short distance upstream, which was destroyed and was never rebuilt. In 1895 a third and major reservoir, Reservoir No. 3, was constructed approximately 2½ miles upstream from Reservoir No. 1. Fig. 1 shows the general interrelation of these reservoirs.

Reservoir No. 3 has a safe available storage of 890 mil. gal. and is contained by an earth-fill dam with a masonry spillway. The capacity curve for this reservoir was computed from a copy of the original contour map drawn at the time of the construction of the reservoir.

During the drier months of the year when the runoff from the watershed of Reservoir No. 1 is insufficient to supply the demands of the City, water is released from Reservoir No. 3 and allowed to flow through 2½ miles of swamp to Reservoir No. 1. During the remainder of the year Reservoir No. 3 is kept full or as nearly full as possible.

The water level in Reservoir No. 1 is normally kept about 2 ft. below the spillway crest so that runoff from heavy rains on the lower watershed can be stored and used.

Safe Yield of Crookfall Brook Watershed. Safe yield is defined as the daily quantity of water which is continuously available from a supply under the lowest rainfall conditions likely to occur. The determination of safe yield is based on three factors; namely, amount of rainfall, runoff resulting from rainfall, and available reservoir storage. Many times these data are not available for a particular watershed, as was true in the case of Woonsocket, where the rate of runoff resulting from rainfall was unknown. Under these conditions, the rainfall and unit runoff from a similar nearby watershed are commonly used. In our studies, data from the Abbott Run watershed on the Pawtucket water supply, located approximately 10 miles from Crookfall Brook was used. Rainfall and runoff records at Abbott Run have been kept continuously since 1907.

Rainfall data were, however, recorded continuously at Reservoir No. 1 from 1886 to 1941. Since that time, rainfall data have been recorded daily at the Woonsocket Sewage Treatment Plant, located about 2 miles north of Reservoir No. 1.

A study of these data indicated that the average rainfall at Reservoir No. 1 exceeds the rainfall on Abbott Run by about 6 per cent. The runoff characteristics of the two watersheds are compara-

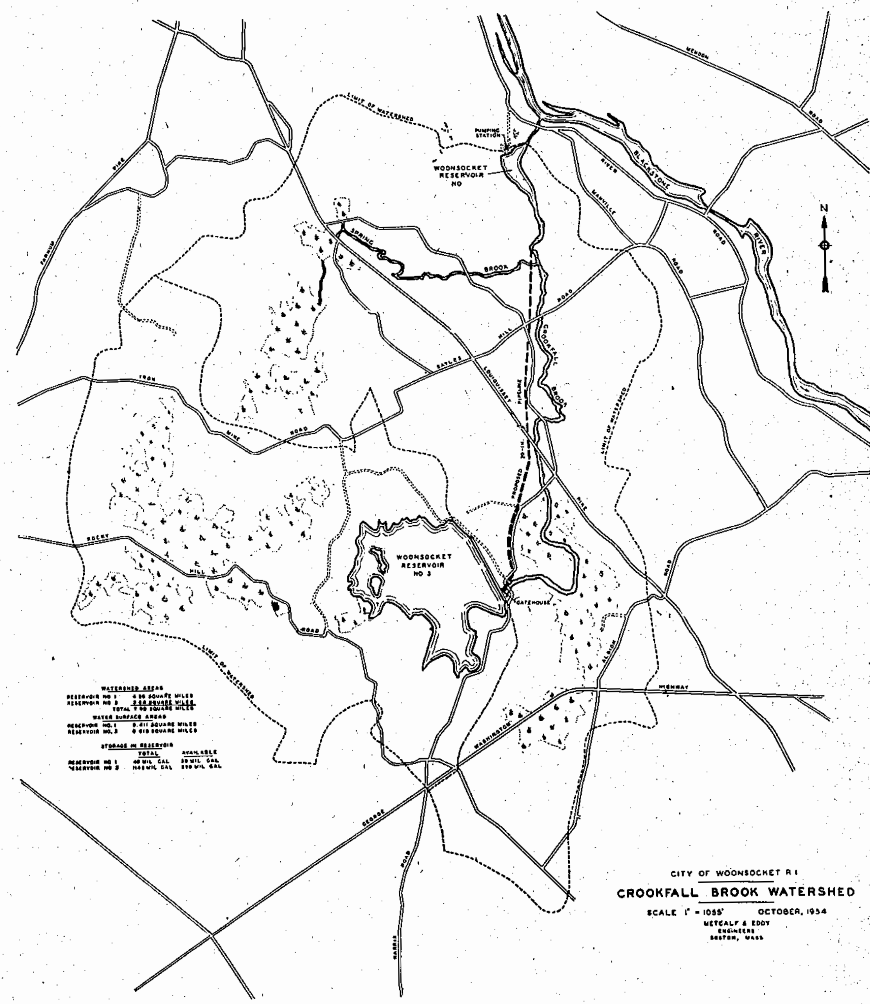

FIG. 1.

ble and it was therefore concluded that the runoff of the Crookfall Brook watershed should be at least equal to that of the Abbott Run watershed. In addition, it was concluded from a study of rainfall frequency at Reservoir No. 1 that an annual rainfall of 32.67 in., the 1930 average rainfall and the lowest annual rainfall at Reservoir No. 1 on record, can be expected only twice in every 100 years.

Utilizing all of these data and diagrams of safe yield prepared by the New England Water Works Association, it was estimated that the safe yield of the Crookfall Brook watershed was 4.5 mgd.

However, in 1930 when the average daily consumption was 3.3 mgd., the water level in Reservoir No. 3 dropped to a dangerously low point. During the preceding winter, the rainfall was below average and Reservoir No. 3 was not full at the beginning of the dry season. During the spring and summer of 1930 the rainfall continued below normal and the quantity of water in storage continued to drop to a dangerous level, although the supply was not entirely depleted. Because of the apparent discrepancy between the safe yield as computed, and the apparent safe yield, as demonstrated by the conditions in 1930, a study was made to justify or discredit our method of computing safe yield. As a result, it was concluded that approximately 1 mgd. of water was lost during the passage of the water through the 2½ miles of brook and swamps from Reservoir No. 3 to Reservoir No. 1. This loss, the result of evaporation, transpiration, and percolation into the swamps along the brook, was demonstrated by comparing the actual water in storage at Reservoir No. 3 during the summer of 1953 with the computed quantity in storage based on actual water consumption, computed runoff, and reservoir losses.

Our first recommendation, therefore, relative to improvement of the Crookfall Brook supply was the installation of a concrete pipeline leading from Reservoir No. 3 to Reservoir No. 1 to eliminate this loss. In addition, installations for measuring the quantity of water discharged from Reservoir No. 3, for recording the water level in that reservoir, and for remote control of the reservoir valves were also recommended.

Groundwater Investigations. As indicated before in Table 1, the future demand is estimated to be 9.5 mgd. The deficit in water supply is, therefore, the difference between this demand and the safe yield of the Crookfall Brook supply or 5.0 mgd. Studies were made to determine the most economical method of increasing the water supply.

An investigation was first carried out to locate possible sources of groundwater which would be capable of rapid development. A seismic search was made within the City limits and in areas adjacent to the City to determine areas suitable for groundwater supplies. This initial work was followed by drilling 2½-in. test wells in those areas where the results of the seismic tests were favorable and in several other areas which also appeared promising after further geological investigation. If test wells yielded less than 30 gpm., they were abandoned because of their limited capacity.

Only two of the fifteen areas so investigated had a sufficient depth of water-bearing sand-and-gravel deposits to warrant further investigation. In these two areas, 8-in. test wells were installed and full-scale pumping tests were made. The safe yield from these two wells was 550 gpm. and 290 gpm., respectively, or a total safe yield of 1.2 mil. gal. Laboratory tests of the water from one of the wells indicated the presence of 1.6 ppm. of manganese and slight bacterial contamination. No manganese and less bacterial contamination were present in the samples taken from the other well, although it appeared reasonable to expect that under prolonged pumping this well might also show some manganese. Both wells contained in excess of 25 ppm. of free carbon dioxide but neither well showed the presence of iron.

It was recommended that water from both wells be conveyed by a common discharge force main to new aeration facilities at Reservoir No. 1 for removal of free carbon dioxide. The aerated well water and the surface water in Reservoir No. 1 would then be blended, treated in a rapid sand filter plant and pumped to the distribution system. It was anticipated that the quantity of iron and manganese in the raw mixture of surface water and well water would slightly exceed the U. S. Public Health Service standard of 0.3 ppm. However, it was our opinion that the water would be acceptable after chlorination during the short period prior to construction of the proposed water treatment plant.

Harris Pond. A study was made of all surface water sources near Woonsocket to determine if any of these sources were possible for economic development. It was our conclusion that Harris Pond on the Mill River met all requirements. The pond and its water rights are privately owned. It lies mostly in Massachusetts, has a safe yield of approximately 6.5 mgd., and was contained by a dam constructed in 1869. Subsequent to the date of our report the dam was breeched by the flash-flood of August 19, 1955 and requires repairs.

Harris Pond water is similar in analysis to Crookfall Brook water and requires only treatment before it can be used by the City. It was, therefore, our recommendation that the water rights and property upon which Harris Pond stood be purchased by the city of Woonsocket.

It was also recommended that a pumping station be installed at Harris Pond and that the pond water be pumped to the same pipeline which carried well water to Reservoir No. 1. This plan, which was also the most economical plan, will allow all water to be treated at a single plant and will ensure that all parts of the system receive water of uniform quality.

Other Surface Water Supplies Investigated. A recommendation made in older reports for a water supply on Tarkiln Brook, southwest of Woonsocket, was reinvestigated. It was determined that this supply would not be sufficient in itself to supply all of Woonsocket's needs, and that it would be necessary to keep the Crookfall Brook supply as an auxiliary supply. The development of the Tarkiln Brook supply would be far more complicated and costly than the development of the Harris Pond supply because, in addition to the facilities required at Harris Pond, it would be necessary to construct and operate treatment plants at both Tarkiln Brook and Reservoir No. 1. Also, the treated water had to be pumped a distance of over 4 miles to reach the northern edge of the City's water distribution system, and about 3 miles of a major roadway passing through the site of the Tarkiln Brook Reservoir would have to be relocated. Consequently, it was determined that development of Tarkiln Brook was not as favorable economically as was development of Harris Pond.

A favorable site on the Mill River above Harris Pond was also investigated for development of a supply. Although the river at the point of investigation could be developed to provide a safe yield of about 14 mgd., the supply would lie wholly within Massachusetts. Its development, therefore, would require legislative action by both Rhode Island and Massachusetts. In addition, there was the problem of compensation for diversion of water involving both the owner of Harris Pond and the owners of the mills lower down on the Blackstone River and the problem of a long pipeline to reach this site. As in the case of Tarkiln Brook, it was our opinion that it was far less expensive to acquire the water rights and develop the Harris Pond supply than to construct a new reservoir on the Mill River in Massachusetts.

Water Treatment Plant. The physical and chemical qualities of Crookfall Brook water are unsatisfactory, exhibiting excessive color, tastes, odors, and iron. These deficiencies were recognized as far back as 1895 when Reservoir No. 3 was constructed. As treatment of water based on scientific principles had only recently been introduced, it was decided to postpone construction until filtration principles had been more fully investigated. Unfortunately, no works were ever constructed although chlorination of the water was initiated in February 1921.

In addition to the Crookfall Brook water, the water from Harris Pond will also require treatment because of the similarity of the two waters and the well water will require treatment to remove excessive manganese and free carbon dioxide. It was therefore decided to aerate the Harris Pond and well supplies at Reservoir No. 1 and to blend the aerated mixture with Crookfall Brook water before treatment. Coagulation tests indicated that the water was readily treatable with conventional alum and lime treatment. It was

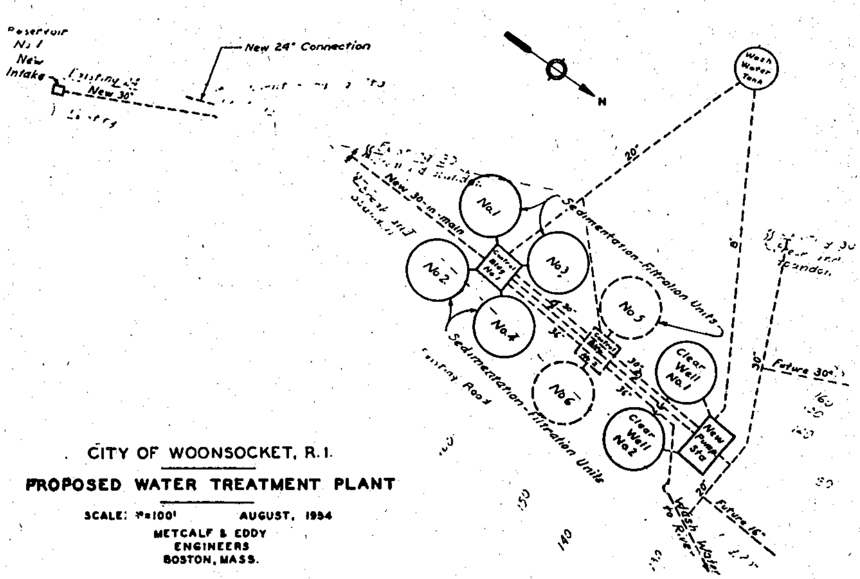

FIG. 2.

recommended that a 10-mgd. treatment plant, with provision for expansion to 15 mgd., be constructed at Reservoir No. 1. The type of plant recommended was of the "Aldrich" type utilizing concentric treatment units. An initial saving of $50,000 per million gallons was indicated in the use of this type plant over a conventional rapid sand filter plant. A layout of the proposed water treatment plant is shown in Fig. 2.

Included in the cost of this new plant was the construction of a new high-service pumping station to replace the existing pumping facilities at Reservoir No. 1.

Summary. A summary of the water supply costs and a recommended program of development is given in Table 2.

TABLE 2
WATER SUPPLY COSTS

Immediate Construction	
1. Development of well supply	$298,500
2. Improvement to Crookfall Brook	176,800
Subtotal	$475,300
Within 5 years	
3. 10-mgd. water treatment plant	1,167,000
4. Development of Harris Pond	242,000
Subtotal	$1,409,000
Future	
5. 5-mgd. water treatment plant addition	306,000
Total	$2,190,300

WATER PUMPING, STORAGE AND DISTRIBUTION

Pumping. The pumping facilities at Reservoir No. 1 consist of two electrically driven pumps installed in 1918 and 1919 and one standby gasoline-engine-driven pump installed in 1946. All pumps are housed in a building constructed in 1895. Tests conducted on the two electric pumps indicated pumping rates of 6.05 and 6.72 mgd., at a head of 223 and 226 ft., respectively. The pump efficiencies were 78.5 and 79.0 per cent, respectively. The gasoline engine pump supplied 8.5 gpm. at a head of 228 ft.

A number of deficiencies exist in the Pumping Station requiring piping changes, replacement of chlorination equipment, installation of new recording instruments, and replacement of "open-front" switchgear and open wiring with modern safe equipment. These changes were recommended if the construction of the new Pumping Station at the Treatment Plant were to be delayed beyond 1956. When the new Pumping Station is constructed, it was further recommended that the three existing pumps be moved to the new station and that the present building be converted to an office and shops for the Water Department to replace their present totally inadequate quarters.

Based upon requirements of the National Board of Fire Underwriters, it was determined that a third electrically driven pump of 8-mgd. capacity is required immediately for fire protection in the event of breakdown of present pumping equipment. In addition, it was determined that the two existing electrically driven pumps should be replaced with two new pumps having a combined capacity of 15.5 mgd. when the maximum daily water consumption exceeds 12 mgd.

Storage. Storage for the water distribution system is provided in five ground storage tanks located on Mt. St. Charles, approximately in the center of the system. Four of these tanks are connected to the low service system and one tank, a standpipe, is connected to the high service area. The total capacity of the four low service tanks is 3.50 mil. gal. and that of the high service tank is 1.55 mil. gal. Water is pumped by a small pumping station located at the tanks from the low to the high service system.

Four of the five storage tanks are of wrought iron or steel construction and are badly in need of painting. The exterior of the fifth tank of concrete construction, has badly disintegrated and requires immediate repairs.

The present high-service Pumping Station at the storage tanks has been in continuous operation for over 20 years. At the present time, the single pump installed in this station is in poor repair and the electrical controls and wiring are obsolete and unsafe. Because the station is too small to house a second pump, it was recommended that a new high service pumping station with duplicate pumps be constructed at the site of the present station. It was also recommended that a positive connection equipped with a relief valve be provided between the high and low service systems. This connection will permit the use of high service water in the low service system during an emergency without danger to the low service piping as a result of excessive pressures.

In the future, it will be necessary to construct additional elevated storage tanks as the need for water in the outlying portions of the City increases. Only one additional tank will be required on the low service system. However, three new high-service booster stations and elevated tanks will be required.

Distribution System. An analyses of the system based on present and future estimated flows, including fire flow, was made on all pipes 12 in. or larger. It was concluded that with several exceptions the principal water mains are of adequate size for present conditions.

Under future conditions, however, a new river crossing near the water treatment plant will be required to supply water to the eastern portions of the City, and in addition, fourteen additional new reinforcing pipelines will be required. A program of these reinforcements was presented to the City.

Also presented was a program of future extensions covering all streets now without City water but requiring water immediately or in the near future.

Summary. A summary of water pumping, storage and distribution costs is given in Table 3.

Stream Pollution and Sewage Treatment

The present Sewage Treatment Plant was built in 1931, for an average sewage flow of approximately 3.3 mgd. Since that time, the flow of sewage and industrial wastes to the plant has nearly doubled. Many items of plant equipment have exceeded their useful life, and it is now impossible for the plant to effectively handle the present load. In order to meet the requirements of the State Department of Health, extensive additions and alterations are required.

Gagings were made and samples collected over a 5-day period of all sewage and industrial wastes reaching the Sewage Treatment Plant. The total volume of sewage and wastes thus gaged was 5.9 mgd., of which 3.0 mgd. was estimated to be infiltration. The per capita contribution of sewage during the gaging period was determined to be 63 gcd.

In addition to sewage and wastes reaching the plant, many industries within the City still discharge their wastes without treatment to the river. If these wastes are to be accepted into the sewerage system, provisions for their treatment must be incorporated in the design of the plant. Consequently, a personal survey of 45 industries was made by our engineers to obtain information regarding the quantity and quality of the industrial wastes. On the basis of the information thus obtained, it was estimated that approximately 0.37 mgd. of industrial wastes are presently being discharged to the sewerage system and that 1.6 mgd. are discharged untreated to the river. If these untreated wastes are intercepted, the total volume of sewage and wastes currently requiring treatment is 7.5 mgd.

TABLE 3
SUMMARY OF WATER PUMPING, STORAGE, AND DISTRIBUTION COSTS

Within 2 years	
Repairs to present pumps	$ 1,700
High-Service Pumping Station	32,000
Repairs and painting storage tanks	58,800
Reinforcements to Distribution System	89,000
High-Service Pressure Reducing Valve	5,000
Repairs to Main Pumping Station	33,000
Subtotal	$219,500
Within 10 years	
Reinforcements to Distribution System	$277,000
Extensions to Distribution System	288,000
Subtotal	$565,000
Future	
Low Service Elevated Tank	$ 60,000
Three high-service districts	564,200
Reinforcements to Distribution System	191,700
Subtotal	$815,900
Total	$1,600,400

Further estimates of the quantity and composition of the sewage and wastes are given in Table 4. The first main heading, namely, "Present" includes only those industrial wastes now discharged to the sewers, whereas the second heading "Design" assumes all industrial wastes have been intercepted.

Stream Survey. To determine the capacity of the Blackstone River to absorb pollution and as a consequence to determine the degree of treatment of the sewage and wastes that may be required, a detailed stream survey was conducted. River sampling stations were established at five points as shown in Fig. 3. Samples were taken at these stations over a 2-day period to determine existing river conditions. Flows in the river during the time of sampling were also determined. As shown in Fig. 4, the critical point with regard to dis-

TABLE 4

QUANTITY AND QUALITY OF SEWAGE AND INDUSTRIAL WASTES

	Present		*Design*	
Population	50,200		55,000	
Population served	40,000		55,000	
Industrial production	Normal	Maximum	Normal	Maximum
Flow, mgd.	5.91	6.09	9.36	10.76
5-day, 20 deg.C., B.O.D., lb./day	7,585	8,285	22,155	29,815
Suspended solids, lb./day	7,680	8,055	31,475	40,775

solved oxygen in the stream occurs at Manville Dam located approximately 2.5 miles below the Sewage Treatment Plant. These data and the time of flow between stations were used to estimate the various river reaeration coefficients.

On the basis of a study of river discharges over the past 25 years

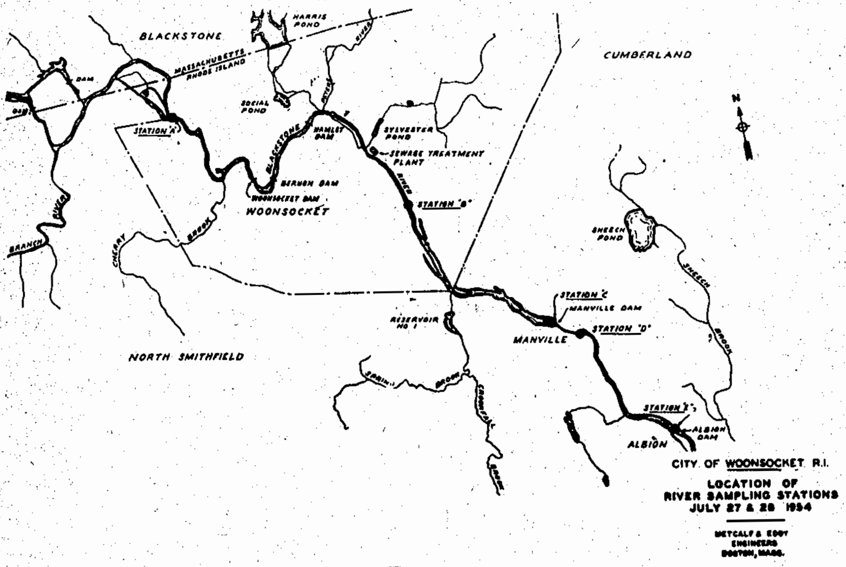

FIG. 3.

of record and the previous data, it was estimated that a satisfactory dissolved oxygen level at Manville Dam can be maintained in the river during the summer months by removal of 85 per cent of the present B.O.D. load, which removal must be increased to 90 per cent by the end of the design period.

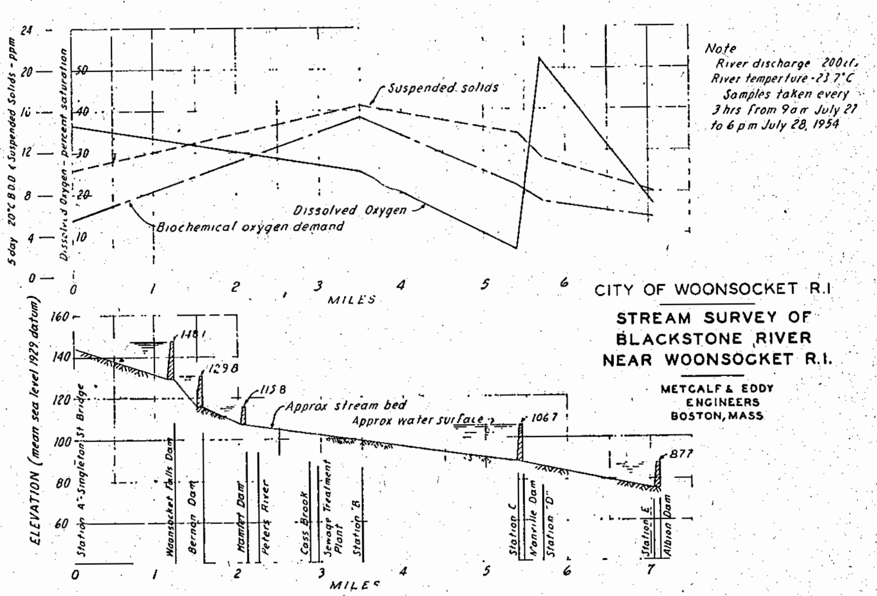

Fig. 4.

Treatment Plant. It was therefore recommended that the present treatment plant be modified to a treatment plant employing the "step-aeration" and "biosorption" processes. These processes, a type of high-rate activated sludge treatment, were selected because only one-half of the aeration tank capacity needed for conventional activated sludge treatment is required; because they are less affected by industrial wastes; and because the sludge is more readily compacted before digestion.

The air requirements of the extremely strong sewage and wastes were estimated to be approximately 2.2 cu. ft. per gallon based on 0.0065 cu. ft. of air per gallon of raw sewage per part per million of 5-day, 20-deg. C. B.O.D. removed in the over-all plant. This quantity of air is also equal to 950 cu. ft. per pound of B.O.D. applied to the tanks, which quantity is comparable to that required by other industrial wastes.

The proposed treatment plant is shown on Fig. 5. Specifically, the additions required to convert the present plant to a 10.8-mgd. plant include a new sewage pumping station, new screening and grit-removal facilities, two new primary settling tanks which will also be used to remove up to 75 per cent of the grease reaching the plant, new sludge pumping facilities, modification and expansion of the existing four aeration tanks, new blower facilities, two new secondary settling tanks, conversion of the existing final settling tanks to chlorine contact tanks, new sludge thickening and digestion facilities, new chlorination facilities, and rehabilitation of the existing sludge drying beds and conversion of several old sand filter beds to sludge lagoons.

The estimated cost of this new plant is $2,966,500. However, construction of the new aeration tanks and of two of the sludge digestion tanks could be deferred until some time in the future. The cost of the plant without these facilities is estimated to be $2,298,500. It was further recommended that this construction be divided into primary treatment facilities constructed within two years at a cost of $1,203,300 and secondary treatment facilities within 5 years at a cost of $1,095,200.

Sewerage System

The sewerage system of Woonsocket is divided into two systems; namely, a "high level" system serving the northeastern portion of the City, and a "low level" system serving the remaining 75 per cent of the City. In general the problems relative to sewerage involved determination of the major sources of infiltration and corrective measures required; modifications or reinforcements to the system required to handle increased flow or to eliminate present overloading; and extensions to the system required to provide service to areas not now served by sewers.

Infiltration. The main trunk sewer of the "low level" system follows the Blackstone River. The sewer, with a maximum diameter of 36 in., is of brick construction and lies for most of its 5,100 ft. length below river level. Inspections indicated that the entire sewer was subject to possible infiltration, particularly in the upper end where heavy infiltration was indicated. It was recommended that the upper portion of the sewer in River Street be cleaned and that a more detailed inspection be made to determine the approximate locations of the excessive infiltration. It was also recommended that, if necessary, this portion of sewer be relaid.

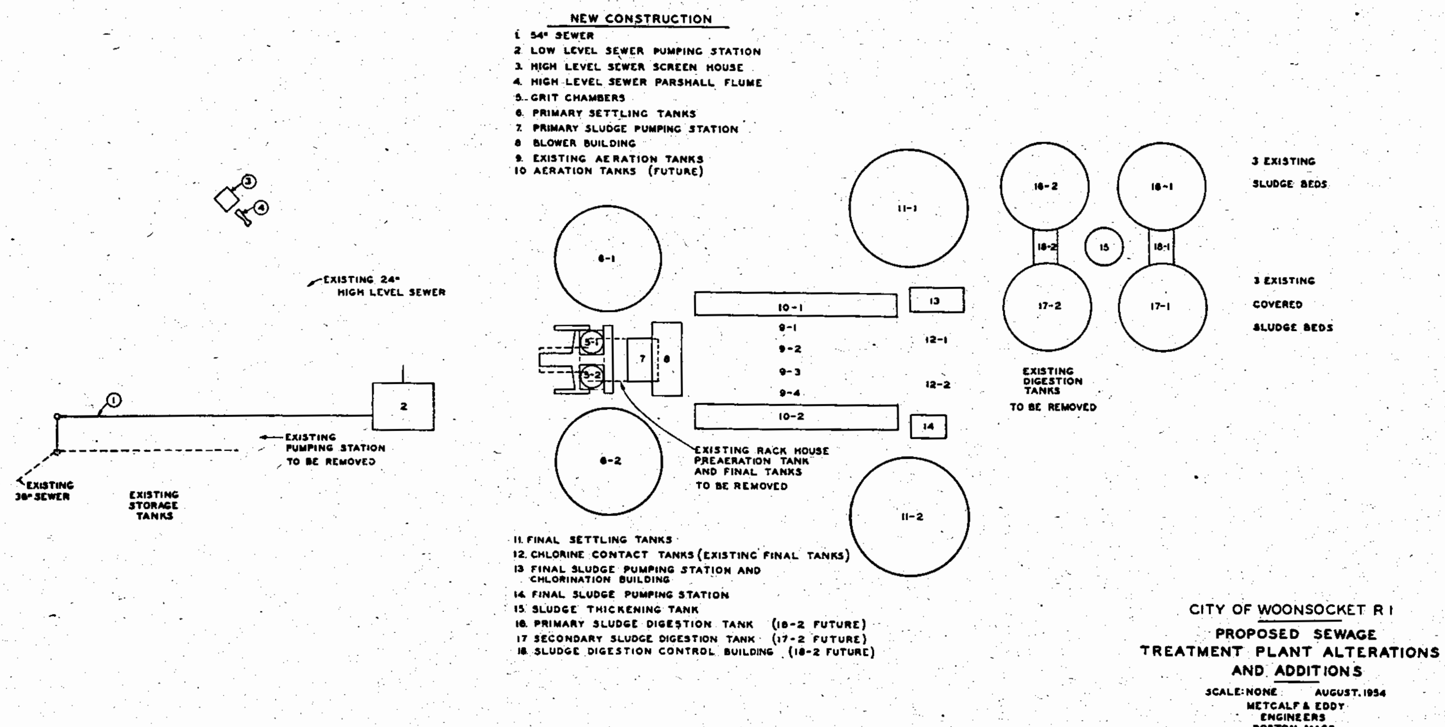
NEW CONSTRUCTION
1 54" SEWER
2 LOW LEVEL SEWER PUMPING STATION
3 HIGH LEVEL SEWER SCREEN HOUSE
4 HIGH LEVEL SEWER PARSHALL FLUME
5 GRIT CHAMBERS
6 PRIMARY SETTLING TANKS
7 PRIMARY SLUDGE PUMPING STATION
8 BLOWER BUILDING
9 EXISTING AERATION TANKS
10 AERATION TANKS (FUTURE)
11 FINAL SETTLING TANKS
12 CHLORINE CONTACT TANKS (EXISTING FINAL TANKS)
13 FINAL SLUDGE PUMPING STATION AND CHLORINATION BUILDING
14 FINAL SLUDGE PUMPING STATION
15 SLUDGE THICKENING TANK
16 PRIMARY SLUDGE DIGESTION TANK (16-2 FUTURE)
17 SECONDARY SLUDGE DIGESTION TANK (17-2 FUTURE)
18 SLUDGE DIGESTION CONTROL BUILDING (18-2 FUTURE)
EXISTING 24" HIGH LEVEL SEWER
EXISTING PUMPING STATION TO BE REMOVED
EXISTING 36" SEWER
EXISTING STORAGE TANKS
EXISTING RACK HOUSE PREAERATION TANK AND FINAL TANKS TO BE REMOVED
EXISTING DIGESTION TANKS TO BE REMOVED
3 EXISTING SLUDGE BEDS
3 EXISTING COVERED SLUDGE BEDS
CITY OF WOONSOCKET R I
PROPOSED SEWAGE TREATMENT PLANT ALTERATIONS AND ADDITIONS
SCALE: NONE AUGUST, 1954
METCALF & EDDY ENGINEERS BOSTON, MASS

FIG. 5.

Other studies and methods of correction were recommended for points of lesser infiltration in other portions of the City.

Modifications and Reinforcements to Trunk Sewer. On the basis of a study of population density and industrial wastes volumes, recommendations were made relative to modifications and reinforcements.

The study indicated that the major trunk lines of the system had adequate capacity to handle present and future flows with the following exceptions:

a. Low Level Trunk Sewer. Additional capacity must be provided in this sewer when all of the industrial wastes now being discharged to the river are diverted to the City's system. The recommended plan involved complete replacement of the older downstream brick section with new pipe of greater capacity and construction of a parallel sewer in the remaining sections to provide additional capacity for future flows.

b. Diamond Hill Trunk Sewer. It was determined that a relief sewer will be required to reinforce the existing sewer as northern areas of the City are sewered.

c. Cass Avenue Trunk Sewer. It was determined that when the eastern areas of the City are sewered, a new relief sewer will be required to prevent overloading of existing sewers. The cost of this sewer was not estimated as its capacity and location were too indefinite.

Extensions. The unsewered areas of the City were divided into seven sewer districts. Each district was studied and recommendations made relative to the size and cost of sewer required in each street. Also recommended was the construction of a total of three small pumping stations.

Summary. A summary of costs of sewerage facilities is given in Table 5.

Rubbish and Garbage Disposal

Household rubbish and ashes are collected by a contractor engaged by the City. Collections are made weekly throughout most of the City although in sparsely settled areas, collections are limited to once in every 2 weeks. All rubbish is deposited on an open dump operated by the City and located along the river near the center of the City. The dump capacity is now nearly exhausted. Either additional dumping areas or alternative new methods of rubbish disposal will be required in the near future. Although efforts have been made in recent years to control the operation of the dump, it remains a source of smoke, odors, debris and provides a breeding place for rodents and insects.

TABLE 5
SUMMARY OF SEWERAGE COSTS

Immediate	
Repair or relay River Street sewer	$ 39,000
Within next 5 years	
Replace and reinforce low-level trunk sewer	791,000
Reinforce Diamond Hill trunk sewer	30,000
Within next 10 years	
Sewer extensions	612,700
Future	
Sewer extensions	1,313,600
Total	$2,786,300

Garbage is collected twice weekly by another contractor engaged by the City for disposal at his piggery in Cumberland, Rhode Island. Although the piggery is clean and well maintained, the future of this method of disposal is questionable because of recent legislation requiring the cooking of garbage prior to its use as a hog feed. Consequently, study was given to alternate methods of garbage disposal.

Methods of Refuse Disposal and Quantities of Refuse. Studies were made of the several methods of garbage and refuse disposal which could be adopted by Woonsocket and it was concluded that the most economical method of disposal was by means of incineration.

To obtain a basis for design, all rubbish delivered to the dump was weighed for a period of 1 week and all garbage collected on an average day was also weighed. It was thus determined that approximately 1.6 and 0.4 lb. per capita of rubbish and garbage, respectively, were collected in Woonsocket on an average day. This information was then compared with the per-capita weight of combustible rubbish in several other cities and as a result it was decided to base the furnace design on a weight of 2.5 lb. per capita per day, the approximate average of the other cities. It was our opinion that the low per-capita contribution in Woonsocket was partially the result of widespread unemployment in the City.

TABLE 6
SUMMARY RECOMMENDED PROGRAM OF EXPENDITURES

	First Cost of Projects				
	Immediate	*Within 2 years*	*Within 5 years*	*Within 10 years*	*Future (as required)*
Water					
Supply and Treatment	$475,300	0	$1,409,000	0	$ 306,000
Pumping, Storage and Distribution	97,500	$ 89,000	365,700	$ 199,300	815,900
Sewerage					
Sewage Treatment	0	1,203,300	1,095,200	668,000	0
Sewers and Pumping Stations	39,000	0	821,000	612,700	1,313,600
Refuse Disposal					
Incinerator	0	520,000	0	0	0
Total Costs	$611,800	$1,812,300	$3,690,900	$1,480,000	$2,435,500

Incinerator. Several sites and sizes of furnaces were studied. It was ultimately recommended that a side hill site near the sewage treatment plant be obtained, and that an incinerator housing two 80-ton furnaces be constructed. These furnaces would provide sufficient capacity to destroy all of the present rubbish in a single 8-hr. shift with both furnaces in operation. At the end of the design period both furnaces would be needed for two 8-hr. shifts.

The cost of the incinerator plant was estimated to be $520,000.

Summary of Recommendations

Because of the large capital expenditures required to carry out the complete program and because many of the items will not be required for a number of years, a step construction program was devised based upon the time when it was estimated that the various facilities would be required. A summary of all of the recommended facilities in this step program is given in Table 6. It should be recognized that this program is not inflexible and that items may be changed from one category to another as the need changes. However, all of the items scheduled within the next 10 years should be completed as expeditiously as possible within the financial capacity of the City to provide an adequate system of sanitary facilities and further to provide a firm base upon which future improvements may be added.

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